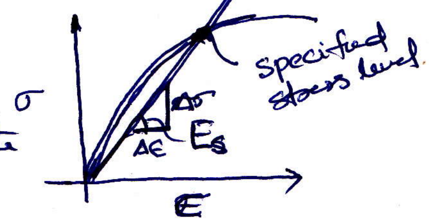
Key for Design of Concrete Structures – I

Class: B.Tech. V Sem. Civil Engg.

i. Define secant modulus? What is the relationship between compressive strength and modulus of elasticity of concrete as per IS:456-2000

slope of the line joining from the origin to the specified stress level ($\approx 1/3 f_{ck}$) on the ~~st~~ uniaxial axial stress strain curve of a concrete

$E = 5000\sqrt{f_{ck}}$



ii. What is a modulus of rupture of concrete? In the absence of test data, how is it measured?

It is the Maximum flexural stress/strength of concrete at the extreme fibre ~~of the stem~~ a ~~section~~ under a three point bending test on a simply supported rectangular beam (std) at failure.

$f_{cr} = 0.7\sqrt{f_{ck}}$.

iii. An isolated T-beam of flange width 2000 mm and rib width of 250 mm is loaded with 20 kN/m load inclusive of self weight. The span of the beam is 8 m and is simply supported. Thickness of slab is 100 mm. Calculate the effective width of flange.

$$b_f = \frac{l_o}{\frac{l_o}{b} + 4} + b_w \le b \Rightarrow \underline{1250\,mm}.$$

iv. Define one way slab and two way slab.

one-way slab: It is the slab in which the flexure/bending predominates only in one direction. ex:- $\square$ bar slab supported on two opposite edges.

Two-way slab:- It is the slab in which the bending dominates in both the directions. in $l_y/l_x < 2$

v. What is an unsupported length and effective length of a column?

unsupported length(l):- clear distance b/w the floor and the shallow beam framing into the columns in each direction at the next higher floor level.

Effective length:- Distance b/w the points of inflection of the comp member in the buckled configuration in a plane.

vi. Under what circumstances the doubly reinforced sections are preferred?

when the dimensions are unaltered due to some architectural reasons and when the Actual BM due to loads exceeds the limiting moment of ~~resistance~~ of ~~a~~ singly reinforced section, then the DRS are preferred.

vii. A rectangular beam of size 300 mm x 450 mm effective is reinforced with 4 no. 16 mm diameter bars in tension. If the grade of steel is (a) Mild steel, (b) Fe415 and (c) Fe500, what is the corresponding limit depth of neutral axes?

(a) $0.53d = 238.5$ mm

(b) $0.48d = 216$ mm

(c) $0.46d = 207$ mm.

viii. A beam of rectangular section having a width of 300 mm and effective depth of 600 mm is subjected to ultimate shear at support is 100 kN. Assuming M-20 grade concrete and Fe-415 HYSD bars. The beam is reinforced with four bars of 25 mm diameter at centre, out of which two bars of 25 mm diameter are bent up at 45^0 near the supports. The shear strength of concrete is 0.25 N/mm^2. Estimate the capacity of bent up bars that is to be considered in the design of shear reinforcement.

$V_u = 100\,kN$ $\quad \tau_c = 0.25\,N/mm^2$ $\quad P_t = \frac{100\,A_{st}}{bd} = 0.545$

$V_{uc} = 45\,kN$ $\quad V_{us} = 55\,kN.$

$V_{ub} = 0.87 f_y A_{sv} \sin\alpha = 250.7\,kN \not> \frac{V_{us}}{2} \Rightarrow \boxed{V_{ub} = 27.5\,kN}$

ix. A short column 600 x 600 mm in section is subjected to a factored load axial load of 1500 kN. Determine the minimum area of longitudinal steel to be provided, assuming M20 grade concrete and Fe415 grade steel.

$P_u = 1500\,kN$ $\quad P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$= 0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}$

$A_{sc} = -ve$ $\quad \therefore A_{sc\,min} = 0.8\% A_c = \frac{0.8}{100} \times \frac{P_u}{0.4 f_{ck}}$

$= 1500\,mm^2$

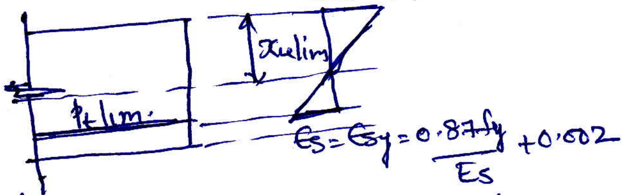
x. What is the difference between primary torsion and secondary torsion and give one example for each.

Primary torsion :- This mainly due to the eccentricity of loading. and is independent of torsional stiffness. Ex:- Cantilever porch

Secondary torsion :- This is mainly due to the compatibility of deformations at the joints. Ex:- when a secondary beam is resting on main beam

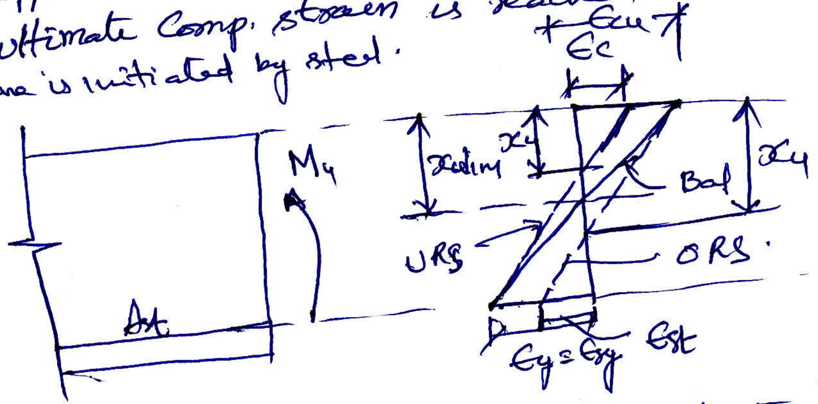
Section - B.

2 (a) **Balanced section:-** ~~at~~ A Balanced section is one in which the area of tension steel is such that at the ultimate limit state, the two limiting conditions are reached simultaneously viz: the Max Comp. strain reaches its ultimate strain ϵ_{cu} and the tensile strain in steel reached its yield strain ϵ_y. It is expected to occur by the simultaneous initiation of crushing of concrete and yielding of steel.



under reinforced section :-

In which the Area of tension steel is such that as the ultimate limit state is approached, the yield strain ϵ_y is reached in the steel before the ultimate Comp. strain is reached in the extreme fibre of the Concrete. Failure is initiated by steel.



$x_u < x_{ulim}$

$p_t < p_{t\,lim}$.

$\epsilon_s = \epsilon_{sy}$

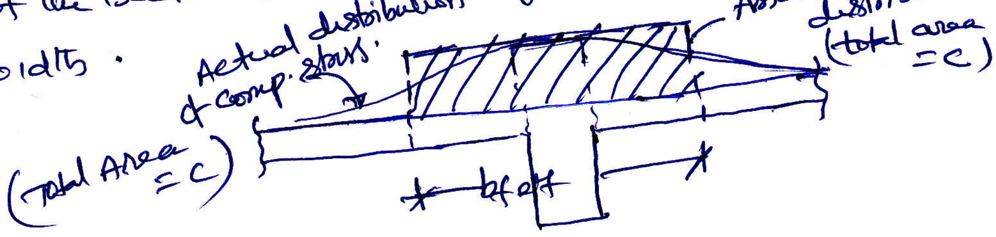
$\epsilon_c < \epsilon_{cu}$.

over reinforced section :- In which the area of tension steel is such that at the ultimate limit state, the ultimate Comp strain in Concrete is reached, however the tensile strain in the reinforcing steel is less than the yield strain ϵ_y. Failure is due to crushing of Concrete & it is sudden.

$x_u > x_{ulim}$; $p_t > p_{t\,lim}$.

$\epsilon_{cu} = \epsilon_{cu}$ $\quad \epsilon_s < \epsilon_{sy}$

(b) when the flange of T-beam is wide, the flexural Comp. stress is not uniform over its width. The stress varies from a maximum in the web region to progressively lower values of points farther away from the web. In order to operate within the framework of the theory of flexure, which assumes a uniform stress distribution across the width of the section, it is necessary to define a reduced effective flange width.

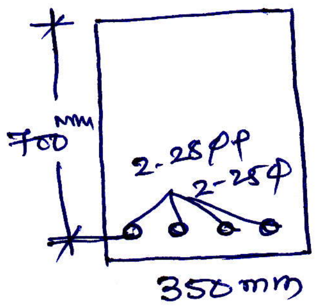


3 (a) When the dimensions of the beam is fixed, and when the actual UBM due to loads exceeds the limiting Moment of resistance of a SRS, the strength of the section can be increased by providing reinforcement in Comp Zone in addition to the reinforcement in Tension Zone. If the section is provided with both tension and comp. reinforcements then the section is called "DRS".

Advantages

* Dead weight of the member can be reduced.
* Good appearence ~~&~~ aesthetically.
* Resists reversal of stresses in case of ~~earthquake~~ dynamic loadings.
* Comp stt acts as anchor bars for holding shear reinforcement.
* Shrinkage & creep strains can be minimized.

(b)



$f_{ck} = 20\,N/mm^2$ $f_y = 415\,N/mm^2$. $A_{st} = 2213.5\,mm^2$

$C_u = T_u$. (Assume URS)

$x_u = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck}\, b} \Rightarrow x_u = 317.13\ mm.$

$x_{ulim} = 0.48d = 336\ mm.$

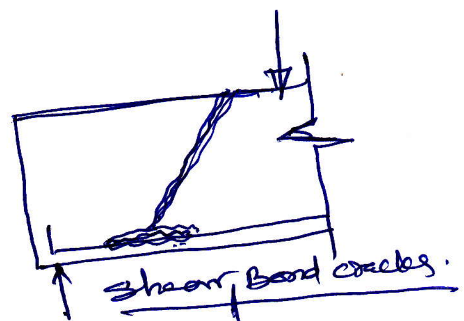
$x_u < x_{ulim} \Rightarrow$ URS $\Rightarrow$ Assumption is correct.

$\therefore M_{uR} = 0.87 f_y A_{st}(d - 0.42x_u) = \underline{452.98\ kN\text{-}m}.$

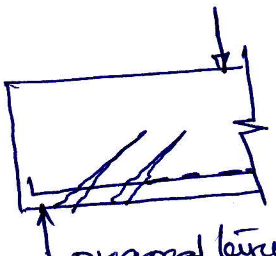
UNIT-II

4 (a) The major types of shear failure modes encountered in reinforced concrete beams are

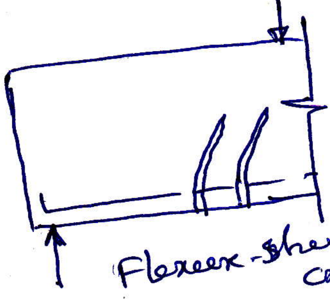
i) Shear-Tension (or) Diagonal tension
ii) Flexure-shear
iii) Shear Compression
iv) Shear Bond.



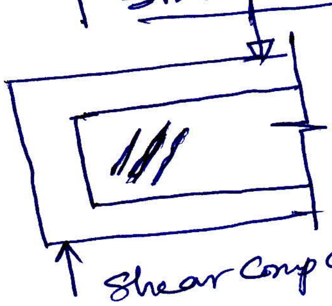
Shear Bond cracks.



Diagonal tension cracks.
$(2.5 < (a/d) < 6)$



Flexure-shear cracks
$(1 < (a/d) < 2.5)$



Shear Comp cracks

(6) Given data

$b = 300\ mm$ $\quad f_{ck} = 20\ N/mm^2$

$d = 600\ mm$ $\quad f_y = 415\ N/mm^2$

$A_{st} = 3 \times 314 = 942\ mm^2$ $\quad A_{sv} = 2 \times 50 = 100\ mm^2.$

$S_v = 200\ mm.$

$$p_t = \frac{100 A_{st}}{bd} = 0.52\%.$$

For $p_t = 0.52\%$ & $f_{ck} = 20\ MPa$, from IS:456 (Table 19)

$\tau_c = 0.48\ N/mm^2.$

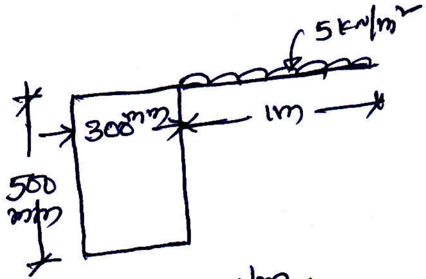
$\therefore\ V_{uc} = \tau_c bd = 86.4\ kN.$

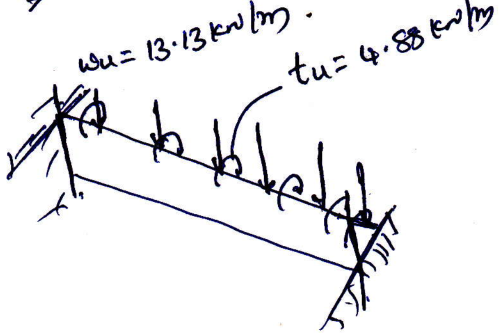
$$V_{us} = \frac{A_{sv} \times 0.87 f_y \times d}{S_v} = 108.3\ kN$$

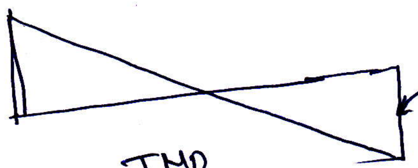
$\therefore$ Total shear resistance of support section

$V_{uR} = V_{uc} + V_{us} = \underline{\underline{194.7}}\ kN.$

(5)

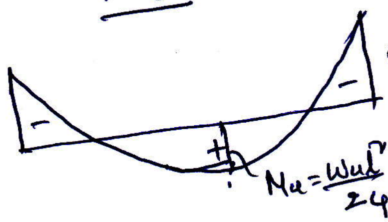






$T_u = \frac{t_u l}{2} = 12.20\ kN\text{-}m$

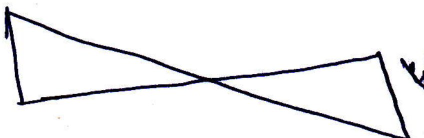
TMD



$M_u = \frac{W_u l^2}{12} = 27.35\ kN\text{-}m$

$M_u = \frac{W_u l^2}{24}.$

BMD



$V_u = \frac{W_u l}{2} = 32.82\ kN.$

SFD

Loads on Beam

From projection : $5 \times 1 = 5\ kN/m.$

From self wt : $25 \times 0.3 \times 0.5 = 3.75\ kN/m$

$\underline{8.75\ kN/m.}$

Load factor $= 1.5.$

$\therefore\ W_u = 8.75 \times 1.5 = 13.13\ kN/m.$

Eccentricity of cantilever load from ℄ of beam $= \frac{1}{2} + \frac{0.3}{2} = 0.65\ m$

$\therefore\ t_u = (5 \times 0.65) \times 1.5 = 4.88\ kN/m.$

Check

Need for torsional rft

$$\tau_{ve} = \frac{V_u + 1.6\,T_u/b}{bd} \quad \text{where } d = 500-30-8 = 462$$

$= 0.706\ N/mm^2 < \tau_{cmax} = 2.8\,MPa$ for M20.

∴ section is adequate.

Shear strength of Concrete

$A_{st} = 402\,mm^2 \Rightarrow p_t = 0.289\,\%$ & M20.

∴ $\tau_c = 0.386\,MPa$ (from Table 19 of IS 456)

∴ As $\tau_{ve} > \tau_c \Rightarrow$ torsional rft is required.

Adequacy of longitudinal reinforcement

$$M_t = T_u\left(\frac{1+D/b}{1.7}\right) = 19.14\ kN\text{-}m$$

$M_{e1} = M_u + M_t = 46.49\ kN\text{-}m$.

$M_{e2} = M_t - M_u < 0 \Rightarrow$ not to be considered.

$M_{uR} = 0.87 f_y A_{st}(d - 0.42 x_u)$ (or) use IS codal expression

$= 61.5\ kN\text{-}m > M_{e1}$ — safe.

Adequacy of side face rft

Area of side face rft $= 0.1\%\,bD = 150\,mm^2$ distributed equally on both the faces.

Area provided is $2-10\phi = 157\,mm^2 > 150\,mm^2$ — safe

Adequacy of transverse rft

$A_{sv} = 157\,mm^2$. (2 legged 10ϕ stirrups).

$s_v = 150\,mm$. $b_1 = 300 - 2\times30 - 16 = 224\,mm$.

$d_1 = 500 - 2\times30 - 16 = 424\,mm$.

$$(A_{sv})_{req} = \left(\frac{T_u}{b_1} + \frac{V_u}{2.5}\right)\left(s_v / d_1(0.87 f_y)\right).$$

$= 66.23\,mm^2 < 157\,mm^2$ provided — ok

Min limit of area of transverse rft

$$(A_{sv})_{req} = \frac{(\tau_{ve} - \tau_c)\,b s_v}{0.87 f_y} = 41.9\,mm^2 < 157\,mm^2 \text{ — o.k.}$$

Further spacing of stirrups provided should satisfy the following.

$$(S_v)_{req} \leq \begin{cases} x_1 = 224+16+10 = 250 \text{ mm} \\ \dfrac{x_1+y_1}{4} = \dfrac{250+450}{4} = 165 \text{ mm} \\ 0.75d = 346.5 \text{ mm} \end{cases}$$

$S_{v\,prov} = 150 \text{ mm} < S_{v\,req} \Rightarrow$ ok.

Hence section provided is adequate in all respects

UNIT-III.

(6) Doglegged Stair Case

Height b/w floors = 4.0 m.

Stair hall : 2.5 x 5 m.

LL = 4 kN/m²

$f_{ck} = 25$ N/mm²

$f_y = 415$ N/mm².

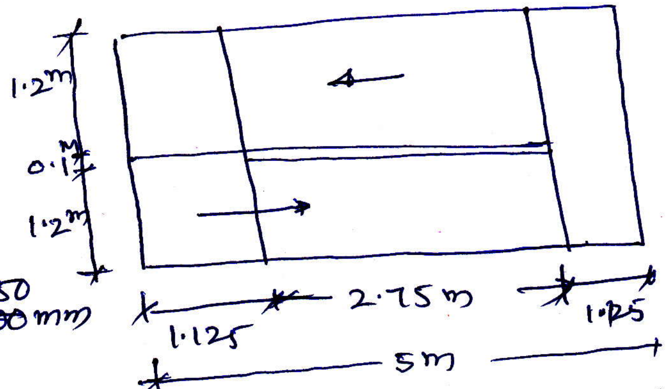
Ht of each flight = 2 m.

Assume R = ~~160.0~~ mm.

No. of Risers ≙ 12 Nos

No of Treads = 11

Assume width of Tread = ~~300~~ 250 mm

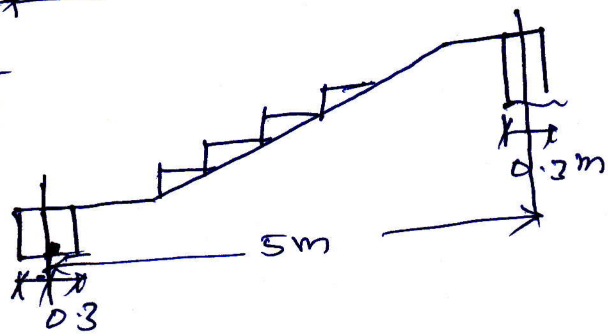


Length of going = 11 x 0.25 = 2.75 m.

∴ width of landing at end = $\dfrac{5-2.75}{2}$

= 1.125 m.

Assume that stair is supported at the ends. width of support = 300 mm



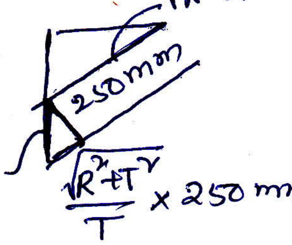
~~Loads on~~

Thickness of waist slab

Assume $D = \dfrac{L}{20} = \dfrac{5.0}{20} = 250$ mm

Assume clear cover = 20 mm (Mild exposure) & 12 mm ϕ bars.

∴ d = 250 − 20 − 6 = 224 mm.



Loads on going (on projected plan area)

Self wt of waist slab $= 0.25 \times \dfrac{302.3}{250} \times 25$

$= 7.56$ kN/m².

$\dfrac{\sqrt{R^2+T^2}}{T} \times 250$ mm

=

Self wt of step $= \frac{1}{2} \times 0.17 \times 25 = 2.125\ kN/m^2$

b

$0.36 f_{ck}\, x_{umax} (d - 0.42 x_{umax})$

$= 0.36 f_{ck} \frac{x_{umax}}{d} (1 - 0.42 \frac{x_{umax}}{d})$

Assume FF $= 0.5\ kN/m^2$

LL $= 4.0\ kN/m^2$

$14.285\ kN/m^2$

Factored load $= 21.42\ kN/m^2$

Loads on Landing

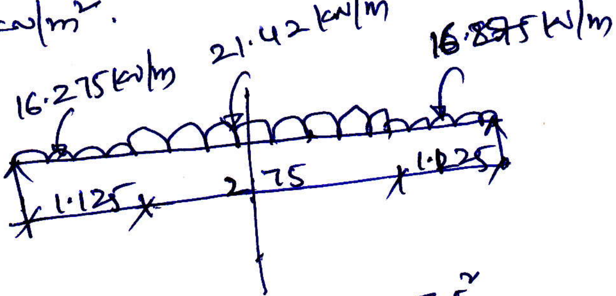
(i) Self wt of slab $= 0.25 \times 25 = 6.25\ kN/m^2$

FF $=$ 0.6 "

LL $=$ 4.0 "

$10.85\ kN/m^2$

Factored load $= 16.275\ kN/m^2$.



Design moment

$R = 16.275 \times 1.125 + 21.42 \times \frac{2.75}{2}$

$= 18.3 + 29.45$

$= 47.75\ kN$

$M_u = 47.75 \times 2.5 - 16.275 \times 1.125 \left(\frac{1.125}{2} + \frac{2.75}{2}\right) - 21.42 \times \frac{2.75^2}{8}$

$= 119.37 - 35.47 - \cancel{30.99}\ 20.24 = \cancel{2.91}\ 63.67\ kN\text{-}m.$

Main Reinforcement $\quad d_{req} = \sqrt{\frac{M_u}{R_b}} = \sqrt{\frac{63.67 \times 10^6}{3.44 \times 1000}} = 136.04 < d_{prov}$

$\rightarrow$ URS.

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

$= 841.92\ mm^2$

Spacing of 12 mm ϕ bars $= 130$ mm.

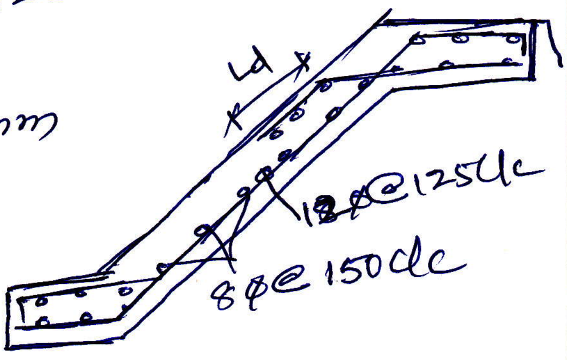
$\boxed{\text{provide } 12\phi\ @\ 125 \text{ mm c/c}}$

Distribution steel $\quad A_{st\,req} = \frac{0.12}{100} \times 1000 \times 250 = 300\ mm^2$

Assume 8 mm ϕ bar.

Spacing $= 166.67$ mm

$\boxed{\text{provide } 8\phi\ @\ 150 \text{ mm c/c}}$



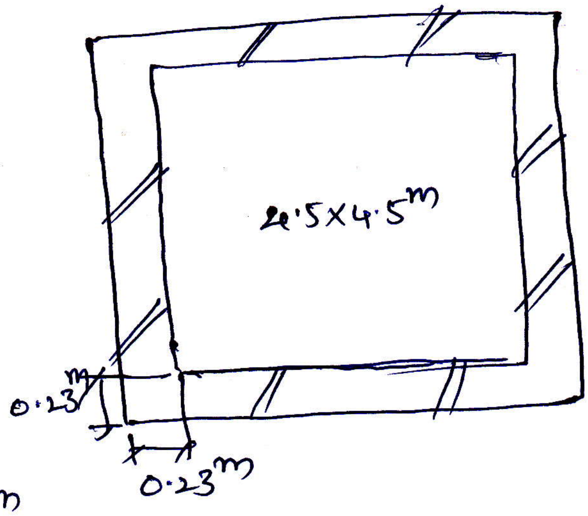
$\boxed{\text{Other checks are to be carried out}}$

(7) **Two-way slab**

$LL = 3\ kN/m^2$

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

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



Thickness of slab

$$d_{req} = \frac{l = 4.5m}{20 \times 1.5} = 150\ mm$$

Assume clear cover = 20 mm (Mild exposure)

10 mm ϕ bars.

$D = 150 + 20 + 5 = 175\ mm$.

$d_x = 150\ mm$; $d_y = 140\ mm$.

Eff span (for SSS) ~~width of support = 230 mm < 1/12 × L~~.

$l_y = 4.5 + 0.15$ (or) $4.5 + 0.23$ } less $= 4.65\ m$.

$l_x =$ ~~4.65 m~~ $4.5 + 0.14$ or $4.5 + 0.23$ } less $= 4.64\ m$.

$$\frac{l_y}{l_x} = \frac{4.65}{4.64} \simeq 1.0.$$

$\alpha_x^+ = 0.062 \quad \alpha_y^+ = 0.062$ (From Table 27 of IS 456)

$M_{ux} = M_{uy} = 0.062 \times 4.65^2 \times w_u$.

Loads: Consider 1m width of slab

$LL = 3\ kN/m^2$

Self wt $= 0.175 \times 25 = 4.375\ kN/m^2$

FF $= 1.0\ kN/m^2$

$8.375\ kN/m^2 \times 1 = 8.375\ kN/m$.

Factored load $= 8.375 \times 1.5 = 12.56\ kN/m$.

Design moment $M_{ux} = M_{uy} = \underline{16.83\ kN\text{-}m}$.

$$d_{req} = \sqrt{\frac{16.83 \times 10^6}{3.44 \times 1000}} = 69.96\ mm < d_{prov}. \Rightarrow URS.$$

$$A_{stx} = A_{sty} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 M_u}{f_{ck} b d^2}}\right] bd$$

$$= 323.3 \text{ mm}^2$$

Min $A_{st} = 0.12\% \, bD = 210 \text{ mm}^2$.

spacing of 10 mm ϕ bars = 242 mm.

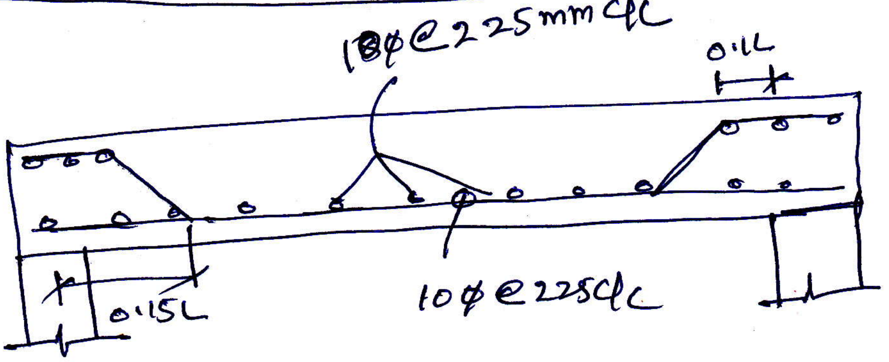
provide $\boxed{10\phi @ 225 \text{ mm c/c}}$ in both ways.

(or)

$\boxed{8\phi @ 150 \text{ mm c/c}}$

At support :- 50% of the mid steel is to be bent up near the supports

$\boxed{\text{other checks are to be carried out}}$



UNIT - IV

(8) Given data : $l = 3400$ mm, $D = 400$ mm, $P_u = 1500$ kN.

(As col is braced) ~~$P_u = 1.5 \times 1500$~~ =

$$\frac{l}{D} = \frac{3400}{400} = 8.5$$

$< 12 \rightarrow$ short column.

Min eccentricity

$$e_{min} = \frac{3400}{500} + \frac{400}{30} = 20.1 \text{ mm} > 20 \text{ mm}.$$

As $0.05D = 20$ mm $\simeq e_{min}$ (20.1 mm), IS. Codal expression for short axially loaded col may be used.

$$\therefore \; P_u = 1.05\left[0.4 f_{ck} A_c + 0.67 f_y A_{sc}\right]$$

$$1500 \times 10^3 = 1.05\left[0.4 \times 25 \times \frac{\pi}{4} \times 400^2 + (0.67 \times 415 - 0.4 \times 25) A_{sc}\right]$$

$$A_{sc} = 642 \text{ mm}^2$$

$P_t = \frac{100\, A_{st}}{A_g} = 0.51\%$

min $A_{se} = 0.8\%\, A_g = 1005\ mm^2 > A_{se\,req}$

$\therefore$ provide $A_{se} = 1005\ mm^2$

ie $\boxed{6 - 16\ mm\phi}$

Spiral reinforcement

Assume a clear cover = 40 mm

Dia of helical rft: $\phi_h \not< \frac{1}{4} \times \phi_L$, $\not< 5\ mm$ } greater

$\boxed{\therefore \phi_h = 6\ mm}$

pitch (p)

$$\frac{\text{Volume of helical rft}}{\text{Volume of Core}} \not< 0.36\left[\frac{A_g}{A_{cr}} - 1\right] \cdot \frac{f_{ck}}{f_y}$$

Dia of core $= D_c = 400 - 2\times40 + 2\times6 = 332\ mm$.

$$\therefore \text{RHS} = \frac{\frac{\pi}{4}\times 6^2 \times \pi(332-6)}{\frac{\pi}{4}\times 332^2 \times p} = \frac{0.334}{p}$$

$$\text{RHS} = 0.36\times\left[\frac{400^2}{332^2} - 1\right]\times\frac{25}{415} = 0.00979.$$

$\therefore 0.334/p \not< 0.00979 \Rightarrow \boxed{p < 34.11\ mm}$

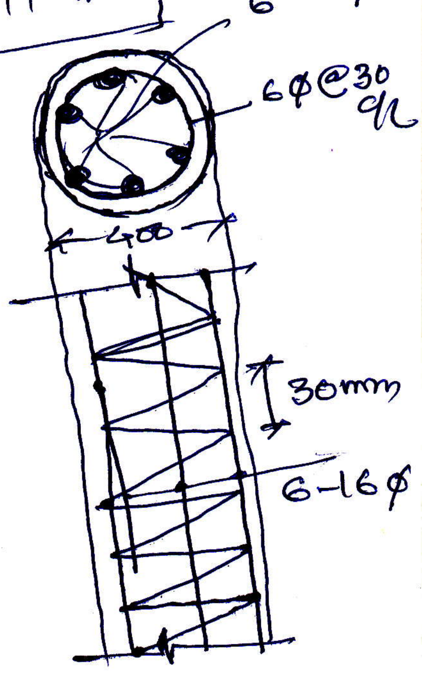
Also $p <$ 75 mm

$< \frac{1}{6} D_c = 55.33\ mm$

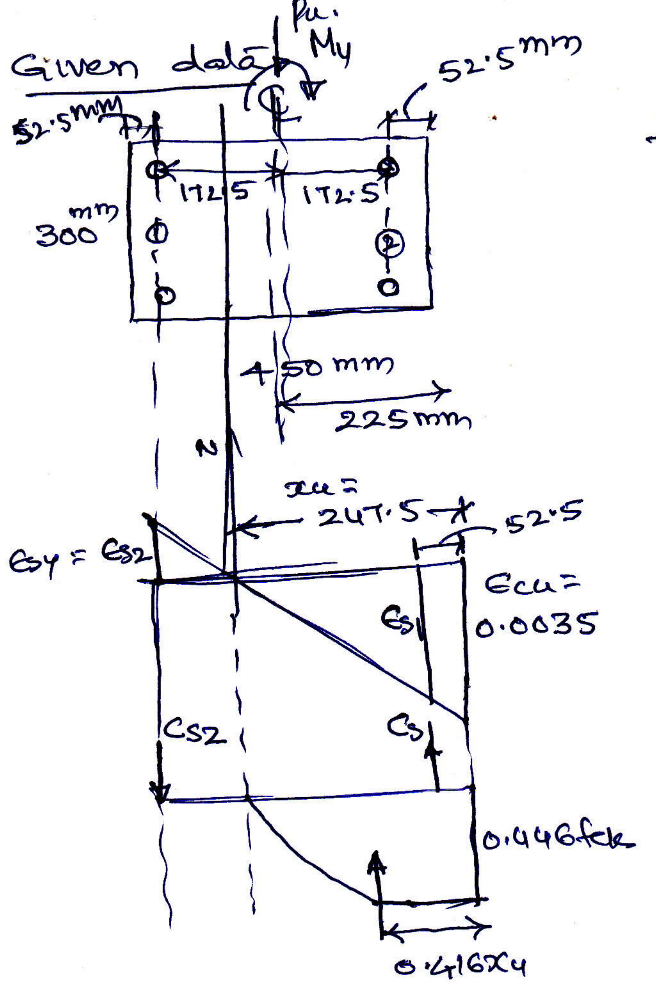
$>$ 25 mm

$> 3\times\phi_h = 18\ mm$

$\therefore$ $\boxed{\text{provide } 6\ mm\ \phi\ @\ 30\ mm\ c/c \text{ as spiral rft}}$



(9) Given data



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

$f_y = 415\ N/mm^2$

Assume clear cover = 40 mm.

$\boxed{x_u = 0.055D = 247.5\,mm < D}$.

$b = 300\ mm;\quad D = 450\ mm.$

$A_{s1} = A_{s2} = 2 \times 491 = 982\ mm^2$

$y_1 = -y_2 = 172.5\ mm.$

strains in steel

$\varepsilon_{s1} = -\varepsilon_{sy} = -0.003805$ (tensile)

$\varepsilon_{s2} = \frac{0.0035}{247.5} \times (247.5 - 52.5)$

$= +0.00275$ (comp)

Design stresses in steel from stress-strain curve of IS 456-2000

$f_{s1} = -0.87 f_y = -360.9\ N/mm^2$

$f_{s2} = 351.8\ N/mm^2$

Design strength components (P_{uR}).

$P_{uR} = P_{uc} + P_{us}.$

$P_{uc} = 0.362 f_{ck}\, b x_u\ \cancel{(d - 0.42)} = 534.6\ kN.$

$P_{us} = \sum_{i=1}^{n} (f_{si} - f_{ci}) A_{si}$

$= -360.9 \times 982 + (351.8 - 0.446 \times 20) \times 982$

$= -354.4 + 336.71 = -17.69\ kN$ (Tensile)

$\therefore \boxed{P_{uR} = +534.6 - 17.69 = 516.91\ kN}$

Design strength components in flexure (M_{uR})

$M_{uR} = M_{uc} + M_{us}$

$M_{uc} = P_{uc} \times \left(\frac{D}{2} - 0.416 x_u\right) = 534.6\left(\frac{450}{2} - 0.416 \times 247.5\right) \times 10^{-3}$

$= \underline{65.16\ kN\text{-}m}.$

$$M_{us} = \sum_{i=1}^{1} P_{usi} \times y_i$$

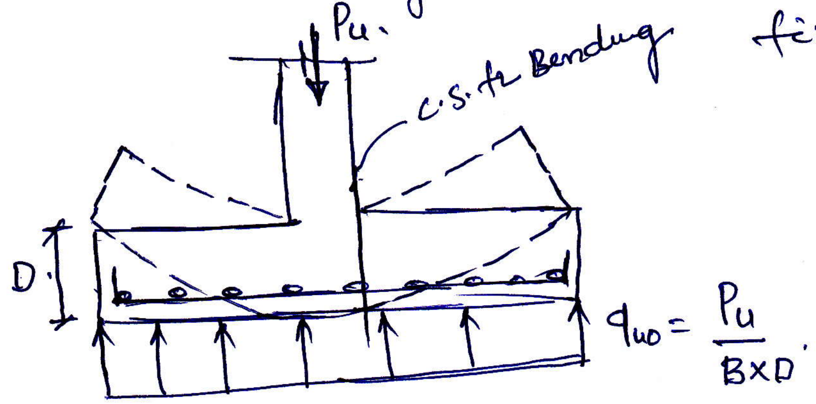
$$= -354.4 \times \frac{(-172.5)}{1000} + 336.71 \times \frac{(172.5)}{1000}$$

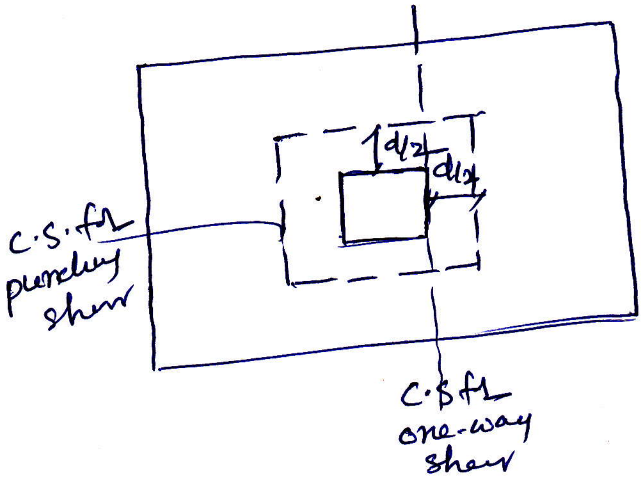
$$= 119.21 \text{ kN-m}.$$

$$\therefore \boxed{M_{uR} = 65.16 + 119.21 = 184.37 \text{ kN-m}}$$

UNIT - V

(10) Isolated Footings → Shallow foundation. → Located on reasonably firm soils.





combined footings. ⇒ when two or more columns are located close to each other and/or they are relatively heavily loaded and/or rest on soil with low SBC, resulting in an overlap of areas if isolated footings are ~~provided~~ attempted. In such case, it is advantageous to provide a single combined footing for the columns. ~~Even in case of property line limit which restricts the extension of footing on one side.~~

Continuous strap footing: columns are aligned in one direction.

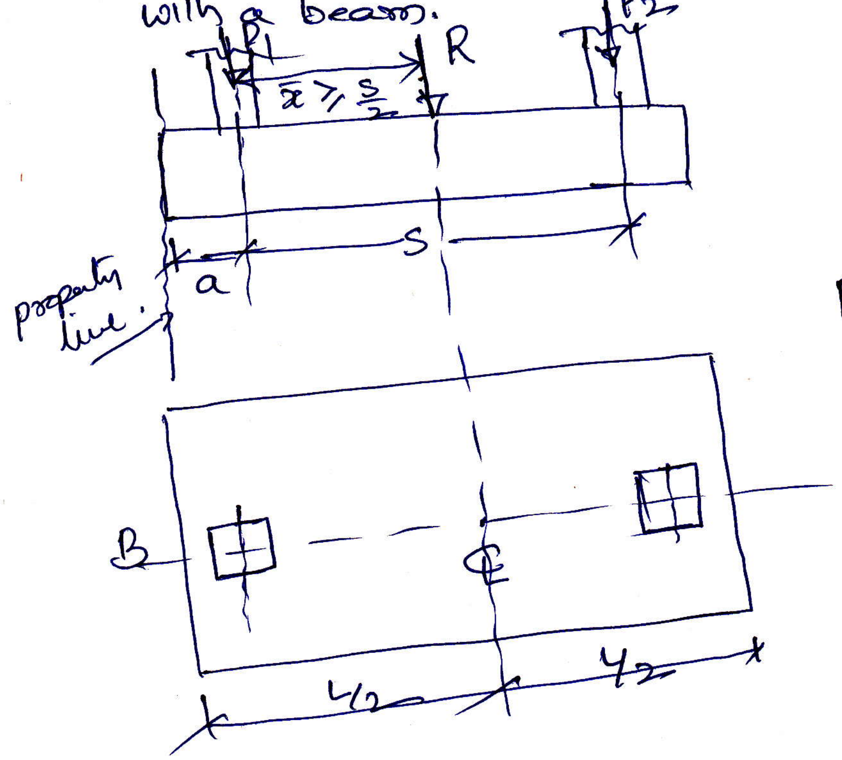
Raft foundation :- There is a grid of multiple columns

The combining of footings contributes to improved integral behaviour of the structure.

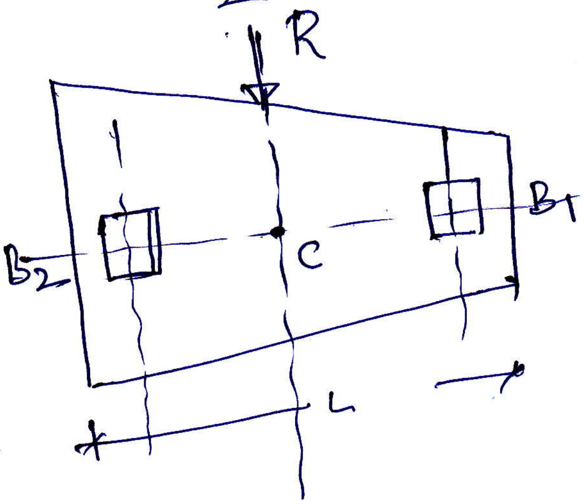
In case of property line limit which restricts the extension of the footing on one side. In this case, the non availability of space near the exterior column is circumvented by combining the footing with that of an interior column. The width of the footing may be kept uniform & tapered. The trapezoid -al shaped footing is required when the exterior column is more heavily loaded than the interior column.

It is sometimes encountered to provide a central beam interconnecting the column bases; this causes the base slab to bend transversely, while the beam alone bends longitudi -nally. (strip footing).

<u>Strap footing</u> :- An alternative to the conventional combined footing is the strap footing, in which the columns are supported essentially on isolated footings, but interconnected with a beam.



If $\bar{x} < \frac{S}{2}$



11 ## Given data

Col : 350 x 350 mm

A_{sc} = 8-20 mm ϕ

$P_u = 1.5 \times 2000 = 3000$ kN $P = 2000$ kN

SBC(q_o) = 300 kN/m^2 @ 1.25 m below GL

$f_{ck} = 25$ N/mm^2 $f_y = 415$ N/mm^2.

Assume $\Delta P = 10\% \, P = 200$ kN.

∴ Base area required $A_{req} = \frac{P + \Delta P}{SBC} = 7.33 m^2$

~~provide~~ Assume a square footing

side $B = \sqrt{7.33} = 2.708$ m.

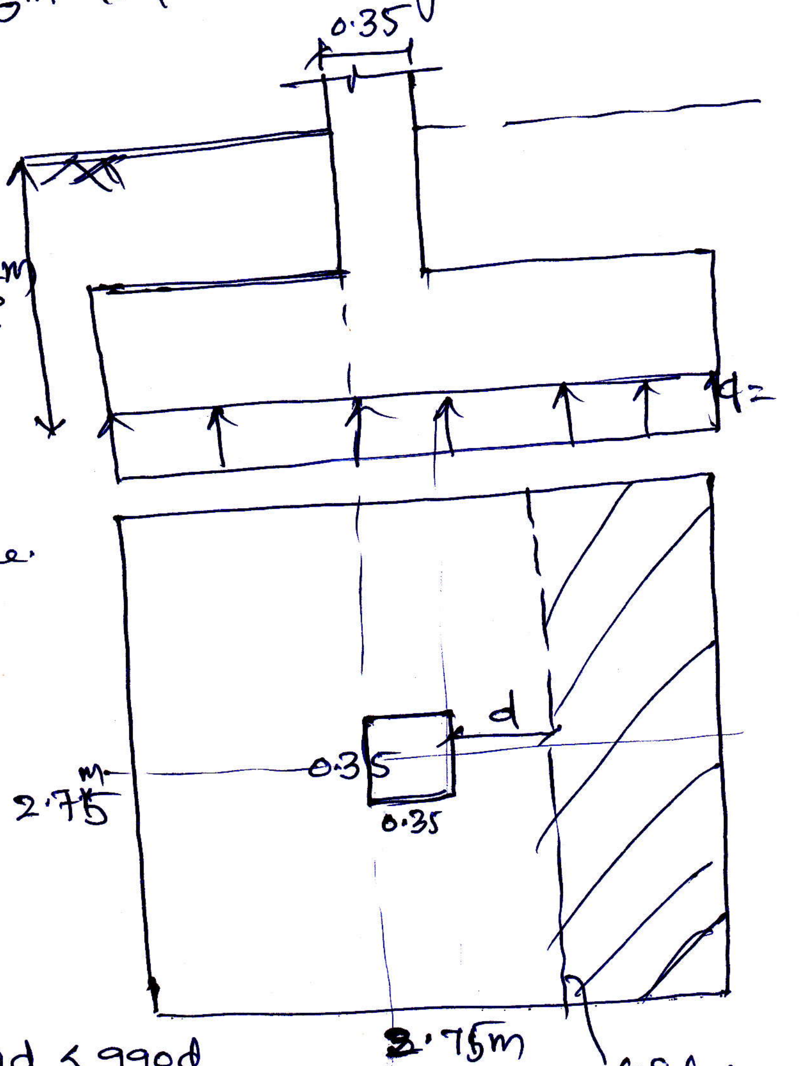
∴ provide 2.75 x 2.75 m square footing

Thickness of footing

Net soil pressure at the base

$q_u = \frac{1.5 \times 2000}{2.75^2} = 396.69$ kN/m^2

$= 0.396$ N/mm^2.



one way shear Consideration

critical section is at 'd' from face of the col.

$V_{u1} = 0.396 \times (1200 - d) \times 2750$

$= (1.306 \times 10^6 - 1089 d)$ N.

Assume $\tau_c = 0.36$ MPa for $P_t = 0.25\%$.

$V_{uc1} = 0.36 \times 2750 \times d$

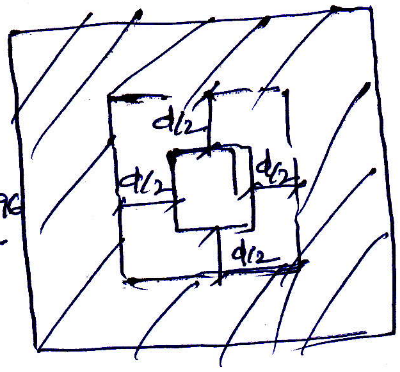
$= 990 d$.

∴ $V_{u1} \leq V_{uc1} \Rightarrow 1.306 \times 10^6 - 1089 d \leq 990 d$

$\Rightarrow \boxed{d \geq 628.18 \text{ mm}}$

Two-way shear

critical section is at $d/2$ from the periphery of the column.



$$V_{u2} = \cancel{0.396 \times (2750 - 350 - d)^2}$$
$$= 0.396 \times \left[2750^2 - (350+d)^2\right]$$
$$= 2.994\times10^6 - 48510 - 2772d - 0.396d^2$$
$$= 2.945\times10^6 - 2772d - 0.396d^2$$

Assume $d = 628$ mm

$$V_{u2} = 1.048\times10^6\ N = 1048\ kN.$$

$$V_{uc2} = k_s \cdot \tau_c \times \left[4\times(350+d)\times d\right]$$

For square col $k = 1$, $\tau_c = 0.25\sqrt{f_{ck}} = 1.25\ N/mm^2$

$$\therefore V_{uc2} = 1\times1.25\times4d\,(350+d)$$
$$= (1750d + 5d^2)\ N.$$

$$V_{u2} \leq V_{uc2} \Rightarrow 1.048\times10^6 \leq 1750d + 5d^2$$
$$\Rightarrow d^2 + 350d - 209.6\times10^3 \geq 0.$$
$$d \geq \frac{-350 \pm \sqrt{350^2 + 4\times209.6\times10^3}}{2} =$$
$$\geq \frac{-350 \pm 980.2}{2} = 315\ mm.$$

$\therefore$ one way shear governs the thickness.

Assume clear cover $= 50$ mm and 16 mm ϕ bars.

$$\therefore D \geq 628 + 50 + \frac{16}{2} = 686\,mm \simeq 700\,mm.$$

$\boxed{D = 750\ mm}$ $\boxed{d = 750 - 50 - 8 = 692\ mm}$

Check for ~~BPSC~~

~~Net~~ Actual gross pressure at the base under service loads.

$$q = \frac{2000}{2.75\times2.75} + 24\times0.7 + 18\times0.55 = 291.1\ kN/m^2$$
$$< SBC \quad \underline{\underline{ok}}$$

Design for flexural reinforcement

$$M_u = (0.396 \times 2750) \times \frac{1200^2}{2} = 784.08 \times 10^6 \text{ N-mm}$$
$$= 784.08 \text{ kN-m.}$$

$$d_{req} = \sqrt{\frac{784.08 \times 10^6}{3.44 \times 2750}} = 287.89 \text{ mm} < d_{prov.} \text{ — safe}$$

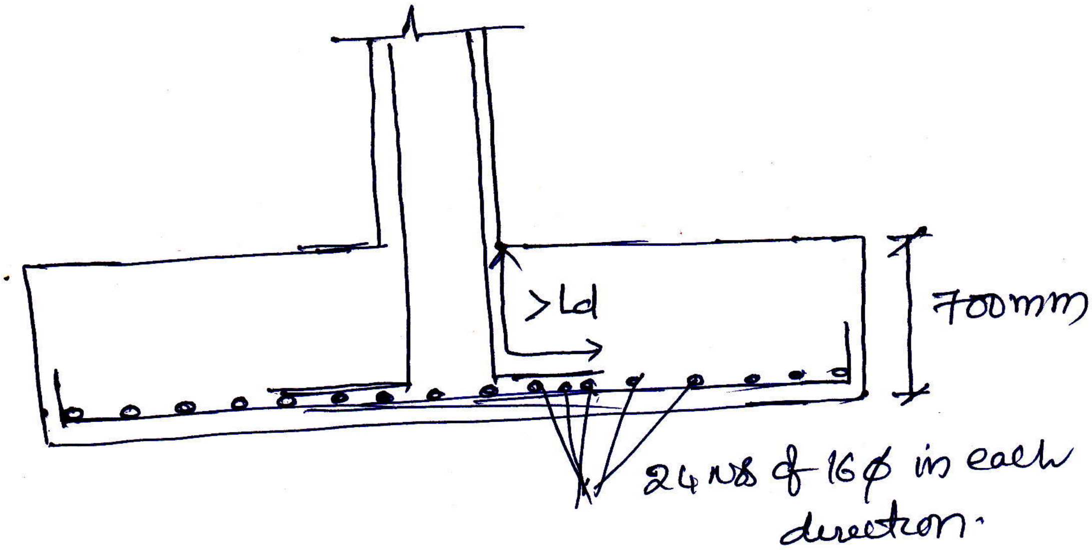
— URS.

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

$$= \underline{\underline{3244.37 \text{ mm}^2}} \qquad p_t = 0.17\% < 0.25\% \text{ for shear.}$$

$$\therefore A_{st} = \underline{\underline{4757.5 \text{ mm}^2}}$$

No. of 16mm ϕ bars = 23.66 ≃ 24 Nos in each direction



Transfer of force at the base should be checked

Extend all the column bars into the footings.