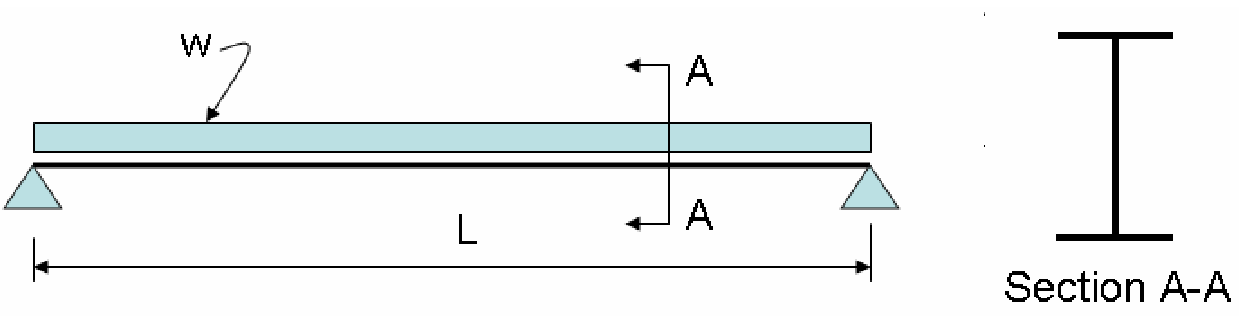
KBC 2009 Example 001

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 6.5 kN/m (D) and 11 kN/m (L). The flexural moment capacity is checked for three unsupported lengths in the weak direction, L_b = 1.75 m, 4 m and 12 m.

GEOMETRY, PROPERTIES AND LOADING



Member Properties
W460x74
E = 205,000 MPa
F_y = 345 MPa

Loading
w = 6.5 kN/m (D)
w = 11.0 kN/m (L)

Geometry
Span, L = 12m

TECHNICAL FEATURES TESTED

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

RESULTS COMPARISON

Independent results are comparing with the results of ETABS.

Output Parameter	ETABS	Independent	Percent Difference
Compactness	Compact	Compact	0.00%
C_b (L_b =1.75m)	1.004	1.002	0.20%
$\phi_b M_n$ (L_b =1.75m) (kN-m)	515.43	515.43	0.00%
C_b (L_b =4m)	1.015	1.014	0.10%
$\phi_b M_n$ (L_b =4m) (kN-m)	394.8	394.2	0.15%
C_b (L_b =12m)	1.136	1.136	0.00%
$\phi_b M_n$ (L_b =12m) (kN-m)	113.48	113.45	0.03%

COMPUTER FILE: KBC 2009 Ex001

CONCLUSION

The results show an acceptable comparison with the independent results.

HAND CALCULATION

Properties:

Material:

E = 205,000 MPa, F_y = 345 MPa

Section: W460x74

b_f = 191 mm, t_f = 14.5 mm, d = 457 mm, t_w = 9 mm

$$h = d - 2t_f = 457 - 2 \bullet 14.5 = 428 \text{ mm}$$

$$h_0 = d - t_f = 457 - 14.5 = 442.5 \text{ mm}$$

S_{33} = 1457.3 cm^3, Z_{33} = 1660 cm^3

I_y =1670 cm^4, r_y = 42 mm, C_w = 824296.4 cm^6, J = 51.6 cm^4

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_{33}}} = \sqrt{\frac{\sqrt{1670 \bullet 824296.4}}{1457.3}} = 50.45 \text{ mm}$$

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

Other:

c = 1.0

L = 12 m

Loadings:

w_u = (1.2w_d + 1.6w_l) = 1.2(6.5) + 1.6(11) = 25.4 kN/m

$$M_u = \frac{w_u L^2}{8} = 25.4 \bullet 12^2/8 = 457.2 \text{ kN-m}$$

Section Compactness:

Localized Buckling for Flange:

$$\lambda = \frac{b_f}{2t_f} = \frac{191}{2 \bullet 14.5} = 6.586$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{205,000}{345}} = 9.263$$

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

Flange is Compact.

<u>*Localized Buckling for Web:*</u>

$$\lambda = \frac{h}{t_w} = \frac{428}{9} = 47.56$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{205,000}{345}} = 91.654$$

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

Web is Compact.

Section is Compact.

Section Bending Capacity:

$$M_p = F_y Z_{33} = 345 \bullet 1660 = 572.7 \text{ kN-m}$$

Lateral-Torsional Buckling Parameters:

<u>*Critical Lengths:*</u>

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76 \bullet 42\sqrt{\frac{205,000}{345}} = 1801.9 \text{ mm} = 1.8 \text{ m}$$

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

$$L_r = 1.95 \bullet 50.45\frac{205,000}{0.7 \bullet 345}\sqrt{\frac{51.6 \bullet 1}{1457.3 \bullet 44.25}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7 \bullet 345}{205,000}\frac{1457.3 \bullet 44.8}{51.6 \bullet 1}\right)^2}}$$

$L_r = 5.25\,\text{m}$

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 = 1.75 m:

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

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{1.75}{12}\right)^2 = 0.995$$

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

$$C_b = 1.002$$

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

$$M_n = M_p = 572.7\,\text{kN-m}$$

$$\phi_b M_n = 0.9 \bullet 572.7$$

$$\phi_b M_n = 515.43\ \text{kN-m}$$

Member Bending Capacity for L_b* = 4 m*:

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

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{4}{12}\right)^2 = 0.972$$

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

$$C_b = 1.014$$

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

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

$$M_n = 1.014\left[572.7 - \left(572.7 - 0.7 \bullet 0.345 \bullet 1457.3\right)\left(\frac{4.00 - 1.80}{5.25 - 1.80}\right)\right] = 437.97 \text{ kN-m}$$

$$\phi_b M_n = 0.9 \bullet 437.97$$

$$\phi_b M_n = 394.2 \text{ kN-m}$$

Member Bending Capacity for L_b* = 12 m*:

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

$$M_A = M_C = 1 - \frac{1}{4}\left(\frac{L_b}{L}\right)^2 = 1 - \frac{1}{4}\left(\frac{12}{12}\right)^2 = 0.750.$$

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

$$C_b = 1.136$$

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

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

$$F_{cr} = \frac{1.136 \bullet \pi^2 \bullet 205,000}{\left(\frac{12000}{50.45}\right)^2} \sqrt{1 + 0.078 \frac{51.6 \bullet 1}{1457.3 \bullet 44.25} \left(\frac{12000}{50.45}\right)^2} = 86.5\,\text{MPa}$$

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

$$M_n = 86.5 \bullet 1457.3 = 126.056\text{kN-m}$$

$$\phi_b M_n = 0.9 \bullet 126.056$$

$$\phi_b M_n = 113.45 \text{ kN-m}$$