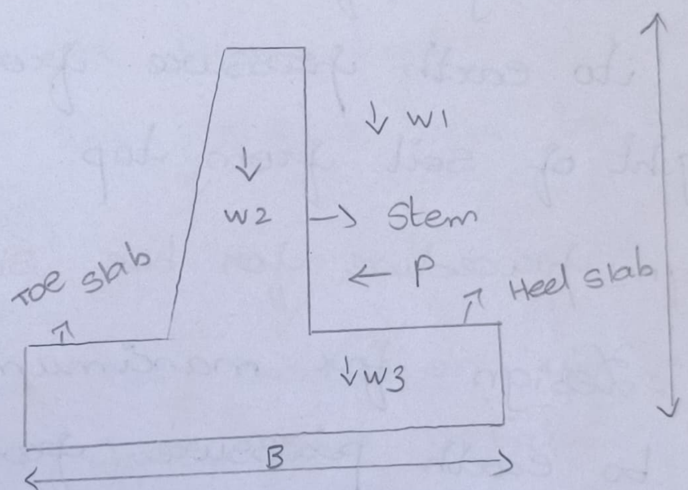
Unit - I

Retaining walls

Retaining walls are relatively rigid walls used for supporting the soil mass laterally show that the soil can be retained at different levels on the two sides.

Dimension of the cantilever retaining wall:



i) Top width of the stem = 200 mm

ii) Bottom width of the stem design for maximum bending moment due to horizontal earth pressure

iii) Base width of the slab

$B = 0.5H$ to $0.6H$

where H is the height of the retaining wall (without surcharge)

$B = 0.5H$ to $0.6H$ (without surcharge)

B = 0.7H (with surcharge)

iv) Toe projection = B/3

v) Thickness of the base slab is equal to bottom width of the stem.

vi) pressure distribution $P_{max/min} = \frac{W}{B}\left[1 \pm \frac{be}{B}\right]$

Design procedure for heel slab:

Design for maximum bending moment due to earth pressure from bottom and weight of soil from top.

Design procedure for toe slab:

Design for maximum bending moment due to earth pressure from bottom and earth soil on the toe slab from the top.

Minimum depth of foundation

$$DF = \frac{qu}{\gamma}\left[\frac{1 - \sin\phi}{1 + \sin\phi}\right]^2$$

where

qu = safe bearing capacity of soil

γ = density of soil

ϕ = angle of repose

Shear key :

shear key is the structural element which is sometime used in footing of the retaining wall to reduce wall's sliding.

check for sliding :

$$FOS = \frac{\mu W}{P} > 1.5$$

μ = is the friction factor

W = is the total load

P = is the pressure due to horizontal soil movement.

Active pressure :

It is the condition in which the earth exerts a force on a retaining system and the member tends to move towards the excavation.

passive pressure :

It is the condition in which the retaining wall exerts a force on the soil.

problem

Cantilever

1. Design the retaining wall to retain horizontal level earth embankment 5m above the soil. A unit weight of the earth is 16 KN/m^3 and angle of repose is 30°. The safe bearing capacity of soil 145 KN/m^2. The coefficient of friction between the soil and concrete is 0.55. use M20 grade concrete and Fe415 HYSD bars.

Given:

$\gamma = 16\ KN/m^3$

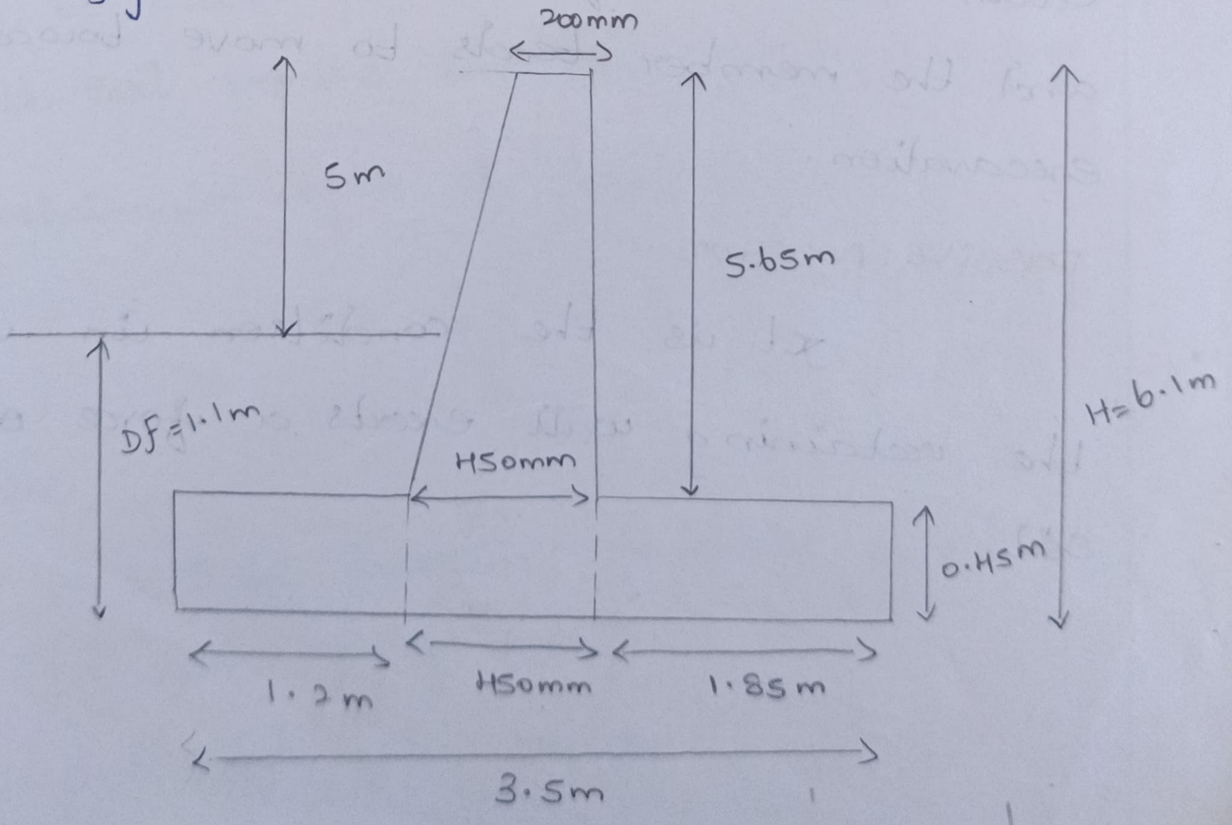
$\phi = 30^\circ$

$q_u = 145\ KN/m^2$

$\mu = 0.55$

$f_{ck} = 20\ N/mm^2$

$f_y = 415\ N/mm^2$



Dimension:

Assume top width of the stem = 200mm

minimum Depth of foundation

$$DF = \frac{q_u}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$

$$= 145/16\left[\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right]^2$$

$$DF = 1.06m \simeq 1.1m$$

Assume base slab thickness as 400 mm to 500mm

Take = 450mm

Base width of the slab $B = 0.5H$ to $0.6H$

$= 0.5 \times 6.1$ to 0.6×6.1

$= 3.05$ to 3.66

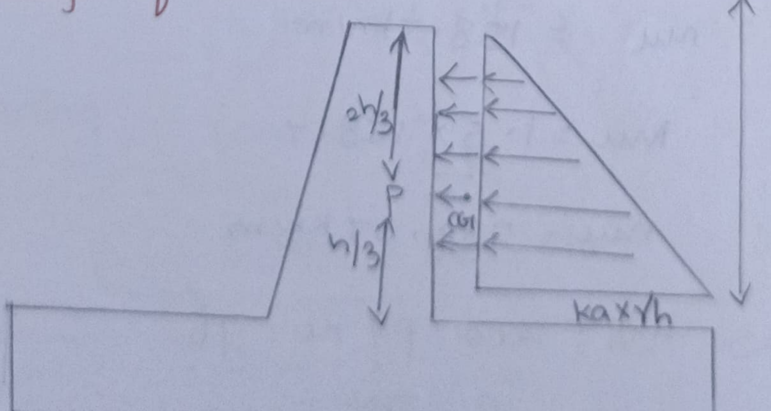
$B \simeq 3.5m$

∴ thickness of the base slab is equal to bottom width of the stem.

Toe width = $B/3$ = $1.16 \simeq 1.2m$

Step: 2

Design of stem:



$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$P = \frac{1}{2} \times b \times h$$

$$= \frac{1}{2} \times K_a \gamma h \times h$$

$$= \frac{1}{2} K_a \gamma h^2$$

$$M_u = P \times x$$

$$= P \times h/3$$

$$= \frac{1}{2} \times b \times h \times h/3$$

$$= \frac{1}{2} \times K_a \times \gamma h \times h \times h/3$$

$$M_u = K_a \gamma h^3/6$$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$= \frac{1-\sin 30^\circ}{1+\sin 30^\circ}$$

$$K_a = 0.33$$

$$M_u = \frac{0.33 \times 16 \times 5.65^3}{6}$$

$$M_u = 158.7 \text{ kNm}$$

$$M_u = 1.5 \times 158.7$$

$$M_u = 238.07 \text{ kNm}$$

Refer IS 456 : 2000 pg no : 96

$Mu_{limit} = 0.36\ X_{umax}/d\ [1 - 0.42\ X_{umax}/d]\ d^2 f_{ck}$

X_{umax}/d value Refer IS 456 : 2000 pg no : 70

provide 50 mm cover and 200 mm stem top width and and gradually tapering to increase the base width the stem as 400mm.

$d = 400\ mm \quad d' = 50mm$

$D = d + d' = 400 + 50 = 450mm$

$\therefore X_{umax}/d = 0.48$

$238.07 \times 10^6 = 0.36 \times 0.48\ [1 - 0.42 \times 0.48] \times 1000 \times 20 \times d^2$

$d = 293.7 < 450mm$

$\therefore$ Hence ok.

main bar :

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

$238.07 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400\ \left[1 - \frac{415 \times A_{st}}{1000 \times 400 \times 20}\right]$

$A_{st} = 1819\ mm^2$

Assume 16 mm ϕ bars

$= 10(\pi/4 \times 16^2) = 2010$

spacing $= 1000\ a_{st}/A_{st}$

$= \frac{1000 \times (\pi/4 \times 16^2)}{1819}$

$= 110\ mm$

provide 16mm ϕ bars @ 110mm c/c spacing.

Distribution bars:

$$(A_{st})_{min} = 0.12\%\ bD$$

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

$$= 540\ mm^2$$

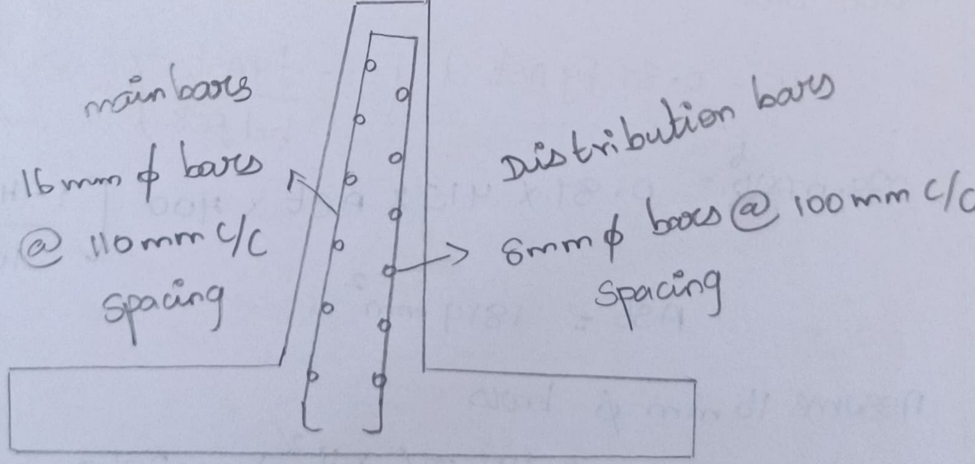
provide 8mm ϕ bars

$$= 11(\pi/4 \times 8^2)$$

$$= 552\ mm^2$$

$$\text{spacing} = \frac{1000(\pi/4 \times 8^2)}{540} = 100\ mm$$

provide 8mm ϕ bars @ 100 c/c spacing.

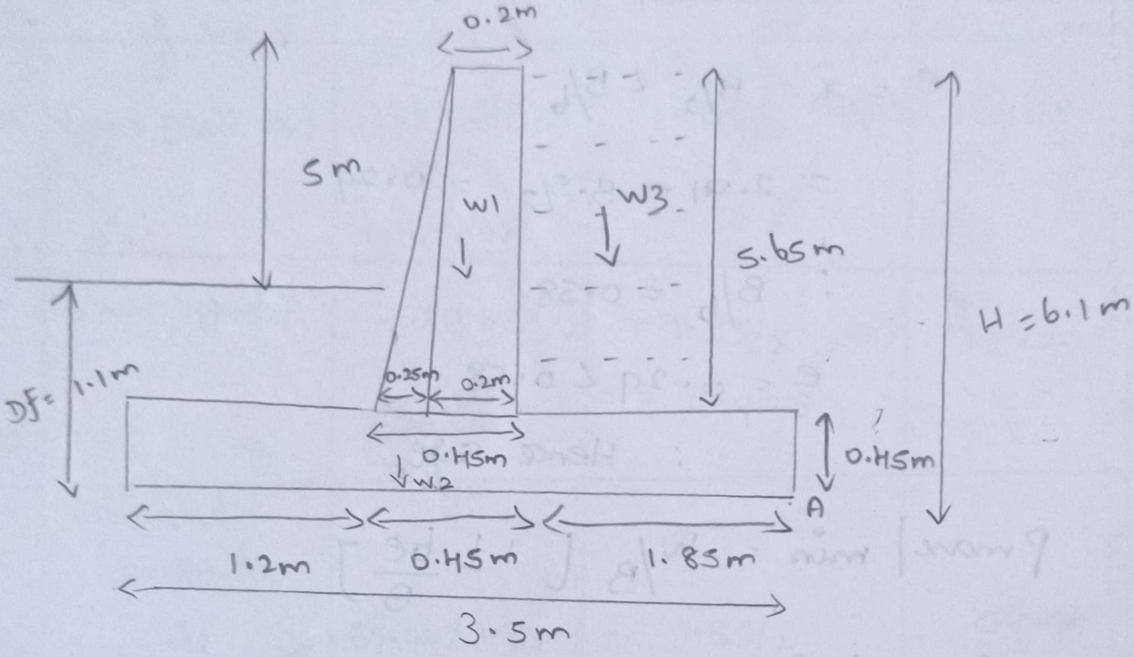


Step: 3

stability calculation

$$e = \bar{x} - B/2 < B/6$$

$$\bar{x} = \Sigma M / \Sigma W \qquad M = W \times \bar{x}$$



S.no	Description of Load	Load in kN	Distance in m	moment in kNm
1.	W_1 □ le	$= 25 \times 0.2 \times 5.65$ $= 28.25$	$1.85 + 0.2/2$ $= 1.95$ m	55.08
	W_1 △ le	$= 25 \times 1/2 \times 0.25 \times 5.65$ $= 17.65$	$= 1.85 + 0.2 + 0.25/3$ $= 2.13$	37.59
2.	W_2 □ le	$= 25 \times 3.5 \times 0.45$ $= 39.37$	$3.5/2$ $= 1.75$	68.95
3.	W_3 □ le	$16 \times 5.65 \times 1.85$ $= 167.4$	$1.85/2$ $= 0.925$	154.69
4.	Horizontal pressure	–	–	$K_a \gamma H^3/6$ $= 199.74$

$$\bar{x} = \Sigma m / \Sigma W = \frac{516.05}{252.67}$$

(6)

$\bar{x} = 2.04$

$e = \bar{x} - B/2 < B/6$

$= 2.04 - 3.5/2 = 0.29$

$\therefore B/6 = 0.58$

$e = 0.29 < 0.58$

$\therefore$ Hence safe.

$$P_{max}/_{min} = W/B \left[1 \pm \frac{6e}{B}\right]$$

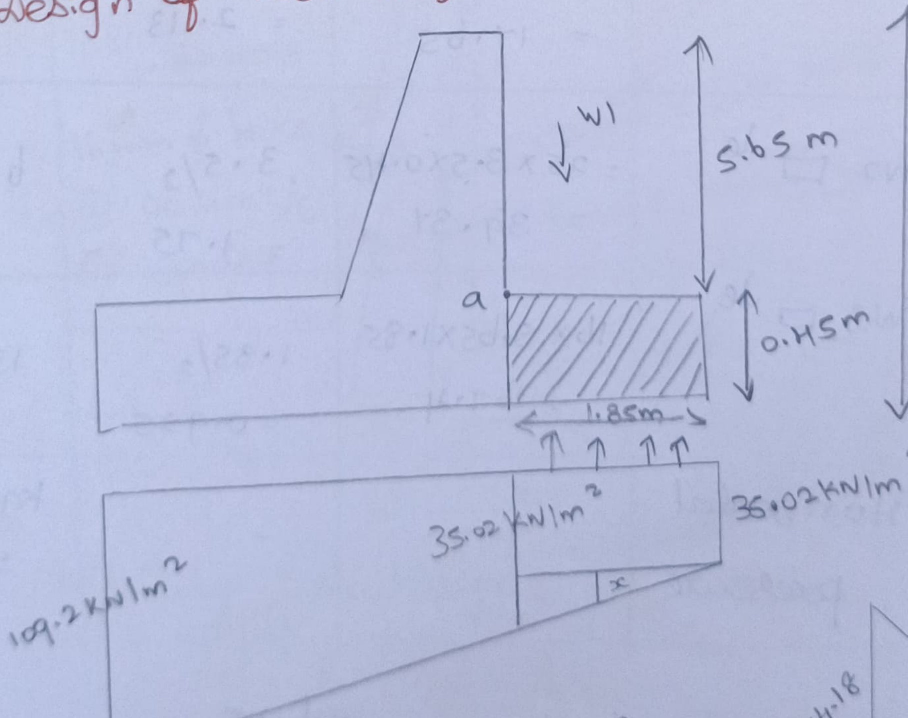
$$P_{max}/_{min} = \frac{252.4}{3.5} \left[1 \pm \frac{6 \times 0.3}{3.5}\right]$$

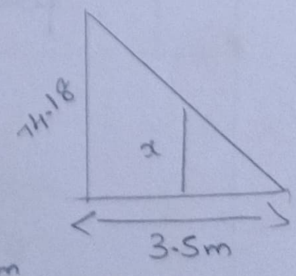
$P_{max} = 109.20 kN/m^2$

$P_{min} = 35.02 kN/m^2$

Step: 4

Design of heel slab:





$$\frac{74.18}{3.5} = \frac{x}{1.85}$$

$x = 39.20$ m

S.No	Description of Load	Load in KN	Distance in m	moment in KNm
1.	↓ W1 (Soil wt)	= 16 × 5.65 × 1.85 = 167.24	1.85/2 = 0.925	154.69
2.	↓ W2 (Slab wt)	= 25 × 0.45 × 1.85 = 20.81	= 1.85/2 = 0.925	19.25 (+)
	upward pressure			(−)
3.	↑ ▭ le	35.02 × 1.85	1.85/2	59.94 (+)
4.	↑ △ le	39.2 × 1.85 × 1/2	1/3 × 1.85	22.11

net moment = 91.89 KNm

$M = 91.89\ KNm$

$Mu = 1.5 \times 91.89 = 137.8\ KNm$

$Mu = 0.87\, fy\, Ast\, d\left[1 - \frac{fy\, Ast}{fck\, bd}\right]$

$137.8 \times 10^6 = 0.87 \times 415 \times Ast \times 400\left[1 - \frac{415 \times Ast}{20 \times 1000 \times 400}\right]$

$Ast = 1007\ mm^2$

Assume 16 mm ϕ bars

$= 6\left(\pi/4 \times 16^2\right)$

$= 1206\ mm^2$

spacing $= 1000\, ast / Ast = \frac{1000\left(\pi/4 \times 16^2\right)}{1007} = 200\ mm$

minimum Ast :

$$(Ast)_{min} = 0.12\% \cdot bD$$

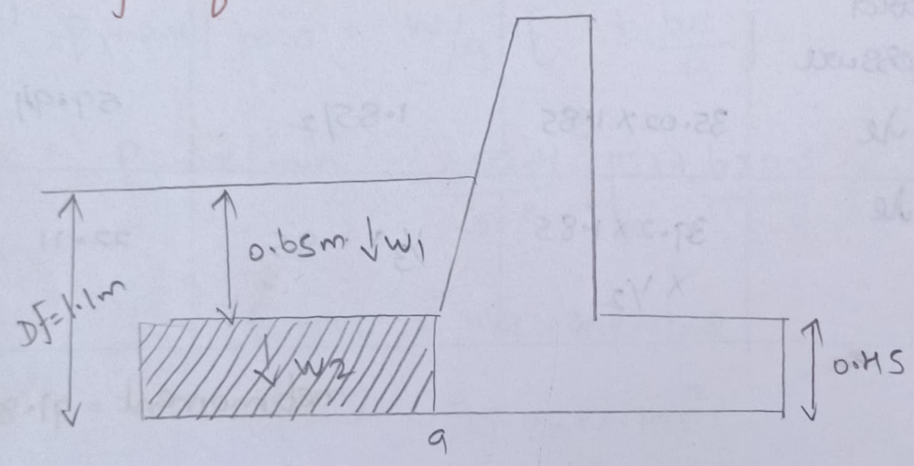
$$= 0.12/100 \times 1000 \times 450$$

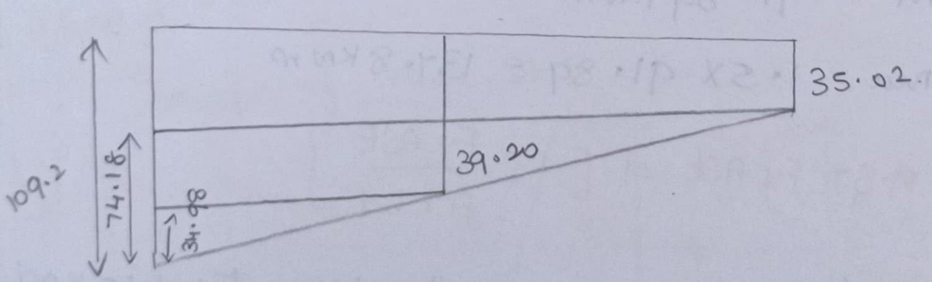
$$= 540 mm$$

$$spacing = 1000\, ast / Ast = \frac{1000 (\pi/4 \times 10^2)}{540} = 150 mm$$

step : 5

Design of toe slab :





S.no	Description of Load	Load in kN	Distance from a (m)	moment in kNm
1.	Soil wt (w1)	16×1.2×0.65	1.2/2	7.48 (+)
2.	Slab wt (W2)	25×1.2×0.45	1.2/2	8.1 (+)
3.	upward pressure			(-)
	▭ le	35.02×1.2	1.2/2	25.4
	▭ le	39.20×1.2	1.2/2	28.2 (+)
	△ le	34.98×1/2 ×1.2	2/3×1.2	16.7
				Σm = 54.64

$M = 54.64$ KNm

$Mu = 1.5 \times 54.64 = 81.96$

$Mu = 0.87 fy\ Ast\ d \left[1 - \frac{fy\ Ast}{fck\ bd}\right]$

$81.96 \times 10^6 = 0.87 \times 415 \times Ast \times 400 \left[\frac{1 - 415 \times Ast}{20 \times 1000 \times 400}\right]$

$Ast = 585\ mm^2$

Assume 16 mm ϕ bars

3 no. of 16 mm ϕ bars

$3(\pi/4 \times 16^2) = 603\ mm^2$

$\text{spacing} = \frac{1000 ast}{Ast} = \frac{1000(\pi/4 \times 16^2)}{585} = 300mm$

$(Ast)_{min} = 0.12/100 \times 1000 \times 450$

$= 540\ mm^2$

Assume 10 mm ϕ bars

$\text{Spacing} = \frac{1000 ast}{Ast} = \frac{1000(\pi/4 \times 16^2)}{540}$

$= 150mm$

check for sliding:

$FOS = \frac{\mu W}{P} > 1.5$

$Pa = Ka \gamma H^2/2$

$= 0.33 \times 16 \times 6.1^2/2$

$P_a = 101.2\ kN/m^2$

$$FOS = \frac{0.55 \times 252.5}{101.2} > 1.5$$

$= 1.3 \ngtr 1.5$

Hence shear key has to be designed.

$P_p = 1/K_a \times P_a$

$P_p = 3.03 \times 101.2$

$= 303.63\ kN/m^2$

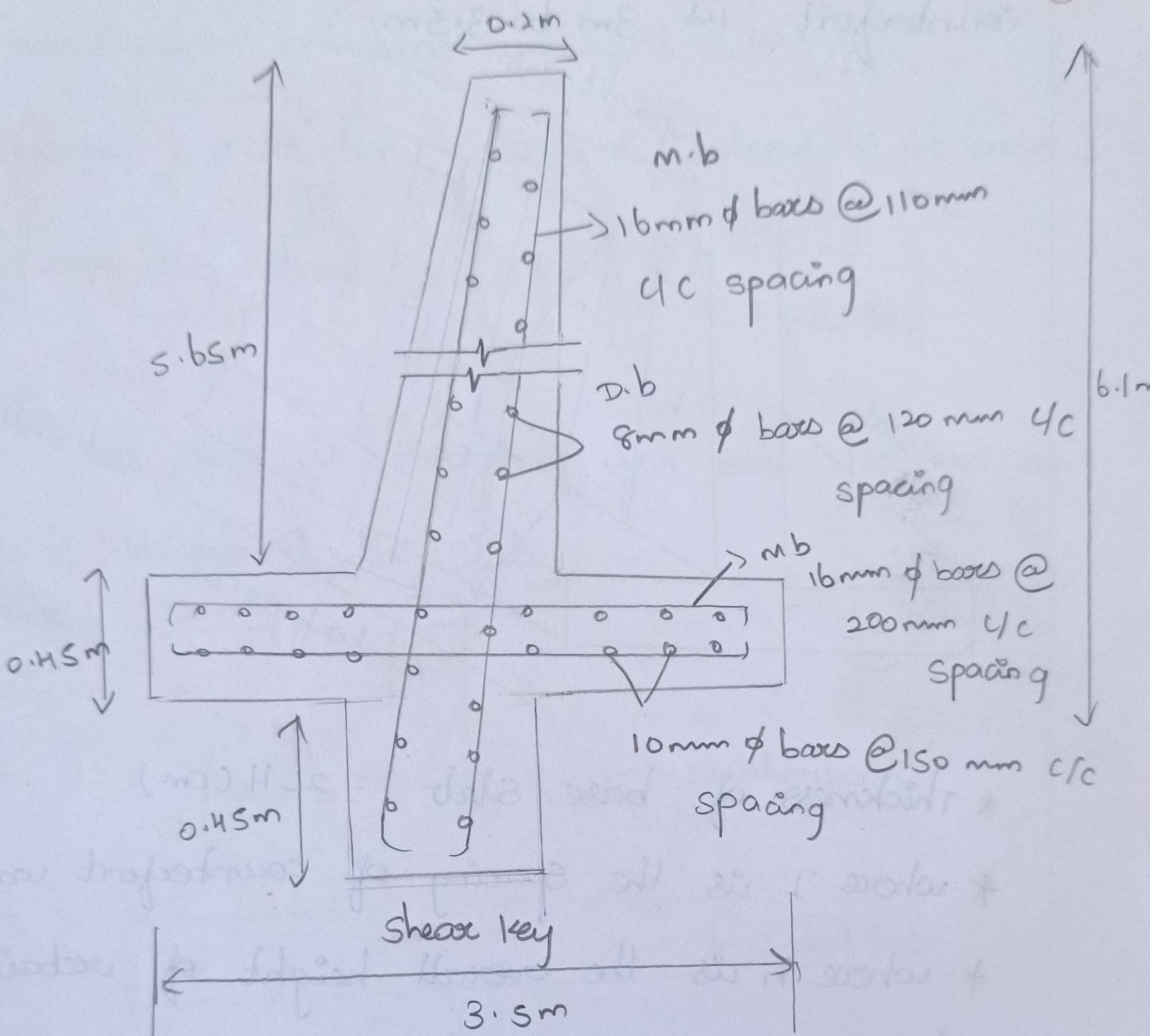
$W = (303.63) \times 0.45 \times 1$

$W = 136.3\ kN$

$$\frac{\mu w + w}{P} = \frac{(0.55 \times 252.2) + (136.3)}{101.2}$$

$2.7 > 1.5$

Hence design is ok.

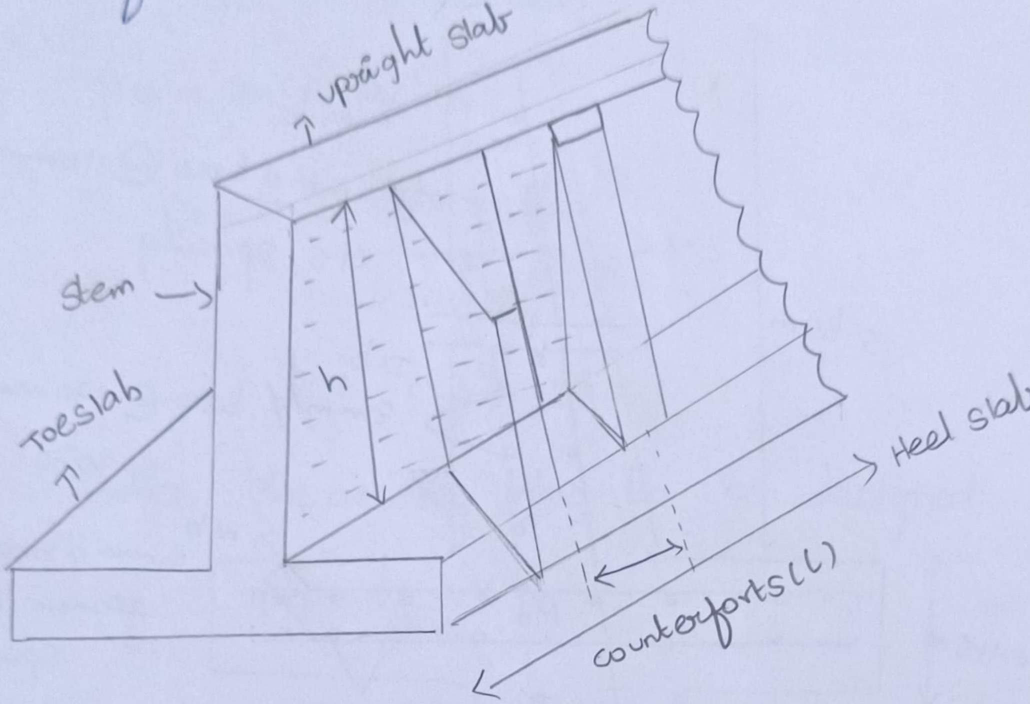


Design of counterfort retaining wall:

* If retaining wall of height over height 5.5 m are usually provided with the counterfort as shown in figure.

* The counterfort retaining consist of stem, heel and toe slabs and also having counterfort walls with regular interval the spacing of

counterfort is 3m to 3.5m



* Thickness of base slab = 2LH (cm)

* where L is the spacing of counterfort walls in m

* where h is the overall height of retaining wall

* Base width equal to 0.6H to 0.7H

* Toe projection = $\frac{\text{width of base}}{4} = \frac{B}{4}$

* Top width of the stem = 200 to 250mm

Design procedure:

Stem:

* stem is designed as continuous slab to span between counterforts.

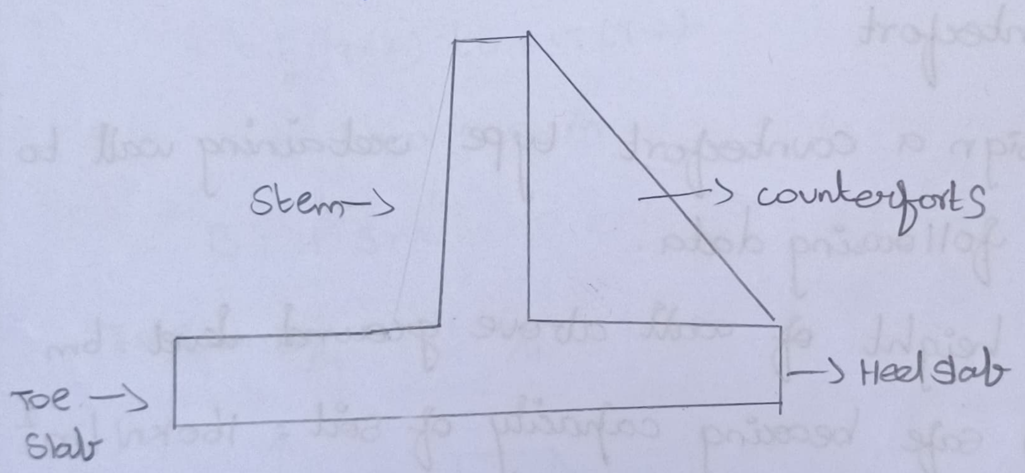
* maximum bending moment = $wl^2/12$

* where w is the pressure intensity at base.

pressure intensity (w) = $\gamma h \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$

Toe Slab:

Toe slab is designed for soil pressure and dead weight of the slab.



Heel Slab:

Heel slab is designed as continuous slab supported between counterfort to resist weight of soil and upward pressure at base.

Counterforts:

* counterfort thickness is same as the base slab.

* counterfort are designed to take lateral earth pressure.

* Maximum bending moment in the counterfort equal to $K_a \gamma h^3/6$

* where h is the height of retaining wall above base.

* where L is the spacing of counterforts

problem

counterfort

Design a counterfort type retaining wall to suit the following data.

i) height of wall above ground level = 6m

ii) safe bearing capacity of soil = 160 kN/m²

iii) Angle of internal friction = 30°

iv) Density of soil = 16 kN/m³

v) At spacing of counterfort = 3m

material used M20 grade Fe415 HYSD bars

soln:

Step: 1

Depth of foundation = $\frac{q_u}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]$

$= \left[160/16\right]\left[\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right]$

$D_f = 1.2$ m

$H = 7.2m$

Thick $= 2LH$

$= 2 \times 3 \times 7.2$ (cm)

Thick $= 43.2$ cm $= 432$

provide $= 450$ mm

$= 0.45$ mm

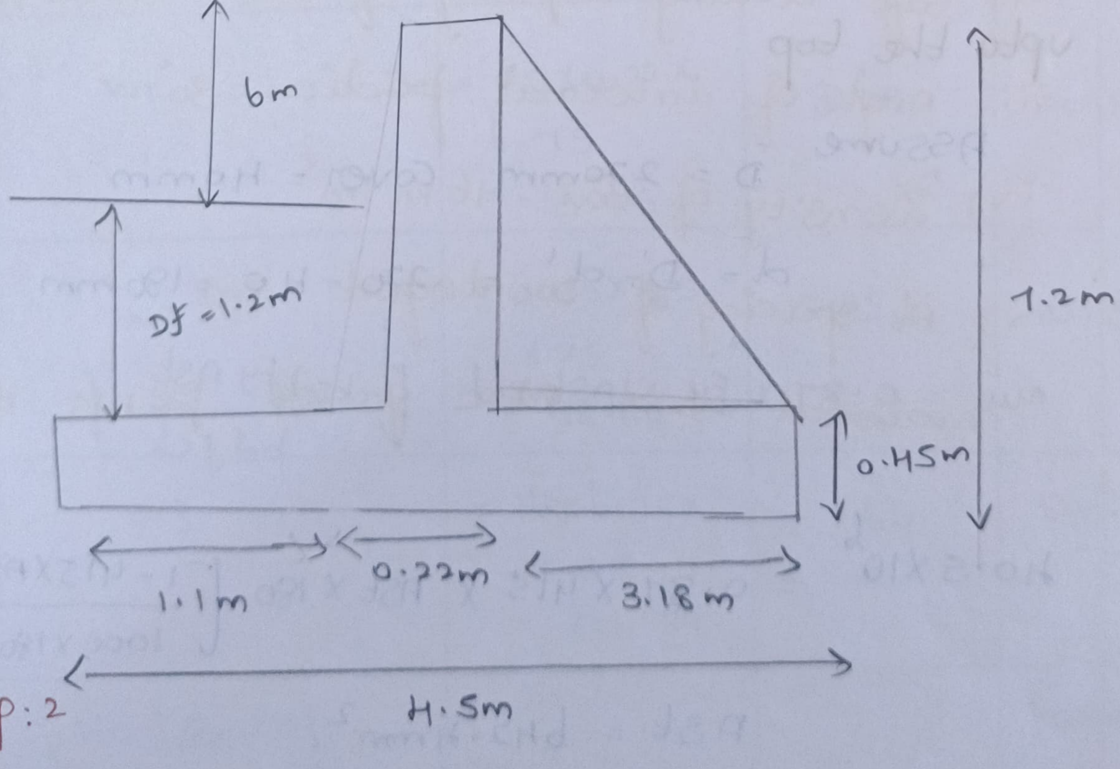
$B = 0.6H$ to $0.7H$

$= 0.6(7.2)$ to $0.7(7.2)$

$= 4.3$ to 5.04 m

$B = 4.5$ m

Toe $= B/4 = 4.5/4 = 1.1$ m



Step: 2

Design of stem:

$M = wl^2/12$

$w = \gamma h \left(\frac{1 - \sin\phi}{1 + \sin\phi} \right)$

$= 16 \times 7.5 \left(\frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} \right)$

$= 35.64 \approx 36 \, kN/m^2$

$M = 36 \times 3^2/12 = 27 \, kNm$

$Mu = 1.5 \times M = 1.5 \times 27 = 40.5 \, kNm$

Assuming an underreinforced section and to provide the suitable thickness to resist the design shear at base of stem.

Adopt an overall thickness of 220 mm constant upto the top.

Assume

$D = 220mm$ Cover = 40mm

$d = D - d' = 220 - 40 = 180 \, mm$

$Mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{fy \, Ast}{bd \, fck} \right]$

$40.5 \times 10^6 = 0.87 \times 415 \times Ast \times 180 \left[1 - \frac{415 \times Ast}{1000 \times 180 \times 20} \right]$

$Ast = 642.4 \, mm^2$

use 16 mm of bars

$$\text{spacing} = \frac{1000\,ast}{Ast} = \frac{1000 \times \pi/4 \times 16^2}{642.4} = 300mm$$

Distribution:

$(Ast)_{min} = 0.12\,\%\; bD$

$= 0.12/100 \times 1000 \times 220$

$= 264\ mm$

provide 8mm ϕ bars

$$\text{spacing} = 1000\,ast/Ast = \frac{1000\,(\pi/4 \times 8^2)}{264}$$

$= 190.39 \simeq 200mm$

Step: 3

stability calculation:

S.No	Description of loads	load in kN	Distance in (m)	moment in kN/m
1.	W1	= 25 × 0.22 × 6.75 = 35.65	3.28	120.80
2.	W2	= 0.45 × 4.5 × 25 = 48.60	2.25	109.35
3.	W3	= 16 × 6.75 × 3.28 = 354.24	1.64	580.95
4.	Horizontal pressure	-	-	$k_a \gamma h^3/6$ = 331.77 kNm

$\bar{x} = \Sigma M / \Sigma V$

$e = \bar{x} - B/2 < B/6$

$= \left[\frac{1142.87}{438.48} \right] - 4.5/2 = 0.41 \text{ m} < B/6$

$= 0.41 < 0.75$

pressure calculation:

$p_{max}/p_{min} = W/B \left[1 \pm 6e/B \right]$

$= 438.49/4.5 \left[1 \pm \frac{6 \times 0.41}{4.5} \right]$

$p_{max} = 150 \text{ kN/m}^2$

$p_{min} = 45 \text{ kN/m}^2$

step: 4

Design of toe slab

S.no	Description of load	Load in kN	Distance in (m)	moment in kNm
1.	W_1 slab	1 × 0.45 × 24 = 10.8	0.5	6.40 } (+)
2.	W_2 (Soil)	= 0.75 × 1 × 16 = 12.0	0.5	6.00
	upward pressure			(−)
3.	pressure 1	126.6 × 1 = 126.6	0.5	63.30 } (+)
4.	pressure 2	0.5 × 1 × 23.4 = 11.7	0.67	7.84
			m =	59.74 kNm

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8870471934

$Mu = 1.5 \times 59.74$

$= 89.61 kNm$

$\therefore d = 400 mm$

main bar:

$Mu = 0.87 \times fy \times Ast \times d \left[\frac{1 - 415 Ast}{b \times d \times fck} \right]$

$(89.61 \times 10^6) = 0.87 \times 415 \times Ast \times 400 \left(\frac{1 - 415 Ast}{1000 \times 450 \times 20} \right)$

$Ast = 644 mm^2$

provide 12 mm ϕ bars @ 150mm c/c spacing

Distribution bars:

$(Ast)_{min} = 0.12\% \; bD$

$= 0.12/100 \times 1000 \times 450$

$= 540 mm^2$

provide 10mm ϕ bars @ 250 mm c/c Spacing

Step: 5

Design of heel slab:

soil weight $= 16 \times 6.75 = 108 kN/m^2$

self weight $= 1 \times 0.45 \times 25 = 10.8 kN/m^2$

upward pressure $= -45.00 kN/m^2$

spacing of counterforts = 3m

$m = \left(73.80 \times 3^2 / 12 \right) = 55.85 kNm$

$mu = 1.5 \times 55.35$

$= 83 KN$

Ast

$mu = 0.87\, fy\, Ast\, d \left[1 - \frac{Ast\, fy}{fck\, bd}\right]$

on applying

$Ast = 600 mm^2$

provide 12 mm ϕ bars @ 150 mm c/c spacing

$Distribution = 540 mm^2$

provide 10 mm ϕ bars @ 280 mm c/c spacing.

Step: 6

Design of counterforts:

Thickness at top = (220 + 220) = 440 mm

Thickness of counterforts = 440 mm

maximum working moment

$m = (ka\, \gamma h^3 / 6 \times L)$

on applying $m = 820.12$ KNm

$mu = 1.5 \times 820.12$

$= 1230$ KNm

$mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{Ast\, fy}{fck\, bd}\right]$

on applying

$Ast = 500\ mm^2$

$Ast = 396\text{s}\ mm^2 \quad \left(AS = \frac{0.85bd}{Fy} \right)$

provide 5# of mm ϕ bars

$h_1 = 6m$ From top

$h_2 = 5.2\left(\frac{5.2}{5}\right) = \left(\frac{h_2}{6.752}\right)$

$h_3 = H - 2\left(\frac{5-3}{5}\right) = \left(\frac{h_3}{6.752}\right)$

connection (upright):

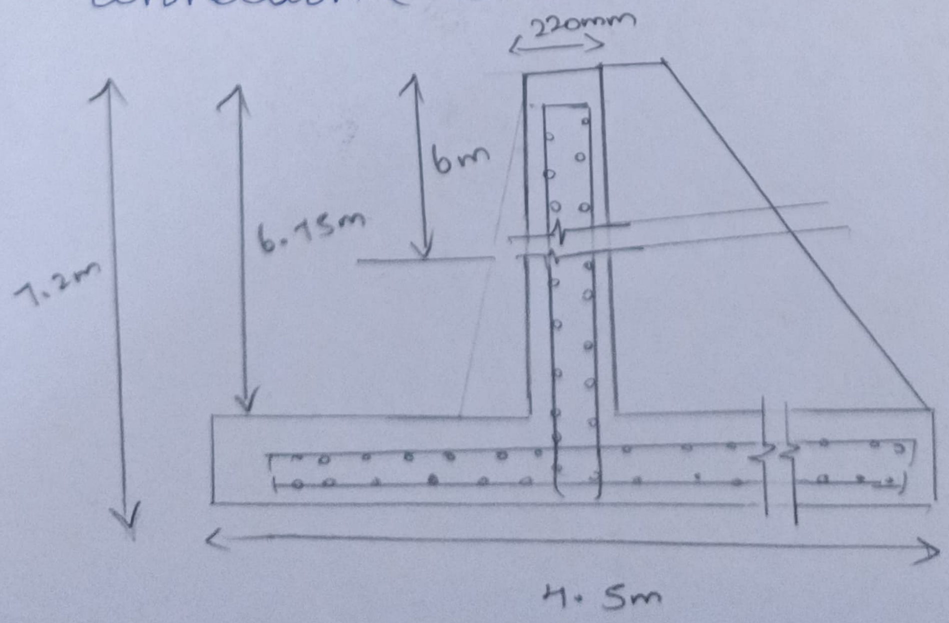
pressure $= 36 KN/m^2$

$F.C = 1.5 \times 91.8$

$= 137.7 KN$

provide 10mm ϕ bars

connection (heel slab) $mu = 1.5 \times 18.9$



Unit -2

Water tank

Design of water tank:

There are three types of reinforced concrete water tank.

* Tank resting on ground
* underground water tank
* Elevated water tank on staging

water tightness:

* water tightness is an important crietia in water tank.
* usually richer mixers with M20, M30 are normally used.
* The tensile stress permitted in concrete are restricted to control tracking in concrete.
* As per IS 3370 part II 1965 are compile in tables.
* For strength calculation stresses in concrete are same as IS 456 : 2000 permissible stresses in steel.

* Direct tension $100 N/mm^2$
* Due to bending steel at water phase $100 N/mm^2$
* Steel away from water phase $125 N/mm^2$

Grade of concrete (N/mm^2)	Permissible stress (N/mm^2)		Shear stress (V/bd)
	Direct tension	Tension due to bending	
M-15	1.1	1.5	1.5
M-20	1.2	1.7	1.7
M-25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

Reinforcement detailing:

* Minimum area of steel = 0.3% of gross area
* It reduces to 0.2% of gross area (upto to 450 thickness)
* For thickness 225m provide 2 layer of reinforcement
* percentage of reinforcement in base (or) floor slab resting directly on ground must not be less than 0.15% of concrete section.
* The minimum cover to all reinforcement should be 25mm (or) the diameter of main base.

Joints between tank wall and floor:

There are three types of joints between tank walls and floors are

* Flexible base (or) free base
* Fixed base
* Hinged base

* In case of flexible or free base between slab and tank walls, the walls are free slide and expand and the hoop tension developed in circular wall can be easily calculated due to hydrostatic pressure.
* However for hinged and fixed base the coefficient of moment and ring tension are calculated by IS 3370 part IV
* coefficient are expressed as function of non-dimensional parameters

$$H^2/Dt$$

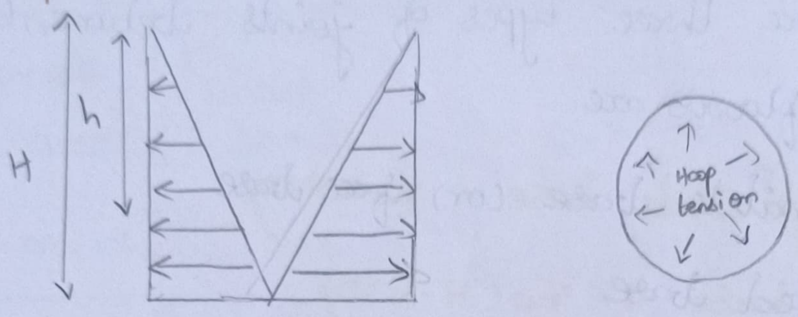
where,

H is the height of water tank

t is the thickness of wall

D is the diameter of tank.

Hoop tension:



Hoop tension = $p \times D/2$

$T = \gamma_w \times H \times D/2$ (Flexible base)

For rigid base:

$$T = \frac{\gamma_w (H-h) D}{2}$$

Design of circular water tank:

Step: 1

Given data and assumption

Step: 2

permissible stresses

i) Direct tensile stress in concrete

$\sigma_{ct} \rightarrow$ 3370 P/II

ii) Direct tensile stress in Steel

$\sigma_{st} = 100 \ N/mm^2$

$m = 280/3\sigma_{cbc}$ j, Q, & K.

Step: 3

Dimensions of tank.

Step: 4

Calculate the hoop tension and steel reinforcement.

For flexible base = $\gamma_w \times H \times D/2$

Rigid base $T = \gamma_w (H-h) D/2$

Step: 5

Thickness of tank wall

$$\sigma_{ct} = \frac{\gamma H D/2}{1000t + (m-1) A_{st}}$$

t is the thickness of tank wall from cracking consideration.

Step: 6

Calculate the reinforcement in tank wall.

Step: 7 Calculate the steel reinforcement of floor slab.

Problem:

Design a circular water tank with a flexible base for capacity of 5 lakh litres the depth of water tank is to be 4m. Free board is 200mm. Use M20 grade and grade I mild steel are used. Take permissible direct tensile stress in concrete 1.2 N/mm² permissible stresses in steel in direct tension = 100 N/mm² sketch the details of reinforcement in water tanks.

Given:

Step: 1

capacity of tank = 5 lakh litres

Depth of tank = 4m

Free board = 200mm

Density of water $\gamma_w = 10 kN/m^3$.

$f_{ck} = 20 N/mm^2$

$f_y = 250 N/mm^2$

Step: 2

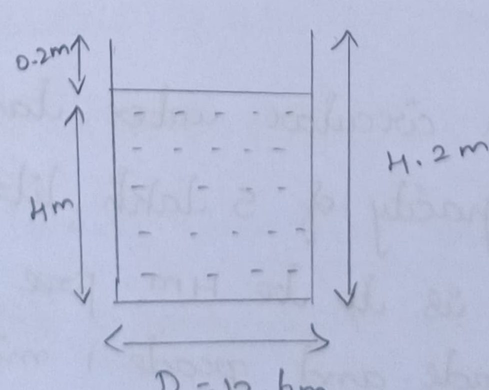
$m = 280 / 3\sigma_{cbc}$ (IS 456: 2000 pg no: 81)

$= \frac{280}{3 \times 7}$

$m = 13.33$

Step: 3

Dimension of water tank.



$V = A \times h$

$500 m^3 = \pi/4 \times D^2 \times H$

$D = 500 / (\pi/4 \times 4)$

$D = 12.6m$

step : 4

Hoop tension $= \frac{\gamma_w \times H \times D}{2}$

$= \frac{10 \times 4.2 \times 12.6}{2}$

$T = 264.6 kN$

$\sigma st = \frac{T}{Ast}$

$Ast = \frac{T}{\sigma st}$

$= \frac{264.6 \times 10^3}{100}$

$Ast = 2646 mm^2$

provide 20 mm ϕ bars

spacing $= 1000\, ast / Ast$

$= \frac{1000 \times \pi/4 \times 20^2}{2646}$

$= 118 \simeq 110mm$

Provide 20 mm ϕ bars @ 110 mm c/c spacing

step : 5

Thickness of wall

$\sigma ct = \frac{\gamma_w HD/2}{1000t + (m-1) Ast}$

$$1.2 = \frac{264.6 \times 10^3}{1000t + (13.33 - 1)\,264.6}$$

$t = 187\ mm$

Reinforcement in tank wall:

spacing of hoop increases towards top.

provide minimum reinforcement 0.3 % of gross area.

$Ast = 0.3\%\ GA$

$= 0.3/100 \times 1000 \times 190$

$= 570\ mm^2$

use 20 mm ϕ bars

$$\text{spacing} = \frac{1000\,ast}{Ast} = \frac{1000 \times \pi/4 \times 20^2}{570}$$

$= 550\ mm$

$$Ast = \frac{\text{Hoop tension}}{\sigma st}$$

$$= \frac{\gamma_w HD/2}{100}$$

$$= \frac{10 \times 2 \times 12.6/2 \times 10^3}{100}$$

$= 1260\ mm^2$

using 20 mm ϕ bars

spacing = 250 mm

Vertical distribution bars :

$$Ast = 0.3\% \text{ gross area}$$

$$= \frac{0.3}{100} \times 1000 \times 190$$

$$= 570 mm^2$$

use 10mm ϕ bars

$$\text{spacing} = 1000 a_{st} / A_{st}$$

$$= 135 mm$$

Base :

provide 160 mm thick

$$Ast = 0.3\% \text{ Gross area}$$

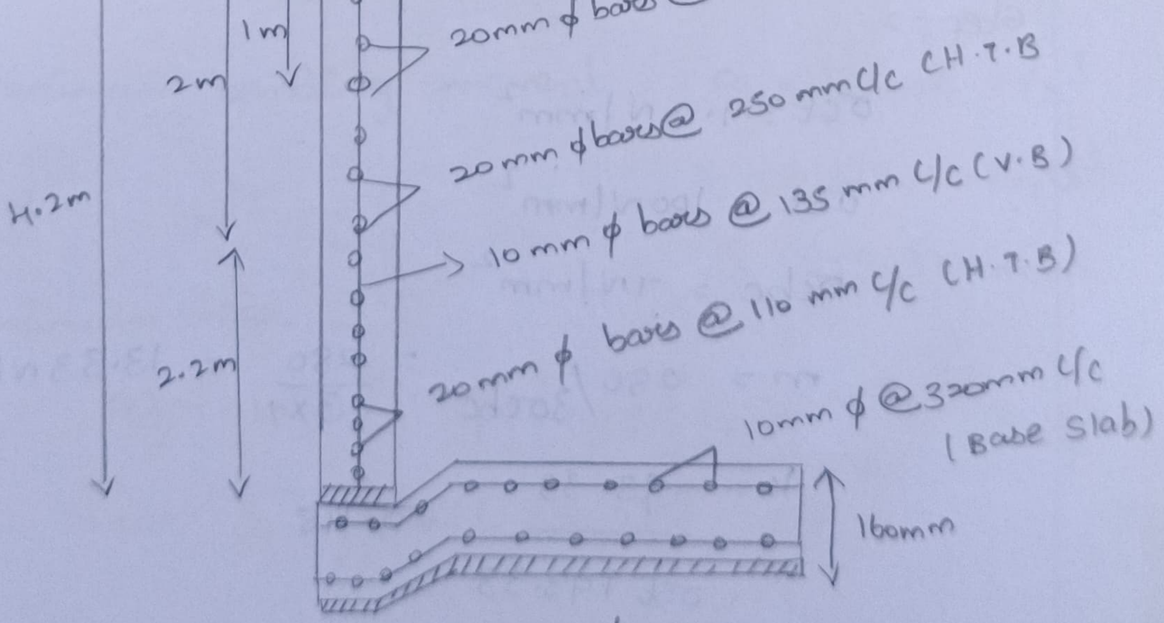
$$= 0.3/100 \times 1000 \times 160$$

$$= 480 mm^2$$

$$Ast/2 = 480/2 = 240 mm^2$$

use 10 mm ϕ bars

$$\text{spacing} = 327 \simeq 320 mm$$



Detailing diagram

2. Design a circular water tank of fixed base with capacity of 4,00,000 litres the depth of water to be 4m free board = 200 mm use M 20 grade concrete and grade 1 mild steel take $\sigma ct = 1.2\ N/mm^2$, $\sigma st = 100\ N/mm^2$. Sketch the details of reinforcement in water tanks. Adopt IS code tables for co-efficient.

Given:

Step: 1

capacity of tank = 4,00,000 litres

depth of tank = 4m

free board = 200mm

Density of water $\gamma_w = 10\ kN/m^3$

$f_{ck} = 20\ N/mm^2$

$f_y = 250\ N/mm^2$

$\sigma st = 100\ N/mm^2$

$\sigma ct = 1.2\ N/mm^2$

Step: 2

$\sigma ct = 1.2\ N/mm^2$

$\sigma st = 100\ N/mm^2$

$\sigma cbc = 7\ N/mm^2$

$m = 280/3\sigma cbc = \frac{280}{3 \times 7} = 13.33\ N/mm^2$

$k = \frac{93.33}{\sigma st + 93.33} = 0.48$

$j = 1 - k/3 = 0.84$

Step : 3

Dimension of tank :

$V = A \times h$

$400 (m^3) = \pi/4 \times D^2 \times 4$

$D = 11.28 m$

Let the thickness of walls and base slab are assumed to be 160 mm thick.

Step : 4

Bending moment, ring tension and shear force.

parameters :

$$\frac{H^2}{Dt} = \frac{4.2^2}{11.2 \times 0.16} = 10$$

Refer IS 3370 part - IV 1967 refer table : 10 pg no : 36

$10 = -0.0122$

maximum bending moment = co-efficient $\times \gamma_H{}^3$

$= -0.022 \times 10 \times 4.2^3$

$= 9.038$ KN m

maximum shear :

Refer table : 11 pg no : 37

Shear co-efficient = 0.158

maximum shear $= 0.158 \times \gamma_{w} \times H^2$

$= 0.158 \times 10 \times 4.2^2$

$= 27.87$ kN

maximum ring tension:

Refer IS 3370 part IV pg no: 38 table: 12

For

$(0.6H) = 0.608$

$N_d = \text{co-efficient} \times \gamma H D/2 \; (0.6H)$

$= \frac{0.608 \times 10 \times 4.2 \times 11.2}{2}$

$N_d = 143$ kN @ (2.52)

calculation of steel for maximum ring tension

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

$= \frac{143 \times 10^3}{100}$

$= 1430 \text{ mm}^2$

use 20 mm ϕ bars

spacing $= 1000 \, a_{st} / A_{st}$ $= 220$ mm

provide 20 mm ϕ bars @ 220 mm c/c

Thickness:

$\sigma_{ct} = \frac{\gamma_{w} H D/2}{1000t + (m-1) A_{st}}$

$$1.2 = \frac{143 \times 10^3}{1000t + (13.33-1) \times 1430}$$

$$1.2\,(1000t + 17631.9) = 143 \times 10^3$$

$$1000t = \frac{143 \times 10^3 - 17631.9}{1.2}$$

$$t = 101.53\ mm < 160\ mm$$

Steel for bending moment:

$D = 160mm \quad d = 130\ mm$

$$Ast = \frac{M}{\sigma st \cdot j \cdot d}$$

$$= \frac{9.03 \times 10^6}{100 \times 0.84 \times 130}$$

$$Ast = 826.92\ mm^2$$

use 12 mm ϕ bars

$$Spacing = \frac{1000\ ast}{Ast} \times 1000$$

$$= 135mm$$

Vertical steel:

$$Ast = 0.3\%\ bD$$

$$= 0.3/100 \times 1000 \times 160$$

$$Ast = 480\ mm^2$$

$$Ast = 240mm^2$$

use 12mm ϕ bars

spacing = 470mm

Assume

150mm thickness slab:

$Ast = 0.3\% \, bD$

$= \frac{0.3}{100} \times 1000 \times 150$

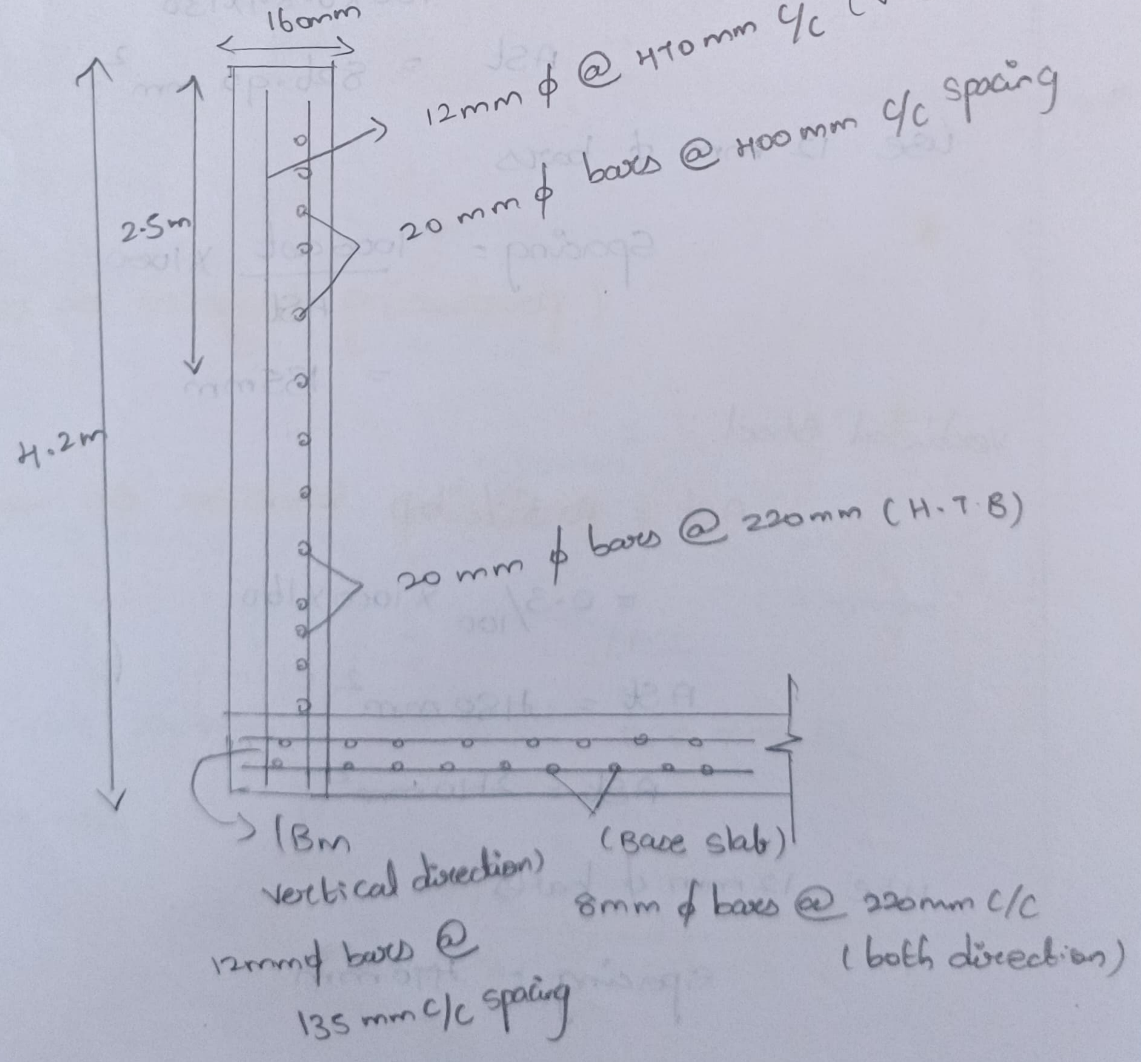
$= 450 mm^2$

$Ast = 450/2 = 225 mm^2$

provide 8mm ϕ bars

spacing = 220mm

Detailing diagram



Design of rectangular tank :

* For tank ratio $L/b < 2$
* For the bottom height $h = H/4$ (or) 1m which ever is more, the bending is in the vertical plan and this portion is designed as cantilever.
* The corners are designed for the maximum moment obtained after moment distribution with the intensity of pressure

$$p = \gamma (H - h)$$

* In the absence of moment distribution the bending moments may be calculated by the following approximate expression.
* Bending moment at centre of the span $= pB^2/16$
  (producing tension on outer face)
* Bending moment at end of the span $= pB^2/12$
  (producing tension on water face)

In addition to the bending moment the walls are subjected to direct tension is given by

Direct tension on long wall

$$T_L = \gamma (H - h) B/2$$

Tension of short wall

$$T_b = \gamma (H - h) L/2$$

therefore design moment $M - T \times x$

$$Ast_1 = \frac{(M - Tx)}{\sigma st \cdot j \cdot d}$$

$$Ast_2 = T/\sigma st$$

$$Ast = Ast_1 + Ast_2$$

Cantilever moment $= \gamma H \times h^2/6$

For ratio $L/b > 2$

Bending moment for long wall $= \gamma H^3/6$

For short wall $= \dfrac{\gamma (H - h) B^2}{16}$

3. A rectangular reinforced water tank with an open top is require to store 80,000 litres of water the inside dimension of tank. may be taken as 4 x 6m. The tank rest on wall all the four side. Design the side wall of the tank using M20 grade and grade I mild steel.

Given:

Step: 1

capacity = 80,000 litres

$L = 6m$

$b = 4m$

$f_{ck} = 20\ N/mm^2$

$f_y = 250\ N/mm^2$

Take free board = 150mm

Step: 2

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

$\sigma_{st} = 100\ N/mm^2$ (on near water face)

$\sigma_{st} = 125\ N/mm^2$ (away from water face)

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

$$k = \frac{93.33}{\sigma_{st}+93.33} = \frac{93.33}{100+93.33} = 0.48$$

$$j = 1 - k/3 = 1 - 0.48/3 = 0.84$$

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

$$= 0.5 \times 7 \times 0.84 \times 0.48$$

$$= 1.41$$

Step: 3

Dimension :

$$V = A \times H$$

$$80 = (6 \times H) \times H$$

$$H = 3.33 \simeq 3.35m$$

Free board = 150mm ⇒ 0.15m

Total height = 3.5m

step : H

pressure

$p = \gamma_w(H-h)$

$= 10(3.5-1)$

$= 25 \ kN/m^2$

Design of long span

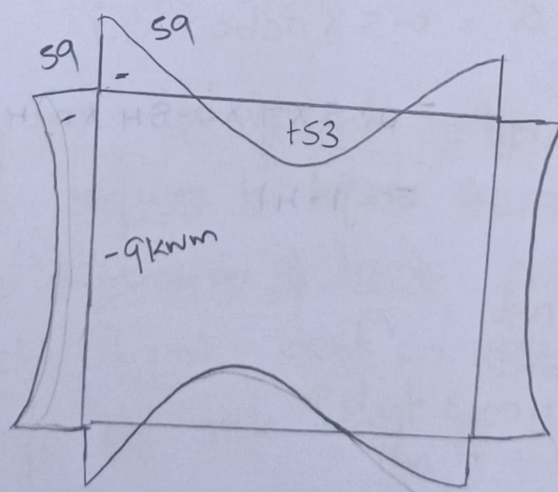
At corner $pL^2/12 = 25 \times 6^2/12 = 75$

at centre $pL^2/8 = 25 \times 6^2/8 = 112.5$

For short wall

At corner $pB^2/12 = 25 \times 4^2/12 = 39$

At centre $pB^2/8 = 25 \times 4^2/8 = 30$



Long wall

BM @ corner = 59 kNm

BM @ centre = 53 kNm

short wall

BM @ corner = 59 kNm

BM @ centre = 9 kNm

$Ast = 2500mm^2$

Half the bars from the innerface @ support (or) bend towards outerface at centre.

providing area @ centre = $2500 - 3517/2$

$Ast = 741mm^2$

provide 16mm ϕ bars

spacing = $271.3 \simeq 270mm$

provide for short wall 50% of reinforcement are provided from long wall reinforcement.

Design of vertical bar:

$B.M = \gamma H h^2/6$

$= 5.8 KNm$

$Ast = \frac{M}{\sigma st j d} = \frac{5.8 \times 10^3}{100 \times 0.84 \times 215} = 321 mm^2$

minimum Ast

$= 0.3\% \, bD$

$= \frac{0.3}{100} \times 1000 \times 250$

$= 750mm^2$

$Ast = 750mm^2$

provide 8mm ϕ spacing 60 mm.

Design of long wall

$$d = \sqrt{M/Qb}$$

$$= \sqrt{\frac{59 \times 10^6}{1.41 \times 1000}}$$

$$d = 215 mm$$

$$D = d + d'$$

$$= 215 + 35 = 250 mm$$

Long wall @ corner:

$$Ast = \frac{M - T x}{\sigma st . j . d} + \frac{T}{\sigma st}$$

$$TL = \frac{\gamma (H-h) B}{2}$$

$$= \frac{10(3.5-1) \times 4}{2}$$

$$TL = 50 kN$$

$$Ast = \left[\frac{59 \times 10^6 - (50 \times 10^3 \times 90)}{100 \times 0.84 \times 25} + \frac{50 \times 10^3}{100} \right]$$

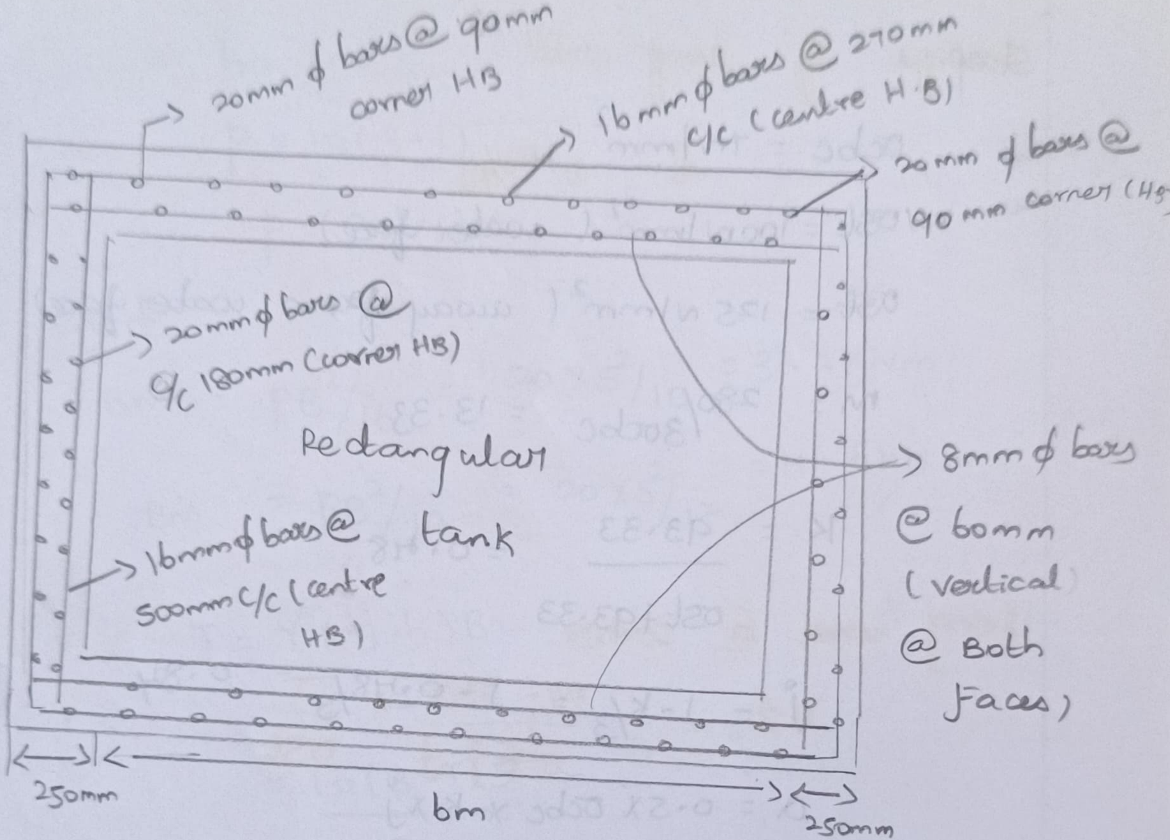
$$= 3517 mm^2$$

provide 20 mm ϕ bars

spacing = 90mm

Long wall @ centre

$$Ast = \left[\frac{53 \times 10^6 - (50 \times 10^3 \times 90)}{125 \times 0.84 \times 215} + \frac{50 \times 10^3}{125} \right]$$



4. Design the side wall of a square of RCC tank of capacity 70,000 litres of water depth of water in the tank equal to 2.8m Free board = 0.2m. Adopt M_{20} grade concrete and grade I mild steel take tensile stress in steel limited to 100 N/mm² at water face 125 N/mm² away from face. Sketch details of reinforcement in the walls of the tank.

Given:

Step: 1

capacity = 70,000 litres

depth of water in the tank = 2.8m

Free board = 0.2m

∴ H = 3m

Step: 2

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

$\sigma_{st} = 100 N/mm^2$ (water face)

$\sigma_{st} = 125 N/mm^2$ (away from water face)

$m = 280/3\sigma_{cbc} = 13.33$

$K = \frac{93.33}{\sigma_{st}+93.33} = 0.48$

$j = 1 - K/3 = 1 - 0.48/3 = 0.84$

$Q = 0.5 \times \sigma_{cbc} \times K \times j$

$= 0.5 \times 7 \times 0.84 \times 0.48$

$= 1.41$

Step: 3

Dimension:

$V = A \times h$

$70 = L^2 \times 2.8$

$L^2 = 25$

$L = 5m$

$5m \times 5m \times 3m$

Design of side walls:

$p = \gamma (H - h)$

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

$= 3/4$ (or) $1m$

$h = 1m$

$P = 10(3-1)$

$P = 20 kN/m^2$

@ centre

$BM = PB^2/16 = 20 \times 5^2/10 = 31.2 kNm$

$BM = PB^2/12 = 20 \times 5^2/12 = 41.6 kNm$

$T = \frac{\gamma(H-h)B}{2}$

$= \frac{10(3-1)5}{2}$

$= 50 kN$

$d = \sqrt{m/Qb}$

$= \sqrt{41.6 \times 10^6 / 1.41 \times 1000}$

$= 172 mm \simeq 220 mm$

$D = d + d'$

$= 220 + 30 = 250 mm$

Corner:

$Ast = \frac{M - Tx}{\sigma st . j . d} + \frac{T}{\sigma st}$

$= \frac{41.6 \times 10^6 - 50 \times 10^3 \times 95}{100 \times 0.84 \times 220} + \frac{50 \times 10^3}{100}$

$Ast = 2507\ mm^2$

provide 20mm ϕ bars

spacing = 120mm

provide @ 20mm ϕ bars @ 120mm c/c

centre:

$Bm = PB^2/16$

$= 31.2 kNm$

$$Ast = \frac{31.2\times10^6 - 50\times10^3\times95}{125\times0.84\times220} + \frac{50\times10^3}{125}$$

$= 1547 mm^2$

$Ast = 1547 - 250/2$

$= 293.5 mm$

provide 10mm ϕ bars

spacing = 260 mm

provide @ 10mm ϕ bars 260mm c/c spacing

vertical reinforcement:

Cantilever moment = $\gamma_H \times h^2/6$

$= 10\times3\times1^2/6$

moment $= 5 kNm$

$$Ast = \frac{M}{\sigma st \cdot j \cdot d} = \frac{5\times10^6}{100\times0.84\times220}$$

$= 270 mm^2$

minimum Ast:

$= 0.3\% \cdot bD$

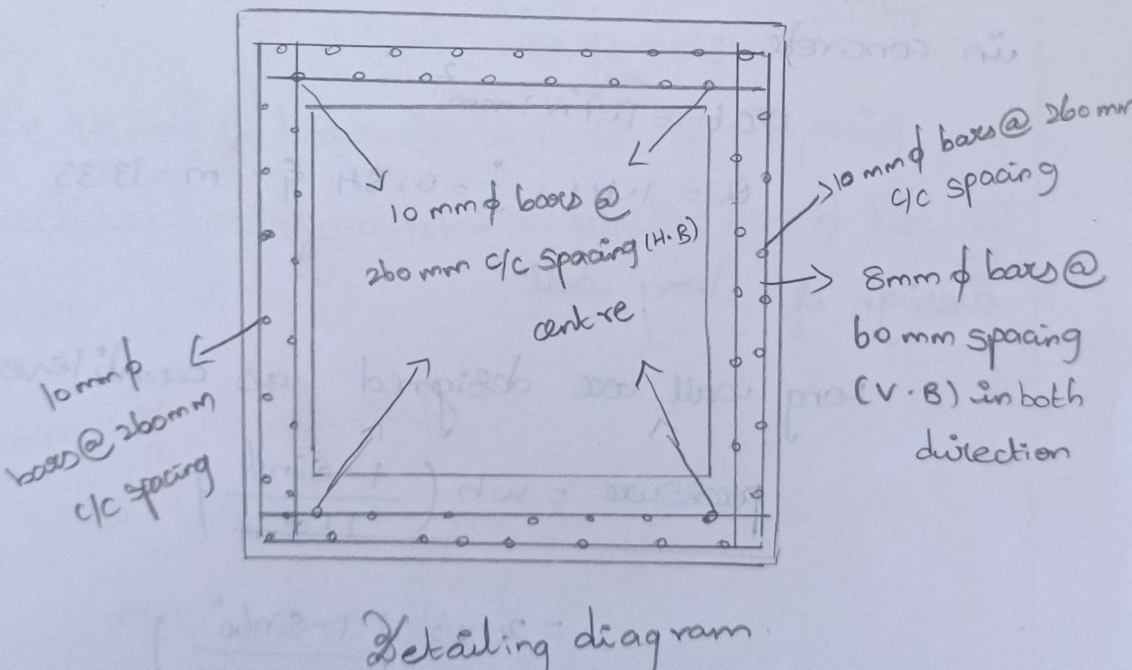
$= \frac{0.3}{100} \times 1000 \times 250$

$= 750 mm^2$

provide 8 mm ϕ bars

Spacing = 60mm c/c

8mm ϕ @ 60mm c/c as a vertical reinforcement from bottom.



Detailing diagram

5. Design of underground water tank design an RC tank of internal dimension 10 X 3 X 3m the tank is to provide under ground the soil surrounding the tank is likely to get wet angle of repose of soil in dry state in 30° and wet state in 6°. Adopt suitable working stresses soil weight in 20 kN/m³. Adopt M20 grade and grade I steel.

Given:

size of tank = 10m X 3m X 3m

Tank surrounded by soil on all sides

Angle of repose of soil in dry state = 30°

wet state = 60°

Density of soil = $20 kN/m^3$

permissible stresses:

$\sigma st = 100 N/mm^2$

permissible stress in tension due to bending in concrete

$\sigma ct = 1.7 N/mm^2$

$Q = 1.41$, $j = 0.84$ & $m = 13.33$

Design of long wall:

Long wall are designed as cantilever ($L/B > 2$)

$$\text{pressure} = wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

$$= 20 \times 3 \left(\frac{1-\sin 60^\circ}{1+\sin 60^\circ}\right)$$

$$p = 48.64 N/m^2$$

$$(ph^2/33.5) = \left(\frac{48.64 \times 3^2}{33.5}\right) = 13.06 kNm$$

$$(ph^2/15) = \left(\frac{48.64 \times 3^2}{15}\right) = 29.18 kNm$$

From cracking consideration:

Design of short walls:

short wall are design

$P = 48.64 \ kN/m^2$

max moment $= Pl^2/12$

$= 48.64 \times 3.3^2/12$

$= 45.48$

$d = \sqrt{\dfrac{45.84 \times 10^6}{1.41 \times 1000}}$

$d = 180mm$

Actual thickness of wall provide $= 350mm$

$Ast = \left(\dfrac{45.48 \times 10^6}{100 \times 0.84 \times 310}\right) = 1746$

use 18mm ϕ bars @ 145 mm c/c spacing

vertical reinforcement = use 10mm ϕ bars @ 150mm

Design of roof slab:

$D.L = 2.4 kN/m^2$

$L.L = 1.5 kN/m^2$

$F.L = 0.6 kN/m^2$

$T.L = 4.5 kN/m^2$

$$M = \left(\frac{\sigma_{ct} \cdot b \cdot D^2}{6} \right) = \left(\frac{1.7 b D^2}{6} \right)$$

$$= 0.28 b D^2$$

$$D = \left(\frac{29.18 \times 10^6}{100 \times 0.84 \times 310} \right)$$

$$= 1120 \text{ mm}^2$$

Spacing of 18 mm ϕ bars

$$\text{Spacing} = \frac{1000 \, a_{st}}{A_{st}} = 226 \text{ mm}$$

provide 18 mm diameter vertical bars @ 220 mm c/c

steel for BM = 13.06 kNm

$$A_{st} = \left(\frac{13.06 \times 10^6}{100 \times 0.84 \times 310} \right)$$

$$= 502 \text{ mm}^2$$

provide 12 mm ϕ bars

$$\text{spacing} = \frac{1000 \, a_{st}}{A_{st}} = 225 \text{ mm c/c}$$

use 12 mm ϕ bars @ 225 mm horizontal reinforcement

$$= \left(\frac{0.3}{100} \times 350 \times 1000 \right)$$

$$= 1050 \text{ mm}^2/\text{m}$$

provide on both faces

$$\text{Area of steel} = 525 \text{ mm}^2/\text{m}$$

use 10 mm ϕ bars @ 150 mm c/c horizontal @ both faces.

UNIT-3

SELECTED TOPICS

Design of staircase :

Staircase are generally provided connecting the successive floor of a building and in small building these are the only means of access between the floor.

The staircase compraises of flight of steps Generally with one or more intermediate Landing provided between the floor level.

The structural component of flight of stair consist of

i) Tread → which forms the horizontal portion of the steps. The Tread is usually 250MM - 300MM wide, depending upon the type of the building.

ii) Riser → It is a vertical distance between the adjucent treads or the vertical project of the step. Generally in the range of 150MM - 190MM Depending up on the type of building.

iii) The width of the stair varies in the range of 1 to 1.5m, With the minimum of 850MM

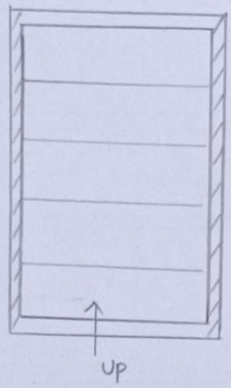
iv) Going forms the horizontal plan protection of an inclined flight off step's between the first and last raiser.

A flight of steps consist of two landings and one going with 10-12 steps.

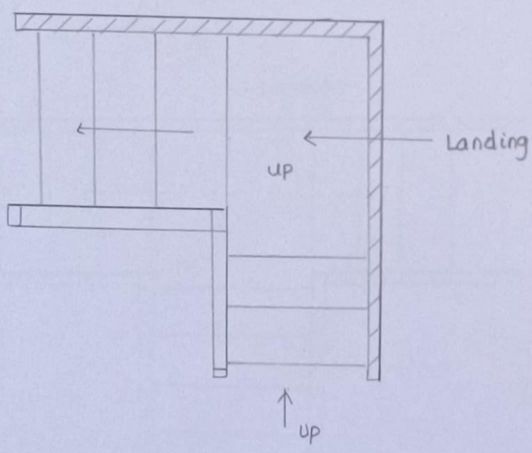
Types of staircase:

1) single straight staircase
2) Dog legged staircase
3) open well staircase
4) Tread and riser type Staircase
5) Cantilever Staircase
6) Spiral staircase and circular staircase
7) Quarter turn staircase
8) Three Quarter turn staircase
9) Bi furcated Staircase.

Straight staircase:

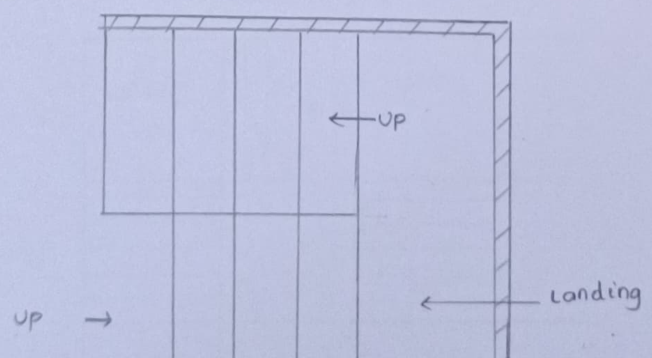


Quater turn stair:

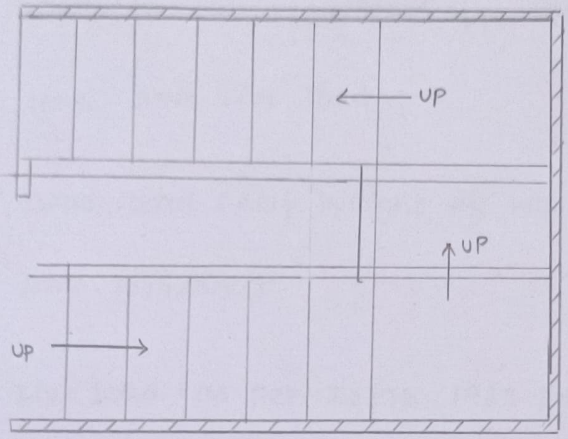


Plan

Dog legged stair:

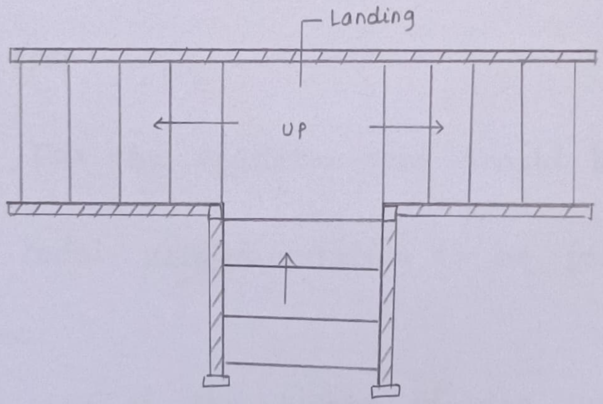


Three Quarter turn stair:



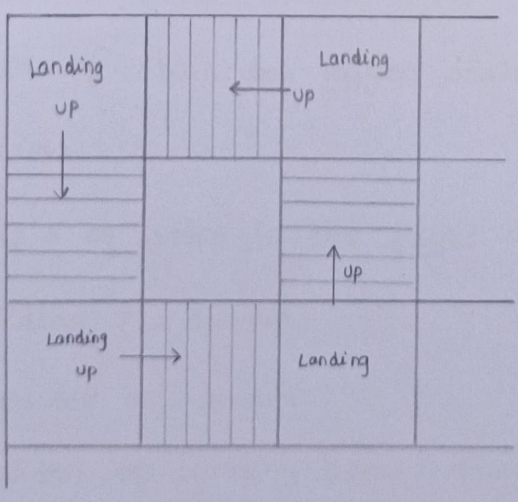
plan

Bifurcated stair:



plan

Multiple flight stair:



Load on Staircase:

Load to be considered for the design of staircase are dead load and Live load.

Dead Load (self weight of waist slab, Tread and riser and finishers)

Live Load (as per IS875 : 1987 Part - II residential building 2kN/m² to 3kN/m²

For public bulding 5kN/m²

Effective span:

For the effective span should be provided by the codal recommondation to be followed as per IS456 :2000

1. Design a one of the flight of dog logged staircase spanning between the Landing beams using the following data.

1. Type of stair dog logged staircase with waist lab, treads, raiser

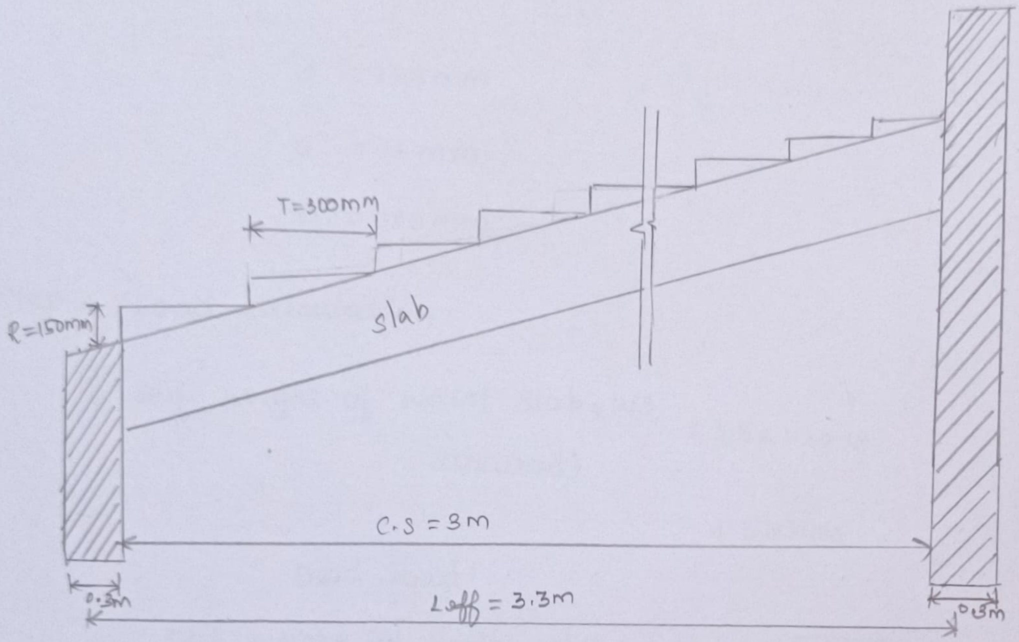
2. No's of steps in the flight = 10

3. Tread T = 300MM

4. Raiser R = 150MM

5. Width of Landing beam = 300MM

6. Materials used M20 grade concrete & Fe415 steel.



Step 1: Effective span calculation:

clear span = No. of steps x Tread

= 10 X 300

= 300mm

= 3m

Leff = C.S + $(\frac{1}{2}b) \times 2$

= 3m + 0.3m

Leff = 3.3m

Step 2: calculate the effective depth of waist slab

[Refer IS456 : 2000 pg.no - 37 clause 23.1.2]

$$\frac{\text{eff span}}{\text{eff depth}} = 20$$

$$\frac{3300}{d} = 20$$

$d = 165\,mm$

$d' = 15\,mm$

$D = 180\,mm$

Step 4 : Load calculation

Self weight of waist slab, WS (Inclined) $= 25 \times 1 \times 0.18$

$= 4.5\,kN/m$

Dead load

Self weight of waist slab $= WS \times \frac{\sqrt{T^2+R^2}}{T}$

$= 4.5 \times \frac{\sqrt{0.3^2+0.15^2}}{0.3}$

Dead load (Slab) $= 5.03\,kN/m$

Dead Load on step:

Self weight of one step $= 25 \times \frac{1}{2} \times b \times D$

$= 25 \times \frac{1}{2} \times 0.3 \times 0.15$

$= 0.56\,kN/step$

Self weight of step/m $= \frac{0.56 \times 1}{0.3}$

$= 1.86\,kN/m$

Live load on staircase (Assume) $= 3\,kN/m^2$

Take finishing load $= 0.125\ kN/m^2$

Total load on stair $=$ (self weight of waist slab lm + self weight of step lm + L.L + F.L)

$= 5.03 + 1.87 + 0.125 + 3$

$W = 10.02\ kN/m$

$W_u = 1.5 \times 10.02$

$= 15.03\ kN/m$

Step 4 : Moment calculation

$$M_u = \frac{W_u L_{eff}}{8}$$

$$= \frac{15.03 \times 3.3^2}{8}$$

$M_u = 20.45\ kNm$

Step 5 : check for depth

[Refer IS 456 : 2000 pg. no. 96]

$$d_{act} = \sqrt{\frac{M_u}{0.138 \times f_{ck} \times b}}$$

$$= \sqrt{\frac{20.45 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d_{act} = 86.07\ mm$

$d_{act} < d_{provided}$

Hence safe.

Step 6 : Reinforcement calculation

[Refer IS456 : 2000 pg. no. 96 cl -G -1.2.1b]

$$Mu = 0.87\, fy\, Ast\, d \left[1 - \frac{fy\, Ast}{fck\, b\, d} \right]$$

$$20.45 \times 10^6 = 0.87 \times 415 \times Ast \times 165 \left[1 - \frac{415\, fck}{20 \times 1000 \times 165} \right]$$

$(Ast)act = 359 mm^2$

$(Ast)min = 0.12\% \cdot bD$

$$= \frac{0.12}{100} \times 1000 \times 180$$

$Ast(min) = 216 mm^2$

Spacing

Use 12mm ϕ bar in (M.B)

$$Spacing = \frac{\pi/4 \times 12^2}{359} \times 1000$$

= 315mm

≃ 300mm

Provide 12mm ϕ @ 300mm c/c as MR

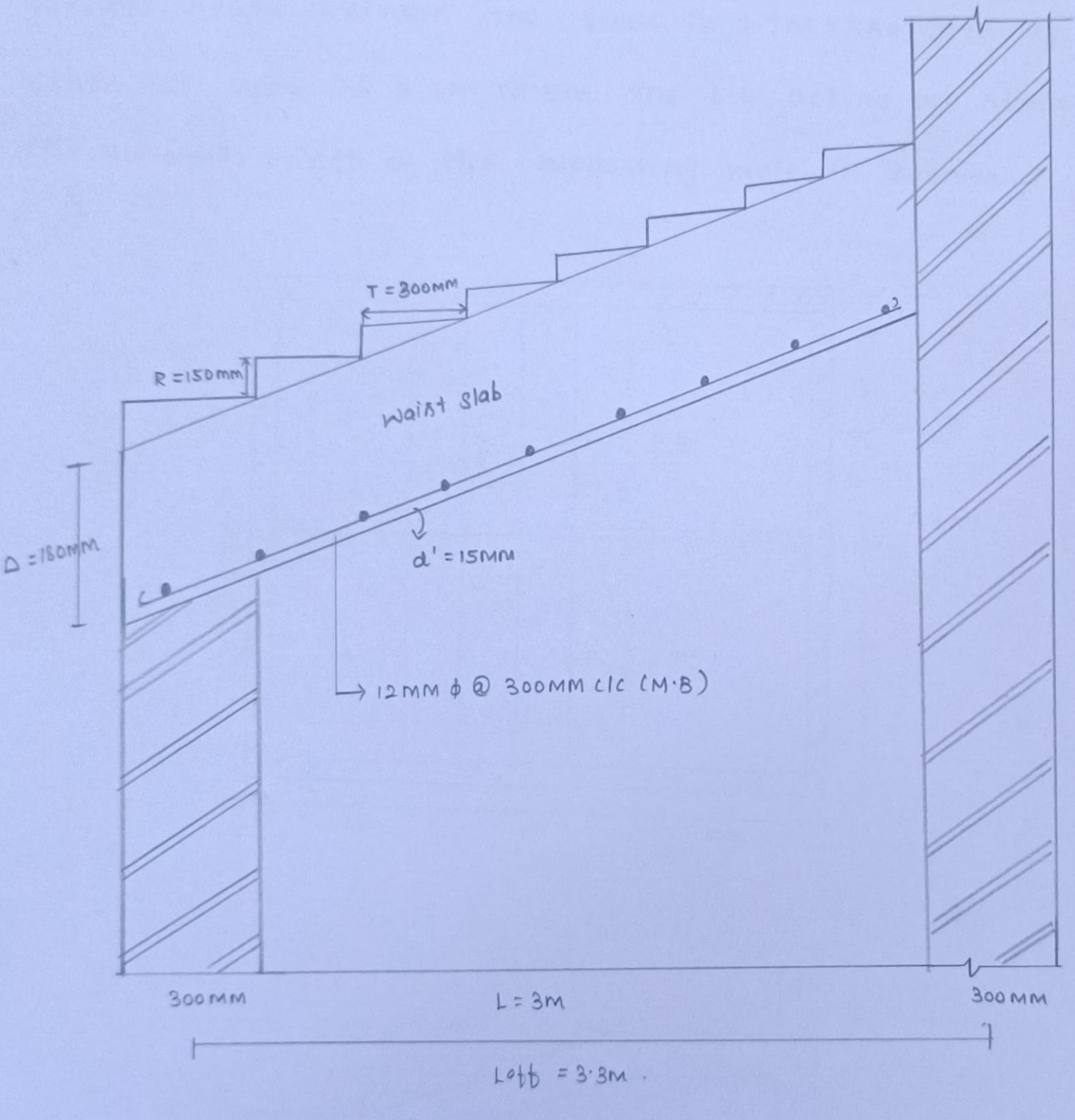
Use 8mm ϕ bar in (DB)

$$Spacing = \frac{\pi/4 \times 8^2}{216} \times 1000$$

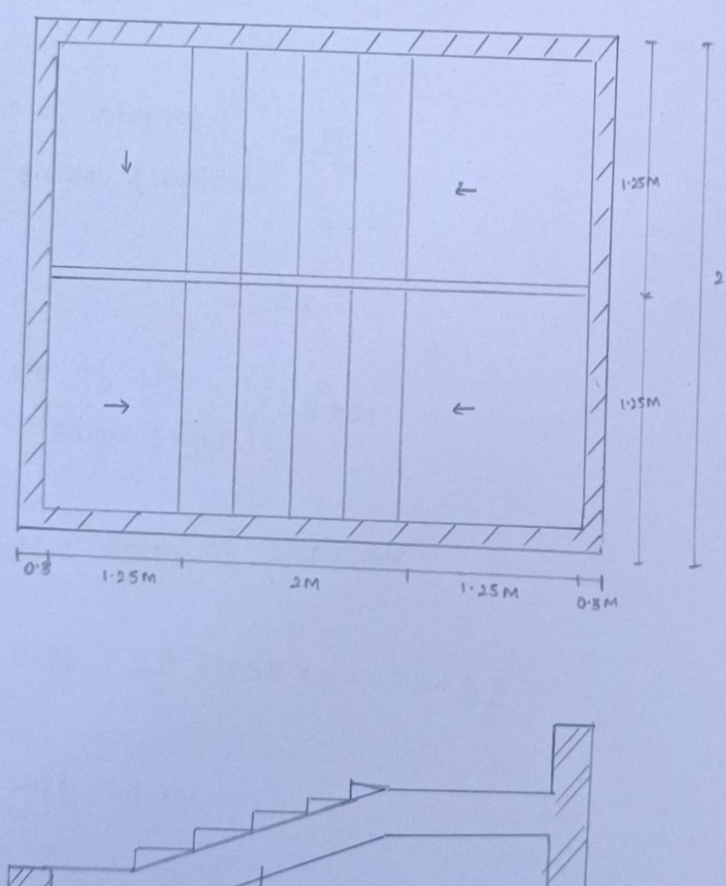
= 233mm ≃ 235mm

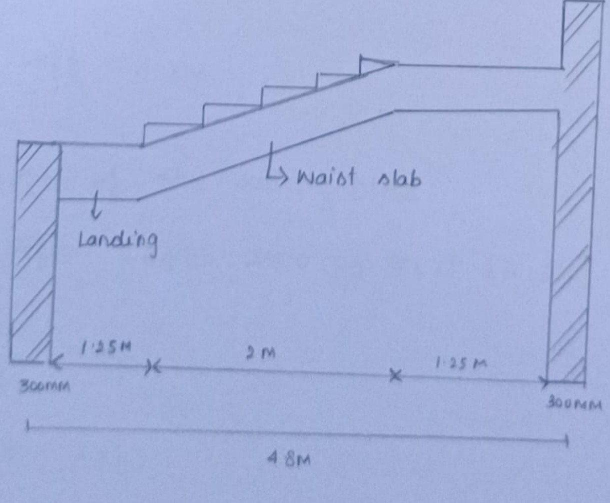
Provide 8mm ϕ @ 235 mm c/c as a DB

Step 7 : Detailing Diagram :



2) Design a dog logged staircase for indermediate RC Multistore Building. Use M20 grade & Fe415 steel. The vertical height between the floor is 2.7M. The size of stair set roof is 4.5M X 2.5M. The L.L acting on stair is 3 KN/m2. width of the supporting wall is 300MM.





Step 1: Number of steps

Take $T = 250MM$

$R = 150MM$

$H = 2.7M$

$$No.of.Slope = \frac{2700}{150}$$

$= 18$ no's

$$\left.\begin{array}{l}\text{No. of step of}\\ \text{single flight}\end{array}\right\} = \frac{18}{2}$$

$= 9$

$T = 8$

$$\left.\begin{array}{l}\text{No. of step}\\ \text{single flight}\end{array}\right\} = 8 \text{ no's}$$

Step 2: Effective span of Staircase

$$L_{eff} = \frac{0.3}{2} + 1.25 + 2 + 1.25 + \frac{0.3}{2}$$

$$L_{eff} = 4.8M$$

Step 3: Depth of the waist slab

[Refer IS456 : 2000 pg. 37 cl. 23.2.1]

$$\frac{span}{d} = 20$$

$$\frac{4800}{d} = 20$$

$d = 240MM$

$d' = 20MM$

$D = 260MM$

Step 4 : Load calculation :

Load on inclined flight

Self weight of waist slab, Ws (inclined) $= 25 \times 1 \times 0.26$

$= 6.5 \, kN/m$

Self weight of waist slab (Horizontal) $= Ws \times \frac{\sqrt{T^2 + R^2}}{T}$

$= 6.5 \times \frac{\sqrt{0.25^2 + 0.15^2}}{0.25}$

$= 7.58 \, kN/m$

One step load $= 25 \times \frac{1}{2} \times 0.25 \times 0.15$

$= 0.46 \, kN/m$

Step load on 1m length

$= \frac{0.46 \times 1}{0.25}$

$= 1.84 \, kN/m$

Total load $W = 7.58 + 1.84 + 3 + 0.125$

$W = 12.54 \, kN/m$

$W_u = 12.54 \times 1.5$

$W_u = 18.81 \, kN/m$.

Load on Landing slab :

Dead Load $= 25 \times 1 \times 0.26$

$= 6.5 \, kN/m$

Total load $= 6.5 + 3 + 0.125$

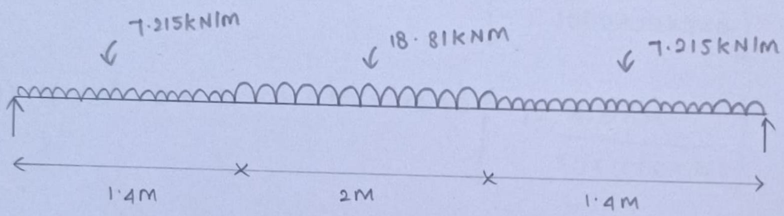
$W = 9.625$ kN/m

$W_u = 9.625 \times 15$

$W_u = 14.48$ kN/m.

Wu / landing slab $= \dfrac{14.43}{2}$

$= 7.215$ kN/m.



ΣM @ B

$$R_A \times 4.8 - (7.215 \times 1.4)\left(\frac{1.4}{2} + 2 + 1.4\right) - (18.81 \times 2)\left(2/2 + 1.4\right) - (7.215 \times 1.4)\left(\frac{1.4}{2}\right) = 0$$

$R_A \times 4.8 = 41.41 + 53.66 + 7.07$

$R_A \times 4.8 = 102.14$

$R_A = 21.27$ kN

$M = (28.9 \times 2.4) - (7.215 \times 1.4)\left(\frac{1.4}{2} + 1\right) - (18.81 \times 1 \times 1/2)$

$M_u = 42.78$ kNm.

step 5:
check for depth.

$$d = \sqrt{\frac{M_u}{0.138 \times f_{ck} \times bD}}$$

$$= \sqrt{\frac{42.78 \times 10^6}{0.138 \times 20 \times 1000}}$$

dact = 125 MM

dprovided = 240 MM

Hence safe.

Ast calculation:

$$Mu = 0.87\, fy\, Ast\, d \left[1 - \frac{fy\, Ast}{fck\, bd}\right]$$

$$= 0.87 \times 415 \times Ast \times 240 \left[1 - \frac{415\, Ast}{1000 \times 20 \times 240}\right]$$

$$42.78 \times 10^6 = 0.87 \times 415\, Ast \times 240 \left[1 - \frac{415\, Ast}{20 \times 1000 \times 240}\right]$$

$$Ast = 516.78\, MM^2$$

Main bar

Use 12 mm ϕ

$$spacing = 1000 \frac{ast}{Ast}$$

$$= \frac{1000 \times \frac{\pi}{4} \times 12^2}{516}$$

$$= 218\, mm$$

$$\simeq 210\, MM$$

provide 12 MM ϕ @ 210 MM c/c

Distribution bar:

$$Ast_{(min)} = 0.12\%\, bD$$

$$= \frac{0.12}{100} \times 1000 \times 240$$

$$Ast_{(min)} = 312\, mm^2$$

spacing:

use 8mm ϕ bar

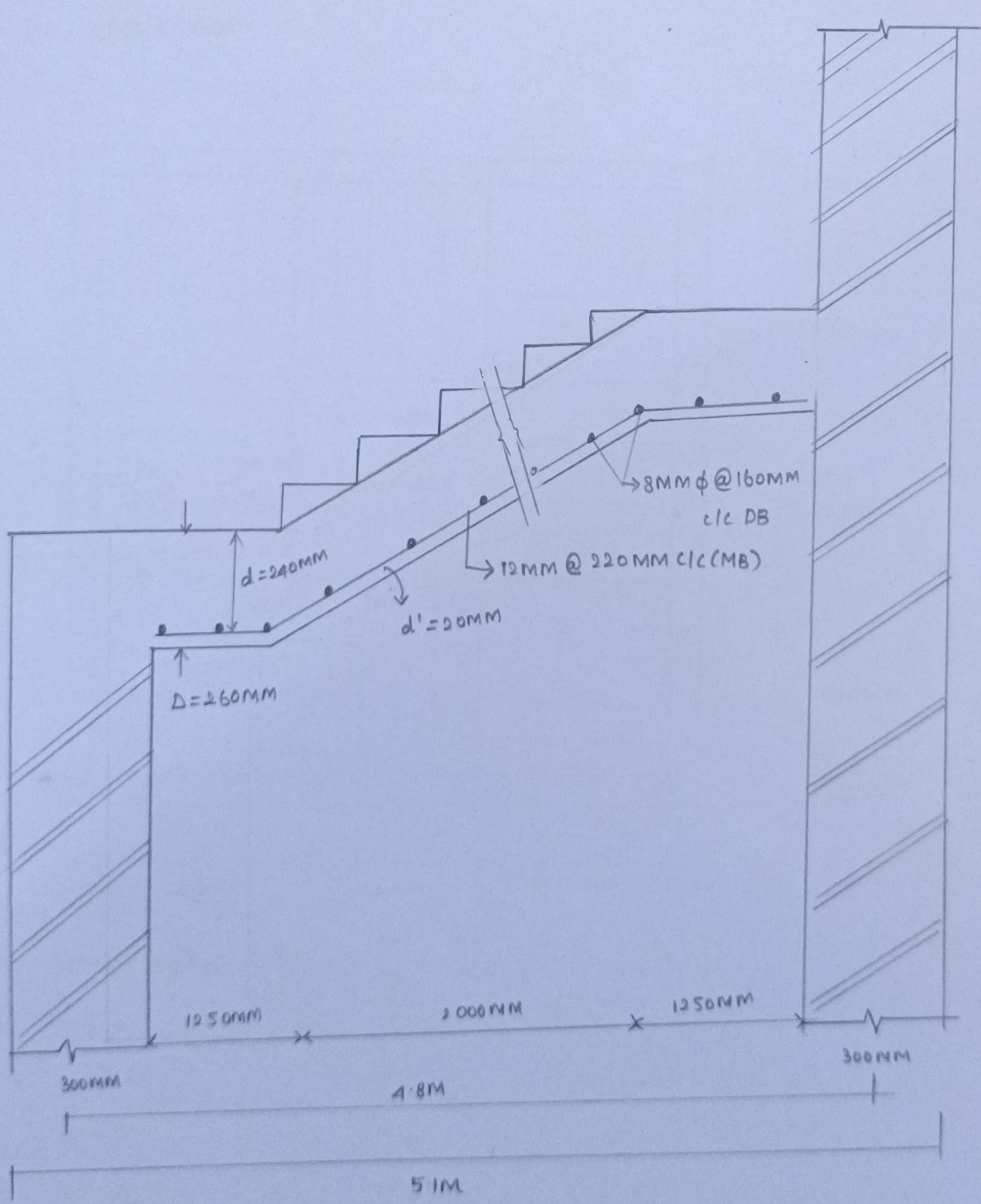
$$\text{spacing} = \frac{1000 \times \pi/4 \times 8^2}{312}$$

$= 161$

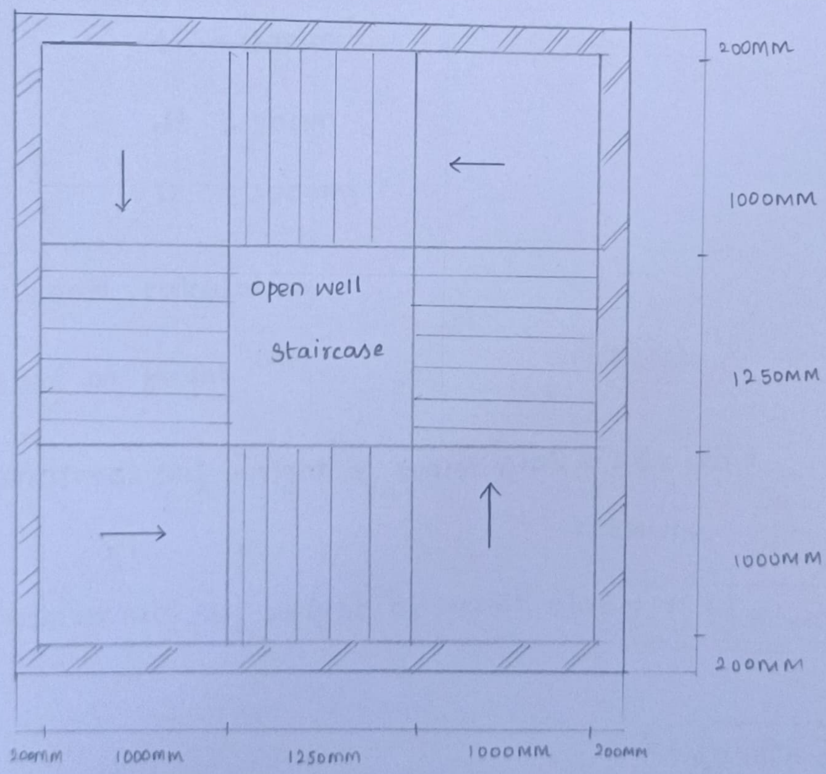
$\simeq 160$ mm

provide 8mm ϕ @ 160mm spacing c/c as DB.

step 7: Detailing Diagram:



The General arrangement of Staircase in a Multi story complex is shown in fig. The raiser are 150MM and the tread are 250MM, the stair slab (waist) is embedded into wall by 200mm the height b/w the floor is 3M The service Live Load is 3 kN/m^2. Adopt M20 grade & Fe415 bars. Design the staircase, flight and draw the longitudinal section and shown in the details of reinforcement in the flight of the staircase.



Step 1: Given data

T = 250mm

R = 150MM

no. of T = 5

no of R = 6

$$\text{no. of step} = \frac{1250}{250}$$

$$= 5 \text{ no's}$$

Step 2: Effective span

$$L_{eff} = 3.45M$$

Step 3: Thickness of the slab (depth of slab)

$$\frac{Span}{d} = 20$$

$$\frac{3450}{d} = 20$$

$$d = 180mm$$

$$d' = 20mm$$

$$D = 200mm$$

Step 4: Load calculation:

Load on flight

(Inclined) self weight of waist slab $= 25 \times 1 \times 0.2$

$= 5KN/m$

(Horizontal) self weight of waist slab $= \frac{W_s \times \sqrt{R^2+T^2}}{T}$

$$= \frac{5 \times \sqrt{0.15^2 + 0.25^2}}{0.25}$$

$$= 5.83KN/m$$

Load on single step $= 25 \times 1/2 \times 0.25 \times 0.15$

$= 0.46 \ KN/step$

Load on step 1m length $= \frac{0.46 \times 1}{0.25}$

$= 1.84$ KN/m

Total load on flight :

= D.L + L.L + F.L

= (5.83 + 1.84) + 3 + 0.125

= 10.79 KN/m

Design load = Wu = 1.5 × 10.79

= 16.19 KN/m

Load on Landing slab :

Self weight of L.S = 25 × 1 × 0.2

= 5KN/m

Live load = 3KN/m

Finishing load = 0.125 KN/m

8.125 KN/m

Total Load = 8.125 KN/m

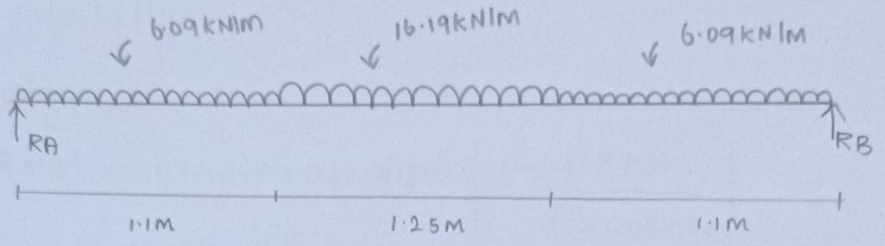
Design load = 1.5 × 8.125

= 12.18 KN/m

For each Landing slab carries a load

$= \frac{12.18}{2}$

= 6.09 KN/m



Moment @ B

$$R_A \times 3.45 - (6.09 \times 1.1)\left(\frac{1.1}{2} + 1.25 + 1.1\right) - (16.19 \times 1.25)\left(\frac{1.25}{2} + 1.1\right) - (6.09 \times 1.1)\left(\frac{1.1}{2}\right) = 0$$

$$R_A = 16.81\ KN$$

Moment @ centre

$$M_u \Rightarrow (16.81 \times 1.75) - (6.09 \times 1.1)\left(\frac{1.1}{2} + 0.625\right) - (16.19 \times 0.625)\left(\frac{0.625}{2}\right)$$

$$M_u = 17.9\ KNM$$

Step 5 : check for depth

$$d = \sqrt{\frac{M_u}{0.138 \times f_{ck} \times b}}$$

$$= \sqrt{\frac{17.9 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 80.53\ MM$$

$d_{act} < d$ provided

Hence safe

Ast calculation:

Main:

$$17.9\times10^6 = 0.87\times415\times Ast\times180\times\left[1-\frac{415\,Ast}{20\times1000\times180}\right]$$

$$17.9\times10^6 = 64989\left[1-1.15277\times10^{-4}\right]$$

$$7.4917875\,Ast^2 - 64989\,Ast + 17.9\times10^6 = 0$$

$$Ast = 284\,mm^2$$

Use 10mm ϕ

$$Spacing = \frac{1000\times ast}{Ast}$$

$$= \frac{1000\times\pi/4\times10^2}{284}$$

$$= 276$$

$$\simeq 270\,mm$$

provide 10mmϕ @ 270mm c/c spacing M.B

$$Ast\,(min) = \frac{0.12}{100}\times1000\times180$$

$$= 216\,mm^2$$

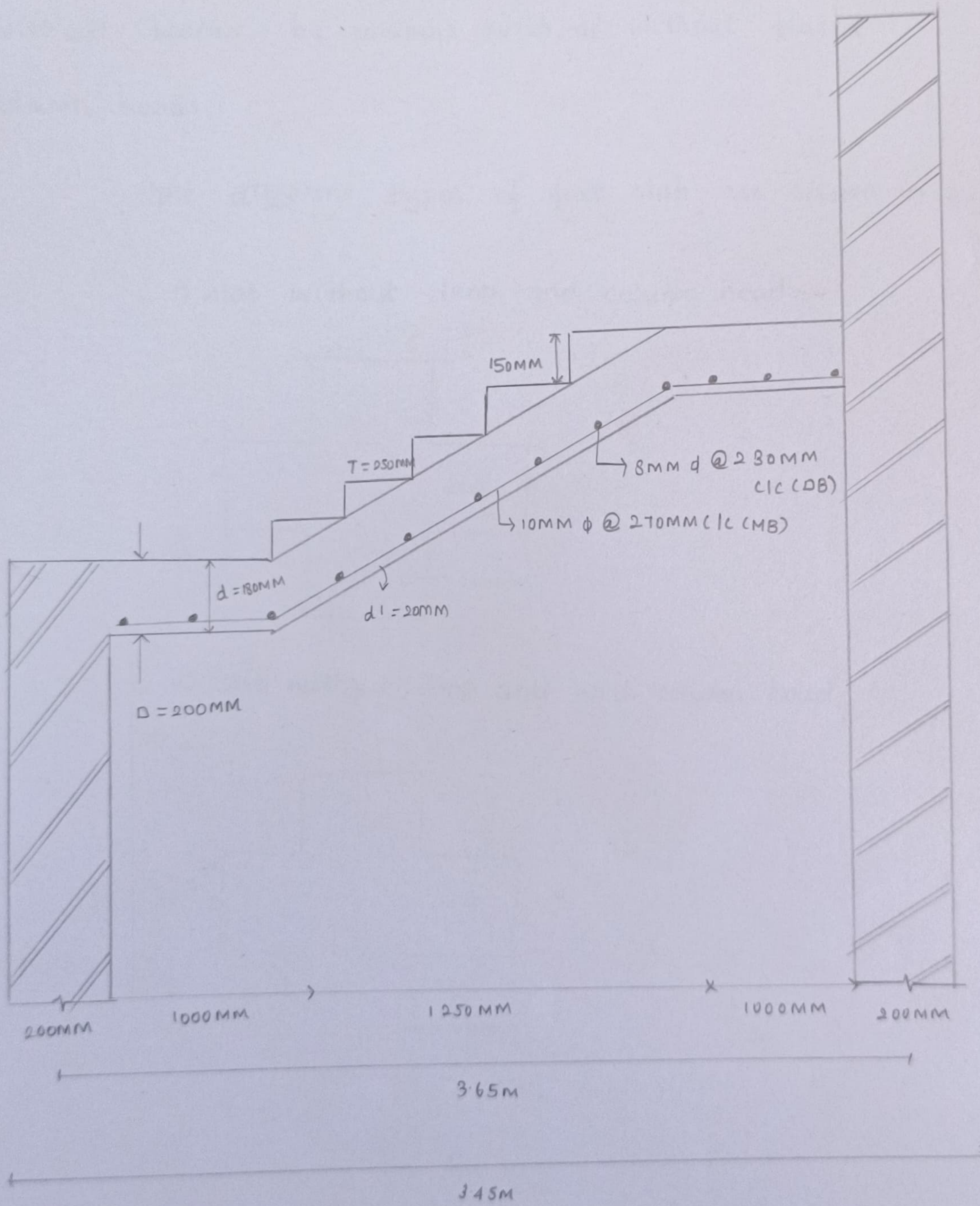
use 8mm ϕ

$$Spacing = \frac{1000\times ast}{Ast}$$

$$= \frac{1000\times\pi/4\times8^2}{216}$$

$$= 230\,mm$$

provide 8mm ϕ @ 230 mm c/c DB

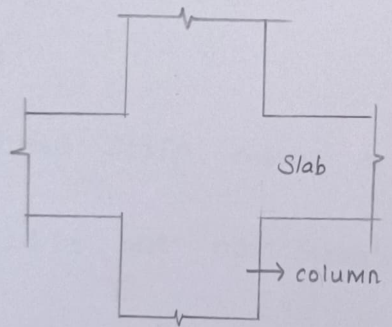

150MM
T=250MM
8MM ϕ @ 230MM C/C (DB)
10MM ϕ @ 270MM C/C (MB)
d=180MM
d' = 20MM
D=200MM
200MM
1000MM
1250MM
1000MM
200MM
3.65M
3.45M

Flat slab :

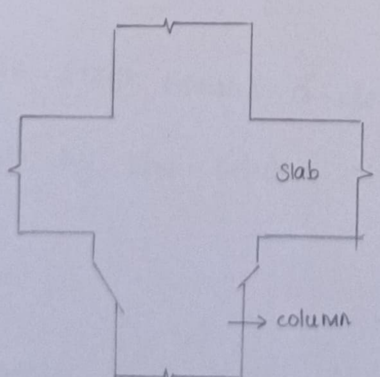
The term flat slab means a reinforced concrete slab with or without drops, supported generally without beams, by columns with or without flared column heads.

The different types of flat slab are shown in fig.

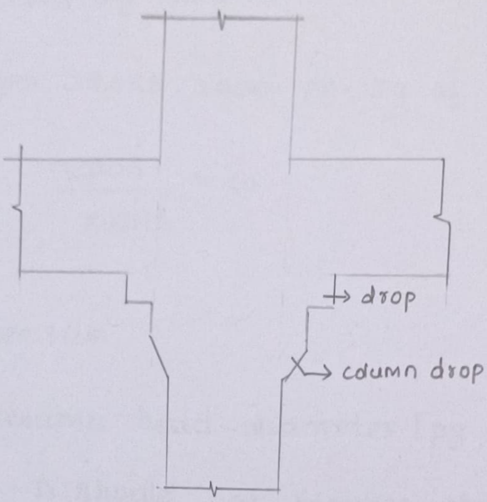
1) slab without drop and column head.



2) Slab without drop and with column head



3) slab with drop and column head:



column strip:

column strip means a design strip having a width of 0.25 l_2 but not Greater than 0.25l_1 on each site of the column centre line.

Middle strip:

Middle strip means a design strip bounded on opposite side by the column strip.

Panel:

Panel moan that part of a slab bounded on each of its four sides by the centre - line of a column or contre line of adjacent spans.

Design procedure for flat slab:

Step 1: Thickness of slab

[Refer IS456 : 2000 pg. 39 of cl - 24.1]

$$\frac{\text{Span}}{\text{Depth}} = 40$$

Step 2 : Dimension

i) column head diameter [pg. NO : 53 cl - 31.2.3]

D should not Greater than 0.25 times of the length (Max length)

ii) Length of the Drop :

It should not less than L/3 in each direction

iii) column & Middle strip dimension

[Refer IS456 : 2000 of pg. 33 cl 31.1.1 a, b]

Step 3 : Load calculation

Step4 : Moment calculation [Refer IS456 : 2000 pg. 55 cl - 3121.2.2]

$$M_D = \frac{WLn}{8}$$

Step 5 : Reinforcement calculation [pg. 96 of G.1.1, b]

Step6 : Detailing Daigram.

Design the interior panel of a flat slab with drop for an office floor to suit the following Data

Size of an office floor = 25X25m

Panel size = 5X5M

Loading class = 4 KN/m²

Use M20, fe 415 steel

Step 1: Thickness of slab

[Refer IS456:2000 pg. 39]

$\frac{Span}{D} = (40 \times 0.8)$

$\frac{5000}{D} = 32$

D = 156mm

D ≃ 150mm

d' = 20mm

d = 130mm

Provide drop depth as 50mm overall thickness of Slab (with drop) = 200mm.

Step 2 : Dimension

i) column Head

dia of column Head

D = L X 0.25

= 5 X 0.25

= 1.25M

Adopt column head dia = 1.25M

ii) Length of drop (should not less than L/3)

= L/3

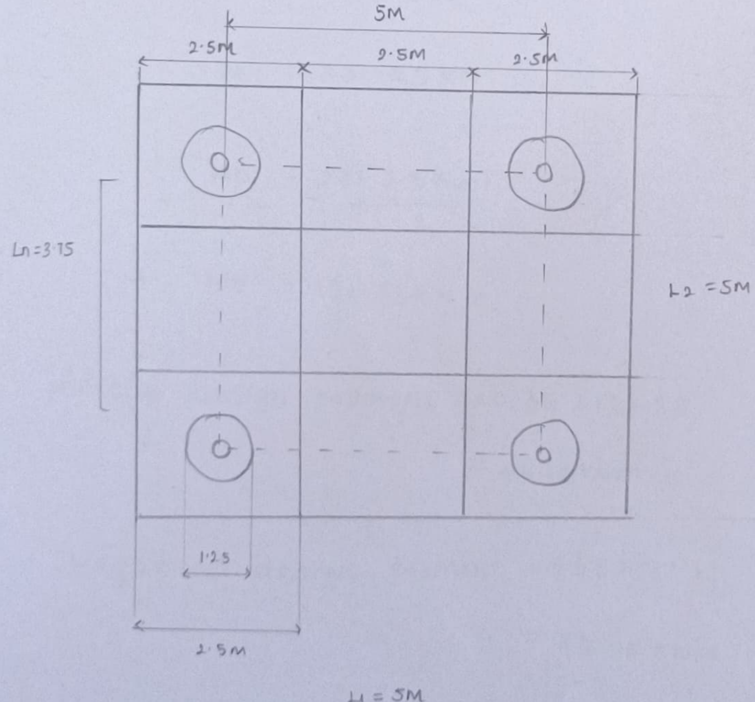
= 5/3

= 1.66M

Adopt length of drop = 2.5M

iii) column & Middle strip

Adopt column strip = Middle strip = 2.5M



step 3 : Load

Self weight of slab = 25 X 1 X 0.20

= 5 KN/M

Live load = 4 KN/M²

= 4 KN/M

Finishing load $= 1 kN/m$.

Total load $= 10 kN/m^2$

Design load $= 1.5 \times 10$

$W_u = 15 kN/m^2$.

Step 4 : Moment calculation

$$M_o = \frac{W l_n}{8}$$

$$W = W_u \times l_2 \times l_n$$

$$= 15 \times 5 \times 3.75$$

$$W = 281.25 kN$$

$$M_o = \frac{281.25 \times 3.75}{8}$$

$$M_o = 131.83 kN \cdot m$$

positive design moment $= 0.35 \times 131.83$

$= 46.14 kNm$

Negative design moment $= 0.65 \times 131.83$

$= 85.68 kNm$.

	column strip	Middle strip
+ve	85% = 39.21	15% = 6.92
-ve	85% = 72.76	15% = 12.84

column strip:

+ve reinforcement

$$Mu = 39.2 \, kNM$$

$$Mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{fy \, Ast}{fck \, b \, d} \right]$$

$$39.2 \times 10^6 = 0.87 \times 415 \times Ast \times 180 \left[1 - \frac{415 \, Ast}{20 \times 1000 \times 180} \right]$$

$$39.2 \times 10^6 = 64989 \, Ast \left[1 - 1.1527 \times 10^{-4} \right] Ast$$

$$39.2 \times 10^6 = 64989 \, Ast - 7.4917 \, Ast^2$$

$$Ast = 652 \, mm^2$$

Use 16mm ϕ

$$Spacing = \frac{1000 \times \pi/4 \times 16^2}{652}$$

$$= 300 \, MM$$

provide 16MM ϕ @ 300MM c/c

-ve reinforcement

$$Mu = 72.76$$

$$72.76 \times 10^6 = 64989 \, Ast - 7.4917 \, Ast^2$$

$$Ast = 1320 \, MM^2$$

Use 20mm ϕ

$$\text{spacing} = \frac{1000 \times \pi/4 \times 20^2}{1320}$$

$= 230\text{mm}$

provide 20mm ϕ @ 230 MM spacing c/c.

Middle strip

+ve $M_u = 6.92$

$$6.92 \times 10^6 = 0.87 \times 415 \times A_{st} \times 130 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 130} \right]$$

$$6.92 \times 10^6 = 46936.5\, A_{st} \left[1 - 1.59615 \times 10^{-4} \right] A_{st}$$

$$6.92 \times 10^6 = 46936.5 A_{st} - 7.49178 A_{st}^2$$

$A_{st} = 151\ \text{mm}^2$

Provide min Ast as $A_{st} < 240\text{MM}^2$

Use 10mm ϕ

$$\text{Spacing} = \frac{1000 \times \pi/4 \times 10^2}{240}$$

$= 300\text{MM}$

provide 10mm ϕ @ 300MM spacing as c/c

-ve

$M_u = 12.84$

$12.84 \times 10^6 = 46936.5\, A_{st} - 7.49178\, A_{st}^2$

$A_{st} = 286 mm^2$

Use 10mm ϕ

Spacing $= \frac{1000 \times \pi/4 \times 10^2}{286}$

$= 300 MM$

Provide 286 MM ϕ @ 300MM spacing as c/c

$A_{st(min)} = 0.12\%\, bD$

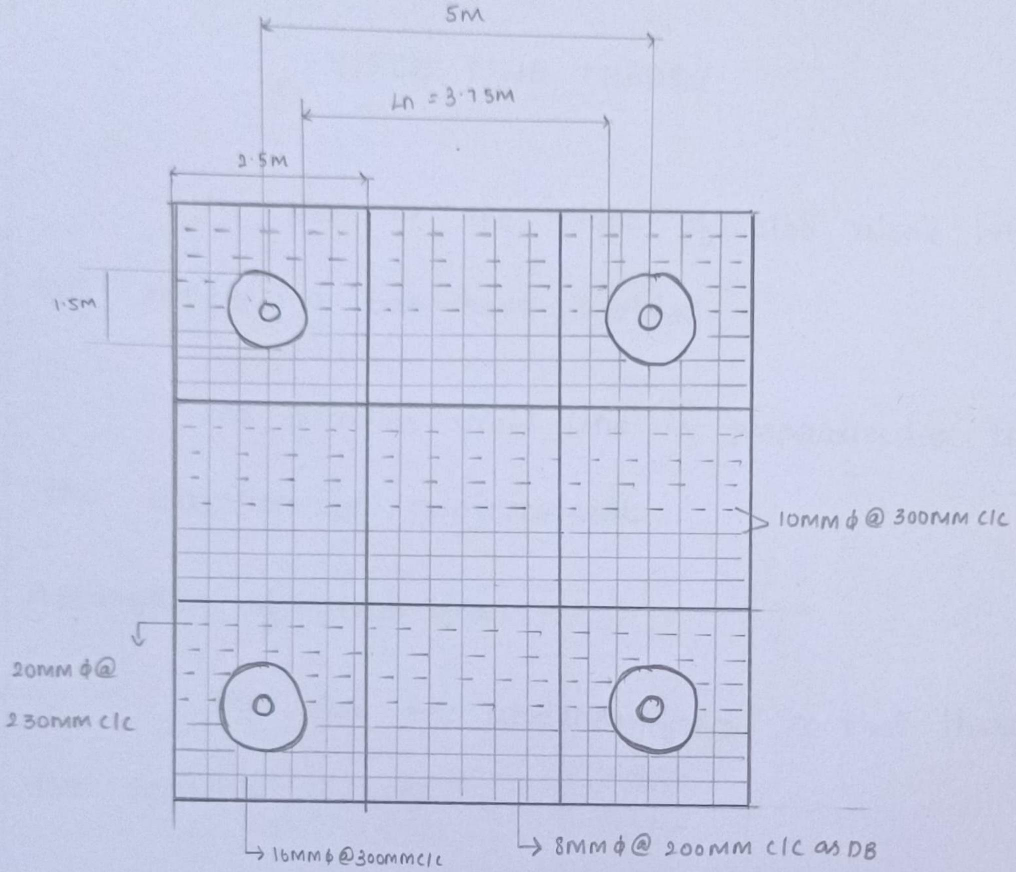
$= \frac{0.12}{100} \times 1000 \times 200$

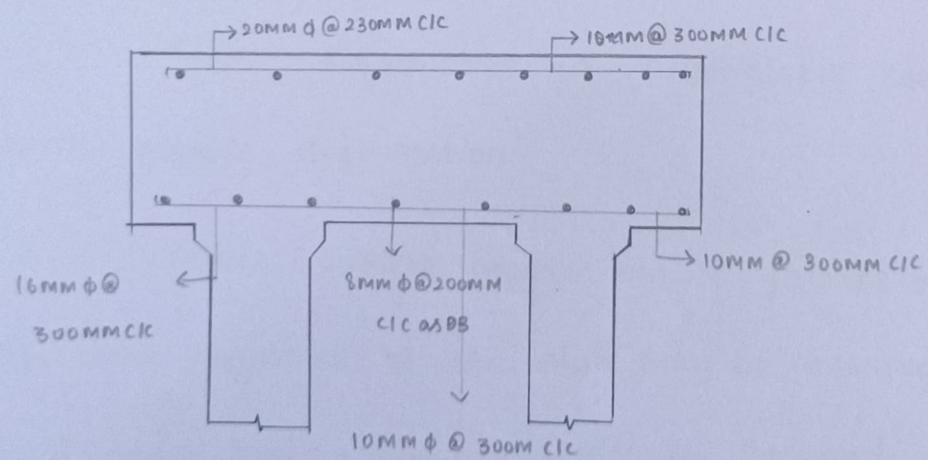
$= 240 mm^2$

Spacing provide 8mm ϕ

$= \frac{1000 \times \pi/4 \times 8^2}{240}$

$= 200 mm^2$


5M
Ln = 3.75M
2.5M
1.5M
10MM ϕ @ 300MM c/c
20MM ϕ@
230MM c/c
16MMϕ@300MMc/c
8MM ϕ@ 200MM c/c as DB


20MM ϕ @ 230MM C/C
10MM@ 300MM C/C
16MM ϕ@
300MMC/C
8MM ϕ@200MM
C/C as DB
10MM @ 300MM C/C
10MM ϕ @ 300M C/C

UNIT - 4

YIELD LINE THEORY

A Line in the plane of slab alone with all reinforcing bar have yielded.

The direction yield Line is perpendicular to the direction of reinforcement.

Assumption of yield Line:

The slab is underreinforced so that there is no tension failure.

Yield Line are straight Lines

Elastic deformation is negeligiable compared with plastic deformation.

After collape mechanism is formed each of the segment of the slab may be treated as rigid body and entire rotation is assumed to take place along the yield Line.

Characteristic features of yield line

Yield Lines are straight Line

Yield Line terminate at the boundary of the slab or intersection of other yield line.

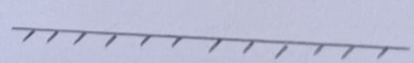
Yield Line att as access of rotation

Access of rotation are along the Line of support and process of column

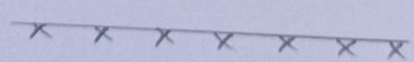
Each segment will be having access rotation on along its boundary.

Yield Line symbol:

1) simply supported

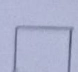


2) Fixed supported

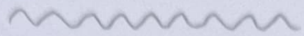


3) Free

4) column



5) positive yield Line

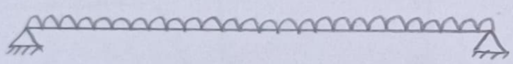


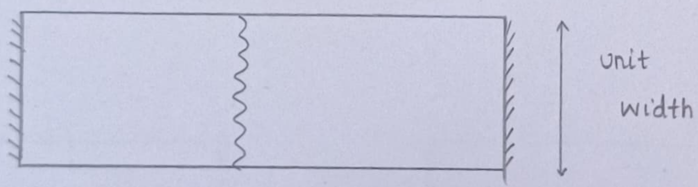
6) Negative yield Line

7) Axis of Rotation

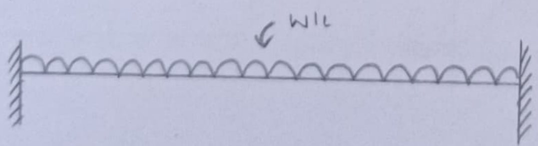
one way slab

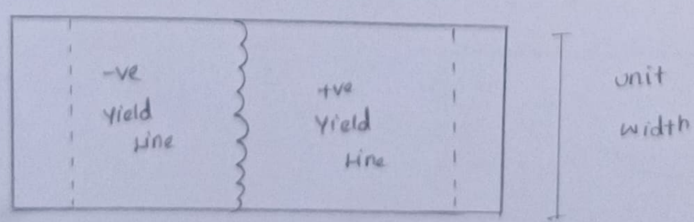
For simply supported:





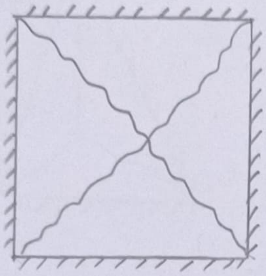
For fixed:

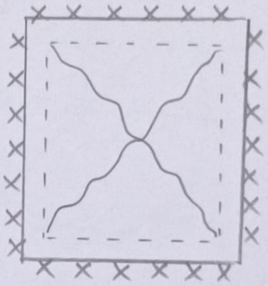




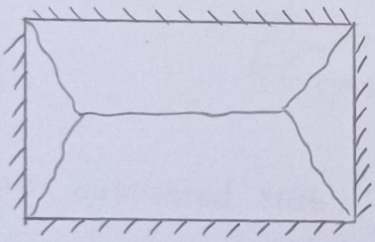
TWO way slab yield Line pattern

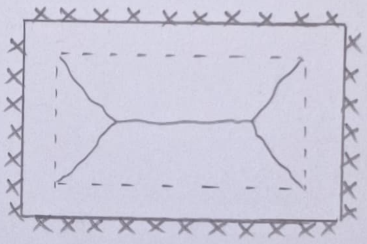
i) square slab:

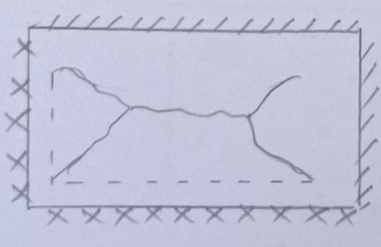


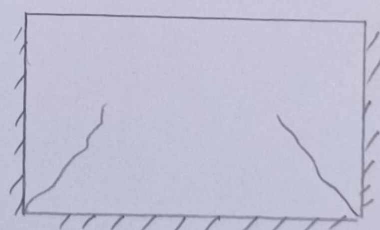


ii) Rectangular slabs:

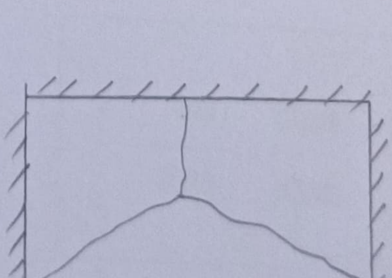




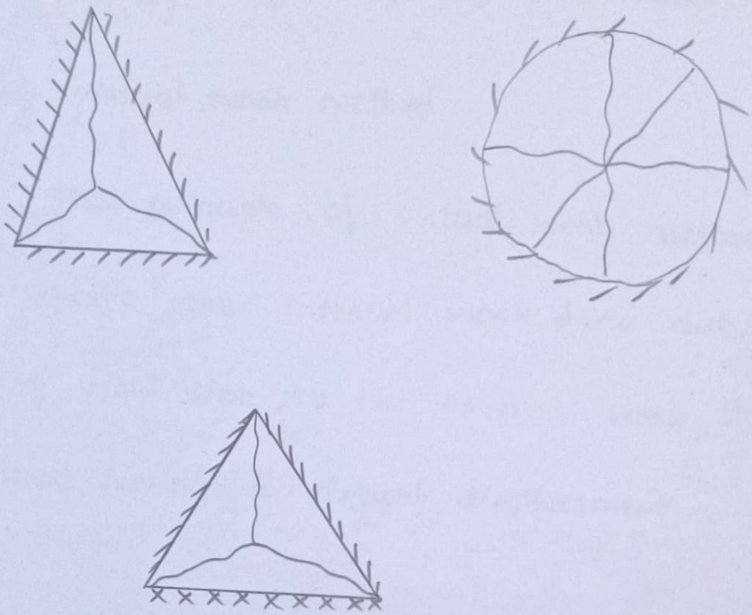




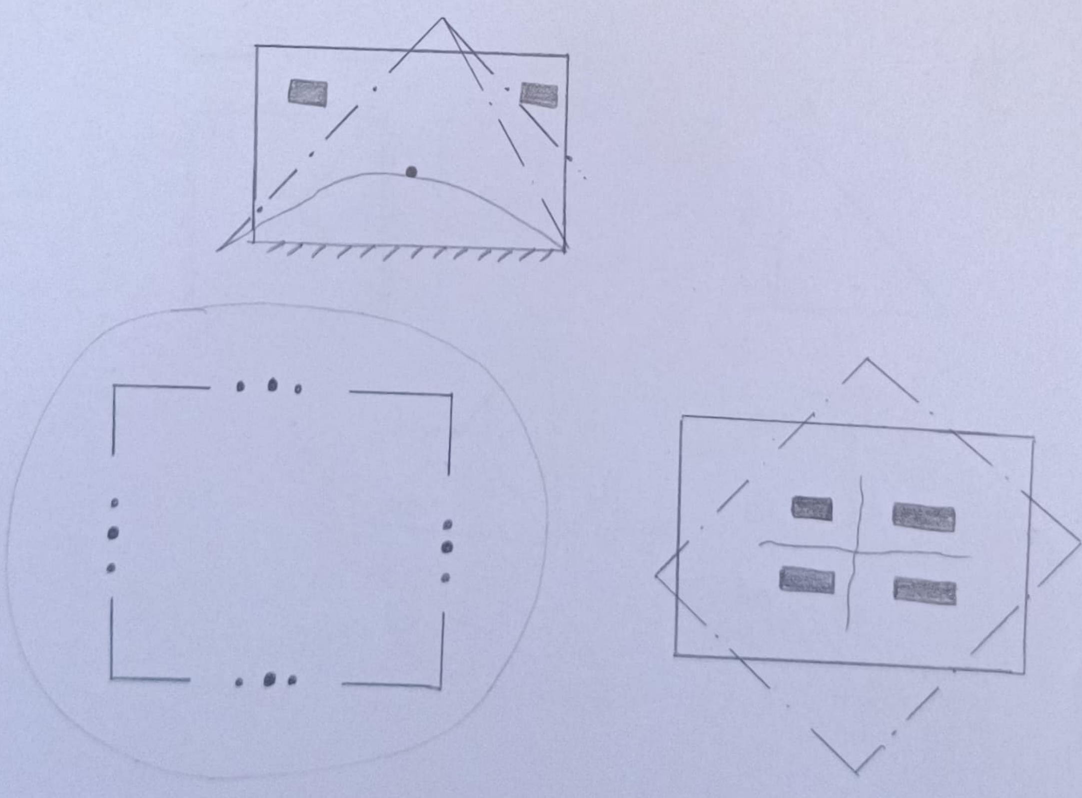
or



iii) other shapes:



Column supported slab:



Isotrophically reinforced square slab is simply supported & supporting uniformly distributed Load ;

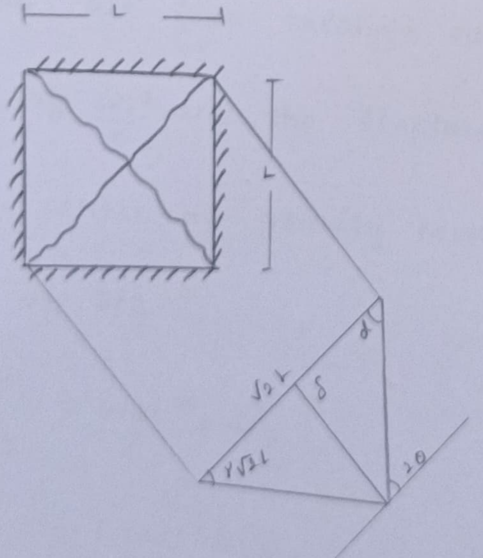
principle of virtual work method :

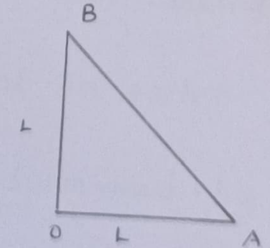
The principle of virtual work method is to equate the internal work done due to rotation of yield Line to the external work done due to Load having a virtual displacement.

External workdone = Force x displacement (Load)

Internal workdone = Moment x Rotation

Diagram :





$BA^2 = OB^2 + OA^2$

$BA^2 = L^2 + L^2$

$BA = \sqrt{2}$

External workdone $= W \times \delta$

Internal workdone $= M \times \theta$

Internal workdone:

= Ultimate moment / unit length of yield Line

$= M_u \times L$

$M = M_u \times \sqrt{2} L$

$\theta = 2\theta$

$I.W.D = M_u \times \sqrt{2L} \times 2 \left(\frac{\delta / \sqrt{2} L}{2} \right)$

$I.W.D = 4 M_u \delta$

$= 8 M_u \delta$

There are four triangle element each carring a Load of $\frac{WL^2}{4}$ and the displacement in each triangle centre of gravity moves downward by a distance of $\delta/3$.

$= \frac{W_u l^2}{4} \times \delta/3$

$= \left(\frac{W_u l^2 \delta}{l^2} \right) \times 4$

$E.W.D = \frac{W_u l^2 \delta}{3}$

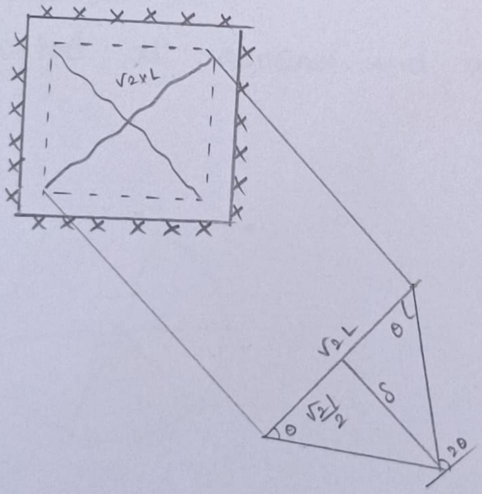
By virtual workdone

Σ External workdone $= \Sigma$ Internal workdone

$$\frac{w_u l^2 \delta}{3} = 8 M_u \delta$$

$$M_u = \frac{w_u l^2}{24}$$

Derive Isotropically reinforced fixed square slab subjected to UDL over entire surface.



Internal workdone = External workdone

In I.W.D $\Rightarrow M = M_u \times \sqrt{2} L$

$\theta = 2\theta$

IWD $= 8 M_u \delta$ (+ve yield line)

$= 8 M_u \delta$ (-ve yield line)

External workdone $= \frac{W_u l^2 \delta}{3}$

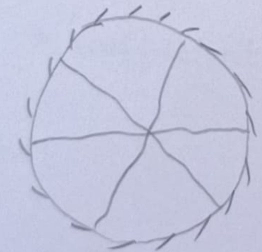
I.W.D $= 16 M_u \delta$

EWD $=$ IWD

$16 M_u \delta = \frac{W_u l^2 \delta}{3}$

$M_u = \frac{W_u l^2}{48}$

For isotropically reinforced circular slab simply supported all around and uniformly loaded all around



$M_u = \frac{W_u r^2}{6}$

where r is radius of circular slab

Derive orthotropically reinforced simply supported Rectangular slab

Let L be the Longer span and αL be the Shorter span

Where Mu is a yield moment across x-axis and $\mu \times Mu$ is a yield moment across y-direction

$$Mu = \frac{Wu \alpha^2 L^2}{24} \left[\sqrt{3+\alpha^2 \mu} - \alpha \sqrt{\mu} \right]^2$$

Where

μ → coefficient of orthotrophy.

Special cases:

For simply supported equalateral triangular slab

$$Mu = \frac{WL^2}{72}$$

For Isotrophical reinforced rectangular Slab.

Where, $\mu = 1$

$$M_u = \frac{W_u \kappa^2 L^2}{24} \left[\sqrt{3+\alpha^2} - \alpha \right]^2$$

Design a simply supported square slab of side 3.6M to carry a service Load of 4 kN/m². Use M20 grade and Fe415 steel.

Given :

$f_y = 415 \text{ N/mm}^2$

$f_{ck} = 24 \text{ N/mm}^2$

$L = 3.6M$

1) Thickness of span :

[Refer IS456 : 2000 of pg : 39]

$\frac{L}{D} = (35 \times 0.8)$

$\frac{3600}{D} = 28$

$D = 128 \text{ MM}$

$\simeq 130 \text{MM}$

$d' = 15 \text{MM}$

$d = 115 \text{MM}$

3) Load

Self weight of slab $= 25 \times 1 \times 0.13$

$= 3.25\ kN/m^2$

Live Load $= 4\ kN/m^2$

Finishing Load $= 1\ kN/m^2$

W $= 8.25\ kN/m^2$

Design Load $W_u = 1.5 \times 8.25$

$W_u = 12.375\ kN/m^2$

4) Moment calculation:

$$M_u = \frac{W_u l^2}{24}$$

$$= \frac{12.375 \times 3.6^2}{24}$$

$$M_u = 6.68\ kNm.$$

5) Check for depth:

$$d = \sqrt{\frac{M_u}{0.138\, f_{ck}\, b}}$$

$$= \sqrt{\frac{6.68 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d = 49.19\ mm < 115\ mm$

Hence safe.

b) Ast calculation:

$$Mu = 0.87\, fy\, Ast\, d \left[1 - \frac{fy\, Ast}{fck\, bd}\right]$$

$$= 0.87 \times 415 \times Ast \times 115 \left[1 - \frac{115\, Ast}{20 \times 1000 \times 115}\right]$$

$$Ast = 165.8 mm^2$$

Spacing use 10mm ϕ

$= 475mm$

$\simeq 300mm$

As the ast value is small so we check the ast min

$$Ast_{min} = 0.12\%.\, bD$$

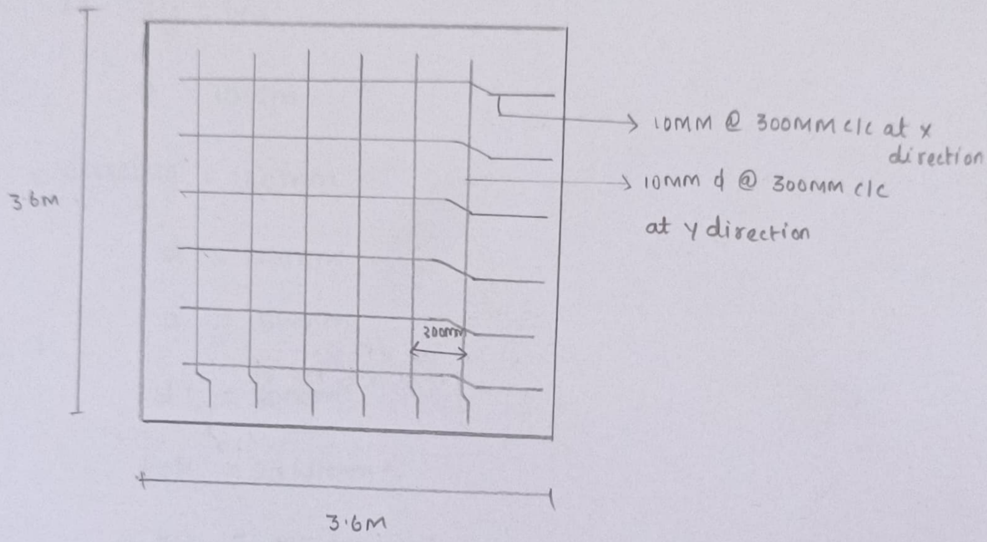
$$= \frac{0.12}{100} \times 1000 \times 130$$

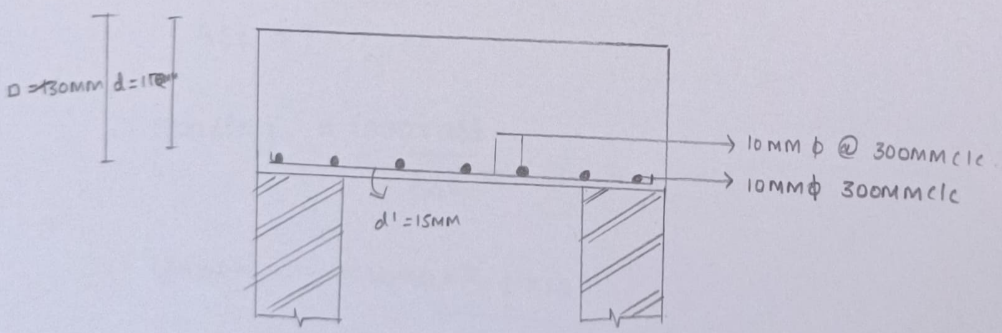
$$Ast_{min} = 156 mm^2$$

$(Ast_{min}) < (Ast_{main})$

Provide 10mmϕ @ 300mm c/c at x & y direction

as Main reinforcement.





2) A simply supported square slab of side 6m by 6m is Reinforced with 10mm dia rod (Fe415 Steel] at a spacing of 180MM in both direction. The average effective depth is nearly taken as 120MM, overall depth as 150MM. Determine a permissible service load if Grade of concrete is M20 and assume it is simply supported on all the edges.

Given :

$Lx = Ly = 6M$

$\phi = 10mm$

Spacing $= 180mm$

$d = 120mm$

$D = 150mm$

$d' = 30mm$

$f_{ck} = 20 N/mm^2$

$f_y = 415 N/mm^2$

$Ast = ?$

$$\text{Spacing} = \frac{1000 \times ast}{Ast}$$

$$180mm = \frac{1000 \times \pi/4 \times 10^2}{Ast}$$

$$Ast = 436.3mm^2$$

$$Mu = 0.87 \, f_y \, Ast \, d \left[1 - \frac{f_y \, Ast}{f_{ck} \, bd}\right]$$

$$Mu = 0.87 \times 415 \times 436.5 \times 120 \left[1 - \frac{415 \times 436.3}{20 \times 1000 \times 120}\right]$$

$$Mu = 17.47 \times 10^6 Nmm$$

$$Mu = 17.47 kNm$$

$$Mu = \frac{Wul^2}{24}$$

$$17.47 = \frac{Wu \times 6^2}{24}$$

$Wu = 11.64\ kN/m$

TOTAL Load $\Rightarrow W = \frac{11.64}{1.5}$

$T.L = 7.76\ kN/m$

Died Load + Live Load + Finishing Load = Total Load

$(25 \times 1 \times 0.15) + L.L + 1 = 7.76$

Live Load $= 3.01\ kN/m$

3) Design a simply supported circular slab of 3.5M diameter to cover an underground sump at supported at periphary to along 200mm thick. Assume Live Load is 4 kN/m². Finishing Load is 1kN/m². Use M20 & Fe 415 bars

Given:

Dia = 3.5M

L.L = 4 kN/m²

F.L = 1kN/m²

D = 200mm

$$M_u = \frac{W_u r^2}{6}$$

$$r = D/2$$

$$r = 1.75m$$

Step 2:

$D = 200mm$

$d' = 20mm$

$d = 180mm$

Step 3: Load calculation:

Dead Load $= 25 \times b \times D$

$= 25 \times 1 \times 0.2$

$= 5 KN/m$

L.L $= 4 KN/m^2$

F.L $= 1 KN/m^2$

$W_u = 1.5 \times 10$

$W_u = 15 KN/m^2$

Step 4: Moment calculation:

$$M_u = \frac{W_u r^2}{6}$$

$$M_u = \frac{15 \times 1.75^2}{6}$$

$M_u = 7.65 KNM$

$$Mu = 0.87 \times fy\ Ast \times d \left[1 - \frac{fy\ Ast}{fck\ bd}\right]$$

$$7.65 \times 10^6 = 0.87 \times 415 \times Ast \times 180 \left[1 - \frac{415\ Ast}{20 \times 1000 \times 180}\right]$$

$$Ast_{(main)} = 118 mm^2$$

Check for $Ast_{(min)} = 0.12\% . bD$

$$= \frac{0.12}{100} \times 1000 \times 200$$

$$Ast_{(min)} = 240 mm^2$$

$$Ast_{(min)} > Ast_{(prov)}$$

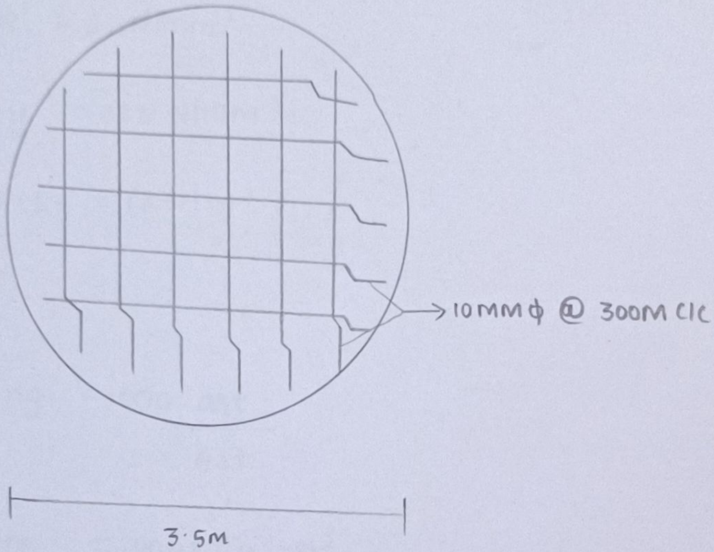
So we provide the min bar as a (Ast) main

use 10mm ϕ

$$Spacing = \frac{1000 \times \pi/4 \times 10^2}{240}$$

$$= 327 mm \nless 300 mm$$

provide 10mm ϕ @ 300 mm c/c as a pheripheral reinforcement.



4) A simply supported circular slab of Radius 2·8m is reinforcement with 10mm dia bar at 180mm c/c spacing in mutually perpendicular direction. The average effective depth 100mm & over all depth 125mm. use M20 and Fe415 HYSD bars. Determine how much it can carry a service Load. Take finishes Load as $1 kN/m^2$.

Given

$r = 2{\cdot}8m$

$\phi = 10mm$

Spacing = 180 mm c/c

$d = 100mm$

$D = 125 mm$

$f_{ck} = 20 N/mm^2$

$f_y = 415 N/mm^2$

$F.L = 1 kN/m^2$

Step 2:

$$Spacing = \frac{1000\ ast}{Ast}$$

$$180mm = \frac{1000 \times \pi/4 \times 10^2}{Ast}$$

$$Ast = 436.3 mm^2.$$

$$Mu = 0.87 \times f_y\ Ast\ d \left[1 - \frac{f_y\ Ast}{f_{ck}\ bd}\right]$$

$$Mu = 0.87 \times 415 \times 436.3 \times 100 \left[1 - \frac{415 \times 436.3}{20 \times 1000 \times 100}\right]$$

$$Mu = 14.3 \times 10^6 Nmm$$

$$Mu = 14.3 kNm.$$

$$Mu = \frac{Wu\gamma^2}{6}$$

$$14.3 = \frac{Wu \times 2.8^2}{6}$$

$W_u = 10.94$ kN/m

Total load $= \frac{10.94}{1.5}$

$= 7.29$ kN/m

Live Load + Dead Load + Finishing Load = 7.29

Live Load = 3.16 kN/m²

5) Design a rectangular slab of size 4m x 6m which is simply supported along the edges and carring a service load of 4 kN/m². Assume coefficient of orthotrophy $\mu = 0.75$. Use M20 grade and Fe415 steel. Design the slab may be restricted to bending only.

Given data :

$L = 6m$

$\alpha L = 4m$

Live Load = 4 kN/m²

$\mu = 0.75$

$f_{ck} = 20$ N/mm²

$f_y = 415$ N/mm²

$\alpha = 0.66$

$\frac{L_y}{L_x} = \frac{4}{6} = 0.67 < 2$

∴ Two way slab

Step 2 :

$$\frac{L(short)}{D} = (35 \times 0.8)$$

$$\frac{4000}{D} = 28$$

$D = 142mm$

$\simeq 150MM$

$d' = 15mm$

$d = 135mm$

Step 3 : Load

D.L $= 25 \times 1 \times 0.15$

$= 3.75 \ KN/m^2$

L.L $= 4 \ KN/m^2$

F.L $= 1 KN/m^2$

T.L $= 8.75 \ KN/m^2$

$wu = 1.5 \times 8.75$

$= 13.125 \ KN/m^2$

Step 4 : Moment calculation :

$$Mu = \frac{wu \, \alpha^2 L^2}{24} \left[\sqrt{3 + \alpha^2 \mu} - \alpha \sqrt{\mu} \right]^2$$

$$= \frac{13.125 \times 0.67^2 \times 6^2}{24} \left[\sqrt{3 + (0.67^2 \times 0.75)} - 0.67\sqrt{0.75} \right]^2$$

$Mu = 13.77 kNM$

Moment [Longer span]

$\mu Mu = 0.75 \times 13.77$

$\mu Mu = 10.32 kNM$

Step 5: Ast for shorter span:

$$13.77 \times 10^6 = 0.87 \times 415\, Ast \times 135 \left[1 - \frac{415\, Ast}{20 \times 1000 \times 135} \right]$$

$Ast = 295 mm^2$

Use 10mm ϕ

$$\text{Spacing} = \frac{1000 \times \pi/4 \times 10^2}{295}$$

$= 260 mm$

provide 260 mm c/c for 10mm ϕ.

Step 6: Ast for longer span:

$$10.32 \times 10^6 = 0.87 \times 415 \times Ast \times 135 \left[1 - \frac{415\, Ast}{20 \times 1000 \times 135} \right]$$

$Ast = 219 mm^2$

Use 10mm ϕ

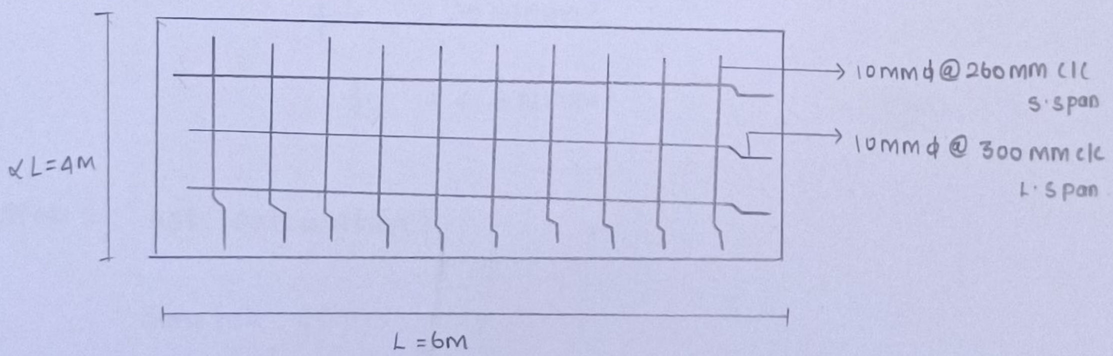
$$\text{spacing} = \frac{1000 \times \pi/4 \times 10^2}{219}$$

$$= 300mm$$

Provide 10mm ϕ bar @ 300mm c/c

$$Ast_{(min)} = \frac{0.12}{100} \times 1000 \times 150$$

$$= 180mm^2$$



6) A Rectangular slab of 5x6m is simply supported at all edges and reinforced with 10mm ϕ bar at 200mm c/c in shorter direction and 10mm ϕ bar at 225 mm c/c in long direction. Determine the super imposed Load carrying the slab. The effective depth of slab is 120mm and overall depth of slab is 150mm. Use M20 grade & Fe415 bar.

Given data :

$L = 6m$

$\alpha L = 5m$

Shorter direction $= 10mm\ \phi$ @ 200 mm c/c

Longer direction $= 10mm\ \phi$ @ 225 mm c/c

$d = 120mm$

$D = 150mm$

$f_{ck} = 20 N/mm^2$

$f_y = 415 N/mm^2$

Step 2: Ast calculation :

Shorter span :

$$A_{st} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{200}$$

$$= 392.6 mm^2$$

Longer span :

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

$$= 349.1 mm^2$$

Step : 3 moment calculation

Shorter span :

$$Mu = 0.87\, fy\, Ast\, d \left[1 - \frac{fy\, Ast}{fck\, bd} \right]$$

$$= 0.87 \times 415 \times 392.6 \times 120 \left[1 - \frac{415 \times 392.6}{20 \times 1000 \times 120} \right]$$

$$Mu = 15.85\, KNm$$

Longer span :

$$\mu Mu = 0.87\, fy\, Ast\, d \left[1 - \frac{fy\, Ast}{fck\, bd} \right]$$

$$= 0.87 \times 415 \times 349.1 \times 120 \left[1 - \frac{415 \times 349.1}{20 \times 1000 \times 120} \right]$$

$$\mu Mu = 14.21\, KNm$$

$$\mu = \frac{14.21}{15.85}$$

$$\mu = 0.96$$

$$Mu = \frac{Wu\, \alpha^2 L^2}{24} \left[\sqrt{3 + \alpha^2 \mu} - \alpha \sqrt{\mu} \right]^2$$

$$15.85 = \frac{Wu \times 0.83^2 \times 6^2}{24}\left[\sqrt{3+0.83^2 \times 0.896} - 0.836\sqrt{0.896}\right]^2$$

$$15.85 = 1.033\,Wu\left[1.9019 - 0.7856\right]^2$$

$$15.85 = 1.033\,Wu \times 1.246$$

$$Wu = \frac{15.85}{1.287}$$

$$Wu = 12.31\ kN/m$$

$$W = \frac{12.31}{1.5}$$

$$W = 8.2\ kN/m^2$$

Total Load = Live Load + Dead Load + Finishing Load

$$8.2 = L.L + (25 \times 1 \times 0.15) + 1$$

$$\text{Live Load} = 3.45\ kN/m^2$$

UNIT - 5

BRICK MASONRY

Size of Standard brick = 190x90 x 90 mm

Standard size of brick with mortor = 200mm x100mm x100mm

Size of Modular brick = 190 x 90 x40mm

Size of Modular brick with mortar = 200x 100 x 50mm

Types & classification of brick:

1. Common clay brick
2. Heavy duty brick.

1. common clay brick:

These type of bricks are used for different type of building work.

1. class I brick:

Uniform colour, throughly uniformly burn brick.

Plane rectangular faces with parallel size & Sharp Straight

2. class II brick:

They shall be uniform colour, may be over burn.

The bricks may distorted & have rough round edge.

3. class III brick.

This bricks may slitely under & over burnt and it is disorted & round edges.

2. Heavy Duty brick:

It is also called as engineering brick (used for roads & bridges)

1. class I brick

It is also called as class A brick.

It shall emit clear ringing sound.

2. class II brick (class B)

It shall have slight disortion and have round edges.

Strength of common brick:

The common building bricks have a Minimum compressive strength or crushing strength is 35 kg/cm^2

Grade A brick - 105 kg/cm^2
(1st class brick)

Grade B brick - not less than 70 kg/cm^2
(2nd class brick)

Grade C brick - Not less than 35 kg/cm^2
(Low class)

Grade AA brick - More than 140 kg/cm^2

Masonry wall.

A wall is a vertical member with width is greater than 4 times its wall thickness.

1. This walls are classified into load bearing wall & non-load bearing wall.

[Refer IS 1905-1987 pg-3 cl 2.14]

The following of the types are load bearing walls.

Solid wall with pier

Solid wall

Cavity wall

Faced wall

Non Load bearing walls:

It has 4 types.

panel wall

partition wall

curtain wall

Free standing walls.

2. Effective height of wall.

[Refer pg-11- cl 4·3] & also [refer Table-4]

3. Effective Length of wall

[Refer page 12 - cl · 4·4] [Table 5]

4. Effective thickness of wall:

[Refer pg-13 cl - 4·5]

5. Slenderness ratio of wall:

[Refer pg-14 cl - 4·6]

$$\text{Slenderness ratio} = \frac{\text{effective height}}{\text{effective thickness}}$$

(or)

$$\text{Slenderness ratio} = \frac{\text{effective length}}{\text{effective thickness}}$$

Which ever is less is known as slenderness ratio.

6. permissible compressive stress on wall:

Refer pg-15 cl 5·4·1

$$f_c = (f_b) \times (k_s) \times (k_a) \times (k_p) \times (k_l)$$

Where,

f_b → basic compressive stress table 8

k_s → stress reduction factor table 9

k_a → Area reduction factor

$k_a = 0.7 + 1.5A$

k_p → shape modefication factor table 10

k_l → Load factor

$[k_l = 1]$

7. Load calculation

$$f_c = \frac{P}{A}$$

8. Bending stress:

$$\frac{P}{A} = M/Z$$

$[Z = I/y]$, $[M = PE]$

Determine the allowable Load (or) Allowable axial Load on the column of size 300x600MM and constructed in a first class brick with cement Mortar is 1:6 using Standard size of brick. The Height of the pier b/w the footing and top of the Slab is 5.1m. The Strength of the Masonry unit May be Assumed as 10.5 N/mm².

Given:

Size = 300x 600mm

cement Mortar Ratio = 1:6

Height of the pier (h) = 5.1M

unit weight of brick

(compr or crushing strength) = 10.5

Standard size of brick = 190x90 x 90mm

Solution:

Refer IS1905

$f_c = f_b \times k_s \times k_a \times k_p \times k_L$

f_b ⇒ Refer IS1905 Pg.No: 16 Tb-8

Grade of Mortar = M_2 type

Refer table 8,

10	10.5	12.5
0.81	x	0.94

on Interpolation,

$$\frac{0.94-0.81}{12.5-10} = \frac{x-0.81}{10.5-10}$$

$$(fb)\ x = 0.836$$

There is no condition given

Effective height $= 1 \times 5.1$ $(1 \times H)$

$= 5.1\text{m}$

Effective Thickness of Wall $= 300\text{mm}$

$= 0.3\text{m}$

Slenderness Ratio $= \frac{5.1}{0.3} = 17$

Refer Table 9,

16	17	18
0.73	x	0.67

on Interpolation

$x = 0.7$

$ks = 0.7$

Refer pg : 16 (5.4.12)

$$ka = 0.7 + 1.5 \text{ Area}$$

$$= 0.7 + 1.5(0.3 \times 0.6)$$

$$ka = 0.97$$

kp pg. no : 16 (5.4.18) Refer table 10

$$kp = 1.09$$

kL is alway 1

$$fc = fb \times ks \times ka \times kp \times kL$$

$$= 0.836 \times 0.7 \times 0.97 \times 1.09 \times 1$$

$$fc = 0.618 \text{ N/mm}^2$$

Find,

$$fc = \frac{P}{A} + \frac{Pe}{z} \quad e(0)$$

$$fc = \frac{P}{A}$$

$$0.618 = P/(1300 \times 600)$$

$$P = 111.24 \times 10^3 N$$

$$P = 111.24 \text{ kN}$$

Design a brick coloumn Masonry wall of room Size 3m x 7m. Reinforced concrete thickness as 10cm and Live Load on roof slab as 1·5 KN/m². The height of the wall is 3m and parapet wall thickness is 2·3 cm and 75 cm height the crushing Strength of brick is 3·5 N/mm². The cement Ratio 1:6 are used and Nominal class brick are used.

Room Size = 3m x 7m

concrete thickness = 10cm = 100mm

Live Load = 1·5 KN/m²
(roof slab)

Height of the wall = 3m

Parapet wall thickness = 23cm = 230mm

parapet wall height = 75cm = 750mm

Ratio = 1:6

Strength = 3·5 N/mm²

Soln :

Assume density of the brick = 18 KN/m 3

Take wall thickness are 300mm.

Load calculation:

consider 1m length of wall

Self weight of parapet well = 18 x 0.75 x 0.23

= 3.105 KN/m

Self wt of roof slab = 25 x 1 x 0.1

= 2.5 KN/m

self wt of wall = 18 x 3 x 0.3

= 16.2 kN/m

Live Load = 1.5 KN/m

T. Load = 23.305 KN/m 2

Stress = P/A = $\frac{23.3 \times 10^3}{1000 \times 300}$

= 0.07 N/mm 2

permissible stress

$f_c = f_b \times k_s \times k_a \times k_c \times k_p$

f_b → Refer PS:16 Table 8 & PS: 6 Table 1

$f_b = 0.35\ N/mm^2$

k_s → Pg. no: 16 Tb 4

Slenderness Ratio ⇒ effective height $= 1 \times H$

$= 1 \times 3$

$= 3m$

Effective Thickness $= 0.3m$

Slenderness Ratio $= 3/0.3 = 10$

Table 9,

$k_s = 0.89$

k_a = Refer PS:16 cl N: 5.4.12

$k_a = 0.7 + 1.5A$

$= 0.7 + 1.5\ (1 \times 0.3)$

$k_a = 1.15$

$K_P \rightarrow$ Table 10

$K_P = 1$

$K_L = 1$

$\left(\because e = 0, K_L = 1 \right.$

$\left. e = \text{value}, K_L = 1.2 \right)$

$f_c = 0.35 \times 0.89 \times 1.15 \times 1 \times 1$

$= 0.358 \ N/mm^2$

$f_c >$ stress

$0.358 > 0.07$

Hence safe

Thus the choise of wall thickness 300mm is satisfied our condition.