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CIVIL ENGINEERING

STEEL STRUCTURES

Volume - 1 : Study Material with Classroom Practice Questions

Design of Steel Structures

Solutions for Volume : I Classroom Practice Questions

Chapter- 2
Riveted Connections

Class Room Practice Questions

01. Ans: 60%

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

$p_{min} = 2.5\phi$

$d = \phi + 1.5$

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

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

$\eta = 60\%$

05. Ans: (b)

Sol: ISA $50 \times 50 \times 6$

$t = 5$ mm

$d = 16$ mm

$\sigma_{bt} = 250$ MPa

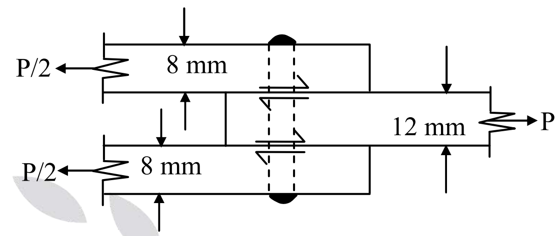
$P_b = d \times t \times \sigma_{bt}$

$= 16 \times 5 \times 250$

$= 20$ 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: $$P_s = 2\times\frac{\pi}{4}(d)^2\times\tau_{vf} = 80\,\text{kN}$$

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

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

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

10. Ans: (d)

Sol: Minimum pitch of rivets in compression zone

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

$t = 10$ mm

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

Pitch = 120 mm

Design of Simple bolted connection

03. Ans: (d)

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

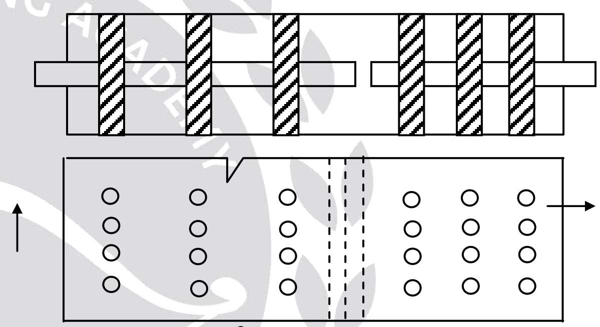
$f_y = 0.6\,f_u$

$= 0.6 \times 400$

$= 240$ N/mm^2

05. 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

$= 3 \times 2 = 6$

06. 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

07. Ans: (d)

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

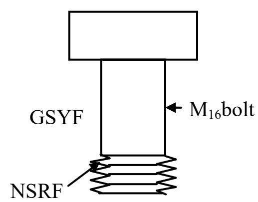
$$T_b = \frac{P_u\cos\theta}{n} = \frac{250}{6}\times\frac{4}{5} = 33.33\text{ kN}$$

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

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

08. 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

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

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

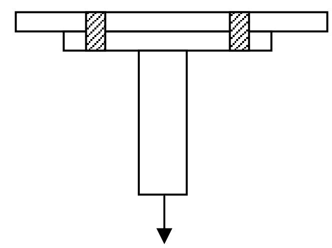
$$T_{db_2} = 45.166 \text{ kN}$$

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

$\therefore T_{db} = 43.86$ kN

09. Ans: (c)

Sol: Hanger connection looks like this one



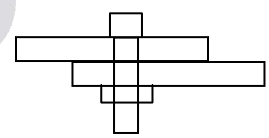
In hanger connections bolts experience only tensile stress. Then we clearly taken, Strength of rivet is equal to the design strength of bolt in tension.

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

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

10. 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%

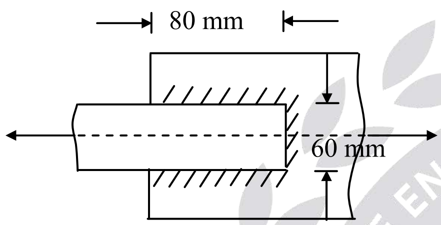
Chapter- 3
Welded Connections

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 ⇒

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

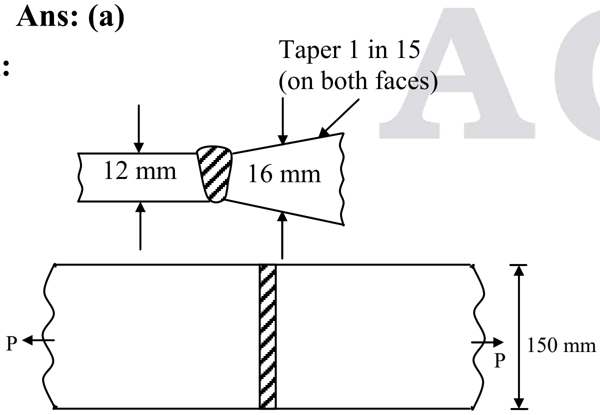
L_W = Effective length of fillet weld

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

$= 135$ 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

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

[∵ In case of single 'V' butt weld

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

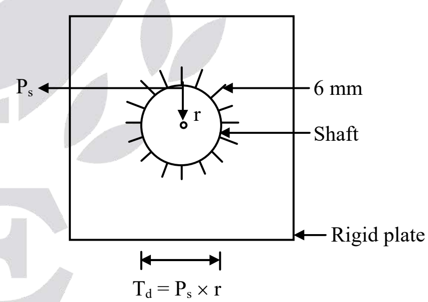
$= 150 \times \dfrac{5}{8} \times 12 \times 150$

$= 168.78 \times 10^3$N

$P = 168.75$ kN

10. Ans: (b)

Sol:

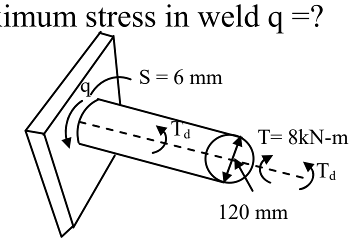


Size of weld (S) = 6 mm

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;

γ_{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 mm $\simeq$ 105 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

Chapter- 4
Eccentric Connections

02. Ans: (b)

Sol:

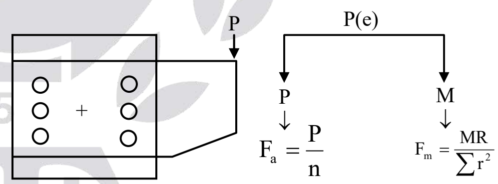
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

$x^2 + y^2 = r^2$ (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

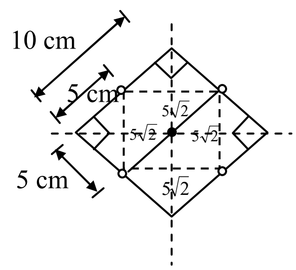
Force in rivet 1 due to direct loading = $\frac{P}{4}$

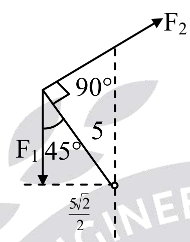
$F_1 = 2.5$ kN

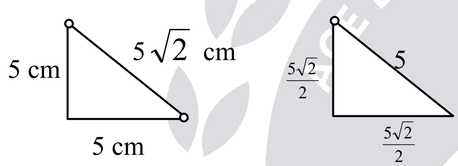
Force in rivet 1 due to twisting moment

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

M = P[150] kN-mm







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

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

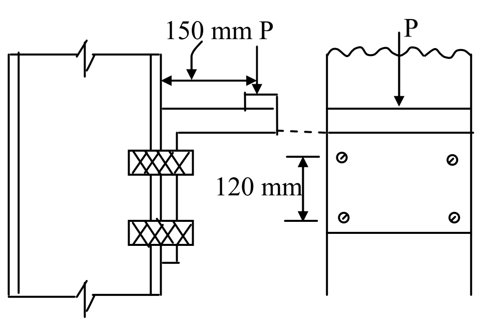
$\theta = 135°$

$F_R = 5.99$ 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

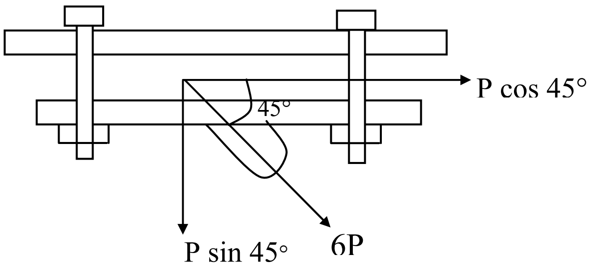
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

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

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

07. Ans: (d)

Sol:



Design shear strength of bolt V_{db} = 30 kN

Design tensile strength of bolt T_{db} = 40 kN

Factored shear force in any bolt due to vertical component of 6P

$$V_b = \frac{P\ \cos\ 45^o}{4} = \frac{P}{4\sqrt{2}}$$

Factored tensile force in any bolt due to vertical component of 6P

$$T_b = \frac{P\ \sin\ 45^o}{n} = \frac{P\ \sin\ 45^o}{4}$$

$$= \frac{P}{4\sqrt{2}}$$

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

$$\left(\frac{P}{4\sqrt{2}} \times \frac{1}{30}\right)^2 + \left[\frac{P}{4\sqrt{2}} \times \frac{1}{40}\right]^2 \le 1.0$$

$$\frac{P^2}{16\times 2\times 900} + \frac{P^2}{16\times 2\times 1600} = 1$$

P = 135.76 kN

The maximum design load applied can be on the joint is 135.76 kN

08. Ans: 173.21 MPa

Sol: Equivalent stress of q_1& q_2 act on the fillet weld is f_e

$$f_e = \sqrt{3(q_{1cal})^2 + (q_{2cal})^2}$$

$$= \sqrt{3\times(50)^2 + (150)^2}$$

= 173.21 MPa

10. Ans: (c)

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

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

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

11. Ans: (c)

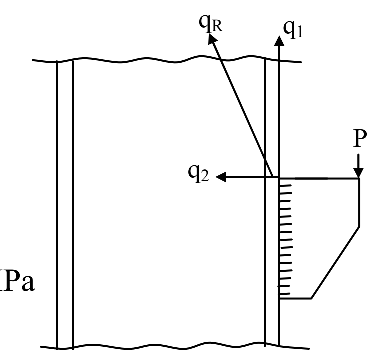
Sol

q_1 = 30 MPa,

q_2 = 120 MPa

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

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



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

Chapter- 5
Tension Members

01. Ans: 240

Sol: $f_y = 400$ MPa

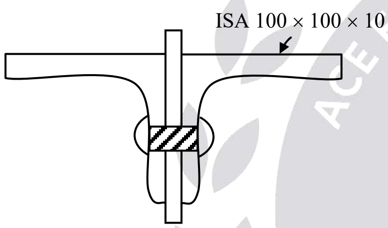
Permissible tensile stress $(\sigma_{at}) = 0.6 \times f_y$

$= 0.6 \times 400$

$\sigma_{at} = 240$ MPa

02. Ans: (d)

Sol:



$$ISA = 100 \times 100 \times 10$$

$$A_1 = 775 \text{ mm}^2$$

$$A_2 = 950 \text{ mm}^2$$

$$A_{net} = A_1 + A_2K_1$$

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

K_1 = Reduction factor = 0.709

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

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

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

03. Ans: (c)

Sol:

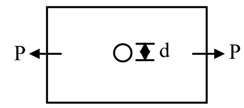


Fig. 1

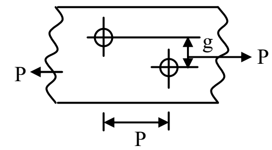


Fig. 2

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

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

$$P_t \propto A_{net}$$

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

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

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

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

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

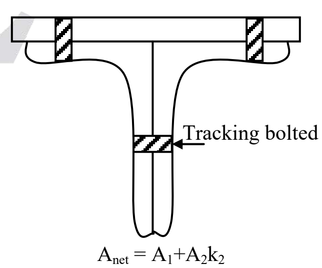
$$\frac{P^2 t}{4g} > dt$$

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

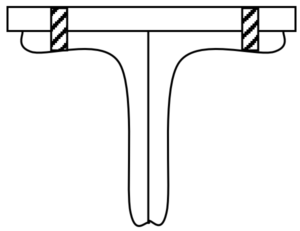
05. Ans: (c)

Sol:

(a)

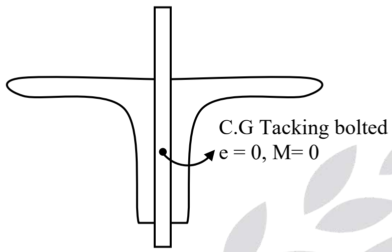


(b)



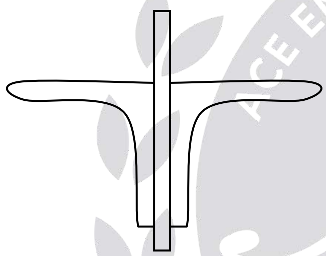
$A_{net} = A_1 + A_2 k_2$

(c)



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

(d)



$A_{net} = A_1 + A_2 k_1$

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

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

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

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

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

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

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

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

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

$$L = 350 \times 4$$

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

Chapter- 6
Compression Members

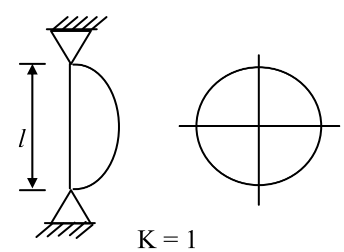
01. Ans: 50

Sol: KL = effective length

K = effective length constant

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

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



K = 1

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

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

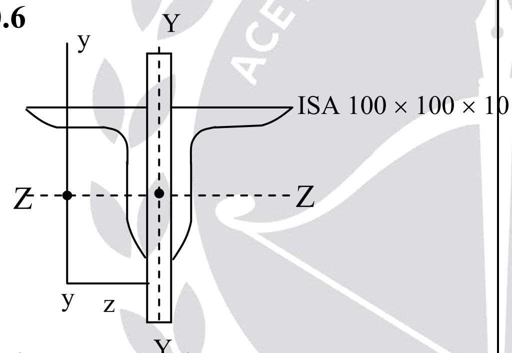
$$= \sqrt{\frac{d^2}{16}} = \frac{d}{4}$$

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

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

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

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

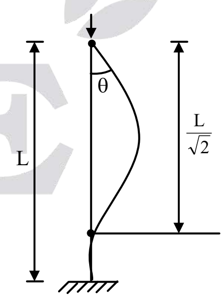
$$= \frac{37.5}{2 \times \sin 45^0}$$

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

= 26.52 kN

08. Ans: (b)

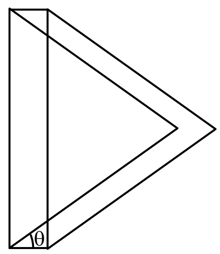
Sol:



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

09. Ans: (d)

Sol:



Skin buckling

Chapter- 7
Column Bases & Column Splices

02. Ans: 9

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

f_{ck} = 20 Mpa

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

03. Ans: (b)

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

Here maximum bearing strength is 0.45 f_{ck}

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

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

∴ Minimum grade of concrete required is M20

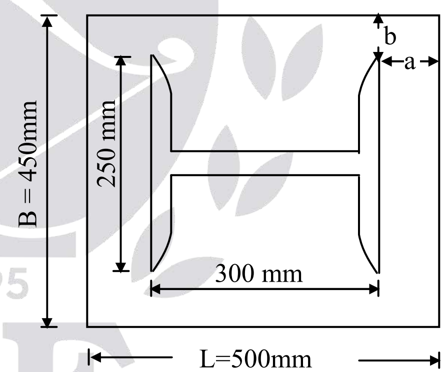
04. Ans: 26.3

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

w = 9.0 N/mm^2;

t_f = 11.6 mm , b_f = 250 mm

D = 300 mm



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

a = 100

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

b = 100

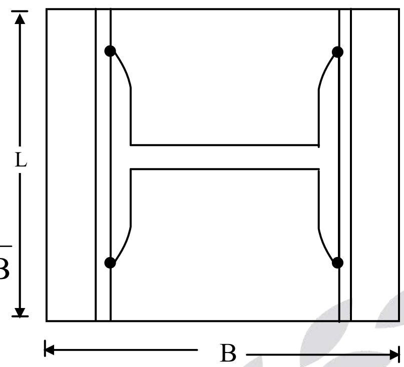
$$t_s = \sqrt{\frac{2.5\times9\times\left(100^2 - 0.3\times100^2\right)\times1.10}{250}}$$

= 26.3 mm> t_f = 11.6mm

05. Ans: (b)

Sol:

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



$$t = C\sqrt{\frac{2.75\,w}{f_y}}$$

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

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

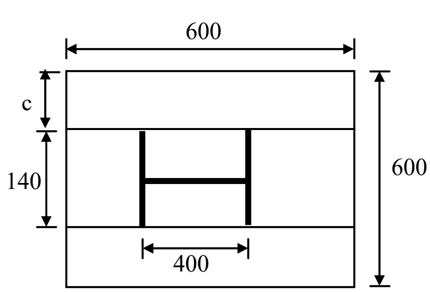
$\Rightarrow$ same

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

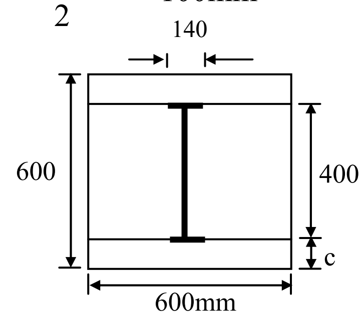
$\therefore$ t α c

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

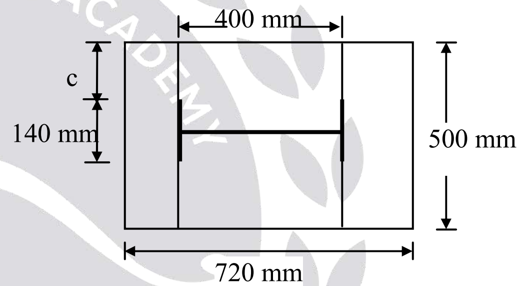
a) $c = \frac{600-140}{2} = 230\,\text{mm}$



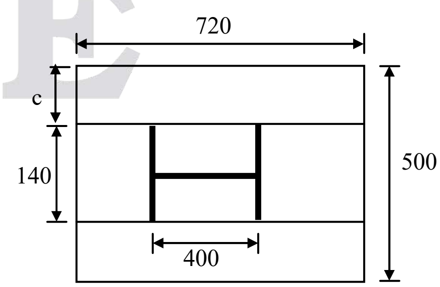
b) $c = \frac{600-400}{2} = 100\text{mm}$



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

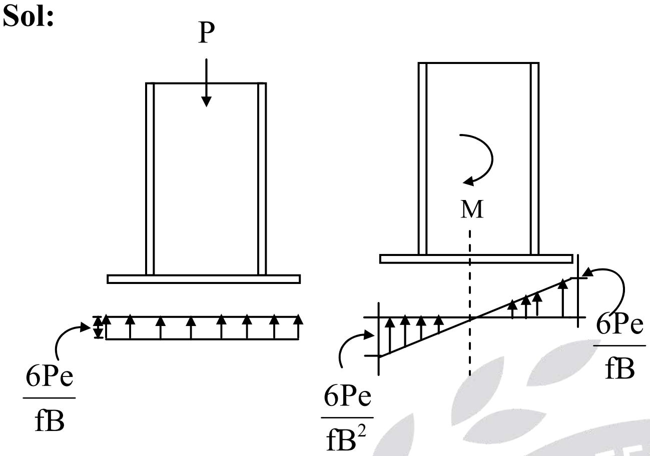


d) $c = \frac{720-400}{2} = 160$ mm



06. Ans: (d)

Sol:



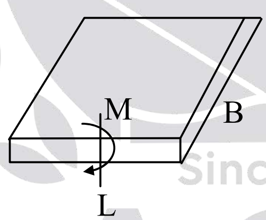
$$\text{Bending stress} = \frac{M}{Z} = \frac{Pe}{\frac{BL^2}{6}} = \frac{6Pe}{BL^2}$$

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

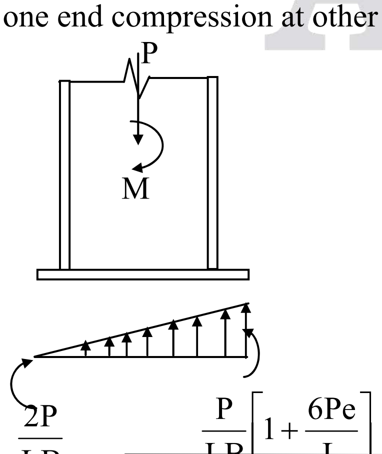
$$= \frac{P}{LB}\left[1 \pm \frac{6e}{L}\right]$$

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

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



$\therefore$ 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

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

Chapter- 8
Beams

08. Ans: 100

Sol: $Z_e = 500$ cm^3, $Z_P = 650$ cm^3

Laterally unrestrained beam semi compact section

Design bending compressive stress $f_{bd} = 200$ MPa

The flexural (or) bending strength

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

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

$$= 500 \times 10^3 \times 200 = 100 \times 10^6 \text{ N-mm}$$

M_d = 100 kN-m

09. Ans : 669.2

Sol: ISMB = 500, h = 500 mm

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

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

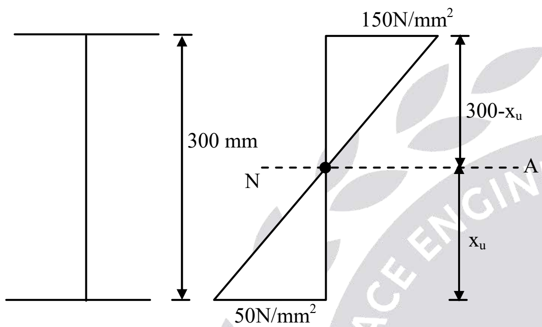
Design shear strength = (V_d) = Design shear stress × shear area

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

$$= \frac{(500 \times 10.2) \times 250}{\sqrt{3} \times 1.1} = 669.20 \text{ kN}$$

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

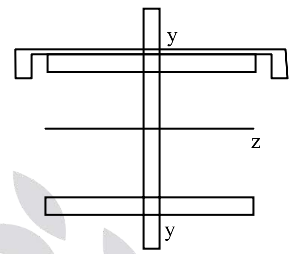
17. Ans: (c)

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.

Chapter- 10
Gantry Girders

01. Ans: (a)

Sol:



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

Chapter- 11
Roof Trusses

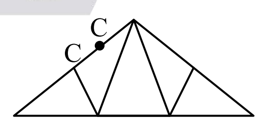
Class Room Practice Solutions

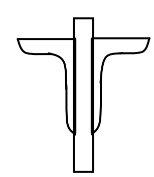
01. Ans: (d)

Sol:

Rigidity due to weld connections

⇒ Rigidity will be there top chord member bottom chord member.

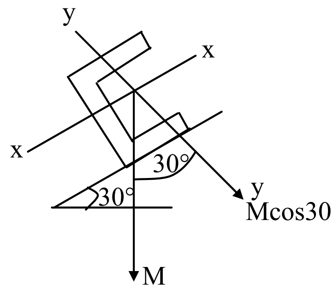




Purlins if placed of intermediate section then cause secondary stresses

02. Ans: (a)

Sol:

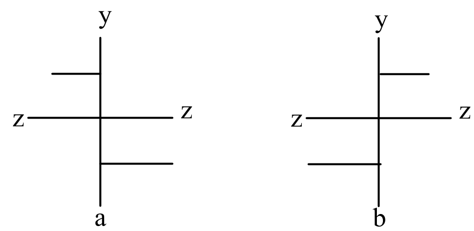


$$M_{xx} = M\cos\theta\left(\theta = 30^0\right) = \frac{\sqrt{3}}{2}M$$

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

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 = $\tan\theta$

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