

A column may be classified based on the following criteria:

1. Shape of cross section.
2. Slenderness ratio.
3. Type of loading.
4. Pattern of lateral reinforcement.

(S.P.E-1F-389)

Column may be classified based on the types of loading:

1. Axially loaded column.
2. A column subjected to axial loading & uniaxial bending.
3. A column subjected to axial loading & biaxial bending.

The reinforced concrete column can also be classified according to the manner in which the longitudinal bars are laterally supported that is:

1. Tied column.
2. Spiral column.

Effective height of the column:-

(Page-94-Table-28)

1508/20/08 - etc

Minimum eccentricity:

Isolated beam (Page-42 - 55.4)

$$e \geq \frac{l}{500} + \frac{10}{300}$$

> 20

Short column under axial compression:

(Page-71-39.3)

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_s$$

Requirements for reinforcement:

There are two kind of reinforcement in a column:

1. Longitudinal reinforcement.

2. Transverse reinforcement.

The purpose of transverse reinforcement is to hold the vertical bars in position providing lateral support, so that the individual bars cannot buckle outward & split the concrete.

The transverse reinforcement does not contribute to the strength of the column directly.

Longitudinal Reinforcement :-

(Page-48-26-5-3)

- Minimum percentage of steel is 0.8%.
- Maximum percentage of steel is 4%, if bars are lapped & 6%, if the bars are not lapped.
- Minimum no. of bars for a rectangular section is 4 & for circular section it is 6.
- Minimum diameter of bar is 12mm.
- Maximum spacing between longitudinal bars is 300mm.
- Minimum percentage of steel for pedestal is 0.15%.
- Minimum nominal cover is 50mm.

Transverse Reinforcement :-

that
Dia of tie or ring / spiral (Should not be less than) ~~less than~~ ~~if~~ ~~for~~

$\frac{\phi_{\text{main}}}{4}$ which even is more
 \downarrow 6 mm

Page-49

$$\begin{aligned} & \rightarrow 800.0 - 10 = 790 \\ & (800.0 - 1) \times 10 = 790 \end{aligned}$$

Date - 02/04/2019

Q1

Design a short column square section to carry an axial load 2000 kN using
(i) mild steel (ii). HYSIO bar of Fe45-5 & the grade of concrete is M20.

Sol)

Axial factored load,

$$P_u = 2000 \text{ kN} \times 1.5 = 3000 \text{ kN} = 3000 \times 10^3 \text{ N.}$$

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

$$A_{sc} = 0.8\%$$

For economy quantity of steel should be adopted, $A_{sc} = 0.8\% = 0.008$

Let us consider the square column has side a .

$$\text{The area of square column} = a^2$$

$$3000 = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

$$A_{sc} = 0.008 a^2$$

$$A_c = a^2 - 0.008 a^2 \\ = a^2 (1 - 0.008)$$

$$P_u = 0.4 \times f_{ck} \times A_c + 0.67 f_y A_s$$

$$\Rightarrow 3000 \times 10^3 = 0.4 \times 20 \times 0.8 (1 - 0.008) + 0.67$$

$$\times 250 \times 0.008 a^2$$

$$\Rightarrow 3000 \times 10^3 = 0.620 \times 7.936 a^3 + 1.34 a^2$$

$$\Rightarrow 9.276 a^2 = 3000 \times 10^3$$

$$\Rightarrow a = \sqrt{\frac{3000 \times 10^3}{9.276}}$$

$$\Rightarrow a = 568.69 \text{ mm}$$

$$= 56.86 \text{ cm} \approx 60 \text{ cm}$$

$$(a^2)_{\text{provided}} = 60 \times 60 = 3600 \text{ cm}^2$$

$$(a^2)_{\text{required}} = 56.86 \times 56.86$$

$$= 3233.05 \text{ cm}^2$$

$$\Rightarrow 3234 \text{ cm}^2$$

The longitudinal reinforcement area required.

$$A_s = 0.008 \times 3234$$

$$= 25.87 \text{ cm}^2$$

Let us provide 20 mm bar

$$n \times \frac{\pi}{4} \times 20^2 = 25.87$$

$$\Rightarrow n = 25.87 \times \frac{4}{\pi} \times \frac{1}{20}$$

$$\Rightarrow n = 8.23$$

Let us provide 8 nos. of 20 mm bars.

$$\text{So, Area of Steel, } A_{sc} = 8 \times \frac{\pi}{4} \times 20^2 \\ = 2513 \text{ cm}^2$$

Diameter & pitch of lateral ties

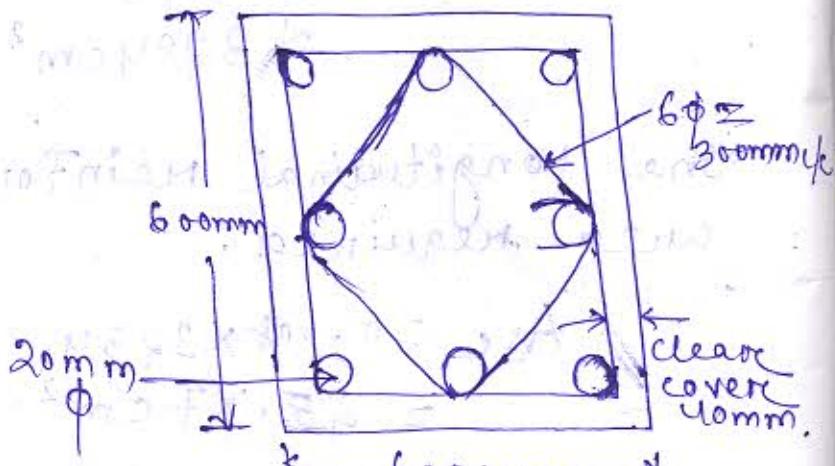
$$\phi = \frac{\phi_L}{4} = \frac{20}{4} = 5 \text{ mm,} \\ 6 \text{ mm}$$

Let us provide 6mm diameter for lateral ties.

$$\text{Pitch} = 600 \text{ mm.}$$

$$\text{less} \left\{ \begin{array}{l} 16\phi_L = 16 \times 20 = 320 \text{ mm} \\ 300 \text{ mm} \end{array} \right.$$

Let us provide 300 mm pitch c/c.



$$f_s = 28 = 30.63 \times \frac{\pi}{4} \times 6^2$$

$$30.63 \times 36 \times 36 \times 3.14 \times 6^2$$

$$30.63 \times 36 \times 3.14 \times 6^2$$

$$P_u = 0.4 \times f_{ck} \times A_c + 0.67 f_y A_{sc}$$

$$\Rightarrow 3000 \times 10^3 = 0.4 \times 20 \times a^2 (1 - 0.008) + 0.67 \times 415 \times 0.008 a^2$$

$$\Rightarrow 3000 \times 10^3 = 7.936 a^2 + 2.2244 a^2$$

$$\Rightarrow 10.16 a^2 = 3000 \times 10^3$$

$$\Rightarrow a = \sqrt{\frac{3000 \times 10^3}{10.16}}$$

$$\Rightarrow a = 543.39 \text{ mm}$$

$$\approx 543.33 \text{ cm}$$

$$\approx 55 \text{ cm}$$

$$(a^2)_{\text{provided}} = 55 \times 55 = 3025 \text{ cm}^2$$

$$(a^2)_{\text{required}} = 54.33 \times 54.33$$

$$\approx 2951.75 \text{ cm}^2$$

The longitudinal reinforcement only,
of area required.

$$A_{sc} = 0.008 \times 2951.75$$

$$\approx 23.61 \text{ cm}^2$$

Let us provide 20 mm dia bar,

$$\pi \times \frac{\pi}{4} \times 2^2 = 23.61$$

$$\Rightarrow n = 23.61 \times \frac{4}{\pi} \times \frac{1}{2}$$

$$= 7.5 \\ \geq 8 \text{ nos.}$$

Let us provide 8 nos. of 20 mm bar.

So, Area of Steel, $A_{sc} = 8 \times \frac{\pi}{4} \times 2^2$

$$= 25.13 \text{ cm}^2$$

diameter & pitch of lateral ties

$$\phi = \frac{\phi_L}{4} = \frac{20}{4} = 5 \text{ mm}$$

6 mm

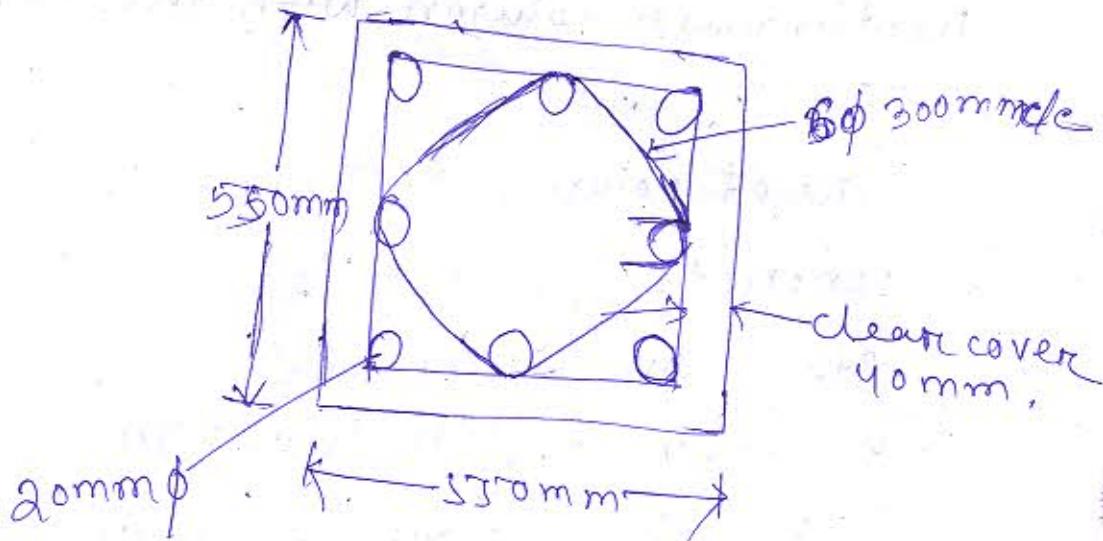
Let us provide 6 mm diameter for lateral ties.

$$\text{Pitch} = 550 \text{ mm}$$

$$\text{6 } \phi_L = 6 \times 20 = 120 \text{ mm}$$

$$300 \text{ mm}$$

Let us provide 300 mm pitch c/c.



Date - 03/04/2019

Design of long column (Slender column)

$\frac{L}{d} < 12$ (short column) (Page - 71, 39, 70)

$\frac{L}{d} \geq 12$ (long column)

$$P_b = (q_1 + \frac{q_2 P}{\sigma_{ck}}) \delta_{ck} b d$$

Rectangular section

$$P_b = (q_1 + \frac{q_2 P_{min}}{\sigma_{ck}}) \delta_{ck} \sigma^2$$

Circular section

q_1 & q_2 are the co-efficients.

- $m m o o o e = m 8 = n e l$
- $m m o o o n = m N = p e l$
- $40 \times 0.25 = 10 \times 0.25 = 1.25$

Q1 Design a ~~slender~~ slender unbraced rectangular column with the following data.

Size of column = 25 cm x 30 cm

Concrete grade = M₂₅

Steel grade = Fe 500 D

Effective length, L_{eff} = 3 m

Effective length, L_{eff} = 4 m

Factored load, P_u = 750 kN

Factored moment in the direction of larger dimension M_u = 25 kNm

Factored moment in the direction of shorter dimension M_u = 15 kNm

The reinforcement is distributed equally to the all four sides.

$$\alpha_1 = 0.2, \alpha_2 = 0.34$$

Axial load corresponding to the maximum compressive stress is 420 kN.

Given data,

$$(m) b = 25 \text{ cm} = 250 \text{ mm}$$

$$D = 30 \text{ cm} = 300 \text{ mm}$$

$$f_{ck} = 25 \text{ MPa}$$

$$f_y = 500 \text{ MPa}$$

$$L_{eff} = 3 \text{ m} = 3000 \text{ mm}$$

$$L_{eff} = 4 \text{ m} = 4000 \text{ mm}$$

$$P_u = 750 \text{ kN} = 750 \times 10^3 \text{ N}$$

$$M_{u1} = 15 \text{ kNm} = 15 \times 10^6 \text{ Nmm}$$

$$\alpha_1 = 0.2$$

$$\alpha_2 = 0.34$$

$$\frac{L_{ey}}{R} = \frac{4000}{300} = 13.33 > 12 \text{ (long column)}$$

The column is slender about the major axis in the direction of larger dimension only.

$$M_{ay} = P_u \times e_y$$

Additional moment $M_{ay} = P_u \times e_y$.

$$\frac{25 \times 10^6}{0.85 - 0.017} \Rightarrow 25 \times 10^6 = 750 \times 10^3 \times e_y$$

$$\Rightarrow e_y = \frac{25 \times 10^6}{750 \times 10^3}$$

$$e_y = 33.33 \text{ mm}$$

Let us assume percentage of steel

is 2%.

$$\text{Net area} = 250 \times 300$$

$$= 75000 \text{ mm}^2$$

$$A_s c = 75000 \times 0.02$$

$$= 1500 \text{ mm}^2$$

$$A_c = 75000 - 1500$$

$$= 73500 \text{ mm}^2$$

$$P_u g = 0.45 f_{ck} * A_c + 0.75 f_y * A_s c$$

$$= 0.45 \times 25 \times 73500 + 0.75 \times 500 \times 1500$$

$$= 1389.375 \text{ kN} \approx 1400 \text{ kN}$$

$$P_b = 420 \text{ kN} = 420 \times 10^3 \text{ N}$$

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} = \frac{1400 - 750}{1400 - 420} \\ = 0.66$$

The reduction moment = $k \times M_{ay}$

$$= 0.66 \times 25 \\ = 16.5 \text{ kNm}$$

Eccentricity,

$$e \leq \frac{L}{500} + \frac{\alpha}{30} = \frac{4000}{500} + \frac{300}{30} \\ = 18 \text{ mm}$$

Let us provide 20mm eccentricity.

So, eccentricity as per IS code which is 20mm is less than the eccentricity provided in the column section.

So, the design is safe.

$$M_u = M_{pl} = P_u \times e = 750 \times 10^3 \times 20 \\ = 15 \text{ kNm}$$

M_u & M_{pl} provided is equal to & more than 15 kNm, so it is safe.

Q. Design a slender ~~bent~~ braced circular column with the following column.

Size of column = 40cm.

Concrete grade = M20.

Steel grade = Fe415.

Effective length = 6m.

Unsupported length = 7m.

Factored load, $P_u = 1200 \text{ kN}$.

Factored moment, $M_u = 75 \text{ kNm}$ at top
& 50 kNm at bottom.

The column is bent in single curvature.

Given data, E = 22 GPa

$$D = 40 \text{ cm} = 400 \text{ mm}$$

~~$f_{ck} = 20 \text{ MPa}$~~

~~$f_y = 415 \text{ MPa}$~~

~~$E = 200 \text{ GPa}$~~

~~$\sigma_{max} = 200 \cdot 0.21 \cdot 0.81 =$~~

Unsupported length $l = 7000 \text{ mm}$,

$$P_u = 1200 \text{ kN}$$

$$P_b = 600 \text{ kN}$$

$$\frac{L}{R} = \frac{6000}{400} = 15.712 \text{ (long column)}$$

The column is slender about the major axis in the direction of larger dimension only.

Additional moment, $M_{\text{ay}} = P_u \times e_y$

$$\Rightarrow 75 \times 10^6 = 1200 \times e_y$$

$$\Rightarrow e_y = \frac{75 \times 10^6}{1200 \times 10^3}$$

$$= 62.5 \text{ mm}$$

Let us assume percentage of steel is 2%.

$$\text{Net area} = \frac{\pi}{4} \times R^2$$

$$= \frac{\pi}{4} \times 400^2$$

$$= 125663.706 \text{ mm}^2$$

$$A_{sc} = 125663.706 \times 0.02$$

$$= 2513.27 \text{ mm}^2$$

$$A_c = 125663.706 - 2513.27$$

$$= 123150.436 \text{ mm}^2$$

$$P_{uz} = 0.45 f_{ck} \times A_c + 0.75 f_y \times A_s$$

$$= 0.45 \times 20 \times 123150 \times 436 + 0.75 \times 415 - \times 2573.27$$

$$= 1890609.21 \text{ N}$$

$$K = \frac{P_{uz} - P_u}{P_{uz} - P_b}$$

$$= \frac{1900 - 1200}{1900 - 600}$$

$$= 0.54$$

The reduction moment = $K \times M_{ay}$

$$= 0.54 \times 75$$

$$= 40.5 \text{ kNm}$$

Eccentricity, (say e_2)

$$e \leq \frac{L}{500} + \frac{\alpha}{300} = \frac{7000}{500} + \frac{400}{300}$$

$$\approx 27.33 \text{ mm} \quad 27.33 \approx 30 \text{ mm}$$

Let us provide $\frac{27.33}{20} \approx 30$ mm eccentricity.

So, eccentricity as per IS code which is 30mm is less than the eccentricity provided in the column section.

So, the design is safe.

$$M = P \times e$$

$$\rightarrow 21 \times 1200 \times 10^3 \times 30 = 36 \text{ kNm}$$

Moment at top & moment at bottom
is more than the ~~max~~ 36 kNm.

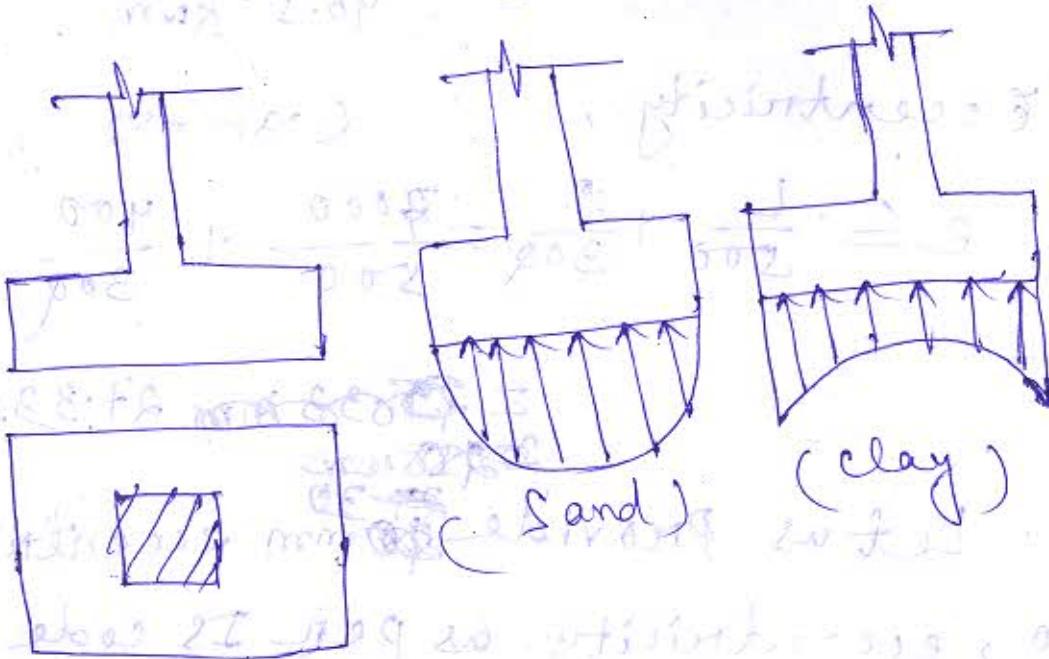
So, ~~the~~ it is safe.

Date - 04/04/2019

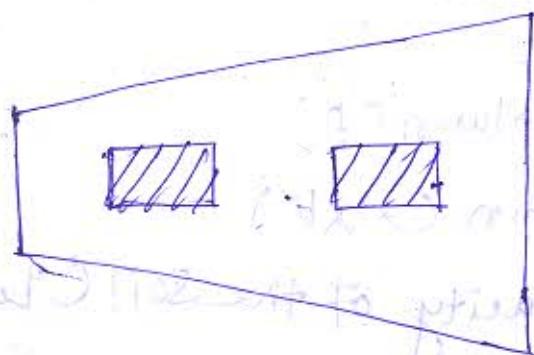
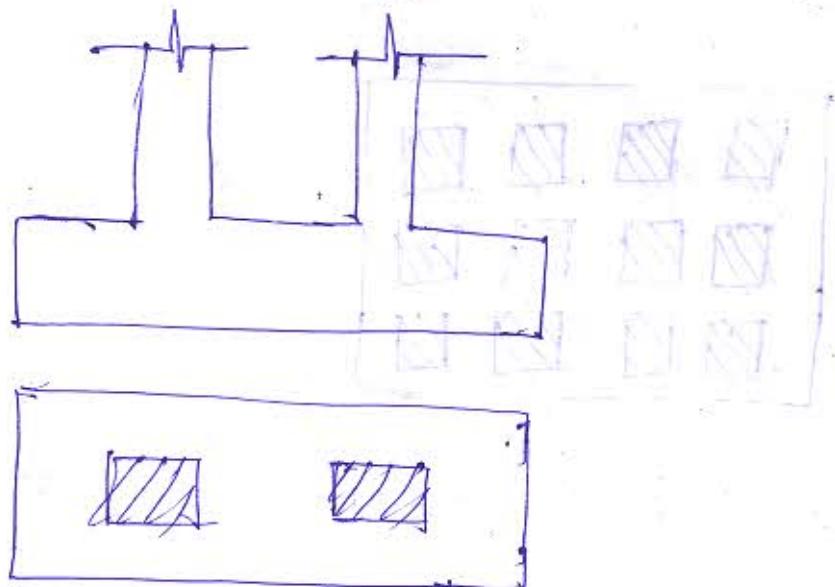
Design of footing:

Types of footing:

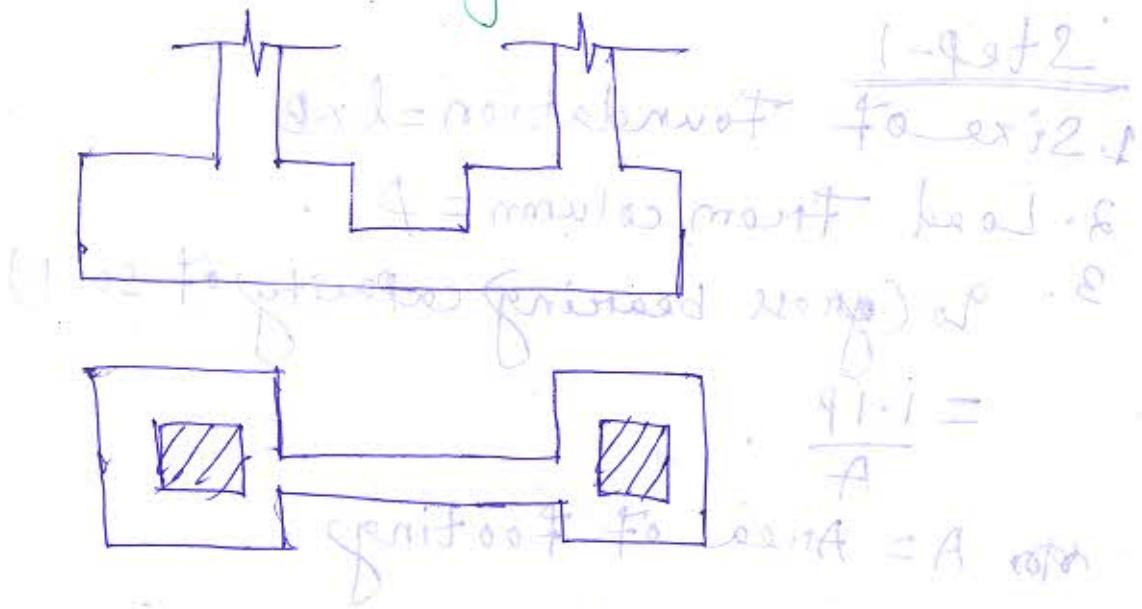
1. Isolated footing:



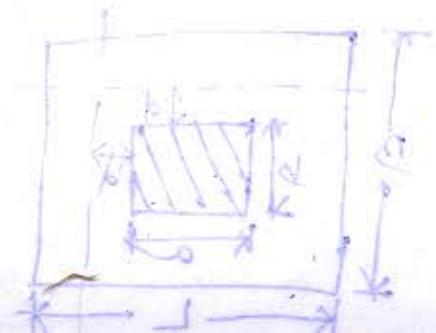
3. combine footing:



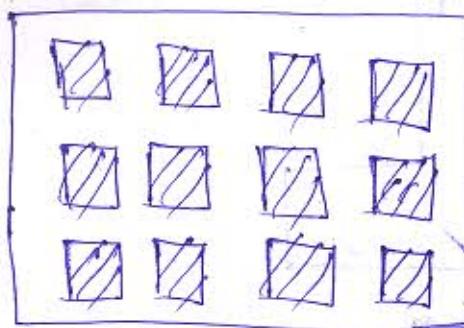
strap footing:



$$\frac{e}{d} = \frac{q}{c}$$



Mat Footing:-



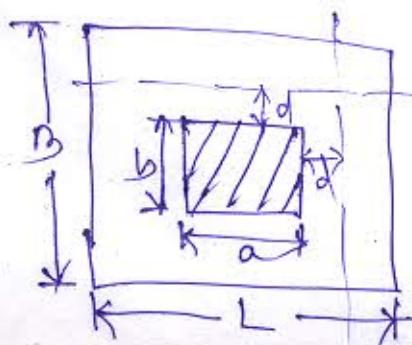
Design of Rectangular isolated Footing:-

Given data

1. Load from column (P)
2. Size of column ($a \times b$)
3. Bearing capacity of the soil (q_s)
4. Grade of concrete & grade of steel.

Step-1

1. Size of Foundation = $L \times B$
 2. Load from column = P .
 3. q_s (gross bearing capacity of soil)
- $= \frac{1.1 P}{A}$
- Where A = Area of Footing.



$$\frac{L}{B} = \frac{a}{b}$$

Net Soil Pressure $w_0 = \frac{1.5P}{A}$

Step-2

calculation of Bending moment :-

$$M = \frac{w_0 l^3}{8}$$

$$M_{max} = \frac{w_0 (B-b)^2}{8}$$

$$M_{yy} = \frac{w_0 (L-a)^2}{8}$$

Step-3

check for shear stress

$$V_{min} = w_0 \left[\frac{(B-b)}{2} - d \right]$$

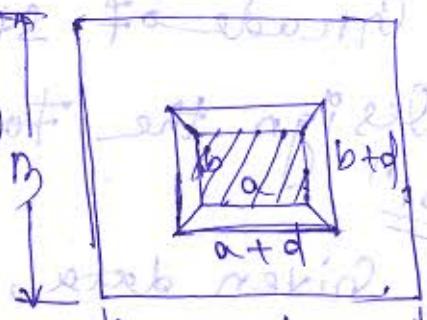
$$V_{yy} = w_0 \left[\left(\frac{L-a}{2} \right) - d \right]$$

Step-4

check for punching shear

Net Punching Force

$$F_p = 1.5P + w_0 (a+d)(b+d)$$



Punching Shear.

= Net Punching Force

* Resisting area

$$\tau_{vp} = \frac{1.5P + w_0 (a+d)(b+d)}{(a+d)(b+d)}$$

As per IS code,

$$T_{np} = \frac{1.5P - 860(a+d)(b+d)}{2[(a+d)+(b+d)]}$$

$$T_{np} \neq k T_c$$

$$k = 0.5 + \frac{b}{a} \geq 1$$

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

Step-5

Calculation of Area of Steel (A_{st}) :-

$$\frac{0.5 f_{ck}}{f_y} (1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b_0 l}}) b_0 l$$

Date - 05/04/2019

Q1 Design a rectangular isolated ~~beam~~ ^{footing} of size 300×500 mm subjected to a load of 1200 kN & 100 kN/m .

q_o = safe bearing capacity
Grade of concrete is M_{25} .
Grade of steel is Fe440.

Design the footing as per limit state method.



Given data,

Size of the column = 300×500

$$P = 1200 \text{ kN}$$

$$q_o = 100 \text{ kN/m}$$

$$f_{uk} = 25 \text{ N/mm}^2$$

$$f_y = 465 \text{ N/mm}^2$$

$$q_0 = \frac{1.1 P}{A}$$

$$\Rightarrow A = \frac{1.1 P}{q_0}$$

$$= \frac{1.1 \times 1200}{100}$$

$$= 13.2 \text{ mm}^2$$

$$A = L \times B$$

Let us consider, $B = 3 \text{ m} \cdot 2.81$

$$13.2 = L \times 3$$

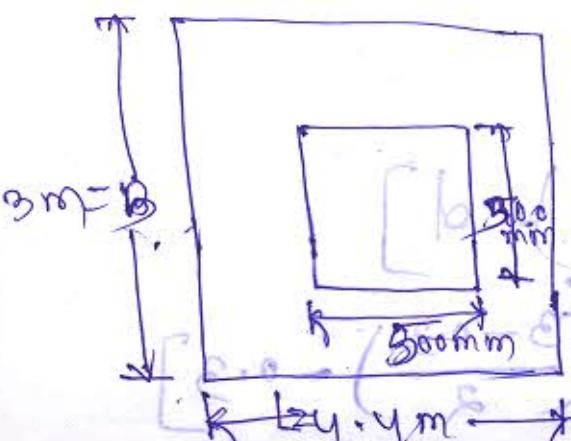
$$\Rightarrow L = \frac{13.2}{3}$$

$$= 4.4 \text{ m}$$

$$w_0 = \frac{1.5 P}{A}$$

$$= \frac{1.5 \times 1200}{3 \times 4.4}$$

$$= 136.36 \text{ KN/m}^2$$



$$M_{max} = \frac{w_0 (B - b)}{8} \times \frac{(3b + 3b)(3 - 0.5)^2}{2}$$

$$= \frac{136 \cdot 36 \times (3 - 0.5)^2}{8}$$

$$= 1261.875 \text{ kNm}$$

$$M_{yy} = \frac{w_0 (L - a)}{8} \times \frac{(4.4 + 0.5)^2}{8}$$

$$= 259.25 \text{ KNm}$$

$$Mu = 0.138 f_{ck} bd^2$$

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

$$= \sqrt{\frac{259.25 \times 10^6}{0.138 \times 25 \times 1000}} = 274.12 \text{ mm}$$

$$\geq 300 \text{ mm}$$

$$V_{nm} = w_0 \left[\left(\frac{B - b}{2} \right) - d \right]$$

$$= 136 \cdot 36 \times \left[\left(\frac{3 - 0.5}{2} \right) - 0.3 \right]$$

$$= 143.17 \text{ kN}$$

$$V_{yy} = W_0 \left[C \frac{(L-a)}{2} - d \right]$$

$$= 136.36 \times \left[C \frac{(4.4-0.3)}{2} - 0.3 \right]$$

$$\approx 224.99 \text{ kN}$$

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

$$= \frac{224.99 \times 10^3}{1000 \times 300}$$

$$= 0.74 \text{ N/mm}^2$$

$$(C_c)_{max} = 3.1 \text{ N/mm}^2$$

$$C_v < (C_c)_{max} \text{ (Safe)}$$

Check for Punching Shear

$$C_{vp} = \frac{1.5P - W_0(a+d)(b+d)}{2[(a+d)+(b+d)]d}$$

$$= \frac{1.5 \times 1200 - 136.36(0.500 + 0.3)(0.3 + 0.3)}{2[(0.5 + 0.3) + (0.3 + 0.3)] \times 0.3}$$

$$= 2064.93 \text{ kN/m}^2$$

$$\therefore \text{unsafe} \quad \frac{2064.93 \text{ N/mm}^2}{1.08} =$$

$$\begin{aligned} \tau_c &= 0.25 \sqrt{f_{ck}} \\ &= 0.25 \times 5 \\ &= 1.25 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} k &= 0.5 + \frac{b}{a} \\ &= 0.5 + \frac{300}{500} \\ &= 1.1 \end{aligned}$$

$$\begin{aligned} k\tau_c &= 1.25 \times 1.1 \\ &= 1.375 \text{ N/mm}^2 \end{aligned}$$

~~Ans~~ ~~k~~ value is more than 1
 So, it is not safe & $\tau_{sp} > k\tau_c$
 So, the design is not safe in punching shear.

$$\begin{aligned} A_{st} &= \frac{0.5 f_{ck}}{f_y} \left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}} \right) b d \\ &= \frac{0.5 \times 25}{415} \left(1 - \sqrt{\frac{4.6 \times 259.25 \times 10^6}{25 \times 1000 \times 300^2}} \right) \\ &= 2841.42 \text{ mm}^2 / \text{m length} \end{aligned}$$

Let us provide 12 mm base

$$n \times \frac{\pi}{4} \times 12^2 = 2841.42$$

$$\Rightarrow n = 2841.42 \times \frac{4}{\pi} \times \frac{1}{12^2}$$

$$\Rightarrow n = 25.12$$

26 no.

Date - 06/04/2019

combined Footing:-

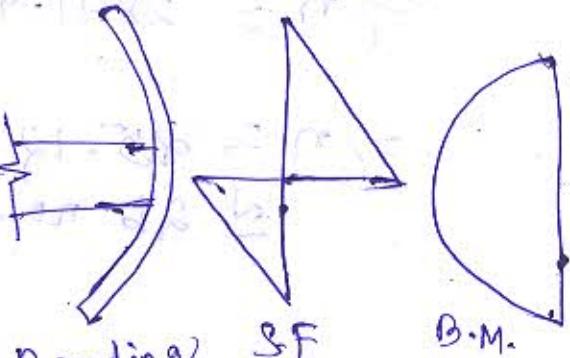
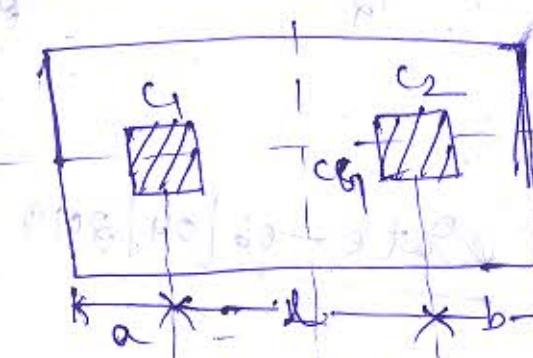
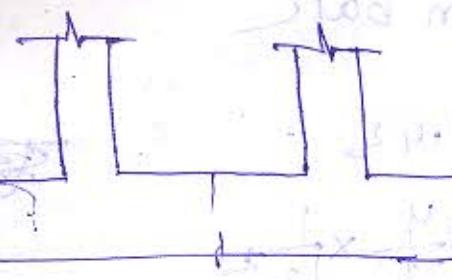
→ combined footing includes two or more columns in a single raft.

→ combined footing is necessary for the following regions.

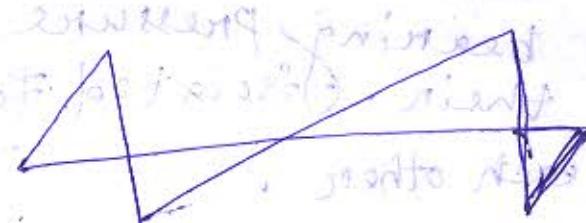
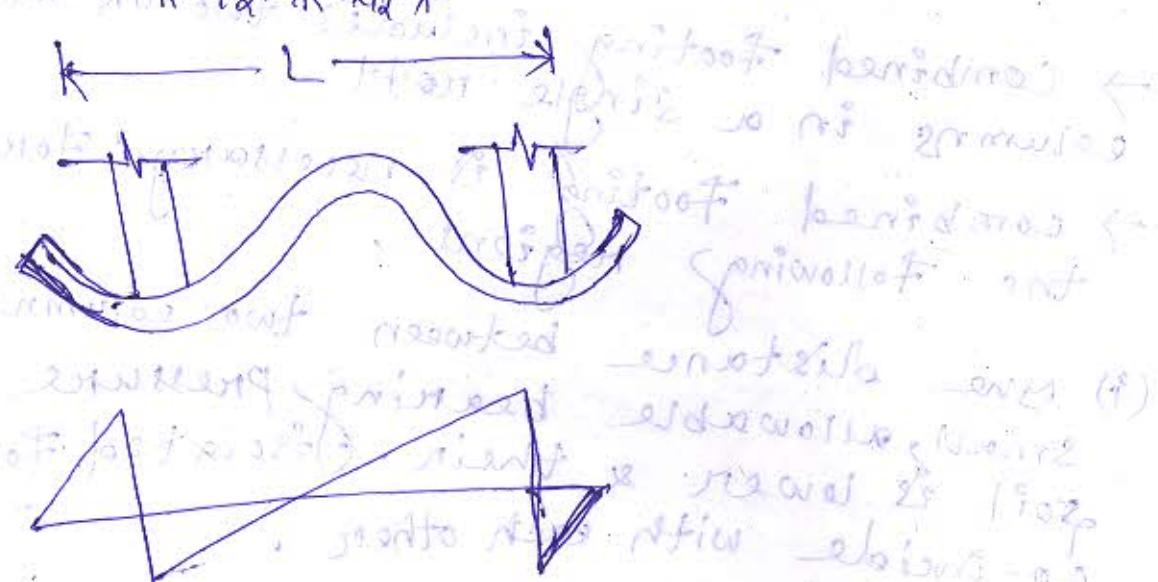
(i) The distance between two columns is small, allowable bearing pressure on soil is lower & their isolated footings coincide with each other.

(ii) When a column is placed at the property line.

(iii) One of the dimension of the footing is restricted to some lower value so that the footings of the columns coincide with each other.



Bending



Optimum eccentricity to develop 25 mmiles a mobil (III)

Optimum eccentricity to mobilize it to 25 mmiles (III)

Optimum eccentricity to mobilize it to 25 mmiles (III)

Ques. Determine the plan & dimension of a combined footing for two axially loaded column with the following data:

- width is not restricted.
- width is restricted to 2.3 m.

Column

C₁

interior

C₂

interior

Type

400x400mm

400x400mm

Size

1000kN

1000kN

P

3 m c/c from C₁ to C₂

spacing

180 kN/m² at 1.6m depth.

ABP

Solⁿ

Let us consider self weight of Footing

be 15% of axial load.

Net pressure on soil = $2 \times 1.15 \times 1000$
 $= 2300 \text{ kN}$.

$$q_0 = \frac{P}{A}$$

$$\Rightarrow A = \frac{P}{q_0}$$

$$= \frac{2300}{150}$$

$$= 15.33 \text{ m}^2$$

$$P = 1000 \text{ kN}$$

$$P_f = 1000 \times 15\% = 1000 \times 0.15$$

$$P + P_f = 1000 + 1000 \times 0.15 = 1000(1+0.15) = 1150 \text{ kN}$$

(a) when width is not restricted.

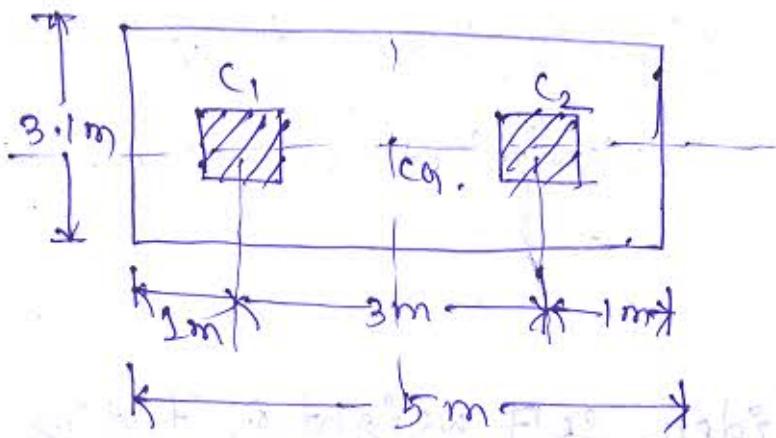
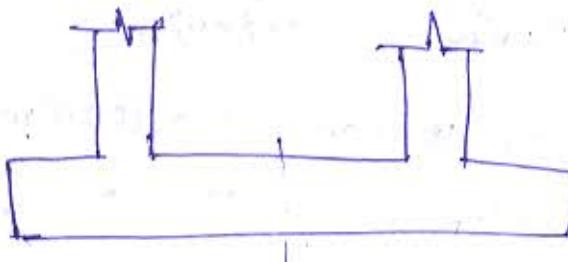
Let us consider 1m projection of both the columns length wise.

$$L = 3 + 1 + 1 = 5 \text{ m}$$

$$L \times B = 15.33 \text{ m}^2$$

$$\Rightarrow 5 \times B = 15.33 \text{ m}^2$$

$$\Rightarrow B = \frac{15.33}{5} = 3.1 \text{ m}$$



(b) When width is restricted to 2.3m.

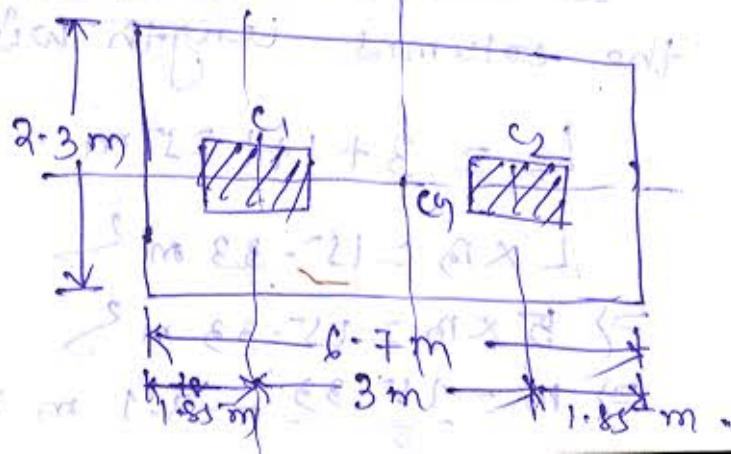
$$B_r = 2.3 \text{ m}$$

$$L \times B_r = 15.33 \text{ m}^2$$

$$\Rightarrow L = \frac{15.33}{2.3} = 6.7 \text{ m}$$

$$a+b+l = L \Rightarrow a+b = \frac{l}{2} - l = 6.7 - 3 = 3.7 \text{ m}$$

$$\frac{L-3}{2} = \frac{6.7-3}{2} = 1.85 \text{ m}$$



Design of Retaining wall

Retaining walls are the structures used to retain earth or other loose material not be able to stand vertically by itself.

Type of retaining wall:-

1. Gravity wall
- 2. counterfort wall
- 3. cantilever wall
4. buttress wall
5. Bridge Abutment
6. Box culvert

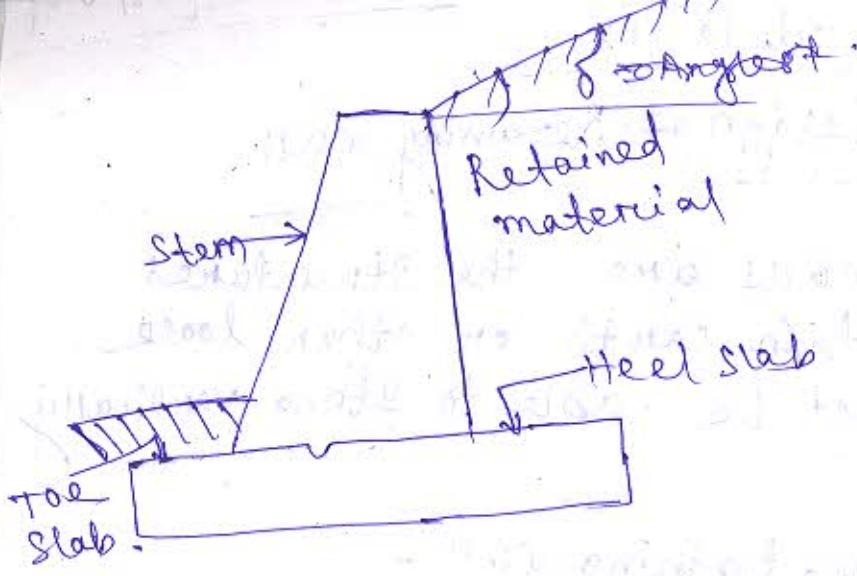
Cantilever retaining wall:-

This is the most commonly used retaining wall. It consists of three components.

1. Vertical walls
2. Heel slab

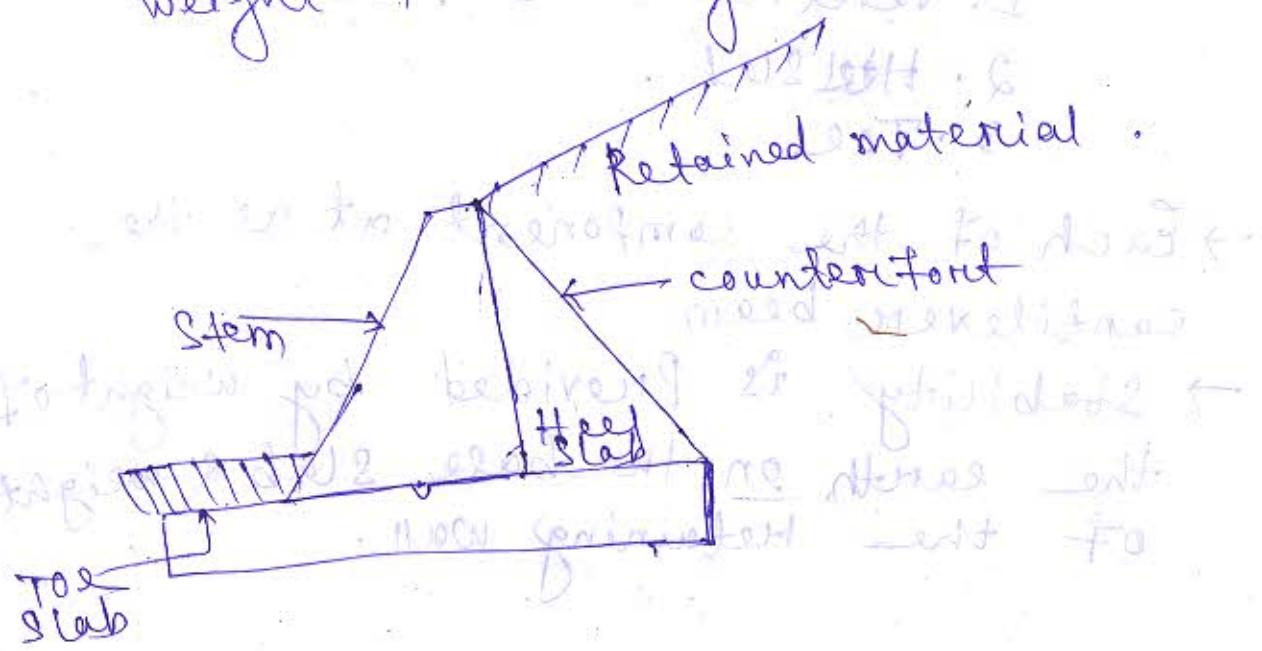
→ Each of the component act as the cantilever beam

→ Stability is provided by weight of the earth on the base slab & weight of the retaining wall.



Counterfort Retaining Wall:

- In counterfort retaining wall the vertical slab & horizontal slab, that is hill & toe are tied together by a counterfort.
- Counterforces are transverse walls spaced at certain interval & act as tension ties to support the vertical wall.
- Stability is provided by weight of the earth on the base slab & the weight of retaining wall.



Forces acting on Retaining wall:-

Generally two types of forces act on retaining wall:

1. Acting Active earth pressure.

2. Passive earth pressure.

$$P = k \gamma h$$

Where, P = earth pressure.

γ = unit weight of retained material

h = Depth of the section below the earth surface.

k = co-efficient of earth pressure that depends on the properties of soil.

If k_a = co-efficient of active earth pressure.

k_p = co-efficient of passive pressure.

$$\text{Then, } P_a = k_a \gamma h$$

$$P_p = k_p \gamma h$$

P_a = active earth pressure

P_p = passive earth pressure.

Net active earth pressure on retaining wall = ~~P_a~~ $P_a = \frac{1}{2} k_a \gamma h^2$.

Net passive earth pressure on retaining wall = $P_p = \frac{1}{2} k_p \gamma h^2$.

$$P_p = \frac{1}{2} k_p \gamma h^2$$

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

ϕ = Angle of repose.

$\gamma + \gamma' \delta$ = Angle of surcharged,

then $P_h = P_a \cos\delta$

$$P_v = P_a \sin\delta$$

Stability requirement:

The Stability requirement of the retaining wall has to satisfy the following conditions:

1. Stability against overturning.
2. Stability against sliding.
3. Base width must be adequate to distribute the load to the foundation soil without exceeding the bearing capacity of the soil.

Date - 10/04/2019

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

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

Factor of safety for overturning moment

$$= \frac{\text{Resisting moment}}{\text{Overturning moment}}$$

$$I = \frac{0.9 W n_y}{1.4 P_h (H/3)}$$

$$\Rightarrow 1.55 = \frac{W n_y}{P_h (H/3)}$$

where, n_y = centre of gravity of vertical loads from the toe.

H = depth of the bottom slab below the earth surface.

P_h = horizontal component of earth pressure.

Factor of safety against sliding :-

$$F.O.S. = \frac{\text{Resisting force}}{\text{Sliding force}}$$

$$F.O.S. \geq 1.4 = \frac{0.9 \mu w}{P_h}$$

$$\Rightarrow 1.55 = \frac{\mu w}{P_h}$$

μ = co-efficient of friction between soil & footing.

Proportioning of cantilever wall:

Height of wall:

Minimum depth of foundation below ground level should be about 1m.

It is necessary to obtain preliminary dimensions of the wall based on certain thumb rule.

Thickness of footing:

The base thickness is usually 10% of the total height with a minimum of about 30 cm the exact thickness will occurs be governed by the bending moment & shear force consideration.

Thickness of vertical wall:

The thickness at top of the wall should not be less than 15 cm. The thickness of vertical wall is determined as required for bending moment & shear force. It may be about 15% of the wall height.

Design of heel:

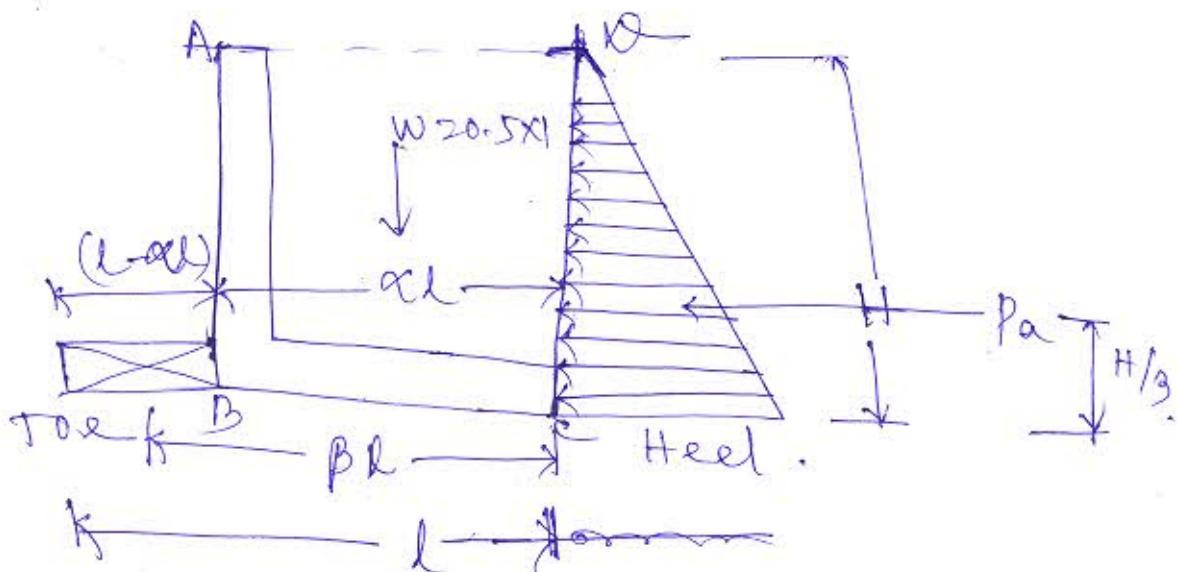
Normally the resultant pressure due to downward weight of earth fill & upward bearing pressure is downward which causes tension on the top face of

~~foot~~

Design of toe :-

Normally weight of earth above the toe is neglected & it is design for upward acting bearing pressure as a cantilever beam.

Position of vertical slab on the base footing:-



$$\frac{e}{H} = \sqrt{\frac{k_a \cos \beta}{(1-m)(1+3m)}} \quad \text{if } \beta \neq 0$$

m = length of toe + ~~length of heel~~

$$q_s = \frac{\gamma h}{P_s}, \quad 1 - \frac{4}{99} \text{ if } \beta = 0 \\ 1 - \frac{3}{89} \text{ if } \beta \neq 0,$$

P_s = Bearing capacity of soil length of base,

h = Depth of top of heel slab.

$$\begin{aligned}
 M_{u,lim} &= 0.138 f_y b d^2 \\
 &= 0.138 \times 20 \times 230 \times 300^2 \\
 &\approx 158.7 \text{ kNm}
 \end{aligned}$$

$$\begin{aligned}
 M_{u2} &= M_u - M_{u,lim} \\
 &= 28200 - 158.7 \\
 &= 41.3 \text{ kNm}
 \end{aligned}$$

$$M_{u2} = f_{sc} \& A_{sc} (d - a)$$

$$\Rightarrow 41.3 \times 10^6 = 0.87 f_y A_{sc} (500 - 30)$$

$$\begin{aligned}
 \Rightarrow f_{sc} &= \frac{41.3 \times 10^6}{0.87 \times 415 \times 480} \\
 &= 260 \text{ mm}^2
 \end{aligned}$$

Let us provide $\frac{1}{16}$ mm dia bar.

$$n \times \frac{\pi}{4} \times 16^2 = 260$$

$$\Rightarrow n = \frac{260 \times 4}{\pi} \times \frac{1}{16^2}$$

$\therefore 4 \text{ no}$ rebar provide 4 nos. 16 mm dia bar

$$\begin{aligned}
 A_{sc} &= 4 \times \frac{\pi}{4} \times 16^2 \\
 &= 804 \text{ mm}^2
 \end{aligned}$$

$$A_{s2} = \frac{A_{sc} f_{sc}}{0.87 f_y} = 260 \text{ mm}^2$$

$$\begin{aligned}
 f_{sc}(M_u)_{lim} &= 0.87 f_y A_{sc} (d - a) \\
 \Rightarrow 158.7 &\times 0.87 \times f_y
 \end{aligned}$$

$$\rightarrow 158.7 \times 10^6 = 0.87 \times 415 \times (A_{st}), (C_{st} = 0.42 \times 240)$$

$$\Rightarrow A_{st,1} = \frac{158.7 \times 10^6}{0.87 \times 415 \times 240} = 1110 \text{ mm}^2$$

$$M_u \text{ lim} = 0.48 \times 380$$

$$\approx 240 \text{ mm}$$

$$A_{st} = A_{st,1} + A_{sp}$$

$$\approx 1110 + 260$$

$$= 1370 \text{ mm}^2$$

Let us provide 20mm dia bars

$$\frac{\pi \times 20^2}{4} = 1370$$

$$\Rightarrow n = 4.36 \div 5 \text{ nos.}$$

$$A_{st,2} = \frac{5 \times \pi}{4} \times 20^2 = 1570 \text{ mm}^2$$

$$1 \times \frac{5}{4} \times 300 = 375$$

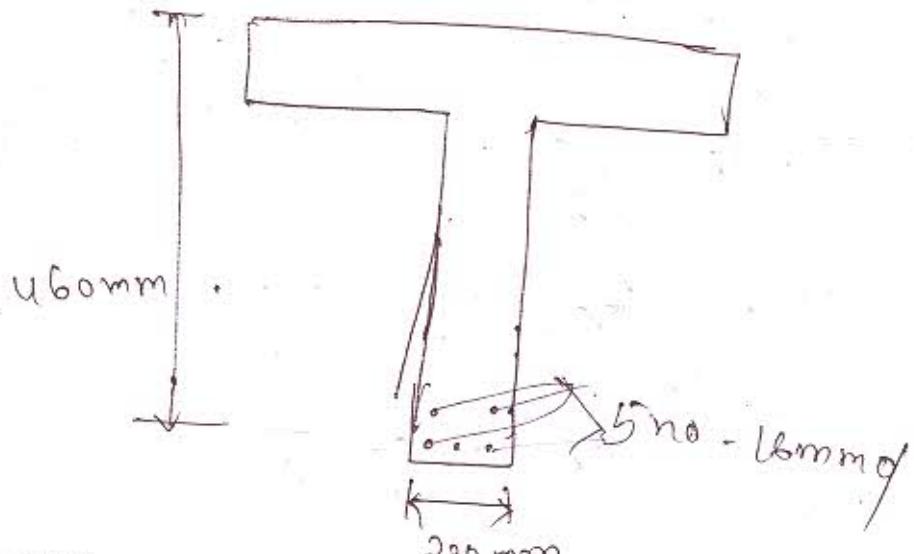
and other material = 2000 mm length required

$$20 \times \pi \times 12 = 754$$

$$5 \text{ mm nos.} =$$

$$5 \text{ mm nos.} = \frac{754}{300} = 2.51$$

Ans - 10, Rebar F8.0 - min GMD
F8x F8.0 = 1000



$$bw = 230 \text{ mm}$$

$$d = 460 \text{ mm}$$

$$V_u = 52.5 \text{ kN}$$

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

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

$$\tau_c = \frac{V_u}{bd} = \frac{52.5 \times 10^3}{230 \times 460} = 0.49 \text{ N/mm}^2$$

$$\tau_c \text{ max} = 2.8 \text{ N/mm}^2$$

$\tau_c < \tau_c \text{ max (Safe)}$

$$P_t = \frac{100 A_s}{bd}$$

$$A_s t = 5 \times \frac{\pi}{4} \times 16^2 = 1005.31 \geq 1006 \text{ mm}^2$$

$$P_t = \frac{100 \times 1006}{230 \times 460} = 0.95 \text{ N}$$

$$\begin{aligned} \epsilon_c &= \frac{0.85 - 0.95 - 0.75}{1 - 0.75} \times 0.62 + \frac{0.95 - 1}{0.75 - 1} \times 0.56 \\ &\geq 0.608 \end{aligned}$$

$$\begin{array}{r} 0.75 - 0.75 \\ 1.00 - 0.62 \end{array}$$

$$\frac{\tau_c}{2} = \frac{0.608}{2} = 0.304$$

$$\tau_v > \frac{\tau_c}{2}$$

$$\tau_v < \tau_c$$

$$\frac{\tau_c}{2} < \tau_v < \tau_c$$

Minimum shear reinforcement will be provided.

$$\frac{A_{sv}}{bsv} \geq \frac{0.4}{0.87 f_y}$$

for stirrups, $f_y = 250 \text{ N/mm}^2$,
let us provide 2 legged 6 mm dia stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 6^2$$

$$\approx 56.54 \text{ mm}^2$$

$$\approx 57 \text{ mm}^2$$

$$\frac{57}{230 \times s_v} \geq \frac{0.4}{0.87 \times 250}$$

$$\Rightarrow s_v \leq \frac{57 \times 0.87 \times 250}{230 \times 0.4}$$

$$\Rightarrow s_v \leq 134.75 \text{ mm.}$$

The spacing shall not be exceed

$$(a) 0.75 \times d = 0.75 \times 460 = 345 \text{ mm}$$

$$(b) S_v = 130 \text{ mm}$$

$$(c) 300 \text{ mm}.$$

Let us provide ~~so~~ 2 leg 6mm dia stirrups
with 130 mm c/c spacing.

$$V_u = 90 \text{ kN}.$$

$$\tau_c = \frac{V_u}{bd} = \frac{90 \times 10^3}{230 \times 460} = 0.85 \text{ N/mm}^2$$

$$A_{st} = 5 \times \frac{\pi}{4} \times 16^2 = 1006 \text{ mm}^2.$$

$$(\tau_c)_{max} = 2.8 \text{ N/mm}^2.$$

$$\tau_c = 0.608 \text{ mm}^2.$$

$$\tau_c > \tau_c.$$

$$V_u = \frac{0.87 f_y A_{sv} d}{S_v}$$

$$\cancel{V_u = V_u - \tau_c = 90 \times 10^3 - 0.608}$$

$$\cancel{= \tau_c b d = 0.608 \times 230 \times 460 = 64326.4 \text{ N}} \\ \cancel{= 64.32 \text{ kN.}}$$

$$V_u = V_u - \tau_c b d = 90 - 64.32 \\ = 25.7 \text{ kN.}$$

$$\cancel{S_v \times 16 \times 2 = 81.07 \text{ mm}}$$

$$\cancel{S_v \times 16 \times 6 = 81.0 \times 46 = 3672 \text{ mm}}$$

$$L_x = 4.0 \text{ m}.$$

$$L_y = 5.5 \text{ m}.$$

$$f_{ck} = 25 \text{ N/mm}^2.$$

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

$$\frac{L}{R} = 35 \times \frac{3.5}{4}$$

$$= 30.62$$

$$\approx 32$$

$$\frac{L}{R} = 32$$

$$\Rightarrow \frac{4800}{R} = 32$$

$$\Rightarrow R = \frac{4800}{32} = 125.00 \text{ mm}.$$

$$\approx 130 \text{ mm}.$$

$$d = 130 - 30 = 100 \text{ mm}.$$

$$(L_{eff})_x = 4.0 + 0.1 = 4.1 \text{ m}$$

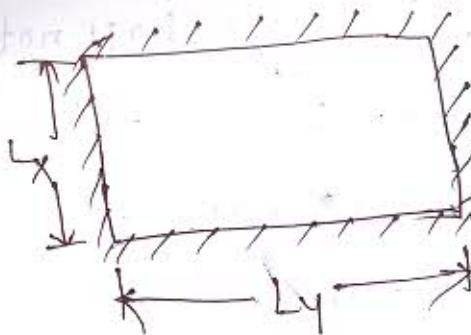
$$(L_{eff})_y = 5.5 + 0.1 = 5.6 \text{ m}.$$

$$\frac{L_{eff}}{R} = \frac{5.6}{130} = 1.36 \approx 1.4$$

Superimposed load = 8 kN/m².

$$WL \text{ of floor} = 25 \times 0.13 = 3.25 \text{ kN/m}^2.$$

$$\text{Floor finishing} = 24 \times 0.13 = 3.12 \text{ kN/m}^2,$$



Total load = 11.37

$$\text{Factored Ver} = 11.37 \times 1.5 = 17.055 \text{ kN/m}^2$$

For 1 m.

$$= 17.055 \text{ kN/m}$$

As per table - 27,

$$\alpha_n = 0.099$$

$$\alpha_y = 0.051$$

$$M_u = 0.099 \times 17.055 \times 4.12 \\ = 26.38 \text{ kNm}$$

$$M_y = 0.051 \times 17.055 \times 4.12 \\ = 14.62 \text{ kNm}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d / 2}} \right] b d$$

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

$$d \geq d_c = \sqrt{\frac{26.38 \times 10^6}{0.138 \times 25 \times 1000}}$$

$$= 90.69 \text{ mm} < 100 \text{ mm} \quad (\text{Safe})$$

Let 100 mm.

$$M_u = 26.38$$

A_{st} 5 Pacing Prent

$$M_y = 14.62$$

930 mm² 84 80 mm
930 mm² 179 170 mm

Area of distribution bar,

$$A_{\text{dist}} = \frac{0.12}{100} \times 1000 \times 130 = 156 \text{ mm}^2$$

Spa = 8 mm & h =

$$\text{spacing} = \frac{1000}{\frac{156}{\pi \times 8^2}} = 322.21 \approx 300 \text{ mm}$$

check for shear

$$V_n = \frac{w h n}{3} \quad V_y = \frac{w h n c}{2 + \gamma c}$$

$$= \frac{17.055 \times 4.1}{3} \quad = \frac{17.055 \times 4.1 \times 1.4}{2 + 1.4}$$
$$= 23.31 \text{ kN} \quad = 28.79$$

$$\tau_v = \frac{28.79 \times 10^3}{1000 \times 100} = 0.28$$

$$\tau_{\text{max}} = 3.1$$

$\tau_v < \tau_{\text{max}}$ leave

check for bond

$$(z_{bd})_n = \frac{V_n}{\sum z_{bd} \times \text{area of bar}}$$