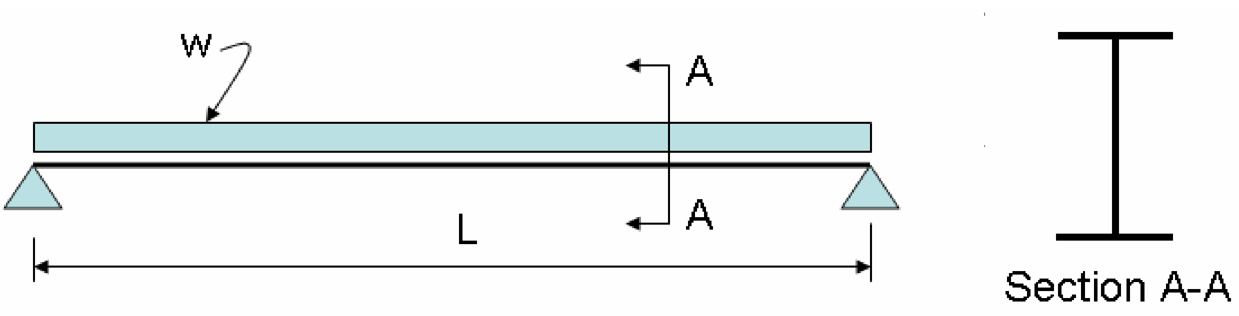
AISC 360-05 Example 001

WIDE FLANGE MEMBER UNDER BENDING

EXAMPLE DESCRIPTION

The design flexural strengths are checked for the beam shown below. The beam is loaded with a uniform load of 0.45 klf (D) and 0.75 klf (L). The flexural moment capacity is checked for three unsupported lengths in the weak direction, L_b = 5 ft, 11.667 ft and 35 ft.

GEOMETRY, PROPERTIES AND LOADING



Member Properties
W18X50
E = 29000 ksi
F_y = 50 ksi

Loading
w = 0.45 klf (D)
w = 0.75 klf (L)

Geometry
Span, L = 35 ft

TECHNICAL FEATURES TESTED

- Section Compactness Check (Bending)
- Member Bending Capacities
- Unsupported length factors

RESULTS COMPARISON

Independent results are comparing with the results of Example F.1-2a from the AISC Design Examples, Volume 13 on the application of the 2005 AISC Specification for Structural Steel Buildings (ANSI/AISC 360-05).

Output Parameter	ETABS	Independent	Percent Difference
Compactness	Compact	Compact	0.00%
C_b (L_b =5ft)	1.004	1.002	0.20%
$\phi_b M_n$ (L_b =5ft) (k-ft)	378.750	378.750	0.00%
C_b (L_b =11.67ft)	1.015	1.014	0.10%
$\phi_b M_n$ (L_b =11.67ft) (k-ft)	307.124	306.657	0.15%
C_b (L_b =35ft)	1.138	1.136	0.18%
$\phi_b M_n$ (L_b =35ft) (k-ft)	94.377	94.218	0.17%

COMPUTER FILE: AISC 360-05 Ex001

CONCLUSION

The results show an acceptable comparison with the independent results.

HAND CALCULATION

Properties:

Material: *ASTM A572 Grade 50 Steel*

E = 29,000 ksi, F_y = 50 ksi

Section: *W18x50*

b_f = 7.5 in, t_f = 0.57 in, d = 18 in, t_w = 0.355 in

$$h = d - 2t_f = 18 - 2 \bullet 0.57 = 16.86 \ in$$

$$h_0 = d - t_f = 18 - 0.57 = 17.43 \ in$$

S_{33} = 88.9 in^3, Z_{33} = 101 in^3

I_y =40.1 in^4, r_y = 1.652 in, C_w = 3045.644 in^6, J = 1.240 in^4

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_{33}}} = \sqrt{\frac{\sqrt{40.1 \bullet 3045.644}}{88.889}} = 1.98 in$$

$R_m = 1.0$ *for doubly-symmetric sections*

Other:

c = 1.0

L = 35 ft

Loadings:

w_u = (1.2w_d + 1.6w_l) = 1.2(0.45) + 1.6(0.75) = 1.74 k/ft

$$M_u = \frac{w_u L^2}{8} = 1.74 \bullet 35^2/8 = 266.4375 \text{ k-ft}$$

Section Compactness:

Localized Buckling for Flange:

$$\lambda = \frac{b_f}{2t_f} = \frac{7.50}{2 \bullet 0.57} = 6.579$$

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

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

Flange is Compact.

Localized Buckling for Web:

$$\lambda = \frac{h}{t_w} = \frac{16.86}{0.355} = 47.49$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{50}} = 90.553$$

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

Web is Compact.

Section is Compact.

Section Bending Capacity:

$$M_p = F_y Z_{33} = 50 \bullet 101 = 5050\, k - in$$

Lateral-Torsional Buckling Parameters:

Critical Lengths:

$$L_p = 1.76\, r_y\sqrt{\frac{E}{F_y}} = 1.76 \bullet 1.652\sqrt{\frac{29000}{50}} = 70.022\ in = 5.835\ ft$$

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_{33}h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_{33}h_o}{Jc}\right)^2}}$$

$$L_r = 1.95 \bullet 1.98\frac{29000}{0.7 \bullet 50}\sqrt{\frac{1.240 \bullet 1.0}{88.9 \bullet 17.43}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7 \bullet 50}{29000}\frac{88.9 \bullet 17.43}{1.240 \bullet 1.0}\right)^2}}$$

$L_r = 16.966\ ft$

Non-Uniform Moment Magnification Factor:

For the lateral-torsional buckling limit state, the non-uniform moment magnification factor is calculated using the following equation:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0 \qquad \textit{Eqn. 1}$$

Where M_A = first quarter-span moment, M_B = mid-span moment, M_C = second quarter-span moment.

The required moments for Eqn. 1 can be calculated as a percentage of the maximum mid-span moment. Since the loading is uniform and the resulting moment is symmetric:

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2$$

Member Bending Capacity for L_b = 5 ft:

$$M_{\max} = M_B = 1.00$$

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{5}{35}\right)^2 = 0.995$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.995) + 4(1.00) + 3(0.995)}$$

$$C_b = 1.002$$

$L_b < L_p$, Lateral-Torsional buckling capacity is as follows:

$$M_n = M_p = 5050\, k - in$$

$$\varphi_b M_n = 0.9 \bullet 5050/12$$

$$\varphi_b M_n = 378.75\ \ k - ft$$

Member Bending Capacity for L_b* = 11.667 ft*:

$$M_{\max} = M_B = 1.00$$

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{11.667}{35}\right)^2 = 0.972$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.972) + 4(1.00) + 3(0.972)}$$

$$C_b = 1.014$$

$L_p < L_b < L_r$, Lateral-Torsional buckling capacity is as follows:

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_{33}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$M_n = 1.014\left[5050 - \left(5050 - 0.7 \bullet 50 \bullet 88.889\right)\left(\frac{11.667 - 5.835}{16.966 - 5.835}\right)\right] = 4088.733\ k - in$$

$$\varphi_b M_n = 0.9 \bullet 4088.733/12$$

$$\varphi_b M_n = 306.657\ \ k - ft$$

Member Bending Capacity for L_b* = 35 ft*:

$$M_{\max} = M_B = 1.00$$

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{35}{35}\right)^2 = 0.750\,.$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.750) + 4(1.00) + 3(0.750)}(1.00)$$

$$C_b = 1.136$$

$L_b > L_r$, Lateral-Torsional buckling capacity is as follows:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_{33} h_o} \left(\frac{L_b}{r_{ts}}\right)^2}$$

$$F_{cr} = \frac{1.136 \bullet \pi^2 \bullet 29000}{\left(\frac{420}{1.983}\right)^2} \sqrt{1 + 0.078 \frac{1.24 \bullet 1}{88.889 \bullet 17.4} \left(\frac{420}{1.983}\right)^2} = 14.133 ksi$$

$$M_n = F_{cr} S_{33} \leq M_p$$

$$M_n = 14.133 \bullet 88.9 = 1256.245 \ k - in$$

$$\varphi_b M_n = 0.9 \bullet 1256.245 / 12$$

$$\varphi_b M_n = 94.218 \ \ k - ft$$