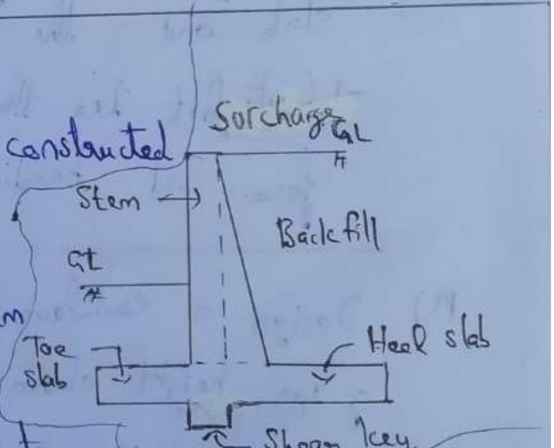
Unit I - Retaining Walls

Reinforced concrete Cantilever and Counterfort Retaining Walls. Horizontal backfill with Surcharge. Design of Shear Key. Design and Drawing.

Retaining walls:

Retaining walls are generally used to retain earth or such materials to maintain unequal levels on its two faces. Retaining walls are used in construction of railways, highways, bridges, canals, basements below ground level, wing walls of bridges, swimming pools and to retain slopes in hilly terrain roads. Retaining walls should be designed to resist lateral earth pressure on wall from sides, soil pressure acting vertically on the footing slab.

Cantilever retaining wall



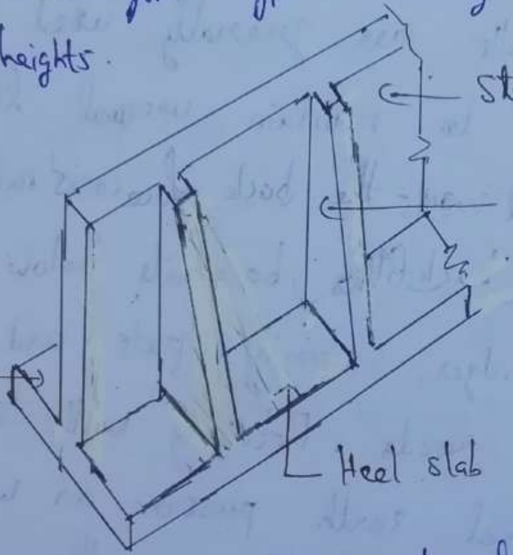
- Cantilever retaining walls are constructed of reinforced concrete
- They consists of relatively thin stem and a base slab.
- The base slab is divided into two parts, the heel and toe. Heel is the part of the base slab at backfill side under the backfill. The toe is the portion of the footing at front of wall
- Stem is the vertical member holding the backfill.
- Shear key projects down the footing of retaining walls to wall's sliding.
- Backfill refers to the soil behind the wall.
- Surcharge is an additional load applied on top of

retaining wall on ground surface

Counterfort retaining wall

→ For larger heights exceeding 5m of earth fill, the bending moment developed in stem, heel and toe slabs are very large resulting in larger thickness of elements which is uneconomical.

→ Hence counterfort type retaining walls are adopted for larger heights.



→ Counterfort retaining wall consists of stem, toe slab and heel slab and the counterforts which subdivide the stems

→ Counterfort tie the slab and base together, reduce shear force and bending moments imposed on wall by soil

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Pb) Design a cantilever retaining wall to retain an embankment of 4m height above ground level. The density of earth is 18 kN/m³ and its angle of repose is 30°. The earth embankment is horizontal at top. The safe bearing capacity of soil is 200 kN/m² and the coefficient of friction between the soil and concrete is 0.5. Adopt M20 grade concrete and Fe 415 HYSD bars.

Retaining wall with horizontal backfill

Design data

→ Height of embankment above ground level = 4m

→ Density of earth, ρ = 18 kN/m³

Angle of repose, $\phi = 30°$

Safe bearing capacity, $\sigma = 200 \, kN/m^2$

Coefficient of friction = 0.5

M20 grade concrete, HYSD bars

Solution

Step 1 - Dimensions of retaining wall

→ Minimum depth of foundation, $d = \left(\frac{\sigma}{e}\right)\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$

$= \left(\frac{200}{18}\right)\left(\frac{1-\sin 30}{1+\sin 30}\right)^2$

$= 1.235 \, m$

Provide depth of foundation, $d = 1.25 \, m$

Overall height of wall, $H = 4 + 1.25 = 5.25 \, m$

→ Thickness of base slab $= H/12 = 5.25/12 = 0.438 \, m \simeq 450 \, mm$

(i) Min. thickness = 300mm

(ii) Thickness of base slab = 450mm

Adopt thickness of of stem as 450mm

→ Width of base slab $= 0.5H$ to $0.6H$

$= (0.5 \times 5.25)$ to (0.6×5.25)

$= 2.625$ to $3.15 \, m$

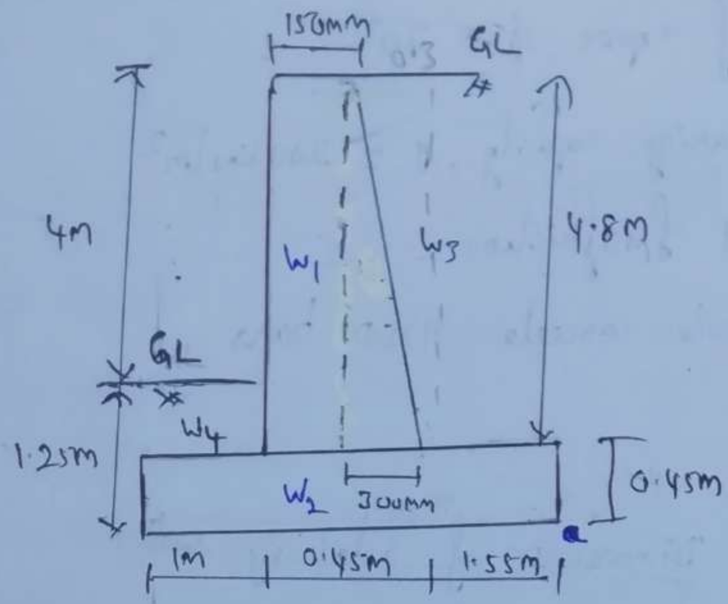
$= 3 \, m$ (say)

→ Height of stem, $h = H -$ base slab thickness

$= 5.25 - 0.45$

$= 4.8 \, m$

→ Toe projection $= b/3 = 3/3 = 1 \, m$



Step 2 - Design of stem

→ Moment, $M = (k_a \rho h^3)/6$

Where $k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = 0.333$

$$M = \frac{0.333 \times 18 \times 4.8^3}{6} = 110.481 \text{ KNm}$$

Factored moment, $M_u = 1.5 \times 110.481 = 165.722$ KNm

→ $M_u = 0.138 f_{ck} b d^2$

$165.722 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$d = 245.039$ mm

$d \simeq 250$ mm

Adopt cover as 50 mm, $D = 250 + 50 = 300$ mm

(i) overall depth = 300mm

(ii) Base slab thickness = 450mm

Adopt thickness as 450mm at bottom and 150mm at top

→ Main bars

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$$

$d = 450 - 50 = 400$ mm

$$165.722 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \times \left[1 - \frac{A_{st} \times 415}{1000 \times 400 \times 20}\right]$$

$$A_{st} = 1225.396\ mm^2$$

Minimum $A_{st} = 0.12\% \, bD$

$$= \frac{0.12 \times 1000 \times 450}{100}$$

$$= 540\ mm^2$$

$$\therefore A_{st} = 1225.396\ mm^2$$

Provide 16mm diameter bars, Spacing $= \frac{1000 \times a_{st}}{A_{st}}$

$$= \frac{1000 \times \left(\frac{\pi}{4} \times 16^2\right)}{1225.396}$$

$$= 164.079\ mm$$

Provide 16mm diameter bars at 160mm c/c

Provided $A_{st} = \frac{1000 \times a_{st}}{\text{spacing}}$

$$= \frac{1000 \times \frac{\pi}{4} \times 16^2}{160}$$

$$= 1256.637\ mm^2$$

→ <u>Distribution bars</u>

Minimum $A_{st} = 0.12\% \, bD$

$$= \frac{0.12 \times 1000 \times 450}{100}$$

$$= 540\ mm^2$$

Provide 10mm diameter bars, Spacing $= \frac{1000 \times \frac{\pi}{4} \times 10^2}{540}$

$$= 145.444\ mm$$

Provide 10mm diameter bars at 140mm c/c

$$\text{Provided } A_{st} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{140} = 561 mm^2$$

Step 3- Stability check

Load	Magnitude (kN)	Distance (m)	Moment @ a (kNm)
W_1 (weight of stem)	$4.8 \times 0.15 \times 25 = 18$	$(0.15/2) + 0.3 + 1.55 = 1.925$	34.65
	$\frac{1}{2} \times 4.8 \times 0.3 \times 25 = 18$	$(\frac{2}{3} \times 0.3) + 1.55 = 1.75$	31.5
W_2 (weight of base slab)	$3 \times 0.45 \times 25 = 33.75$	$3/2 = 1.5$	50.63
W_3 (weight of earth fill)	$1.55 \times 4.8 \times 18 = 133.92$	$1.55/2 = 0.78$	104.46
	$\frac{1}{2} \times 4.8 \times 0.3 \times 18 = 12.96$	$(\frac{1}{3} \times 0.3) + 1.55 = 1.65$	21.38
W_4 (weight of earth fill)	$(1.25 - 0.45) \times 1 \times 18 = 14.4$	$(\frac{1}{2}) + 0.45 + 1.55 = 2.5$	36
Moment due to earth pressure			110.481

$\Sigma W = 231.03 kN$, $\Sigma M = 389.101 kNm$

Point of resultant force acting from base, $z = \frac{\Sigma M}{\Sigma W} = \frac{389.101}{231.03} = 1.684 m$

Eccentricity, $e = z - b/2 = 1.684 - (3/2) = 0.184$

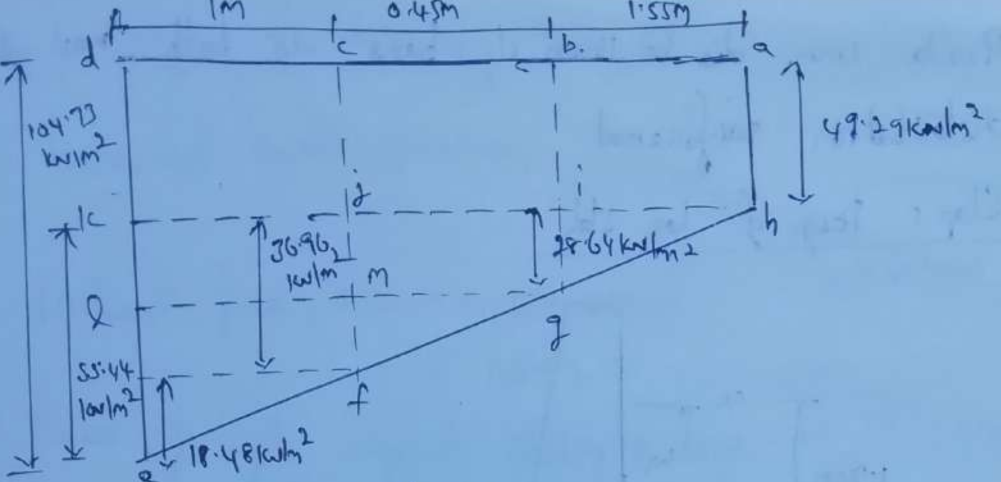
Maximum eccentricity, $e = b/6 = 3/6 = 0.5$

Hence safe

$\rightarrow \sigma_{max/min} = \frac{\Sigma W}{b}\left[1 \pm \frac{6e}{b}\right] = \frac{231.03}{3}\left[1 \pm \frac{6 \times 0.18}{3}\right]$

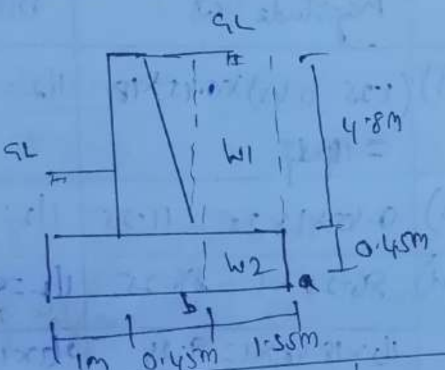
$\sigma_{max} = 104.73 kN/m^2 < 200 kN/m^2$

$\sigma_{min} = 49.29 kN/m^2$



Step 4 - Design of heel slab

→ Net moment on structure



Load	Magnitude (kN)	Distance (m)	Moment @ b (kNm)
w_1 (weight of earthfill)	$1.55 \times 4.8 \times 18 = 133.92$	$1.55/2 = 0.78$	104.46
w_2 (weight of base slab)	$1.55 \times 0.45 \times 25 = 17.44$	$1.55/2 = 0.78$	13.6
upward pressure (abhi)	$49.29 \times 1.55 = 76.40$	$1.55/2 = 0.78$	59.59
upward pressure (igh)	$1/2 \times 28.64 \times 1.55 = 22.2$	$1/3 \times 1.55 = 0.52$	11.54
Net moment on structure = 118.06 − 71.13			46.93

$M_u = 1.5 \times 46.93 = 70.40$ kNm

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$ where $d = 450 - 50 = 400$ mm, $b = 1000$ mm

$$70.4 \times 10^6 = 0.87 \times 415 \times 400 \times A_{st} \left[1 - \frac{A_{st} \times 415}{1000 \times 400 \times 20}\right]$$

$A_{st} = 500.46 \text{ mm}^2$

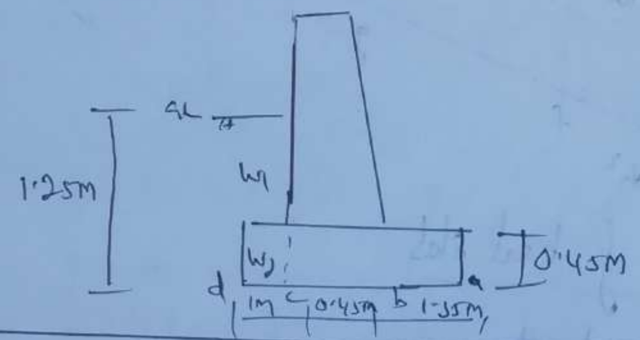
Minimum $A_{st} = 0.12\% bD = \frac{0.12}{100} \times 1000 \times 450 = 540 \text{ mm}^2$

Provide 10mm dia bars, spacing $= \frac{1000 \times \left(\frac{\pi}{4} \times 10^2\right)}{540} = 145.44$ mm

Provide spacing 140mm, $A_{st} = \frac{1000 \times \left(\frac{\pi}{4} \times 10^2\right)}{140} = 561 \text{ mm}^2$

Provide 10mm dia @ 140mm c/c bars as both main and distribution reinforcement.

Step 5 - Design of toe slab.



Load	Magnitude (kN)	Distance (m)	Mom @ c (kNm)
W_1 (weight of earthfill)	$(1.25-0.45)\times1\times18$ = 14.4	$1/2 = 0.5$	7.20
W_2 (weight of baseslab)	$0.45\times1\times25 = 11.25$	$1/2 = 0.5$	5.63
Upward pressure (dcnt)	$86.25\times1 = 86.25$	$1/2 = 0.5$	43.13
Upward pressure (nfe)	$1/2\times18.48\times1 = 9.24$	$2/3\times1 = 0.67$	6.19
Net moment of structure = 12.83 ~ 49.32			36.49

$M_u = 1.5\times36.49 = 54.74$ kNm

$M_u = 0.87 f_y A_{st} d\left[1-\frac{A_{st}f_y}{bdf_{ck}}\right]$ where $d = 450-50 = 400$mm, $b = 1000$mm

$54.74\times10^6 = 0.87\times415\times A_{st}\times400\left[1-\frac{A_{st}\times415}{1000\times400\times20}\right]$

$A_{st} = 386.79\text{mm}^2$

Minimum $A_{st} = 0.12\%\ bD = \frac{0.12}{100}\times1000\times450$

$= 540\text{mm}^2$

Provide 10mm dia bars, Spacing $= \frac{1000\times\frac{\pi}{4}\times10^2}{540} = 145.44$mm

Provide spacing 140mm, $A_{st} = \frac{1000\times\frac{\pi}{4}\times10^2}{145.44} = 561\text{mm}^2$

Provide 10mm dia @ 140mm c/c bars as both main and distribution reinforcement.

Step 6 - Design of Shear key

→ Horizontal earth pressure, $P = \frac{k_a \rho H^2}{2} = \frac{0.333 \times 18 \times 5.25^2}{2}$

$= 82.6 kN$

Frictional force, $\mu W = 0.5 \times 231.03$

$= 115.52 kN$

Factor of safety against sliding $= \frac{\mu W}{P} = \frac{115.52}{82.6} = 1.4 < 1.5$

Hence shear key is to be provided.

→ Intensity of earth pressure, $P_p = k_p P$

where $k_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 30}{1-\sin 30} = 3$

$\therefore P_p = 3 \times 82.6 = 247.8 kN$

Assuming depth of shear key as 450mm,

Pressure force at key, $P_f = P_p \times 0.45 = 111.51 kN$

Factor of safety $= \frac{\mu W + P_f}{P} = \frac{115.52 + 111.51}{82.6} = 2.75 > 1.5$

→ Minimum $A_{st} = 0.3\% bD = \frac{0.3}{100} \times 1000 \times 450 = 1350 mm^2$

Provide 16 mm dia bars, Spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1350} = 148.93 mm$

Providing 140mm Spacing, $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{140} = 1436.16 mm^2$

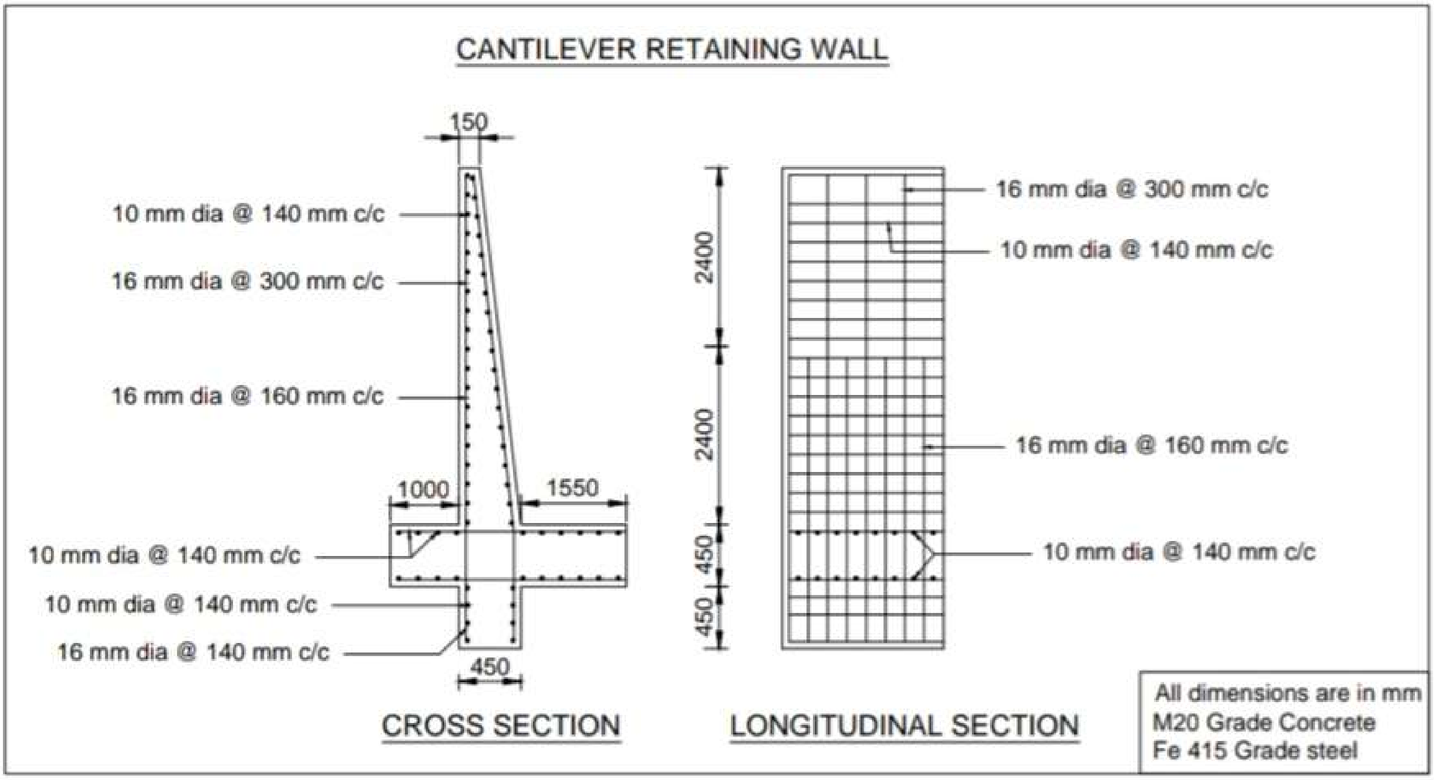
Provide 16mm dia bars at 140mm c/c as main reinforcement

→ Minimum $A_{st} = 0.12\% bD = \frac{0.12}{100} \times 1000 \times 450 = 540 mm^2$

Provide 10mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 10^2}{540} = 145.44 mm$

Provide 140mm Spacing, $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{140} = 561 mm^2$

Provide 10mm dia bars at spacing of 140mm c/c as distribution reinforcement.


CANTILEVER RETAINING WALL
150
10 mm dia @ 140 mm c/c
16 mm dia @ 300 mm c/c
16 mm dia @ 160 mm c/c
1000
1550
10 mm dia @ 140 mm c/c
10 mm dia @ 140 mm c/c
16 mm dia @ 140 mm c/c
450
CROSS SECTION
2400
2400
450
450
16 mm dia @ 300 mm c/c
10 mm dia @ 140 mm c/c
16 mm dia @ 160 mm c/c
10 mm dia @ 140 mm c/c
LONGITUDINAL SECTION
All dimensions are in mm
M20 Grade Concrete
Fe 415 Grade steel

Pb) Design a cantilever retaining wall for the following data

Height of wall above ground level = 3 m

Unit weight of soil = 18 kN/m³ (e)

Angle of internal friction = 30° (ϕ) (Angle of repose)

Coefficient of friction between soil and concrete = 0.5 (μ)

Surcharge angle = 16° (α)

Safe bearing capacity of soil = 100 kN/m² (σ)

Solution: **Retaining wall with sloping backfill or surcharge**

Step 1 – Dimensions of retaining wall

→ Minimum depth of foundation, $d = \frac{\sigma}{e} k_a^2$

$$k_a = \cos\alpha \frac{(\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi})}{(\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi})}$$

$\cos\alpha = 0.961$

$\cos\phi = 0.866$

$$= \cos 16 \frac{(\cos 16 - \sqrt{\cos^2 16 - \cos^2 30})}{(\cos 16 + \sqrt{\cos^2 16 - \cos^2 30})}$$

$= 0.379$

$\therefore d = \frac{100 \times 0.379^2}{18} = 0.8 \text{ m} \simeq 1 \text{ m}$

Overall height of wall, $H = 3 + 1 = 4$ m

→ Thickness of base slab $= H/12 = \frac{4}{12} = 0.33$ m $= 333$ mm

(i) Minimum thickness = 300 mm

(ii) Thickness of base slab = 333

Adopt thickness of base slab as 350 mm

→ Width of base slab $= 0.5H$ to $0.6H = 2$ to 2.4 m

$b = 2.4$ m

(sloping backfill)

→ Toe projection $= \alpha b$ where $\alpha = 1 - \frac{\sigma}{2.7eH} = 1 - \frac{100}{2.7 \times 18 \times 4} = 0.486$

$\therefore$ Toe projection $= 0.486 \times 2.4 = 1.166 \simeq 1.2$ m

Step 2 – Design of stem

Moment, $M = \frac{k_a \gamma h^3}{6} \times \cos\alpha$, where h = height of stem

$= 4 - 0.35 = 3.65m$

$= \frac{0.379 \times 18 \times 3.65^3}{6} \times \cos 16$

$= 55.289\ kNm$

Factored moment, $M_u = 1.5 \times 55.289 = 82.934\ kNm$

→ $M_u = 0.138 f_{ck} b d^2$

$82.934 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

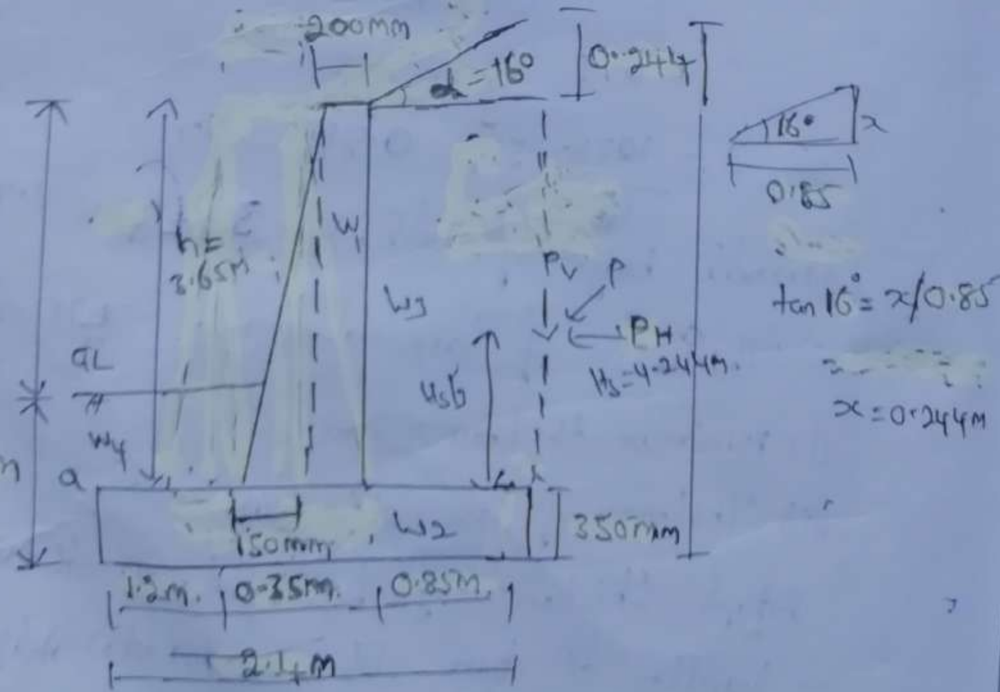
$d = 173.35mm \approx 180mm$

Adopt cover as 40mm, $D = 180 + 40 = 220mm$

(i) Overall depth = 220mm

(ii) Base slab thk = 350mm

Hence adopt thickness as 350mm at bottom and 200mm at top



$\tan 16° = x/0.85$

$x = 0.244m$

→ Main bars

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$ where $d = 350 - 50 = 300mm$

$82.934 \times 10^6 = 0.87 \times 415 \times A_{st} \times 300 \times \left[1 - \frac{A_{st} \times 415}{1000 \times 300 \times 20}\right]$

$A_{st} = 811.188 mm^2$

Minimum $A_{st} = 0.12\% \, bD$

$= \frac{0.12}{100} \times 1000 \times 350$

$= 420 mm^2$

Provide 12mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12}{811.188} = 139.42 mm$

Provide spacing 130mm, $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{130} = 869.98 mm^2$

Provide 12mm dia bars at 130mm c/c as main bars.

→ Distribution bars

Minimum $A_{st} = 0.12\% \, bD$

$= \frac{0.12}{100} \times 1000 \times 350$

$= 420 mm^2$

Provide 10 mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 10^2}{420} = 187 mm$

Provide spacing 180mm, $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{180} = 436.33 mm^2$

Provide 10 mm dia bars at 180mm c/c as distribution bars

Step 3 - Stability check

Earth pressure acting parallel to surcharge, $P = \frac{k_a e H_s^2}{2} = \frac{0.379 \times 18 \times 4.244^2}{2}$

$= 61.437 kN$

Horizontal and vertical components $P_H = P \cos \alpha = 61.437 \times \cos 16 = 59.057 kN$

$P_V = P \sin \alpha = 61.437 \times \sin 16 = 16.934 kN$

Step 3 - Stability check

Load	Magnitude (kN)	Distance (m)	Moment @a (kNm)
W_1 (weight of steam	$0.2 \times 3.65 \times 25 = 18.25$	$\frac{0.2}{2} + 0.15 + 1.2 = 1.45$	26.463
	$\frac{1}{2} \times 0.15 \times 3.65 \times 25 = 6.844$	$(\frac{2}{3} \times 0.15) + 1.2 = 1.3$	8.897
W_2 (weight of base slab)	$2.4 \times 0.35 \times 25 = 21$	$2.4/2 = 1.2$	25.2
W_3 (weight of earth fill)	$0.85 \times 3.65 \times 18 = 55.845$	$\frac{0.85}{2} + 0.35 + 1.2 = 1.975$	110.294
	$\frac{1}{2} \times 0.85 \times 0.244 \times 18 = 1.867$	$(\frac{2}{3} \times 0.85) + 0.35 + 1.2 = 2.117$	3.952
W_4 (weight of earth fill)	$1.2 \times (1 - 0.35) \times 18 = 14.04$	$\frac{1.2}{2} = 0.6$	8.424
Earth pressure due to surcharge	16.934	2.4	40.642

$\Sigma W = 134.78\,kN$, $M_R = 223.872\,kNm$

Overturning Moment, $M_O = P_H \times \frac{H_s}{3} = 59.057 \times \frac{4.244}{3}$
$= 83.546\,kNm$

Net moment, $\Sigma M = M_R - M_O = 223.872 - 83.546 = 140.326\,kNm$

Point of resultant force acting from base, $z = \Sigma M / \Sigma W = 1.041$

Eccentricity, $e = \frac{b}{2} - z = \frac{2.4}{2} - 1.041 = 0.159$

Maximum eccentricity, $e = b/6 = \frac{2.4}{6} = 0.4$

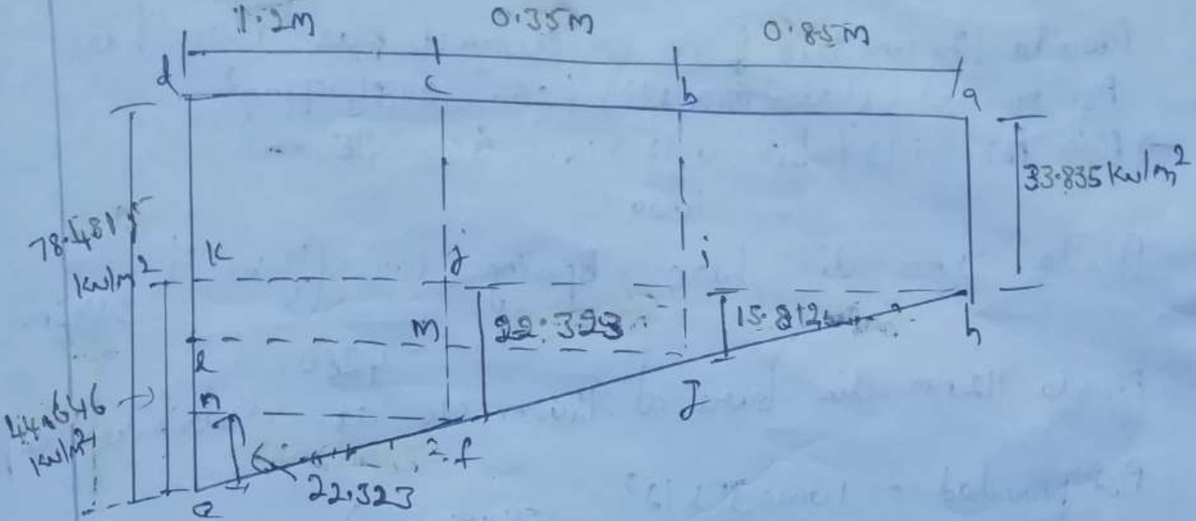
Hence safe

$$\rightarrow \sigma_{max/min} = \frac{\Sigma W}{b}\left[1 \pm \frac{6e}{b}\right]$$

$$= \frac{134.78}{2.4}\left[1 \pm \frac{6 \times 0.159}{2.4}\right]$$

$\sigma_{max} = 78.481\,kN/m^2 < 100\,kN/m^2$

$\sigma_{min} = 33.835\,kN/m^2$



Step 4 – Design of heel slab

Load	Magnitude (kN)	Distance (m)	Moment @b (kNm)
W_1 (Earth fill)	$0.85 \times 3.65 \times 18 = 55.85$	$0.85/2 = 0.425$	23.736
	$1/2 \times 0.85 \times 0.244 = 0.104$	$2/3 \times 0.85 = 0.567$	0.059
W_2 (Slab)	$0.85 \times 0.35 \times 25 = 7.438$	$0.85/2 = 0.425$	3.161
pressure (abhi)	$33.835 \times 0.85 = 28.76$	$0.85/2 = 0.425$	12.223
pressure (igh)	$1/2 \times 15.812 \times 0.85 = 6.72$	$\frac{1}{3} \times 0.85 = 0.283$	1.902
Surcharge	16.934	0.85	14.394
Net moment on structure = 41.35 ∩ 14.125			27.225

$\rightarrow M_u = 1.5 \times 27.225 = 40.838 \text{ kNm}$

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$ where $d = 300\text{mm}$, $b = 1000\text{mm}$

$40.838 \times 10^6 = 0.87 \times 415 \times A_{st} \times 300 \left[1 - \frac{415 \times A_{st}}{1000 \times 300 \times 20}\right]$

$A_{st} = 387.411 \text{mm}^2$

Minimum $A_{st} = 0.12\%\, bD = \frac{0.12}{100} \times 1000 \times 350 = 420 \text{mm}^2$

Provide 12 mm dia, Spacing $= \frac{1000 \times \left(\frac{\pi}{4} \times 12^2\right)}{420} = 269.28 \text{ mm}$

Provide 12 mm dia bars @ 260mm c/c as main bars

A_{st} provided $= (1000 \times \pi/4 \times 12^2) / 260 = 434.99 mm^2$

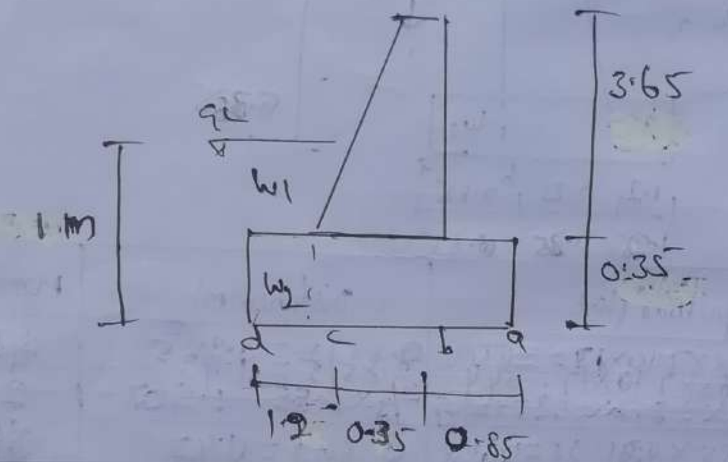
→ Min $A_{st} = 0.12\% \, bD = \frac{0.12}{100} \times 1000 \times 350 = 420 mm^2$

Provide 12mm dia bars, Spacing $= \frac{1000 \times (\frac{\pi}{4} \times 12^2)}{420} = 269.28 mm$

Provide 12mm dia bars at 260mm c/c as distribution reinforcement

A_{st} provided $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{260} = 434.99 mm^2$.

Step 5 - Design of toe slab



Load	Magnitude (kN)	Distance (m)	Moment @ c (kNm)
W1 (weight of fill)	(1-0.35) × 1.2 × 18 = 14.04	1.2/2 = 0.6	8.424
W2 (weight of slab)	1.2 × 0.35 × 25 = 10.5	1.2/2 = 0.6	6.3
Pressure (dcnf)	55.708 × 1.2 = 66.85	1.2/2 = 0.6	40.11
Pressure (nfa)	1/2 × 22.323 × 1.2 = 13.394	$\frac{2}{3}$ × 1.2 = 0.8	10.715
Net moment	on structure = 14.724 ~ 50.825		36.101

→ $M_u = 1.5 \times 36.101 = 54.152 kNm$

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$ where d = 300mm, b = 1000mm

$54.152 \times 10^6 = 0.87 \times 415 \times A_{st} \times 300 \left[1 - \frac{415 A_{st}}{1000 \times 300 \times 20}\right]$

$A_{st} = 518.551\ mm^2$

Min $A_{st} = 0.12\%.bD = \frac{0.12}{100} \times 1000 \times 350 = 420\ mm^2$

Provide 12mm dia, Spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{518.55} = 218.103\ mm$

Provide 12mm dia @ 210 mm c/c as main reinforcement

Ast provided $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{210} = 538.6\ mm^2$

→ Min $A_{st} = 0.12\%.bD = \frac{0.12}{100} \times 1000 \times 350 = 420\ mm^2$

Provide 12mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{420} = 269.5\ mm$

Provide 12 mm dia bars at 260mm c/c as distribution st.

Ast provided $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{260} = 434.99\ mm^2$

Step 6 - Design of Shear key (Stability against sliding)

→ ~~Horizontal earth pressure, $R = \frac{k_a e H^2}{2} = \frac{0.7 \times 17 \times 7.25^2}{2}$~~

~~$= 357.37\ kN$~~

Frictional force $\mu w = 0.5 \times 134.78$

$= 67.39\ kN$

Factor of safety against sliding $= \frac{\mu w}{P_H} = \frac{67.39}{59.057} = 1.141$ < 1.5

Hence shear key is to be provided

→ Intensity of earth pressure, $P_p = k_p P = \frac{1}{k_a} \times P$

$K_p = 1/0.379 = 2.639$, Pressure at jn of toe slab & key

$P_p = 2.639 \times (33.835 + 22.323) = 148.201$ kN

Assuming depth of shear key as 350mm,

Pressure force at key, $P_f = P_p \times 0.35 = 148.201 \times 0.35$

$= 51.87$ kN

Factor of safety $= \frac{\mu w + P_f}{P_H} = \frac{67.39 + 51.87}{59.057}$

$= 2.019 > 1.5$ (Hence safe)

→ Min $A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 350 = 1050 \, mm^2$

Provide 16mm dia bars, Spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1050} = 191.48$ mm

Provide 16mm dia bars @ 190mm c/c as main reinforcement

Provided $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{190} = 1058.22 \, mm^2$

→ Min $A_{st} = 0.12\% \, bD = \frac{0.12}{100} \times 1000 \times 350 = 420 \, mm^2$

Provide 12mm dia bars, Spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{420}$

$= 269$ mm

Provide 12mm dia bars @ 260mm c/c as distribution bars,

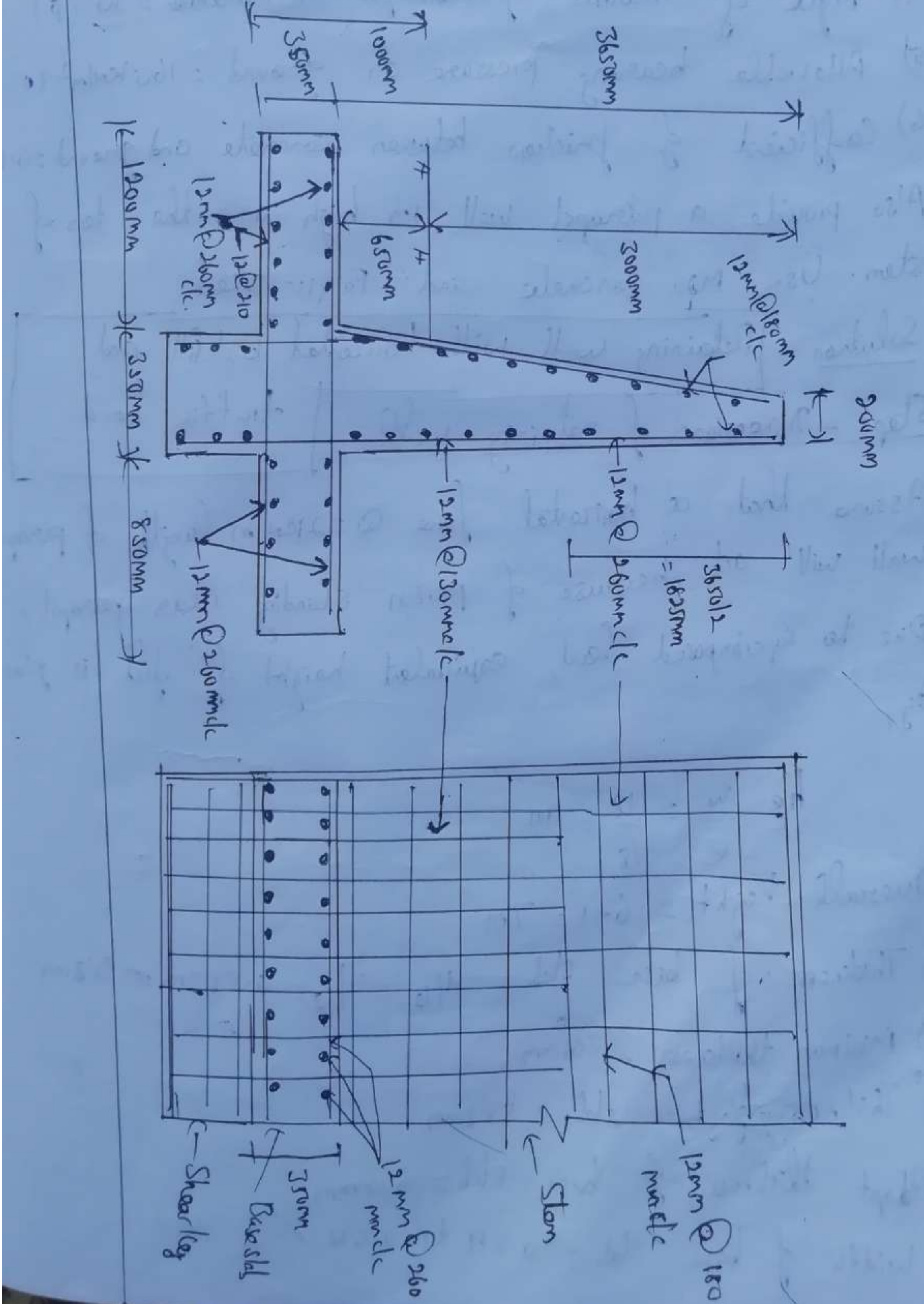
Provided $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{260} = 434.99 \, mm^2$

Step 7 - Stability against overturning

Factor of safety against overturning $= \frac{M_R}{M_O}$

$= \frac{223.872}{83.546} = 2.68 > 2$

Hence safe



P1) Design a Cantilever retaining wall for a road for the following requirements

1) Height of wall from the bottom of base to top of stem = 6m

2) Superimposed load due to road traffic = 18 kN/m³ (w)

3) Unit weight of fill = 18 kN/m³ (e)

4) Angle of internal friction for fill material = 30° (ϕ)

5) Allowable bearing pressure on ground = 160 kN/m² (σ)

6) Coefficient of friction between concrete and ground = 0.4

Also provide a parapet wall 1m high on the top of stem. Use M20 concrete and Fe415 steel

Solution: Retaining wall with horizontal backfill and traffic load

Step 1 - Dimensions of retaining wall

→ Assume that a horizontal force Q = 2 kN/m length of parapet wall will act because of person standing near parapet.

Due to superimposed load, equivalent height of fill is given by,

$$h_e = \frac{w}{e} = \frac{18}{18} = 1m$$

Overall height = 6+1 = 7m

→ Thickness of base slab = H/12 = 7/12 = 0.583m ≃ 583mm

(i) Minimum thickness = 300mm

(ii) Thickness of base slab = 583mm

Adopt thickness of base slab = 600 mm

→ Width of base slab = 0.5H to 0.6H

$= (0.5 \times 7)$ to (0.6×7)

$= 3.5m$ to $4.2m$

Width of base slab, $b = 4m$

→ Toe projection $= \alpha b$

where $\alpha = 1 - \frac{\sigma}{2.2 e H}$ ← traffic load

$= 1 - \frac{160}{2.2 \times 18 \times 7}$

$= 0.423$

Toe projection $= 0.423 \times 4 = 1.692m$

Keep toe projection $= 1.7m$

∴ Height of stem, $h = 6 - 0.6 = 5.4m$

Step 2 - Design of stem

Moment, $M = \frac{k_a e h^3}{6} + \frac{k_a w h^2}{2} + Q(H_1 + 1)$

$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30}{1 + \sin 30} = 0.333$

$\therefore M = \frac{0.333 \times 18 \times 5.4^3}{6} + \frac{0.333 \times 18 \times 5.4^2}{2} + 2(5.4 + 1)$

$= 157.367 + 87.393 + 12.8$

$M = 257.5$ KNm

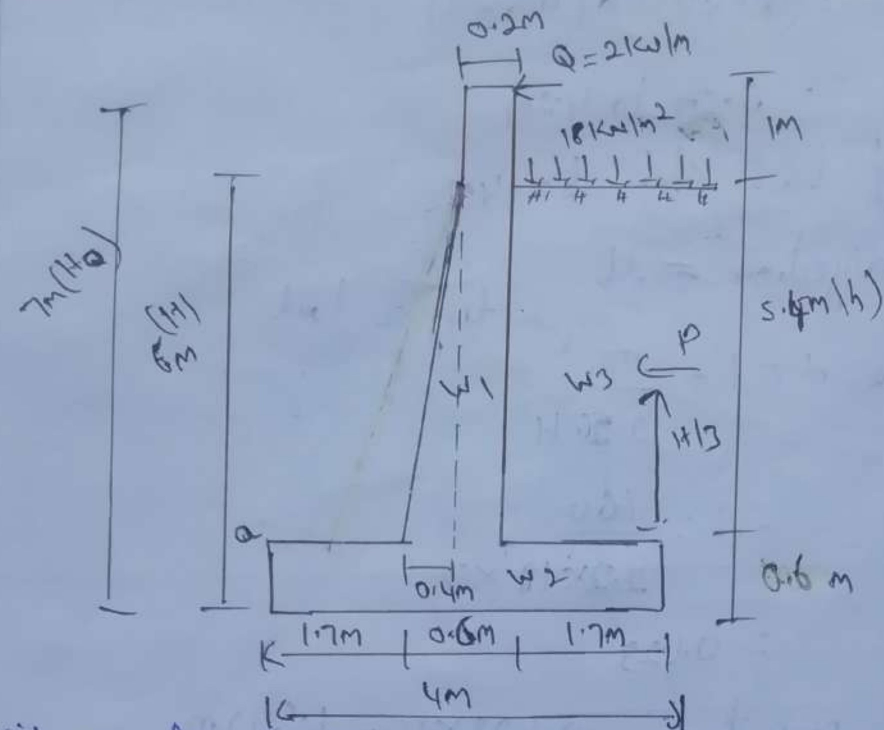
Factored moment, $M_u = 1.5 \times 257.5 = 386.25$ KNm

$M_u = 0.138 f_{ck} b d^2$

$386.25 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$\Rightarrow d = 374.09$ mm $\simeq 400$mm

Adopt cover as 50mm, $D = 400 + 50 = 450$mm



(i) Overall depth = 450mm

(ii) Base slab thickness = 600mm

∴ Provide thickness of stem as 600mm at bottom and 200mm at top

→ Main bars

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$$ where $d = 600 - 50 = 550mm$

$$386.25 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{415 \times A_{st}}{1000 \times 550 \times 20}\right]$$

$$A_{st} = 2113.628 mm^2$$

Minimum $A_{st} = \frac{0.12}{100} \times bD = \frac{0.12}{100} \times 1000 \times 600 = 720mm^2$

Provide 20mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 20^2}{2113.628} = 148.635mm$

Provide 20mm dia bars at 140mm c/c as main bars

Provided $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 20^2}{140} = 2243.995mm^2$

Minimum $A_{st} = 0.12\% \, bD = \frac{0.12}{100} \times 1000 \times 600$

$= 720 mm^2$

Provide 12 mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{720}$

$= 157.08 mm$

Provide 12mm dia bars at 150 mm c/c as distribution reinforcement

Provided $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{150} = 753.982 mm^2$

Step 3 - Stability Calculation

Load	Magnitude (kN)	Distance (m)	Moment @ a (kNm)
w_1 (weight of stem)	$0.2 \times 6.4 \times 25 = 32$	$(0.2/2) + 0.4 + 1.7 = 2.2$	70.4
	$1/2 \times 0.4 \times 5.4 \times 25 = 27$	$(2/3 \times 0.4) + 1.7 = 1.967$	53.109
w_2 (weight of base slab)	$4 \times 0.6 \times 25 = 60$	$4/2 = 2$	120
w_3 (weight of earth fill)	$1.7 \times 5.4 \times 18 = 165.24$	$(1.7/2) + 0.6 + 1.7 = 3.15$	520.506
weight due to load	$1 \times 1.7 \times 18 = 30.6$	$(1.7/2) + 0.6 + 1.7 = 3.15$	96.39

$\Sigma W = 314.84 kN$, Resisting moment, $M_R = 860.405 kNm$

→ Overturning moment due to earth pressure $= P \times \frac{H}{3}$

where P = Earth pressure $= \frac{k_a w H^2}{2} = \frac{0.333 \times 18 \times 6^2}{2}$

$= 107.892 kN$

∴ overturning moment due to earth pressure $= \frac{107.892 \times 6}{3} = 215.784 kN$

Overturning moment due to horizontal force $Q = 2 \times H_q = 2 \times 7 = 14 kNm$

Overturning moment due to traffic load $= \frac{k_a w H^2}{2}$

$$= \frac{0.333 \times 18 \times 6^2}{2} = 107.892 \text{ kNm}$$

$\therefore$ Total overturning moment, $M_o = 215.784 + 14 + 107.892$

$$= 337.676 \text{ kNm}$$

→ Net moment, $\Sigma M = M_R - M_o$

$$= 860.405 - 337.676$$

$$= 522.729 \text{ kNm}$$

→ Point of resultant force acting from base, $z = \frac{\Sigma M}{\Sigma W} = \frac{522.729}{314.84}$

$$z = 1.66 m$$

→ Eccentricity, $e = \frac{b}{2} - z = \frac{4}{2} - 1.66 = 0.34 m$

Maximum eccentricity, $e = b/6 = 4/6 = 0.667$

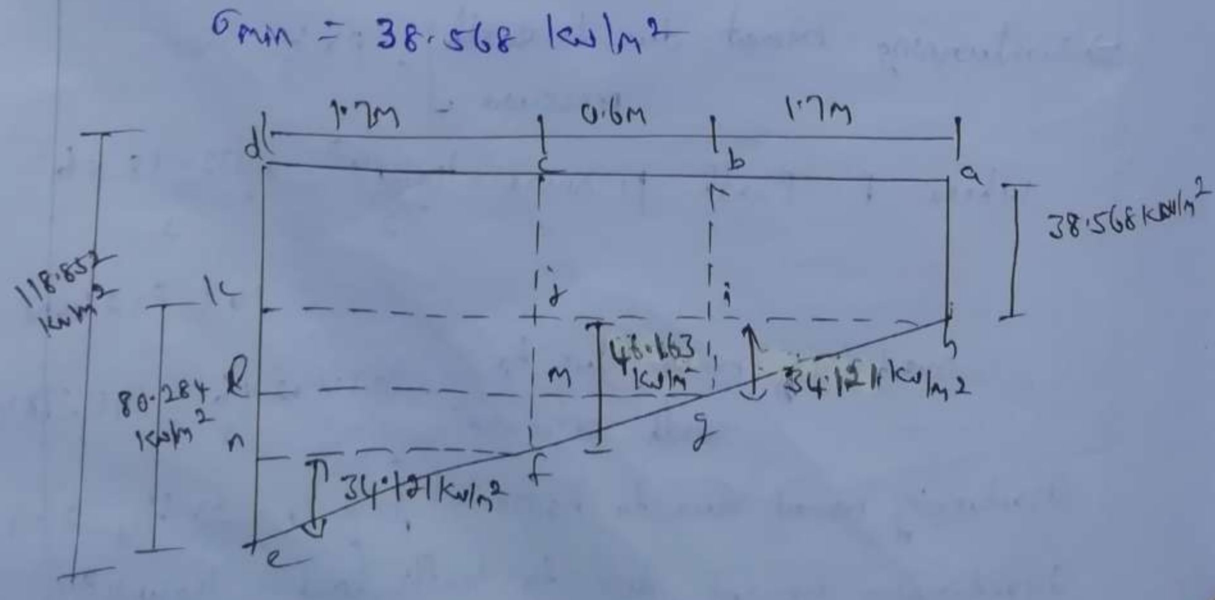
Hence safe.

→ $$\sigma_{\substack{max \\ min}} = \frac{\Sigma W}{b}\left[1 \pm \frac{6e}{b}\right]$$

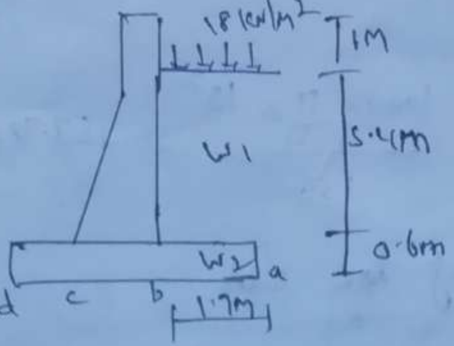
$$= \frac{314.84}{4}\left[1 \pm \frac{6 \times 0.[illegible]}{4}\right]$$

$\sigma_{max} = 118.852 \text{ kN/m}^2 < 160 \text{ kN/m}^2$. Hence safe

$\sigma_{min} = 38.568 \text{ kN/m}^2$



Step 4 - Design of heel slab



Load	Magnitude (kN)	Distance (m)	Moment (kNm) @ b
W_1 (Earth fill)	1.7×5.4×18 = 165.24	1.7/2 = 0.85	140.454
W_2 (Slab)	1.7×0.6×25 = 25.5	1.7/2 = 0.85	21.675
Traffic load	18×1×1.7 = 30.6	1.7/2 = 0.85	26.01
Pressure (cbhi)	38.568×1.7 = 65.566	1.7/2 = 0.85	55.731
Pressure (igh)	1/2×34.12×1.7 = 29	1/3(1.7) = 0.567	16.443
Net moment on structure = 188.139 ~ 72.174			115.965

→ $M_u = 1.5 \times 115.965 = 173.948$ kNm

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$ where $d = 600 - 50 = 550$ mm

$173.948 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \times \left[1 - \frac{415 \times A_{st}}{1000 \times 550 \times 20}\right]$

$A_{st} = 907.007 \text{ mm}^2$

Provide 16mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{907.007}$

$= 221.676$ mm

Provide 16mm dia bars at 220mm c/c as main bars

A_{st} provided $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{220} = 913.918 \text{ mm}^2$

→ Minimum $A_{st} = 0.12\% bD = \frac{0.12}{100} \times 1000 \times 600 = 720 \text{ mm}^2$

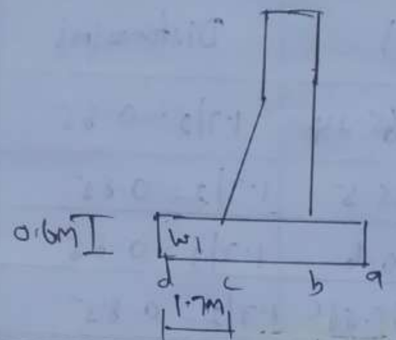
Provide 12mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{720}$

$= 157.08$ mm

Provide 12mm dia bars at 150mm c/c as distribution reinforcement

$$A_{st}\ \text{provided} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{150} = 753.982\ mm^2$$

Step 5 - Design of toe slab



Load	Magnitude	Distance (m)	Moment @ c (kNm)
w_1 (Slab)	$1.7 \times 0.6 \times 25 = 25.5$	$1.7/2 = 0.85$	21.675
Pressure (dcnf)	$84.731 \times 1.7 = 144.043$	$1.7/2 = 0.85$	122.437
Pressure (nfe)	$1/2 \times 34.121 \times 1.7 = 29$	$2/3 \times 1.7 = 1.133$	32.857
Net moment on structure = 21.675 ~ 155.294			133.619

→ Factored moment, $M_u = 1.5 \times 133.619 = 200.429$ kNm

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right] \quad \text{where } d = 600 - 50 = 550mm$$

$$200.429 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{415 A_{st}}{1000 \times 550 \times 20}\right]$$

$$A_{st} = 1050.997 mm$$

Provide 16mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1050.997} = 191.306mm$

Provide 16mm dia bars at 190mm c/c as main bars

$$A_{st}\ \text{provided} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{190} = 1058.221\ mm^2$$

Min $A_{st} = 0.12\%\ bD = 720mm^2$, Provide 12mm dia @ 150mm c/c as distribution rft

Step 6 - Design of shear key (Stability against sliding)

Frictional force, $\mu \Sigma W = 0.4 \times 314.84 = 125.936$ kN

Factor of safety against sliding $= \frac{\mu \Sigma W}{P_H}$

Where P_H = Total horizontal pressure $= P + Q$

$= 2 + 107.892$

$= 109.892$

$\therefore$ FOS $= \frac{125.936}{109.892} = 1.146 < 1.5$

Hence shear key is to be provided.

Intensity of earth pressure, $P_p = k_p / P = \frac{1}{k_a} \times P$

$\left(k_p = 1/k_a \text{ or } \frac{1+\sin\phi}{1-\sin\phi} \right)$

$= \frac{1}{0.333} \times (46.163 + 38.568)$ (Pressure at jn of key + toe slab)

$= 254.193$ kN

Assuming depth of shear key as 600mm,

Pressure force at key, $P_f = P_p \times 0.6$

$= 254.193 \times 0.6$

$= 152.516$ kN

Factor of safety against sliding $= \frac{\mu w + P_f}{P_H} = \frac{125.936 + 152.516}{109.892}$

$= 2.534 > 1.5$

Hence safe

→ Min Ast $= 0.3\% \cdot bD = \frac{0.3}{100} \times 1000 \times 600$

$= 1800 mm^2$

Provide 16mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1800} = 201.062mm$

Provide 16mm dia bars at 200mm c/c as main reinforcement

Ast provided $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{200} = 1809.557 mm^2$

→ Min Ast $= 0.12\% \cdot bD = \frac{0.12}{100} \times 1000 \times 600 = 720 mm^2$

Provide 12mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{720} = 157.08mm$

Provide 12mm dia bars at 150mm c/c as distribution reinforcement.

Ast provided $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{150} = 753.982 mm^2$

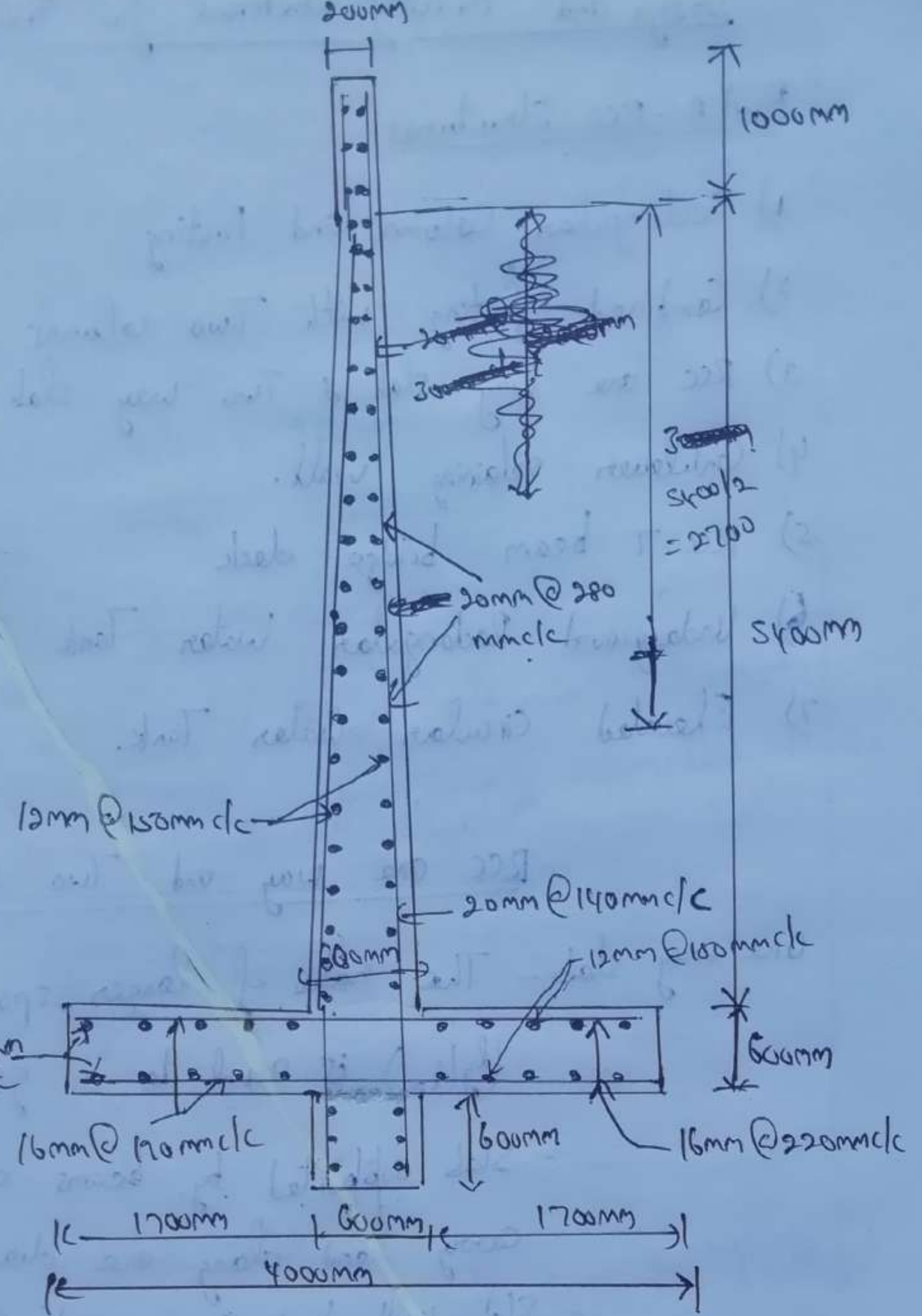
Step 7 - Stability against overturning

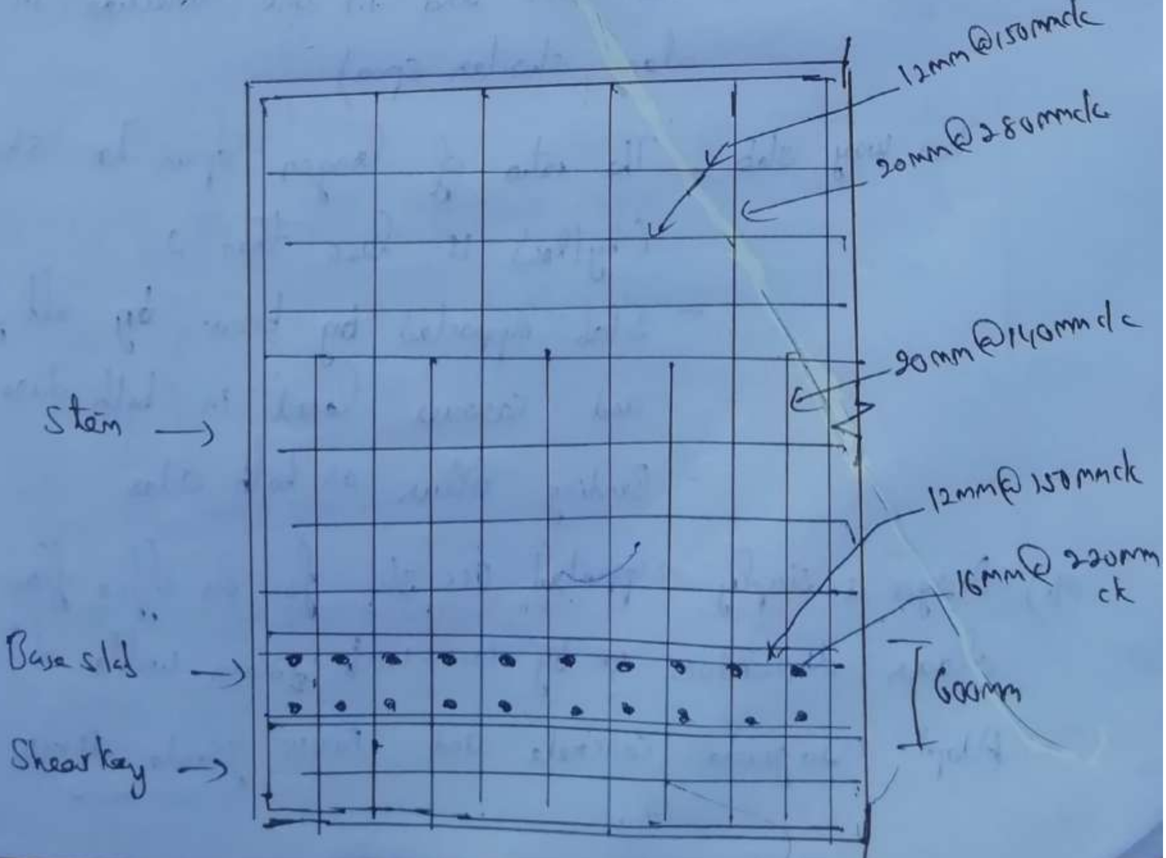
Factor of safety against overturning $= \frac{M_R}{M_O}$

$= \frac{860.405}{337.676}$

$= 2.548 > 2$

Hence safe


200mm
1000mm
5400/2
= 2700
5400mm
20mm @ 280
mmc/c
12mm @150mm c/c
20mm @140mmc/c
600mm
12mm @100mmc/c
12mm @150mm
c/c
16mm @190mmc/c
600mm
600mm
16mm @220mmc/c
1700mm
600mm
1700mm
4000mm


12mm @150mmc/c
20mm @280mmc/c
20mm @140mm c/c
Stem
12mm @150mmc/c
16mm @220mm
c/c
Base slab
600mm
Shear key

Ex No. 2 **COUNTERFORT RETAINING WALL**

DATE:

AIM

Design a counterfort retaining wall using the following details.

Height of wall above ground level = 6m

Safe bearing capacity of soil at site = 160 kN/m^2

Angle of internal friction = 33°

Density of soil =16 kN/m^3

Spacing of counterfort = 3m

Adopt M20 grade concrete and Fe415 HYSD bars.

Draw the following,

(i) Sectional elevation at midway of counterfort.

(ii) Sectional elevation between counterfort.

(iii) Sectional plan at base of counterfort

SOLUTION

<u>Step 1 – Dimensions of retaining wall</u>

Minimum depth of foundation, $d = \left(\frac{\sigma}{\rho}\right)\left(\frac{1-\sin\emptyset}{1+\sin\emptyset}\right)^2$ = 0.84 m

Provide depth of foundation, d = 1 m

Overall height of wall, H = 6+1 = 7 m

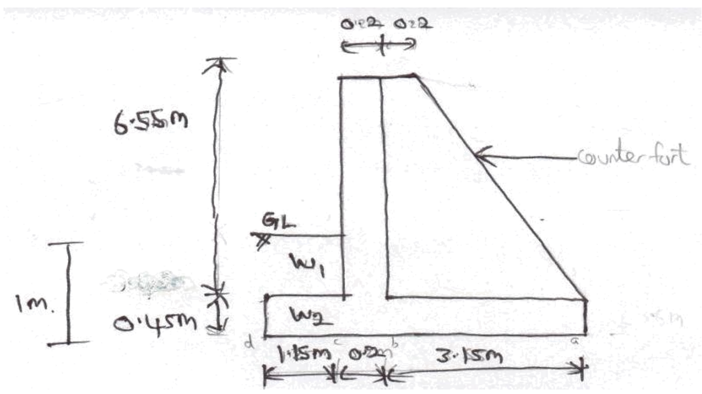
Spacing of counterfort, L = (1/3) H to (1/2) H = 2.33 to 3.5 = 3m

Thickness of base slab, t = 2LH = 2 x 3 x7 = 42 cm = 450 mm (say)

Width of base slab, D = 0.6 H to 0.7 H = 4.2 to 4.9 m = 4.5 m (say)

Height of stem, h = H – base slab thk = 7 – 0.45 = 6.55 m

Toe projection = (1/4) D= 1.13 = 1.15 m



Step 2 – Design of stem

Pressure intensity at base , $W = k_a \rho h$

$k_a = (1-\sin\phi) / (1+\sin\phi) = (1-\sin33)/(1+\sin33) = 0.29$

$W = 0.29 x 16 x 6.55 = 30.39\ kN/m^2$

Working moment, $M = (WL^2/12) = (30.39 \times 3^2)/12 = 22.79$ kNm

Working moment, $M_u = 1.5 \times 22.79 = 34.19$ kNm

$M_u = 0.138\ f_{ck}\ bd^2$

$34.19 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$,

Hence d = 111.3 mm = 150 mm

Assuming cover as 50 mm, Overall depth = 150+50 = 200 mm

Main bars

$M_u = 0.87\ f_y\ A_{st}\ d\ [1- (A_{st}\ f_y/bdf_{ck})$ where d = 200-50 = 150 mm, b=1000 mm

$A_{st} = 698.87\ mm^2$

Minimum A_{st} = 0.12% b D = 240 mm^2

Provide 12 smm dia bars, Spacing = $(1000x(\pi/4)x12^2) / 698.87 = 161.83 = 160$ mm

Provide 12 mm dia bars @ 160 mm c/c as main reinforcement

Provided $A_{st} = (1000x(\pi/4)x12^2) / 160 = 706.86\ mm^2$

Distribution bars

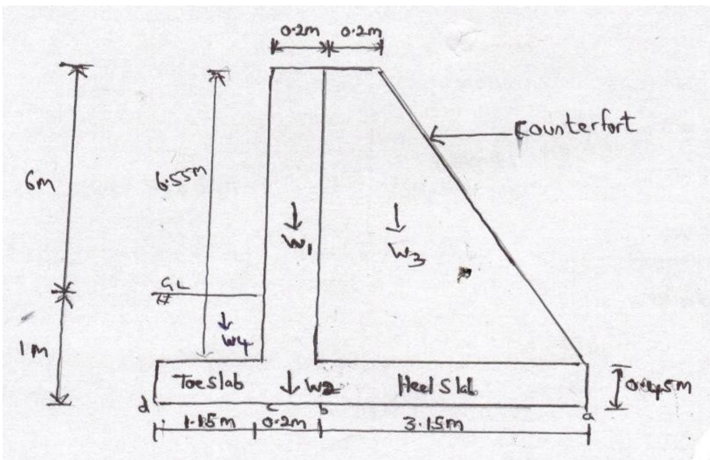
Minimum A_{st} = 0.12% b D = 240 mm^2,

Provide 8 mm dia bars, Spacing = $(1000x(\pi/4)x8^2) / 240 = 209.444 = 200$ mm

Provide 8 mm dia bars @ 200 mm c/c as main reinforcement

Provided $A_{st} = (1000x(\pi /4)x8^2) / 200 = 251.33\ mm^2$

Step 3 – Stability Check



Load	Magnitude (kN)	Distance (m)	Moment @ a
W1 (Weight of stem)	6.55 x 0.2 x 25	(0.2/2)+3.15	106.44
W2 (Weight of base slab)	4.5 x 0.45 x 25	4.5/2	113.92
W3 (Weight of earth fill)	3.15 x 6.55 x 16	3.15/2	519.94
W4 (Weight of earth fill)	(1 – 0.45) x 1.15 x 16	(1.15/2) + 0.2 + 3.15	34.54
Moment due to earth pressure =$(k_a\rho\, h^3)/6$ = $(0.29x16x6.55^3)/6$			217.32

ΣW = 423.62 kN, ΣM = 992.16 kNm

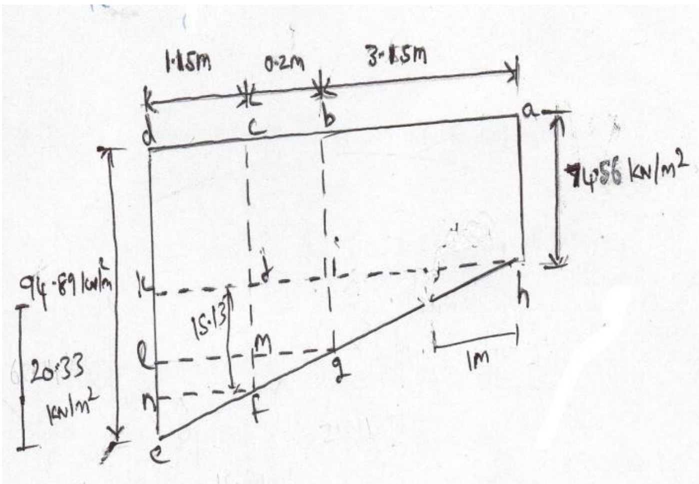
Point of resultant force acting from base, z = Σ M/Σ W = 2.34 m

Eccentricity, e = z- (b/2) = 2.34 – (4.5/2) = 0.09

Maximum eccentricity = b/6 = 5/6 = 0.83 Hence safe.

$\sigma_{max,\ min}$ = ΣW/b [1 ± 6e/b] = 423.62/4.5 [1 ± (6x0.09)/4.5]

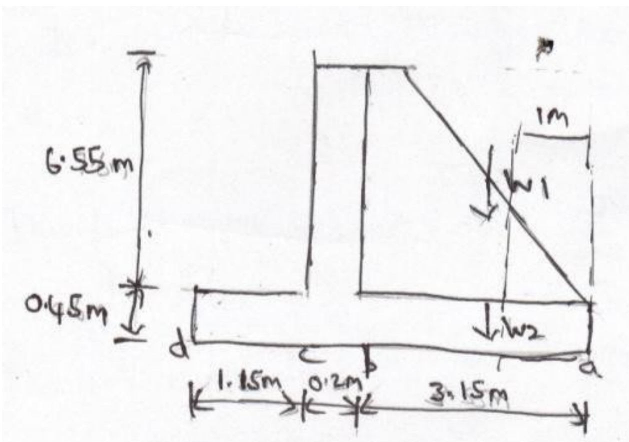
σ_{max} = 94.89 kN/m² < 160 kN/m² σ_{min} = 74.56 kN/m²



Step 4 – Design of heel slab

Net moment on structure

Consider 1 m strip from ‘a’ on heel slab



Load	Pressure (kN/m^2)
W1 (Weight to earth fill)	6.55 x 16 = 104.8
W2 (Weight of base slab)	0.45 x 25 = 11.25
Upward pressure (abhi)	74.56
Net pressure on structure = 116.05 ~ 74.56 = 41.49 kN/m^2	

Working moment, M = ($WL^2/12$)= (41.49 x 3^2)/12 = 10.87 kNm

M_u = 1.5 x 10.87 = 16.31 kNm

M_u = 0. 87 f_y A_{st} d [1- (A_{st} f_y/bdf_{ck}) where d = 450-50 = 400 mm, b=1000 mm

A_{st} = 113.6 mm^2

Minimum A_{st} = 0.12% b D = 540 mm^2,

Provide 10 mm dia bars, Spacing = (1000 x (π/4)x10^2) / 540 = 145.444 = 140 mm

Provided A_{st} = (1000x(π/4)x10^2) / 140 = 561 mm^2

Provide 10 mm dia @ 140 mm c/c bars as both main and distribution reinforcement

Step 5 – Design of toe slab

Net moment on structure

Load	Magnitude (kN)	Distance (m)	Moment @ c
W1 (Weight to earth fill)	(1 – 0.45) x 1.15 x 16	1.15/2	5.82
W2 (Weight of base slab)	1 .15x 0.45 x 25	1.15/2	7.44
Upward pressure (dcnf)	86.69 x 1.15	1.15/2	57.32
Upward pressure (nfe)	(1/2) x 5.2 x 1.15	(2/3) x 1.15	2.29
Net moment on structure = 13.26 ~ 59.61			46.35

M_u = 1.5 x 46.35 = 69.53 kNm

M_u = 0. 87 f_y A_{st} d [1- (A_{st} f_y/bdf_{ck}) where d = 450-50 = 400 mm, b=1000 mm

A_{st} = 494.11 mm^2

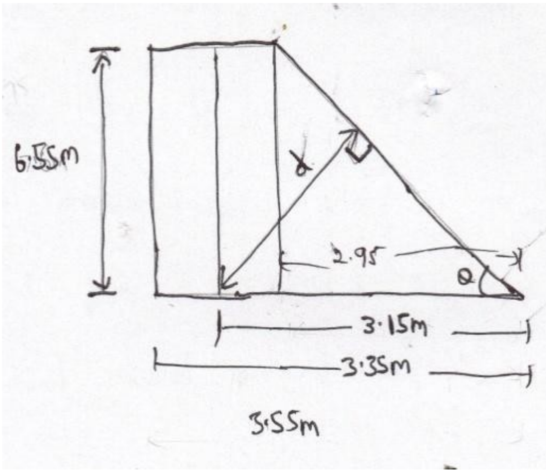
Minimum A_{st} = 0.12% b D = 540 mm^2,

Provide 10 mm dia bars, Spacing = (1000x(π/4)x10^2) / 540 = 145.444 = 140 mm

Provided A_{st} = (1000x(π/4)x10^2) / 140 = 561 mm^2

Provide 10 mm dia @ 140 mm c/c bars as both main and distribution reinforcement

Step 6 – Design of counterfort



Moment, $M=[(k_a \rho H^3)/6] \times L=[(0.29 \times 16 \times 7^3)/6] \times 3 = 795.76$ kNm

Factored moment = 1.5 x 795.76 = 1193.64 kNm

$M_u = 0.87 f_y A_{st} d [1- (A_{st} f_y/bdf_{ck})]$

tan θ = 6.55/2.95, θ = 65.75

sin 65.75 = d/3.15, d = 2.87 m

Thickness of counterfort, b = 0.2 + 0.2 = 0.4 m

A_{st} = 1176.96 mm^2

Minimum reinforcement is given $A_s/bd = 0.85 f_y$,

$A_s/(400 \times 2870) = 0.85 \times 415$, $A_s = 2351.33$ mm^2

Provide 5 no's of 28 mm dia bars (A_{st} = 3078.76 mm^2)

Step 7 – Connection between counterfort and stem

Pressure intensity @ base = 30.39 kN/m^2

Consider the bottom 1 m height of stem,

Lateral pressure transferred = 30.39 x (3.15-0.2) x1 = 89.65 kN

Factored force = 1.5 x 89.65 = 134.48 kNm

Reinforcement required per metre length = $F/0.87f_y = (134.48 \times 10^3)/(0.87 \times 415)$

= 372.47 mm^2

Minimum A_{st} = 0.12% b D = 540 mm^2,

Provide 10 mm dia bars, Spacing = $(1000 \times (\pi/4) \times 10^2) / 540 = 145.44 = 140$ mm

Provided $A_{st} = (1000 \times (\pi/4) \times 10^2) / 140 = 561$ mm^2

Provide 10 mm dia bars @ 140 mm c/c for connection between counterfort & stem

<u>Step 8 – Connection between counterfort and heel slab</u>

Pressure intensity @ base = 41.49 kN/m^2

Consider the bottom 1 m height of stem,

Lateral pressure transferred = 41.49 x (3.15-0.2) x1 = 122.4 kN

Factored force = 1.5 x 122.4 = 183.6 kNm

Reinforcement required per metre length = $F/0.87f_y$ = $(183.6 \times 10^3)/(0.87 \times 415)$

= 508.52 mm^2

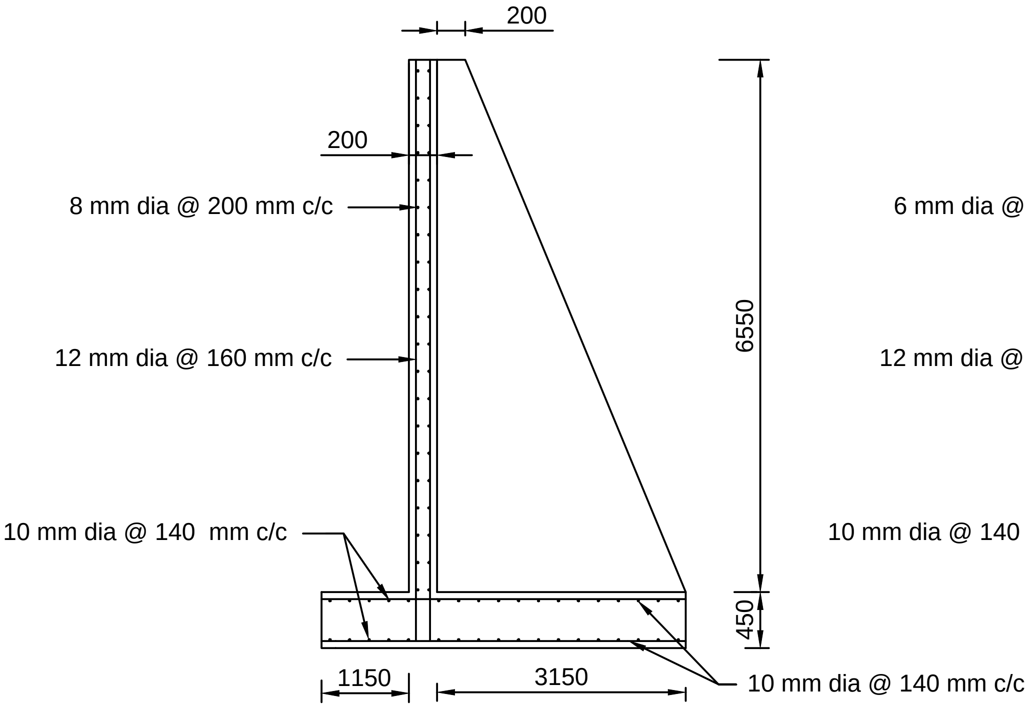
Minimum A_{st} = 0.12% b D = 540 mm^2,

Provide 10 mm dia bars, Spacing = $(1000 \times (\pi/4) \times 10^2) / 540$ = 145.44 = 140 mm

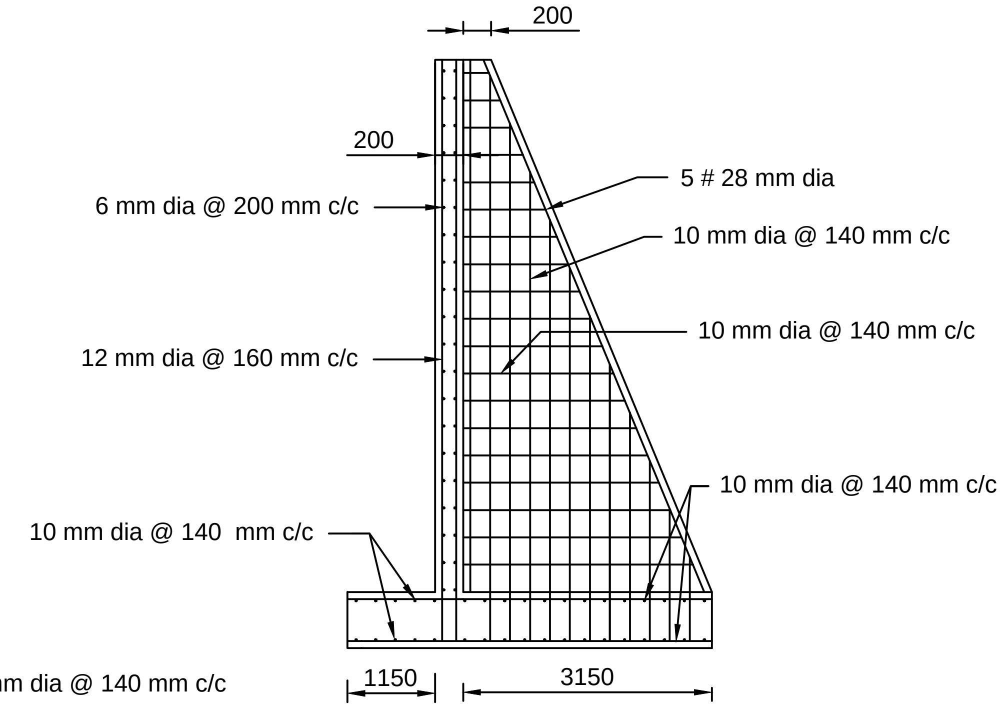
Provided A_{st} = $(1000 \times (\pi/4) \times 10^2) / 140$ = 561 mm^2

Provide 10 mm dia bars @ 140 mm c/c for connection between counterfort & heel slab

COUNTERFORT RETAINING WALL



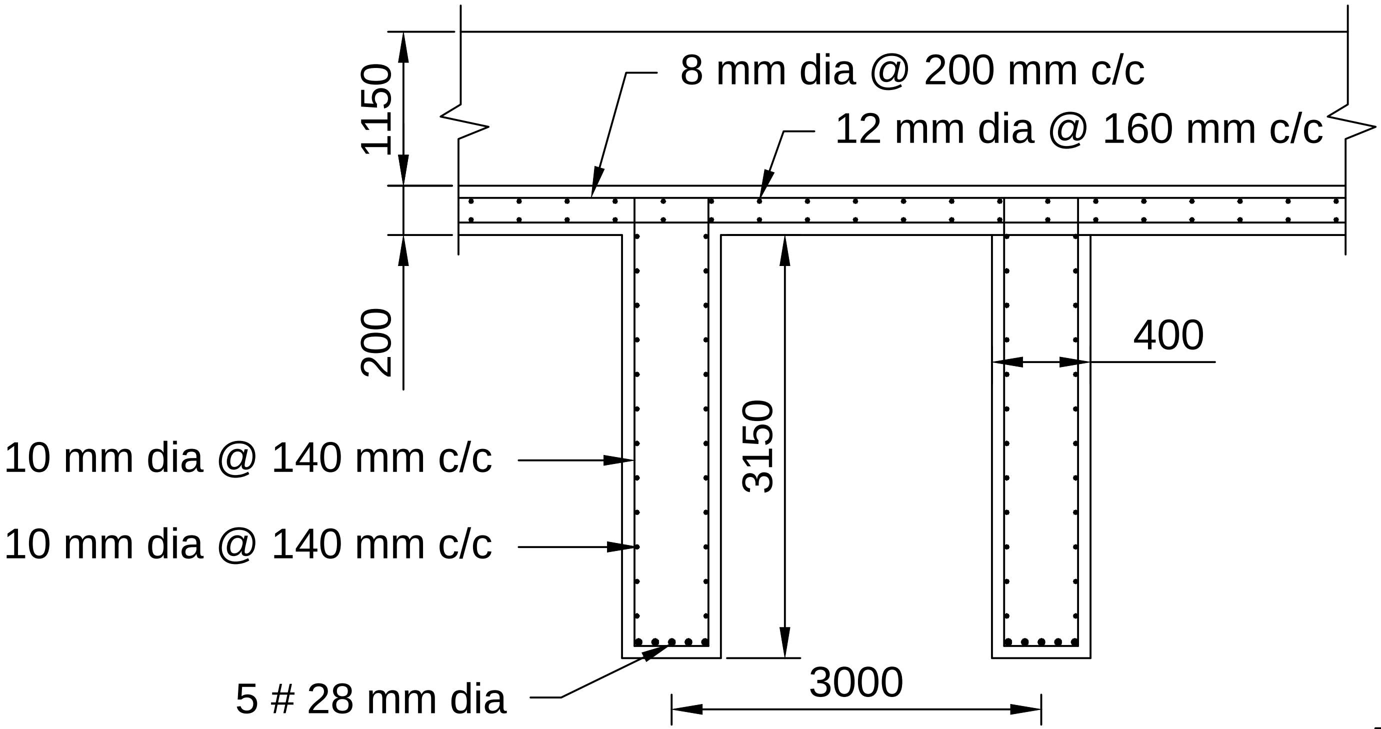
SECTIONAL ELEVATION AT MIDWAY OF COUNERFORT



SECTIONAL ELEVATION AT MIDWAY OF COUNERFORT

All dimensions are in mm
M20 Grade Concrete
Fe 415 Grade steel

COUNTERFORT RETAINING WALL



SECTIONAL PLAN AT BASE OF COUNTERFORT

All dimensions are in mm
M20 Grade Concrete
Fe 415 Grade steel

Unit II – Flat Slabs and Bridges

Design of Flat slabs with and without drops by Direct Design Method of IS Code – Design and Drawing – IRC Specifications and Loading – RC solid slab bridge – Steel Foot over Bridge – Design and Drawing

Design of Flat slab

Definition

A flat slab is a reinforced concrete slab supported directly over columns without beam. Generally used when headroom is limited and hence used in large industrial structures, warehouses, high rise buildings and hotels

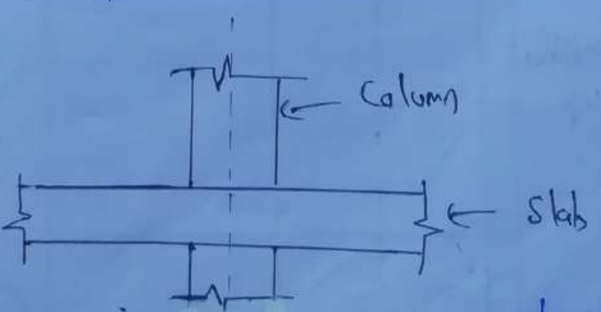
Advantages

- → Reduces overall height of structure
- → Flat slab are capable of carrying concentrated loads
- → Requires less formwork.
- → Better appearance, quality control, fire resistant.

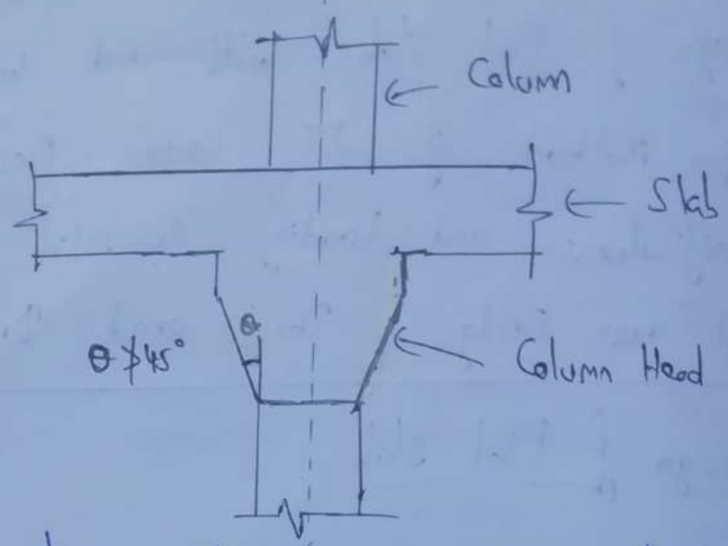
Types of Flat Slab

The different types of flat slab are

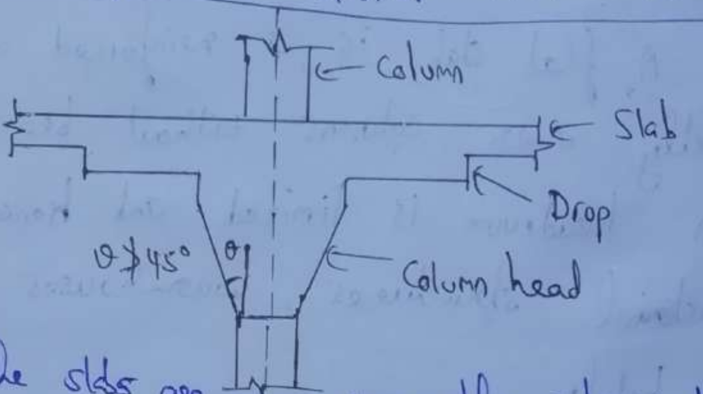
→ Slab without drops and column without column head



→ Slab without drop and column with column head
The column is widened at its head to reduce punching shear in slab. The widened portion is called column head.



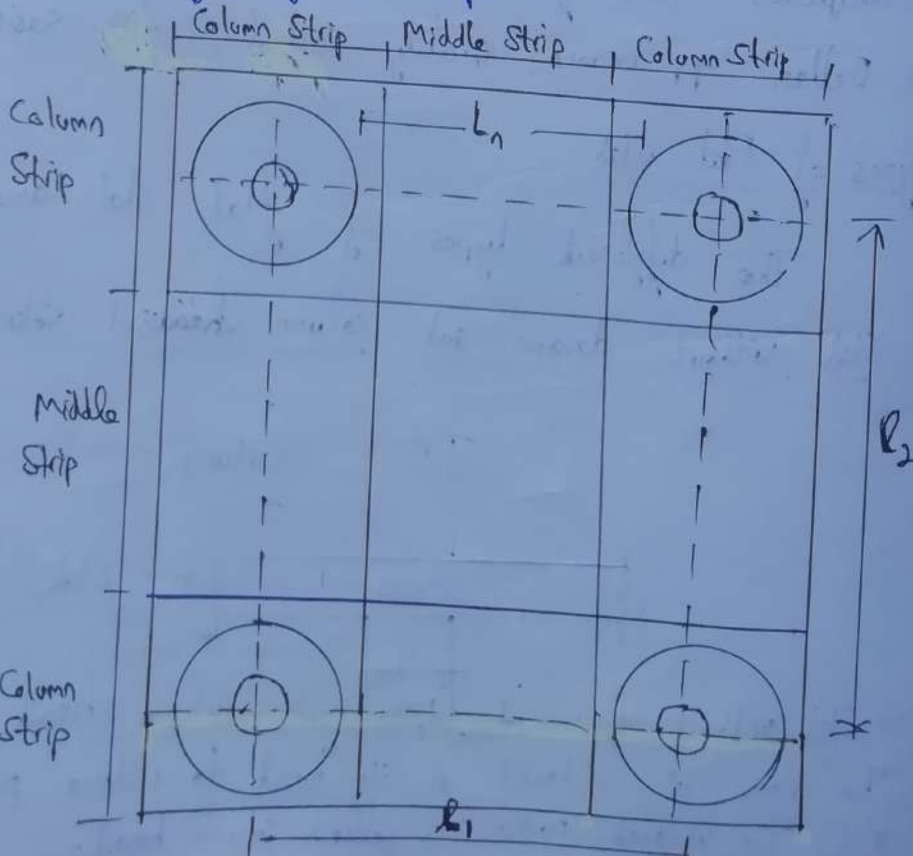
→ Slab with drop and column with column head



Moments in the slabs are more near the column. Hence the slab is thickened near the columns by providing drops.

Panel Divisions

Panel is that part of the slab bounded on each of its sides by centre line of columns or centre line of adjacent spans.



Column strip is a design strip having a width of $0.25l_2$ but not greater than $0.25l_1$ where 'l_1' is span in direction moments are being determined, measured c/c of supports and 'l_2' is span in transverse direction to l_1, measured c/c of supports (IS 456 - Pg 53)

Middle strip means a design strip bounded on each of its opposite sides by column strip. (IS 456 Pg 53)

Proportioning of flat slabs

→ Drops – Length of drop = $\frac{1}{3}$ × Panel length in that direction

– Width of drop = $\frac{1}{2}$ × Panel length

→ Column head – $\frac{1}{2}$ × Column strip

→ Thickness of slab – If drops are provided

(span/eff depth ratio) = 40 (mild steel)

= 32 (Fe415 (or) Fe500)

– If drops are not provided

= 40×0.9 = 36 (mild steel)

= 32×0.9 = 28.8

Thickness should not be less than 125 mm (IS 456 - Pg 53 31.2.1)

Determination of BM & SF

1) Direct design method

2) Equivalent frame method

1) Direct design method

Total moment (sum of +ve and -ve), $M_o = \frac{W L_n}{8}$ (IS Pg 55 - 31.4.2.2)

where, M_o → Total moment

W → Design load on area $L_2 \times L_n$

L_n → Clear span extending face to face of columns ≮ $0.65L_1$

L_1 → Length of span in direction of M_o

L_2 → Length of span transverse to L_2

Distribution of moments

In interior span (IS 456 - Pg 55 - 31.4.3.2)

Negative design moment - $0.65 M_0$

Positive " " - $0.35 M_0$

Along column strip and middle strip (IS 456 31.5.5.1, 31.5.5.3 31.5.5.4(a))

Moment	Column strip	Middle strip
-ve	75% of total -ve moment	Moment not resisted by column strip
+ve	60% of total +ve moment	"

Check for Shear

$$\tau_v = \frac{V}{b_o d}$$ (IS456 - Pg 57 - 31.6.2.1)

where V = Shear force to be resisted

b_o = Periphery of critical section

d = Effective depth

Permissible shear stress $= k_s \tau_c$ (IS456 - Pg 58 - 31.6.3.1)

where $k_s = 0.5 + \beta_c \not> 1$ where β_c = Ratio of short to long side of column

$\tau_c = 0.25 \sqrt{f_{ck}}$

Reinforcement

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$$ (IS456 - Pg 96 G-1.1(b))

where f_y = strength of reinforcement

d = Effective depth

A_{st} = Area of tension reinforcement

b = width

f_{ck} = Compressive strength of Concrete

Pb) Design an interior panel of a flat slab of size 5m×5m without providing drop and column head. Size of column is 500×500mm and live load on the panel is 4 kN/m². Take floor finishing load as 1 kN/m². Use M20 concrete and Fe415 steel

Solution

Step 1 - Thickness of slab

Drops are not provided, thickness is given by

$$\frac{\text{Span}}{\text{Effective depth}} = 32 \times 0.9 = 28.8$$

$$\frac{5000}{d} = 28.8$$

$\Rightarrow d = 173.61\text{mm} \simeq 175\text{mm} \nless 125\text{mm}$ (IS 456 Pg 53 - 31.2.1)

Overall depth = d + cover = 175 + 25 = 200mm

Step 2 – Panel dimensions

Length of panel = width of panel = 5m

$l_1 = l_2 = 5m$

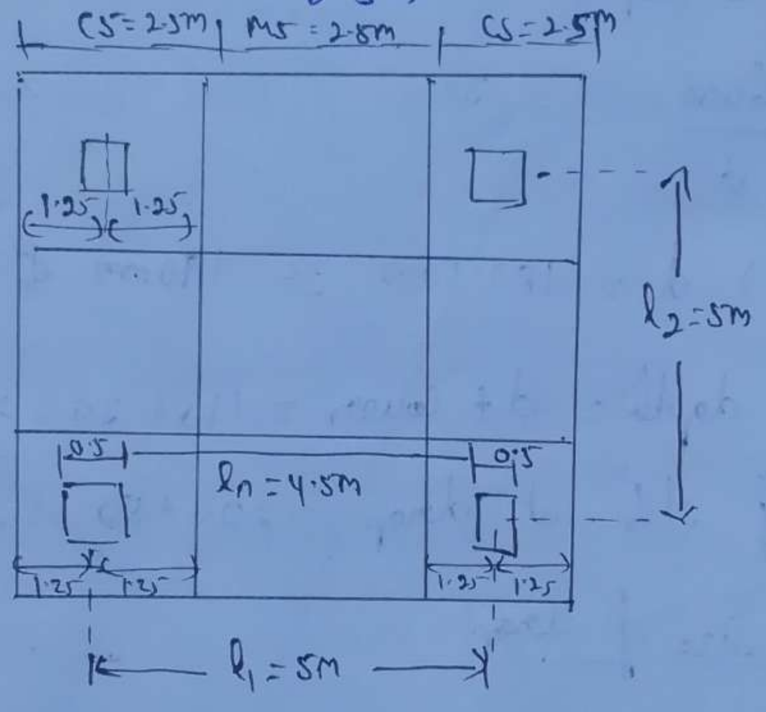
Width of column strip = $0.25 l_2 \ngtr 0.25 l_1$ (IS456 - Pg 53)

$= 0.25 \times 5 = 1.25m$ on each side of column centre line

Width of middle strip = $l_1 - 1.25 - 1.25$

$= 5 - 1.25 - 1.25$

$= 2.5m$



Clear span, $l_n = 5 - \frac{0.5}{2} - \frac{0.5}{2} = 4.5m$

Step 3 - Loads

Self weight of slab = 0.20 × 25 kN/m³ = 5 kN/m²

Live load = 4 kN/m²

Finishing load = 1 kN/m²

Total working load = 10 kN/m²

Total factored load = 1.5 × 10 = 15 kN/m²

Step 4 Moments

Total moment, $M_o = \frac{W l_n}{8}$ (IS Pg 55 - 31.4.2.2)

where W = Design load on area $l_2 l_n$

l_2 = Span in transverse direction to l_1 = 5m

l_n = clear span = $l_1 - \frac{\text{col dia}}{2} - \frac{\text{col dia}}{2}$

$= 5 - \frac{0.5}{2} - \frac{0.5}{2}$

$= 4.5m$

$\therefore W = 15 \times 5 \times 4.5 = 337.5\ kN$

$M_o = \frac{337.5 \times 4.5}{8}$

In interior span $M_o = 189.84\ kNm$

The total design moment shall be distributed in foll. proportions, (IS 456 Pg 55 - 31.4.3.2)

Negative design moment = 0.65×189.84 = 123.40 kNm

Positive design moment = 0.35×189.84 = 66.44 kNm

The BM is distributed across column strip (IS 456 - Pg 57 - 31.5.5.1 & 31.5.5.3) and middle strip (IS 465 - Pg 57 - 31.5.5.4(a)) as below. 75, 60

Moment	Column Strip (kNm)	Middle Strip (kNm)
-ve	$0.75 \times 123.40 = 92.55$	$0.25 \times 123.40 = 30.85$
+ve	$0.6 \times 66.44 = 39.86$	$0.4 \times 66.40 = 26.58$

Check for limiting moment

$M_{ulim} = 0.138 f_{ck} bd^2$ (SP 16 - Pg 10, Table c for Fe415)

where b = Width of Column strip = 2500 mm

d = 175mm

f_{ck} = 20 N/mm2

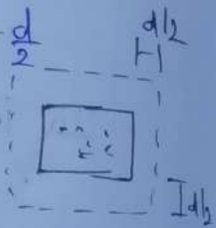
$M_{ulim} = 0.138 \times 20 \times 2500 \times 175^2 = 2.113 \times 10^8\ Nmm = 211.31\ kNm$

All the moments are within limit, hence safe.

Step 5 - Check for shear

(IS456 - Pg 57 - 31.6.1)

The critical section for shear is at a distance 'd/2' from the column face. Hence periphery of critical section around a column is square of size = Column size $+ \frac{d}{2} + \frac{d}{2}$



$= 500 + \frac{175}{2} + \frac{175}{2}$

$= 675 mm$

Shear force to be resisted by critical section = Total load on panel − Load on square area

$= (15 \times 5 \times 5) - (15 \times 0.675 \times 0.675)$

$V = 368.17 \text{ kN}$

Shear force/m of perimeter $= (368.17) / (4 \times 0.675)$

$= 136.36 \text{ kN}$

Nominal shear stress, $\tau_v = \frac{V}{b_o d}$ (IS456 - Pg 57 - 31.6.2.1)

$= \frac{368.17 \times 10^3}{4 \times 675 \times 175}$

$\tau_v = 0.78 N/mm^2$

Permissible shear stress $= k_s \tau_c$ (IS456 - Pg 58 - 31.6.3.1)

$k_s = 0.5 + \beta_c$ where $\beta_c = \frac{L_1}{L_2} = \frac{5}{5} = 1$

$= 0.5 + 1$

$= 1.5 > 1$

$\therefore k_s = 1.0$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25 \times \sqrt{20} = 1.12\ N/mm^2$

$k_s \tau_c = 1.0 \times 1.12 = 1.12\ N/mm^2$

$\tau_v < k_s \tau_c$

Hence safe

Step 6 - Reinforcements

(i) Column strip

For -ve moment $M_u = 92.55\ kNm$

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$ IS456 - Pg 96 - G 1.1(b)

$92.55 \times 10^6 = 0.87 \times 415 \times A_{st} \times 175 \left[1 - \frac{A_{st} \times 415}{2500 \times 175 \times 20}\right]$ (usually 1000)

$A_{st} = 1583.74\ mm^2$

Provide 12mm dia bars, $\text{spacing} = \frac{2500 \times a_{st}}{A_{st}}$

$= \frac{2500 \times \frac{\pi}{4} \times 12^2}{1583.74} = 178.53\ mm$

∴ Provide 12mm dia bars at 175 mm c/c

(usually 1000)

$A_{st}\ \text{provided} = \frac{2500 \times a_{st}}{\text{spacing}} = \frac{2500 \times \frac{\pi}{4} \times 12^2}{175} = 1615.68\ mm$

For +ve moment, $M_u = 39.86\ kNm$

$39.86 \times 10^6 = 0.87 \times 415 \times A_{st} \times 175 \times \left[1 - \frac{A_{st} \times 415}{2500 \times 175 \times 20}\right]$

$A_{st} = 650.96\ mm^2$

Provide 10mm dia bars, $\text{spacing} = \frac{2500 \times \frac{\pi}{4} \times 10^2}{650.96} = 301.63\ mm$

Spacing should be less than 3d (3×175) or 300mm (IS Pg 46 26.3.3(b) ...

Provide 10mm bars at 300 mm c/c

$$A_{st}\ \text{provided} = \frac{2500 \times \frac{\pi}{4} \times 10^2}{300} = 654\,mm^2$$

(ii) Middle strip

For -ve moment, M_u = 30.85 kNm & For +ve moment

M_u = 26.58 kNm (Taking maximum)

$$30.85 \times 10^6 = 0.87 \times 415 \times A_{st} \times 175 \times \left[1 - \frac{A_{st} \times 415}{2500 \times 175 \times 20}\right]$$

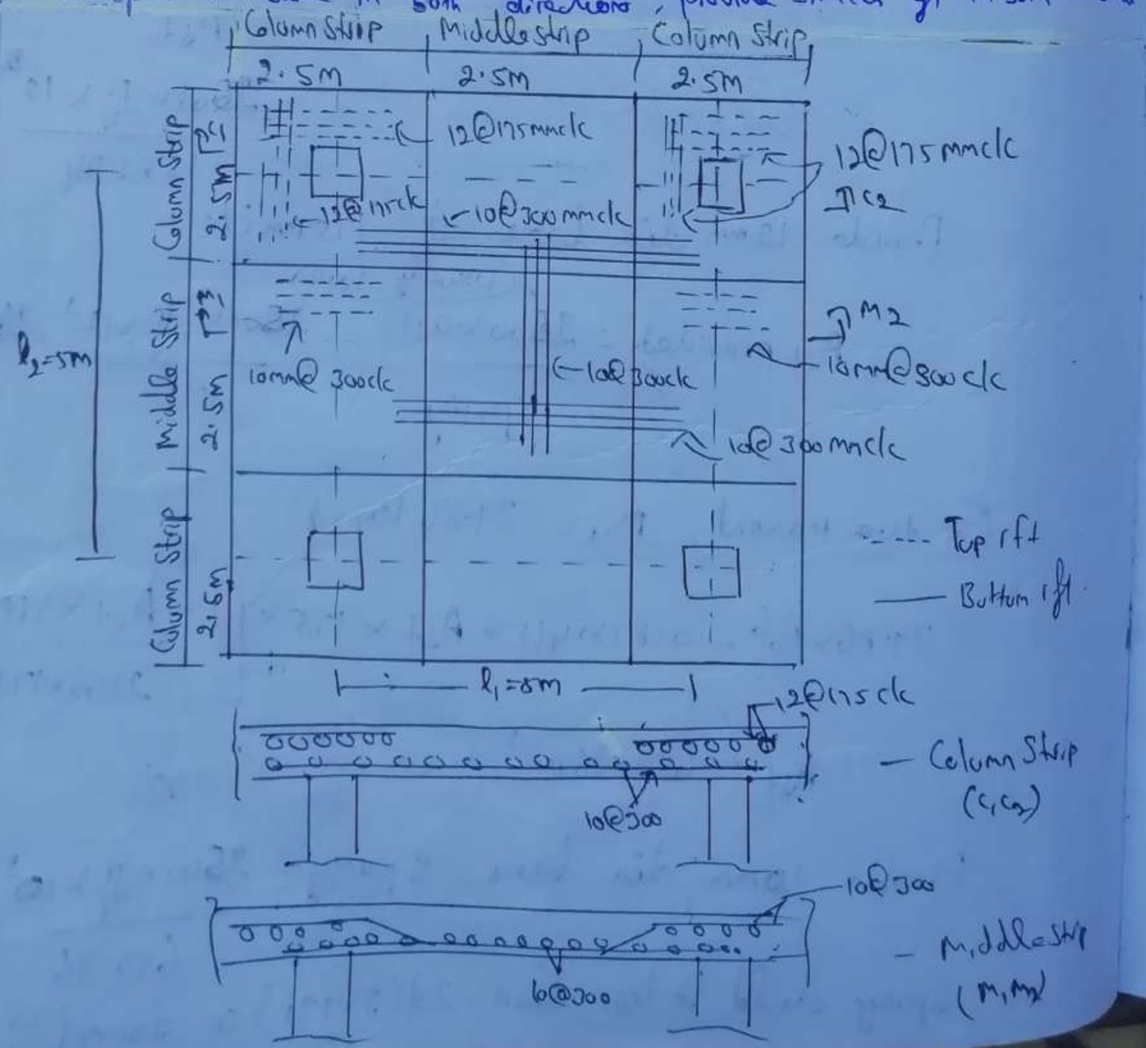
$$A_{st} = 500.12\,mm^2$$

Provide 10mm dia bars, $\text{Spacing} = \frac{2500 \times \frac{\pi}{4} \times 10^2}{500.12} = 392.6\,mm$

Provide 10mm dia bars at 300 mm c/c in

$$\text{Provided } A_{st} = \frac{2500 \times \frac{\pi}{4} \times 10^2}{300} = 654\,mm^2$$

Since span is same in both directions, provide similar rft in both direction



Pb) Design an interior panel of a flat slab with panel size 6x6m supported by columns of size 500mm x 500mm. Provide suitable drop. Take live load as 4 kN/m². Use M20 grade concrete and Fe 415 steel.

Solution

Step 1 - Thickness of slab

Drops are provided, thickness is given by,

$$\frac{\text{Span}}{\text{Effective depth}} = 32$$

$$\frac{6000}{d} = 32$$

$\Rightarrow d = 187.5\text{mm} \simeq 190\text{mm}$ (IS 456 - Pg 53 - 31.2.1)
$< 125\text{mm}$

overall depth = 190+30 = 220mm

Depth of slab at drop = 220+50 = 270mm.

Step 2 - Panel dimensions

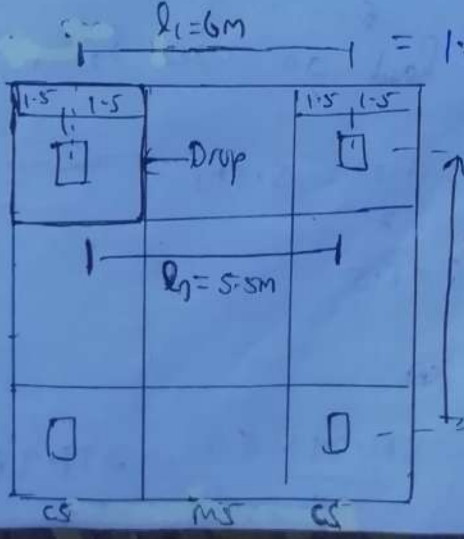
Length of panel = Width of panel = 6m

$l_1 = l_2 = 6\text{m}$

Width of column strip = $0.25 l_2 \not> 0.25 l_1$

$= 0.25 \times 6$

$= 1.5\text{m}$ on each side of column centre line



Width of middle strip = 6 − 1.5 − 1.5

= 3 m

Clear span, $l_n = 6 - \frac{0.5}{2} - \frac{0.5}{2}$

= 5.5 m

Step 3 - Size of drop

Length of drop = $\frac{1}{3}$ × Panel lath

$= \frac{1}{3} \times 6$

= 2 m

However keep length of drop equal to column strip (3 m)

∴ Provide drop of size 3 m × 3 m

Step 4 - Loads

Self weight of slab = 0.27 × 25 = 6.75 kN/m²

Live load = 4 kN/m²

Finishing load = 1 kN/m²

Total working load = 11.75 kN/m²

Total factored load = 1.5 × 11.75 = 17.63 kN/m²

Step 5 - Moments

Total moment, $M_o = \frac{W l_n}{8}$

where W = Design load on area $l_2 l_n$

$l_n = 6 - \frac{0.5}{2} - \frac{0.5}{2} = 5.5$ m

W = 17.63 × 6 × 5.5

= 581.79 kN

Total moment, $M_o = \frac{581.79 \times 5.5}{8} = 399.98 \simeq 400$ kN

In interior span, the total design moment shall be distributed in foll. proportions (IS 456 Pg 55 - 31.4.3.2)

Negative design moment $= 0.65 \times 400 = 260\ KNm$

Positive design moment $= 0.35 \times 400 = 140\ KNm$

The BM is distributed across column strip (IS 456 Pg 57 - 31.5.5.1 & 31.5.5.3) and middle strip (IS 456 - Pg 57 - 31.5.5.4(a)) as below,

Moment	Column Strip (KNm)	Middle Strip (KNm)
-ve	$0.75 \times 260 = 195$	$0.25 \times 260 = 65$
+ve	$0.6 \times 140 = 84$	$0.4 \times 140 = 56$

<u>Check for limiting moment</u>

$M_{ulim} = 0.138 f_{ck} b d^2$ (SP16 - Pg 10, Table C for Fe415)

where b = width of column strip = 3000 mm

$d = 270 - 30 = 240\ mm$

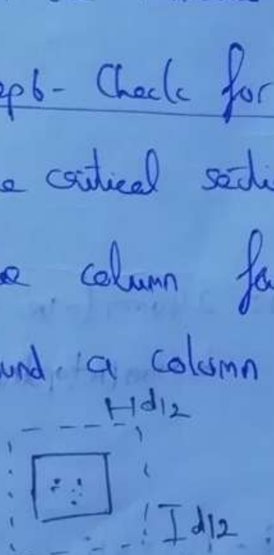
$M_{ulim} = 0.138 \times 20 \times 3000 \times 240^2 = 4.769 \times 10^8\ Nmm$

$= 476.928\ KNm$

All the moments are within limit, hence safe.

<u>Step 6 - Check for shear</u>

The critical section for shear is at a distance 'd/2' from the column face. Hence pheriphery of critical section around a column is square of size = Column size $+ \frac{d}{2} + \frac{d}{2}$



$= 500 + \frac{240}{2} + \frac{240}{2}$

$= 740\ mm$

Shear force to be resisted by critical section = Total load on panel − load on square area

$= (17.63 \times 6 \times 6) - (17.63 \times 0.74 \times 0.74)$

$= 625.026$ kN

Nominal shear stress, $\tau_v = \frac{V}{b_o d}$ (IS456 - Pg 57 - 31.6.2.1)

$= \frac{625.026 \times 10^3}{4 \times 740 \times 240}$

$\tau_v = 0.88 \, N/mm^2$

Permissible shear stress $= k_s \tau_c$ (IS 456 - Pg 58 - 31.6.3.1)

$k_s = 0.5 + \beta_c$ where $\beta_c = \frac{L_1}{L_2} = \frac{6}{6} = 1$

$= 0.5 + 1$

$= 1.5 \not> 1$

$k_s = 1$

$\tau_c = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{20} = 1.12 \, N/mm^2$

$k_s \tau_c = 1 \times 1.12 = 1.12 \, N/mm^2$

$\tau_v < k_s \tau_c$

Hence safe.

Step 7 - Reinforcements

(i) Column strip

For -ve moment, $M_u = 195$ kNm , $d = 240$ mm

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$

$195 \times 10^6 = 0.87 \times 415 \times A_{st} \times 240 \times \left[1 - \frac{A_{st} \times 415}{3000 \times 240 \times 20}\right]$

$A_{st} = 2419.023 \, mm^2$

Provide 12mm dia bars, spacing $= \frac{3000 \times \frac{\pi}{4} \times 12^2}{2419.023} = 140.26mm$

Provide 12mm dia @ 140 mm c/c

Provided $A_{st} = \frac{3000 \times \frac{\pi}{4} \times 12^2}{140} = 2423.514 \ mm^2$

For +ve moment, $M_u = 84$ kNm, $d = 190mm$

$$84 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \times \left[1 - \frac{A_{st} \times 415}{3000 \times 190 \times 20}\right]$$

$A_{st} = 1284.569 mm^2$

Provide 10mm dia bars, spacing $= \frac{3000 \times \frac{\pi}{4} \times 10^2}{1284.569} = 183.42$ mm

Provide 10mm dia @ 180 mm c/c.

Provided $A_{st} = \frac{3000 \times \frac{\pi}{4} \times 10^2}{180} = 1308.997 \ mm^2$

(ii) Middle strip

For -ve moment, $M_u = 65$ kNm, $d = 190mm$ (there is no drop)

$$65 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \times \left[1 - \frac{A_{st} \times 415}{3000 \times 190 \times 20}\right]$$

$A_{st} = 982.682 mm^2$

Provide 10mm dia bars, spacing $= \frac{3000 \times \frac{\pi}{4} \times 10^2}{982.682} = 239.77mm$

Provide 10mm dia bars at 230mm c/c

Provided $A_{st} = \frac{3000 \times \frac{\pi}{4} \times 10^2}{230} = 1024.432 mm^2$

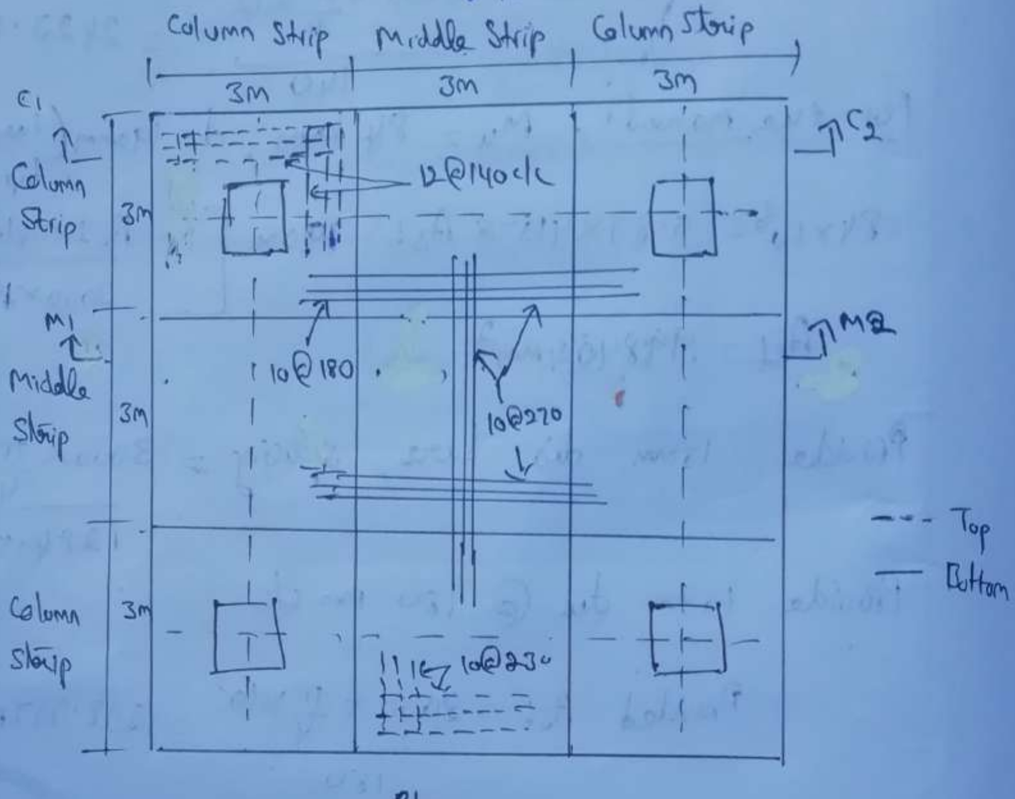
For +ve moment, $M_u = 56$ kNm, $d = 190mm$ (there is no drop)

$$56 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \times \left[1 - \frac{A_{st} \times 415}{3000 \times 190 \times 20}\right]$$

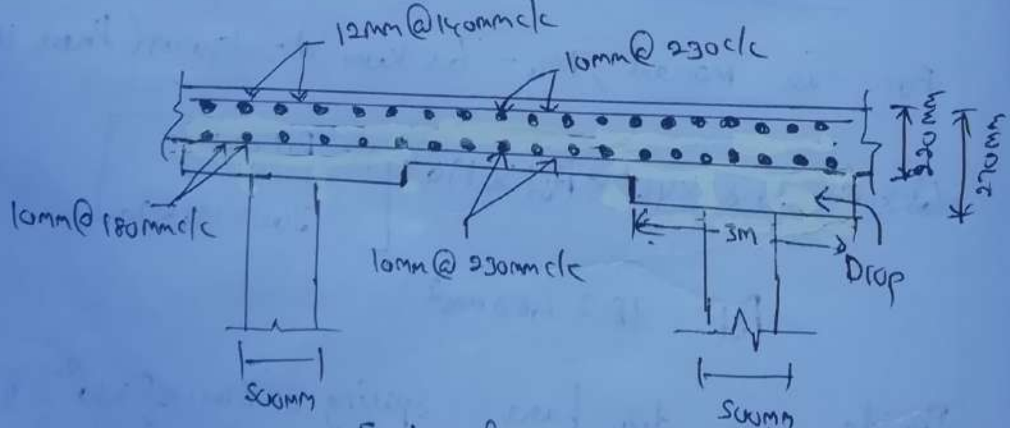
$A_{st} = 842.15 mm^2$

Provide 10mm dia bars, spacing $= \dfrac{3000 \times \frac{\pi}{4} \times 10^2}{842.15} = 279.783mm$

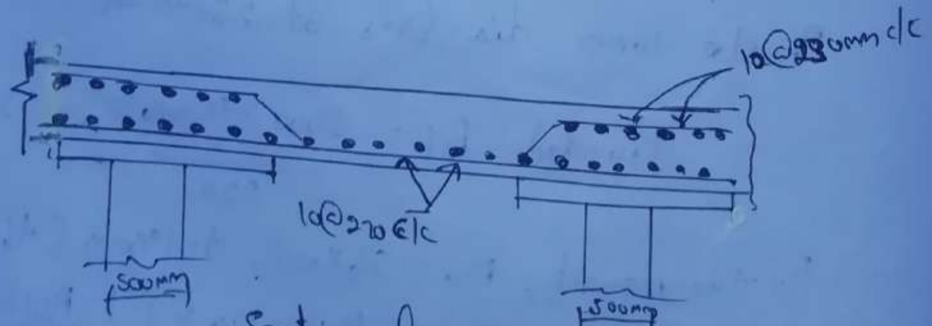
Provide spacing 270mm, Provided $A_{st} = \dfrac{3000 \times \frac{\pi}{4} \times 10^2}{270} = 872.665 mm^2$



Plan



Section along C_1C_2 (Column Strip)



Section along M_1M_2 (Middle Strip)

Pb) Design the interior panel of a flat slab 5.6m × 6.6m in size, for a superimposed load of 7.75 kN/m². Provide two way reinforcement. Use M20 concrete and Fe 415 steel. Use M20 Concrete and Fe 415 steel.

Solution

Step 1 - Thickness of slab

Drops are provided, thickness is given by,

$$\frac{\text{Span}}{\text{Effective depth}} = 32$$

Taking maximum dimension, $\frac{6600}{d} = 32$

$d = 206.25\,mm \simeq 210\,mm$

Overall depth = d + cover = 210 + 15 = 225 mm

Depth of slab at drop = 225 + 50 = 275 mm

Step 2 - Panel dimensions

Length of panel, $L = l_1 = 6.6m$; Width of panel, $B = l_2 = 5.6m$

Along length

Width of column strip $= 0.25\,l_2 \not> 0.25\,l_1$

$= 0.25 \times 5.6 \not> 0.25 \times 6.6$

$= 1.4m$ on either side of column centre line

Width of middle strip = 6.6 − 1.4 − 1.4 = 3.8m

Along width

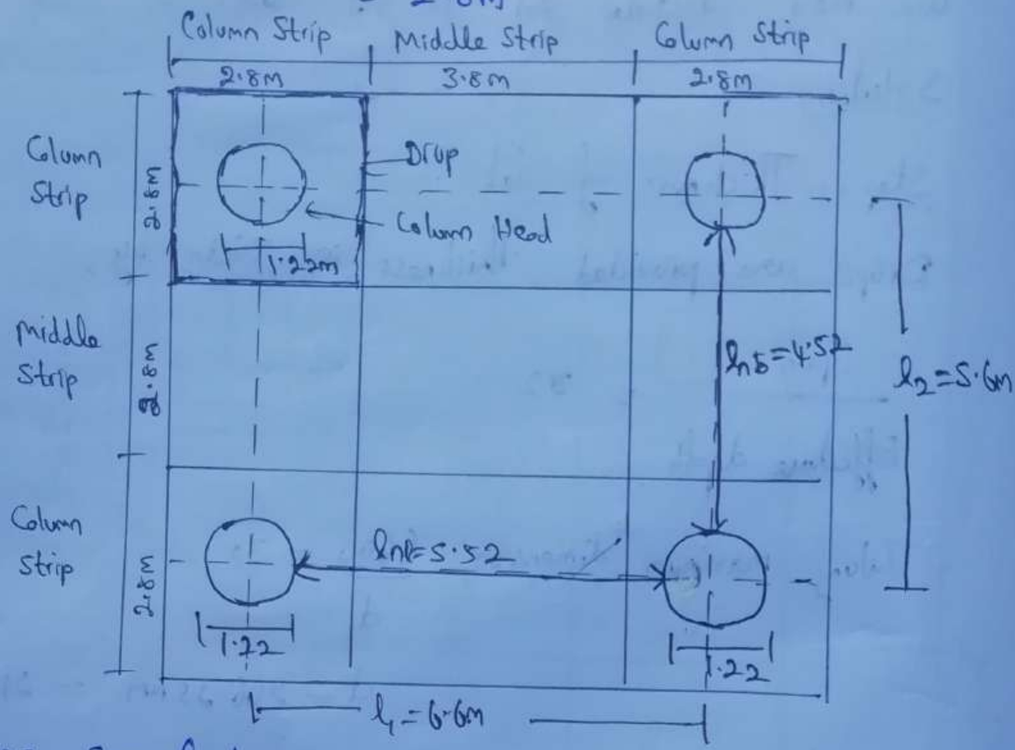
Width of column strip $= 0.25\,l_2 \not> 0.25\,l_1$

$= 0.25 \times 6.6 \not> 0.25 \times 5.6$

= 1.4m on either side of column centre line

Width of middle strip = 5.6 − 1.4 − 1.4

= 2.8m



Step 3 – Size of drop

Along length

Length of drop = $\frac{1}{3}$ × Panel length = $\frac{1}{3} \times 6.6 = 2.2m$.

However keep length of drop equal to column strip (2.8m)

∴ Provide drop of size 2.8m × 2.8m

Along width

Length of drop = $\frac{1}{3}$ × Panel length = $\frac{1}{3} \times 5.6 = 1.867m$

However keep length of drop equal to column strip (2.8m)

∴ Provide drop of size 2.8m × 2.8m

Step 4 – Loads

Self weight of slab = 0.275 × 25 = 6.875 kN/m^2

Live load = 7.75 kN/m^2.

Total working load = 14.625 kN/m^2

Total factored load = 1.5 × 14.625 = 21.94 kN/m^2

Step 5 - Moments along longer span

Let the column have a column head of diameter one fifth of average span.

$$\text{Average span} = \frac{1}{2}(L+B) = \frac{1}{2}(6.6+5.6) = 6.1\,m$$

$$\text{Column head (dia)} = \frac{1}{5} \times 6.1 = 1.22\,m$$

Assume height of column head as 500mm, column diameter as 400mm

$$\text{Total moment, } M_o = \frac{W l_n}{8}$$

where W = Design load on area $l_2 l_{n1}$.

$$\text{Equivalent square, } a^2 = \frac{\pi}{4} d^2$$

$$a^2 = \frac{\pi}{4} \times 1.22^2$$

$$a = \sqrt{\frac{\pi}{4} \times 1.22^2}$$

$$a = 1.08\,m.$$

$$l_{n1} = 6.6 - \frac{1.08}{2} - \frac{1.08}{2} = 5.52\,m$$

$$W = 21.94 \times 5.6 \times 5.52$$

$$= 678.21\,kN$$

$$\text{Total moment, } M_o = \frac{678.21 \times 5.52}{8} = 467.96\,kNm$$

In interior span, the total design moment shall be distributed in foll. proportions (IS 456 Pg 55 - 31.4.3.2)

Negative design moment = 0.65 × 467.96 = 304.17 kNm

Positive design moment = 0.35 × 467.96 = 163.79 kNm

The BM is distributed across column strip (IS 456 Pg 57 - 31.5.5.1, 31.5.5.3) and middle strip (IS 456 - Pg 57 - 31.5.5.4(a) as below,

Moment	Column Strip (KNm)	Middle strip (KNm)
-ve	0.75×304.17 = 228.13	0.25×304.17 = 76.04
+ve	0.6×163.79 = 98.27	0.4×163.79 = 65.52

Step 6 – Moments along shorter span

Total moment, $M_0 = \frac{W l_n}{8}$

Where W = Design load on area $l_1 l_{n_b}$

$l_{nb} = l_2 - \frac{1.08}{2} - \frac{1.08}{2}$

$= 5.6 - \frac{1.08}{2} - \frac{1.08}{2}$

$= 4.52 m$

$\therefore W = 21.94 \times 6.6 \times 4.52$

$= 654.51 kN$

Total moment, $M_0 = \frac{654.51 \times 4.52}{8} = 369.8 kNm$

Design moment is distributed as,

Negative design moment = 0.65 × 369.8 = 240.37 kNm

Positive design moment = 0.35 × 369.8 = 129.43 kNm

Moment distribution across column and middle strip,

Moment	Column Strip (KNm)	Middle Strip (KNm)
-ve	0.75×240.37 = 180.28	0.25×240.37 = 60.09
+ve	0.6×129.43 = 77.66	0.4×129.43 = 51.77

Check for limiting moment

$M_{ulim} = 0.138 f_{ck} b d^2$

Where b = Width of column strip = 2800mm

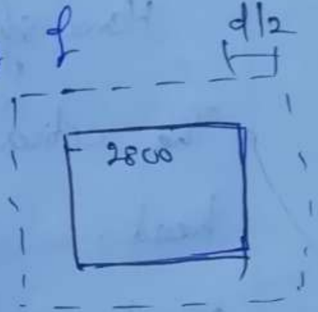
$d = 275 - 15 = 250mm$

$M_{ulim} = 0.138 \times 20 \times 2800 \times 250^2 = 4.83 \times 10^8$ Nmm

= 483 kNm, Hence safe

Step 7 - Check for shear

The critical section for shear is at a distance d/2 from face of drop. Hence pheriphery of critical section is square of



$size = 2800 + \frac{d}{2} + \frac{d}{2}$

$= 2800 + \frac{250}{2} + \frac{250}{2}$

= 3050mm

Shear force to be resisted = Shear force on panel - Shear force on square area

$= (21.94 \times 6.6 \times 5.6) - (21.94 \times 3.05 \times 3.05)$

= 606.806 kN

Nominal shear stress, $\tau_v = \frac{V}{b_o d}$

$= \frac{606.806 \times 10^3}{4 \times 3050 \times 250}$

$\tau_v = 0.199 N/mm^2$

Permissible shear stress $= k_s \tau_c$

$k_s = 0.5 + \beta_c$ where $\beta_c = \frac{L_1}{L_2} = \frac{6.6}{5.6} = 1.179$

$k_s = 0.5 + 1.179 = 1.679 \not> 1$

$\therefore k_s = 1$, $\tau_c = 0.25\sqrt{f_{ck}} = 0.25 \times \sqrt{20} = 1.12\ N/mm^2$

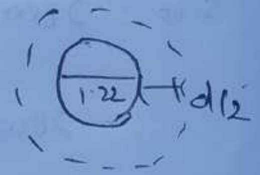
$k_s \tau_c = 1 \times 1.12$

$= 1.12\ N/mm^2$

$\tau_v < k_s \tau_c$

Hence safe

The critical section is at a distance $d/2$ from column head.



$\text{Diameter} = 1.22 + \frac{0.25}{2} + \frac{0.25}{2}$

$= 1.45m$

Shear force to be resisted $= (21.94 \times 6.6 \times 5.6) - \left(21.94 \times \frac{\pi}{4} \times 1.45^2\right)$

$= 774.673\ kN$

Nominal shear stress, $\tau_v = \frac{V}{b_o d}$

where b_o = circumference $= \pi \times d = \pi \times 1.45$

$\therefore \tau_v = \frac{774.673 \times 10^3}{\pi \times 1.45 \times 10^3 \times 250}$

$= 0.68\ N/mm^2 < k_s \tau_c$

Step 8- Reinforcement along longer span

(i) Column strip

For -ve moment, $M_u = 228.13$ kNm, $d = 225$mm, $b = 2800$mm

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$$

$$228.13 \times 10^6 = 0.87 \times 415 \times A_{st} \times 225 \times \left[1 - \frac{415 \times A_{st}}{2800 \times 225 \times 20}\right]$$

$$A_{st} = 3131.14 mm^2$$

Provide 12mm dia bar, spacing $= \frac{2800 \times \frac{\pi}{4} \times 12^2}{3131} = 163.956$mm

Provide 12mm dia bars @ 160mm c/c

$$\therefore \text{Provided } A_{st} = \frac{2800 \times \frac{\pi}{4} \times 12^2}{160} = 3208.564 mm^2$$

For +ve moment, $M_u = 98.27$ kNm, $d = 210$mm, $b = 2800$mm

$$98.27 \times 10^6 = 0.87 \times 415 \times A_{st} \times 210 \times \left[1 - \frac{415 \times A_{st}}{2800 \times 210 \times 20}\right]$$

$$A_{st} = 1361.503 mm^2$$

Provide 10mm dia bars, spacing $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{1361.503} = 161.52$mm

Provide 10mm dia bars @ 160mm c/c

$$\text{Provided } A_{st} = \frac{2800 \times \frac{\pi}{4} \times 10^2}{160} = 1374.447 mm^2$$

(ii) Middle strip

For -ve moment, $M_u = 76.04$ kNm and for +ve moment,

$M_u = 65.52$ kNm, $d = 210$mm, $b = 2800$mm

$$76.04 \times 10^6 = 0.87 \times 415 \times A_{st} \times 210 \times \left[1 - \frac{415 \times A_{st}}{2800 \times 210 \times 20}\right]$$

$$A_{st} = 1041.148 mm^2$$

Provide 10mm dia bars, spacing $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{1041.148} = 211.22$ mm

Provide 10mm dia bars @ 200mm c/c

A_{st} provided $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{200} = 1099.56 mm^2$

Step 9 – Reinforcement along short span

(i) Column strip

For -ve moment, $M_u = 180.28$ kNm, $d = 225$ mm, $b = 2800$ mm

$$180.28 \times 10^6 = 0.87 \times 415 \times A_{st} \times 225 \times \left[1 - \frac{415 \times A_{st}}{2800 \times 225 \times 20}\right]$$

$A_{st} = 2410.6 mm^2$

Provide 12mm dia bars, spacing $= \frac{2800 \times \frac{\pi}{4} \times 12^2}{2410.6} = 131.367$ mm

Provide 12mm dia bars @ 130mm c/c

A_{st} provided $= \frac{2800 \times \frac{\pi}{4} \times 12^2}{130} = 2435.943 mm^2$

For +ve moment, $M_u = 77.66$ kNm, $d = 210$ mm, $b = 2800$ mm

$$77.66 \times 10^6 = 0.87 \times 415 \times 210 \times A_{st} \times \left[1 - \frac{415 \times A_{st}}{2800 \times 210 \times 20}\right]$$

$A_{st} = 1064.229 mm^2$

Provide 10mm dia bars, spacing $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{1064.229} = 206.639$ mm

Provide 10mm dia @ 200mm c/c,

A_{st} provided $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{200} = 1099.56 mm^2$

(ii) Middle strip

For -ve moment, $M_u = 60.09$ kNm and +ve moment, $M_u = 51.77$ kNm

$d = 210$ mm, $b = 2800$ mm

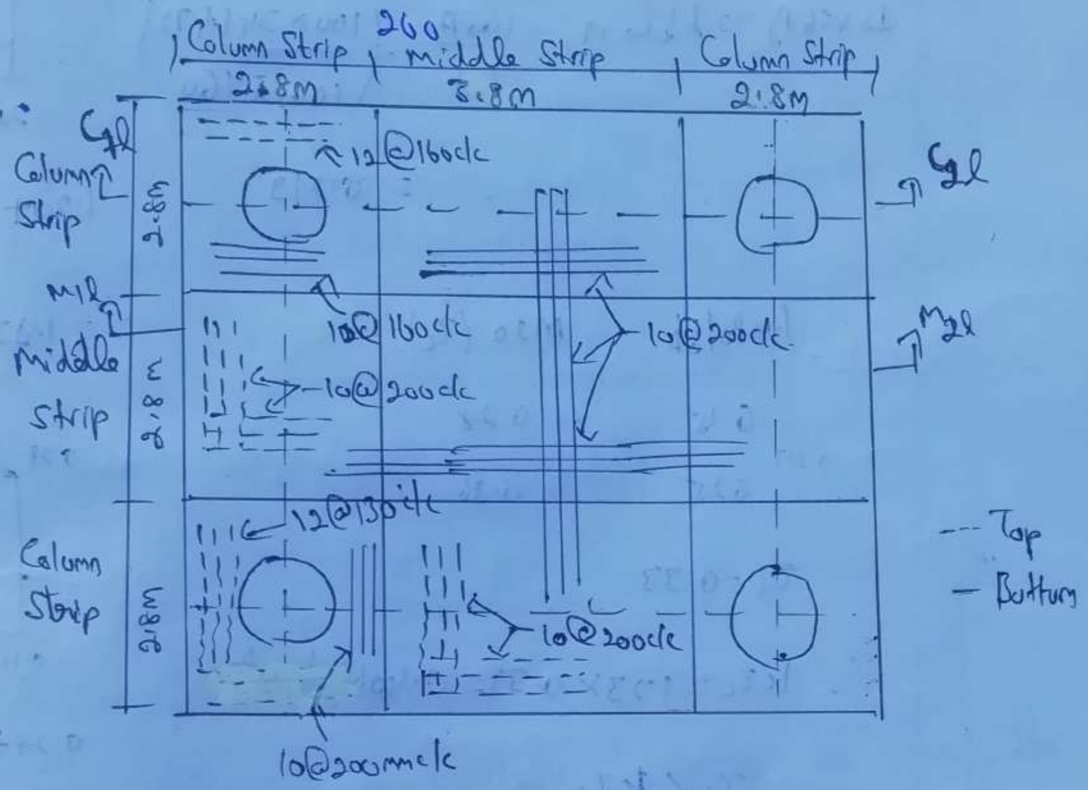
$$60.09 \times 10^6 = 0.87 \times 415 \times A_{st} \times 210 \times \left[1 - \frac{415 \times A_{st}}{2800 \times 210 \times 20}\right]$$

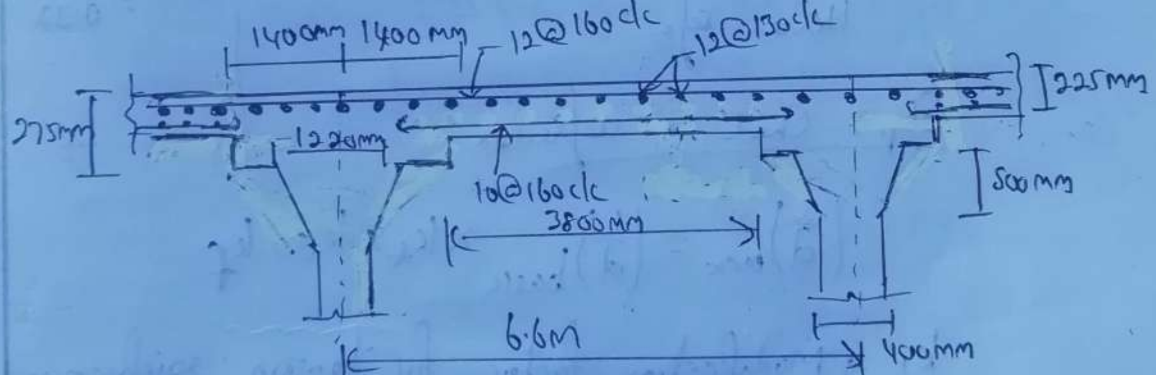
$A_{st} = 816.029 mm^2$

Provide 10 mm dia bars, spacing $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{816.029} = 269.49$ mm

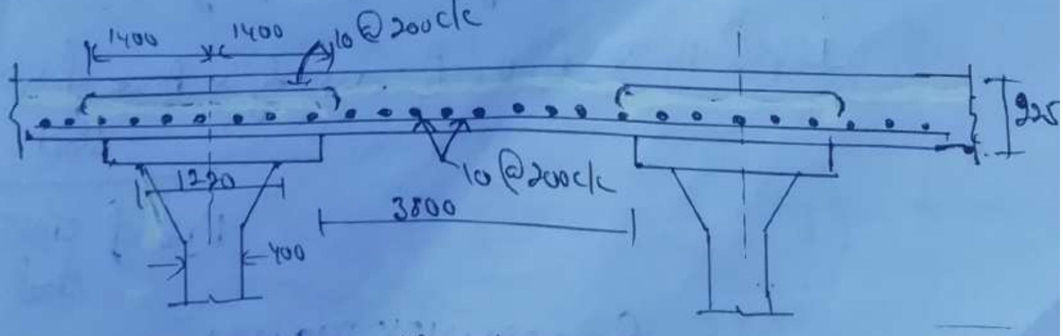
Provide 10mm dia bars at 260mm c/c

Ast provided $= \frac{2800 \times \frac{\pi}{4} \times 10^2}{260} = 845.813 \text{ mm}^2$





Section along column strip (C_{1l}, C_{2l}) - longer span



Section along middle strip (M_{1l}, M_{2l}) - longer span

Ex No. 6 **RCC DECK SLAB (or) SLAB CULVERT**

DATE:

AIM

Design a RCC culvert for a national highway to suit following data carriage way = 7.5 m wide, foot path = 1m on either side, clear span = 7m take loading IRC class AA tracked vehicle. Sketch the details of reinforced in the longitudinal and cross section of the slab.

DESIGN DATA

Clear span = 7 m

Wearing coat = 80mm thk (Assume)

Width of carriage way = 7.5m (2 lane)

Width of foot path = 1m (on either side)

Grade – M25 & Fe 415

Codes – IS 456 & IRC 21

Step 1 – Permissible stresses

Permissible flexural compressive stress, σ_{cb}= 8.33 N/mm^2 (IS 21 – 2000, Table 9)

Permissible stress in steel, σ_{st} = 200 N/mm^2 ((IS 21 – 2000, Table 10)

m = 280/3 σ_{cbc} = 280/(3*7) = 11.2

k = 1/[1+ (σ_{st}/m σ_{cbc})] = 0.32

j = 1-k/3 = 0.89

Q = 0.5 σ_{cbc} k j = 1.19

Step 2 – Depth of slab

Deck slab thk = 80mm/m of span = 80*7 = 560 = 600 mm

Effective thk = 600-25-(25/2) = 562.5 mm

Width of bearing = 400mm

Effective span

c/c of support = 7 + 0.4 = 7.4

Clear span + d = 7 + 0.5625 = 7.5625

Effective span = 7.4 m

Step 3 – Dead Load BM & SF

Self weight of slab = 0.6*25 = 15 kN/m^2

Self weight of wearing coat = 0.08*22 = 1.76 kN/m^2

Total DL = 16.76 kN/m^2

DL BM = $Wl^2/8$ = 114.722 kNm

DL SF = Wl/2 = 62.012 kN

Step 4 – Live Load BM

Effective length

Effective length = 3.6 + (2*(0.6+0.08)) = 4.96

Effective width

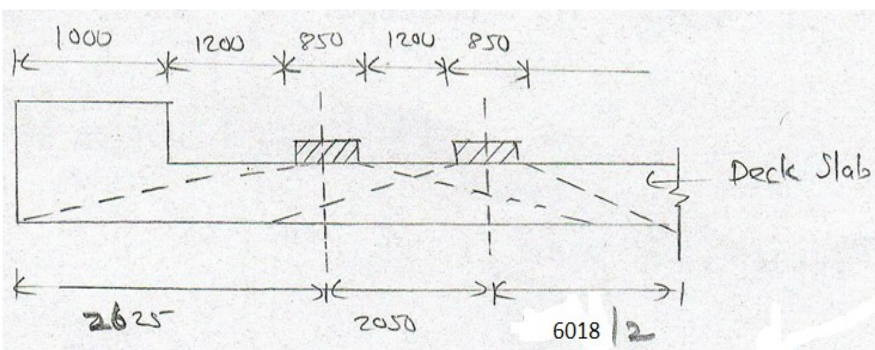
Effective width, b_e = kx (1-x/L) + b_w

Maximum BM occurs at centre of span, x = 7.4/2 = 3.7 m

L = 7.4 m, B = 7.5 + 1 + 1 = 9.5 m, For B/L = 1.284, k = 2.707 (IRC 21, Pg 53 – Simply supported slab)

b_w = Wheel base + (2*wearing coat) = 0.85+(2*0.08) = 1.01m

Substituting the values, Effective width, b_e = 6.018 m



Net Effective width = 2.625 + 2.05 + (6.018/2) = 7.684 m

Load

Load for IRC class AA tracked vechicle = 700 kN

Impact factor for 7.4 m span = 16% (IRC 6 – Pg 16)

Load with impact = 700 * 1.16 = 812 kN

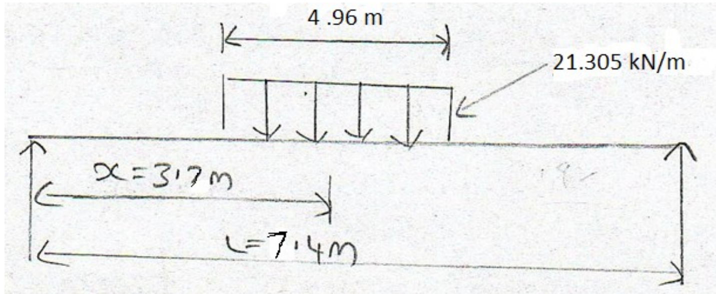
Average intensity of load = 812 / (7.684*4.96) = 21.305 kN/m^2

Bending Moment

Total downward load = 21.305 * 4.96 = 105.673 kN

Reaction = 105.073 / 2 = 52.537 kN

BM @ centre = (52.537 x 3.7) – (21.305 x 2.48 x (2.48/2)) = 128.87 kNm



Step 5 – Live Load SF

Effective length

Effective length = 3.6 + (2*(0.6+0.08)) = 4.96

Effective width

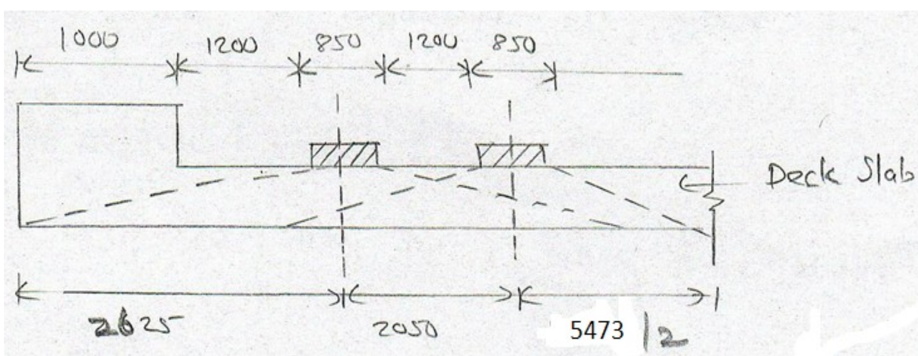
Effective width, $b_e = kx\,(1\text{-}x/L) + b_w$

Maximum SF occurs at support, x = 4.96/2 = 2.48 m

L = 7.4 m, B = 7.5 + 1 + 1 = 9.5 m, For B/L = 1.284, k = 2.707 (IRC 21, Pg 53 – Simply supported slab)

b_w = Wheel base + (2*wearing coat) = 0.85+(2*0.08) = 1.01m

Substituting the values, Effective width, b_e = 5.473 m



Net Effective width = 2.625 + 2.05 + (5.473/2) = 7.412 m

Load

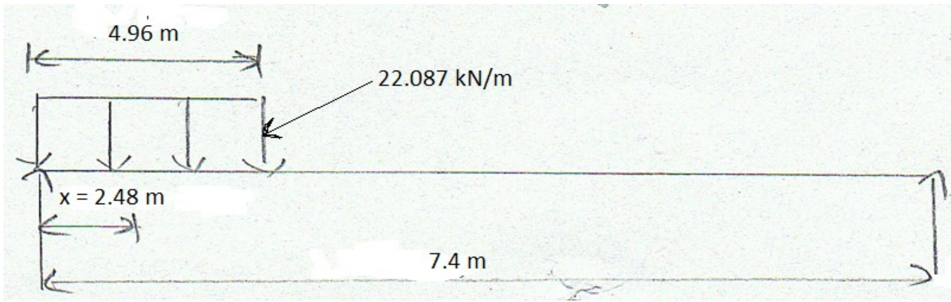
Load for IRC class AA tracked vechicle = 700 kN

Impact factor for 7.4 m span = 16% (IRC 6 – Pg 16)

Load with impact = 700 * 1.16 = 812 kN

Average intensity of load = 812 / (7.412*4.96) = 22.087 kN/m^2

Shear force



Total downward load = 21.305 * 4.96 = 105.673 kN

Reactions R_B = 36.715 kN & R_A = 72.837 kN

SF @ support = 72.837 kN

Step 6 – Design of deck slab

Main reinforcement

Total Moment=Dead load moment + Live load moment = 114.722 + 128.87 = 243.592 kNm

Hence safe

Provide 25 mm dia bars, S = [1000* (p/4) *25^2] / 2432.879 = 201.767 mm

Provide 25 mm dia bars at 200 mm c/c (A_{st} = 2454.369 mm^2)

Distributor reinforcement

Total Moment=$0.3M_L + 0.2M_D$ = (0.3 x 128.87) + (0.2 x 114.722) = 61.605 kNm

Provide 12 mm dia bars, S = [1000* (p/4) *12^2] / 615.281 = 183.814 mm

Provide 12 mm dia bars at 180 mm c/c (A_{st} = 628.319 mm^2)

Step 7 – Check for shear stress

Total Shear = Dead load shear + Live load shear = 62.012 + 72.837 = 134.849 kN

Permissible shear stress for slabs without shear reinforcement is given as

$k_1 = 1.14 – 0.7\ d \geq 0.5$

$= 1.14 – (0.7 \times 0.5625) = 0.746$

$k_2 = 0.5 + 0.25\ p \geq 1$

$k_2 = 0.5 + (0.25 \times 0.436) = 0.609 \geq 1 = 1$

Hence

Also, hence provide minimum shear reinforcement.

Minimum shear reinforcement is given by A_{sv}/ (b*S_v) = 0.4 / (0.87*f_y)

Provide 2 legged 10 mm dia stirrups at 140 mm c/c.

Step 8 – Design of kerb

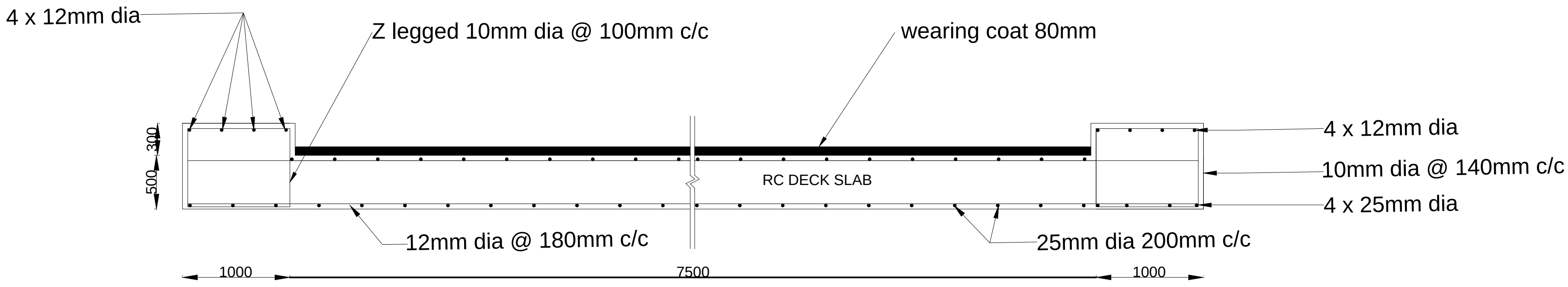
Assuming depth of kerb above deck slab as 300 mm, total depth = 300 + 562.5 = 862.5 mm

At bottom,

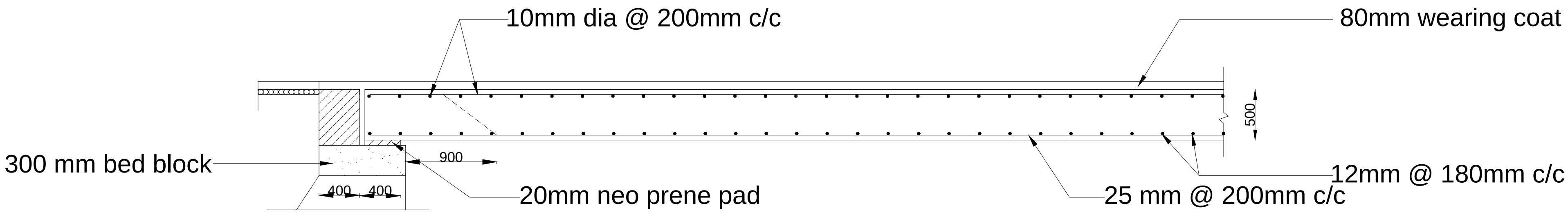
Provide 4 No's of 25 mm dia (A_{st} = 1963.495 mm^2)

At top, A_{st} = 0.12 % bd = (0.12/100) x 1000 x 300 = 360 mm^2

Provide 4 No's of 12 mm dia (A_{st} = 482.389 mm^2)



CROSS SECTION OF RC DECK SLAB



LONGITUDINAL SECTION OF RC DECK SLAB

Unit III – Liquid Storage Structures

RCC water tanks – On ground, elevated circular, underground Rectangular Tanks – Hemispherical Bottomed Steel Water Tank – Design and Drawing

I) Concrete Water Tanks

Tanks resting on ground

1) Rectangular Water Tank with L/B ratio > 2
2) Rectangular Water Tank with L/B ratio < 2
3) Circular Water Tank with open top (fixed base)
4) Circular Water Tank with open top (flexible base)
5) Circular water Tank with domical top and flat base supported on masonry tower (flexible base)
6) Circular water tank with doomed bottom and top

Tanks resting underground

1) Rectangular water tank

Elevated water tanks

1) Intze type water tank

2) Circular water tank

II) Steel water tanks

1) Hemispherical bottomed steel water tank

Design the sidewalls of a rectangular reinforced concrete water tank of dimensions 6m by 2m and having a maximum depth of 2.5m, using M20 grade concrete and Fe 415 HYSD bars

Rectangular WT with L/B > 2

on ground

Given

Size of tank, L×B = 6m × 2m

Depth of tank, H = 2.5m

Materials - M20 grade concrete and Fe415 HYSD bars

Solution

Step 1 - Permissible Stress

From IS 456-2000 - Table 21,

Permissible stress in direct compression, $\sigma_{cc} = 5 N/mm^2$

Permissible stress in bending compression, $\sigma_{cbc} = 7 N/mm^2$

Permissible stress in steel, $\sigma_{st} = 0.6 f_y = 0.6 \times 250$

$= 150 N/mm^2$

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times7} = 13.33$$

$$k = \frac{1}{1+\frac{\sigma_{st}}{m\sigma_{cbc}}} = \frac{1}{1+\frac{150}{13.33\times7}} = 0.38$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.38}{3} = 0.87$$

$$Q = 0.5\,\sigma_{cbc}\,kj = 0.5\times7\times0.38\times0.87 = 1.16$$

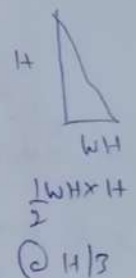
Step 2 – Dimensions of tank

L = 6m and B = 2m

Ratio $L/B = 6/2 = 3 > 2$

Long walls are designed as vertical cantilevers fixed at base and short walls are designed as horizontal slabs between long walls. These horizontal slabs bend horizontally.

Step 3 – Design of ~~long~~ side walls (Vertical eff)



Maximum BM in long walls $= \left[\frac{1}{2} WH \times H\right] \times \frac{H}{3}$

$$= \frac{WH^3}{6}$$

$$= \frac{10 \times 2.5^3}{6}$$

$$M_L = 26.04 \text{ kNm}$$

$M = Qbd^2$

$26.04 \times 10^6 = 1.16 \times 1000 \times d^2$

$\Rightarrow d = 149.83 \text{ mm}$

Adopt effective depth, d = 150mm

Overall depth, D = d + cover = 150 + 30 = 180mm

$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{26.04 \times 10^6}{150 \times 0.87 \times 150} = 1330.27 \text{ mm}^2$

Provide 16mm diameter bars, spacing $= \frac{b \times a_{st}}{A_{st}}$

$$= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1330.27}$$

$$= 151.14 \text{ mm}$$

Provide 16mm diameter bars at 150mm c/c as vertical reinforcement $\left(A_{st} \text{ provided} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{150} = 1340.41 mm^2\right)$

Step 4 – Design of long walls (horizontal reinforcement)

Intensity of water pressure, $p = w(H-h)$ where $h = H/4$ or 1m whichever is greater

$h = 2.5/4$ (or) 1m =

$= 0.63$ (or) 1m

$\therefore h = 1m$

Intensity, $p = 10(2.5-1) = 15 kN/m^2$

Direct tension in long wall, $T_x = \frac{PB}{2} = \frac{15 \times 2}{2} = 15 kN$

$A_{st1} = \frac{T_x}{\sigma_{st}} = \frac{15 \times 10^3}{150} = 100 mm^2$

Min $A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 180 = 540 mm^2 = 270 mm^2$ on each face

Providing 10mm diameter bars,

$$\text{Spacing} = \frac{b \times a_{st}}{A_{st}} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{270}$$

$= 290.89 mm$

Provide 10mm diameter bars at 280mm c/c on both faces of long wall in horizontal direction

Step 5 – Design of short wall (horizontal reinforcement)

Direct tension in short wall $= P \times 1$ (per m)

$= 15 \times 1$

$T_y = 15 kN$

Bending moment, $M = \frac{PB^2}{12}$ where $B = 2m + \frac{0.18}{2} + \frac{0.18}{2}$ (c/c)

$B = 2.18 m$

$$M = \frac{15 \times 2.18^2}{12} = 5.94 \, kNm$$

$$A_{st} = \frac{M - T_y}{\sigma_{st} j d} + \frac{T_y}{\sigma_{st}}$$

$$= \frac{(5.94 \times 10^6) - (15 \times 10^3)}{150 \times 0.87 \times 150} + \frac{15 \times 10^3}{150}$$

$$A_{st} = 402.68 mm^2$$

$$\text{Min } A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 180 = 540 mm^2 = 270 mm^2 \text{ on each face}$$

Providing 10mm diameter bars,

$$\text{Spacing} = \frac{b \times a_{st}}{A_{st}} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{270}$$

$$= 290.89 mm$$

Provide 10mm diameter bars at 280mm c/c on both faces of short wall in horizontal direction.

<u>Step 6 – Design of reinforcement for cantilever action</u>

$$\text{Cantilever moment} = \left(\frac{1}{2} w H \times h\right) \times \frac{h}{3}$$

$$= \frac{1}{2} \times 10 \times 2.5 \times 1 \times \frac{1}{3}$$

$$= 4.17 \, kNm$$

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{4.17 \times 10^6}{150 \times 0.87 \times 150} = 213.03 mm^2$$

$$\text{Min } A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 180 = 540 mm^2 = 270 mm^2 \text{ on each face}$$

Provide 10mm diameter bars at 280mm c/c at junction of side wall and base slab on both faces

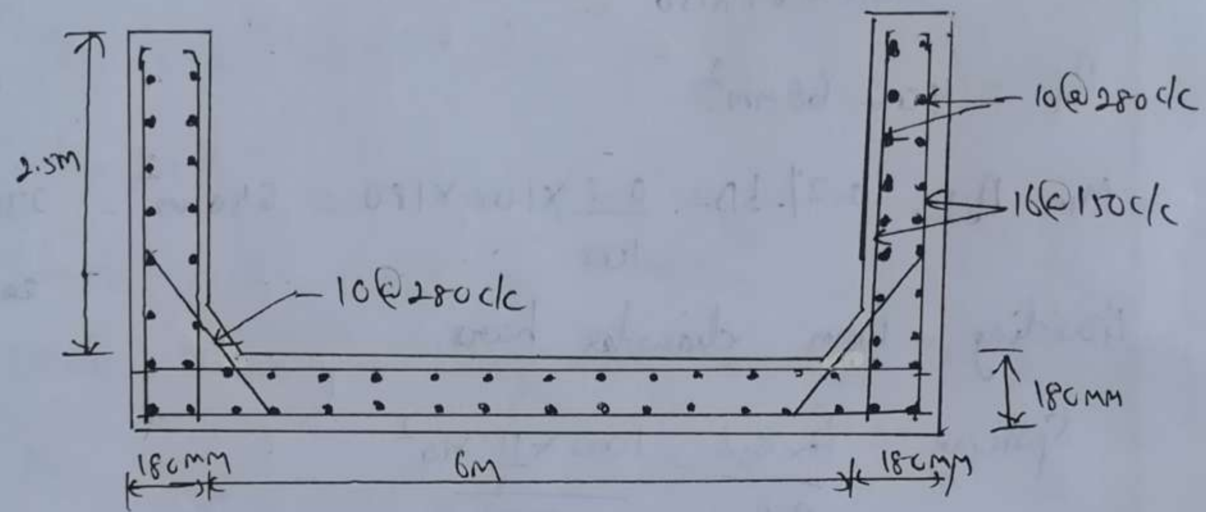
Step 7- Design of base slab

Take overall thk = 180mm and effective thk = 150mm

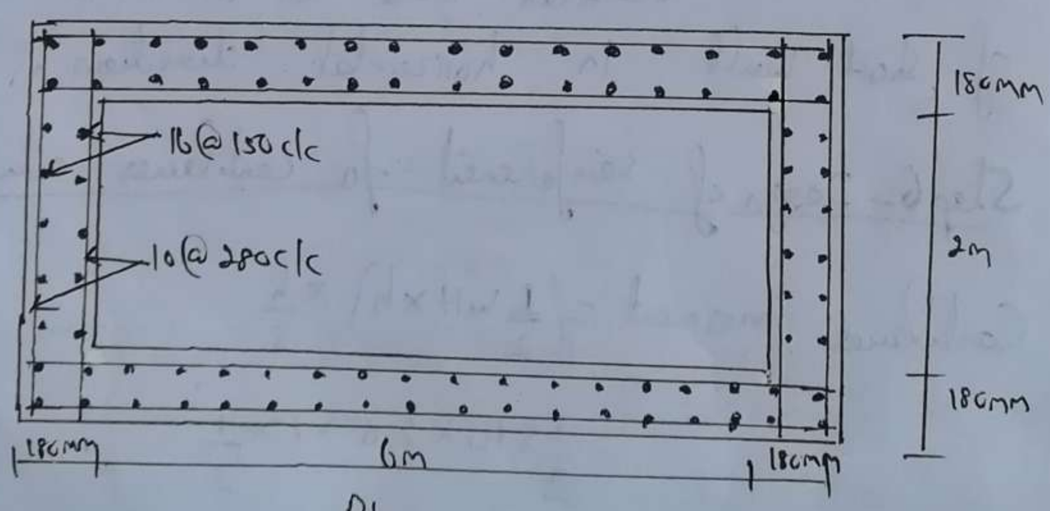
Minimum $A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 180 = 540mm^2$

$= 270mm^2$ on each face

Provide 10mm @ 280mm c/c as main and distribution bar on both faces.



Sectional Elevation



Plan.

A rectangular RCC water tank resting on ground with an open top is required to store 80000 litres of water. The inside dimension of the tank may be taken as 6 x 4 m. The tank rests on wall on all four sides. Design the side walls of the tank using M20 concrete and Fe 415 steel.

Draw the following,

(i) Cross sectional elevation of rectangular water tank

(ii) Plan of rectangular water tank

DESIGN DATA

Volume of tank = 80000 litres

Size of tank = 6 m x 4 m

Grade – M20 & Fe415

SOLUTION

Step 1 – Permissible stresses

From IS : 456 – 2000 – Table 21,

Permissible stress in direct compression, $\sigma_{cc} = 5$ N/mm^2

Permissible stress in bending compression, $\sigma_{cbc} = 7$ N/mm^2

Permissible stress in steel, $\sigma_{st} = 0.6$ fy = 150 N/mm^2 (Assume)

$m = 280/3\ \sigma_{cbc} = 280/(3*7) = 13.333$ $k = 1/[1+ (\sigma_{st}/m\ \sigma_{cbc})] = 0.38$

$j = 1\text{-}k/3 = 0.87$

$Q = 0.5\ \sigma_{cbc}\ k\ j = 1.16$

Step 2 – Dimensions of tank

Depth of tank = Volume/Area =(80000 x 10^{-3})/(6 x 4) = 3.33 m Assuming free board as 150 mm, Depth = 3.33 + 0.15 = 3.48 m

Hence take depth of tank as 3.5 m

L/B = 6/4 = 1.5 < 2, Hence walls are designed as continuous slab subjected to water pressure above an height of H/4 or 1m, whichever is greater, h = 3.5/4 (or) 1 = 1m

Intensity of water pressure, p=ρ (H-h)=10(3.5 – 1)=25 kN/m^2

Step 3 – Moment on side walls

Long wall

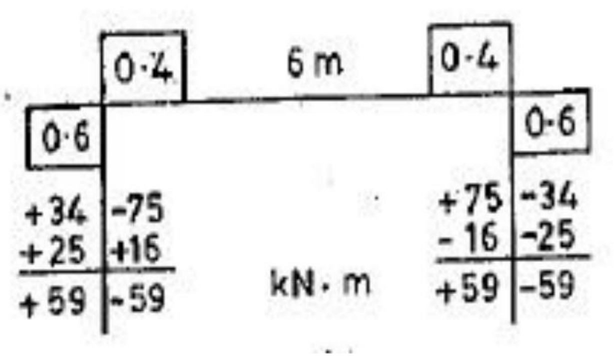
BM at fixed end of long wall = (pL^2)/12 = (25 x 6^2)/12 = 75 kNm

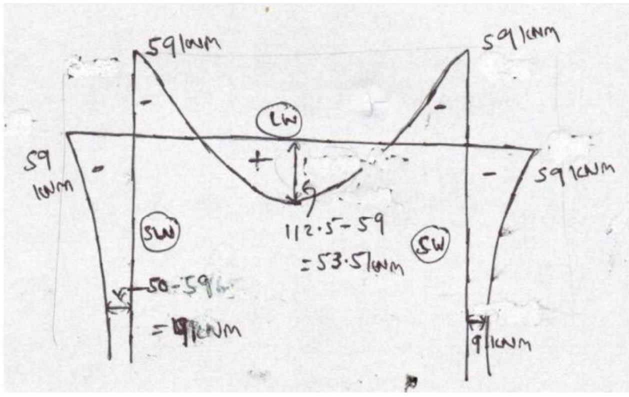
BM in centre of long wall = (pL^2)/8 = (25 x 6^2)/8 = 112.5 kNm

Short wall

BM at fixed end of short wall = (pB^2)/12 = (25 x 4^2)/12 = 34 kNm

BM in centre of short wall = (pB^2)/8 = (25 x 4^2)/8 = 50 kNm





Step 4 – Design of side walls (vertical reinforcement)

Maximum moment, Qbd^2 = 59 kNm

d = 225.53 mm, Eff depth = 225 mm, Overall depth = 250 mm

Minimum A_{st} = 0.3% b D = 750 mm^2

Provide 12mm dia bars, Spacing = (1000x(π/4)x12^2) / 750= 150.08 = 150 mm

Provide 12mm dia bars @ 150 mm c/c as vertical reinforcement in side walls

Provided A_{st} = (1000x(π/4)x12^2) / 160 = 753.98 mm^2

Step 5 – Design of long walls (horizontal reinforcement)

Direct tension in long wall, T_x = pB/2 =(25 x 4)/2 = 50kN

Moment at long wall ends, M = 59 kNm

A_{st} (long wall corners) = $\frac{M - T_x}{\sigma_{st}\, jd} + \frac{T_x}{\sigma_{st}} = 2341\ \text{mm}^2$

Provide 20 mm dia bars, Spacing = (1000x(π/4)x20^2) / 2341= 134.20 = 130 mm

Provide 20 mm dia bars @ 130 mm c/c as horizontal reinforcement at corner of long wall.

Provided A_{st} = (1000x(π /4)x20^2) / 130 = 2416.61 mm^2

Moment at long wall centre, M = 53.5 kNm

A_{st} (long wall centre) = 2153.68 mm^2

Provide 20 mm dia bars, Spacing = (1000x(π/4)x20^2) / 2153.68= 145.87 = 130 mm

Provide 20 mm dia bars @ 130 mm c/c as horizontal reinforcement at centre of long wall.

Provided A_{st} = (1000x(π /4)x20^2) / 130 = 2416.61 mm^2

<u>Step 6 – Design of short walls (horizontal reinforcement)</u>

Direct tension in short wall, T_y = pL/2 =(25 x 6)/2 = 75kN

A_{st} (short wall corners) = $\frac{M-T_y}{\sigma_{st}\ jd} + \frac{T_y}{\sigma_{st}} = 2506.81\ \text{mm}^2$

Provide 20 mm dia bars, Spacing = (1000x(π/4)x20^2) / 2506.81= 125.32 = 120 mm

Provide 20 mm dia bars @ 120 mm c/c as horizontal reinforcement at corner of short wall.

Provided A_{st} = (1000x(π/4)x20^2) / 120 = 2617.99 mm^2

Moment at short wall centre, M = 9 kNm

A_{st} (short wall centre) = 803.96 mm^2

Provide 12 mm dia bars, Spacing = (1000x(π/4)x12^2) / 803.96= 140.68 = 120 mm

Provide 12 mm dia bars @ 120 mm c/c as horizontal reinforcement at centre of short wall.

Provided A_{st} = (1000x(π/4)x12^2) / 120 = 942.48 mm^2

<u>Step 7 – Design of reinforcement for cantilever action</u>

Cantilever moment = (1/2x3.5x1x10) x ((1/3) x 1) = 5.83 kNm

A_{st} = M/σstjd = (5.83 x 10^6)/(150 x0.87x225) = 198.55 mm^2

Minimum A_{st} = 0.3% b D = 750 mm^2

Provide 12mm dia bars, Spacing = (1000x(π/4)x12^2) / 750= 150.08 = 150 mm

Provide 12 mm dia bars @ 150 mm c/c at junction of side wall and base slab

Provided A_{st} = (1000x(π /4)x12^2) / 160 = 753.98 mm^2

Step 8 – Design of base slab

Taking overall thickness of base slab as 250 mm, effective depth = 225 mm (Cover = 25 mm)

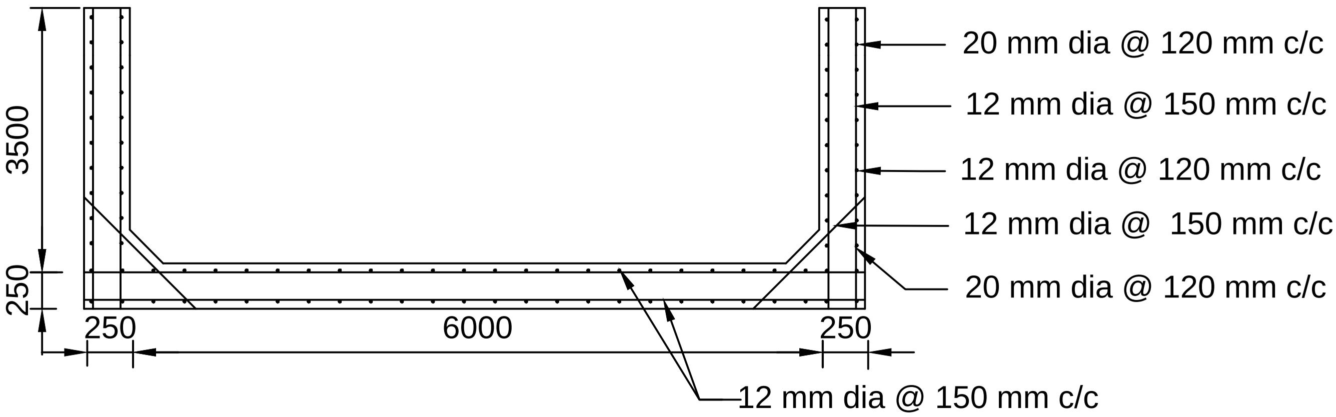
Minimum Ast = 0.3% b D = 750 mm^2

Provide 12mm dia bars, Spacing = (1000x(π /4)x122) / 750= 150.08 = 150 mm

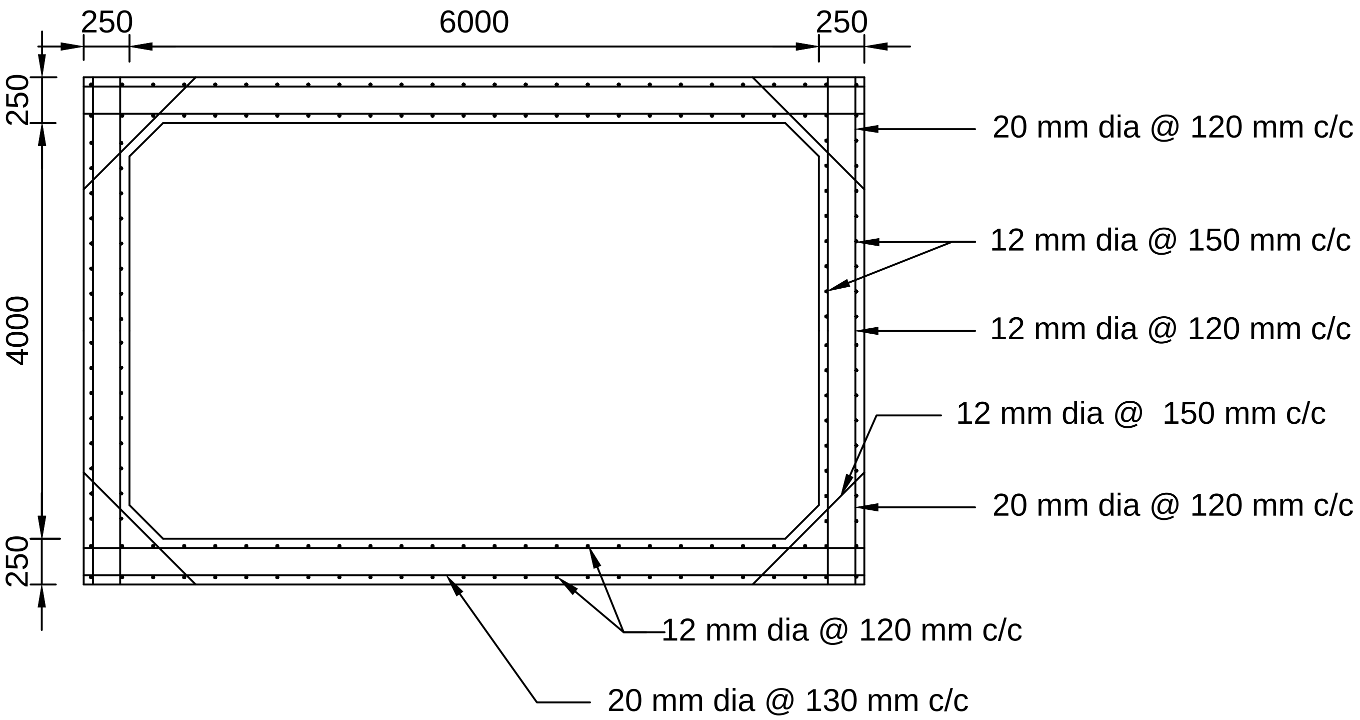
Provide 12mm dia bars @ 150 mm c/c as vertical reinforcement in side walls

Provided A_{st} = (1000x(π /4)x122) / 160 = 753.98 mm^2

RECTANGULAR WATER TANK



CROSS SECTION



PLAN

All dimensions are in mm
M20 Grade Concrete
Fe 415 Grade steel

Circular Water Tank

Circular tanks on ground may be designed either with flexible connection of the wall with the base or with a rigid connection of the wall with base. In flexible connection, expansion or contraction of side walls is possible while in rigid connection, the walls are monolithic with the base.

(i) Circular tank with rigid connection (wall restrained at base)

The wall resist the water pressure partly by hoop action and partly by cantilever action, while hoop action is predominant. These tanks are analysed by following methods.

- → Reissner's method
- → Carpenter's method
- → Approximate method
- → IS Code method

IS code method

The bending moments, hoop tension and shear at base for the tank wall of circular tank may be determined by using appropriate coefficients given by using IS code.

These coefficients depend on the ratio H^2/Dt

→ Hoop tension per metre height = Coefficient × wHR (N/m)

→ Bending moment per metre run = Coefficient × wH^3 (Nm/m)

→ Shear force at base of wall = Coefficient × wH^2 (N)

(Pb) Design a circular tank 12m diameter and 4 metre high. The tank rests on firm ground. The walls of the tank are restrained at the base. Use M20 concrete and Fe 415 steel.

Given

Circular water tank with fixed base (open top)

Diameter, D = 12 m

Height, H = 4 m

Restrained at base

M20 grade concrete and Fe415 grade steel

Solution

Step 1 - Permissible stresses

Permissible stress in direct tension (tank wall), $\sigma_{ct} = 1.2 N/mm^2$
(IS 3370, Part II, Table 1)

Permissible stress in steel, $\sigma_{st} = 115 N/mm^2$ (Fe 250)
(IS 3370 - Part II)

Permissible stress in direct compression, $\sigma_{cc} = 5 N/mm^2$

bending compression, $\sigma_{cbc} = 7 N/mm^2$

(IS 456-200 - Table 21)

$m = 280/3\sigma_{cbc}$

$= 280/(3\times7)$

$= 13.333$

$k = 1/\left(1 + \frac{\sigma_{st}}{m\sigma_{cbc}}\right) = 1/\left(1 + \frac{115}{13.333\times7}\right)$

$= 0.448$

$j = 1 - k/3 = 1 - \frac{0.448}{3} = 0.851$

$Q = 0.5\sigma_{cbc}kj = 0.5\times7\times0.448\times0.851 = 1.334$

Step 2 - Dimensions of tank

$D = 12m$, $H = 4m$

Thickness of wall is taken as greater of following,

(i) 150mm

(ii) $(3H + 5)$ cm $= (3 \times 4) + 5 = 17$cm $= 170$mm

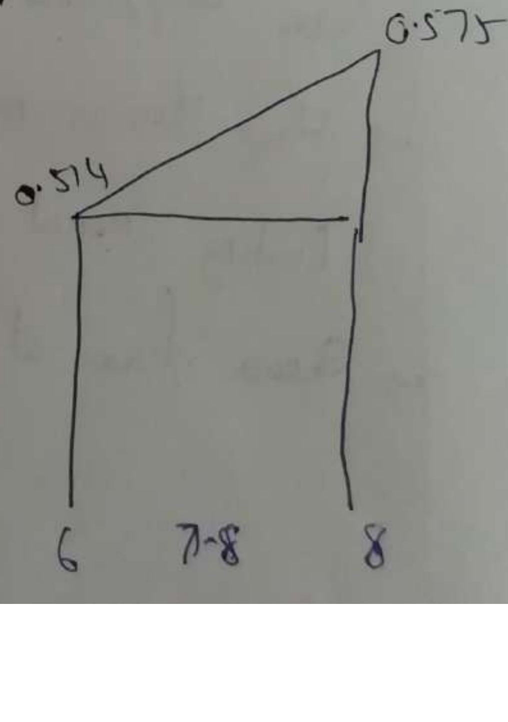
∴ Take ~~depth~~ thickness = 170mm, Effective ~~depth~~ thick = $170 - 30 = 140$mm

Step 3 - Design of side walls for hoop tension

$$\frac{H^2}{Dt} = \frac{4^2}{12 \times 0.17} = 7.8$$

IS 3370-IV, Table 9, Pg 35, for 0.6H,

H^2/Dt	Coefficient
6	0.514
8	0.575



Coefficient for hoop tension $= 0.514 + \left(\frac{0.575 - 0.514}{8-6}\right) \times (7.8 - 6)$

$= 0.569$

Hoop tension per metre height = Coefficient × WHR

$= 0.569 \times 9810 \times 4 \times 6$

$F_t = 133965.36 N$

$A_{st} = \frac{F_t}{\sigma_{st}}$

σ_{st} for Fe 250 = $115 N/mm^2$ ($150 N/mm^2$ for Fe 415)

$A_{st} = \frac{133965.36}{115}$

$= 1164.916 mm^2$

A_{st} for each face $= \frac{1164.916}{2} = 582.458 mm^2$

Provide 12 mm dia bars, spacing $= \frac{b \times a_{st}}{A_{st}}$

$= \frac{1000 \times \frac{\pi}{4} \times 12^2}{582.458}$

$= 194.173 mm$

Provide 12 mm diameter bars @ 190 mm c/c as horizontal reinforcement (A_{st} provided $= 595.249 mm^2$) on each face

Vertical reinforcement $= 0.3\% bD$

$= \frac{0.3}{100} \times 1000 \times 170 = 510 mm^2$

A_{st} for each face $= \frac{510}{2} = 255 mm^2$

Provide 8 mm dia bars. Spacing $= \dfrac{1000 \times \frac{\pi}{4} \times 8^2}{255}$

$= 197.12\,mm$

Provide 8mm dia bars at 190mm c/c as vertical reinforcement (A_{st} provided $= 264.555\,mm^2$) on each face

<u>Step 4 - Design of side wall for cantilever moment</u>

IS 3370 - Part IV, Table 10, Pg 36 for 1.0H,

H^2/Dt	Coeff
6	0.0187
8	0.0146

Coefficient for moment $=$ ~~0.0187 − 0.0187 − 0.01~~

Coefficient for moment $= 0.0187 - \dfrac{(0.0187 - 0.0146)}{8-6} \times (7.8-6)$

$= 0.016$

Moment $=$ coefficient $\times w H^3$

$= 0.016 \times 9810 \times 4^3$

Moment $= 10045.44\,Nm$

$A_{st} = \dfrac{M}{\sigma_{st} j d} = \dfrac{10045.44 \times 10^3}{115 \times 0.857 \times 140} = 733.185\,mm^2$

Minimum $A_{st} = 0.3\%\, bD = \dfrac{0.3}{100} \times 1000 \times 170 = 510\,mm^2$

~~Provide~~ A_{st} on one face $= \dfrac{733.185}{2} = 366.593\,mm^2$

Provide 8mm dia bars, $A_{st} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{366.593} = 137.115mm$

Provide 8mm dia bars at 130mm c/c as cantilever reinforcement on both faces ($A_{st} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{130} = 386.658 mm^2$)

Step 5 - Design of base slab

Provide base slab of thickness = 200mm, d = 170mm

$A_{st} = 0.3\%.bD$

$A_{st} = \frac{0.3}{100} \times \frac{1000 \times 200}{1} = 600mm^2$

A_{st} on each face $= \frac{600}{2} = 300mm^2$

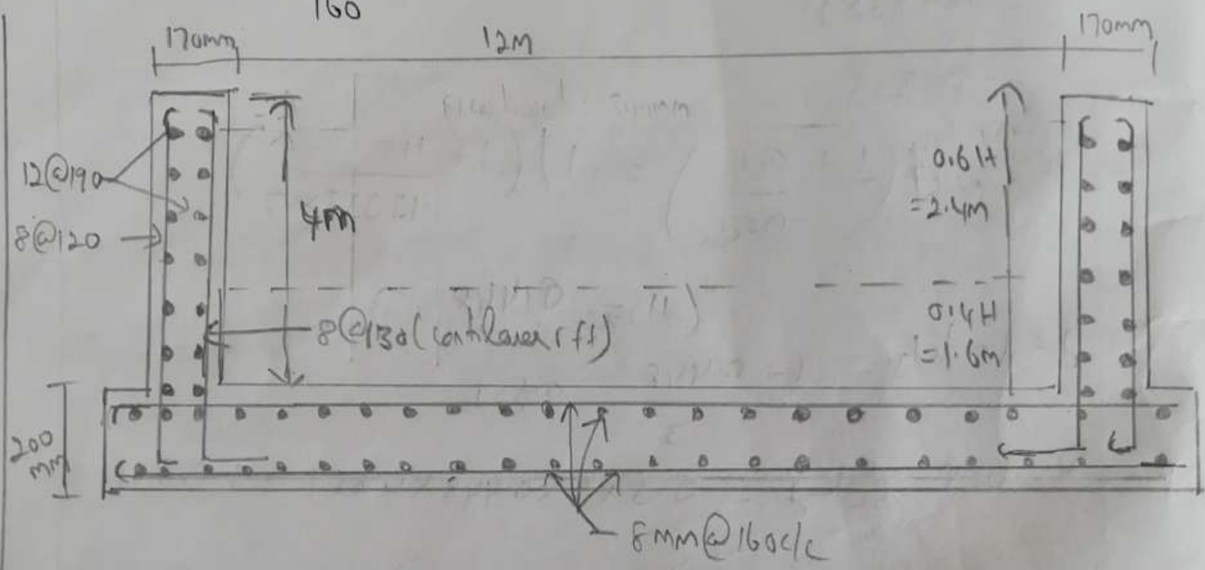
Provide 8mm dia bars, spacing $= \frac{b \times a_{st}}{A_{st}}$

$$= \frac{1000 \times \frac{\pi}{4} \times 8^2}{300}$$

$$= 167.552\,mm$$

Provide 8mm dia bars at 160mm c/c on both faces.

$$\left(A_{st} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{160} = 314.159\,mm^2\right)$$



Pb) Design a circular tank resting on firm ground to the following particulars.

Circular water tank with flexible base (open top)

(i) Diameter of tank = 3.50m

(ii) Depth of water = 3M

(iii) The wall and base are not monolithic with each other

(iv) Specific weight of water = 9810 N/m^3

Use M25 concrete and Fe415 steel

Step 1 - Permissible stress

Permissible stress in direct tension = 1.2 N/mm^2 (IS 3370 - Part II Table 1)

Permissible stress in steel = 150 N/mm² (Fe415 grade)

Permissible stress in direct Compression, $\sigma_{cc} = 5 N/mm^2$

Permissible stress in bending compression, $\sigma_{cbc} = 7 N/mm^2$

(IS 456 - 2001, Table 1)

$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.333$

$k = 1/\left(1 + \frac{\sigma_{st}}{m\sigma_{cbc}}\right) = \frac{1}{\left(1 + \frac{150}{13.33 \times 7}\right)} = 0.384$

$j = 1 - \frac{k}{3} = 1 - \frac{0.384}{3} = 0.87$

$Q = 0.5\,\sigma_{cbc} k j = 0.5 \times 7 \times 0.387 \times 0.87 = 1.16$

Step 2 - Dimensions of tank

D = 3.5m, H = 3m

Thickness of wall is taken as greater of following.

(i) 150mm

(ii) (3H + 5) cm = (3 × 3) + 5 = 14cm = 140mm

Take thickness = 150mm

Step 3 - Design of side wall

Consider 1m height of wall.

~~Hoop tension = $\gamma_w \times H \times \frac{D}{2}$ = $9.81 \times 1 \times \frac{3.5}{2}$ = 17.168 kN~~

Hoop tension = $\gamma_w \times H \times \frac{D}{2} = 9.81 \times 2.5 \times \frac{3.5}{2} = 42.919$ kN

$F_{st} = 42.919$ kN

$A_{st} = \frac{F_{st}}{\sigma_{st}} = \frac{42.919 \times 10^3}{115} = 373.209 mm^2$

Minimum $A_{st} = 0.3\% bD = \frac{0.3}{100} \times 1000 \times 150 = 450 mm^2$

A_{st} on one face $= 450/2 = 225 mm^2$

Provide 10mm dia bars, ~~Spt~~ Spacing = $\frac{b \times ast}{Ast}$

$= \frac{1000 \times \frac{\pi}{4} \times 10^2}{225}$

$= 349.066mm$

Provide 10mm dia bars @ 300mm c/c (Ast provided = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{300} = 261.799mm^2$) an horizontal and vertical reinforcement on both faces.

Permissible stress in tank wall = $\frac{F_t}{A_c + mA_{st}}$

$= \frac{42.919 \times 10^3}{(1000 \times 150) + (13.33 \times 2 \times 261.799)}$

$= 0.273 N/mm^2$

$< 1.2 N/mm^2$

Step 4 - Design of base slab

Provide base slab of thickness 200mm, d = 170mm

$A_{st} = 0.3\%.bD = \frac{0.3}{100} \times 1000 \times 200$

$= 600mm^2$

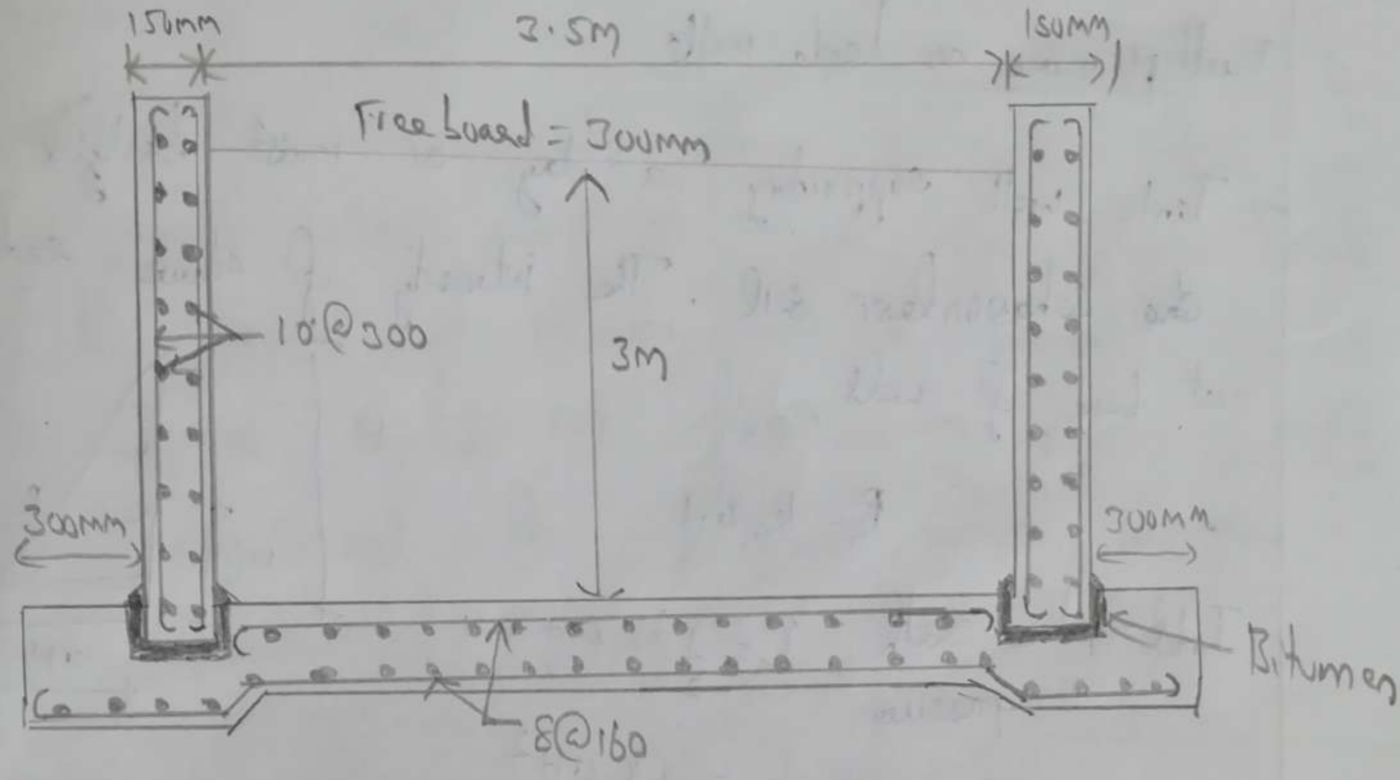
Ast on each face = $\frac{600}{2} = 300mm^2$

Provide 8mm dia bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 8^2}{300}$

$= 167.552mm$

Provide 8mm dia bars at 160mm c/c on both faces

Ast provided = $\frac{1000 \times \frac{\pi}{4} \times 8^2}{160} = 314.159mm^2$)



150mm
3.5m
150mm
Free board = 300mm
10@300
3m
300mm
300mm
Bitumen
8@160

Design an RCC circular tank resting on ground with a flexible base and a spherical dome for a capacity of 500000 litres. The depth of storage is to be 4m. And free board is 200mm. Materials used are M20 grade concrete and Fe 415 HYSD bars. Draw the following,

(i) Cross section of the tank showing reinforcement details in dome, tank walls and floor slabs.

(ii) Plan of the tank showing reinforcement details.

DESIGN DATA

Capacity of tank = 500000 litres = 500 m^3

Depth of storage = 4 m

Grade – M20 & Fe415

Codes – IS 456 & IS 3370

SOLUTION

<u>Step 1 – Permissible stresses</u>

Permissible stress in direct tension (tank wall), σ_{ct}= 1.2 N/mm^2 (IS 3370 (Part II) – 1965, Table 1)

Permissible stress in direct tension (dome & ring beam), σ_{ct} = 2.8 N/mm^2 (IS 456 -2000, Pg 80)

Permissible stress in steel, σ_{st} = 0.6 fy = 150 N/mm^2 (IS 800)

Permissible stress in direct compression, σ_{cc} = 5 N/mm^2 (IS 456 – 2000, Table 21)

Permissible stress in bending compression, σ_{cbc} = 7 N/mm^2 (IS : 456 – 2000, Table 21)

m = 280/3 σ_{cbc} = 280/(3*7) = 13.333

k = 1/[1+ (σ_{st}/m σ_{cbc})] = 0.38

j = 1-k/3 = 0.87

Q = 0.5 σ_{cbc} k j = 1.16

<u>Step 2 – Dimensions of tank</u>

Depth of tank = 4 + 0.2 = 4.2 m

Volume of tank = (πD^2/4) * 4.2 = 500

$D = 12.93$ m

Central rise = (1/5 to 1/6) D = (1/6) D = 2.16 m

Radius of dome, $R^2 = [6.465^2 + (R-2.16)^2]$

$R = 10.755$ m

sin q = 6.465/10.755 = 0.6, cos q= 8.595/10.755 = 0.8, q = 36.87

Step 3 – Design of top spherical dome

Thickness of top dome, t = 100 mm (Assume)

Load calculation

Self weight = 0.1 * 25 = 2.5 kN/m^2

Live load & finishes = 2 kN/m^2

Total load, w = 4.5 kN/m^2

Meridional stress

Meridional thrust, T_1 = wR / 1+cos q = (4.5*10.755) / (1+0.8) = 26.888 kN/m

Meridional stress = T_1 / t = 26.888 / 100 = 0.269 $N/mm^2 < 5\ N/mm^2$

Hoop stress

Circumferential force, T_2 = wR{ cos q - (1 / [1+ cos q] }

= 4.5 * 10.755 *{0.8 - (1 / [1+ 0.8]} = 11.831 kN/m

Hoop stress = T_2 / t = 11.831/100 = 0.118 $N/mm^2 < 5\ N/mm^2$

Reinforcement

A_{st} = 0.3 % bd = (0.3/100) * 1000 * 100 = 300 mm^2

S = [1000 * (π/4) * 8^2] / 300 = 167.55 mm

Provide 8mm dia bars at 160mm c/c circumferentially & meridionally

Step 4 – Design of top ring beam

Reinforcement

Hoop tension, F_t = T_1 * cos q * D_t/2 = 26.888 * 0.8 * (12.93/2) = 139.065 kN

$A_{st} = F_t / \sigma_{st}$ = (139.065 * 10^3) / 150 = 927.1 mm^2

Provide 3 no's of 20 mm dia bars (A_{st} = 942.478 mm^2)

Minimum shear reinforcement is given by $A_{sv} / (b*S_v) = 0.4 / (0.87 * f_y)$

Provide 2 legged 6 mm dia stirrups at 250mm c/c.

Size

Permissible stress in ring beam = $F_t / (A_c + mA_{st})$

$$2.8 = (139.065*10^3) / (A_c + 13.33 * 942.478)$$

$$A_c = 37102.84$$

Provide top ring beam of size 200 x 200 mm

Step 5 – Design of tank walls

Horizontal reinforcement

Hoop tension, $F_t = g_w * H * D_t/2 = 9.81 * 4.2 * 12.93/2 = 266.371$ kN/m

$A_{st} = F_t / \sigma_{st} = (266.371 * 10^3) / 150 = 1775.807$ mm^2/m

A_{st} on one face = 1775.807 / 2 = 887.904 mm^2/m

Provide 16 mm dia bars, S = [1000* (π/4) *16^2] / 887.904 = 226.446 mm

Provide 16 mm dia bars at 200 mm c/c on both faces (A_{st}=2010.62 mm^2)

Size

Permissible stress in tank wall = $F_t / (A_c + mA_{st})$

$$1.2 = (266.371 *10^3) / (A_c + 13.33 * 2010.62)$$

$$A_c = 195174.269$$

$$1000 * t = 195174.269$$

Provide tank wall of thickness 200 mm throughout the tank wall

Vertical reinforcement

A_{st} = 0.3 % bd = (0.3/100) * 1000 * 200 = 600 mm^2

A_{st} on one face = 600/2 = 300 mm^2

Provide 10 mm dia bars, S = [1000 * (π/4) *10^2] / 300 = 261.8 mm

Provide 10 mm dia bars at 250 mm c/c on both faces (A_{st} = 628.319 mm^2)

Step 6 – Design of tank floor slab

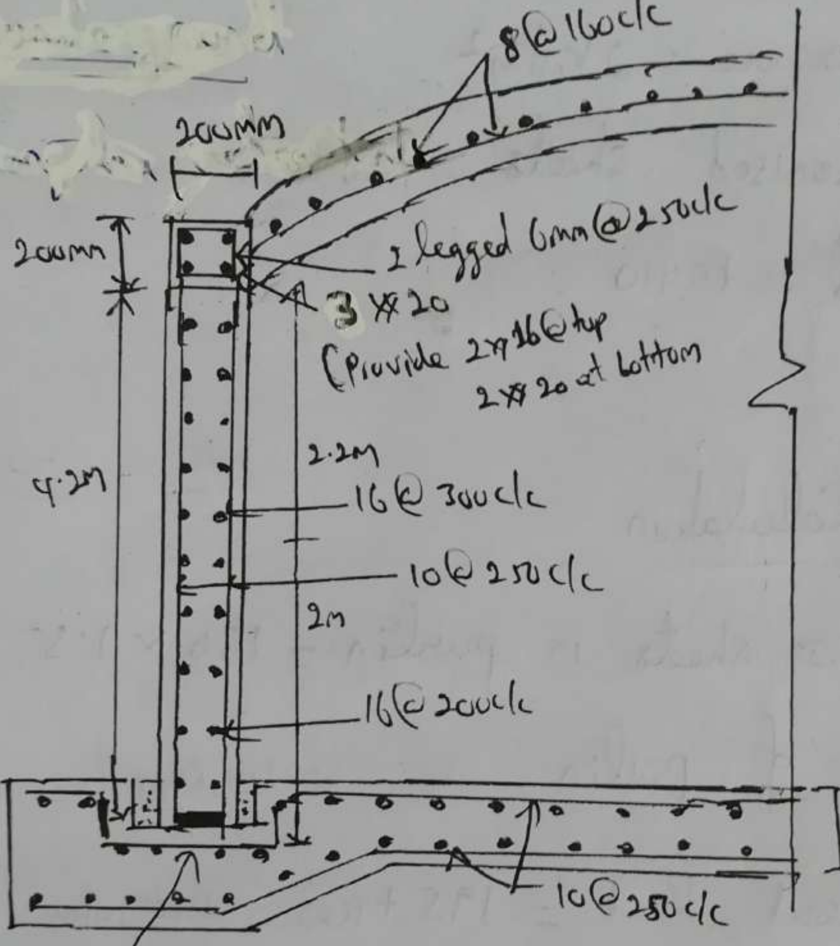
Reinforcement

A_{st} = 0.3 % bd = (0.3/100) * 1000 * 200 = 600 mm^2

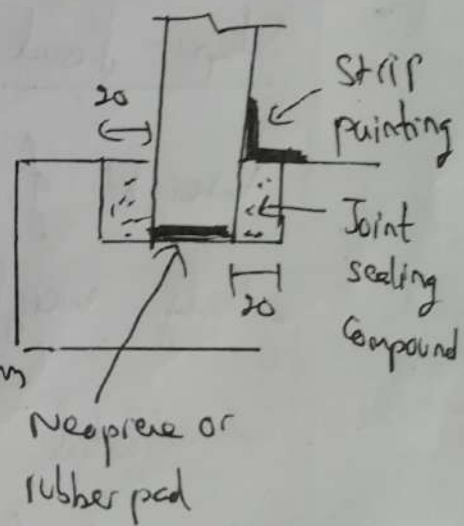
A_{st} on one face = 600/2 = 300 mm^2

Provide 10 mm dia bars, S = [1000 * (π/4) *10^2] / 300 = 261.8 mm

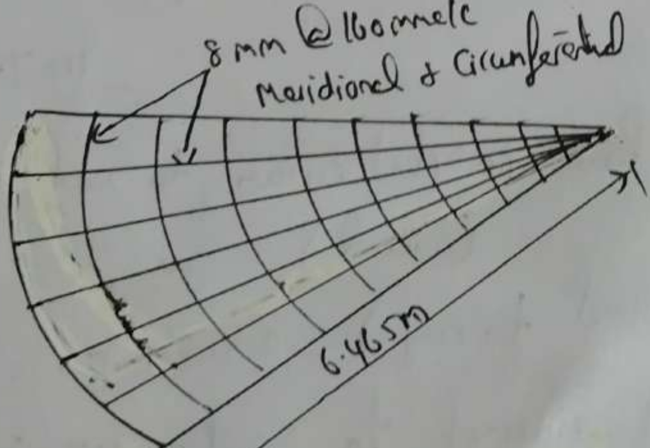
Provide 10 mm dia bars at 250 mm c/c on both faces (A_{st} = 628.319 mm^2)



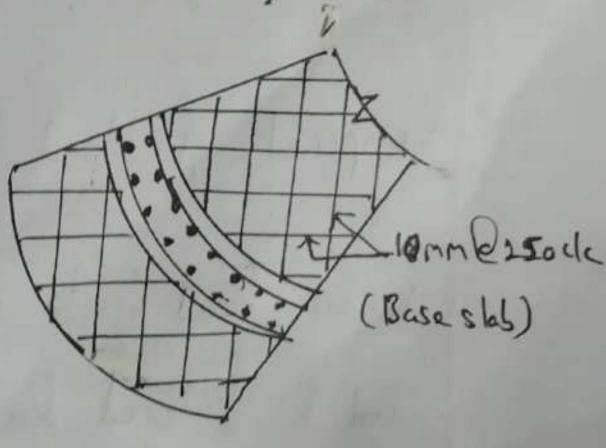
Sectional Elevation



Detail at A



Reinforcement in top dome



Reinforcement in base slab

Plan

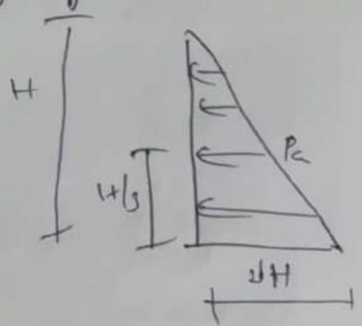
Underground water tanks

Underground water tanks are commonly used for storage of water recieved from water supply mains operating at low pressure. Underground water tanks. Underground water tanks are usually of two shapes circular shape and rectangular. For tanks of smaller capacity the cost of shuttering for circular tanks becomes high, hence rectangular tanks are used in such circumstances. Rectangular tanks are normally not used for large capacities since they are uneconomical and analysis is difficult.

When circular and rectangular tanks are situated underground, the walls of the tank to be designed for earth pressure as well as water pressure acting separately and also acting simultaneously. Similarly the floors of tanks are to be designed for hydrostatic water pressure (if water table is higher) acting upwards.

Earth pressure on tank walls

→ Tank wall supporting a dry or moist backfill of cohesionless soil. The intensity of active earth pressure at base of wall,



$$P_a = k_a \gamma H$$

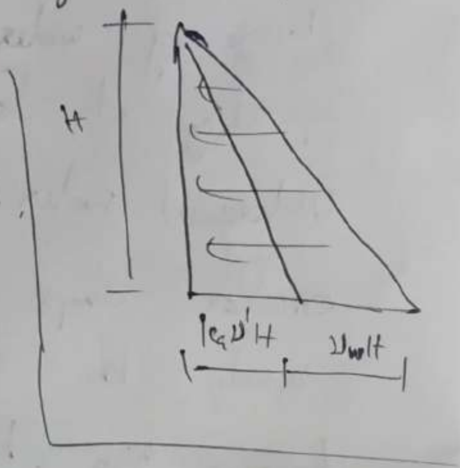
Total active earth pressure, $P_a = \frac{1}{2} k_a \gamma H \times H$

$$= \frac{1}{2} k_a \gamma H^2$$

acting at H/3 above base

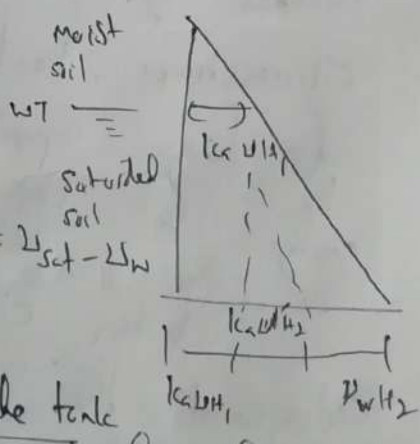
→ In case of submerged backfill (backfill saturated with water). intensity at base of wall

$$P_a = k_a \gamma' H + \gamma_w H$$



→ If backfill is partly submerged (ie) the backfill is moist to a depth H_1 below ground level and then it is submerged.

$$P_a = k_a \gamma H_1 + k_a \gamma' H_1 + \gamma_w H_2$$



where γ → Unit wt of soil

γ' → submerged unit weight $= \gamma_{sat} - \gamma_w$

γ_w → Unit weight of water

γ_{sat} → saturated unit wt

Uplift pressure on the floor on the tank

If the water table is below the floor level, the floor of the tank is designed for the load of tank wall, tank roof etc assumed to distributed evenly

The weight of water standing on the floor and the self weight of floor are assumed to pass directly to the foundation. If the sub soil water level (or ground water level) is above the floor level of the tank uplift pressure will be induced. When tank is empty it should not float. The weight of empty tank must exceed the flotation value to give a small factor of safety 1·1 to 1·25

Pb) Design an underground water tank 4m × 10m × 3m deep The sub soil consists of sand having angle of repose of 30° and saturated unit weight of 17 kN/m³. The water table is likely to rise upto ground level. Use M20 Concrete and Fe 415 HYSD bars. Take unit weight of water as 9·81 kN/m³

Solution

Step 1 - Permissible stresses

Permissible stress in steel under direct tension, $\sigma_{st} = 150 N/mm^2$

Permissible compressive stress in columns subjected to direct load (IS 3370 - Part II), $\sigma_{sc} = 175 N/mm^2$

Permissible stress in direct compression, $\sigma_{cc} = 5 N/mm^2$

" " bending ", $\sigma_{cbc} = 7 N/mm^2$

(IS 456, Table 21)

$$M = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m\sigma_{cbc}}} = \frac{1}{1 + \frac{150}{13.33 \times 7}} = 0.38$$

$j = 1 - \frac{k}{3} = 1 - \frac{0.38}{3} = 0.87$

$Q = 0.5\, \sigma_{cbc}\, kj = 0.5 \times 7 \times 0.38 \times 0.87$

$= 1.16$

Step 2 - Dimensions of tank

Length, L = 10m, Breadth, B = 4m, Depth, D = 3m

The base slab will be designed for uplift pressure as water table is above ground level.

$\frac{L}{B} = \frac{10}{4} = 2.5 > 2$

Long walls are designed as vertical cantilevers fixed at base and short walls are designed as horizontal slabs between long walls in top portion and bottom one metre height is designed as cantilever.

Step 3 - Design of long walls

(a) Tank empty with pressure of saturated soil from outside

(i) Main bars (Vertical rft)

In case of submerged backfill, intensity of p at base of wall,

$p_a = k_a \gamma' H + \gamma_w H$

where γ' → Submerged unit wt $= \gamma_{sat} - \gamma_w$

γ_w → Unit wt of water $= 9.81 kN/m^3$

γ_{sat} → Saturated unit wt $= 17 kN/m^3$

H → Height of tank = 3m

$k_a \rightarrow$ Coefficient of active earth pressure

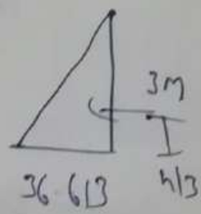
$$= \frac{1-\sin\phi}{1+\sin\phi}$$

$\gamma' = \gamma_{sat} - \gamma_w = 17 - 9.81 = 7.19 \text{ kN/m}^3$

$$k_a = \frac{1-\sin 30}{1+\sin 30} = 0.333$$

$P_a = (0.333 \times 7.19 \times 3) + (9.81 \times 3)$

$= 36.613 \text{ kN/m}^2$.



Total active earth pressure, $P_a = \frac{1}{2} \times 36.613 \times 3$

$= 54.92 \text{ kN/m}$

Moment $= P_a \times \frac{h}{3} = 54.92 \times \frac{3}{3} = 54.92 \text{ kNm}$

$M = Qbd^2$

$54.92 \times 10^6 = 1.16 \times 1000 \times d^2$

$\Rightarrow d = 217.589 \text{ mm}$

Provide effective depth, $d = 225$ mm, cover = 25 mm

Overall depth = eff depth + cover

$= 225 + 25$

$= 250$ mm

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{54.92 \times 10^6}{150 \times 0.87 \times 225} = 1870.413 \text{ mm}^2$$

Provide 16mm dia bars, spacing $= \frac{b \times a_{st}}{A_{st}}$

$$\text{Spacing} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{1870.413}$$

$= 107.496 mm$

Provide 16mm dia bars @ 100mm c/c on outside face at bottom of long wall as vertical reinforcement.

<u>Curtailment of reinforcement</u>

As BM is proportional to h^3, we have

$$\frac{Ast_h}{Ast_H} = \frac{h^3}{H^3}$$

$$\Rightarrow h = \left(\frac{Ast_h}{Ast_H}\right)^{1/3} \times H$$

→ Half the bars are curtailed, ∴ $Ast_h = \frac{1}{2} Ast_H$

$$h = \left(\frac{Ast_h}{2 Ast_h}\right)^{1/3} \times H$$

$$= \left(\frac{1}{2}\right)^{1/3} \times 3$$

$= 2.38 m$

Height from base $= 3 - 2.38 = 0.62m$

As per IS456, the bars are to be continued for a distance of

(i) $12\phi = 12 \times 16 = 192$ mm

(ii) $d = 225$mm

whichever is more

Hence half the bars are curtailed at a distance of

$0.62 + 0.225 = 0.845$mm from base

→ One fourth bars are curtailed, $A_{st\,h} = \frac{1}{4} A_{st\,H}$

$$h = \left(\frac{A_{st\,h}}{A_{st\,H}}\right)^{1/3} \times H$$

$$= \left(\frac{A_{st\,h}}{4 A_{st\,h}}\right)^{1/3} \times H$$

$$= \left(\frac{1}{4}\right)^{1/3} \times 3$$

$$= 1.89\,m$$

Height from base = 3 - 1.89 = 1.11 mm

As per IS 456, the bars are to be continued for a distance of

(i) $12\phi = 12 \times 16 = 192\,mm$

(ii) $d = 225\,mm$

whichever is more

Hence 3/4th of bars are curtailed at a distance of

1.11 + 0.225 = 1.335 m from base

$$\text{Min } A_{st} = \frac{0.3}{1}\%\ bD = \frac{0.3}{100} \times 1000 \times 250 = 750\,mm^2$$

$$1/2\,A_{st\,H} = \frac{1}{2} \times 1870.413 = 953.207\,mm^2 > 750\,mm^2$$

$$1/4\,A_{st\,H} = \frac{1}{4} \times 1870.413 = 467.603\,mm^2 \ngtr 750\,mm^2$$

Hence curtailment of half the bars is alone possible

∴ Spacing of 16 mm bars for 1/2 A_{st} = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{953.207}$

= 210.932 mm

Provide 16mm diameter bars at 100 mm c/c at base for a height of 0.845m and 16mm diameter at 200mm c/c above height of 0.845m as vertical reinforcement

(ii) Distribution rft (horizontal)

→ Minimum A_{st} = 0.3% bD = $\frac{0.3}{100} \times 1000 \times 250 = 750 mm^2$

A_{st} on each face = $\frac{750}{2} = 375 mm^2$ 375mm²

Provide 10mm dia bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{375} = 209.44$ mm

Provide 10mm diameter bars at 200mm c/c as ~~distribution bars~~ (horizontal rft) distribution bars

(iii) Direct compression in long walls

The earth pressure acting on short walls will cause compression in long walls because top portion of short walls act as slab supported on long walls. Bottom portion height h/4 (or) 1m whichever is greater

3/4 (or) 1m = 0.75 (or) 1m = 1m, So H = 3-1 = 2m

Intensity at base of wall, $P_a = K_a \gamma' H + \gamma_w H$

$P_a = (0.333 \times 7.19 \times 2) + (9.81 \times 2)$

$= 24.409 \; kN/m^2$

Direct compression developed on long walls $= P_a \times \frac{B}{2} = 24.409 \times \frac{4}{2}$

$= 48.818 \; kN$

$A_{st} = \frac{48.818 \times 10^3}{150}$

$= 325.453 mm^2$

which is less than area of distribution steel. Hence distribution steel will take care of direct compression

(b) Tank full with water, and no earth fill outside

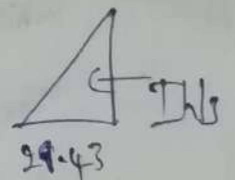
Main bars (vertical rft)

Intensity at base of wall, $P_a = \gamma_w H$

$= 9.81 \times 3$

$= 29.43\ kN$

Total active earth pressure $= \frac{1}{2} \times 29.43 \times 3$



$P_a = 44.145\ kN/m$

Moment $= P_a \times \frac{h}{3} = 44.145 \times \frac{3}{3} = 44.145\ kNm$

$A_{st} = \frac{M}{\sigma_{st}\, j\, d} = \frac{44.145 \times 10^6}{150 \times 0.87 \times 225} = 1503.448\ mm^2$

Provide 16mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1503.448} = 133.734\ mm$

Provide 16mm diameter bars at 130mm c/c at inside face at bottom of long wall as vertical reinforcement

<u>Curtailment of reinforcement</u>

As BM is proportional to h^3 we have,

$\frac{A_{st\,h}}{A_{st\,H}} = \frac{h^3}{H^3}$

$\Rightarrow h = \left(\frac{A_{st\,h}}{A_{st\,H}}\right)^{1/3} \times H$

→ Half the bars are curtailed, $A_{st\,h} = \frac{1}{2} A_{st\,H}$

$h = \left(\frac{A_{st\,h}}{2 A_{st\,h}}\right)^{1/3} \times 3$

$= \left(\frac{1}{2}\right)^{1/3} \times 3$

$= 2.38\ m$

Height from base $= 3 - 2.38 = 0.62\ m$

As per IS 456, the bars are to be continued for a distance of

(i) $12\phi = 12 \times 16 = 192\ mm$

(ii) $d = 225\ mm$

whichever is more

Hence half of bars are curtailed at a distance of 0.62 + 0.225 = 0.845m from base

$\text{Min } A_{st} = 0.3\%\, bD = \frac{0.3}{100} \times 1000 \times 250 = 750\text{mm}^2$

$\frac{1}{2} A_{st_H} = \frac{1}{2} \times 1503.448 = 751.724\text{mm}^2 > 750\text{mm}^2$

Spacing of 16mm dia bars for $\frac{1}{2} A_{st}$ $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{751.724}$

$= 267.468\text{mm}$

Provide 16mm diameter bars at 130mm c/c at base for a height of 0.845m and 16mm dia bars at 260mm c/c above height of 0.845m as vertical reinforcement at inside face

(PTO.)

(ii) Distribution rft (Horizontal rft)

Minimum $A_{st} = 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 250 = 750mm^2$

A_{st} on each face $= \frac{750}{2} = 375mm^2$

Provide 10mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 10^2}{375} = 209.44mm$

Provide 10mm diameter bars at 200mm c/c as horizontal reinforcement (distribution rft)

(iii) Direct tension in long walls

Since the top portion of short walls act as slab supported on long walls, the water pressure acting on short walls will cause tension in long walls

Bottom portion height h/4 (or) 1m whichever is greater

3/4 (or) 1m = 0.75 (or) 1m = 1m, So H = 3-1 = 2m

Intensity at base of wall, $P_a = \gamma_w H = 9.81 \times 2 = 19.62 \, kN/m^2$

Direct ~~compression~~ tension developed on long walls $= P_a \times \frac{B}{2} = 19.62 \times \frac{4}{2}$

$= 39.24 kN$

$A_{st} = \frac{39.24 \times 10^3}{150} = 261.6mm^2$

which is less than area of distribution steel. Hence distribution steel will take direct tension

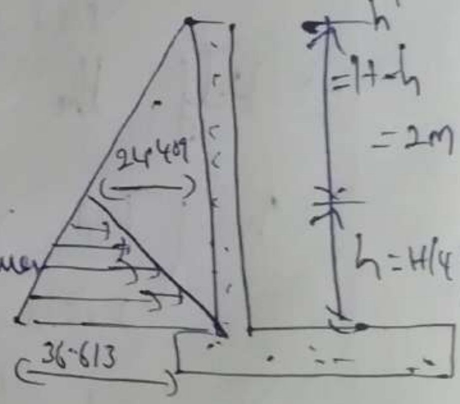
(PTO)

Step 4 - Design of short walls

(a) Tank empty with pressure of saturated soil from outside

(i) Top portion

Bottom portion height $h/4$ or 1m whichever is greater



$h = 3/4$ (or) 1m whichever is greater = 1m

Intensity at base of wall, $P_a = k_a \gamma' h' + \gamma_w h'$

$$P_a = (0.333 \times 7.19 \times 2) + (9.81 \times 2)$$

$$= 24.409 \ kN/m^2$$

M_f (Fixed moment at supports) $= \dfrac{P_a B^2}{12} = \dfrac{24.409 \times 4^2}{12} = 32.545 \ kNm$

BM at Centre = $\frac{P_a B^2}{8} = \frac{24.407 \times 4^2}{8} = 48.818$ KNm

Net moment at centre = BM - M_f = 48.818 - 32.545

= 16.273 KNm

At supports, $A_{st} = \frac{M}{\sigma_{st} j d}$

$= \frac{32.545 \times 10^6}{150 \times 0.87 \times 225}$

$= 1108.387 mm^2$

Using 12 mm dia bars, spacing = $\frac{b \times a_{st}}{A_{st}}$

$= \frac{1000 \times \frac{\pi}{4} \times 12^2}{1108.387}$

= 102.038 MM

Provide 12mm diameter bars at 100mm c/c at outer face at 2m below the top as distribution st (horizontal)

At mid span, $A_{st} = \frac{M}{\sigma_{st} j d}$

$= \frac{16.273 \times 10^6}{150 \times 0.87 \times 225}$

$= 554.21 mm^2$

Using 12 mm dia bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{554.21}$

= 204.069 mm

Hence provide 12mm diameter bars at ~~outside~~ spacing 200mm at inner face in ~~vertical direction~~ at 2m below the top as distribution st (horizontal)

(ii) <u>Bottom portion</u>

Intensity of ~~earth pressure at bottom of~~ pressure at base of wall,

$$P_h = k_a \gamma' H + \gamma_w H$$

$$\gamma' = \gamma_{sat} - \gamma_w = 17 - 9.81 = 7.19\ kN/m^3$$

P_a = 3m at base from top

$P_a = (0.333 \times 7.19 \times 3) + (9.81 \times 3) = 36.613\ kN/m^2$

Total pressure $= \frac{1}{2} \times 36.613 \times 1 = 18.307\ kN$

Moment $= 18.307 \times \frac{1}{3} = 6.102\ kNm$

$A_{st} = \frac{M}{\sigma_{st}\, j\, d}$

$= \frac{6.102 \times 10^6}{150 \times 0.87 \times 225}$

$= 207.816\ mm^2$

Min $A_{st} = 0.3\%\ bD = \frac{0.3}{100} \times 1200 \times 250 = 750\ mm^2$

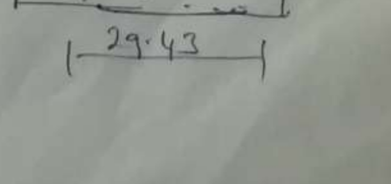
Spacing of 12mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{750}$

$= 150.796\ mm$

Provide 12mm diameter bars at 150mm c/c at outside face in the vertical direction for bottom 1m height. This space was doubled for upper portion. Hence provide 12mm diameter bars at 300mm c/c at a height above 1m from bottom

(b) Tank full with water and no earthfill outside

(i) Top portion



Bottom portion height h/4 or 1m whichever is greater

$h = 3/4$ (or) 1m = 1m

Intensity $= \gamma_w h' = 9.81 \times 2 = 19.62\ kN/m^2$

M_f (Fixed moment at supports) $= \frac{p B^2}{12} = \frac{19.62 \times 4^2}{12} = 26.16\ kNm$

BM at centre $= \frac{P_a B^2}{8} = \frac{19.62 \times 4^2}{8} = 39.24$ kNm

Net moment at centre $= BM - M_f = 39.24 - 26.16 = 13.08$ kNm

At supports, $A_{st} = \frac{M}{\sigma_{st} j d}$

$= \frac{26.16 \times 10^6}{150 \times 0.87 \times 225}$

$= 890.932 mm^2$

Using 12 mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{890.932} = 126.943$ mm

Provide 12 mm dia bars at 120 mm c/c at outer face at 2m below the top as distribution reinforcement (horizontal)

At mid span, $A_{st} = \frac{M}{\sigma_{st} j d}$

$= \frac{13.08 \times 10^6}{150 \times 0.87 \times 225}$

$= 445.466 mm^2$

Using 12mm dia bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{445.466} = 253.885$ mm

Hence provide 12mm diameter bars at 250mm c/c at inner face at 2m below top as distribution reinforcement (horizontal)

(ii) ~~Distri~~ Bottom portion

Intensity of pressure at base of wall, $P_a = w H$

$= 9.81 \times 3$

$= 29.43$ kN/m²

~~Moment $= \frac{1 \times 29.43}{2}$ x~~ Total pressure $= \frac{1}{2} \times 29.43 \times 1 = 14.715$ kN

Moment $= 14.715 \times \frac{1}{3} = 4.905$ kNm

$$A_{st} = \frac{M}{\sigma_{st} j d}$$

$$= \frac{4.905 \times 10^6}{150 \times 0.87 \times 225}$$

$$= 167.05\ mm^2$$

Min $A_{st} = 0.3\%\ bD = \frac{0.3}{100} \times 1000 \times 250 = 750\ mm^2$.

Provide 12mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{750}$

$= 150.796\ mm$

Provide 12mm diameter bars at 150mm c/c at inner face in the vertical direction for bottom 1m height. This space was doubled for upper portion. Hence provide 12mm diameter bars at 300mm c/c at height 1m from bottom

<u>Step 5 - Design of top slab</u>

$\frac{L}{B} = \frac{10}{4} = 2.5 > 2$

Hence the slab will be designed as one way slab

<u>Load calculation</u>

Assuming slab thickness as 15cm, $(d = 150 - 25 = 125\ mm)$

Self weight of slab $= 0.15 \times 1 \times 25 = 3.75\ kN/m^2$

Live load (assume) $= 2\ kN/m^2$

Total load, w $= 5.75\ kN/m^2$

<u>Effective span</u>

c/c of supports $= 4 + \frac{0.25}{2} + \frac{0.25}{2} = 4.25\ m$

$l_{eff} = 4.25\ m$

Moment

$$M = \frac{wl^2}{8} = \frac{5.75\times4.25^2}{8} = 12.982 \text{ KN m}$$

Check for depth

$$M = Qbd^2$$

$$12.982\times10^6 = 1.16\times1000\times d^2$$

$$\Rightarrow d = 105.789\text{mm} < 125\text{mm} \text{ (Hence safe)}$$

Reinforcement

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{12.982\times10^6}{150\times0.87\times125} = 795.831\text{mm}^2$$

$$\text{Min } A_{st} = 0.3\% \, bD = \frac{0.3}{100}\times1000\times150 = 450\text{mm}^2$$

Provide 12mm diameter bars, spacing $= \frac{1000\times\frac{\pi}{4}\times12^2}{795.831} = 142.112\text{mm}$

Provide 12mm diameter bars at 140mm c/c as Main bars

Distributor bars, provide 10mm bars, spacing $= \frac{1000\times\frac{\pi}{4}\times10^2}{450} = 174.533\text{mm}$

Provide 10mm diameter bars at 170mm c/c as distribution bars

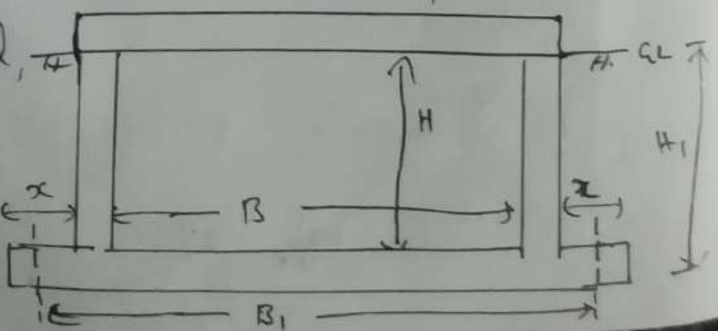
Step 6 - Design of bottom slab

If there were no sub soil water only nominal (min) reinforcement will be required. Because of saturated soil there will be uplift pressure on bottom slab.

Assuming thickness of bottom slab as 300mm,

height upto ground level, H

$$H_1 = 3+0.3 = 3.3\text{m}$$



Uplift pressure on bottom slab, ∴ $p_u = \gamma_w H_1 = 9.81 \times 3.3$

$= 32.373 \text{ kN/m}^2$

The whole tank must be checked against flotation when the tank is empty.

Total upward flotation force, $P_u = p_u \times B \times L$

$= 32.373 \times 4 \times 10$

$= 1294.92 \text{ kN}$

Total downward force is computed from weight of tank,

→ Weight of base slab $= 4 \times 10 \times 0.3 \times 25 = 300 \text{ kN}$

→ Weight of long wall $= 0.25 \times 10 \times 3 \times 25 \times 2 = 375 \text{ kN}$

→ Weight of short wall $= 0.25 \times 4 \times 3 \times 25 \times 2 = 150 \text{ kN}$

→ Weight of roof slab $= 4 \times 10 \times 0.15 \times 25 = 150 \text{ kN}$

975 kN

Downward force is less than the upward flotation force. Hence provide projections of base slab beyond the face of vertical walls by a distance 'x' all around so that weight of soil column supported by the projections will provide additional downward force. It is assumed that if the tank is floated, the earth would rupture on vertical planes shown by dotted lines.

→ Weight of soil supported by projection 'x' $=$ Vol × unit wt

$= 2(L+B) \times H \times x \times \gamma_{soil}$

$= 2(10+4) \times 3 \times x \times 17$

$= 1428x \text{ kN}$

→ Weight of base slab $= [4+(2\times0.25)+2x] \times [10+(2\times0.25)+2x]$

$\times 0.3\times25$

$= (4.5+2x)(10.5+2x)\times7.5$ kN

→ Weight of long wall $= 375$ kN

→ Weight of short wall $= 150$ kN

→ Weight of roof slab $= 150$ kN

~~Total downward force $= 2103 + (4.5+2x)(10.5+2x)7.5$~~

~~$= 2103 + (47.25+9x+21x+4x^2)7.5$~~

~~$= 2103 + 354.375 + 225x + 30x^2$~~

~~Total downward force $= 2457.325 + 225x + 30x^2$ — ①~~

Total downward force $= 1428x + (4.5+2x)(10.5+2x)7.5$

$+ 375 + 150 + 150$

$= 1428x + (47.25+9x+21x+4x^2)7.5$

$+ 675$

$= 1428x + 354.375 + 225x + 30x^2$

$+ 675$

Total downward force $= 30x^2 + 1653x + 1029.375$ — ①

Total uplift force (or) upward floatation force $= 32.373[4+(2\times0.25)+2x] +$

$[10+(2\times0.25)+2x]$

$= 32.373[4.5+2x][10.5+2x]$

$= 32.373[47.25+9x+21x+4x^2]$

Total upward force $= 1529.624 + 971.19x + 129.492x^2$ — ②

As the bottom slab is projected on its sides, height for calculating uplift pressure 'H_1' gets reduced to (3.3 − 0.3) 3m.

∴ Uplift pressure on bottom slab, p_u = 1 × 9.81 × 3

$= 29.43 \ kN/m^2$

Total upward flotation force, $P_u = p_u \times B \times L$

$= 29.43 [4 + (2 \times 0.25) + 2x][10 + (2 \times 0.25) + 2x]$

$= 29.43 [4.5 + 2x][10.5 + 2x]$

$= 29.43 [47.25 + 9x + 21x + 4x^2]$

Total upward force $= 1390.568 + 882.9x + 117.72x^2$ — (2)

Equating (1) & (2)

$\Rightarrow 30x^2 + 1653x + 1029.375 = 1390.568 + 882.9x + 117.72x^2$

$87.72x^2 - 770.1x + 361.193 = 0$

$x = 8.282m, 0.497m$

Taking $x = 0.497m$ (least) $\simeq 0.5m$

Check for flotation

Sub value of x in eqn (1) & (2).

Total downward force = $30x^2 + 1653x + 1029.375$

$= (30 \times 0.5^2) + (1653 \times 0.5) + 1029.375$

$= 1863.375$ KN

Total upward force = $1390.568 + 882.9x + 117.72x^2$

$= 1390.568 + (882.9 \times 0.5) + (117.72 \times 0.5^2)$

$= 1861.448$ KN

Factor of safety against flotation = $\frac{\text{Total downward force}}{\text{Total upward force}}$

$= \frac{1863.375}{1861.448}$

$= 1.001$

A factor of safety of 1.1 is needed

$\therefore$ FOS $= 1.1 = \frac{30x^2 + 1653x + 1029.375}{1390.568 + 882.9x + 117.72x^2}$

$x = 0.836\text{m} \simeq 0.85\text{m}$

Now, Total downward force $= (30 \times 0.85^2) + (1653 \times 0.85) + 1029.375$

$= 2456.1$ KN

Total upward force $= 1390.568 + (882.9 \times 0.85) + (117.72 \times 0.85^2)$

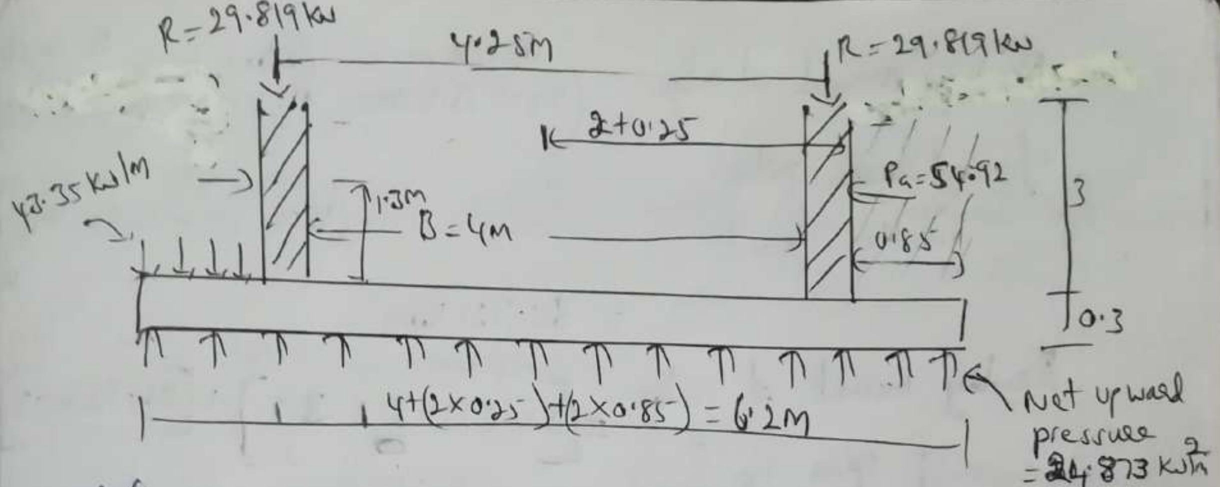
$= 2226.086$ KN

F/S against flotation $= \frac{2456.1}{2226.086} = 1.103$

Hence ok

The base slab will be designed by one way slab.

Considering 1 metre length of slab,



→ Uplift pressure, $P_u = \gamma_w H_1 = 9.81 \times 3.3 = 32.373\ kN/m^2$

Self wt of slab (per m) $= 1 \times 1 \times 0.3 \times 25 = 7.5\ kN/m^2$ (L B t)

∴ Net upward pressure $= 32.373 - 7.5 = 24.873\ kN/m^2$

→ Weight of wall per m $= 0.25 \times 3 \times 1 \times 25 = 18.75\ kN/m^2$ (t H L)

→ Weight of roof slab transferred to each wall per m $= 0.15 \times (2+0.25) \times 1 \times 25 = 8.438\ kN/m^2$ (t B L)

→ Weight of earth on projection $= 0.85 \times 3 \times 1 \times 17 = 43.35\ kN/m$ (B H L γ_{sat})

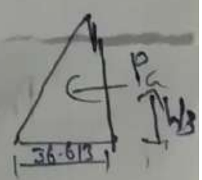
Net unbalanced force/m = Total upward force − Weight of tank

$= (32.373 \times 6.2 \times 1) - [2 \times (18.75 + 8.438 + 43.35)]$ (B L)

$= 59.637\ kN$

Reaction on each wall $= 59.637/2 = 29.819\ kN$

→ Soil pressure from sides, Intensity, $P_a = K_a \gamma' H + \gamma_w H$



where $\gamma' = \gamma_{sat} - \gamma_w = 17 - 9.81 = 7.19\ kN/m^3$

$P_a = (0.333 \times 7.19 \times 3) + (9.81 \times 3) = 36.613\ kN/m^2$

Total active earth pressure $= \frac{1}{2} \times 36.613 \times 3$

$= 54.92\ kN/m$ acting at $h/3$

from base $0.3 + \frac{3}{3} = 1.3\ m$

Bending moment at edge of cantilever portion (bottom face)

$$= \left(24.873 \times 0.85 \times \frac{0.85}{2}\right) + (54.92 \times 1.3) - \left(43.35 \times 0.85 \times \frac{0.85}{2}\right)$$

(up pressure, d/s, centroid; Pa × d/s; soil on projection, d/s, centroid)

$$= 64.721\ KNM$$

Bending moment at centre of span (top face)

$$= \left[24.873 \times \frac{3.2}{2} \times \frac{3.2}{2}\right] + \left[54.92 \times 1.3\right] - \left[43.35 \times 0.85 \times \left(\frac{0.85}{2} + 2.25\right)\right] - \left(29.819 \times \frac{4.25}{2}\right) - \left(18.75 \times \frac{4.25}{2}\right) - \left(8.438 \times \frac{4.25}{2}\right)$$

(up pressure, d/s, centroid; Pa, d/s; soil on projection, d/s, centroid; R, d/s; wall, d/s)

$$= 119.515 + 71.396 - 98.567 - 63.365 - 39.844 - 17.931$$

$$= -28.796\ KNM$$

→ $M = Qbd^2$, Taking max moment, $64.721 \times 10^6 = 1.16 \times 1000 \times d^2$

$d = 236.21\ mm$

Take $d = 250\ mm$, $D = d + cover = 250 + 50 = 300\ mm$ (Same as assumed)

→ A_{st} (bottom face) $= \frac{M}{\sigma_{st} j d} = \frac{64.721 \times 10^6}{150 \times 0.87 \times 250} = 1983.785\ mm^2$

Provide 16mm dia bars, Spacing: $\frac{1000 \times \frac{\pi}{4} \times 16^2}{1983.785} = 101.353\ mm$

Provide 16mm diameter bars at 100mm c/c at bottom face

→ A_{st} (top face) $= \frac{M}{\sigma_{st} j d} = \frac{28.796 \times 10^6}{150 \times 0.87 \times 250} = 882.636\ mm$

Provide 12mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{882.636} = 128.136\ MM$

Provide 12mm diameter bars at 120 mm c/c at top face

→ Distribution reinforcement $= 0.3\% \, bD = \frac{0.3}{100} \times 1000 \times 300 = 900\ mm^2$

Long walls

Summary

Tank empty with soil outside – outside face – vertical rft – pressure (γ_w, γ')
– horizontal rft – min Ast

Tank full with water and no earth outside – inner face – vertical rft – pressure (γ_w)
– horizontal rft – min Ast

Short walls

Tank empty with soil outside – top portion – support moment – outer face – ~~horizontal~~ horizontal
– midspan – inner face – ~~horizontal~~ horizontal
– bottom portion – cantilever – vertical – outside

Tank fill with water and no earth outside – – do –
– bottom portion – cantilever – vertical – inside

Ex No. 4 INTZE TYPE WATER TANK

Design the Intze type water tank with capacity of one million litres, supported on an elevated tower comprising of 8 columns. The base of the tank is 16 m above the ground level and the depth of the foundation is 1 m below the ground level. Adopt M20 grade concrete and Fe 415 Steel.

DESIGN DATA

- Capacity of tank = 1 million litres = 1000 m^3
- Base of tank = 16 m above ground level
- Depth of foundation = 1 m above ground level
- Grade – M20 & Fe415
- Codes – IS 456 & IS 3370

SOLUTION

Step 1 – Permissible stresses

Permissible stress in direct tension (tank wall), σ_{ct}= 1.2 N/mm^2 (IS 3370 (Part II) – 1965, Table 1)

Permissible stress in direct tension (dome & ring beam), σ_{ct} = 2.8 N/mm^2 (IS 456 -2000, Pg 80)

Permissible stress in steel, σ_{st} = 0.6 fy = 150 N/mm^2 (IS 800)

Permissible stress in direct compression, σ_{cc} = 5 N/mm^2 (IS 456 – 2000, Table 21)

Permissible stress in bending compression, σ_{cbc} = 7 N/mm^2 (IS : 456 – 2000, Table 21)

m = 280/3 σ_{cbc} = 280/(3*7) = 13.333

k = 1/[1+ (σ_{st}/m σ_{cbc})] = 0.38

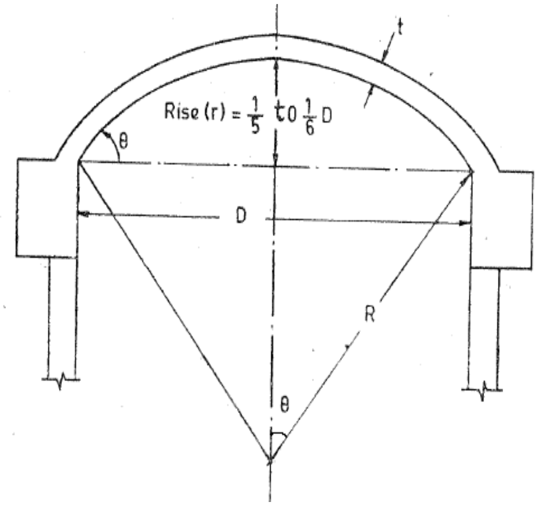
j = 1-k/3 = 0.87

Q = 0.5 σ_{cbc} k j = 1.16

Step 2 – Dimensions of tank

- Depth of tank = 0.65 D to 0.75 D = 0.75 D_t where 'D_t' is the diameter of tank at top

  Volume of tank = (πD_t^2/4) * 0.75 D_t = 1000



$D_t = 12m$

- Depth of tank = 0.75 D_t = 9 m
- Central rise = (1/5 to 1/6) D_t = (1/6) D_t = 2 m
- Radius of dome, $R^2 = [6^2 + (R-2)^2]$

$R = 10$ m

- sin θ = 6/10 = 0.6, cos θ= 8/10 = 0.8, θ = 36.87

Step 3 – Design of top spherical dome

- Thickness of top dome, t = 100 mm (Assume)
- Load calculation

Self weight = 0.1 * 25 = 2.5 kN/m^2

Live load & finishes = 2 kN/m^2

Total load, w = 4.5 kN/m^2

- Meridional stress

Meridional thrust, T_1 = wR / 1+cos θ = (4.5*10) / (1+0.8) = 25 kN/m

Meridional stress = T_1 / t = 25 / 100 = 0.25 $N/mm^2 < 5\ N/mm^2$

- Hoop stress

Circumferential force, T_2 = wR{ cos θ - (1 / [1+ cos θ] }

= 4.5 * 10 *{0.8 - (1 / [1+ 0.8]} = 11 kN/m

Hoop stress = T_2 / t = 11/100 = 0.11 $N/mm^2 < 5\ N/mm^2$

- Reinforcement

A_{st} = 0.3 % bd = (0.3/100) * 1000 * 100 = 300 mm^2

S = [1000 * (π/4) * 8^2] / 300 = 167.55 mm

Provide 8mm dia bars at 160mm c/c circumferentially & meridionally

Step 4 – Design of top ring beam

- Reinforcement

✓ Hoop tension, $F_t = T_1 * \cos\theta * D_t/2 = 25 * 0.8 * 6 = 120$ kN

✓ $A_{st} = F_t / \sigma_{st} = (120 * 10^3) / 150 = 800\ mm^2$

Provide 4 no's of 16 mm dia bars ($A_{st} = 804.25\ mm^2$)

✓ Minimum shear reinforcement is given by $A_{sv} / (b*S_v) = 0.4 / (0.87 * f_y)$

Provide 2 legged 6 mm dia stirrups at 250mm c/c.

- <u>Size</u>

Permissible stress in ring beam = $F_t / (A_c + mA_{st})$

$$2.8 = (120*10^3) / (A_c + 13.33 * 804.25)$$

$$A_c = 32136.49$$

Provide top ring beam of size 200 x 200 mm

<u>Step 5 – Design of tank walls</u>

- <u>Horizontal reinforcement</u>

✓ Hoop tension, $F_t = \gamma_w * H * D_t/2 = 9.81 * 9 * 6 = 529.74$ kN/m

✓ $A_{st} = F_t / \sigma_{st} = (529.74 * 10^3) / 150 = 3531.6\ mm^2/m$

A_{st} on one face = 3531.6 / 2 = 1765.8 mm^2/m

Provide 20 mm dia bars, S = $[1000* (\pi/4) *20^2] / 1765.8 = 177.91$mm

Provide 20 mm dia bars at 170mm c/c on both faces (A_{st}=3695.99 mm^2)

Height (from top)	**Height (range from top)**	**F_t**	**A_{st} on one face**	**A_{st} provided**
3	0-3	176.58	588.6	12 @ 190
6	3-6	353.16	1177.2	16 @ 170
9	6-9	529.74	1765.8	20 @ 170

- <u>Size</u>

Permissible stress in tank wall = $F_t / (A_c + mA_{st})$

$$1.2 = (529.74 *10^3) / (A_c + 13.33 * 3695.99)$$

$$A_c = 392182.45$$

$1000 * t = 392182.45$

Provide tank wall of thickness 400 mm at bottom and gradually reduced to 200 mm at top.

- Vertical reinforcement

✓ A_{st} = 0.3 % bd = (0.3/100) * 1000 * 300 = 900 mm^2

✓ A_{st} on one face = 900/2 = 450 mm^2

Provide 10 mm dia bars, S = [1000 * (π/4) *10^2] / 450 = 174.53 mm

Provide 10 mm dia bars at 170 mm c/c on both faces (A_{st} = 923.99 mm^2)

Step 6 – Design of bottom ring beam

- Reinforcement

✓ Load due to top spherical dome = T * sin θ = 25*0.6 = 15 kN/m

Load due to top ring beam = 0.2 * 0.2 * 25 = 1 kN/m

Load due to tank wall = 0.3 * 9 * 25 = 67.5 kN/m

Assuming size of bottom ring beam as 1.2m x 0.6m, load due to bottom ring beam

= 1.2 * 0.6 * 25 = 18 kN/m

Total vertical load = 101.5 kN/m

Total horizontal load = 101.5 * cot 45 = 101.5 kN/m

✓ Hoop tension due to vertical load, F_t = 101.5 * D_t/2 = 609 kN

✓ Hoop tension due to water, $F_t = \gamma_w * H * h * D_t / 2$ = 9.81 * 9 * 0.6 * 6 = 317.84 kN

✓ Hoop tension, F_t = 609 + 317.84 = 926.84 kN

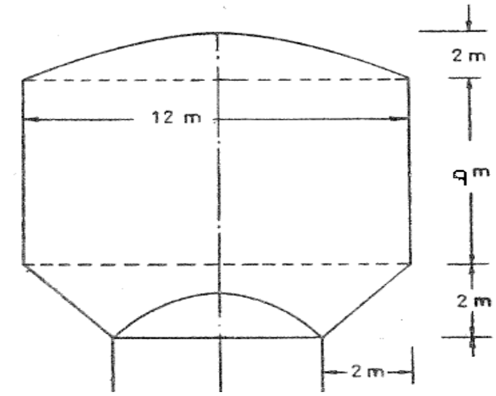
✓ $A_{st} = F_t / \sigma_{st}$ = (926.84*10^3)/150 = 6178.93 mm^2

Provide 8 no's of 32 mm dia bars (A_{st} = 6433.98 mm^2)

✓ Minimum shear reinforcement is given by $A_{sv}/(b*S_v) = 0.4 / (0.87*f_y)$

Provide 2 legged 8 mm dia stirrups at 150mm c/c.

Step 7 – Design of conical dome



- Dimensions

Length of bottom of tank = 12-2-2 = 8 m

Average dia of conical dome, D_c = (12+8) / 2 = 10 m

Average depth of water, H_c = 9 + (2/2) = 10 m

- Load Calculation

Weight of water above conical dome = $(\pi D_c * H_c * 2) * 9.81 = 6163.8$kN

Assuming thickness of conical dome as 600 mm, self weight of conical dome

$= (\pi D_c * 0.6 * \sqrt{(2^2 + 2^2)}) * 25 = 1332.865$kN

Total horizontal load = $101.5 * \pi D_t = 101.5 * \pi * 12 = 3826.46$ kN

Total load = 11323.125kN

Load/m length=11323.125 / $(\pi * D_b)$ = 11323.125 / $(\pi*8)$ = 450.533 kN/m

- Meridional stress

Meridional thrust, T_1 = 450.533 * cosec 45 = 637.15 kN/m

Meridional stress=T_1 / t=637.15/600 =1.062 N/mm^2 < 5 N/mm^2

- Horizontal Reinforcement

✓ Hoop tension, $F_t = (p \operatorname{cosec} \theta + q \cot \theta) * D_t/2$

Where p = Water pressure = $9.81 * D_b = 9.81 * 8 = 78.4$ kN/m^2

q = Self weight of conical dome = 0.6 * 25 = 15 kN/m^2

F_t = (78.4 cosec 45 + 15 cot 45) * 12/2 = 755.246 kN

✓ $A_{st} = F_t / \sigma_{st} = (755.246 * 10^3) / 150 = 5034.973$ mm^2

A_{st} on one face = 5034.973 / 2 = 2517.487 mm^2

Provide 20 mm dia bars, S= $[1000 * (\pi/4) * 20^2]$ / 2517.487= 124.791 mm

Provide 20 mm dia bars at 120mm c/c on both faces (A_{st}=5235.988 mm^2)

- Vertical reinforcement

✓ A_{st} = 0.3 % bd = (0.3/100) * 1000 * 600 = 1800 mm^2

✓ A_{st} on one face = 1800/2 = 900 mm^2

Provide 12 mm dia bars, S = [1000 * (π/4) *12^2] / 900 = 125.66 mm

Provide 12 mm dia bars at 120mm c/c on both faces (A_{st} = 1884.956 mm^2)

- Stress check

Permissible stress in conical dome = F_t / (A_c + mA_{st})

= (755.246 * 10^3) / (600*1000 + 13.33*5235.988)

= 1.128 N/mm^2 < 2.8 N/mm^2

Step 8 – Design of bottom spherical dome

- Diameter at bottom, D_b = 8m

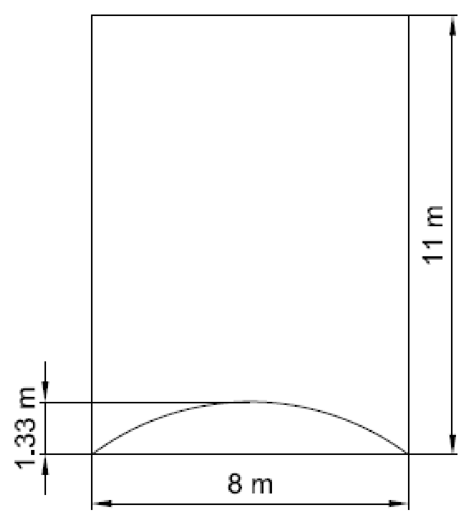
  Central rise = (1/5 to 1/6) D_b = (1/6) D_b = 1.33m

  Radius of dome, R^2 = [4^2+$(R-1.33)^2$]

  R = 6.68m

  sin θ = 4/6.68 = 0.6, cos θ =5.35 / 6.68 = 0.8, θ= 36.87

- Thickness of bottom dome = 300mm (Assume)



- Load calculation

Self weight = (2π* 6.68 * 1.33) * 0.3 * 25 = 418.667 kN

Volume of water = [π r^2h – (2/3) * π r^2h]

= [π* 4^2 *11 – (2/3) * π * 4^2 *1.33] = 508.352m^3

Weight of water = 508.352 * 9.81 = 4986.933 kN

Total load = 5405.6 kN

Load/m^2 = 5405.6/(π*4^2) = 107.541 kN/m^2

- Meridional stress

Meridional thrust, T_1 = wR / 1+cos θ = (107.541 * 6.68) / (1+0.8) = 399.097 kN/m

Meridional stress= T / t = 399.097/300 = 1.33 N/mm^2 < 5 N/mm^2

- Hoop stress

Circumferential force, T_2 = wR{cos θ-(1/[1+ cos θ]}

= 107.541 * 6.68 *{0.8-(1/[1+ 0.8]} = 175.603 kN/m

Hoop stress = 175.603/300 = 0.585 N/mm^2 < 5 N/mm^2

- Reinforcement

A_{st} = 0.3 % bd = (0.3/100) * 1000 * 300 = 900 mm^2

S = [1000 * (π/4) *12^2] / 900 = 125.66 mm

Provide 12mm dia bars at 120mm c/c circumferentially & meridionally

Step 9 – Design of girder

- Thrust from conical dome, T_1 = 637.15 kN/m, α = 45

  Thrust from bottom spherical dome, T_2 = 399.097 kN/m, β = 36.87

- Stress check

  Horizontal force = T_1 cos α - T_2 cos β

  = 131.256 kN/m

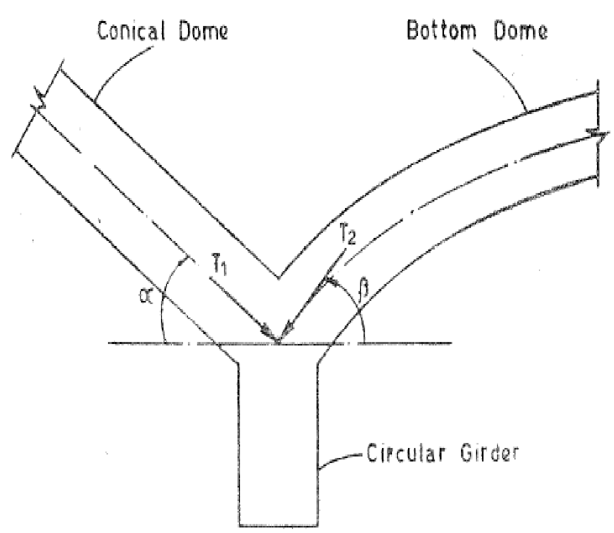
  Hoop tension, T = 131.256*D_b/2

  = 131.256*8/2

  = 525.024 kN

  Hoop stress = (525.024*10^3) / (600*1200)

  = 0.729 N/mm^2 < 5 N/mm^2



- Load on girder

✓ Vertical load on beam = T_1 sin α + T_2 sin β = 689.99 kN/m

✓ Assuming size of girder as 0.6m x 1.2m, load due to self weight of girder

= 0.6*1.2*25 = 18 kN/m

Total load, w = 707.99 kN/m

Total design load on girder, W = 707.99 * π * D_b = 707.99 * π * 8 = 17793.73 kN

- BM & SF

For 8 columns,

- ✓ Negative BM = 0.0083*W*R = 0.0083*17793.73*4 = 590.752 kNm
- ✓ Positive BM = 0.0041* W*R = 0.0041*17793.73*4 = 291.817 kNm
- ✓ Torsional moment=0.0006*W*R = 0.0006* 17793.73*4 = 42.705 kNm
- ✓ Shear force at support = [w*R*(π/4)]/2 = [707.99*4*(π/4)]/2 = 1112.108 kN
- ✓ SF at maximum tension =1112.108-[w*R*(9.55*π/180)]

= 1112.108-[707.99*4*9.55*π/180)] = 640.08 kN

TABLE 4.1 Moment Coefficients in Circular Girders Supported on Columns
Moment Coefficients

Number of columns n		Negative Bending moment at support K_1	Positive Bending moment at centre of spans K_2	Maximum Twisting moment or Torque K_3	Angular distance for maximnm torsion
4	90°	0.0342	0.0176	0.0053	19° – 12′
5	60°	0 0148	0.0075	0.0015	12° – 44′
8	45°	0.0083	0.0041	0.0006	9° – 33′
10	36°	0.0054	0.0023	0.0003	7° – 30′
12	30°	0.0037	0.0014	0.0017	7° – 15′

A_{st} at support:

M = 590.752 kNm, V = 1112.108 kN

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{590.752 \times 10^6}{1.16 \times 600}} = 921.293\ mm\ < 1200\ mm$$

Hence safe

Adopt effective depth = 1150 mm, Cover = 50 mm

$$A_{st} = \frac{M}{\sigma_{st} jd} = \frac{590.752 \times 10^6}{150 \times 0.87 \times 1150} = 3936.378\ mm^2$$

Minimum A_{st} = 0.3 % $bd = \left(\frac{0.3}{100}\right) \times 600 \times 1200 = 2160\ mm^2$

Provide 5 no's of 32 mm diameter (A_{st} = 4021.24 mm^2)

$$\tau_v = \frac{V_u}{bd} = \frac{1112.102 \times 10^3}{600 \times 1150} = 1.612\ {}^{N}/_{mm^2}$$

$\frac{100A_{st}}{bd} = \frac{100\times4021.24}{600\times1150} = 0.583$, Hence $\tau_c = 0.327\ {}^{N}/_{mm^2}$

Also $\tau_c < \tau_v$, hence provide shear reinforcement.

$V_s = V_u - \tau_c bd = (1112.102 \times 10^3) - (0.327 \times 600 \times 1150) = 806.472\ kN$

Provide 4 legged 12 mm dia stirrups, $A_{sv} = 4 \times ({}^{\pi}/_{4}) \times 10^2 = 314.159\ mm^2$

Spacing is given by $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

Substituting the values, S_v = 161.743 mm

Provide 4 legged 12 mm dia stirrups at 160 mm c/c

A_{st} at middle:

M = 291.817 kNm, V = 640.08 kN

$$A_{st} = \frac{M}{\sigma_{st} jd} = \frac{291.817 \times 10^6}{150 \times 0.87 \times 1150} = 1944.47\ mm^2$$

Minimum $A_{st} = 0.3\ \%\ bd = \left(\frac{0.3}{100}\right) \times 600 \times 1200 = 2160\ mm^2$

Provide 5 no's of 25 mm diameter (A_{st} = 2454.37 mm^2)

$$\tau_v = \frac{V_u}{bd} = \frac{640.08 \times 10^3}{600 \times 1150} = 0.928\ {}^{N}/_{mm^2}$$

$$\frac{100A_{st}}{bd} = \frac{100 \times 2454.37}{600 \times 1150} = 0.356$$

Hence $\tau_c = 0.25\ {}^{N}/_{mm^2}$

Also $\tau_c < \tau_v$, hence provide shear reinforcement.

$V_s = V_u - \tau_c bd = (640.08 \times 10^3) - (0.25 \times 600 \times 1150) = 467.58\ kN$

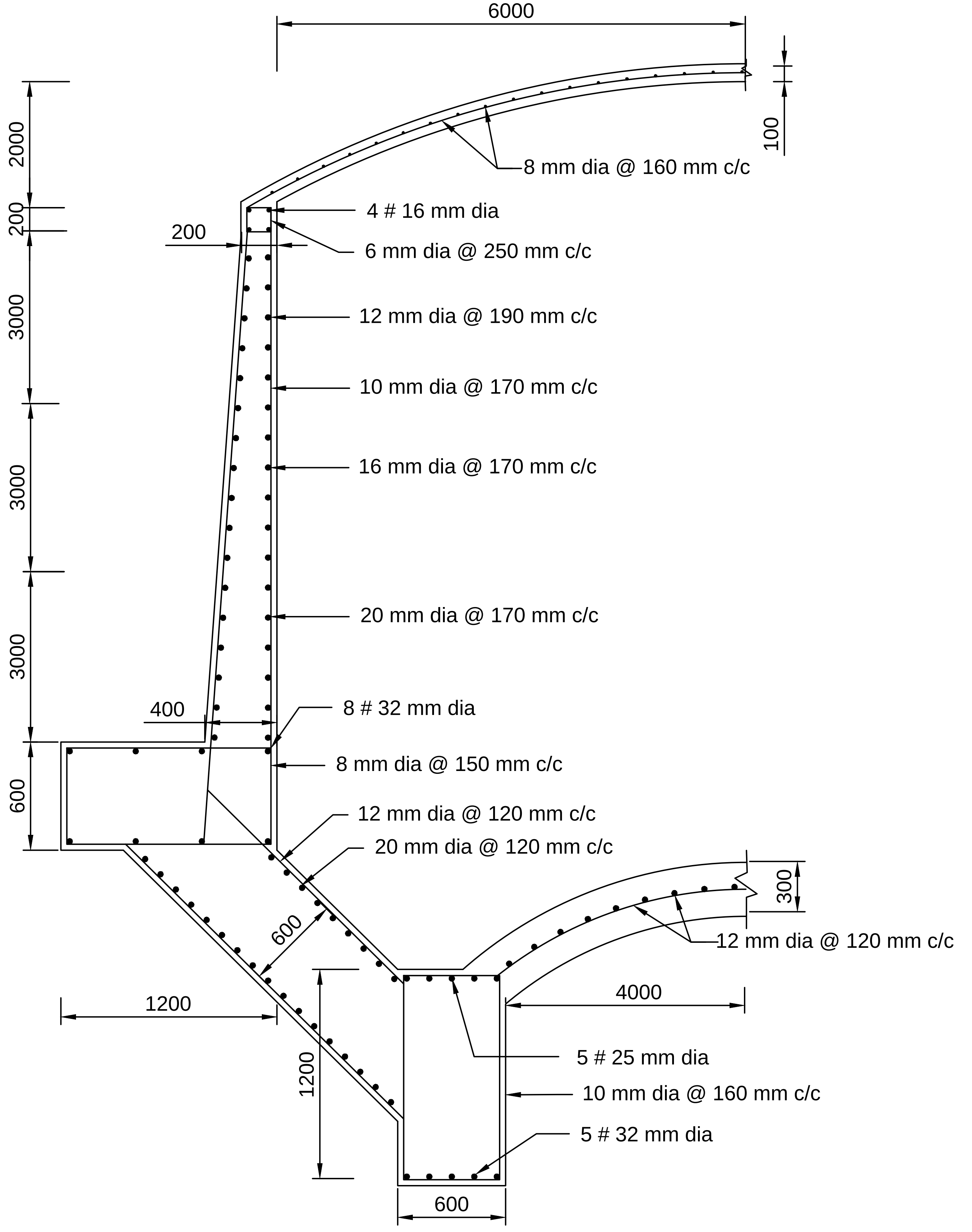
Provide 4 legged 12 mm dia stirrups, $A_{sv} = 4 \times ({}^{\pi}/_{4}) \times 10^2 = 314.159\ mm^2$

Spacing is given by

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$$

Substituting the values, S_v = 278.97 mm

Provide 4 legged 12 mm dia stirrups at 250 mm c/c



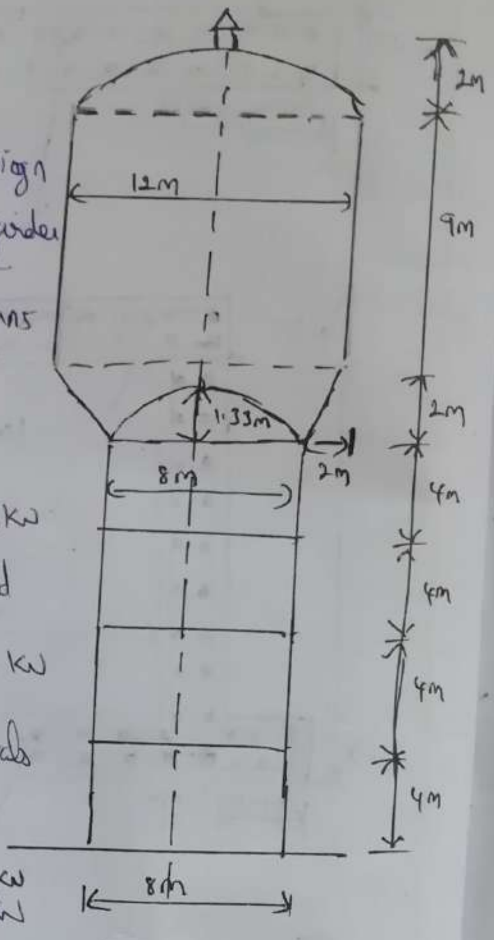
CROSS SECTION OF INTZE TYPE WATER TANK

All dimensions are in mm
M20 Grade Concrete
Fe 415 Grade steel

Step 10 - Design of Columns of supporting tower

- The supporting tower comprises of 8 columns equally spaced on a circle of 8m diameter. (Equal to bottom dome diameter)
- Spacing of bracing = 4m
- Base of tank is 16m above ground level

Load on columns (weight)



- Vertical load on each column due to weight = Total design load on girder / No of Columns

$= \frac{17793.73}{8}$

$= 2224.216 kN$

- Self weight of column of height 16m and dia 650mm $= \frac{\pi}{4} \times 0.65^2 \times 16 \times 25 = 132.732$ kN
- Self weight of bracing (3 no's at 4m intervals) size of bracing is 500mm x 500mm

$= 3 \times (0.5 \times 0.5 \times 8 \times \pi) \times 25 = 471.239$ kN

For one column $= \frac{471.239}{8} = 58.905$ kN

Total vertical load on each column due to weight $= 2224.216 + 132.732 + 58.905$

$= 2415.853 kN$ — (1)

Wind force on columns

Intensity of wind pressure $= 1.5 kN/m^2$

Reduction coefficient for circular shapes = 0.7

- Wind force on top dome and cylindrical wall $= (9 + \frac{2}{2}) \times 12 \times 0.7 \times 1.5$ (H, D)

$= 126 kN$

- Wind force on bottom ring beam $= 1.2 \times 8 \times 0.7 \times 1.5 = 10.08 kN$ (b, L)

→ Wind force on Conical dome $= (9+2) \times 2 \times 0.7 \times 1.5 = 21.0$ kN (BD + proj. L)

→ Wind force on five columns $= 5 \times 0.65 \times 16 \times 0.7 \times 1.5 = 54.6$ kN

(5 columns gets exposed in one direction)

→ Wind force on bracings $= 3 \times 0.5 \times 8 \times 1.5 = 18$ kN

∴ Total wind force $= 126 + 10.08 + 21 + 54.6 + 18 = 226.68$ kN

Assuming point of contraflexure at mid; ∴ height of column and fixed at base due to raft foundation, moment at base of column is calculated as,

Fixing Moment, M_F = Total wind force × $\frac{\text{column height}}{2}$

$$= 226.68 \times \frac{4}{2}$$

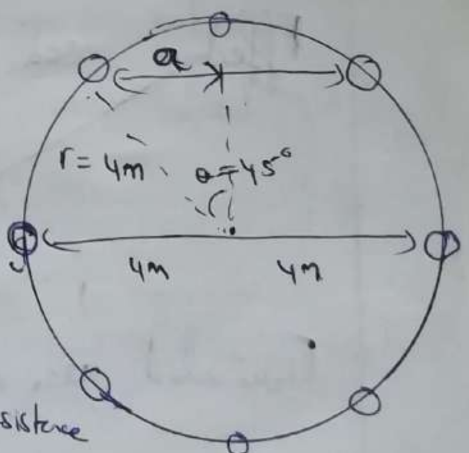
$$= 453.36 \text{ kN}$$

If M_R is moment of resistance at base of columns due to wind loads,

(Till top dome; top wall + dome; Till dome; Conical dome; (Till beam); Bracing)

$$M_R = \left(126 \times \left[18 + \left(\frac{9+2}{2}\right)\right]\right) + \left[21 \times \left(16 + \frac{2}{2}\right)\right] + (10.08 \times 16) + (6 \times 12) + (6 \times 8) + (6 \times 4)$$

$M_R = 3623.28$ kNm



$$M_R = M_F + \frac{V}{r} \Sigma a^2$$

Where
M_F → Fixing Moment, M_R → Moment of resistance
V → Reaction
r → Radius
a → Distance of column from centre

$$3623.28 = 453.36 + \frac{V}{4}\left[(2 \times 4^2) + (4 \times 2.828^2) + (2 \times 0^2)\right]$$

(↑ no. of columns, a)

$\sin\theta = \frac{a}{4}$

$\sin 45 = \frac{a}{4}$

$a = 2.828$ m

$$3623.28 - 453.36 = 16V$$

Reaction, $V = 198.12\,kN$ — ②

Total load on column = Vertical load due to weight + Reaction due to wind load (①+②)

$$= 2415.853 + 198.12$$

$$P = 2613.973\,kN$$

Moment in each column = $\frac{\text{Fixing moment}}{\text{No of columns}} = \frac{453.36}{8}$

$$M = 56.67\,kNm$$

Reinforcement in column

Eccentricity, $e = \frac{M}{P} = \frac{56.67 \times 10^6}{2613.973 \times 10^3} = 21.68\,mm$

IS 456, Pg 42, 25.4, Minimum eccentricity is 20mm

Use 12 bars of 32mm diameter and lateral ties of 10mm diameter at 300mm c/c.

$$A_{sc} = 12 \times \frac{\pi}{4} \times 32^2 = 9650.973\,mm^2$$

Effective area, $A_e = A_c + m \cdot A_{st}$

$$= \left(\frac{\pi \times 650^2}{4}\right) + (13 \times 13.333 \times 6433.98)$$

$$= 4.605 \times 10^5\,mm^2$$

Equivalent area of column = $\left(\frac{\pi}{4} \times D^2\right) + m\,A_{sc}$

$$= \left(\frac{\pi}{4} \times 650^2\right) + [13.33 \times 9650.973]$$

$$= 460507.147\,mm^2$$

Equivalent moment of inertia, $I_e = \frac{\pi D^4}{64} + \frac{(m-1) A_{sc} d^2}{8}$

$d = D - \text{cover} = 650 - 50 = 600\,mm$

$$= \frac{\pi \times 650^4}{64} + \frac{(13.333-1) \times 9650.973 \times 600^2}{8}$$

$$= 1.412 \times 10^{10}\ mm^4$$

According to IS code, when effect of wind load is considered, the permissible stress in materials may be increased by $33\frac{1}{3}\%$. ($33.33\% = 1.333$)

For safety of column, we have

$$\frac{\sigma_{cc}'}{\sigma_{cc}} + \frac{\sigma_{cbc}'}{\sigma_{cbc}} < 1 \quad -①$$

where σ_{cc}' → Direct compressive stress $= \frac{P}{A_e}$

A_e → Equivalent area of column

σ_{cbc}' → Bending stress in column $= \frac{M}{Z}$

Z → I_e/y

I_e → Equivalent moment of inertia

y → Centroid $= D/2$

P, M → Total load and moment on column

σ_{cc}, σ_{cb} → Permissible compressive stress (direct & bending)

$$\sigma_{cc}' = \frac{P}{A} = \frac{2613.973 \times 10^3}{460507.147} = 5.676\ N/mm^2$$

$$\sigma_{cbc}' = \frac{M}{Z}, \quad Z = \frac{I_e}{y} = \frac{1.412 \times 10^{10}}{d/2} = \frac{1.412 \times 10^{10}}{650/2} = 4.345 \times 10^7\ mm^3$$

$$= \frac{56.67 \times 10^6}{4.345 \times 10^7} = 1.304\ N/mm^2$$

Sub in ① ⇒ $\frac{5.676}{1.333 \times 5} + \frac{1.304}{1.333 \times 7} = 0.991 < 1$

Step 11 - Design of braces

Moment in brace = 2 × Moment in column

$= 2 \times 56.67 = 113.34$ kNm

~~Ast = Moment~~

~~$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{160.287 \times 10^6}{249 \times 0.909 \times}$~~

~~$\sigma_{st} = 0.6 f_y = 0.6 \times 415 = 249$ N/mm²~~

~~$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.333$~~

~~$k = \frac{1}{1 + \frac{\sigma_{st}}{m\sigma_{cbc}}} = \frac{1}{1 + \frac{249}{13.333 \times 7}} = 0.273$~~

~~$j = 1 - \frac{k}{3} = 1 - \frac{0.273}{3} = 0.909$~~

$A_{st} = \frac{M}{\sigma_{st} j d}$

where $d = D - d' = 500 - 50 = 450$ mm

$\sigma_{st} = 150$ N/mm²

$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.333$

$k = \frac{1}{1 + \frac{\sigma_{st}}{m\sigma_{cbc}}} = \frac{1}{1 + \frac{150}{13.33 \times 7}} = 0.384$

$j = 1 - \frac{k}{3} = 1 - \frac{0.384}{3} = 0.8727$ ~~$Q = 0.5 \sigma_{cbc} k j = 1.17$~~

~~$= 0.5 \times 7 \times 0.273 \times 0.909$~~

$A_{st} = \frac{113.34 \times 10^6}{150 \times 0.871 \times 450}$

$= 1930.013$ mm²

Provide 4 25mm diameter at top and bottom ($4 \times \frac{\pi}{4} \times 25^2 = 1963.5$ mm²) and 10mm 2 legged stirrups at 300mm c/c as stirrups.

Step 12 - Design of foundations

A circular girder with a raft slab is provided

Total load on foundation due to columns = Total vertical load on column × No. of columns

= 2415.853×8

= 19326.824 kN

Self weight of foundations (Assume) = 10% of total load (column)

$= \frac{10}{100} \times 19326.824$

= 1932.682 kN

∴ Total load on foundations = 19326.824 + 1932.682

= 21259.506 kN

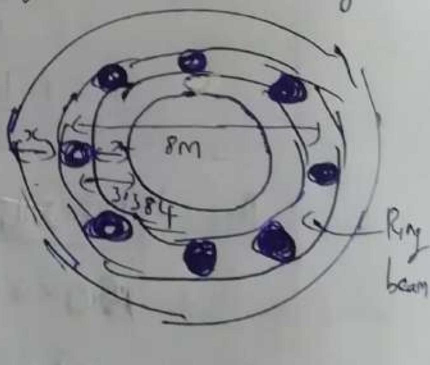
Safe bearing capacity of soil = 250 kN/m²

∴ Area of foundation = $\frac{\text{Total load}}{\text{SBC}} = \frac{21259.506}{250}$

= 85.038 m²

Design of raft slab

Providing a raft slab with equal projections on either side of circular ring beam and if 'b' is width of raft slab.



Area = $\pi d \times b = 85.038$

$\Rightarrow (\pi \times 8) \times b = 85.038$

$b = 3.384$ m

~~Adopt a raft slab having~~ 5m inner ~~diameter and~~ 11 m ~~outer diameter~~

Inner diameter = 8 − 3.384 = 4.616 m

Outer diameter = 8 + 3.384 = 11.384 m

Area of annular portion $= \frac{\pi}{4}(11.384^2 - 4.616^2) = 85.409$ m²

Moment of Inertia $= \frac{\pi}{64}(11.384^4 - 4.616^4) = 802.136\ m^4$

The foundation will be designed for an average pressure of, p = Total load on foundation due to columns / Area of annular raft

$= \frac{19336.824}{85.409}$

$= 226.286\ kN/m^2$

Overhang 'x' $= \frac{1}{2}\left[\frac{1}{2}(11.384 - 4.616) - 0.7\right]$

$x = 1.342\ m$

Bending moment, $M = P \times x \times \frac{x}{2} = 226.286 \times 1.342 \times \frac{1.342}{2}$

$= 203.766\ kNm$

$M = Qbd^2$

$203.766 \times 10^6 = 0.869 \times 1000 \times d^2$

$\Rightarrow d = 484.235\ mm$

Provide 525 mm thick slab with effective depth = 525 − 40

$d = 485\ mm$

$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{203.766 \times 10^6}{249 \times 0.909 \times 485}$

$= 1856.208\ mm^2$

Provide 20 mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 20^2}{1856.208}$

$= 169.248\ mm$

Provide 20 mm diameter bars at 160 mm c/c

Minimum $A_{st} = 0.3\% \cdot bD = \frac{0.3}{100} \times 1000 \times 525 = 1575\ mm^2$

Ast on both faces = $\frac{1575.57}{2}$

Provide 10 mm dia bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{1575.57} = 199.46mm$

Provide 10mm diameter at 190mm c/c as distribution reinforcement.

Design of circular girder

Total design load on girder, W = 19326.824 kN (Load on columns)

Load per metre run on girder, $w = \frac{19326.824}{\pi \times 8} = 768.99$ kN/m

For 8 columns,

Negative BM = 0.0083 WR = 0.0083 × 19326.824 × 4 = 641.651 kNm

Positive BM = 0.0041 WR = 0.0041 × 19326.824 × 4 = 316.96 kNm

Torsional moment = 0.0006 WR = 0.0006 × 19326.824 × 4 = 46.384 kNm

Shear force at support = $[wR(\pi \times 4)]/2 = [768.99 \times 4 \times (\pi/4)]/2 = 1207.927$ kN

Shear force at maximum tension = $1207.927 - \left[wR \times \left(\frac{9.55 \times \pi}{180}\right)\right]$

$= 1207.927 - \left[768.99 \times 4 \times \left(\frac{9.55 \times \pi}{180}\right)\right]$

= 695.229 kN

At support, M = 641.551 kNm, V = 1207.927 kN

$M = Qbd^2$, Assuming width of girder is 750mm

$641.55 \times 10^6 = 0.869 \times 750 \times d^2$

$\Rightarrow d = 992.144mm$

Adopt effective depth, d = 1000mm, Overall depth = 1000 + 50 = 1050mm

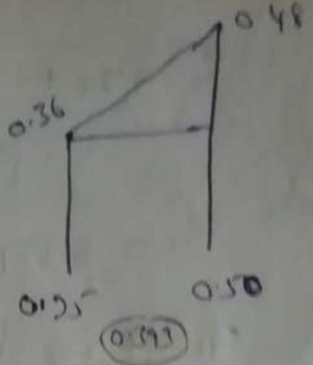
$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{641.55 \times 10^6}{249 \times 0.909 \times 1000} = 2834.44 mm^2$

Min A_{st} = 0.3% bD = $\frac{0.3}{100} \times 750 \times 1050 = 2363 mm^2$

Provide 6 no's of 25mm diameter ($A_{st} = 6 \times \frac{\pi}{4} \times 25^2 = 2945.24 mm^2$)

$\tau_v = \frac{V_u}{bd} = \frac{1207.927 \times 10^3}{750 \times 1000} = 1.611 N/mm^2$

$\frac{100 A_{st}}{bd} = \frac{100 \times 2945.24}{750 \times 1000} = 0.393$



For M20 grade

$\frac{100 A_{st}}{bd}$	τ_c
0.25	0.36
0.50	0.48

$\tau_c = 0.36 + \frac{(0.48 - 0.36)}{(0.50 - 0.25)}(0.393 - 0.25) = 0.429 N/mm^2$

$\tau_v > \tau_c$, hence provide shear reinforcement,

$V_{us} = V_u - \tau_c bd = (1207.927 \times 10^3) - (0.429 \times 750 \times 1000) = 886177 N$

Provide 4 legged 12 mm dia stirrups, $A_{sv} = 4 \times \frac{\pi}{4} \times 12^2 = 452.389 mm^2$

Spacing is given from, $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$886177 = \frac{0.87 \times 415 \times 452.389 \times 1000}{S_v}$

$S_v = 184.314 mm$

Provide 4 legged 12 mm dia stirrups at 180 mm c/c

At middle, M = 316.9511 KNm, V = 695.229 kN

$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{316.96 \times 10^6}{249 \times 0.909 \times 1000} = 1400.365 mm^2$

~~$Min A_{st} = 0.3\% bD = \frac{0.3}{100} \times 750 \times 1050 = 2362.5 mm^2$~~

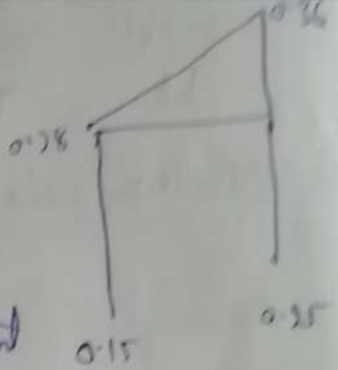
Provide 3 no's of 25 mm diameter ($A_{st} = 3 \times \frac{\pi}{4} \times 25^2 = 1472.6 mm^2$)

$\tau_v = \frac{V_u}{bd} = \frac{695.229 \times 10^3}{750 \times 1000} = 0.927 N/mm^2$

$\frac{100 A_{st}}{bd} = \frac{100 \times 1472.6}{750 \times 1000} = 0.196$

For M20 grade

$\frac{100 A_{st}}{bd}$	τ_c
0.15	0.28
0.25	0.36



$$\tau_c = 0.28 + \frac{(0.36-0.28)}{(0.25-0.15)}(0.196-0.15)$$

$$\tau_c = 0.317 \text{ N/mm}^2$$

$\tau_v > \tau_c$, hence provide shear reinforcement

$$V_{us} = V_u - \tau_c b d = (695.229\times10^3) - (0.317\times 750\times1000)$$

$$= 457479 \text{ N}$$

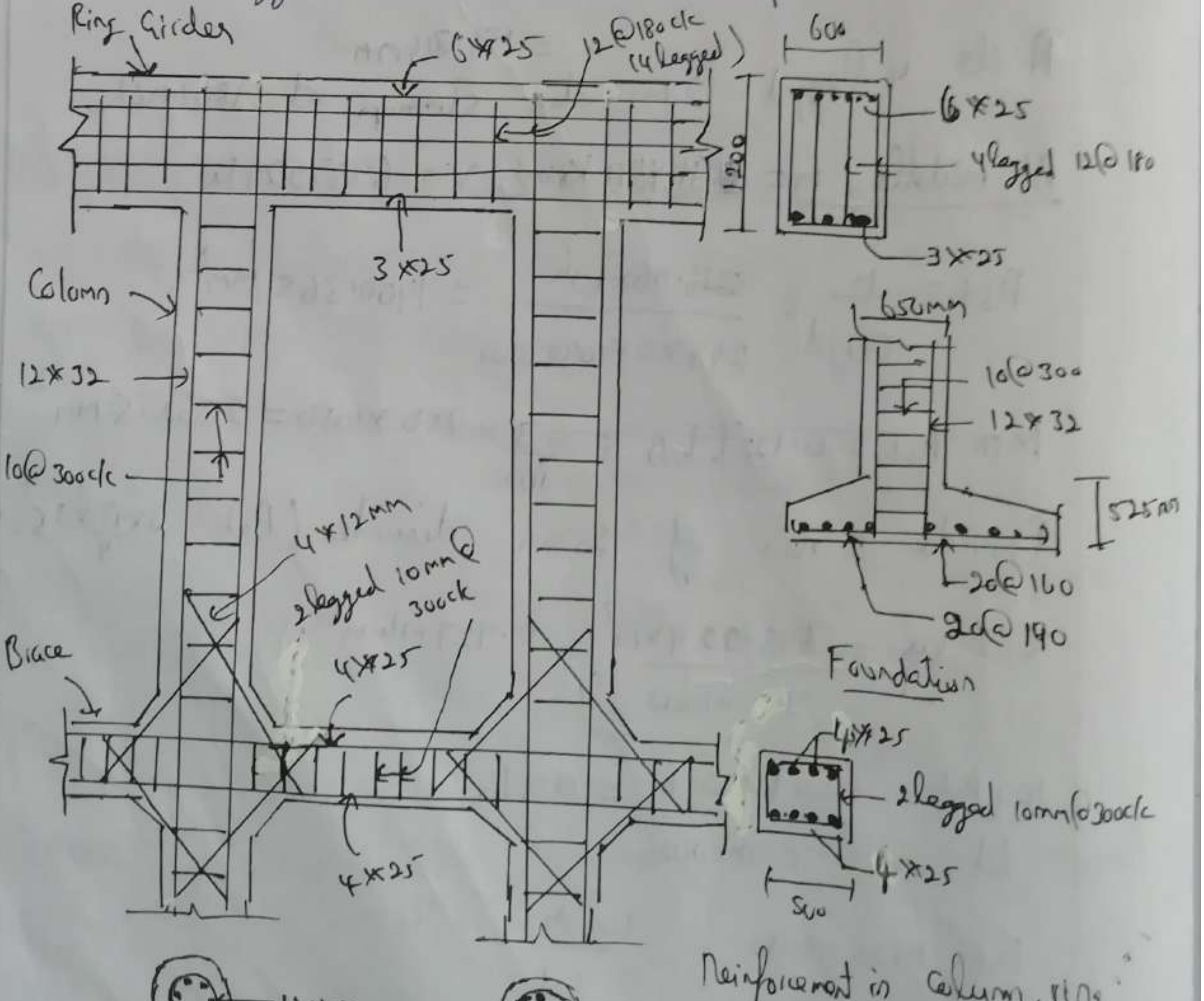
Provide 4 legged 10mm dia stirrups $\left(A_{sv} = 4\times\frac{\pi}{4}\times10^2 = 314.159\text{mm}^2\right)$

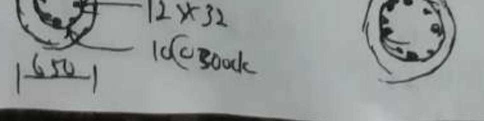
Spacing is given from $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$$457479 = \frac{0.87\times415\times314.159\times1000}{S_v}$$

$$S_v = 247.939\text{mm}$$

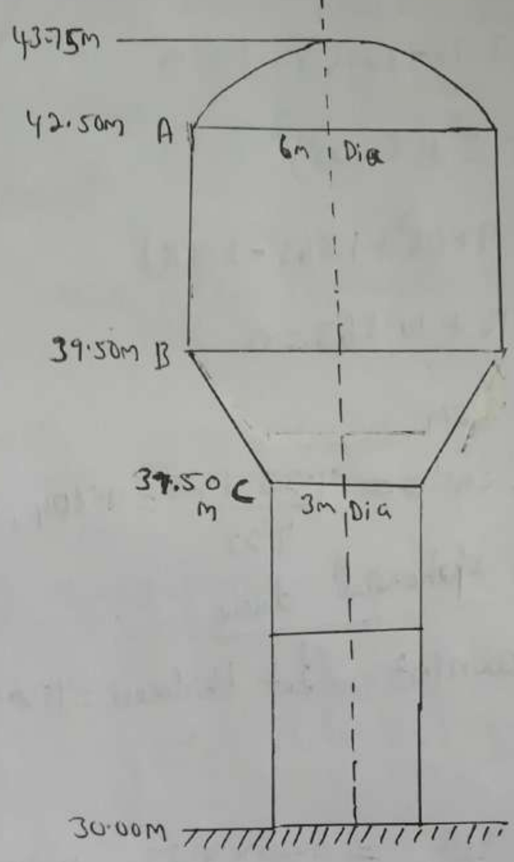
Provide 4 legged 12 mm diameter stirrups at 240mm c/c





Reinforcement in column, ring girder and braces

1b Figure shows an arrangement of an overhead tank. Design the tank to the centre line dimensions shown in figure. The equivalent uniformly distributed load on the dome may be taken as 6000 N/metre².



Step 1 - Permissible stresses

→ Permissible stress in direct tension (tank wall) = 1.2 N/mm²
(IS 3370 - Part II - 1965 - Table 1)

→ Permissible stress in direct tension (dome & ring beam) = 2.8 N/mm²

→ Permissible stress in steel = 150 N/mm²

→ Permissible stress in direct compression = 5 N/mm²

→ Permissible stress in bending compression = 7 N/mm²

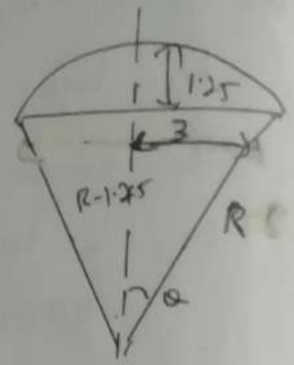
$$M = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times 7} = 13.33$$

$$k = \frac{1}{1+\frac{\sigma_{st}}{m\sigma_{cbc}}} = \frac{1}{1+\frac{150}{13.33\times 7}} = 0.384$$

$$j = 1 - k/3 = 1 - 0.384/3 = 0.872$$

$$Q = 0.5\sigma_{cbc}\, kj = 0.5\times 7\times 0.384\times 0.872 = 1.16$$

Step 2 - Dimensions of tank



Depth of tank = 42.5 − 39.5 = 3m

Diameter of tank = 6m

Central rise of tank = 43.75 − 42.50 = 1.25m

Radius of dome, $R = 3^2 + (R-1.25)^2$

$$R = 9 + (R^2 + 1.563 - 2.5R)$$

$$R^2 - 3.5R + 10.563 = 0$$

$$R = 4.23m$$

$\sin\theta = 3/4.23 = 0.709$, $\cos\theta = \frac{4.23-1.25}{4.23} = 0.704$, $\theta = 45.257$

Step 3 - Design of top spherical dome

Load on top dome = 6000N/m², Let thickness = 150mm

Meridional stress

Meridional thrust, $T_1 = \frac{WR}{1+\cos\theta} = \frac{6000 \times 4.23}{1+0.704} = 14894.366 N/m$

$= 14.894 kN/m$

Meridional stress $= \frac{T_1}{t} = \frac{14.894}{150} = 0.099 N/mm^2 < 5N/mm^2$

Hoop stress

Circumferential force, $T_2 = WR\left[\cos\theta - \frac{1}{1+\cos\theta}\right]$

$= 6000 \times 4.23\left[0.704 - \frac{1}{1+0.704}\right]$

$= 2973.154 N/m$

$= 2.973 kN/m = 2.973 N/mm$

Hoop stress $= \frac{T_2}{t} = \frac{2.973}{150} = 0.02 N/mm^2 < 5N/mm^2$

Reinforcement

Min Ast = 0.3% bD $= \frac{0.3}{100} \times 1000 \times 150 = 450mm^2$

Provide 8mm diameter bars, Spacing $= \dfrac{1000 \times \frac{\pi}{4} \times 8^2}{450}$

Provide 8mm diameter bars at 100mm c/c both circumferentially and meridionally. $= 111.701\text{mm}$

Step 4 - Design of top ring beam

Reinforcement

→ Hoop tension, $F_t = T_1 \times \cos\theta \times D_t/2 = 14.894 \times 0.704 \times 6/2$

$= 31.456\ \text{kN}$

$A_{st} = \dfrac{F_t}{\sigma_{st}} = \dfrac{31.456 \times 10^3}{150} = 209.707\text{mm}^2$

Provide 4 no's of 10mm diameter $\left(4 \times \frac{\pi}{4} \times 10^2 = 314.159\text{mm}^2\right)$

→ Minimum shear reinforcement is given by,

$$\frac{A_{sv}}{b S_v} = \frac{0.4}{0.87 f_y}$$

Provide 2 legged 6 mm diameter stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 6^2 = 56.549\text{mm}^2$$

$$\frac{56.549}{100 \times S_v} = \frac{0.4}{0.87 \times 415}$$

$$S_v =$$

Size

Permissible stress in ring beam $= \dfrac{F_t}{A_c + m A_{st}}$

$$2.8 = \frac{31.456 \times 10^3}{A_c + (13.33 \times 314.159)}$$

$$A_c = 7046.546\text{mm}^2$$

$$1.2 = \frac{31.456 \times 10^3}{A_c + (13.33 \times 314.159)}$$

$$A_c = 22025.594 \, mm^2$$

Provide top ring beam of size 150×150mm ($A_c = 22{,}500 \, mm^2$)

Shear reinforcement

Minimum shear reinforcement in given by

$$\frac{A_{sv}}{b S_v} = \frac{0.4}{0.87 f_y}$$

Where $A_{sv} = 2 \times \frac{\pi}{4} \times 6^2 = 56.549 \, mm^2$,

$$\frac{56.549}{150 \times S_v} = \frac{0.4}{0.87 \times 415}$$

$$S_v = 340.284 \, mm$$

∴ Provide 6mm diameter 2 legged stirrups at 300mm c/c

Step 5 - Design of tank wall

Horizontal reinforcement

Hoop tension, $F_t = \gamma_w \times H \times \frac{D_t}{2} = 9.81 \times 3 \times \frac{6}{2} = 88.29 \, kN/m$

$$A_{st} = \frac{F_t}{\sigma_{st}} = \frac{88.29 \times 10^3}{150} = 588.6 \, mm^2$$

$$A_{st} \text{ on one face} = \frac{588.6}{2} = 294.3 \, mm^2$$

Provide 8mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 8^2}{294.3}$

$= 170.97 \, mm$

Provide 8mm diameter bars at 170mm c/c on both faces

(A_{st} provided $= \frac{1000 \times \frac{\pi}{4} \times 8^2}{170} = 295.679 \, mm^2$ on ~~both~~ one face)

A_{st} on both faces $= 2 \times 295.679 = 591.358 \, mm^2$)

Size

Permissible stress in tank wall = $\frac{F_t}{A_c + m A_{st}}$

$$1.2 = \frac{88.29 \times 10^3}{A_c + (13.33 \times 591.358)}$$

$A_c = 65,692.198\ mm^2$

Provide tank wall of thickness 150mm ($A_c = 1000 \times 150 = 150000 mm^2$)

Vertical reinforcement

$A_{st} = 0.3\%\ bD = \frac{0.3}{100} \times 1000 \times 150 = 450 mm^2$

A_{st} on each face $= \frac{450}{2} = 225 mm^2$

Provide 8mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 8^2}{225}$

$= 223.402$ mm

Provide 8mm diameter bars at 220mm c/c on both faces.

Step 6 - Design of bottom ring beam

Reinforcement

→ Load due to spherical dome = $T \sin\theta = 14.894 \times 0.709$

$= 10.56$ KN/m

Load due to top ring beam $= 0.15 \times 0.15 \times 25 = 0.563$ kN/m

Load due to tank wall $= 0.15 \times 3 \times 25 = 11.25$ kN/m

Assuming size of bottom ring beam as 0.15×0.15 m, load due to bottom ring beam $= 0.15 \times 0.15 \times 25 = 0.563$ kN/m

∴ Total vertical load $= 10.56 + 0.563 + 11.25 + 0.563$

$= 22.936$ kN/m

Total horizontal load $= 22.936 \times \cot 45 = 22.936$ kN/m

→ Hoop tension due to vertical load $= 22.936 \times \frac{D_t}{2}$

$= 22.936 \times \frac{6}{2}$

$= 68.808 \, kN$

→ Hoop tension due to water $= \gamma_w \times H \times h \times \frac{D_t}{2}$

$= 9.81 \times 3 \times 0.15 \times \frac{6}{2}$

$= 13.244 \, kN$

→ Total hoop tension $= 68.808 + 13.244 = 82.052 \, kN$

→ $A_{st} = \frac{F_t}{\sigma_{st}} = \frac{82.052 \times 10^3}{150} = 547.013 \, mm^2$

Provide 4 no's of 16mm diameter bars ($A_{st} = 4 \times \frac{\pi}{4} \times 16^2 = 804.25 \, mm^2$)

→ Minimum shear reinforcement is given by

$\frac{A_{sv}}{b s_v} = \frac{0.4}{0.87 f_y}$

Provide 2 legged 6mm diameter stirrups,

$A_{sv} = 2 \times \frac{\pi}{4} \times 6^2 = 56.549 \, mm^2$

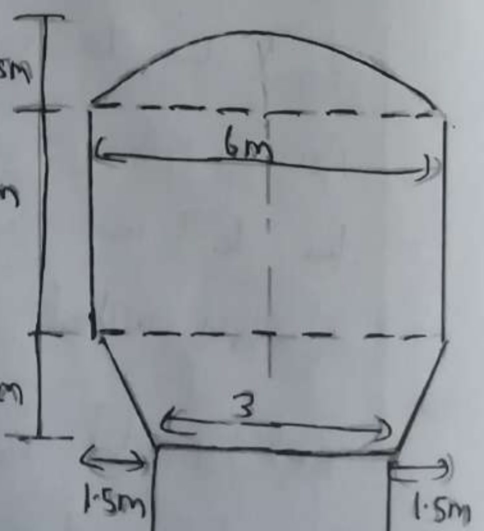
$\therefore \frac{56.549}{150 \times S_v} = \frac{0.4}{0.87 \times 415}$

$S_v = 340.284 \, mm$

Provide 2 legged 6mm diameter stirrups at 300mm c/c

Step 7 – Design of conical dome

→ Dimensions



Length of bottom of tank (Diameter) $= 6 - 1.5 - 1.5$

$= 3 \, m$

Average diameter of conical dome (D_c) $= \frac{6+3}{2} = 4.5 \, m$

Average depth of water, $H_c = 3 + 2/2 = 4 \, m$

→ Load Calculation

Weight of water above conical dome $= (\pi D_c \times H_c \times 2) \times 9.81$

$= \pi \times 4.5 \times 4 \times 2 \times 9.81$

$= 1109.485 kN$

Total horizontal load (from components above) $= 22.936 \times \pi D_t$

$= 22.936 \times \pi \times 6$

$= 432.333 kN$

Assuming thickness of conical dome as 150mm, self weight of dome $= \pi D_k \times 0.15 \times \sqrt{2^2+1.5^2} \times 25$

$= \pi \times 4.5 \times 0.15 \times \sqrt{2^2+1.5^2} \times 25$

$= 132.536 kN$

$\therefore$ Total load $= 1109.485 + 432.333 + 132.536$

$= 1674.354 kN$ — ①

Load/m length $= 1674.354 / (\pi \times D_b) = 1674.354 / (\pi \times 3)$

$= 177.654 kN/m$

→ Meridional stress

Meridional thrust, $T_1 = 177.654 \times \operatorname{cosec} 45 = 251.241 kN$

Meridional stress $= \frac{T_1}{t} = \frac{251.241}{150} = 1.675 < 5 N/mm^2$

→ Horizontal reinforcement

Hoop tension, $F_t = (p \operatorname{cosec}\theta + q \cot\theta) \times \frac{D_t}{2}$

where p = water pressure $= 9.81 \times D_b = 9.81 \times 3 = 29.43 kN/m^2$

q = self weight of conical dome $= 0.15 \times 25 = 3.75 kN/m^2$

$\therefore F_t = [(29.43 \times \operatorname{cosec} 45) + (3.75 \times \cot 45)] \times \frac{6}{2}$

$= 136.111 kN$

$A_{st} = \frac{F_t}{\sigma_{st}} = \frac{136.111 \times 10^3}{150} = 907.407 mm^2$

A_{st} on one face $= 907.407/2 = 453.703 mm^2$

Provide 12mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{453.703}$

$= 249.276 mm$

Provide 12 mm diameter bars at 240 mm c/c on both faces ($A_{tp} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{240} \times 2 = 942.478 mm^2$)

→ Vertical reinforcement

$A_{st} = 0.3\%\ bD = \frac{0.3}{100} \times 1000 \times 150 = 450 mm^2$

A_{st} on each face $= 450/2 = 225 mm^2$

Provide 8mm diameter bars, spacing $= \frac{1000 \times \frac{\pi}{4} \times 8^2}{225}$

$= 223.402$ mm

Provide 8mm diameter bars at 220mm c/c on both faces.

→ Stress check

Permissible stress in conical dome $= \frac{F_t}{A_c + m A_{st}}$

$= \frac{136.111 \times 10^3}{(1000 \times 150) + (13.333 \times 942.478)}$

$= 0.837 N/mm^2 < 2.8 N/mm^2$

Step 8 - Design of Circular base slab

The slab will be designed as supported all around the edge over the circular ring girder. Maximum bending moment at centre of circular base slab is given by,

$M = \frac{3}{16} wr^2$ per metre width of slab

where w → weight on slab

r → Radius at bottom $= 3/2 = 1.5$m

→ Weight of water over slab $= 9.81 \times (3+2) = 49.05 kN/m^2$

→ Self weight of slab (assuming 200mm thick) $= 0.2 \times 25 = 5 kN/m^2$

Total load, $w = 54.05 kN/m^2$

$\therefore M = \frac{3}{16} \times 54.05 \times 1.5^2$

$= 22.802$ kNm

$\rightarrow M = Qbd^2$

$\Rightarrow 1.16 \times 1000 \times d^2 = 22.802 \times 10^6$

$d = 140.203$ mm

Hence provide effective depth, d = 170mm, cover = 30mm, overall thickness = 170 + 40 = 200mm

$\rightarrow A_{st} = \frac{M}{\sigma_{st} j d}$

$= \frac{22.802 \times 10^6}{150 \times 0.872 \times 170}$

$A_{st} = 1025.454$ mm^2

Provide 16mm dia bars, Spacing = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{1025.454}$

$= 196.071$mm

Provide 16mm diameter bars at 190mm c/c (A_{st} provided = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{190} = 1058.221$mm^2)

circular

Step 9 – Design of circular or ring girder

→ Load

From ①, total load of water, load from other components and self weight of conical dome } = 1674.354 kN

Self wt of circular slab = $\frac{\pi d^2}{4} \times t \times 25 = \frac{\pi \times 3^2}{4} \times 0.2 \times 25$

= 35.343 kN

Assuming size of ring girder as 300×500mm, self weight of girder (d = 450mm) = $(0.3 \times 0.5 \times 25) \times (\pi \times 3)$ (D_b)

= 35.343 kN

Total design load on girder, W = 1674.354 + 35.343 + 35.343

= 1745.04 kN

Total design load per m, w = $\frac{1745.04}{\pi \times 3}$ (D_b) = 185.154 kN/m

→ BM & SF

Let us provide 6 columns,

Negative BM = 0.0148 WR = 0.0148 × 1745.04 × 1.5 (D/2) = 38.74 kNm

Positive BM = 0.0075 WR = 0.0075 × 1745.04 × 1.5 = 19.632 kNm

Torsional moment = 0.0015 WR = 0.0015 × 1745.04 × 1.5 = 3.926 kNm

Shear force at support = $\frac{wR \times (\pi/4)}{2} = \frac{185.154 \times 1.5 \times (\pi/4)}{2}$

= 109.065 kN

Shear force at maximum tension = $109.065 - \left(w \times R \times 12°44' \times \frac{\pi}{180}\right)$

= $109.065 - \left(185.154 \times 1.5 \times 12.733 \times \frac{\pi}{180}\right)$

= 47.344 kN

At support, M = 38.74 kNm, V = 109.065 kN

$M = Qbd^2$

$38.74\times10^6 = 1.16\times300\times d^2$

$\Rightarrow d = 333.649mm < 450mm$

$A_{st} = \frac{M}{\sigma_{st} jd} = \frac{38.74\times10^6}{150\times0.872\times450} = 658.172mm^2$

Min $A_{st} = 0.3\%\, bD = \frac{0.3}{100}\times300\times500 = 450mm^2$

Provide 6 #12 mm diameter bars ($A_{st} = 6\times\frac{\pi}{4}\times12^2 = 678.584mm^2$) at top.

$\tau_v = \frac{V_u}{bd} = \frac{109.065\times10^3}{300\times450} = 0.808 N/mm^2$

$\frac{100A_{st}}{bd} = \frac{100\times678.584}{300\times450} = 0.5 \Rightarrow \tau_c = 0.3 N/mm^2$ (IS456, pg 84)

$\tau_v > \tau_c \Rightarrow$ Provide shear reinforcement.

$V_{us} = V_u - \tau_c bd = (109.065\times10^3) - (0.3\times300\times450) = 68565N$
$= 68.565kN$

Provide 2 legged 6mm dia stirrups, $A_{sv} = 2\times\frac{\pi}{4}\times8^2 = 100.53mm^2$

Spacing is given by $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$68.56\times10^3 = \frac{0.87\times415\times100.53\times450}{S_v}$

$S_v = 238.93mm$

Provide 2 legged 6mm diameter stirrups at 230mm c/c

At middle, M = 19.632 kNm, V = 47.344 kN

$A_{st} = \frac{M}{\sigma_{st} jd} = \frac{19.632\times10^6}{150\times0.872\times450} = 333.54mm^2$

Min $A_{st} = 0.3\%\, bD = \frac{0.3}{100}\times300\times500 = 450mm^2$

Provide 4 no's of 12 mm diameter bars at bottom ($A_{st} = 452.39mm^2$)

Provide 2 legged 6mm dia stirrups at 230mm c/c.

Step 10 - Design of Columns

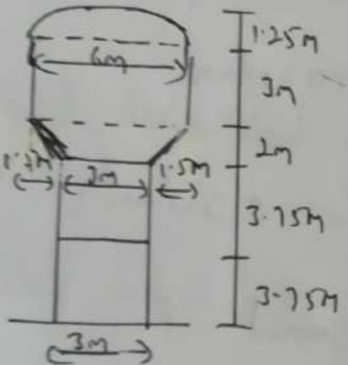
The supporting tower comprises of 6 columns equally spaced on a circle of 3m diameter

Spacing of bracing = 3.75m (7.5/2)

Base of tank is 7.5m above ground level.

Load on columns (weight)

→ Vertical load on each column due to weight = Total design load on girder / No. of columns



$= \frac{1745.04}{6}$

$= 290.84\,kN$

→ Self weight of column of height 7.5m and diameter 300mm

$= \frac{\pi}{4} \times 0.3^2 \times 7.5 \times 25 = 13.25\,kN$

→ Self weight of bracing (300mm × 300mm) = 1 no's × (0.3 × 0.3) × (π × 3) × 25

$= 21.21\,kN$

Bracing weight on one column $= \frac{21.21}{6} = 3.54\,kN$

Total vertical load on each column due to weight $= 290.84 + 13.25 + 3.54 = 307.63\,kN$ — (11)

Wind force on columns

Intensity of wind pressure = 1.5 kN/m³ (assume)

Reduction coefficient for circular shapes = 0.7

→ Wind force on top dome and cylindrical wall $= \left(3 + \frac{1.25}{2}\right) \times 6 \times 0.7 \times 1.5$

$= 22.84\,kN$

→ Wind force on bottom ring beam $= 0.15 \times 3 \times 0.7 \times 1.5 = 0.47\,kN$

→ Wind force on conical dome $= (3+1.5)\times 2\times 0.7\times 1.5 = 9.45\,kN$

→ Wind force on three exposed columns $= 3\times 0.3\times 7.5\times 0.7\times 1.5$

$= 7.09\,kN$

→ Wind force on bracing $= 1\times 0.5\times 3\times 1.5 = 2.25\,kN$

Total wind force $= 22.84 + 0.47 + 9.45 + 7.09 + 2.25$

$= 42.1\,kN$

Assuming point of contraflexure at mid height of column, fixed at base due to raft foundation, moment at base of column is calculated as,

Fixing moment, $M_F = \text{Total wind force} \times \dfrac{\text{Column height}}{2}$

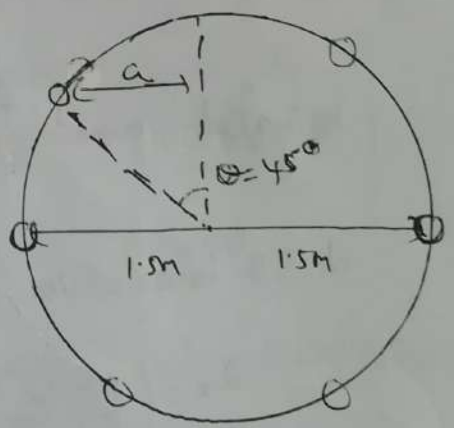
$= 42.1 \times \dfrac{7.5}{2}$

$= 157.88\,kNm$

If M_R is the moment of resistance at base of column due to wind loads,

$M_R = 22.84\times\left(9.5 + \dfrac{3+1.25}{2}\right) + 9.45\times\left(7.5+\dfrac{2}{2}\right) + (0.47\times 7.5)$
$+ (2.25\times 3.75)$

$= 357.8\,kNm$



$M_R = M_F + \dfrac{V}{r}\Sigma a^2$

where M_F → Fixing moment

M_R → Moment of resistance

V → Reaction

a → Distance of column from centre

$\sin 45 = \dfrac{a}{1.5}$

$a = 1.06\,m$

$357.8 = 157.88 + \dfrac{V}{1.5}\left[(4\times 1.06^2) + (2\times 1.5^2)\right]$

$357.8 = 157.88 + 6V$

Reaction, $V = 33.32\,kN$ — (2)

Total load on columns = vertical load due to weight + Reaction due to dead load

$$= 307.63 + 33.32$$

$$P = 340.95\,kN$$

Moment in each column = $\frac{\text{Fixing moment}}{\text{No. of columns}} = \frac{157.88}{6}$

$$M = 26.31\,kNm$$

<u>Reinforcement in Column</u>

Eccentricity, $e = \frac{M}{P} = \frac{26.31 \times 10^6}{340.95 \times 10^3} = 77.17\,mm$

IS 456, Pg 42, 25.4, minimum eccentricity is 20mm

Use 8 bars of 32mm diameter and lateral ties of 10mm diameter at 300mm c/c

$$A_{sc} = 8 \times \frac{\pi}{4} \times 32^2 = 6433.98\,mm^2$$

Effective area, $A_e = A_c + m A_{st} = \left(\frac{\pi D^2}{4}\right) + m A_{sc}$

$$= \left(\frac{\pi \times 300^2}{4}\right) + (13.33 \times 6433.98)$$

$$A_e = 156450.788\,mm^2$$

Equivalent moment of inertia, $I_e = \frac{\pi D^4}{64} + \frac{(m-1) A_{sc} d^2}{8}$

$d = D - \text{Cover} = 300 - 30 = 260\,mm$

$$I_e = \left(\frac{\pi \times 300^4}{64}\right) + \frac{(13.333-1) \times 6433.98 \times 260^2}{8} = 1.068 \times 10^9\,mm^4$$

For safety of column, $\frac{\sigma_{cc}'}{\sigma_{cc}} + \frac{\sigma_{cbc}'}{\sigma_{cbc}} < 1$ (Permissible stress in materials may be increased by $33\frac{1}{3}\%$ (1.333)

where $\sigma_{cc}' = \frac{P}{A_e} = \frac{340.95 \times 10^3}{156450.788} = 2.179\,N/mm^2$

Step 11 – Design of braces

Moment in brace = 2× Moment in column

$= 2 \times 26.63 = 53.26$ kNm

$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{53.26 \times 10^6}{150 \times 0.872 \times 250} = 1628.75 mm^2$

Provide 6 × 20mm diameter at top and bottom $(6 \times \frac{\pi}{4} \times 20^2 = 1884.96 mm^2)$ and 10mm 2 legged stirrups at 300mm c/c as stirrups.

Step 12 – Design of foundations

A circular girder with raft slab is provided.

Total load on foundations due to columns = Total vertical load on columns × No of columns

$= 307.63 \times 6 = 1845.78$ kN

Self weight of foundation (assume) = 10% of total load (columns)

$= \frac{10}{100} \times 1845.78 = 184.578$ kN

Total load on foundations = 1845.78 + 184.578 = 2030.358 kN

Safe bearing capacity of soil = 250 kN/m²

Area of foundation = $\frac{\text{Total load}}{\text{SBC}} = \frac{2030.358}{250} = 8.121 m^2$

Design of raft slab

Providing a raft slab with equal projections on either side of circular ring beam, if 'b' is the width of raft slab

Area = $\pi d \times b$

$\Rightarrow (\pi \times 3) \times b = 8.121$

$b = 0.862 m$

Inner diameter = $3 - 0.862 = 2.138 m$

Outer diameter = $3 + 0.862 = 3.862 m$

Area of annular portion = $\frac{\pi}{4}(3.862^2 - 2.138^2) = 8.124 m^2$

Moment of inertia = $\frac{\pi}{64}(3.862^4 - 2.138^4) = 9.894 mm^4$

Foundation will be designed for average pressure of, $P = \frac{\text{Total load on foundation due to column}}{\text{Area of annular raft}}$

$= \frac{1845.78}{8.124}$

$= 227.201 kN/m^2$

Overhang, $x = \frac{1}{2}\left[\frac{1}{2}(3.862 - 2.138) - 0.3\right] = 0.281 m$

Bending moment, $M = P \times x \times \frac{x}{2} = 227.201 \times 0.281 \times \frac{0.281}{2}$

$= 8.97 kNm$

$M = Qbd^2 \Rightarrow 8.97 \times 10^6 = 1.16 \times 1000 \times d^2$

$d = 87.936 mm$

Provide effective depth, d = 90 mm, cover = 30 mm, Overall depth = 120 mm

$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{8.97 \times 10^6}{150 \times 0.872 \times 90} = 761.978 mm^2$

Provide 12 mm dia bars, Spacing = $\frac{1000 \times (\pi/4) \times 12^2}{761.978} = 148.426 mm$

Provide 12mm diameter bars at 140mm c/c as main bars

Min A_{st} = 0.3% bD = $\frac{0.3}{100} \times 1000 \times 120 = 360 mm^2$

Provide 10 mm diameter bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{360}$ = 218.166mm

Provide 10 mm diameter bars at 210mm c/c as distribution bars.

Design of circular girder

Total design load on girder, W = 1845.78 (load on columns)

Total design load per m = w = $\frac{1845.78}{\pi \times 3 (Dtb)}$ = 195.843 kN/m

As the design load of circular ring girder in foundation is more or less equal to the loads in circular girder at tank provide similar depth and reinforcements.

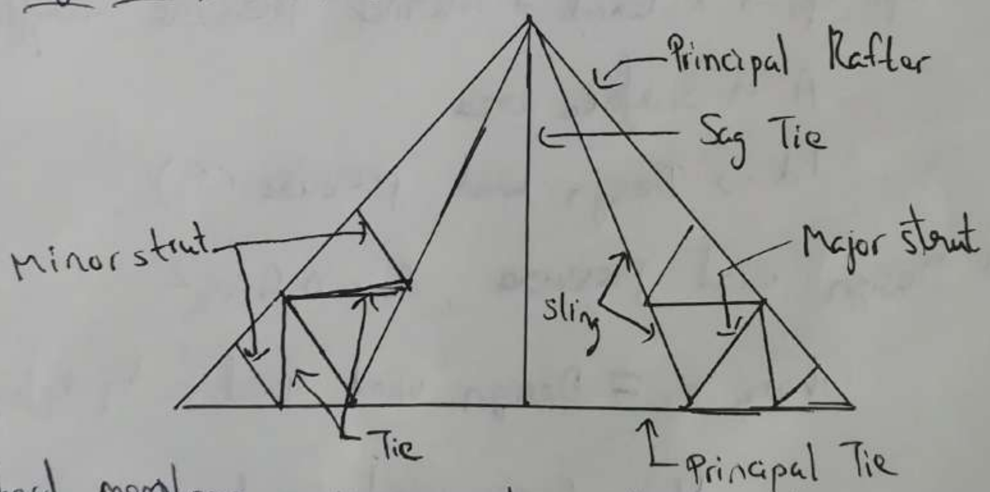
Unit IV - Industrial Structures

Structural steel framing - Steel Roof Trusses - Roofing elements. Beam Columns - Codal provisions - Design and Drawing

Roof Truss

Large column free areas are required for auditoriums, assembly halls, workshops etc. To get column free area, roofing system is provided which has roof truss connected with purlins which inturn supports roof sheeting made of GI sheets, Aluminium sheets or Asbestos Cement (AC) sheets. The roof truss are supported on walls or columns on both sides.

Elements of a roof truss



Top chord members (or) Principal Rafter – Uppermost members along uppermost line of truss passing through peak and support. They support purlins which supports sheet.

Bottom chord members (or) Principal Ties – Lowermost members extending from one support to another.

Struts – Members subjected to compression forces other than top and bottom chord members

Strings or Tie – Members subjected to tension forces other than top and bottom chord members

Sag Tie - Provided to reduce sag of peak

Load on roof truss

1) Dead Load

Unit weight of sheet (IS875 - Part I) → CI sheet - 85 N/m²
→ AC sheet - 130 N/m²

Weight of purlin - 100 to 120 N/m

Weight of truss - $\left(\frac{span}{3}+5\right) \times 10$ (or) 10% of load on truss.

Weight of bracing - 12 to 15 N/m² of plan area.

2) Live load

IS875 Part II - Roof slope ≤ 10° - Access provided = 1.5 kN/m²
- No access = 0.75 kN/m²

> 10° - $[0.75 - 0.02(\theta - 10)]$

3) Wind load

→ Wind force, $F = (C_{pe} - C_{pi}) A P_d$

C_{pe}, C_{pi} → External & internal pressure coefficient

A → Surface area

P_d → Design wind pressure (P_z)

→ Design wind pressure, $P_z = 0.6 V_z^2$

where V_z = Design wind speed $= V_b k_1 k_2 k_3$

V_b → Basic wind speed

k_1 → Risk coefficient (Depending on importance of bldg)

k_2 → Terrain, height and structure, size factor

k_3 → Topography factor

4) Snow load

Snow load = 2.5 N/m² per mm depth of snow

When roof slope > 50°, snow load is neglected

5) <u>Load Combination</u>

-> Dead Load + Live Load

-> Dead Load + Snow Load

-> Dead Load + Wind Load

Pb) Determine the loads acting on the roof of a Fink truss for the following data.

-> Overall length of building - 48M

-> Overall width of building - 16.5M

-> Width (c/c of roof truss) - 16m

-> c/c spacing of truss - 8M

-> Rise of truss - 1/4 of span

-> Self weight of purlins - 318N/m

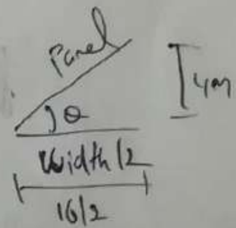
-> Height of columns = 11M

-> Roofing and size coverings - AC sheet (171N/mm^2)

The building is located in industrial area Naini, Allahabad. Both the ends of truss are hinged. Use steel of grade Fe410

<u>Solution</u>

<u>Step 1 - Truss Geometry</u>



Rise of truss $= \frac{1}{4}$ of span

$= \frac{1}{4} \times 16$

$= 4M$

Slope, $\tan\theta = \frac{4}{8} = 0.5$

$\Rightarrow \theta = \tan^{-1}(0.5) = 26.565^\circ$

Length of panel $= \sqrt{8^2 + 4^2} = 8.944m$

Pitch (Rise) $= 1/5$

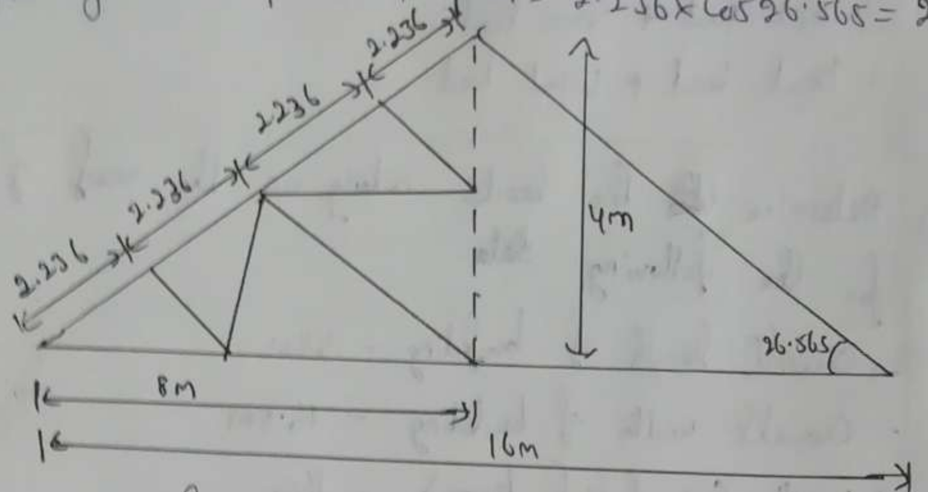
$\frac{Ht}{Span} = \frac{1}{5}$

$\frac{Ht}{16} = \frac{1}{5}$

Width of bldg = span of truss + 0.5M

Length of each panel on one side = 8.944/4 = 2.236m

Length of each panel in plan = 2.236 × cos 26.565 = 2m



Step 2 - Dead load

Self weight of AC sheet = 171 N/m²

Self weight of purlin = 318 N/m

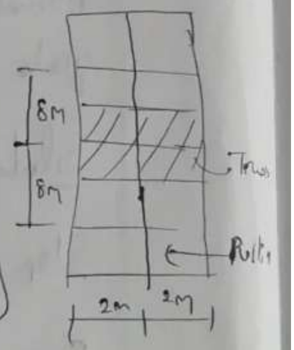
Self weight of truss = $\left(\frac{\text{Span}}{3}+5\right) \times 10 = \left(\frac{16}{3}+5\right) \times 10$

$= 103.33 \text{ N/m}^2 \simeq 110 \text{ N/m}^2$

Self weight of bracing = 12 N/mm² (Assume)

Total dead load (N/m²) = 171+110+12 = 293 N/m²

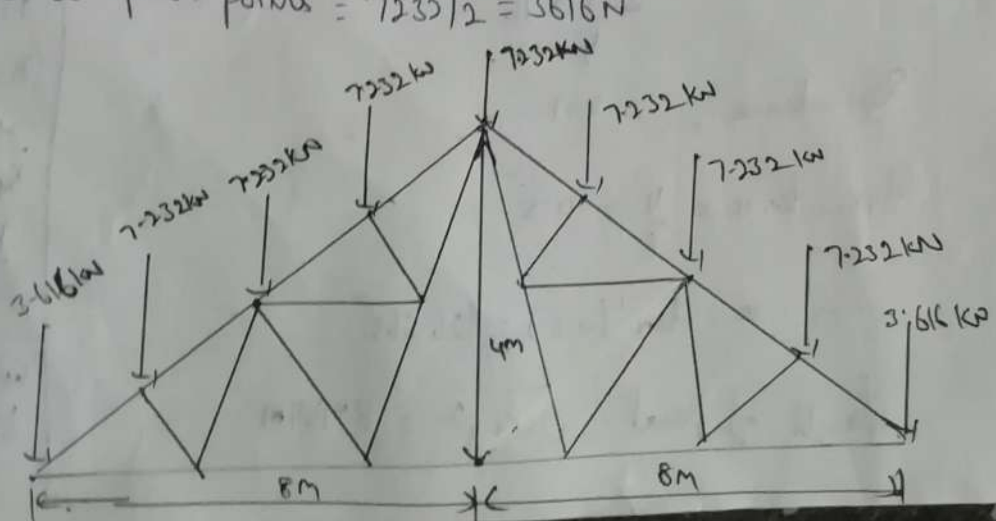
Total dead load (N/m) = 318 N/m



Load at intermediate panels = (293 × 8 × 2) + (318 × 8)

= 7232 N

Load at panel points = 7232/2 = 3616 N



Step 3 - Live load

Live load for $\theta > 10°$, Live load $= 0.75 - 0.02(\theta - 10°)$

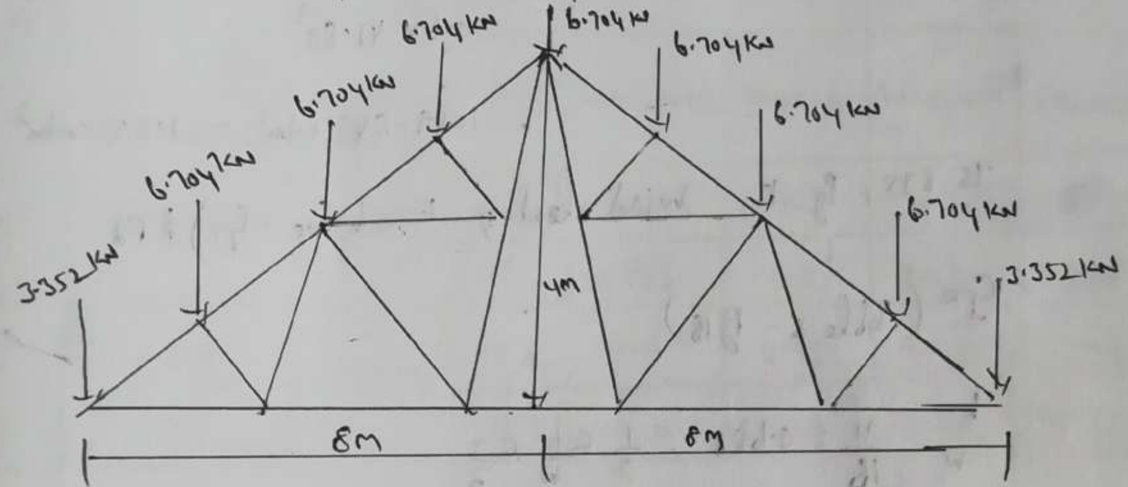
$= 0.75 - 0.02(26.565 - 10)$

$= 0.419\ kN/m^2$

Load at intermediate panels $= 0.419 \times 8 \times 2$

$= 6.704\ kN$

Load at end panel points $= 6.704/2 = 3.352\ kN$



Step 3 - Wind Load

→ $V_z = V_b k_1 k_2 k_3$ (IS 875 - Part III - Pg 8)

$V_b = 47\ m/s$ (Allahabad) - (IS 875 - Part III - Pg 53, Pg 9 Fig 1)

$k_1 = 1.0$ (general buildings) - IS 875 Part III, Pg 11, Table 1)

k_2

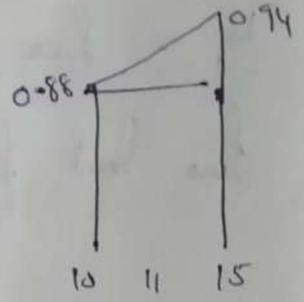
Category 3 - Numerously closed spaced obstructions with building height upto 10m (IS 875 - Part III - Pg 8)

Class B - Greatest dimension - 48m (between 20 to 50m)

From Table 2,

Height	k_2
10	0.88
(11)	
15	0.94

$$k_2 = 0.88 + \left(\frac{0.94 - 0.88}{15 - 10}\right) \times (11 - 10)$$



$k_2 = 0.892$

$k_3 = 1$ (Flat topography)

$\therefore V_z = 47 \times 1 \times 0.892 \times 1 = 41.83\ m/s$

Design wind pressure, $P_z = 0.6 V_z^2$

$= 0.6 \times 41.83^2$

$= 1049.849\ N/m^2 = 1.05\ kN/m^2$

IS 875, Pg 13, Wind load, $F = (C_{pe} - C_{pi})\, A\, P_d$

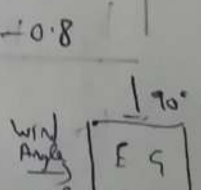
C_{pe} (Table 5 - Pg 16)

$\frac{h}{w} = \frac{11}{16} = 0.688$, $\frac{1}{2} < \frac{h}{w} < \frac{3}{2}$

Roof angle	Wind Angle 0°		Wind angle 90°	
	EF	GH	FG	FH
20	-0.7	-0.5	-0.8	-0.6
30	-0.2	-0.5	-0.8	-0.8

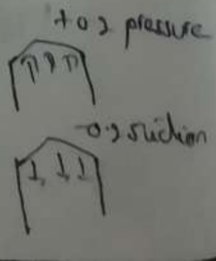
0° → EF - Windward, GH - Leeward

90° → EG - Windward, FH - Leeward



Roof angle	Wind Angle 0°		Wind Angle 90°	
	EF	GH	FG	FH
26.565	-0.372	-0.5	-0.8	-0.731

C_{pi}



IS875, Pg 27 - 6.2.3.1, $C_{pi} = \pm 0.2$

Area (A)

Area = 8 × 2.236 = 17.888 mm^2

→ Wind Force

$F = (C_{pe} - C_{pi}) A P_d$ → Simple calculation

$= (-0.372 - 0.2) \times 17.888 \times 1.05$

$= -10.744$ kN

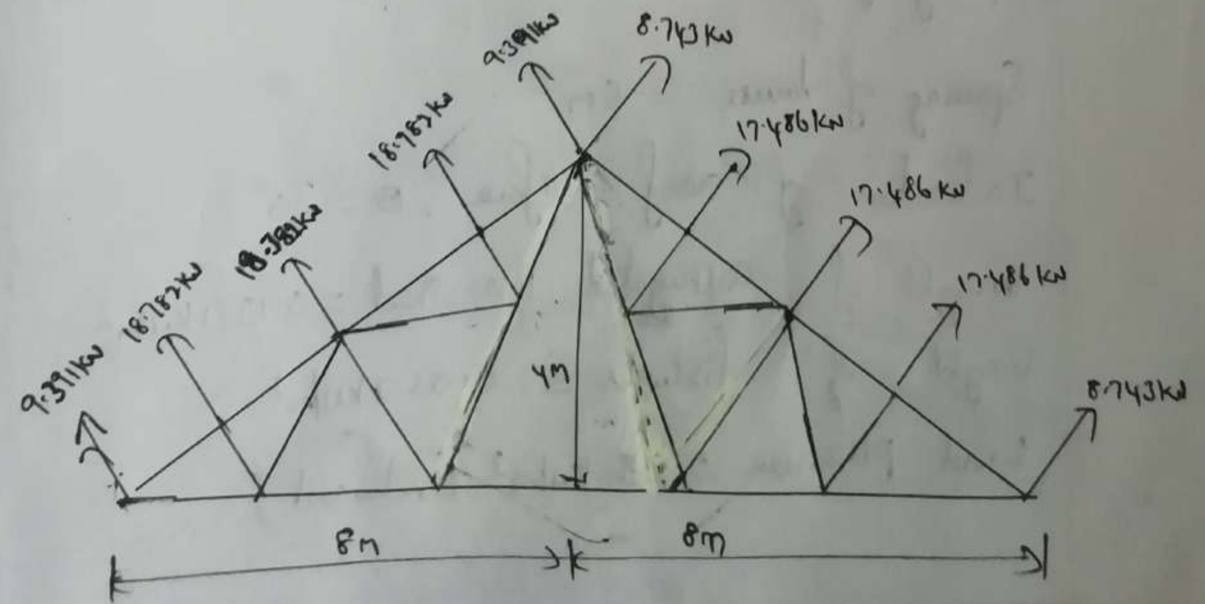
Wind Angle	Pressure Coefficients			$C_{pe} - C_{pi}$		Wind force (F)	
	C_{pe}		C_{pi}	Windward	Leeward	Windward	Leeward
	Windward	Leeward					
0°	−0.372	−0.5	+0.2	−0.572	−0.7	−10.744	−13.148
			−0.2	−0.172	−0.3	−3.231	−5.635
90°	−0.8	−0.731	+0.2	−1.0	−0.931	−18.782	−17.486
			−0.2	−0.6	0.531	−11.269	−9.973

Windward force at intermediate panel points = −18.782 kN

" " " end panel points = −18.782/2 = −9.391 kN

Leeward force at intermediate panel points = −17.486 kN

" " end panel points = −17.486/2 = −8.743 kN



(Pb) Design a roof truss to suit the following requirements,

Span of truss = 16m

Rise of truss = 4m

Spacing of truss = 4m

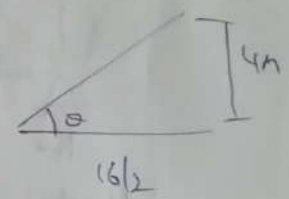
Roofing shall be of GI sheets

Live load = 50kg/m^2

Wind pressure = 120 kg/m^2 normal to roof.

Solution

Step 1 - Truss geometry



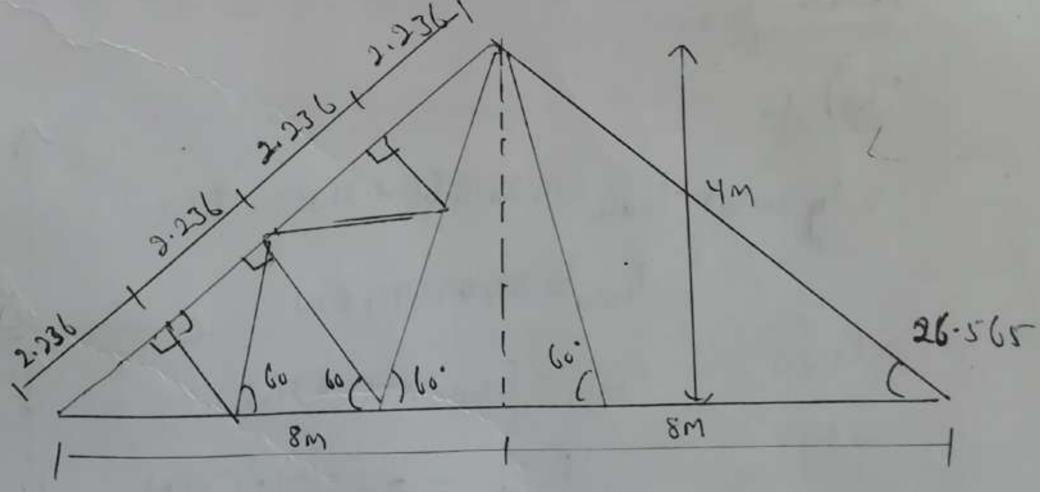
Slope, $\tan\theta = \frac{4}{8} = 0.5$

$\theta = \tan^{-1} 0.5 = 26.565$

Length of panel $= \sqrt{8^2+4^2} = 8.944$m

Length of each panel in one side $= 8.944 / 4 = 2.236$m

Length of each panel in plan $= 2.236 \times \cos 26.565 = 2$ m



Step 2 - Dead Load

Self weight of GI sheets = 150 N/m^2 (assume)

Self weight of purlin = 80 N/m^2 (assume)

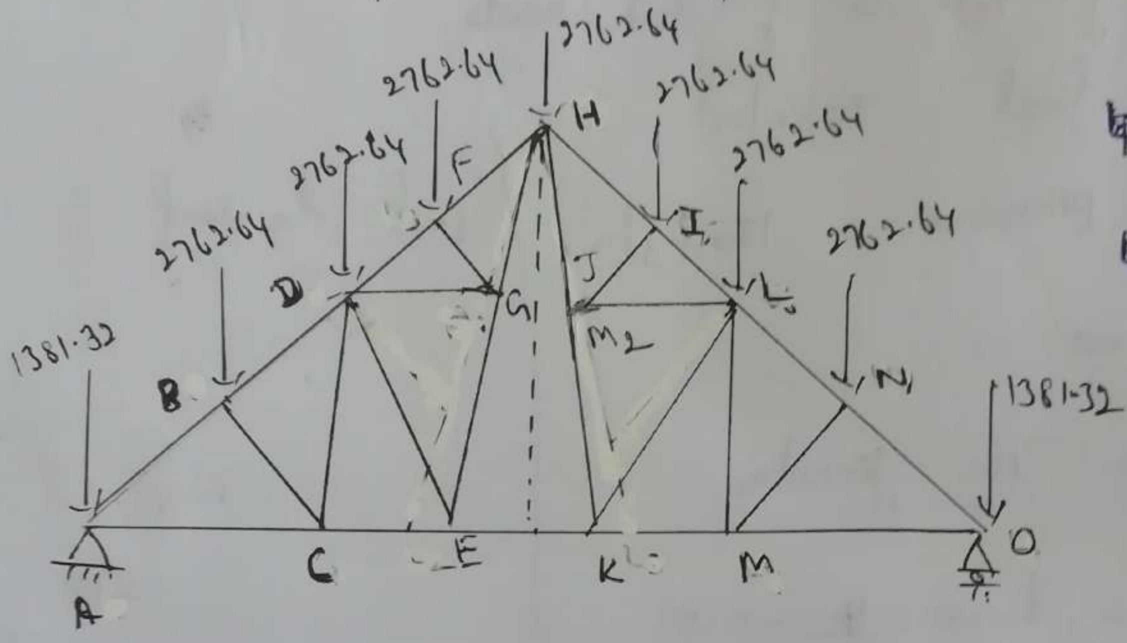
Weight of truss $= \left(\frac{\text{span}}{3}+5\right) \times 10 = \left(\frac{16}{3}+5\right) \times 10 = 103.33$ N/m^2

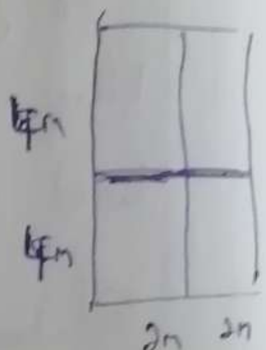
Self weight of bracing = 12 N/m² (Assume)

Total dead load = 150 + 80 + 103.33 + 12 = 345.33 N/m²

Load at intermediate panels = 345.33 × 4 × 2 = 2762.64 N (Purlin spacing (assume))

Load at end points = 2762.64/2 = 1381.32 N





Reactions

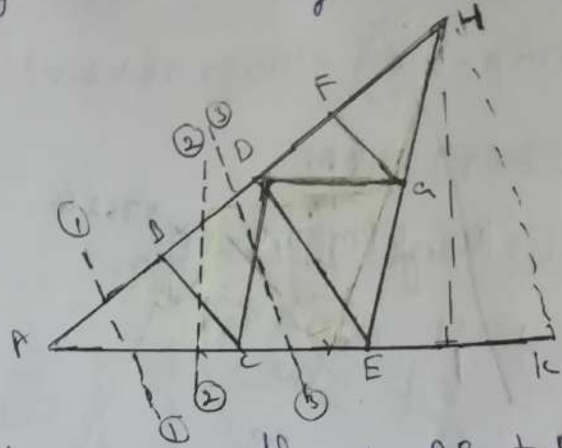
Total downward load = (2762.64 × 7) + (1381.32 × 2)

= 22101.12 kN

Reaction at A, and O = 22101.12/2 = 11050.56 kN

Member forces

Analysing the truss by method of joints,



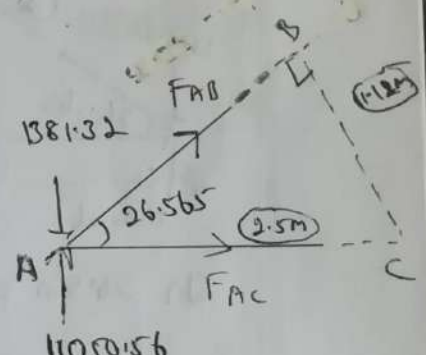
Cut section ①-① through AB & AC,

Taking moment about C, $\Sigma M_C = 0$

$\Rightarrow (11050.56 \times 2.5) - (1381.32 \times 2.5)$

$+ (F_{AB} \times 1.12) = 0$

$\Rightarrow$ $F_{AB} = -21583.348$ kN



$\sin 26.565 = \frac{BC}{AC}$

$\sin 26.565 = \frac{BC}{2.50}$

$BC = 1.12m$

$\cos 26.565 = \frac{AB}{AC}$

$\cos 26.565 = \frac{2.236}{AC}$

$AC = 2.50m$

Cut section ②-② through BD, BC & AC,

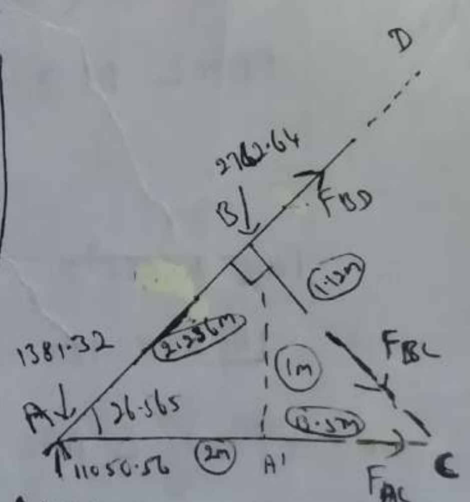
$\Sigma M_B = 0 \Rightarrow (11050.56 \times 2) - (1381.32 \times 2)$

$- (F_{AC} \times 1.12) = 0$

$F_{AC} = 19338.48N$

$\Sigma M_A = 0 \Rightarrow (F_{BC} \times 2.236) + (2762.64 \times 2)$

$\Rightarrow F_{BC} = -2471.055N$



$\Sigma M_C = 0$

$(F_{BD} \times 1.12) - (2762.64 \times 0.5)$

$- (1381.32 \times 2.5)$

$+ (11050.56 \times 2.5) = 0$

$\Delta AA'B$

$\sin 26.565 = \frac{A'B}{AB}$

$= A'B / 2.236$

$A'B = 1m$

$\Delta ABC,$

$\sin 26.565 = \frac{BC}{AC}$

$= BC / 2.5$

$BC = 1.12m$

$\Delta AA'B$

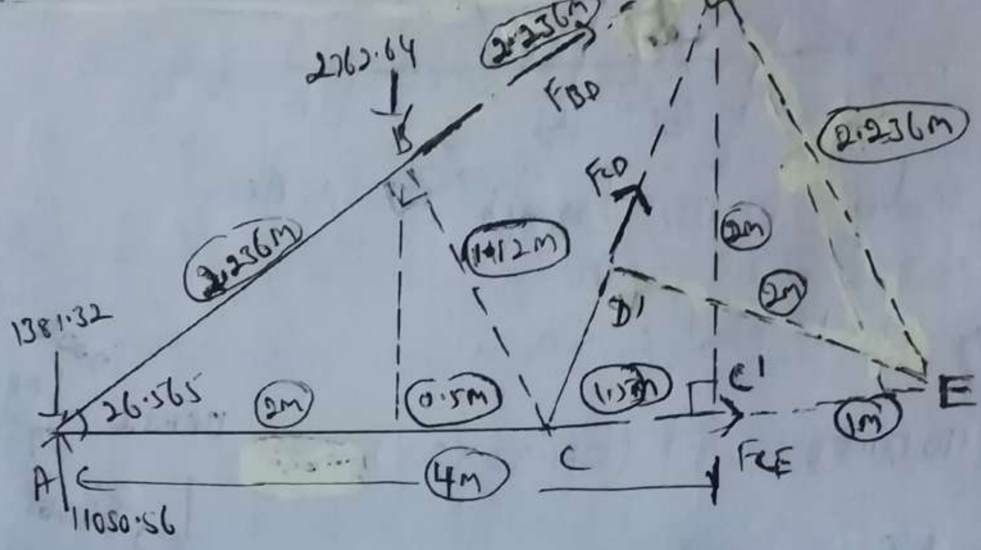
$\cos 26.565 = \frac{AA'}{AB}$

$= \frac{AA'}{2.236}$

$AA' = 2m$

$\Rightarrow F_{BD} = -20349.804\,N$

Cut section ③-③ through BD, CD, CE



$\sin 26.565 = \frac{C'D}{AD} = \frac{C'D}{4.472} \Rightarrow C'D = 2m$

$\cos 26.565 = \frac{AC'}{AD} = \frac{AC'}{4.472} \Rightarrow AC' = 4m$

$\Sigma M_D = 0 \Rightarrow -(F_{CE} \times 2) - (1381.32 \times 4) + (11050.56 \times 4) - (2762.64 \times 2)$

$\Rightarrow F_{CE} = 16575.84\,N$

~~$\Sigma M_A = 0 \Rightarrow (F_{DE} \times 4.472) + (2762.64 \times 2) + (2762.64 \times 4) = 0$~~

~~From Δ AED, $\cos 26.565 = \frac{4.472}{AE}$~~

~~AE = 5m~~

~~From Δ CC'E,~~

~~∠ACB = 180 -~~

From Δ ABC, ∠ACB = 180 - 90 - 26.565 = 63.435°

From Δ BCD, $\tan \angle BCD = \frac{2.236}{1.12} \Rightarrow \angle BCD = 63.394°$

∠ECD' = 180 - 63.435 - 63.394 = 53.171°

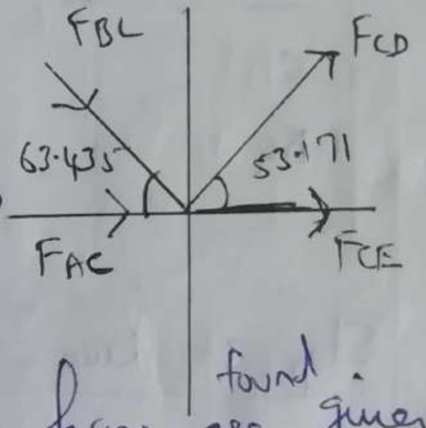
From ~~Δ ECD'~~

Analysing by method of joints,

Joint C

$\Sigma F_y = 0 \Rightarrow F_{CD} \sin 53.171 - F_{BC} \sin 63.435 = 0$

$F_{CD} \sin 53.171 - (-2471.055 \times \sin 63.435) = 0$



$F_{CD} = 2761.247 N$

Similarly find all forces, Various forces found are given below,

$F_{AB} = 21583.348 N$ (Compressive)

$F_{AC} = 19338.48 N$ (tensile)

$F_{BC} = 2471.055 N$ (compressive)

$F_{BD} = 20349.804 N$ (compressive)

$F_{CE} = 16575.84 N$ (tensile)

$F_{CD} = 2761.247 N$ (tensile)

The forces are tabulated below,

Member	Compressive (N)	Tensile (N)
Top Chord members		
AB	21583.348	
BD	20349.804	
DF	19185.189	
FH	17986.115	
Bottom Chord Members		
AC		19338.48
CE		16575.84
EK		10991.515
Strut members		
BC	2471.055	
DE	4942.11	
FG	2471.055	
Tie members		
CD		2761.247
DG		2761.247

Step 3 - Live load.

Live load = 50 kg/m²

= 0.5 kN/m²

Load at intermediate panels = 0.5 × 4 × 2 = 4 kN
= 4000 N

Load at end points = $\frac{4000}{2}$ = 2000 N

The forces determined by live load will be $\frac{4000}{2762.64}$ (LL/DL) times the dead load forces

Step 4 - Wind load

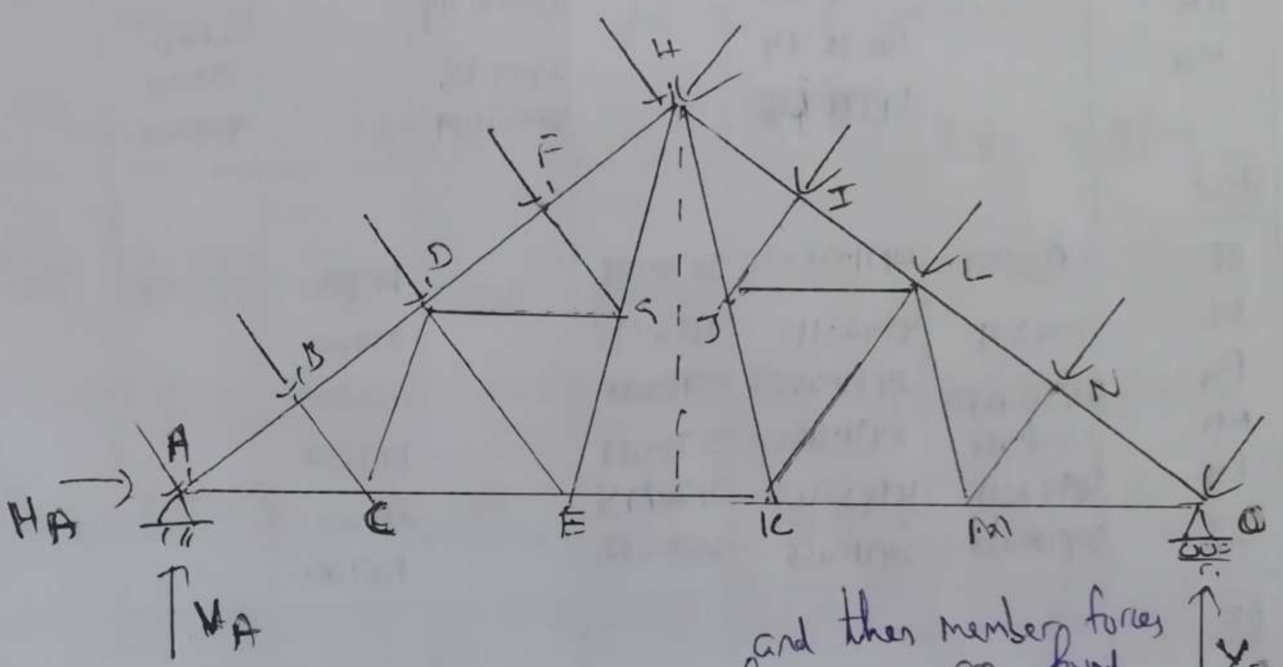
Wind pressure = 120 kg/m² normal to roof

= 120 × 10

= 1200 N/m²

Load at intermediate panels = 1200 × 4 × 2.236 = 1073.28 N

Load at end panels = 1073.28/2 = 536.64 N



Reactions H_A, V_A and V_O are found and then member forces are found by method of sections and method of joints.

Step 5 - Load Combination

The total load is determined from the summation of dead load + greater of live load or wind force. Summary of forces is given below,

Member	Dead load C	Dead load T	Live Load C	Live Load T	Wind Load C	Wind Load T	Design Force C	Design Force T
Top Chord								
AB	21583.348		31252.688		48300		69883.35	
BD	20349.804		29466.516		48300		68649.804	
DF	19185.189		27780.154		48300		67485.189	
FH	17986.115		26043.895		48300		66286.115	
HI	17986.115		26043.895		48300		66286.115	
IK	19185.189		27780.154		48300		67485.189	
LN	20349.804		29466.516		48300		68649.804	
NO	21583.348		31252.688		48300		69883.35	
Bottom Chord								
AC		19338.48		28002.119		60000		79338.48
CE		16575.84		24001.816		48000		64575.84
EK		10991.515		15915.714		24000		34991.515
KM		16575.84		24001.816		28800		45375.84
MO		19338.48		28002.119		40800		60138.48
Strut								
BC	2471.055		3578.088		10700			13171.055
DE	4942.11		7156.175		21400			26342.11
FG	2471.055		3578.088		10700			13171.055
NM	2471.055		3578.088		10700			13171.055
LK	4942.11		7156.175		21400			26342.11
IJ	2471.055		3578.088		10700			13171.055
Tie								
CD		2761.247		3998.286		12000		14761.247
DG		2761.247		3998.286		12000		14761.247
ML		2761.247		3998.286		12000		14761.247
LJ		2761.247		3998.286		12000		14761.247

Step 6 - Design of Purlin

Dead load

→ Weight of GI sheet = 150 N/m²,

→ Self weight of purlin = 80 N/m²

Total dead load = 150 + 80 = 230 N/m² = 230 × 2 (Purlin Spacing) = 460 N/m

Component of dead load normal to roof = 460 × cos 26.565

$= 411.437 N/m$

Component of dead load parallel to roof $= 460 \times \sin 26.565$

$= 205.718 N/m$

Live Load

Live load $= 0.5 kN/m^2 = 500 N/m^2$

Component of live load normal to roof $= 500 \times \cos 26.565$

$= 447.214 N/m$

Component of live load parallel to roof $= 500 \times \sin 26.565$

$= 223.606 N/m$

Moment

$$\text{Moment} = \frac{w_{DL} l^2}{10} + \frac{w_{LL} l^2}{9}$$

BM parallel to major principal axis (UU axis)

$$M_{UU} = \frac{411.437 \times 4^2}{10} + \frac{447.214 \times 4^2}{9} = 1453.346 \text{ N m}$$

BM parallel to minor principal axis (VV axis)

$$M_{VV} = \frac{205.718 \times 4^2}{10} + \frac{223.606 \times 4^2}{9} = 726.671 \text{ N m}$$

Required section modulus, $Z_U = \frac{M_{UU}}{Z_b}\left(1 + \frac{M_{VV}}{M_{UU}} \cdot \frac{Z_{UU}}{Z_{VV}}\right)$

~~$= \frac{1453.346}{165}\left[1 + \frac{726.671}{1453.346} \times 7\right]$~~

~~$= 39.637 m^3$~~

$$= \frac{1453.346 \times 10^3}{165}\left[1 + \frac{726.671 \times 10^3}{1453.346 \times 10^3} \times 7\right]$$

$= 39636.624 mm^3 = 39.636 cm^3$

Choose ~~ISA 150x~~ ISJB 150 @ 7.1 kg/m with $Z_{xx} = 42.9 cm^3$

$Z_{yy} = 3.7 cm^3$

$Z_{UU} = Z_{xx} = 42.9 \times 10^3 mm^3$; $Z_{VV} = Z_{yy} = 3.7 \times 10^3 mm^3$

$$\sigma_b = \frac{M_{uu}}{Z_{uu}}\left[1 + \frac{M_{vv}}{M_{uu}} \cdot \frac{Z_{uu}}{Z_{vv}}\right]$$

$$= \frac{1453.346\times10^3}{42.9\times10^3}\left[1 + \frac{726.671\times10^3}{1453.346\times10^3} \times \frac{42.9\times10^3}{3.7\times10^3}\right]$$

$$= 230.275 \, N/mm^2 > 165 \, N/mm^2$$

Try another section ISLB 125 @ 11.9 kg/m, with

$Z_{uu} = Z_{xx} = 65.1\times10^3 \, mm^3$, $Z_{vv} = Z_{yy} = 11.6\times10^3 \, mm^3$

$$\sigma_b = \frac{1453.346\times10^3}{65.1\times10^3}\left[1 + \frac{726.671\times10^3}{1453.346\times10^3} \times \frac{65.1\times10^3}{11.6\times10^3}\right]$$

$$= 84.969 \, N/mm^2 < 165 \, N/mm^2$$

Step 7 - Design of Compression members

Top chord member AB & BN has maximum compressive force of 69883.5N with ~~peak~~ member length 2.236m

IS800, Pg 44, Table 10, Buckling class 'C' for angle section with ~~ed~~ $f_{cd} = 90 \, N/mm^2$ (Assume)

Compressive force, $P_d = A f_{cd}$

$$69883.35 = A \times 90$$

$$A = 776.482 \, mm^2$$

For single angle area $= 776.482/2 = 388.241 \, mm^2 = 3.88 \, cm^2$

Choose ISA 50×50×5 mm with area $= 4.79 \, cm^2 = 479 \, mm^2$

with $r_{xx} = 1.52$ cm (SP6, Pg 8, Table III)

Effective length $= 0.7$ to $0.85 L$ (IS 800 Pg 48 - 7.5.2.1)

$$= 0.85 \times 2.236 = 1.901 m$$

Slenderness ratio, $\lambda = \frac{kL}{r_{min}} = \frac{1.0 \times 1.901 \times 10^3}{1.52 \times 10}$ (k=1.0 for steel)

$= 125.066$

For BC – C, $\lambda = 125$, Table 9(C), IS800, Pg 42

$f_y = 250\ N/mm^2$

λ	f_{cd}
120	83.7
130	74.3

For $\lambda = 125$, $f_{cd} =$ 79 N/mm²

$P = A \times f_{cd} = 2 \times 479 \times 79 = 75682\ N > 69883.35\ N$

Hence safe.

Step 8 – Design of tension members

Bottom chord member AC has maximum tensile force of

79338.48N

Gross area, $A_g = \frac{T_u \gamma_{mo}}{f_y}$ (IS800 Pg 32, 6.2)

$= \frac{79338.48 \times 1.1}{250}$

$= 349.089\ mm^2$

For ~~single angle~~ area $= 349.089/2 = 174.545\ mm^2 = 1.7456\ cm^2$

Gross area is increased by 25 to 40%,

$\therefore$ Area $= 349.089 + \frac{25}{100} \times 349.089 = 436.361\ mm^2$

For single angle, Area $= \frac{436.361}{2} = 218.181\ mm^2 = 2.182\ cm^2$

(SP6, Pg 8, Table III)

Choose ISA 50×50× 3mm with Area $= 2.95\ cm^2 = 295\ cm^2$

$r_x = 1.53\ cm$

Strength of angle, $T_u = \frac{A_g f_y}{\gamma_{mo}} = \frac{2 \times 295 \times 250}{1.1} = 134090.91\ N$

$> 79338.48\ N$

Step 6 - Slenderness ratio = $\frac{l}{r} = \frac{2500}{15.3} = 163.399 < 350$ (IS 800, Pg 20, t3(c))

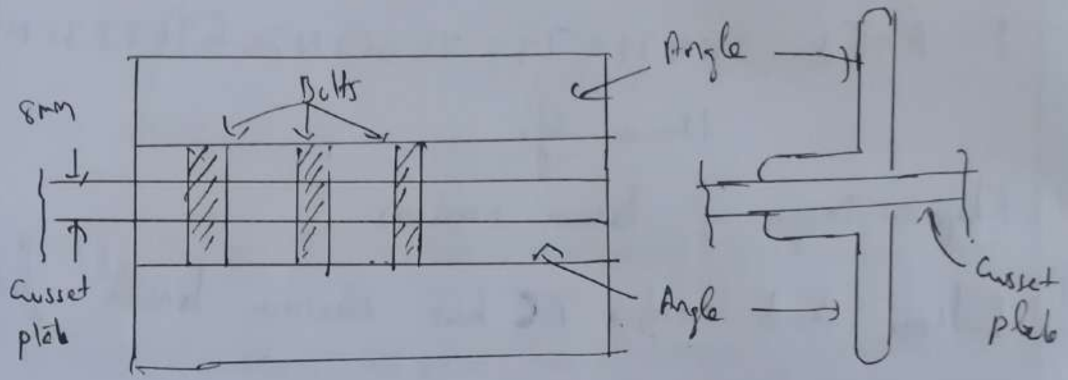
Step 7 - Design of joints

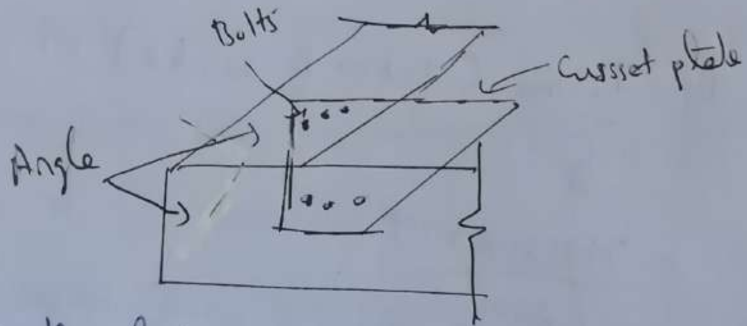
Thickness of gusset plate, t = 8mm

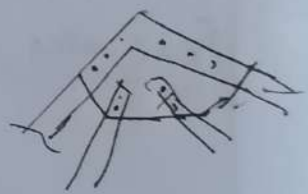
Nominal Diameter of rivets, d = 16mm

Gross diameter of rivets, $d_0 = 16 + 2 = 18$ mm

The angle sections are connected by gusset plate by bolts as in fig.







Strength of bolt in shear (IS 800 Pg 75, 10.3.3)

$V_{dsb} = V_{nsb} / \gamma_{mb}$

where $V_{nsb} = \frac{f_u}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}]$

→ $A_{nb} = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.828 \ mm^2$, $A_{sb} = \frac{\pi}{4} \times 16^2 = 201.062 \ mm^2$

→ $n_n = n_s = 1$

→ $f_u = 400 N/mm^2$

$\therefore V_{nsb} = \frac{400}{\sqrt{3}} [(1 \times 156.828) + (1 \times 201.062)]$

$= 82651.155 N$

$V_{dsb} = 82651.155 / 1.25 = 66120.924\ N$

Strength of bolts in bearing

$V_{dpb} = V_{npb} / \gamma_{mb}$

where $V_{npb} = 2.5\, k_b\, d\, t\, f_u$ ~~e = 1.5 d₀ = 1.5×18~~

k_b is least of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$e = 1.5 d_0 = 1.5 \times 18 = 27 \simeq 30\,mm$

$p = 2.5 d = 2.5 \times 16 = 40\,mm$

$\therefore k_b = \frac{30}{3\times18}, \frac{40}{3\times18} - 0.25, \frac{400}{410}, 1.0$

$= 0.556, 0.491, 0.976, 1.0$

$\therefore k_b = 0.491$

$V_{npb} = 2.5 \times 0.491 \times 16 \times 8 \times 410 = 64419.2\,N$ ~~(2)~~

~~Rivet value is least of (1) & (2), R = 51~~

$V_{dpb} = 64419.2 / 1.25 = 51535.36\,N$ — (2)

Rivet value is the least of (1) & (2),

$R = 51535.36\,N$

No. of bolts (member AB) $= \frac{\text{Force}}{\text{Rivet value}} = \frac{69883.35}{51535.36} = 1.356 \simeq 2$

Similarly provide rivets for other members.

Step 10 - Design of end support

Maximum normal reaction of bearing = 125 kN (Assume)

No of rivets required for connection of shoe angles with gusset plate $= \frac{\text{Reaction}}{\text{Rivet value}} = \frac{125\times10^3}{51355.36} = 2.426 \simeq 3$

4 rivets are provided to connect shoe angles with gusset plate. 4 rivets are also provided to connect shoe angles with base plate. Two ISA 80 mm × 80mm × 8mm, 450mm long are used for shoe angles.

Bearing plate

Normal reaction = 125 kN

Length of base plate = 450mm

Width of bearing plate = 80+80+10 = 170mm

Bearing pressure on concrete bearing pad = $\frac{P}{A}$

$$= \frac{125 \times 10^3}{450 \times 170}$$

$$= 1.634 \text{ N/mm}^2$$

Consider 1mm strip of base plate, bending moment,

$$M = \frac{1.634 \times (80-8)^2}{2} = 4235.33 \text{ Nmm} \quad -①$$

Moment of resistance of base plate = $185 \times 1 \times \frac{t^2}{6}$ ②

Equating ① & ② ⇒ $\frac{185t^2}{6} = 4235.33$

$$t = 11.72\text{mm}$$

∴ Thickness of base plate required, $t_1 = 11.72 - 8 = 3.72$mm

Provide 6mm thick base plate 450mm × 170mm × 6mm bearing plate below the base plate. An elliptical hole is kept on each side of shoe angles and base plate. The base plate can slide over bearing plate

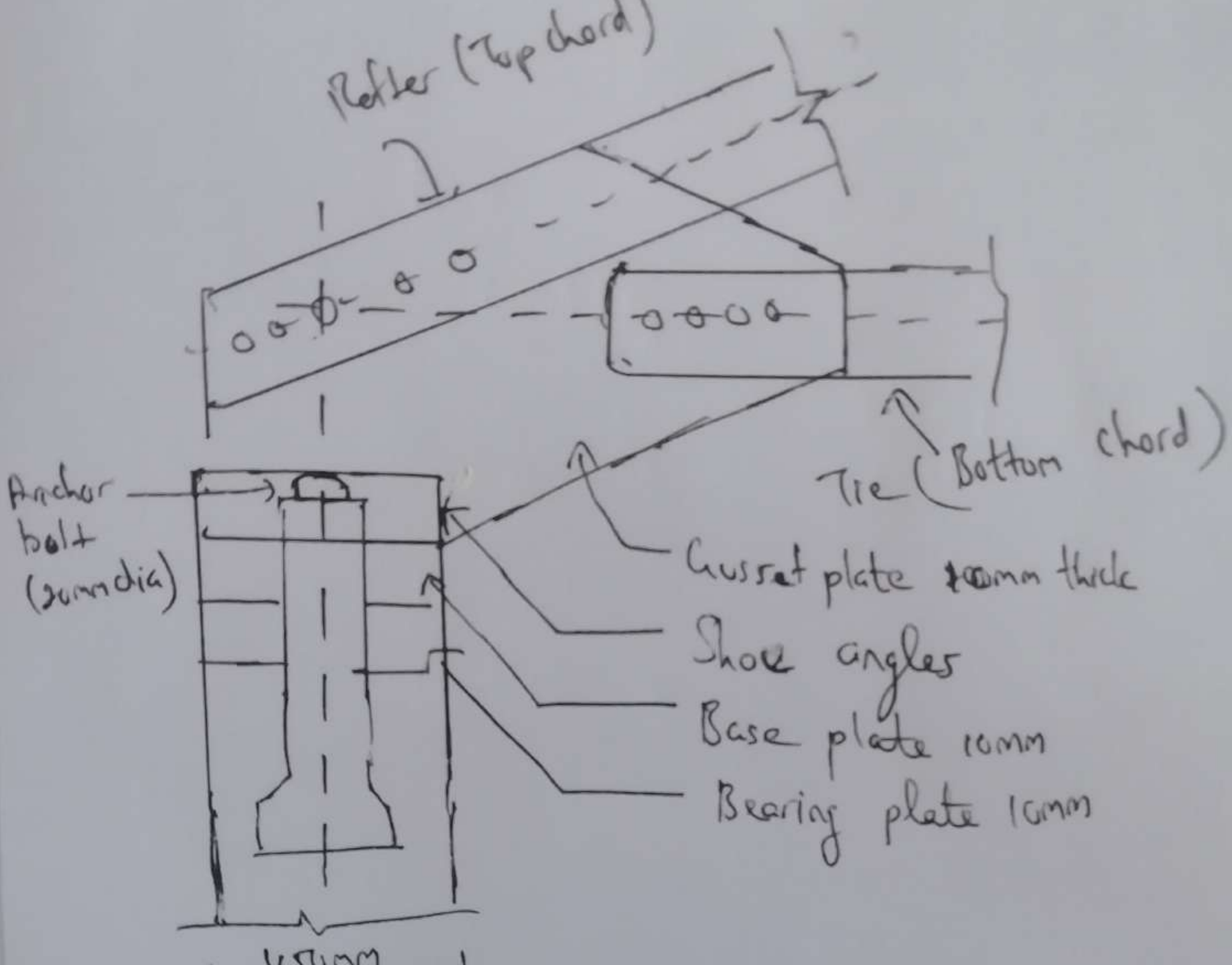
Anchor plate

Pull in anchor bolt = 7.50 kN

Allowable axial tension in anchor bolt is $0.6 \times 260 = 156 \text{ N/mm}^2$

Area required at the roof of thread $\frac{7.50 \times 10^3}{156} = 48.07 \text{ mm}^2$

Two nominal 20mm diameter anchor bolts are provided on each side of shoe angles.



Rafter (Top chord)
Anchor bolt (20mm dia)
Tie (Bottom chord)
Gusset plate mm thick
Shoe angles
Base plate 10mm
Bearing plate 10mm
450MM

Purlin

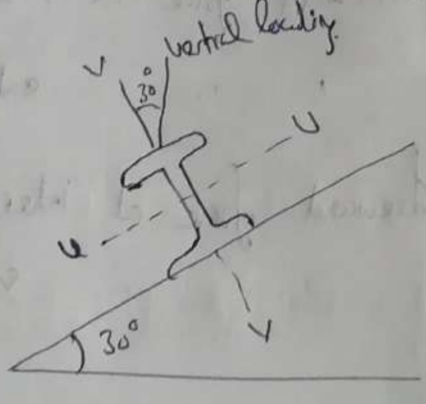
Purlins are structural members subjected to transverse loads and rest on top chord members of ~~rod~~ truss. The purlin supports the sheeting that ~~carries the~~ covers the roof truss.

Pb) Design an I section purlin to support galvanized corrugated iron sheet roof. The purlins are 1·25m apart over roof trusses spaced 5m centre to centre. The roof surface has an inclination of 30° to the horizontal. The weight of corrugated iron sheet is 0·1331 kN/m². The weight of fixtures at 0·053 kN/m². The design wind pressure for medium permeability is 1·50 kN/m² (outward) parallel to ridge.

Solution

~~Step 1 – Dead Load~~

~~Weight of iron sheet =~~



Given

Spacing of purlin = 1·25m

Spacing of truss = 5m

Inclination of roof surface, $\theta = 30°$

Weight of corrugated iron sheet = 0·1331 kN/m²

Weight of fixtures = 0·053 kN/m²

Wind pressure = 1·50 kN/m² (outward)

Solution

Step 1- Dead load

Weight of galvanized iron sheet = 0.1331 × 1.25 = 0.1664 kN/m

Weight of fixtures = 0.053 × 1.25 = 0.0663 kN/m

Self weight of purlin (assumed) = 0.12 kN/m

Total dead load = 0.1664 + 0.0663 + 0.12 = 0.3527 kN/m

≃ 0.36 kN/m

→ Component of dead load normal to roof = 0.36 × cos 30

= 0.312 kN/m

→ Component of live load parallel to roof = 0.36 × sin 30

= 0.18 kN/m

Step 2 - Live load

Live load for sloping roof with slope greater than 10° subject to a minimum of 0.4 kN/m² = 0.75 − (θ − 10) 0.02

= 0.75 − [(30 − 10) 0.02]

= 0.35 kN/m² < 0.4 kN/m²

∴ Live load = 0.4 kN/m²

~~Component of live load normal to roof = 0.4 × cos 30~~

Total live load = 0.4 × 1.25 = 0.5 kN/m

→ Component of live load normal to roof = 0.5 × cos 30

= 0.433 kN/m

→ Component of live load parallel to roof = 0.5 × sin 30

= 0.25 kN/m

Step 3- Wind load

Wind load (parallel to ridge) = 1.50 kN/m²

Step 4 - Combination of Loads

(i) DL + LL

(ii) DL + LL + WL (parallel to ridge). In case the design wind pressure acts outward (negative) the imposed live load shall not be considered.

Step 5 - Design of purlin for DL+LL

$$\text{Moment} = \frac{W_{DL} l^2}{10} + \frac{W_{LL} l^2}{9}$$

→ BM due to DL+LL parallel to the major principal axis (UU axis)

$$M_{UU} = \frac{0.312 \times 5^2}{10} + \frac{0.433 \times 5^2}{9} = 1.983\ \text{kNm}$$

→ BM due to DL+LL parallel to minor principal axis (VV axis)

$$M_{VV} = \frac{0.18 \times 5^2}{10} + \frac{0.25 \times 5^2}{9} = 1.144\ \text{kNm}$$

Required section modulus,

$$Z_{UU} = \frac{M_{UU}}{\sigma_b}\left(1 + \frac{M_{VV}}{M_{UU}} \cdot \frac{Z_{UU}}{Z_{VV}}\right) \quad \text{— (1)}$$

Assuming $Z_{UU}/Z_{VV} = 7$ for I section purlin and

$\sigma_b = 0.66 f_y = 0.66 \times 250 = 165\ \text{N/mm}^2$

$$Z_{UU} = \frac{1.983 \times 10^6}{165}\left(1 + \frac{1.144 \times 10^6}{1.983 \times 10^6} \times 7\right)$$

$$= 60.552 \times 10^3\ \text{mm}^3$$

From SP 6 - Pg 2, Table 1, choose ISLB 125 @ 11.9 kg/m

with $Z_{xx} = 65.1\ \text{cm}^3$ (Pg 3), $Z_{yy} = 11.6\ \text{cm}^3$

$Z_{UU} = Z_{xx} = 65.1 \times 10^3\ \text{mm}^3$; $Z_{VV} = Z_{yy} = 11.6 \times 10^3\ \text{mm}^3$

Sub in ①

$$\Rightarrow \sigma_b = \frac{M_{UU}}{Z_{UU}}\left(1+\frac{M_{VV}}{M_{UU}}\cdot\frac{Z_{UU}}{Z_{VV}}\right)$$

$$= \frac{1.983\times10^6}{65.1\times10^3}\left[1+\frac{1.144\times10^6}{1.983\times10^6}\times\frac{65.1\times10^3}{11.6\times10^3}\right]$$

$$= 129.02\,N/mm^2 < 165\,N/mm^2$$

Step 6 - Design of purlin for DL+~~LL~~+WL

The wind load acts outward (negative), hence live load is not considered with the combination. BM due to DL+WL along parallel to major principal axis

$$\text{Moment} = \frac{W_{DL}\,l^2}{10} + \frac{W_{WL}\,l^2}{10}$$

$$M_{UU} = \frac{0.312\times5^2}{10} + \left(\frac{-1.5\times5^2}{10}\right)$$

$$= -2.97\,kNm$$

~~$M_{VV} = \frac{0.18\times5^2}{10} + (0)$~~

BM due to DL+WL parallel to minor principal axis,

$$M_{VV} = \left(\frac{0.18\times5^2}{10}\right) + 0$$

$$= 0.45\,kNm$$

$$\text{Required } Z_{UU} = \frac{M_{UU}}{\sigma_b}\left(1+\frac{M_{VV}}{M_{UU}}\cdot\frac{Z_{UU}}{Z_{VV}}\right)$$

$$= \frac{2.97\times10^6}{165}\left(1+\frac{0.45\times10^6}{2.97\times10^6}\times7\right)$$

$$= 37.09\times10^3\,mm^3$$

Provided section has $Z_{xx} = 65.1\,cm^3$. Hence ok

Sub in ①

$$\sigma_b = \frac{2.97\times10^6}{65.1\times10^3}\left[1+\frac{0.45\times10^6}{2.97\times10^6}\times7\right]$$

$$= 94.01\ N/mm^2 < 165\ N/mm^2$$

(Pb) Design an iron angle purlin for a trussed roof for the following data.

Span of roof truss = 12 m

Spacing of roof truss = 5 m

Spacing of purlins along the slope of roof truss = 1.2 m

Slope of roof truss = 1 vertical to 2 horizontal

Wind load on roof surface normal to roof = 1.04 kN/m^2

Vertical load from roof sheeting = 0.200 kN/m^2

__Solution__

__Step 1 – Slope of roof truss__

$$\text{Slope} = \tan\theta = \frac{\text{vertical}}{\text{horizontal}} = \frac{1}{2} = 0.5$$

$$\theta = \tan^{-1}(0.5) = 26.565^\circ$$

__Step 2 – Vertical load on purlin (DL+LL)__

Vertical load from roof sheeting = 0.2 kN/m^2 = 0.2×1.2 = 0.24 kN/m

Self weight of purlin (assume) = 0.12 kN/m

Total load (vertical) = 0.36 kN/m

__Step 3 – Wind load__

Wind load normal to roof = 1.04 kN/m^2 = 1.04 × 1.2 = 1.248 kN/m

__Step 4 – Design of purlin__

Total load normal to roof = 0.36 + 1.248 = 1.608 kN/m

Moment, $M = \frac{WL^2}{10} = \frac{1.608 \times 5^2}{10} = 4.02$ KNM

→ Required section modulus, $Z = \frac{M}{\sigma}$ where $\sigma = 0.66 f_y$

$= 0.66 \times 250$

$= 165 N/mm^2$

$\therefore Z = (4.02 \times 10^6) / 165 = 24.63 \times 10^3 mm^3$

→ Depth of angle purlin $= \frac{L}{45} = \frac{5000}{45} = 111.11mm$

→ width of purlin

As wind load is considered, stress can be increased by $33\frac{1}{3}\%$ (1.333), $\sigma = 1.333 \times 165 = 219.945 N/mm^2$

$$Z = \frac{4.02 \times 10^6}{219.945} = 18.277 \times 10^3 mm^2$$

→ Depth of angle purlin $= \frac{L}{45} = \frac{5000}{45} = 111.11mm$

→ Width of angle purlin $= \frac{L}{60} = \frac{5000}{60} = 83.33mm$

Choose ISA 125×95×6mm @ 0.129 kN/m with $Z = 23.4 \times 10^3 mm^3$

Gantry Girder

Definition

Overhead travelling cranes are used in industrial buildings to lift and transport ~~heavy~~ heavy machineries and assembled parts from one place to another. For movement of the crane, wheels are attached to their ends. The wheels move over rails which are inturn placed over steel I beams called as Gantry Girder.

Loads Considered

- → Reaction from the crane girder acting vertically downwards
- → Longitudinal thrust due to starting or stoping of crane acting in longitudinal direction
- → Lateral thrust due to starting and stopping of crab acting horizontally, normal to gantry girder
- → Longitudinal horizontal force along the crane rail.

Assumptions made

- → The vertical loads are resisted by the entire section of girder
- → The horizontal loads are resisted by the compression flange

Design Procedure

1) Maximum wheel load is determined

(i) Weight of trolley and lifted load are considered as moving load

(ii) Self weight of crane girder is considered as udl

(iii) The maximum wheel load is half the vertical force transferred from crane girder to gantry girder

2) Maximum Bending Moment is determined

This consist of

(i) BM due to wheel load (with impact)

(ii) BM due to dead load of girder and rails. The BM due to dead load is maximum at centre of span

3) Maximum Shear Force is determined

This consist of

(i) SF due to wheel load (with impact)

(ii) SF due to dead load of ~~gantry~~ girder and rails

The SF is maximum when one of the wheels is at support

4) Selection of trial section

Trial section is choosed such that

(i) Economic depth is 1/12th of span

(ii) Compression flange width is kept 1/25th of span

(iii) Section modulus should be 40 to 50% more than the calculated.

5) Calculation of sectional properties

Properties like I_{xx}, I_{yy}, Z_{ex}, Z_{ey}, Z_{px}, Z_{py} are calculated

6) Section classification

Section is classified based on b/t_f and d/t_w value. as plastic, semicompact or compact. Plastic sections are preferred

8) Check for moment capacity

The girder is laterally supported and hence bending strength is given as

$$M_{dz} = \frac{\beta_b Z_p f_y}{\gamma_{m0}} < \frac{1.2 Z_e f_y}{\gamma_{m0}}$$

The bending strength obtained should be greater than applied bending moment.

7) Check for shear

$$\text{Design shear}, = \frac{A_v f_{yw}}{\sqrt{3} \times \gamma_m}$$

This should be greater than applied shear force

8) Check for biaxial bending

Interaction formula $\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1.0$ is checked

9) Check for web buckling and bearing

Buckling - $\frac{d}{t_w} < 67$, if not stiffeners to be provided

Bearing resistance, $F_w = \frac{(b_1 + n_2) t_w f_y}{\gamma_{m0}}$

10) Design of bolts or welds

11) Check for deflection

$$\delta = \frac{WL^3 \times \left(\frac{3a}{4L} - \frac{a^3}{L^3}\right)}{6EI} < \frac{\text{Span}}{750}$$

Where L → Span

$a = \frac{L-C}{2}$

C → Wheel base

Example 14.3. Design a simply supported gantry girder to carry one electric over head travelling crane.

Crane capacity = 300 kN

Weight of crane excluding trolley = 190 kN

Weight of trolley = 100 kN

Minimum approach of crane hook = 1.2 metres

Distance between centres of crane wheel = 3.5 metres

Distance between centres of crane wheel = 18 metres

Span of gantry girder = 6 metres

Weight of rail section = 0.300 kN/m

Height of rail section = 75 mm

Design :

Step 1 : Maximum wheel load

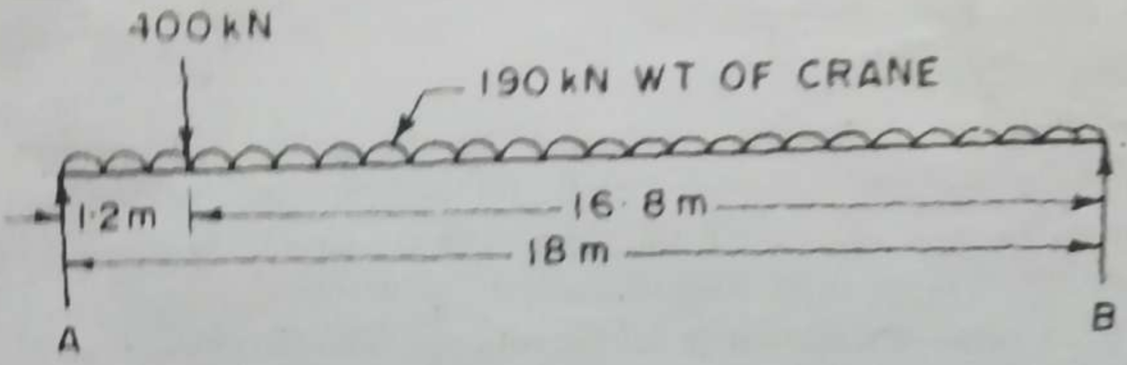


Fig. 14.5.

Weight of trolley + lifted load = (300 + 100) = 400 kN

The weight of crane (excluding trolley) 190 kN acts as uniformly distributed live load as shown in Fig. 14.5.

The vertical reaction on each wheel of crane would be maximum, when trolley is at nearest distance to trolley girder as shown in Fig. 14.5.

Take moment about B, then reaction at A

$$R_A = \frac{1}{18}\left[400 \times 16.8 + 190 \times \frac{18}{2}\right] = 468 \text{ kN}$$

This vertical load at one end of the crane bridge is transferred to the gantry girder through two wheels.

Maximum vertical load on each wheel of crane, = (1/2 x 468) = 234 kN

Step 2' : Maximum bending moment (due to D.L. + L.L. + I.L.)

The maximum bending moment in the gantry girder under a moving load occurs when the life of action of that load and c.g. of the loads are at equal distance from the centre of span. That is,

EC = CF = 0.875 (Fig. 14.6)

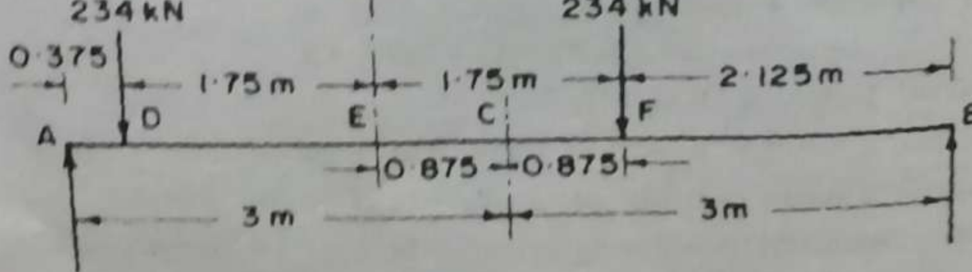


Fig. 14.6.

The reaction at the supports A and B are as follows :

$$R_A = 234 \times \frac{1}{6}\,[(6 - 0.375) + 2.125] = 302.23 \text{ kN}$$

$$R_B = 2 \times 234 - 302.23) = 165.77 \text{ kN}$$

Maximum bending moment due to moving load

$$M_F = (165.77 \times 2.125) = 352.3 \text{ kN}$$

Add 25 per cent impact moment vix., 88.1 kN-m

(1) **Live load moment** = (352.3 + 88.1) = 440.4 kN-m

Assume self-weight of the girder as 2 kN/m

Weight of rail section is 0.300 kN/m, Total dead load = 2.3 kN/m

Maximum bending moment due to dead load

$$\left(\frac{wl^2}{8}\right) = \left(\frac{2.3 \times 6 \times 6}{8}\right) = 10.35 \text{ kN–m}$$

(2) **Dead load moment** = 10.35 kN-m

(3) **Total vertical moment** = (440.4 + 10.35) = 450.75 kN/m

Assume allowable bending compressive stress, = (0.66 x 250) = 165 N/mm^2

The section modulus required for bending moment is vertical plane (approximately)

$$Z = \left(\frac{450.75 \times 1000 \times 1000}{165}\right) = 2731.8 \times 10^3 \text{ mm}^3$$

From steel section tables, try WB 600, @ 1.337 kN/m and LC 300, @ 0.331 kN/m. The section of the gantry is shown in Fig. 14.7.

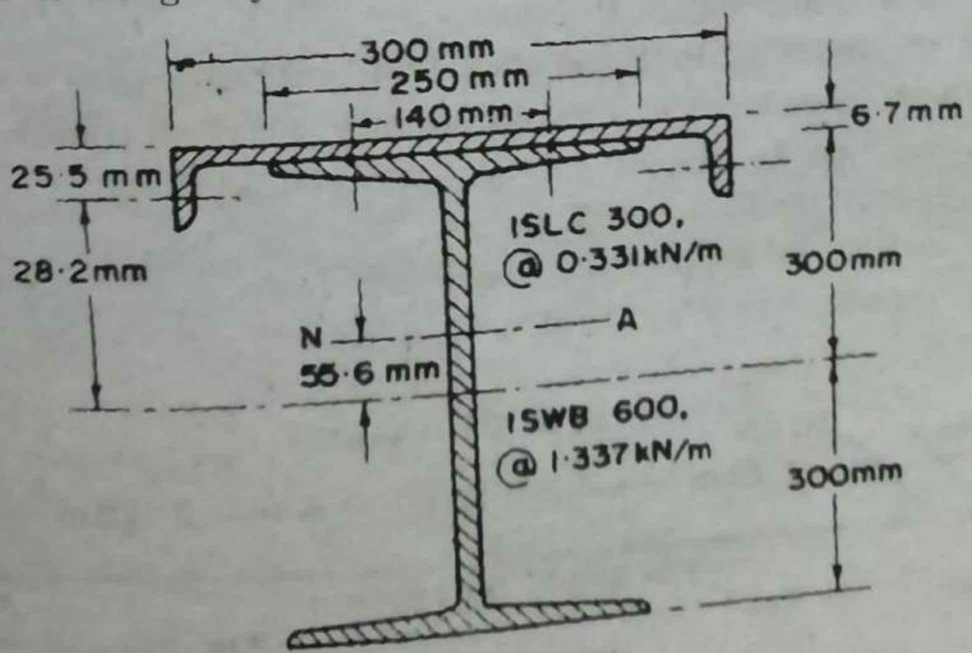


Fig. 14.7.

Sectional area of beam section is 17038 mm^2

Section area of channel sectio is 4211 mm^2

∴ Total section area is 21249 mm^2

Thickness of flange of beam section, t_f is 21.3 mm

Let y be the distance of neutral axis of built-up section from neutral axis of beam section

Moment of inertia of built-up section about xx-axis

I_{yy} (gross) = [106198.5 + 170.38 x 5.56^2 + 346 + 42.11 x $(28.12 - 5.56)^2$] x 10^4 mm^2 = 133334.5 x 10^4 mm^4

Moment of inertia of built-up section about yy-axis

I_{yy}(gross) = [4702.5 + 6047.9] x 10^4 = 10750.4 x 10^4 mm^4

Bending stress due to vertical loading

Actual bending compressive stress for vertical loading

$$\sigma_{bc.x.cal} = \left(\frac{450.75 \times 1000 \times 1000 \times 251.1}{133334.5}\right) = 84.8867 \text{ N/mm}^2$$

Actual bending tensile stress for vertical loading

$$\sigma_{bc.x.cal} = \left(\frac{450.75 \times 1000 \times 1000 \times 355.6}{122224.5 \times 10^4}\right) = 119.4 \text{ N/mm}^2$$

$$< (1.10 \times 165) = 181.5 \text{ N/mm}^2$$

Step 3 : Maximum bending moment due to horizontal (transverse) force

Horizontal force transverse to the rail

10 percent of (weight of trolley + lifted load) = 1/10 x (300 + 100) = 40 kN

Horizontal force transverse to the rail on each wheel or crane, = 20 kN

Horizontasl reaction at support A (Figs. 14.8 and 14.8)

= 20/234 x 302.33 = 25.83 kN

Horizontal reaction at support B = 14.17 kN

Horizontal moment, 14.17 x 2.125 = 30.1 kN-m

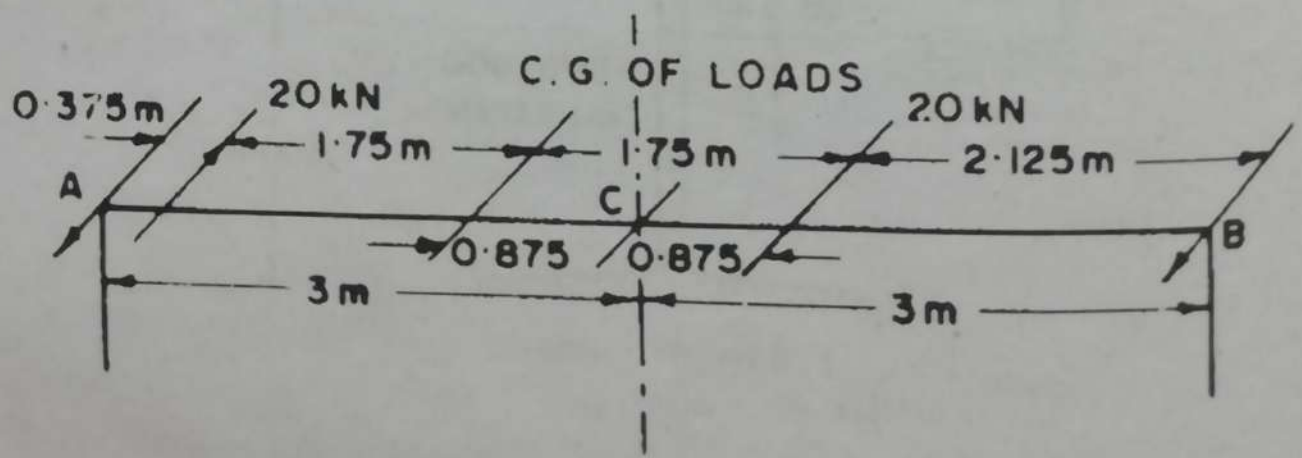


Fig. 14.8.

Step 4 : Bending moment in horizontal plane

Horizontal moment = 30.10 kN-m

The moment of inertia of compression flange about yy-axis (considering I_{yy} of compression flange of beam section as half of that for beam section)

I_{yy} = [6047.9 + 1/2 x 4702.5] x 10^4 = 8399.6 x 10^4 mm^4

Bending compressive stress in horizontal plane (Bottom flange is neglected).

$$\sigma_{bc.y.cal} = \left(\frac{30.1 \times 1000 \times 1000 \times 150}{8399.6 \times 10^4}\right) = 53.58 \text{ N/mm}^2$$

Step 5 : Allowable stress in horizontal plane

Let $\bar{y}_1$ be the distance of compression flange from top fibre

$$\bar{y}_1 = \left[\frac{4211 \times 25.5 + 250 \times 21.3\,(6.7 + 10.65)}{4211 + 250 \times 21.3}\right] = 20.9 \text{ mm}$$

Distance between c.g. to c.g. of top and bottom flanges

h = (605.7 - 20.9 - 10.6) = 575.2 mm

Section modulus about xx-axis reference to the compression flange

$$Z_{xx} = \left[\frac{133334.5 \times 10^4}{(300 + 6.7 - 55.6)}\right] = 5308.8 \times 10^3 \text{ mm}^3$$

$$\omega = \left(\frac{\text{Moment of inertia of comp. flange about yy–axis}}{\text{Moment of inertia of built up section about yy–axis}}\right)$$

$$\omega = \left(\frac{8399.6 \times 10^4}{10750.4 \times 10^4}\right) = 0.78$$

From IS : 800-1984, k_1 = 0.28

Effective length of compression flange = 6000 mm

Radius of gyration of the completion section about yy-axis

$$r_y = \left(\frac{10750.4 \times 10^4}{21249}\right)^{1/2} = 79.58 \text{ mm}$$

$$\text{Slenderness ratio} = \left(\frac{6000}{79.58}\right) = 75.39$$

Overall depth, D = 606.7 mm

Mean thickness of flange T = (t_f = 21.3 + 6.7) = 28.0

Ratio (D/T) = 21.668

From IS : 800-1984, Table 14.2, X = 632.02 and Y = 503.27

From Eq. 14., the elastic critical stress

f_{cb} = k_1 (X + k_2.Y) c_2/c_1, = 1.0 (632.02 + 0.28 x 503.27) x (3067/300)

$= 790.20\ N/mm^2$ (MPa)

Let the value of yield stress for the structural steel be $250\ N/mm^2$

$$\text{Ratio}\left(\frac{T}{t_w}\right) = \left(\frac{28}{11.2}\right) = 2.5 > 2.0$$

$\therefore f_{cb}$ is not increased by 20 percent. From IS : 800-1984, Table 14.2, $\sigma_{cb} = 145\ N/mm^2$

Step 6 : Check for combined bending compressive stress in extreme fibre

$(\sigma_{bcx.cal} + \sigma_{bcy.cal}) = (84.498 + 53.58)$

$137.98\ N/mm^2 < 1.1 \times 145 = 159.5\ N/mm^2$

Hence design is safe and satisfactory.

Step 7 : Horizontal (longitudinal) force along the rails

$$5\% \text{ of the static wheel load} = \left(\frac{1}{20} \times 2 \times 234\right) = 23.4\ kN$$

Height of rail = 75 mm

Bending moment in the longitudinal direction, $= 23.4 \times (75 + 251.1) = 7630.74$ mm-kN

Stress in longitudinal direction

$$\left(\frac{P}{A} + \frac{M}{Z}\right) = \left(\frac{23.4 \times 1000}{21249} + \frac{7630.74 \times 1000}{5308 \times 104}\right) N/mm^2$$

$(1.10 + 14.376) = 2.538\ N/mm^2$ (Very small)

Shear force

Maximum shear force in the gantry girder

$$\left(234 + 234 \times \frac{2.5}{6.0}\right) = 331\ kN$$

Add 25% for impact = 82.75 kN

$$\text{Dead load shear} = \left[\frac{(1337 + 331)\ 6}{2 \times 1000}\right] = 5.61\ kN$$

Total shear = 419.36 kN

Intensity of horizontal shear stress per mm length

$$f_y = (FQ/I)\ (Q = A \cdot \bar{y})$$

Consider the portion of web of flange only.

Area = $(6.7 \times 300) = 2000\ mm^2$

From NA, $\bar{y} = 251.1 - 1/2 \times 6.7 = 247.75$ mm

$$\tau_{va} = \left(\frac{419.36 \times 2000 \times 247.75 \times 1000}{13334.5 \times 10^4}\right) = 155.84\ N/mm^2$$

Step 8 : Rivet value

Use 22 mm diameter power driven rivets.

Strength of power driven rivets in single shear

$$\left(\frac{\pi}{4}\frac{(23.5)^2 \times 100}{1000}\right) = 43.35 \text{ kN}$$

Strength of rivet in bearing

$$\left(23.5 \times \frac{6.7 \times 300}{1000}\right) = 47.235 \text{ kN}$$

Rivet value, R = 43.35 kN

$$\text{Pitch of rivets} = \left(\frac{43.35 \times 1000}{155.84}\right) = 278.17 \text{ mm}$$

Rivets are provided in two lines

$$\therefore\ 2.p = 556.34 \text{ mm}$$

Maximum allowable pitch in compression

$$= (12 \times 6.7) = 80.4 \text{ mm}$$

Provide rivets at 80 mm pitch throughout the length of gantry girder.

Plate Girder

A plate girder is a I beam ~~but~~ built up of steel plates using bolting or welding. It is a deep flexural member used to carry heavy loads on longer spans. Plate girders are normally used in bridges and sometimes in buildings when it is required to support heavy concentrated loads.

Advantages & Dis of plate girders over trusses

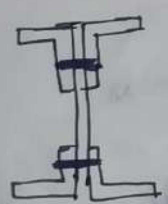
The usual practical alternative to plate girders is trusses as they are economical. However plate girders have following advantages :-

- Cost of fabrication is low when compared to trusses
- Erection is faster and cheaper when " "
- Plate girders requires small vertical clearance
- Resist vibration and impact loads
- Plate girders are safe (bending of plates is safe than members)
- Can be easily painted

Dis advantages

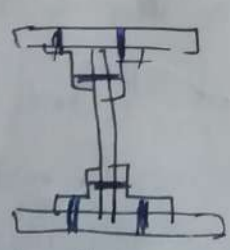
- Heavier than truss
- Low torsional stiffness
- Need large number of connections b/w web and flange
- large exposed area to wind.

Types of sections

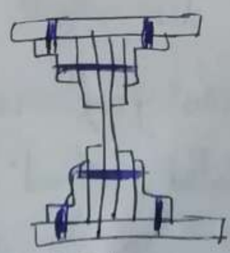


- Simplest type.
- Angles connected to web & used as flange.

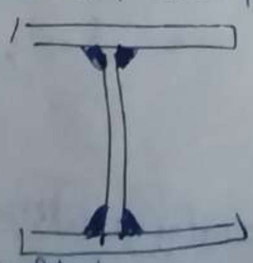
Bolted w/o cover plates



Bolted with cover plates



Bolted with cover plates & side plates.



welded

Box girder

Types of plate girders

Bolted / Riveted plate girder	Welded plate girder
* spans - 15 to 30m	* Span upto 100m
* Self weight is high due to provision of extra flange angles and flange cover plates	× Economic compared to bolted / riveted plate girder due to reduction in self weight

Elements of Plate girder

-> Web plate
-> Flange plate
-> Flange angles or flange cover plates
-> Stiffeners - ~~bearing, transverse and longitudinal~~
-> Splices - web & flange.
-> Connection between flange and web
-> a " web and stiffeners

Stiffeners

1) **Bearing stiffeners (or) Load carrying stiffeners**

-> Used in ends to transfer the load from beam to support
-> Used to avoid crushing of web at the ends.
-> Used when concentrated load acts on the girder.

2) **Intermediate stiffeners**

-> These stiffeners are provided to ~~effect~~ avoid buckling of web
-> Also called as stability stiffeners bearing & intermediate
-> They are of two types namely horizontal (longitudinal) stiffeners and ~~transverse~~ vertical (transverse) stiffeners

Horizontal stiffeners - Used to increase buckling strength resistance when buckling caused by bending
- Generally located in compression zone (at a depth of 0.2d from compression flange)
- Provided b/w vertical stiffener

Vertical stiffeners - Used to ~~resist~~ increase buckling strength ~~against~~ resistance when buckling caused by shear

3) Load carrying stiffeners - Provided when comp forces applied thru flange exceeds buckling strength of web.

Steps involved in design of plate girder

1) Assume self weight of girder, $w = \frac{W}{200}$ where
W is total factored load on girder in kN
w is self weight of girder in kN/m

2) Calculate total bending moment (M) and shear force (V)

3) Calculate economical depth of plate girder

$$d = \left(\frac{Mk}{f_y}\right)^{1/3}$$

Where M – BM, f_y = 250 N/mm²

$$k = \frac{d}{t_w} = 67$$

(i) $k = \frac{d}{t_w} \leq 67$ (plate girder designed as ordinary beam w/o stiffeners ~~except bearing stiffeners~~)

(ii) $k = \frac{d}{t_w} = 67$ to 200 (when transverse stiffeners are not provided except bearing stiffeners)

(iii) $k = \frac{d}{t_w} < 200$ to 270 (when only transverse stiffeners are provided)

(iv) $k = \frac{b}{t_w} < 250$ to 340 (when transverse & longitudinal stiffeners provided at one level)

(v) $k = \frac{d}{t_w} = 340$ to 400 (when a second longi stiffener is provided)

(Pg 59 - 8.4.2.1 & Pg 63 - 8.6.1)

Calculate web thk from assumed d/tw value

4) Determine flange area required

$$A_f = \frac{M \gamma_{m0}}{f_y d}$$

Flange width is suitably taken as 0.3 times depth of web and thk to satisfy section classification

($\frac{b}{t_f}$ < 9.4 → Plastic, < 10.5 → Compact, < 15.7 – Semi Compact)
S – Section classification

5) Check shear resistance of web using simple post critical method (8.4.2.2 – Pg 59) or tension field method (8.4.2.2 – Pg 60)

8) Check for bending strength depending upon whether the plate girder is laterally supported (8.2.1.2 Pg 53) or laterally unsupported (8.2.2 – Pg 54) weld & connectors

10) Design connection between flange & web plate, by welding or bolting

Weld

$$\text{Shear force} = \frac{V A_f \bar{y}}{2 I_f} \quad (1)$$

$$\text{Strength of weld per unit length} = \frac{L_w t_e f_u}{\sqrt{3}} = \frac{t_e f_u}{\sqrt{3}} \quad (2)$$

Equating (1) & (2) te is obtained.

$$t_e = 0.7 \times S \Rightarrow S = t_e / 0.7$$

From above equation size of weld is calculated

Bolt

Bolt value = Least of strength of bolt in shear bearing, tension

No of bolts = Load / Bolt value.

1) Design a welded plate girder 24m in span and laterally restrained throughout. It has to support a uniform load of 100 kN/m throughout the span exclusive of self wt. Design the girder without intermediate transverse stiffeners. The steel for the flange and web plates is of grade Fe410. Yield stress may be assumed as 250 MPa. Design as, end load bearing stiffener & connections.

Step 1 - load Calculation

Factored LL load = $100 \times 1.5 = 150$ kN/m

Self weight, $w = \frac{W}{200} = \frac{150 \times 25}{200} = 18$ kN/m

Factored self wt $= 1.5 \times 18 = 27$ kN/m

Step 1 - load Calculation

Live load = 100 kN/m

Self weight $= \frac{W}{200} = \frac{100 \times 24}{200} = 12$ kN/m

∴ Total load $= 100 + 12 = 112$ kN/m

Total FL $= 1.5 \times 112 = 168$ kN/m

Step 2 - BM & SF

$SF = \frac{wl}{2} = \frac{168 \times 24}{2} = 2016$ kN

$BM = \frac{wl^2}{8} = \frac{168 \times 24^2}{8} = 12096$ kNm

Step 3 - Economical depth & thk of web

Economical depth, $d = \left(\frac{Mk}{f_y}\right)^{1/3}$

$k = \frac{d}{t_w} = 67$ to 200 (when no intermediate transverse stiffeners are provided)

Assume $k = 110$ (decrease this value)

$$d = \left(\frac{12096 \times 10^6 \times 110}{250}\right)^{1/3}$$

$= 1745.95$ mm

Economical depth, $d = 1800$ mm

$\frac{d}{t_w} = 110 \Rightarrow \frac{1800}{t_w} = 110 \Rightarrow t_w = 16.36$ mm $\simeq 16$ mm

Provide web plate of size 1800 × 16 mm. ($A_w = 1800 \times 16 = 28{,}800$ mm^2)

Step 4 - Flange area

Required flange area, $A_f = \frac{M \gamma_{mo}}{f_y d}$

$$= \frac{12096 \times 10^6 \times 1.1}{250 \times 1800}$$

$A_f = 29568$ mm^2

Flange width = 0.3 × depth of web

$= 0.3 \times 1800$

$= 540$ mm $\simeq 600$ mm

Thickness of flange $= \frac{\text{Flange area}}{\text{Flange width}} = \frac{29568}{600}$

$= 49.28$ mm

Provide Flange plate of size 600 × 50 mm (A_f = 2 × 600 × 50 = 60,000 m

Step 5- Section classification

$$\frac{b}{t_f} = \frac{600/2}{50} = 6 < 9.4\,\varepsilon \rightarrow \text{Plastic}$$

Step 6 - Check for shear resistance

Use simple post critical method (w/o transverse stiffeners at intermediate)

Pg 59, $V_n = V_{cr} = A_v \tau_b$

where $A_v = d t_w = 1800 \times 16 = 28800 \text{ mm}^2$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}}$$

$$\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)(d/t_w)^2}$$

$k_v = 5.35$ (transverse stiffeners provided only at support and not at intermediate points)

$$\tau_{cre} = \frac{5.35 \times \pi^2 \times 2\times10^5}{12(1-0.3^2) \times 170^2} = 79.924 \text{ N/mm}^2$$

$$\lambda_w = \sqrt{\frac{250}{\sqrt{3}\times 79.924}} = 1.344$$

$\lambda_w \geq 0.8 \Rightarrow$

$\tau_b = f_{yw}/\sqrt{3}\lambda_w^2 = 250/(\sqrt{3}\times 1.344^2) = 79.906 \text{ N/mm}^2$

$\therefore V_{cr} = 28800 \times 79.906 = 2.301 \times 10^6 \text{ N}$

$= 2301.29 \text{ kN}$

$2016 < 2301.29 \text{ kN}$

Bearing stiffener is essential

Step 7 - Check for bending strength:

$V < 0.6 V_d$ (f_y 53)

$\Rightarrow \quad M_d = \beta_b Z_p f_y / \gamma_{m0} < 1.2 Z_e f_y / \gamma_{m0}$

where $\beta_b = 1.0$ (plastic section) or $\frac{Z_e}{Z_p}$ (semi compact section)

Since flanges resist ~~bending~~ bending moment, Z_p & Z_e are calculated for flanges only.

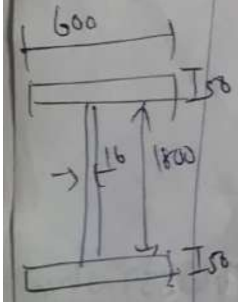
$Z_p = Z_{p, flange}$

$= Ah/2$

~~$= \left(1800 \times 16 \times \frac{1900}{2}\right) + \left(2 \times 600 \times 50 \times \frac{1900}{2}\right)$~~

$= \left(60{,}000 \times \frac{1900}{2}\right)$

$= 5.7 \times 10^7 \ mm^3$



Since flanges resist BM, I is calculated only for flanges

$Z_e = I/y$ where $I = \left(\frac{600 \times 50^3}{12}\right) + (600 \times 50)\left(\frac{50}{2} - 950\right)^2 + \left(\frac{600 \times 50^3}{12}\right) + (600 \times 50)\left(50 + 1800 + \frac{50}{2} - 950\right)^2$

$= (2.5675 \times 10^{10}) + (2.5675 \times 10^{10}) + (7.77 \times 10^{9})$

5.135×10^{10}

$= \quad mm^4$

$Z_e = I/y = (5.135 \times 10^{10}) / 950 = 5.405 \times 10^7 \ mm^3$

When section is plastic

$M_d = \frac{1.0 \times 5.7 \times 10^7 \times 250}{1.1} < \frac{1.2 \times 5.405 \times 10^7 \times 250}{1.1}$

$= 1.295 \times 10^{10}$ Nmm $< 1.474 \times 10^{10}$

$= 12950$ kNm < 14740

$= 12950$ kNm

12096 < 12950 kNm

When section is semi compact

$$M_d = \frac{Z_e}{Z_p} Z_p f_y / \gamma_{m0} < 1.2 Z_e f_y / \gamma_{m0}$$

$$= \frac{Z_e f_y}{\gamma_{m0}} < 1.2 Z_e f_y / \gamma_{m0}$$

$$= \frac{Z_e f_y}{\gamma_{m0}} = \frac{5.405 \times 10^{7} \times 250}{1.1}$$

$$= 1.228 \times 10^{10} \text{ Nmm}$$

M_d = 12,280 kNm

12096 < 12280 kNm

Step 8 - Design of end bearing stiffener or load carrying stiffener

(i) Design force

Pg 67 - 8.7.4

$F_w = (b_1 + n_2)\, t_w f_{yw} / \gamma_{m0}$

where $b_1 = 125$ mm (Assume)

$n_2 = 2.5(t_f + t_1) = 2.5(50) = 125$ mm

$\therefore F_w = \frac{(125 + 125) \times 16 \times 250}{1.1} = 9.09 \times 10^5$ N

$= 909$ kN

909 kN ≯ 2016 kN

Hence end bearing stiffeners are required

[Pg 68 – 8.7.6], Bearing stiffeners should be designed for applied load or reaction less the local capacity of web (909 kN)

Design force = Reaction – Local capacity of web

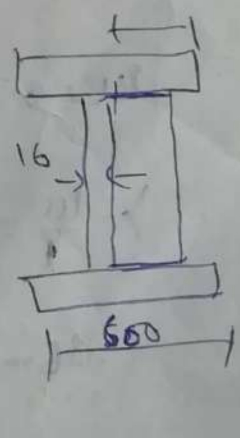
= SF – 〃

$F_x = 2016 - 909 = 1107$ kN

(ii) Size

[Pg 65 – 8.7.1.2], Outstand of stiffener $= 14\, t_q\, \varepsilon$

where $t_q \rightarrow$ Thk of stiffener = 16 mm (web thk – assume)



$\varepsilon = \left(\frac{250}{250}\right)^{1/2} = 1$

∴ Outstand $= 14 \times 16 \times 1 = 224$ mm

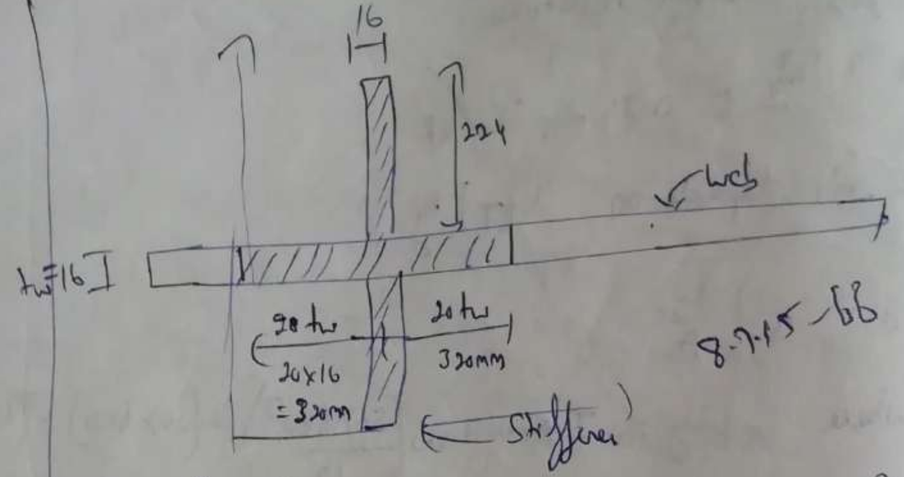
Outstand available $= \frac{600 - 16}{2} = 292$ mm

∴ Provide stiffener of size 224 × 16 mm

(iii) Check for buckling

Pg 68 – 8.7.5.1 $\quad \lambda = \frac{KL}{r}$ where $r = \sqrt{\frac{I}{A}}$

Curve – c



Core Area of stiffener with web area $= (2\times224\times16)+(2\times320\times16)$

$= 17408 mm^2$

$I = I_{xx} + A(y-\bar{y})^2$ where $\bar{y} = \frac{224+16+224}{2} = 232mm$

$= \left[\left(\frac{16\times224^3}{12}\right)+(16\times224)\left(\frac{224}{2}-232\right)^2\right]$

$+\left[\left(\frac{16\times224^3}{12}\right)+(16\times224)\left(224+16+\frac{224}{2}-232\right)^2\right]$

$+\left[\left(\frac{640\times16^3}{12}\right)+(640\times16)\left(224+\frac{16}{2}-232\right)^2\right]$

$= (6.66\times10^7)+(6.66\times10^7)+(2.185\times10^5)$

$I = 1.336\times10^8 mm^4$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1.336\times10^8}{17408}} = 87.605 mm$

$KL = 0.7L = 0.7\times1800$ (depth) $= 1260mm$

$\lambda = \frac{KL}{r} = \frac{1260}{87.605} = 14.383$

BC-C, $\lambda = 14.383 \Rightarrow$

λ	fcd
10	227
20	224

$\frac{3}{10}\times4.383 = 1.315$

$\therefore f_{cd} = 227-1.315 = 225.685 N/mm^2$

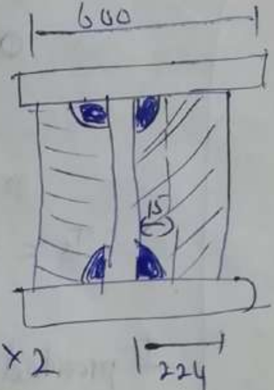
$P_d = A\times f_{cd} = 17408\times225.685 = 3.929\times10^6 N = 3929 kN$

$> 2016 kN$

(iv) Check for bearing

Pg 68 - 8.7.5.2, $F_{psd} = \frac{A_q f_{yq}}{0.8\,\gamma_{m0}} \geq F_x$

Since the stiffener will be coped to manage with accomodate the fillet weld of flange plate to web, the available effective width of stiffeners will be lesser than actual width. Let the stiffener plate be coped by 15mm



Width available for bearing = 224 - 15

= 209mm

$\therefore$ Area of stiffener in contact with flange = 209 × 16 × 2

= 6688 mm^2

$$F_{psd} = \frac{6688 \times 250}{0.8 \times 1.10}$$

$= 1.9 \times 10^6 N = 1900 kN$

$F_x \rightarrow$ load transferred = Design force = 1107 kN

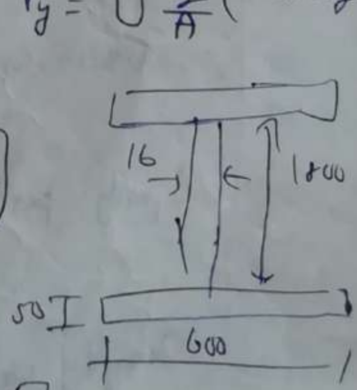
1900 > 1107 kN

Check for torsional restraint

Pg 68 - 8.7.9

$\therefore I_s \geq 0.34 \alpha_s D^3 T_{cf}$

where α_s depends on L_{LT}/r_y where $r_y = \sqrt{\frac{I_y}{A}}$ (radius of gyration of beam)



$$I_y = \left[\left(\frac{50 \times 600^3}{12}\right) + 50 \times 600 \times \left(\frac{600}{2} - \frac{600}{2}\right)^2\right]$$

+ ,,

db^3

$$+\left[\left(\frac{1800 \times 16^3}{12}\right) + (16 \times 1800)\left(\frac{600}{2} - \frac{600}{2}\right)^2\right]$$

6.144×10^5

$= (9 \times 10^8) + (9 \times 10^8) + (~~7.776 \times 10^9~~) = 1.801 \times 10^9 mm^4$

$A = (2 \times 600 \times 50) + (1800 \times 16) = 88800\ mm^2$

$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1.801 \times 10^9}{88800}} = 142.413$

$L_{LT} = KL = 24M = 24 \times 10^3 M$

~~$\lambda = KL$~~ $\frac{L_{LT}}{r_y} = \frac{24 \times 10^3}{142.413} = 168.524 > 100$

$\Rightarrow \alpha_s = \frac{30}{(L_{LT}/r_y)^2} = \frac{30}{168.524^2} = 1.056 \times 10^{-3}$

$D = 1800 + 2(50) = 1900$

$\therefore I_s = 0.34 \times 1.056 \times 10^{-3} \times 1900^3 \times 50 = 1.231 \times 10^8\ mm^4$

~~I_s provided $= \frac{bd^3}{12} = \frac{2 \times 16 \times 224^3}{12} = 2.997 \times 10^7$~~ $> 1.336 \times 10^8\ mm^4$

Step 9 - Connection b/w flange & web plate

Sheer force, $q_s = \frac{V A_f \bar{y}}{2 I_f}$

$= \frac{2016 \times 10^3 \times 660 \times 50 \times 950}{2 \times 5.135 \times 10^{10}}$

$= 559.455 \ N/mm$ — ①

Strength of weld per unit length $= \frac{L_w t_e f_u/\sqrt{3}}{L_w \gamma_m}$

$= \frac{t_e f_u/\sqrt{3}}{\gamma_m}$

$= \frac{t_e \times 410/\sqrt{3}}{1.25}$

$= 189.371 \ t_e \ N/mm$ — ②

Equating ① & ② ⇒ $559.455 = 189.371 t_e$

⇒ $t_e = 2.954$

$0.7 \times s = 2.954$

⇒ $s = 4.22$ mm ⇒ 5mm.

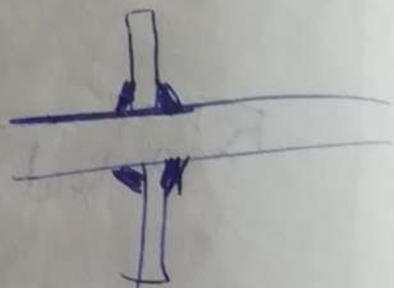
Let us provide weld of size 5mm.

Step 18 - Connection b/w web plate and stiffener

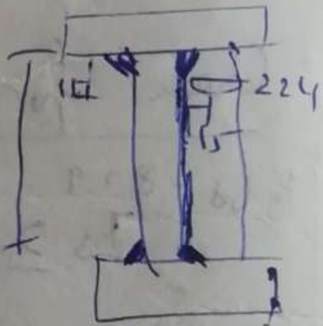
Width available = 224 - 15 = 209 mm

Pg 32 - 6.3.1

Tension capacity of plates $= \dfrac{0.9 A_n f_u}{\gamma_{m1}}$



Where $A_n = \left[b - n d_h + \Sigma \dfrac{p_{si}^2}{4 g_i} \right] t$



$A_n = 209 \times 16 = 3344 \text{ mm}^2$

$\therefore$ Tension capacity $= \dfrac{0.9 \times 3344 \times 410}{1.25} = 9.8715 \times 10^5 \text{ N} = 987.15 \text{ kN}$

Capacity per unit length $= \dfrac{987.15 \times 10^3}{2(1800 - 2 \times 15)} = 278.856 \text{ N/mm}$ — (1)

Strength of weld per unit length $= \dfrac{L_w t_e f_u / \sqrt{3}}{\gamma_m}$

$= \dfrac{1 \times t_e \times 410 / \sqrt{3}}{1.25} = 189.37 t_e$ — (2)

Equating ① & ② ⇒ 278.856 = 189.371 te

⇒ te = 1.473 mm

0.7 × s = 1.473

s = 2.104 mm

Provide weld size of 5 mm (min Thk for 10–20 mm plates = 5 mm IS 800 – Table 21)

2) Redesign the plate girder using intermediate transverse stiffeners. Connections need not be designed. Use post critical method of design

~~Step 3 - Economical depth & thk of web~~

~~Let $\frac{d}{t_w} = 180$ ($3d \geq c \geq d$)~~

Step 1 - Load Calculation

$F_L = 168 kN/m$

Step 2 - SF & BM

$SF = 2016 kN$

$BM = 12096 kNm$

Step 3 - Economical depth & thk of web

$$d = \left(\frac{M k}{f_y}\right)^{1/3}$$

$k = \frac{d}{t_w} \leq 200$ to 270, ≤ 200 ($3d \geq c \geq d$)

Assume $k = 190$

$$d = \left(\frac{12096 \times 10^6 \times 190}{250}\right)^{1/3} = 2094.844 \simeq 2100mm$$

$\frac{d}{t_w} = 180 \Rightarrow \frac{2100}{t_w} = 190 \Rightarrow t_w = 11.053mm$

∴ Provide web plate of size 2100×12 mm ($A_w = 2100 \times 12 = 25200$)

Spacing - $3d \geq c \geq d$

$3 \times 2100 \geq c \geq 2100$

$6300 \geq c \geq 2100$

Provide transverse stiffeners at a spacing of $c = 2.5m$.

Step 4 - Flange area

$A_f = \frac{M_u \gamma_{m0}}{f_y h} = \frac{12096 \times 10^6 \times 1.1}{250 \times 2100} = 25344 mm^2$, width $= 0.3 \times 2100 = 630mm$

Flange - 600×50 ($A = 2 \times 600 \times 50 = 60000 mm^2$)

Thk $= \frac{25344}{630}$

$= 40.229$

$= 50 mm$

Step 5 - Section classification

$\frac{b}{t_f} = \frac{600/2}{50} = 6 < 9.4\varepsilon \rightarrow$ Plastic

~~$\frac{d}{t_w} = 180$~~ $\frac{d}{t_w} = 190 \nleq 126$

Step 6 - Check for shear resistance

Pg 59, $V_n = V_{cr} = A_v \tau_b$

where $A_v = d t_w = 2100 \times 12 = 25200 mm^2$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\, \tau_{cr,e}}}$$

$$\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)(d/t_w)^2}$$

$c/d = \frac{2500}{2100} = 1.19 \geq 1.0$

$$k_v = 5.35 + \frac{4.0}{(c/d)^2} = 5.35 + \frac{4.0}{1.19^2} = 8.175$$

$$\tau_{cr,e} = \frac{8.175 \times \pi^2 \times 2\times10^5}{12(1-0.3^2)\times 190^2} = 40.934\ N/mm^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3}\times \text{~~45.604~~}\ 40.934}} = 1.878$$

$$\therefore \lambda_w \geq 1.2,\quad \tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2} = \frac{250}{\sqrt{3}\times 1.878^2} = \text{~~45.607~~}\ 40.925\ N/mm^2$$

$$V_{cr} = 25200 \times \text{~~45.607~~}\ 40.925 = 1.031\times10^6\ N = 1031\ kN$$

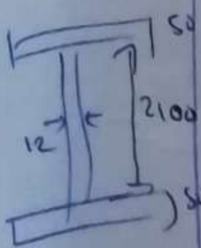
Step 7 - Check for bending strength

~~0.6 Vd = 0.6×~~

$$M_d = \beta_b Z_p f_y/\gamma_{m0} < 1.2 Z_e f_y/\gamma_{m0}$$

where $\beta_b = 1.0$ (plastic section)

~~Zp~~ = Since flange resist BM, Z_p & Z_e are calculated for flanges only.



$$Z_p = Z_{p\,flange} = Ah/2 = 60000 \times \left(\frac{2100+50+50}{2}\right) = 6.6\times10^7\ mm^3$$

$$Z_e = I/y = \frac{5.135\times10^{10}}{\left(\frac{2100+50+50}{2}\right)} = 4.668\times10^7\ mm^3$$

$$M_d = \frac{1.0\times6.6\times10^7\times250}{1.1} < \frac{1.2\times4.668\times10^7\times250}{1.1}$$

$$= 1.5\times10^{10} < 1.273\times10^{10}$$

$$= 15000 < 12730 = 12730\ kNm$$

$$12096 < 12730$$

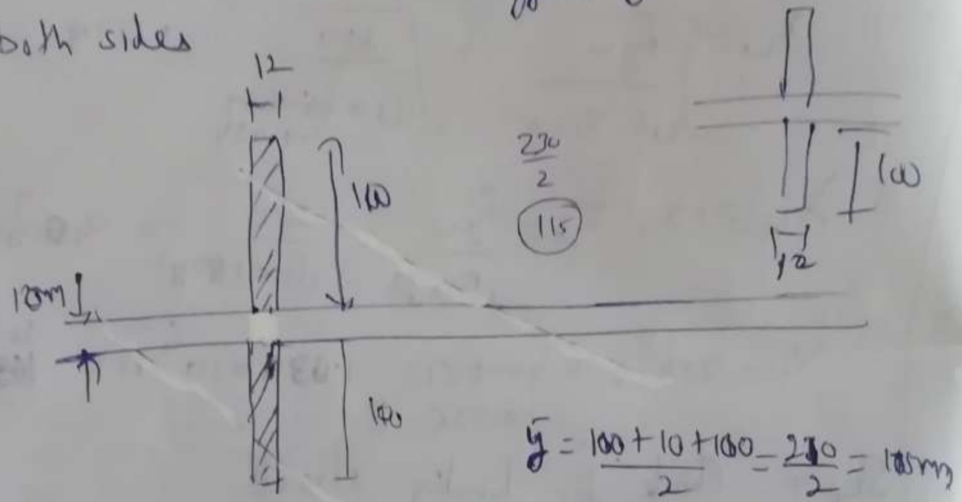
Step 8 - Design of intermediate transverse stiffeners

Pg 66 - 8.7.2.4, $\frac{c}{d} = \frac{2500}{2100} = 1.19 < \sqrt{2}\,(1.414)$

$\Rightarrow I_s = \frac{1.5 d^3 t_w^3}{c^2}$

$= \frac{1.5 \times 2100^3 \times 12^3}{2500^2} = 3.841 \times 10^6\ mm^4$

Try intermediate transverse stiffener of size 100×12 on both sides



$\bar{y} = \frac{100 + 10 + 100}{2} = \frac{210}{2} = 105mm$

$I = \left(\frac{12 \times 100^3}{12}\right) + (12 \times 100)($

$I = \left[\left(\frac{12 \times 100^3}{12}\right) + \left(12 \times 100 \times \left(\frac{100}{2} - 105\right)^2\right]\right. +$

$\left(\frac{12 \times 100^3}{12}\right) + (12 \times 100) \times \left(100 + 10 + \frac{100}{2} - 105\right)^2\Big]$

$= (~~8.26~~\ 4.63 \times 10^6) + (~~8.26~~\ 4.63 \times 10^6)$

$= 1.652\ (9.26) \times 10^6\ mm^4 > 3.841 \times 10^6\ mm^4$

(i) Check for buckling

Pg 67, 8.7.2.5, ~~Buckling~~ Design resistance ; $F_q = \frac{V - V_{cr}}{\gamma_{m0}}$

Stiffener force, $F_q = \frac{V - V_{cr}}{\gamma_{m0}}$

Where V ⇒ Factored SF near stiffener $= \frac{w(l - c)}{2}$

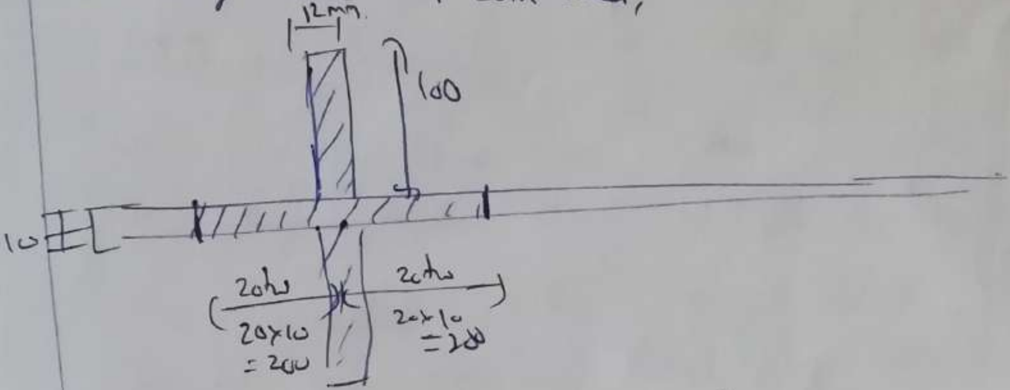
$$= \frac{168 \times (24 - 2.5)}{2}$$

$$= 1806\ kN$$

$V_{cr} = 1031\ kN$

$$F_q = \frac{1806 - 1031}{1.1} = 704.545\ kN$$

Considering $20t_w$ on both sides,



Area $A = (2 \times 200 \times 10) + (2 \times 100 \times 12) = 6400\ mm^2$

$I_{xx} = I_{xx} + A(y - \bar{y})^2$ where $\bar{y} = \frac{100 + 10 + 100}{2} = 105\ mm$

$$= 9.26 \times 10^6 + \left[\left(\frac{400 \times 10^3}{12}\right) + (400 \times 10) \times \left(100 + \frac{10}{2} - 105\right)^2\right]$$

$I = 9.293 \times 10^6\ mm^4$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{9.293 \times 10^6}{6400}} = 38.11$$

$KL = 0.7 \times L = 0.7 \times 2100 = 1470\ mm$

$KL/r = 1470/38.11 = 38.573$

BC-C, $\lambda = 38.573$ ⇒

λ	f_{cd}
30	211
40	198

$\frac{13}{10} \times 8.573 = 11.145$

$f_{cd} = 211 - 11.145 = 199.855\ N/mm^2$

$P_d = A \times f_{cd} = 6400 \times 199.855 = 1.279 \times 10^6\ N = 1279\ kN$

$704.545 < 1279\ kN$

UNIT-5 Connections (welded)

Beams may be connected to supporting by welding (or) bolting. In practice welded connections are commonly used instead of bolted connection.

The end of the beam may be designed to transferred shear to supporting column by
→ framed connection
→ unstiffened seated connection
→ stiffened seated connection

The end of the beam may be designed to transfer shear as well as moment by → Moment Resistant connection

Framed connection:- (Shear force)

1. An ISMB 400 beam is connected to ISHB 250 column to transfer end force of 140 kN. Design double plated welded connection. [width of plate = 50] assume

Soln:

Factored Shear Force $= 140 \times 1.5$
$= 210\,kN$

Using 50 mm wide plate, factored moment on weld connecting plate and beam

Moment = load x Plate width
$= 210 \times 10^3 \times 50$
$= 10.5 \times 10^6\ Nmm$

Thickness of plate should be 1.5 mm more than web thickness of beam.

From SP 6, Table 1, Pg 2, $t_w = 8.9$ mm (ISMB 400)

Plate thickness = Thickness of beam + 1.5
$= 8.9 + 1.5$
$= 10.4$ mm (for lathe plate thickness + 2mm $= 10.4 \simeq 10+2 = 12$ mm)

use lathe of size 50mm x 12mm

Strength of weld:-

IS 800, Pg 79, 10.5.7.1.1

Design of strength of filled weld, $f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

where, $f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}}$

$\gamma_{mw} = 1.5$ (field weld) - Pg 30, Table 4.
$\gamma_{mw} = 1.25$ (Shop weld)

Design is made for field weld, Same is adopted for shop weld

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$= \frac{236.71}{1.5} = \frac{236.71}{1.5}$$

$f_{wd} = 157.809 \; N/mm^2$ (For field weld)

$$f_{wd} = \frac{236.71}{1.25}$$

$f_{wd} = 189.368$ (For shop weld)

} Assume one value.

Shop weld connecting plate and web of beam (weld B)

Assume 6mm size of weld,

Throat thickness $= 0.7 \times 6$

$T_e = 4.2$ mm

Depth of weld, $h = \sqrt{\frac{6M}{2 t_e f_{wd}}}$

$$= \sqrt{\frac{6 \times 10.5 \times 10^6}{2 \times 4.2 \times 157.809}}$$

$h = 218.0$ mm.

The above depth can resist bending moment alone, Additional depth of 20% is provided to resist shear.

$$\text{Depth} = 218 + \left(\frac{20}{100} \times 218\right)$$

$h = 261.8 \simeq 260$ mm

Direct Shear stress $q_1 = \frac{\text{Shear force}}{\text{Area}}$

$$= \frac{V}{2 t_e h}$$

$$= \frac{210 \times 10^3}{2 \times 4.2 \times 260}$$

$q_1 = 96.15 \; N/mm^2$

Stress due to bending $q_2 = \frac{M}{2Z}$ where $Z = \frac{t_e h^2}{6}$

$$Z = \frac{(4.2)^2}{6} = 2.94 \text{ mm}$$

$$Z = \frac{4.2 \times (260)^2}{6}$$

$$Z = 47{,}320$$

$$q_2 = \frac{10.5 \times 10^6}{2 \times Z}$$

$$q_2 = 110.94 \ N/mm^2$$

Resultant Stress, $q = \sqrt{q_1^2 + q_2^2}$

$$q = 146.81 \ N/mm^2 < 157.81 \ (\text{Assume})$$

Hence provide 6mm size, 260mm long.

Field weld connecting plate to column:

Strength of weld = Area x Stress

$= 2 \times t_e \times h \times f_{wd}$

$= 2 \times t_e \times 260 \times 157.809 \longrightarrow ①$

Shear Force $= 210 \times 10^3 \longrightarrow ②$

Equating eqn ① & ②

$$2 \times t_e \times 260 \times 157.809 = 210 \times 10^3$$

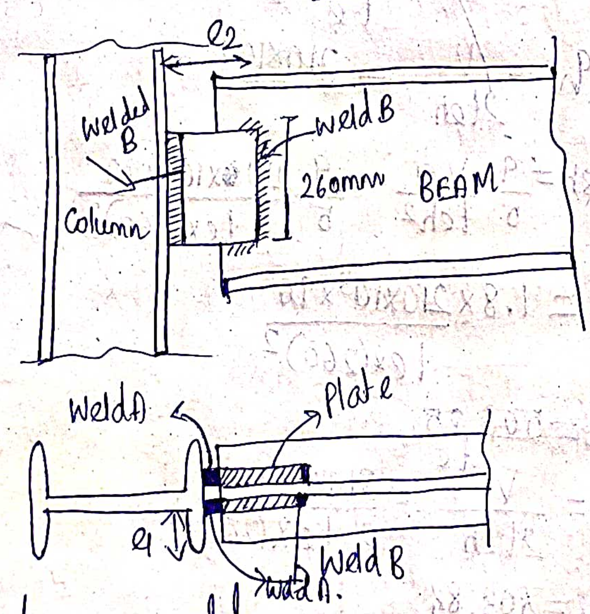
$$\boxed{t_e = 2.56mm}$$

Throat thickness, $t_e = 0.7 \times S$

$$2.56 = 0.7 \times S$$

$$S = \frac{2.56}{0.7}$$

$$\boxed{S = 3.66 \ mm}$$



Provide 4mm weld

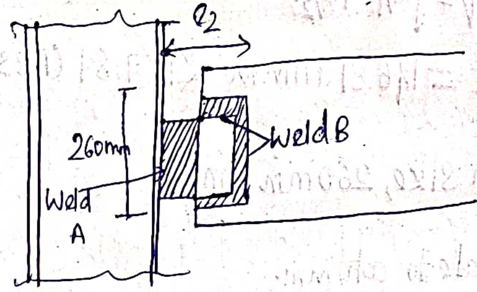
2). An ISMB 400 beam is connected to ISHB 250 column to transfer end force of 140 kN. Design double angle connection.

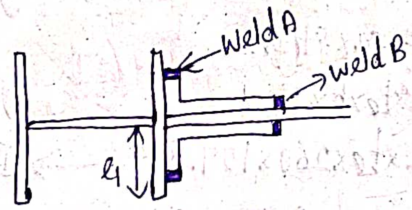
Soln

Assume: use 2ISA 90 60 angles

Let the depth of angle be 260 mm

Design of weld A (field weld)





Design of weld A (field weld) ($e_1 = 90mm$, $e_2 = 60mm$)

Factored Shear Force $= 140 \times 1.5$

$V = 210 kN$

Reaction on each angle (weld), $R = \frac{210}{2} = 105 kN$

Horizontal Shear, $q_{sh} = \frac{9}{5} \frac{V e_1}{t_e h^2} = \frac{9}{5} \frac{105 \times 10^3 \times 90}{t_e \times (260)^2}$

$q_{sh} = 1.8 \times \frac{105 \times 10^3 \times 90}{t_e \times (260)^2}$

$q_{sh} = \frac{251.63}{t_e}$

Vertical Shear, $q_v = \frac{V}{2 t_e h} = \frac{210 \times 10^3}{}$

Horizontal shear, $q_{sh} = \frac{9}{5} \frac{V e_1}{t_e h^2} = \frac{9}{5} \frac{210 \times 10^3 \times 90}{t_e \times (260)^2}$

$= 1.8 \times \frac{210 \times 10^3 \times 90}{t_e \times (260)^2}$

$q_{sh} = \frac{503.25}{t_e}$

Vertical Shear, $q_v = \frac{V}{2 t_e h} = \frac{210 \times 10^3}{2 \times t_e \times (260)}$

$q_v = \frac{403.84}{t_e}$

$$\text{Resultant} = \sqrt{q_{sh}^2 + q_v^2}$$

$$= \sqrt{\left(\frac{503.25}{te}\right)^2 + \left(\frac{403.84}{te}\right)^2}$$

$$= \frac{645.25}{te} \longrightarrow ①$$

Strength of weld,

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}} = 236.71$$

$$\text{For field weld} = \frac{236.71}{1.5}$$

$$f_{wd} = 157.809\ N/mm^2 \longrightarrow ②$$

Eqn ① & ②

$$\frac{645.25}{te} = 157.809$$

$$\boxed{te = 4.08\,mm}$$

$$te = 0.7 \times 8$$

$$\frac{4.08}{0.7} = S$$

$$S = 5.84 = 6\,mm$$

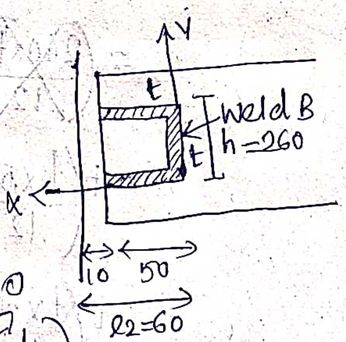
Provide 6mm weld.

Design of shop weld (B)

$$\text{Strength of weld, } f_{wd} = \frac{f_{wn}}{\gamma_{mw}} \Rightarrow f_{wn} = \frac{410}{\sqrt{3}} = 236.71$$

$$f_{wn} = \frac{236.71}{1.25}$$

$$f_{wd} = 189.368\ N/mm^2 \longrightarrow ①$$



$$\bar{x} = \frac{A_1x_1 + A_2x_2 \cdots}{A_1 + A_2}$$

$$= \frac{\left(50 \times t \times \frac{50}{2}\right) + \left(50 \times t \times \frac{50}{2}\right) + \left(260 \times t \times \frac{t}{2}\right)}{(50 \times t) + (50 \times t) + (260 \times t)}$$

$$= \frac{1250t + 1250t}{360t} = \frac{2500t}{360t}$$

$$\bar{x} = 6.94\,mm, \text{ From end} = 60 - 6.94 = 53.06\,mm$$

$\bar{y}_1 = \frac{260}{2} = 130mm$

Shear force per weld $= \frac{210}{2} = 105 kN$

Moment $= 105 \times 10^3 \times 53.06$

$= 5.57 \times 10^6$ Nmm

$I_{xx} = I_{xx} + A(y-\bar{y})^2$

($I_{xx} = \frac{bd^3}{12}$)

$= \left(\frac{t \times 260^3}{12}\right) + (t \times 260)\left(\frac{260}{2} - 130\right)^2$

$+ \left(\frac{50 \times t^3}{12}\right)^0 + (t \times 50)\left(\frac{t}{2}^0 - 130\right)^2$

$+ \left(\frac{50 \times t^3}{12}\right)^0 + (t \times 50)\left(t + 260 - 2t + \frac{t}{2}^0 - 130\right)^2$

$\Downarrow$

$\left(-\frac{t}{2}^0 + 260 - 130\right)^2$

$= \frac{17.57 \times 10^6 t}{12} + (260t)(0)$

$+ 0 + 50t(16.9 \times 10^3)$

$+ 0 + 50t(130)^2$

$I_{xx} = 3.15 \times 10^6 t \; mm^4$

$I_{yy} = I_{yy} + A(x - \bar{x})^2$

($I_{yy} = \frac{db^3}{12}$)

$= \left(\frac{260 \times t^3}{12}\right) + (260 \times t)\left(\frac{t}{2}^0 - 6.94\right)^2$

$+ \left(\frac{t \times 50^3}{12}\right) + (t \times 50)\left(\frac{50}{2} - 6.94\right)^2$

$+ \left(\frac{t \times 50^3}{12}\right) + (t \times 50)\left(\frac{50}{2} - 6.94\right)^2$

$= \left[260t \times (6.94)^2\right] + \left[10.42 \times 10^3 t + 50t(326.16)\right]$

$+ \left[10.42 \times 10^3 t + 50t(326.16)\right]$

$= \left[260t \times (6.94)^2\right] + 26.73 \times 10^3 t + 26.73 \times 10^3 t$

$=12.52\times10^{3}t+26.73\times10^{3}t+26.73\times10^{3}t$

$=65.98\times10^{3}t\ mm^4$

$I_{yy}=6.598\times10^{4}t\ mm^4$

$I_{pp}=I_{xx}+I_{yy}$

$=3.15\times10^{6}t+6.598\times10^{4}t=3.216\times10^{6}t\ mm^4$

$I_{pp}=3.216\times10^{6}t\ mm^4$

$r=\sqrt{130^2+6.94^2}$

$=130.18\ mm$

$\theta=\tan^{-1}\left(\frac{130}{6.94}\right)$

$\theta=86.94$

Stress due to twisting (horizontal) $q_r=\frac{M}{I_p}\times r$

$=\frac{5.57\times10^{6}}{3.216\times10^{6}t}\times130.18$

$=225.88/t=\frac{225.8}{t}$

Shear stress (vertical), $q_v=\frac{F}{A}$

$=(140\times10^{3})$ should not be taken

Shear force per weld, $F=105\ kN$

$q_v=\frac{105\times10^{3}}{(260\times t)+(50\times t)+(50\times t)}$

$=\frac{105\times10^{3}}{260t+50t+50t}$

$=\frac{105\times10^{3}}{360t}$

$q_v=\frac{291.67}{t}$

Resultant $=\sqrt{q_v^2+q_r^2+2q_vq_r\cos\theta}$

$=\sqrt{\left(\frac{291.67}{t}\right)^2+\left(\frac{225.8}{t}\right)^2+2\left(\frac{291.67}{t}\right)\left(\frac{225.8}{t}\right)\cos 86.94}$

Resultant $= \frac{378.260}{t}$ N/mm² ⟶ ②

Solving eqn ① & ②.

$$189.368 = \frac{378.260}{t}$$

$$t = \frac{378.260}{189.368}$$

$t = 1.99$ mm $\simeq 2$ mm.

$0.7 \times S = 2$

$$S = \frac{2}{0.7}$$

$S = 2.85$ mm

Use 3mm weld

(X.) 3) An ISMB 400 transfer an end reaction of 160 kN and an end moment of 80 kNm to the flange of ISHB 300. Design Moment Resistant Connection.

Step 1: Design of Tension plate

Force in tension plate $= \frac{\text{Moment}}{\text{Depth of Beam}}$

$$= \frac{80 \times 10^6}{400}$$

$= 2 \times 10^5$ N $= 200$ kN

Factored Force $= 1.5 \times 200$

$= 300$ kN

Strength of plate, $T_{dn} = \frac{0.9 A_n f_u}{\gamma_m} = \frac{0.9 \times A_n \times 410}{1.25} = 295.2 A_n$ ⟶ ②

(Pg: 46 IS800 6.3.1)

where, A_n → Area of plate

f_u → ultimate stress $= 410$ N/mm²

γ_m → Safety factor $= 1.25$

Eqn ① & ②

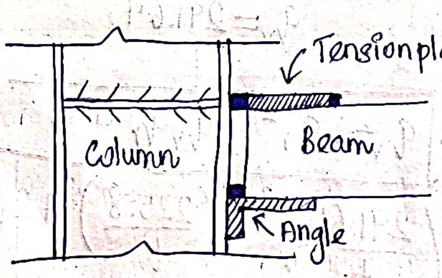
$300 \times 10^3 = 295.2 A_n$

$A_n = 1016.26$ mm²

$A_n = 1016.26$ mm²

Thickness $= \frac{1016.26}{110}$

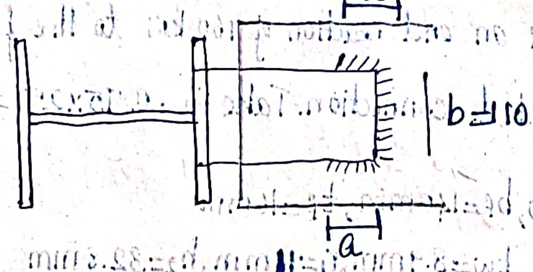
$= 9.24$ mm $\simeq 10$ mm



width of flange of ISMB 400 is 140 mm
Selecting width of flange as 110 mm,

Provide plate of width 110 mm and thickness 10 mm, Connect the plate to column by full penetration for butt weld.

[Bigger seat angle i.e. unstiffened Seated condition]



Strength of weld $= l_w t_e f_{wd}$

$= l_w t_e \frac{f_{wd}}{\gamma_{mw}}$

$= l_w t_e \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$

Assume size of weld as 8mm, $t_e = 0.7 \times S$

$= 0.7 \times 8$

$= 5.6 \text{mm}$

$f_u = 410 \text{N/mm}^2$

$\gamma_{mw} = 1.25$ (Shopweld)

$l_w = 2a + 110$

Strength of weld $= (2a+110) \times 5.6 \times \frac{410}{\sqrt{3} \times 1.25}$

$= (2a+110) \times 1060.47$

$= 2a(1060.47) + 110(1060.47)$

$= 2120.95a + 116.65 \times 10^3 \longrightarrow ③$

Eqn ① & ③

$300 \times 10^3 = 2120.95a + 116.65 \times 10^3$

$183.348 \times 10^3 = 2120.95a$

$a = 86.48 \text{ mm} \simeq 90 \text{ mm}$

$a = 90\text{mm}$

Length of weld $= 2a + 110 \Rightarrow 2 \times 90 + 110 = 290 \text{mm}$

Provide weld of length 290mm

Step 2:- Design of Seating angle.

An ISMB 400 transfers an end reaction of 160 kN to the flange of ISHB 300. Designed unstiffened seated connection. Take $F_B = 0.75 \times 250 = 187.5\ N/mm^2$.

For ISMB 400, $b_f = 140\ mm$ (→ width of flange), $t_f = 16\ mm$

(From SP6) $t_w = 8.9\ mm$, $r_1 = 14\ mm$, $h_2 = 32.8\ mm$

$$B = \frac{F}{f_b t_w}$$

$$= \frac{160 \times 10^3}{187.5 \times 8.9}$$

$$\boxed{B = 95.9}$$

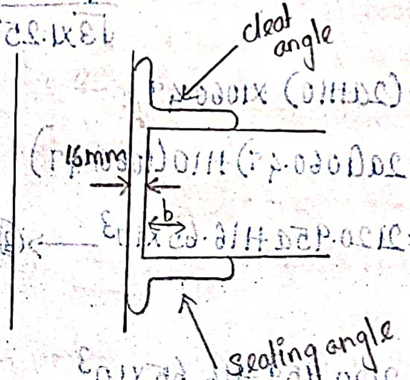
$$b = B - \sqrt{3} h_2 \not< \frac{B}{2}$$

$$= 95.9 - \sqrt{3} \times 32.8 \not< \frac{95.9}{2}$$

$$b = 39.08 \not< 47.95$$

$$\therefore b = 47.95$$

Let the bearing length of seating angle (b) be 47.95



Step 1: Design of seating angle:

Assuming 15mm thick angle of size 150x115, the distance of reaction from critical section.

$$= \text{clearance} + \left(\frac{1}{2} \times b\right) - (t + r_1)$$

$$= 10 + \left(\frac{1}{2} \times 47.9\right) - (15 + 11)$$

$$= 10 + 23.95 - 26$$

$$= 7.95\ mm$$

Moment of critical section = Force × disturbance of critical section

$$= 160 \times 10^3 \times 7.95 = 1.27 \times 10^6\ Nmm$$

Factored Moment $= 1.5 \times 1.27 \times 10^6$

$= 1.9 \times 10^6$ Nmm $\longrightarrow$ ①

Assume Length of seating angle equal to width of flange of beam [h] = 140mm

Moment of resistance, $M_d = \frac{f_y Z_p}{\gamma_{mw}}$

where $f_y = 250 N/mm^2$

$Z_p = \frac{bt^2}{4} = \frac{140 \times t^2}{4}$

$\gamma_{mw} = 1.25$

$$M_d = \frac{250 \times \frac{140 \times t^2}{4}}{1.25}$$

$M_d = 7000 t^2 \longrightarrow$ ②

Equating equation ① & ②

$1.9 \times 10^6 = 7000 t^2$

$t = 16.47$ mm

But we assumed 15mm thickness, but we got thickness = 16.47mm

So assume thickness = 20 mm. (angle thickness) = t

Design of weld:

$q_r = \frac{F}{A}$

$= \frac{160 \times 10^3 \times 1.5}{2 \times 140 \times t}$

$q_r = \frac{857.14}{t}$

Distance of reaction $= 10 + \frac{b}{2}$

$= 10 + \frac{47.95}{2}$

$= 33.97$ mm

Moment $= 1.5 \times 160 \times 10^3 \times 33.97$

$= 8.15 \times 10^6$ Nmm

$q_2 = \frac{M}{2Z_p}$, $Z_p = \frac{th^2}{4}$

$Z_p = \frac{t \times 140^2}{4} = 4900 \times t$

$$q_2 = \frac{8.15 \times 10^6}{2 \times 4900\,t}$$

$$q_2 = \frac{831.63}{t}$$

$$\text{Resultant, } q = \sqrt{q_1^2 + q_2^2}$$

$$= \sqrt{\left(\frac{857.14}{t}\right)^2 + \left(\frac{831.63}{t}\right)^2}$$

$$= \frac{1.19 \times 10^3}{t} \longrightarrow ①$$

Strength of weld, $f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

$$= \frac{236.71}{1.25}$$

$$f_{wd} = 189.368 \longrightarrow ②$$

Solving equation ① & ② we get

$$\frac{1.19 \times 10^3}{t} = 189.368$$

$$t = 6.3 \text{ mm}$$

$$\text{Throat thickness} = 0.7 \times S$$

$$6.3 = 0.7 \times S$$

$$S = 9 \text{ mm}$$

Provide 9mm weld

Assume seat angle of size ISA 100x100x6? @ top and use 6mm weld.