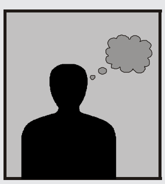
2020

MADE EASY WORKBOOK



Detailed Explanations of Try Yourself *Questions*

Civil Engineering
Design of Steel Structures

MADE EASY
MADE EASY Publications

1 Structural Fasteners

Detailed Explanation of Try Yourself Questions

T1 : Solution

For Fe 410 grade of steel: f_u = 410 MPa
For bolts of grade 4.6: f_{ub} = 400 MPa
γ_{mb} = partial safety factor for the material of bolt = 1.25

A_{nb} = net tensile stress are of 20 mm diameter bolt = $0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{mm}^2$

(a) The bolts will be in single shear and bearing.
Diameter of bolt, d = 20 mm
The strength of bolt in single shear,

$$V_{sb} = A_{nb}\frac{f_{ub}}{\sqrt{3}\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26 \text{ kN}$$

The strength of bolt in bearing,

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}} \quad (f_u \text{ will be lesser of } f_u \text{ and } f_{ub})$$

For 20 mm diameter bolt,
Diameter of hole, $d_0 = d + 2 = 22$ mm
Edge distance, $e = d_0 \times 1.5 = 33$ mm
Pitch = $2.5 \times d = 50$ mm

k_b is least of $\frac{e}{3d_o} = \frac{33}{3 \times 22} = 0.5$; $\frac{p}{3d_o} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.5$; $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$; and 1.0.

Hence, $k_b = 0.5$

$$\Rightarrow \quad V_{pb} = 2.5 \times 0.5 \times 20 \times 12 \times \frac{400}{1.25} = 96.0 \text{ kN}$$

The strength of the bolt will be minimum of the strength in shear and bearing and is 45.26 kN.

(b) The strength of bolt in single shear = 45.26 kN

The strength of bolt in bearing,

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

t is minimum of combined thickness of cover plates and thickness of main plate = 10 mm

$$\therefore \quad V_{pb} = 2.5 \times 0.5 \times 20 \times 10 \times \frac{410}{1.25} \times 10^{-3} = 82.0 \text{ kN}$$

The strength of the bolt will be minimum of the strength in shear and bearing and is 45.26 kN.

(c) The strength of bolt in double shear.

$$V_{sb} = 2 \times A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 2 \times 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 90.52 \text{ kN}$$

The strength of the bolt in bearing,

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

t is minimum of combined thickness of cover plates and thickness of main plate = 12 mm

$$\therefore \quad V_{pb} = 2.5 \times 0.5 \times 20 \times 12 \times \frac{410}{1.25} \times 10^{-3} = 98.4 \text{ kN}$$

The strength of the bolt will be minimum of the stength in shear and bearing and is 90.52 kN.

T2 : Solution

For Fe 410 grade steel, f_y = 250 MPa.

(a) In case of single-*V* groove weld, incomplete penetration of weld takes place; therefore as per the specifications,

Throat thickness, $t_e = \frac{5}{8} t = \frac{5}{8} \times 14 = 8.75$ mm

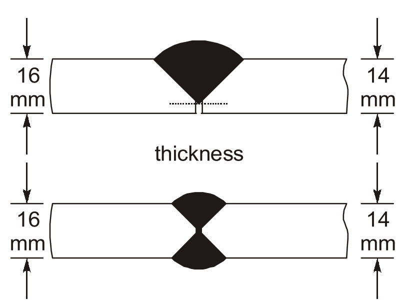
For shop weld: partial safety factor for material = γ_{mw} = 1.25

Effective length of the weld, L_w = 175 mm

Strength of the weld, $T_{dw} = L_w\, t_e \frac{f_y}{\gamma_{mw}} = 175 \times 8.75 \times \frac{250}{1.25} \times 10^{-3} = 306.25 \text{ kN} < 430 \text{ kN}$

Hence joint is not safe.

(b) In the case of double-*V* groove weld, complete penetration of the weld takes place;



Throat thickness, t_e = thickness of thinner plate = 14 mm

$$\text{Strength of the weld, } T_{dw} = L_w\, t_e \frac{f_y}{\gamma_{mw}} = 175 \times 14 \times \frac{250}{1.25} \times 10^{-3}$$

= 490 kN > 430 kN which is adequate and safe.

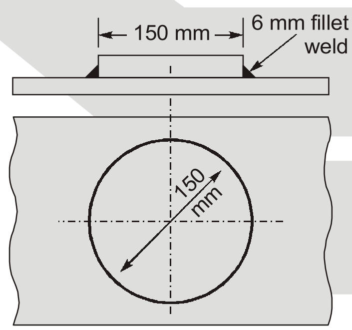
T3 : Solution

For Fe 410 grade steel, f_u = 410 MPa
For shop welding: partial safety factor for material γ_{mw} = 1.25
Size of weld: S = 6 mm
Effective throat thickness = KS = 0.7 × 6 = 4.2 mm

$$\text{Strength of weld per mm length} = 1 \times t_t \times \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = 1 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25} = 795.36 \text{ N/mm}$$

$$\text{Total length of the weld provided} = \pi\, d = \pi \times 150 = 471.24 \text{ mm}$$

$$\text{Greatest twisting moment} = 795.36 \times 471.24 \times \frac{150}{2} = 28110408.48 \text{ Nmm} = 28.11 \text{ kNm}$$



T4 : Solution

For Fe 410 grade of steel, f_u = 410 MPa
For bolts of grade 4.6, f_{ub} = 400 MPa
Partial safety factor for the material of bolt, γ_{mb} = 1.25

$$A_{nb} = \text{stress area of 20 mm diameter bolt} = 0.78 \times \frac{\pi}{4} d^2 = 245 \text{ mm}^2$$

Given: diameter of bolt, d = 20 mm; pitch, p = 80 mm; edge distance, e = 40 mm
For d = 20 mm, d_0 = 20 + 2 = 22 mm
Strength of the bolt in single shear,

$$V_{sb} = A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26 \text{ kN}$$

Strength of the bolt in bearing,

$$V_{pb} = 2.5\, k_b d t \frac{f_u}{\gamma_{mb}}$$

Diameter of bolt hole, d_0 = 22 mm

k_b is least of $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606; \frac{p}{3d_0} - 0.25 = \frac{80}{3 \times 22} - 0.25 = 0.96, \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$; and 1.0.

Hence, $k_b = 0.606$

$$V_{pb} = 2.5 \times 0.606 \times 20 \times 9.1 \times \frac{410}{1.25} \times 10^{-3} = 90.44 \text{ kN}$$

Hence, strength of the bolt, $V_{sd} = 45.26$ kN.

Let, P_1 be the factored load.

Service load, $P = \frac{P_1}{\text{load factor}} = \frac{P_1}{1.50}$

The bolt which is stressed maximum is *A*.

Total number of bolts in the joint, $n = 10$

The direct force, $F_1 = \frac{P_1}{n} = \frac{P_1}{10}$

The force in the bolt due to torque, $F_2 = \frac{Pe_0 r_n}{\Sigma r^2}$

$$r_n = \sqrt{(80+80)^2 + \left(\frac{120}{2}\right)^2} = 170.88 \text{ mm}$$

$$\Sigma r^2 = 4 \times [(160^2 + 60^2) + (80^2 + 60^2)] + 2 \times 60^2 = 164000 \text{ mm}^2$$

$$F_2 = \frac{P_1 \times 200 \times 170.88}{164000} = 0.2084\, P_1$$

$$\cos\theta = \frac{60}{\sqrt{60^2 + 160^2}} = 0.3511$$

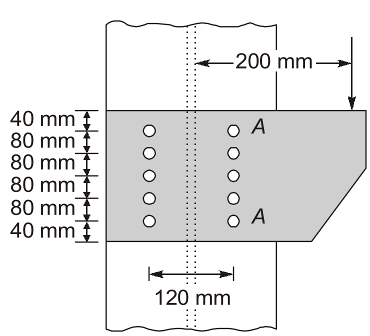
The resultant force on the bolt should be less than or equal to the strength of bolt.

$$45.26 \geq \sqrt{\left(\frac{P_1}{10}\right)^2 + (0.2084 P_1)^2 + 2 \times \frac{P_1}{10} \times 0.2084\, P_1 \times 0.3511}$$

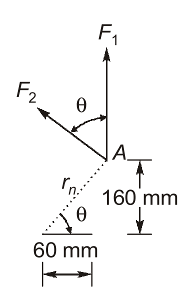
$\Rightarrow$ $0.2609\, P_1 \leq 45.26$

$\Rightarrow$ $P_1 = 173.48$ kN

The service load, $P = \frac{P_1}{\text{load factor}} = \frac{173.48}{1.5} = 115.65$ kN.

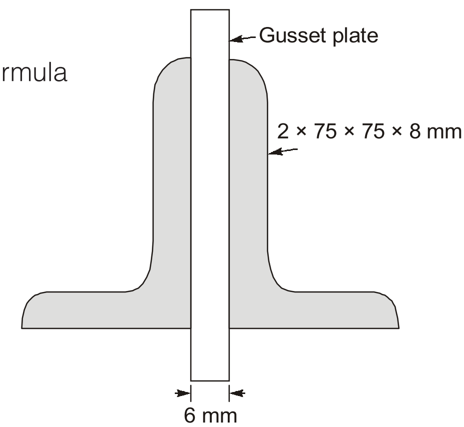


(a)



(b)

T5 : Solution



For angles of 8 mm thickness, diameter of rivets using Unwin's formula

$$d = 6.05\sqrt{t} = 6.05\sqrt{8}$$

$$d = 17.11 \text{ mm} \simeq 18 \text{ mm}$$

Let us provide 18 mm diameter rivets

Diameter of hole $d_h = d + 1.5 = 19.5$ mm

Shearing strength of one rivet in double shearing

$$F_s = 2\times\frac{\pi}{4}d_h^2\times f_s$$

$$= 2\times\frac{\pi}{4}\times 19.5^2\times 100\times 10^{-3}$$

$$= 59.73 \text{ kN}$$

Bearing strength of one rivet

$$f_b = \pi d_h \times t\times f_b$$

t is minimum of thickness of gusset plate and combined thickness of angles = 6 mm

$$f_b = \pi \times 19.5 \times 6 \times 300 \times 10^{-3}$$

$$f_b = 110.27 \text{ kN}$$

Rivet value is minimum of shearing and bearing strength of rivet i.e.,

$$R_V = 59.73 \text{ kN}$$

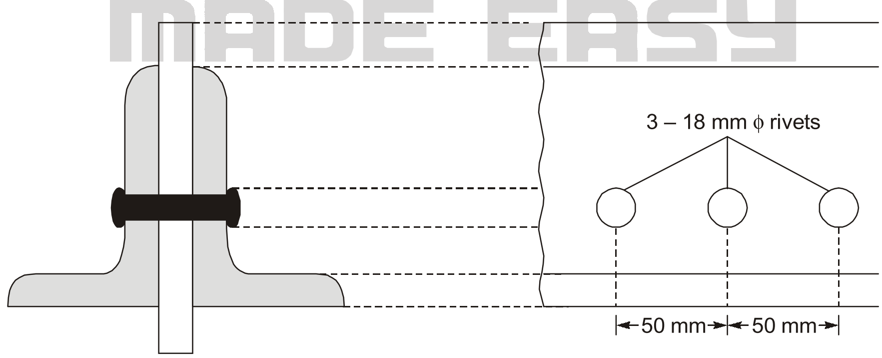
Number of rivets required $= \dfrac{\text{Load}}{\text{Rivet value}} = \dfrac{150}{59.73} = 2.5 \simeq 3$

Provide 3 rivets at guage distance from face of angle.

Pitch distance, $p = 2.5 \times$ nominal diameter of rivet

$= 2.5 \times 18 = 45$ mm

Adopt $p = 50$ mm



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2 Tension Members

T1 : Solution

For Fe 410 grade steel: $f_u = 410$ MPa, $f_y = 250$ MPa

Partial safety factors for material $\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

$A_{vg} = (2 \times 125) \times 10 = 2500 \text{ mm}^2$

$A_{vn} = (2 \times 125) \times 10 = 2500 \text{ mm}^2$

$A_{tg} = 250 \times 10 = 2500 \text{ mm}^2$

$A_{tn} = 250 \times 10 = 2500 \text{ mm}^2$

The block shear strength will be minimum of T_{db1} and T_{db2} as calculated below.

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9\,A_{tn} f_u}{\gamma_{m1}}$$

$$= \left[\frac{2500 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2500 \times 410}{1.25}\right] \times 10^{-3} = 1066.04$$

$$T_{db2} = \frac{0.9\,A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \left[\frac{0.9 \times 2500 \times 410}{\sqrt{3} \times 1.25} + \frac{2500 \times 250}{1.1}\right] \times 10^{-3} = 994.27 \text{ kN}$$

Hence, the block shear strength of the tension member is 994.27.

T2 : Solution

For Fe 410 grade of steel: $f_y = 250$ MPa

Diameter of bolt, $d = 18$ mm

Diameter of bolt hole, $d_o = 20$ mm

(a) Net area of connected leg $= \left(100 - 20 - \frac{10}{2}\right) \times 10 = 750 \text{ mm}^2$

Net area of outstanding leg $= \left(75 - \frac{10}{2}\right) \times 10 = 700 \text{ mm}^2$

Total net area = 750 + 700 = 1450 mm^2

Since only one leg of the angle is connected, the net area will be reduced depending upon the number of bolts used for making the connection.

$$A_n = \alpha A$$

where, α = 0.6 for one or two bolts
= 0.7 for three bolts
= 0.8 for four or more bolts or welds.

Hence, effective net area, $A_n = 0.7 \times 1450 = 1015 \text{ mm}^2$

(b) Net area of connected leg $= \left(100 - \dfrac{10}{2}\right) \times 10 = 950 \text{ mm}^2$

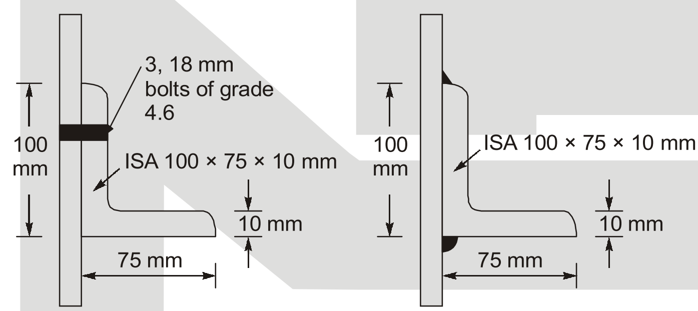
Net area of outstanding leg $= \left(75 - \dfrac{10}{2}\right) \times 10 = 700 \text{ mm}^2$

Total net area $= 950 + 700 = 1650 \text{ mm}^2$

Since only one leg of the angle is connected, the net area will be reduced

$\alpha = 0.8$ for welded joints.

Hence, effective net area, $A_n = 0.8 \times 1650 = 1320 \text{ mm}^2$.



(a) (b)

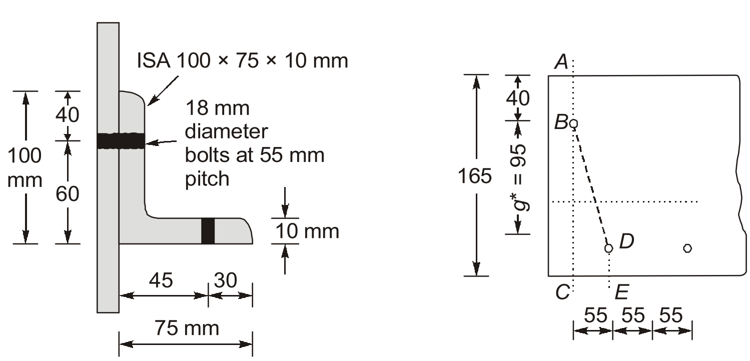
T3 : Solution

For Fe 410 grade of steel: $f_y = 250$ MPa

Diameter of bolt, $d = 18$ mm

Diameter of bolt hole, $d_o = 20$ mm

For calculating the net area of the angle section, the outstanding leg of the angle may be rotated and the total section may be visualized as a plate, as shown in respective figures.



(a) $g^* = g_1 + g_2 - t = 60 + 45 - 10 = 95$ mm

Net area along path $A - B - C$,

$$A_{n1} = (B - nd_o)t = (165 - 1 \times 20) \times 10 = 1450 \text{ mm}^2$$

Net area along path *A - B - D - E*,

$$A_{n2} = \left(B - nd_o + \frac{n'p^2}{4g}\right)t$$

$$n = 2,\ n' = 1,\ p = 55,\ g = 95$$

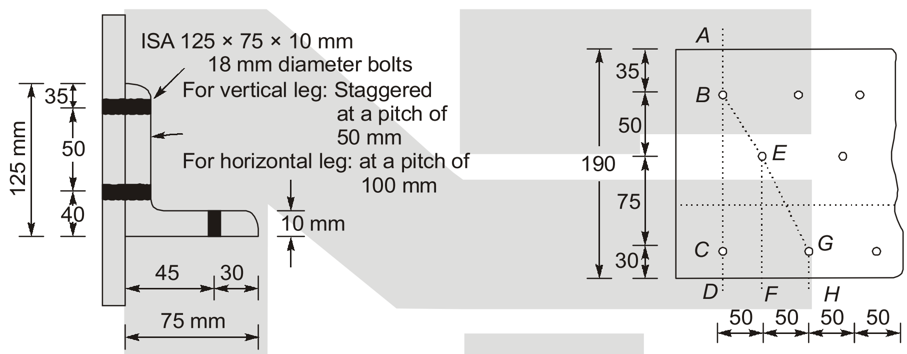
$$A_{n2} = \left(165 - 2\times 20 + \frac{1\times 55^2}{4\times 95}\right)\times 10$$

$$= 1329.60 \text{ mm}^2$$

The minimum of A_{n1} and A_{n2} will be the net area of the section. Since both the legs of the angle section are connected, no reduction is net area will be made.

Hence, effective net area = 1329.60 mm^2.

(b)



$$g^* = 40 + 45 - 10 = 75 \text{ mm}$$

Net area along path *A - B - C - D*,

$$A_{n1} = (B - nd)t = (190 - 2\times 20)\times 10 = 1500 \text{ mm}^2$$

Net area along path *A - B - E - F*,

$$n = 2,\ n' = 1,\ p = 50 \text{ mm},\ g = 50 \text{ mm}$$

$$A_{n2} = \left(B - nd + \frac{n'p^2}{4g}\right)t = \left(190 - 2\times 20 + \frac{1\times 50^2}{4\times 50}\right)\times 10 = 1625 \text{ mm}^2$$

Net area along path *A - B - E - G - H*,

$$n = 3,\ n'_1 = n'_2 = 1,\ p_1 = p_2 = 50 \text{ mm},\ g_1 = 50 \text{ mm},\ g_2 = 75 \text{ mm}$$

$$A_{n3} = \left(B - nd + \frac{n'_1p_1^2}{4g_1} + \frac{n'_2p_2^2}{4g_2}\right)t$$

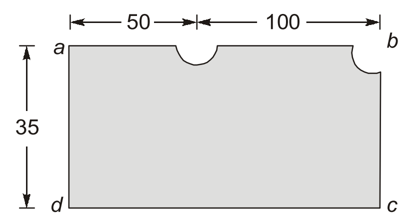
$$= \left(190 - 3\times 20 + \frac{1\times 50^2}{4\times 50} + \frac{1\times 50^2}{4\times 75}\right)\times 10 = 1508.33 \text{ mm}^2$$

The least of A_{n1}, A_{n2} and A_{n3} will be the net area of the section. Since both the legs of the angle section are connected, no reduction in net area will be made.

Hence, effective net area, A_n = 1500 mm^2

T4 : Solution

For Fe 410 grade steel: $f_u = 410$ MPa, $f_y = 250$ MPa



Partial safety factors for material: $\gamma_{m0} = 1.1$

$$\gamma_{m1} = 1.25$$

The shaded area shown in figure will shear out.

$$A_{vg} = (1 \times 100 + 50) \times 8 = 1200 \text{ mm}^2$$

$$A_{vn} = \left(1 \times 100 + 50 - \left(2 - \frac{1}{2}\right) \times 18\right) \times 8 = 984 \text{ mm}^2$$

$$A_{tg} = 35 \times 8 = 280 \text{ mm}^2$$

$$A_{tn} = \left(35 - \frac{1}{2} \times 18\right) \times 8 = 208 \text{ mm}^2$$

The block shear strength will be minimum of T_{db1} and T_{db2} as calculated below:

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \left[\frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 208 \times 410}{1.25}\right] \times 10^{-3}$$

$$= 218.86 \text{ kN}$$

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \left[\frac{0.9 \times 984 \times 410}{\sqrt{3} \times 1.25} + \frac{280 \times 250}{1.1}\right] \times 10^{-3}$$

$$= 231.34 \text{ kN}$$

Hence, the block shear strength of the tension number is 218.86 kN.

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3 Compression Members

T1 : Solution

I_z of ISHB 250 = 7983.9 × 10^4 mm^4 and A = 6971 mm^2, and t_f = 9.7 mm.

$$I_z \text{ for plates} = 2[I_a + A_p y_1^2]$$

$$= 2\left[\frac{300\times 20^3}{12} + 300\times 20\times(125+10)^2\right] = 21910\times 10^4 \text{ mm}^4$$

$$\text{Total } I_z = 7983.9\times 10^4 + 21910\times 10^4 = 29893.9\times 10^4 \text{ mm}^4$$

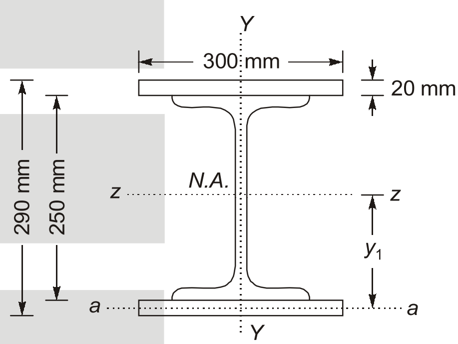
$$\text{Area of the built-up section} = 6971 + 2\times 300\times 20 = 18971 \text{ mm}^2$$

$$r_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{29893.9\times 10^4}{18971}} = 125.52 \text{ mm}$$

$$I_y \text{ of ISHB 250 @ 536.6 N/M} = 2011.7\times 10^4 \text{ mm}^4$$

$$I_y \text{ of plates} = 2\times\frac{20\times(300)^3}{12}$$

$$= 9000\times 10^4 \text{ mm}^4$$



Total I_y = 2011.7 × 10^4 + 9000 × 10^4 = 11,011.7 × 10^4 mm^4

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{11011.7\times 10^4}{18971}}$$

$$= 76.19 \text{ mm}$$

Hence, least radius of gyration will be minimum of r_z and r_y i.e., 76.19 mm.

T2 : Solution

For steel of grade Fe 410: f_y = 250 MPa
Partial safety factor for material: γ_{m0} = 1.10
The column ends are restrained in direction and position; K = 0.65.
The properties of ISHB 350 @ 710.2 N/m from IS Hand book No. 1 are as follows.

$$h = 350 \text{ mm}, b_f = 250 \text{ mm}, t_f = 11.6 \text{ mm}, t_w = 10.1 \text{ mm}$$

$$A = 9221 \text{ mm}^2, r_z = 146.5 \text{ mm}, r_y = 52.2 \text{ mm}$$

$$\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2$$

$$t_f = 11.6 \text{ mm} \le 40 \text{ mm}$$

From IS 800 : 2007 Table 10,

The buckling curve to be used along *ZZ*-axis will be curve *a*, and that about *YY* axis will be curve *b*.

Since $r_y < r_z$ the column will buckle about *YY*-axis and the design compressive strength will be governed by effective slenderness ratio λ_y.

Design compressive stress about *Y-Y* axis:

$$\text{Effective slenderness ratio} = \lambda_y = \frac{KL}{r_y} = \frac{0.65 \times 3.5 \times 10^3}{52.2} = 43.58$$

For buckling curve *b*, the imperfection factor $\alpha = 0.34$.

$$\text{Euler buckling stress } f_{cc} = \frac{\pi E}{\left(\frac{KL}{r_y}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{43.58^2} = 1039.33$$

The non-dimensional slenderness ratio,

$$\lambda_y = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{1039.33}} = 0.490$$

$$\phi_y = 0.5[1 + \alpha(\lambda_y - 0.2) + \lambda_y^2]$$

$$= 0.5[1 + 0.34 \times (0.490 - 0.2) + 0.490^2] = 0.669$$

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi_y + (\phi_y^2 - \lambda_y^2)^{0.5}} = \frac{250/1.1}{0.669 + (0.669^2 - 0.490^2)^{0.5}}$$

$$= 202.11 \text{ N/mm}^2$$

The design compressive strength, $P_d = A_e f_{cd} = 9221 \times 202.11 \times 10^{-3} = 1863.65$ kN

T3 : Solution

For steel of grade Fe 410: $f_y = 250$ MPa

The relevant properties of the angle sections used are as follows:

ISA 110 mm × 110 mm × 10 mm

$A = 2106$ mm^2, $I_z = I_y = 238.4 \times 10^4$ mm^4, $r_z = r_y = 33.6$ mm, $c_{xx} = c_{yy} = 30.8$ mm

ISA 130 mm × 130 mm × 15 mm

$A = 3681$ mm^2, $I_z = I_y = 574.6 \times 10^4$ mm^4, $r_z = r_y = 39.5$ mm, $c_{xx} = c_{yy} = 37.8$ mm

Let the distance of the centroidal axis *zz* from the face *aa* of the section be $\bar{y}$.

Taking the moment of the area about the axis *aa*,

$$(2106 + 3681 + 2106)\,\bar{y} = 2106 \times (30.8 + 15) + 3681 \times 37.8 + 2106 \times (180 - 30.8)$$

or $$7893\,\bar{y} = 96454.8 + 139141.8 + 314215.2$$

or $$\bar{y} = \frac{549811.8}{7893} = 69.658 \text{ mm}$$

Moment of inertial about *zz*-axis (I_z) can be found as follows:

$$I_z = I_{z1} + I_{z2} + I_{z3}$$

Moment of inertia of angle section 1 about centroidal axis *zz*,

$$I_{z1} = 238.4 \times 10^4 + 2106 \times (69.658 - 30.8 - 15)^2$$
$$= 358.27 \times 10^4 \text{ mm}^4$$

Moment of inertia of angle section 2 about centroidal axis *zz*,

$$I_{z2} = 574.6 \times 10^4 + 3681 \times (69.658 - 37.8)^2$$
$$= 948.19 \times 10^4 \text{ mm}^4$$

Moment of inertia of angle section 3 about centroidal axis *zz*,

$$I_{z3} = 238.4 \times 10^4 + 2106 \times (180 - 69.658 - 30.8)^2$$
$$= 1570.85 \times 10^4 \text{ mm}^4$$
$$I_z = (358.27 + 948.19 + 1570.85) \times 10^4$$
$$= 2877.31 \times 10^4 \text{ mm}^4$$

The two angle sections (1) and (3) are placed in such a way that the moment of inertia about *yy*-axis will be same as that about the *zz*-axis. Hence r_z and r_y will be equal.

Minimum radius of gyration, $r = r_z = \sqrt{\dfrac{I_z}{A}} = \sqrt{\dfrac{2877.31\times 10^4}{2106+3681+2106}} = 60.37$ mm

Effective length, $l = KL = 1.0 \times 4800 = 4800$ mm

Slenderness ratio, $\lambda = \dfrac{Kl}{r} = \dfrac{4800}{60.37} = 79.5$

For $\dfrac{KL}{r} = 79.5$, $f_y = 250$ MPa, and buckling curve ($\alpha = 0.49$).

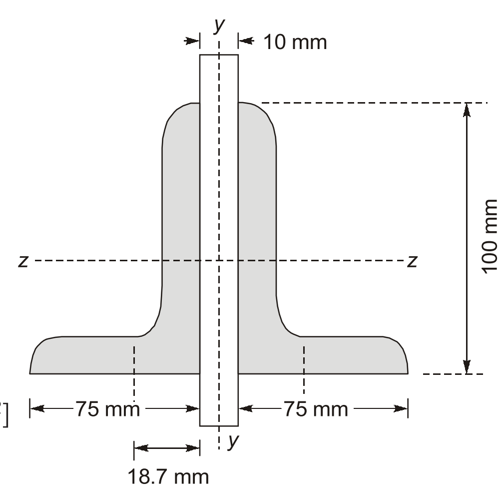
Euler buckling stress, $f_{cc} = \dfrac{\pi^2 E}{(KL/r)^2} = \dfrac{\pi^2 \times 2\times 10^5}{79.5^2} = 312.32$ N/mm²

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{312.32}} = 0.895$$
$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$
$$= 0.5[1 + 0.49(0.895 - 0.2) + 0.895^2] = 1.071$$

Design compressive stress, $f_{cd} = \dfrac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda^2}} = \dfrac{250/1.1}{1.071 + \sqrt{1.071^2 - 0.895^2}} = 136.9$ N/mm²

Design compressive strength, $P_d = A_e f_{cd} = 7893 \times 136.9 \times 10^{-3} = 1080$ KN

T4 : Solution



Case-I: Longer leg connected back to back of a gusset plate

Relevant properties of ISA 100 × 75 × 8 mm

$$A = 1336 \text{ mm}^2$$
$$I_{zz} = 131.6 \times 10^4 \text{ mm}^4$$
$$I_{yy} = 63.3 \times 10^4 \text{ mm}^4$$
$$c_y = 18.7 \text{ mm}$$

For stut, $A_e = 2 \times A = 2672$ mm²

$$I_z = 2 \times 131.6 \times 10^4$$
$$= 263.2 \times 10^4 \text{ mm}^4$$
$$I_y = 2 \times [63.3 \times 10^4 + 1336 \times (18.7 + 5)^2]$$
$$= 276.68 \times 10^4 \text{ mm}^4$$

∴ $I_{min} = I_z = 263.2 \times 10^4$ mm⁴

$$r_{min} = \sqrt{\frac{I_{min}}{A_e}} = \sqrt{\frac{263.2 \times 10^4}{2 \times 1336}} = 31.38 \text{ mm}$$

For sturt effective slendernes ratio,

$$\frac{kL}{r} = \frac{0.85 \times 3000}{31.38} = 81.26$$

Euler buckling stress, $f_{cc} = \frac{\pi^2 E}{(kL/r)^2} = \frac{\pi^2 \times 2 \times 10^5}{81.26^2} = 298.93 \text{ N/mm}^2$

Non-dimensional effective slenderness ratio,

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{298.93}} = 0.914$$

For built-up section, buckling curve = c

∴ Imperfection factor, $\alpha = 0.49$ (for buckling curve c)

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$

$$= 0.5[1 + 0.49(0.914 - 0.2) + 0.914^2] = 1.093$$

Design compressive stress,

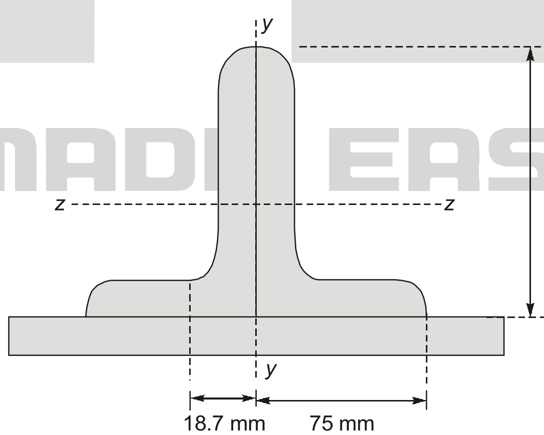
$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{250/1.1}{1.093 + \sqrt{1.093^2 - 0.914^2}} = 134.29 \text{ N/mm}^2$$

Load carrying capacity,

$$P = A_c \times f_{cd} = 2 \times 1336 \times 134.29$$

$$P = 358.83 \text{ kN}$$

Case-II: Shorter legs connected on same side of gusset plate,



$$I_z = 2 \times 131.6 \times 10^4 = 263.2 \times 10^4$$

$$I_y = 2 \times [63.3 \times 10^4 + 1336 \times 18.7^2] = 220.04 \times 10^4 \text{ mm}^4$$

∴ $I_{min} = I_y = 220.04 \times 10^4 \text{ mm}^4$

$$r_{min} = \sqrt{\frac{I_{min}}{A_e}} = \sqrt{\frac{220.04 \times 10^4}{2 \times 1336}} = 28.7 \text{ mm}$$

Effecgive slenderness ratio,

$$\frac{kL}{r} = \frac{0.85 \times 3000}{28.7} = 88.85 \text{ mm}$$

Euler buckling stress,

$$f_{cc} = \frac{\pi^2 E}{(kL/r)^2} = \frac{\pi^2 \times 2 \times 10^5}{88.85^2} = 250.04 \text{ N/mm}^2$$

Non-dimensional effecgive slenderness ratio,

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{250.4}} = 1$$

$$\phi = 0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$$
$$= 0.5\,[1 + 0.49\,(1 - 0.2) + 1^2]$$
$$= 1.196$$

Design compressive stress, $$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{250/1.1}{1.196 + \sqrt{1.196^2 - 1^2}} = 122.71 \text{ N/mm}^2$$

Load carrying capacity, $$P = A_e \times f_{cd}$$
$$P = 2 \times 1336 \times 122.71 = 327.89 \text{ kN}$$

Percentage change, $$= \left(\frac{358.83 - 327.89}{358.83}\right) \times 100 = 8.6\%$$

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