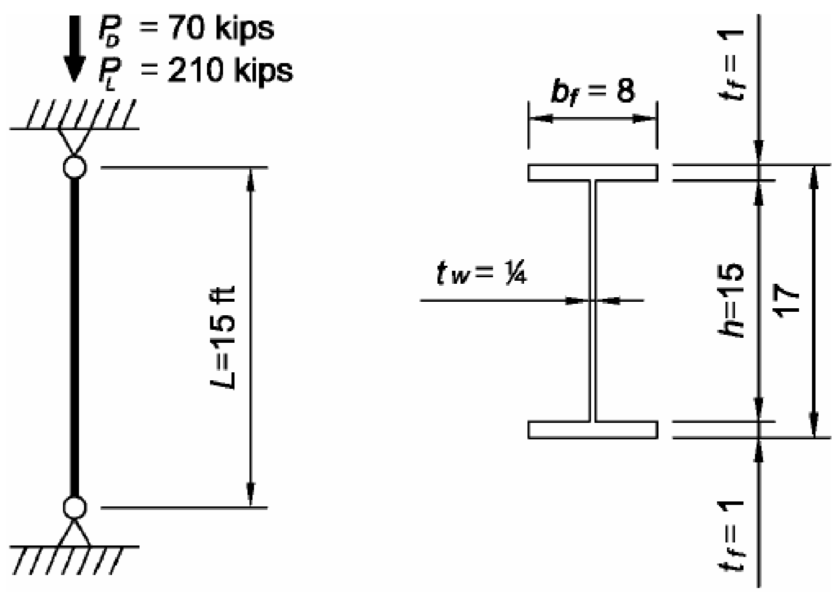
AISC 360-10 Example 002

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

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 360-10).

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

COMPUTER FILE: AISC 360-10 EX002

CONCLUSION

The results show an exact comparison with the independent results.

HAND CALCULATION

Properties:

Material: ASTM A572 Grade 50

E = 29,000 ksi, F_y = 50 ksi

Section: Built-Up Wide Flange

d = 17.0 in, b_f = 8.00 in, t_f = 1.00 in, h = 15.0 in, t_w = 0.250 in.

Ignoring fillet welds:

A = 2(8.00)(1.00) + (15.0)(0.250) = 19.75 in^2

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

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

$$I_x = \sum Ad^2 + \sum I_x$$

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

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

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

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

Member:

K = 1.0 for a pinned-pinned condition

L = 15 ft

Loadings:

P_u = 1.2(70.0) + 1.6(210) = 420 kips

Section Compactness:

Check for slender elements using Specification Section E7

Localized Buckling for Flange:

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

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

$\lambda < \lambda_p$, No localized flange buckling

Flange is Compact.

Localized Buckling for Web:

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

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

$\lambda > \lambda_r$, Localized web buckling

Web is Slender.

$\boxed{\text{Section is Slender}}$

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.

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

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

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.

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

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

Therefore, the flexural buckling limit state controls.

$F_e = 38.18$ ksi

<u>Section Reduction Factors</u>

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 $Q = 1.0$

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

So

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

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

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

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

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

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

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

$$P_n = F_{cr}A_g = 28.5 \bullet 19.75 = 562.3\,\text{kips}$$
$$\phi_c = 0.90$$
$$\phi_c P_n = F_{cr}A_g = 0.90(562.3) = 506.1\,\text{kips} > 420\text{ kips}$$
$$\boxed{\phi_c P_n = 506.1\,\text{kips}}$$