02. Riveted Connections

01. Ans: 60%
Sol: \( \eta = \frac{P - d}{P} \times 100 \)
\[ p_{\text{min}} = 2.5 \phi \]
\[ d = \phi + 1.5 \]
\[ = \frac{2.5 \phi - \phi}{2.5} \times 100 \]
\[ = \frac{1.5 \phi}{2.5} \times 100 \]
\[ \eta = 60\% \]

05. Ans: (b)
Sol: ISA 50 x 50 x 6
\( t = 5 \text{ mm} \)
\( d = 16 \text{ mm} \)
\( \sigma_{bt} = 250 \text{ MPa} \)
\( P_b = d \times t \times \sigma_{bt} \)
\[ = 16 \times 5 \times 250 \]
\[ = 20 \text{ kN} \]

06. Ans: (43.29)
Sol:
\[
\begin{align*}
P/2 & \quad 8 \text{ mm} \\
P & \quad 12 \text{ mm} \\
P/2 & \quad 8 \text{ mm}
\end{align*}
\]
Nominal dia of rivet (\( \phi \)) = 16 mm

For field Rivets permissible shear stress in Rivet.
\( \tau_{vf} = 90 \text{ MPa} \)
Bearing stress in Rivet \( \sigma_{pf} = 270 \text{ MPa} \)
Rivet value
\( R_v = \) lesser of \( P_s \) & \( P_b \)
\( \phi = 16 \text{ mm} (\phi \leq 25 \text{ mm}) \)
\( d = \phi + 1.5 = 17.5 \text{ mm} \)
\[ P_s = 2 \times \frac{\pi}{4} (d)^2 \times \tau_{vf} \]
\[ = 2 \times \left( \frac{\pi}{4} \right) (17.5)^2 \times 90 \]
\[ P_s = 43.29 \times 10^3 \text{ N} \]
\[ = 43.29 \text{ kN} \]
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
\( t (\text{MP}) = 12 \text{ mm} \)
\( t (\text{cp + cp}) = 8 + 8 = 16 \text{ mm} \)
\[ P_b = 17.5 \times 12 \times 270 \]
\[ P_b = 56.7 \text{ kN} \]
\( R_v = \) lesser of \( P_s \) & \( P_b \)
\[ \therefore R_v = 43.29 \text{ kN} \]
07. Ans: (d)

Sol:
\[ P_s = \frac{2\pi}{4} d^2 \tau_{vf} = 80 \text{kN} \]
\[ P_s = \frac{\pi d^2}{4} \tau_{vf} = 40 \text{kN} \]
\[ P_s = 40 \text{kN}, \quad P_b = 60 \text{kN}, \quad P_{tr} = 70 \text{kN} \]
\[ n = \frac{P}{P_s} = \frac{200}{40} = 5 \]

09. Ans: (d)

Sol: Minimum pitch of rivets in compression zone
\[ \text{whichever is minimum} \]
\[ 12t \text{mm}, \quad 200 \text{mm} \]
\[ t = 10 \text{mm} \]
\[ 12t = 12 \times 10 = 120 \text{ mm}, \quad 200 \text{ mm} \]
\[ \text{whichever is minimum} \]
Pitch = 120 mm

03. Ans: (d)

Sol: \( f_u = 400 \text{ N/mm}^2 \)
\[ f_y = 0.6 f_u \]
\[ = 0.6 \times 400 \]
\[ = 240 \text{ N/mm}^2 \]

04. Ans: (d)

Sol:
\[ V_{d,db} = \frac{f_{sb}}{3\gamma_{mb}} \left( n_n A_{mb} + n_s A_{sb} \right) \]
\[ n_n (or) n_s \]
\[ = 3 \times 2 = 6 \]

05. Ans: (b)

Sol: \( P = 240 \text{ kN}; \quad V_{dsb} = 40 \text{ kN}; \quad V_{dpb} = 50 \text{ kN}; \)
\( T_{db} = 30 \text{ kN}; \quad V_{db} = \text{lesser of } V_{dsb}, V_{dpb} \)
\[ n = \frac{P}{V_{db}} = \frac{240}{40} \]
\[ n = 6 \text{ no's} \]

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 M16 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

\[
T_{db1} = \frac{A_{sh} f_{yb}}{\gamma_{mo}} = \frac{\frac{\pi}{4} \times (16)^2 \times 240}{1.1} = 43.86 \times 10^3 \text{ N} = 43.86 \text{ kN}
\]

\( T \) is based on net section rupture

\[
T_{db2} = 0.9 A_{mh} f_{ub} \frac{\pi}{4} \times (16)^2 \times 400 = 45.166 \text{ kN}
\]

\( T_{db} \) is lesser of \( T_{db1} \) & \( T_{db2} \)

\[
\therefore T_{db} = 43.86 \text{ kN}
\]

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.

\( P_{db} = 45 \text{ kN} = \text{Design bolt strength} \)

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

09. Ans: (b)

Sol: Design strength values

\[
\begin{align*}
V_{dpb} &= 1,50,000 \text{ N} \\
T_{dp} &= 1,80,000 \text{ N} \\
T_{sp} &= 2,40,000 \text{ N} \\
V_{dsb} &= 1,60,000 \text{ N}
\end{align*}
\]

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

\[
V_{dc} \text{ is lesser of } \left( \frac{V_{dp} T_{db}}{V_{dpb} T_{dp}} \right)
\]

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

\[
\eta = 62.5\%
\]

01.

Sol Axial load \( P = 180 \) kN

Permissible shear stress \( \tau_{vf} = 80 \text{ N/mm}^2 \)

Permissible bearing stress \( \sigma_{pf} = 250 \text{ N/mm}^2 \)

Size of angle ISA 90 \times 90 \times 8

Thickness of gusset plate \( t_g = 10 \text{ mm} \)
Assume nominal diameter of rivet using unwin’s equation
\[ \phi = 6.04 \sqrt{t} = 6.04 \sqrt{10} = 19.1 \text{ mm} \]
\[ \approx 20 \text{ mm} \]
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
\[ P_s = 2 \times \frac{\pi}{4} \times d^2 \times \tau_v \]
\[ = 2 \times \frac{\pi}{4} \times (21.5)^2 \times 80 \times 10^3 \]
\[ P_s = 58.08 \times 10^3 \text{ kN} \]
Bearing strength of one rivet
\[ P_b = d \times t \times \sigma_{pf} \times 10^3 \text{ N} \]
\[ P_b = 5370 \text{ kN} \]
Rivet value \( (R_v) \) = Lesser of \( P_s \) and \( P_b \)
\[ = 53.70 \text{ kN} \]
Number of rivets are required to resist on axial load ‘P’ is
\[ n = \frac{\text{Axial load}}{\text{Rivet value}} = \frac{P}{R_v} = \frac{180}{5370} = 3 \approx 4 \text{ No’s} \]
Using 4 No’s - \( \phi 20 \text{ mm} \) rivets arranged in single line as shown in figure.

Minimum end distance
\[ e = e_{\text{min}} = 1.5 d = 1.5 \times 21.5 = 35 \text{ mm} \]
Minimum pitch distance
\[ P = P_{\text{min}} = 2.5 \phi = 2.5 \times 20 = 50 \text{ mm} \]

02.

Sol: Factored (or) Design load,
\[ P = \gamma_f \times P_s = 1.5 \times 300 = 450 \text{ kN} \]
Assume shank diameter of Bolt
\[ d = 6.04 \sqrt{t} \]
\[ = 6.04 \sqrt{10} = 19 \text{ mm} \approx 20 \text{ mm} \]
Diameter of bolt hole, (for ordinary bolts)
\[ d_o = d + 2.0 = 22 \text{ mm} \]
Minimum pitch of bolt:
\[ P = P_{\text{min}} = 2.5d = 2.5 \times 20 = 50 \text{ mm} \]
Minimum end distance:
\[ e = e_{\text{min}} = 1.5d_o = 1.5 \times 22 = 33 \text{ mm} \]
Assume, designing a bolted connected as a double cover bolt connection as it will give higher efficiency.
Assume, thickness of each cover plate
\[ t = 0.625 \times \text{thickness of main plate} \]
\[ t = 0.625 \times 10 = 6.25 \text{ mm} \]
\[ t = 6 \text{ mm} \]
(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, \( V_{db} = \text{Minimum of } V_{dab} & V_{dpb} \)
Design shear strength of one bolt
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) \( \frac{e}{3d_o} = \frac{33}{3 \times 22} = 0.50 \)

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

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

(iv) 1.0

\:. \: K_b = 0.51

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

$$V_{dpb} = 80 \text{ kN}$$

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

$V_{db} = 80 \text{ kN}$

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

\[ n = \frac{450}{80} = 5.6 \]

$n \approx 6$ No’s

Arrange 6 No’s ; M20 bolts using diamond pattern as shown in figure.

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

$$T_{dp1-1} = \frac{0.9 A_{n} f_{u}}{\gamma_{mb}} = \frac{0.9 \left((200 - 22) \times 10\right) \times 410}{1.25}$$

$$T_{dp1-1} = 525.46 \text{ kN} \geq P = 450 \text{ kN}$$

Design tensile strength of cover plate of section 3-3

$$T_{dp3-3} = \frac{0.9 A_{n} f_{u}}{\gamma_{ml}}$$

\[ T_{dp3-3} = \frac{0.9 \left((200 - 3 \times 22) \times 12\right) \times 410}{1.25} \]

$$T_{dp3-3} = 474.5 \text{ kN} \geq P = 450 \text{ kN}$$

Hence connection is safe

$V_{dc}$ = lesser of $T_{dp1-1}$ and $T_{dp3-3} = 474.5 \text{ kN}$

Also, $V_{dc} \geq P$

Hence safe.
03. Welded Connections

07. **Ans: (20)**
**Sol:** Permissible stress in the weld are reduced by 20% when the welding is done in the field.

08. **Ans: (d)**
**Sol:**

![Weld Diagram]

\[
S = 8 \text{ mm}, \quad \tau_{vf} = 110 \text{ N/mm}^2
\]

Working stress method ⇒

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

\[
L_w = \text{Effective length of fillet weld}
\]

\[
= (80+80+60) \times (0.7 \times 8) \times 110
\]

\[
= 135 \text{ kN}
\]

09. **Ans: (a)**
**Sol:**

![Weld Diagram]

**Given data:**
Permissible tensile stress in the plate
\[
\sigma_{at} = 150 \text{ MPa}
\]

What is maximum tension \(P = ?\)
We can allow Maximum tension load up to strength of butt weld
\[
P = T_s = l_w \cdot t_e \cdot \sigma_{at}
\]

[\( \because \) In case of single ‘V’ butt weld
\[
t_e = \frac{5}{8} \times \text{thickness of thinner member in case of single V}
\]

\[
= 150 \times \frac{5}{8} \times 12 \times 150
\]

\[
= 168.78 \times 10^3 \text{N}
\]

\[
P = 168.75 \text{ kN}
\]

10. **Ans: (b)**
**Sol:**

![Weld Diagram]

Size of weld (S) = 6 mm
Torque \(T = 8 \text{ kN-m}\)

\[
= 8 \times 10^6 \text{ N-mm}
\]

Maximum stress in weld \(q = ?\)

Twisting moment capacity of weld
\[ T_d = P_s \times r = P_s \times \frac{d}{2} \]
\[ = L_w \cdot t_w \cdot q \cdot \frac{d}{2} \]
\[ T = \pi d (k_s) q \cdot \frac{d}{2} \]
\[ 8 \times 10^6 = \pi \times 120 \times (0.7 \times 6) \times q \times \frac{120}{2} \]
\[ q = 84.2 \text{ N/mm}^2 \approx 85 \text{ MPa} \]

11. Ans: (b)

Sol: 
\[ S = 10 \text{ mm}; \quad f_y = 250 \text{ Mpa} = f_{yw}, \quad f_u = 410 \text{ Mpa}; \]
\[ \gamma_{mw} = 1.25; \quad P = 270 \text{ kN} \]
\[ = f_{yw} \]
\[ \therefore L_w = l_j + l_j = 2l_j \]
\[ P \leq P_{dw} = L_w \times t_j \times \frac{f_u}{\sqrt{3} \gamma_{wm}} \]
\[ \Rightarrow 270 \times 10^3 = (2 \times l_j) \times (k \times S) \times \frac{f_u}{\sqrt{3} \gamma_{wm}} \]
\[ \Rightarrow 270 \times 10^3 = (2 \times l_j) \times (0.7 \times 10) \times \frac{410}{\sqrt{3} \times 1.25} \]
\[ l_j = 101.8 \text{ mm} \approx 105 \text{ mm} \]

12. Ans: 60

Sol: Throat thickness = 0.7 \times 6 \text{ mm}
\[ = 4.2 \text{ mm} \]
Effective length of weld = 100 + 100 + 50
\[ = 250 \text{ mm} \]
Permissible stress in the weld = 150 MPa
Strength of weld = (4.2 \times 250) \times 150 \text{ N}
\[ = 115.5 \text{ kN} \]

Strength of plate = [50 \times 8] \times 150 \text{ N}
\[ = 60 \text{ kN} \]

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
\[ f_u = 410 \text{ MPa} \quad \& \quad f_y = 250 \text{ MPa} \]
\[ \gamma_{mo} = 1.10; \quad \gamma_{mw} = 1.25 \]
Size of fillet weld, S = 6 mm
For angle ISA 90 \times 90 \times 10
\[ A = 1703 \text{ mm}^2 \]
\[ C_{xx} = C_{zz} = 25.9 \text{ mm} \]
Design tensile strength of an angle based on cross section yielding.
\[ P = T_{dg} = A \times f_y \frac{f_y}{\gamma_{mo}} = 1703 \times \frac{250}{1.10} \]
\[ P = 387 \text{ kN} \]

Size of fillet weld, S = 6 mm
Effective throat thickness
\[ t_t = KS \]
\[ t_t = 0.7 \times 6 \]
\[ t_t = 4.2 \text{ mm} \]
Design shear strength of fillet weld (\( P_{dw} \))
\[ P_{dw} = L_w \cdot t_t \times \frac{f_u}{\sqrt{3} \gamma_{mw}} \]
Equating \( P = P_{dw} = L_w \cdot t_t \times \frac{f_u}{\sqrt{3} \gamma_{mw}} \)
387 \times 10^3 = L_w (4.2) \frac{410}{\sqrt{3} \times 1.25}

L_w = 486.6 \text{ mm}

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

\begin{align*}
x + y + 90 &= L_w = 486.6 \text{ mm} \\
x + y &= 486.6 - 90 \\
x + y &= 396.6 \text{ mm} 
\end{align*}

\( \rightarrow (1) \)

\[
x + y + 90 = L_w = 486.6 \text{ mm}
\]

\[
x + y = 396.6 \text{ mm}
\]

\( \rightarrow (1) \)

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

\[
y \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25} \times 90 + 90 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25} \times \frac{90}{2} - 387 \times 10^3 \times (90 - 25.9) + 0 = 0
\]

\[
y = 301.5 \text{ mm} \rightarrow (2)
\]

from equation (1) and equation (2)

\[
x = 95.04 \text{ mm}
\]
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 \( \leq 3t \) or 25 mm whichever is greater.

\[ \therefore \text{ width} = 3 \times 7.6 = 22.8 \]

Hence adopt width = 25 mm

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

\[ 274.16 = 4 l \]

\[ l = 68.54 \approx 70 \text{ mm} \]

### 03. Ans: (c)

**Sol:**

\[ F_a \propto r \]

\[ F_m \propto r \]

\[ F_a \leq 0.5 V_{db} \]

\[ F_m \leq r \]

### 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 = \( \frac{P}{4} \)

\[ F_1 = 2.5 \text{ kN} \]

Force in rivet 1 due to twisting moment

\[ = \frac{Mr_e}{\Sigma r^2} \]

\[ M = P[150] \text{ kN-mm} \]

### 04. Eccentric Connections

**02. Ans: (b)**

**Sol:**

\[ \left( \frac{V_b}{V_{db}} \right)^2 + \left( \frac{T_b}{T_{db}} \right)^2 \leq 1.0 \]

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

It is circle equation
\[ F_2 = \left[ \frac{10 \times 150 \times 50}{4 \times 50} \right] = 7.5 \text{ kN} \]

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

\( \theta = 135^\circ \)

\( F_R = 5.99 \text{ kN} \)

05. Ans: (c)
Sol: Design shear stress \( V_{db} = 20 \text{ kN} \)
Design tensile capacity \( \tau_{bd} = 15 \text{ kN} \)

1) \( P \rightarrow \) cause shear force in bolt = \( P/8n \).
2) \( M = P \times e = P \times 150 = 150P \text{ kN} - \text{ mm} \).

\[ V_b = \frac{P}{n} = \frac{P}{4} \]

\[ M = \frac{f}{y} \]

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

\[ T_b = f \times A = \frac{M}{I} y \times A \]

\( M = 150P \text{ kN} - \text{mm} \)

\( y = \frac{120}{2} = 60 \text{mm} \)

\( I_{XX} = [ICG + Ag]4 \)

\[ \frac{\pi d^4}{64} + A(60)^2 \]

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

\[ T_b = \frac{M}{I} \times y \times A \]

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

Interaction formula

\[ \left( \frac{V_b}{V_{db}} \right)^2 + \left( \frac{T_b}{T_{db}} \right)^2 \leq 1.0 \]

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

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 (\( F_m \))

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

\[ F_{r max} = \sqrt{E^2 + F_m^2 + 2E_aF_m \cos \theta} = 50 \]

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

\[ = 69.32 \text{ kN} \]
09. Ans: (c)  
Sol  
\[ q_1 = 30 \text{ MPa}, \quad q_2 = 120 \text{ MPa}, \quad q = \sqrt{30^2 + 120^2} = 120 \text{ MPa} \]  
\[ R = 300 \text{ kN}, \quad P = \frac{R}{2} = 150 \text{ kN} \]

No. of rows to be provided \((m) = 2\)  
Eccentricity \((e) = 200 \text{ mm}\)  
\[ \Rightarrow \text{Moment (M)} = P.e = 30 \text{ kN-m} \]
Thickness of plate = 10 mm  
Using M20 bolts of Grade 4.6 (fully threaded)  
\[ [\phi = 20 \text{ mm}, \ f_{ub} = 400 \text{ MPa}, \ f_{yb} = 240 \text{ MPa}] \]
→ Design shear capacity  
\[ (V_{dsb}) = \frac{f_{ub}}{\sqrt{3}} \left[ n_s.A_{sh} + n_n.A_{nb} \right] \]
\[ = 46.38 \text{ kN} \]

Adopting minimum pitch \((p) = 2.5\phi = 50 \text{ mm}\)  
Minimum end distance \((e) = 1.5d_h = 33 \text{ mm}\)  
→ Design bearing capacity  
\[ (V_{dpb}) = 2.5K_b \phi t.A_{s} f_{u} \gamma_{mb} \]
\[ = 2.5 \times 0.5 \times 20 \times 10 \times \frac{400}{1.25} \]
\[ = 80 \text{ kN} \]

→ Design capacity of bolt \((V_{db}) = 46.38 \text{ kN}\)
Design of connections:

→ No. of bolts required per row = \( \sqrt{\frac{6M}{m.p.V_{db}}} \)

= 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

\( (F_1) = \frac{P}{14} \)

1 due to direct force Force P = 10.7 kN

Shear force developed in bolt 1 due to twisting moment

\( (F_2) = \frac{M}{\Sigma r_i^2} \times r_i \)

= 27.92 kN

\[ \cos \theta = \frac{45}{156.6} \]

Resultant shear force developed in bolt 1

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

\( F_{res} < V_{db} \quad \text{Hence safe.} \)

02.
Sol:

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Let size of weld be 'S' mm
Throat thickness \( (t_t) = 0.75 \)
From figure 3, \( \bar{x} = 47 \) mm
\[ \Rightarrow \text{Eccentricity (e)} = 353 \text{ mm} \]

**From figure 2,**
\[ I_p = I_{zz} + I_{yy} \]
\[ = (A_1 \times 225^2) \times 2 + t_t \times \frac{450^3}{12} + \]
\[ 2 \times \left[ t_t \times \frac{200^3}{12} + A_1(53)^2 \right] + A_2 \times (47)^2 \]
\[ = 31.29 \times t_t \times 10^6 \text{ mm}^4 \]

[Neglecting higher powers of \( t_t \)]

**From figure 3,**
\( t_1 = 200 \times t_t ; x_1 = 100 \)
\( t_2 = 450 \times t_t ; x_2 = 0 \)
\( t_3 = 200 \times t_t ; x_3 = 100 \)
\[ \bar{x} = \frac{A_1 x_1 + A_2 x_2 + A_3 x_3}{A_1 + A_2 + A_3} = 47 \text{ mm} \]

Total force \( (P) = 200 \text{ kN} \)
Twisting Moment \( (T) = P.e = 70.6 \text{ kN-m} \)
**At most critical point A:**
Shear stress developed due to direct shear force \( P \), \( (F_1) = \frac{P}{A_{\text{weld}}} \)

\[ = \frac{200 \times 10^3}{(200 + 450 + 200)t_t} = \frac{235.3}{t_t} \text{ MPa} \]
Shear stress developed due to moment \( T \)
\[ (F_2) = \frac{T}{I_p} \times (r_A = 231.15) \]
\[ = \frac{70.6 \times 10^6}{31.29t_t \times 10^6} \times 231.15 = \frac{521.5}{t_t} \text{ MPa} \]
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
\[ t_t \text{ req} = \frac{619.19}{110} \text{ mm} = 5.62 \text{ mm} \]
\[ S_{\text{reqd}} = \frac{5.62}{0.7} = 8.04 \text{ m} \]
Minimum size required = 8.04 mm

**03. Sol:**

Let size of weir be 'S' mm.
Given:
Load along Y-Y \( (P_Y) = 50 \text{ kN} \)
Load along Z-Z \( (P_Z) = 50 \text{ kN} \)
eccentricity \( (e) = 100 \text{ mm} \)
Moment along Y-Y (M_{YY}) = P_{Z} \cdot e = 5 \text{kN.m}

Moment along Z-Z (M_{ZZ}) = P_{Y} \cdot e = 5 \text{kN.m}

Weld:

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

\[
\Rightarrow I_{ZZ} = I_{YY} = 2.25 \times 10^{6} \text{ mm}^{4}
\]

(neglecting higher power of \( t_{t} \))

Area of weld (\( A_{w} \)) = 4 \times 150 \times t_{t}

\[
= 600 t_{t} \text{ mm}^{2}
\]

**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

\[
M_{yy} \ (f_{1}) = \frac{M_{zz}}{I_{yy}} \times (Z = 75 \text{ mm})
\]

\[
= \frac{166.67}{t_{t}} \text{ MPa}
\]

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 \( f_{eq} \leq f_{wd} \)

\[
\frac{390.8}{t_{t}} \leq \left( \frac{f_{uw}}{\sqrt{3} \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} \right)
\]

\[
\Rightarrow t_{t} \geq 2.06 \text{ mm}
\]

\[
\Rightarrow S \geq \frac{t_{t}}{0.7} = 2.95 \text{ mm}
\]

For 6 mm plate, \( S_{min} = 3 \text{ mm} \)

Hence, weld of 3 mm may be considered safe.
05. Tension Members

01. Ans: 240
Sol: \( f_y = 400 \text{ MPa} \)
Permissible tensile stress \( (\sigma_{at}) = 0.6 \times f_y \)
\[ \sigma_{at} = 0.6 \times 400 \]
\[ \sigma_{at} = 240 \text{ MPa} \]

02. Ans: (d)
Sol:
\[
\begin{align*}
\text{ISA} & = 100 \times 100 \times 10 \\
A_1 & = 775 \text{ mm}^2 \\
A_2 & = 950 \text{ mm}^2 \\
A_{net} & = A_1 + A_2 K_1 \\
K_1 & = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 775}{(3 \times 775 + 950)} = 0.709 \\
A_{net} & = A_1 + A_2 K_1 \\
& = (2 \times 775) + (2 \times 950) \times 0.709 \\
A_{net} & = 2899 \text{ mm}^2 
\end{align*}
\]
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

\[ r_{\text{min}} = \sqrt{\frac{1}{A}} = \sqrt{\frac{\pi d^4}{64}} = \frac{d}{4} \]

\[ r_{\text{min}} = \frac{16}{4} = 4 \text{m} \]

\[ \lambda = \frac{L}{r_{\text{min}}} \]

\[ L = 350 \times 4 \]

\[ L = 1400 \text{ mm} \]

---

6. Ans: (d)
Sol: Attachment to the member should capable of developing 40% in excess of force in outstanding leg of the angle.

Assume double cover butt joint connected with 4.6 grade bolts.

Design strength of connection = Least of

\[ V'_{\text{dsb}}, V'_{\text{dpb}}, T_{\text{dp}} \]
\( V'_{dsb} = \text{Design shear strength of bolts} \)
\[
= (n_s A_{sb} + n_n A_{nb}) \cdot \frac{f_{sb}}{\sqrt[3]{3} \gamma_{mb}}
\]
Assuming both shear planes to intercept with threading
\[
V'_{dsb} = \left( 0 + 2 \times 0.78 \times \frac{\pi \times 25^2}{4} \right) \cdot \frac{400}{\sqrt[3]{3} \times 1.25} \times 6
\]
\( = 848.85 \text{ kN} \)
\[
V'_{dpb} = \left( \frac{2.5d't}{\gamma_{mb}} \right) f_u k_b \times 6
\]
\( K_b = \text{Least of} \)

(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) \( \frac{f_{sb}}{f_u} = \frac{400}{410} = 0.97 \)

(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.

\[
T_{dp} = \frac{0.9A_n f_u}{\gamma_{mi}}
\]
\[
A_n = (B - d_o)t = (320 - 28)16 = 4672 \text{ mm}^2
\]
\[
T_{dp} = \frac{0.9 \times 4672 \times 410}{1.25} = 1379.17 \text{ kN}
\]

**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.
\[
T_{dp} = \frac{0.9A_n f_u}{\gamma_{mi}}
\]
\( A_n = (320 - 3 \times 28)16 \times 2 = 7552 \text{ mm}^2 \)
\[
T_{dp} = \frac{0.9 \times 7552 \times 410}{1.25} = 2229 \text{ kN}
\]

\( \therefore \) Design strength of bolted connection
\( = \text{Least of } V'_{dsb}, V'_{dpb} & T_{dp} = 803.3 \text{ kN}. \)

**02.**

**Sol:** Length of tie member, \( = 1.8 \text{ m} \)
Axial tensile load, \( P = 155 \text{ kN} \)
Yield stress, \( f_y = 250 \text{ MPa} \)
\( \tau_{vf} = 100 \text{ MPa} \)
\( \sigma_{pf} = 300 \text{ MPa} \)
\( \tau_{vw} = 108 \text{ MPa} \)
Permissible tensile stress,
\( \sigma_{at} = 0.6 f_y \)
\( = 0.6 \times 250 = 150 \text{ MPa} \)
Net sectional area required to resist tensile load $P$ is $A_{\text{net}}$

$$A_{\text{net}} = \frac{P}{\sigma_{at}} = \frac{155 \times 10^3}{150}$$

$$= 1033.2 \text{ mm}^2$$

Gross sectional area of trial section

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

$$A_g = 1291.1 \text{ mm}^2$$

Available sections for angle

<table>
<thead>
<tr>
<th>Angle size</th>
<th>Sectional area (mm$^2$)</th>
<th>Minimum radius of gyration ($r_{\text{min}}$)</th>
<th>C$y_y$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISA70$\times$45$\times$10</td>
<td>1052</td>
<td>9.50</td>
<td>24.8</td>
</tr>
<tr>
<td>ISA100$\times$75$\times$6</td>
<td>1014</td>
<td>15.9</td>
<td>30.1</td>
</tr>
<tr>
<td>ISA100$\times$65$\times$8</td>
<td>1257</td>
<td>13.9</td>
<td>32.6</td>
</tr>
<tr>
<td>ISA 90 $\times$60 $\times$8</td>
<td>1137</td>
<td>12.8</td>
<td>27.2</td>
</tr>
<tr>
<td>ISA90$\times$60$\times$10</td>
<td>1401</td>
<td>12.7</td>
<td>28.1</td>
</tr>
</tbody>
</table>

Choose an angle (usually $\geq A_g$)

ISA 100$\times$65$\times$8

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

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

Assume nominal diameter of rivet

$$\phi = 6.04 \sqrt{t}$$

$t$ → thickness of thinner connected member

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

$$= 18 \text{ mm}$$

Gross dia of rivet

$$d = 18 + 1.5 = 19.5 \text{ mm}$$

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

Net & Effective sectional area of single angle

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

$A_1$ – net sectional area of connected leg

$$= (100 – 19.5 – 8/2) \times 8$$

$$= 612 \text{ mm}^2$$

$A_2$ = cross sectional area of outstanding leg

$$A_2 = (65 – 8/2) \times 8 = 488 \text{ mm}^2$$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 612}{3 \times 612 + 488}$$

$$= 0.79$$

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

$$= 612 + 488 \times 0.79$$

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

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

Hence trial section is unsafe

(i.e., $P_{\text{tr}} = A_{\text{net trial}} \times \sigma_{at}$

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

$$= 149.6 \times 10^3$$

$$= 149.6 \text{ kn} < 155 \text{ kN not safe}$$
Iteration –II:

Choose a trial section
ISA 90 × 60 × 10
A = 1401 mm²

rₘᵋᵦᵦᵧ = 12.7 mm
nominal diameter of rivet
φ = 6.04 √10 = 19.04 mm
= 20 mm

Gross diameter of rivet
d = 21.5 mm

net sectional area of connected leg

\[
A₁ = \left(90 - 21.5 - \frac{w}{2}\right) \times w
\]

A₁ = 635 mm²

Gross sectional area of outstanding leg

\[
A₂ = \left(60 - \frac{10}{2}\right) \times w = 350 \text{ mm}²
\]

Reduction factor, \(K₁ = \frac{3A₁}{3A₁ + A₂}\)

\[
K₁ = \frac{3 \times 635}{3 \times 635 + 550} = 0.78
\]

∴ \(A_{net} = A₁ + A₂K₁\)

\[
A_{net} = 635 + 550 \times 0.78
\]

\(A_{net \ trial} = 1064 \text{ mm}² \geq A_{net \ required}\)

\(P_{trial} = A_{net \ trial} \times σ_{at}\)

\[
P_{trial} = 1064 \times 150
\]

\(P_{trial} = 159.6 kN > 155 kN\)

Hence trial section is safe
Number of rivets required

\[
n = \frac{A_{Axial \ tensile \ load}}{\text{Rivet value}}
\]

Rivet value = \(R_V = \text{Smaller of } P_s & P_b\)

Strength of one rivet in single shear

\[
P_s = \frac{\pi}{4} d^2 \times τ_vf
\]

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

\[
= 36.3 \times 10^3 \text{ N}
\]

Pₕ = 36.3 kN

Strength of one rivet in bearing

Pₖ = d \times t \times σₚf = 21.5 \times 10 \times 300

Pₖ = 64.5 kN

Rivet value, \(R_V = 36.3 \text{ kN}\)

\[
n = \frac{P}{P_V} = \frac{155}{36.3} = 4.26\]

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 mm²

rₘᵋᵦᵦᵧᵦᵦᵧ = 13.9 mm

Net sectional area of connected leg

\[A₁ = \left(100 - \frac{8}{2}\right) \times 8 = 768 \text{ mm}²\]

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

Let \(L_{w1}\) and \(L_{w2}\) be the length of top & bottom length of weld respectively.

\[L_{w} = L_{w1} + L_{w2} = 341.7 \quad \rightarrow (1)\]

Effective throat thickness
\[t_i = KS = 0.7 \times 6 = 4.2 \text{ mm}\]

equating, \(P = Ps\)
\[155 \times 10^3 = L_{w} \times t_i \times \tau_{vf}\]
\[155 \times 10^3 = L_{w} \times 4.2 \times 108\]

\[L_w = 341.7 \text{ mm}\]

Consider moments of weld strength and load on top edge of an angle.
\[P_2 \times 100 + P_1 \times 0 - P \times 32.6 = 0\]
\[L_{w2} \times t_i \times \tau_{vf} \times 100 = 155 \times 10^3 \times 32.6\]
\[L_{w2} \times 4.2 \times 108 \times 100 = 155 \times 10^3 \times 32.6\]

\[L_{w2} = 111.39 \text{ mm}\]

From equation (1) & (2)
\[L_{w1} = 230.31 \text{ mm}\]
\[L_{w2} = 111.39 \text{ mm}\]

03.

**Sol:** Working tensile load, \(T_s = 200\) kN

Length of member, \(L = 2.5\) m

For Fe410 grade steel,
\[f_u = 410 \text{ MPa} \& f_y = 250 \text{ MPa}\]

For grade 8.8 bolts
γ_m₀ = 1.1; γ_m₁ = 1.25; γ_mb = 1.25
Factored (or) Design load, T = γ_f × T_s
T = 1.5 × 200 = 300 kN
Gross sectional area of trial section required to resist τ is A_g:

\[ A_g = \frac{T}{\gamma_m \frac{f_y}{\gamma_m}} \]

\[ A_g = \frac{300 \times 10^3}{250 \times 1.1} = 1320 \text{ mm}^2 \]

Choose a trial section ISA 100×75×8
\[ A_g = 1336 \text{ mm}^2 \]
\[ r_{min} = 15.9 \text{ mm} \]

Sections available for angle are

<table>
<thead>
<tr>
<th>Angle size</th>
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<th>Minimum radius of gyration (r_{min})</th>
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<td>13.9</td>
</tr>
<tr>
<td>ISA80×80×8</td>
<td>1221</td>
<td>15.5</td>
</tr>
<tr>
<td>ISA90×60×10</td>
<td>1401</td>
<td>12.7</td>
</tr>
</tbody>
</table>

Design of bolted connections:

Number of bolts required
\[ n = \frac{T}{V_{db}} \]

Shank diameter of Bolt, 
\[ d = 20 \text{ mm} \]
Diameter of bolt hole, 
\[ D_o = 20 + = 22 \text{ mm} \]
Minimum pitch, P = 2.5 × 20
\[ P = 50 \text{ mm} \approx 60 \text{ mm} \]
Minimum end distance
\[ e = 1.5 \times D_o = 1.5 \times 22 \approx 33 \approx 40 \text{ mm} \]

Design shear strength of one Bolt
\[ V_{db} = \frac{f_{ub} A_n b}{\sqrt{3 \gamma_{mb}}} (n_A A + n_A A_{sb}) \]
\[ = \frac{800}{\sqrt{3 \times 1.25}} \left[ 1 \times 1 \times 0.78 \times \frac{\pi}{4} (20)^2 + 0 \right] \]
\[ = 90.51 \text{ kN} \]

Design bearing strength of one bolt
\[ V_{dpb} = 2.5 f_{ub} K_b f_{db} / \gamma_{mb} \]
\[ = 2.5 \times 20 \times 8 \times 800 \times 0.61 \]
\[ \text{K_b is lesser of (bearing factor)} \]
\[ \bullet e = \frac{40}{3d_o} = 0.61 \]
\[ \bullet \frac{P}{2d_o} = 0.25 = 0.66 \]
\[ \bullet \frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95 \]
\[ \bullet 1.0 \]
Design tensile strength of trial section

$$T_{\text{trial}} = \min(T_{\text{dg}}, T_{\text{dn}}, T_{\text{db}})$$

Design tensile strength based on Gross section yielding

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

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

Design tensile strength based on net section rupture.

$$T_{\text{dn}} = 0.9A_{\text{nc}} f_u / \gamma_{\text{ml}} + \beta A_{\text{go}} f_y / \gamma_{\text{ma}}$$

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

$$\leq \frac{f_u}{f_y} \frac{\gamma_{\text{mo}}}{\gamma_{\text{ml}}}$$

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

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

$$w = 75 \text{mm}; \quad t = 8 \text{mm}$$

$$f_u = 410 \text{MPa}; f_y = 250 \text{MPa}$$

$$b_s = \text{shear leg distance} = w + w_1 - t = 75 + 60 - 8 = 127 \text{mm}$$

$$L_c = \text{length of end connection} = 3 \times 60 = 180 \text{mm}$$

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

$$\beta = 1.09$$

$$\therefore \text{Design tensile strength of trial section.}$$

By

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

$$T_{\text{dn}} = 315.47 \text{kN}$$

Design tensile strength based on block shear failure

Gross area of tension plane

$$A_{\text{tg}} = 40 \times 8 = 320 \text{mm}^2$$

Net area of tension plane

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

Gross area of shear plane

$$A_{\text{vg}} = (3 \times 60 + 40) \times 8 = 1760 \text{mm}^2$$

Net area of shear plane

$$A_{\text{vn}} = [(3 \times 60 + 40) - 3 \times 22 \times 8 - \frac{22}{2}]$$
\[ A_{vn} = 1144 \, \text{mm}^2 \]
\[ T_{db1} \text{ based on shear rupture tension yield} \]
\[ T_{db1} = 0.9A_{vn} \frac{f_u}{\sqrt{3} \gamma_{ml}} + A_{tg} \frac{f_y}{\gamma_{mo}} \]
\[ = 0.9 \times 1144 \times \frac{410}{\sqrt{3} \times 1.25} + 320 \times 250 \times 1.10 \]
\[ T_{db1} = 267.7 \, \text{kN} \]
\[ T_{db2} \text{ based on shear yielding & tension rupture} \]
\[ T_{db2} = A_{vg} \frac{f_y}{\sqrt{3} \gamma_{mo}} + 0.9At_u \frac{f_u}{\gamma_{ml}} \]
\[ = 1760 \times \frac{250}{\sqrt{3} \times 1.1} + 0.9 \times (232) \times \frac{410}{1.25} = 299.426 \]
\[ T_{db} = \text{lesser of } T_{db1} \& T_{db2} \]
\[ T_{d_{trial}} = \text{lesser of } T_{dg} = 303.6 \, \text{kN} \]
\[ d_u = 315.85 \, \text{kN} \]
\[ T_{db} = 267.7 \, \text{kN} \]
\[ T_{d_{trial}} = 267.7 \, \text{kN} < T = 300 \, \text{kN} \]
Hence, trial section not safe
Increase pitch and end distance
\[ P = 100 \, \text{mm}, \ e = 80 \, \text{mm} \]
\[ A_{vg} = (3 \times 100 + 80) \times 8 \, \text{mm}^2 \]
\[ A_{vg} = 3040 \, \text{mm}^2 \]
\[ A_{vn} = (380 - 3 \times 22 - \frac{22}{2}) \times 8 \]
\[ A_{vn} = 2424 \, \text{mm}^2 \]
\[ A_{tg} = (80 \times 8) = 640 \, \text{mm}^2 \]
\[ A_{tg} = \left(80 - \frac{22}{2}\right) \times 8 = 522 \, \text{mm}^2 = 552 \, \text{mm}^2 \]
\[ \therefore \ T_{db1} = 0.9 \times 2424 \times \frac{410}{\sqrt{3} \times 625} + 640 \times 250 \times 1.10 \]
\[ T_{db1} = 558.5 \, \text{kN} \]
\[ \therefore \ T_{db2} = 3040 \times \frac{250}{\sqrt{3} \times 1.1} + 0.9 \times 552 \times 410 \times 1.25 \]
\[ T_{db2} = 561.8 \, \text{kN} \]
Lesser of \( T_{db1} \& T_{db2} = T_{db} \)
\[ T_{db} = 558.5 \, \text{kN} \]
Design tensile strength of trial section (\( T_d \))
\[ T_{d_{trial}} = \text{less of, } T_{dg} = 303.6 \, \text{kN} \]
\[ T_{db} = 558.5 \, \text{kN} \text{ and } T_{dn} \]
\[ T_{d_{trial}} = 303.6 > 300 \, \text{kN} \]
Hence, trial section is safe
Slenderness ratio
\[ \frac{L}{r_{min}} = 2.5 \]
\[ = 13.6 \times 10^{-3} \]
\[ = 182.82 < 350 (\lambda_{\text{limit}}) \]
\[ \therefore \text{Hence it is safe from buckling.} \]

06. Compression Members

01. Ans: 50
Sol: \( KL = \text{effective length} \)
\[ K = \text{effective length constant} \]
\[ \frac{KL}{r} = 200 \]
\[ \frac{l}{d} = ? \]
Radius of gyration, \( r_{min} = \sqrt{\frac{I}{A}} \)
\[ K = 1 \]
\[ d = \sqrt{\frac{\pi d^4}{64} \times \frac{4}{\pi d^2}} = \frac{d^2}{\sqrt{16}} = \frac{d}{4} \]

\[ KL = \frac{1.0\ell}{r_{\min}} = 200 \quad (\because k = 1) \]

\[ \ell = \frac{200}{4} = 50 \]

04. Ans: 30.6

Sol:

Symmetric with respect to yy axis

\[ A = 1903 \text{ mm}^2 \]

\[ I_{zz} = I_{yy} = 177 \times 10^4 \text{ mm}^4 \]

\[ I_{yy} = 2(I_{yy} + A z^2) \]

Gusset plates one at joint member only.

\[ I_{zz} = 2I_{zz} = 2 \times 177 \times 10^4 \]

\[ I_{zz} = 354 \times 10^4 \text{ mm}^4 \]

06. Ans: 37.5

Sol: Service load = 1000 kN

Factored load = 1.5 \times 1000 = 1500 kN

\[ V = 2.5\% \text{ of factored column load} \]

\[ V = \frac{2.5}{100} \times P = \frac{2.5}{100} \times 1500 = 37.50 \text{ kN} \]

07. Ans: (d)

Sol: Force in lacing member

\[ F = \frac{V}{N \sin \theta} \]

\[ N = \frac{37.50}{2 \times \sin 45^\circ} = 26.52 \text{ kN} \]

08. Ans: (b)

Sol:

\[ \frac{L}{\sqrt{2}} = 0.707 L \approx 0.80 L \]

09. Ans: (d)

Sol:

Skin buckling
01. Given data: \( L = 6 \text{ m} \)

Since both ends are pinned, \( l = kL, \quad k = l \)

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

\( l_y = kL_y = 0.8 \text{ L}_y = 0.8 \times 3 = 2.4 \text{ m} \)

Slenderness ratio, \( \lambda_x = \frac{\ell_x}{r_{xx}} = \frac{6 \times 10^3}{142.9} = 41.98 \)

\( \lambda_y = \frac{\ell_y}{r_{yy}} = \frac{2.4 \times 10^3}{28.4} = 84.5 \)

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

\[
\begin{align*}
P_d &= f_c d_y A_e \\
f_{cd} &= \frac{f_y}{\gamma_m} = \frac{f_y}{\phi + \left(\frac{\phi^2 - \lambda^2}{r}\right)^{1/2}}
\end{align*}
\]

\[
\begin{align*}
\lambda &= \sqrt{\frac{f_y}{f_{cc}}} \\
f_{cc} &= \frac{\pi^2 E}{(K L/r)^2} \\
\phi &= 0.5 \left[ 1 + \alpha(\lambda - 0.2) + \lambda^2 \right] \\
\alpha &= 0.34 \\
\gamma &= 0.5 \left[ 1 + 0.34 \left( 0.951 - 0.2 \right) + 0.951^2 \right] \\
\phi &= 2.159
\end{align*}
\]

\[
f_{cd} = \frac{250/1.1}{2.159 + \sqrt{2.159^2 - 0.951^2}} = 55.47 \text{ N/mm}^2
\]

\[
P_d = f_{cd} \times A_e = 55.47 \times 6671 = 370 \text{ kN
}
\]

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

02. Sol: An equal angle, ISA 100\times 100\times 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,
\[ P_C = \sigma_{ac} \times A_c = \sigma_a \times 1903 \]

The permissible axial stress in compression are given below

<table>
<thead>
<tr>
<th>KL/r_min</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
<th>140</th>
<th>150</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{ac} )</td>
<td>90</td>
<td>80</td>
<td>72</td>
<td>64</td>
<td>57</td>
<td>51</td>
<td>45</td>
</tr>
</tbody>
</table>

To calculate slenderness ratio

1. If end conditions are not given, assume given length as \( L_{\text{eff}} \)

2. For \( r_{\text{min}} \) is with respect to major & minor axis

\[ KL = 2.4 \text{ m} = 2400 \text{ mm} \]

Major and Minor moment of inertia of an angle

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

\[ \frac{I_{UU}}{I_{VV}} = \frac{177 \times 10^4 + 177 \times 10^4}{2} = 104.4 \times 10^4 \text{ mm}^4 \]

Minimum radius of gyration

\[ r_{\text{min}} = \sqrt{\frac{I_{\text{min}}}{A}} = \sqrt{\frac{72.6 \times 10^4}{1903}} \]

\[ r_{\text{min}} = 19.5 \text{ mm} \]

Effective slenderness ratio

\[ \frac{KL}{r_{\text{min}}} = \frac{2400}{19.5} = 123 \]

For \( \frac{KL}{r_{\text{min}}} = 123, \sigma_{ac} = ? \)

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

\[ \sigma_{ac} = 61.97 \text{ N/mm}^2 \]

Safe compressive load carrying capacity of an angle strut

\[ P_C = \sigma_{ac} \times A_c = 1.97 \times 19.23 \]

\[ P_C = 117.8 \text{ kN} \]

03.

Sol:

Design of braced built up column service

Axial load, \( P_s = 750 \text{ kN} \)

Effective length of column, \( KL = 10 \text{ m} \)

For M16 Bolt, \( d = 16 \text{ mm}, \quad d_o = 18 \text{ mm} \)

For grade 4.6 Bolt, \( f_{ub} = 400 \text{ MPa} \)

ISM\( C 300@ 363 \text{ N/m} \)

\[ A = 4630 \text{ mm}^2, \quad r_{zz} = 118 \text{ mm} \]

\[ g = 50 \text{ mm}, \quad r_{yy} = 26 \text{ mm} \]

\[ C_{yy} = 23.5 \text{ mm} \]
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.

\[ r_{yy} \leq r \Rightarrow r_{yy} \geq r_{zz} \]

Min radius of gyration of built up section.

\[ r_{min} = r = r_{zz} = 118 \text{ mm} \]

Effective slenderness ratio of built up column

\[ kL = \frac{kL}{r_{min}} = \frac{kL}{r_{zz}} = \frac{10000}{118} = 84.74 \]

Increased slenderness ratio

\[ 1.05 \times 84.7 = 88.97 \]

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

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

\[ f_{cd} = 122.54 \text{ N/mm}^2 \]

Design compressive strength of built up section

\[ P_d = f_{cd} \times A_c = 122.54 \times (2 \times 4630) = 1134.76 \times 10^3 \text{ N} \]

\[ P_d = 1134.76 \text{ kN} \geq P = 1125 \text{ kN} \]

Hence built up section is safe

Let \( S \) be the back to back spacing between channels. Equating.

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

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

\[ S = 182.69 \text{ mm} = 183 \text{ mm} \]

**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

<table>
<thead>
<tr>
<th>CL/r</th>
<th>( f_{cd} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>168</td>
</tr>
<tr>
<td>70</td>
<td>152</td>
</tr>
<tr>
<td>80</td>
<td>136</td>
</tr>
<tr>
<td>90</td>
<td>121</td>
</tr>
<tr>
<td>100</td>
<td>107</td>
</tr>
<tr>
<td>110</td>
<td>94</td>
</tr>
<tr>
<td>120</td>
<td>83</td>
</tr>
<tr>
<td>130</td>
<td>74</td>
</tr>
</tbody>
</table>
\( \frac{L}{r_{yy}} = \frac{566}{26} = 21.7 \leq 50 \leq 59.3 \)

\( \Rightarrow \) 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,

\( \frac{KL}{r_{min}} \leq 145 \)

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

For bolting,

\( Kl/l = 400 \) mm

Min width of flat lacing member

\( b = 3d = 3 \times 16 = 50 \) mm

Minimum thickness of single flat lacing

\( t \geq \frac{\ell}{40} \geq \frac{400}{40} = 10 \) mm

Adopt size of flat lacing member as 50ISF 10

Minimum radius of gyration of flat lacing member

\[
 r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{50 \times 10^3}{12} \frac{1}{50 \times 10}} = \frac{10}{\sqrt{12}} = 0.288 \text{ mm}
\]

\( KL = 400 \) r_{min} \leq 145

Hence lacing member is free from local buckling

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

\[
\frac{L}{r_{min}} \leq 50
\]

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

\( \Rightarrow \) whichever is lesser
(c) \(40^\circ \leq \theta \leq 70^\circ\)

Hence ok,

Transverse shear force

\[ V = 2.5 \times \text{Design load column load} \]

\[ = 2.5 \times \frac{1125}{100} \]

\[ V = 28.10 \text{kN} \]

Design axial force in lacing member

\[ F = \frac{V}{\sin \theta \times 2} \]

\[ F = 19.88 \text{kN} \]

For \( \frac{kL}{r_{\text{min}}} = 138.8 \)

Use \( \frac{kL}{r_{\text{min}}} = 130 \)

\( f_{\text{cd}} = 74 \)

\( 65 \)

\[ \therefore f_{\text{cd}} = 74 - \frac{(74 - 65)}{(140 - 130)} (138.8 - 130) \]

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

Design compressive strength of lacing member

\[ P_d = f_{\text{cd}} \times A_e \]

\[ = 66.08 \times 50 \times 10 \]

\[ P_d = 33.04 \geq F = 19.88 \text{N} \]

Which is safe

Design tensile strength of flat based on gross section yielding

\[ T_{\text{dg}} = A_g \frac{f_y}{\gamma_{m_i}} \]

\[ = 50 \times 10 \times \frac{250}{1.10} \]

\[ T_{\text{dg}} = 113.6 \text{kN} \geq F = 19.88 \text{kN} \]

Design tensile strength based on rupture

\[ T_{\text{dn}} = 0.9A_n \frac{f_u}{\gamma_{m_i}} = 0.9(50 - 1\times18) \times \frac{410}{1.25} \times 10 \]

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

07. Column Bases & Column Splices

02. Ans: 9

Sol: Bearing strength of concrete is 0.45 \( f_{\text{ck}} \)

\( f_{\text{ck}} = 20 \text{ Mpa} \)

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

03. Ans: (b)

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

Here maximum bearing strength is 0.45 \( f_{\text{ck}} \)

\[ \therefore 0.45 \frac{f_{\text{ck}}}{650 \times 420} = \frac{2000 \times 10^3}{650 \times 420} \]

\[ \therefore f_{\text{ck}} = 16.28 \]

\[ \therefore \text{Minimum grade of concrete required is M20} \]

04. Ans: 26.3

Sol: \( t_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)}{f_y}} > t_r \)

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

\( t_r = 11.6 \text{ mm}, \) \( b_r = 250 \text{ mm} \)

\( D = 300 \text{ mm} \)
Gusset plate to be provided along full width of base plate cantilever projection is available along length of base plate direction.

05. Ans: (b)

Sol:

\[ t = C \sqrt{\frac{2.75w}{f_y}} \]

\[ W = \frac{P}{L \times B} \]

\[ t \propto a \]

\[ t \propto c \]

\[ t = C \sqrt{\frac{2.75w}{f_y}} \]

\[ W = \frac{P}{L \times B} \]

\[ t \propto a \]

\[ t \propto c \]
d) \[ c = \frac{720 - 400}{2} = 160 \text{ mm} \]

06. Ans: (d)
Sol:

\[ \text{Bending stress} = \frac{M}{Z} = \frac{P e}{6} = \frac{6P e}{f B^2} \]

\[ \text{Combined stress} = \frac{P}{LB} + \frac{6P e}{BL^2} \]

\[ Z = \frac{BL^2}{12} \times \left( \frac{2}{L} \right) = \frac{BL^2}{6} \]

\[ \therefore \text{ Zero at one end compression at other end.} \]

10. Ans: (a)
Sol:

Total axial compressive load = 1600 kN
Rivet valve \( R_V = 40 \text{ kN} \)
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) = 250 mm
- Thickness of flange (t) = 13.6 mm
- Yield stress of steel (f_y) = 250 N/mm^2
- Partial safety factor (γ_m) = 1.10
Sol:

From the given figure:

Area of base plate \((A_P) = 500 \times 400 \text{ mm}^2\)

Thickness of base plate \((t) = 16 \text{ mm}\)

Larger & shorter projection \((a \& b) = 75 \text{ mm} \& 75 \text{ mm} \) respectively

- Concrete pressure \((w) = \frac{\text{factored load } (P_u)}{A_P} = \frac{1800 \times 10^3}{500 \times 400} = 9 \text{ MPa}\)

For M20 concrete, bearing strength = 0.45\(f_{ck}\) = 9 MPa

Hence concrete is safe in bearing.

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

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

\(t_{\text{required}}) = 19.74 > 16 \text{ mm}\)

Hence thickness provided of slab is not safe.

Minimum thickness should have been 19.74 mm \(
\approx 20 \text{ mm}\)

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 185 N/mm² and permissible bearing stress in concrete pedestal is 4 N/mm².

Sol:

Given load \((P) = 2500 \text{ kN}\)

Concrete grade = M15

Depth of column = 300 mm

Width of flange = 350 mm

Permissible bearing stress in slab \((\sigma_{bs}) = 185 \text{ MPa}\)

Permissible bearing stress in concrete \((w) = 4 \text{ MPa}\)
• Area of base plate required to avoid bearing failure of concrete

\[
A_{(P, \text{required})} = \frac{P}{4 \text{ MPa}} = \frac{2500 \times 10^3}{4} = 625000 \text{ mm}^2
\]

• To design most efficient section, the dimensions must be designed such that \( a = b \)

\[
L = (300 + 50 + 2a) = (350 + 2a)
\]

\[
B = (350 + 2b)
\]

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

\[
\Rightarrow a = b = 220.28 \text{ mm} \approx 225 \text{ mm}
\]

Design \( L = B = 350 + 2 \times 225 = 800 \text{ mm} \)

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

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

\[
\Rightarrow \text{Design plate of dimensions are}
\]

800 mm \( \times \) 800 mm \( \times \) 55 mm

08. Beams

08. Ans: 100

Sol: \( Z_c = 500 \text{ cm}^3, Z_p = 650 \text{ cm}^3 \)
Laterally unrestrained beam semi compact section

Design bending compressive stress \( f_{bd} = 200 \text{ MPa} \)

The flexural (or) bending strength

\[
M_d = \beta_b \cdot Z_p \cdot f_{bd} = \frac{Z_c}{Z_p} \cdot Z_p \cdot f_{bd}
\]

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

\( M_d = 100 \text{ kN-m} \)

09. Ans: 669.2

Sol:

ISMB = 500, \( \text{h} = 500 \text{ mm} \)

\( f_y = 250 \text{ N/mm}^2, \) shear stress \( = \frac{f_y}{\sqrt{3}} \)

\( f_u = 410 \text{ MPa}, \gamma_{m0} = 1.10, \gamma_m = 1.25 \)

Design shear strength \( = (V_d) = \text{Design shear stress} \times \text{shear area} \)

\[
= \frac{f_y}{\sqrt{3} \gamma_{m0}} \times (h \cdot t_w)
\]

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

\[
\frac{x_u}{50} = \frac{300 - x_u}{150}
\]

\( x_u = 75 \text{ mm} \)

17. Ans: (c)
Sol: Flanges and cover plates in cover plates are un stiffened, the outstand of flange or cover 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 \( 200 \times 20 \text{ mm} \) 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

\[
\begin{align*}
\text{Design moment (M) at C} &= R_B \times 3 \\
R_A &= \frac{200}{2} + 200 \times \frac{0.5}{6} \\
&= 350 \text{ kN-m}
\end{align*}
\]

- Loading condition for maximum shear force is shown below

\[
\begin{align*}
\text{Design shear force (V) at A} &= R_A = 200 + 200 \times \frac{3.5}{6} \\
&= 316.67 \text{ kN}
\end{align*}
\]
1. Section:

Area of web \( (A_w) = 800 \times 6 = 4800 \text{ mm}^2 \)

\[ I_{ZZ} \text{ of beam} = 2 \times \left\{ \frac{200 \times 20^3}{12} + 200 \times 20 \times 410^3 \right\} + \left\{ \frac{6 \times 800^3}{12} \right\} \]

\[ = 1601.06 \times 10^6 \text{ mm}^4 \]

Check for bending stress

Bending stresses developed @ extreme fibres (\( f_{b, \text{developed}} \))

\[ f_{b, \text{developed}} = \frac{M}{I} \times y \]

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

\[ = 91.8 \text{ MPa} \]

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

Check for shear

Shear stress developed (\( \tau_{\text{develop}} \))

\[ \tau_{\text{develop}} = \frac{V}{A_w} \]

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

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

Hence safe in shear

\( \therefore \) Given section is safe in bending & shear.

02.

Sol: Effective span, \( L = 5 \text{ m} \)

Service loads:

- UDL and DL, \( w_d = 20 \text{ kN/m} \)
- UD Load LL, \( w_l = 20 \text{ kN/m} \)
- Yield strength, \( f_y = 250 \text{ MPa} \)

Note: for calculating deflections, service loads only used.

For moment of shear, factored loads only used.

\[ \gamma_{mo} = 1.10; \quad \gamma_f = 1.50 \]

Failures considered:

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

Plastic classification:

(i) \( \frac{b}{f_y} \leq 9.4 \)

(ii) \( \frac{d}{t_w} \leq 67\varepsilon \)

Calculation of design (or) factored loads:

Factored dead load = \( \gamma_f \times w_d \)

\[ = 1.5 \times 20 = 30 \text{ kN/m} = \text{FLL} \]

Factored DL & LL

\[ w = 60 \text{ kN/m} \]
Design BM:
\[ M = \frac{w\ell^2}{8} = \frac{60 \times 5^2}{8} = 187.5 \text{ kN-m} \]

Design SF:
\[ V = \frac{w\ell}{2} = \frac{60 \times 5}{2} = 150 \text{ kN} \]

Section dessification
\[ b = \frac{b_f}{2} = \frac{165}{2} = 82.5 \text{ mm} \]
\[ t_f = 11.4 \text{ mm}; \ t_w = 7.4 \text{ mm} \]

Depth of web between root of fillets
\[ d = h - 2 \times (t_f + r_1) = 350 - 2 \times (11.4 + 16) = 295.2 \text{ mm} \]

Hence the section is plastic section

Check for low shear case:

Design shear strength of section.
\[ V_d = A_v \times \frac{f_y}{\sqrt{3} \gamma_{mo}} = (h \times t_w) \times \frac{f_y}{\sqrt{3} \gamma_{mo}} \]
\[ V_d = 350 \times 7.4 \times \frac{250}{\sqrt{3} \times 1.10} = 339.85 \text{ kN} \geq V = 150 \text{ kN} \]

(Which is safe against shear)

When low shear case \( V \leq 0.6 \ V_d \)
\[ 150 \text{ kN} = 0.6 \times 339.85 = 203.91 \text{ kN} \]

Design bending strength of section
\[ M_d = \beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}} \leq 1.2 Z_c \cdot \frac{f_y}{\gamma_{mo}} \]

(For plastic section \( \beta_b = 1.0 \))
\[ M_d = 1.0 \times 857.11 \times 10^3 \times \frac{250}{1.10} = 193.43 \times 10^6 \leq 1.2 \times 151.9 \times 10^3 \times \frac{250}{1.10} = 205.06 \text{ kN-m} \]

Also,
\[ M_d = 193.43 \text{ kN-m} \leq M = 187.5 \text{ kN-m} \]

Which is safe bending strength

Design for deflections:

Limit deflections for Britliness
\[ \varepsilon = \frac{300}{L} \times \frac{5000}{300} = 16.67 \text{ mm} \]

Maximum calculated deflect under service loads.
\[ \Delta_{cal} = \frac{5}{384} \times \frac{w \ell^4}{EI} \]
\[ \Delta_{\text{cal}} = \frac{5}{384} \times \frac{40 \times 5000^4}{2 \times 10^5 \times (751.9 \times 10^3 \times \frac{350}{2})} \]

\[ \Delta_{\text{cal}} = 12.36 \text{ mm} \leq \Delta_{\text{limt}} = 16.67 \text{ mm} \]

Which is safe against deflections

03.

Sol: Effective span of beam \( L = 5.0 \text{ m} \)

Yield stress of steel \( f_y = 250 \text{ Mpa} \)

Section classification

Section classification \( \epsilon = \sqrt{\frac{250}{f_y}} = 1.0 \)

Outstand of flange \( b = \frac{b_f}{2} = \frac{165}{2} = 82.5 \text{ mm} \)

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

Moment of inertia of joist about major axis

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

\[ I_{zz} = 129.47 \times 10^6 \text{ mm}^4 \]

Elastic section modulus about major axis \( Z_{ez} \)

\[ Z_{ez} = \frac{I_{zz}}{y} = \frac{129.47 \times 10^6}{(350/2)} = 7.39 \times 10^3 \text{ mm}^3 \]

Plastic section modulus about major axis \( Z_{pz} \)

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

\[ Z_{pz} = 834.97 \times 10^3 \text{ mm}^3 \]

Since \( V \leq 0.6 V_d \) this is low shear case

Design bending strength of laterally supported beam

\[ M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \leq 1.2 Z_e \frac{f_y}{\gamma_{mo}} \]

For plastic section \( \beta_b = 1 \)

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

\[ = 189.76 \times 10^6 \text{ N-mm} = 189.76 \text{ kN-m} \]

\[ \leq 1.2 Z_e \frac{f_y}{\gamma_{mo}} = 1.2 \times 739.83 \times 10^3 \times \frac{250}{1.10} \]
ESE-Postal Coaching Solutions

\[
\Delta \text{limit} = \Delta_{cat} = \frac{5}{384} \frac{wL^4}{EI}
\]

16.67 = \frac{5}{384} \frac{(w_s)(5000)^4}{2 \times 10^5 \times 129.47 \times 10^6}

\Rightarrow w_s = 53.04 \text{ kN/m} \quad (3)

Factored distributed load on beam
\[
w = \gamma_f \times w_s = 1.5 \times 53.04 = 79.56 \text{ kN/m}
\]

Factored distributed load on beam inclusive of self weight
\[
w = 60.72 \text{ kN/m}
\]

10. Gantry Girders

01. Ans: (a)

Sol:

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

Limiting deflection for a simply supported beam (assuming brittle cladding)
\[
\Delta_{\text{limit}} = \frac{10}{300} = \frac{5000}{300} = 16.66 \text{mm}
\]

Calculated maximum deflection under service loads

Service UDL on beam \(w_s = w/\gamma_f\)

\[
= 10/1.5 = 6.67 \text{ kN/m}
\]
11. Roof Trusses

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:

\[ M_{xx} = M \cos \theta \left( \theta = 30^\circ \right) = \frac{\sqrt{3}}{2} M \]
\[ M_{yy} = M \sin \theta = \frac{M}{2} \]

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)
01. Ans: (b)
Sol: Slope of roof = \( \tan \theta \)
      \( \tan \theta = 1 \Rightarrow \theta = \tan^{-1} = 45^\circ \)

Conventional Practice Solutions

01.
Sol:

Given:
Slope of truss (\( \theta \)) = 30°
Wind pressure (P) = 2 kN/m²
       = 2 kN/m² \times 1.5 m
       = 3 kN/m
Self weight = 166.77 N/m of purlin
Weight of galvanised sheet = 130 N/m²
       = 130 N/m² \times 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
\[ (M_{zz}) = \gamma_f \times (P + G \cos \theta) \times \frac{r^2}{10} \]
Design moment capacity along y-y ($M_{dyy}$)

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

$$= 6.58 \text{ kN-m}$$

Design shall be considered safe as per IS 800:2007 if interaction equation is satisfied

$$\left( \frac{M_{uzz}}{M_{dzz}} \right) + \left( \frac{M_{uzy}}{M_{dyy}} \right) = 0.955 < 1$$

Hence the design is safe

Total factored moment developed along Y-Y ($M_{uyy}$)

$$= \gamma_f \times G \sin \theta \times \frac{f^2}{10}$$

$$= 1.95 \text{ kN-m}$$

As per IS 800:2007

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

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

$$= 25.95 \text{ kN-m}$$