Design of Column by LSM

Pg 69 of IS code 456:2000.

Assumption:

1. All assumption discussed for beam / slab (for flexural member) as per 38.1 (IS code) (a to f) are valid for column also.

In addition, there are two more assumption:-

1. Max^m compressive strain in concrete in axial compression in case of column is 0.002.
2. Max^m compressive strain at highly compressed extreme fibre, in case the section subjected to axial compression and bending and when there is no tension in the section will be.

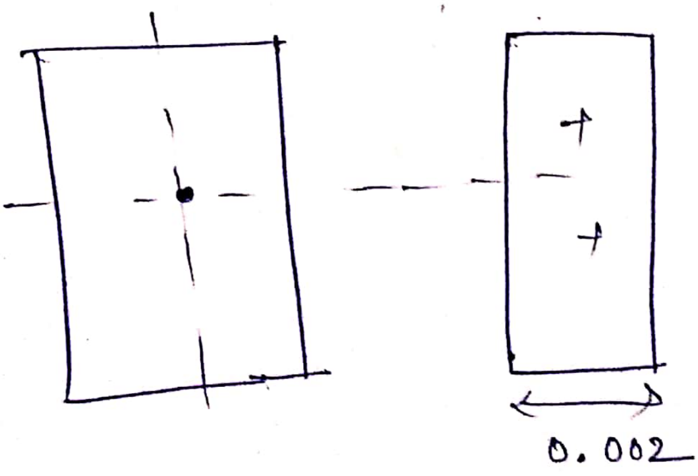
= 0.0035 − 0.75 times the strain at least compressed extreme fibre.

$$\epsilon_{hc} = 0.0035 - 0.75\,\epsilon_{lc}$$

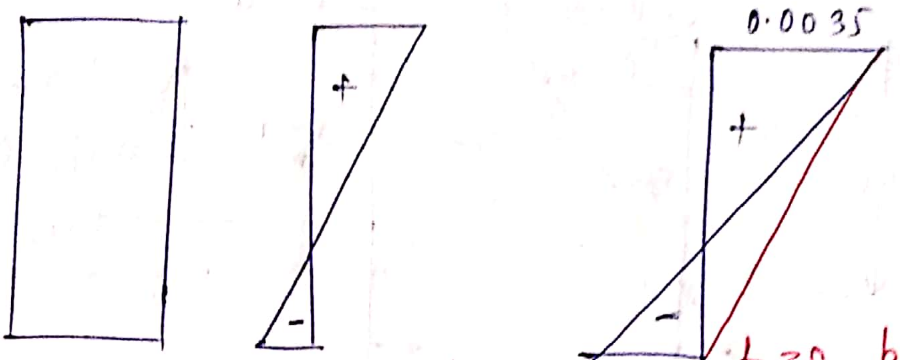
3. $E_{concrete} = E_{steel}$ & $\sigma - \epsilon$ relationship for steel is same in compression & tension.

There are three cases:

① Axial Compression.



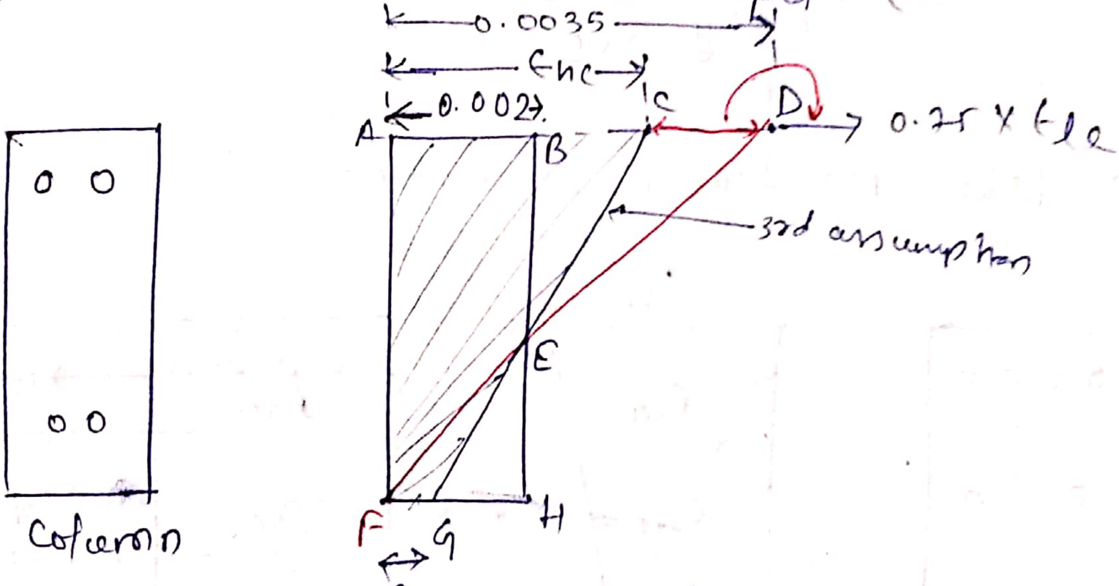
② Bending stress
(Axial Comp + Bending)
when tension is developed.



stress diag.

$t=0$ but strain will not exceed 0.0035.

③ When tension is not developed. (2nd assumption)



ϵ_{lc} (least strain fibre)
strain diag.

$\Delta BED \simeq \Delta EFH$

(BD = AD − AB)

$$\frac{BD}{FH} = \frac{ED}{EF} = \frac{0.0015}{0.002} = 0.75$$

$\Delta CED \simeq \Delta EFG$

$$\frac{ED}{EF} = \frac{CD}{FG} = 0.75$$

$$CD = 0.75\,FG = 0.75\,\epsilon_{lc}$$

$$\epsilon_{hc} = AC = AD - CD = 0.0035 - 0.75\,\epsilon_{lc}$$

(Proved)

A column is defined as a compression member, the effective length of which exceeds three times its lateral dimension. Compression members whose lengths do not exceed three times their least lateral dimension are classified as pedestals.

R.C. Columns concrete has a high compressive strength and a low tensile strength. Hence, theoretically concrete should need no reinforcement when it is subjected to compression. Reinforcements are provided in order to reduce the size of columns. Though a column is mainly a compression member, it is liable to some moment due to eccentricity of loads or transverse loads or due to its slenderness. Such moments may occur in any direction and so it is necessary to provide reinforcement near all faces of the column. These reinforcements form the longitudinal steel. In order to maintain the position of the longitudinal reinforcement and also to prevent their buckling which may cause splitting of concrete, it is necessary to provide transverse reinforcements in the form of lateral ties or spirals at close pitch. The transverse reinforcement also assists in confining the concrete.

Classification of columns A column may be classified on the basis of its shape, its slenderness ratio, the manner of loading and the type of lateral reinforcement provided. A column may have a section which may be square, rectangle, circular or a desired polygon.

Depending on the slenderness ratio, a column may be a short or a long column. The slenderness ratio of a column is the ratio of the effective length of the column to its least lateral dimension. A column whose slenderness ratio exceeds 12 is a long column. A column whose slenderness ratio does not exceed the above limit is a short column.

Based on the manner of loading, columns may be classified into

(*i*) Axially loaded columns

(*ii*) Columns subjected to axial load and uni-axial bending

(*iii*) Columns subjected to axial and bi-axial bending.

Columns may also be classified based on the type of lateral reinforcement provided. On this basis, columns are classified into

(*i*) Tied columns in which separate or individual ties are provided surrounding the longitudinal reinforcement

(*ii*) Spirally reinforced columns in which helical bars are provided surrounding the longitudinal reinforcement.

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Effective Length of Compression Members [I.S. 456]

Degree of end restraint of compression member	*Theoretical value of effective length*	*Recommended value of effective length*
1 Effectively held in position and restrained against rotation at both ends. (*i.e.*, both ends are fixed).	0.50 l	0.65 l
2 Effectively held in position at both ends, restrained against rotation at one end (*i.e.*, fixed at one end and hinged at the other end).	0.70 l	0.80 l
3 Effectively held in position at both ends but not restrained against rotation (*i.e.*, both ends are hinged).	1.00 l	1.00 l
4 Effectively held in position and restrained against rotation at one end, and the other restrained against rotation but not held in position.	1.00 l	1.20 l
5 Effectively held in position and restrained against rotation at one end, and at the other partially restrained against rotation but not held in position.	—	1.50 l
6 Effectively held in position at one end, but not restrained against rotation, and at the other end restrained against rotation but not held in position.	2.00 l	2.00 l
7 Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end (*i.e.*, fixed at one end and free at the other end).	2.00 l	2.00 l

Some other I.S code provisions:

1) Min^m % of steel (main bar) for a column:

= 0.80 % of Gross C/s area.

2) Max^m % of steel (Main bar).

= 6 % of Ag (when none of the bars are overlapped)

= 4 % of Ag (when bars are lapped).

Cover

- Min^m cover to a column reinforcement equals to 40mm or diameter of bar whichever is greater.

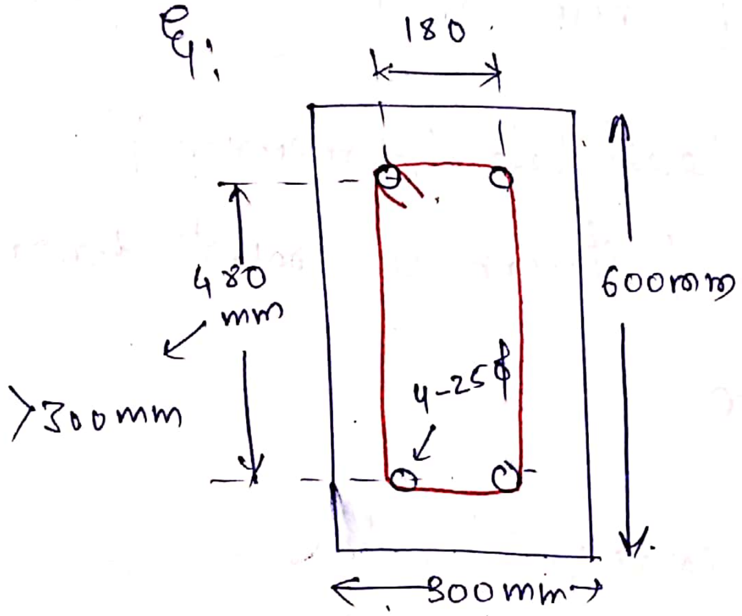
(3) Minm dia of Main bar for column = 12mm

(4) Minimum number of main bars:

Square or Rectangular = 4
Circular = 6.

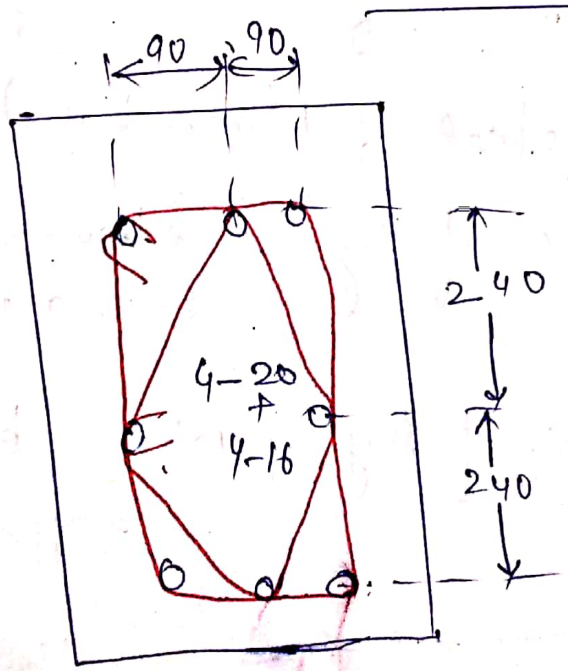
(5) Maxm distance b/w main bars = 300mm

Eg:



← wrong detailing.

So Correct detailing: (by keeping Ast constant.)
Decreasing dia & increasing nos.



Design of lateral ties:

a) **Diameter**: shall be max^m of following.

Max^u
(I) $\frac{\phi_{main}}{4}$ (ϕ_{main} = dia of largest diameter bar)
(II) 5mm / 6mm

b) **Spacing**: shall be min^m of the following.

Min^m
(I) LLD (least lateral dimension)
(II) $16\phi_{main}$ (ϕ_{main} of min^m dia bar)
(III) 300 mm
(IV) 48ϕ (where ϕ is diameter of transverse reinforcement)

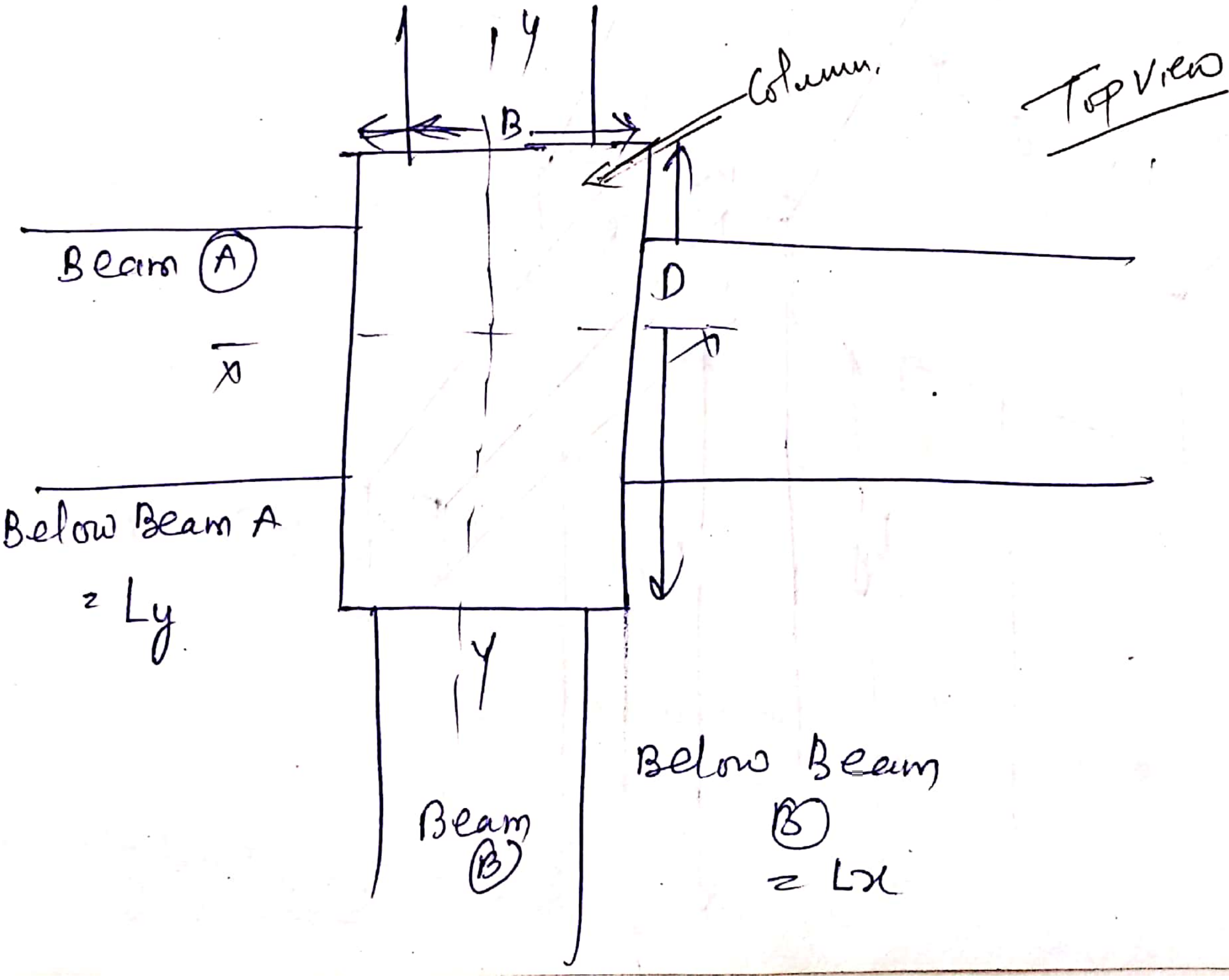
39.2 Minm eccentricity:

All column shall be designed for minm eccentricity of

$$\left(\frac{\text{Unsupported length of column}}{500}\right) + \left(\frac{\text{Lateral dimension}}{30}\right)$$

or 20 mm

whichever is more.



39.3 Design of a short, axially loaded column

If the minm eccentricity calculated as per 39.2 (of IS456: 2000) is less than 0.05 × lateral dimension,

The column can be designed using following eqn:

$$\boxed{P_u = 0.40 f_{ck} A_c + 0.67 f_y A_{sc}} \quad - (A)$$

The formulae can be used only when following condition are fulfilled.

1. The column should be short column

$$\left(\frac{L_{ex}}{D}\right) \text{ or } \left(\frac{L_{ey}}{B}\right) < 12$$

2. The column should be axially loaded column
- above formulae cannot be used, if the column is subjected to any moment or if the load is acting at an eccentricity.

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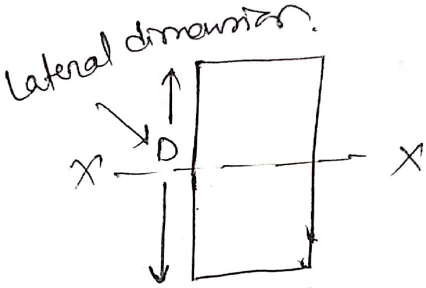
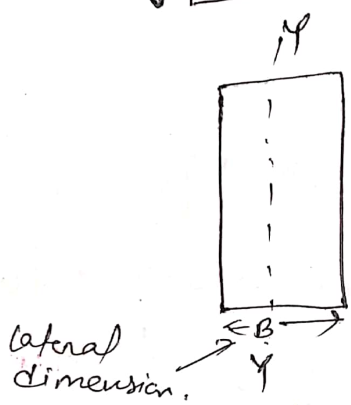
(3) $e_{min\,xx} \nless 0.05 \times lat_{Dim}$

$$\left(\frac{L_x}{500} + \frac{D}{30}\right) \nless 0.05 \times D$$



(4) $e_{min\,yy} \nless 0.05 \times lat_{dim}$

$$\left(\frac{L_y}{500} + \frac{B}{30}\right) \nless 0.05 \times B$$



Q. A R.C.C column of size 460 × 600 mm having eff. length of 3 m for an axial load of 2500 kN (service load) use M20 / Fe 415 use LSM.

Sol) As we can use the code formulae.

$$P_u = 0.40 f_{ck} A_c + 0.67 f_y A_{sc}$$

if following cond^n are fulfilled.

(1) Load on Column

Axial Column = 2500 kN

$P_u = 1.5 \times 2500 = 3750$ kN

No moment OK

(2) Slenderness ratio:

$$\frac{L_{eff}}{LLD} = \frac{3000}{460} = 6.52 < 12$$

It is called short - Column (OK)

(3) Minm ecc about xx.

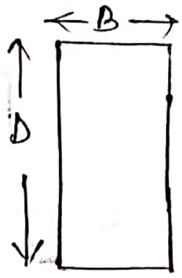
$$e_{min\,xx} = \frac{L_x}{500} + \frac{D}{30}$$

$$= \frac{3000}{500} + \frac{600}{30}$$

$= 26.00 > 20$ mm

Note:

[It is a rectangular Column].



Here B < D

If it was a square Column then B = D

& If it was a Circular Column B = D = diameter of Circular Column

$0.05D = 0.05 \times 600 = 30$ mm

$e_{min\,xx} < 0.05D$ — okay

4. e_{min} about y-y.

$$e_{min\,yy} = \frac{L_y}{500} + \frac{B}{30}$$

$$= \frac{3000}{500} + \frac{460}{30}$$

$$= 22.53 > 20 \text{ mm}$$

$0.05B = 0.05 \times 460 = 23$ mm

$e_{min\,yy} < 0.05B$ okay

(A) So we can use.

$$P_u = 0.40 f_{ck} A_c + 0.67 f_y A_{sc}$$

$$\Rightarrow 3750 \times 10^3 = 0.40 \times 20 \times (460 \times 600 - A_{sc})$$
$$+ 0.67 \times 415 \times A_{sc}.$$

$$\therefore A_{sc} = 5710 \text{ mm}^2$$

No. of 25 mm ϕ bars

$$\text{No.s} = \frac{5710}{\frac{\pi}{4}(25)^2} = 11.6 \text{ say } 12.$$

provide 12 – 25 mm ϕ.

(B) Design of lateral ties:

(1) Dia of i) $\frac{25}{4} = 6.25$; ii) 6 mm — use 8 mm

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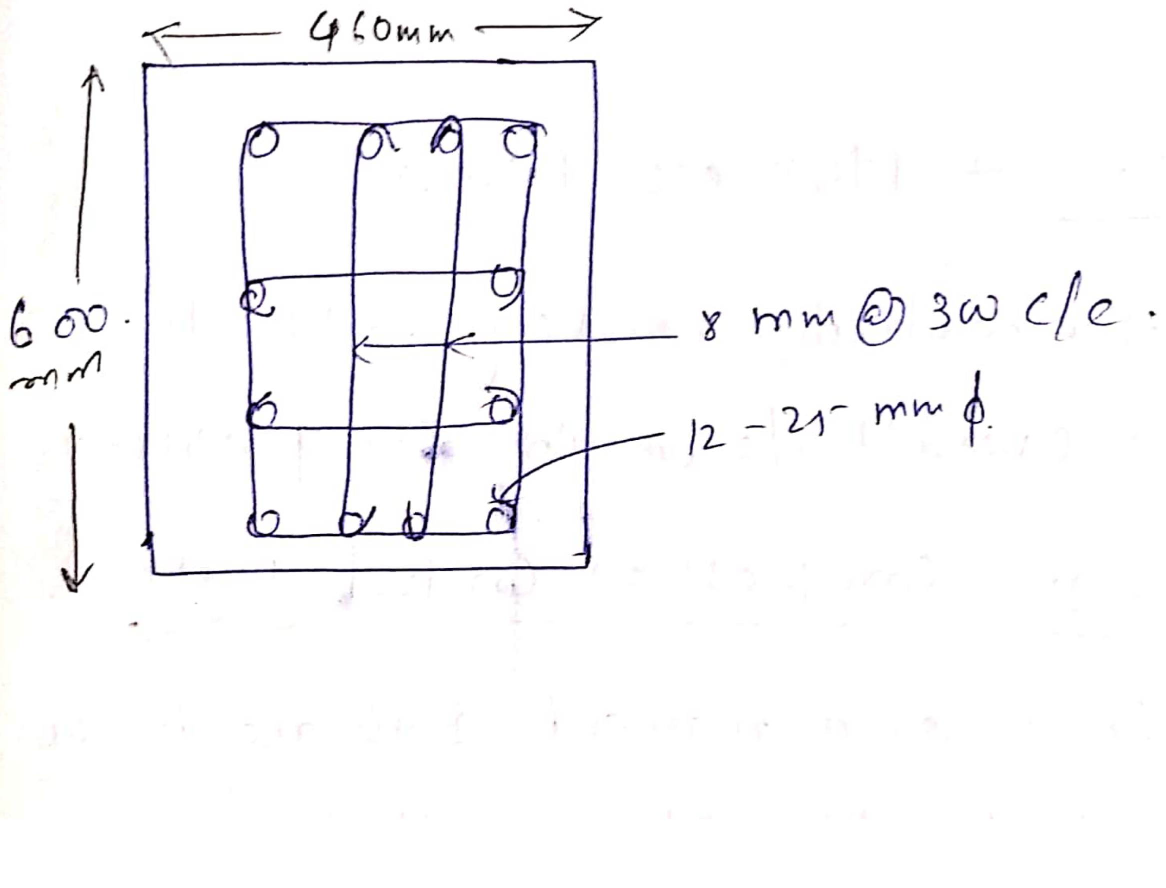
(iii) Spacing i) LLD = 460 mm
ii) 16 X 25 = 400 mm
iii) 300 mm
(iv) 48 X 8 = 384 mm

@ 300 c/c.



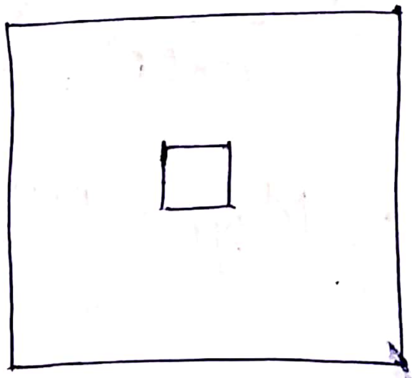
foundation:

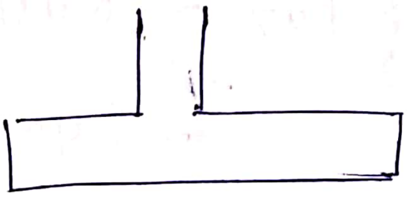
→ **Types of foundation:**

① Isolated footing:

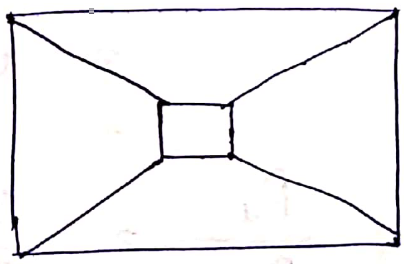
Types of Isolated footing:

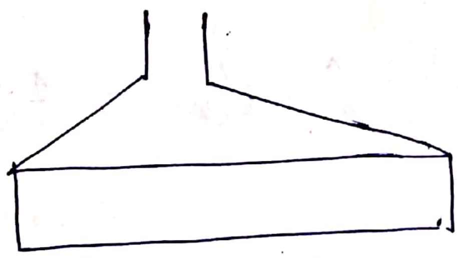
- → Rectangular
- → square
- → circular
- → Uniform thickness
- → sloped footing:





Uniform thickness.

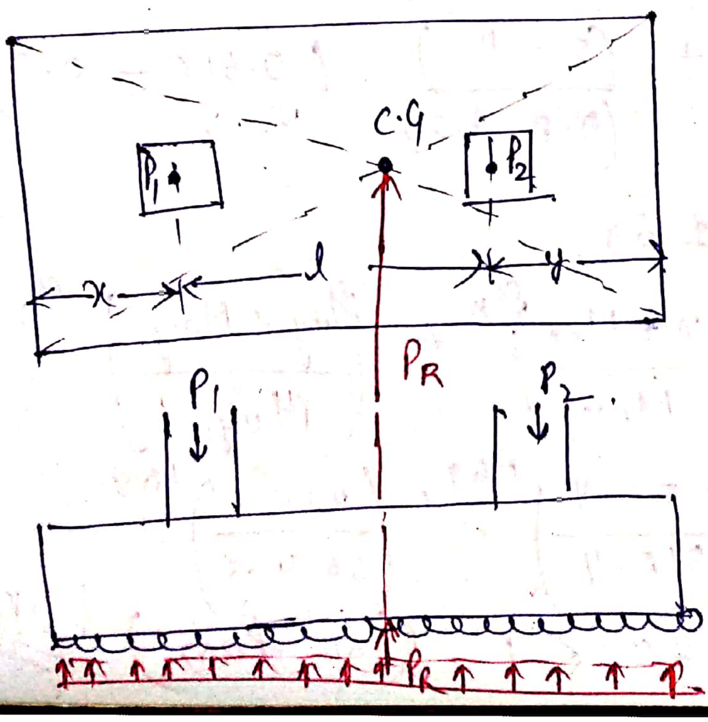




sloped footing

② Combined footing:

1 foundation for two columns.



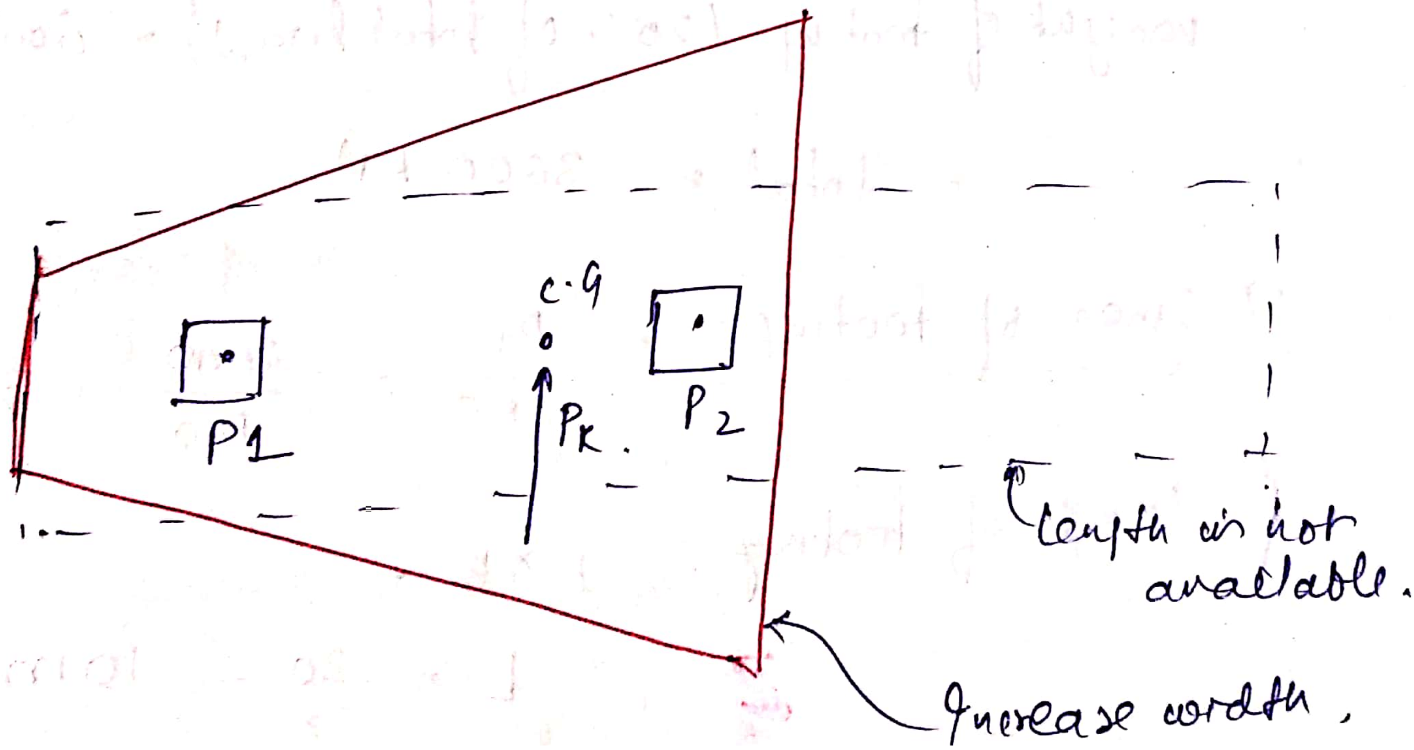
so pressure $\boxed{\frac{P_R}{A}}$ is uniform

In case of combined footing :

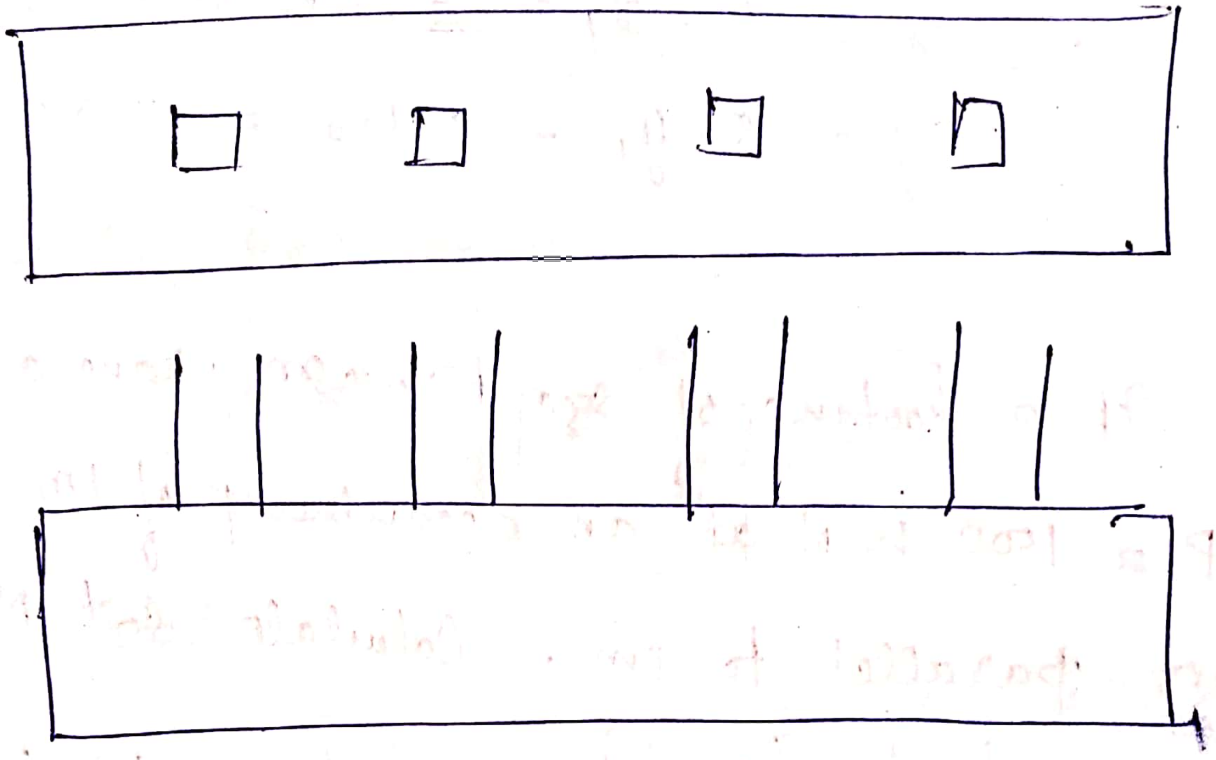
C.G of load is matched with C.G of foundation, So that soil pressure over foundation is uniform.

Combined footing :

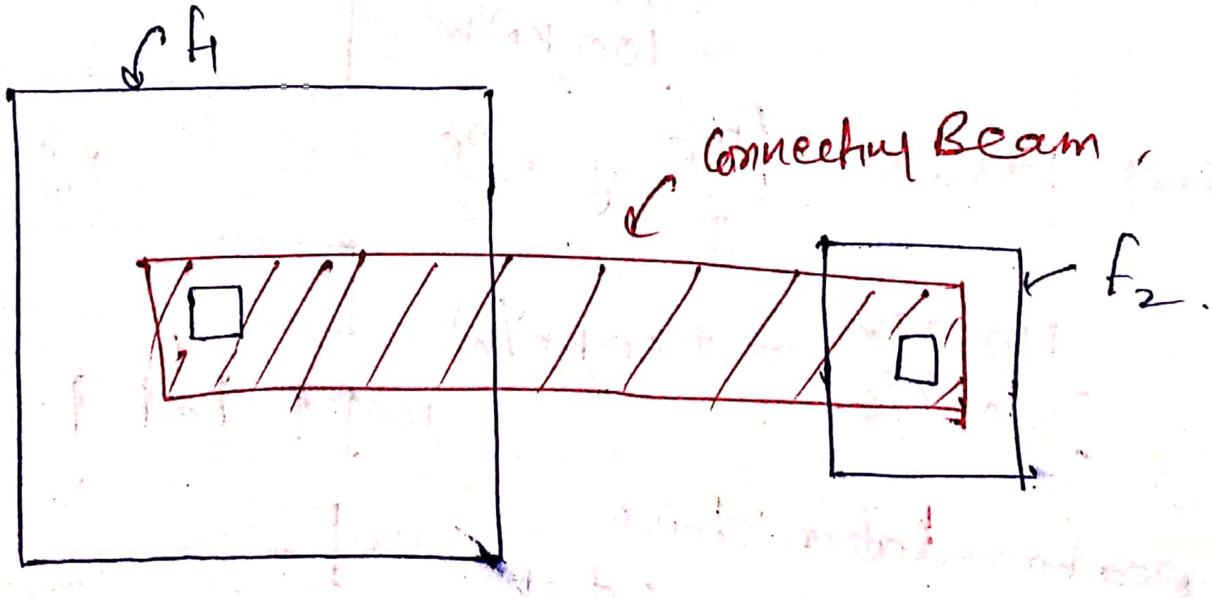
- → Rectangular.
- → Trapezoidal (If projection in the dirn of length is not available, trapezoidal footing is provided to keep C.G. of load matching with C.G of footing.) .



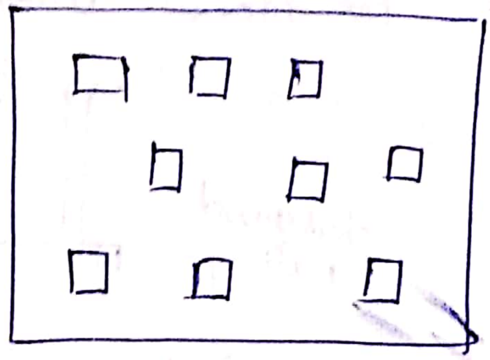
③ strip footing :



④ strap footing :



(5) Raft footing:



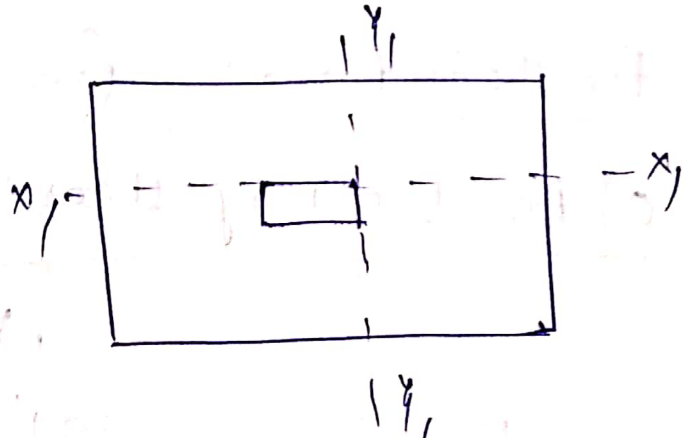
Design of Isolated footing:

Critical Section:

(1) for Bending Moment: (34 . 2.3.2 p-65).

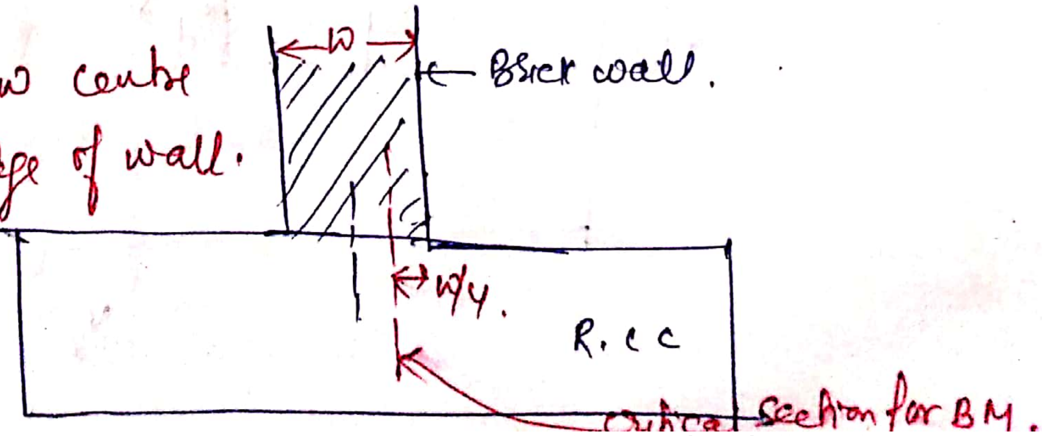
(a) If foundation is provided for a R.CC column or R.CC wall.

Then critical section is at the face of the column.

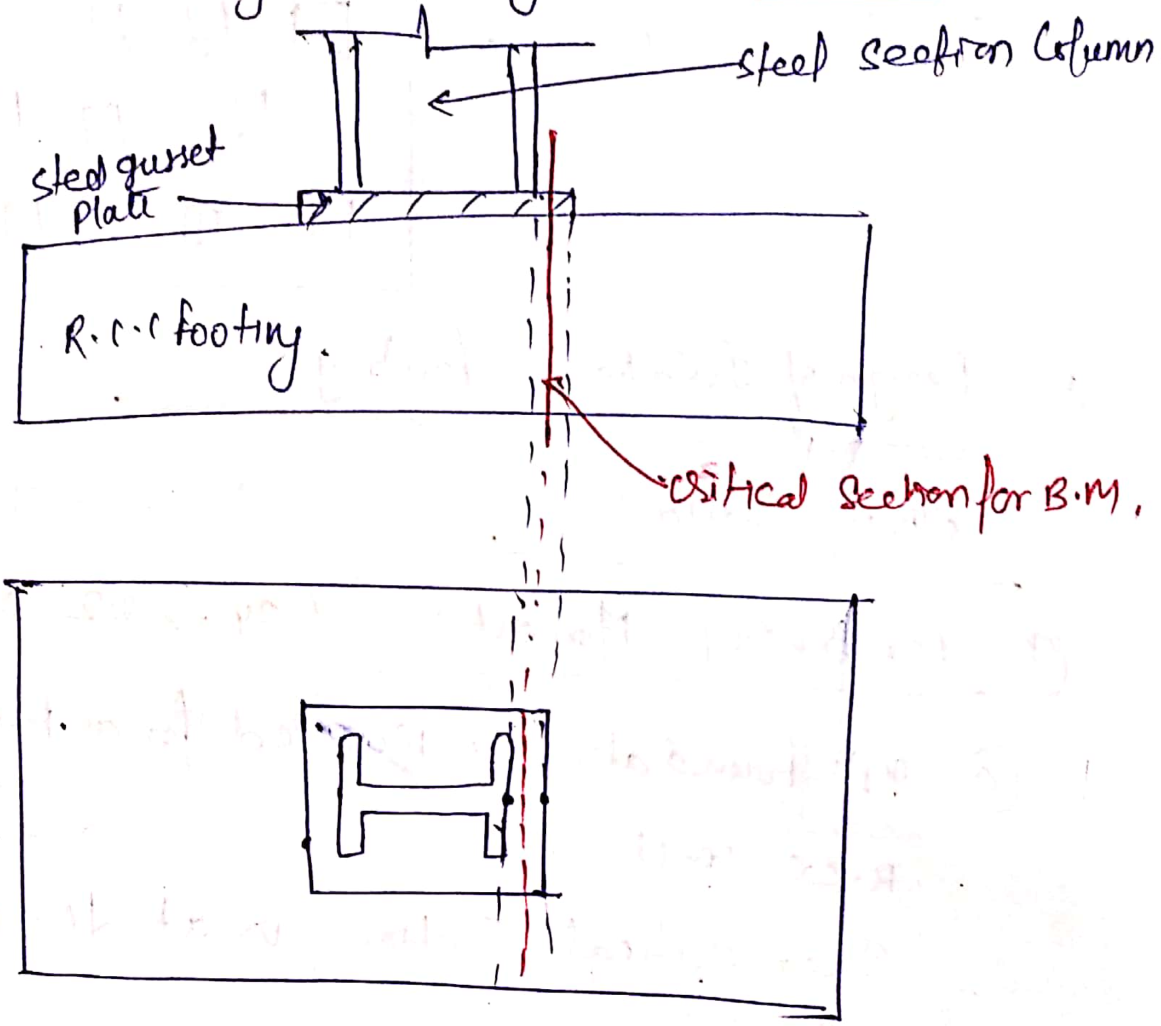


(b) If the R.C.C foundation is provided for a brick wall.

p-65 Halfway b/w centre line and Edge of wall.

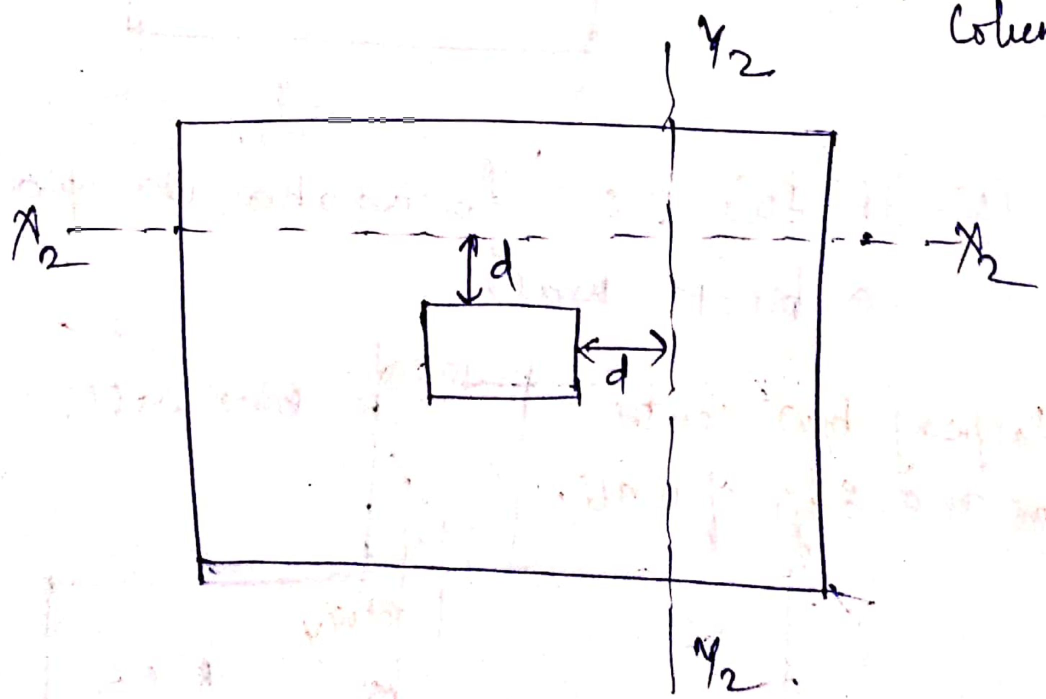


(c) for footing under gusseted bases:



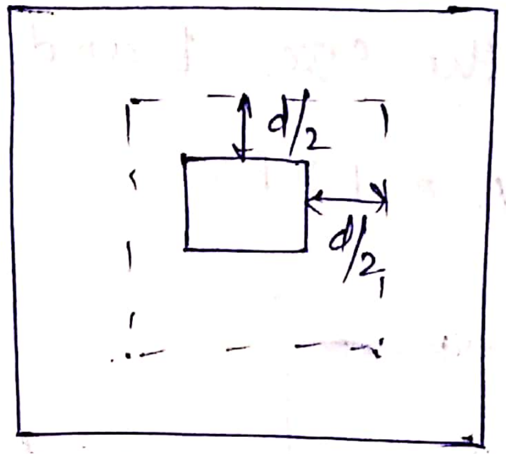
(2) for shear force: (§2.2.4)

a) for one way shear: (At d distance from face of column).



b) for two way shear (or punching shear).
[at $d/2$ distance from the face of column].

d → effective depth of footing.

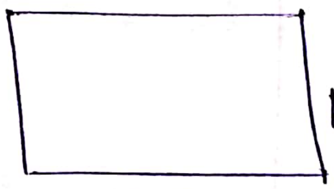


Design of a rectangular footing for a R.C.C column:

Given values:-

① Load from column (working load) = P

② Size of column = a×b



③ Safe bearing capacity of soil = q_o

④ Grade of concrete and steel. (f_{ck} / f_y).

Step 1: Size of foundation:

Same for WSM and LSM. Load not to be factored for LSM.

Load from column = P.

Weight of footing (10 to 20% of P) = 0.2P

∴ Total load P_T = 1.20 P

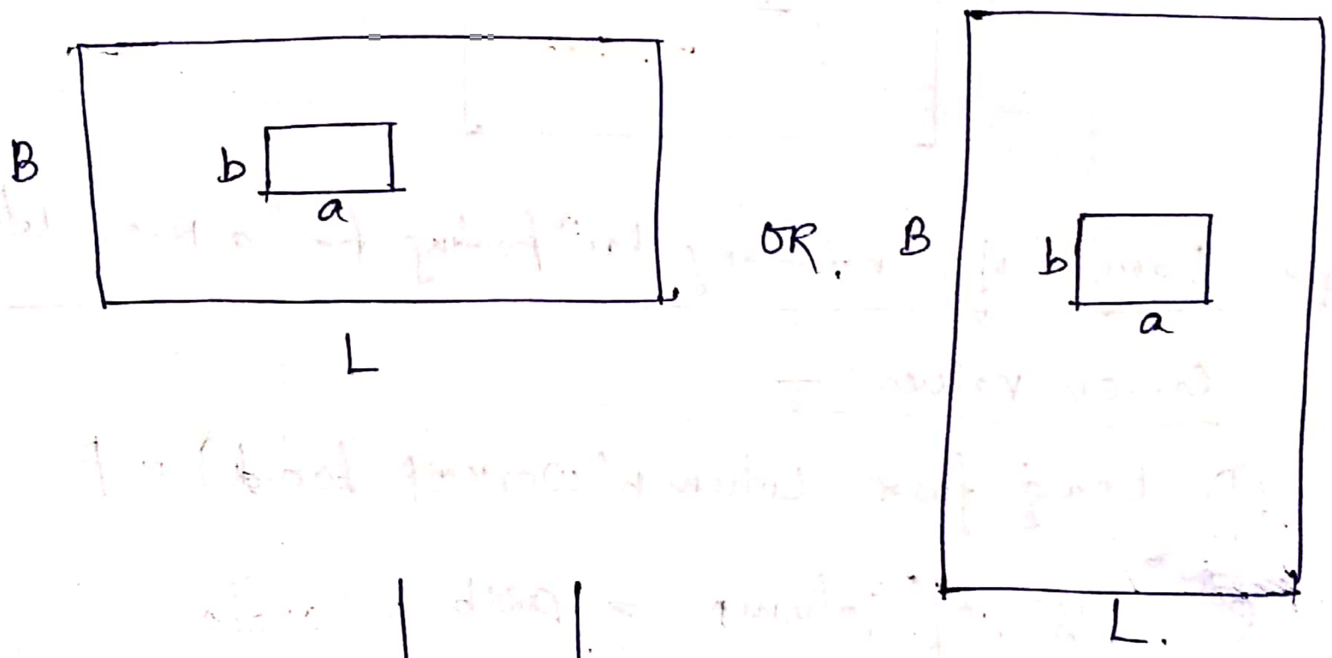
RCC.

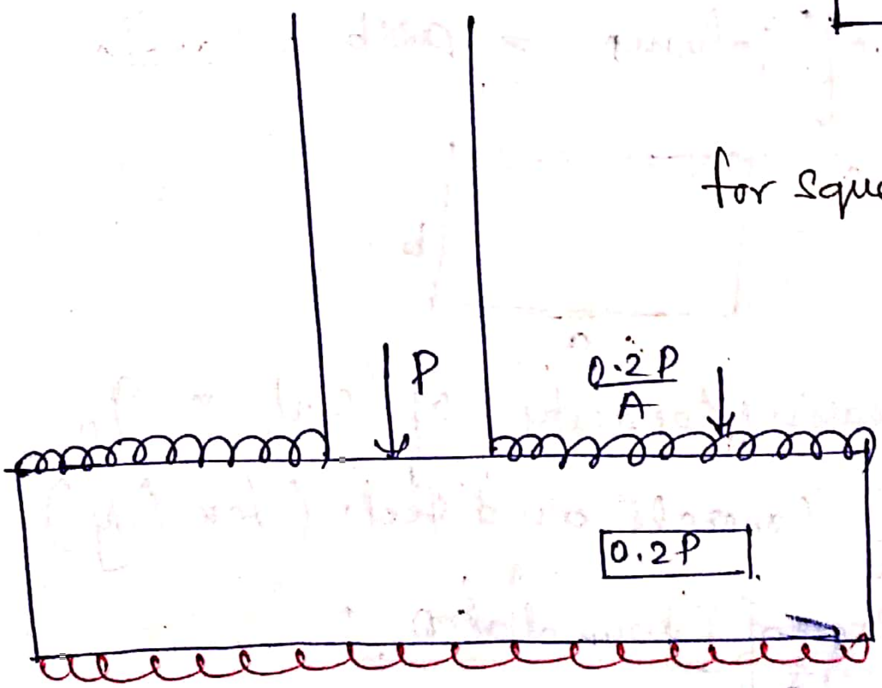
Area of footing req:

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

Decide the size L and B.

$$A = L \times B.$$





for square footing $B = L$

So $A = L^2$ or B^2

$$q_0 = \left(\frac{1.2P}{A}\right)$$

Net soil pressure acting over foundation (used for design purpose).

$$w_o = q_o - \frac{0.2P}{A}$$

$$= \frac{1.20P}{A} - \frac{0.2P}{A}$$

$$\therefore \boxed{w_o = \frac{P}{A}} \leftarrow \text{WSM.}$$

In LSM

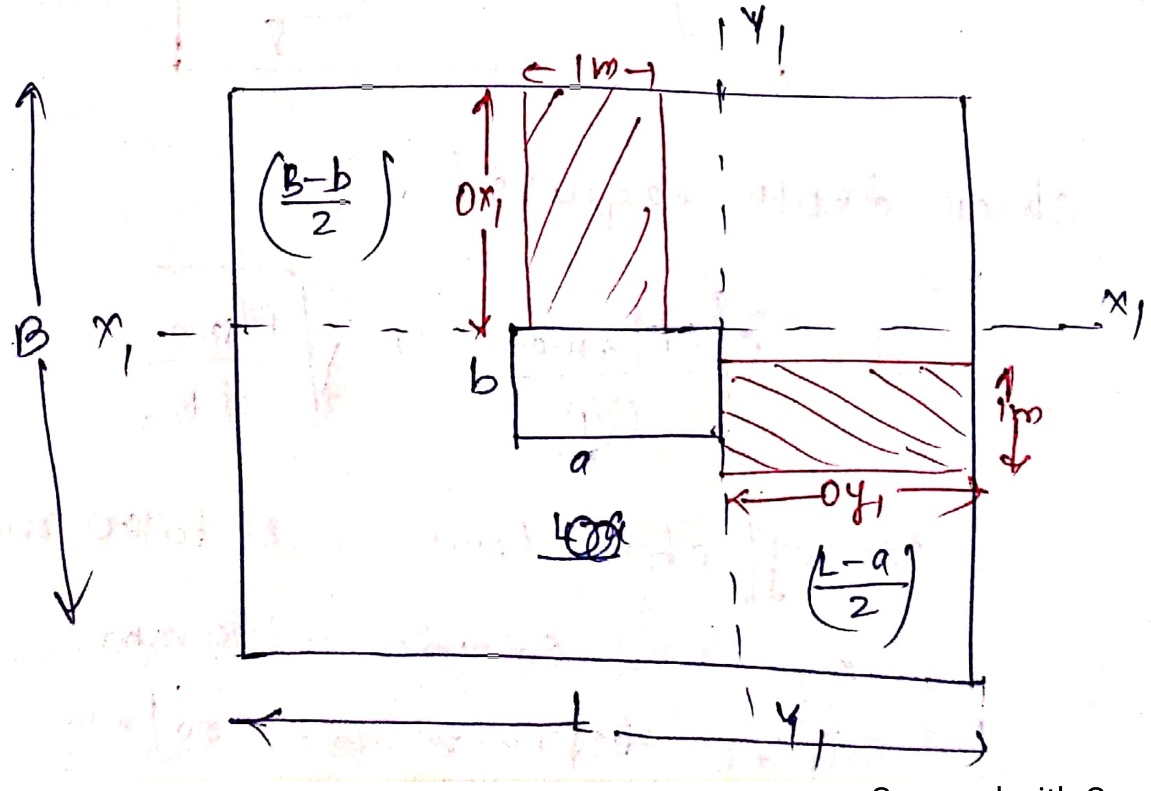
factored design soil pressure

$$w_{uo} = 1.5 w_o = 1.5\left(\frac{P}{A}\right)$$

Step (2). Depth of Footing as per B.M

(check for Bending Moment).

Critical section at the face of column.



(1) At section $X_1 - X_1$

Overhang $O_{x_1} = \left(\frac{B-b}{2}\right)$

$$BM_x = w_o \times 1m \times O_{x_1} \times \frac{O_{x_1}}{2}$$

$$= w_o \times \left(\frac{B-b}{2}\right) \times \frac{1}{2}\left(\frac{B-b}{2}\right)$$

$$\therefore \boxed{M_x = w_o \times \frac{(B-b)^2}{8}} \text{ — WSM}$$

$$\boxed{M_{ux} = w_{uo} \times \frac{(B-b)^2}{8}} \text{ — LSM}$$

(2) At section Y – Y overhang: $O_{y_1} = \left(\frac{L-a}{2}\right)$

Similarly:

$$\boxed{M_y = w_o \times \frac{(L-a)^2}{8}} \text{ — WSM}$$

$$\boxed{M_{uy} = w_{uo} \times \frac{(L-a)^2}{8}} \text{ — LSM}$$

Check depth required.

$$d = \sqrt{\frac{M_{u,max}}{QB}} \text{ or } \sqrt{\frac{M_{max}}{QB}}$$

$Q = 0.148 f_{ck} \rightarrow Fe250$
$= 0.138 f_{ck} \rightarrow Fe415$
$= 0.133 f_{ck} \rightarrow Fe500$
LSM

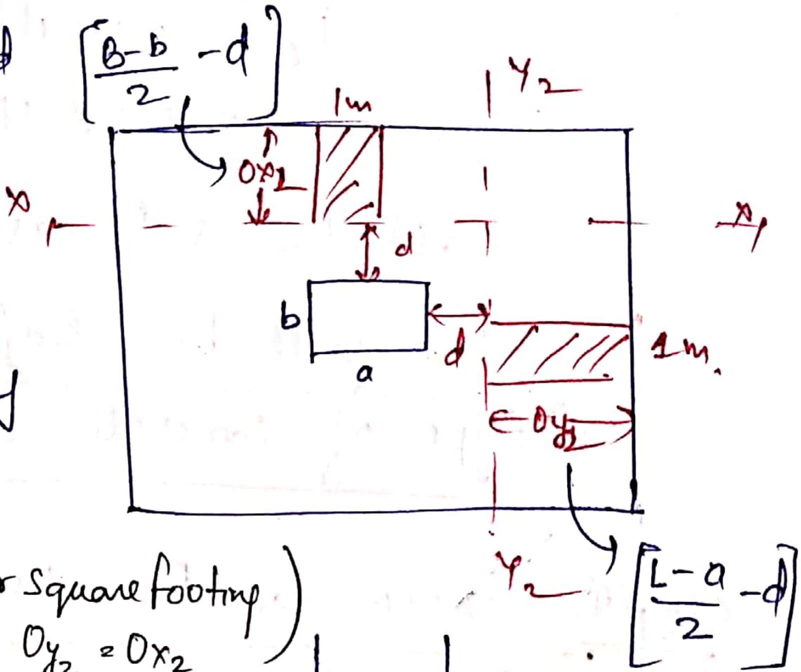
Use effective cover = 80 to 100 mm

Consider = 100 mm.

Step 3: Check for one-way shear:

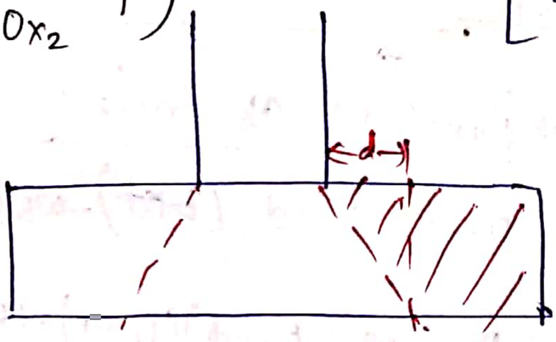
Critical section at distance of d from column.

- One way shear is checked at d distance from face of column
- Most critical section will be where overhang is max.



i.e. if $Oy_2 > Ox_2$ (for square footing $Oy_2 = Ox_2$)

So At $y_2'-y_2$.



shear force (for 1m width)

$$V_y = w_0 \times 1 \times Oy_2$$

$$= w_0\left[\frac{L-a}{2}-d\right] \quad - \text{WSM}.$$

$$V_{uy} = w_{u0}\left[\frac{L-a}{2}-d\right] \quad - \text{LSM}.$$

Check Nominal shear stress.

$$\tau_v = \left[\frac{V_u}{B_1 d} \text{ or } \frac{V_{uy}}{B_1 d}\right] \not> \tau_c.k.$$

$[\text{here } B_1 = 1000\text{ mm}]$

Note: A foundation must be safe in shear. No shear reinforcement is provided.

for design purpose :

Consider $\tau_c = \tau_{c,min}$

$\tau_c = 0.18\ N/mm^2$ — WSM

$= 0.28\ N/mm^2$ — LSM.

if $\tau_v < \tau_c$ as above :

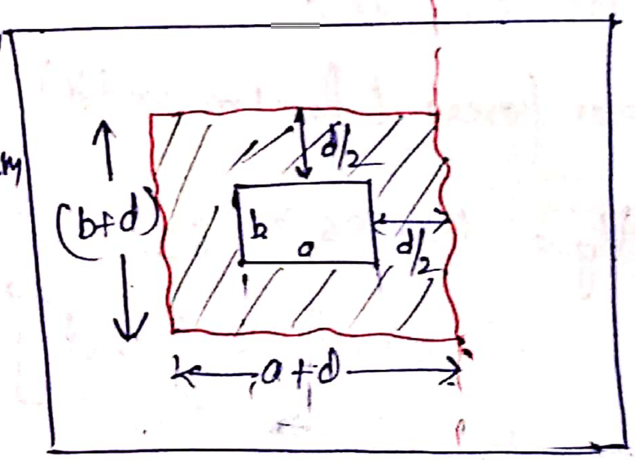
Section will be safe in shear for all type of concrete with any % of steel.

Step (4) check for Punching shear: (Two-Way shear)

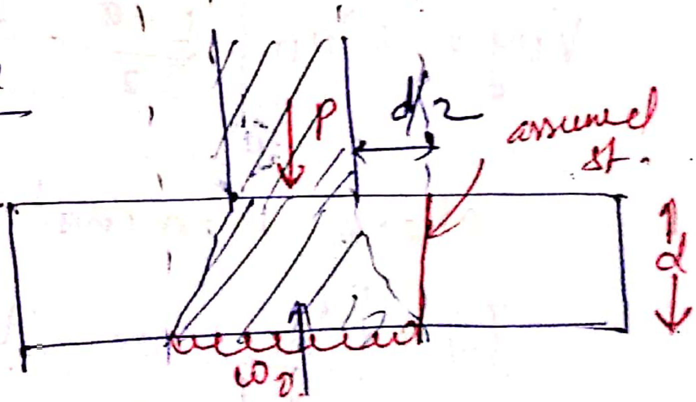
Net Punching force :

$= P - w_0(a+d)(b+d)$ — WSM

$= P_u - w_{u0}(a+d)(b+d)$ — LSM



Punching shear stress developed.



$$\tau_{v(dev)} = \frac{\text{Net Punching force}}{\text{Resisting area}}$$

$$= \frac{P - w_0(a+d)(b+d)}{2[(a+d)+(b+d)] \times d} \quad \text{WSM}$$

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$$= \frac{P_u - w_{uo}(a+d)(b+d)}{2[(a+d)+(b+d)] \times d} \quad \text{— LSM}.$$

and τ_{vp} (dev) $\ngtr$ τ_{vp} (permissible).

① For WSM:

τ_{vp} (per) $= K_\beta \times 0.16\sqrt{f_{cu}}$.

$$\boxed{K_B = \left(0.5 + \frac{b}{a}\right) \ngtr 1.0}$$

② For LSM

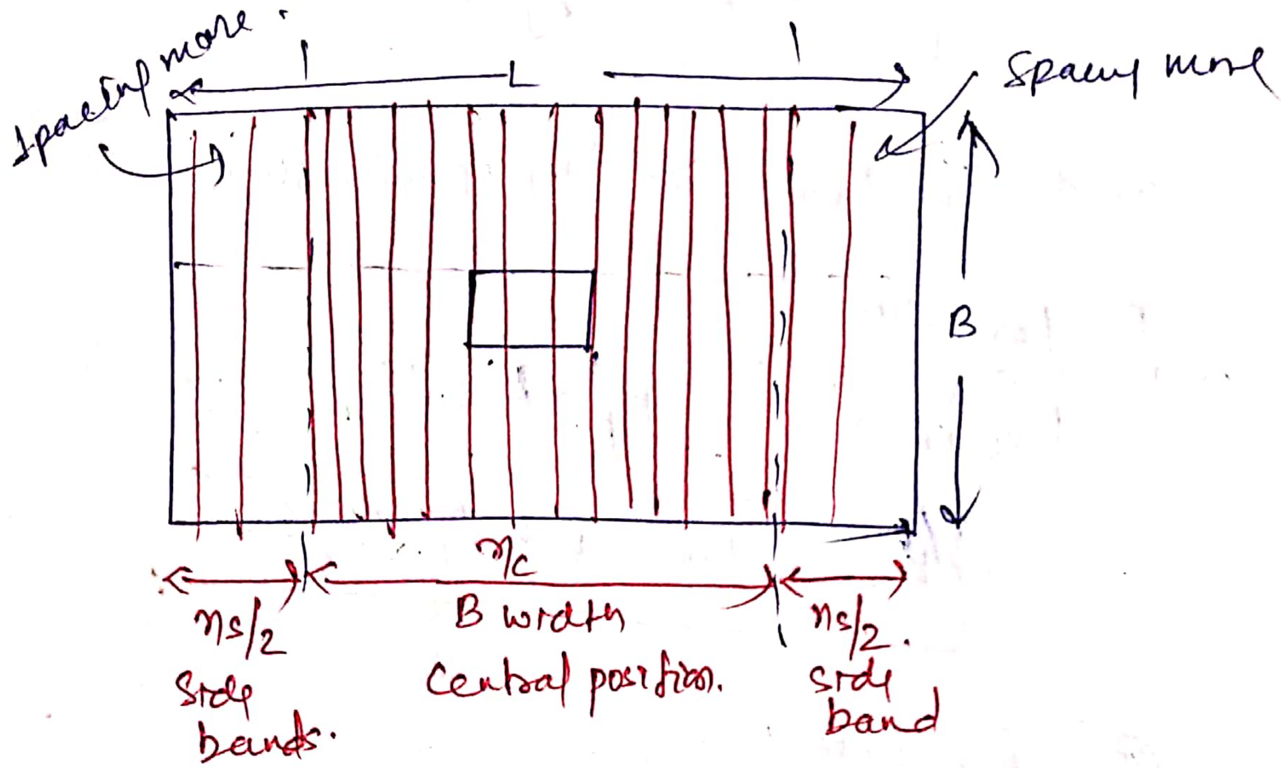
τ_{vp} (per) $= K_\beta \times 0.25\sqrt{f_{cu}}$

Final depth is considered such that section is safe in ① BM ② Single shear ③ Punching shear } Max^m d req is used.

Step ⑤ Design of Area of steel:

Design of Area of steel.

① for M_x or M_{ux}.



Note: As per your syllabus only square footing question will come in exam. So it becomes simple. ie properties from both x-axis & y-axis will be same. So need to check from either one axis only.

Area of steel req. for M_x/M_{ux}.

{for 1 m width of footing}.

$$A_{st} = \frac{M_x}{\sigma_{st}\, j d} \quad \text{— WSM}$$

$$= \frac{M_{ux}}{0.87 f_y\, j d} \quad \text{— LSM}.$$

$$= \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 M_{ux}}{f_{ck} B d^2}}\right] Bd.$$

Area of steel required for total 'L' width

$= A_{st} \times$ 'L'.

→ Total number of bars required;

$$n_T = \frac{A_{st} \times L}{\frac{\pi}{4}(\phi)^2} \quad (\text{Assume } \phi)$$

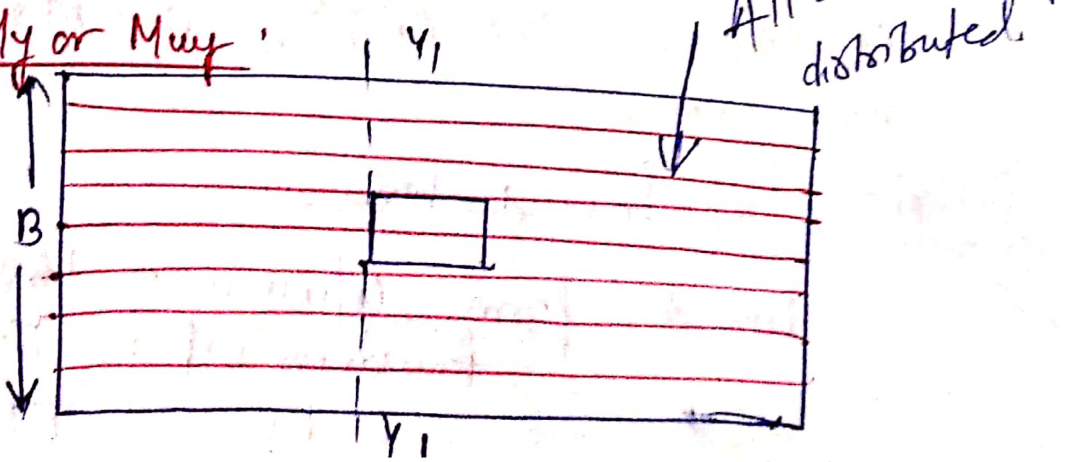
→ Number of bars to be provided in central Band of 'B' width.

$$n_c = n_T \times \frac{2}{(1 + L/B)}$$

→ Remaining reinforcement is distributed on two side bands.

$$\therefore n_s = \frac{(n_T - n_c)}{2}.$$

② For My or Muy:



→ Area of steel (for M_y or M_{uy})
(For 1m width)

$$A_{sty} = \frac{M_y}{\sigma_{st}\, j d} \quad \text{— WSM}$$

$$= \frac{M_{uy}}{0.87 f_y\, j' d} \quad \text{or 0.5 formula — LSM.}$$

→ Area of steel required for full width 'B'

$$= A_{st} \times 'B'.$$

→ No. of bars $n = \dfrac{A_{st} \times B}{\frac{\pi}{4}(\phi)^2}$

Distribute this total number in full width 'B' uniformly.

Qu ① Design a Square footing for a column of size 500 mm × 500 mm for a load of 1600 kN. If SBC = 120 kN/m², Use M 25 & Fe 415 - Use LSM.

Sol Step I: Size of foundation.

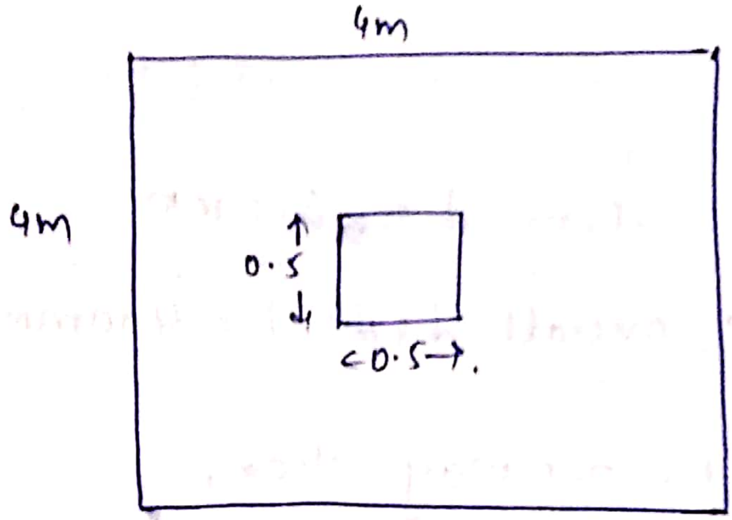
load from column = 1600 kN
foundation wt = 320 kN

∴ P_T = 1920 kN

Area of footing required $= \dfrac{P_T}{q_0} = \dfrac{1920}{120} = 16\,m^2$.

$\therefore$ size of footing $= \sqrt{16} = 4\,m$.

So provide $4m \times 4m$ footing.

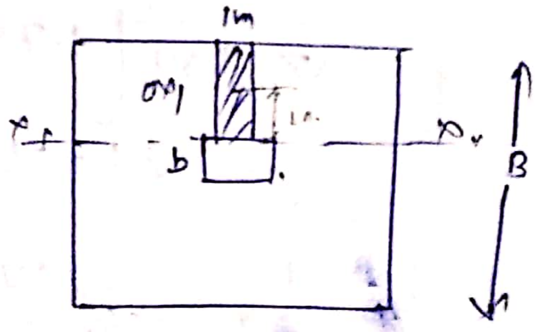


Net soil pressure for LSM: $W_{uo} = 1.5\,\dfrac{P}{A} = \dfrac{1.5 \times 1600}{16}$

$= 150\ kN/m^2$.

Step (2) Check for Bending Moment:

Maxm B.M



$$M_{ux} = W_{uo} \times \left(\frac{B-b}{2}\right) \times 1 \times \left(\frac{B-b}{4}\right)$$

$$= \frac{W_{uo}(B-b)^2}{8}$$

$$= \frac{150\,(4-0.5)^2}{8}$$

$$= 229.70\ kN\text{-}m.$$

check depth required :

$$d = \sqrt{\frac{M_{ux}}{Q\ B_1}} = \sqrt{\frac{229.70 \times 10^6}{0.138 \times 25 \times 1000}}$$

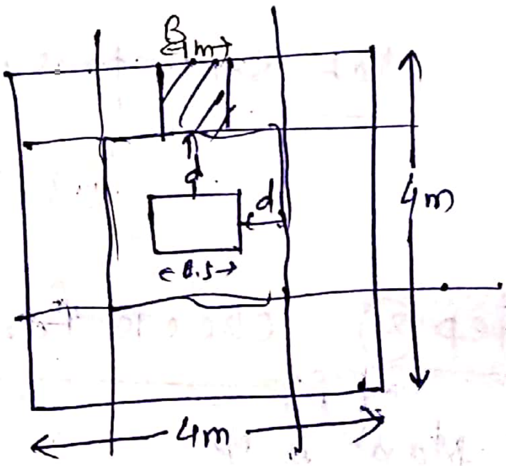
$$= 258.03 m.$$

$\therefore$ Take $d = 300\,mm$

$\therefore$ overall depth $D = 400\,mm$

<u>steps</u> : <u>check for one way shear</u> :

Maxm S.F at critical section



$$V_{ux} = W_{uo}\left[\frac{B-b}{2} - d\right] \times 1$$

$$= 150\left[\frac{4-0.5}{2} - 0.3\right]$$

$$= 150\,(1.75 - 0.30)$$

$$= 217.50\ KN$$

$\tau_v = \frac{V}{Bd}$

$$\text{So } \tau_v = \frac{217.50 \times 10^3}{1000 \times 300} = 0.725\ N/mm^2$$

$> 0.28\ N/mm^2$ (L2m)

so it is failed

$$\text{So } d_{required} = \frac{V_u}{\tau_c \cdot B_1}$$

$$= \frac{217.5 \times 10^3}{0.28 \times 1000} = 777\,mm$$

So $d_{required} = \underbrace{\frac{777 + 300}{2}}_{avg.} = 558.$

$\therefore$ Choose $d = 600$ mm

$\therefore V_u = 150 \times (1.75 - 0.6)$

$= 172.5$ kN.

$\therefore \tau_v = \frac{172.5 \times 10^3}{1000 \times 600} = 0.2875$

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

(OK).

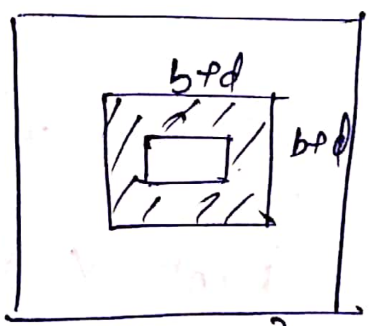
$\therefore d_{required} = 600$ mm

$\therefore D_{overall} = 700$ mm.

step 4: Check for Punching shear stress:

or 2-way shear.

Punching shear stress developed.



$\tau_{vp(dev)} = \frac{P_u - w_{u0}(b+d)^2}{4 \times (b+d) \times d}$

$= \frac{1.5 \times 1600 \times 10^3 - 150(0.5 + 0.6)^2 \times 10^3}{4 \times [500 + 600] \times 600}$

$= 0.84$ N/mm^2.

$\tau_{vp(perm)} = K_s \cdot 0.25\sqrt{f_{ck}}$

$= \left(0.5 + \frac{0.5}{0.5}\right) = 1.50 \not> 1.0.$

$= 1 \times 0.25 \times \sqrt{25}$

$= 1.25\ N/mm^2.$

$\therefore$ [vp (der)] < [vp (perm)]

So safe.

Steps! **Area of steel:**

$M_{ux} = 229.70$ KN-m.

(for 1 m width).

Area of steel: [9t's v.R.S. as do provided on large]

$$A_{st} = \frac{0.5 \times 25}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 229.7 \times 10^6}{25 \times 1000 \times 600^2}}\right]$$

$= 1093\ mm^2$. (for 1m) $\times 1000$ $\times 600$

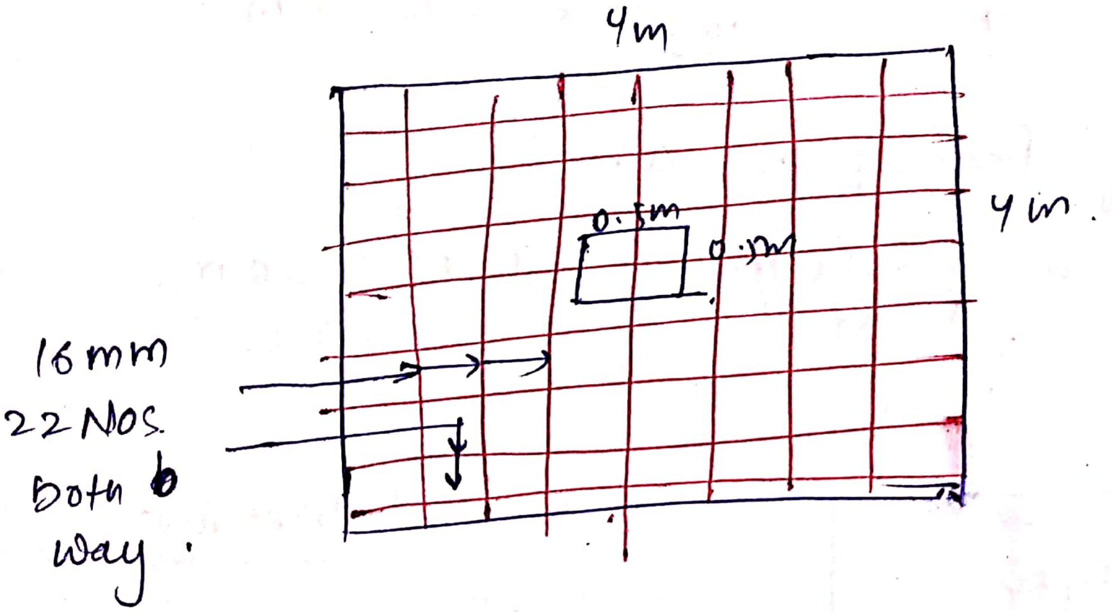
Min % of steel $= \frac{0.12}{100} \times 1000 \times 600 = 720\ mm^2$
(B.) (D) < A_{st} provided OK

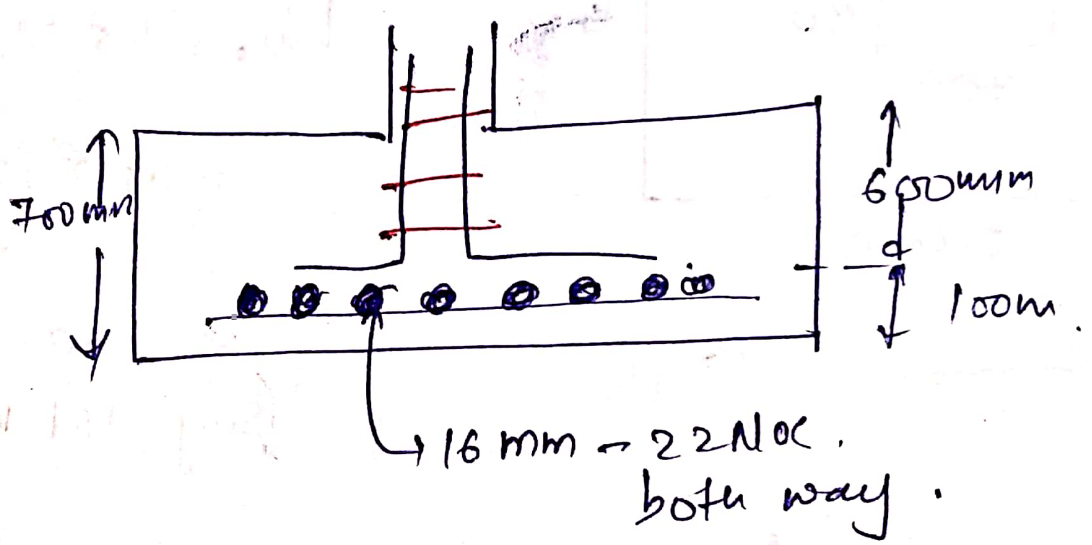
Total area of steel req $= 4 \times 1093$

$= 4372\ mm^2$

No. of 16mm ϕ bars $= \frac{4372}{\frac{\pi}{4}(16)^2} = 21.7$

Say 22 Nos.


4m
4m.
0.5m
0.5m
16mm
22 NOS.
Both
way.


700mm
600mm
d
100m.
16 mm → 22 NOS.
both way.