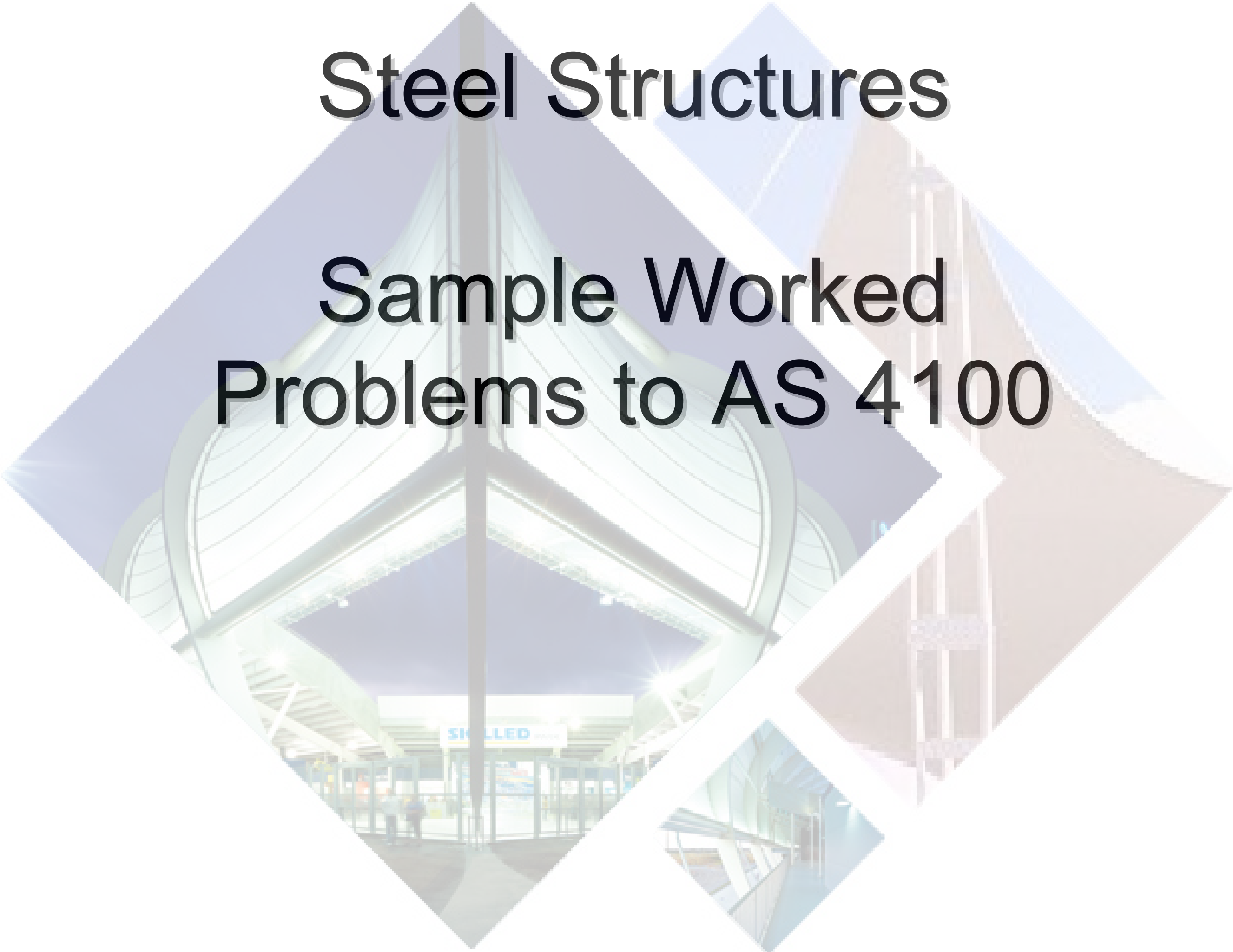
AUSTRALIAN STEEL INSTITUTE

Steel Structures

Sample Worked Problems to AS 4100



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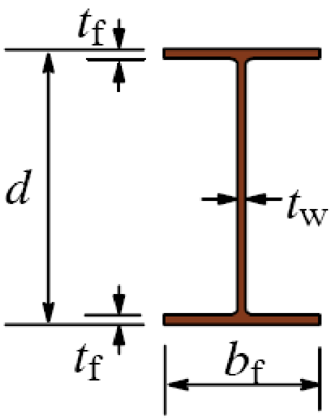
NOTE: All references to Clauses or Tables in the Examples are references to Clauses/Tables in AS 4100 “Steel Structures” unless noted otherwise

Example 1 - Determine the Design Section Moment Capacity of a Universal Beam with Full Lateral Restraint

Determine the design section moment capacity of a 460UB82.1 beam of Grade 300PLUS steel which has full lateral restraint.



Geometric Data
From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition

d = 460 mm
b_f = 191 mm
t_f = 16 mm
t_w = 9.9mm
S_x = 1840 x 10^3 mm^3
Z_x = 1610 x 10^3 mm^3
Flange f_{yf} = 300 MPa
Web f_{yw} = 320 MPa

Choose f_y = 300 MPa

1. Calculate the flange slenderness λ_{ef} (Clause 5.2.2)

$$\lambda_{ef} = \left(\frac{b}{t}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{191 - 9.9}{2 \text{ x} 16}\right)\sqrt{\frac{300}{250}}$$

$= 6.20 < \lambda_{ep} = 9$ (Table 5.2, uniform compression, HR)

2. Calculate the web slenderness λ_{ew} (Clause 5.2.2)

$$\lambda_{ew} = \left(\frac{b}{t}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{460 - 2 \text{ x} 16}{9.9}\right)\sqrt{\frac{320}{250}}$$

$= 49.0 < \lambda_{ep} = 82$ (Table 5.2)

$\therefore$ Section is compact.

$$Z_{ex} = [S_x, 1.5Z_x]_{min}$$
$$Z_{ex} = 1840 \times 10^3 mm^3$$

3. Determine the design section moment capacity M_x^*

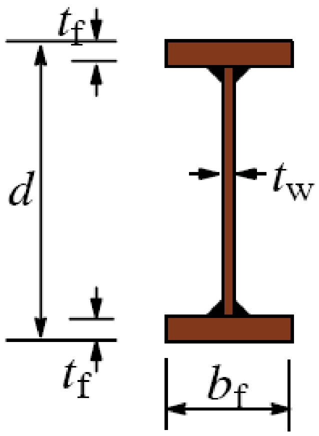
$\phi M_{sx} = \phi \times f_{yf} \times Z_{ex} = (0.9 \text{ x } 300 \text{ x } 1840 \text{ x } 10^3)/10^6$ (Clause 5.1, 5.2.1)

= 497 kNm

4. Design Capacity Tables, Student Edition 2009, Table 5.3, ϕM_{sx} = 496 kNm

Example 2 - Determine the Design Member Moment Capacity of a Laterally Unrestrained Universal Beam

Determine the maximum design uniform bending moment M* that a 530UB82.0 of Grade 300PLUS steel can sustain. The beam is simply supported over a span of 5.0 m, and is fully restrained at the supports against lateral deflection and twist rotation and restrained against lateral rotation. Restraint arrangement FF, load on top flange



Geometric Data
From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition

d = 528 mm
b_f = 209 mm
t_f = 13.2mm
t_w = 9.6 mm
Z_x = 1810 x 10^3 mm^3
S_x = 2070 x 10^3 mm^3
I_y = 20.1 x 10^6 mm^4
J = 526 x10^3 mm^4
S_x = 1330 x 10^9 mm^3
I_w = 1330 x 10^9 mm^6
E = 200 x 10^3 MPa
G = 80 x 10^3 MPa
Flange f_{yf} = 300 MPa
Web f_{yw} = 320 MPa

Choose f_y = 300 MPa

1. Calculate the flange slenderness λ_{ef} (Clause 5.2.2)

$$\lambda_{ef} = \left(\frac{b}{t}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{209-9.6}{2\,x13.2}\right)\sqrt{\frac{300}{250}} = 8.27 < \lambda_{ep} = 9 \quad \text{(Table 5.2)}$$

∴ Section is compact.

$$Z_{ex} = [S_x, 1.5Z_x]_{min} = 2070 \times 10^3 mm^3$$

2. Determine the nominal section moment capacity M_s (Clause 5.2.1)

$$M_s = f_y \text{ x } Z_{ex} = 300 \text{ x } 2070 \text{ x } 10^3 / 10^6 = 621\,kNm$$

3. Determine the effective length l_e (Clause 5.6.3) FF restraints

k_t = 1.0, k_l = 1.0, k_r = 1.0 (Tables 5.6.3(1), (2), (3))

$l_e = k_t\, k_l\, k_r\, l$ = 1.0 × 1.0 × 1.0 × 5000 = 5000 mm

4. Determine the reference buckling moment M_o (Clause 5.6.1.1)

$$M_o = \sqrt{\frac{\pi^2 EI_y}{l_e^2}\left[GJ + \left(\frac{\pi^2 EI_w}{l_e^2}\right)\right]}$$

$$= \left[\sqrt{\frac{\pi^2 \times 200000 \times 20.1 \times 10^6}{5000^2}\left[80000 \times 526 \times 10^3 + \left(\frac{\pi^2 \times 200000 \times 1330 \times 10^9}{5000^2}\right)\right]}\right] / 10^6$$

= 483.2 kNm

5. Determine the slenderness reduction factor α_s (Clause 5.6.1.1)

$$\alpha_s = 0.6\left[\sqrt{\left(\frac{M_s}{M_{oa}}\right)^2 + 3} - \frac{M_s}{M_{oa}}\right]$$

$$= 0.6\left[\sqrt{\left(\frac{621}{483.2}\right)^2 + 3} - \frac{621}{483.2}\right] \qquad \text{(Equation 5.6.1.1(2))}$$

= 0.523

From Clause 5.6.1.1(a)(i), $\alpha_m = 1.0$

6. Determine the design member moment capacity (Clauses 5.1, 5.6.1.1) that the beam can carry

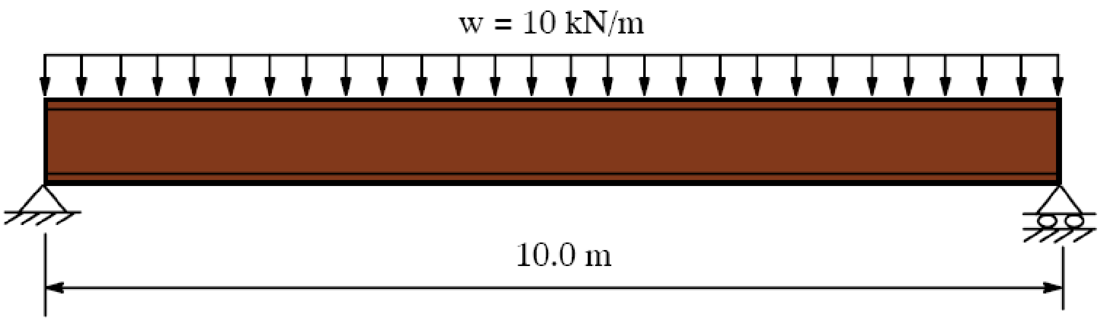
$\phi M_b = \phi \alpha_m \alpha_s M_s$ = 0.9 x 1.0 x 0.523 x 621

= 292 kNm

7. Design Capacity Tables, Student Edition, Table 5.5, ϕM_{bx} = 292 kNm

Example 3 - Design a Laterally Unrestrained Beam

A simply supported beam with a span of 10.0 metres has a uniformly distributed live load of 10 kN/m on the top flange. The beam is restrained against lateral displacement and twist at both ends, and is free to rotate in plan. Design a suitable UB section of Grade 300PLUS steel.



1. Assume f_y = 280 MPa, and that the section is compact.

Allow 1.25 kN/m for beam self weight

a. Calculate the design bending moment

$$M^*_{DL} = 1.2 \times M_G = 1.2 \times \frac{1}{8} \times 1.25 \times 10^2 = 18.8 \text{ kNm}$$

$$M^*_{LL} = 1.5 * M_Q = 1.5 \times \frac{1}{8} \times 10 \times 10^2 = 187.5 \text{ kNm}$$

$$M^* = 18.8 + 187.5 = 206.3 \text{ kNm}$$

From Table 5.6.1 of AS 4100, α_m = 1.13 for uniform loading

Guess α_s = 0.2

b. Calculate the required member moment capacity ϕM_b (Clause 5.1, 5.6.1.1)

$$M^* \leq \phi M_b = \phi \alpha_m \alpha_s M_s$$

$$M_b \geq \frac{M^*}{\phi \alpha_m \alpha_s} = \frac{206.3}{0.9 \times 1.13 \times 0.2} = 1014 \text{ kNm}$$

c. Calculate the required effective section modulus Z_{ex} (Clause 5.2.1)

$$Z_{ex} \geq \frac{M_b}{f_y} = \frac{1014 \times 10^6}{280} = 3620 \times 10^3 \text{ mm}^3$$

d. Determine the effective length l_e (Clause 5.6.3)

k_t = 1.0 (FF), k_l = 1.4 (load on top flange), k_r = 1.0 (PP)
$l_e = k_t\, k_l\, k_r\, l$ = 1.0 × 1.4 × 1.0 × 10000 = 14000 mm

2. Try using a 610 UB125 (S_x = 3680 x 10^3 mm^3). From *OneSteel "Hot Rolled and Structural Steel Products" 4th Edition*,

b_f = 229 mm
t_f = 19.6 mm
t_w = 11.9 mm
S_x = 3680 x 10^3 mm^3
I_y = 39.3 x 10^6 mm^4
J = 1560 x 10^3 mm^4
I_w = 3450 x 10^9 mm^6

f_{yf} = 280 MPa
f_{yw} = 300 MPa

Choose f_y = 280 MPa

a. Calculate the flange slenderness λ_{ef} (Clause 5.2.2)

$$\lambda_{ef} = \left(\frac{b}{t}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{229 - 11.9}{2x19.6}\right)\sqrt{\frac{280}{250}} = 5.86 < \lambda_{ep} = 9$$

$\therefore$ Section is compact

$Z_{ex} = S_x$ = 3680 x 10^3 mm^3

b. Determine the nominal section moment capacity M_s (Clause 5.2.1)

$$M_s = f_y \text{ x } Z_{ex} = \left(280 \text{ x } 3680 \text{ x } 10^3\right) / 10^6 = 1030 \text{ kNm}$$

Using the Design Capacity Tables, Student Edition 2009, Table 5.3,

ϕM_{sx} = 927 kNm,

$\therefore$ M_{sx} = 927 / 0.9 = 1030 kNm

c. Determine the reference buckling moment M_o (Equation 5.6.1.1(3))

$$M_o = \sqrt{\frac{\pi^2 EI_y}{l_e^2}\left[GJ + \left(\frac{\pi^2 EI_w}{l_e^2}\right)\right]}$$

$$= \sqrt{\frac{\pi^2 \text{ x } 200000 \text{ x } 39.3 \text{ x } 10^6}{14000^2}\left[80000 \text{ x } 1560 \text{ x } 10^3 + \left(\frac{\pi^2 \text{ x } 200000 \text{ x } 3450 \text{ x } 10^9}{14000^2}\right)\right]} / 10^6$$

= 251.3 kNm

d. Determine the slenderness reduction factor α_s (Equation 5.6.1.1(2))

$$\alpha_s = 0.6\left[\sqrt{\left(\frac{M_s}{M_{oa}}\right)^2 + 3} - \frac{M_s}{M_{oa}}\right]$$

$$= 0.6\left[\sqrt{\left(\frac{1030}{251.3}\right)^2 + 3} - \frac{1030}{251.3}\right]$$

= 0.210

e. Check the design member moment capacity (Clauses 5.1, 5.6.1.1)

$\phi M_b = \phi \alpha_m \alpha_s M_s = 0.9 \times 1.13 \times 0.210 \times 1030$

= 221 kNm > M* = 206.3 kNm SATISFACTORY

Adopt a 610UB125 – actual dead load = 125 kgs/m (1.23 kN/m)

Allowed 1.25 kN/m in design SATISFACTORY

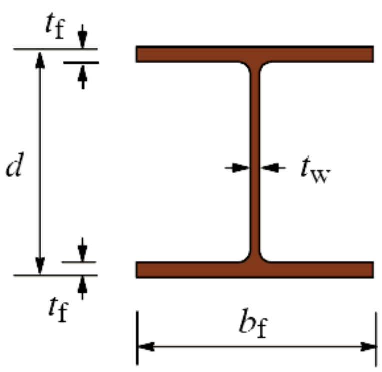
Note: Design Capacity Tables, Student Edition 2009, Table 5.5,

ϕM_{bx} = 195 kNm for α_b = 1.0, l_e = 14m

Hence, ϕM_{bx} = 220 kNm for α_b = 1.13, l_e = 14m

Example 4 - Determine the Design Section Capacity for a Concentrically Loaded Member in Compression

Determine the design section capacity of a concentrically loaded 250UC72.9 compression member of Grade 300PLUS steel if the effective length about each axis is 4.0 metres.



<u>Geometric Data</u>
From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition

d = 254 mm
b_f = 254 mm
t_f = 14.2 mm
t_w = 8.6 mm
A = 9320 mm^2
r_x = 111 mm
r_y = 64.5 mm

$l_{ex} = l_{ey}$ = 4000 mm

1. Determine the form factor k_f (Clause 6.2.2)

From *OneSteel "Hot Rolled and Structural Steel Products" 4th Edition*,

f_{yf} = 300 MPa
f_{yw} = 320 MPa

Choose f_y = 300 MPa

k_f = 1.0

2. Determine the nominal section capacity N_s (Clause 6.2.1)

$N_s = k_f A_n f_y = 1.0 \times 9320 \times 300 / 10^3 = 2796$ kN

3. Calculate the modified member slenderness λ_n (Clause 6.3.3)

$$\lambda_{nx} = \left(\frac{l_{ex}}{r_x}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \left(\frac{4000}{111}\right)\sqrt{1.0}\sqrt{\frac{300}{250}} = 39.5$$

$$\lambda_{nx} = \left(\frac{l_{ey}}{r_y}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \left(\frac{4000}{64.5}\right)\sqrt{1.0}\sqrt{\frac{300}{250}} = 67.9$$

$\therefore\ \lambda_n = 67.9$ since y-axis buckling controls.

4. Calculate the compression member section constant α_b and member slenderness reduction factor α_c, using linear interpolation.

For hot rolled UC sections, $\alpha_b = 0$ (Table 6.3.3(1))

$\alpha_c = 0.779 - (0.779 - 0.748) \times \dfrac{(67.9 - 65)}{(70 - 65)} = 0.761$ (Table 6.3.3(3))

5. Determine the design section capacity in compression (Clause 6.3.3)

$\phi N_c = \phi \alpha_c N_s = 0.9 \times 0.761 \times 2796 = 1915$ kN

The design axial capacity can also be determined using the Design Capacity Tables, Student Edition 2009.

From Table 6.4, $\phi N_{cy} = 1920$ kN

Example 5 - Design a Universal Column Compression Member

A concentrically loaded compression member of Grade 300PLUS steel is restrained so that its effective lengths are l_{ex} = 10.0 m and l_{ey} = 5.0 m. If the nominal dead and live axial loads are 600 kN and 1200 kN respectively, design a suitable UC section.

1. Calculate the design axial force N* (AS1170.0) for the permanent and imposed action load combination

N* = 1.2G + 1.5Q = 1.2 × 600 + 1.5 × 1200 = 2520 kN

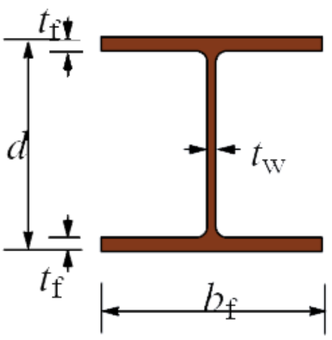
2. Guess a trial section

Guess f_y = 280MPa, k_f = 1.0, α_b = 0, λ_n = 80
From Table 6.3.3(3), α_c = 0.681

$$N^* \le \phi N_c = \phi\alpha_c N_c = \phi\alpha_c k_f A_n f_y$$

$$\therefore A_n \ge \frac{N^*}{\phi\alpha_c k_f f_y} = \frac{2520 \times 10^3}{0.9 \times 0.681 \times 1.0 \times 280} = 14684\,\text{mm}^2$$

3. Try using a 310 UC118



Geometric Data
From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition

d = 315 mm
b_f = 307 mm
t_f = 18.7 mm
t_w = 11.9 mm
A = 15000 mm^2
r_x = 136 mm
r_y = 77.5 mm

a. Determine the form factor k_f (Section 6.2.2)

From *OneSteel "Hot Rolled and Structural Steel Products" 4th Edition*,

f_y = 280 MPa
k_f = 1.0

b. Calculate the modified member slenderness λ_n (Clause 6.3.3)

$$\lambda_{nx} = \left(\frac{l_{ex}}{r_x}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \left(\frac{10000}{136}\right)\sqrt{1.0}\sqrt{\frac{280}{250}} = 77.8$$

$$\lambda_{ny} = \left(\frac{l_{ey}}{r_y}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}} = \left(\frac{5000}{77.5}\right)\sqrt{1.0}\sqrt{\frac{280}{250}} = 68.3$$

$\therefore\ \lambda_n = 77.8$ since x-axis buckling controls.

c. Calculate the compression member section constant α_b and member slenderness reduction factor α_c

From Table 6.3.3(1), $\alpha_b = 0$

From Table 6.3.3(3) using linear interpolation,

$$\alpha_c = 0.715 - \frac{0.715 - 0.681}{(80 - 75)} \text{ x } (77.8 - 75) = 0.696$$

d. Determine the design member compression capacity ϕN_c (Clause 6.3.3)

$\phi N_{cx} = \phi \alpha_c k_f A_n f_y = 0.9 \times 0.696 \times 1.0 \times 15000 \times 280 / 10^3$

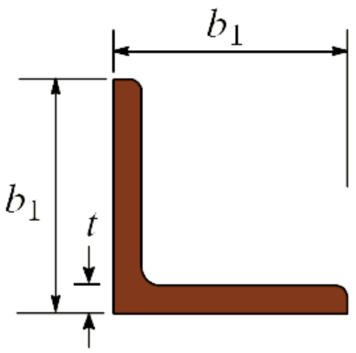
$= 2631 > N^* = 2550$ kN SATISFACTORY

Hence adopt a 310 UC118 section.

e. Using the Design Capacity Tables, Student Edition 2009, Table 6.3, $\phi N_{cx} = 2630$ kN

Example 6 - Determine the Design Capacity for an Eccentrically Connected Single Angle Member in Tension

Determine the design capacity of a tension member consisting of a single 100x100x10EA angle of Grade 300PLUS steel which is connected eccentrically through one leg by a single line of 16 mm bolts (18 mm holes).



Geometric Data *From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition* b_1 = 100 mm t = 9.5 mm A = 1810 mm^2 k_t = 0.85 (Table 7.3.2, configuration (i))

From *OneSteel "Hot Rolled and Structural Steel Products" 4th Edition*,

f_y = 320 MPa
f_u = 440 MPa

1. Calculate the net area of the cross-section A_n (Clause 7.2)

A_g = 1810 mm^2
A_n = 1810 − 1 × (16 + 2) × 9.5 = 1639 mm^2

(NB: holes are 2mm larger than the bolt diameter)

2. Determine the nominal section capacity N_t (Clause 7.2)

Member yield $N_t = A_g f_y = 1810 \times 320 / 10^3 = 579$ kN

Section fracture $N_t = 0.85 k_t A_n f_u = 0.85 \times 0.85 \times 1639 \times 440 / 10^3$

$= 521$ kN

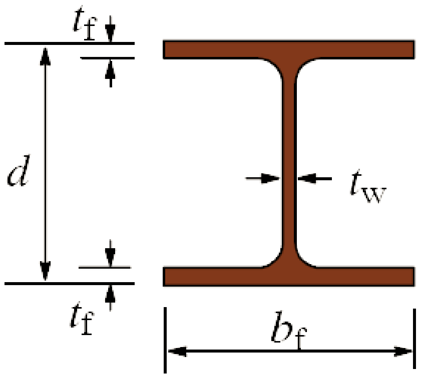
$\therefore$ N_t = 521 kN, section fracture governs

3. Determine the maximum design axial tension force N* that the angle can carry (Clause 7.1)

$N^* \leq \phi N_t = 0.9 \times 521 = 469$ kN

Example 7 - Determine the Design Moment Capacity for the Major Axis for a Section also subject to Axial Compression

Determine the design major axis section moment capacity of a 310UC118 of Grade 300PLUS steel which has a design axial compression force of N*= 180 kN.



Geometric Data
From OneSteel "Hot Rolled and Structural Steel Products" 4th Edition

d = 315 mm
b_f = 307 mm
t_f = 18.7 mm
t_w = 11.9 mm
A = 15000 mm^2
S_x = 1960 x 10^3 mm^3
Z_x = 1760 x 10^3 mm^3

From *OneSteel "Hot Rolled and Structural Steel Products" 4th Edition*,

f_{yf} = 280 MPa
f_{yw} = 300 MPa

Choose f_y = 280 MPa

1. Calculate the flange slenderness λ_{ef} (Clause 5.2.2)

$$\lambda_{ef} = \left(\frac{b}{t}\right)\sqrt{\frac{f_y}{250}} = \left(\frac{307 - 11.9}{2 \times 18.7}\right)\sqrt{\frac{280}{250}} = 8.35 < \lambda_{epf} = 9$$

∴ Section is compact.

2. Calculate the nominal section moment capacity M_{sx} (Clause 5.2.1)

$S_x = 1960 \times 10^3$ mm^3

$< 1.5Z_x = 1.5 \times 1760 \times 10^3 = 2640 \times 10^3$ mm^3 (Clause 5.2.3)

$\therefore Z_e = S_x$

$M_{sx} = f_y \times Z_e = 280 \times 1960 \times 10^3 / 10^6 = 549$ kNm (Clause 5.2.1)

Using the Design Capacity Tables, Student Edition 2009, Table 5.4 and 8.2

$\phi M_{sx} = 494$ kNm

$\therefore M_{sx} = 494 / 0.9 = 549$ kNm

3. Determine the nominal section compression capacity N_s (Clause 6.2.1)

$\lambda_{ef} = 8.35 < \lambda_{eyf} = 16$ (Table 6.2.4)

$\lambda_{ew} = 24.69 < \lambda_{eyw} = 45$ (Table 6.2.4)

$\therefore k_f = 1.0$ (Clause 6.2.2)

$N_s = k_f A_n f_y = 1.0 \times 15000 \times 280 / 10^3 = 4200$ kN (Clause 6.2.1)

Using the Design Capacity Tables, Student Edition 2009, Table 8.2,

$\phi N_s = 3780$ kN

$\therefore N_s = 3780 / 0.9 = 4200$ kN

4. Determine the design major axis section capacity reduced by axial force (Clause 8.3.2)

(a) Method 1

$$M_{rx} = M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right) = 549 \text{ x } \left(1 - \frac{180}{0.9 \text{ x } 4200}\right)$$

$= 523$ kNm $< M_{sx} = 549$ kNm

(b) Method 2 (for compact doubly symmetric I-sections): (Clause 8.3.2)

1. Determine the section moment capacity reduced by axial force M_{rx}

$$M_{rx} = 1.18 M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right) = 1.18 \text{ x } 549 \text{ x } \left(1 - \frac{180}{0.9 \text{ x } 4200}\right)$$

$= 617$ kNm $> M_{sx} = 549$ kNm

$\therefore M_{rx} = 549$ kNm (Clause 8.3.2)

Under Clause 8.3.2, either calculated values of M_{rx} can be adopted, however adopt the larger value

$\therefore M_{rx} = 549$ kNm.

2. Determine the design major axis reduced section capacity ϕM_{rx} (Clause 8.3.2)

$$\phi M_{rx} = 0.9 \text{ x } 549 = 494 \text{ kNm}$$

3. Using the Design Capacity Tables, Student Edition 2009, Table 8.2

$\phi M_{rx\,(comp)} = 582\,(1\text{-}n)$ $= 582 \text{ x } (1 - N^* / \phi N_s)$
$= 582 \text{ x } (1 - 180 / 3780)$
$= 554$
But must $\leq \phi M_{sx} = 494$ kNm

$\therefore \phi M_{rx\,(comp)} = 494$ kNm