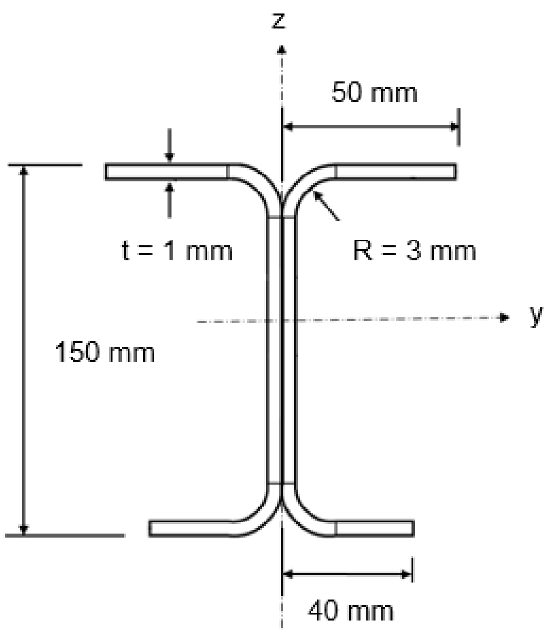
EC3 1-3 2006 CFFD Example 002

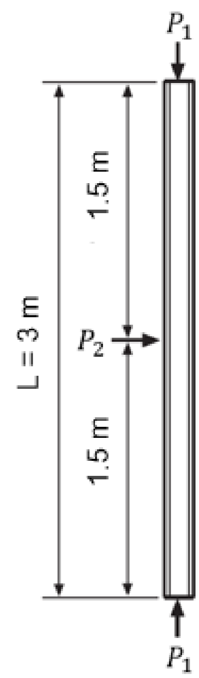
I-Section Member Under Combined Compression, Bending, and Shear

Example Description

Compression, moment, and shear capacities and demand/capacity ratio are calculated for I section at mid-height as shown below. It is simply supported with a length of 3.0 meters. The member is fully braced flexural buckling about z-z axis.

Geometry, Properties and Loading





Dead: $P_1 = 2000\ N, P_2 = 250\ N$
Live: $P_1 = 5000\ N, P_2 = 500\ N$

Technical Features Tested

- Axial compressive strength
- Major moment strength
- Shear strength
- Demand/Capacity ratio.

COMPUTER FILE: EC3 1-3 2006 CFFD EX002

Applicable Programs

- SAP2000

RESULTS COMPARISON

Independent results are hand calculated.

CONCLUSION

The results show exact match with independent results.

Benchmarks: SAP2000

Output Parameter	Program	Independent	Percent Difference
Axial - Flexural buckling $N_{b,Rd}$ (N)	65718.9	65738.2	0.03%
Axial – Torsional-Flexural buckling $N_{b,Rd}$ (N)	26485.5	26508.5	0.09%
Axial – Local & Distortional Buckling $N_{c,Rd}$ (N)	76173	76174	0.00%
Flexure – Lateral-Torsional Buckling $M_{b,Rd}$ $(N-mm)$	2271718	2271718	0.00%
Flexure – Local & Distortional Buckling $M_{c,Rd}$ $(N-mm)$	4660295	4660250	0.00%
Shear $V_{b,Rd}$ (N)	47573.9	47573.9	0.00%
D/C Ratio	0.931	0.931	0.00%

HAND CALCULATION

Properties:

Material: $E = 210{,}000\ N/mm^2$, $G = 80{,}770\ N/mm^2$, $f_{yb} = 350\ N/mm^2$

Section: $h = 150\ mm$, $b_1 = 50\ mm$, $b_2 = 40\ mm$, $t = 1\ mm$, $r = 3\ mm$

$\rightarrow h_p = h - t = 150 - 1 = 149\ mm$

$\rightarrow b_{1p} = b_1 - t/2 = 50 - 1/2 = 49.5\ mm$

$\rightarrow b_{2p} = b_2 - t/2 = 40 - 1/2 = 39.5\ mm$

Check for the effect of rounding of the corners:

$$\frac{r}{t} = \frac{3}{1} = 3 < 5 \rightarrow OK$$

$$\frac{r}{b_{1p}} = \frac{3}{49.5} = 0.061 < 0.1 \rightarrow OK$$

$$\frac{r}{b_{2p}} = \frac{3}{39.5} = 0.076 < 0.1 \rightarrow OK$$

Therefore, the effect of rounding of the corners can be neglected in calculation of section properties:

$A_g = 476\ (mm^2)$

$I_y = 1534605.3\ (mm^4)$

$I_z = 125987.6\ (mm^4)$

$i_y = 56.78\ (mm)$

$i_z = 16.27\ (mm)$

$W_{el,c} = 21502.2\ (mm^3)$

$W_{el,t} = 19768.1\ (mm^3)$

$I_t = 158.67\ (mm^4)$

$I_w = 604830242\ (mm^6)$

$y_0 = 0.0\ (mm)$

$z_0 = 21.17\ (mm)$

Member: $K_y = K_T = 1.0$ for a pinned-pinned condition

$L_y = L_z = L_T = 3000\ mm$

Member is modeled to be braced against flexural buckling about z-z axis by setting $K_z = 0.0001$

Loadings: Dead: $P_1 = 2000\ N, P_2 = 250\ N$

Live: $P_1 = 5000\ N, P_2 = 500\ N$

Required strengths: for the section in the middle

$$N_{Ed} = 1.2D + 1.6L = 1.2 \times 2000 + 1.6 \times 5000 = 10400\ (N)$$

$$M_{Ed} = 1.2D + 1.6L = 1.2 \times \frac{250 \times 3000}{4} + 1.6 \times \frac{500 \times 3000}{4} = 825000\ (N - mm)$$

$$V_{Ed} = 1.2D + 1.6L = 1.2 \times \frac{250}{2} + 1.6 \times \frac{500}{2} = 550\ (N)$$

Member Compression Capacity: the compression capacity is calculated considering the limit states of global buckling, and local buckling. Distortional buckling is not considered as there is no lip stiffener.

1. Local buckling:

The effective width method is utilized to calculate the nominal axial strength in consideration of local buckling with the compressive stress of $f_{yb} = 350\ (N/mm^2)$.

Check for the applicability of the method as the following conditions are satisfied:

$$\frac{b_1}{t} = \frac{50}{1} = 50 = 50 \rightarrow OK$$

$$\frac{b_2}{t} = \frac{40}{1} = 40 < 50 \rightarrow OK$$

$$\frac{h}{2t} = \frac{150}{2 \times 1} = 75 < 500 \rightarrow OK$$

As the section is subjected to uniform compression and both flanges are considered outstand unstiffened elements:

Top flange:

$$\psi = 1$$

$$k_\sigma = 0.43$$

$$\varepsilon = \sqrt{\frac{235}{f_{yb}[N/mm^2]}} = \sqrt{\frac{235}{350}} = 0.8194$$

$$\bar{\lambda}_{1p,b} = \frac{b_{1p}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{49.5/1}{28.4 \times 0.8194\sqrt{0.43}} = 3.244 > 0.748$$

$$\rho = \frac{\bar{\lambda}_{1p,b} - 0.188}{\bar{\lambda}_{1p,b}^2} = \frac{3.244 - 0.188}{3.244^2} = 0.2904 \leq 1.0$$

$$b_{1eff} = \rho b_{1p} = 0.2904 \times 49.5 = 14.375\ (mm)$$

Bottom flange:

$$\psi = 1$$

$$k_\sigma = 0.43$$

$$\varepsilon = \sqrt{\frac{235}{f_{yb}[N/mm^2]}} = \sqrt{\frac{235}{350}} = 0.8194$$

$$\bar{\lambda}_{2p,b} = \frac{b_{2p}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{39.5/1}{28.4 \times 0.8194\sqrt{0.43}} = 2.5885 > 0.748$$

$$\rho = \frac{\bar{\lambda}_{2p,b} - 0.188}{\bar{\lambda}_{2p,b}^2} = \frac{2.5885 - 0.188}{2.5885^2} = 0.3583 \leq 1.0$$

$$b_{2eff} = \rho b_{2p} = 0.3583 \times 39.5 = 14.152\ (mm)$$

The web is considered an internal (stiffened) element under uniform compression:

$$\psi = 1$$

$$k_\sigma = 4$$

$$\bar{\lambda}_{p,b} = \frac{h_p/(2t)}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{149/2}{28.4 \times 0.8194\sqrt{4}} = 1.6 > 0.673$$

$$\rho = \frac{\bar{\lambda}_{p,b} - 0.055(3+\psi)}{\bar{\lambda}_{p,b}^2} = \frac{1.6 - 0.055(3+1)}{1.6^2} = 0.539 \le 1.0$$

$$h_{eff} = \rho h_p = 0.539 \times 149 = 80.3\ (mm)$$

$$h_{e1} = h_{e2} = 0.5h_{eff} = 0.5 \times 80.3 = 40.15\ (mm)$$

$$A_{eff} = 2th_{eff} + 2tb_{1eff} + 2tb_{2eff}$$

$$= 2 \times 80.3 + 2 \times 14.375 + 2 \times 14.152 = 217.64\ (mm^2)$$

$$A_{eff} = 217.64\ (mm^2) < 476\ (mm^2) = A_g$$

$$\rightarrow N_{c,Rd} = \frac{A_{eff} f_{yb}}{\gamma_{M0}} = \frac{217.64 \times 350}{1.0} = 76174\ (N)$$

Because the section is symmetric about z-z axis, its effective properties are also symmetric about z-z axis, resulting in $e_{Nz} = 0 \rightarrow \Delta M_{z,Ed} = 0$

$$\bar{y} = \frac{\sum_i A_i y_i}{A} = \frac{2th_p \frac{h_p}{2} + 2tb_{2p}h_p}{A} = \frac{2 \times 149 \frac{149}{2} + 2 \times 39.5 \times 149}{476} = 71.37\ (mm)$$

$$\bar{y}_{eff} = \frac{\sum_i A_{eff,i} z_i}{A_{eff}} = \frac{2th_{e1}\frac{h_{e1}}{2} + 2th_{e2}\left(h_p - \frac{h_{e2}}{2}\right) + 2tb_{2eff}h_p}{A_{eff}}$$

$$= \frac{2 \times 40.15 \times \frac{40.15}{2} + 2 \times 40.15\left(149 - \frac{40.15}{2}\right) + 2 \times 14.152 \times 149}{217.64} = 74.35\ (mm)$$

$$e_{Ny} = \bar{z}_{eff} - \bar{z} = 74.35 - 71.37 = 2.98\ (mm)$$

$$\Delta M_{y,Ed} = N_{Ed} e_{Ny} = 10400 \times 2.98 = 30992\ (N - mm)$$

2. Global buckling: includes flexural buckling and torsional and flexural-torsional buckling
   i. Flexural buckling: since the member is fully braced against flexural buckling about z-axis. In the program, this bracing is modeled by assigning such a small value of effective length factor K_{2minor} for the element.

$$N_{cr,y} = \frac{\pi^2 EI_y}{\left(K_y L_y\right)^2} = \frac{\pi^2(210{,}000)1534605.3}{(1.0 \times 3000)^2} = 353405\ (N)$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{(K_z L_z)^2} = \frac{\pi^2(210{,}000)125987.6}{(0.0001 \times 3000)^2} = 2.9 \times 10^{12}\ (N)$$

$$\bar{\lambda}_y = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr,y}}} = \sqrt{\frac{217.64 \times 350}{353405}} = 0.464$$

For I section with different top and bottom flanges, the buckling curve is c and $\alpha = 0.49$

$$\Phi_y = 0.5\left[1 + \alpha\left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^2\right] = 0.5[1 + 0.49(0.464 - 0.2) + 0.464^2] = 0.672$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}} = \frac{1}{0.672 + \sqrt{0.672^2 - 0.464^2}} = 0.863$$

$$N_{by,Rd} = \frac{\chi_y A_{eff} f_{yb}}{\gamma_{M1}} = \frac{0.863 \times 217.64 \times 350}{1.0} = 65738.2\ (N)$$

ii. Torsional and flexural-torsional buckling:

$$i_0 = \sqrt{i_y^2 + i_z^2 + y_0^2 + z_0^2} = \sqrt{56.78^2 + 16.27^2 + 0.0^2 + 21.17^2} = 62.744\ (mm)$$

$$N_{cr,T} = \frac{1}{i_0^2}\left[GI_t + \frac{\pi^2 EI_w}{L_T^2}\right] = \frac{1}{62.74^2}\left[80{,}770 \times 158.67 + \frac{\pi^2 210{,}000 \times 604830242}{(1.0 \times 3000)^2}\right] = 38641\ (N)$$

$$\beta = 1 - \frac{y_0^2 + z_0^2}{i_0^2} = 1 - \frac{0.0^2 + 21.17^2}{62.74^2} = 0.886$$

$$N_{cr,TF} = \frac{N_{cr,z}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,z}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,z}}\right)^2 + 4\left(\frac{z_0}{i_0}\right)^2 \frac{N_{cr,T}}{N_{cr,z}}}\right]$$

$$= \frac{2.9 \times 10^{12}}{2 \times 0.886}\left[1 + \frac{38641}{2.9 \times 10^{12}} - \sqrt{\left(1 - \frac{38641}{2.9 \times 10^{12}}\right)^2 + 4\left(\frac{21.17}{62.74}\right)^2 \frac{38641}{2.9 \times 10^{12}}}\right] = 38641\ (N)$$

As $N_{cr,TF} = 38641\ (N) = N_{cr,T}$

$$\rightarrow \bar{\lambda}_T = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr,TF}}} = \sqrt{\frac{217.64 \times 350}{38641}} = 1.404$$

For I section with different top and bottom flanges, the buckling curve for torsional-flexural buckling is c and $\alpha = 0.49$

$$\Phi_T = 0.5\left[1 + \alpha\left(\bar{\lambda}_T - 0.2\right) + \bar{\lambda}_T^2\right] = 0.5[1 + 0.49(1.404 - 0.2) + 1.404^2] = 1.781$$

$$\chi_T = \frac{1}{\Phi_T + \sqrt{\Phi_T^2 - \bar{\lambda}_T^2}} = \frac{1}{1.781 + \sqrt{1.781^2 - 1.404^2}} = 0.348$$

$$N_{bT,Rd} = \frac{\chi_T A_{eff} f_{yb}}{\gamma_{M1}} = \frac{0.348 \times 217.64 \times 350}{1.0} = 26508.5\ (N)$$

Member Flexural Capacity: the flexural capacity is calculated considering the limit states of lateral-torsional buckling, and local buckling. Distortional buckling is not considered as there is no lip stiffener.

1. Local buckling:

   The effective width method is utilized to calculate the nominal flexural strength in consideration of local and distortional buckling with the compressive stress in the top flange of $f_{yb} = 350\ (N/mm^2)$. As the section is subjected to positive moment, the top flange is under compression and it is considered an outstand unstiffened element:

$$\psi = 1$$
$$k_\sigma = 0.43$$
$$\varepsilon = \sqrt{\frac{235}{f_{yb}[N/mm^2]}} = \sqrt{\frac{235}{350}} = 0.8194$$
$$\bar{\lambda}_{1p,b} = \frac{b_{1p}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{49.5/1}{28.4 \times 0.8194\sqrt{0.43}} = 3.244 > 0.748$$
$$\rho = \frac{\bar{\lambda}_{1p,b} - 0.188}{\bar{\lambda}_{1p,b}^2} = \frac{3.244 - 0.188}{3.244^2} = 0.2904 \leq 1.0$$
$$b_{1eff} = \rho b_{1p} = 0.2904 \times 49.5 = 14.375\ (mm)$$

The bottom flange is in tension and calculation of its effective width is not carried out.
The neutral axis of the section with effective top flange measured from the centerline of the top flange is:

$$\bar{y} = \frac{\sum_i A_i y_i}{A} = \frac{2th_p\left(\frac{h_p}{2}\right) + 2tb_{2p}h_p}{2tb_{1eff} + 2th_p + 2tb_{2p}} = \frac{2 \times 149 \times \frac{149}{2} + 2 \times 39.5 \times 149}{2 \times 14.375 + 2 \times 149 + 2 \times 39.5} = 83.726\ (mm)$$

The web is considered an internal (stiffened) element under stress gradient:

$$\sigma_1 = f_{yb} = 350\ (N/mm^2)$$
$$\sigma_2 = -f_{yb}\frac{149 - 83.726}{83.726} = -272.86\ (N/mm^2)$$
$$\psi = \frac{\sigma_2}{\sigma_1} = -\frac{272.86}{350} = -0.78$$
$$k_\sigma = 7.81 - 6.29\psi + 9.78\psi^2 = 7.81 - 6.29 \times (-0.78) + 9.78 \times (-0.78)^2 = 18.66$$
$$\bar{\lambda}_{p,b} = \frac{h_p/(2t)}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{149/2}{28.4 \times 0.8194\sqrt{18.66}} = 0.741 > 0.673$$
$$\rho = \frac{\bar{\lambda}_{p,b} - 0.055(3+\psi)}{\bar{\lambda}_{p,b}^2} = \frac{0.741 - 0.055(3 - 0.78)}{0.741^2} = 1.127 \leq 1.0 \rightarrow \rho = 1.0$$

The web is fully effective.
The neutral axis of the effective section is as calculated previously:

$$\bar{y} = \frac{\sum_i A_i y_i}{A} = 83.726\ (mm)$$
$$I_y = 2\frac{b_{1eff}t^3}{12} + 2b_{1eff}t\bar{y}^2 + 2\frac{b_{2p}t^3}{12} + 2b_{2p}t\left(h_p - \bar{y}\right)^2 + \frac{2th_p{}^3}{12} + 2h_pt\left(\frac{h_p}{2} - \bar{y}\right)^2$$
$$= 2\frac{14.375 \times 1^3}{12} + 2 \times 14.375 \times 1 \times 83.726^2 + 2\frac{39.5 \times 1^3}{12} + 2 \times 39.5 \times 1(149 - 83.726)^2$$
$$+ \frac{2 \times 1 \times 149^3}{12} + 2 \times 149 \times 1\left(\frac{149}{2} - 83.726\right)^2 = 1114833(mm^4)$$
$$W_{eff,c} = \frac{I_y}{\bar{y}} = \frac{1114833}{83.726} = 13315\ (mm^3)$$

$$W_{eff,t} = \frac{I_y}{h_p - \bar{y}} = \frac{1114833}{149 - 83.726} = 17079\ (mm^3)$$

$$M_{c,Rd} = \frac{W_{eff,c} f_{yb}}{\gamma_{M0}} = \frac{13315 \times 350}{1.0} = 4660250\ (N - mm)$$

2. <u>Lateral-torsional buckling:</u>
   Due to the concentrated loading and simply support condition at both ends of the column:
   $C_1 = 1.365$, $C_2 = 0.553$, $C_3 = 1.73$
   $k_w = 1.0$ and $K_{LTB} = 1.0$
   The neutral axis of the gross section measured from top fiber is 71.37 (mm) → $z_a = 71.37\ (mm)$ as the load is applied on the top flange
   $z_g = z_a - z_s = z_a - z_0 = 71.37 - 21.17 = 50.2\ (mm)$
   $z_j = 22.33\ (mm)$
   $L_{cr} = 3000\ (mm)$
   $I_z = 125987.6\ (mm^4)$
   $I_w = 604830242\ (mm^6)$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L_{cr}^2}\left\{\left[\left(\frac{K_{LTB}}{k_w}\right)\frac{I_w}{I_z} + \frac{L_{cr}^2 G I_T}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j\right)^2\right]^{0.5} - \left(C_2 z_g - C_3 z_j\right)\right\}$$

$$= 3330252.7\ (N - mm)$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff,y} f_{yb}}{M_{cr}}} = \sqrt{\frac{13315 \times 350}{3330252.7}} = 1.183$$

   The applicable buckling curve is b and $\alpha_{LT} = 0.34$

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right] = 0.5[1 + 0.34(1.183 - 0.2) + 1.183^2] = 1.367$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = \frac{1}{1.367 + \sqrt{1.367^2 - 1.183^2}} = 0.48733 \le 1.0$$

$$M_{b,Rd} = \chi_{LT} W_{eff,y} \frac{f_{yb}}{\gamma_{M1}} = 0.48733 \times 13315 \frac{350}{1.0} = 2271718\ (N - mm)$$

Member Shear Capacity:

$$\bar{\lambda}_w = 0.346 \frac{s_w}{2t}\sqrt{\frac{f_{yb}}{E}} = 0.346 \frac{149}{2 \times 1}\sqrt{\frac{350}{210000}} = 1.05234$$

$$\rightarrow 0.83 < \bar{\lambda}_w < 1.40$$

$$\rightarrow f_{bv} = \frac{0.48 f_{yb}}{\bar{\lambda}_w} = \frac{0.48 \times 350}{1.05234} = 159.644\ (N/mm^2)$$

$$V_{b,Rd} = \frac{h_w (2t) f_{bv}}{\gamma_{M0}} = \frac{149 \times 2 \times 1 \times 159.644}{1.0} = 47573.9\ (N)$$

Combined D/C ratio:

As the column is simply supported about both y-y and loaded with a concentrated load at midspan, α_h is taken as zero.

$C_{my} = C_{mLT} = 0.9 + 0.1\alpha_h = 0.9 + 0.1 \times 0 = 0.9$

$$k_{yy} = C_{my}\left(1 + 0.6\bar{\lambda}_y \frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}\right) \leq C_{my}\left(1 + 0.6\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}\right)$$

$$= C_{my}\left(1 + 0.6\bar{\lambda}_y \frac{N_{Ed}}{N_{by,Rd}}\right) \leq C_{my}\left(1 + 0.6\frac{N_{Ed}}{N_{by,Rd}}\right)$$

$$= 0.9\left(1 + 0.6 \times 0.464\frac{10400}{65738.2}\right) \leq 0.9\left(1 + 0.6\frac{10400}{65738.2}\right)$$

$$= 0.94 < 0.985$$

There is no loading about minor z-z axis, $\Psi = 1$

$C_{mz} = 0.6 + 0.4\Psi = 0.6 + 0.4 \times 1 = 1.0$

$$N_{cr,z} = \frac{\pi^2 EI_z}{(K_z L_z)^2} = \frac{\pi^2(210{,}000)125987.6}{(0.0001 \times 3000)^2} = 2.9 \times 10^{12}\ (N)$$

$$\bar{\lambda}_z = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr,z}}} = \sqrt{\frac{217.64 \times 350}{2.9 \times 10^{12}}} = 0.000162$$

For I section with different top and bottom flanges, the buckling curve is c and $\alpha = 0.49$

$$\Phi_z = 0.5\left[1 + \alpha\left(\bar{\lambda}_z - 0.2\right) + \bar{\lambda}_z^2\right] = 0.5[1 + 0.49(0.000162 - 0.2) + 0.000162^2]$$

$$= 0.451$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{0.451 + \sqrt{0.451^2 - 0.000162^2}} = 1.11 > 1.0 \rightarrow \chi_z = 1.0$$

$$N_{bz,Rd} = \frac{\chi_z A_{eff} f_{yb}}{\gamma_{M1}} = \frac{1.0 \times 217.64 \times 350}{1.0} = 76174\ (N)$$

$$k_{zz} = C_{mz}\left(1 + 0.6\bar{\lambda}_z \frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right) \leq C_{mz}\left(1 + 0.6\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right)$$

$$= C_{mz}\left(1 + 0.6\bar{\lambda}_z \frac{N_{Ed}}{N_{bz,Rd}}\right) \leq C_{my}\left(1 + 0.6\frac{N_{Ed}}{N_{bz,Rd}}\right)$$

$$= 1.0\left(1 + 0.6 \times 0.000162\frac{10400}{76174}\right) \leq 1.0\left(1 + 0.6\frac{10400}{76174}\right)$$

$$= 1.0 < 1.08$$

$$k_{zy} = \left[1 - \frac{0.05\bar{\lambda}_z}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right] \geq \left[1 - \frac{0.05}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right]$$

$$= \left[1 - \frac{0.05\bar{\lambda}_z}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{N_{bz,Rd}}\right] \geq \left[1 - \frac{0.05}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{N_{bz,Rd}}\right]$$

$$= \left[1 - \frac{0.05 \times 0.000162}{(0.9 - 0.25)} \times \frac{10400}{76174}\right] \geq \left[1 - \frac{0.05}{(0.9 - 0.25)} \times \frac{10400}{76174}\right]$$
$$= 1.0 > 0.989$$

$$k_{yz} = k_{zy} = 1.0$$

The combination D/C ratio by Equation 6.61 in Eurocode 3 1-1 2005 is:

$$\frac{D}{C} = \frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$
$$= \frac{10400}{65738.2} + 0.94 \frac{825000 + 30992}{2266420} + 1.0 \frac{0 + 0}{\frac{M_{z,Rk}}{\gamma_{M1}}} = 0.513$$

The combination D/C ratio by Equation 6.62 in Eurocode 3 1-1 2005 is:

$$\frac{D}{C} = \frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$
$$= \frac{10400}{76174} + 1.0 \frac{825000 + 30992}{2266420} + 1.0 \frac{0 + 0}{\frac{M_{z,Rk}}{\gamma_{M1}}} = 0.514$$

The combination D/C ratio by Equation 6.36 in Eurocode 3 1-3 2006 is:

$$\frac{D}{C} = \left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0.8} + \left(\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{by,Rd}}\right)^{0.8} + \left(\frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{bz,Rd}}\right)^{0.8}$$
$$= \left(\frac{10400}{26508.5}\right)^{0.8} + \left(\frac{825000 + 30992}{2271718}\right)^{0.8} + \left(\frac{0 + 0}{M_{bz,Rd}}\right)^{0.8} = 0.931$$

The largest ratio $\frac{D}{C} = 0.931$ is by Equation 6.36 in Eurocode 3 1-3 2006 and governs the design.