ESE | GATE | PSUs

CIVIL ENGINEERING

STEEL STRUCTURES

Text Book : Theory with worked out Examples and Practice Questions

3 Design of Steel Structures

(Solutions for Text Book Practice Questions)

02. Riveted Connections

01. Ans: 60%

Sol: $\eta = \dfrac{p-d}{p}\times 100$

$p_{min} = 2.5\phi$

$d = \phi + 1.5$

$= \dfrac{2.5\phi - \phi}{2.5\phi}\times 100$

$= \dfrac{1.5\phi}{2.5\phi}\times 100$

$\eta = 60\%$

05. Ans: (b)

Sol: ISA 50 × 50 × 6

$t = 5$ mm

$d = 16$ mm

$\sigma_{bt} = 250$ MPa

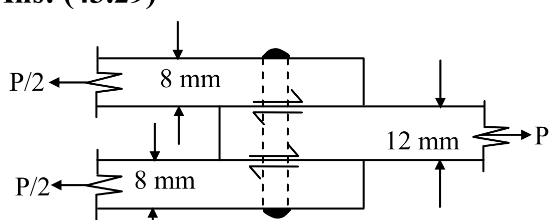
$P_b = d \times t \times \sigma_{bt}$

$= 16 \times 5 \times 250$

$= 20$ kN

06. Ans: (43.29)

Sol:



Nominal dia of rivet (ϕ) = 16 mm

For field Rivets permissible shear stress in Rivet .

$\tau_{vf} = 90$ MPa

Bearing stress in Rivet $\sigma_{pf} = 270$ MPa

Rivet value

R_v = lesser of P_s & P_b

$\phi = 16$ mm ($\phi \le 25$ mm)

$d = \phi + 1.5 = 17.5$ mm

$P_s = 2\times\frac{\pi}{4}(d)^2\times\tau_{vf}$

$= 2\times\left(\frac{\pi}{4}\right)(17.5)^2\times 90$

$P_s = 43.29 \times 10^3$ N

$= 43.29$ kN

Bearing strength of one rivet

$P_b = d \times t \times \sigma_{pf}$

t = thickness of main thinner plate (or) sum of cover plates thickness, which ever is minimum in cause of a butt joint

t (MP) = 12 mm

t (cp + cp) = 8 + 8 = 16 mm

$P_b = 17.5 \times 12 \times 270$

$P_b = 56.7$ kN

R_v = lesser of P_s & P_b

$\therefore\ R_v = 43.29$ kN

07. Ans: (d)

Sol: $$P_s = 2\times\frac{\pi}{4}(d)^2\times\tau_{vf} = 80\,kN$$

$$P_s = \frac{\pi}{4}d^2\times\tau_{vf} = 40\,kN$$

$P_s = 40$ kN, $P_b = 60$ kN, $P_{tr} = 70$ kN

$$n = \frac{P}{R_v} = \frac{P}{P_s} = \frac{200}{40} = 5$$

09. Ans: (d)

Sol: Minimum pitch of rivets in compression zone

$$\left.\begin{array}{l}12\,t\,mm\\200\,mm\end{array}\right\}\text{ whichever is minimum}$$

t = 10 mm

$$\left.\begin{array}{r}12t = 12\times10 = 120\text{ mm}\\200\,mm\end{array}\right\}\text{ whichever is minimum}$$

Pitch = 120 mm

Design of Simple bolted connection

03. Ans: (d)

Sol: $f_u = 400$ N/mm^2

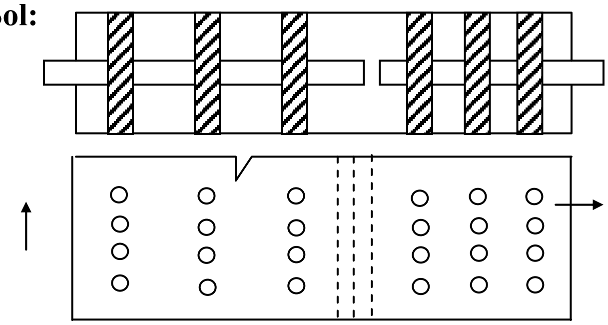
$f_y = 0.6\,f_u$

$= 0.6 \times 400$

$= 240$ N/mm^2

04. Ans: (d)

Sol:



$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$$

n_n (or) n_s

$= 3 \times 2 = 6$

05. Ans: (b)

Sol: P = 240 kN; V_{dsb} = 40 kN; V_{dpb} = 50 kN; T_{db} = 30 kN; V_{db} = lesser of V_{dsb}, V_{dpb}

$$n = \frac{P}{V_{db}} = \frac{240}{40}$$

n = 6 no's

06. Ans: (d)

Sol: Tensile force in each bolt due to $P_u\cos\theta$

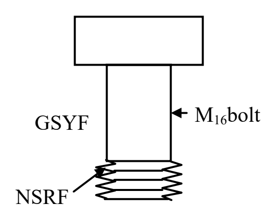
$$T_b = \frac{P_u\cos\theta}{n} = \frac{250}{6}\times\frac{4}{5} = 33.33\text{ kN}$$

Shear force in each bolt due to $P_u\sin\theta$

$$V_b = \frac{P_u\sin\theta}{n} = \frac{250}{6}\times\frac{3}{5} = 25\text{ kN}$$

07. Ans: (d)

Sol:



For M_{16} bolt d = 16 mm

For Grade 4.6; f_{ub} = 400 MPa

f_{yb} = 240 MPa

Design tensile strength of bolt T_{db} = ?

T_{db} is based on Gross section

$$T_{db_1} = \frac{A_{sb} f_{yb}}{\gamma_{mo}}$$

$$T_{db_1} = \frac{\frac{\pi}{4} \times (16)^2 \times 240}{1.1}$$

$$= 43.86 \times 10^3 \text{ N} = 43.86 \text{ kN}$$

T is based on net section rupture

$$T_{db_2} = \frac{0.9 A_{nb} f_{ub}}{\gamma_{mb}}$$

$$= \frac{0.9(0.78 \times \frac{\pi}{4} \times (16)^2 \times 400)}{1.25}$$

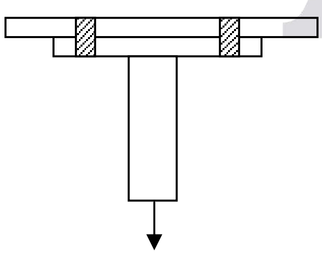
$$T_{db_2} = 45.166 \text{ kN}$$

T_{db} is lesser of T_{db_1} & T_{db_2}

$\therefore$ T_{db} = 43.86 kN

08. Ans: (c)

Sol: Hanger connection looks like this one



In hanger connections bolts experience only tensile stress. Then we clearly taken,

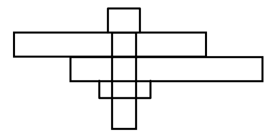
Strength of rivet is equal to the design strength of bolt in tension.

$\overline{P}_{db} = 45 \text{kN}$ = Design bolt strength

Number of bolts required = $\frac{180\text{kN}}{45\text{kN}}$ = 4 no's

09. Ans: (b)

Sol: Design strength values



V_{dpb} = 1,50,000 N

T_{dp} = 1,80,000 N

T_{sp} = 2,40,000 N

V_{dsb} = 1,60,000 N

$$\eta = \frac{\text{design strength of bolted connection } (V_{dc})}{\text{design strength of solid plate} (T_{sp})} \times 100$$

V_{dc} is lesser of $\begin{pmatrix} V_{dsp} & T_{db} \\ V_{dpb} & T_{dp} \end{pmatrix}$

$$\eta = \frac{1,50,000}{2,40,000} \times 100 = 62.5\%$$

η = 62.5%

Conventional Practice Solutions

01.

Sol Axial load P = 180 kN

Permissible shear stress τ_{vf} = 80 N/mm^2

Permissible bearing stress σ_{pf} = 250 N/mm^2

Size of angle ISA 90 × 90 × 8

Thickness of gusset plote t_g = 10 mm

Assume nominal diameter of rivet using unwin's equation

$\phi = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.1$ mm

$\simeq 20$ mm

Gross diameter of rivet $d = \phi + 1.5 = 20 + 1.5 = 21.5$

Rivet value = Lesser of P_s and P_b

Strength of one rivet in double shear

$$P_s = 2\times\frac{\pi}{4}\times d^2\times\tau_{vf}$$

$$= 2\times\frac{\pi}{4}\times(21.5)^2\times 80\times 10^3$$

$P_s = 58.08 \times 10^3$ kN

Bearing strength of one rivet

$P_b = d\times t \times\sigma_{pf}\times 10^3$ N

$P_b = 5370$ kN

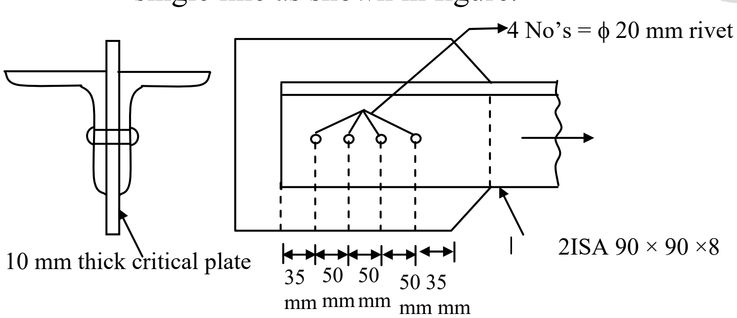
Rivet value (R_v) = Lesser of P_s and P_b

= 53.70 kN

Number of rivets are required to resist on axial load 'P' is

$$n = \frac{\text{Axial load}}{\text{Rivet value}} = \frac{P}{R_v} = \frac{180}{5370} = 3 \cong 4 \text{ No's}$$

Using 4 No's - ϕ 20 m rivets arranged in single line as shown in figure.



Minimum end distance

$e = e_{min} = 1.5\,d = 1.5 \times 21.5 = 35$ mm

Minimum pitch distance

$P = P_{min} = 2.5\,\phi = 2.5 \times 20 = 50$ m

02.

Sol: Factored (or) Design load,

$P = \gamma_f \times P_s = 1.5 \times 300 = 450$ kN

Assume shank diameter of Bolt

$d = 6.04\sqrt{t}$

$d = 6.04\sqrt{10} = 19$ mm ≈ 20 mm

Diameter of bolt hole, (for ordinary bolts)

$d_o = d + 2.0 = 22$ mm

Minimum pitch of bolt:

$P = P_{min} = 2.5d = 2.5 \times 20 = 50$ mm

Minimum end distance:

$e = e_{min} = 1.5d_o = 1.5 \times 22 = 33$ mm

Assume, designing a bolted connected as a double cover bolt connection as it will give higher efficiency.

Assume, thickness of each cover plate

$t = 0.625 \times$ thickness of main plate

$t = 0.625 \times 10 = 6.25$ mm

$t = 6$ mm

(Round mostly to higher value for safety)

Number of bolts required on each side of butt connection.

$$n = \frac{\text{Design load}}{\text{Design strength of one bolt}} = \frac{P}{V_{db}}$$

Where, V_{db} = Minimum of V_{dsb} & V_{dpb}

Design shear strength of one bolt

(V_{dsb}):

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}\left[n_n A_{nb} + A_{sb}.n_s\right]$$

Number of shear plates intersecting a rivet

$= n_n + n_s = 2$

Assume, shear area intercepts thread portion is, $n_s = 0$

$\therefore\ n_n = 2$

$$V_{dsb} = \frac{400}{\sqrt{3}\times 1.25}\left[1\times 2\times 0.78\times \frac{\pi}{4}(20)^2 + 0\right]$$

$V_{dsb} = 90.54$ kN

Design bearing strength of one bolt (V_{dpb}):

$$V_{dpb} = 2.5dt\,f_{ub}\frac{K_b}{\gamma_{mb}}$$

K_b is bearing factor lesser of

(i) $\dfrac{e}{3d_o} = \dfrac{33}{3\times 22} = 0.50$

(ii) $\dfrac{P}{3d_o} - 0.25 = \dfrac{50}{3\times 22} - 0.25 = 0.51$

(iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97$

(iv) 1.0

$\therefore\ K_b = 0.51$

$$V_{dpb} = 2.5 \times 20 \times 10 \times \frac{400\times 0.5}{1.25}$$

$V_{dpb} = 80$ kN

V_{db} = minimum of V_{dsb} and V_{dpb}

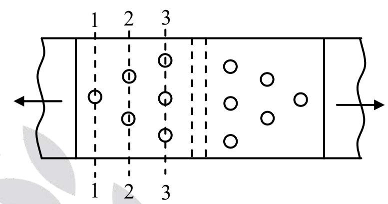
$V_{db} = 80$ kN

Number of bolts, $n = \dfrac{P}{V_{db}}$

$$n = \frac{450}{80} = 5.6$$

$n \approx 6$ No's

Arrange 6 No's ; M_{20} bolts using diamond pattern as shown in figure.



Design tensile strength of main plate at section 1-1:

$$T_{dp1-1} = \frac{0.9 A_n f_u}{\gamma_m}$$

$$= \frac{0.9(((200-22)\times 10)\times 410)}{1.25}$$

$T_{dp_{1-1}} = 525.46\,\text{kN} \geq P = 450\,\text{kN}$

Design tensile strength of cover plate of section 3-3

$$T_{dp3-3} = 0.9\frac{A_n f_u}{\gamma_{m1}}$$

$$= 0.9[(200 - 3\times 22)\times 12]\times \frac{410}{1.25}$$

$T_{d_{p3-3}} = 474.5\,\text{kN} \geq P = 450\,\text{kN}$

Hence connection is safe

V_{dc} = lesser of T_{dP1-1} and $T_{dp3-3} = 474.5$ kN

Also, $V_{dc} \geq P$

Hence safe.

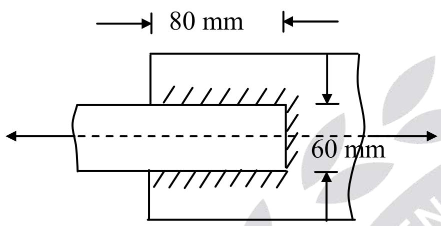
03. Welded Connections

07. Ans: (20)

Sol: Permissible stress in the wield are reduced by 20% when the welding is done in the field.

08. Ans: (d)

Sol:



$S = 8$ mm, $\tau_{vf} = 110$ N/mm^2

Working stress method $\Rightarrow$

$\therefore \quad P_s = L_w \times t_t \times \tau_{vf}$ $\quad [\therefore t_t = 0.7 \times S = 0.7 \times 8]$

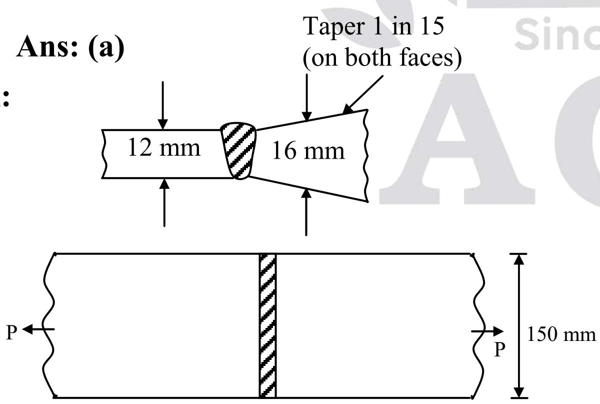
L_W = Effective length of fillet weld

$= (80+80+60) \times (0.7 \times 8) \times 110$

$= 135$ kN

09. Ans: (a)

Sol:



Single 'V' butt weld

Given data:

Permissible tensile stress in the plate

$\sigma_{at} = 150$ MPa

What is maximum tension P =?

We can allow Maximum tension load up to strength of butt weld

= Axial strength of butt weld

$P = T_s = l_w.t_e.\sigma_{at}$

[$\because$ In case of single 'V' butt weld

$t_e = \frac{5^{th}}{8}$ thickness of thinner member in case of single V]

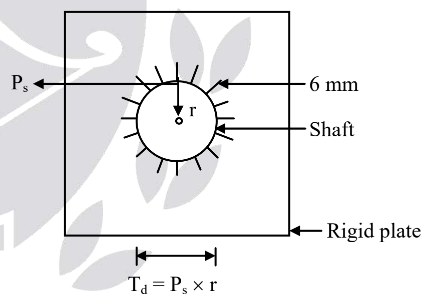
$= 150 \times \frac{5}{8} \times 12 \times 150$

$= 168.78 \times 10^3$N

$P = 168.75$ kN

10. Ans: (b)

Sol:

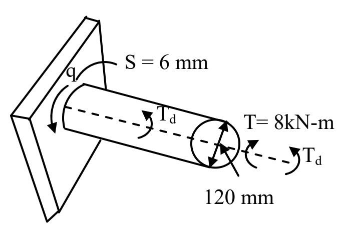


Size of weld (S) = 6 mm

Torque $\quad T = 8$ kN-m

$= 8 \times 10^6$ N-mm

Maximum stress in weld q =?



Twisting moment capacity of weld

$$T_d = P_s \times r = P_s \times \frac{d}{2}$$

$$= L_w.t_t.q.\frac{d}{2}$$

$$T = \pi d(ks)q.\frac{d}{2}$$

$$8\times10^6 = \pi\times120\times(0.7\times6)\times q\times\frac{120}{2}$$

$$q = 84.2 \text{ N/mm}^2 \simeq 85 \text{ MPa}$$

11. Ans: (b)

Sol: S = 10 mm; $f_y = 250$ Mpa = f_{yw}; $f_u = 410$ Mpa;

$\gamma_{mw} = 1.25$; P = 270 kN

$= f_{yw}$

$\therefore L_w = l_j + l_j = 2l_j$

$$P \le P_{dw} = L_w \times t_t \times \frac{f_u^1}{\sqrt{3}\gamma_{wm}}$$

$$\Rightarrow 270\times10^3 = (2\times\ell_j)\times(k\times S)\times\frac{f_u^1}{\sqrt{3}\gamma_{mw}}$$

$$\Rightarrow 270\times10^3 = (2\times\ell_j)\times(0.7\times10)\times\frac{410}{\sqrt{3}\times1.25}$$

$$l_j = 101.8 \text{ mm} \simeq 105 \text{ mm}$$

12. Ans: 60

Sol: Throat thickness = 0.7× 6 mm

= 4.2 mm

Effective length of weld = 100 + 100 +50

= 250 mm

Permissible stress in the weld = 150 MPa

Strength of weld = (4.2×250] ×150 N

= 115.5 kN

Strength of plate= [50 × 8] ×150 N

= 60 kN

Then before the failure of weld joint plate fails.

The permissible load allowable = 60 kN

Conventional Practice Solutions

01.

Sol: For Fe 410 grade steel

$f_u = 410$ MPa & $f_y = 250$ MPa

$\gamma_{mo} = 1.10$; $\gamma_{mw} = 1.25$

Size of fillet weld, S = 6 mm

For angle ISA 90 × 90 × 10

$A = 1703 \text{ mm}^2$

$C_{xx} = C_{zz} = 25.9$ mm

Design tensile strength of an angle based on cross section yielding.

$$P = T_{dg} = A_g \frac{f_y}{\gamma_{mo}} = 1703\times\frac{250}{1.10}$$

P = 387 kN

Size of fillet weld, S = 6 mm

Effective throat thickness

$t_t = KS$

$t_t = 0.7 \times 6$

$t_t = 4.2$ mm

Design shear strength of fillet weld (P_{dw})

$$P_{dw} = L_w.t_t.\frac{f_u'}{\sqrt{3}\gamma_{mw}}$$

Equating $P = P_{dw} = L_w.t_t \times \frac{f_u'}{\sqrt{3}\gamma_{mw}}$

$$387\times10^3 = L_w(4.2)\frac{410}{\sqrt{3}\times1.25}$$

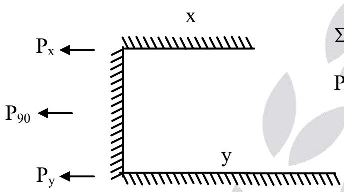
$L_w = 486.6$ mm

Effective length of weld to be arranged on top edge, bottom and vertical edge of an angle as shown in figure.

$x + y + 90 = L_w = 486.6$ mm

$x + y = 486.6 - 90$

$x + y = 396.6$ mm $\rightarrow (1)$



$\Sigma F_x = 0$

$P_x + P_y + P_{90} = P$

$$x.t_t.\frac{f_u'}{\sqrt{3}\gamma_{mw}} + y\times t_t\times\frac{f_c'}{\sqrt{3}\gamma_{mw}}.$$

$$+90\times t_t.\frac{f_c'}{\sqrt{3}.\gamma_{mw}} = L_w\times t_t\times\frac{f_u}{\sqrt{3}\gamma_{mw}}$$

$x + y + 90 = L_w$

Consider moments of weld strength and load about top edge of an angle.

$$y\times4.2\times\frac{410}{\sqrt{3}\times1.25}\times90 + 90\times4.2\times\frac{410}{\sqrt{3}\times1.25}\times\frac{90}{2}$$

$-387\times10^3\times(90-25.9) + 0 = 0$

$y = 301.5$ mm $\rightarrow (2)$

from equation (1) and equation (2)

$x = 95.04$ mm

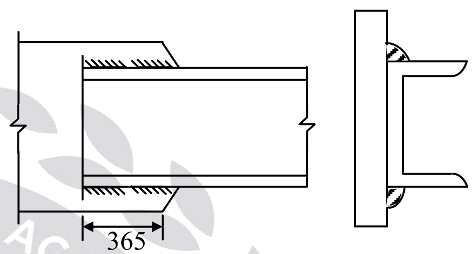
02.

Sol: Given data: ISMC300, $t_g = 16$ mm

Overlap length (L) = 365 mm

Sectional properties of ISMC 300,

$t_f = 13.6$ mm, $t_f = 7.6$ mm, $A = 4564\ mm^2$



Maximum size of weld (s_{max}) = t – 1.5

= 7.6 – 1.5 = 6.1 mm

$\therefore$ Adopt 's' = 6 mm

Design strength of fillet weld (P_{dw}) = (0.75 l_w) $\frac{f_u}{\sqrt{3}\gamma_{mw}}$

Assuming weld to be designed for maximum strength of tension member,

Design strength of tension member

$$(T_{dg}) = \frac{A_g f_y}{\gamma_{mo}}$$

$$= \frac{4564\times250}{1.1}$$

= 1037.27 kN

$\therefore P_{dw} = 1037.27\times10^3$

$$= 0.7\times6\times l_w\times\frac{410}{\sqrt{3}\times1.25}$$

$l_w = 1304.16$ mm

Assuming & welding to be on 3 sides, maximum length of weld that can be provided

$$= 2\times 365 + 300 = 1030 \text{ mm} < l_w$$

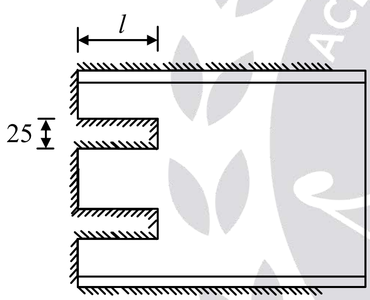
Since length of weld available is not sufficient, Adopt plug weld.

As per specification,

Width of slot weld $\nless$ 3t or 25 mm whichever is greater.

$\therefore$ width = 3 × 7.6 = 22.8

Hence adopt width = 25 mm



Excess length of weld to be provided as slot weld = 1304.16 – 1030 = 274.16 mm

274.16 = 4 l

l = 68.54 ≈ 70 mm

04. Eccentric Connections

02. Ans: (b)

Sol:

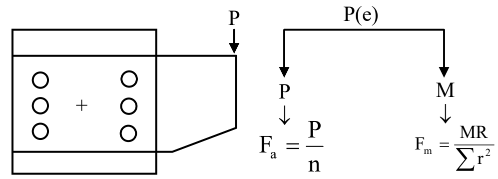
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

$$x^2 + y^2 = r^2 \quad (r = 1)$$

It is circle equation

03. Ans: (c)

Sol:



(1) $F_a \propto r$ (2) $F_m \propto r$

(3) $F_a < 0.5\, V_{db}$ (4) $F_m \le r$

04. Ans: 5.99

Sol: Given load P = 10 kN

Eccentricity = 150 mm

Number of rivets =4

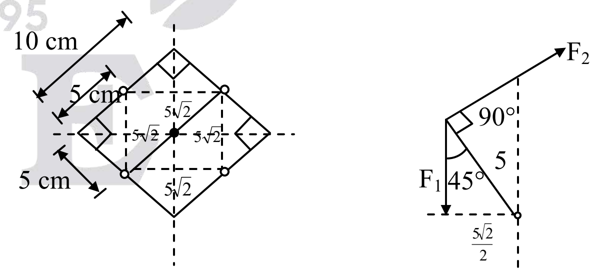
Force in rivet 1 due to direct loading = $\frac{P}{4}$

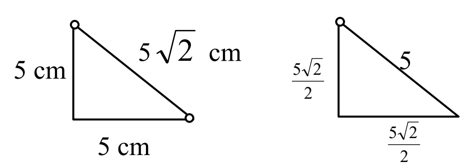
F_1 = 2.5 kN

Force in rivet 1 due to twisting moment

$$= \frac{M.r_e}{\Sigma r^2}$$

M = P[150] kN-mm





$$F_2 = \frac{[10]\times 150\times 50}{4\times [50]^2} = 7.5 \text{ kN}$$

Resultant force $F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

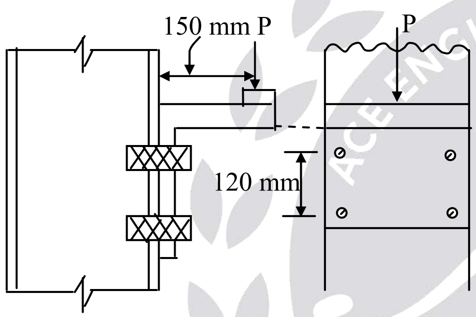
$\theta = 135°$

$F_R = 5.99$ kN

05. Ans: (c)

Sol: Design shear stress $V_{db} = 20$ kN

Design tensile capacity $\tau_{bd} = 15$ kN



1) P → cause shear force in bolt= P/ 8n.

2) $M = P \times e = P \times 150$

$= 150P$ kN − mm.

$$V_b = \frac{P}{n} = \frac{P}{4}$$

$$\frac{M}{I} = \frac{f}{y}$$

$$f = \left(\frac{M}{I}\right)y$$

$$T_b = f.\ A = \frac{M}{I} y.A$$

$M = 150$ P kN − mm

$$y = \frac{120}{2} = 60\text{mm}$$

$I_{XX} = [I_{CG} + Ag\ ]4$

$$= \left[\frac{\pi d^4}{64} + A(60)^2\right]4 \qquad \left(\frac{\pi d^4}{64} \text{ neglected}\right)$$

because it is very smaller than (60^2) A.

$= 14400$ A mm^4

$$T_b = \frac{M}{I} \times y \times A$$

$$= \frac{150P}{14400A} \times 60 \times A = \frac{5P}{8}$$

Interaction formula

$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

$$= \left(\frac{P}{4}\times\frac{1}{20}\right)^2 + \left(\frac{5P}{8}\times\frac{1}{15}\right)^2 \le 1.0$$

$$\left(\frac{P}{80}\right)^2 + \left(\frac{P}{24}\right)^2 \le 1.0$$

08. Ans: (c)

Sol: Force in each bolt due to direct concentric load (F_a)

$$F_a = \frac{p}{n} = \frac{130}{4} = 32.5$$

Force in critical bolt due to moment (Fm)

$$F_m = \frac{Per}{\Sigma r^2} = \frac{130\times 200\times 130}{4\times 16900}$$

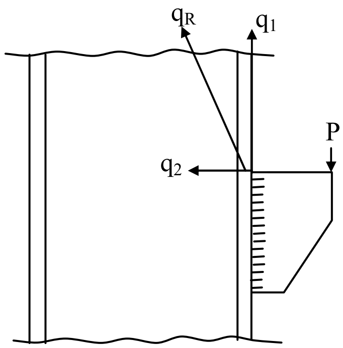
$$F_{R\,max} = \sqrt{E_a^2 + F_m^2 + 2F_aF_m\cos\theta} = 50$$

$$= \sqrt{(325)^2 + (50)^2 + 2\times 32.5\times 50\times \frac{50}{130}}$$

$= 69.32$ kN

09. Ans: (c)

Sol



$P_u.\sin\theta = 4/5$

$q_1 = 30$ MPa,

$q_2 = 120$ MPa

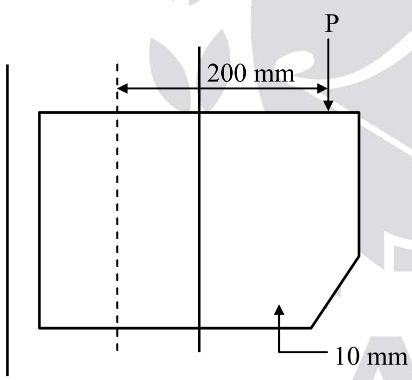
$$q_R = \sqrt{30^2 + 120^2}$$

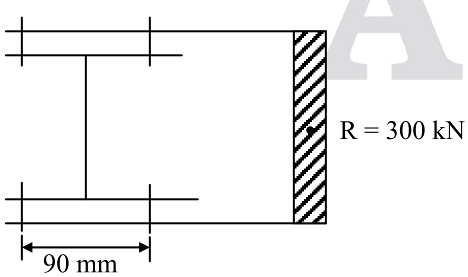
$$= 120\,\text{MPa} = 132\,\text{MPa}$$

Conventional Practice Solutions

01.

Sol:





From the figure

Force acting on each connection

(P) = R/2 = 150 kN

No. of rows to be provided (m) = 2

Eccentricity (e) = 200 mm

$\Rightarrow$ Moment (M) = P.e = 30 kN-m

Thickness of plate = 10 mm

Using M20 bolts of Grade 4.6 (fully threaded)

[ϕ = 20 mm, f_{ub} = 400 MPa, f_{yb} = 240 MPa]

$\rightarrow$ Design shear capacity

$$(V_{dsb}) = \frac{f_{ub.}}{\sqrt{3}\gamma_{mo}}[n_s.A_{sb} + n_n.A_{nb}]$$

$$= 46.38 \text{ kN}$$

$$\left[\begin{array}{l} n_s = 0;\ n_n = 1 \\ A_{sb} = \frac{\pi}{4}(20^2) \\ A_{nb} = 0.78\frac{\pi}{4}(20)^2 \end{array}\right]$$

Adopting minimum pitch (p) = 2.5ϕ = 50 mm

Minimum end distance (e) = 1.5d_h = 33 mm

$\rightarrow$ Design bearing capacity

$$(V_{dpb}) = 2.5K_b.\phi.t.\frac{f_u}{\gamma_{m_b}}$$

$$= 2.5 \times 0.5 \times 20 \times 10 \times \frac{400}{1.25}$$

[K_b = 0.5 for p_{min} & e_{min}]

$$= 80 \text{ kN}$$

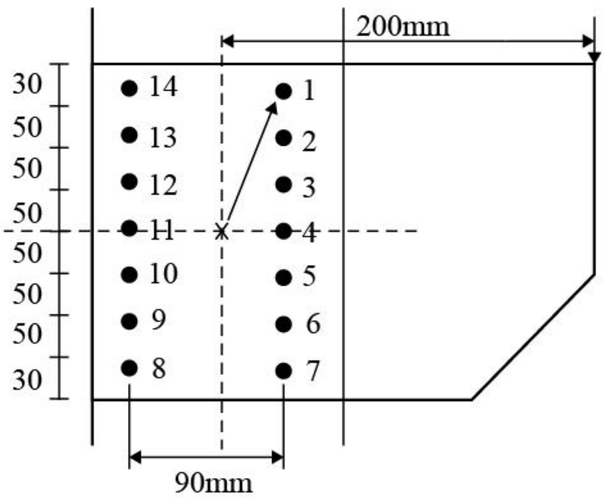
$\rightarrow$ Design capacity of bolt (V_{db}) = 46.38 kN

Design of connections:

$\rightarrow$ No. of bolts required per row $= \sqrt{\dfrac{6M}{m.p.V_{db}}}$

$= 6.23$

Providing 7 bolts per row as shown in figure



Check:

Most critical bolt is bolt 1

At bolt 1:

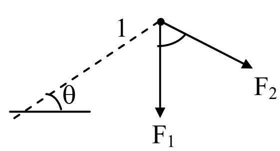
Direct shear force developed on bolt

$(F_1) = \dfrac{P}{14}$

1 due to direct force Force P = 10.7 kN

Shear force developed in bolt1 due to twisting

moment $(F_2) = \dfrac{M}{\Sigma r_i^2} \times r_1 = 27.92$ kN



$\cos\theta = \dfrac{45}{156.6}$

Resultant shear force developed in bolt 1

$(F_{req}) = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta} = 31.9$ kN

$F_{res} < V_{db}$ Hence safe.

02.

Sol:

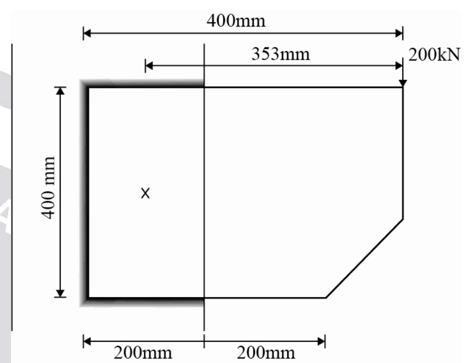


Fig. 1

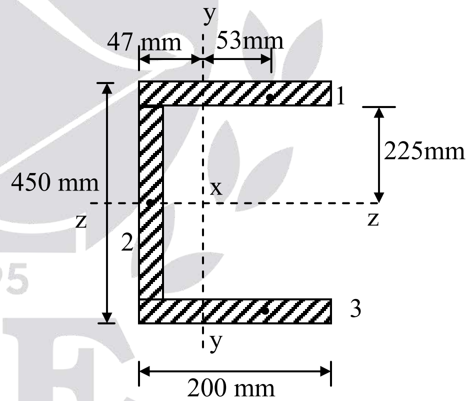


Fig. 2

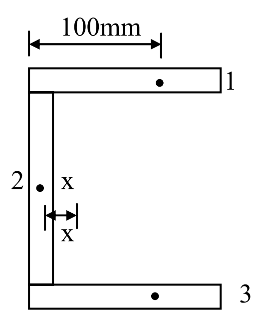


Fig. 3

Let size of weld be 'S' mm

Throat thickness (t_t) = 0.75

From figure 3, $\bar{x} = 47$ mm

$\Rightarrow$ Eccentricity (e) = 353 mm

From figure 2,

$I_P = I_{zz} + I_{yy}$

$$= (A_1 \times 225^2) \times 2 + t_t \times \frac{450^3}{12} +$$

$$2 \times \left[t_t \times \frac{200^3}{12} + A_1(53)^2 \right] + A_2 \times (47)^2$$

$= 31.29\ t_t \times 10^6\ mm^4$

[Neglecting higher powers of t_t]

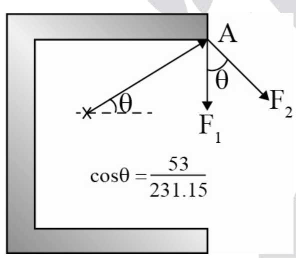
From figure 3,

$t_1 = 200 \times t_t$; $x_1 = 100$

$t_2 = 450 \times t_t$; $x_2 = 0$

$t_3 = 200 \times t_t$; $x_3 = 100$

$$\bar{x} = \frac{A_1x_1 + A_2x_2 + A_3x_3}{A_1 + A_2 + A_3} = 47 \text{ mm}$$



Total force (P) = 200 kN

Twisting Moment (T) = P.e = 70.6 kN-m

At most critical point A:

Shear stress developed due to direct shear force P, $(F_1) = \dfrac{P}{A_{weld}}$

$$= \frac{200 \times 10^3}{(200 + 450 + 200)t_t} = \frac{235.3}{t_t} \text{MPa}$$

Shear stress developed due to moment T $(F_2) = \dfrac{T}{I_P} \times (r_A = 231.15)$

$$= \frac{70.6 \times 10^6}{31.29 t_t \times 10^6} \times 231.15 = \frac{521.5}{t_t} \text{MPa}$$

Resultant Shear stress developed

$$(F_{res}) = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos\theta} = \frac{619.19}{t_t} \text{MPa}$$

Allowable shear stress = 110 MPa

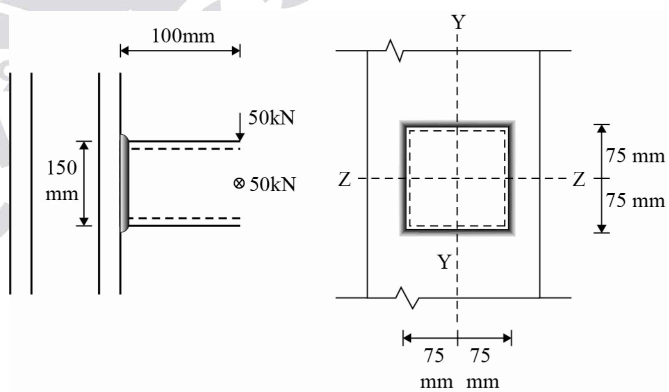
$$t_{t\ req} = \frac{619.19}{110} \text{mm} = 5.62 \text{ mm}$$

$$S_{reqd} = \frac{5.62}{0.7} = 8.04 \text{ m}$$

Minimum size required = 8.04 mm

03.

Sol:



Let size of weir be 'S' mm.

Given:

Load along Y-Y (P_Y) = 50 kN

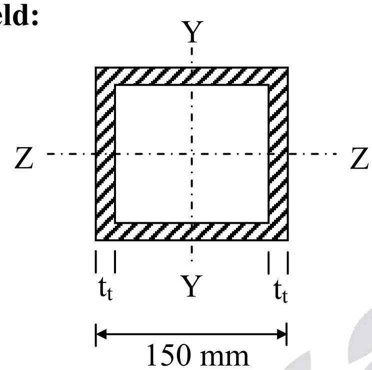
Load along Z-Z (P_Z) = 50 kN

eccentricity (e) = 100 mm

Moment along Y-Y (M_{YY}) = $P_Z.e$ = 5 kN.m

Moment along Z-Z (M_{ZZ}) = $P_y.e$ = 5 kN.m

Weld:



$$I_{ZZ} = 2\left[t_t \times \frac{150^3}{12} + 150t_t \times 75^2\right] = I_{YY}$$

$\Rightarrow I_{ZZ} = I_{YY} = 2.25 \times 10^6$ mm^4

(neglecting higher power of t_t)

Area of weld (A_w) = 4×150×t_t

= 600t_t mm^2

Due to direct shear force (P_Y & P_Z)

Shear stress developed along Y (q_y) in weld

$$= \frac{P_y}{A_w} = \frac{83.3}{t_t}\text{MPa}$$

Shear stress developed along Z (q_z) in weld

$$= \frac{P_z}{A_w} = \frac{8.3}{t_t}\text{MPa}$$

Resultant shear stress developed due to direct shear force (q) = $\sqrt{q_y^2 + q_z^2}$

$$= \frac{117.85}{t_t}\text{MPa}$$

Due to Moments (M_{yy} & M_{zz})

Maximum bending stress developed is at extreme fibres

At extreme fibres

- Bending (or normal) stress developed due to

$$M_{yy}\ (f_1) = \frac{M_{yy}}{I_{yy}} \times (Z = 75\,\text{mm})$$

$$= \frac{166.67}{t_t}\text{MPa}$$

Bending (or normal) stress developed due to

$$M_{zz}\ (f_2) = \frac{M_{zz}}{I_{zz}}(y = 75\,\text{mm})$$

$$= \frac{166.67}{t_t}\text{MPa}$$

Resultant bending (or normal) stress developed due to M_{yy} or M_{zz} (f) = $f_1 + f_2$

$$= \frac{333.3}{t_t}\text{MPa}$$

Due to combined shear force (P_z & P_y) and Moment (M_{yy} & M_{zz}), equivalent stress developed (f_{eq}) = $\sqrt{3q^2 + f^2}$

$$= \frac{390.8}{t_t}\text{MPa}$$

For safe connection $f_{eq} \le f_{wd}$

$$\frac{390.8}{t_t} \le \left(\frac{f_{uw}}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25}\right)$$

$\Rightarrow t_t \ge 2.06$ mm

$$\Rightarrow S \ge \frac{t_t}{0.7} = 2.95\,\text{mm}$$

For 6 mm plate, S_{min} = 3 mm

Hence, weld of 3 mm may be considered safe.

05. Tension Members

01. Ans: 240

Sol: $f_y = 400$ MPa

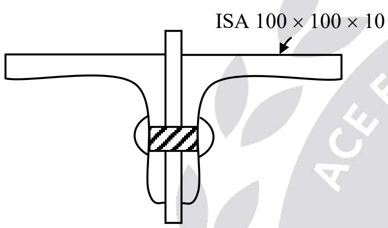
Permissible tensile stress $(\sigma_{at}) = 0.6 \times f_y$

$= 0.6 \times 400$

$\sigma_{at} = 240$ MPa

02. Ans: (d)

Sol:



ISA = 100 × 100 × 10

$A_1 = 775\ mm^2$

$A_2 = 950\ mm^2$

$A_{net} = A_1 + A_2K_1$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3\times 775}{(3\times 775 + 950)} = 0.709$$

K_1 = Reduction factor = 0.709

$A_{net} = A_1 + A_2 \times K_1$

$= (2 \times 775) + (2 \times 950) \times 0.709$

$A_{net} = 2899\ mm^2$

03. Ans: (c)

Sol:

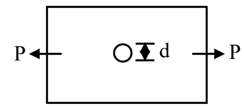


Fig. 1

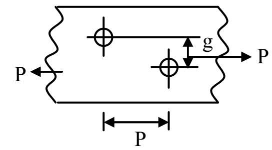


Fig. 2

$$P_t = A_{net} \times \sigma_{at}$$

$(\sigma_{at} = 0.6\ f_y$ is same (1) & (2))

$$P_t \propto A_{net}$$

$$P_{(t)_2} > P_{t_{(1)}}$$

$$A_{net(2)} > A_{net(1)}$$

$$(B - 2d)t + \frac{P^2 t}{4g} > (B - d)t$$

$$Bt - 2dt + \frac{P^2 t}{4g} > Bt - dt$$

$$-dt - dt + \frac{P^2 t}{4g} > -dt$$

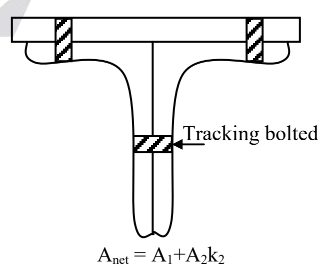
$$\frac{P^2 t}{4g} > dt$$

$$\Rightarrow P^2 > 4gd$$

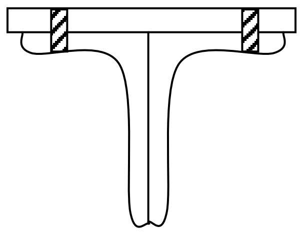
05. Ans: (c)

Sol:

(a)

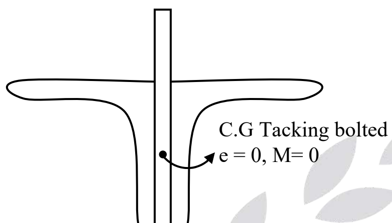


(b)



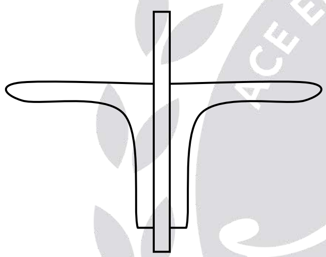
$A_{net} = A_1 + A_2 k_2$

(c)



$A_{net} = A_g -$ Area of rivet hole

(d)



$A_{net} = A_1 + A_2 k_1$

$$K_1 = \frac{3A_1}{3A_1 + A_2}\ ;\ \ K_2 = \frac{5A_1}{5A_1 + A_2}$$

$$P_t = A_{net} \times \sigma_{at}$$

$$w(a) = w(b) = w(c) = w(d)$$

$$A_{(c)net} > A_{net\,(a)} > A_{net\,(b)} = A_{net\,(d)}$$

$$\therefore\ \ P_{(t)\,c} > P_{t(a)} > P_{t(b)} = P_{(t)d}$$

06. Ans: (d)

Sol: Attachment to the member should capable of developing 40% in excess of force in outstanding leg of the angle.

10. Ans: 1400

Sol: A member normally acting as a tie in a roof truss (or) a bracing system, when is subjected to possible reversal of stress resulting from the action of wind (or) earth forces, the maximum slenderness ratio is 350

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{\frac{\pi}{64}d^4}{\frac{\pi}{4}d^2}} = \frac{d}{4}$$

$$r_{min} = \frac{16}{4} = 4\text{m}$$

$$\lambda = \frac{L}{r_{min}}$$

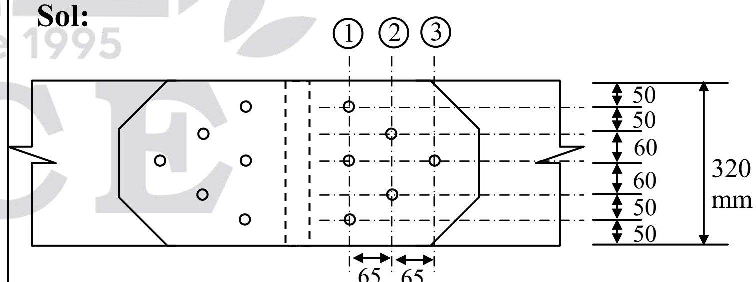
$$L = 350 \times 4$$

$$L = 1400 \text{ mm}$$

Conventional Practice Solutions

01.

Sol:



Assume double cover butt joint connected with 4.6 grade bolts.

Design strength of connection = Least of V'_{dsb}, V'_{dpb} & T_{dp}

V'_{dsb} = Design shear strength of bolts

$$= (n_s A_{sb} + n_n A_{nb}) \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}$$

Assuming both shear planes to intercept with threading

$$V'_{dsb} = \left(0 + 2 \times 0.78 \times \frac{\pi \times 25^2}{4}\right)\frac{400}{\sqrt{3} \times 1.25} \times 6$$

$= 848.85$ kN

$$V'_{dpb} = \frac{(2.5dt')f'_u k_b}{\gamma_{mb}} \times 6$$

K_b = Least of

(i) $\frac{e}{3d_o} = \frac{50}{3 \times 28} = 0.595$

(ii) $\frac{p}{3d_o} - 0.25 = \frac{65}{3 \times 28} - 0.25 = 0.523$

(iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

(iv) 1.0

$\therefore K_b = 0.523$

$$V'_{dpb} = \frac{2.5 \times 20 \times 16 \times 400 \times 0.523}{1.25} \times 6 = 803.3\,\text{kN}$$

Design Strength of connection $\Rightarrow$ Least of strength of main plate & strength of cover plates

Design Strength of main plate $\Rightarrow$ w.r.t rupture at section 3-3

The critical cross section for failure of main plate in tearing is section (3) which is nearer to the application of force.

$$\therefore T_{dp} = \frac{0.9 A_n f_u}{\gamma_{m_1}}$$

$A_n = (B - d_o)t = (320 - 28)16 = 4672$ mm^2

$$T_{dp} = \frac{0.9 \times 4672 \times 410}{1.25} = 1379.17\,\text{kN}$$

Design strength of cover plates in tearing:

Thickness of each cover plate

$$= \frac{5}{8} \times 16 = 10\,\text{mm}$$

The critical c/s for failure of cover plates in tearing is section (1) which is nearer to the centre of joint.

$$T_{dp} = \frac{0.9 A_n f_u}{\gamma_{m_1}}$$

$A_n = (320 - 3 \times 28)16 \times 2 = 7552$ mm^2

$$T_{dp} = \frac{0.9 \times 7552 \times 410}{1.25} = 2229\,\text{kN}$$

$\therefore$ Design strength of bolted connection

= Least of V'_{dsb}, V'_{dpb} & T_{dp} = 803. 3 kN.

02.

Sol: Length of tie member, = 1.8 m

Axial tensile load, P = 155 kN

Yield stress, f_y = 250 MPa

τ_{Vf} = 100 MPa

σ_{pf} = 300 MPa

τ_{vw} = 108 MPa

Permissible tensile stress,

$\sigma_{at} = 0.6\, f_y$

$= 0.6 \times 250 = 150$ MPa

Net sectional area required to resist tensile load P is A_{net}

$$A_{net} = \frac{P}{\sigma_{at}} = \frac{155\times10^3}{150}$$

$= 1033.2\ mm^2$

Gross sectional area of trial section

$A_g = 1.25\ A_{net} = 1.25 \times 1033.2$

$A_g = 1291.1\ mm^2$

Available sections for angle

Angle size	**Sectional area (mm^2)**	**Minimum radius of gyration (r_{min})**	**Cyy (mm)**
ISA70×45×10	1052	9.50	24.8
ISA100×75×6	1014	15.9	30.1
ISA100×65×8	1257	13.9	32.6
ISA 90 ×60×8	1137	12.8	27.2
ISA90×60×10	1401	12.7	28.1

Choose an angle (usually $\geq A_g$)

ISA 100×65 ×8

$A = 1257\ mm^2$; $r_{min} = 13.9$ mm

(i) Power driven rivets are used at a joint:

Assume nominal diameter of rivet

$\phi = 6.04\sqrt{t}$

$t \rightarrow$ thickness of thinner connected member

$\phi = 6.04\sqrt{8} = 17.08\,mm$

$= 18$ mm

Gross dia of rivet

$d = 18 + 1.5 = 19.5$ mm

Assuming at a section, angle section is reduced by one rivet hole.

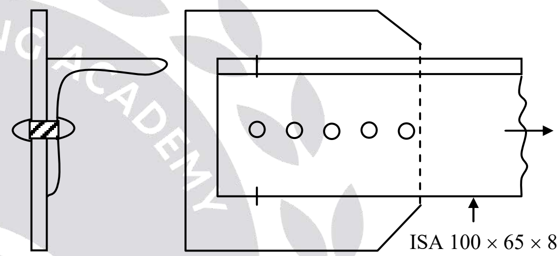
Net & Effective sectional area of single angle

$A_{net} = A_1 + A_2 K_1$

A_1 – net sectional area of connected leg

$= (100 - 19.5 - 8/2) \times 8$

$= 612\ mm^2$



A_2 = cross sectional area of outstanding leg

$A_2 = (65 - 8/2) \times 8 = 488\ mm^2$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3\times612}{3\times612+488}$$

$= 0.79$

$\therefore\ A_{net} = A_1 + A_2 K_1$

$= 612 + 488 \times 0.79$

$A_{net\ trial} = 997.52\ mm^2 \leq 1033\ mm^2$

$A_{net\ trial} = \leq A_{net\ required}$

Hence trial section is unsafe

(i.e., $p_{t\ trial} = A_{net\ trial} \times \sigma_{at}$

$P_{t\ axial} = 99732 \times 150$

$= 149.6 \times 10^3$

$= 149.6$ kn < 155 kN not safe

Iteration –II:

Choose a trial section

ISA 90 × 60 × 10

A = 1401 mm^2

r_{min} = 12.7 mm

nominal diameter of rivet

$\phi = 6.04\sqrt{10} = 19.04$ mm

= 20 mm

Gross diameter of rivet

d = 21.5 mm

net sectional area of connected leg

$$A_1 = \left(90 - 21.5 - \frac{w}{2}\right) \times w$$

A_1 = 635 mm^2

Gross sectional area of outstand leg

$$A_2 = \left(60 - \frac{10}{2}\right) \times w = 350\ \text{mm}^2$$

Reduction factor, $K_1 = \dfrac{3A_1}{3A_1 + A_2}$

$$= \frac{3 \times 635}{3 \times 635 + 550}$$

K_1 = 0.78

$\therefore \quad A_{net} = A_1 + A_2K_1$

= 635 + 550 × 0.78

$A_{net\ trial}$ = 1064 mm^2 ≥ A_{net} required

$P_{trial} = A_{net\ trial} \times \sigma_{at}$

= 1064 ×150

$P_{t_{trial}} = 159.6\,\text{kN} > 155\,\text{kN}$

Hence trial section is safe

Number of rivets required

$$n = \frac{\text{Axial tensile load}}{\text{Rivet value}}$$

Rivet value = R_V = Smaller of P_s & P_b

Strength of one rivet in single shear

$$P_s = \frac{\pi}{4} d^2 \times \tau_{vf}$$

$$= \frac{\pi}{4}(21.5)^2 \times 100$$

= 36.3 × 10^3 N

P_s = 36.3 kN

Strength of one rivet in bearing

$P_b = d \times t \times \sigma_{pf}$ = 21.5× 10 ×300

P_b = 64.5 kN

Rivet value, R_V = 36.3 kN

$$n = \frac{P}{P_V} = \frac{155}{36.3} = 4.26$$

n ≈ 5 No's

Minimum pitch, P = 2.5×20 = 50 mm

Minimum end distance, e = 1.5×21.5 ≈ 35 mm

Use 5 no's - ϕ20 mm PDS rivets as shown in figure.

(ii) Fillet weld used at a joint:

choose a trial section ISA 100 ×65 ×8 mm

A = 1257 mm^2

r_{min} = 13.9 mm

Net sectional area of connected leg

$$A_1 = \left(100 - \frac{8}{2}\right) \times 8 = 768\,\text{mm}^2$$

Net connected area of outstanding leg

$$A_2 = \left(65 - \frac{8}{2}\right) \times 8 = 488\ \text{mm}^2$$

Reduction factor

$$K_1 = \frac{3A_1}{3A_1 + A_2}$$

$$= \frac{3 \times 768}{3(768) + 488} = 0.825$$

Net effective sectional area of trial section

$A_{net} = A_1 + A_2 K_1$

$= 768 + 488 \times 0.82$

$= 1170.6\ \text{mm}^2 \geq 1033\ \text{mm}^2$

$A_{net} \geq A_{net}$

Hence trial section is safe

Slenderness ratio of trial section

$$\lambda = \frac{L}{r_{min}} = \frac{1800}{13.9}$$

$= 129 \leq \lambda_{limit} = 350$

Hence ok.

Design of fillet weld:

Let S and L_w be the size and effective length of fillet weld minimum size of fillet weld, $S_{min} = 3$ mm

Maximum size of fillet weld, $S_{max} = \frac{3}{4} \times 8$

$= 6$ mm

Adopt size of fillet weld, S = 6 mm

Arranging effective length of weld. On top & bottom edge of an angle.

Let Lw_1 and Lw_2 be the length of top & bottom length of weld respectively.

$L_w = L_{w1} + L_{w2} = 341.7 \quad \rightarrow (1)$

Effective throat thickness

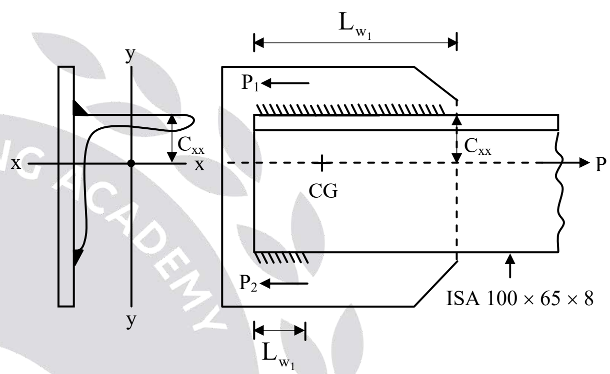
$t_t = KS = 0.7 \times 6 = 4.2$ mm

equating, P = Ps

$155 \times 10^3 = L_w \times t_t \times \tau_{vf}$

$155 \times 10^3 = L_w \times 4.2 \times 108$

$L_w = 341.7$ mm



Consider moments of weld strength and load on top edge of an angle.

$P_2 \times 100 + P_1 \times 0 - P \times 32.6 = 0$

$L_{w2} \times t_t \times \tau_{vf} \times 100 = 155 \times 10^3 \times 32.6$

$L_{w2} \times 4.2 \times 108 \times 100 = 155 \times 10^3 \times 32.6$

$L_{w2} = 111.39$ mm

From equation (1) & (2)

$L_{w1} = 230.31$ mm

$L_{w2} = 111.39$ mm

03.

Sol: Working tensile load, $T_s = 200$ kN

Length of member, L = 2.5 m

For Fe410 grade steel,

$f_u = 410$ MPa & $f_y = 250$ MPa

For grade 8.8 bolts

$\gamma_{mo} = 1.1$; $\gamma_{m1} = 1.25$; $\gamma_{mb} = 1.25$

Factored (or) Design load, $T = \gamma_f \times T_s$

$T = 1.5 \times 200 = 300$ kN

Gross sectional area of trial section required to resist τ is A_g:

$$T = T_d = T_{dg} = A_g \frac{f_y}{\gamma_{mo}}$$

$$A_g = \frac{T}{\frac{f_y}{\gamma_{mo}}}$$

$$A_g = \frac{300\times10^3}{\frac{250}{1.1}} = 1320\,\text{mm}^2$$

Choose a trial section ISA 100× 75 × 8

$A_g = 1336$ mm^2

$r_{min} = 15.9$ mm

Sections available for angle are

Angle size	Sectional area (mm^2)	Minimum radius of gyration (r_{min})
ISA100×75×8	1336	15.9
ISA100×65×8	1257	13.9
ISA80×80×8	1221	15.5
ISA90×60×10	1401	12.7

Design of bolted connections:

Number of bolts required

$$n = \frac{\text{Design tensile load}}{\text{Dsign strength of one Bolt}}$$

$$n = \frac{T}{V_{db}}$$

Shank diameter of Bolt,

d = 20 mm

Diameter of bolt hole,

$D_o = 20 + = 22$ mm

Minimum pitch, $P = 2.5 \times 20$

$P = 50$ mm≈ 60 mm

Minimum end distance

$e = 1.5\, d_o = 1.5 \times 22$

$= 33 \approx 40$ mm

Design shear strength of one Bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}\left(n_n A_n b + n_s A_{sb}\right)$$

$$= \frac{800}{\sqrt{3}\times1.25}\left[1\times1\times0.78\times\frac{\pi}{4}(20)^2 + 0\right]$$

= 90.51 kN

Design bearing strength of one bolt

$$V_{dpb} = 2.5\,dt\,f_{ub}K_b/\gamma_{mb}$$

$$= 2.5\times20\times8\times800\times\frac{0.61}{1.25}$$

K_b is lesser of (bearing factor)

- $\dfrac{e}{3d_o} = \dfrac{40}{3\times22} = 0.61$
- $\dfrac{P}{3d_o} - 0.25 = 0.66$
- $\dfrac{f_{ub}}{f_u} = \dfrac{800}{410} = 1.95$
- 1.0

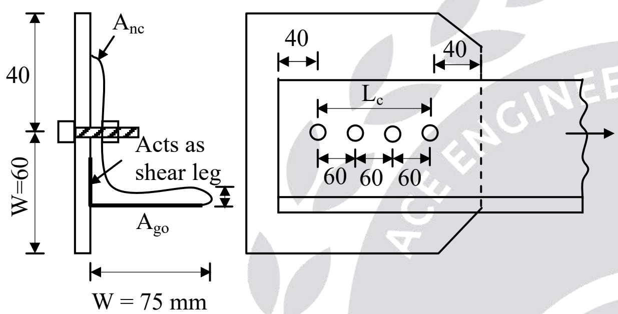
V_{dpb} = 156.16 kN

V_{db} = minimum of V_{dsp} and V_{dpb}

V_{db} = 90.51 kN

$$n = \frac{T}{V_{db}} = \frac{300}{90.51} \approx 4\,\text{no's}$$

Use 4 no's of M20 bolts arranged in single line as shown in figure for connection.



Design tensile strength of trial section

T_{dtrial} = Min. of T_{dg}, T_{dn} & T_{db}

Design, tensile strength based on Gross section yielding

$$T_{dg} = A_g \frac{f_y}{\gamma_{mo}} = 1336 \times \frac{250}{1.1}$$

$$T_{dg} = 303.64\,\text{kN}$$

Design tensile strength based on net section rupture.

$$T_{dn} = 0.9 A_{nc} \frac{f_u}{\gamma_{m1}} + \beta . A_{go} \frac{f_y}{\gamma_{ma}}$$

$$\beta = 1.40 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_e}\right) \geq 0.7$$

$$\leq \frac{f_u}{f_y} . \frac{\gamma_{mo}}{\gamma_{m1}}$$

$$A_{nc} = \left(100 - 22 - \frac{8}{2}\right) \times 8 = 592\,\text{mm}^2$$

$$A_{go} = \left(75 - \frac{8}{2}\right) \times 8 = 568\,\text{mm}^2$$

w = 75 mm; t = 8 mm

f_u = 410 MPa; f_y = 250 MPa

b_s = shear leg distance = w + w_1 – t

= 75 + 60 – 8

= 127 mm

L_c = length of end connection

= 3× 60 = 180 mm

$$\beta = \left[1.40 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{127}{180}\right)\right]$$

β = 1.09

∴ Design tensile strength of trial section.

$$\text{By } T_{dn} = 0.9 \times 592 \times \frac{410}{1.25} + 1.09 \times 568 \times \frac{250}{1.1}$$

T_{dn} = 315.47 kN

Design tensile strength based on block shear failure

Gross area of tension plane

A_{tg} = 40 × 8 = 320 mm^2

Net area of tension plane

$$A_{tn} = \left(40 - \frac{22}{2}\right) \times 8 = 232\,\text{mm}^2$$

Gross area of shear plane

A_{vg} = (3 ×60 + 40) ×8

= 1760 mm^2

Net area of shear plane

$$A_{vn} = [(3\times 60 + 40) - 3\times 22 \times 8 - \frac{22}{2})]$$

$A_{vn} = 1144 \text{ mm}^2$

T_{db1} based on shear rupture tension yield

$$T_{dbi} = 0.9A_{vn}\cdot\frac{f_u}{\sqrt{3}\gamma_{m1}} + A_{tg}\frac{f_y}{\gamma_{mo}}$$

$$= 0.9\times1144\times\frac{410}{\sqrt{3}\times1.25} + \frac{320\times250}{1.10}$$

$T_{db1} = 267.7$ kN

T_{db2} based on shear yielding & tension rupture

$$T_{db_2} = A_{vg}\frac{f_y}{\sqrt{3}\gamma_{mo}} + 0.9At_n\frac{f_u}{\gamma_{m1}}$$

$$= 1760\times\frac{250}{\sqrt{3}\times1.1} + 0.9(232)\times\frac{410}{1.25} = 299.426$$

T_{db} = lesser of T_{db1} & T_{db2}

Design tensile strength of trial section (T_d)

T_{dtrial} = lesser of $T_{dg} = 303.6$ kN

$d_n = 315.85$ kN

$T_{db} = 267.7$ kN

$T_{d\ trial} = 267.7 \text{ kN} < T = 300$ kN

Hence, trial section not safe

Increase pitch and end distance

P = 100 mm, e = 80 mm

$A_{vg} = (3\times 100 + 80)\times 8 \text{ mm}^2$

$A_{vg} = 3040 \text{ mm}^2$

$$A_{vn} = \left(380 - 3\times22 - \frac{22}{2}\right)\times8$$

$A_{vn} = 2424 \text{ mm}^2$

$A_{tg} = (80\times8) = 640 \text{ mm}^2$

$$At_n = \left(80 - \frac{22}{2}\right)\times8 = 522\,\text{mm}^2 = 552 \text{ mm}^2$$

$$\therefore\ T_{db_1} = 0.9\times2424\times\frac{410}{\sqrt{3}\times625} + \frac{640\times250}{1.1}$$

$T_{db1} = 558.5$ kN

$$\therefore\quad T_{db_2} = 3040\times\frac{250}{\sqrt{3}\times1.1} + 0.9\times\frac{552\times410}{1.25}$$

$T_{db2} = 561.8$ kN

Lesser of T_{db1} & $T_{db_2} = T_{db}$

$T_{db} = 558.5$ kN

Design tensile strength of trial section (T_d)

$T_{d\ trial}$ = lesser of, $T_{dg} = 303.6$ kN

$T_{db} = 558.5$ kN and T_{dn}

$T_{d\,trial} = 303.6 > 300\,\text{kN}$

Hence, trial section is safe

Slenderness ratio

$$\frac{L}{r_{min}} = \frac{2.5}{13.6\times10^{-3}}$$

$= 182.82 < 350\ (\lambda_{limit})$

$\therefore$ Hence it is safe from buckling.

06. Compression Members

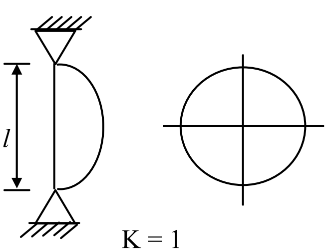
01. Ans: 50

Sol: KL = effective length

K = effective length constant

$$\left(\frac{KL}{r}\right) = 200$$

$$\frac{\ell}{d} = ?$$



K = 1

Radius of gyration, $r_{min} = \sqrt{\dfrac{I}{A}}$

$$= \sqrt{\frac{\pi d^4}{64} \times \frac{4}{\pi d^2}}$$

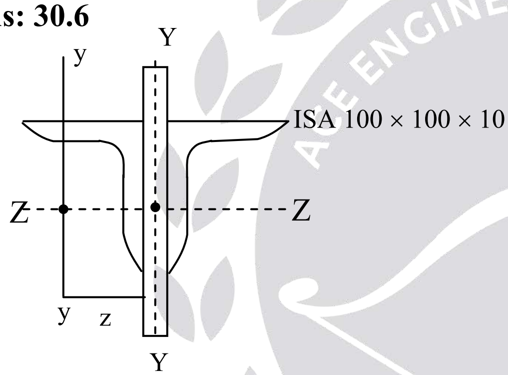
$$= \sqrt{\frac{d^2}{16}} = \frac{d}{4}$$

$$\frac{KL}{r_{min}} = \frac{1.0\ell}{\frac{d}{4}} = 200 \qquad (\therefore k = 1)$$

$$\frac{\ell}{d} = \frac{200}{4} = 50$$

04. Ans: 30.6

Sol:



Symmetric w.r.t yy axis

$A = 1903\ mm^2$

$I_{zz} = I_{yy} = 177 \times 10^4\ mm^4$

$I_{yy} = 2\left(I_{yy} + Az^2\right)$

Gusset plates one at joint member only.

$I_{ZZ} = 2.I_{zz} = 2 \times 177 \times 10^4$

$= 354 \times 10^4\ mm^4$

$I_{min} = 2 \times I_{ZZ}$

$$r_{min} = \sqrt{\frac{I_{ZZ}}{A}}$$

$$= \sqrt{\frac{2 \times 177 \times 10^4}{2 \times 1903}} = \sqrt{\frac{354 \times 10^4}{3806}}$$

$= 30.49\ mm \simeq 30.5\ mm$

06. Ans: 37.5

Sol: Service load = 1000 kN

Factored load = 1.5×1000

= 1500 kN

V = 2.5% of factored column load

$$V = \frac{2.5}{100} \times P$$

$$= \frac{2.5}{100} \times 1500 = 37.50\ kN$$

07. Ans: (d)

Sol: Force in lacing member

$$F = \frac{V}{N \sin\theta}$$

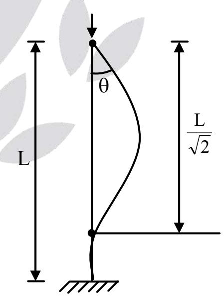
$$= \frac{37.5}{2 \times \sin 45^0}$$

($\because$ N = 2 for single lacing system)

= 26.52 kN

08. Ans: (b)

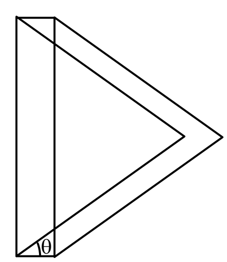
Sol:



$$\frac{L}{\sqrt{2}} = 0.707\ L \approx 0.80\ L$$

09. Ans: (d)

Sol:



Skin buckling

Conventional Practice Solutions

01. Given data: L = 6 m

Since both ends are pinned, l = k.L, k = l

In Z – Z axis, $\therefore l_x = 6\text{ m} = l_z$

In Y-Y direction, it is supported at mid height

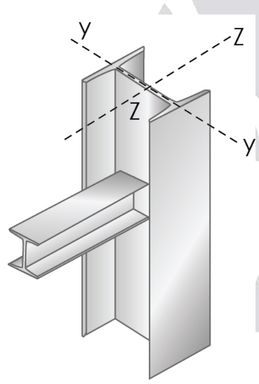
$$\therefore l_y = 3\text{m}$$

Since one end is hinged and other acts as fixed end

$$l_y = kL_y = 0.8\ L_y = 0.8 \times 3 = 2.4\text{ m}$$

Slenderness ratio, $\lambda_x^1 = \frac{\ell_x}{r_{xx}} = \frac{6\times10^3}{142.9} = 41.98$

$$\lambda_y^1 = \frac{\ell_y}{r_{yy}} = \frac{2.4\times10^3}{28.4} = 84.5$$



Since slenderness ratio is more about Y-axis, it is critical in that direction.

$$P_{d_y} = f_c d_y A_e$$

$$f_{cd} = \frac{f_y/\gamma_{m_o}}{\phi + \left[\phi^2 - \lambda^2\right]^{1/2}}$$

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

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}, \qquad f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$f_{cc} = \frac{\pi^2 \times 2\times10^5}{84.5^2}$$

$$= 276.45\text{ N/mm}^2$$

$$\lambda = \sqrt{\frac{250}{276.45}} = 0.951$$

Imperfection factor 'α_Y'

$\frac{h}{b_f} = \frac{350}{140} = 2.5 > 1.2$, $t_f = 14.2 < 40$ mm

$\therefore$ Buckling class about Y-axis is 'b'

$$\alpha = 0.34$$

$$\phi = 0.5\ [1 + 0.34\ (0.951 - 0.2) + 0.951^2]$$

$$= 2.159$$

$$\therefore f_{cd} = \frac{250/1.1}{2.159 + \left[2.159^2 - 0.951^2\right]^{1/2}} = 55.47\ \text{N/mm}^2$$

$$P_d = f_{cd} \times A_e$$

$$= 55.47 \times 6671$$

$$= 370\text{ kN}$$

Service load = $\frac{370}{1.5} = 246.7\text{ kN}$

02.

Sol: An equal angle, ISA 100× 100× 10

$A = 1903\text{ mm}^2$

$I_{xy} = I_{yy} = 177 \times 10^4\text{ mm}^4$

$I_{xy} = 104.4 \times 10^4\text{ mm}^4$

Length of strut = 2.4 m

Safe compressive load carrying capacity,

$P_C = \sigma_{ac} \times A_e = \sigma_a \times 1903$

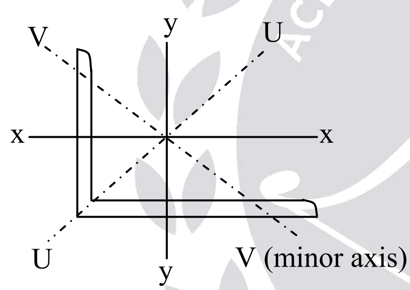
The permissible axial stress in compression are given below

KL/ r_{min}	90	100	110	120	130	140	150
σ_{ac}	90	80	72	64	57	51	45

To calculate slenderness ratio

1. If end conditions are not given, assume gn length as L_{eff}

2. For r_{min} is with respect to major & minor axis



Not about X and Y axis, hence, it has to be proved that, xx of yy axis are not major and minor axes.

Assume given length of a strut is effective length.

KL = 2.4 m = 2400 mm

Major and Minor moment of inertia of an angle

$$\frac{I_{UU}}{I_{VV}} = \frac{I_{XX}+I_{YY}}{2} \pm \sqrt{\left(\frac{I_{XX}-I_{YY}}{2}\right)^2 + \left(I_{XY}\right)^2}$$

$$\frac{I_{UU}}{I_{VV}} = \frac{177\times10^4 + 177\times10^4}{2} \pm 104.4\times10^4$$

Major moment of inertia, of angle strut

$I_{UU} = 177 \times 10^4 + 104.4 \times 10^4$

$= 281.4 \times 10^4$ mm^4

Minor moment of Inertia of angle strut

$I_{VV} = 177 \times 10^4 - 104.4\times10^4$

$= 72.6 \times 10^4$ mm^4

Minimum radius of gyration

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{72.6\times10^4}{1903}}$$

r_{min} = 19.5 mm

Effective slenderness ratio

$$\frac{KL}{r_{min}} = \frac{2400}{19.5} = 123 \qquad \text{for } \frac{KL}{r_{min}} = 123, \sigma_{ac} = ?$$

$$\therefore\ \sigma_{ac} = 64 - \left(\frac{64-57}{130-120}\right)(123-120)$$

σ_{ac} = 61.97 N/mm^2

Safe compressive load carrying capacity of an angle strut

$P_C = \sigma_{ac} \times A_e = 1.97\times19.23$

P_C = 117.8 kN

03.

Sol:

Design of braced built up column service Axial load, P_s = 750 kN

Effective length of column, KL = 10 m

For M16 Bolt, d = 16 mm, d_o = 18 mm

For grade 4.6 Bolt, f_{ub} = 400 MPa

ISMC 300@ 363 N/m

A = 4630 mm^2, r_{zz} = 118 mm

g = 50 mm, r_{yy} = 26 mm

C_{yy} = 23.5 mm

The built up column section is arranged in such way the minimum radius of gyration to be maximum .

The radius of gyration of built up section about yy axis should not be less than radius of gyration about zz axis of built up column.

$r_{yy} \not< r \Rightarrow r_{yy} \geq r_{zz}$

Min radius of gyration of built up section.

$r_{min} = r = r_{zz} = 118$ mm

Effective slenderness ratio of built up column

$$\frac{kL}{r_{min}} = \frac{kL}{r_{zz}} = \frac{kL}{r_{zz}}$$

$$= \frac{10000}{118} = 84.74$$

Increased slenderness ratio

$1.05 \times 84.7 = 88.97$

For $1.05 \times \frac{kL}{r_{min}} = 88.97$

$$f_{cd} = 136 - \frac{(136-121)}{(90-80)}(88.97-80)$$

$f_{cd} = 122.54$ N/mm^2

CL/r	**f_{cd} (MPa)**
60	168
70	152
80	136
90	121
100	107
110	94
120	83
130	74

Design compressive strength of built up section

$P_d = f_{cd} \times A_e = 122.54 \times (2 \times 4630)$

$= 1134.76 \times 10^3$ N

$P_d = 1134.76$ kN $\geq P = 1125$ kN

Hence built up section is safe

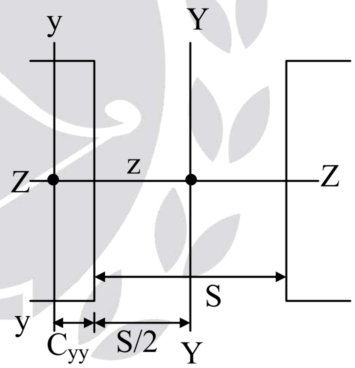
Let S be the back to back spacing between channels. Equating.

$2(I_{xy} + Az^2) = 2 \times (I_{zz} + Ay^2)$

$$\left(313\times10^4 + 4630\left(\frac{S}{2}+23.5\right)^2\right)$$

$= (6420 \times 10^4 + 4630\ (0)^2)$

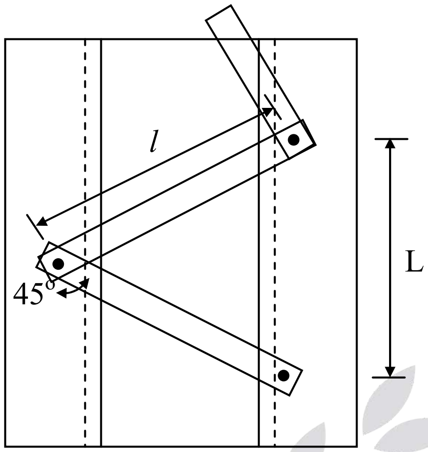
S = 182.69 mm = 183 mm

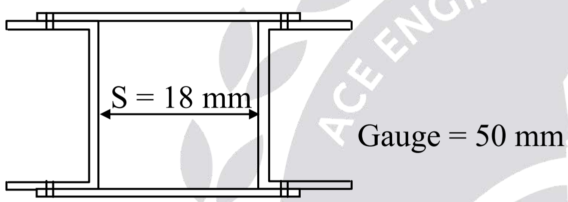


Design of lacings:

Provide single flat lacings with an angle 45° with respect to longitudinal axis of built up column. Let L and *l* be the spacing of lacings and length of lacing member respectively.

Distance between centroid of bolts





a = 183 + 50 + 50 = 283 mm

$$\tan 45° = \frac{a}{L/2} = \frac{283}{L/2}$$

L = 2 × 283 = 566 mm

$$\sin 45° = \frac{a}{\ell}$$

$$\ell = \frac{a}{\sin 45°} = \frac{283}{\sin 45°} = 400\,\text{mm}$$

(a) Check the local buckling failure of individual column component

$$\frac{L}{r_{min}^2} \le 50$$

$$\le 0.7 \times \left(\frac{KL}{r}\right) \text{of built up column}$$

⇒ whichever is lesser

$$\frac{L}{r_{yy}} = \frac{566}{26} = 21.7 \le 50 \le 59.3$$

⇒ whichever is less

Hence individual column component is free from local buckling

(b) Check the local buckling failure of lacing member.

Slenderness ratio of lacing member,

$$\frac{KL}{r_{min}} \le 145$$

Effective length of lacing member (for welding = 0.7 l)

For bolting,

Kl = l = 400 mm

Min width of flat lacing member

b = 3d = 3 × 16 = 50 mm

Minimum thickness of single flat lacing

$$t \ge \frac{\ell}{40} \ge \frac{400}{40} = 10\,\text{mm}$$

Adopt size of flat lacing member as 50ISF 10

Minimum radius of gyration of flat lacing member

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{50\times10^3}{12}\frac{1}{50\times10}}$$

$$= \frac{10}{\sqrt{12}} = 0.288\text{ mm}$$

$$\frac{KL}{r_{min}} = \frac{400}{2.88} = 138.8 \le 145$$

Hence lacing member is free form local buckling

(c) $40° \le \theta \le 70°$

Hence ok,

Transverse shear force

V = 2.5 × Design load column load

$$= 2.5 \times \frac{1125}{100}$$

V = 28.10 kN

Design axial force in lacing member

$$F = \frac{V}{N\sin\theta} = \frac{28.10}{2\sin 45°}$$

F = 19.88 kN

For $\frac{kL}{r_{min}} = 138.8$

Use $\frac{kL}{r_{min}} = 130 \quad 140$

$f_{cd} = 74 \quad 65$

$$\therefore \quad f_{cd} = 74 - \frac{(74-65)}{(140-130)}(138.8-130)$$

$$\left(\text{for} \frac{kL}{r_{min}} = 138.8\right)$$

$f_{cd} = 66.08\ \text{N/mm}^2$

Design compressive strength of lacing member

$P_d = f_{cd} \times A_e$

$= 66.08 \times 50 \times 10$

$P_d = 33.04 \ge F = 19.88$ N

Which is safe

Design tensile strength of flat based on gross section yielding

$$T_{dg} = A_g \frac{f_y}{\gamma_{m_o}}$$

$$= 50 \times 10 \times \frac{250}{1.10}$$

$T_{dg} = 113.6\ \text{kN} \ge F = 19.88$ kN

Design tensile strength based on rupture

$$T_{dn} = 0.9A_n \frac{f_u}{\gamma_{m_1}} = 0.9(50 - 1\times18) \times \frac{410}{1.25} \times 10$$

$T_{dn} = 94.2\ \text{kN} > F = 19.88$ kN

07. Column Bases & Column Splices

02. Ans: 9

Sol: Bearing strength of concrete is $0.45\ f_{ck}$

$f_{ck} = 20$ Mpa

$= 0.45 \times 20 = 9\ \text{N/mm}^2$

03. Ans: (b)

Sol: We know that allowable bearing strength of concrete $= \frac{\text{factored load (P)}}{\text{area of base slab}}$

Here maximum bearing strength is $0.45\ f_{ck}$

$$\therefore 0.45\ f_{ck} = \frac{2000\times10^3}{650\times420}$$

$\therefore \quad f_{ck} = 16.28$

$\therefore$ Minimum grade of concrete required is M20

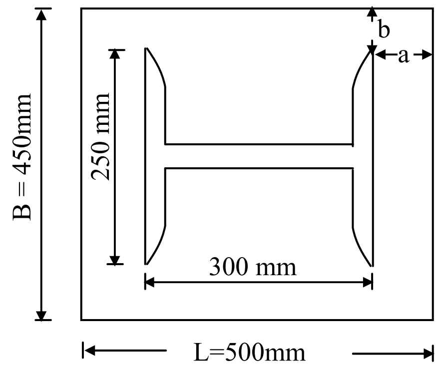
04. Ans: 26.3

Sol: $t_s = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)\gamma_{mo}}{f_y}} > t_f$

$w = 9.0\ \text{N/mm}^2$;

$t_f = 11.6$ mm , $b_f = 250$ mm

D = 300 mm



$$a = \frac{500-300}{2} \qquad a = 100$$

$$b = \frac{450-250}{2} \qquad b = 100$$

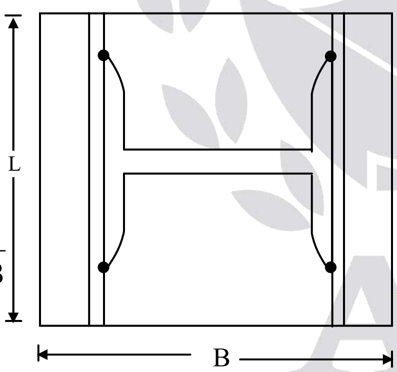
$$t_s = \sqrt{\frac{2.5\times 9\times\left(100^2 - 0.3\times 100^2\right)\times 1.10}{250}}$$

$= 26.3$ mm$>$ $t_f = 11.6$mm

05. Ans: (b)

Sol:

$$w = \frac{P}{L\times B}$$



$$t = C\sqrt{\frac{2.75\,w}{f_y}} \qquad W = \frac{P}{L\times B}$$

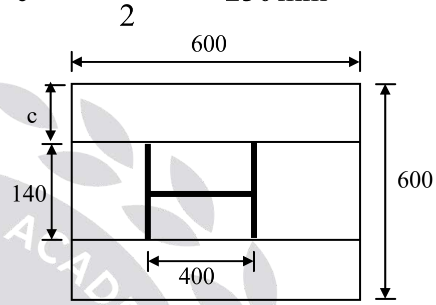
$\therefore$ If area$\uparrow$, the W$\downarrow$ then t$\downarrow$ but here area $\Rightarrow$ same

So $\sqrt{\dfrac{W}{f_y}} \Rightarrow$ constant for all options

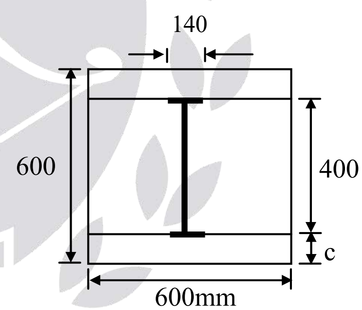
$\therefore$ t α c

Gusset plate to be provided along full width of base plate cantilever projection is available along length of base plate direction

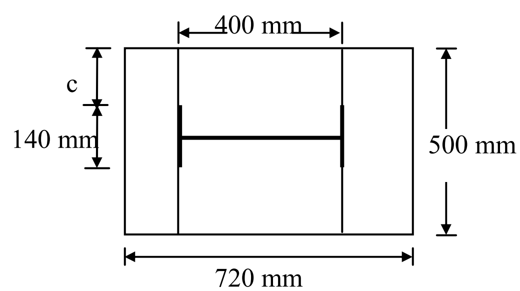
a) $c = \dfrac{600-140}{2} = 230\,\text{mm}$



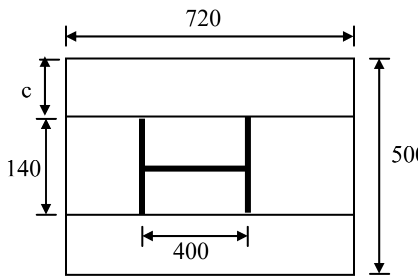
b) $c = \dfrac{600-400}{2}$ =100mm



c) $c = \dfrac{500-140}{2} = 180\,\text{mm}$

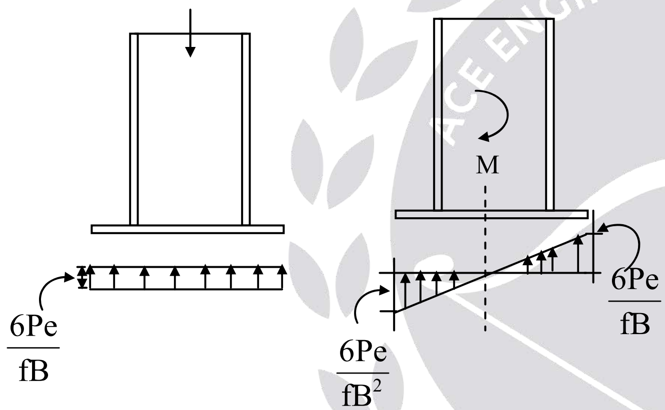


d) $c = \dfrac{720-400}{2} = 160$ mm



06. Ans: (d)

Sol:

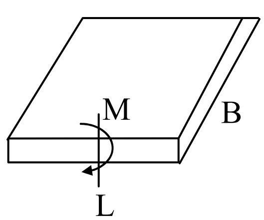


$$\text{Bending stress} = \frac{M}{Z} = \frac{Pe}{\frac{BL^2}{6}} = \frac{6Pe}{BL^2}$$

$$\text{Combined stress} = \frac{P}{LB} \pm \frac{6Pe}{BL^2}$$

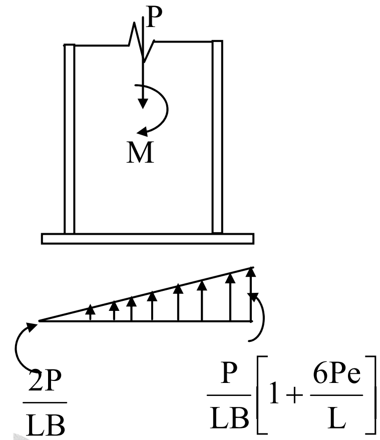
$$= \frac{P}{LB}\left[1 \pm \frac{6e}{L}\right]$$

$$Z = \frac{I}{y} = \frac{BL^3}{12} \times \left(\frac{2}{L}\right) = \frac{BL^2}{6}$$



$$= \frac{BL^2}{6}$$

$\therefore$ Zero at one end compression at other end.



10. Ans: (a)

Sol: Total axial compressive load = 1600 kN

Rivet valve R_V = 40 kN

Number of rivets required for 4 sides

$$= \frac{1600}{40} = 40 \text{ no's}$$

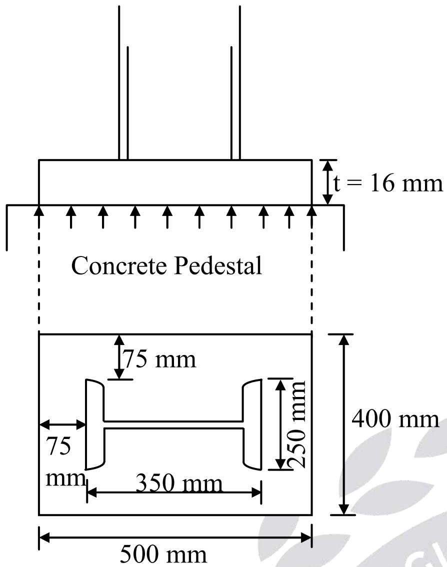
Number of rivets required on each side

$$= \frac{40}{4} = 10 \text{ no's}$$

Conventional Practice Solutions

01. A slab base for steel column of **ISHB 350** consists of steel plate 500×400×16mm, supported on concrete pedestal made of M20 concrete. If the column is subjected to factored axial load of 1800 kN. Check the adequacy of the slab base provided. The properties of ISHB350@ 486 N/m are Depth of beam section (h) = 350 mm Width of flange (b_f)= 250 mm Thickness of flange (t_f) = 13.6 mm
Yield stress of steel (f_y) = 250 N/mm^2
Partial safety factor (γ_{mo}) = 1.10

Sol:



From the given figure

Area of base plate (A_P) = 500 × 400 mm^2

Thickness of base plate (t) = 16 mm

Larger & shorter projection (a & b) = 75 mm & 75 mm respectively

- Concrete pressure (w) $= \dfrac{\text{factored load } (P_u)}{A_P}$

$$= \frac{1800\times10^3}{500\times400} = 9 \text{ MPa}$$

For M20 concrete, bearing strength = $0.45f_{ck}$ = 9 MPa

Hence concrete is safe in bearing

- Minimum thickness required of plate to prevent bending failure of plate,

$$(t_{required}) = \sqrt{\frac{2.5w(a^2-0.3b^2)}{\frac{f_y}{\gamma_{mo}}}}$$

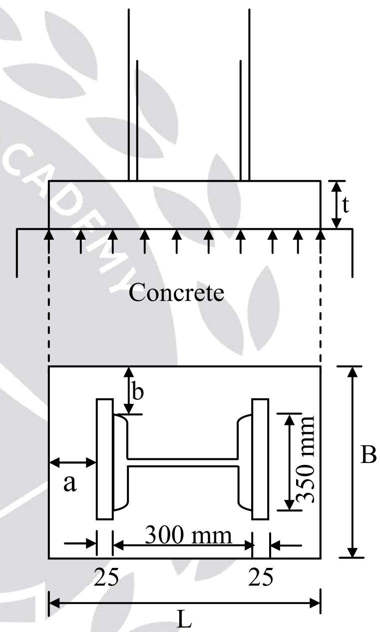
$t_{required)}$ = 19.74 > 16 mm

Hence thickness provided of slab is not safe

Minimum thickness should have been 19.74 mm ≃ 20 mm

02. Design a slab base for a column of one ISHB 300 and one cover plate 350×25mm on each side column flange carries an axial load of 2500 kN. The column is to be supported on concrete pedestal of M15 grade. The permissible bearing stress in slab base **185N/mm^2** and permissible bearing stress in concrete pedestal is 4 N/mm^2

Sol:



Given load (P) = 2500 kN

Concrete grade = M15

Depth of column = 300 mm

Width of flange = 350 mm

Permissible bearing stress in slab (σ_{bs}) = 185 MPa

Permissible bearing stress in concrete (w) = 4 MPa

- Area of base plate required to avoid bearing failure of concrete

$$A_{(P,\text{required})} = \frac{P}{4\ \text{MPa}}$$

$$= \frac{2500\times10^3}{4}$$

$$= 625000\ \text{mm}^2$$

- To design most efficient section, the dimensions must be designed such that a = b

L = (300 + 50 + 2a) = (350 + 2a)

B = (350 + 2b)

$L \times B = 625000\ \text{mm}^2$

$\Rightarrow$ a = b = 220.28 mm $\simeq$ 225 mm

Design L = B = 350 + 2 × 225 = 800 mm

- Minimum thickness required of plate to prevent bending failure of plate

$$(t) = \sqrt{\frac{3w(a^2 - 0.25b^2)}{\sigma_{bs}}} = 50.3\ \text{mm}$$

$\Rightarrow$ Design plate of dimensions are

800 mm × 800 mm × 55 mm

↓ ↓ ↓

(L) (B) (t)

08. Beams

08. Ans: 100

Sol: $Z_e = 500\ \text{cm}^3$, $Z_P = 650\ \text{cm}^3$

Laterally unrestrained beam semi compact section

Design bending compressive stress f_{bd} = 200 MPa

The flexural (or) bending strength

$M_d = \beta_b . Z_p . f_{bd}$

$$= \frac{Z_e}{Z_P}.Z_p.f_{bd}$$

$$= 500\times10^3 \times 200 = 100\times10^6\ \text{N-mm}$$

M_d = 100 kN-m

09. Ans : 669.2

Sol: ISMB = 500, h = 500 mm

$f_y = 250\ \text{N/mm}^2$, shear stress $= \dfrac{f_y}{\sqrt{3}}$

f_u = 410 MPa, $\gamma_{mo} = 1.10$, $\gamma_{m1} = 1.25$

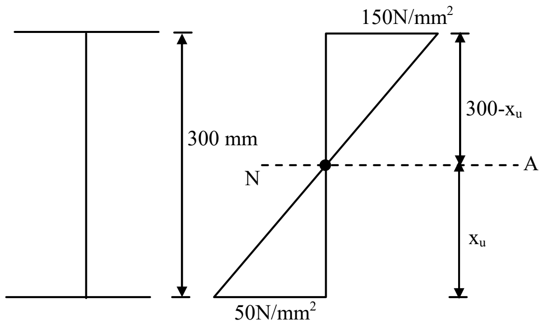
Design shear strength = (V_d) = Design shear stress × shear area

$$= \frac{f_y}{\sqrt{3}\gamma_{m0}} \times (h.t_w)$$

$$= \frac{(500\times10.2)\times250}{\sqrt{3}\times1.1} = 669.20\ \text{kN}$$

16. Ans: (c)

Sol:



We know that stress variation is linear let us assume height of neutral axis from bottom of beam is x_u.

From similar triangles

$$\frac{x_u}{50} = \frac{300 - x_u}{150}$$

$x_u = 75$ mm

17. Ans: (c)

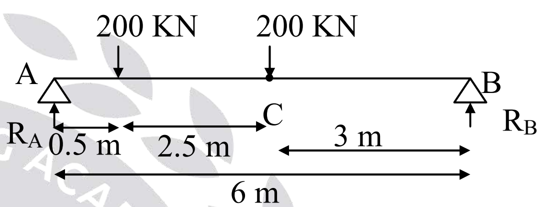
Sol: Flanges and cover plates in cover plates are un stiffened, the outstand of flange or cove plate from the time of connection should not exceed $\frac{256t}{\sqrt{f_y}}$ subjected to maximum of 16t.

Conventional Practice Solutions

01. Two wheels, placed at a distance of 2.5 m apart, with a load of 200 kN on each of them, are moving on a simply supported girder (I-section) of span 6.0 m. The top and bottom flanges of the I-section are of **200 x 20 mm** and the size of web plate is 800 x 6 mm. If the allowable bending compressive, bending tensile and average shear stresses are 110 MPa, 165 MPa and 100 MPa respectively, check the adequacy of the section against bending and shear stresses, self weight of the girder may be neglected.

Sol: Given

- Loading condition for maximum bending moment is shown below



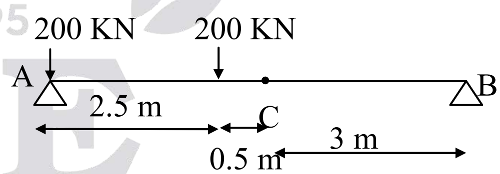
(span of Girder)

Design moment (M) at C = $R_B \times 3$

$$\left[R_A = \frac{200}{2} + 200 \times \frac{0.5}{6}\right]$$

= 350 kN-m

- Loading condition for maximum shear force is shown below

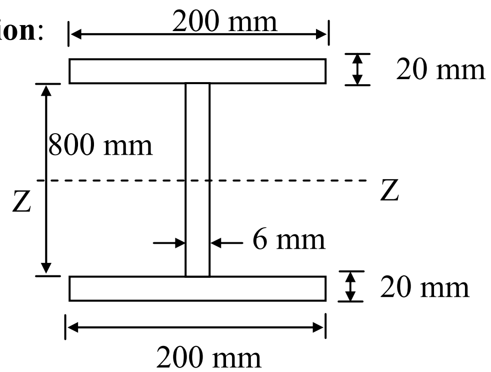


$\Rightarrow$ Design shear force (V) at A

$$= R_A = 200 + 200 \times \frac{3.5}{6}$$

= 316.67 kN

I – Section:



Area of web (A_w) = 800 × 6 = 4800 mm^2

I_{ZZ} of beam

$$= 2\times\left\{\frac{200\times 20^3}{12}+200\times 20\times 410^2\right\}+\left\{\frac{6\times 800^3}{12}\right\}$$

$= 1601.06 \times 10^6$ mm^4

Check for bending stress

Bending stresses developed @ extreme fibres ($f_{b,\,developed}$) $= \dfrac{M}{I}\times y$

$$= \frac{350\times 10^6}{1601\times 10^6}\times 420 \text{ MPa}$$

$= 91.8$ MPa

$f_{b,developed} < 110$ MPa (perm. Bending stress in compression), hence safe in bending

Check for shear

Shear stress developed ($\tau_{develop}$) $= \dfrac{V}{A_w}$

$$= \frac{316.67\times 10^3}{4800}\text{N/mm}^2 = 65.97 \text{ MPa}$$

$\tau_{develop} < 100$ MPa (Permissible Shear stress)

Hence safe in shear

∴ Given section is safe in bending & shear.

02.

Sol: Effective span, L = 5 m

Service loads:

UDL and DL, $w_d = 20$ kN/m

UD Load LL, $w_l = 20$ kN/m

Yield strength, $f_y = 250$ MPa

Note: for calculating deflections, service loads only used.

For moment of shear, factored loads only used.

$\gamma_{mo} = 1.10$; $\gamma_f = 1.50$

Failures considered:

- Tension flange yielding $= \dfrac{f_y}{\gamma_{mo}}$
- Compression flange buckling = restrained (or) not
- Web buckling $= \dfrac{d}{t_w} > 67$
- Web yielding $= V \le 0.6\ V_d$

Plastic classification:

(i) $\dfrac{b}{f_y} \le 9.4$

(ii) $\dfrac{d}{t_w} \le 67\varepsilon$

Calculation of design (or) factored loads:

Factored dead load $= \gamma_f \times w_d$

$= 1.5 \times 20 = 30$ kN/m = FLL

Factored DL & LL

w = 60 kN/m

Design BM:

$$M = \frac{w\ell^2}{8} = \frac{60\times 5^2}{8}$$

M = 187.5 kN-m

Design SF:

$$V = \frac{w\ell}{2} = \frac{60\times 5}{2} \qquad V = 150 \text{ kN}$$

Section dessification

$$b = \frac{b_f}{2} = \frac{165}{2} = 82.5\text{mm}$$

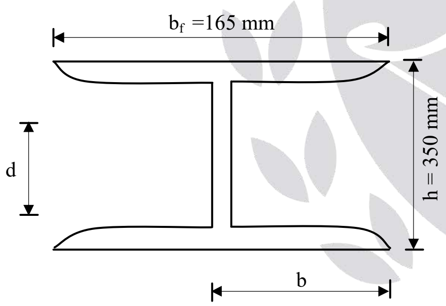
$t_f = 11.4$ mm; $t_w = 7.4$ mm

Depth of web between root of fillets

$d = h - 2 \times (t_f + r_1)$

$= 350 - 2 \times (11.4 + 16)$

d = 295.2 mm



h = 350 mm; b_h = 165 mm

t_w = 7.4 mm; t_f = 11.4 mm

r_1 = 16 mm

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.24 \le 9.4$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} \le 67\varepsilon = 67\times 1 = 67$$

Hence the section is plastic section

Check for low shear case:

Design shear strength of section.

$$V_d = A_v \times \frac{f_y}{\sqrt{3}\gamma_{mo}} = (h\times t_w)\times\frac{f_y}{\sqrt{3}\gamma_{mo}}$$

$$V_d = 350\times 7.4\times\frac{250}{\sqrt{3\times 1.10}}$$

V_d = 339.85 kN ≥ V = 150 kN

(which is safe against shear)

When low shear case V ≤ 0.6 V_d

150 kN = 0.6 × 339.85 = 203.91 kN

Design bending strength of section

$$M_d = \beta_b.z_p.\frac{f_y}{\gamma_{mo}} \le 1.2Z_e.\frac{f_y}{\gamma_{mo}}$$

(For plastic section β_b = 1.0)

$$M_d = 1.0\times 857.11\times 10^3\times\frac{250}{1.10}$$

$$= 193.43\times 10^6 \le 1.2\times 151.9\times 10^3\times\frac{250}{1.10}$$

M_d = 193.43 kN-m ≤ 205.06 kN-m

Also,

M_d = 193.43 kN-m ≥ M = 187.5 kN-m

Which is safe bending strength

Design for deflections:

Limit deflections for Brittlesss

$$\text{cladding} = \frac{L}{300} = \frac{5000}{300} = 16.67 \text{ mm}$$

Maximum calculated deflect under service loads.

$$\Delta_{cal} = \frac{5}{384}\times\frac{w\ell^4}{EI}$$

$$\Delta_{cal} = \frac{5}{384} \times \frac{40 \times 5000^4}{2 \times 10^5 \times (751.9 \times 10^3 \times \frac{350}{2})}$$

$\Delta_{cal} = 12.36$ mm $\leq \Delta_{limt} = 16.67$ mm

Which is safe against deflections

03.

Sol: Effective span of beam $L = 5.0$ m

Yield stress of steel $f_y = 250$ Mpa

Section classification

Section classification $\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$

Outstand of flange $b = \frac{b_f}{2} = \frac{165}{2} = 82.5\,mm$

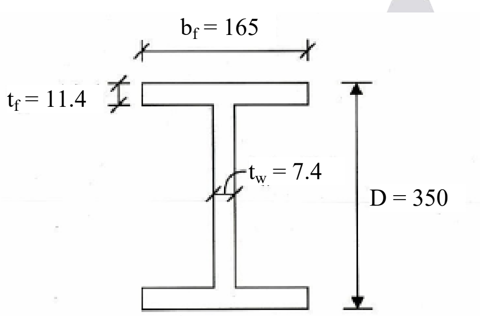
Depth of web $d = h - 2\times(t_f + R_1)$

$= 300 - 2\times(13.1+14)$

$= 245.8$mm

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.23 < 9.4$$

Hence section is plastic section



Built up I section

(All dimensions are in mm)

Moment of inertia of joist about major axis

$$I_{zz} = 2\times\left(\frac{165\times 11.4^3}{12} + 165\times 11.4\times\left(\frac{350}{2} - \frac{11.4}{2}\right)^2\right) + \frac{7.4\times(350-22.8)^3}{12}$$

$I_{zz} = 129.47\times 10^6$ mm^4

Elastic section modulus about major axis Z_{ez}

$$Z_{ez} = \frac{I_{ZZ}}{y} = \frac{129.47\times 10^6}{(350/2)} = 739.83\times 10^3\,\text{mm}^3$$

Plastic section modulus about major axis Z_{pz}

$$Z_{pz} = 2\times 165\times 11.4\times\frac{(350-11.4)}{2} + 2\times\left[7.4\times\left(\frac{350}{2} - 11.4\right)\times\frac{\left(\frac{350}{2} - 11.4\right)}{2}\right]$$

$= 834.97 \times 10^3$ mm^3

Since $V \leq 0.6\ V_d$ this is low shear case

Design bending strength of laterally supported beam

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \leq 1.2 Ze \frac{f_y}{\gamma_{mo}}$$

For plastic section $\beta_b = 1$

$$M_d = 1.0\times 834.97\times 10^3 \times \frac{250}{1.10}$$

$= 189.76\times 10^6$ N-mm $= 189.76$ kN-m

$$\leq 1.2 Ze\frac{f_y}{\gamma_{mo}} = 1.2\times 739.83\times 10^3\times\frac{250}{1.10}$$

$= 201.77\times10^6$ N-mm = 201.77 kN-m

Which is all right

Assume w be factored distributed load on beam inclusive of self weight in kN/m

Design bending moment about bending axis

$$M_{zz} = \frac{wl^2}{8} = \frac{w\times(5)^2}{8} = 3.125\ \text{w kN-m}$$

Equating $M_{zz} = M_d$

$3.125\ w = 189.76$

$\Rightarrow w = 60.72$ kN/m (1)

Design shear strength of joist V_d

$$V_d = A_v \frac{f_y}{\sqrt{3}\gamma_{mo}} = h.\times t_w \frac{f_y}{\sqrt{3}\gamma_{mo}}$$

$$= 350\times7.4\frac{250}{\sqrt{3}\times1.10} = 339.85\ kN$$

Design shear force V

$$V = \frac{wl}{2} = \frac{w\times(5)}{2} = 2.5\ w$$

Equating $V = V_d$

$2.5\ w = 339.85$

$\Rightarrow w = 135.94$ kN/m (2)

Limiting deflection for a simply supported beam (assuming brittle cladding)

$$\Delta_{limit} = \frac{\ell}{300} = \frac{5000}{300} = 16.66\text{mm}$$

Calculated maximum deflection under service loads

Service UDL on beam $w_s = w/\gamma_f$

$= 10/1.5 = 6.67$ kN/m

$$\Delta l_{imit=}\Delta_{cal} = \frac{5}{384}\frac{w_s L^4}{EI}$$

$$16.67 = \frac{5}{384}\times\frac{(w_s)(5000)^4}{2\times10^5\times129.47\times10^6}$$

$\Rightarrow$ $w_s = 53.04$ kN/m (3)

Factored distributed load on beam

$w = \gamma_f \times w_s = 1.5 \times 53.04 = 79.56$ kN/m

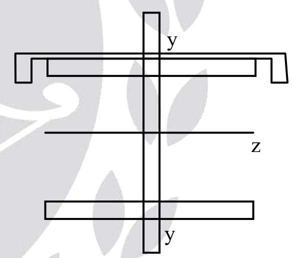
Factored distributed load on beam inclusive of self weight

$w = 60.72$ kN/m

10. Gantry Girders

01. Ans: (a)

Sol:



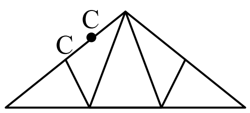
$I_{yy} = (I_{yy\ of\ ISWB} + I_{zz\ channel})$

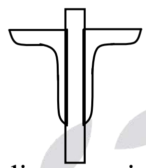
11. Roof Trusses

01. Ans: (d)

Sol: Rigidity due to weld connections

$\Rightarrow$ Rigidity will be there top chord member bottom chord member.

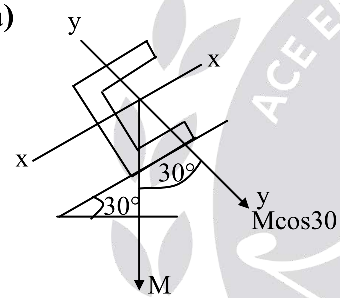




Purlins if placed of intermediate section then cause secondary stresses

02. Ans: (a)

Sol:

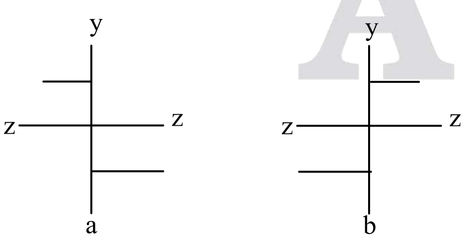


$$M_{xx} = M\cos\theta\left(\theta = 30^0\right) = \frac{\sqrt{3}}{2}M$$

$$M_{yy} = M\sin\theta = \frac{M}{2}$$

03. Ans: (c)

Sol:



Both above arrangements are provide same section modulus about y-y & z-z axis, hence both arrangements are Equally & efficient.

04. Ans: (a) **05. Ans: (c)** **08. Ans: (a)**

10. Ans: (b)

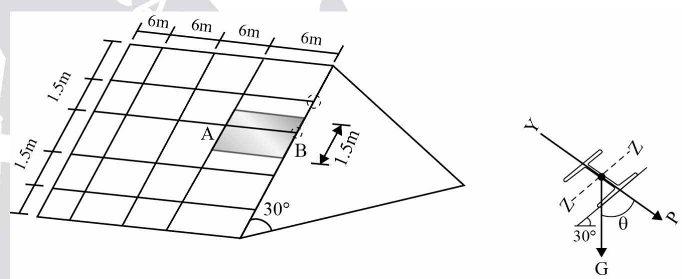
Sol: Slop of roof = tanθ

$\tan\theta = 1 \Rightarrow \theta = \tan^{-1} = 45^o$

Conventional Practice Solutions

01.

Sol:



Given:

Slope of truss (θ) = 30°

Wind pressure (P) = 2 kN/m^2

= 2 kN/m^2 × 1.5 m

= 3 kN/m

Self weight = 166.77 N/m of purlin

Weight of galvanised sheet = 130 N/m^2

= 130 N/m^2 × 1.5 = 195 N/m

Total gravity load(G)=195 N/m+166.77 N/m

= 361.77 N/m

span of purlin (l) = 6 m

Total factored moment developed along Z-Z

$$(M_{uzz}) = \gamma_f \times (P + G\cos\theta) \times \frac{\ell^2}{10}$$

$$= 1.5 \times (3 + 0.361) \times \frac{6^2}{10}$$

= 18.14 kN-m

Total factored moment developed along Y-Y

$$(M_{uyy}) = \gamma_f \times G \sin\theta \times \frac{\ell^2}{10}$$

= 1.95 kN-m

As per IS 800:2007

Design moment capacity along z-z (M_{dzz})

$$= \frac{f_y}{\gamma_{mo}} \times Z_{pzz}$$

= 25.95 kN-m

Design moment capacity along y-y (M_{dyy})

$$= \frac{f_y}{\gamma_{mo}} \times Z_{pyy}$$

= 6.58 kN-m

Design shall be considered safe as per IS800:2007 if interaction equation is satisfy

$$\left(\frac{M_{uzz}}{M_{dzz}}\right) + \left(\frac{M_{uyy}}{M_{dyy}}\right)$$

= 0.955 < 1 Hence the design is safe