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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 × 50 × 6

t = 5 mm

d = 16 mm

$\sigma_{bt} = 250$ MPa

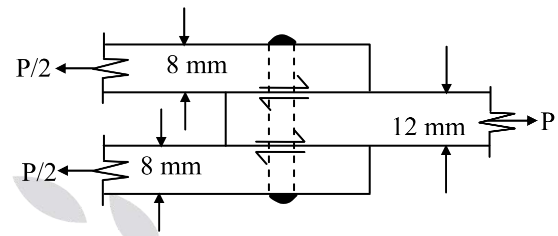
$P_b = d \times t \times \sigma_{bt}$

$= 16 \times 5 \times 250$

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

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

$P_s = 40$ 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

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

t = 10 mm

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

Pitch = 120 mm

Design of Simple bolted connection

03. Ans: (d)

Sol: $f_u = 400$ N/mm²

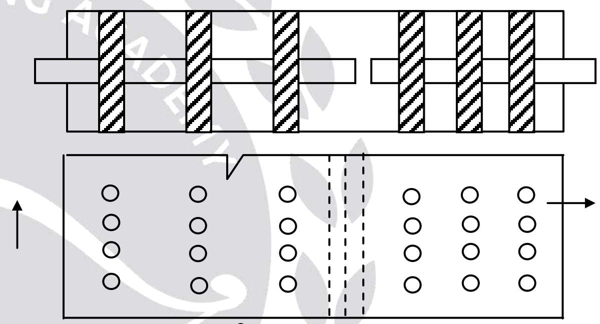
$f_y = 0.6\, f_u$

$= 0.6 \times 400$

$= 240$ N/mm²

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 $P_u \cos\theta$

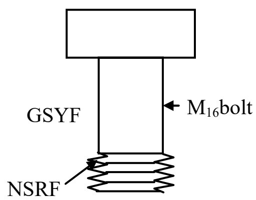
$$T_b = \frac{P_u\cos\theta}{n} = \frac{250}{6}\times\frac{4}{5} = 33.33\ \text{kN}$$

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

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

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

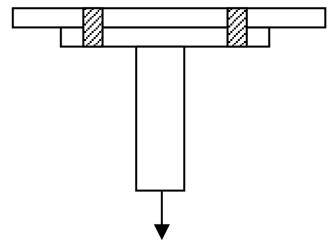
$$T_{db_2} = 45.166 \text{ kN}$$

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



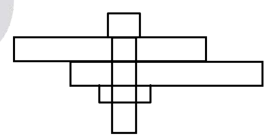
In hanger connections bolts experience only tensile stress. Then we clearly taken, Strength of rivet is equal to the design strength of bolt in tension.

$\overline{P}_{db} = 45 \text{kN}$ = 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%

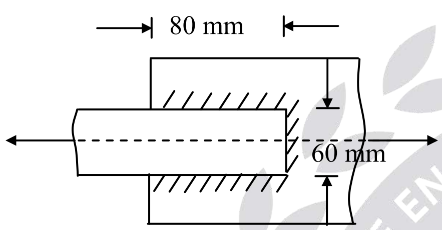
Chapter- 3
Welded Connections

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 ⇒

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

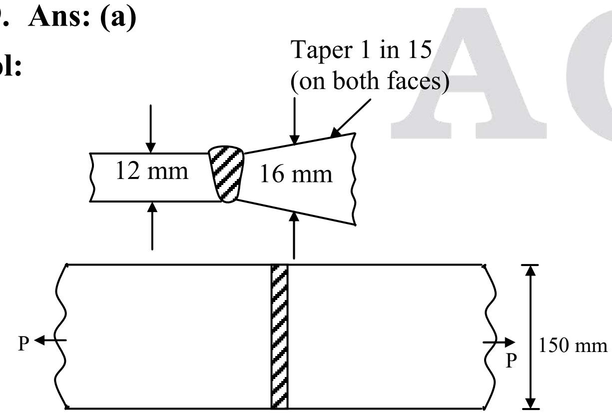
L_W = Effective length of fillet weld

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

$= 135$ 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

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

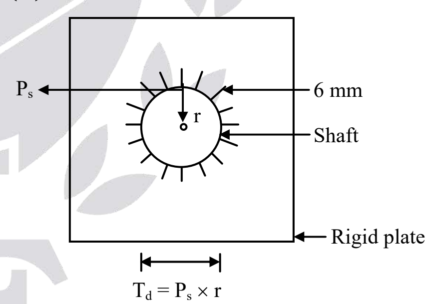
$= 150 \times \frac{5}{8} \times 12 \times 150$

$= 168.78 \times 10^3$N

$P = 168.75$ kN

10. Ans: (b)

Sol:

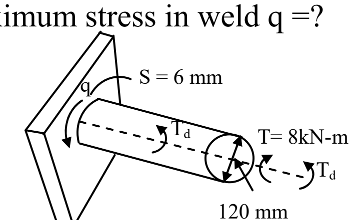


Size of weld (S) = 6 mm

Torque $\quad T = 8$ kN-m

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

Maximum stress in weld q =?



Twisting moment capacity of weld

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

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

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

$$8 \times 10^6 = \pi \times 120 \times (0.7 \times 6) \times q \times \frac{120}{2}$$

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

11. Ans: (b)

Sol:

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

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

$= f_{yw}$

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

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

$$\Rightarrow 270 \times 10^3 = (2 \times \ell_j) \times (k \times S) \times \frac{f_u^1}{\sqrt{3}\gamma_{mw}}$$

$$\Rightarrow 270 \times 10^3 = (2 \times \ell_j) \times (0.7 \times 10) \times \frac{410}{\sqrt{3} \times 1.25}$$

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

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 = $(4.2 \times 250] \times 150$ N

= 115.5 kN

Strength of plate = $[50 \times 8] \times 150$ N

= 60 kN

Then before the failure of weld joint plate fails.

The permissible load allowable = 60 kN

Chapter- 4
Eccentric Connections

02. Ans: (a)

Sol: r = Max (1, 3, 4 & 6) θ = min (1)

Hence critical bolt is Bolt No. 1

03. Ans: (b)

Sol:

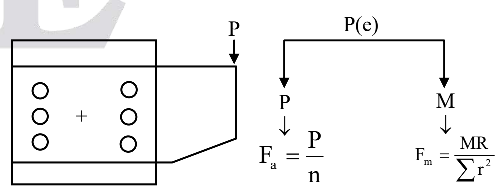
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

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

It is circle equation

04. Ans: (c)

Sol:



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

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

05. Ans: 5.99

Sol: Given load P = 10 kN

Eccentricity = 150 mm

Number of rivets =4

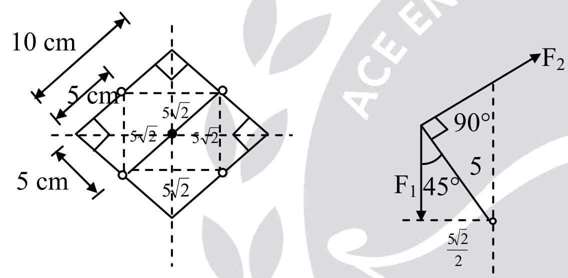
Force in rivet 1 due to direct loading = $\frac{P}{4}$

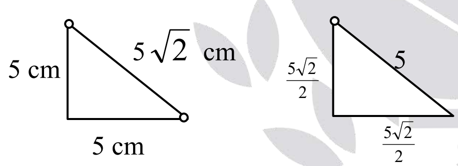
$F_1 = 2.5$ kN

Force in rivet 1 due to twisting moment

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

M = P[150] kN-mm





$$F_2 = \frac{[10]\times150\times50}{4\times[50]^2} = 7.5 \text{ kN}$$

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

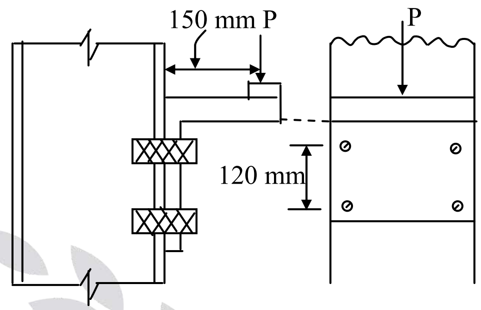
$\theta = 135°$

$F_R = 5.99$ kN

06. 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 × e = P × 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.

$= 14400 \text{ A mm}^4$

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

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

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: 156.20

Sol: An eccentric load may be replaced by set of one direct concentric load (p) and in plane moment (M)

$M = p.e$

$= (100)\,(600)$

$= 6 \times 10^4$ kN-mm

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

$$F_a = \frac{p}{n} = \frac{100}{5} = 20\text{kN}$$

Force in critical bolt due to moment (F_m)

$$= \frac{p.e.r}{r^2}$$

$$= \frac{M.r}{\Sigma r^2} = \frac{6 \times 10^4 \times 75\sqrt{2}}{4 \times \left(75\sqrt{2}\right)^2 + 0}$$

$$= \frac{6 \times 10^4 \times 75\sqrt{2}}{4 \times 75^2 \times 2}$$

$= 141.42$ kN

Maximum Resultant force in critical bolt 1 and 2 is

$$F_{Rmax} = \sqrt{F_m^2 + F_a^2 + 2F_a F_m \cos\theta}$$

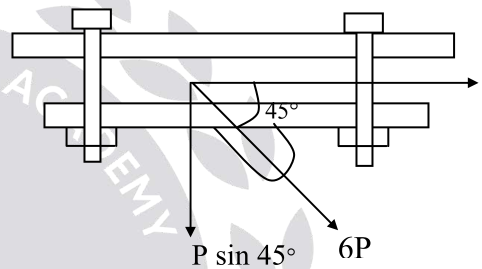
$$= \sqrt{(141.42)^2 + (20)^2 + 2 \times 20 \times (141.42) \times \left(\frac{1}{\sqrt{2}}\right)}$$

$= 156.20$ kN

08. Ans: 20 kN

10. 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

11. 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

Chapter- 5
Tension Members

01. Ans: 240

Sol: $f_y = 400$ MPa

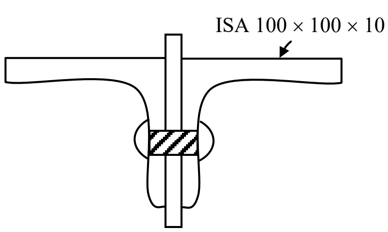
Permissible tensile stress $(\sigma_{at}) = 0.6 \times f_y$

$= 0.6 \times 400$

$\sigma_{at} = 240$ MPa

02. Ans: (d)

Sol:



ISA = 100 × 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:

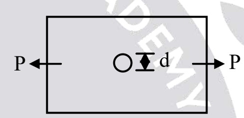


Fig. 1

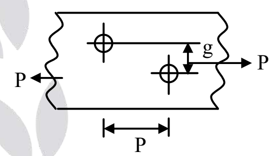


Fig. 2

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

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

$P_t \propto A_{net}$

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

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

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

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

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

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

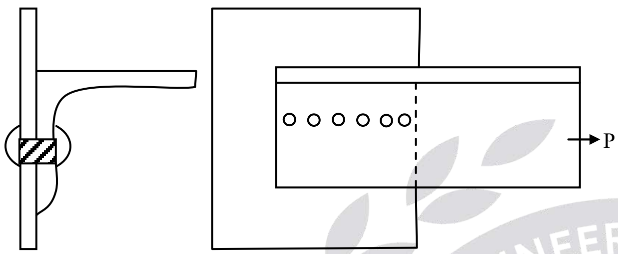
$$\Rightarrow P^2>4gd$$

04. Ans: (d)

Sol: ISA: $100 \times 100 \times 10$

$\sigma_{at} = 150\ N/mm^2$

Nominal dia of rivet $\phi = 20$ mm



Gross dia (d) = $\phi + 1.5$

= 21.5 mm (ϕ = 20 mm)

Safe tensile strength = $(P_t) = A_{net} \times \sigma_{at}$

$A_{net} = A_1 + A_2K_1$

$A_1 = (x - d - t/2)\, t$

$= (100 - 21.5 - \frac{10}{2})\, 10$

$= 735\ mm^2$

$A_2 = (y - t/2)\, t$

$= \left(100 - \frac{10}{2}\right)10$

$= 950\ mm^2$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 735}{3 \times 735 + 950}$$

$K_1 = 0.70$

$A_{net} = A_1 + A_2K_1$

$= 735 + 950 \times 0.70 = 1400\ mm^2$

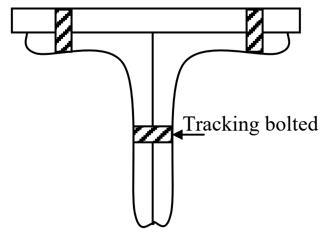
$P_t = A_{net} \times \sigma_{at} = 1400 \times 150$

$P_t = 210$ kN

07. Ans: (c)

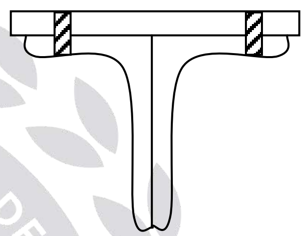
Sol:

(a)



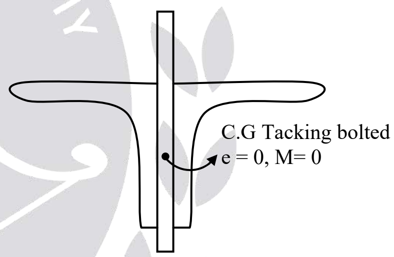
$A_{net} = A_1 + A_2k_2$

(b)



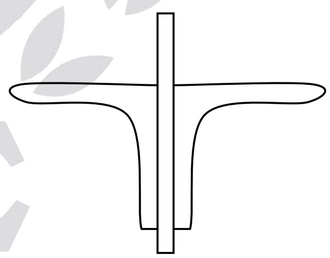
$A_{net} = A_1 + A_2k_2$

(c)



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

(d)



$A_{net} = A_1 + A_2k_1$

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

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

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

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

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

08. Ans: (c)

Sol: $r_{min} = 200$ mm

Slenderness ratio of trial section $\leq \lambda_{limit}$

$$\frac{\ell_{max}}{r_{min}} \leq 350$$

$$\ell_{max} \leq 350 \times r_{min}$$

$$\ell_{max} \leq 350 \times 6`$$

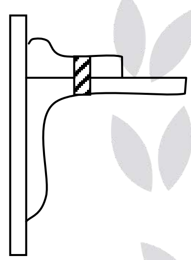
$$\ell_{max} \leq 2100 \text{ mm (or)} \leq 7.0 \text{ m}$$

$$\ell_{max} = 2.10 \text{ m}$$

09. Ans: 40

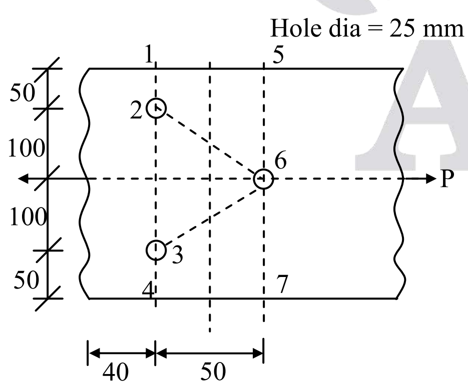
Sol: $\eta_2 \times R_V = 1.40\ T_2$

$= 40\%$



11. Ans: (b)

Sol:



Possible failure sections are

- 1–2–3–4
- 1–2–6–7
- 1–2–6–3–4

A_n along section 1–2–3–4

$$A_n = [B - nd_o] \times t$$

$$= [300 - 2 \times 25] \times 10 = 2500 \text{ mm}^2$$

A_n along section 1–2–6–7

$$A_n = (B - nd_o)t + \frac{P_1^2 t}{4g_1}$$

$$= (300 - 2 \times 25) \times 10 + \frac{50^2 \times 10}{4 \times 100}$$

$$= 2562.5 \text{ mm}^2$$

A_n along section 1–2–6–3–4

$$A_n = (B - nd_o)t + \frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}$$

$$= (300 - 3 \times 25) \times 10 + \frac{50^2 \times 10}{4 \times 100} + \frac{50^2 \times 10}{4 \times 100}$$

$\therefore A_n = 2375 \text{ mm}^2$

Hence Critical sectional area $A_n = 2375 \text{ mm}^2$

Chapter- 6
Compression Members

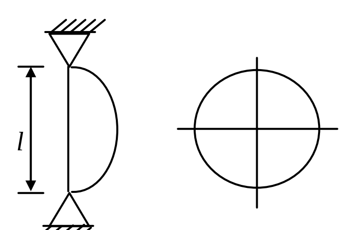
01. Ans: 50

Sol: KL = effective length

K = effective length constant

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

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



K = 1

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

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

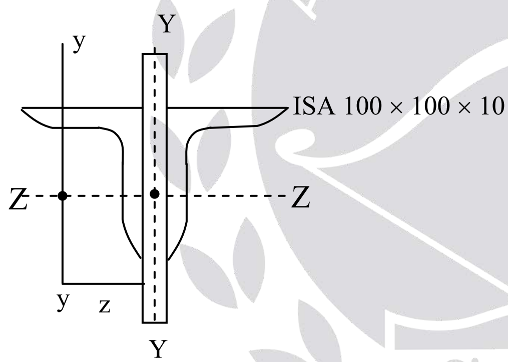
$$= \sqrt{\frac{d^2}{16}} = \frac{d}{4}$$

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

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

05. 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.

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

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

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

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

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

$= 30.49\ mm \simeq 30.5\ mm$

07. Ans: 37.5

Sol: Service load = 1000 kN

Factored load = 1.5×1000

= 1500 kN

V = 2.5% of factored column load

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

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

08. Ans: (d)

Sol: Force in lacing member

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

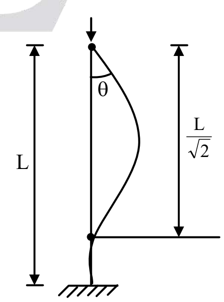
$$= \frac{37.5}{2 \times \sin 45^0}$$

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

= 26.52 kN

09. Ans: (b)

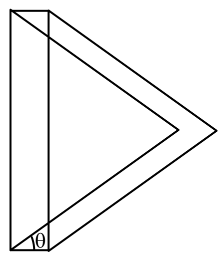
Sol:



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

10. 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

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

03. Ans: (b)

Sol: We know that allowable bearing strength of

$$\text{concrete} = \frac{\text{factored load (P)}}{\text{area of base slab}}$$

Here maximum bearing strength is 0.45 f_{ck}

$$\therefore\ 0.45\, f_{ck} = \frac{2000 \times 10^3}{650 \times 420}$$

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

$\therefore$ Minimum grade of concrete required is M20

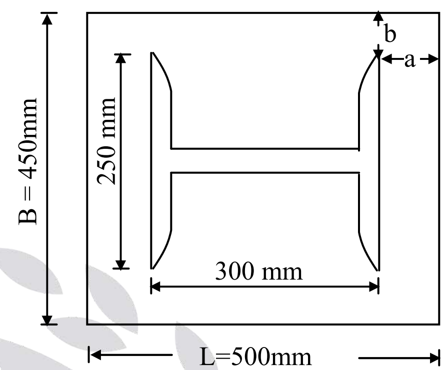
04. Ans: 26.3

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



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

a = 100

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

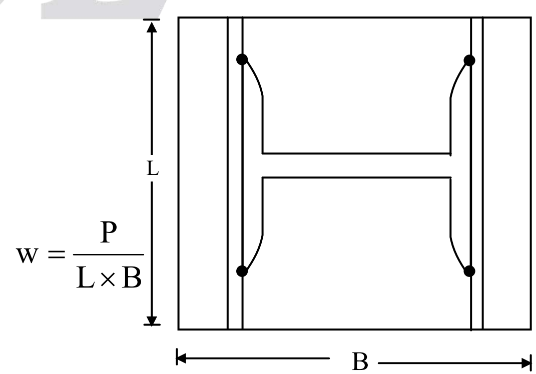
b = 100

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

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

05. Ans: (b)

Sol:



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

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

$\therefore$ If area↑, the W↓ then t↓ but here area

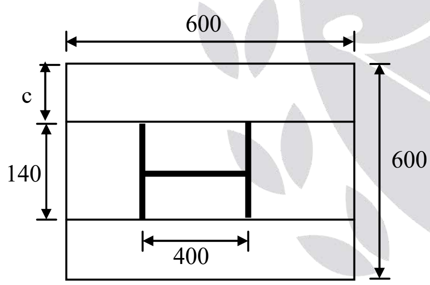
$\Rightarrow$ same

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

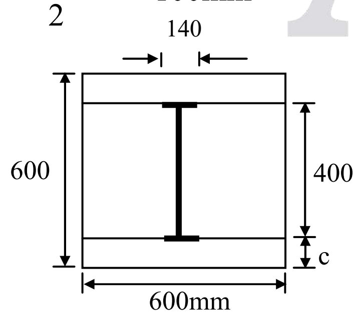
$\therefore$ t α c

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

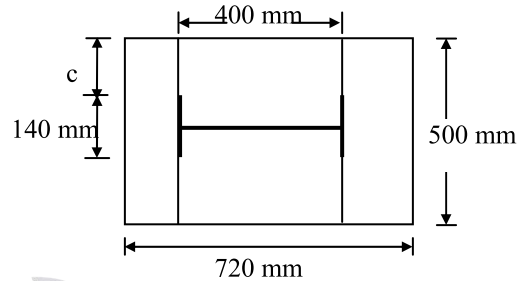
a) $c = \frac{600-140}{2} = 230\,\text{mm}$



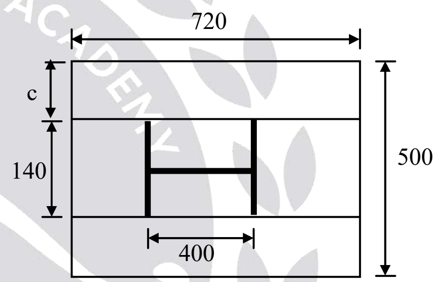
b) $c = \frac{600-400}{2}$ =100mm



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

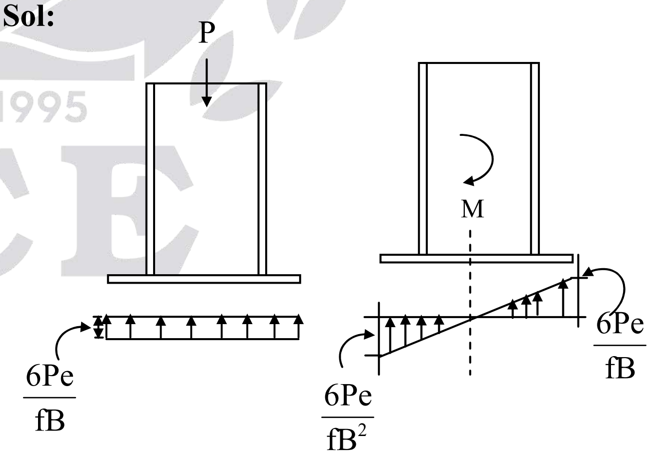


d) $c = \frac{720-400}{2} = 160$ mm



06. Ans: (d)

Sol:



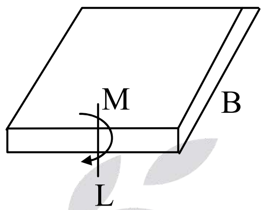
$$\text{Bending stress} = \frac{M}{Z} = \frac{Pe}{\frac{BL^2}{6}} = \frac{6Pe}{BL^2}$$

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

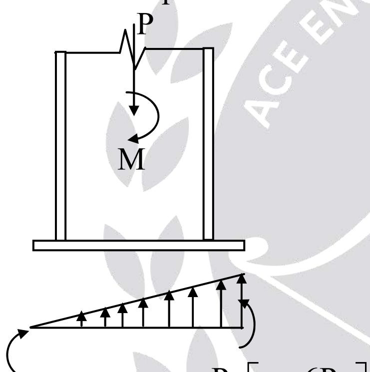
$$= \frac{P}{LB}\left[1 \pm \frac{6e}{L}\right]$$

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

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



∴ Zero at one end compression at other end.



$\frac{2P}{LB}$ $\quad \frac{P}{LB}\left[1 + \frac{6Pe}{L}\right]$

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 $= \frac{f_y}{\sqrt{3}}$

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

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

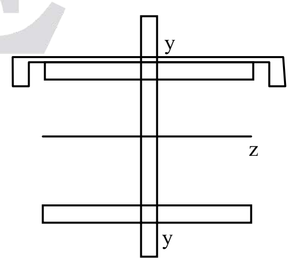
$$= \frac{f_y}{\sqrt{3}\gamma_{m0}} \times (h.t_w)$$

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

Chapter- 10 Gantry Girders

01. Ans: (a)

Sol:



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