Design of RCC and Steel Structures
(18CV72)

Module 1 : RCC structures

* Footing: Design of Rectangular slab type combined footing

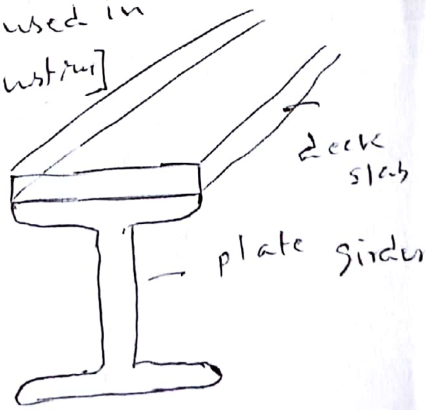
* Retaining wall :
  - Cantilever retaining wall [≤ 6m]
  - Counterfort retaining wall [> 6m]

* Water tank :
  - Circular water tank
  - Rectangular water tank

* Portal Frame : [Footing + Column + Beam]
  - Fixed Support
  - Hinged Support

Module 2 : Steel structures

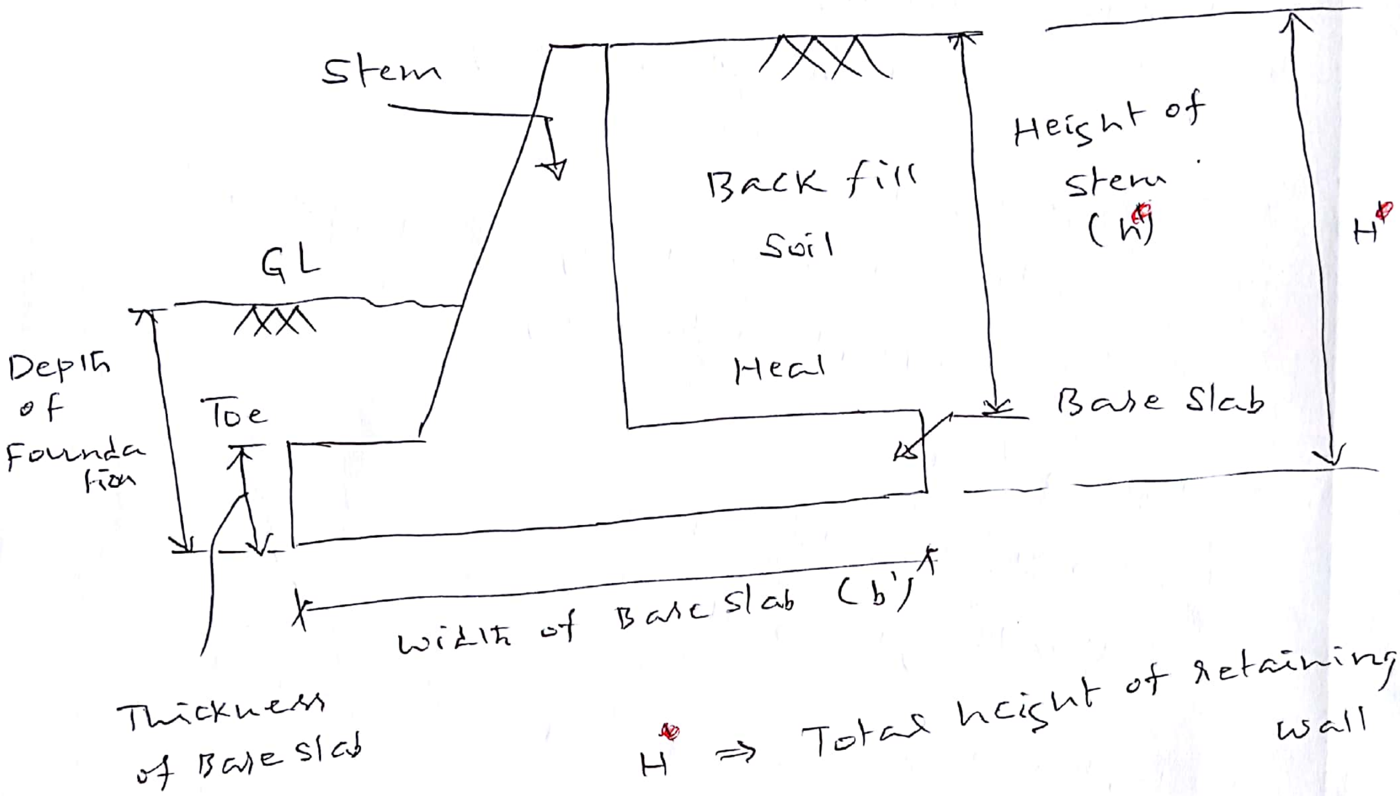
Roof truss [Industrial buildings]
plate girder [Strong I-section] above which deck slab is placed
Gantry girder [Normally used in granite Industries]



Module 1 : RCC structures

Retaining wall

Cantilever Retaining wall [≤ 6m]



H ⇒ Total height of retaining wall

Cantilever Retaining wall

1) Design a cantilever retaining wall to retain the levelled earth embankment 5 m height above ground level. The unit weight of earth is 16 kN/m² & the angle of repose is 30°. The SBC of soil is 145 kN/m². The coefficient of friction between soil and concrete is 0.55. Use M20 grade concrete & Fe415 grade steel. Also design the shear key. Draw the following reinforcement details.

a) The section of retaining wall showing the details of steel in stem, base slab & shear key

b) Longitudinal section for 2 m showing the reinforcement of stem, base slab & shear key

c) plan of base slab through centre showing the reinforcement.

i) Data: unit weight of soil, $\gamma = 16$ kN/m³

SBC = 145 kN/m²

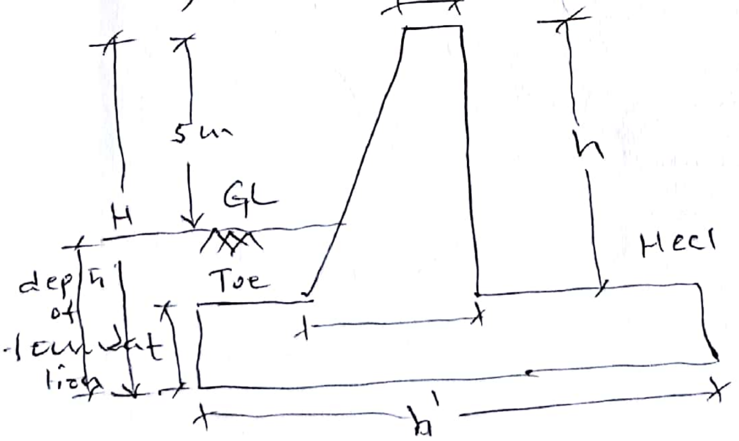
Angle of repose, $\phi = 30°$

$f_{ck} = 20$ N/mm²

f_e or f_y = 415 N/mm²

Dimensions
↓
Load
↓
Bending moment, M_u
↓
Area of steel, A_{st}
↓
No. of bars / Spacing of bars

ii) Dimensions of Retaining wall



Top width of stem = 150 mm

Depth of foundation $= \frac{SBC}{r}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$ or 1m min

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

$= 1.006$ m or 1m

Take depth of foundation as 1m

Total height of retaing wall = 1m + 5m

= 6m

width of base slab, $b^1 = 0.6H$

$= 0.6(6)$

$\boxed{b^1 = 3.6\ m}$

Thickness of base slab $= \frac{H}{12} = \frac{6}{12} = 0.5$ m

Height of stem, h = H − Thickness of base slab

$= 6 - 0.5$

$\boxed{h = 5.5m}$

width of Toe $= \frac{b^1}{3}$ or 1m min

$= \frac{3.6}{3}$ or 1m

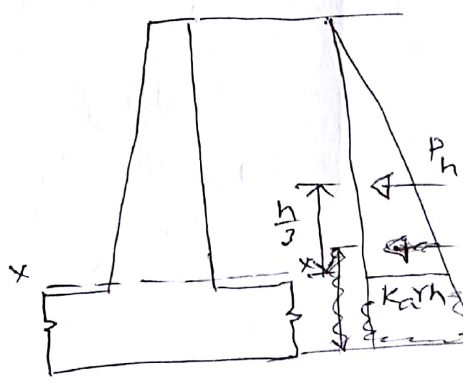
$= 1.2$ m

Heel projection :

i) Bottom stem Thickness

Coefficient of active earth pressure

$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = 0.33$



$P_h = \frac{1}{2} \times K_a r h \cdot h$

$\therefore P_h = \frac{1}{2} \times 0.33 \times 16 \times 5.5 \times 5.5$

$= 79.86$ KN

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$M_{xx} = P_h \cdot \frac{h}{3} = 79.86\ kN \times \frac{5.5}{3} = 146.41\ kNm$

$M_u = 1.5\ M_{xx} = 1.5 \times 146.41 = 219.62\ kNm$

$M_{umax} = 0.36\ \frac{x_{umax}}{d}\left[1 - 0.42\ \frac{x_{umax}}{d}\right] f_{ck}\ bd^2$

$219.62 \times 10^6 = 0.36 \times 0.48\left[1 - 0.42 \times 0.48\right] 20 \times 1000 \times d^2$

$d = 282.12\ mm$

using an effective cover of 60 mm

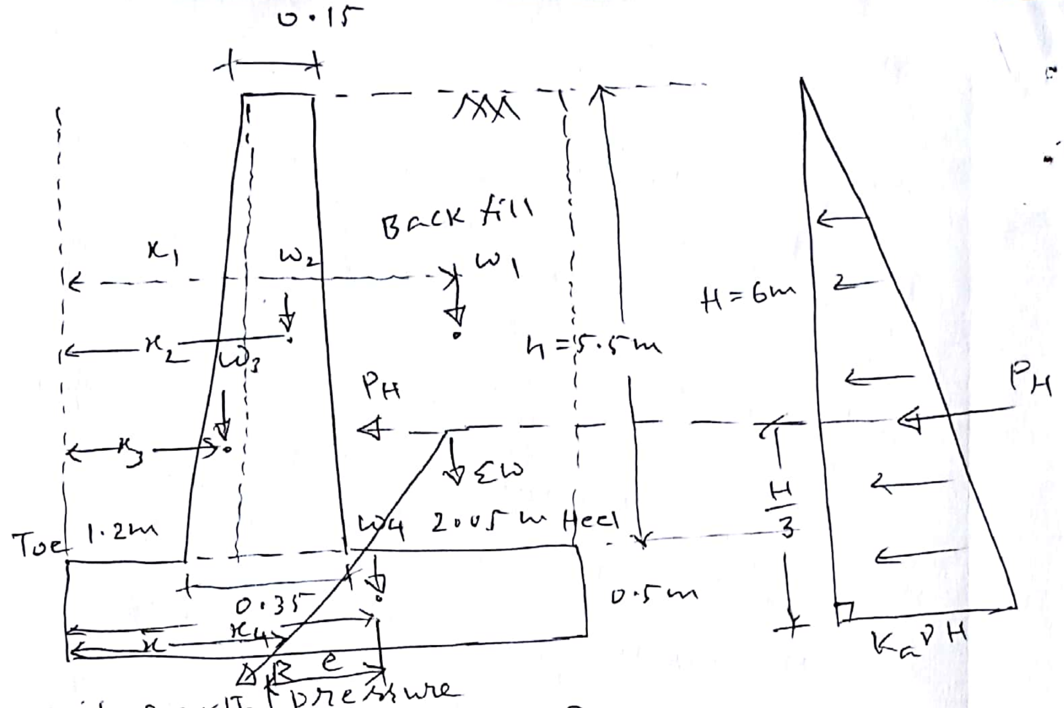
$D = 282.12 + 60 = 342.12 \simeq 350\ mm$

Heel projection $= 3.6 - 1.2 - 0.35 = 2.05\ m$

ii) Check for Stability of Retaining wall

a) Check for Soil earth pressure

Components	Weight (kN)	Distance upto Toe (m)	Moment (kNm)
Backfill	$w_1 = 16\ kN/m^3 \times [2.05 \times 5.5 \times 1]$ $= 180.4\ kN$	$\frac{2.05}{2} + 0.35 + 1.2$ $x_1 = 2.57\ m$	$w_1 x_1$ $= 180.4 \times 2.57$ $= 464.5\ kNm$
Stem	$w_2 = 25\ kN/m^3 \times [0.15 \times 5.5 \times 1]$ $= 20.63\ kN$	$\frac{0.15}{2} + 0.2 + 1.2$ $x_2 = 1.475\ m$	$w_2 x_2$ $= 30.43\ kNm$
	$w_3 = 25\ kN/m^3 \times \left[\frac{1}{2} \times 0.2 \times 5.5 \times 1\right]$ $= 13.75\ kN$	$1.2 + \frac{2}{3}(0.2)$ $x_3 = 1.33\ m$	$w_3 x_3$ $= 18.33\ kNm$
Base slab	$w_4 = 25\ kN/m^3 \times [3.6 \times 0.5 \times 1]$ $= 45\ kN$	$x_4 = \frac{3.6}{2} = 1.8\ m$	$w_4 x_4$ $= 81\ kNm$
	$\Sigma W = 259.76\ kN$		ΣM_R $= 594.29\ kNm$



Check for soil earth pressure

$$\sigma_{min} / \sigma_{max} = \frac{\Sigma W}{b'}\left[1 \pm \frac{6e}{b'}\right] \quad -(1)$$

$$e = \frac{b'}{2} - \bar{x} \quad -(2)$$

where e = eccentricity of Resultant force

$=$

$\bar{x}$ = point of application of Resultant from Toe

$$\bar{x} = \frac{\Sigma M_R - M_o}{\Sigma W} \quad -(3)$$

Overturning Moment

$$M_o = P_H \times \frac{H}{3}$$

$$= \frac{1}{2} \times (0.33 \times 16 \times 6^2) \times \frac{6}{3}$$

$$= 190.08 \text{ KNm}$$

$$\bar{x} = \frac{594.29 - 190.08}{259.79}$$

$$\boxed{\bar{x} = 1.56m}$$

Substituting $\bar{x}$ in equation (2)

$$e = \left[\frac{3.6}{2} - 1.56\right] \Rightarrow \boxed{e = 0.25m}$$

$$\sigma_{min} / \sigma_{max} = \frac{259.79}{3.6}\left[1 \pm \frac{6\times0.25}{3.6}\right]$$

$$\sigma_{min} = \frac{259.79}{3.6}\left[1 - \frac{6\times0.25}{3.6}\right] = 42.09\ kN/m^2$$

$$\sigma_{max} = \frac{259.79}{3.6}\left[1 + \frac{6\times0.25}{3.6}\right] = 102.2\ kN/m^2$$

$< SBC\ [145\ kN/m^2]$

Hence safe

ii) <u>Check for overturning</u>

Factor of safety against overturning

$$= \frac{\text{Resultant Moment } (\Sigma M_R)}{\text{overturning Moment } (M_o)} \not< 1.55$$

$$= \frac{594.29}{190.08} > 1.55$$

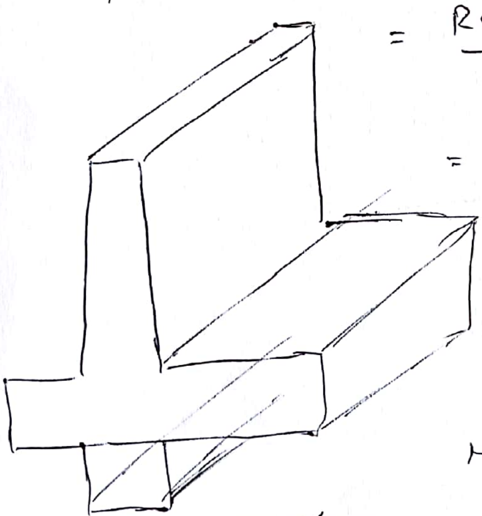
$3.12 > 1.55$

Hence Safe

c) <u>Check for sliding</u>

Factor of safety against sliding

$$= \frac{\text{Resisting Force}}{\text{Sliding Force}} = \frac{\mu \Sigma W}{P_H}$$

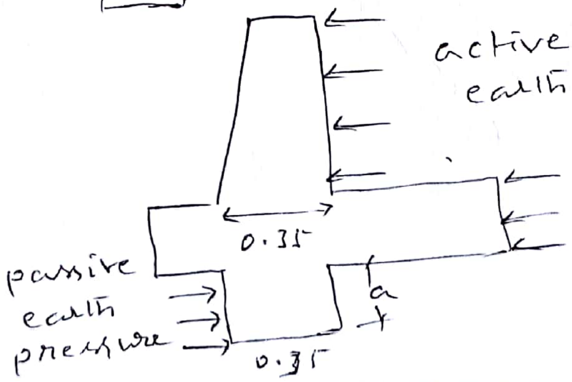


$$= \frac{0.55\times 259.78}{\underbrace{\frac{1}{2}\times(0.33\times16\times6)\,6}_{95.04}}$$

$= 1.5 < 1.55$

$\not< 1.55$

Hence Not Safe.



active earth pressure

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

Assume $a = 200$ mm

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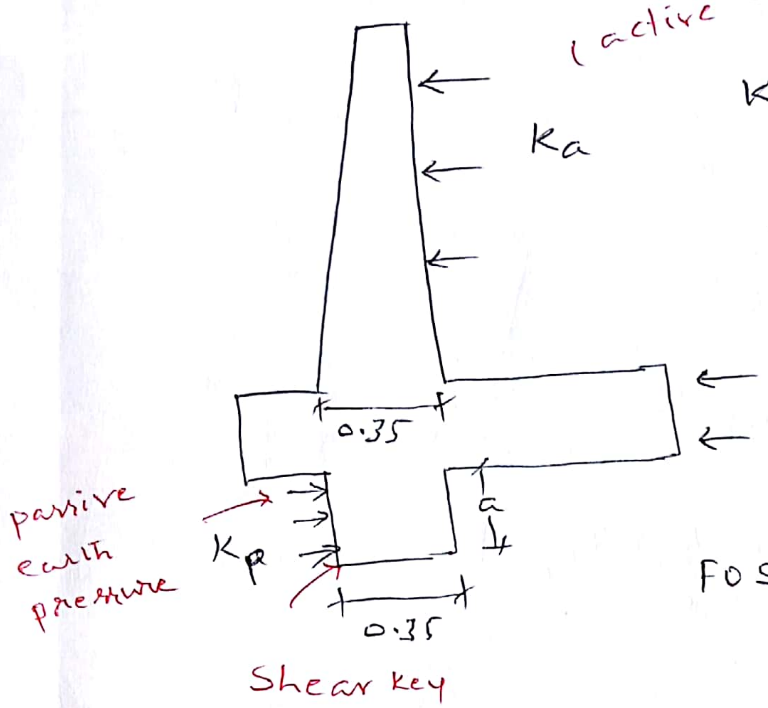
Check for stability (Sliding)

(active earth pressure)



$$K_p = \frac{1+\sin 30^\circ}{1-\sin 30^\circ} = 3$$

assume a = 200 mm

FOS against sliding

$$= \frac{\mu \Sigma W + \text{Resisting force from Shear key}}{P_H}$$

Resisting Force = $K_p \times$ upward soil pressure $\times$ area

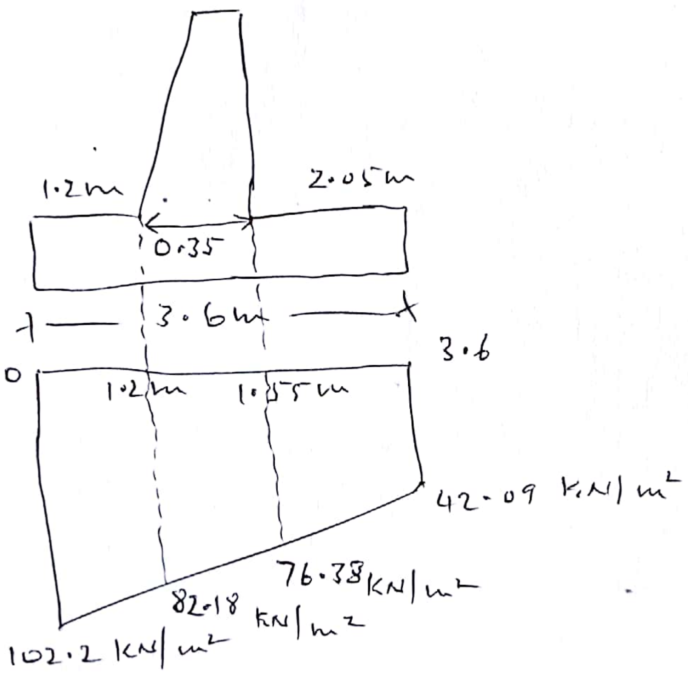
$= 3 \times 82.18\ kN/m^2 \times 0.2m \times 1m$

$= 49.31\ kN$

stress $= \frac{P}{A} \Rightarrow P = S \times A$

$$F.O.S = \frac{0.55 \times 259.79 + 49.31}{95.04} = 2.02 > 1.55$$

Hence Safe



0 → 102.2 kN/m²

1.2 → ~~76.38~~ 82.18 kN/m²

3.6 → 42.09 kN/m²

1.55 → 76.38 kN/m²

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iv) Design of Stem

$M_u)_{stem} = 219.62$ kNm [From Bottom Stem Thickness]

$D = 350$ mm

effective cover = 60 mm

$d = 350 - 60 = 290$ mm

Area of steel (Ast)

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

$$219.62 \times 10^6 = 0.87 \times 415 \times A_{st} \times 290 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 290}\right]$$

$A_{st} = 2570$ mm^2

Assume 16 mm ϕ

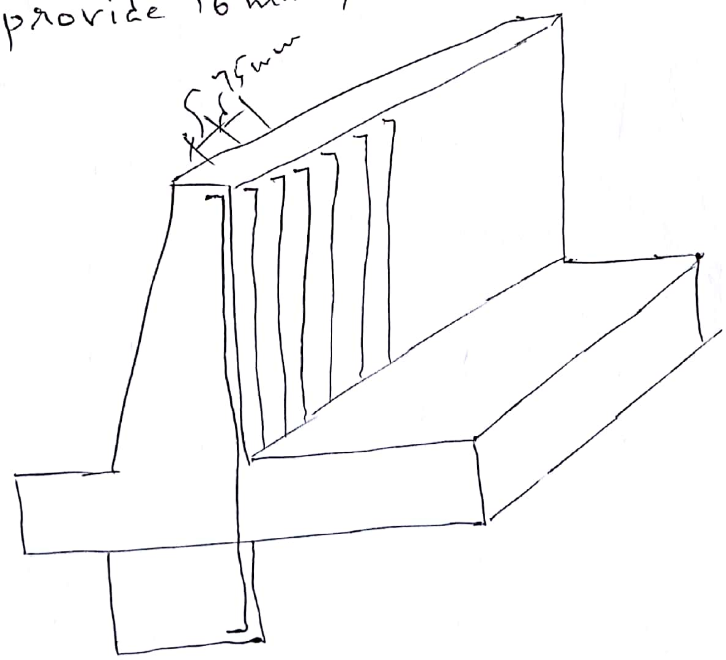
$$\text{spacing of bars} = \frac{\frac{\pi}{4}(16)^2 \times 1000}{2570} = 78.25 \text{ mm}$$

spacing is Least of the following

$3d = 3 \times 290 = 870$ mm

300 mm

Let provide 16 mm ϕ bars @ 75 mm c/c



Area of distribution bars

$A_{st})_{min} = 0.12\%$ of gross area

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

$= 420 \ mm^2$

Assuming 10 mm dia bars

Spacing $= \frac{\frac{\pi}{4}(10)^2 \times 1000}{420}$

$= 187$ mm

Spacing is least of the following

$5d = 5 \times 290 = 1450$ mm

450 mm

provide 10 mm ϕ @ 150 mm c/c

<u>Check for Shear</u>

i) Nominal Shear stress (τ_v) [Page 72 of IS456]

$\tau_v = \frac{V_u}{bd}$

$= \frac{1.5 \times 79.86 \times 10^3}{1000 \times 290}$

$\tau_v = 0.41 \ N/mm^2$

ii) Maximum shear stress (τ_{cmax}

Table 20, Page 73, IS 456

For M_{20} grade $\tau_{cmax} = 2.8 \ N/mm^2$

$\tau_v \ (0.41 N/mm^2) < \tau_{cmax} \ (2.8 \ N/mm^2)$

iii) Permissible shear strength of concrete (τ_c)

from table 19, page 73, IS 456

$$\frac{100\,A_{st}}{bd} = \frac{100 \times 2570}{1000 \times 290} = 0.88$$

$$\tau_c = 0.59\ N/mm^2$$

$\tau_v\ (0.41 N/mm^2) < \tau_c\ (0.59 N/mm^2)$

Hence safe

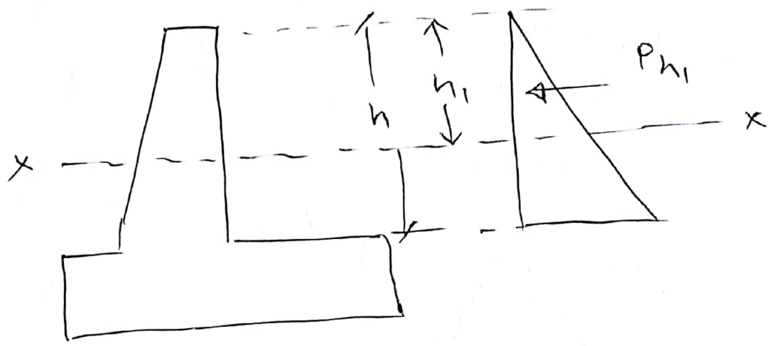
Curtailment of stem reinforcement

@ the top provide a spacing of $75 \times 2 = 150$ mm c/c

Moment will also reduced at the top

$$50\% = \frac{M_u}{2} = \frac{219.62}{2} = 109.81\ kNm$$

$$M = \frac{109.81}{1.5} = 73.2\ kNm$$



$$M_{xx} = P_{h_1} \times \frac{h_1}{3}$$

$$= \frac{1}{2} \times (K_a \cdot \gamma \cdot h_1) \cdot h_1 \cdot \frac{h_1}{3}$$

$$73.2 = \frac{1}{2} \times 0.33 \times 16 \times \frac{h_1^3}{3} \Rightarrow h_1 = 4.36\ m \text{ from top}$$

From bottom of stem $= 5.5 - 4.36$

$= 1.14\,m$

According to the code extend the bar upto $L_d = 47\phi$

$= 47 \times 16$

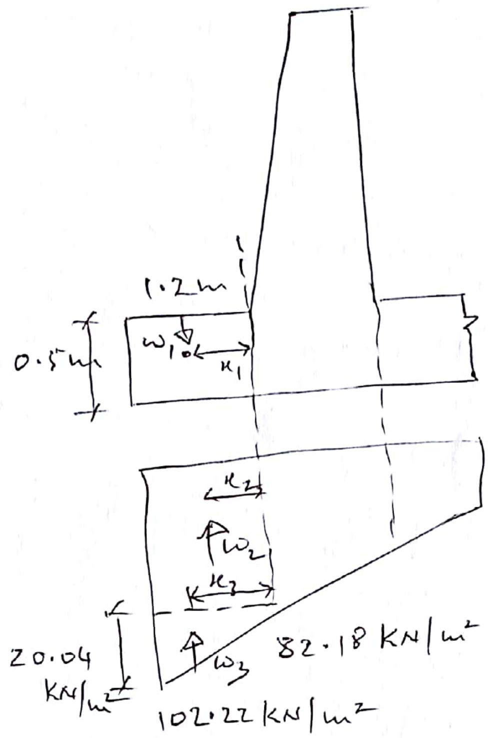
$= 752\,mm$

From bottom $= 1.14 + 0.752$

$= 1.89$

$\simeq 1.9\,m$ from bottom of stem

Design of Toe slab



Thickness of base slab, $D = 500\,mm$

Effective depth, $d = 500 - 60 = 440\,mm$

$M_1 = w_1 \cdot x_1 = (25\,KN/m^3 \times 1.2\,m \times 0.5\,m \times 1\,m) \frac{1.2}{2}$

$M_1 = 9\,KNm$ (↻)

$M_2 = w_2 \cdot x_2 = (82.18\,KN/m^2 \times 1.2\,m \times 1\,m) \times \frac{1.2}{2}$

$= 59.16\,KNm$ (↺)

$M_3 = w_3 \cdot x_3 = (20.04\,KN/m^2 \times \frac{1}{2} \times 1.2\,m \times 1\,m) \times \frac{2}{3}(1.2\,m)$

$= 9.62\,KNm$

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Net Moment, $M = -M_1 + M_2 + M_3$

$= -9 + 59.16 + 9.62$

$= 59.78$ kNm

$M_u = 1.5 \times 59.78 = 89.67$ kNm

$\boxed{M_u = 89.67 \text{ kNm}}$

Check for effective depth

$M_{ulim} = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42\frac{x_{umax}}{d}\right] f_{ck}\, b\, d^2$

$89.67 \times 10^6 = 0.36 \times 0.48 \left[1 - 0.42 \times 0.48\right] \times 20 \times 1000 \times d^2$

$d = 180.27$ mm $<$ d) provided $= 440$ mm

Hence safe.

Area of Main Steel (Ast)

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

$89.67 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$

$A_{st} = 580.33 \text{ mm}^2$

Assuming 12 mm dia bars

$\text{Spacing} = \frac{\frac{\pi}{4}(12)^2 \times 1000}{580.33} = 194.88$ mm

Spacing is least of the following

i) $3d = 3 \times 440 = 1320$ mm

ii) 300 mm

175 mm

provide 12 mm ϕ @ 190 mm c/c

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Area of distribution steel

$$A_{st})_{min} = \frac{0.12}{100} \times 1000 \times 500$$

$$= 600 \text{ mm}^2$$

providing 10mm ϕ bars

$$\text{Spacing} = \frac{\frac{\pi}{4}(10)^2 \times 1000}{600}$$

$$= 130 \text{ mm}$$

Spacing is least of the following

i) $5d = 5 \times 440 = 2200 \text{ mm}^2$

ii) 450 mm

provide 10 mm ϕ bars @ 125 mm c/c

Check for shear

$$\text{shear force}, V_u = 1.5 \times [-w_1 + w_2 + w_3]$$

$$= 1.5 [(-25 \times 1.2m \times 0.5m \times 1m) + (82.18 \times 1.2m \times 1m) + (20.04 \times \tfrac{1}{2} \times 1.2 \times 1m)]$$

$$V_u = 143.43 \text{ kN}$$

i) Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{143.43 \times 10^3}{1000 \times 440}$$

$$\tau_v = 0.32 \text{ N/mm}^2$$

ii) max. permissible shear stress ($\tau_{c\,max}$)

M_{20} concrete, Table 20, page 73, IS 456

$$\tau_{c\,max} = 2.8 \text{ N/mm}^2$$

τ_v (0.32 N/mm²) $< \tau_{c\,max}$ (2.8 N/mm²)

Hence safe

iii) Design Shear strength of concrete (τ_c)

Table 19, page 73, IS456

$$\frac{100 \, Ast}{b d} = \frac{100 \times 580.35}{1000 \times 440} = 0.132$$

$$\tau_c = 0.28 \, N/mm^2$$

τ_v ($0.32 \, N/mm^2$) $>$ τ_c ($0.28 \, N/mm^2$)

unsafe

using

$$\tau_v = \frac{V_u}{b d}$$

$\downarrow$

τ_c

$$0.28 = \frac{143.43 \times 10^3}{1000 \times d}$$

$$d = 512.27 \text{ mm}$$

effective cover of 60 mm

580

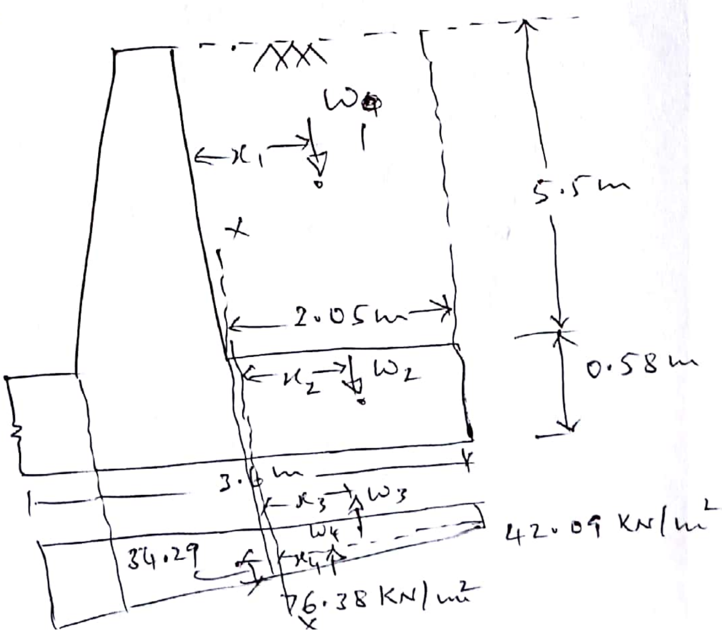
D = 512.27 + 60 = 572.27 $\simeq$ 600 mm

580

d = 600 − 60 = 540 mm

520

vi) Design of Heel Slab



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$M_1 = W_1 \cdot x_1 = \left[16\,KN/m^3 \times 2.05 \times 5.5\,m \times 1\,m\right] \times \frac{2.05}{2}$

$= 184.91\,KNm$ (A)

$M_2 = W_2 \cdot x_2 = \left[25\,KN/m^3 \times 2.05\,m \times 0.\cancel{55}\,58\,m \times 1\,m\right] \times \frac{2.05}{2}$

$= 30.47\,KNm$ (A)

$M_3 = W_3 \cdot x_3 = \left[42.09\,KN/m^2 \times 2.05\,m \times 1\,m\right] \times \frac{2.05}{2}$

$= 88.44\,KNm$ (B)

$M_4 = W_4 \cdot x_4 = \left[34.29\,KN/m^2 \times \frac{1}{2} \times 2.05 \times 1\,m\right] \times \frac{1}{3}(2.05)$

$= 24.02\,KNm$ (B)

Net Moment, $M = +184.91 + 30.47 - 88.44 - 24.02$

$M = 102.92\,KNm$

$M_u = 1.5 \times 102.92 = 154.38\,KNm$

Check for effective depth (d)

$M_{ulim} = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42 \frac{x_{umax}}{d}\right] f_{ck} b d^2$

$154.38 \times 10^6 = 0.36 \times 0.48 \left[1 - 0.42 \times 0.48\right] \times 20 \times 1000\, d^2$

$d = 236.54\,mm < d)$ provided $= 520\,mm$

Hence safe

$D = 580\,mm$

effective cover $= 60\,mm$

$d = 580 - 60$

$= 520\,mm$

Area of main steel (A_{st})

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

$154.38 \times 10^6 = 0.87 \times 415 \times A_{st} \times 520 \left[1 - \frac{415\,A_{st}}{20 \times 1000 \times 520}\right]$

$A_{st} = 851.19\,mm^2$

provide 12 mm dia (ϕ) bar

$$\text{Spacing} = \frac{\frac{\pi}{4}(12)^2 \times 1000}{851.19} = 132.87 \text{ mm}$$

Spacing is least of the following

i) $3d = 3 \times 520 = 1560$

ii) ~~450~~ 300 mm

provide 12 mm ϕ bars @ 125 mm c/c

Area of distribution steel

$$A_{st})_{min} = \frac{0.12}{100} \times 1000 \times 580 = 696 \text{ mm}^2$$

Assuming 10 mm dia bars

$$\text{spacing} = \frac{\frac{\pi}{4}(10)^2 \times 1000}{696} = 112.8 \text{ mm}$$

Spacing is least of the following

i) $5d = 5 \times 520 = 2600$ mm

ii) 450 mm

provide 10 mm ϕ bars @ 100 mm c/c

Check for shear

i) check for nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} \quad \text{— ①}$$

$$V_u = 1.5 \times [-w_1 - w_2 + w_3 + w_4]$$ $w_1 = 16 \text{ KN/m}^3 \times 2.05 \times 5.5$

$$= 1.5 \times [-180.4 - 29.73 + 86.28 + 35.14]$$

V_u = ~~828.71~~ KN   Neglect -ve sign

-133.06

From ① $\Rightarrow$ $\tau_v = \frac{133.06 \times 10^3}{1000 \times 520} = 0.26\ N/mm^2$

ii) Max. permissible shear strength of concrete for M20 grade concrete, from table 20, page 73

$\tau_{c\,max} = 2.8\ N/mm^2$

Comparing

$\tau_v\ (0.26\ N/mm^2) < \tau_{c\,max}\ (2.8\ N/mm^2)$

Hence safe

iii) Design shear strength of concrete, τ_c

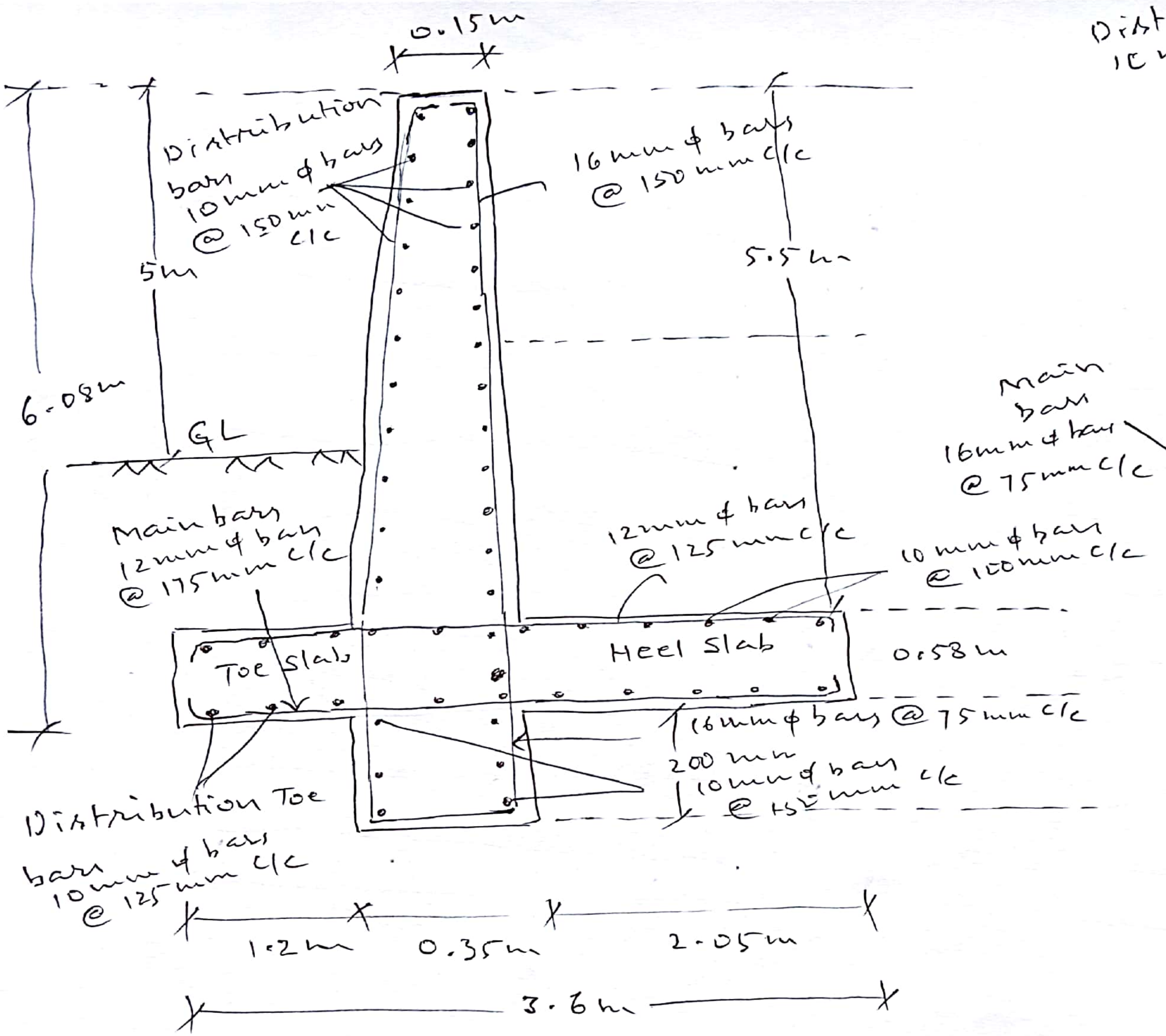
$\frac{100\,A_{st}}{bd} = \frac{100 \times 851.18}{1000 \times 520} = 0.163$

From table 19 of IS 456, page 73

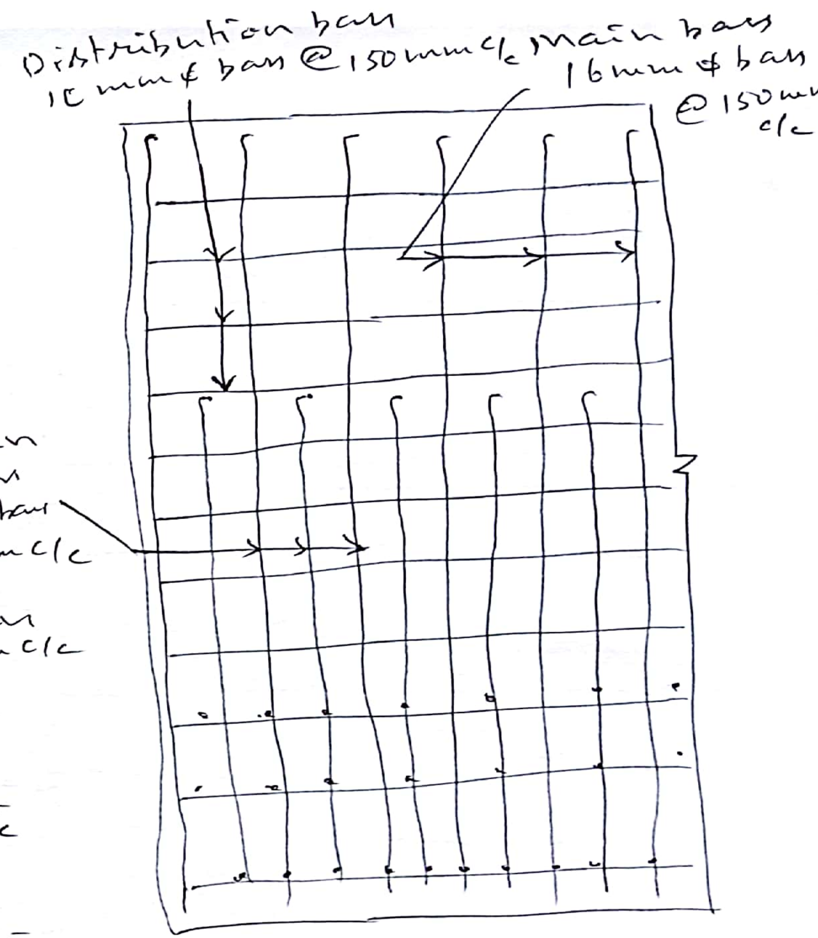
$\tau_c = 0.3\ N/mm^2$

$\tau_v\ (0.26\ N/mm^2) < \tau_c\ (0.3\ N/mm^2)$

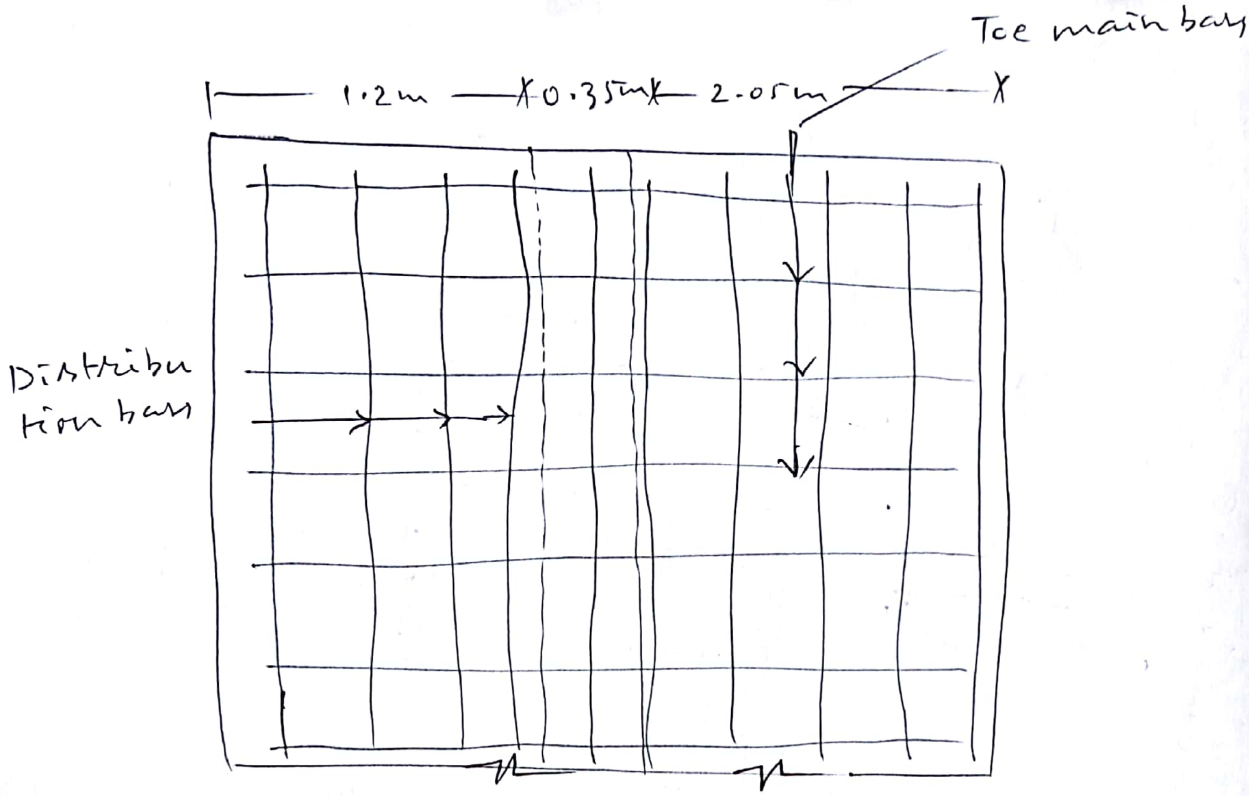
Hence safe.



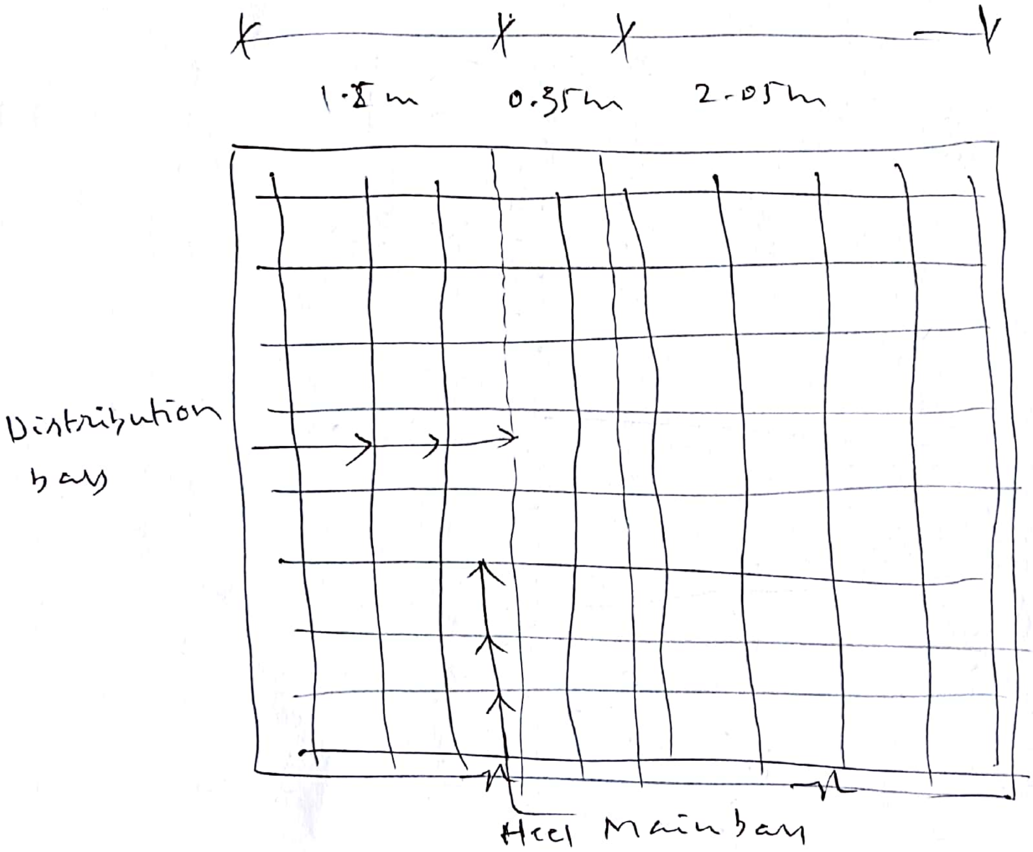
Reinforcement details in cantilever retaining wall



Longitudinal section of wall

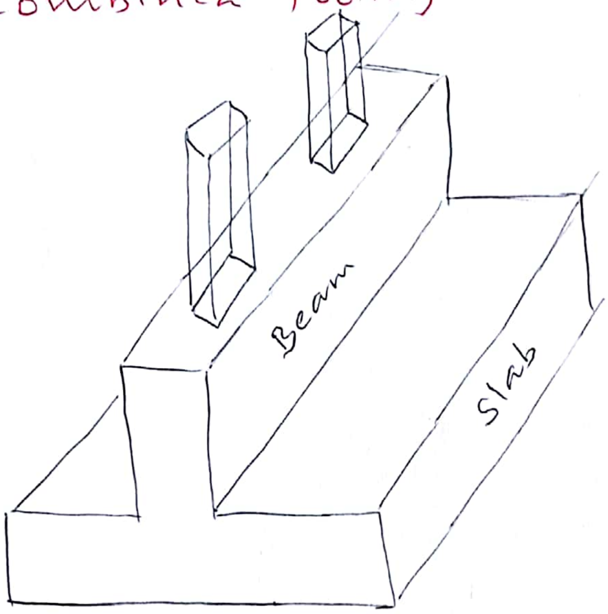


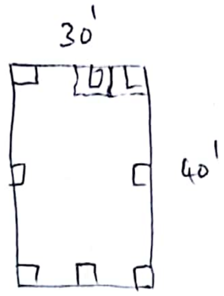
Bottom plan of base slab



Top plan of base slab

Combined Footing





Beam and Slab type combined Footing

1) Design a rectangular slab and beam type Combined footing for 2 columns spaced @ 4 m c/c. The first column is 300 mm x 300 mm and carries 800 kN. The second column is 400 mm x 400 mm and carries 1200 kN. The width of footing is restricted to 1.8 m. Use SBC of soil as 180 kN/m². Use M25 grade concrete & Fe 415 grade steel.

i) Data:

1st column

Size = 300 mm x 300 mm

Load = 800 kN

2nd column

Size = 400 mm x 400 mm

Load = 1200 kN

SBC = 180 kN/m²

M_{25} → Concrete

Fe 415 → Steel

ii) Dimensions of footing

Total load = Load from (Column A + Column B)

= 800 + 1200

= 2000 kN

[w = density x vol.
= 25 kN/m³ x b D L]

Self weight of column = 200 kN
[10 % of Column load]

Stress = $\frac{Load}{Area}$

Area = $\frac{Load}{Stress}$

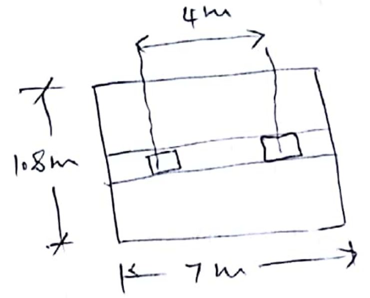
Total Load = 2200 kN

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

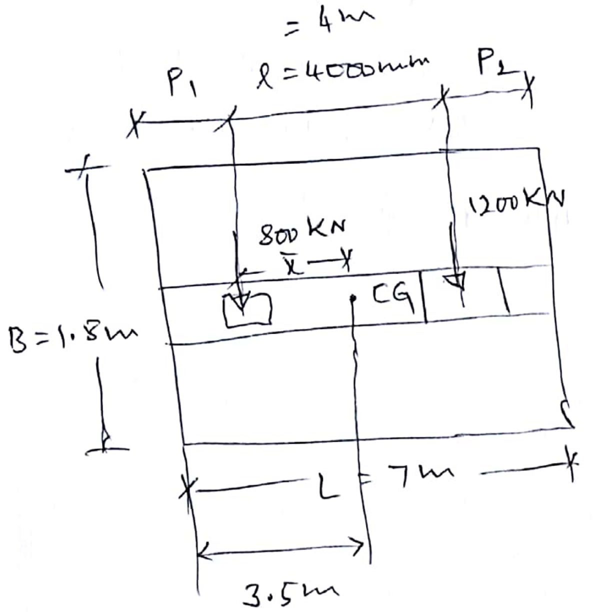
$$L \times B = \frac{2200 \text{ kN}}{180 \text{ kN/m}^2}$$

$$L \times 1.8 = \frac{2200}{180} = 6.79 \text{ m} \simeq 7 \text{ m}$$

$\boxed{L \times B = 7m \times 1.8m}$



iii) projections



$$\bar{x} = \frac{a_1x_1 + a_2x_2 + \cdots}{a_1 + a_2 + \cdots}$$

$$P_1 + \bar{x} = 3.5 \quad \text{— (1)}$$

$$\bar{x} = \frac{w_1x_1 + w_2x_2}{w_1 + w_2}$$

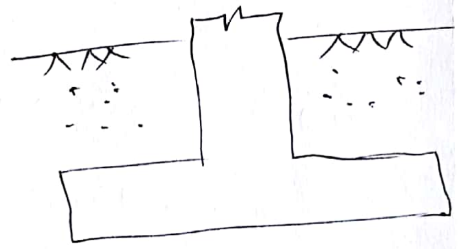
$$= \frac{800 \times 0 + 1200 \times 4}{800 + 1200}$$

$$\bar{x} = 2.4\,m$$

Stress = only column load / Area

$$= \frac{2000\,kN}{7 \times 1.8}$$

$$= 158.73\,kN/m^2$$



From (1)

$$P_1 + \bar{x} = 3.5$$

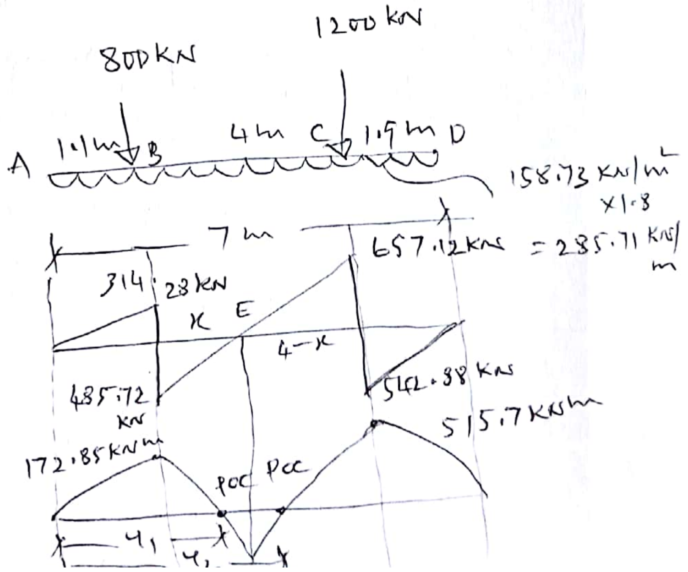
$$P_1 = 3.5 - 2.4$$

$$\boxed{P_1 = 1.1\,m}$$

$$1.1 + 4m + P_2 = 7m$$

$$\boxed{P_2 = 1.9\,m}$$

iv) BMD and SFD



SFD

S.F Left of A = 0

SF Right of A = 0

SF Left of B = 285.71 × 1.1 = 314.28 kN

SF Right of B = 314.28 − 800 = − 485.72 kN

SF Left of C = 285.71 × 5.1 − 800 = 657.12 kN

SF Right of C = 657.12 − 1200 = − 542.88 kN

SF Left of D = 285.71 × 7 − 800 − 1200 = 0

SF Right of D = 0

BMD

$M_A = M_D = 0$

$M_B = 285.71 \times 1.1 \times \frac{1.1}{2} = 172.85$ kNm

$M_C = 285.71 \times 5.1 \times \frac{5.1}{2} - 800 \times 4 = 515.7$ kNm

Using similar triangle concept

$\frac{x}{4-x} = \frac{485.72}{\cancel{542.88}\ 657.12} \Rightarrow x = (4-x)\,0.739$

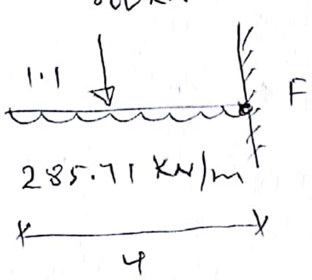
$x = 2.956 - 0.739x$

$x = 1.7$ m

$M_E = 285.71 \times \frac{(1.1+1.7)^2}{2} - 800 \times 1.7$

$= -240.01$ kNm

POC



$M_F = 285.71 \times \frac{y^2}{2} - 800\,(y - 1.1)$

$0 = 285.71 \frac{y^2}{2} - 800y + 880$

$142.855y^2 - 800y + 880 = 0$

$y^2 - 5.6y + 6.16 = 0$

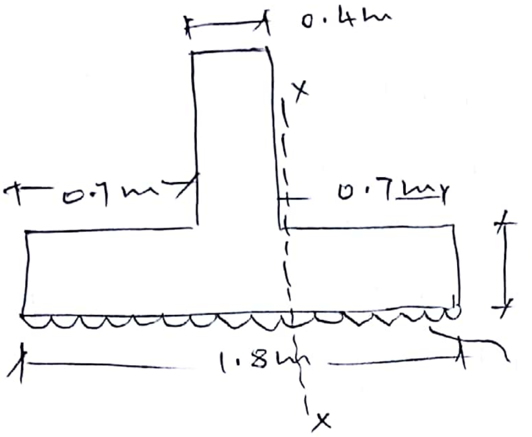
$y = \frac{+5.6 \pm \sqrt{5.6^2 - 4(1)(6.16)}}{2.1}$

$= \frac{5.6 \pm 2.59}{2}$

$y = 4.09$ m, 1.5 m

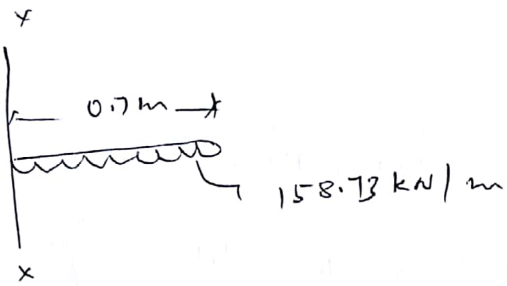
$\boxed{y_1 = 1.5\text{ m} \quad y_2 = 4.09\text{ m}}$

v) Design of footing slab



Assume width of beam as 400 mm

158.73 kN/m² × 1m strip
= 158.73 kN/m



$M_{xx} = \frac{158.73 \times 0.7^2}{2} = 38.89$ kNm

$\boxed{M_u = 1.5 \times 38.89 = 58.33 \text{ kNm}}$

* To find effective depth (d)

$M_{ulim} = 0.36 \frac{x_{umax}}{d} \left[1 - 0.42 \frac{x_{umax}}{d}\right] f_{ck} b d^2$

$58.33 = 0.36 \times 0.48 \left[1 - 0.42 \times 0.48\right] \times 25 \times 1000 \times d^2$

$d = 130.05$ mm

using 60 mm effective cover

$D = 130.05 + 60 = 190.05 \simeq 200$ mm

From shear consideration double the depth

$D = 2 \times 200 = 400$ mm

$\boxed{d = 400 - 60 = 340 \text{ mm}}$

* Area of Main Steel (A_{st})

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

$$53.33 \times 10^6 = 0.87 \times 415 \times A_{st} \times 340 \left(1 - \frac{415 A_{st}}{25 \times 1000 \times 340}\right)$$

$$A_{st} = 486.73 \text{ mm}^2$$

Check for A_{st} min

$$A_{st\,min} = \frac{0.12}{100} \times b \times D$$

$$= \frac{0.12}{100} \times 1000 \times 400$$

$$= 480 \text{ mm}^2$$

$$A_{st} = 486.73 \text{ mm}^2 > A_{st\,min} = 480 \text{ mm}^2$$

Hence safe.

Assuming 12 mm dia bars

$$\text{Spacing} = \frac{\frac{\pi}{4}(12)^2}{486.73} \times 1000$$

$$= 232.3 \text{ mm}$$

* Spacing is least of the following

i) $3d = 3 \times 340 = 1020$ mm

ii) 300 mm

provide 12 mm ϕ bars @ 225 mm c/c

* Area of distribution steel

$$A_{st\,min} = 480 \text{ mm}^2$$

Assuming 8 mm dia bars

$$\text{Spacing} = \frac{\frac{\pi}{4}(8)^2}{480} \times 1000 = 104.72 \text{ mm}$$

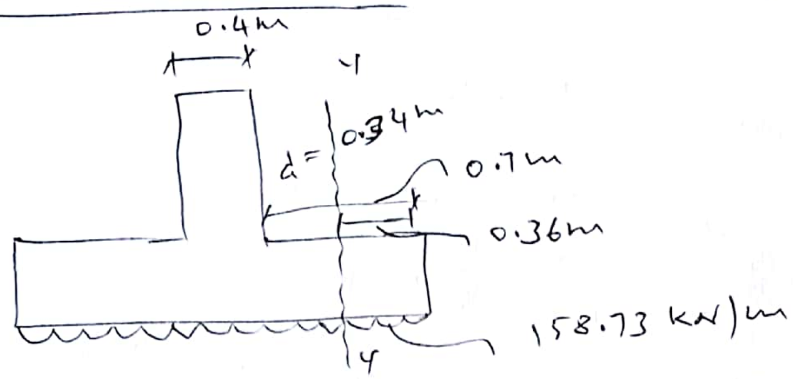
Spacing is least of the followings

i) $5d = 5 \times 340 = 1700$ mm

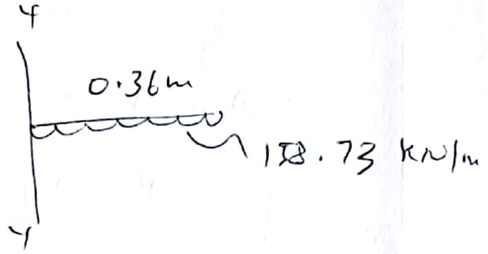
ii) 450 mm

provide 8 mm ϕ bars @ 100 mm c/c

× <u>Check for shear</u>



Shear force will be maximum at a distance 'd' from the face of the beam



S.F about y-y axis $= 158.73 \times 0.36$

$= 57.14$ kN

$V_u = 1.5 \times 57.14$

$= 85.71$ kN

i) Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{85.71 \times 10^3}{1000 \times 340}$$

$\tau_v = 0.25$ N/mm²

ii) Max. permissible shear stress ($\tau_{c\,max}$)

for M_{25} grade concrete, from table 20 page 73 of IS 456

$\tau_{c\,max} = 3.1$ N/mm²

comparing τ_v (0.25 N/mm²) $< \tau_{c\,max} = 3.1$ N/mm²

Hence safe.

iii) Design Shear strength of Concrete (τ_c)

$$\frac{100\,A_{st}}{bd} = \frac{100 \times 486.73}{1000 \times 340} = 0.14$$

from table 19, page 73, IS 456

$\tau_c = 0.29\ N/mm^2$

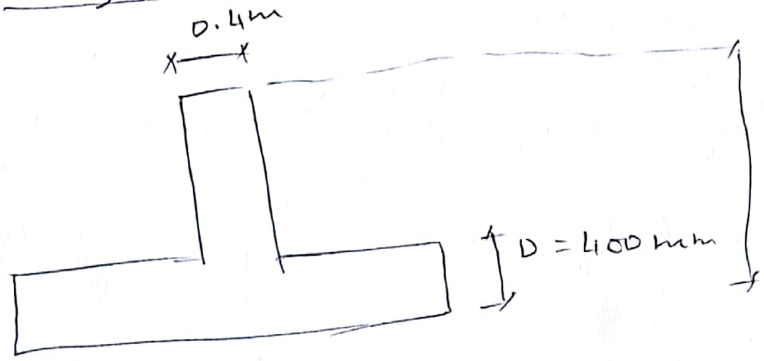
for slab, final $\tau_c = k\,\tau_c$

$= 1 \times 0.29\ N/mm^2$

$\tau_v = 0.25\ N/mm^2 < \tau_c = 0.29\ N/mm^2$)

Hence safe.

vi) Design of beams



From BMD, highest value is

$M = 515.70\ kNm$

$M_u = 1.5 \times 515.70 = 773.55\ kNm$

* To find effective depth (d')

$$M_{ulim} = 0.36\ \frac{x_{umax}}{d}\left[1 - 0.42\ \frac{x_{umax}}{d}\right] f_{ck}\, bd^2$$

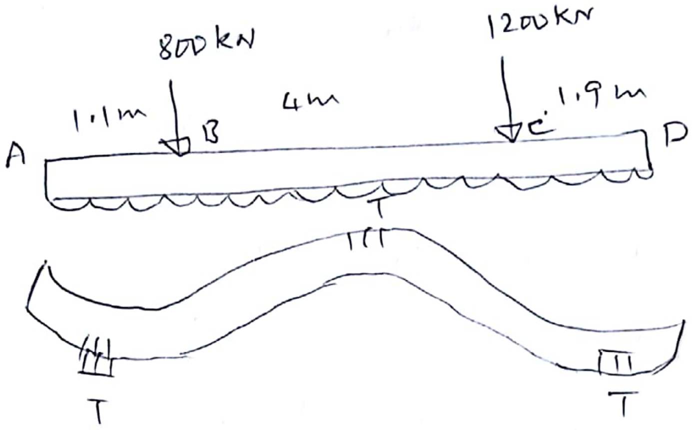
$$773.55 \times 10^6 = 0.36 \times 0.48 \left[1 - 0.42 \times 0.48\right] 25 \times 400 \times d_1'^2$$

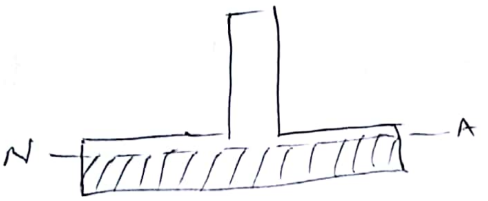
$d' = 748.79\ mm$

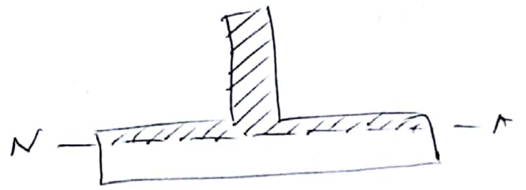
$\simeq 750\ mm$

using 50 mm effective cover

$D' = 750 + 50 = 800\ mm$







AB and CD portion

Mid portion DC

Tension lies with in the flange

∴ design it as rectangular beam $(b = b_w)$

Tension lies with in the web

∴ design it as T-beam $(b = b_f)$

* AB portion

Rectangular beam $(b = b_w = 400\,mm)$

$M = 172.85$ kNm

$M_u = 1.5 \times 172.85 = 259.27$ kNm

$D' = 800$ mm

$d' = 750$ mm

Area of Steel (Ast)

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

$$259.27 \times 10^6 = 0.87 \times 415 \times A_{st} \times 750 \left\{ 1 - \frac{415 \times A_{st}}{25 \times 400 \times 750} \right]$$

$A_{st} = 1014.39\ mm^2$

Assuming 20 mm of bars

$$\text{No. of bars} = \frac{1014.39}{\frac{\pi}{4}(20)^2} = 3.02 \simeq 4$$

* BC portion

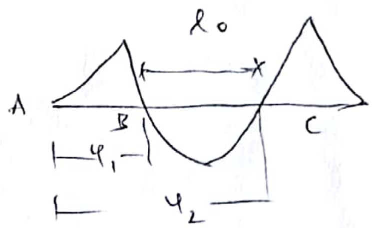
$b = b_f$

from page 37 of IS 456

For isolated T beam

$$b_f = \frac{l_o}{\left(\frac{l_o}{b}\right) + 4} + b_w$$

width of flange



$l_o = y_2 - y_1$

$= 4.09 - 1.5$

$$\boxed{l_o = 2.59\ m}$$

$$b_f = \frac{2.59 \times 1000}{\frac{2590}{1800} + 4} + 400$$

$b_f = 876.2\ mm$

from BMD, $M = 240.06\ kNm$

$M_u = 1.5 \times 240.06$

$M_u = 360\ kNm$

* Area of Steel (Ast)

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

$$360 \times 10^6 = 0.87 \times 415 \times A_{st} \times 750 \left\{1 - \frac{415 A_{st}}{25 \times \cancel{4000}\ 876.2 \times 750}\right\}$$

$A_{st} = 1377.24\ mm^2$

Assume 20 mm ϕ bars

$$\text{No. of bars} = \frac{1377.24}{\frac{\pi}{4}(20)^2} = 4.38 \simeq 5$$

* CD portion of beam

$M = 515.17$ KNm

$M_u = 1.5 \times 515.17 = 773.53$ KNm

$b = b_w = 400$ mm

* Area of steel (Ast)

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

$$773.53 \times 10^6 = 0.87 \times 415 \times A_{st} \times 750 \left[1 - \frac{415 \times A_{st}}{25 \times 400 \times 750}\right]$$

$A_{st} = 3556.38$ mm^2

Assuming 32 mm ϕ bars

$$\text{No. of bars} = \frac{3556.38}{\frac{\pi}{4}(32)^2} = 4.4 \simeq 5$$

$$A_{st} \text{ provided} = 5 \times \frac{\pi}{4}(32)^2 = 4021.24 \text{ mm}^2$$

Check for shear

i) Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd}$$

$V = 657.12$ kN

$V_u = 1.5 \times 657.12 = 985.69$ kN

$$= \frac{985.69 \times 10^3}{400 \times 750}$$

$\tau_v = 3.28$ N/mm^2

ii) Max. permissible shear stress [$\tau_{c\,max}$]

For M_{25} grade concrete, from table No. 20, Page 73 of IS 456

$\tau_{c\,max} = 3.1\ N/mm^2$

Comparing $\tau_v\ (3.28\ N/mm^2) > \tau_{c\,max}\ (3.1\ N/mm^2)$

unsafe

WKT $\tau_v = \frac{V_u}{bd}$

$\tau_{c\,max}$

$3.1 = \frac{985.69 \times 10^3}{400 \times d}$

$d = 794.91 \simeq 800\ mm.$

effective cover $= 50\ mm$

$D = 800 + 50 = 850\ mm$

So nominal shear stress $(\tau_v) = \frac{V_u}{bd}$

$= \frac{985.69 \times 10^3}{400 \times 800}$

$\tau_v = 3\ N/mm^2$

Comparing $\tau_v\ (3\ N/mm^2) < \tau_{c\,max}\ (3.1\ N/mm^2)$

hence safe.

iii) Design shear strength of concrete (τ_c)

$\frac{100\ Ast\ provided}{bd} = \frac{100 \times 4021.24}{400 \times 800} = 1.25$

page 73, table 19 of IS456

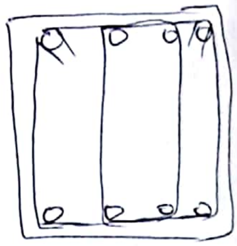
$\tau_c = 0.7\ N/mm^2$

Comparing $\tau_v = 3\ N/mm^2 > \tau_c = 0.7\ N/mm^2$

provide shear reinforcement in the form of vertical stirrups

Let us provide 4 legged 8 mm ϕ vertical stirrups

$A_{sv} = \frac{\pi}{4}(8)^2 \times 4 = 201.06 \text{ mm}^2$



To find Spacing of vertical stirrups

page 73 of IS 456

$V_{us} = V_u - \tau_c b d$ —①

where $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$ —②

substituting ② in ①

$\frac{0.87 \times 415 \times 201.06 \times 800}{S_v} = 985.69 \times 10^6 - 0.7 \times 400 \times 800$

$S_v = 76.24$ mm

* spacing is least of the following

i) $0.75d = 0.75 \times 800 = 600$ mm

ii) 300 mm

provide 8 mm dia 4L v.s @ 75 mm c/c near the column & 300 mm c/c in the other portion.

Slab Type Combined Footing

Two rectangular columns 300 mm x 300 mm & 500 mm x 500 mm in size carry axial load of 600 kN & 850 kN respectively. The columns are spaced @ 3.5 m c/c. SBC of the soil is 150 kN/m^2. The property line is 0.8 m from the centre of first column. use M20 grade concrete & Fe415 grade steel. Design the footing as slab type combined footing.

i) Data:

1st column = 300 mm x 300 mm

Load = 600 kN

2nd column = 500 mm x 500 mm

Load = 850 kN

S.B.C = 150 kN/m^2

M20 grade concrete

Fe 415 grade steel

ii) Size of the footing:

Total column load = 600 + 850 = 1450 kN

Assume self weight of column = 145 kN

[10% of total column load]

Total Load = 1595 kN

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

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

* Area of footing

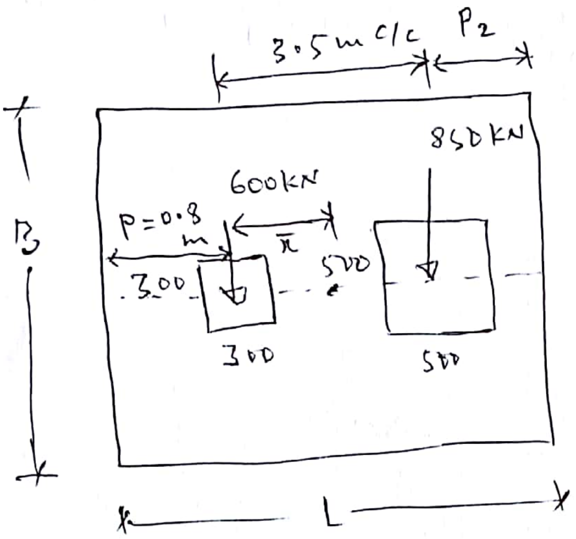
$$\text{Area} = \frac{\text{Total Load}}{\text{SBC of soil}}$$

$$L \times B = \frac{1595 \text{ kN}}{150 \text{ kN/m}^2}$$

$$L \times B = 10.63 \text{ m}^2 \quad \text{— (1)}$$

Note: Do not assume the width of the footing as projection information is given

* To find projections



For uniform upward soil pressure C.G. of footing should coinside with C.G. of columns.

$$\bar{x} = \frac{w_1 x_1 + w_2 x_2}{w_1 + w_2}$$

$$= \frac{600 \times 0 + 850 \times 3.5}{600 + 850}$$

$$\boxed{\bar{x} = 2.05 \text{ m}}$$

From figure $P_1 + \bar{x} = \frac{L}{2}$

$$0.8 + 2.05 = \frac{L}{2}$$

$$L = 5.7 \text{ m}$$

From (1) $\Rightarrow$ $L \times B = 10.63$

$$5.7 \times B = 10.63$$

$B = 1.86\,m \simeq 1.9\,m$

provide $L \times B = 5.7\,m \times 1.9\,m$

* To find P_2

From fig

$P_1 + 3.5 + P_2 = L$

$0.8 + 3.5 + P_2 = 5.7$

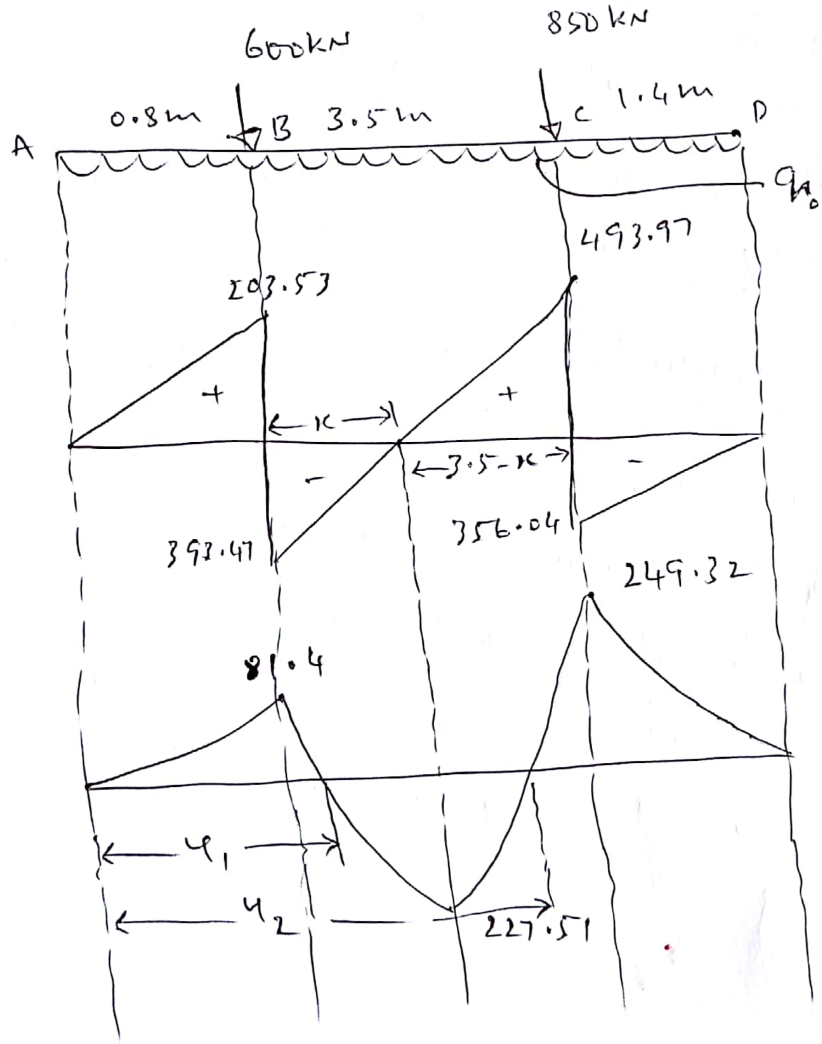
$P_2 = 1.4\,m$

* Net upward Soil pressure, $q_0 = \dfrac{\text{only Column load}}{\text{Area}}$

$= \dfrac{1450\,KN}{5.7 \times 1.9}$

$= 133.9\,KN/m^2$

BMD & SFD



Breadth of footing

$q_0 = 133.90\,KN/m^2 \times 1.9\,m$

$= 254.41\,KN/m$

SF calculation:

- SF Left of A = 0
- SF Right of A = 0
- SF Left of B = + 254.41 × 0.8 = 203.53 KN
- SF Right of B = 203.53 − 600 = −397.47 KN
- SF Left of C = −397.47 + 254.41 × 3.5 = 493.97 KN
- SF Right of C = 493.97 − 850 = −356.04 KN
- SF Left of D = −356.04 + 254.41 × 1.4 = 0
- SF Right of D = 0

BM calculation:

$M_A = 0$

$M_D = 0$

$M_B = 254.41 \times 0.8 \times \frac{0.8}{2} = 81.4$ kNm

$M_C = 254.41 \times 4.3 \times \frac{4.3}{2} - 600 \times 3.5 = 249.32$ kNm

using similar △le concept

$$\frac{x}{393.47} = \frac{3.5 - x}{493.97} \qquad x = 1.55\text{ m}$$

$$M_E = 254.41 \times \frac{(0.8 + 1.55)^2}{2} - 600 \times 1.55 = 227.51 \text{ kNm}$$

To find the position of point of contraflexure

equating BM_x b/w B and C = 0

ie $254.41 \times \frac{y^2}{2} - 600(y - 0.8) = 0$

$127.205y^2 - 600y + 480 = 0$

Solving $y = 1.02$ m and 3.695 m.

* Depth of footing

$M_u = 1.5 \times 249.32$
$= 373.98$ KNm

$$M_{ulim} = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42 \frac{x_{umax}}{d}\right] f_{ck}\, b d^2$$

$$373.98 \times 10^6 = 0.36 \times 0.48 \left[1 - 0.42 \times 0.48\right] 20 \times 1000 \times d^2$$

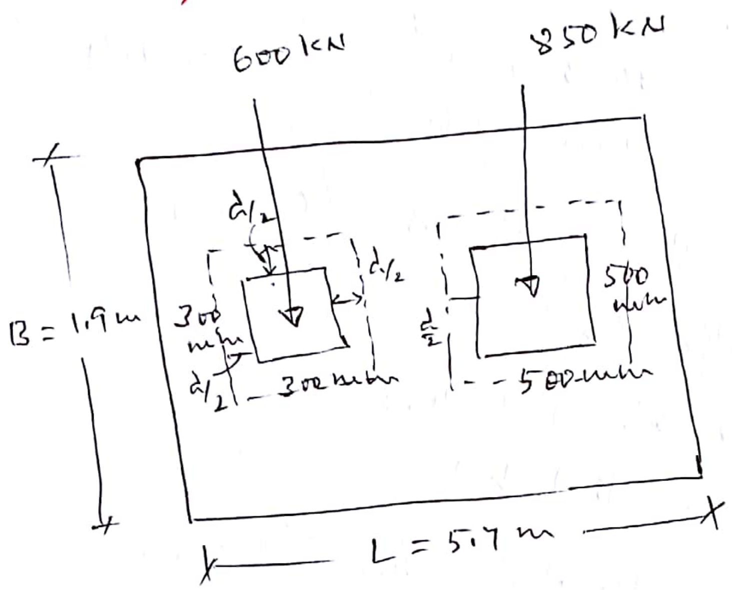
$d = 368.15$ mm

$d \simeq 400$ mm

providing 60 mm effective cover

$D = 400 + 60 = 460$ mm

* Design of Footing Slab



* Check for two-way shear

V_u = Column Load − stress × area of critical section — ①

1st column

$P = 600$ KN

$P_u = 1.5 \times 600 = 900$ KN

$q_o = 133.90$ KN/m²

Area of critical section $= B^2$

$$= \left[\frac{400}{2} + 300 + \frac{400}{2}\right]^2$$

$= 490000$ mm²

$= 0.49$ m²

From ① ⇒

$$V_u = [900 - 133.9 \times 0.49] = 834.389 \text{ kN}$$

* Nominal shear stress [τ_v]

$$\tau_v = \frac{V_u}{b'd} \quad — ②$$

b' = perimeter of critical section

$$= \left[\frac{400}{2} + 300 + \frac{400}{2}\right] 4$$

$$= 2800 \text{ mm}$$

From ② ⇒

$$\tau_v = \frac{834.389 \times 10^3}{2800 \times 400} = 0.745 \text{ N/mm}^2$$

* permissible shear stress [τ_{per}]

$$\tau_{per} = K_s \cdot \tau_c$$

from page 58 of IS456

$$K_s = (0.5 + \beta_c) \ngtr 1$$

Then $\beta_c = \dfrac{\text{shorter dimension of column}}{\text{Longer dimension of column}}$

$$= \frac{300}{300}$$

$$= 1$$

$$K_s = 0.5 + 1 = 1.5 > 1$$

Hence take $K_s = 1$

$$\tau_c = 0.25 \sqrt{f_{ck}} = 0.25 \cdot \sqrt{20} = 1.118 \text{ N/mm}^2$$

$$\tau_{per} = K_s \cdot \tau_c = 1 \times 1.118 = 1.118 \text{ N/mm}^2$$

Comparing τ_v (0.745 N/mm²) < τ_{per} (1.118 N/mm²)

Hence safe.

For 2nd Column

$P = 850\ kN$

$P_u = 1.5 \times 850 = 1275\ kN$

$q_o = 133.90\ kN/m^2$

area of critical section $= \left[\frac{400}{2} + 500 + \frac{400}{2}\right]^2$

$= 900^2$

$= 810000\ mm^2$

$= 0.81\ m^2$

From ① $\Rightarrow$

$V_u = 1275 - 133.9 \times 0.81 = 1166.54\ kN$

Nominal Shear stress $[\tau_v]$

$$\tau_v = \frac{V_u}{b'd} \quad - ③$$

b' = perimeter of critical section

$= 4\left[\frac{400}{2} + 500 + \frac{400}{2}\right] = 3600\ mm$

③ $\Rightarrow$ $\tau_v = \frac{1166.54 \times 10^3}{3600 \times 400} = 0.81\ N/mm^2$

permissible shear stress $[\tau_{per}]$

$\tau_{per} = k_s \cdot \tau_c$

where $k_s = [0.5 + \beta_c] \ngtr 1$

where $\beta_c = \frac{\text{shorter dimension of column}}{\text{Longer dimension of column}}$

$= \frac{500}{500} = 1$

$k_s = 0.5 + 1 = 1.5 > 1$

Take $k_s = 1$

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

$= 0.25 \sqrt{20}$

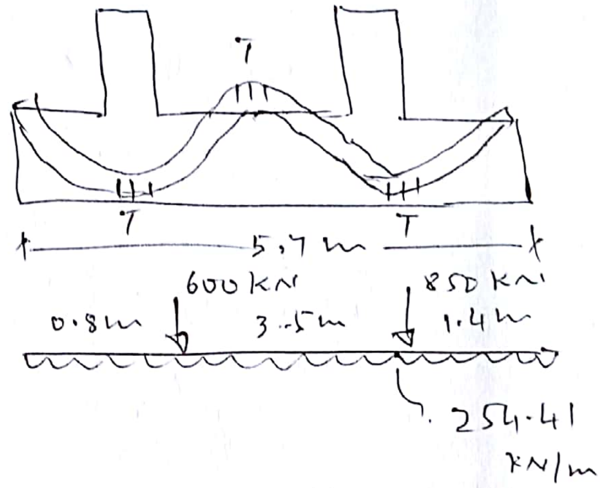
$= 1.118 \ N/mm^2$

$\tau_{per} = 1 \times 1.118 = 1.118 \ N/mm^2$

comparing

$\tau_v \ (0.81 \ N/mm^2) < \tau_{per} \ (1.118 \ N/mm^2)$

Hence safe



* Reinforcement along longitudinal direction

i) Span AB

$M = 81.41 \ kNm$

$M_u = 1.5 \times 81.41 = 122.115 \ kNm$

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

$$122.115 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 A_{st}}{20 \times 1900 \times 400}\right]$$

breadth of footing

$A_{st} = 866.01 \ mm^2$

Assuming 16 mm ϕ bar

$$\text{No. of bars} = \frac{866.01}{\frac{\pi}{4} \times 16^2} = 4.3 \simeq 5 \ \text{No's}$$

ii) Span CD

$M = 249.32$ KNm

$Mu = 1.5 \times 249.32 = 373.98$ KNm

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

$373.98 \times 10^6 = 0.87 \times 415\, Ast \times 400 \left[1 - \frac{415\, Ast}{20 \times 1900 \times 400}\right]$

$Ast = 2804.03\ mm^2$

Assuming 16 mm dia bars

No. of bars $= \frac{2804.03}{\frac{\pi}{4}(16)^2} = 13.94 \simeq 14$

BC portion

$M = 190.646$ KNm

$Mu = 1.5 \times 190.646$ KNm $= 285.96$ KNm

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

$285.96 \times 10^6 = 0.87 \times 415\, Ast \times 400 \left[1 - \frac{415\, Ast}{20 \times 1900 \times 400}\right]$

$Ast = 2100.48\ mm^2$

Assuming 16 mm ϕ bars

No. of bars $= \frac{2100.48}{\frac{\pi}{4}(16)^2} = 10.4 \simeq 11$

* Design of Shear reinforcement

i) AB portion

$V = 396.47$ KN

$Vu = 1.5 \times 396.47 = 594.705$ KN

a) Nominal Shear stress (τ_v)

$$\tau_v = \frac{V_u}{b d} = \frac{594.705 \times 10^3}{1900 \times 400} = 0.78 \ N/mm^2$$

b) Maximum permissible shear stress [τ_c max]

M_{20} grade concrete from Table 20, page 73, IS456

$$\tau_{c max} = 2.8 \ N/mm^2$$

Comparing $\tau_v = 0.78 \ N/mm^2 < \tau_{cmax} = 2.8 \ N/mm^2$

Hence safe

c) Design Shear strength of Concrete (τ_c)

$$\frac{100 A_{st}}{b d} = \frac{100 \times 866.01}{1900 \times 400} = 0.1139$$

From Table 19, page 73, IS456

$$\tau_c = 0.28 \ N/mm^2$$

Comparing $\tau_v \ (0.78 N/mm^2) > \tau_c \ (0.28 N/mm^2)$

provide shear reinforcement in the form of vertical stirrups

Assuming 4 legged 8 mm ϕ vertical stirrups

$$A_{sv} = 4 \times \frac{\pi}{4} (8)^2 = 201.06 \ mm^2$$

From page 72, IS456

$$V_{us} = V_u - \tau_c b d \quad — (1)$$

where $$V_{us} = \frac{0.87 f_y \cdot A_{sv} \cdot d}{S_v} \quad — (2)$$

(1) = (2)

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$$\frac{0.87 \times 415 \times 201.06 \times 400}{S_V} = 594.705 \times 10^3 - 0.28 \times 1900 \times 400$$

$S_V = 76.02$ mm

* Spacing is least of the following

i) $0.75d = 0.75 \times 400 = 300$ mm

ii) 300 mm

provide 4L # 8 mm ϕ V.S @ 75 mm c/c

* CD portion

i) Nominal shear stress (τ_v)

From SFD
$V = 493.97$ KN
$V_u = 1.5 \times 493.97$ KN
$= 740.95$ KN

$$\tau_v = \frac{V_u}{b \cdot d}$$

$$= \frac{740.95 \times 10^3}{1900 \times 400}$$

$\tau_v = 0.975$ N/mm^2

ii) Max. permissible shear stress [$\tau_{c\,max}$]

M_{20} grade concrete, from table 20 page 73, IS 456

$\tau_{c\,max} = 2.8$ N/mm^2

Comparing τ_v (0.975 N/mm^2) $< \tau_{c\,max} = 2.8$ N/mm^2

Hence safe

iii) Design shear strength of concrete [τ_c]

$$\frac{100 A_{st}}{b\,d} = \frac{100 \times 2804.03}{1900 \times 400} = 0.36$$

Page 73, IS 456, Table 19

$\tau_c = 0.412$ N/mm^2

comparing

τ_v (0.975 N/mm²) > τ_c (0.412 N/mm²)

provide the shear reinforcement in the form of vertical stirrups

provide 4 legged 8 mm ϕ V-S

$$A_{sv} = 4 \times \frac{\pi}{4}(8)^2 = 201.06 \text{ mm}^2$$

from page 72, IS 456

$$V_{us} = \frac{0.87 f_y \cdot A_{sv} \cdot d}{S_v} \quad - (3)$$

$$V_{us} = V_u - \tau_c bd$$

$$740.95 \times 10^3 - 0.412 \times 1900 \times 400 = \frac{0.87 \times 415 \times 201.06 \times 400}{S_v}$$

$$S_v = 69.96 \text{ mm}$$

Spacing is least of the following

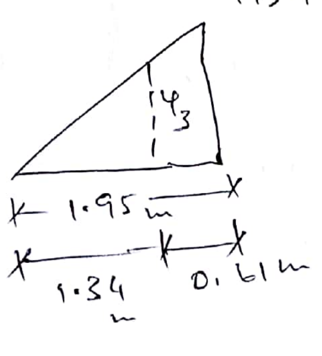
i) 0.75 x 400 = 300 mm

ii) 300 mm

provide 8 mm ϕ V.S 4 legged @ 60 mm c/c

* BC portion

Consider the triangle



Applying similar triangle concept

$$\frac{1.34}{y_3} = \frac{1.95}{493.97}$$

$$y_3 = 339.44 \text{ kN}$$

$V_u = 1.5 \times 339.44 = 509.16$ kN

i) Nominal shear stress [τ_v]

$$\tau_v = \frac{V_u}{b.d}$$

$$= \frac{509.16 \times 10^3}{1900 \times 400} = 0.669 \text{ N/mm}^2$$

ii) Max. permissible shear stress [τ_c max]

for M20 concrete, table 20, page 73, IS456

$\tau_{c\,max} = 2.8$ N/mm^2

comparing $\tau_v = 0.669$ N/mm^2 < $\tau_{c\,max} = 2.8$ N/mm^2

Hence safe

iii) Design shear strength of concrete [τ_c]

$$\frac{100\,A_{st}}{bd} = \frac{100 \times 2100.48}{1900 \times 400} = 0.276$$

from table 19, page 73, IS456

$\tau_c = 0.372$ N/mm^2

comparing τ_v (0.669 N/mm^2) > τ_c (0.372 N/mm^2)

Hence provide shear reinforcement in the form of VS

provide 4 Legged 8 mm ϕ bars

$$A_{sv} = 4 \times \frac{\pi}{4}(8)^2 = 201.06 \text{ mm}^2$$

from page 72, IS456

$$V_{us} = \frac{0.87 f_y \cdot A_{sv}\, d}{S_v} \quad -\ (4)$$

$V_{us} = V_u - \tau_c b d$ — (5)

(4) = (5)

$$509.16 \times 10^3 - 0.372 \times 1900 \times 400 = \frac{0.87 \times 415 \times 201.06 \times 400}{S_v}$$

$S_v = 128.23$ mm

Spacing is least of the following

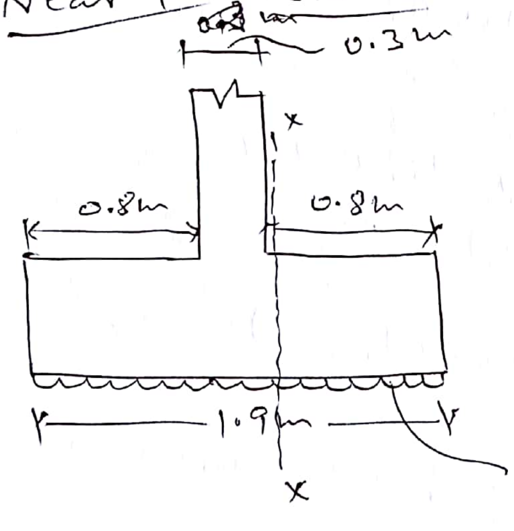
i) $0.75d = 0.75 \times 400 = 300$ mm

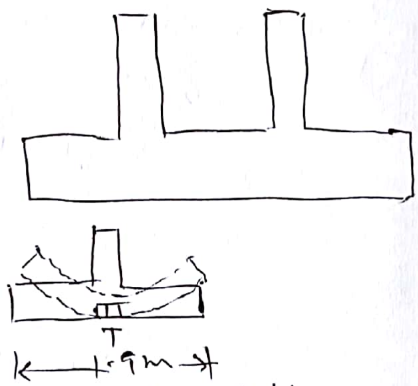
ii) 300 mm

provide 4L #8 mm ϕ V.S @ 125 mm c/c

Transreverse reinforcement

Near 1st Column





$q_{u_0} = 133.90$ kN/m² × 1m

$= 133.90$ kN/m

$$M_{xx} = 133.90 \times 0.8 \times \frac{0.8}{2} = 42.848 \text{ kNm}$$

$M_u = 1.5 \times 42.848$ kNm $= 64.27$ kNm

Area of Steel (Ast)

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

$$64.27 \times 10^6 = 0.87 \times 415 A_{st} \times 400 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 400}\right]$$

$A_{st} = 455.81 \ mm^2$

* Check for $A_{st\,min}$

$$A_{st\,min} = \frac{0.12}{100} \times 1000 \times 460 = 552 \ mm^2$$

A_{st} ($455.8 \ mm^2$) < $A_{st\,min}$ ($552 \ mm^2$)

provide $A_{st\,min} = 552 \ mm^2$

Assuming 12 mm dia bars

$$\text{Spacing} = \frac{\frac{\pi}{4}(12)^2}{552} \times 1000 = 204.8 \ mm$$

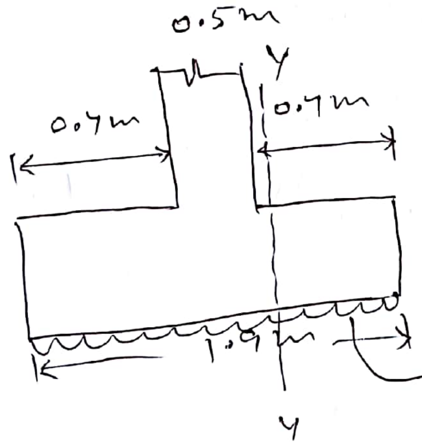
Spacing is least of the following

i) $3d = 3 \times 400 = 1200 \ mm$

ii) 300 mm

provide 12 mm ϕ bars @ 200 mm c/c

* Near column 2



$133.90 \ KN/m^2 \times 1m$

$= 133.90 \ KN/m$

$$M_{yy} = 133.90 \times 0.7 \times \frac{0.7}{2} = 32.8 \ KNm$$

$M_u = 1.5 \times 32.8 = 49.2 \ KNm$

* Area of Steel (A_{st})

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

$$49.2 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 400}\right]$$

$A_{st} = 346.91$ mm

* Check for $A_{st\,min}$

$$A_{st\,min} = \frac{0.12}{100} \times 1000 \times 460 = 552 \text{ mm}^2$$

A_{st} (346.91 mm²) < $A_{st\,min}$ (552 mm²)

provide $A_{st\,min} = 552$ mm²

Assuming 12 mm ϕ bars

$$\text{Spacing} = \frac{\frac{\pi}{4}(12)^2 \times 1000}{552} = 204.8 \text{ mm}$$

Spacing is least of the following

i) $3d = 3 \times 400 = 1200$ mm

ii) 300 mm

provide 12 mm ϕ bars @ 200 mm c/c

Scanned by CamScanner