10/1/16

Combined footing and Retaining Walls.

UNIT-I

A footing supporting more than a single column is called a combined footing.

** Conditions under which a combined footing is provided.

1) When soils have very low bearing capacity

2) When columns are so closely spaced that isolated footing may overlap.

3) Soil is not uniform and unequal settlements may occur.

4) When the columns are located along the periphery of the building and cannot be extended, one (or) two sides beyond the property the line.

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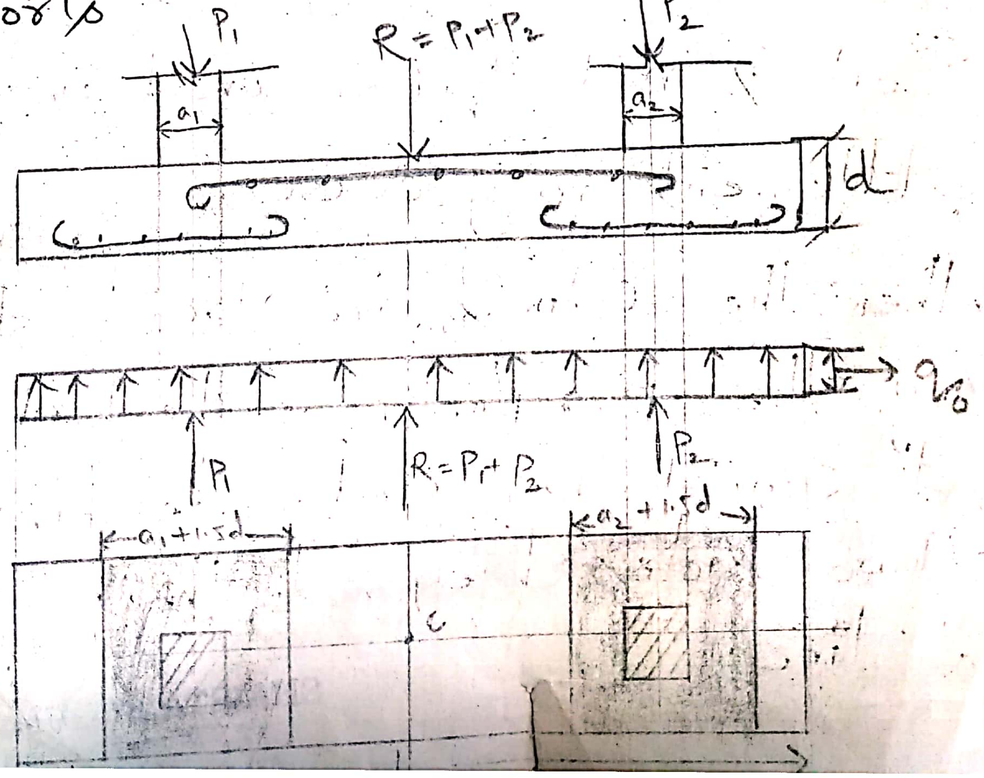
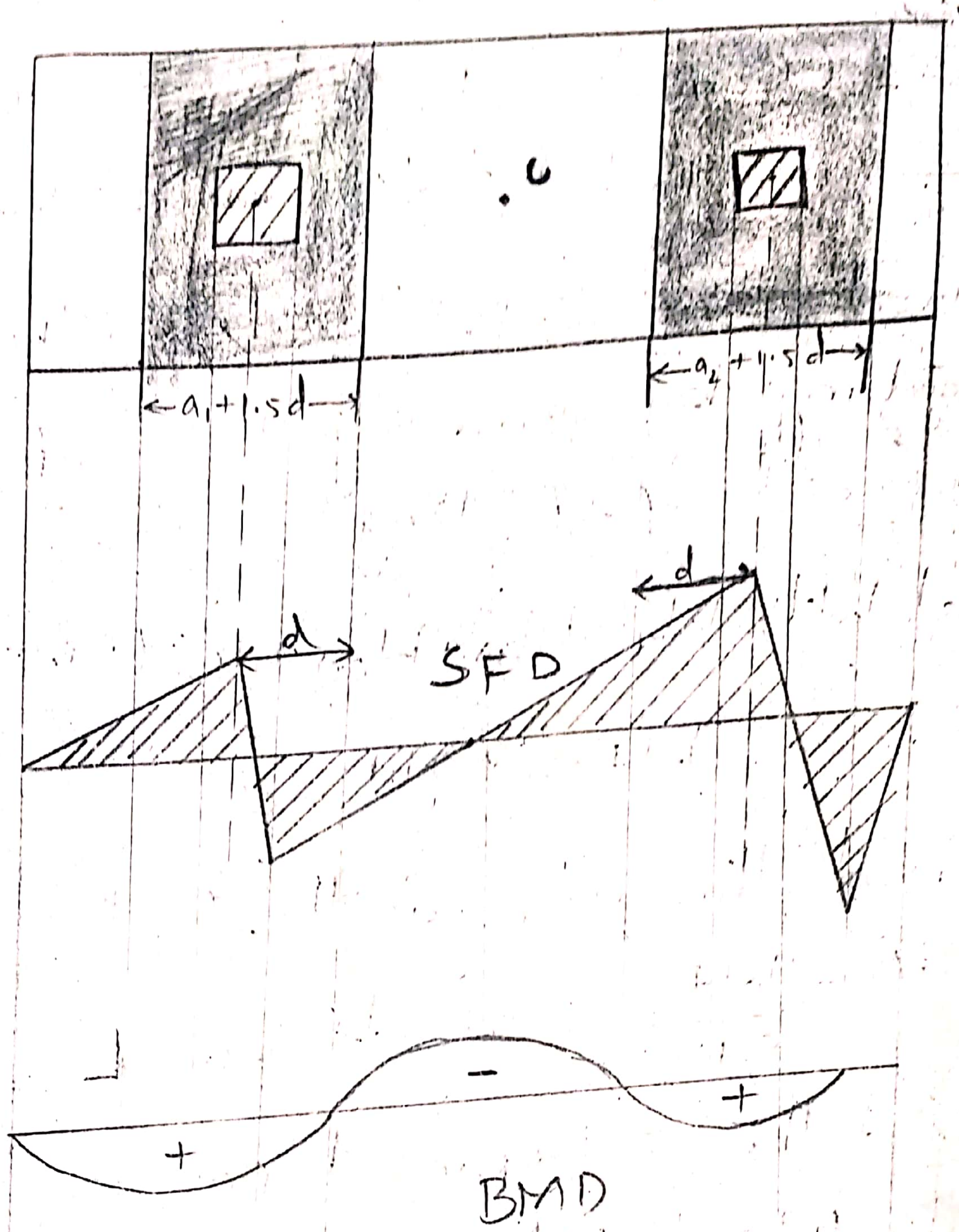
Role 1:- The geometry of the footing based, should be so selected that the centroid of the footing area coincides with the resultant of columns loads including moments. This will results in uniform distribution of soil pressure which is desirable to avoid possible tilting of footing.

* The shape may be trapezoidal (or) rectangular, depending on the relative magnitude of loads on the two columns which the footing supports





Load Transfer Mechanism.

Design Steps

Type-1 (Combined footing with equal loads)

1) Find the total loads on the footing by assuming 10% self weight of footing.

Total Wt, $W_t = W_{(1)} + W_{(2)} + 10\% (2W)$.

$$W_t = 2.2W$$

2) Find the area of the footing,

$A = \frac{W_t}{SBC}$ and decide 'L' and 'B'

3) Find the net upward pressure intensity, $w = \frac{W+W}{A}$

4) Find B.M at the face of the column and at mid point between the two columns.

5) Find depth (thickness) of the footing considering higher moment and find area of steel 'Ast' required.

5) Check for one-way shear (beam shear) and two-way shear (punching shear).

13-01-2016

Q) Design a combined rectangular footing for two RCC column carrying axial load of 1000 KN each. These columns are seperated by a distance 3m c/c width of footing is restricted to 2m, safe bearing capacity of soil is 200 KN/m². Draw neat sketches showing reinforcement details?

Soln:- Given data

$l = 3m\ c/c$, $B = 2m$

Adopt M_{20} concrete and Fe415

1) $W_1 = 1000 KN = W_2$ SBC = 200 KN/m²

$W_t = 2.2 \times 1000$ $\left(W_1 + W_2 + \frac{10}{100}(W_1+W_2)\right)$ self wt of footing

$= 2200 KN$

2) Area of footing $= \frac{W_t}{SBC} = \frac{2200}{200} = 11 m^2$

given, B = 2m

$A = L \times B$

$L = A/B = \frac{11}{2} = 5.5m$

3) NUPI, $W = \frac{W_1 + W_2}{A}$

$= \frac{2000}{2 \times 5.5}$

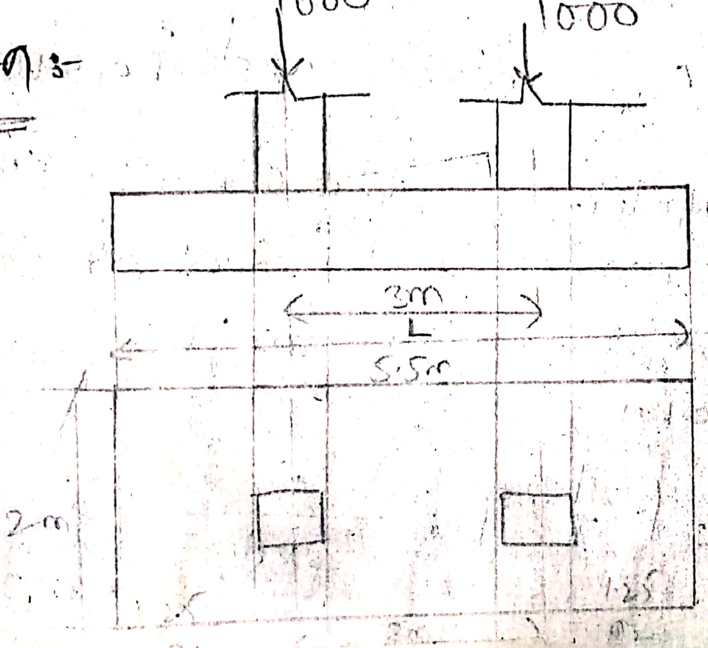
$W = 181.82 \ KN/m^2$

Note :- This pressure intensity is for 1m wide footing, ∴ the net upward pressure intensity for 2m wide footing

$= 181.82 \times 2$

$= 363.64 \ KN/m$.

) BM :-



Cantilever projections

$a_1 = a_2 = \frac{5.5 - 3}{2}$

$= 1.25m$

* Cantilever projections from face of Column is equal to

$a_c = 1.25 - 0.15$ (assume size (or) 300X300)

$a_c = 1.1 m$

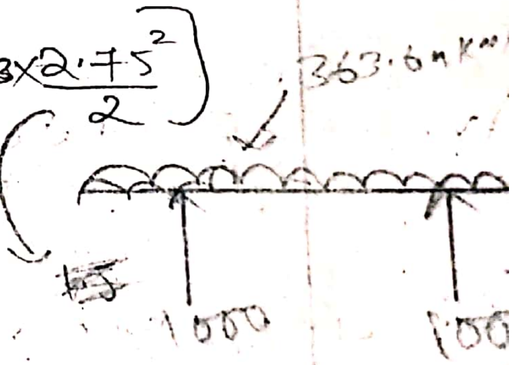
* Sagging B.M in cantilever portion

$$M_c = 1.5 \frac{w a_c^2}{2}$$

$$= \frac{1.5 \times 363.64 \times 1.1^2}{2} = 330 \text{ KN-m}$$

* Hogging B.M b/w two ends

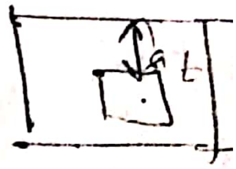
$$M_{hog} = 1.5\left[1000 \times \frac{3}{2} - 363.63 \times \frac{2.75^2}{2}\right]$$



$M_{hog} = 187.3$ KN-m

* Transverse Cant B.M $= 1.5 \frac{w a_t^2}{2}$

$$a_t = \frac{2 - 0.3}{2} = 0.85$$



* Cantilever projections from face of Column is equal to

$a_c = 1.25 - 0.15$ (assume size (or) 300×300)

$a_c = 1.1 m$

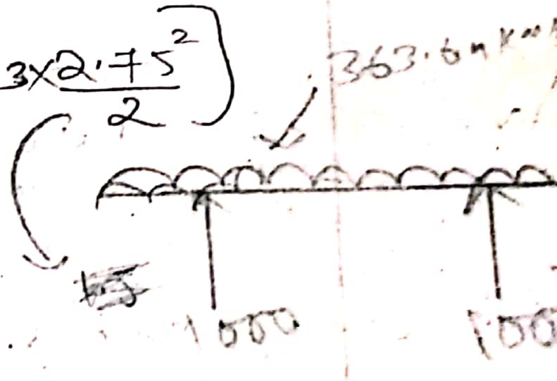
* Sagging B.M in cantilever portion

$$M_c = 1.5 \frac{w a_c^2}{2}$$

$$= \frac{1.5 \times 363.64 \times 1.1^2}{2} = 330 \text{ KN-m}$$

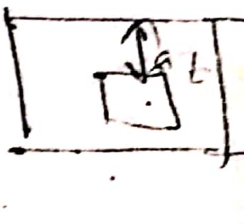
* Hogging B.M b/w two cols

$$M_{hog} = 1.5 \left[1000 \times \frac{3}{2} - 363.63 \times \frac{2.75^2}{2}\right]$$



$M_{hog} = 187.3$ KN-m

* Transverse Cant. B.M $= 1.5 \frac{w a_t^2}{2}$



$$a_t = \frac{2 - 0.3}{2} = 0.85$$

$M_{tv} = \frac{1.5 \times 363.64 \times 0.85^2}{2}$

$M_{tv} = ~~131.43~~$ ~~KN-m~~

197.02 KN-m

18-01-16

effective depth of footing

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

$330 \times 10^6 = 0.138 \times 20 \times 2000 \times d^2$

$d \simeq 250mm$

eff cover $d' = 50mm$

Overall depth $= D = 300mm$

Area of steel (At center zone of footing).

$187.4 \times 10^6 = 0.87 f_y A_{st_1} (d - 0.42 x_{umax})$

$187.4 \times 10^6 = 0.87 \times 415 \times A_{st_1} (250 - 0.42 \times 0.48 \times$ 250

$A_{st_1} = 2600mm^2$

.3x7

provide 20mm ϕ

Steel in

Mu

$$M_{tv} = \frac{1.5 \times 363.64 \times 0.85^2}{2}$$

~~$M_{tv} = 131.43$ KN-m~~

197.02 KN-m

18-01-16

effective depth of footing

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

$330 \times 10^6 = 0.138 \times 20 \times 2000 \times d^2$

$d \simeq 250mm$

eff cover $d' = 50mm$

Overall depth = $D = 300mm$

Area of steel (At center zone of footing).

$187.4 \times 10^6 = 0.87 f_y A_{st_1} (d - 0.42 x_{umax})$

$187.4 \times 10^6 = 0.87 \times 415 \times A_{st_1} (250 - 0.42 \times 0.48 \times 250)$

$A_{st_1} = 2600 mm^2$

.3×7

provide 20mm

steel

=165.3×4

Provide 20mm ϕ @ $\frac{\frac{\pi(20)^2}{4}}{2600} \times 1000$

= 120mm c/c

Steel in cantilever portion

$M_{cant} = 0.87 f_y A_{st_2} (d - 0.42 x_{u\,max})$

$330 \times 10^6 = 0.87 \times 415 \times A_{st_2} (250 - 0.42 \times 0.48 \times 250)$

$A_{st_2} = 4578 \text{ mm}^2$

provide 25mm ϕ

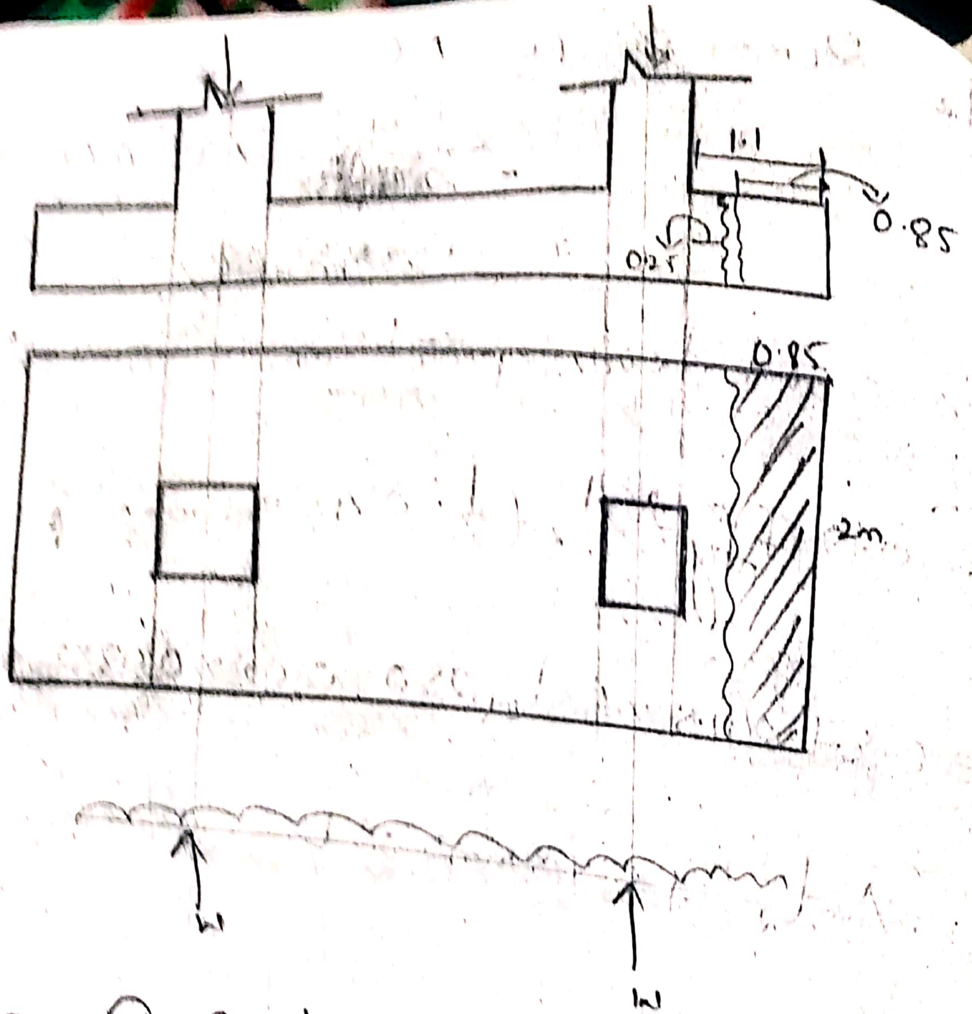
$= \frac{\frac{\pi(25)^2}{4}}{4578} \times 1000$

= 100mm c/c

Check for one-way shear

Typical section occurs at a distance 'd' = 250mm from the face of column.

Scanned with CamScanner



S.F @ Critical Section.

$V_u = 1.5 \times (w) \times \text{shaded area}$ (w → NUPI)

$= 1.5 \times 181.82 \times 0.85 \times 2$

$V_u = 463.64\ \text{kN}$

$\tau_{vu} = \dfrac{V_u}{bd} = \dfrac{463.64 \times 10^3}{2000 \times 250} = 0.927\ \text{N/mm}^2$

* Shear Stress (τ_c)

$\%\ \text{steel} = \dfrac{100 A_{st}}{b \times d}$

(11)

$= \frac{100 \times 4580}{2000 \times 250}$

$= 0.916\%$

* M_{20} grade conc & $P_t = 0.916\%$

table 19 (IS 456)

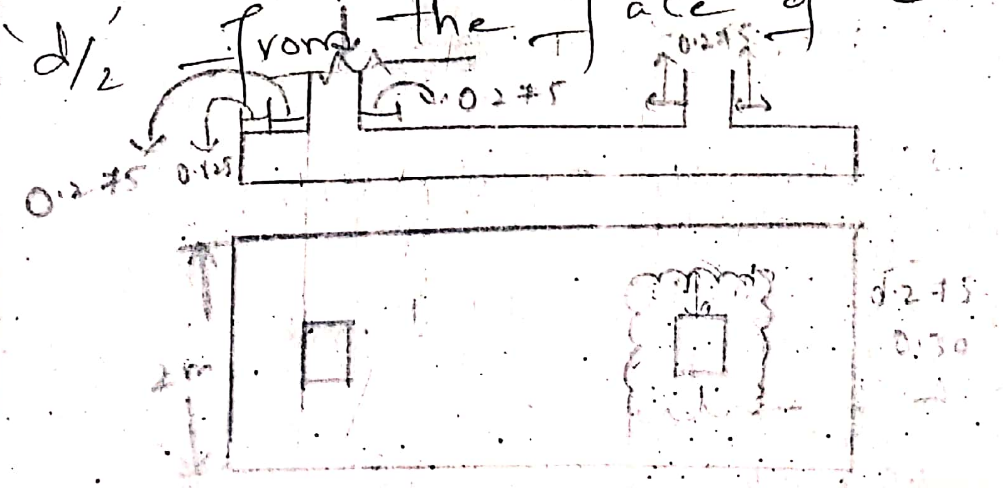
$\tau_c =$

$\tau_v > \tau_c$ (one way shear fail)

There fore, increase the depth of the footing to 550mm with the cover of 50mm. How ever the problem can continue with the same depth.

Check for two way shear: (punching shear).

* Critical section occurs at a dist 'd/2' from the face of column.



S.F. @ Critical section

$V_u = 1.5 [1800 - w \times \text{shaded area}]$

$= 1.5 [1800 - 181.82 \times 0.85 \times 0.85]$

$V_u = 1302.92 = 1303 \text{ kN}$

$\tau_{v_u} = \dfrac{1303 \times 10^3}{(4 \times 850) \times 550} = 0.7 \text{ N/mm}^2$

(4 × 850 → perimeter)

Per shear stress τ_c

per punching stress $= k_s \tau_c$

$k_s = 0.5 + \beta_c \ngtr 1$

$\beta_c = \dfrac{b}{d} = \dfrac{300}{300} = 1$

$K_s = 1.5 > 1 \Rightarrow K_s = 1$

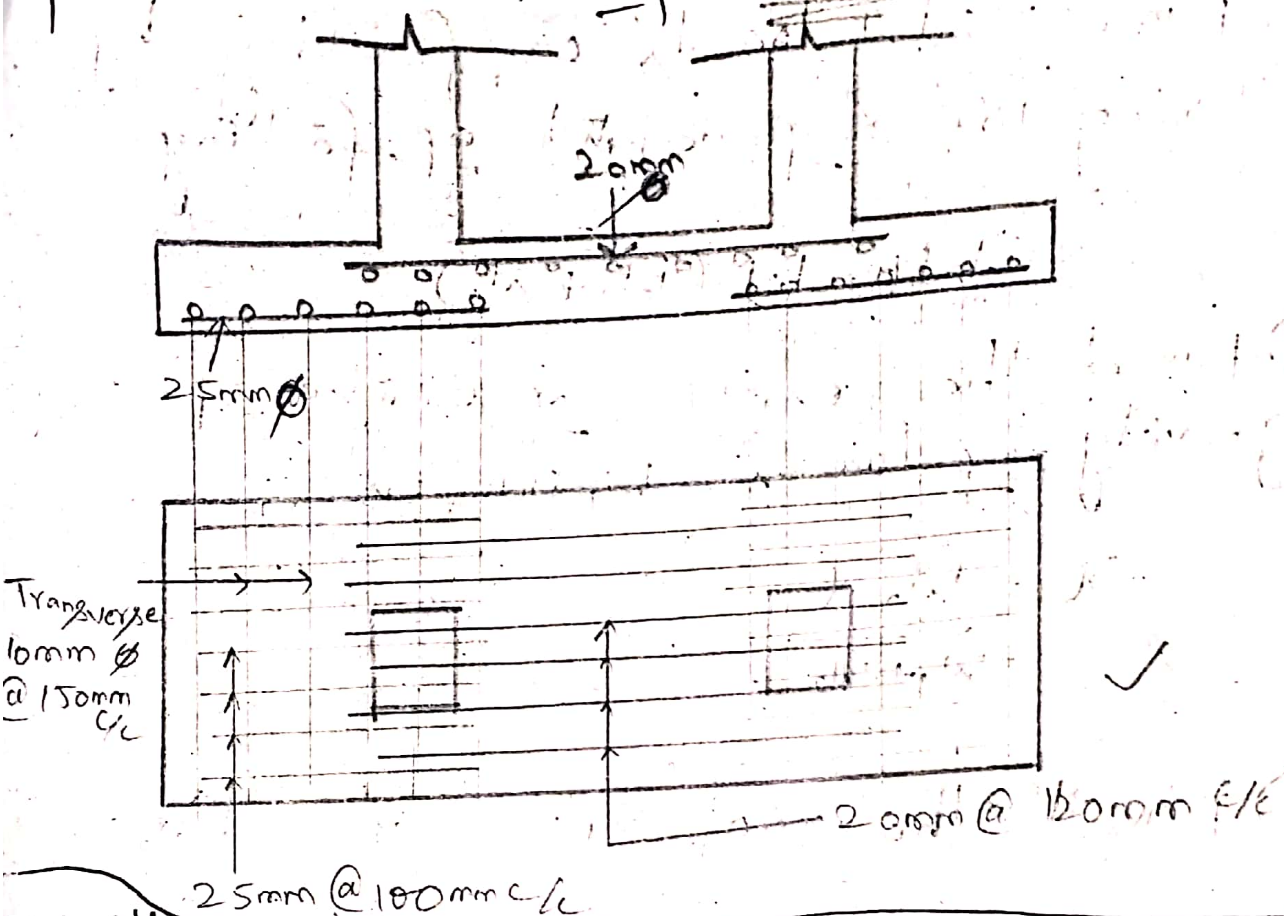
$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.12 \text{ N/mm}^2$

$\tau_c = 1 \times 1.12 = 1.12 \text{ N/mm}^2$

$\tau_{vu} = 0.7\ N/mm^2$

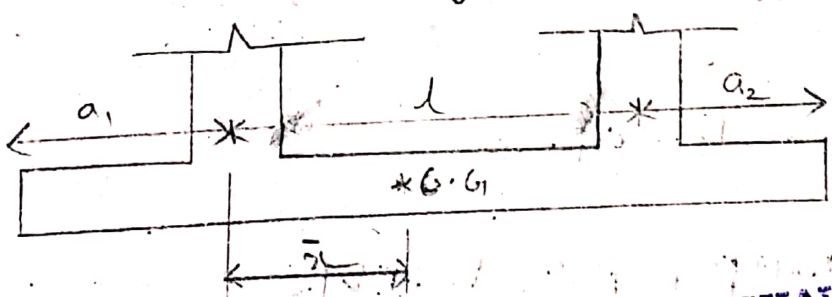
$\tau_{vu} < \tau_c$ ∴ Hence safe

provide thickness of 550mm.



20/01/16

II Combined footing with unequal loads.



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Steps:-

1) Given loads, W_1, W_2 (W_1, ~~W_2~~) (CL.H.S), SBC of soil, width of footing 'B' and 'l' (center to center dist b/w columns.

3) Find the area of footing

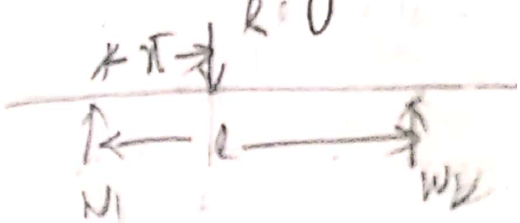
$$A = \frac{1.1(W_1 + W_2)}{SBC}$$ & decide L & B

2) Find total loads on footing by assuming 10% self weight of footing

$$W_t = 1.1\ (W_1 + W_2)$$

4) Find the position of centre of gravity of loads W_1 & W_2 using

$$\bar{x} = \frac{W_2\, l}{W_1 + W_2}$$



5) Find the projections 'a_1' and 'a_2'.

~~$a_1 + \bar{x} = l/2$~~ $\quad a_1 + \bar{x} = L/2$

~~$a_1 = l/2 - \bar{x}$~~ $\quad a_1 = L/2 - \bar{x}$

$$a_2 = L - a_1 - l$$

6) Find NUPI, $w = \frac{W_1 + W_2}{A}$.

7) Draw SFD and find position where SF change its sign i.e point of max B.M b/w columns.

8) Find cantilever moment, moment b/w columns and transverse moment. Determine the thickness of footing considering max B.M

9) Calculate area of steel for each moment seperately

10) Check for one way shear and two way shear and Draw neat sketches showing reinforcement details.

Q) Two RCC columns seperated by a distance of 2.9m, carry axial loads of intensity 1100 KN and 900 KN. SBC of soil is 180 KN/m² and width of footing is restricted 2m. Design a combined footing and draw neat sketches showing reinforcement details?

Soln :- Given, $W_1 = 1100\,KN$, $W_2 = 900\,KN$

$l = 2.9m$, $SBC = 180\,KN/m^2$

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

$f_y = 415\,N/mm^2$

Adopt
Size of Col = 400mm x 400mm

(1) $W_t = 1.1 (W_1 + W_2)$

$= 1.1 (1100 + 900)$

$= 2200 \text{ KN}$

(2) $\text{Area} = \frac{1.1 (W_1 + W_2)}{SBC} = \frac{2200}{180} = 12.22 \text{ m}^2$

Given, $B = 2m$

$A = L \times B \Rightarrow 12.22 = L \times 2$

$L = 6.11 m$

(3) $\bar{x} = \frac{W_2 d}{W_1 + W_2}$

$\bar{x} = \frac{900 \times 2.9}{2000} = 1.3 m$

(4) $a_1 = \frac{L}{2} - \bar{x} = \frac{6.1}{2} - 1.3 = 1.75 m$

$a_2 = (L - a_1 - d) = (6.1 - 1.75 - 2.9)$

$a_2 = 1.45 m$

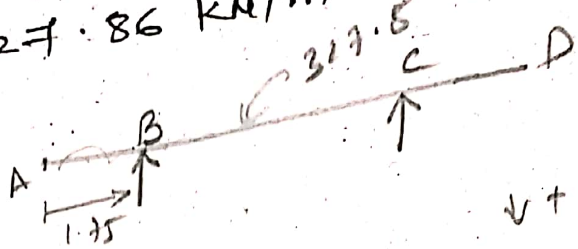
= 165.3 x 7

(5) $\omega = \frac{h l_1 + h l_2}{A} = \frac{2000}{12.2} = 163.93$ KN/m² (for 1m wide)

for 2m wide

⇒ 2 x 163.93

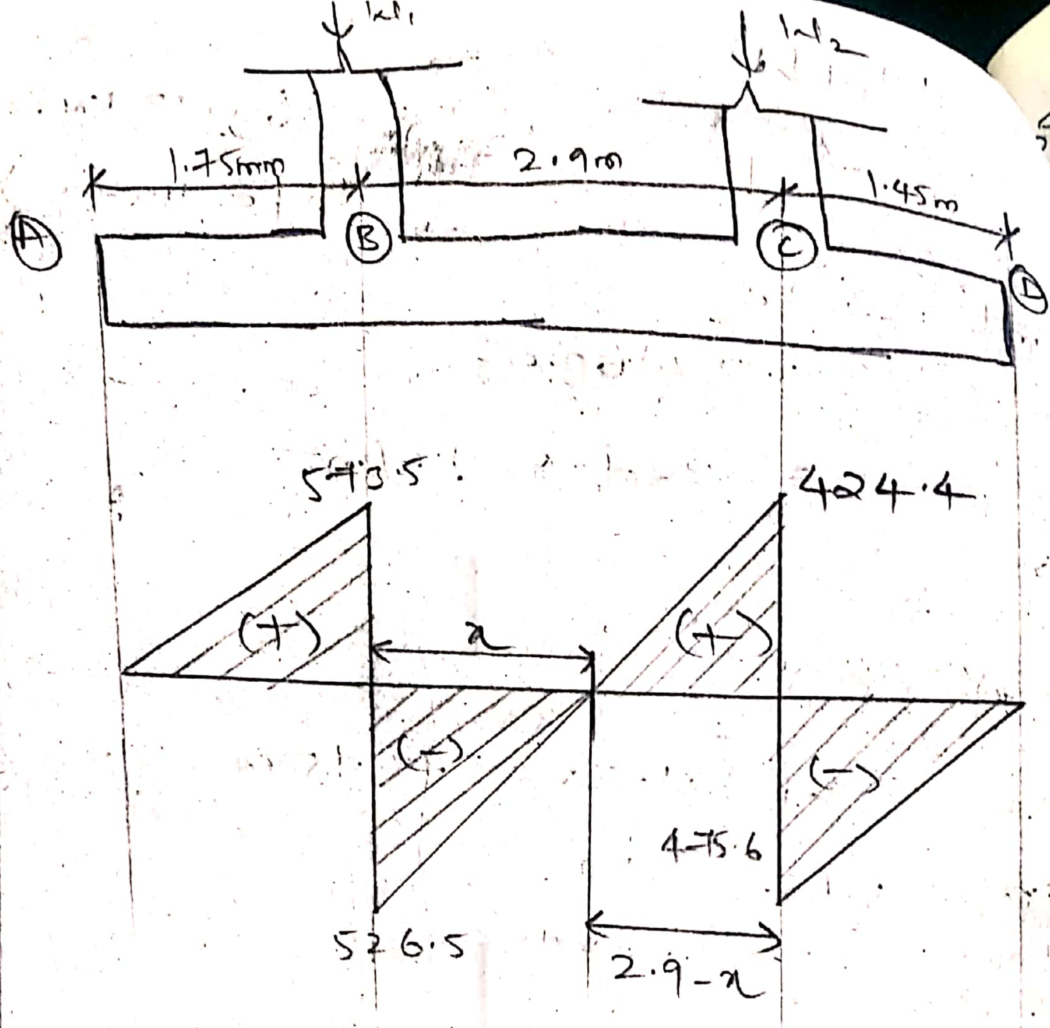
⇒ 327.86 KN/m²



(6) SFD

↓+ ↑−

* S.F @ A = 0
* S.F @ B = 327.8 x 1.75 = 573.75 KN
* S.F @ B_1 = 573.5 − 1100 = −526.5 KN
* S.F @ C = −526.5 + 327.82 x 2.9 = +424.4 KN
* SF @ C_1 = +424.4 − 900 = −475.6
* SF @ D = −475.6 + (327.8 x 1.45) ≈ 0.0 KN.



Position where SF changes sign.

from similar △le

$$\frac{x}{526.5} = \frac{2.9-x}{424.4}$$

$$x = 1.6m$$

7) Bending Moments

i) Cantilever moment

$$a_{1c} = 1.75 - \frac{0.4}{2} = 1.55m$$

$$M_{cant} = \frac{1.5 W a_c^2}{2}$$

$$= \frac{1.5 \times 327.86 \times 1.55^2}{2}$$

$$M_{cant} = 590.76 \text{ KN-m.}$$

M_{cant} (2) less because dist is less.

ii) Max B.M b/w two column

@ 'E' i.e @ $x = 1.6m$

$$M_{uE} = 1.5\left[327.86 \frac{(1.75+1.6)^2}{2} - 1100 \times 1.6\right]$$

$$M_{uE} = 119.55 \text{ KN-m}$$

iii) Transverse BM

$$a_{tr} = \frac{2 - 0.4}{2} = 0.8m$$

$$M_{tra} = 1.5 \times \frac{W a_t^2}{2} = \frac{1.5 \times 327.86 \times 0.8^2}{2}$$

$$M_{tr} = 157.5 \text{ KN-m.}$$

thickness of footing (Consider highest)

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

$590.76 \times 10^6 = 0.138 \times 20 \times 2000 \times d^2$

$d = 330mm$

Assume $d' = 50mm$

$D = 380mm$

* A_{st} at centre

$M_{hog} = 0.87 f_y A_{st_1} (d - 0.42 x_{umax})$

$A_{st_1} = 1261.47mm^2$

* provide 16mm ∅ @ $\frac{\frac{\pi(16)^2}{4}}{1261.47} \times 1000 = 159mm\ c/c$

* $M_{can} = 0.87 f_y A_{st_2} (d - 0.42 x_{umax})$

$A_{st_2} = 6210.25mm^2$

* Provide 25mm ∅ $\frac{\frac{\pi(25)^2}{4}}{6210.25} \times 1000$

$= 79mm\ c/c$

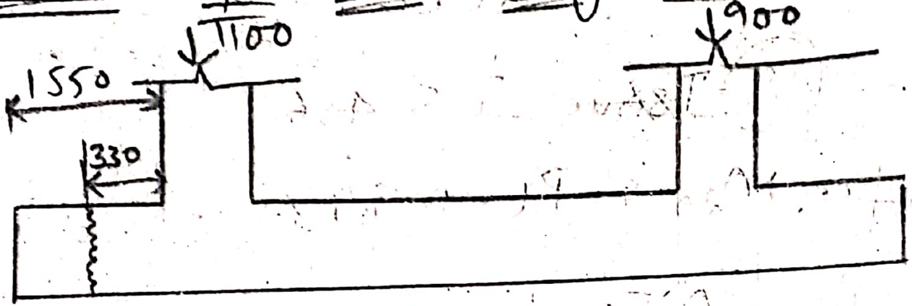
$M_{u} = 0.87 f_y A_{st_3} (d - 0.42 x_{umax})$

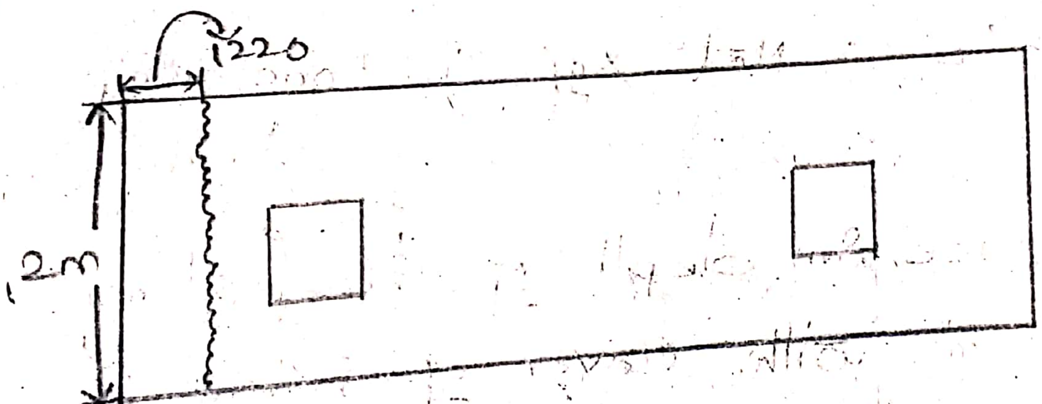
$A_{st_3} = 1346.581 mm^2$

* Provide 16mm ø @ $\frac{\frac{\pi (16)^2}{4} \times 1000}{1346.581}$

25/01/16 $= 149mm$ c/c

Check for one way shear.





* S.F causing failure $= 1.5$ [NUOPI × Area]

$= 1.5 \times 163.9 \times 2 \times 1.22$

S.F = 600kN

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* Shear stress $\tau_{vu} = \frac{V_u}{bd}$ $= \frac{600 \cdot 165.3 \times 10^3}{2000 \times}$

$\tau_{vu} = 0.9\ N/mm^2$

* Permissible stress (τ_c)

$P_t = \frac{100 A_{st}}{b \times d} = \frac{100 \times 6210.5}{2000 \times 300} = 0.94\%$

table (19) ~~Issue~~ IS 456

for M_{20} & $P_t = 0.94\%$

$\tau_c = 0.55\ N/mm^2$

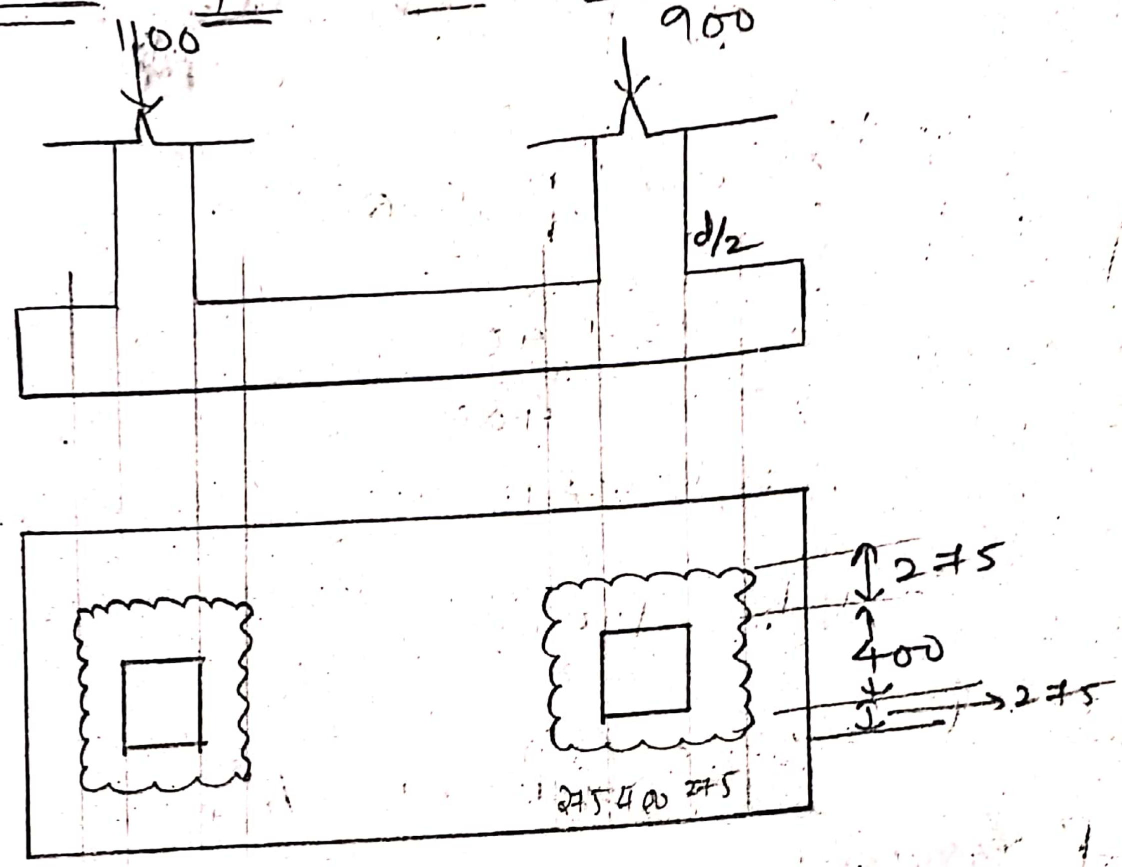
$\tau_v > \tau_c$ ∴ Not safe in one way shear.

* Increase in depth of the footing to 550mm with cover of 50mm.

However, the problem can be continued with the same depth

$= 165.3 \times 7$

Check for Two way Shear.



* Critical Section occurs at a dist d/2 i.e $\frac{500}{2}$ mm from the face of the column.

$V_u = 1.5\,[1100 - w \times \text{shaded area}]$

$= 1.5\,[1100 - 163.93 \times 0.95 \times 0.95]$

$= 1428.12$ KN

* $\tau_{vu} = \frac{V_u}{bd} = \frac{1428.2 \times 10^3}{(4 \times 950)\,550}$ (perimeter)

$= 0.68$ N/mm^2

* Permissible punching stress $= k_s \tau_c$

$k_s = 0.5 + \beta_c \ngtr 1.0$

$= 0.5 + \frac{400}{400}$

$= 1.5 \ngtr 1.0$

$k_s = 1.0$

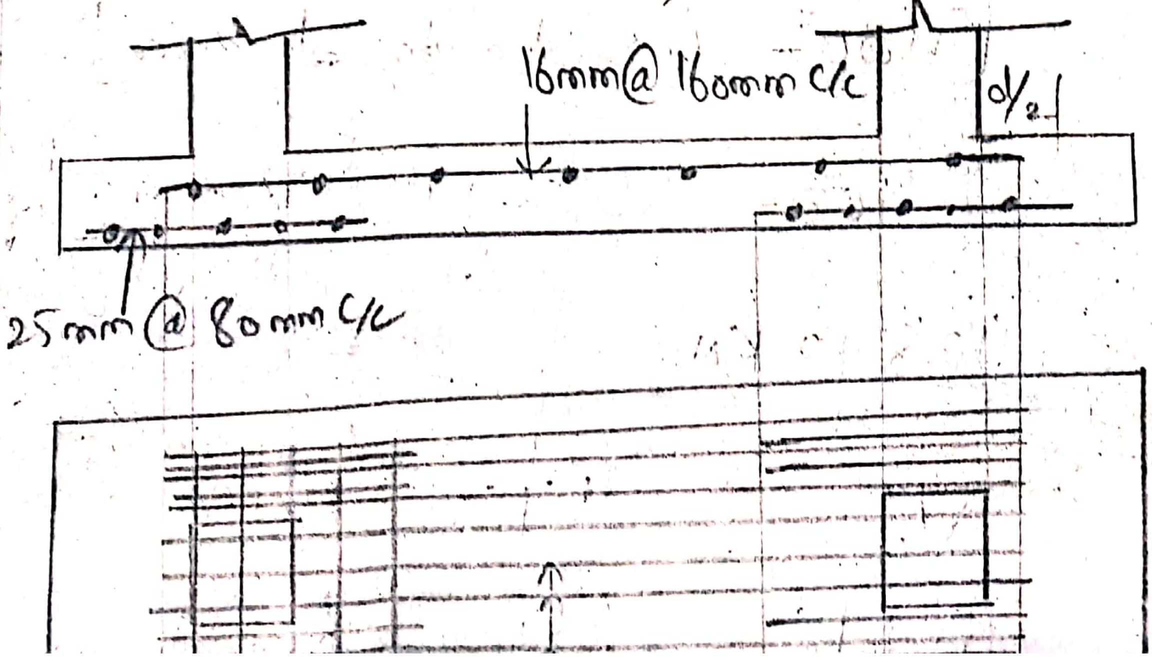
$\tau_c = 0.25\sqrt{f_{ck}} = 1.12\ N/mm^2$

* Permissible punching stress $= k_s \tau_c$

$= 1.12\ N/mm^2$

$\tau_v < \tau_c$: (Safe in two way shear).

(diagram Abbias)



Trapezoidal footing

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(Q) Two RCC- Column Carrying axial load of 1200 KN and 1000 KN each are seperated by distance of 3.5m c/c The projections on either sides are limited to 1.2m and 0.8m respectively. SBC of soil may be taken as 250 KN/m². Adopting suitable size of column and grades of steel and concrete, design a footing?

Soln: Given $W_1 = 1200$ KN, $W_2 = 1000$ KN

$L = 3.5$ m c/c
$a_1 = 1.2$m
$a_2 = 0.8$m

SBC = 250 KN/m²
$f_{ck} = 20$ N/mm²
$f_y = 415$ N/mm²

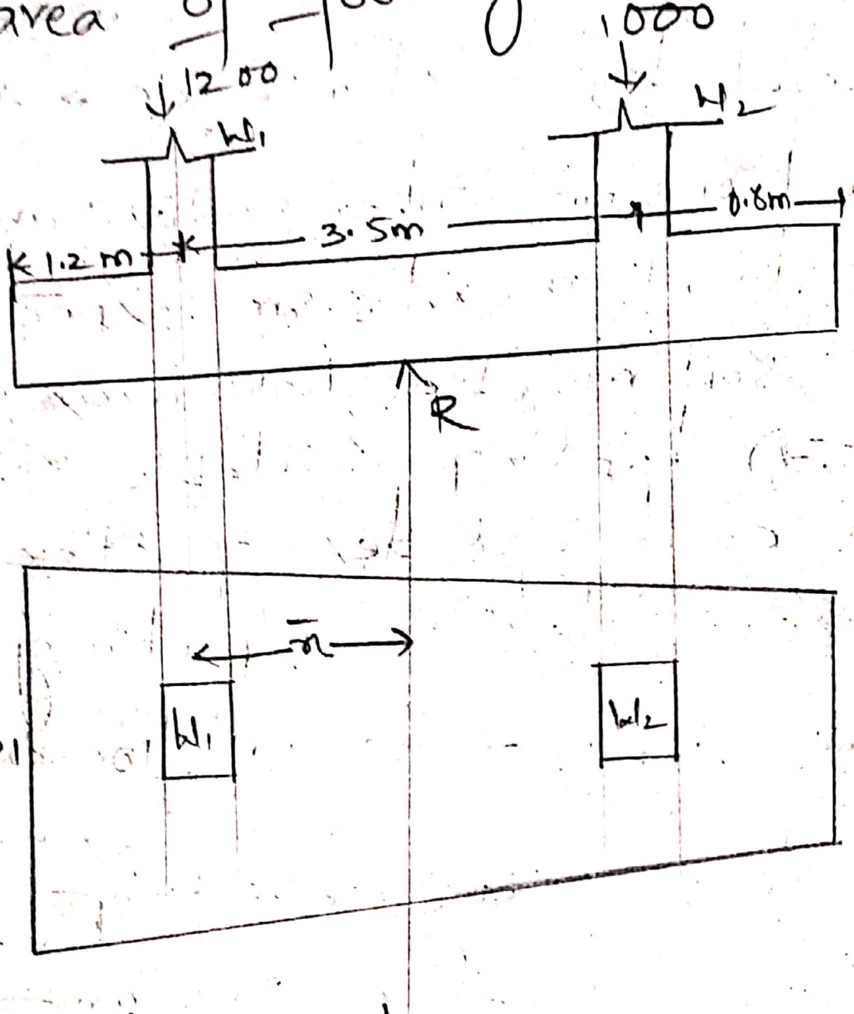
Size of col : 300mm X 300mm

* Since the lengths of projections is restricted, design a trapezoidal footing.

* let B_1, B_2 be the width of footing at left and right.

* B_1 & B_2 be ...

Such that the resultant of loads of columns must coincide with ... of area of footing



① Total weight

$$W_t = W_1 + W_2 + 10\%(W_1 + W_2)$$

$$= 1200 + 1000 + 0.1(2200)$$

$$= 2420 \text{ KN.}$$

② Area of footing $= \frac{W_t}{SBC} = \frac{2420}{250} = 9.68 \, m^2$

Q3) length of footing = 1·2+3·5+0·8 = 5·5m

$$Area = L \times \left[\frac{B_1 + B_2}{2}\right]$$

$$9.68 = \frac{5.5}{2}\left[B_1 + B_2\right]$$

$$B_1 + B_2 = 3.52 \longrightarrow (1)$$

* Dist of C.G $\bar{x}$ (of wt's) from L.H.S

$$\bar{x} = \frac{W_2 \, l}{W_1 + W_2} = 1.59m$$

* C.g of area (trap)

(from L.H.S)

$$\bar{x} = \left(\frac{2a+b}{a+b}\right)\times\frac{h}{3}$$

$$\bar{X} = \left(\frac{2B_1 + B_2}{B_1 + B_2}\right) \times \frac{5.5}{3}$$

from (R.H.S)

$$\bar{x} = \frac{(2b+a)}{(a+b)} \times h/3$$

* equating with $(1.2 + \bar{x})$

$$\left(\frac{2B_1 + B_2}{B_1 + B_2}\right)\frac{5.5}{3} = 1.2 + 1.59$$

$$11B_1 + 5.5B_2 = 8.37 B_1 + 8.37 B_2$$

$$2.63 B_1 - 2.87 B_2 = 0 \longrightarrow (2)$$

$B_1 = 1.84m$ $B_2 = 1.68m$

4) NUPI, $\omega = \frac{W_1 + W_2}{A}$

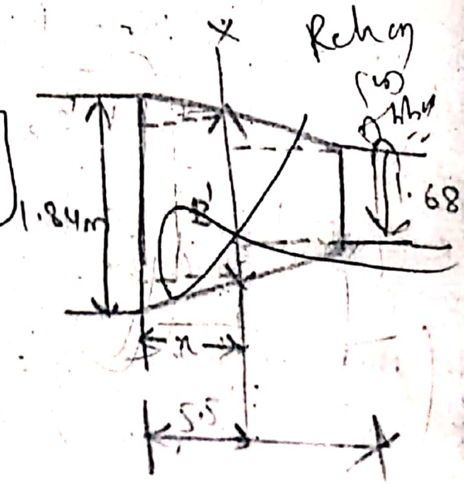
$= \frac{1200 + 1000}{9.68}$

$\omega = 227 \ kN/m^2$

5) to find BM at diff section

* Since, the width of footing is varying, an expression is to be formulated for BM.
* Width of footing at a distance 'x' from L.H.S.

$B' = 1.84 - \left[(1.84 - 1.68)\frac{x}{5.5}\right]$



$A_x = \int_0^x B' \, dx$

$= 1.84 - \left[0.16 \times \frac{x}{5.5}\right]$

$y = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)$

$= 1.84 - [0.029 \times x]$

$= 1.84 - [0.029x]$

$$A_x = \int_0^x B'x \, dx$$

$$= \int_0^x (1.84 - 0.029x) \, dx$$

$$A_x = 1.84x - \frac{0.029x^2}{2}$$

(iii) S.F @ dist 'x' from left edge

$$V_x = NOPI (A_x)$$

$$V_x = 227\left[1.84x - \frac{0.029x^2}{2}\right]$$

$$= 417.68x - 3.29x^2$$

iv) B.M

$$V_x = \frac{dM_x}{dx}$$

$$dM_x = V_x \cdot dx$$

$$\int_0^x dM_x = \int_0^x V_x \, dx$$

$$M_x = \int_0^x (417.6x - 3.29x^2)\,dx$$

$$M_x = \frac{417.6x^2}{2} - \frac{3.29x^3}{3}$$

$$M_x = 208.84x^2 - 1.1x^3$$

* Max. B.M in cant portion @ B

$x = 1.2 - 0.15$
$= 1.05$

* $M_B = 208.84(1.05)^2 - 1.1(1.05)^3$

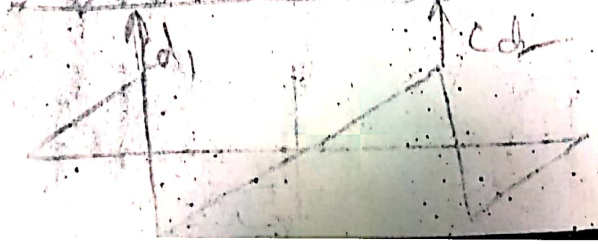
$M_B = 229.03$ kN-m.

$M_B = 1.5 \times 229.03$
$= 343.5$ kN-m (sagging)

* Max hogging B.M occurs where SF changes sign (or S.F = 0)

$V_x = 0$

~~417~~ $417.68x - 3.29x^2 - 1200 = 0$



$x = 2.94\,mm$

Subst $x = 2.94\,mm$ in M_x eqⁿ

$M_{max}(E) = 208.84\,(2.94)^2 - 1.1\,(2.94)^3 + 1200\,(2.94 - 1.2)$

$M_E = -310.82\,kN\text{-}m$

$M_{uE} = 1.5\,(-310.82)$
$= -466.2\,kN\text{-}m$ (hogging B.M)

Step 6:

Thickness of footing [for max B.M]

$466.2 \times 10^6 = 0.138 \times f_{ck}\, B' d^2$

$B' = 1.84 - 0.029\,(x)$

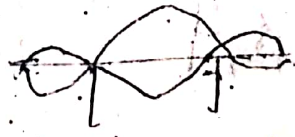
$1.84 - 0.029\,(2.94) = 1.75$

$B' = 1.75\,m$

$466.2 \times 10^6 = 0.138 \times 20 \times 1750\, d^2$

$d = 310.7\,mm$

$D = 360\,mm$



Step-I: Area of steel

1. Area of steel in cantilever (sagging ...)

$M_B = 0.87 f_y A_{st_1} (d - 0.42 x_{u\,max})$

$393.5 \times 10^6 = 0.87 \times 415 \times A_{st_1} (310.7 - 0.42 \times 0.48 \times 310.7)$

$A_{st_1} = 3935.2 \, mm^2$

provide 25mm ∅ bars

$$\frac{\frac{\pi (25)^2}{4}}{3835.2} \times 1000$$

$= 126 \, mm \; c/c$

* Area of steel in hogging B.M.

$M_E = 0.87 f_y A_{st_2} (d - 0.42 x_{u\,max})$

$+466.2 \times 10^6 = 0.87 \times 415 \times A_{st_2} (310.7 - 0.42 \times 0.48 \times 310.7)$

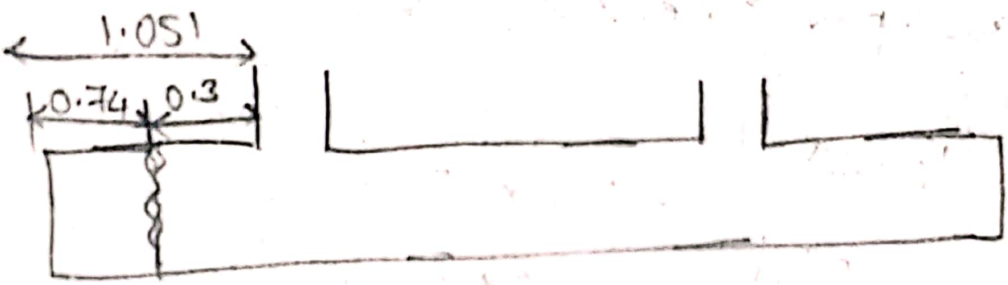
$A_{st_2} = 5205 \, mm^2$

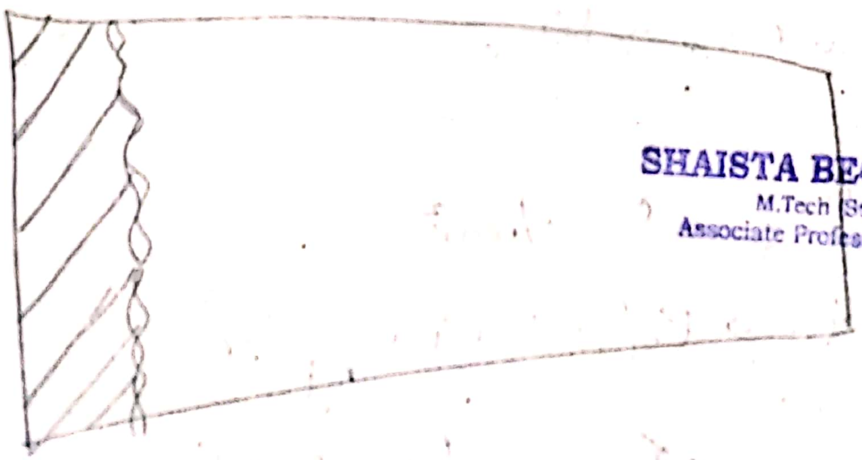
provide 28 mm Ø

$$\frac{\frac{\pi(28)^2}{4}}{5205} \times 1000 = 117 \text{ mm c/c}$$

Step 8 :- Check for one way shear.

* Critical section occur @ dist d = 310mm from the face of column.





$V_x = (417.68x - 3.27x^2)\,1.5$

$V_x = (417.68(0.74) - 3.27(0.74)^2)\,1.5$

$V_x = 461.92$ kN

$\tau_{vu} = \frac{V_{ux}}{B''d}$ $B'' = 1.84 - 0.02\ 75.3\ldots$

$= 1.84 - 0.029$

$B'' = 1.81m$

* $\tau_{vu} = \frac{460.92 \times 10^3}{1810 \times 310}$

$\tau_{vu} = 0.82 N/mm^2$

* Max. per. Shear Stress:

$P_t = \frac{100 A_{st}}{B''d} = \frac{100 \times 3835}{1810 \times 310} = 0.68\%$

for P_t 0.68% & M_{20}

table

$\tau_c = 0.5 N/mm^2$

$\tau_v > \tau_c$ (Section fails)

* Section fails for one-way shear, hence increase the thickness of footing to 500mm with a cover of 50mm.

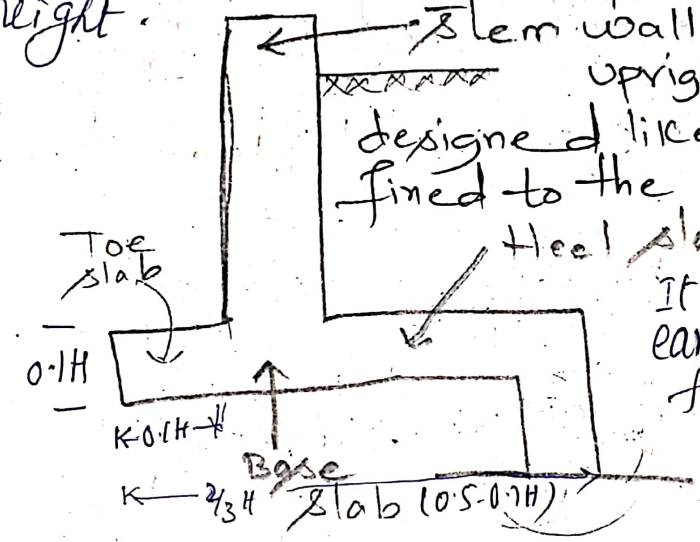
However ~~continue~~ continue the with the ~~same~~
thickness (d = 500 mm).

<u>01/02/16</u>

* Walls which retain soil at different levels on two of its sides are known as retaining walls.

1) <u>Cantilever Retaining Walls</u>:- These walls are provided upto a height of ~~6m~~ height. (6m)

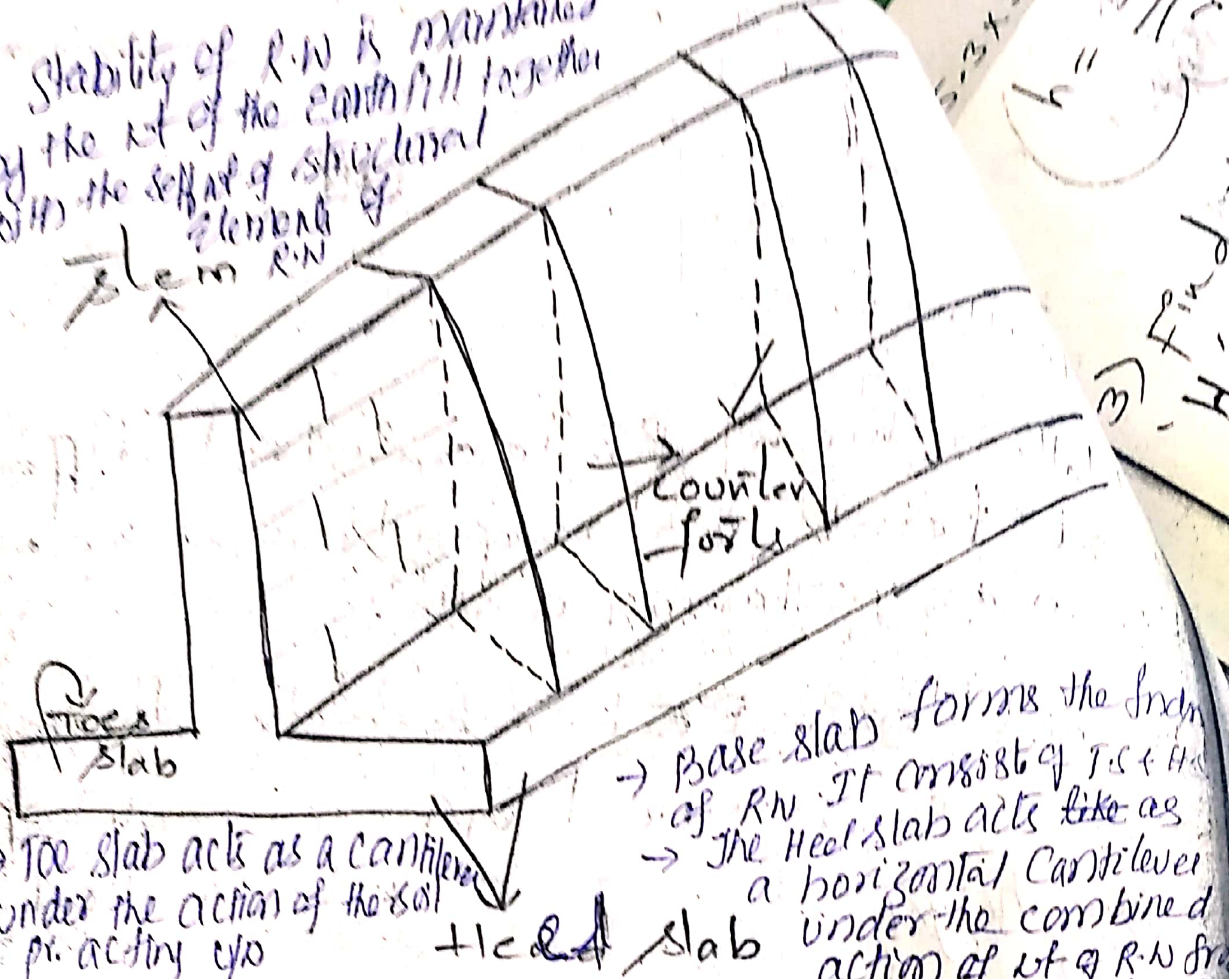
* The vertical stem wall (stem) (or) upright slab) is designed like a cantiliver fixed to the base slab.



2) <u>Counter fort Retaining Walls</u>:-

* These walls are provided for a height above 6m, the vertical (stem) is designed like a continous slab supported on counter fort.

• Stability of R.W is maintained by the wt of the earth fill together with the self wt of structural elements of R.W



→ Base slab forms the foundation of R.W. It consists of T.S & H.S.

→ The Heel slab acts as a horizontal cantilever under the combined action of wt of R.W from top & soil pr acting from soffit.

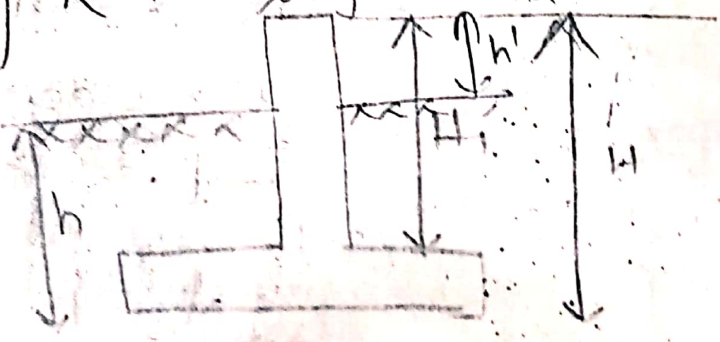
→ Toe slab acts as a cantilever under the action of the soil pr. acting u/p

Design Of Cantilever Retaining Walls

Steps for the design of stem.

1) Given height of back fill, SBC of soil, unit weight of soil, and angle of repose (ø).

2) Find the depth of foundation (h) using Rankine's formula.



$$h = \frac{SBC}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

3) Find the total height of the wall 'H₁' and determine BM using

$$M_u = \frac{1.5 K_a \gamma H_1^3}{6}$$

where K_a = Co-efficient of active earth pressure

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

<u>Note</u>:- $K_a = \cos\beta \cdot \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$

(When the back fill is slopping at an angle β).

4) Find the thickness of stem using the expression $M_u = 0.138 f_{ck} b d^2$

and determine area of steel using

5) provide distribution steel in horizontal direction equal to

0.12% (or) 0.15% cross sectional depending tor (or) mild steel

Steps for design of base slab

① Adopt width of base slab equal to 0.5H - 0.6H

② provide Toe projection $= \frac{1}{3}$ width of base slab

③ Find eccentricity 'e' of the resultant from the mid point of the base slab.

$$e = \frac{b}{2} - \bar{x}$$

where $\bar{x} = \frac{\text{Resultant moment}}{\Sigma W}$

$$\bar{x} = \frac{\Sigma M_{Resisting} - \Sigma M_{overturning}}{\Sigma W}$$

④ Determine the pressure intensities at the edge of Toe and heel slab. $P_{max} = W/b \left(1 + \frac{6e}{b}\right)$

$$P_{min} = \frac{W}{b}\left(1 - \frac{6e}{b}\right)$$

5) Calculate moments in Toe slabs and heel slab assuming them fix to the stem. The moment is calculated considering the upward pressure intensities.

6) Determine the thickness of Toe slab and heel slab and find the corresponding area steel.

7) Check for stability (IS 456 pg 33)

(a) overturning

$$FOS = \frac{\text{Restoring Moment}}{\text{overturning Moment}} \geq 1.4$$

$$= \frac{0.9 \times M_{rest}}{\text{overturning moment}} \geq 1.4$$

(b) Sliding

$$FOS = \frac{\text{frictional force}}{\text{sliding force}} \geq 1.4$$

$$= \frac{0.9\,\mu W}{\frac{1}{2} K_a \gamma H^2} \geq 1.4$$

Q1) A Cantilever retaining wall retaines level back fill upto 4.5m from ground level. The backfill is subjected to a traffic load of 10KN/m², angle of repose of soil is 30° and its unit weight is 18KN/m³. SBC of soil is 150 KN/m². Design

(a) Stem (or) upright

(b) toe & heel slab

(c) If co-efficient of friction μ b/w soil and concrete is 0.5, Check for sliding and if required provide a key.

Soln: Given data

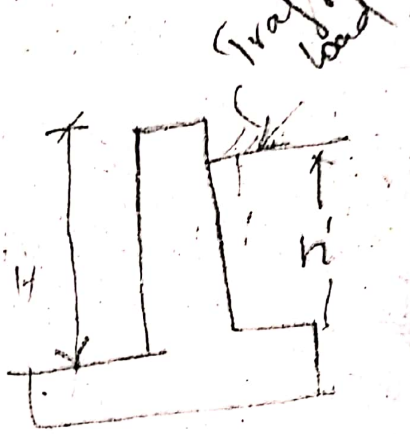
$h' = 4.5\,m$ (backfill)

traffic load $= 10\ KN/m^2$

$SBC = 150\ KN/m^2$

$\gamma = 18\,KN/m^3$, $\phi = 30^\circ$

(2) $h = \frac{SBC}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$



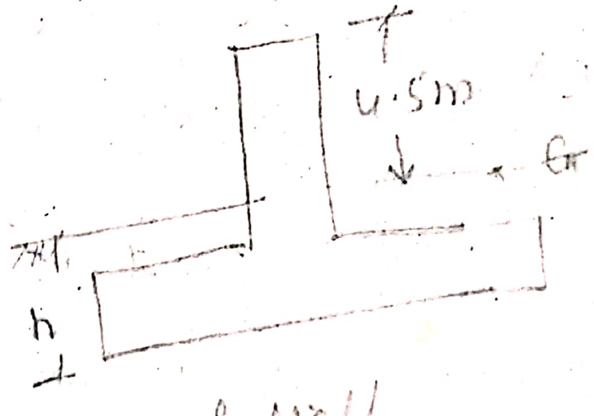
$h = 0.92m$

~~(1)~~ total height and B.M

(3) $M_u = \frac{1.5\,K_a\,\gamma\,H^3}{6}$

$H = h' + h +$ equivalent height due to traffic load (h_2)

$h_2 = \frac{\text{traffic intensity}}{\text{unit wt of soil}}$



$h_2 = 0.55m$

$H = h' + h + h_2$

$H = 5.97m$

Ht of wall = Backfill ht + ... − thick of ...

$K_a = 0.33$

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$M_u = 1.5 \times 0.33 \times 18 \times 5.97^2 / 6$

~~= 297.24~~

$M_u = 315.97$ KN-m

(4) $M_u = 2.76\, bd^2$

$$d = \sqrt{\frac{320 \times 10^6}{0.138 \times 20 \times 1000}}$$

$d = 340.5$ mm

$d' = 60$ mm

$D =$ ~~60~~ 400 mm

Area of steel (A_{st})

$M_u = 0.87 f_y A_{st} (d - 0.42 x_{umax})$

$A_{st} = 3264\ mm^2$

Spacing provide 25 mm ø

$$= \frac{\frac{\pi \times 25^2}{4}}{3264} \times 1000 = 150mm\ c/c$$

3/02/16

* Dist. steel (using for steel)

= 0.12% of gross c/c area

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

$$= 480mm^2$$

* provide 8mmϕ bars @ $\frac{\frac{\pi (8)^2}{4}}{480} \times 1000$

$$= 100\ mm\ c/c$$

Design of base slab

1) width of base slab, b = 0.5H to 0.6H

= 0.6 (5.97)

≃ 3.6m

2) width of toe slab = $\frac{1}{3}b$

$$= \frac{1}{3} \times 3.6 \Rightarrow 1.2m$$

$$= \frac{\frac{\pi \times 25^2}{4}}{3264} \times 1000 = 150mm \text{ c/c}$$

3/02/16

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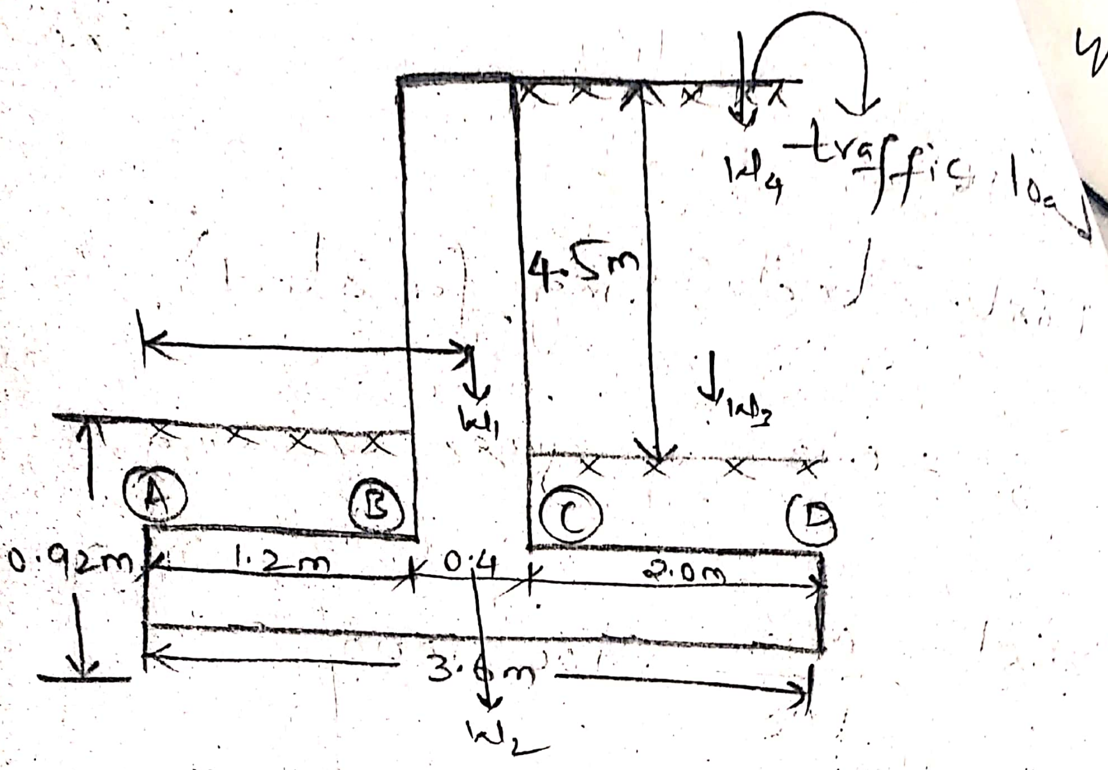
= 0.6 (5.97)

≃ 3.6m

2) width of toe slab = $\frac{1}{3}$ b.

$= \frac{1}{3} \times 3.6 \Rightarrow 1.2m$

(iii) Width of heel slab = 3.6 − 0.4 − 1.2
= 2.0 m.



* To find eccentricity (e)

Restoring moment: Assume base slab thickness = 0.4m

Member	load	Dist form A	Moment about (A) (KN-m).
stem	(0.4×1×5.57×25) = 55.7	1.4m	78.0
Base slab	(0.4×3.6×1×25) = 36	3.6/2 = 1.8m	64.8
Back fill	4.5×2×1×18 = 162	2.6	421.2
Traffic load	(1×2.0)×10 = 20	2.6	52

$\Sigma M_R =$ ~~626.4 KN-m~~ 616 KN-m

$\Sigma W =$ ~~2770.7 KN~~ 273.7 KN

* dist at which resultant of loads ~~intersects~~ intercepts the base slab from 'A',

ΣM_o = over turning moment of stem = 320 KN-m

$$\bar{x} = \frac{\Sigma M_R - \Sigma M_o}{\Sigma W}$$

$$\bar{x} = \frac{616 - 320}{273.7}$$

$\bar{x} = 1.08m$ from A

* eccentricity 'e' $= b/2 - \bar{x}$

$$= \frac{3.0}{2} - 1.08$$

e = ~~0.72m~~ 0.42m

* e < b/6 [Unsafe for over turning]

~~Hence increase base width to~~

Hence reduce b to 0.5H → 0.5 × ~~0.57~~ 5.97

b ≃ 3.0m

Continue the problem with revised b = 3m

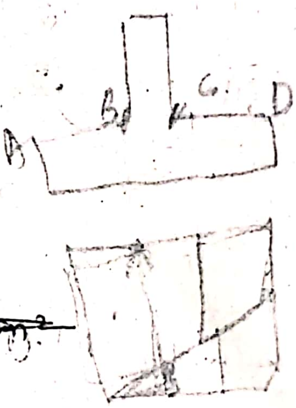
) pressure distribution.

$$P_{max} = \frac{W}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{273.7}{3.6}\left(1 + \frac{6(0.42)}{3.6}\right)$$

= ~~164.7 kN/m²~~ ~~164.86 kN-m~~

164.2 kN-m



$$P_{min} = \frac{W}{b}\left(1 - \frac{6e}{b}\right)$$

P_{min} = ~~72.67 kN/m²~~

18.2 kN/m²

* pressure intensity @ the junction of toe slab & stem (@ B)

$$P_B = 164.2 - \left\{164.2 - 18.2\right\}\left(\frac{1.0}{3.0}\right)$$

$P_B = 115.53$ kN/m²

$\bar{a}$ l (C)

* $164.2 - (164.2 - 18.2)\frac{1.4}{3}$

$P_c = 96.06$ KN/m²

* BM in the toe slab is

$M_u = 1.5\{$load in trapezium × centroidal dist from (A) $\}$

$= 1.5\left\{\underbrace{\left(\frac{164.2+115.52}{2}\right)\times 1}_{\text{load}} \times \underbrace{\left(\frac{2\times164.2+115.52}{164.2+115.2}\times\frac{1.0}{3}\right)}_{\bar{x}\text{ (cdist)}}\right\}$

$\frac{2a+b}{a+b}\times\frac{h}{3}$

* $M_u = 111$ KN-m

* <u>Thickness of toe slab (base slab)</u>.

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

$111\times10^6 = 0.138\times20\times1000\times d^2$

$d = 200$ mm $d' = 50$ mm

D = 2...

* $M_u = 0.87 f_y A_{st} (d - ...)$

$A_{st} = 1925\ mm^2$

* provide 20mm ϕ dia bar

$$\frac{\frac{\pi (20)^2}{4}}{1925} \times 1000 \approx 160mm\ c/c$$

Dist steel = 0.12%

* for heel slab

~~Mu~~ B.M

$M_u = 1.5 \{ \text{load in trapezium} \times \text{Centroidal dist from A} \}$

$$= 1.5 \left\{ \left(\frac{96.06 + 18.2}{2} \right) \times 1 \times \left(\frac{1.6}{3} \right) \left(\frac{2 \times 96.06 + 18.2}{96.06 + 18.2} \right) \right\}$$

$M_u = 134.6\ kN\cdot m$

$M_u = 2.76\, bd^2$

$134.6 \times 10^6 = 2.76 \times 1000 \times d^2$

$d = 220mm$ $d' = 50mm$

$D = 270\,mm$

$M_u = 0.87 f_y A_{st} (d - 0.42 x_{u\,max})$

$A_{st} = 2122.43 mm^2$

provide 20mm ϕ bars.

$$\frac{\frac{\pi (20)^2}{4}}{2122} \times 1000 = 145 mm\ c/c$$

dist. steel reinforcement

$$A_{st\,dis} = \frac{0.12}{100} \times 1000 \times 270$$

$= 324 mm^2$

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provide 8mm ø bars

$$\frac{\frac{\pi(8)^2}{4}}{324} \times 1000 = 155 mm c/c$$

Checks

1) Overturning

$$F.S = \frac{0.9 \Sigma M_R}{\Sigma M_o} = \frac{0.9 \times 616}{320} = 1.73$$

≥ 1.4

Safe

2) Sliding

$$F.S = \frac{0.9 \mu W}{\text{hor. force}} = \frac{0.9 \times 0.5 \times 273.7}{\frac{1}{2} \times \frac{1}{3} \times 18 \times 5.97^2}$$

hor. force → $\frac{1}{2} K_a \gamma H^2$

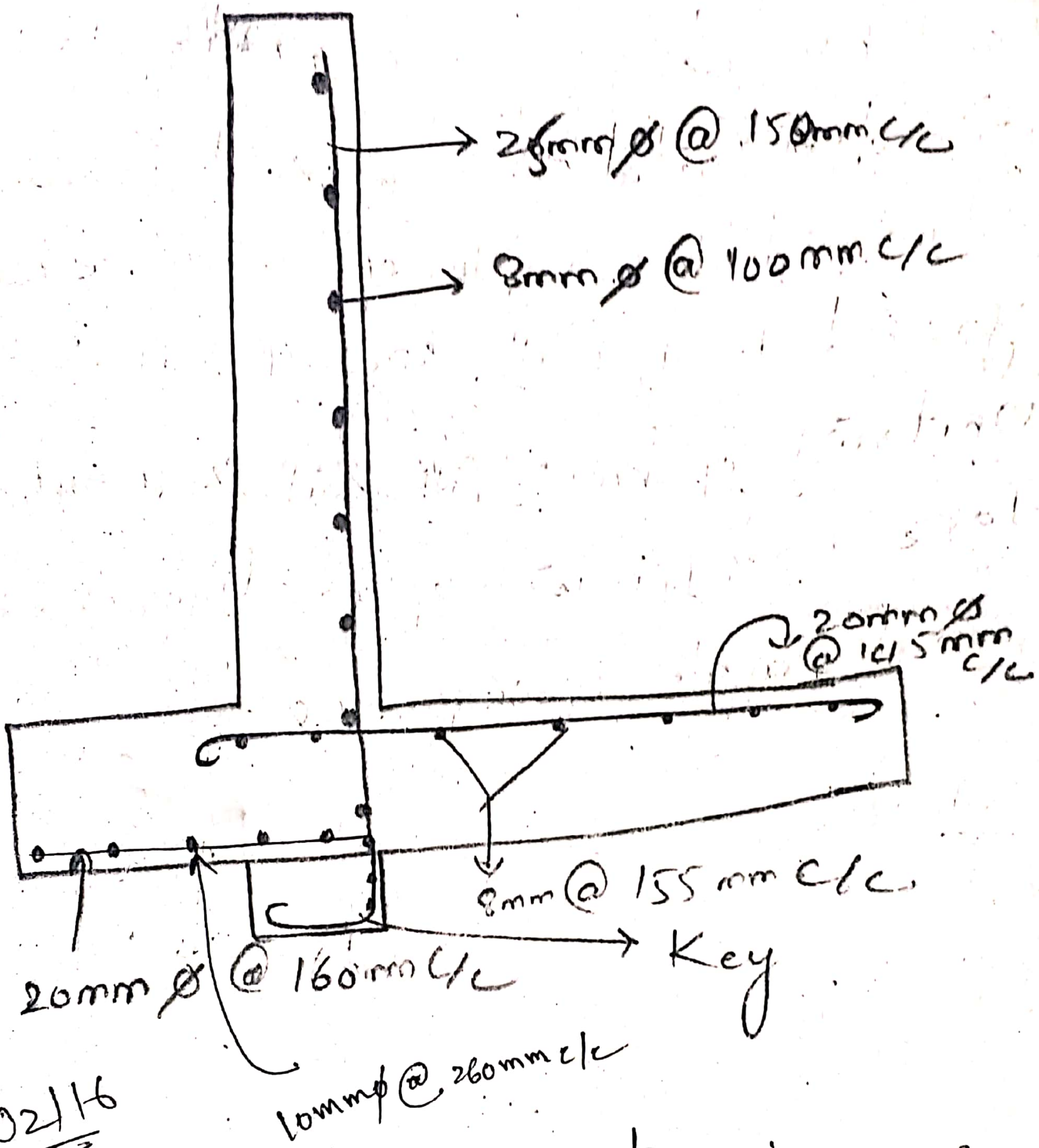
123

106.92 ×273.7

$= 1.15 < 1.4$

(Unsafe)

+ provide a key of size (0.4×0.8)m

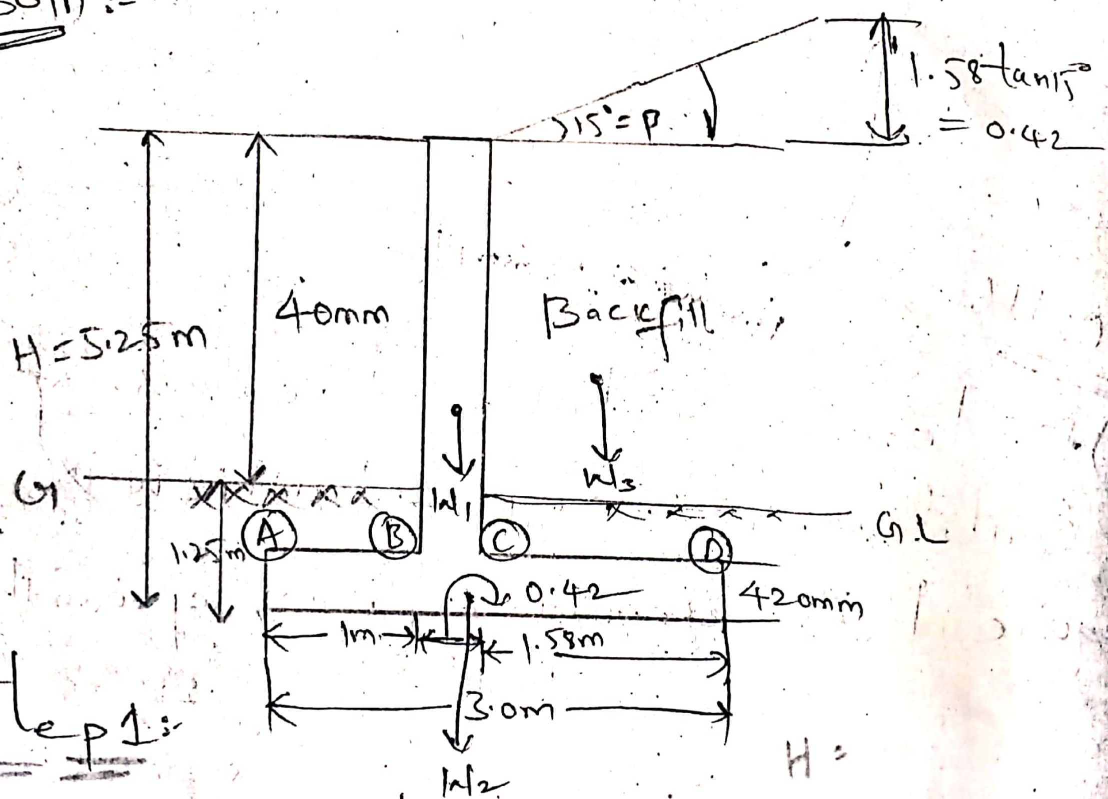


8/02/16

Q) Determine suitable dimensions of a cantilever retaining wall which is required to support a bank of earth 4m above ground level on the toe side of wall?

Consider the back fill surface be inclined at an angle of 1[illegible] with horizontal. Assume soil foundation at a depth of 1.25m below ground level, with SBC of soil 160kN/m². Unit wt of back fill soil is 16 kN/m³. Take co-efficient of friction μ = 0.5 and angle of repose ϕ = 30°

Soln :-



Step 1:

$SBC = 160 kN/m^2$, $\mu = 0.5$

$\gamma = 16 kN/m^3$, $\phi = 30°$

$H = 5.25m$, $\beta = 15°$

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

$K_a = 0.373$

* $K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3.0$

Step 2: (Assume thickness of base slab) ($H/12$)

* Thickness of base slab $\simeq 0.08H$

$\simeq 0.08 \times 5.25 m$

$= 0.42 m$

* Width of base slab $\simeq 0.6H$

$\simeq 3.15 \simeq 3.0 m$

* Width of toe slab $= \frac{1}{3} \times b$

$= \frac{1}{3} \times 3 = 1.0 m$

* Assume thickness of stem $= 420 mm$

* Width of heel slab $= 3.0 - 1.0 - 0.42$

$= 1.58 m$

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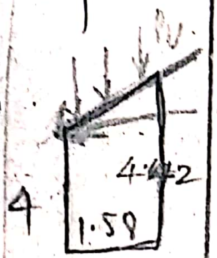
Steps Stability of wall

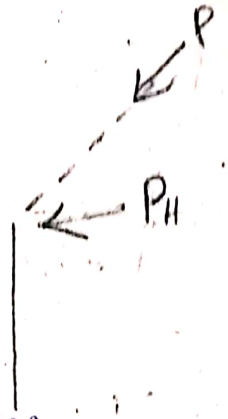
Member	Force (W)	lever arm upto (A)	Moment
Stem	$1 \times 0.42 \times (5.25 - 0.47) \times 25$ $= 50.71$ KN	$1 + \frac{0.42}{2}$ $= 1.21$ m	61.36 KN-m
Base slab	$1 \times 3 \times 0.42 \times 25$ $= 31.5$ KN	$3/2 = 1.5$ m	47.25 KN-m
Back fill 	~~1×3×~~ $\left(\frac{4+4.4}{2}\right) \times 1.58 \times 16$ 106.42 KN	$\frac{1.58}{2} + 1.42$ $= 2.2$ m	235.18 KN-m
P_v	$P \cdot \sin 15°$ $\frac{K_a \gamma H^2}{2} \sin 15°$ $96 \sin 15° =$ 24.85 KN	0	0

$$P = \frac{K_a \gamma H^2}{2} = \frac{0.373 \times 16 \times (5.25 + 0.42)}{2} = 96 \text{ KN}$$

$\Sigma W = 213.48\ KN$ $\Sigma M_R = 343.79\ KN\text{-}m$

(×1.5 for design)

Over-turning moment ΣM_o



$$\Sigma M_o = (P_a \cos\beta) \cdot H/3$$

$$= (9.6 \cos 15) \times \frac{5.67}{3}$$

$$\Sigma M_o = 175.25\ KN\text{-}m$$ ×1.5 ← for design

* $$\bar{x} = \frac{\Sigma M_R - \Sigma M_o}{\Sigma W}$$

$$\bar{x} = 0.78\,m < b/6 = \frac{3.0}{6} = 0.54\,m$$

* F.S against over turning

$$= \frac{0.9\ \Sigma M_R}{\Sigma M_o}$$

$$= \frac{0.9 \times 343.79}{175.25} = 1.767\ 1.4$$

F.S against over-turning

$e = \frac{b}{2} - \bar{x}$

$= 1.5 - 0.78$

$e = 0.716m > b/6$

Step 5:

$P_{max} = \frac{W}{b}\left(1 + \frac{6e}{b}\right)$

$= \frac{213.48}{3.0}\left(1 + \frac{6 \times 0.716}{3}\right)$

$= 173.06\ KN/m^2 < SBC$

Step 6: Check for Sliding

$F.S = \frac{0.9 \mu W}{P_a \cos\beta} > 1.4$

$= \frac{0.9 \times 0.5 \times 213.48}{97 \cos 15}$

F.S = 1.02 < 1.4

Fails

Hence provide a Key (or) a

Key may be provided to mobilise the force through passive resistance

∴ Assume a shear key of size 300 × 300 mm at a dist of 1.3 m from toe

10/2/16 Counterfort Retaining Wall

* In counterfort walls counterforts are provided at a spacing of about $\frac{1}{3}$ to $\frac{1}{2}$ of the ht of walls.

* The traingular shaped counterforts are provided on the rear side of the wall inter connecting the stem with heel slab.

* Counterforts enable the use of stem and base slab of much small thickness.

* For initial calculations the stem and base thickness may be taken as 4% of the height of wall but not less than 300mm.

* Some small buttresses are provided in the front side below the level of ground interconnecting the toe slab with lower portion of stem.

If buttresses are not provided, thickness of toe slab shall be taken as, in case of cantilever wall.

* Thickness of counter forts may be taken as about 5% of the height of wall.

Note:-

The counter fort shall be designed as vertical cantilever fixed at its base

* Counter fort may be designed as T-beams with depth of the section varying from top to bottom.

Design of counterfort retaining walls

* Design of stem

1) The stem of counterfort retaining wall acts as a continuous slab supported on counter forts

2) Design of heel slab

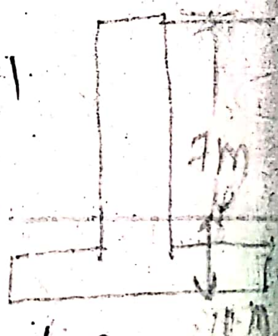
3) Design of toe slab

4) " " Counter forts.

problems:-

Q) Design a counterfort retaining wall to retain 7m high embankment above ground level. The foundation is to be taken 1m deep where the S.B.C of soil is 180 KN/m². The top of the earth retain is horizontal and soil weighs 18 KN/m³ with an angle of internal friction $\phi = 30°$. coefficient of friction μ b/w concrete and soil is 0.5. Use M_{20} & Fe 415 grades.

Soln:- 1) Given, total ht, H = 7+1
= 8m



$\mu = 0.5$, $f_{ck} = 20\,N/mm^2$

$f_y = 415\,N/mm^2$, SBC = 180 KN/m²

$\gamma = 18\,KN/m^3$, $\phi = 30°$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-1/2}{1+1/2} = 1/3$$

Dimensions

i) Thickness of base slab = 300mm

2) " " stem = 300mm

3) Width of base slab = 0.5 H
= 0.5×8

$b = 4.0m$

iv) Width of toe slab = $\alpha \cdot b$ (more precise)
(⅓ b)

where $\alpha = \left(1 - \frac{SBC}{2.2\gamma H}\right) = 0.43$

∴ width of toe slab = 0.43 × 4
= 1.72m

v) Width of heel slab = 4 − 1.72 − 0.3
= 2.0m

vi) Spacing of counter fort

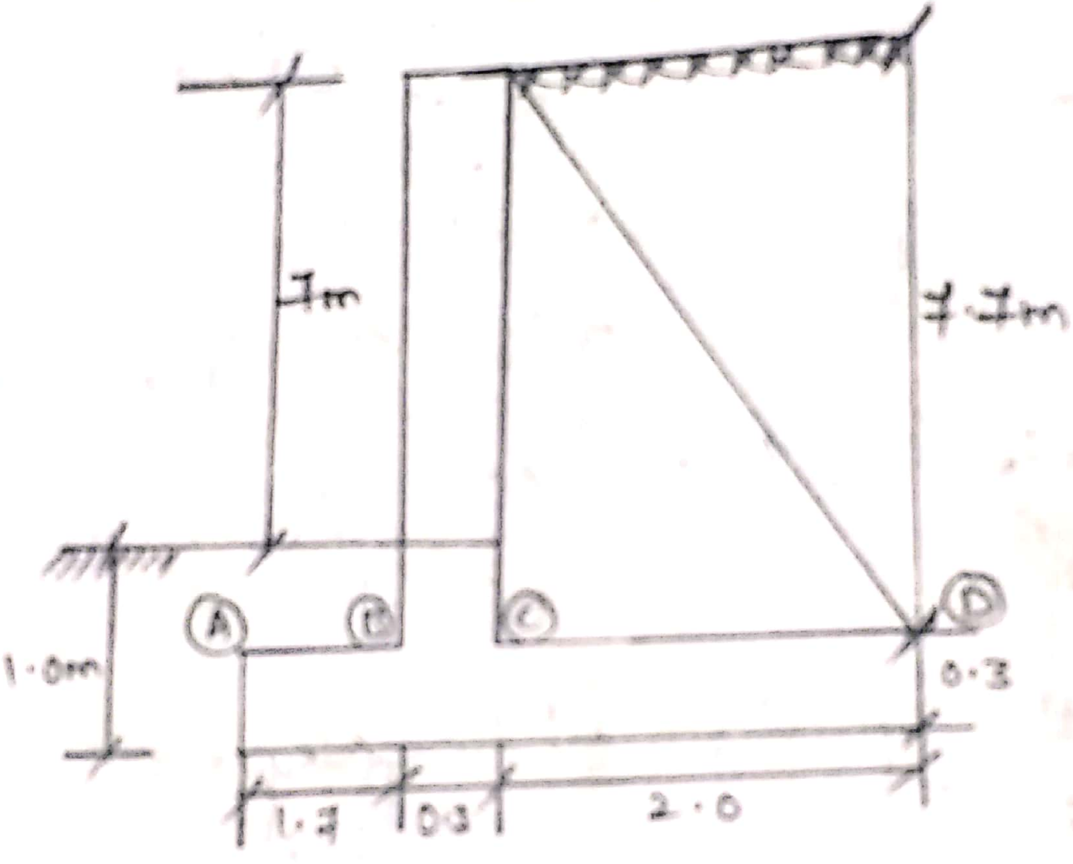
$l = 3.5\left(H/\gamma\right)^{1/4}$

$l = 3.5\left(8/18\right)^{1/4}$

$l = 2.86$ m c/c
≃ 3.0 m c/c

* Keep the spacing of counterfort

Note: provide counterforts over the slab also upto ground level at spacing of 3m c/c



* Pr. Intensity at base (max)

$= K_a \gamma H$ (ht of stem)

$= \left(\frac{1}{3} \times 18 \times 7.7 \times 1\right)$

$= 46.20\ kN/m$ (UDL)

Max B.M (M_u) = $\frac{1.5 w l^2}{12}$

$= \frac{1.5 \times 46.20 \times 3^2}{12}$

Max B.M = 51.97 KN-m

$M_u = ~~0.36~~ \; 0.138 f_{ck} b d^2$

$d = \sqrt{\frac{51.97 \times 10^6}{0.138 \times 20 \times 1000}}$

d = 137 mm

step-3:-

Stability of Wall:-

Member	load	Lever Arm	Moment
W_1	$0.3 \times 7.7 \times 1 \times 25 = 57.75$	$\frac{0.3}{2} + 1.7 = 1.85$	106.83
W_2	$0.3 \times 4 \times 1 \times 25 = 30$	$\frac{4}{2} = 2m$	60
W_3	$7.7 \times 2 \times 1 \times 18 = 277.2$	$\frac{2}{3} + 0.3 + 1.7 = 3m$	831.6
ΣW	364.95		$\Sigma M_R = 998.4$

Overturning Moment (ΣM_o)

$P = k_a \gamma H^2/2$

$= \frac{1}{3} \times 18 \times \frac{7.7^2}{2} = 177.87 \ kN/m^2$

$\Sigma M_o = P \times H/3 = 177.87 \times \frac{7.7}{3} = 456.5 \ kN\text{-}m$

$\bar{x} = \frac{\Sigma M_R - \Sigma M_o}{\Sigma W} = 1.48m.$

→ $e = \frac{b}{2} - \bar{x} = \frac{4}{2} - 1.48 = 0.52m.$

$\frac{b}{6} = \frac{4}{6} = 0.667$

$e < \frac{b}{6}$ ∴ (Safe).

* **Checks:-**

$(F.S)_{overturning} = \frac{0.9 \Sigma M_R}{\Sigma M_o} = \frac{0.9 \times 998.6}{456.5} = 1.66 > 1.4$ (safe)

$(F.S)_{sliding} = \frac{0.9 \mu W}{\frac{1}{2} k_a \gamma H} = \frac{0.9 \times 0.5 \times 365}{\frac{1}{2} \times \frac{1}{3} \times 18 \times 7.7^2}$

$= 0.92 < 1.4$

Hence provide a shear key of size 300mm X 300mm.

Overturning Moment (ΣM_0)

$P = k_a \gamma H^2/2$

$= \frac{1}{3} \times 18 \times \frac{7.7^2}{2} = 177.87 \ kN/m^2$

$\Sigma M_0 = P \times H/3 = 177.87 \times \frac{7.7}{3} = 456.5 \ kN\text{-}m$

$\bar{x} = \frac{\Sigma M_R - \Sigma M_0}{\Sigma W} = 1.48m.$

→ $e = \frac{b}{2} - \bar{x} = \frac{4}{2} - 1.48 = 0.52m.$

$\frac{b}{6} = \frac{4}{6} = 0.667$

$e < \frac{b}{6}$ = (safe).

* checks:-

$(F.S)_{overturning} = \frac{0.9 \Sigma M_R}{\Sigma M_0} = \frac{0.9 \times 998.6}{456.5} = 1.66 > 1.4$ (safe)

$(F.S)_{sliding} = \frac{0.9 \mu W}{\frac{1}{2} k_a \gamma H} = \frac{0.9 \times 0.5 \times 365}{\frac{1}{2} \times \frac{1}{3} \times 18 \times 7.7^2}$

$= 0.92 < 1.4$.

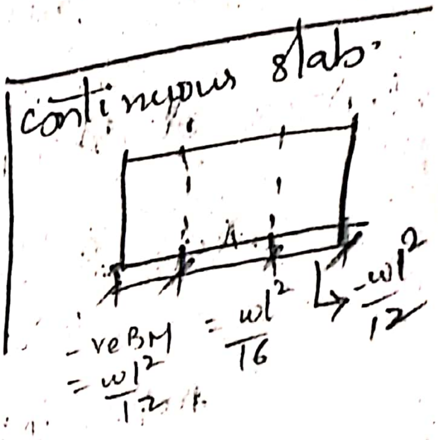
Hence provide a shear key of size 300mm X 300mm.

Step 4 :- (Design of Stem):

The Stem acts as a continuous slab considering in strip.

$w = P = k_a \gamma H = \frac{1}{3} \times 18 \times 7.7 = 46.2 \text{ kN/m}^2$.

$(-ve)$ B.M $= 1.5 \times \frac{wl^2}{12}$ (near support)



$(-M_u) = \frac{1.5 \times 46.2 \times (3)^2}{12}$

$(-M_u) = 51.97 \text{ kN-m}$.

* $(+BM) = \frac{1.5 P l^2}{16}$ (middle strip)

$= \frac{1.5 \times 46.62 \times (3)^2}{16} = 38.98 \text{ kN-m}$.

Thickness of stem :-

$51.97 \times 10^6 = 0.138 \times 20 \times 1000 d^2$

$d = 137.23 \text{ mm}$

* However, Provide a thickness of stem as 300mm + 50mm cover, to take care of shear failure.

~~Area~~

Area of steel (for -ve B.M)

$$M_U = 0.87 f_y \times A_{st} (d - 0.42 x_{umax})$$

$A_{st} = 601 mm^2$

Provide 12mm ϕ bars @ $\frac{\frac{\pi (12)^2}{4}}{601} \times 1000 = 180mm$ c/c

* Area of steel (+ve B.M)

$$A_{st} = \frac{88.98 \times 10^6}{0.87 \times 415 (300 - 0.48 \times 0.42 \times 300)}$$

$A_{st} = 450 mm^2$

Spacing provide 12mm ϕ bars @

$$\frac{\frac{\pi (12)^2}{4}}{450} \times 1000 = 250mm \text{ c/c}$$

Step5 (Design of heel slab)

* Heel slab also has to be design as a continuous slab.

* Pressure distribution at the base slab.

$$P_{max} = \frac{W}{b}\left(1 + \frac{6e}{b}\right) = \frac{365}{4}\left(1 + \frac{6 \times 0.52}{4}\right)$$

$= 162 kN/m^2$

$480 kN/m^2$

Distributory Steel

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

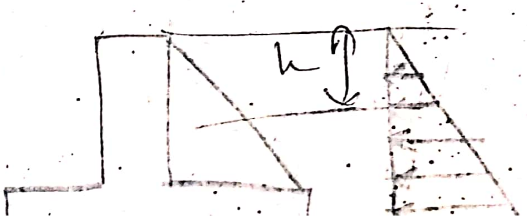
$$= 420 mm^2$$

provide 8mm ∅ @ 120mm c/c

$$\frac{\frac{\pi (8)^2}{4}}{420} \times 1000 = 120mm\ c/c$$

Design of Counter forts :-

* Assume thickness of counter fort = 500mm spaced @ 3m c/c
* Countour fort recieves earth pressure from a width of 3m and downward reaction from heel slab for a width of 3m.
* At any section at depth 'h' below top of stem, the pressure acting on each Counter fort.



* Each counter fort $= K_a \gamma h \times 3.0$

$= \frac{1}{3} \times 18h \times 3$

$= 18h$

* ∴ net downward pressure on heel at

$D = 18h + 0.3 \times 25 - 20$

$= (18 \times 7.7) + (7.5) - 20$

$= 126.1 \ KN/m^2$

* At C

$= (18 \times 7.7) + (0.3 \times 25) - 91$

$= 55.1 \ KN/m^2$

* Hence, γ_{m} transvered to each counter fort will be at D

$= 126.1 \times 3 = 378.3 \ KN/m$

At C ∴ $55.1 \times 3 = 165.3 \ KN/m$.

* The critical section for counter fort will be at (F), since, below this anormous depth will be available to resist bending.

Net u/w pr at B = 101.6 − 7.5

= 94.1 kN/m

& P = w = 154.8 kN/m

(near support)

−ve B.M $= \frac{-wl^2}{12}$

$= -\frac{154.5 \times 3^2}{12}$

= −115.87

$M_u = -1.5 \times 115.87$

= −173.8 kN-m

$M_u = 0.87 f_y A_{st} (d - 0.42 x_{umax})$

$A_{st} = \frac{173.8 \times 10^6}{0.87 \times 415 \times (300 - 0.42 \times 0.48 \times 300)}$

$A_{st(-)} = 2009.7 mm^2$

provide 16mm ø @ 100mm c/c

+ + B

$$B.M_{(+)} = \frac{+wl^2}{16}$$

$$= \frac{+154.5 \times 3^2}{16}$$

$$= 86.90 \text{ KN-m}$$

$$M_u = 1.5 \times 86.90$$

$$M_u = 130 \text{ KN-m}$$

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_{umax})$$

$$A_{st} = \frac{130 \times 10^6}{0.87 \times 415 \times (300 - 0.42 \times 0.48 \times 300)}$$

$$A_{st} = 1503.2 \text{ mm}^2$$

provide 16mm Ø @ 133 mm c/c

$$\frac{\frac{\pi (16)^2}{4}}{1503.2} \times 1000 = 133 \text{mm}$$

$$M_{+ve} = \frac{+wl^2}{16}$$

$$= \frac{+126.1 \times 3^2}{16}$$

$$= 70.9 \text{ KN-m}$$

$$M_u = 1.5 \times 70.9$$

$$= 106.39 \text{ KN-m}$$

$$M_{u(+ve)} = 0.87 f_y A_{st} (d - 0.42 x_{u\,max})$$

$$A_{st(+ve)} = \frac{106.4 \times 10^6}{0.87 \times 20 \times (300 - 0.42 \times 0.48 \times 300)}$$

$$A_{st(+ve)} = 1225.73 \text{ mm}^2$$

provide 16mmø spacing

$$\frac{\frac{\pi(16)^2}{4}}{1225.73} \times 1000$$

$= 160\text{mm c/c}$

+ Distributary Steel

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

$= 420 mm^2$

provide 8mm Ø @ 120mm c/c

$\frac{\frac{\pi (8)^2}{4}}{420} \times 1000 = 120mm$ c/c

Design of toe slab

+ Since the toe slab is also of large span (1.7m), provide buttress below ground level at spacing of 3m c/c. Thus, toe slab behaves as a continuous slab.

+ Pr at (A)

D/w pr (self wt) = $0.3 \times 25 \times 1 = 7.5$ KN/m

U/w pr at (A) = 1×162 KN/m² = 162 KN/m

∴ Net u/w pr at A = $162 - 7.5 = 154.5$ KN/m

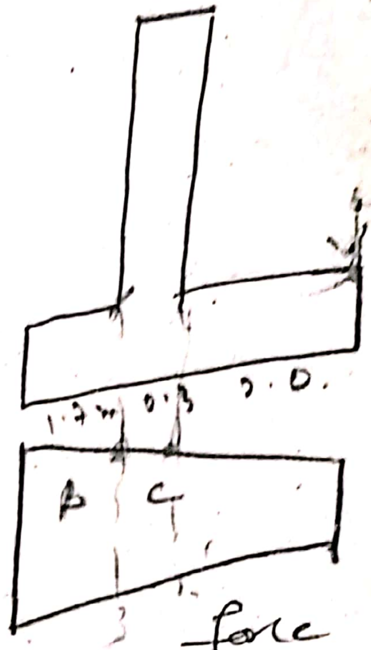
$$P_{min} = \frac{365}{4}\left(1 - \frac{6 \times 0.52}{4}\right) = 20.07 KN/m$$

P_D

9-2-14

$P_B = 104.6 KN/m^2$

$P_c = 91 KN/m^2$



force

* Clear spacing b/w counterforts / continuous slab.

= span of continuous slab

$l = 3m$ c/c

* At (D), downward load due to back fill $= \gamma h = 18 \times 7.7$
$= 138.6 KN/m^2$

+ Downward load of self wt of heel slab
$= 0.3 \times 25$
$= 7.5 KN/m^2$

+ upward pr from soil $= 20 KN/m^2$

+ Net downward $= 138.6 + 7.5 - 20$
$= 126.1 KN/m^2$

* Max -ve B.M $= \frac{wl^2}{12}$
(at supports)

$= \frac{126.14 \times 3^2}{12}$

$= 94.59$ kN-m

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

$1.5 \times 94.59 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$

$d = 226\text{mm} \approx 226\text{mm}$

However from shear consideration 'd' shall be taken as 300mm

$M_u = 0.87 f_y A_{st} (d - 0.42 x_{umax})$

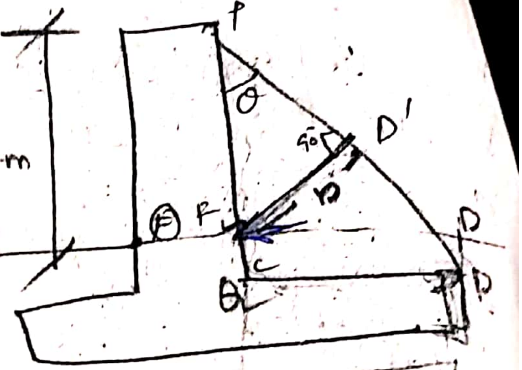
$A_{st} = 1640.3\text{mm}^2$
(-ve)

provide 16mm ø at spacing

$\frac{\frac{\pi (16)^2}{4}}{} \times 1000 = 122$ mm c/c

* $S.F = \frac{wL}{2} = \frac{165.3 \times 7}{2}$

$= 578.5\ KN$



* $B.M = 578.5 \times 7/3$

$= 1350\ KN\text{-}m$

* $M_u^{**} = 0.138 \times f_{ck}\, b\, d^2$

$d = \sqrt{\frac{1.5 \times 1350 \times 10^6}{0.138 \times 20 \times 1810}}$

$d = 636mm$

* provide $d = 700mm$

$d' = 50mm$

for b

ΔPQD

$\tan\theta = \frac{QD}{QP}$

$= \frac{2}{7}$

$= \approx 15°$

$\Delta PFD'$

$\sin 15° = \frac{FD'}{PF}$ ← 7m

* $M_u = 0.87 f_y A_{st} (d - 0.42 x_{umax})$ $FD' = b = 1.81$

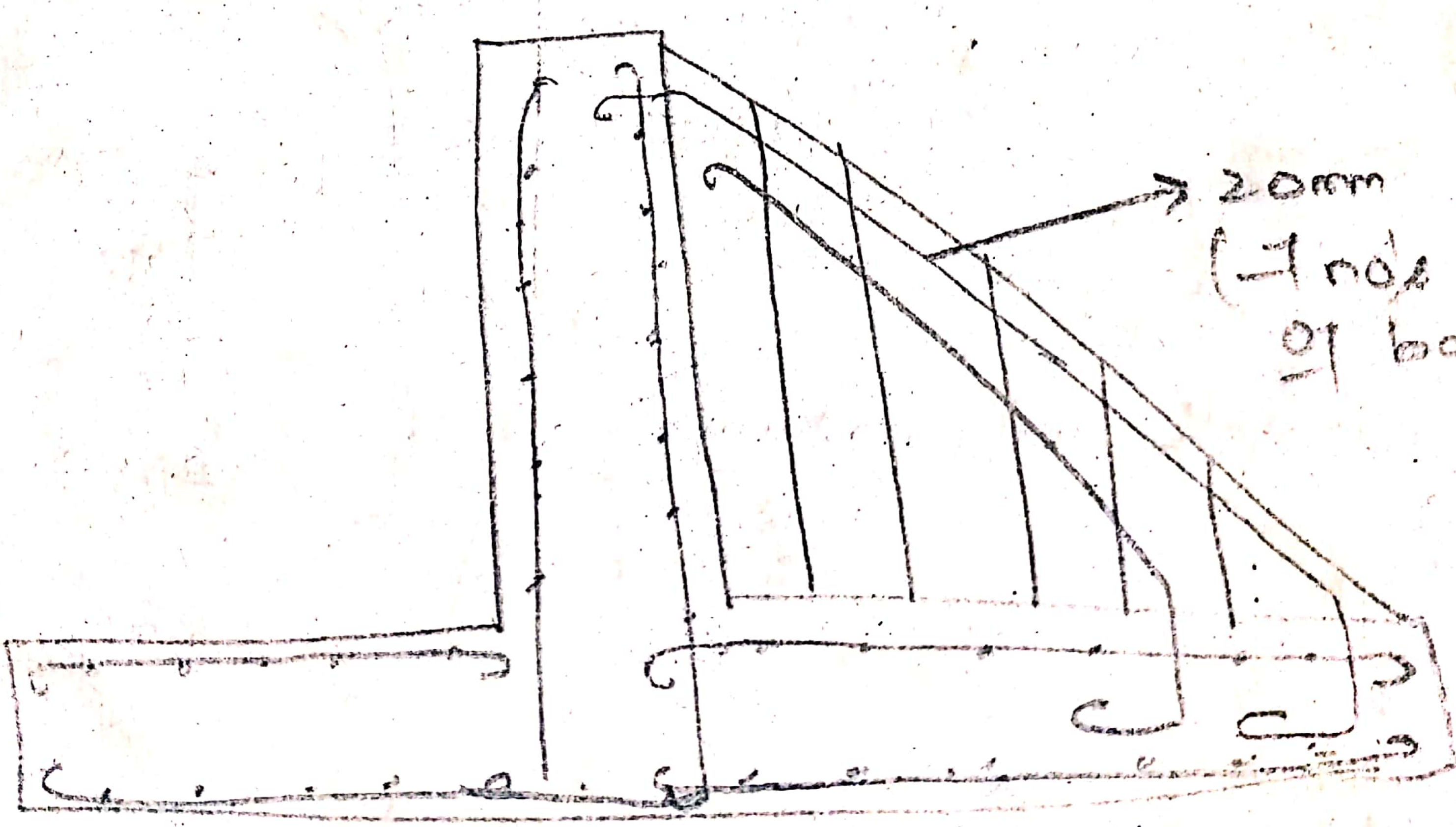
$A_{st} = \cancel{135}\ \cancel{t_5} = 3881.133\ mm^2$

* In this formula we are taking the value of 'b' in place of 'd' because counter fort.

* Ka Asme ander steel provide Karte, depth mein nahi Karte.

* provide the reinforcement in two layers.

* No of bars = $\dfrac{3881/2}{\dfrac{\pi(20)^2}{4}} \simeq 6$ bars



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