LECTURE NOTES OF Dr. B.L.P. SWAMI.

SEDDI [R.C.C]

STRUCTURAL ENGINEERING-DESIGN & DETAILING.

LECTURE NOTES OF Dr. B.L.P. SWAMI

TOPICS COVERED.

1. FUNDAMENTALS OF WORKING STRESS DESIGN (R.C.C)
2. R.C.C COMBINED FOOTINGS.
3. RETAINING WALLS.
4. WATER TANKS OF VARIOUS TYPES.
5. R.C.C. BRIDGES
   a) SLAB BRIDGES
   b) GIRDER BRIDGES.

[IS-456(2000), Water Tanks tables and IRC-Code Specifications are to be provided during the examination]

UNIT-I

TOPICS.

1) Fundamentals of working stress design

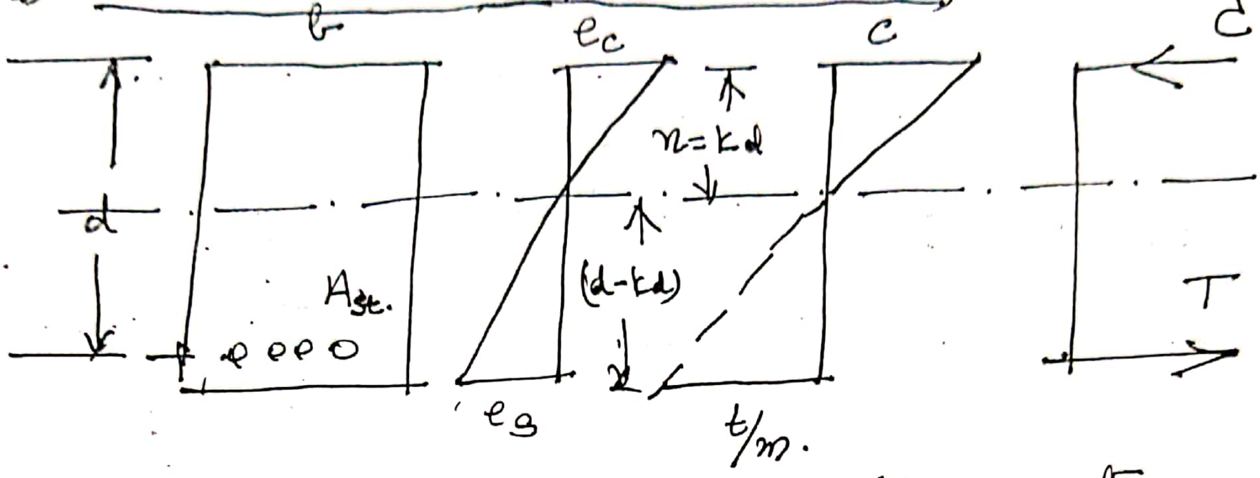
2) Combined Footings

3) Retaining walls.

WORKING STRESS DESIGN _ FUNDAMENTALS.

Considering a Singly reinforced rectangular Section,

a) All the Sectional details of the beam are known)



Since there is no resultant force across the Section, equating

Total Compression = Total tension, $C = T$

Neglecting the tensile stress in Concrete, we get

$$\frac{1}{2} c\, b\, kd = t \cdot A_{st}$$

Stress in equivalent Concrete area $= t/m$

Hence by Similar triangles, $\dfrac{c}{t/m} = \dfrac{kd}{(d-kd)}$ or $\dfrac{mc}{t} = \dfrac{kd}{d(1-k)}$

or $mc(1-k) = tk$ or $mc - mck = tk$

or $mc = k(mc+t)$, $\dfrac{c}{k} = \dfrac{(mc+t)}{m} = (c + \dfrac{t}{m})$

$k = \dfrac{mc}{mc+t}$ or $\cancel{k} = \dfrac{c}{(\cancel{c+t/m})}$ $\therefore k = \dfrac{1}{1+\dfrac{t}{mc}}$

or $k = \dfrac{1}{1+r/m}$, where r = stress ratio $= t/c$.

We have, $k = \frac{mc}{mc+t}$

By taking maximum permissible stresses in concrete and steel,

We get $k_c = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

k_c = (Critical N.A depth factor).

or Hence critical N.A depth $= n_c = k_c d = \left[\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right] d.$

When the stresses in concrete and steel reach their max. permissible values then, it is a <u>BALANCED SECTION</u>.

<u>Modular Ratio</u>

As per IS 456-2000, $m = \frac{280}{3\sigma_{cbc}}$, or $m\sigma_{cbc} = \frac{280}{3} = 93.33.$

We can write,

$$k_c = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{93.33}{93.33 + \sigma_{st}} = \frac{1}{1+0.0107\sigma_{st}}$$

Hence, it is clear that the N.A depth factor does not depend on the grade of concrete. It depends only on grade of steel.

b) <u>Percentage of steel reinf.</u>

Area of steel for the section is given by,

Percentage steel $p = \frac{A_{st}}{bd} \times 100$

We have, from the stress diagram,

$$\frac{1}{2} c\, b\, kd = t \cdot A_{st}.$$

or $$\frac{A_{st}}{bd} = k \cdot \frac{c}{2t}.$$

Hence $$p = \frac{kc}{2t} \times 100 = 50\frac{c}{t} \times k = 50\frac{c}{t} \times \left(\frac{mc}{mc+t}\right)$$

or $$p = \frac{50mc^2}{t(mc+t)}.$$

In terms of permissible stresses $$\boxed{p = \frac{50\, m\, \sigma_{cbc}^2}{\sigma_{st}(m\, \sigma_{cbc} + \sigma_{st})}}$$

(b) <u>If the dimensions of the beam are completely known.</u>

we have,

$$b \cdot kd \cdot \frac{kd}{2} = m A_{st}(d - kd)$$

or $k^2 \cdot \frac{bd^2}{2} + m A_{st} kd - m d A_{st} = 0$, Dividing by $\frac{bd^2}{2}$,

or $k^2 + 2km \frac{A_{st}}{bd} - 2m \frac{A_{st}}{bd} = 0$, $\frac{A_{st}}{bd} = p'$

Then $k^2 + 2kmp' - 2mp' = 0$.

$$\text{or } k = \frac{-2mp' \pm \sqrt{4m^2 p'^2 + 4mp'}}{2}$$

Hence knowing the dimensions of the beam, area of steel and concrete mix, the N.A factor can be computed.

MOMENT OF RESTANCE.

If 'jd' is the lever arm (Distance between Centre of Compression and centre of tension, then the lever arm is given by

$$a = jd = d - \frac{kd}{3} = d\cdot\left(1 - k/3\right).$$

or Lever arm factor 'j' $= \left(1 - \frac{k}{3}\right)$

Now

$$M\cdot R = \frac{1}{2} c k d \cdot b \cdot (jd) = \frac{1}{2} c k j (bd^2) = t \cdot A_{st}\, jd$$

$$\text{or } M\cdot R = R b d^2, \quad R = M\cdot R \text{ factor}.$$

In the Case of a balanced Section

The stresses in Concrete and Steel reach their permissible values Simultaneously.

While designing R.C.C Sections, balanced design is carried out. Hence in terms of permissible values, we can write,

$$\cancel{M_r} = \cancel{\tfrac{1}{2}} \quad M\cdot R \text{ factor} = R_c = \frac{1}{2} \sigma_{cbc}\, j_c\, k_c.$$

In terms of steel area, $M_r = \sigma_{st} A_{st}\, j_c\, d.$

$$\text{or} \quad A_{st} = \frac{M_r}{\sigma_{st}\, j_c\, d}.$$

We have the other Cases like Under reinforced and over reinforced.

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COMBINED FOOTINGS

DEF: The footing that supports two or more columns is termed as Combined footing. When isolated footings are provided, some times they overlap due to closer spacing of columns. Hence Combined footings are necessary. If eccentric footings are provided at the boundaries, some times they may be overturn due to excessive pressures. Due to all these reasons Combined footings are provided.

TYPES OF COMBINED FOOTINGS.

1) For two columns.
2) For a row of columns.

Depending on the shape the types are

1) Rectangular
2) Trapezoidal
3) Beam and slab type.

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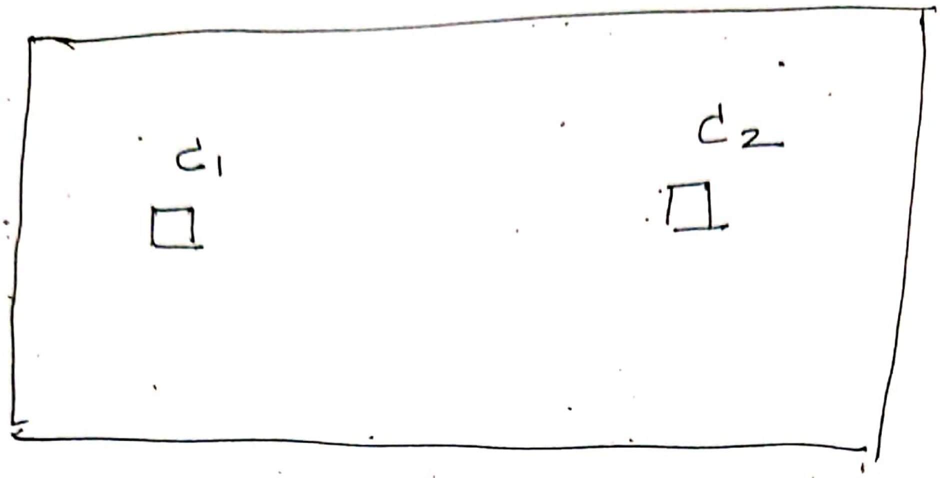
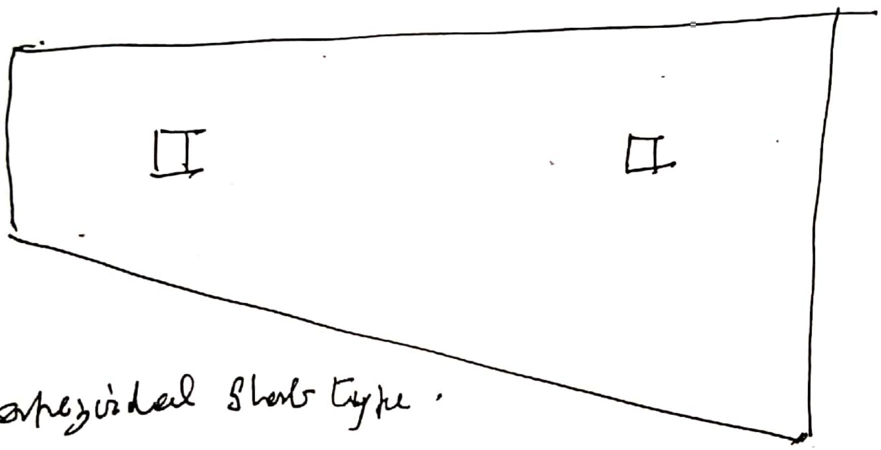
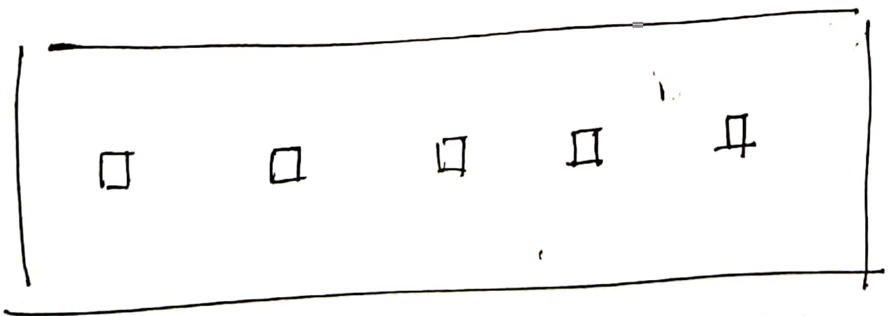


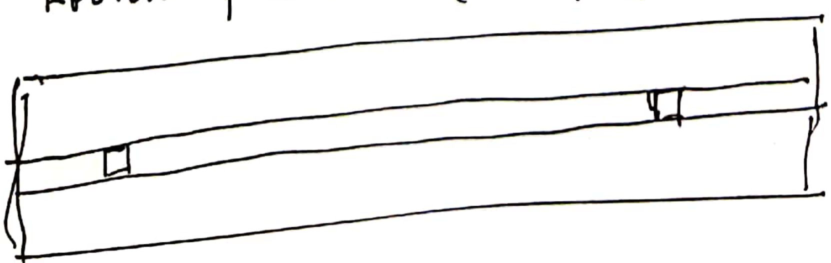
1) Rectangular slab type footing.



2) Trapezoidal slab type.



3) For rows of columns (strip footing)



4) Beam-Slab combined footing.

DESIGN PRINCIPLES OF RECTANGULAR SLAB TYPE COMBINED FOOTING:

Let P_1 and P_2 be the loads on the two columns, $P_1 < P_2$.

The distance of P_1 from boundary line $= x_1$

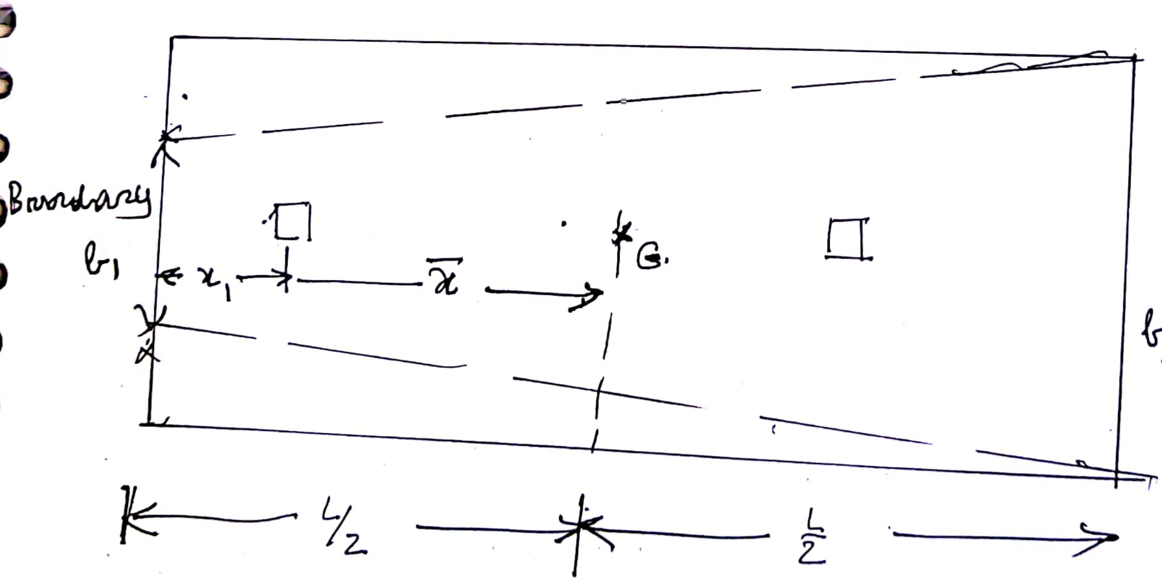
The centre of gravity of the footing is given by (Taking moments)

$$\bar{x} = \frac{P_1 \times 0 + P_2 l}{(P_1 + P_2)}$$

Taking $x_1 + \bar{x} = \frac{1}{2} \times$ length of the footing, $L = 2(x_1 + \bar{x})$

Area required for the footing $= \dfrac{1.1\,(P_1 + P_2)}{\text{Safe B.C. of the soil}}$ (10% is added for self wt. of footing)

Hence, Width of the footing $= \dfrac{A}{L}$



[Rectangular or trapezoidal footing (Combined)]

DESIGN PROCEDURE.

Step (1) : From the given loads acting on the columns fix the dimensions of the footing knowing the safe bearing capacity of the soil. Calculate the net upward pressure of the soil.

Step (2) Treating the combined footing as an overhanging beam over the column supports under the UDL of soil pressure draw the S.F and B.M diagrams. Note the critical sections of bending moment and S.F.

Step (3) Check for one way shear and depth of the footing. From the maximum value of one way shear, determine the depth (thickness required) for the footing. Depth is checked from two way shear (Punching) and cantilever moment acting on the face of the column from the projection.

Step. (4) Compute the area of steel and spacing of reinforcement against Max. -ve and +ve moments. Provide the transverse reinforcement for the cantilever projection. In the remaining provide minimum reinforcement at 0.12% (for tor steel).

Step. (5) Provide shear reinforcement if necessary.

Step (6) Detailing of reinforcement is shown by neat sketches.

DESIGN EXAMPLE.

Design an RCC rectangular Combined footing for two columns located at 4.5m apart. The overall sizes of the columns are 400x400mm and 600x600mm and they are transferring 600 KN and 1000 KN respectively. The Centre of the lighter column is 0.4m from the property line. The S.B.C of the Soil is 150 KN/m² Use M20 and Fe 415. Sketch the reinforcement details.

SOLUTION: Step ①.

From the Centre of lighter column the C.g of the loads is

$$\bar{x} = \frac{600 \times 0 + 1000 \times 4.5}{(600+1000)} = 2.81\ m.$$

$x_1 = 0.4m.$

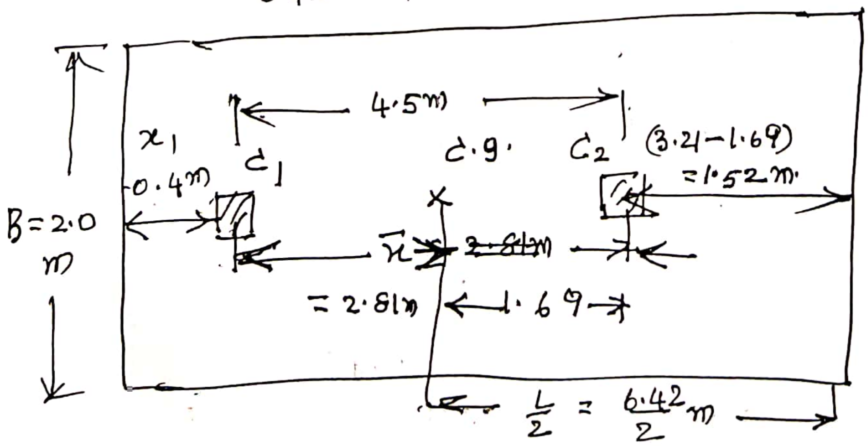
Hence, length of the footing $= (2.81+0.4)2 = 6.42\ m.$

Step ②

S.B.C = 150 KN/m²

Assuming 10% for Self weight, Area of the footing $A = \frac{1.1\,(600+1000)}{150} = 11.73\ m^2$

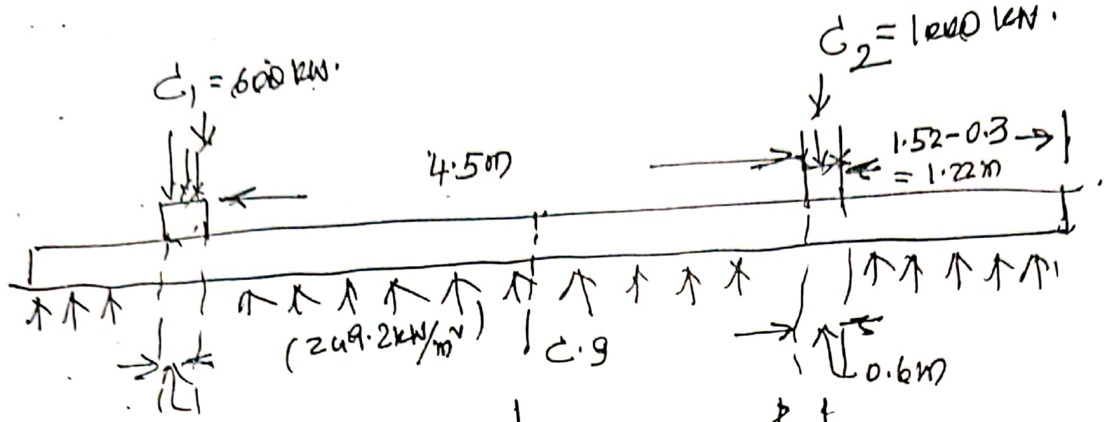
Hence $B = \frac{11.73}{6.42} = 1.83m$, Provide B = 2.0m.

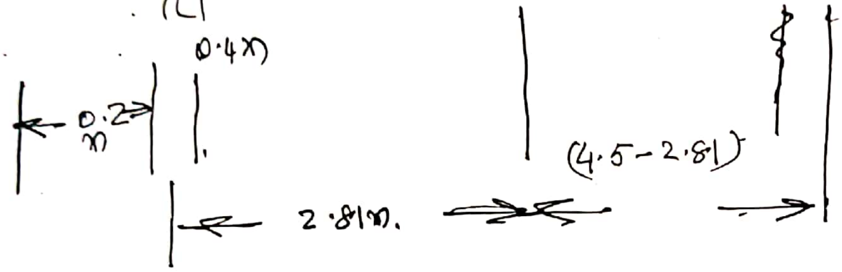


c_1 = 400x400mm

c_2 = 600x600 mm.

Step 3   Moments and Shears

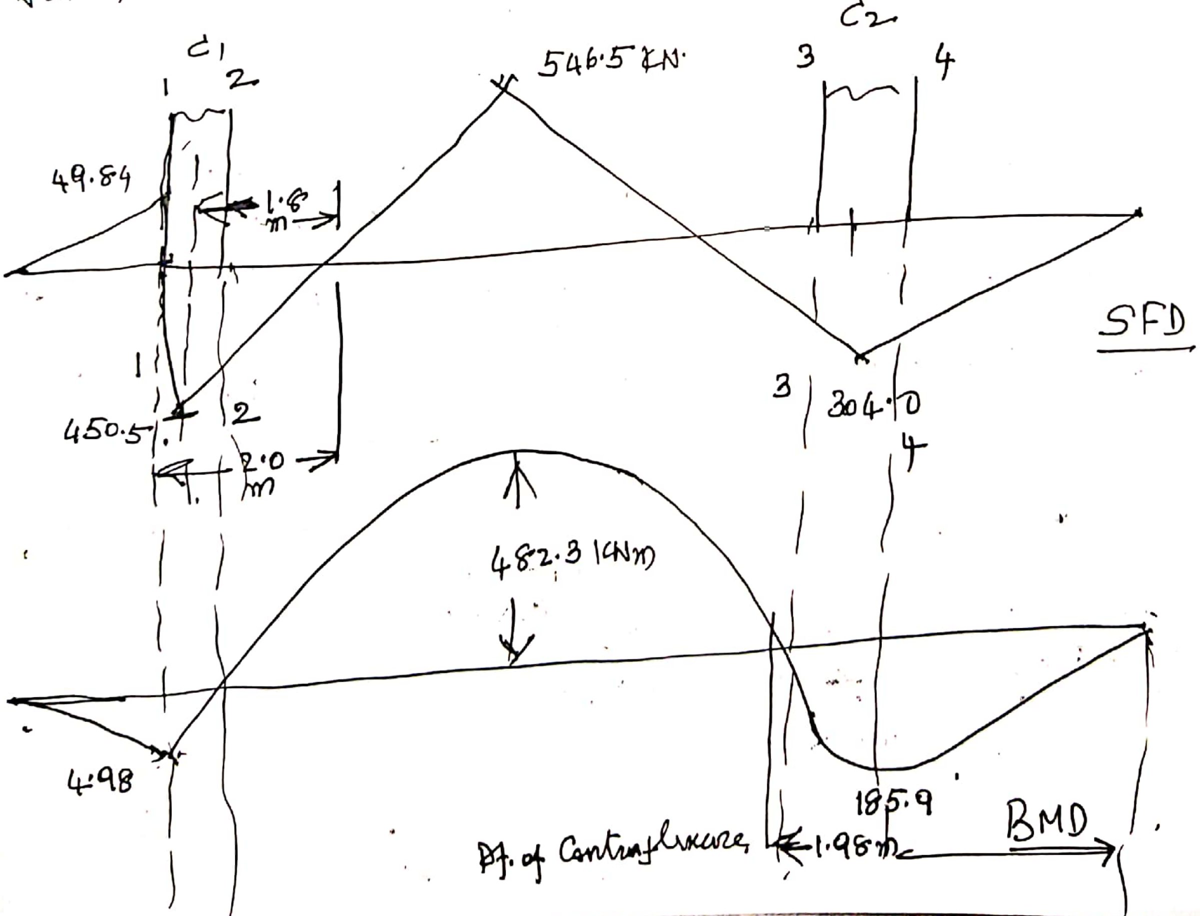




(Along the length)

Soil pressure per unit ~~length~~ area $= q = \frac{600+1000}{6.42} = 249.2 \text{ kN/m}^2$ (1m width).

Hence, the S.F and BMD are drawn as shown.



S.F

At 1-1 = $249.2 \times 0.2 = 49.84$ kN.

At 2-2 = $249.2 \times 0.6 - 600 = -450.5$ kN.

At 3-3 = $-249.2 \times 1.82 + 1000 = +546.5$ kN.
(from right end)

At 4-4 = $-249.2 \times 1.22 = -304$ kN.

S.F = 0 at $\frac{450.5}{249.2} = 1.8$ m from 2-2 (i) at 2 m from the centre of column (1).

B.M's

at 1-1 = $249.2 \times \frac{0.2^2}{2} = 4.98$ kNm.

at 4-4 = $249.2 \times \frac{1.2^2}{2} = -185.9$ kNm.

Max. +ve B.M (at S.F = 0) = $M_{max} = -249.2 \times \frac{2.4^2}{2} + 600 \times 2$

$= 482.3$ kNm.

Point of contra flexure occurs where B.M = 0. (Shown in the figure step (4).

Critical section for moment is where S.F = 0. (1.8 m from the centre of c_1)

Max. Moment at this point = 482.3 kNm.

Factored moment $M_u = 1.5 \times 482.3 = 723.5$ kNm.

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Critical Section for Shear. (one way)

It occurs at 'd' from the face of the Column (d = depth of the footing), in mm

S.F at this Section = $\left[546.5 - 249.2 \times \frac{d}{1000}\right]$ kN.

(Factored) $V_u = 1.5\left[546.5 - 249.2 \times \frac{d}{1000}\right]$ kN.

Critical Section for two way shear occurs at $'\frac{d}{2}'$ from the face of the column.

<u>one way shear</u>.

Critical Section at 'd' from the face of heavier Column.

$V = \left(546.50 - 249.2 \times \frac{d}{1000}\right)$ kN ('d' is in 'mm') (working)

Hence, ~~$V_u = \left[546.50 - 249.2 \times \frac{d}{1000}\right] 1.50$ kN~~

<u>Two way Shear</u>

It is at $'\frac{d}{2}'$ from the Column face.

<u>Step (4)</u> . <u>Depth of footing</u>

$V = \left(546.5 - 249.2 \times \frac{d}{1000}\right) 10^3$ N (one way Shear)

Assuming a minimum of 0.2% reinforcement, the Shear strength of Concrete is $0.32 N/mm^2$

· Equating to the resisting S.F,

$$\left[546.5 - 249.2\frac{d}{1000}\right]1000 = B.d\times(0.32) \quad, B = 2m$$
$$= 2000\ mm.$$

Hence $[546.5\times1000 - 249.2\times d] = 2000\times d\times0.32$

Hence $d = 614.6\ mm$., Provide $d = 620$ with $D = 680\ mm$ including Cover.

By LSD, for Fe 415 steel, $M_u = 0.138 f_{ck}\, b\, d^2$

Hence $M_{u\,(Lim.)} = 0.138\times20\times2000\times620^2$
$= 2121.9\times10^6\ Nmm > 723.6\times10^6$ (actual).

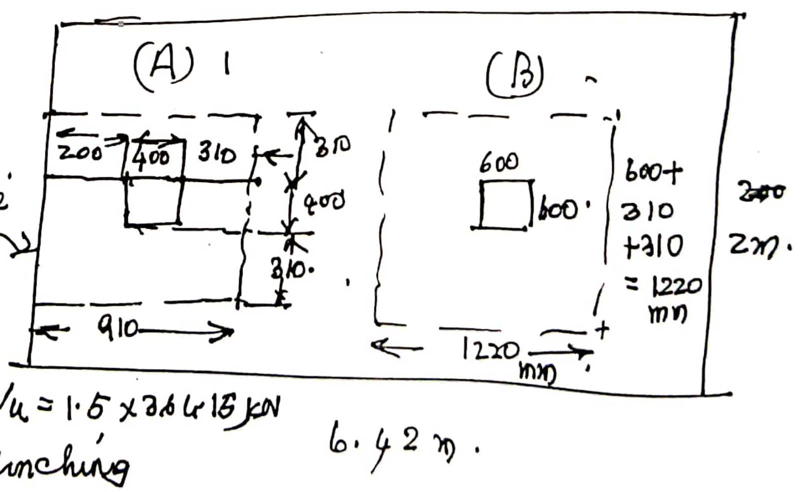
Hence, depth is Sufficient.

Considering 2-way Shear at column!

·Perimeter of only 3 Sides for Column (A) (C_1)

$= 2\times910 + 1020.$

$= 2840\ mm.$



S.F over the Shear area

$= 600 - 249.2\times.910\times\cancel{1.20}\ 1.020$

$= 364.15\ KN.$, $V_u = 1.5\times364.15\ kN$

Resisting Shear against punching

$= (2840\times620)\,\tau_c'$

Equating, $(1.5\times364.15\times1000) = 2840\times620$, $\tau_c = 0.308\ N/mm^2$

τ_u (Allowable) $= 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\ N/mm^2$, Hence okay.

Two way Shear at Column (B) (C_2)

Perimeter $= 1220 \times 4$ mm, $d = 620$ mm

$= 1.22$ m

$V = (1000 - 249.2 \times 1.22 \times 1.22) = 629.09$ kN

$V_u = (1.5 \times 629.09)\, 10^3$ N

Equating, $(1.5 \times 629.09)\, 10^3 = \tau_v \times 1220 \times 620$

Hence, $\tau_v = 0.312$ N/mm² $< \tau_c$ (Allowable)

Hence, okay.

Hence, $d = 620$ is sufficient from all considerations.

Step 5 Reinforcement in longitudinal direction.

From the BMD, Max. +ve $M_u = 1.5 \times 482.3 = 723.5$ kNm.

We have from LSD, $M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st}}{bd} \times \frac{f_y}{f_{ck}}\right)$

Substituting,

$$723.5 \times 10^6 = 0.87 \times 415 A_{st}\, 620 \left[1 - \frac{A_{st}}{2000 \times 620} \times \frac{415}{20}\right]$$

Simplifying, $3232 = A_{st}\left[1 - \frac{A_{st}}{59759}\right]$

or $A_{st}^2 - 67469.9\, A_{st} + (3543.24 \times 67469.9) = 0$

Solving by trial, $A_{st} = 3428$ mm², For 16 mm ϕ, Area $= \frac{\pi}{4} \times 16^2$

$= 2.01$ mm²

Hence, spacing of 16 mm ϕ over 2 m width $= \frac{2.01}{3428} \times 2000$

$= 117$ mm.

Hence provide 16 ϕ @ 110 mm c/c at top.

Max. −ve moment = 185.9 × 1.5 kNm.

We can write,

$$1.5\times186.9\times10^6 = 0.87\times415\,A_{st}\times620\left[1-\frac{A_{st}}{2000\times620}\times\frac{415}{20}\right]$$

Simplifying, $A_{st}^2 - 59759A_{st} + (243\times5959) = 0$.

Solving, $A_{st} = 1270\ mm^2$. It is very less.

Providing, Min. steel at 0.12% = $\frac{0.12}{100}\times2000\times680 = 1632\ mm^2$

Spacing of 16 mm ϕ = $\frac{\pi/4\times16^2}{1632}\times2000 = 24632$ mm.

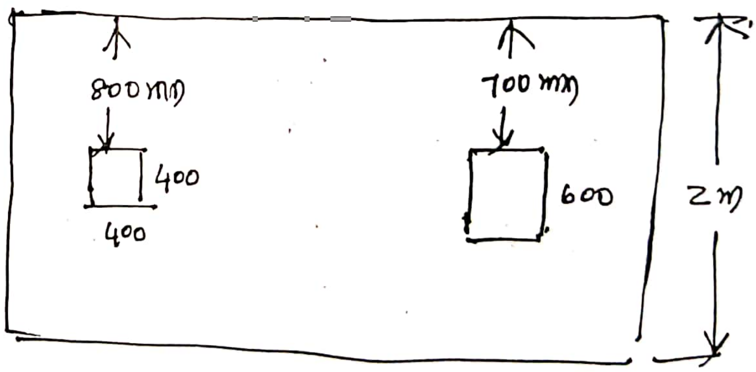
Hence provide 16 ϕ at ~~240~~ 220 mm c/c at bottom.

Step. 6 (Transverse steel)

Cantilever in the transverse direction = 800 mm (Under Column C_1)

Upward pressure = 249.2 kN/m^2.

Considering 1m width,



$$M_{max} = 249.2\times0.8^2 = 159.49\ kNm.$$

Factored moment = 159.49 × 1.5 = 239.23 kNm.

Eff. depth for transverse direction = $620 - \frac{16}{2} = 612$ mm

Hence,

$$239.23\times10^6 = 0.87f_yA_{st}d\left(1-\frac{A_{st}}{bd}\times\frac{f_y}{f_{ck}}\right)$$

or $$239.23\times10^6 = 0.87\times415\,A_{st}\times612\left(1-\frac{A_{st}}{1000\times612}\times\frac{415}{20}\right)$$

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$$239.23\times10^{6} = 0.87\times415\,A_{st}\times612\left[1-0.333\times10^{-3}A_{st}\right]$$

$$\cancel{239.23\times10^{6} = 220962.6A_{st} - 0.073A_{st}^{2}}$$

$$\cancel{0.073A_{st}} + \cancel{220962.6}^{\,73.58}A_{st}^{2} + 239.23\times10^{6} = 0$$

$$-2,20,962\,A_{st}$$

$A_{st}^{2} - 3003.2\,A_{st} + 3.25\times10^{6}$; Solving, A_{st} = Very less.

$$A_{st}(\text{Min}) = \frac{0.12}{100}\times1000\times680 = \cancel{2442.4}\ 816\,mm^{2}$$

$$\text{spacing of } 16\phi = \frac{\pi/4\times16^{2}}{816}\times1000 = 246.4\,mm.$$

Provide 16ϕ @ 220mm c/c in the lateral direction.

Under column C_2

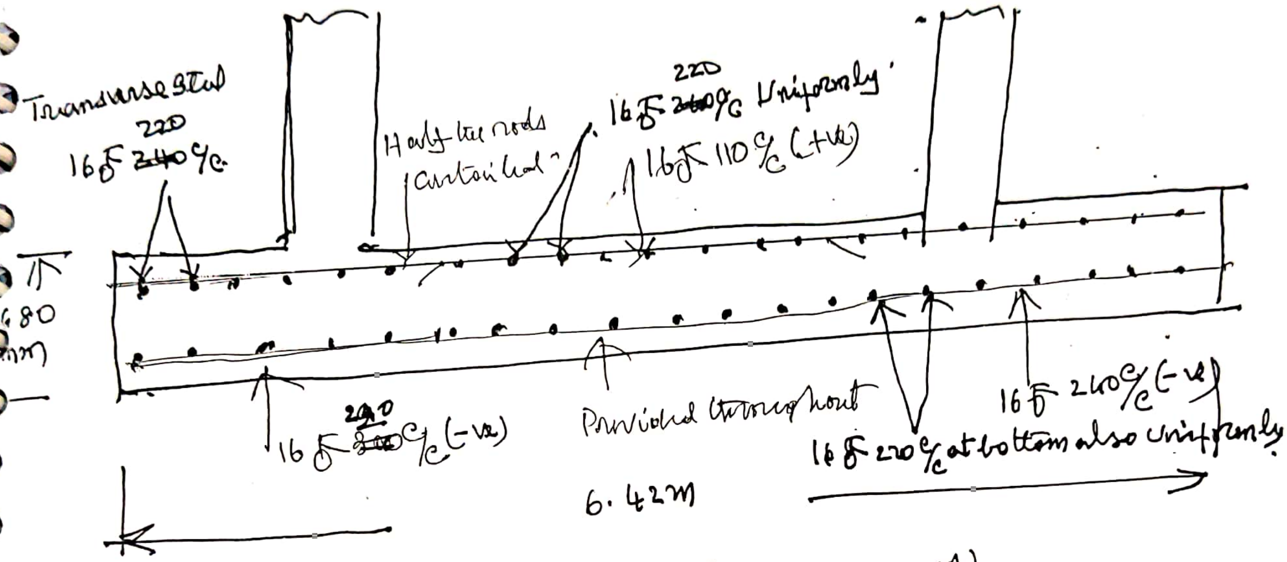
Length of the Cantilever projection is less and hence B.M is less.

~~Pa~~ Same ~~lateral~~ transverse reinforcement is provided at 'C_2' also

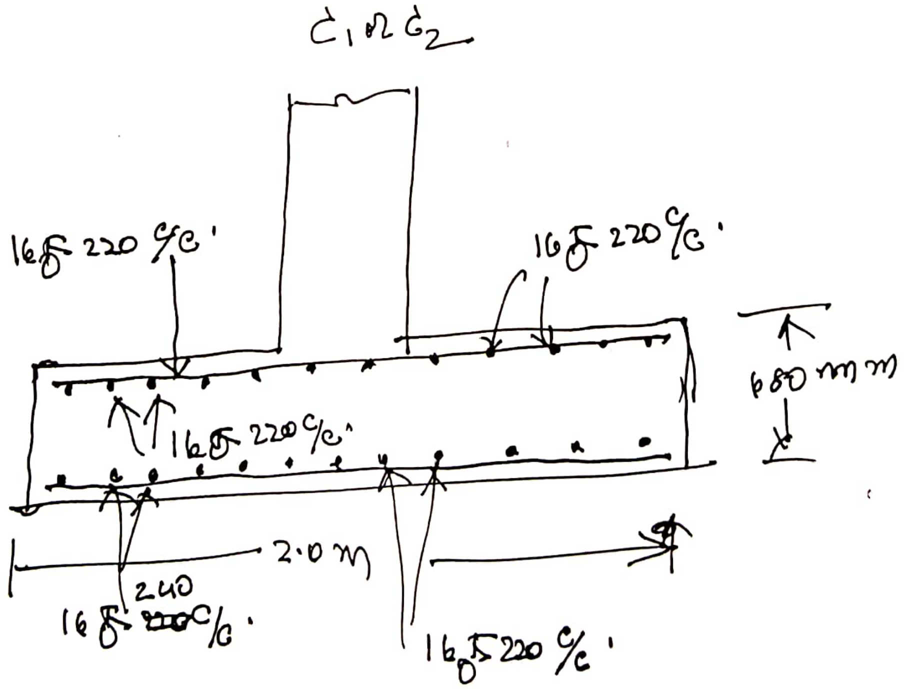
As the depth was decided from shear considerations which is more than the depth from B.M considerations, the section is safe in shear. Shear reinforcement is not necessary.

~~SET~~

STEP-7 REINFORCEMENT DETAILS.



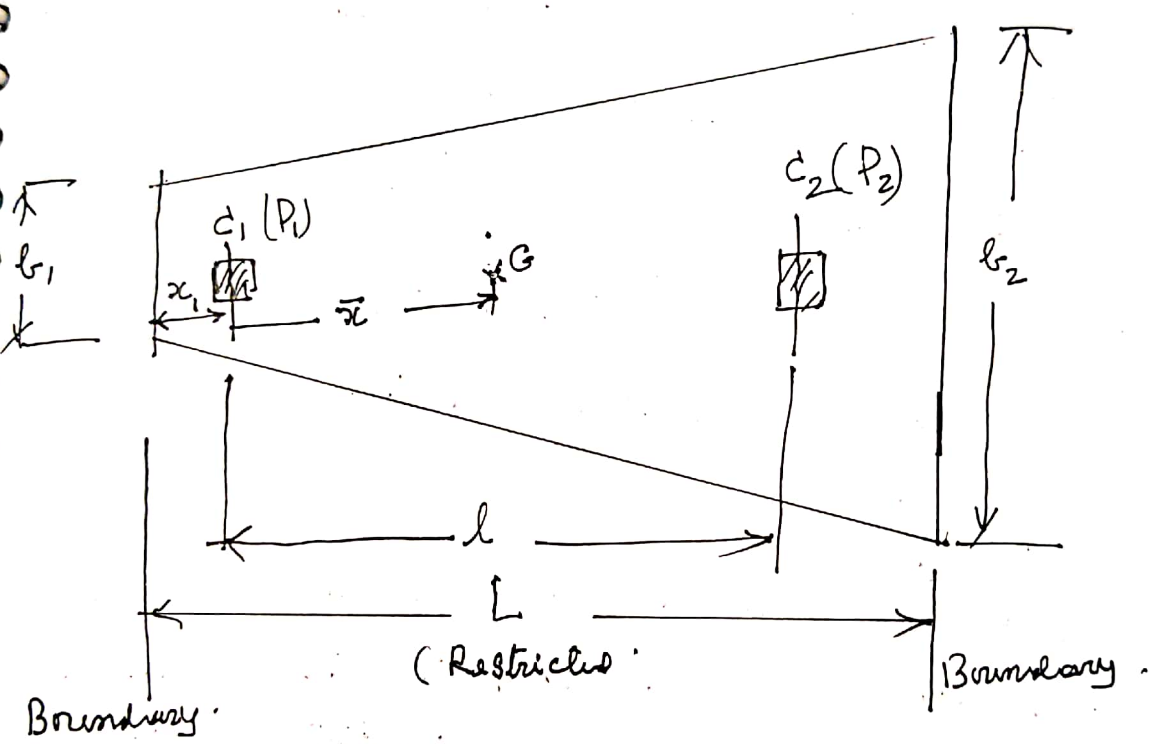
CROSS SECTION. (LONGITUDINAL)



CROSS SECTION (TRANSVERSE)

TREPEZOIDAL COMBINED FOOTING.

If there is a restriction of space on both sides of the columns for which combined footing is to be designed, then the length of footing is fixed. In such cases, it is not possible to coincide the c.g of the loads with c.g of the footing. In such cases trapezoidal footing is provided. In the figure



The distance of c.g of P_1 and P_2 from the centre of $P_1 = \bar{x}$

$$\bar{x} = \frac{P_2 \cdot l}{(P_1+P_2)}, \quad A = \frac{1.1\,(P_1+P_2)}{SBC},$$

Area of the combined footing $= \dfrac{(b_1+b_2)}{2} \cdot L = A$. (1)

Distance of c.g from the edge of the footing near column 'C_1' $= x_1 + \bar{x}$ (2)

By trapezium formula, distance of c.g from the same edge

$$= \frac{L}{3} \frac{(b_1 + 2b_2)}{(b_1 + b_2)} \qquad (3)$$

Equating (2) and (3). $\frac{L}{3} \frac{(b_1 + 2b_2)}{(b_1 + b_2)} = x_1 + \bar{x} \qquad (4)$

Eq. (1) is $\frac{(b_1 + b_2)}{2} \cdot L = A \qquad (1)$

(Again)

Solving eq. (1) and (4) for b_1 and b_2, the footing dimensions are found.

The net upward pressure of the soil = $q_n = \left(\frac{P_1 + P_2}{A}\right) kN/m^2$

Knowing the upward soil pressure and dimensions of the footing S.F and B.M diagrams can be constructed. Then, design is carried out as usual by the Limit State Method.

DESIGN EXAMPLE:

There are two columns 400x400 mm and 600x600 mm carrying loads of 600 kN and 1000 kN respectively. The centre to centre distance of the columns is 4 m. The property line is at a distance of 0.3 m from 600 kN column. The length of the footing is to be restricted to 5 m. Prepare the layout of the footing and show the loading on longitudinal section. SBC of the soil is 150 kN/m².

SOLUTION

Area of the footing required 'A' $= \frac{(600+1000)1.1}{150} = 11.733 m^2$

Distance of c.g of the loads from the lighter Column (400 × 400 mm)

is $\bar{x} = \frac{1000 \times 4 + 600 \times 0}{1000 + 600} = 2.5 m.$

Distance of the boundary from the lighter Column $= 0.3 m = x_1$

Distance of c.g from property line $= \bar{x} + x_1 = 2.5 + 0.3 = 2.8 m.$

Length of footing = 5m (Restricted).

If b_1 and b_2 are the widths of the footing at the two ends,

Distance of the c.g of the trapezium from smaller side 'b_1' is

$= \frac{L}{3} \cdot \frac{(b_1+2b_2)}{(b_1+b_2)} = \frac{5}{3} \frac{(b_1+2b_2)}{(b_1+b_2)}$, To make it equal to 2.8 m,

$\frac{5}{3} \frac{(b_1+2b_2)}{b_1+b_2} = 2.8 m$, $(b_1+2b_2) = \frac{3}{5}(b_1+b_2) \times 2.8$

or $b_1+2b_2 = 1.68(b_1+b_2)$ or $b_1+2b_2 = 1.68 b_1 + 1.68 b_2$

$2b_2 - 1.68 b_2 = 1.68 b_1 - b_1$ or $0.32 b_2 = 0.68 b_1$

$b_2 = \frac{0.68}{0.32} b_1$ or $b_2 = 2.125 b_1$ (1).

Area of trapezium $= \frac{(b_1+b_2)}{2} L = A.$

or $\frac{(b_1+b_2)}{2} \times 5 = 11.733.$ (2)

By putting $b_2 = 2.125\, b_1$ from eq. (1) into eq. 2.

$$\frac{b_1 + 2.125\, b_1}{2} \times 5 = 11.733, \quad \text{Hence } b_1 = 1.5\,m.$$

$b_2 = 2.125 \times 1.5 = 3.2\,m.$, Variation $= \frac{(3.2 - 1.5)}{5} = 0.34/m.$

Intensity of upward soil pressure $= \frac{600 + 1000}{\frac{(1.5+3.2)}{2} \times 5} = 136.17\,kN/m^2$

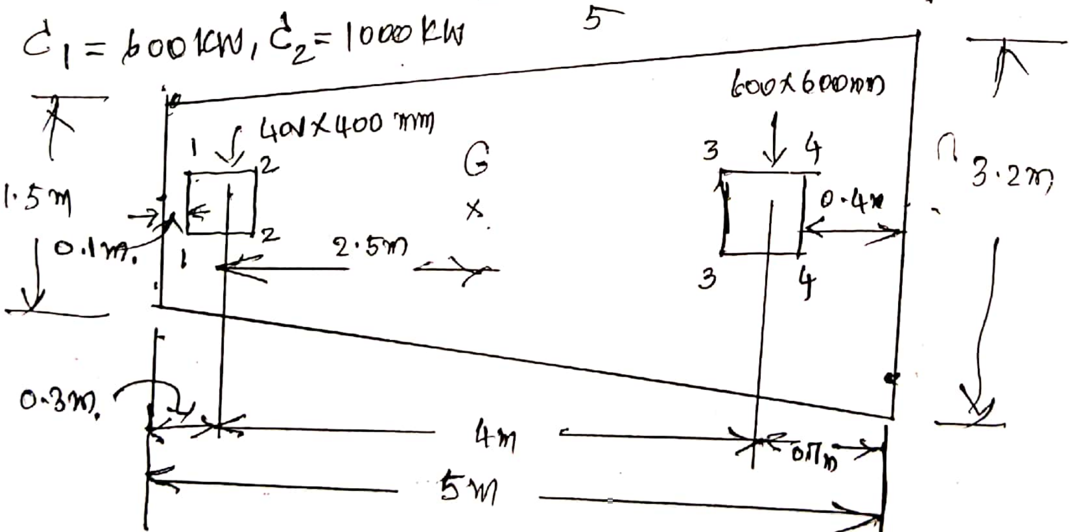
Soil pressure/unit length at the end of $b_1 = 1.5 \times 136.17 = 204.25\,kN/m$

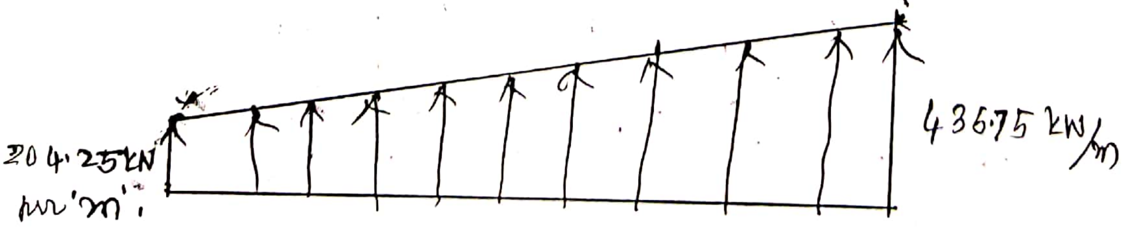
" at the end of $b_2 = 3.2 \times 136.17 = 435.75\,kN/m.$

(The details are shown in the fig.)

Variation of soil pressure $= \frac{(436.75 - 204.25)}{5} = 46.5\,kN/meter.$

$C_1 = 600\,kN$, $C_2 = 1000\,kN$





To draw the SFD,

Soil pressures at

1-1 (at 0.1m from the left edge) $= 204.25 + 0.1 \times 46.5 = 205.715$ kN/m.

2-2 (at 0.5m) $= 204.25 + 0.5 \times 46.5 = 227.5$ kN/m.

From right hand side,

At 3-3 $= 436.75 - \cancel{0.9}^{\,1.0 \times 46.5} = 390.25$ kN/m.

At 4-4 $= 436.75 - 0.4 \times 46.5 = 416.15$ kN/m.

S.F^s: (Average pressures are considered)

At 1-1 $= \dfrac{(204.25 + 205.715)}{2} \times 0.1 = 204.98 \times 0.1 = 20.5$ kN

At 2-2 $= \dfrac{204.25 + 227.5}{2} \times 0.5 - 600 = 215.875 \times 0.5 = \cancel{107.94\text{ kN}} - 600 = -492.06$ kN.

At 3-3 $= -\dfrac{(\cancel{390.25} + \cancel{416.15})^{\,390.25 + 436.75}}{2} \times 1.0 + 1000 = +\cancel{595.8}^{\,586.50}$ kN

At 4-4 $= -\dfrac{(416.15 + 436.75)}{2} \times 0.4 = -170.98$ kN.

B.M^S.

At 1-1 = Ave. pressure $\times \frac{0.1^2}{2} = 204.98 \times \frac{0.1^2}{2} =$ ~~10.25~~ 1.02 KNm. (−)

4-4 = $\frac{(418.15 + 436.75)}{2} \times \frac{0.4^2}{2} =$ 34.2 KNm. (−).

Total variation over 3.5 m = ~~565~~ $\frac{586.50 + 492.06}{3.5}$ = 308.16 m.

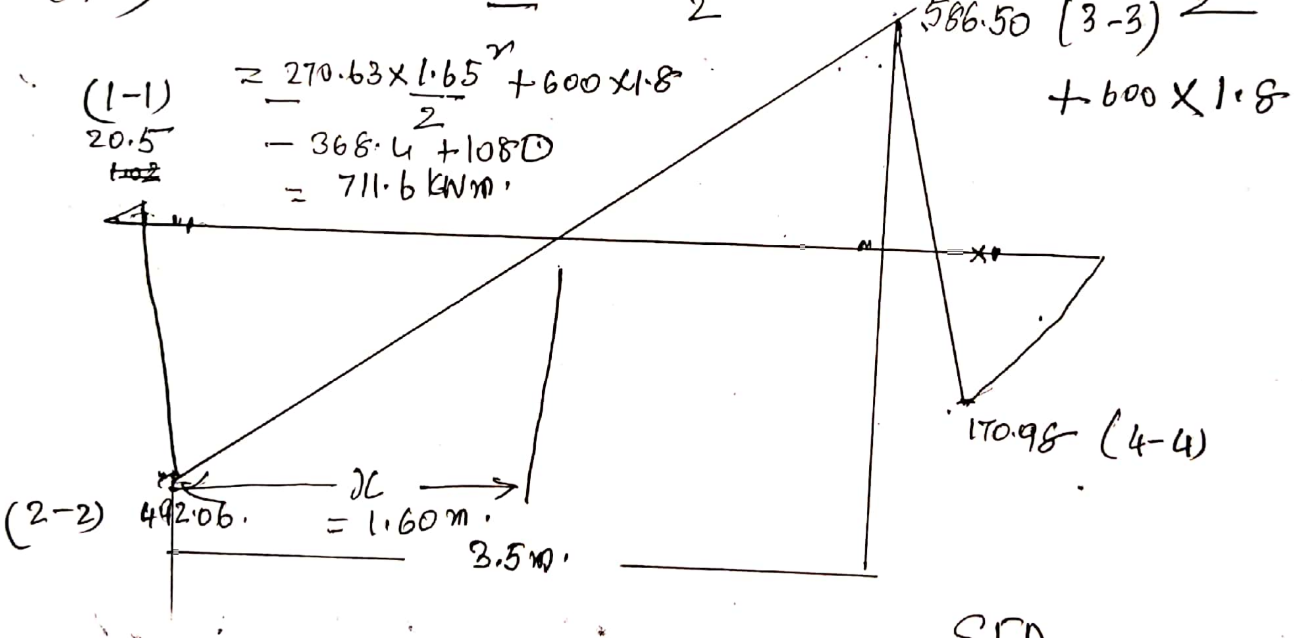
Hence, S.F = 0 at $\left(\frac{492.06}{308.16}\right)$ m from 2-2. = 1.6 m from 2-2.

Max. B.M (1.60 m from 2-2) (+ve) = $\frac{[204.25 + 204.25 \times (1.60 + 0.5)]}{2} \times \frac{(1.60+0.5)^2}{2}$ + 600 × 1.8

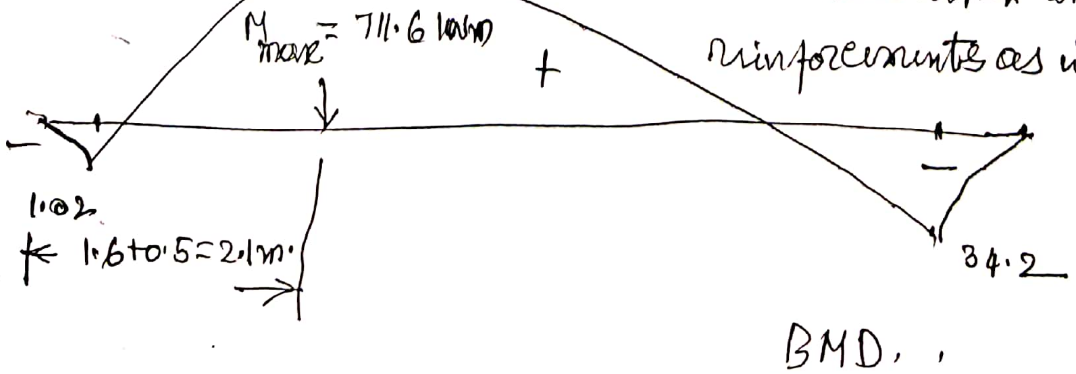
= $270.63 \times \frac{1.65^2}{2}$ + 600 × 1.8

− 368.4 + 1080

= 711.6 KNm.



SFD



Determine the depth and reinforcements as in the ~~previous~~ previous case

BMD.

CANTILEVER RETAINING WALL.

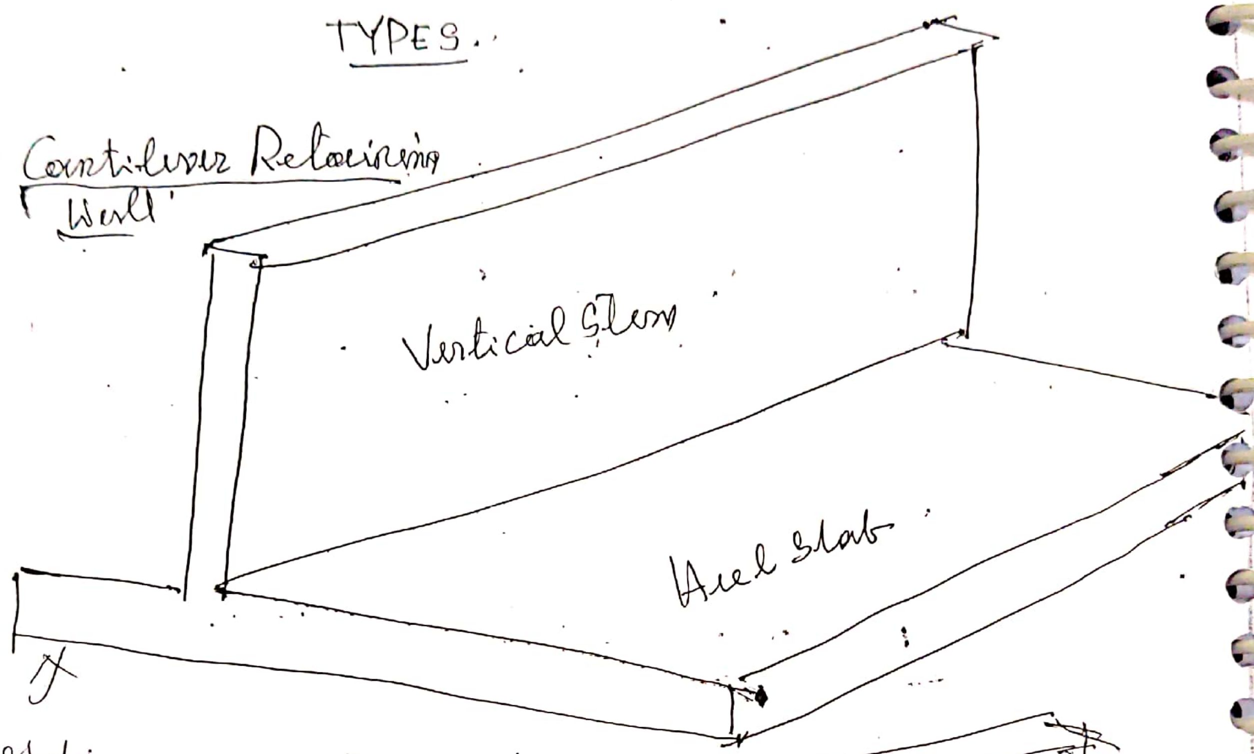
Retaining walls are provided to retain back fill consisting of earth mass to a greater height. They are used in railway and road projects, building ~~at~~ basements etc. The back fill usually consists of gravelly or sand filling. Proper draining arrangements are to be made to drain away water to avoid water pressure on the wall.

A cantilever retaining wall consists of a vertical stem and a base slab below the G.L. The base slab consists of front toe portion and back heal portion. All the three components vertical stem, toe and heal slabs are designed as cantilevers and hence the wall is termed as a cantilever retaining wall. The vertical stem is under the action of earth pressure exerted by the back fill in the lateral direction. The heal slab is under the action of vertical load of soil and the upward soil pressure from the base. The toe portion is mainly under the action of soil pressure ~~at~~ from the base. In the case of roads or railway tracks laid on the back fill, there will be surcharge loads to be considered in the design.

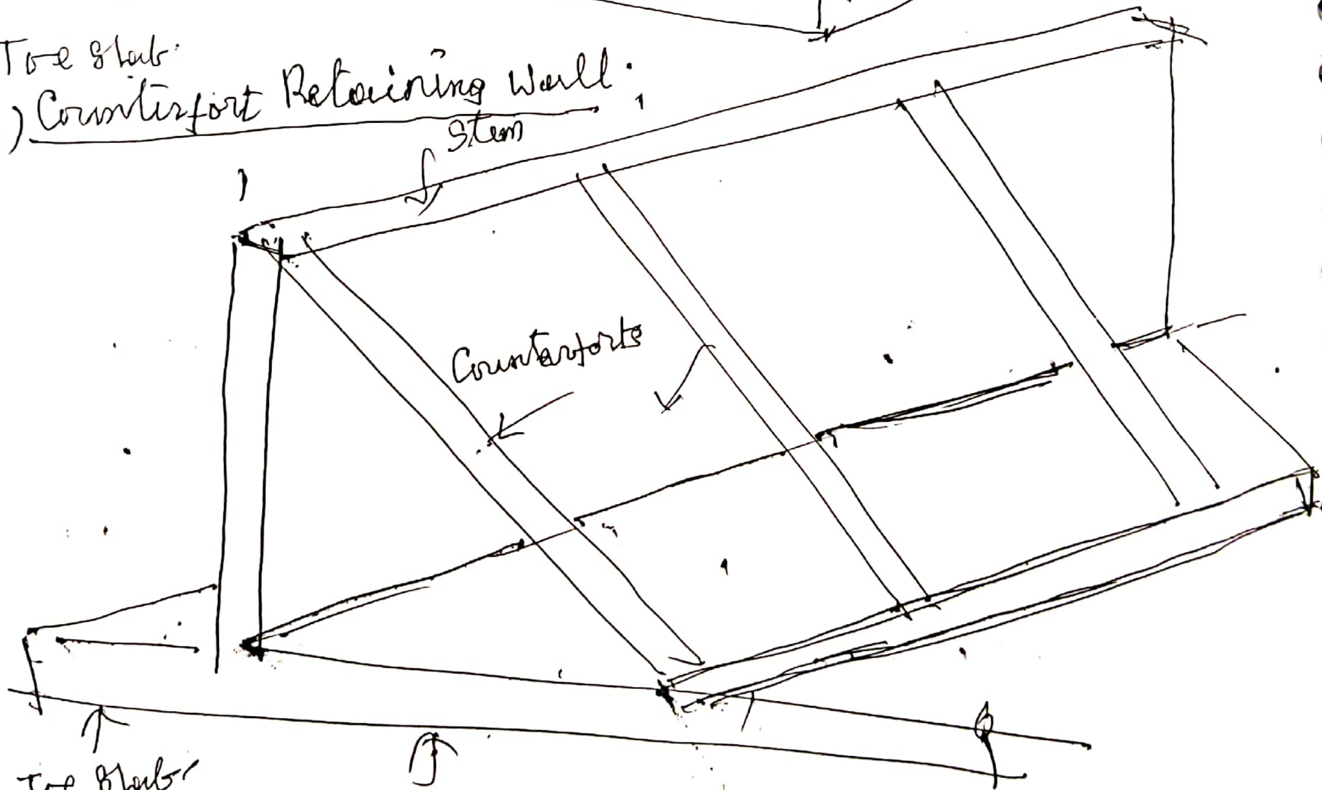
RETAINING WALLS.

TYPES.

a) Cantilever Retaining Wall



b) Counterfort Retaining Wall



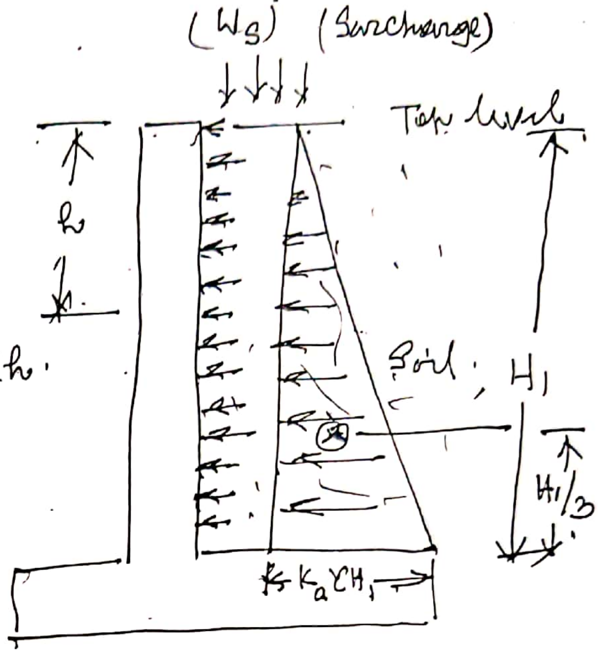
USUAL SIZES.

If 'H' is the total height of the wall above the foundation, usually,

(B) Base width = 0.48H to 0.56H, Toe projection = $\frac{B}{3}$.
Thickness of the base slab = H/12, Top thickness of the stem = 150 to 300 mm

a) Estimation of lateral Soil pressure on the Wall.

on the Vertical Stem



By Rankine's formula,

at any height of 'h' from top,

the lateral earth pressure $p_a = K_a \gamma h$.

γ: Unit Weight of Soil

K_a: Rankine's Coefficient given by

$K_a = \frac{1-\sin\phi}{1+\sin\phi}$, ϕ: Angle of internal friction of Soil.

The pressure varies linearly in the form of a triangle

Total force acting (Area of the triangle) $= \frac{1}{2} K_a \gamma H_1 \times H_1 = \frac{1}{2} K_a \gamma H_1^2 = P$

acting through the c.g at $\frac{H_1}{3}$ above the base

The max. moment produced at the base of the wall $= \frac{1}{2} K_a \gamma H_1^2 \times H_1/3$.

$= \frac{1}{6} K_a \gamma H_1^3$.

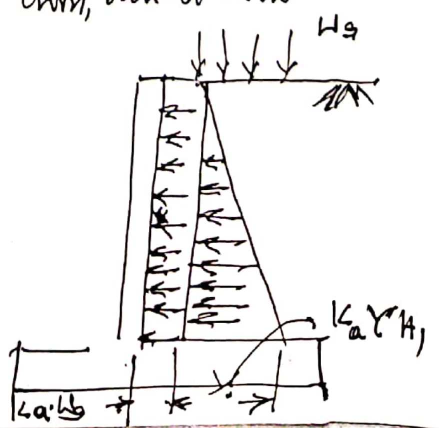
If there is Surcharge at top on the Soil (W_s) then, due to this

there is a Constant lateral pressure $= K_a W_s$

In this Case the total moment about the base

$= \frac{1}{6} K_a \gamma H_1^3 + K_a W_s H_1 \cdot \frac{H_1}{2}$

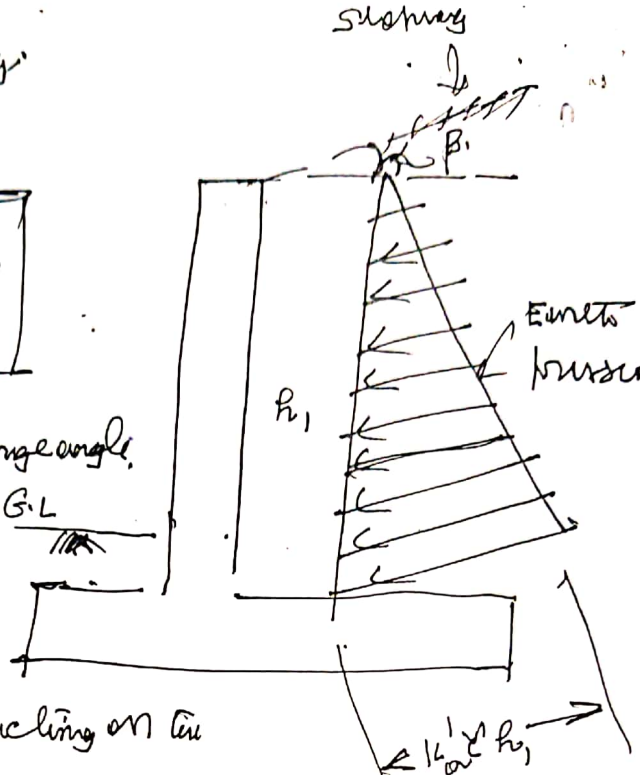
$= \left[\frac{1}{6} K_a \gamma H_1^3 + K_a \frac{W_s H_1^2}{2}\right]$



If the backfill is Sloping:

In this Case, Rankine's Coeff: is

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



β: Angle of slope also known as Surcharge angle.

ϕ: Angle of internal friction

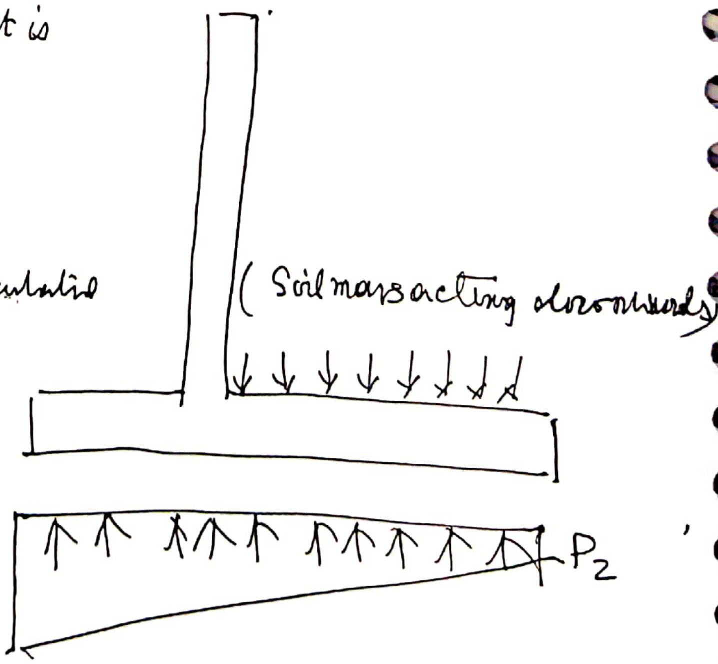
$$p_a = K_a' \gamma \cdot h_1$$

The earth pressure (passive) acting on the front side of the wall over a small height below the ground level is neglected.

Soil Reaction on base (Toe and Heel.)

on the heel slab weight of soil mass over it is Considered.

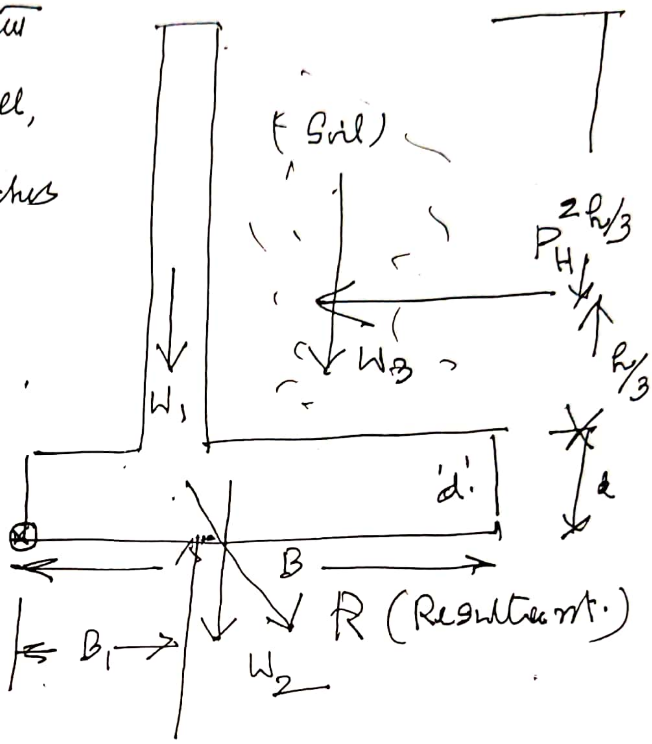
on the whole base slab, net upward Soil pressure will be acting. This is Calculated by Considering the stability of the retaining wall.



Checks for stability

By taking the moments about the edge of the base slab of all the forces acting at the toe end of the retaining wall, the point where the resultant touches the bottom is given by

$$B_1 = \frac{\sum \text{Moments}}{\sum W \text{ (Vertical)}}$$



Hence eccentricity of this point from the centre of the base slab is given by $e = \left(\frac{B}{2} - B_1\right)$

The following are the checks to be conducted for the stability of the wall.

1) Check for Middle third rule.

The resultant should pass through the middle third of the width of the base slab to avoid tension (i.e) $e = \left(\frac{B}{2} - B_1\right) < \frac{B}{6}$.

2) Check for over turning

The sum of all the vertical forces like self weight of the wall, soil load on the heel produces a stabilising moment about the toe, whereas the moment caused by the horizontal soil pressure produces moment in the opposite direction and it tries to overturn the wall about the toe. Hence we have,

Factor of safety against overturning = $\frac{\text{Stabilising Moment } (M_S)}{\text{overturning moment } (M_O)}$

> 1.4

(As per IS 456-2000, only 0.9 × characteristic dead load should be considered for this purpose.)

Hence $\frac{0.9 M_S}{M_O} > 1.4$.

Check for Sliding

The earth pressure from the back fill tries to push and slide away the wall. The factor of safety against sliding is

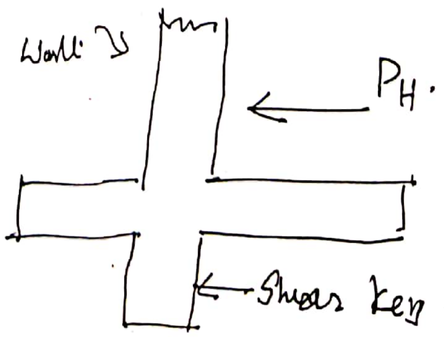
Hence $\frac{0.9\mu(\Sigma W)}{P_H} > 1.4$ (μ is Coeff. of friction

If this check fails then a shear key is to be provided for proper grip.

Depth of footing

By Rankine's formula, the depth of foundation below ground level is

$$d_{(min)} = \frac{q_0}{\gamma} \cdot \left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$



DESIGN EXAMPLE.

Design a Cantilever retaining wall to retain earth over a height of 3.5 m above G.L. The top of the earth is horizontal; Density of Soil is $18 kN/m^3$, Angle of internal friction 'ϕ' = $30°$

SBC of the Soil is $200 kN/m^2$, Coeff. of friction 'μ' = 0.50.

Use M20 Concrete and Fe 415 steel.

SOLUTION.

Earth pressure Coefficient and depth of foundation

Coeff. of active earth pressure $k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{2} \times \frac{2}{3} = \frac{1}{3}$.

Minimum depth of foundation by Rankine's formula is $\frac{q_0}{\gamma} \cdot k_a^2$

$= \frac{200}{18} \times \left(\frac{1}{3}\right)^2 = 1.23 m$ below G.L, Provide 1.25 m depth below G.L

Hence, total height of the wall $= 3.5 + \cancel{1.5}\ 1.25 = \cancel{5.0 m}$ 4.75 m.

Preliminary Dimensions of the wall.

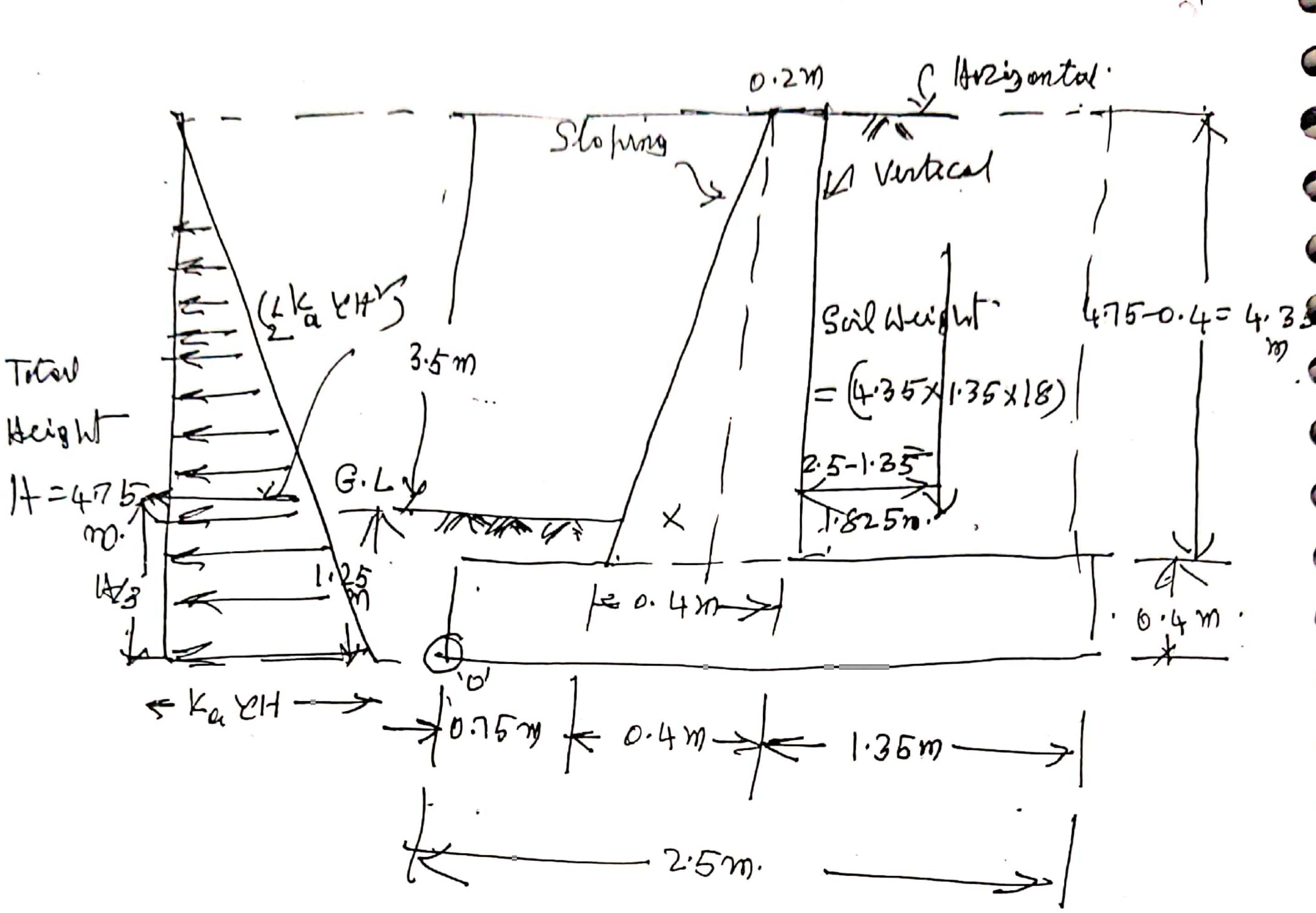
$B = 0.48H$ to $0.56H$ = 2.375 m to 2.66 m, try 2.50 m.

The projection of toe $= \frac{B}{3} = \frac{2.50}{3} = 0.81 m$, try 0.75 m.

Thickness of Stem $= \frac{H}{12} = \frac{4.75}{12}$ = Say 0.4 m at bottom (also provide the same for base slab)

Let the top width of the Stem = 0.2 m, The stem tapers down to 0.4 m.

Hence, the wall has dimensions as shown in the figure.



Earth Pressure.

P_H = Total earth pressure = Area of the triangle $= \frac{1}{2} K_a \gamma H^2$

$K_a = \frac{1}{3}$ $(\phi = 30^\circ)$. (acting at $\frac{H}{3}$ above the base.)

Hence, overturning moment about the base $= \frac{1}{2} K_a \gamma H^2 \times \frac{H}{3}$ $\cancel{= 67.688}$.

Substituting $M_O = \underbrace{\left[\frac{1}{2} \times \frac{1}{3} \times 18 \times 4.75^2\right]}_{(67.688)} \times \frac{4.75}{3} = 67.688 \times \frac{4.75}{3} = 107.17$ KNm

For vertical forces acting on the wall, refer to the table.

Table (Stability Calculations)

Sl. No	Component	Weight in kN	Dist. of C.G from toe end.	Moment
1	Weight of Back fill	$W_1 = 1.35 \times 4.35 \times 18 = 105.7 kN$	1.825 m	192.90 (Anticlockwise) kNm
2.	Rectangular Portion of Stem.	~~0.75+0.4-0.1~~ $0.2 \times 4.35 \times 25 = 21.75$ kN.	$0.75 + 0.4 - 0.1 = 1.05$	22.84 kNm.
3.	Triangular Portion of Stem.	$\frac{1}{2} \times 0.2 \times 4.35 \times 25 = 10.88$ kN.	~~0.75+~~ $0.75 + \frac{2}{3} \times 0.2 = 0.88$	9.61 kNm
4.	Base Slab.	$0.4 \times 2.5 \times 25 = 25$ kN. 163.33 kN	$\frac{2.5}{2}$	31.25 kNm. 256.60 kNm.
5	Over turning Moment	—	—	107.17 kNm.

$\Sigma W = 105.7 + 21.75 + 10.88 + 25.0 = 163.33$ kN.

Factor of Safety against overturning $= \frac{0.9 \times 256.6}{107.17} = 2.15 > 1.40$ (As per IS 456)

Safety against Sliding $= \frac{0.9 \Sigma W \times \mu}{P_H} = \frac{0.9 \times 163.33 \times 0.5}{67.688} = 1.09 < 1.40$

Hence, additional safety against friction is provided by a shear key of suitable dimensions directly under the stem below the heel slab.

Upward Soil pressure below the heel slab:

Total moment about the toe end 'O' = 256.60 − 107.17 = 149.43 kN·m (clockwise)

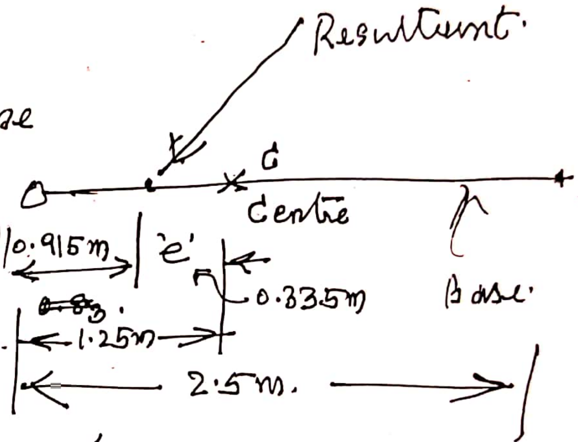
Total vertical load = ΣW = 163.33.

Hence, distance from 'O' where the resultant intersects the base.

$$= \frac{\text{Net moment}}{\text{Total vertical load}} = \frac{149.43}{163.33} = 0.915\,m \text{ from 'O'}.$$

Hence, eccentricity w.r.t. the centre of the base

$= 1.25 - \cancel{0.83} = \cancel{0.42}\,m.$
$0.915 = 0.335\,m$



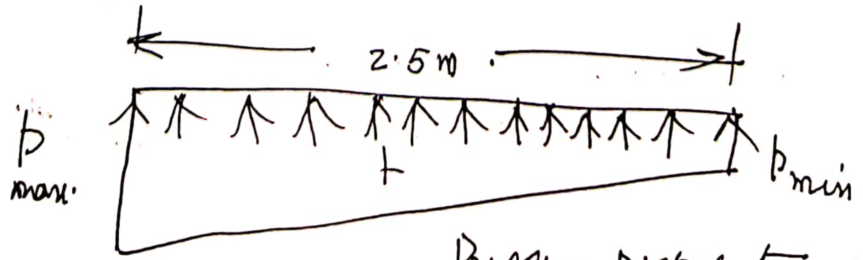
Hence, upward pressures at the base $= \frac{\Sigma W}{B}\left[1 \pm \frac{6e}{B}\right]$

$$= \frac{163.33}{2.5}\left[1 \pm \frac{6 \times 0.335}{2.5}\right] = 65.332\left[1 \pm 0.804\right]$$

Hence $p_{max} = 65.332 \times 1.804 = 117.86\ kN/m^2$ (+), $p_{min} = 65.332 \times 0.196 = \cancel{13.39}$ 12.80 (+) kN/m^2.

Maximum = 117.86 < Safe B.C of 150, Hence, it is within the limits.

Min, p_{min} = + ~~13.39~~ 12.80 kN/m^2. Hence, there is no tension on the base.



Pressure Distribution at base.

DESIGN OF STEM.

Stem is designed as a Cantilever Slab of height 4.35 m above the base.

Pressure varies from 0.0 at top to $K_a \gamma h$ at base horizontally.

(i) ~~0.0 at top~~ Hence total pressure $= \frac{1}{2} \times (K_a \gamma h) \cdot h = \frac{K_a \gamma h^2}{2}$ (Area of triangle)

Hence, the Max. moment at the base $= \frac{K_a \gamma h^2}{2} \times \frac{h}{3} = \frac{K_a \gamma h^3}{6}$

$= \frac{1}{6} \times \frac{1}{3} \times 18 \times 4.35^3 = 82.31$ kNm.

Factored Moment $= M_u = 1.5 \times 82.31 = 123.47$ kNm.

Hence, $0.138 \times 20 \times 1000 \times d^2 = 123.47 \times 10^6$

Solving, $d = 211.50$ mm., Provide $d = 350$ mm with $D = 400$ mm.
Provide $D_{overall} = 200$ mm at top.

Then, we have

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

Hence, $123.47 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \left[1 - \frac{A_{st}}{1000 \times 350} \times \frac{415}{20}\right]$

Solving for A_{st}, we get, $A_{st} = 1041$ mm²

Using 12 ϕ, spacing $= \frac{\pi/4 \times 12^2}{1041} \times 1000 = 108$ mm, Provide at 100 c/c. Vertically.

A_{st} (act.) $= \frac{1000}{100} \times \pi/4 \times 12^2 = 1130$ mm²/m.

Distribution steel at 0.12% $= \frac{0.12}{100} \times 1000 \times \frac{(200 + 400)}{2} = 360$ mm².
(Avg thickness)

Spacing of 8 ϕ $= \frac{\pi/4 \times 8^2}{360} \times 360 = 558$ mm, i.e. $\frac{558}{2}$ mm on each face

Hence, provide 8 ϕ C 270 c/c on each face horizontally.

$\frac{1}{3}$ of the vertical bars at 1.5 m above the base and another $\frac{1}{3}$ at 3 m from the base may be curtailed.

Shear force at the base = Total horizontal force = 107.17 KN.

Factored S.F = 1.5 x 107.17 = 160.75 KN.

Hence, Ave. shear stress = $\frac{160.75 \times 10^3}{1000 \times 400} = 0.4 N/mm^2$ (τ_c)

Percentage steel = $\frac{\cancel{\frac{\pi}{4} \times 12^2 \times 100}}{\cancel{400 \times 100}} = \frac{1130}{400 \times 1000} \times 100 = 0.283$ Percent.

$\tau_c = 0.4$ < Average stress. Hence, no shear reinf. is necessary.

TOE SLAB,

From the pressure diagram,

Pressure on the face of the toe

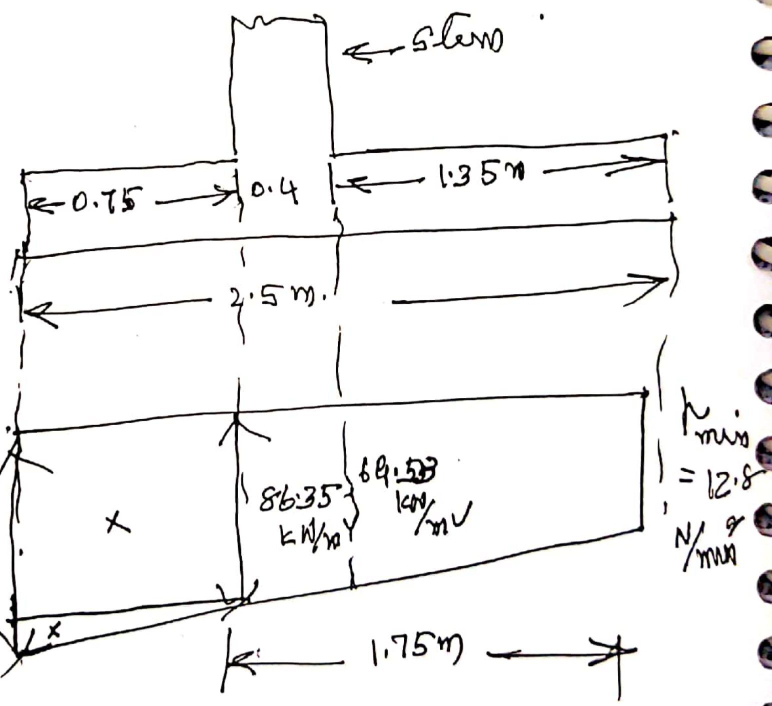
$= 12.8 + \frac{(117.86 - 12.8)}{2.5} \times 1.75$

$= 86.35 \, KN/m^2$.

Max. moment about the face of the stem $= 86.25 \times \frac{0.75^2}{2}$

$+ \frac{1}{2}(117.86 - 86.25) 0.75 \times \frac{2(0.75)}{3}$

$= 30.19$ KNm,



Factored moment = 1.5 x 30.19 = 45.285 KNm.

'd' provided was 350 mm. (Self wt of toe slab is ignored as it is of a small length)

Hence steel is given by

$$45.215\times10^6 = 0.87\times415\times A_s\times350\left[1-\frac{A_{st}}{1000\times350}\times\frac{415}{20}\right].$$

$$358.36 = A_{st}\left[1-\frac{A_{st}}{16867.50}\right]$$

Solving for A_{st}', $A_{st} = 366\ mm^2$

$A_{st}\ (Min) = \frac{0.12}{100}\times1000\times\underset{(D)}{400} = 480\ mm^2$, Provide $A_{st}(Min)$.

Spacing of $12\phi = \frac{\frac{\pi}{4}\times12^2}{480}\times1000 = 235\ mm$

Provide 12ϕ 300 c/c both directions.

HEEL SLAB

Width of the heel slab = 1.35 m.

Pressure varies from $12.8\ kN/m^2$ to $69.53\ kN/m^2$.

Weight of the back fill soil on the heel slab $= \gamma H_1 = 18\times4.35 = 78.3\ kN/m$.

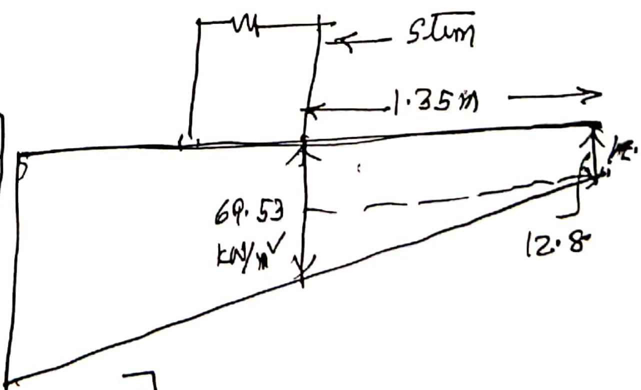
Self weight $= 0.4\times1\times25 = 10\ kN/m$.

Total downward load $= 78.3+10 = 88.3\ kN/m$.

Hence $M_{more} = \frac{88.3\times1.35^2}{2}$

$$-12.8\times\frac{1.35^2}{2}-\frac{1}{2}\left[(69.53-12.8)\times1.35\times\frac{1.35}{3}\right]$$

$= 51.57\ kNm$.

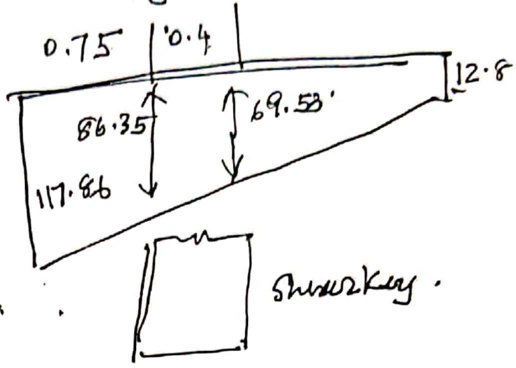


A_{st} is given by

$$51.57\times10^6 = 0.87\times415\times A_{st}\times350\left[1-\frac{A_{st}}{1000\times350}\times\frac{415}{20}\right],\ So$$

$$408.1 = A_{st}\left[1 - \frac{A_{st}}{16867.5}\right], \quad A_{st} = 418 \text{ mm}^2$$

· Provide Min. A_{st} of 12 ϕ 225 c/c.



SHEAR KEY

Provide 200 mm deep Shear key

~~SKETCHES OF REINFORCEMENT~~.

Pressure on the face of Shear key = 86.35 kN/m.

Coeff. of passive earth pressure $K_p = \frac{1}{K_a} = \frac{1}{1/3} = 3$

If 'a' is the projection below the base slab of the Shear key, resistance offered to passive earth pressure = K_p · vertical pressure

Vertical pressure = 86.35 × 'a' where 'a' is the projection

Hence, resistance = K_p × 86.25 × a = 3 × 86.35 a = 259.05 a kN.

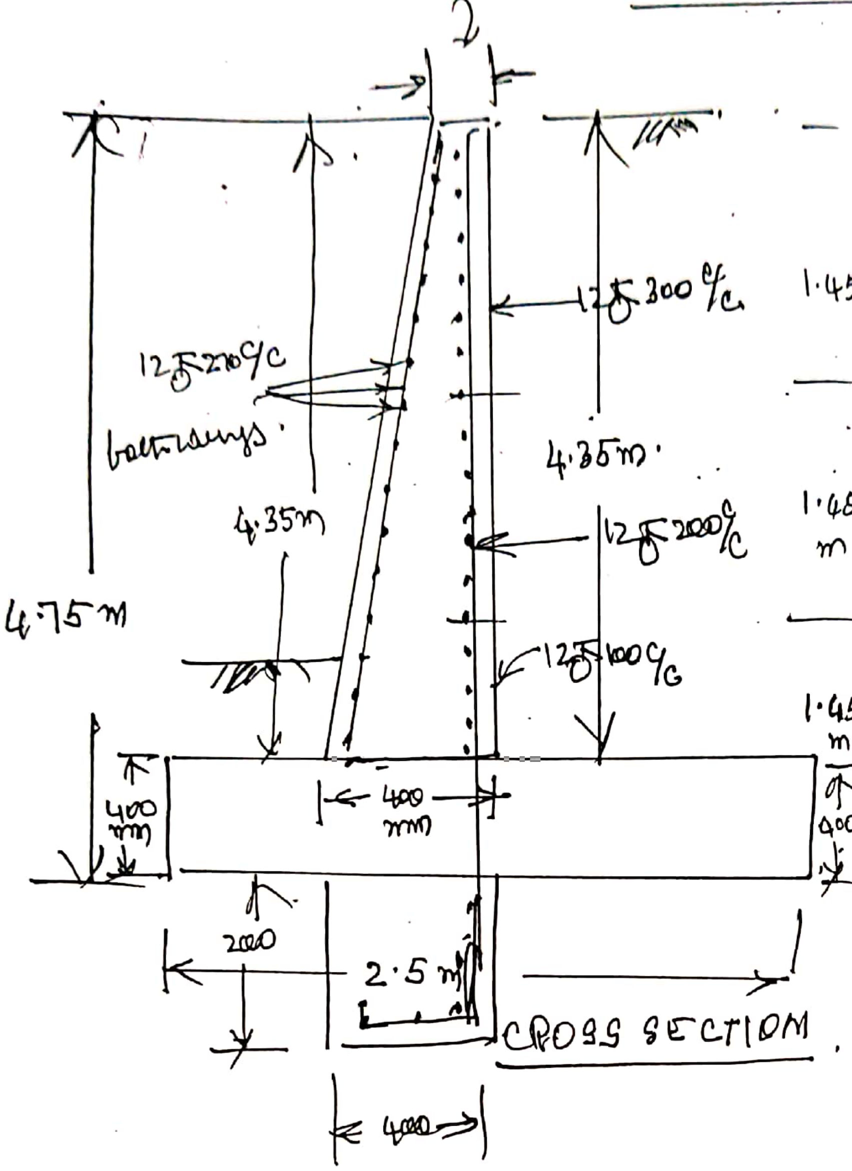
Hence, Factor of Safety against Sliding is

$$= F_2 = \frac{0.9\mu \Sigma W + 259.05 a}{\text{Horiz. pressure } 67.688} = 1.4$$

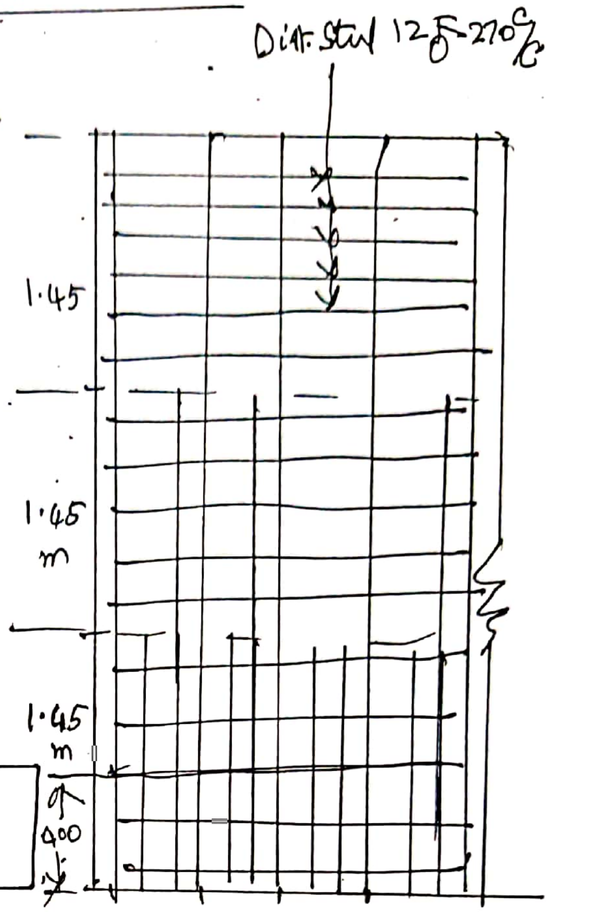
Hence, a = 0.085 m ≐ 85 mm

Provide 200 mm. Continue the reinforcement from the stem into the Shear Key.

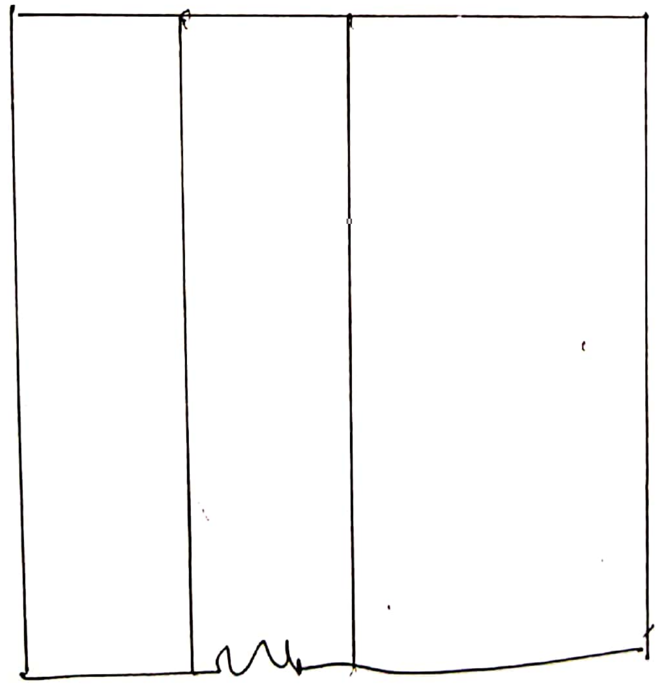
REINFORCEMENT DETAILS OF CANTILEVER RETAINING WALL.



CROSS SECTION



LONGITUDINAL SECTION



SECTIONAL PLAN.

DESIGN PRINCIPLES. COUNTERFORT RETAINING WALL

Stem

It is designed as a vertical continuous slab supported over the counterforts and reinforcement is provided accordingly.

HEEL slab.

It is designed as a horizontal continuous slab supported over the counterforts. Vertical loads are self weight, back fill and upward soil pressure.

Toe slab

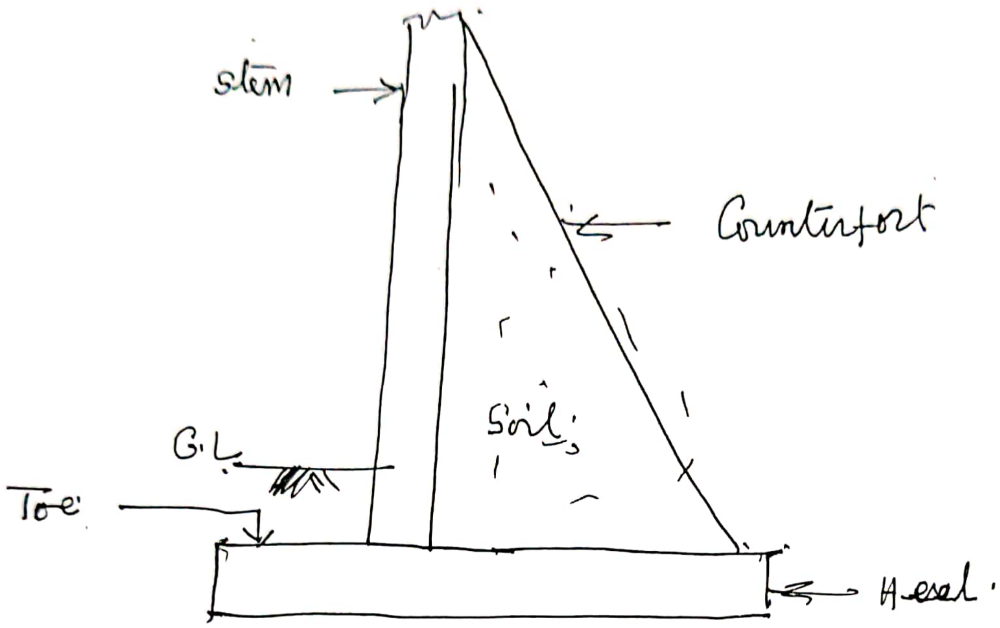
It is again designed as a cantilever against the upward soil pressure and the self weight.

Counterforts

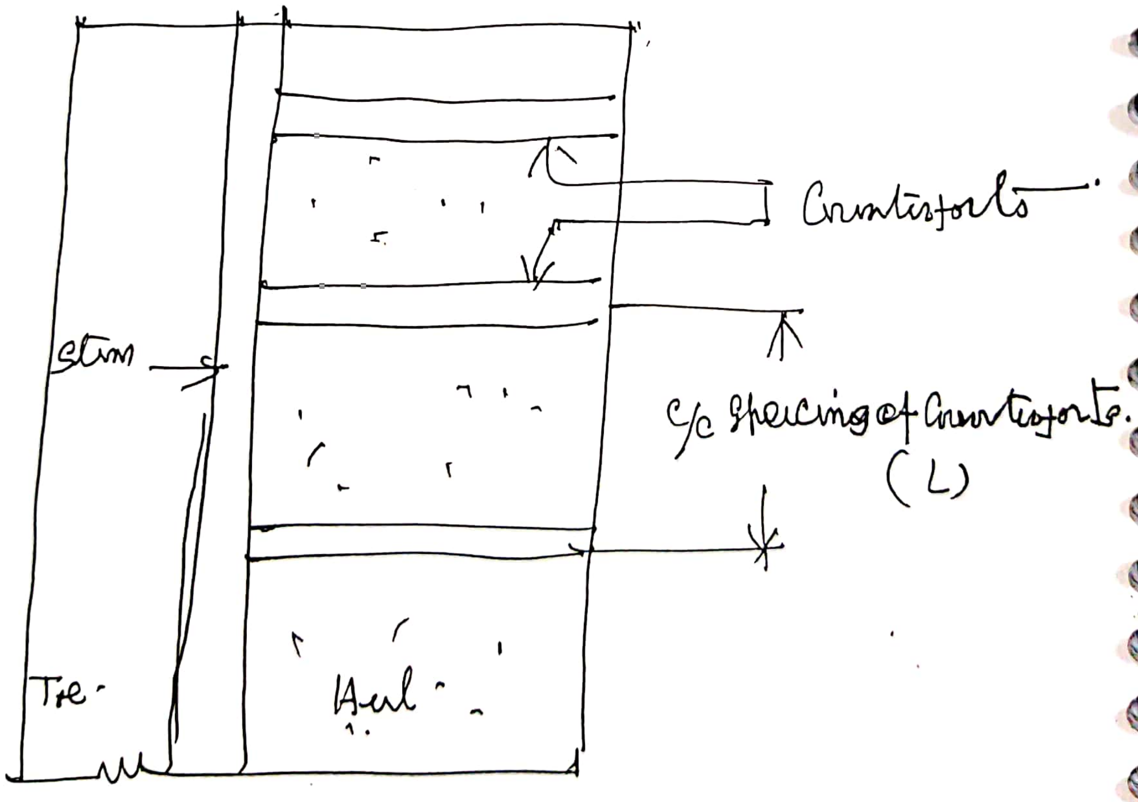
Usually they are provided at 3m to 3.5m spacing.

They are designed as a 'T' beam subjected to the load from the back fill acting in the lateral direction. Hence the max. B.M at the base $= \frac{1}{6} k_a \gamma h^3 \times L$ where 'L' is the spacing of the counterforts. The counterfort is provided with horizontal and vertical reinforcement so that it is anchored to the base slab and the stem.

COUNTERFORT RETAINING WALL.



FRONT VIEW



PLAN.

DESIGN EXAMPLE

Design a Counterfort retaining wall if the height of the wall above the ground level is 5.5 m. SBC of the soil is 180 kN/m²; $\phi = 30^\circ$ and unit weight of backfill is 18 kN/m³. Spacing of Counterforts is 3 m; $\mu = 0.50$. Adopt M20 and Fe 415 steel.

SOLUTION.

Height of the wall above G.L = 5.5 m (H_1)

Active earth pressure Coeff. $K_a = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$.

Max. depth of foundation below G.L = $K_a^2 \cdot \frac{q_0}{\gamma} = \left(\frac{1}{3}\right)^2 \times \frac{180}{18} = 1.111 m$.

Provide the depth as 1.3 m.

Hence, the total height of the wall above the foundation = 5.5+1.3 = 6.8 m (H)

Taking the base width as 0.5H to 0.6H = 3.4 m to 4.08 m

Let the base width be, b = 4.0 m.

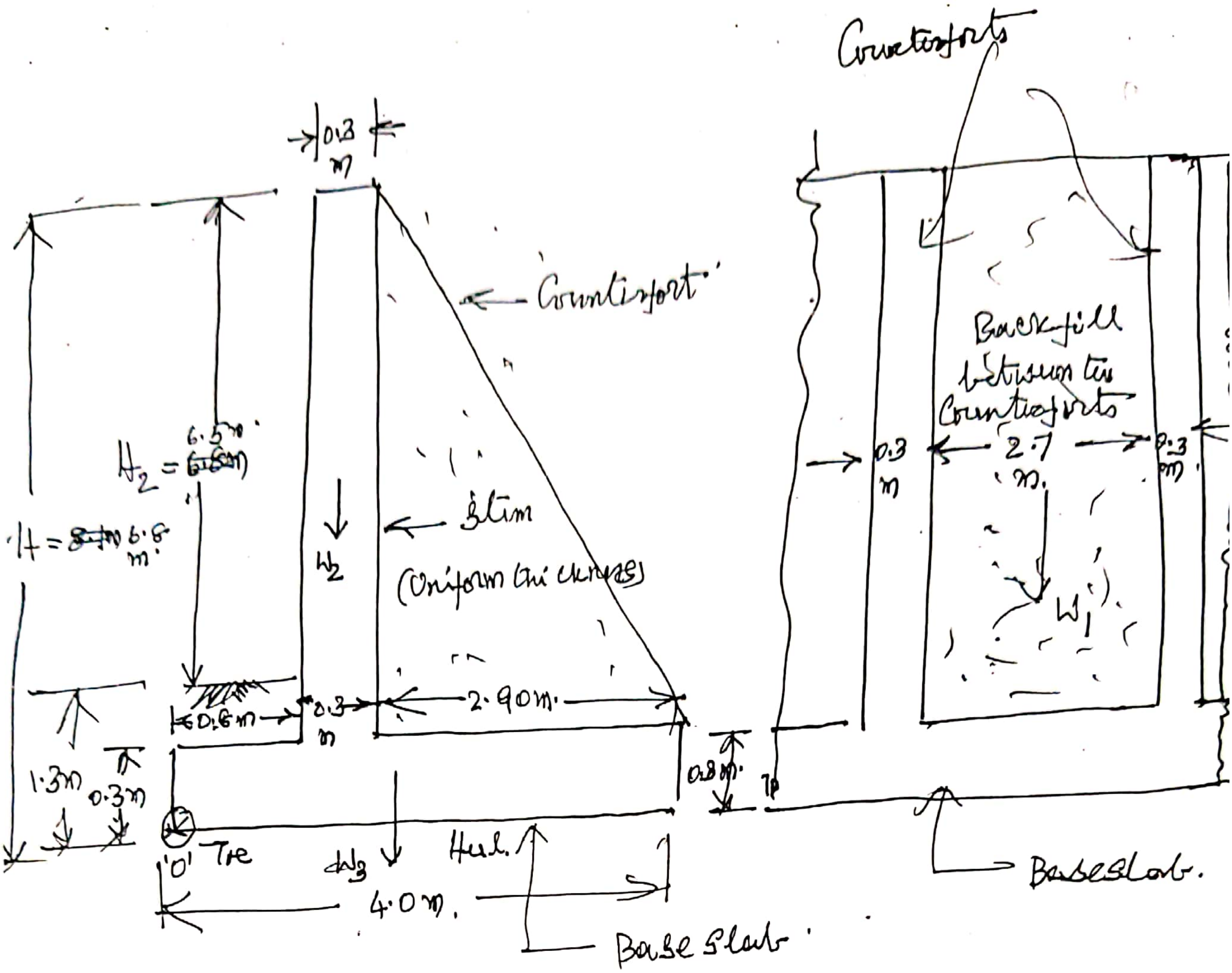
Toe = $\frac{1}{4}$ to $\frac{1}{5}$ of b = $\frac{4.0}{5}$ = 0.8 m.

Width of the Counterforts is generally = 0.03H to 0.06H

= 0.13 to 0.16 m, Provide 0.3 = 300 mm.

Thickness of the stem of the retaining wall = Thickness of the base slab

$= \frac{H}{25} = \frac{6.8}{25}$, Provide D = 300, d = 240 mm



Stability Calculations

Sl. No.	Weight of the Component kN.	$\bar{x}$ 'm'	Moment M_s (kNm)
1.	W_1 = Backfill. $2.7 \times (6.8 - 0.3) \times 18 = 315.9$ kN.	$\left(4 - \frac{2.9}{2}\right) = 2.55$ m	805.5
2.	Weight of Stem = W_2 $= 0.3 \times 6.5 \times 25 = 48.75$	0.95 m	46.30.
3.	Weight of base slab = W_3 $= 0.3 \times 4.0 \times 25 = 30.$	2.0	60.00
	Total Vertical force $= \Sigma W = 394.65$ kN		$\Sigma M_s = 911.80$ kNm.

Due to horizontal earth pressure on the wall, $M_0 = \frac{K_a \gamma h^3}{6}$.

$= \frac{1}{3} \times \frac{1}{6} \times 18 \times 6.8^3 = 314.4 \, kNm$,

Hence, factor of safety against overturning $= \frac{0.9 M_s}{M_0} = \frac{0.9 \times 911.8}{314.40}$

$= > 1.4$, Hence, Safe.

Sliding force $= P_a = K_a \frac{\gamma h^2}{2} = \frac{1}{3} \times \frac{18 \times 6.8^2}{2} = 138.72 \, kN$.

Resisting vertical force $= \mu (0.9 w_1 + w_2 + w_3)$

$= 0.5 \left[0.9 \times 315.9 + 48.75 + 30 \right] = 181.53$

Hence, Factor of Safety against Sliding $= \frac{181.53}{138.72} = 1.3 < 1.4$

Hence, provide a Shear key of 300mm depth below the base slab.

Soil Pressure:

Net moments about the toe $= M_s - M_0 = 911.80 - 314.40$

$= 597.4 \, kNm$.

The distance of the point where the resultant cuts the base from 'O'

$= \frac{\Sigma M}{\Sigma W} = \frac{597.4}{394.15} = 1.514 m \; (\bar{x})$

$e = \frac{b}{2} - \bar{x} = 2.0 - 1.514 = 0.486 m$ (towards left of centre)

$\frac{b}{3} = \frac{2.0}{3} = 0.67 m$; $e < \frac{b}{6}$, Hence, no tension occurs at the base.

Hence, Soil pressures = $\frac{\Sigma W}{b}\left[1 \pm \frac{6e}{b}\right]$

$= \frac{394.65}{4}\left[1 \pm \frac{6 \times 0.486}{4}\right]$

Hence $p_{max} = 170.6\ kN/m^2$ & $p_{min} = 26.7\ kN/m^2$, Hence no tension is occurring.

$p_{max} = 170.6 < 180.0$ (SBC of Soil). Hence Safe.

B) STEM

Stem is designed as a Continuous slab supported over the Counterforts.

Eff. span = 2.7 + 0.3 = 3.0 m.

Horizontal earth pressure at the base = $K_a \gamma H_1$

$= \frac{1}{3} \times 18 \times 6.5 = 39\ kN/m^2$

$M_{max} = \pm \frac{wL^2}{12} = \frac{39 \times 3^2}{12} = 29.25\ kNm.$

M_u (Factored) $= 1.5 \times 29.25 = 43.9\ kNm$

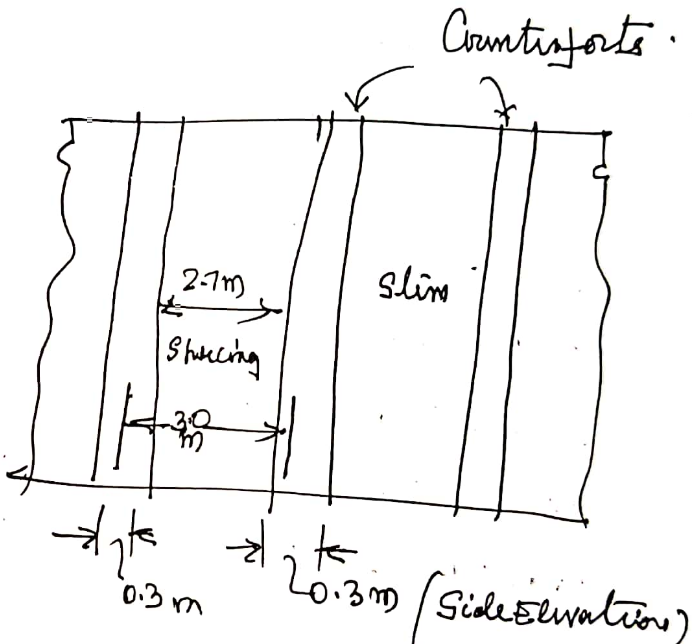
$M_u (Lim) = 0.138 \times 20 \times 1000 \times 260^2$

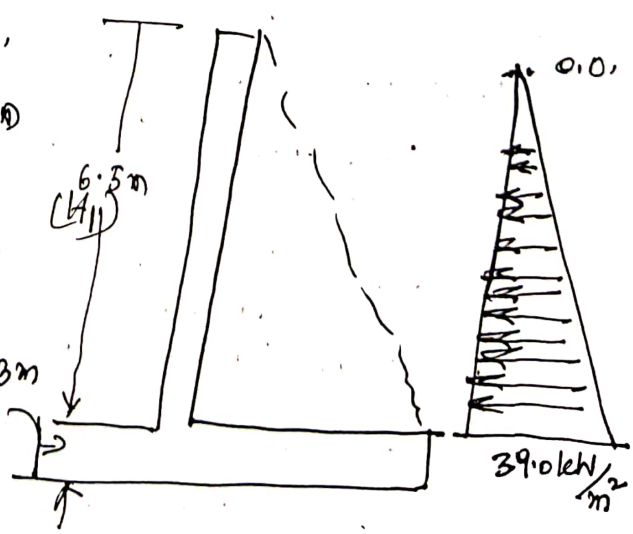
(Taking $d = 300 - 40 = 260$)

$M_u (Lim) = 186.57 \times 10^6\ Nmm$

$> 43.9 \times 10^6$. Hence okay.

(Act.)





The -ve reinforcement is given by

$$M_u \frac{\cancel{}}{} = 0.87 \times 415 \times A_{st} \times 260 \left[1 - \frac{A_{st}}{1000 \times 260} \times \frac{415}{20}\right]$$

It is simplified as $467.65 = A_{st}\left[1 - \frac{A_{st}}{12,530}\right]$

or $A_{st}^2 - 12530\,A_{st} + 467.65 \times 12,530 = 0.$

Solving, $A_{st} = 486\ mm^2.$

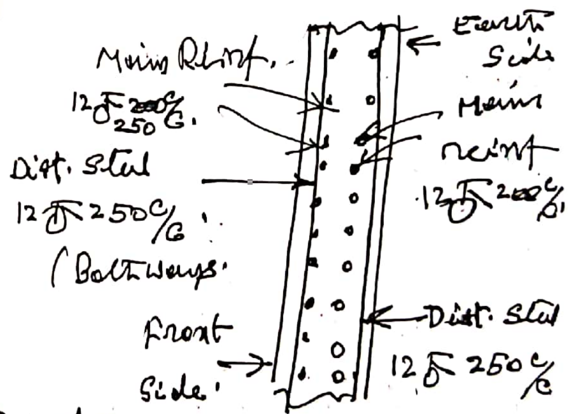
Using 12 ϕ, spacing $= \frac{\frac{\pi}{4} \times 12^2}{486} \times 1000 = 232\ mm.$

Provide 12 ϕ 200 c/c horizontally on the front Side ~~of the~~ and rear Wall to take care of ve and -ve BM?

Dist. steel $= \frac{0.12}{100} \times 1000 \times 260 = 312\ mm^2$, spacing $= 312\ mm^2$

12 ϕ 250 c/c is provided Vertically on the ~~front~~ center face.

Provide 12 ϕ 250 c/c on the front Side of the wall both ways. This will take Care of the positive moment also.



Check for Shear

Max. S.F at the Support at the Counterfort.

$$= \frac{39(3-0.3)}{2} = 52.65\ kN, \quad V_u = 52.65 \times 1.5 = 79\ kN$$

(clear)

$$\tau_v = \frac{79 \times 1000}{260 \times 1000} = 0.303\ N/mm^2$$

Percentage steel provided $= \frac{\text{Area of one rod}}{\text{depth} \times \text{Spacing}} = \frac{\frac{\pi}{4} \times 12^2}{260 \times 200} \times 100 = 0.18\%$

Hence, from tables $\tau_c = < \tau_v$. Hence Safe

Increase the spacing of longitudinal bars to 300 c/c towards top.

TOE SLAB

Cantilever moment about the fixed end

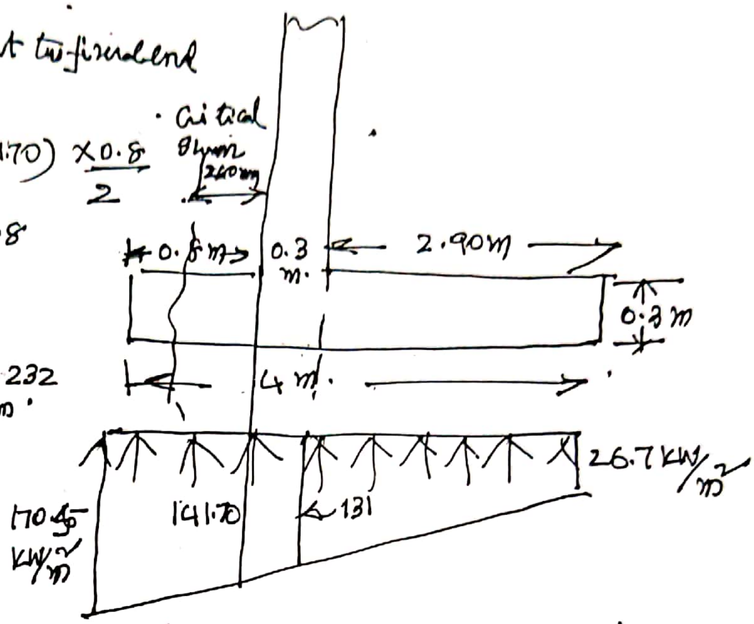
$$= 141.70 \times \frac{0.8^2}{2} + (170.5 - 141.70) \times \frac{0.8}{2} \times \frac{2}{3} \times 0.8$$

$$= 51.488 \text{ kNm}$$

Factored $M_u = 1.5 \times 51.488 = 77.232$ kNm.

Taking $d = 0.26$ m (260 mm)

Area of steel required is



Soil Pressure Distribution.

$$77.232 \times 10^6 = 0.87 \times 415 \times A_{st} \times 260 \left[1 - \frac{A_{st}}{1000 \times 260} \times \frac{415}{20}\right]$$

Hence,

$$822.7 = A_{st}\left[1 - \frac{A_{st}}{12,530}\right], \text{ Solving, } A_{st} = 885 \text{ mm}^2$$

Using 16 ϕ, spacing $= \frac{\pi/4 \times 16^2}{885} \times 1000 = 227$ mm, Provide 16 ϕ 220 c/c at the bottom.

Check for Shear

Critical Section at 'd' from the face of the stem

$$\text{Pressure} = 26.7 + \frac{(141.70 - 26.7)}{4} \times (3.2 + 0.260) = 126.2 \text{ kN/m}^2$$

~~Shear force~~ Hence, total S.F up to the Critical Section

$$= \frac{1}{2}(170.5 + 126.2)(0.80 - 0.26) = 80.10 \text{ kN}.$$

$$V_u = 1.5 \times 80.10 = 120.15 \text{ kN}.$$

$$\tau_v = \frac{120.15 \times 1000}{1000 \times 260} = 0.462\ N/mm^2$$

$$\text{Percentage reinf. } p = \frac{\frac{\pi}{4} \times 16^2}{} = \frac{\text{Area of Steel in 1m}}{100} \times \frac{100}{bd}$$

$$A_{st} \text{ (act.)} = \frac{\pi}{4} \times 16^2 \times \text{No. of rods in 1m.}$$

$$= \frac{\pi}{4} \times 16^2 \times \frac{1000}{220} = \cancel{914}\ 913.45\ mm^2$$

$$\text{Percentage steel} = \frac{913}{260 \times 1000} \times 100 = 0.35\%$$

From table 19 of IS 456-2000, Design Shear Stress τ_c (Allowable) $= 0.41\ N/mm^2$.

Hence,

The $\tau_v > \tau_c$ (Allowable)

Hence increase the thickness to $D = 350$ mm with $d = 300$ mm.

$$\text{Hence } \tau_v = \frac{120.15 \times 1000}{1000 \times 300} = 0.40 < 0.41.$$

Hence okay.

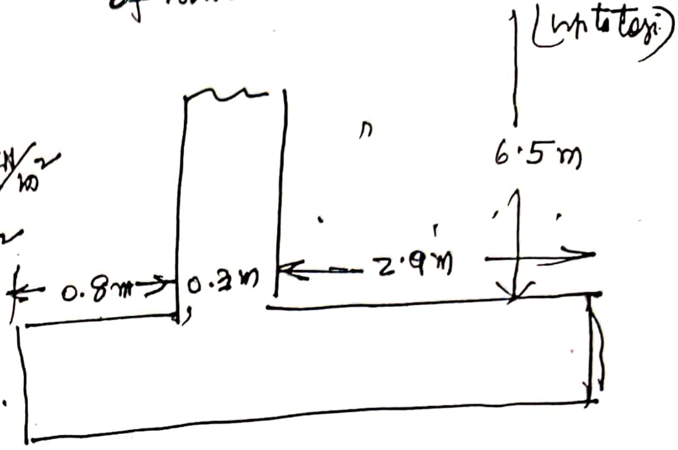
~~HEEL SLAB~~

The reinforcement details are shown in the sketch.

HEEL SLAB.

From the soil pressure diagram, soil pressure at the junction of heel slab with the stem

$= 131\ kN/m^2$



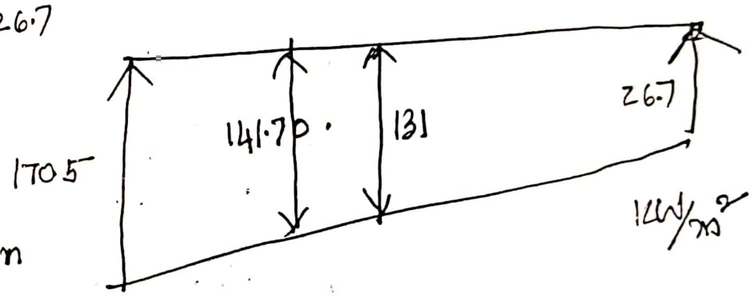
Load from back fill $= 6.5 \times 18 = 117\ kN/m^2$

Self weight heel slab $= 0.3 \times 1 \times 25 = 7.5\ kN/m^2$

Total downward load $= 117 + 7.5 = 124.5\ kN/m^2$

Net downward pressure at the edge of the heel $= 124.5 - 26.7$

$= 97.8\ kN/m^2$



Hence $M_{max} = \pm \dfrac{97.8 \times 3^2}{12} = 73.35\ kNm$

Factored $M_u = 1.5 \times 73.35 = 110.0\ kNm$.

The heel slab is designed as a ~~Cantilever~~ Continuous slab between the counterforts.

Longitudinal steel is

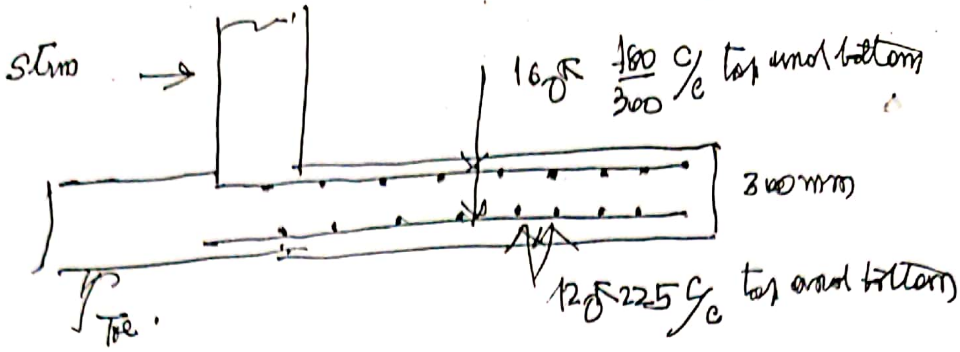
$$110.0 \times 10^6 = 0.87 \times 415 A_{st} \times 300 \left[1 - \frac{A_{st}}{1000 \times 300} \times \frac{415}{20}\right]$$

Simplified as $1015 = A_{st}\left[1 - \dfrac{A_{st}}{14457.8}\right]$, Solving $A_{st} = 1098\ mm^2$

Spacing of $16\phi = \dfrac{\frac{\pi}{4} \times 16^2}{1098} \times 1000 = 181\ mm$, Provide 16ϕ 180 c/c at top and bottom

The spacing may be increased to 300mm towards the junction

Dist. steel of 12ϕ 225 c/c is provided in the other direction.

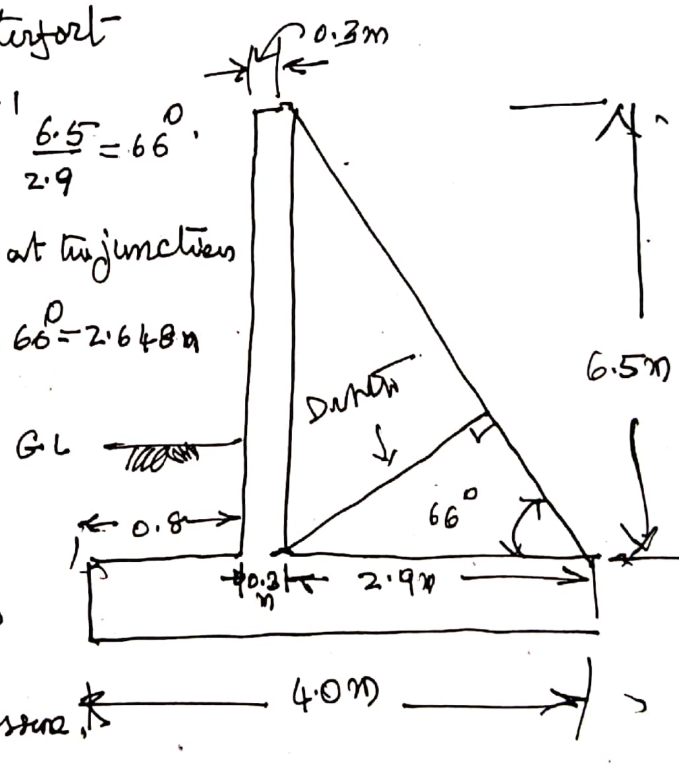


COUNTER FORT. (As a 'T' beam)

The Counterfort is designed as a 'T' beam spanning between the base slab and stim with varying section

Inclination of the Counterfort

with the horizontal = $\tan^{-1}\frac{6.5}{2.9} = 66^0$.



Depth of the Counterfort ~~with~~ at the junction

$= 2.9\sin\theta = 2.9\times\sin 66^0 = 2.648\,m$

If 'h' is the height of the ~~Coun~~ Counterfort, the max. moment at the base $= \frac{\cancel{k_a}\,\gamma h^3}{6}$ is calculated from the earth pressure.

The total earth pressure on the stim $= P = \frac{\gamma h^2}{2}\cdot k_a /m.$

moment about the base $= \frac{\gamma h^3}{6} k_a\cdot$ Tolo

Total moment to be taken by the Counterfort supporting the stim

$= \frac{\gamma h^3}{6}\times L$ kNm (L is the spacing of the Counterforts.)

Hence $M_{max} = \frac{1}{3} \times 18 \times \frac{6.5^3}{6} \times 3 = 823.875$ kNm

M_u (Factored) = $1.5 \times 823.875 = 1235.8$ kNm.

Area of steel required is given by

$$1235.8 \times 10^6 = 0.87 \times 415 \times A_{st} \times 2648 \left[1 - \frac{A_{st}}{300 \times 2648} \times \frac{415}{20}\right].$$

$$1292 = A_{st}\left[1 - \frac{A_{st}}{38284}\right].$$

Solving, $A_{st} = 1339\ mm^2$.

Min. reinforcement $= \frac{0.85bd}{fy}$ (In the case of beams)

$= \frac{0.85 \times 300 \times 2648}{415} = 1627\ mm^2$ is provided.

Providing 4 Nos 25 ϕ, A_{st}(provided) $= 4 \times \frac{\pi}{4} \times 25^2 = 1963\ mm^2$

Horizontal ties

Considering 1 m height, the earth pressure at the bottom = 39 kN/m²

Hence, total earth pressure over 3 m spacing = 39 (3 − 0.3) = 105.3 kN.
(clear)

Factored force = 1.5 × 105.3 = 158 kN

Hence, Area of tensile steel required $= \frac{158 \times 1000}{0.87 \times 415} = 437.5\ mm^2$
(in 1 m height)

Spacing of 10 ϕ $= \frac{\frac{\pi}{4} \times 10^2}{437.5} \times 1000 = 179.5$ mm.

Hence, provide 10 ϕ 170 c/c horizontally in the counterfort etc.

Vertical Reinforcement

2

The Downward force near the end

= Load from back fill + Self weight of base slab

— Soil pressure at the heel end = 117 + 7.5 − 26.7 = 97.8 kN/m^2

Hence, Factored force = 1.5 × 97.8 = 146.7 kN/m^2

Hence, Steel required for direct tension = $\frac{146.7 \times 1000}{0.87 \times 415}$

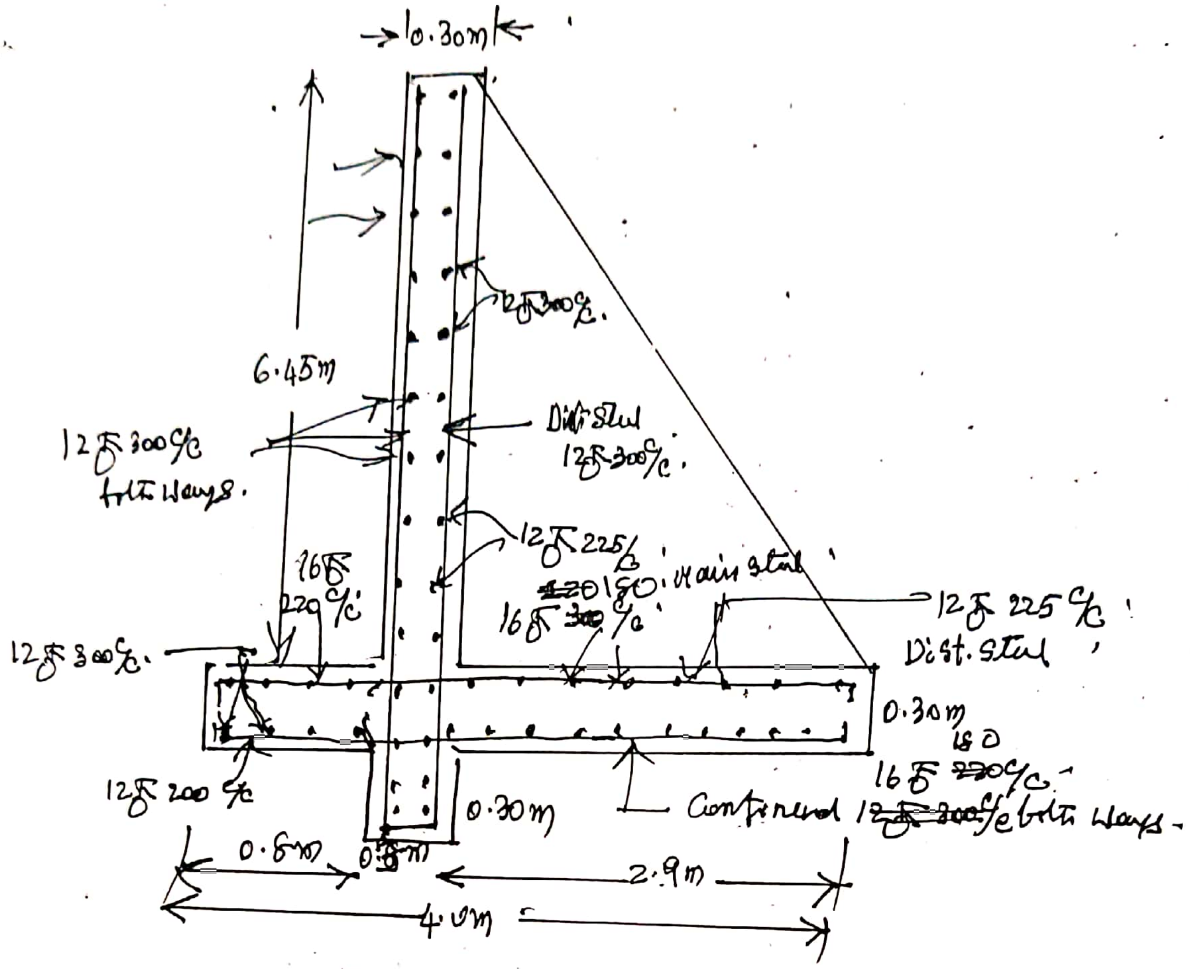
= 406 mm^2

↑ (Partial Safety factor)

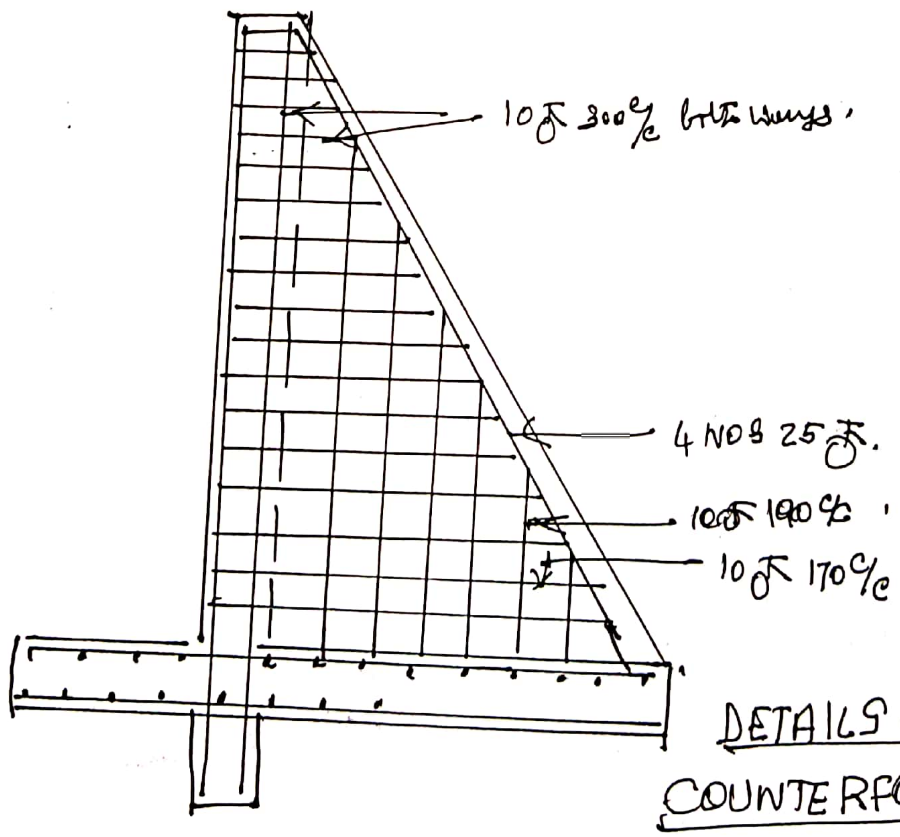
Providing 10 Φ, spacing = $\frac{\pi/4 \times 10^2}{406} \times 1000$ = 193 mm.

Provide 10 Φ 190 c/c.

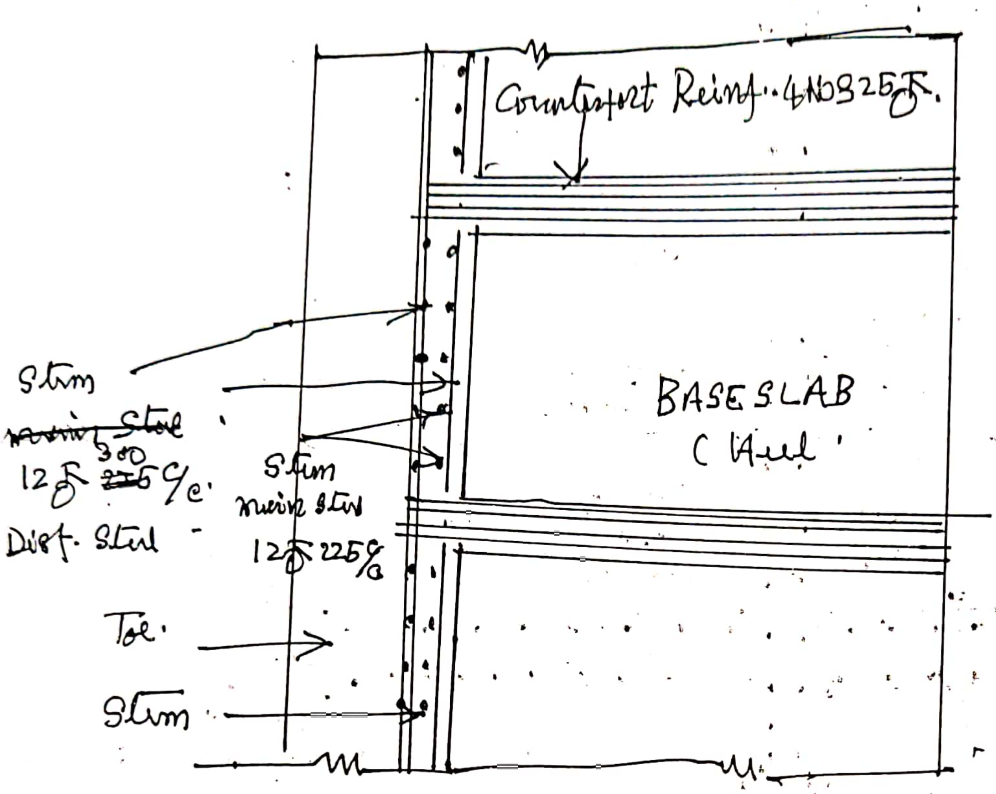
The details of reinforcement are shown by sketches.



CROSS SECTION.



DETAILS OF COUNTERFORT



SECTIONAL PLAN.