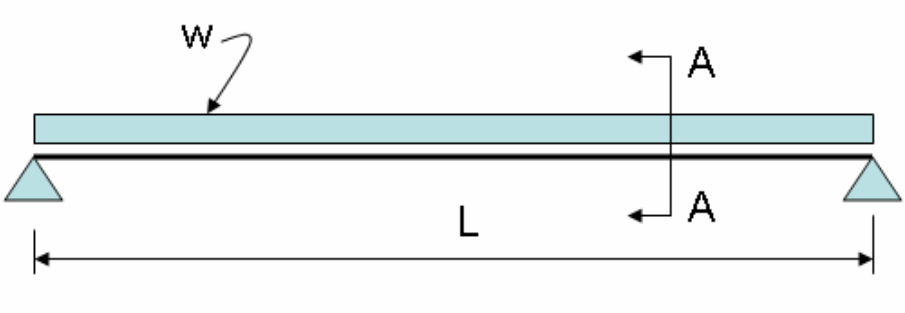
AISC ASD-89 Example 001

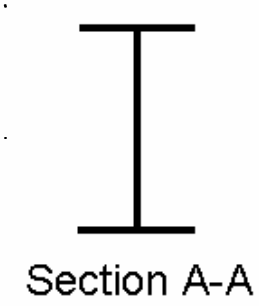
WIDE FLANGE MEMBER UNDER BENDING

EXAMPLE DESCRIPTION

The beam below is subjected to a bending moment of 20 kip-ft. The compression flange is braced at 3.0 ft intervals. The selected member is non-compact due to flange criteria.

GEOMETRY, PROPERTIES AND LOADING





Member Properties
W6X12, M10X9, W8X10
E = 29000 ksi

Loading
w = 1.0 klf

Geometry
Span, L = 12.65 ft

TECHNICAL FEATURES TESTED

- Section compactness check (bending)
- Member bending capacity

RESULTS COMPARISON

Independent results are taken from Allowable Stress Design Manual of Steel Construction, Ninth Edition, 1989, Example 3, Page 2-6.

Output Parameter	SAP2000	Independent	Percent Difference
Compactness	Non-Compact	Non-Compact	0.00%
Design Bending Stress, f_b (ksi)	30.74	30.74	0.00%
Allowable Bending Stress, F_b (ksi)	32.70	32.70	0.00 %

COMPUTER FILE: AISC ASD-89 EX001

CONCLUSION

The results show an exact match with the independent results.

HAND CALCULATION

Properties:

Material: ASTM A572 Grade 50 Steel

E = 29,000 ksi, F_y = 50 ksi

Section: W8x10

b_f = 3.94 in, t_f = 0.205 in, d = 7.98 in, t_w = 0.17 in

$h = h - 2t_f = 7.89 - 2 \bullet 0.205 = 7.48$ in

Member:

L = 12.65 ft

l_b = 3 ft

Loadings:

w = 1.0 k/ft

$$M = \frac{wL^2}{8} = 1.0 \bullet 12.65^2/8 = 20.0 \text{ k-ft}$$

Design Bending Stress

$$f_b = M / S_{33} = 20 \bullet 12 / 7.8074$$

$$\boxed{f_b = 30.74 \text{ ksi}}$$

Section Compactness:

Localized Buckling for Flange:

$$\lambda = \frac{b_f}{2t_f} = \frac{3.94}{2 \bullet 0.205} = 9.610$$

$$\lambda_p = \frac{65}{\sqrt{F_y}} = \frac{65}{\sqrt{50}} = 9.192$$

$$\lambda_r = \frac{95}{\sqrt{F_y}} = \frac{95}{\sqrt{50}} = 13.435$$

$\lambda > \lambda_p$, Localized flange buckling is present.

$\lambda < \lambda_r$,

Flange is Non-Compact.

Localized Buckling for Web:

$$\lambda = \frac{d}{t_w} = \frac{7.89}{0.17} = 46.412$$

No axial force is present, so $f_a = \frac{P}{A} = 0$ and $\frac{f_a}{F_y} = 0 \le 0.16$, so

$$\lambda_p = \frac{640}{\sqrt{F_y}}\left(1 - 3.74\frac{f_a}{F_y}\right) = \frac{640}{\sqrt{50}}\left(1 - 3.74 \bullet \frac{0}{50}\right) = 90.510$$

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

Web is Compact.

Section is Non-Compact.

Section Bending Capacity

Allowable Bending Stress

Since section is Non-Compact

$$F_{b33} = \left(0.79 - 0.002\frac{b_f}{2t_f}\sqrt{F_y}\right)F_y$$

$$F_{b33} = \left(0.79 - 0.002 \bullet 9.61 \bullet \sqrt{50}\right)50$$

$F_{b33} = 32.70$ ksi

Member Bending Capacity for L_b = 3.0 ft:

Critical Length, l_c:

$$l_c = \min\left\{\frac{76b_f}{\sqrt{F_y}}, \frac{20{,}000A_f}{dF_y}\right\}$$

$$l_c = \min\left\{\frac{76 \bullet 3.94}{\sqrt{50}}, \frac{20{,}000 \bullet 3.94 \bullet 0.205}{7.89 \bullet 50}\right\}$$

$$l_c = \min\left\{42.347, 40.948\right\}$$

$$l_c = 40.948 \text{ in}$$

$$l_{22} = l_b = 3 \bullet 12 = 36 \text{ in}$$

$l_{22} < l_c$, section capacity is as follows:

$$\boxed{F_{b33} = 32.70 \text{ ksi}}$$