Design of Reinforced Cement Concrete

Exam based Notes

Dedicate to
my
All friends 014-Civil

Features of Note

1. Clear vision of question and given values or terms
2. tips
3. Design step [clear Design step]
4. Design Example

Prepared By
Puran Shrestha
014-023
Civil 1 'A'

Doubly Beam Design

Given Load, span, size of beam (May not be given)

if not give $\left[d = \frac{l_e}{12},\ b = d/2\right]$

To find

Size of beam, Area of Steel
(b×D)

Tips Factor load (design load) = 1.5 * working load
OR
superimposed load

examples

* question मा load कुन type को हो Analyze गर्ने; question मै Factor load, design load भनेर दियो भने 1.5 ले multiply गर्नु पर्दैन

* question मा size of Beam 300×600 effective भनेको भने 600 mm = d हो
if 300*600 overall भनेको भने 600mm = D हुन्छ,

* Beam of, clear span = 5.5 भनेमा leff निकाल्ने
already effective span भनेमा given span नै effective span हुन्छ,,

* TOR steel = Fe415, TMT steel = Fe500
Mild exposure = M20

BM को लागि leff use गर्ने
SF को लागि l मात्र use गर्ने

For simply supported Beam
$BM = \frac{wl_e^2}{8}$
$BM = \frac{wl}{2}$

$l_e = l_c + t/2 + t/2$ or $l_c + d/2 + d/2$ Take minimum

* question मा Bearing / thickness of wall भनेमा $l_e = l_c + t/2 + t/2$ use गर्ने

Steps for Design

Step 1 / size calculation of Beam, load calculation, Moment calculation (M_u)

Step 2 / Calculation of Limiting Moment

$$M_{lim} = 0.138 f_{ck} bd^2 \quad \text{for Fe 415}$$
$$= 0.133 f_{ck} bd^2 \quad \text{Fe 500}$$

$M_u > M_{lim}$ Here doubly section

calculation of steel on tension zone

A_{st1}

$$M_{lim} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$$

Step 3 calculation of steel on compression zone

Extra moment $= M_u - M_{lim} = (f_{sc} - f_{cc}) A_{sc} (d - d')$

For equilibrium

$$T_2 = C_2$$

$$0.87 f_y A_{st2} = (f_{sc} - f_{cc}) A_{sc\,provided}$$

Total steel on tension zone

$$A_{st} = A_{st1} + A_{st2}$$

Step 4. Check for min steel $\Rightarrow A_s = \frac{0.85 bd}{f_y}$

max steel $= 4\%$ of bD

Step 5 Check for shear.

Nominal shear stress $(\tau_v) = \frac{V_u}{bd}$

Shear strength of concrete (τ_c) Pg 40, τ_{max}

if $\tau_v < \tau_c$, $\tau_v > \tau_{max}$

stirrup is needed

$$V_{us} = V_u - \tau_c * bd = 0.87 f_y A_{sv} \frac{d}{a}$$

Step 6 Check for development length.

Design a Rectangular beam for a clear span of 5.0m. The superimposed load is 50 kN/m and size of the beam is limited to 250*500 mm effective. Use TOR steel and Mild explosure condition

2

Step-1 Given $L_c = 5.0$ m

load = 50 kN/m

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

$f_y = 415$ N/mm^2

$b = 250$ mm

$d = 500$ mm

overall depth $D = d + 25 + \phi/2$ (cover)

$= 500 + 25 + 20/2$ (Assume)

$= 535$ mm

Self weight of Beam $= 25 \times b \times D$ (unit wt of RCC)

$= 25 * 0.25 \times 0.535$

$= 3.34$ kN/m

Total load $= 53.34$ kN/m $(50 + 3.34)$

Factored load $= 1.5 * 53.34$

$= 80.01$ kN/m

$l_{eff} = 5.0 + \frac{0.500}{2} + \frac{0.500}{2}$ ← depth of Beam

$= \cancel{6.}\ 5.5$ m

Moment $M_u = \frac{wl^2}{8} = \frac{80.01 * 5.5^2}{8}$

$= 302.53$ kN-m

Sher force $V_u = \frac{wl}{2} = \frac{80.01 * 5.0}{\cancel{8}\ 2}$

$= \cancel{50\text{ kN}}$

200.02

(21)

Step 2

Limiting moment $M_{lim} = 0.138 f_{ck} b d^2$ $Fe\,415$

$= 0.138 \times 20 \times 250 \times 500^2 \times 10^{-6}$ kN-m

$= 172.5$ kN-m

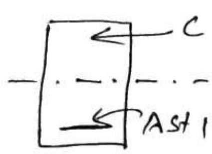
$M_u > M_{lim}$ Hence Doubly section needed

For A_{st1}

$\therefore M_{lim} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b}\right)$

$172.5 \times 10^6 \, (N\text{-}mm) = 0.87 * 415 \, A_{st} \left(500 - \frac{415 * A_{st}}{20 * 250}\right)$

$\Rightarrow \boxed{A_{st_1} = 1191.02 \text{ mm}^2}$

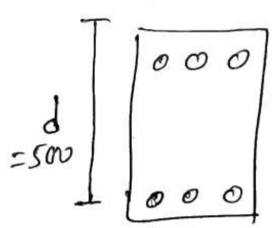


Step 3 Extra moment $= M_u - M_{lim}$

$= 302.53 - 172.5$

$= 130.03$ kN-m

$= (f_{sc} - f_{cc}) \, A_{sv} (d - d')$



$d' = 25 + \phi/2 = 35$ mm

For F_{sc} pg 38

$d'/d = \frac{35}{500} = 0.07 \simeq 0.1$

$f_{sc} = 353$ N/mm²

$F_{cc} = 0.446 f_{ck}$

$= 8.42$

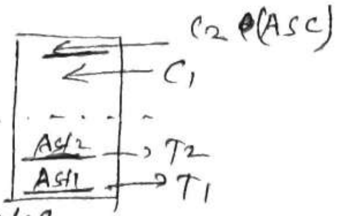
$130.03 \times 10^6 \, (N\text{-}mm) = (353 - 8.42) \, A_{sv} (500 - 35)$

$\Rightarrow \boxed{A_{sv} = 811.52 \text{ mm}^2}$

provide $\boxed{3\text{-}20\text{mm}\,\phi}$ $\left[3 \times \frac{\pi}{4} 20^2 = 942 \text{ mm}^2\right]$

For equilibrium

$T_2 = C_2$



$0.87\, f_y\, A_{st2} = (f_{sc} - f_{cc})\, A_{sc\,provided}$

or $0.87 \times 415 \times A_{st2} = (353 - 8.92) * 942$

$\Rightarrow \boxed{A_{st2} = 897.72\ mm^2}$

Total steel in tension zone

$A_{st} = A_{st1} + A_{st2} = 1191.02 + 897.72$

$\boxed{A_{st} = 2088.79}$

provide $\boxed{5 - 25\,mm\ \phi}$ $(A_{st} = 2454\ mm^2)$

Step 4 Check for steel

min steel $= \dfrac{0.85 \times 250 \times 500}{415}$

$= 256.02$ ok

max steel $= \dfrac{4}{100} \times 250 \times 535$

$= 5350$ ok

Step 5 Check for shear

nominal shear stress $(\tau_v) = \dfrac{V_u}{bd} = \dfrac{200.02 \times 10^3\ N}{250 \times 500\ mm^2}$

$= 1.6\ N/mm^2$

Shear strength of concrete, M20

$\tau_c = 0.7841$

$\tau_{max} = 2.8$

$\tau_v > \tau_c$ or $\tau_{max} > \tau_v$

Here

stirrup is needed

$1.75 \rightarrow 0.75$

$2.0 \rightarrow 0.79$

$1.9632 \Rightarrow x = 0.7841$

For τ_c / pg 40

$\% p = \dfrac{100\, A_{st\,provided}}{bd}$

$= \dfrac{100 \times 2454}{250 \times 500}$

$= 1.9632$

~~0.75 → 0.56~~

~~1.0 → 0.62~~

~~1.9632 → x =~~

(31)

provide 8mm ϕ 2 legged vertical stirrups

$V_{us} = V_u - \tau_c b d$

$= 200.02 \times 10^3 - 0.7841 * 250 \times 500$

$= 102007\ N = 102\ kN$

Asu =
$2 \times \frac{\pi}{4} 8^2$
$= 100$

$= 0.87 f_y A_{sv} \times \frac{d}{x}$ [Pg 40]

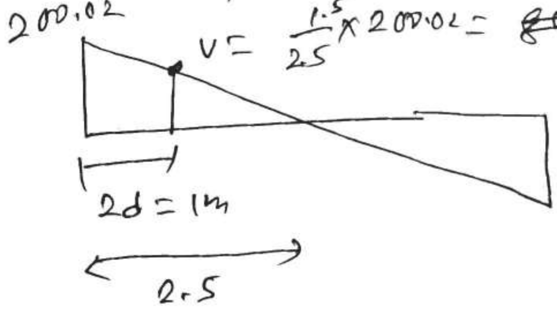
or $102007 = 0.87 \times 415 \times 100 \times \frac{500}{x}$

$\boxed{\Rightarrow x = 176mm \simeq 150mm}$ $> 100mm$ $\not> 300mm$

provide 8mm 2 legged stirrup @ 150mm

Again shear force at 2d distance

$V = \frac{1.5}{2.5} \times 200.02 = \cancel{80.008}\ 120$



$\overset{120}{\cancel{80.008}} \times 10^3 - 0.7841 \times 250 \times 500 = 0.87 f_y A_{sv} * \frac{d}{x}$

$\Rightarrow \boxed{x = 821 \not> 300}$

provide 8mm 2 legged @ 300mm c/c

Step 6 Check for deflection

$L_d = 1.3\frac{M_1}{V} + L_o$ (Pg 13)

$\frac{0.87 \times f_y \phi}{4\, \tau_{bd}}$ (Pg 11) $= 1.3\frac{M_1}{V} + L_o$

Assume Ubent $L_o = 16\phi$

$M_1 = 0.87 f_y \underset{\downarrow}{Ast_{pro}} \left(d - \frac{f_y\, Ast_{pro}}{f_{ck}\, b}\right)$

$= \quad 2454$

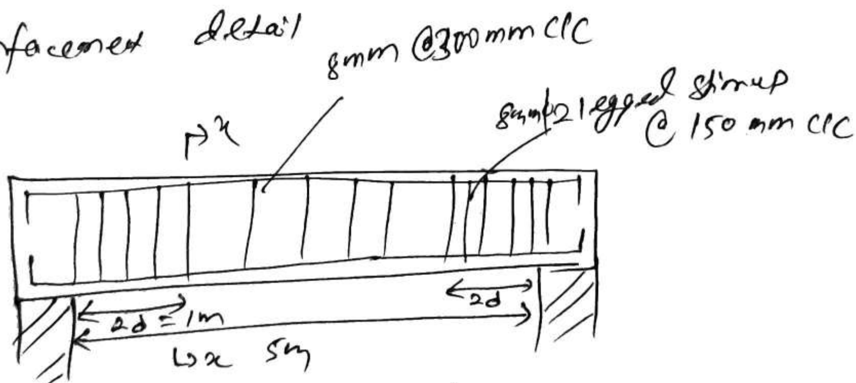
262.54 kN-m

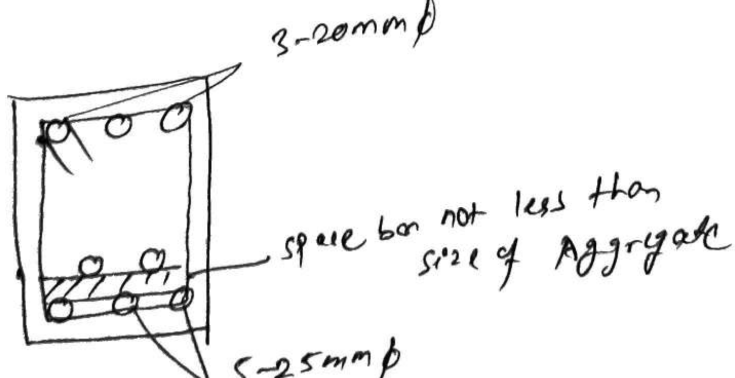
$\tau_{bd} = 1.2 \times 1.6$ (From Pg 12)

$\frac{0.87 \times 415\,\phi}{4 \times 1.2 \times 1.6} = 1.3 \times \frac{262.54 \times 10^6}{200 \times 10^3} + 16\phi$

$\Rightarrow \phi = 55.06 > \phi_{provided}$ OK

Reinforcement detail





X-Y

(4')

Cantilever Beam

Design a cantilever 3.5 meter long load carrying a superimposed load of 12000 N/m use M20 concrete

Soln

Step 1) Depth, breadth calculation.

Effective depth = $\frac{span}{7}$

$= \frac{3500}{7} = 500mm$

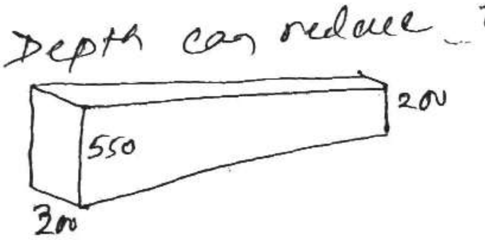
$D = 500 + 25 + \phi/2$ → 16 assume

$= 533 \simeq 550 mm$

$d = 550 - (25 + \phi/2) = 517 mm$

adopt $b = 0.5D = 275 \simeq 300mm$

Depth can reduce to 200mm at free end



Step 2 load calculation

Dead load of cantilever $= \frac{0.55+0.2}{2} \times 0.3 \times 3.5 \times 25$ kN

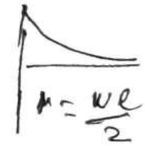
$= 9.84$ kN

CG from support $= \frac{d_1 + 2d_2}{d_1 + d_2} \times \frac{span}{3}$

$= \frac{0.55 + 2 \times 0.2}{0.55 + 0.2} \times \frac{3.5}{3}$

$= 1.47$ m

Moment calculation



M_u = Moment due to self wt + given load

$= 9.84 \times 1.47 (kN\text{-}m) + 12 \times \frac{3.5^2}{2}$

$= 88.05$ kN-m

factored BM $(M_u) = 1.5 \times 88.05 = 132.07$ kN-m

Step 3 depth calculation

Limiting moment $(M_{lim}) = 0.138 f_{ck} b d^2$

$132.07 \times 10^6 = 0.138 \times 20 \times 300 \times d^2$

$\Rightarrow d = 399.38 <$ d provided Ok

Steel calculation

$132.07 \times 10^6 = 0.87 \times 415 \times A_{st} \left(517 - \frac{415 \times A_{st}}{20 \times 300}\right)$

$\Rightarrow A_{st} = 621.8\ mm^2$

provide 4-16mm ϕ $\left(A_{st} = 4 \times \frac{\pi \phi^2}{4} = 804.24\ mm^2\right)$

check for minimum steel

$A_o = \frac{0.85 bd}{f_y} = \frac{0.85 \times 300 \times 517}{415}$

$= 317.6\ mm^2$ Ok

(5')

Step 4 Design of shear

shear force = 9.84 + 12 * 3.5

= 51.84 kN

factored shear force = 1.5 × 51.84

$(V_u) = 77.76$ kN

Now $\tau_v = \frac{V_u}{bd} = \frac{77.76 \times 10^3}{300 \times 517}$

$\tau_v = 0.5\ N/mm^2$

For τ_c $\frac{100 A_{st}}{bd} = \frac{100 \times 804}{300 \times 517} = 0.51$

0.5 → 0.48

0.75 → 0.56

0.51 ⇒ x = 0.48

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

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

$\tau_c < \tau_v$, $\tau_v \not> \tau_{max}$ Here Stirrup is needed

For Stirrups provide 8mm 2legged vertical stirrup

$A_{sv} = 2 \times \frac{\pi}{4} 8^2 = 100 mm^2$

$V_{us} = 77.76 \times 10^3 - 0.48 \times 300 \times 517$

= 3.312 kN = 3312 N

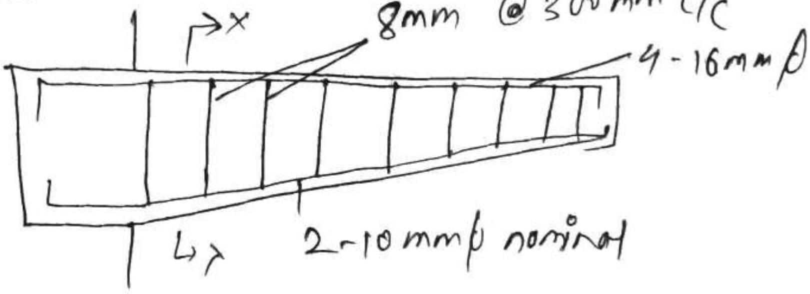
$= 0.87 \times f_y A_{sv} \frac{d}{x}$

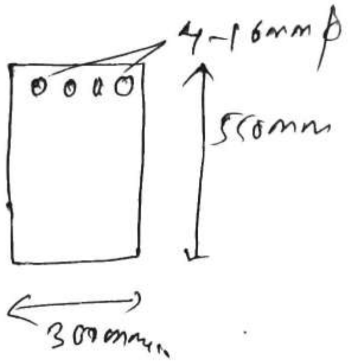
⇒ x = 563.5 mm $\not>$ 300 mm

$\not>$ 0.75d = 387

provide 8mm 2legged stirrup @ 300mm c/c

Reinforcement detail

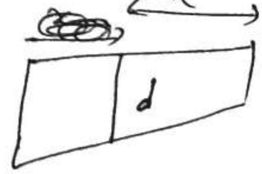




editing

<u>Curtailment of top Bar</u>

Reinforcement can be curtailed towards the free end of cantilever because bending moment decrease very rapidly. let us curtail 2-16mm bar at distance x from the free end



~~d = 500~~
~~517 - 25 -~~
$\cancel{d = 517 - 25 - \frac{16}{2}}$

$$d = 200 - 25 - \frac{16}{2} + \frac{550-200}{3500} x$$

$$= (159 + 0.1x)$$

$$\text{Moment} = \frac{132.07 \times x^2 \times 10^6}{3500^2} = 10.78\, x^2 \ \text{N-mm}$$

Now

$$M = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck}\, b} \right)$$

$$A_{st\,remaining} = 2 \times \frac{\pi}{4} 16^2 = 402.12\, mm^2$$

(6)

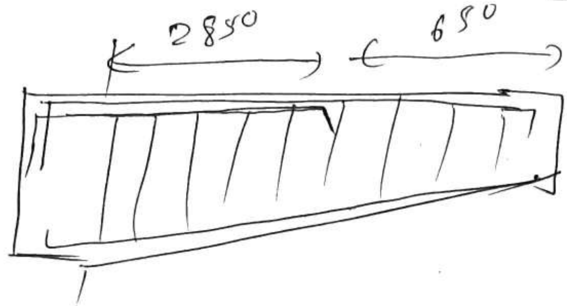
$$10.78 * x^2 = 0.87 * 415 * 402\left(159 + 0.1x - \frac{415 \times 402}{20 \times 30}\right)$$

$$10.78x^2 = 14514.21x - 1904119x.1$$

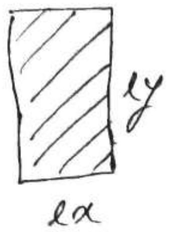
$\Rightarrow x = 673.20 \simeq 650$

Distance from free end $= 3500 - 650$

$= 2850$

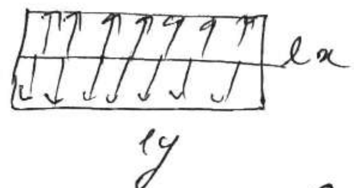


Slab Design

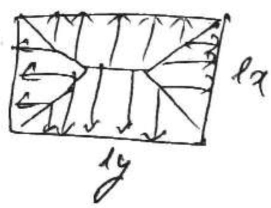


* short span = lx
long span = ly } are effective lengths.

* one way slab ($ly/lx > 2.0$)
→ load transfer in one direction



* two way slab ($ly > lx < 2.0$)
→ load transfer in two direction



Slab detail

- deflection $d = \dfrac{lx}{\alpha\beta\gamma\lambda\delta}$

- min steel = 0.12 of bd
max steel = $\frac{1}{8} * bD$
spacing of bar For main bar : 3d or 300 } smaller

For distribution bo 5d 450mm } smaller

cover = 15mm
width = 1.0m

(7)

Design of one way slab

Step 1: calculate depth $d = \dfrac{l_x}{(\alpha \beta \gamma \delta \lambda)}$ (pg-6)

$(\alpha\beta\gamma\delta\lambda) \rightarrow 30\text{-}35$ assume

Step 2: load and moment calculation (same as beam)

Step 3: Design of sectional size and steel calculation

Step 4: Check for shear, development length, deflection

Step 5: Figure is necessary — plan - 1, section - 2

Design a simply supported roof slab for a hall 9m × 4m clear size.

Given load = $4\,kN/m^2$

concrete M20

steel Fe 415

Soln

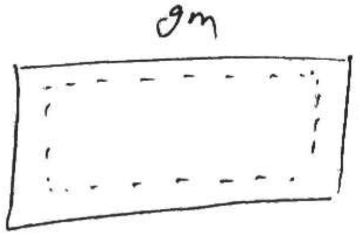
$l_x = 4m$

$l_y = 9m$

$\dfrac{l_y}{l_x} = \dfrac{9}{4} = 2.25 > 2$

One way slab

9m



4m

let $d = \dfrac{l_x}{30} = \dfrac{400}{30} = 133.33\,mm$

$\simeq 140\,mm$

$D = 140 + 15 + \dfrac{10}{2} \rightarrow \phi$ Assume

$= 160\,mm$

Self wt of slab = $25 \times b \times D$

$= 25 \times 1.0 \times 0.16$

$= 4\ kN/m$

~~Fo~~ given load = $4\ kN/m^2 \times 1m$

$= 4\ kN/m$

Total load = $8\ kN/m$

Factored load = $1.5 \times 8 = 12\ kN/m$

<u>for Moment</u>

$l_{eff} = l_{ct} + \frac{t}{2} + \frac{t}{2} = 4.0 + \frac{0.16}{2} + \frac{0.16}{2}$

$= 4.14\ m$

Now $BM = \frac{wl^2}{8} = \frac{\overset{12}{\cancel{8}} \times 4.14^2}{8} = 25.7094\ kN\text{-}m$

<u>Step 3 Design</u>

(a) <u>Size</u> $M_{um} = 0.138 f_{ck} b d^2$

$25.7094 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$d \simeq 96.51 <$ d provided <u>ok</u>

provide depth of slab $D = 160mm$

(b) <u>Steel bar</u>

(i) <u>main bar</u>

$M_{um} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$

$25.7094 \times 10^6 = 0.87 \times 415 A_{st} \left(140 - \frac{415 A_{st}}{20 \times 1000} \right)$

$\boxed{\Rightarrow A_{st} = 554.13}$

number of bar = $\frac{A_{st}}{A_\phi}$

spacing = $\frac{1000}{n} = \frac{1000 A_\phi}{A_{st}}$

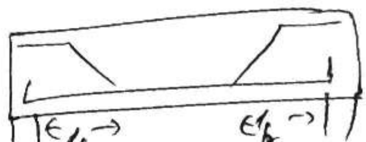
(8) let us provide 10mm ϕ $\Rightarrow$ $A_{10} = 78.5 mm^2$

$$\text{spacing} = \frac{1000 \times 78.5}{554.13}$$

$$= 141.66$$

$$\simeq 140mm \ c/c$$

provide 10mm @ 140mm c/c with alternative bentup by $l/7$ from face of support



$$A_{st}\text{provided} = \frac{1000 A_0}{\text{spacing}} = \frac{1000 \times 78.5}{140}$$

$$\boxed{A_{st}P = 560.71 \ mm^2}$$

<u>steel check</u>

$$\text{min steel} = 0.12\% \text{ of } bD$$

$$= 0.12 \times \frac{1}{100} \times 1000 \times 160$$

$$= 192 \quad (OK)$$

$$\text{max spacing} = 3d = 3 \times 140$$

$$= 420 > 140mm$$

(OK)

(ii) Distribution bar

$$A_D = 0.12\% \text{ of } bD$$

$$= 192 mm^2$$

let 8mm ϕ ; spacing $= \dfrac{1000 \times 50}{192}$

$A_8 = 50mm^2$

$$= 260 mm \simeq 250mm$$

check spacing $\not> 5d$
$\not> 450mm$

provide 8mm ϕ @ 250mm c/c

4 (a) Check for shear

Shear force $V_u = \frac{w l_x}{2} = \frac{12 * 4}{2} = 24$ kN

nominal shear stress $(\tau_v) = \frac{V_u}{bd}$

$= \frac{24 \times 10^3 \text{ N}}{1000 \times 140}$

$= 0.17 \text{ N/mm}^2$

Shear strength of Concrete

$\tau_c = 0.32$

Modified shear stress

$\tau_c = k \times \tau_c$

$= 1.3 \times 0.32$

$= 0.416$

$k = 1.3$ (pg 39)
For $d = 150$ or less

For τ_c

$p = \frac{100(A_{st}/2)}{bd}$ → support पर steel half होता, Because of Alternative bent up

$= \frac{100 \times 560.71/2}{1000 \times 140}$

$= 0.20$ (pg-40)

$0.15 \rightarrow 0.28$

$0.25 \rightarrow 0.36$

$0.20 \rightarrow x = 0.32$

$\tau_c > \tau_v$ Here

no need for shear Reinforcement

(b) Development length

$M_1 = 0.87 * f_y * \frac{A_{st}}{2} \left(d - \frac{f_y A_{st}/2}{f_{ck} b} \right)$

$= 0.87 \times 415 \times \frac{560.71}{2} \left(140 - \frac{415 \times 560.71/2}{20 \times 1000} \right)$

$= \cancel{25.98}\ 13.58$

$l_d = 1.3 \frac{M_1}{V} + l_0$

$\frac{0.87 * 415\, \phi}{4 \times 1.2 \times 1.6} = 1.3 \frac{13.58 \times \cancel{25.98} \times 10^6}{24 \times 10^3} + 16\, \phi$

$\Rightarrow \phi = \cancel{65}$ mm 23.71 mm $> \phi$ provided

OK

(9) (c) Deflection

$$d = \frac{l_x}{\alpha\beta\gamma\delta\lambda}$$

(Pg-006) $\alpha = 20$
$\beta = 1$

$$f_s = 0.58 \times f_y \frac{A_{st}^{r}}{A_{st}^{p}} = 0.58 \times 415 \times \frac{554.13}{560.7}$$
$$= 238.12$$

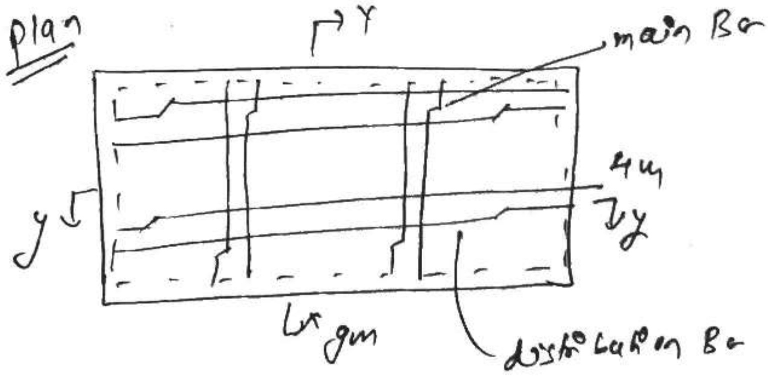
$P = \frac{100 A_{st}}{bd}$ → Take full Ast Because Deflection occur at centre

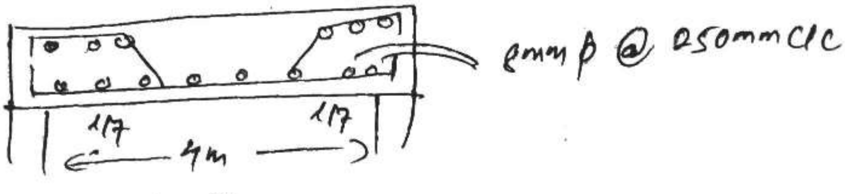
$$= \frac{100 \times 560.7}{1000 \times 140} = 0.4005$$

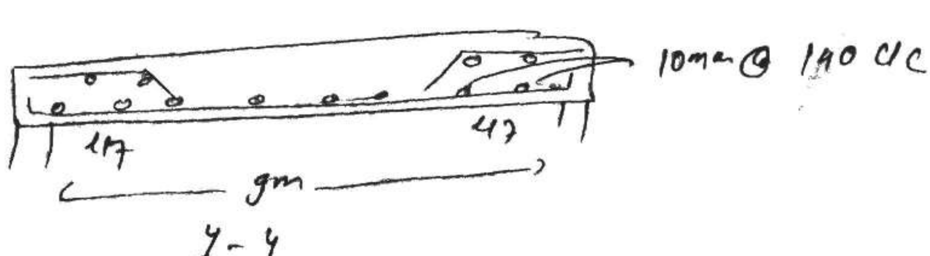
$\boxed{\text{Pg 007 Fig-4}}$
⇒ $\gamma = 1.62$

$$d = \frac{4000}{20 \times 1.62} = 123.45 < d_{provided}$$
OK

plan





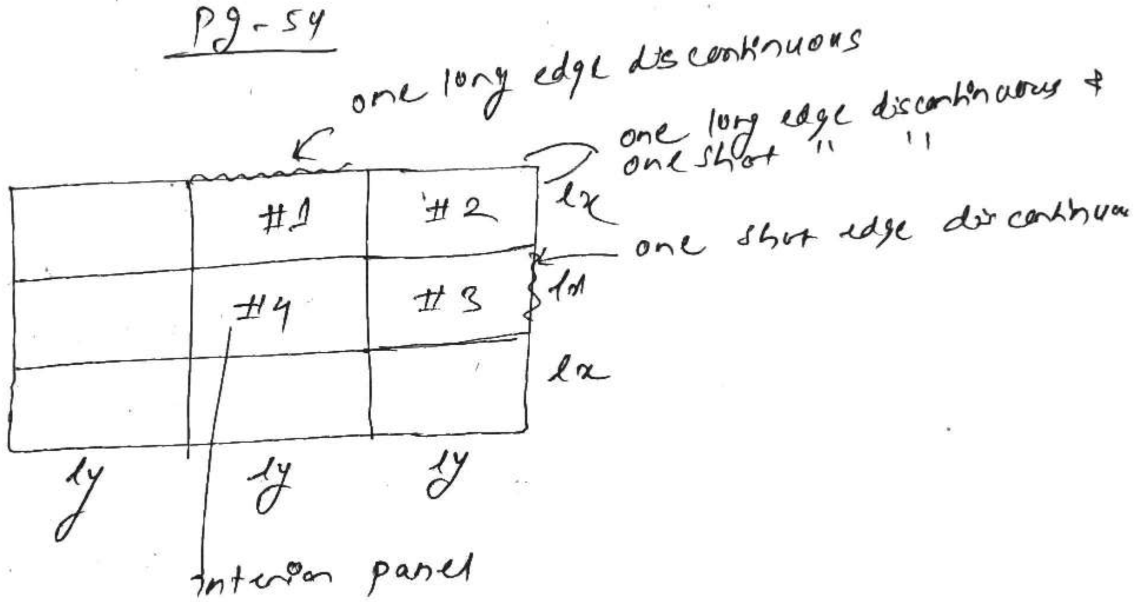


Design of two way slab ($l_y/l_x < 2.0$)

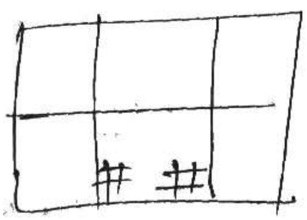
- load distribute on Both sides.
- Both sides Reinforcements are main bars calculated

$M_x = \alpha_x w l_x^2$
$M_y = \alpha_y w l_x^2$
use l_x for Both side moment calculation

Pg - 54



Torsional Reinforcement for discontinuous edge

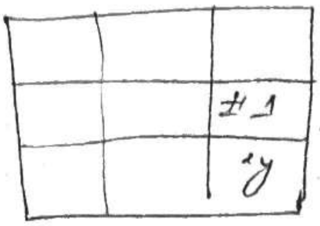


mesh length $= \frac{l_x}{5}$

Area of steel = 75% of main steel

(10)

Design the panel #1 as indicated in the typical floor as shown in fig. The clear dimension of panel is 5.5x4.5m clear and is rest on 250mm wide beams. It is subjected to live load of $4 kN/m^2$ and consists of 15mm, thick pcc floor finish on the slab. Assume M20 concrete and Fe415 steel. Sketch the Reinforcement detailing



$l_x = 4.5m$

$l_y = 5.5m$

$d = \frac{l_x}{30} = \frac{4500}{30}$

$= 150mm$

$D = 150 + 15 + \frac{10}{2}$

$= 170mm$

Step 1 Calculation of Moment.

Self wt of slab = $25 \times 1.0 \times 0.17$

$= 4.25 \ kN/m$

pcc flooring = $24 \times 1 \times 0.015$

$= 0.36 \ kN/m$

live load = $4 \ kN/m^2 \times 1m = 4 kN/m$

Total load = $8.61 \ kN/m$

factored load = 1.5×8.61

$= 12.915 \ kN/m$

$BM_x = \alpha_x w l_x^2$

$BM_y = \alpha_y w l_x^2$

(11)

To α_x, α_y

our panel that is one short edge discontinuous.

Pg. 64, Table 26,

$l_y : L_y + \frac{t}{2} + \frac{t}{2}$ or $L_y + \frac{t}{2} + \frac{t}{2}$

$l_y = 5.5 + \frac{0.17}{2} \times 2$

$= 5.67\,m$

$l_x : 4.67\,m$

$l_y / l_x = 5.67 / 4.67 = 1.21$

$\alpha_y^- = 0.037$		1.2	1.21	1.3
$\alpha_y^+ = 0.028$	α_x^-	0.048	p	0.051
$\alpha_x^- = 0.0483$	α_x^+	0.036	p	0.039
$\alpha_y^+ = 0.0363$	using interpolation			
	p = 0.0483			
	p = 0.0363			

$BM_x = 0.0483 \times 12.915 \times 4.67^2$

$= 13.60\ kN\text{-}m$

$BM_y = 0.037 \times 12.915 \times 4.67^2$

$= 10.42\ kN\text{-}m$

Maximum Bending moment $= 13.60\ kN\text{-}m$

Step 3 Design

(a) sectional size

$13.60 \times 10^6 = 0.138 \times \cancel{415}\,20 \times 1000 \times d^2$

$\Rightarrow d = 70.19 <$ d provided OK

(ii) (b) Steel

For shorter span

$$BM_x = 0.87 f_y A_{st}\left(d - \frac{f_y A_{st}}{f_{ck} b}\right)$$

$$13.60\times10^6 = 0.87\times415\, A_{st}\left(150 - \frac{415 * A_{st}}{20\times1000}\right)$$

$$\Rightarrow \boxed{A_{st} = 260.50\ mm^2}$$

provide 8mm ϕ ($A_\phi = 50$)

$$\text{spacing} = \frac{1000 A_\phi}{A_{st}} = \frac{1000\times50}{260.5}$$

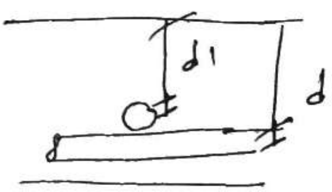
$$= 191.9\ mm$$

$$\simeq 190mm > 100mm$$

provide spacing 150mm c/c

$$\boxed{A_{st}P = \frac{1000\times50}{150} = 333.33\ mm^2}$$

(b) Steel for longer span



$$d' = d - \phi/_2 - \phi/_2$$

$$= 150 - 8/_2 - 8/_2$$

$$= 142\ mm$$

$$M_y = 10.42\times10^6 = 0.87\times415 A_{st}\left(142 - \frac{415\, A_{st}}{20\times1000}\right)$$

$$\Rightarrow A_{st} = 209.66\ mm^2$$

provide 8mm ϕ bar

$$\text{spacing} = \frac{1000\times50}{209.66} = 238.48$$

provide 8mm ϕ @ 200 c/c

Check min steel = $0.12\% bD$

$$= \frac{0.12\times1000\times170}{100} = 204\ mm^2 < A_{st}\ \text{ok}$$

Step 3 (a) Check for shear

$$V_u = \frac{w l_x}{2} = \frac{12.915 \times 4.5}{2}$$
$$= 29.05 \text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{29.05 \times 10^3}{1000 \times 150}$$
$$= 0.1936 \text{ N/mm}^2$$

For τ_c, $$p = \frac{100 A_{st}/2}{bd} = \frac{100 \times 333.3/2}{1000 \times 150}$$

Pg - 40 $$= 0.11$$

$\tau_c = 0.28$ for $p < 0.15$
$> \tau_v$ ok

(b) Check for ~~shea~~ development length

$$M_1 = 0.87 f_y \frac{A_{st}}{2} \left(d - \frac{f_y \, A_{st}/2}{f_{ck} \, b}\right)$$ ($A_{st} \to 333.33$)

$$= 8.81 \text{ kN-m}$$

$$l_d = 1.3 \frac{M_1}{V} + l_0$$

Pg - 4

$$\frac{0.87 f_y \phi}{4 \tau_{bd}} = 1.3 \frac{M_1}{V} + l_0$$

$$\frac{0.87 \times 415 \phi}{4 \times 1.2 \times 1.6} = 1.3 \times \frac{8.81 \times 10^6}{29.05 \times 10^3} + 16\phi$$

$\phi = 12.71 > \phi$ provided ok

121

(c) deflection

$$d = \frac{l_x}{\alpha \beta \gamma \delta \lambda}$$

$$f_s = 0.58 \times 415 \times \frac{260}{333}$$

$$= 187.93$$

$$\%P = \frac{100 A_{st}}{bd} \quad (\rightarrow 333)$$

$$= 0.22\ldots$$

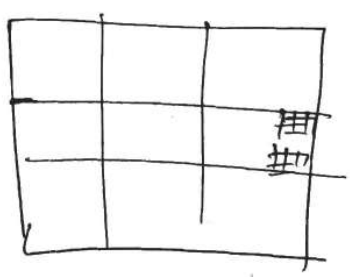
$$\Rightarrow \gamma = 2.0$$

pg-6

$$\alpha = \frac{20+26}{2} = 23$$

$$d = \frac{4500}{23 \times 2} = 97.82 < d_{provided}\text{ ok}$$

(d) Torsional reinforcement

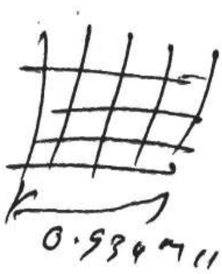


$$\text{mesh length} = \frac{l_x}{5} = \frac{4.67}{5} = 0.934\text{ mm}$$

$$A_{st} = 0.75 \times 333$$

$$= 250.2\text{ mm}^2$$

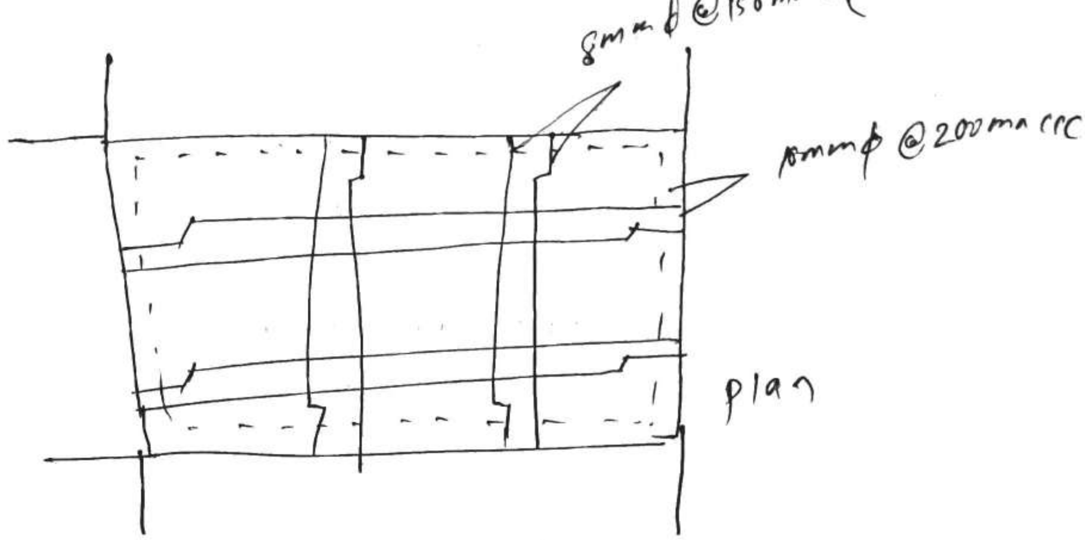
no of bar = 5 nos



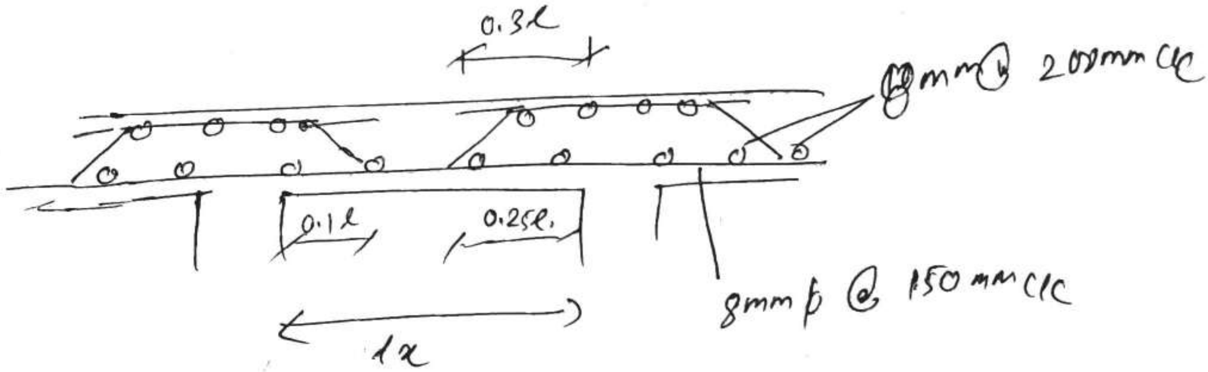
0.934 mm

0.934 mm

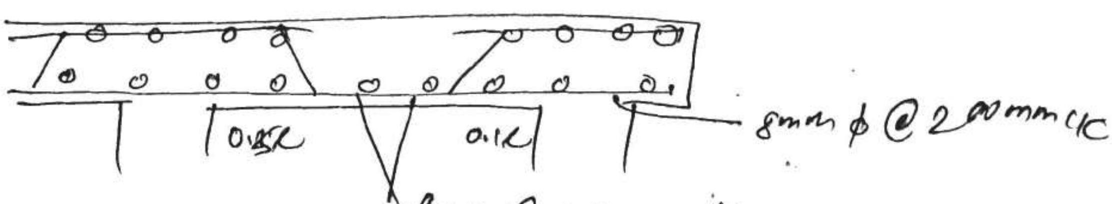
Reinforcement detail



plan



section through short span

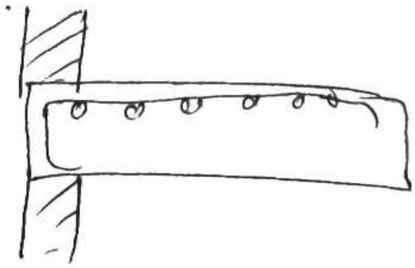


Section through long span

(3) Design of cantilever slab (Balcony)

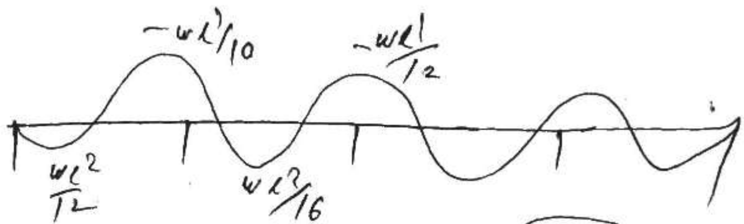
* eff depth = $\frac{span}{10 \to 12}$
* BM = $\frac{wl_x^2}{2}$

  SF = wl



All design steps are same

Design of continuous slab

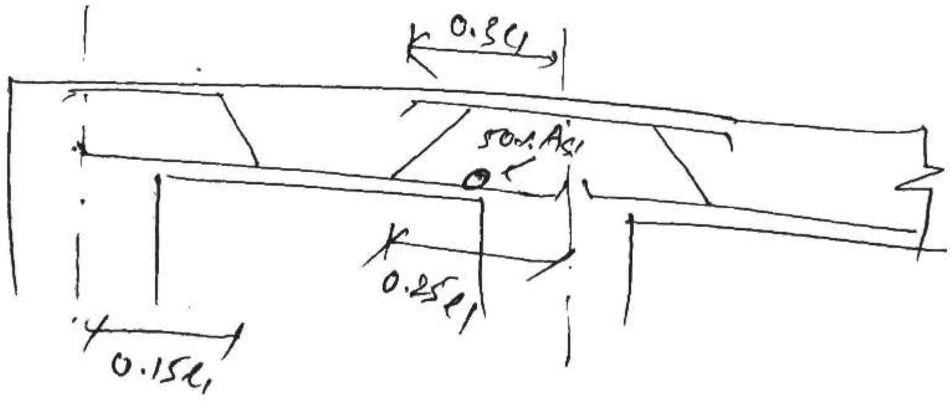


BM = αwl^2

SF = αwl

IS 5

α taken from Table 12



Column

① Short column ($\lambda < 12$)
- → axial load and uniaxial bending
- → axial load and biaxial bending

② long column ($\lambda > 12$)
- → axial load and uniaxial bending
- → axial load and biaxial bending

Biaxial Bending गर्दा पनि always

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1.0$$ formula use हुन्छ।

(Pg-34)

Check for steel

min steel = $0.8\% \text{ of } bD$

max steel = $4\% \text{ of } bD$

no of bar (minimum)

① Rectangular section = 4 nos

② circular section = 6 nos.

Design of tie

diameter = $\frac{1}{4} \times \phi_{LL}$ or 6mm } greater

8mm in real field

pitch = $16 \times \phi_{LS}$, 300mm, least dimension } minimum

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$ (Pg - 34)

$A_c = (A_g - A_{sc})$

A_{sc} → 2.5% of A_g for column

Short column with without Bending

Given load = 2000 kN

f_{ck} = 20 N/mm²

f_y = 415 N/mm²

Soln factored load = 1.5 P = 1.5×2000 = 3000 kN

we have

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

$A_c = (A_g - A_{sc})$

assume p = 2.5 %

$$3000 \times 10^3 = 0.4 \times 20\left(A_g - \frac{2.5}{100}A_g\right) + 0.67 \underbrace{f_y}_{415} \frac{2.5}{100} A_g$$

$\Rightarrow A_g = \cancel{208.3725}\ 203372.5$ mm²

Adopt ~~500mm×500mm~~ 500×500 mm² column (b × D)

$$A_{sc} = \frac{2.5}{100} \times 500 \times 500$$

$= 6250$ mm²

provide 8-32mm ϕ Reinforcement

check, max steel = 4% of bD = 10,000 (OK)

lateral tie

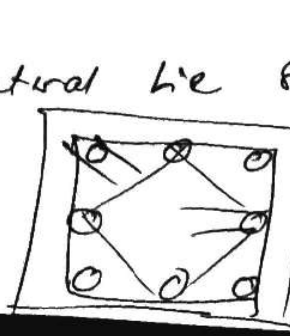
$\phi_{LL} = \frac{\phi_l}{4} = \frac{32}{4} = 8$ or 6 mm } greater

pitch = 16 × ϕ_{LL} = 512 mm

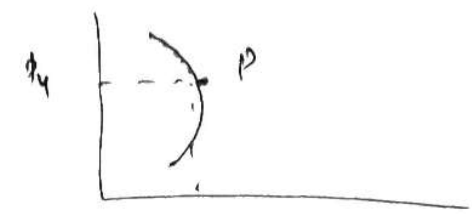
300 mm

500 mm } least

adopt lateral tie 8mm @ 300mm c/c



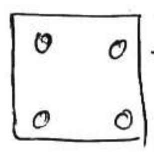
Short column with axial load and uniaxial bending



interaction diagram

Design a short column for uniaxial bending size of column = 450mm x 450mm, factored load P_u = 1000 kN, factored moment M_u = 75 kN-m, use f_{ck} = 20, f_y = 415

Soln



$d' = 40 + \phi/2 = 40 + 20/2 = 50mm$

$d'/D = \frac{50}{450} = 0.11 \simeq 0.15$ मा लेंगे

Chart 45, pg 76 for f_y = 415, d'/D = 0.15, rectangular

Now $\frac{P_u}{f_{ck} b D} = \frac{1000 \times 10^3}{20 \times 450 \times 450} = 0.247$

$\frac{M_u}{f_{ck} b D^2} = \frac{75 \times 10^6}{20 \times 450 \times 450^2} = 0.04115$

from chart $P_u/f_{ck} = 0$

But min steel = 0.8% of bD
= $1620 mm^2$

provide 8-20mm ϕ (A_{st} = 2513.27 mm^2)

Tie design
pitch design } same as previous

8

3) Design a symmetrically reinforced column 300×450 mm restrained against sway with following data.

effective length for bending parallel to larger dimension 6.1m

effective length for bending parallel to smaller dimension = 5.9 m

unsupported length = 7m

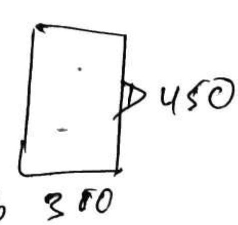
factored axial load (Pu) = 1000 kN

factored moment in the direction of larger dimension 105 kNm at top
75 kNm at bottom

factored moment in the direction of shorter dimension 40 kNm at top
30 kNm at bottom

Concrete mix M20
Reinforcement Fe 415

Sol^n $\lambda_x = \frac{l_x}{D}$



$= \frac{6.1 \times 10^3}{450} = 13.55$

> 12

$\left(\begin{array}{c} 300 \times 450 \\ b \times D \end{array} \right)$

$\lambda_y = \frac{5.9 \times 10^3}{300}$

$= 19.66 > 12$

long column

Now Additional moment

$$M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

$$= \frac{1000 * 450}{2000}\left(\frac{\cancel{5.9}\ 6.1 * 10^3}{450}\right)^2$$

$$= \cancel{38.07}\ 41.34 \text{ KN-m}$$

$$M_{ay} = \frac{1000 * 300}{2000}\left(\frac{5.9 * 10^3}{300}\right)^2$$

$$= 58.016 \text{ KN-m}$$

Now addition moment can reduce by factor $k = \frac{P_{uz} - P_u}{P_{uz} - P_b}$

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

Assume $\% P = 2.5\%$

$$P_{uz} = 0.45 * \cancel{415}\ 20\left(A_g - \frac{2.5}{100}A_g\right) + 0.75 * 415\left(\frac{2.5}{100}A_g\right)$$

where $A_g = 300 * 450$

$= 2235.09$ KN

For P_b

$k_1 = 0.196$

$k_2 = 0.203$

$\frac{d'}{d} = \frac{50}{450} = \cancel{0.1428}\ 0.11 \simeq 0.15$

$$\frac{P_b}{20 * 300 * 450} = 0.196 + 0.203 * \frac{2.5}{20}$$

$\Rightarrow P_b = 597.212$ KN

Now $k = \frac{2235.09 - 1000}{2235.09 - 597.2} = 0.75411$

Now reduced Bending moment:

$$\boxed{\begin{aligned} M_x' &= 0.754 * 41.34 = 31.17 \text{ KN-m} \\ M_y' &= 0.754 * 58.016 = 43.74 \text{ KN-m} \end{aligned}}$$

Design initial moments due to double curvature

$M_{ix} = 0.6 * 105 - 0.4 * 75 = 33$ KN-m

$M_{iy} = 0.6 * 40 - 0.4 * 30 = 12$ KN-m

Again initial moment due to eccentricity

$e_x = \frac{L}{500} + \frac{D}{200} = \frac{7000}{500} + \frac{450}{200}$

$= 16.25$ mm > 20 mm

$e_y = \frac{7000}{500} + \frac{300}{200} = 15.5$ mm > 20 mm

$M_{ixe} = 1000 \text{ KN} * \left(\frac{16.25}{1000}\right)$ m

$= 16.25$ KN-m < 33 KN-m

$M_{iye} = 1000 * \left(\frac{15.5\text{ m}}{1000}\right)$

$= 15.5$ KN-m > 12 KN-m

adop $M_{ix} = 33$ KN-m

$M_{iy} = 15.5$ KN-m

design moments are

$M_x = 31.17 + 33 = 64.17$ KN-m

$M_y = 43.74 + 15.5 = 59.24$ KN-m

Now

$$\left(\frac{Mux}{M'ux}\right)^{\alpha} + \left(\frac{Muy}{M'uy}\right)^{\alpha} \leq 1$$

$\frac{d'}{D} = \frac{50}{450} = 0.11 \simeq 0.15$

let $P = 2.5\%$

$P/f_{ck} = 2.5/20 = 0.125$

$\frac{P}{f_{ck} bD} = 0.32032$

From chart 45

$\frac{Mux}{f_{ck} bD^2} = 0.15$

$\Rightarrow Mux = 182.25$ kN-m

$\frac{Muy}{f_{ck} b^2 D} = 0.15 = 121.5$ kN-m

For α_1 $\quad Pu/Puz = \frac{1000}{2235.09} = 0.4474$

	0.2	0.8	0.4474
$\alpha \rightarrow$	1.0	2.0	

$x = 1.0 + \frac{2-1}{0.8-0.2}(0.447-0.2)$

$= 1.416$

$\alpha = 1.416$

Now $\left(\frac{64.17}{182.25}\right)^{1.416} + \left(\frac{59.24}{121.5}\right)^{1.416} \leq 1$

$0.589 \leq 1$ ok

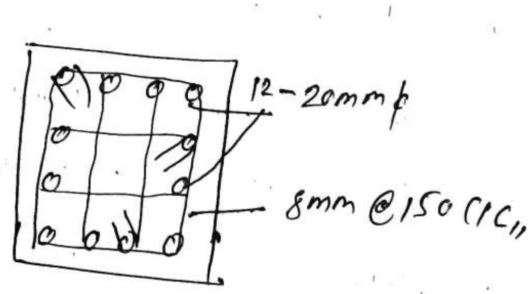
Total Area of steel $= \frac{2.5}{100} * 300 * 450$
$= 3375\ mm^2$

provide 20mm ϕ in 12 nos
Asprovided $= 12 * \pi \frac{20^2}{4}$
$= 3769\ mm^2$,

lateral tie design $\frac{\phi_L}{6} = \frac{20}{6} = 3.3$
as
6mm

pitch least of $b = 300$
$16\phi_L = 320$

provide 8mm ϕ 150 c/c



where β is the ratio of the long side to the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portion of the footing.

(5) Nominal Reinforcement

(*i*) Minimum reinforcement and spacing shall be as per requirement of solid slab.

(*ii*) The nominal reinforcement for concrete sections greater than 1 m thick shall be 360 mm^2 per m length in each direction, if it works out lesser as per requirements of solid slab.

(6) Cover to Reinforcement

A minimum clear cover of 50 mm shall be provided to the reinforcement in foundations under soil.

14.3.2. Design Steps for Isolated Rectangular Footing

Given: Load on column, safe bearing capacity of soil, grade of concrete and steel.

1. Find design constants $\frac{x_{u\max}}{d}$ and R_u for given steel and concrete grades from Table 4.2 and Table 4.3.

2. Calculate area of footing as follows:

$$A = \frac{w_c + w_f}{q_0}$$

where w_c = load on column

w_f = self wt of footing + pedestal if provided (usually considered as 10% of w_c)

q_0 = safe bearing capacity of soil

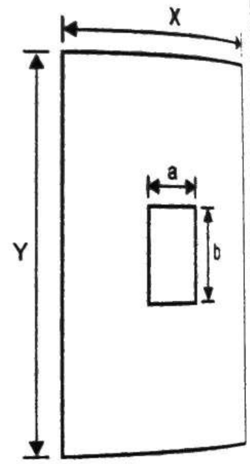


Fig. 14.8.

3. Calculate the size of footing:

(*a*) For square footing, side of footing $S = \sqrt{A}$, round off

(*b*) For rectangular footing, assume one dimension (say *X*) and calculate the other dimensions (say *Y*) as follows:

$$Y = \frac{A}{X} \text{ round off to nearest 5 or 10 cms.}$$

Alternatively, if ratio of width to length of column and footing are assumed to be similar (say *a*/*b*), then bending moment is same in both directions.

4. Calculate the soil pressure due to factored column load only, as follows:

$$P_u = \frac{1.5\, w_c}{X.Y}$$

where w_c = column load

X = shorter dimension of footing

Y = longer dimension of footing

5. Depth of footing is calculated by the following three criteria and highest value so calculated is adopted in the design:

(*a*) **By one way shear criterion:** The critical section for one way shear is taken at a distance *d* (effective depth) from the column's face (Fig. 14.9).

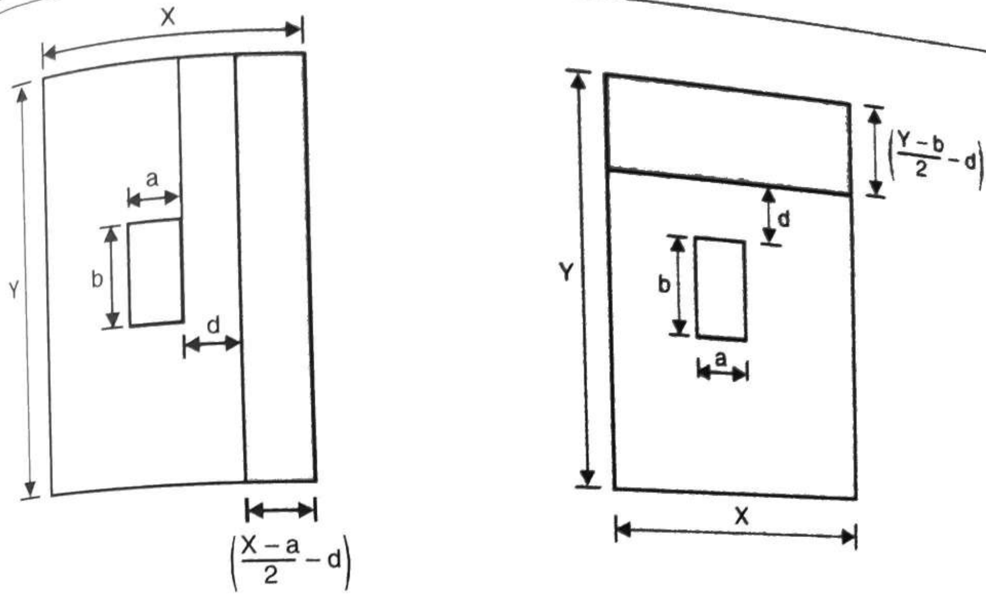


Fig. 14.9

Shear force at the critical section

$$V_u = p_u \times X \times \left(\frac{Y-b}{2}-d\right) \quad ...(i)$$

Shear force resisted by concrete $= \tau_c Xd$ ...(ii)

Equating (*i*) and (*ii*)

$$\tau_c \, Xd = p_u \times X \times \left(\frac{Y-b}{2}-d\right)$$

As exact percentage of reinforcement to be provided is not yet known, τ_c may be assumed as that corresponding to minimum reinforcement, *i.e.*, 0.2%. For M20, this value may be taken as 0.32 N/mm^2.

(*b*) **By two way shear criterion:** The critical section for two way shear or punching shear as it is commonly called, is at a distance *d*/2 from the face of the column.

Referring to Fig. 14.10, perimeter of critical section

$$= 2\left(a+\frac{d}{2}+\frac{d}{2}+b+\frac{d}{2}+\frac{d}{2}\right)$$

$$= 2(a+b+2d)$$

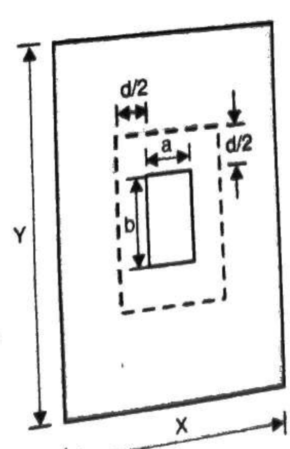


Fig. 14.10.

Area of concrete resisting punching shear

$$A = 2(a+b+2d) \times d$$

Punching shear on the critical section

$$= p_u\,(XY-(a+d)\times(b+d)) \quad ...(iii)$$

Punching shear resisted by the section

$$= \tau_c \times A \quad ...(iv)$$

$$= \tau_c \times 2\,(a+b+2d)\times d$$

where $\tau_c = 0.25\sqrt{f_{ck}}$

by equating the two expressions, (*iii*) and (*iv*) we can calculate the depth of footing.

(*c*) **By bending moment criterion:** The critical section for bending moment is shown in Fig. 14.11.

$$p_u\left(\frac{X-a}{2}\right)\left(\frac{X-a}{4}\right) = \frac{p_u}{8}(X-a)^2 \quad ...(v)$$

$$\text{B.M. in } Y \text{ direction} = \frac{p_u}{8}(Y-b)^2 \quad ...(vi)$$

Moment of resistance of section

$$= 0.36\, f_{ck} \cdot \frac{x_{u\lim}}{d}\left(1-\frac{0.42\, x_{u\lim}}{d}\right) Yd^2$$

Equating the (*v*) and (*vi*) with the moment of resistance we get the value of *d*.

The highest value of depth as obtained in steps (*a*), (*b*) and (*c*) above shall be adopted as effective depth of the footing.

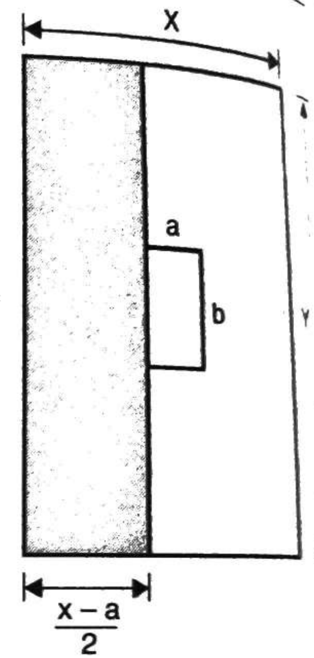


Fig. 14.11.

6. Determine the area of reinforcement required by following equation.

$$M_u = 0.87\, f_y\, A_{st}\, d\left(1-\frac{A_{st}\, f_y}{Xd\, f_{ck}}\right)$$

The reinforcement area so calculated should not be less than the minimum reinforcement and distributed as per IS code provisions.

NOTE:	**For design of square footing, follow the above mentioned procedure and substitute *X* = *Y* and *a* = *b***

SOLVED EXAMPLES

(B)

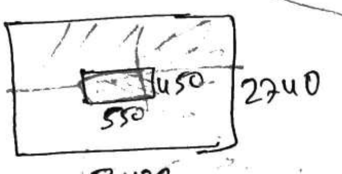
4) Design a rectangular footing which has 550mm × 450mm size of column with 8-25mm longitudinal bars, and carrying a load of 900kN, Assume soil with a safe bearing capacity of 120kN/m² at depth of 1.5m below ground. Assume M20 grade concrete and Fe415.

⇒ Soln

Step 1

App Area of footing $= \frac{900}{120} = 7.5\,m^2$

Ratio of L:B of footing $= \frac{550}{450} = \frac{5.5}{4.5} = \frac{1.25}{1}$



$(1.25x)(x) = 7.5$

$\Rightarrow x = 2.449$ (B)

$1.25x = 3.061$ (L)

self wt of foundation $= 20 * 1.5 * 7.5$

$= 225\,kN$

True Area foundation (Actual Area) $= \frac{900+225}{120}$

$= 9.375$

$(1.25x)(x) = 9.375$

$\Rightarrow L = 1.25x = 3.42\,m$

$B = x = 2.74\,m$

Step 2

uplift pressure $= \frac{1.5 * 900}{LB} = \frac{1.5 * 900}{9.375}$

$= 144\,kN/m^2$

$BM = \frac{wl^2}{2} = \frac{P * B}{2}\left(\frac{L - 550}{2}\right)^2$

$= \frac{144 * 2.740}{2}\left(\frac{3.420 - 0.550}{2}\right)^2$

$= 406.24\,kN\text{-}mm$

Step 3

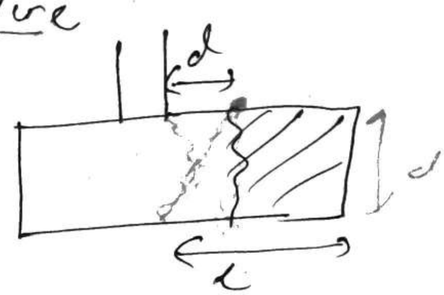
(a) depth calculation

$$406.24 \times 10^6 = 0.138 \underset{\geq 20}{f_{ck}} \underset{\downarrow \; 2740}{b} d^2$$

$$\Rightarrow d = 231.77 \text{ mm}$$

(b) one way shear failure

According to IS code
One way shear acts
d distance from column



Shear force at section

$X\text{-}X = V_u = p * B * (l - d)$

$= 144 * 2.740 \left(\frac{3.420 - 0.55}{2} - d\right)$

$= 394.56 \, (1.435 - d)$

Assume % p = 0.15 %

$\tau_c = 0.28$

Shear force balance by $\tau_c = 0.28 * b * d$

$= \cancel{0.28 * 2.744 * d}$ 2740

$= 0.28 * 10^3 (kN/m^2) * 2.740 * d$

$= 767.2 \, d$

Now

$394.56 (1.435 - d) = 767.2 \, d$

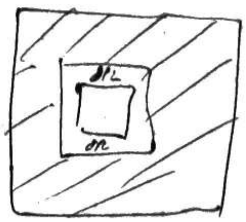
$\Rightarrow d = 0.487 \text{ m}$

$= 487.3 \text{ mm}$

(c) by two way shear / Punching (15)

According to IS code two way shear act d/2 distance from column

$V_u = p * (\text{Area of Shaded})$



$= 144 * (3.420 * 2.740 - (0.450 + d)(0.55 + d))$

modified shear stress $\tau' = k * \tau_c$

$= (0.5 + \beta_c) * \tau_c$

$= \left(0.5 + \frac{450}{500}\right) * 0.25\sqrt{20}$

$= 1.5652 \ N/mm^2$

Permissible shear force $= 1.5652 * b_o d$

$= 1.5652 * 10^3 \ KN/m^2 * 2[0.55 + d + 0.45 + d] * d$

equating we get

$144(3.420 * 2.740 - (0.45 + d)(0.550 + d))$

$= 1.5652 * 10^3 * 2(0.55 + 0.45 + 2d) * d$

on solving

$\Rightarrow d = 0.2644$

$= \underline{264.4 \ mm}$

adopt maximum depth $d = 482 m$

$\simeq 500 mm$

$D = 500 + 50 + \frac{20}{2}$

$= 560 \ mm$

Area of steel

Along x

$406.24 * 10^6 = 0.87 * 415 * Ast \left(500 - \frac{415 * Ast}{20 * 3420}\right)$

$Ast = 2315.37 \ mm^2$

provide 12mm ϕ 113.1 mm^2 reinforcement bar

$Spacing = \frac{3420 * 113.1}{2315.37} = 167.0$

$\simeq 150mm \ c/c$

Along y

$\cancel{406.24} \ My * 10^6 = 0.87 * 415 \ Ast \left(500 - \frac{415 \ Ast}{20 * 2740}\right)$

$Ast = 2332.70$

provide ~~Ast~~ 12mm ϕ

$Spacing = \frac{2740 * 113.1}{2332.0} = 132$

$\simeq 130 \ c/c$

development length

$ld = 1.3 \frac{M_1}{V} + lo$

available length $= 3420 - 50 - 550$

$= 2820 \ mm$

$ld = \frac{0.87 fy \ \phi_{bar}}{4 \tau_{bd}}$

$= \frac{0.87 * 415 * 12}{4 * 1.2}$

$= 902.62 < 2820 \ mm$

OK

load transfer

nominal bearing stress on column = $\frac{1.5 * 900 * 10^3}{550 * 450}$

= 5.54

allowable comp strength of concrete = ~~0.45fck~~ $0.446 \times f_{ck}$

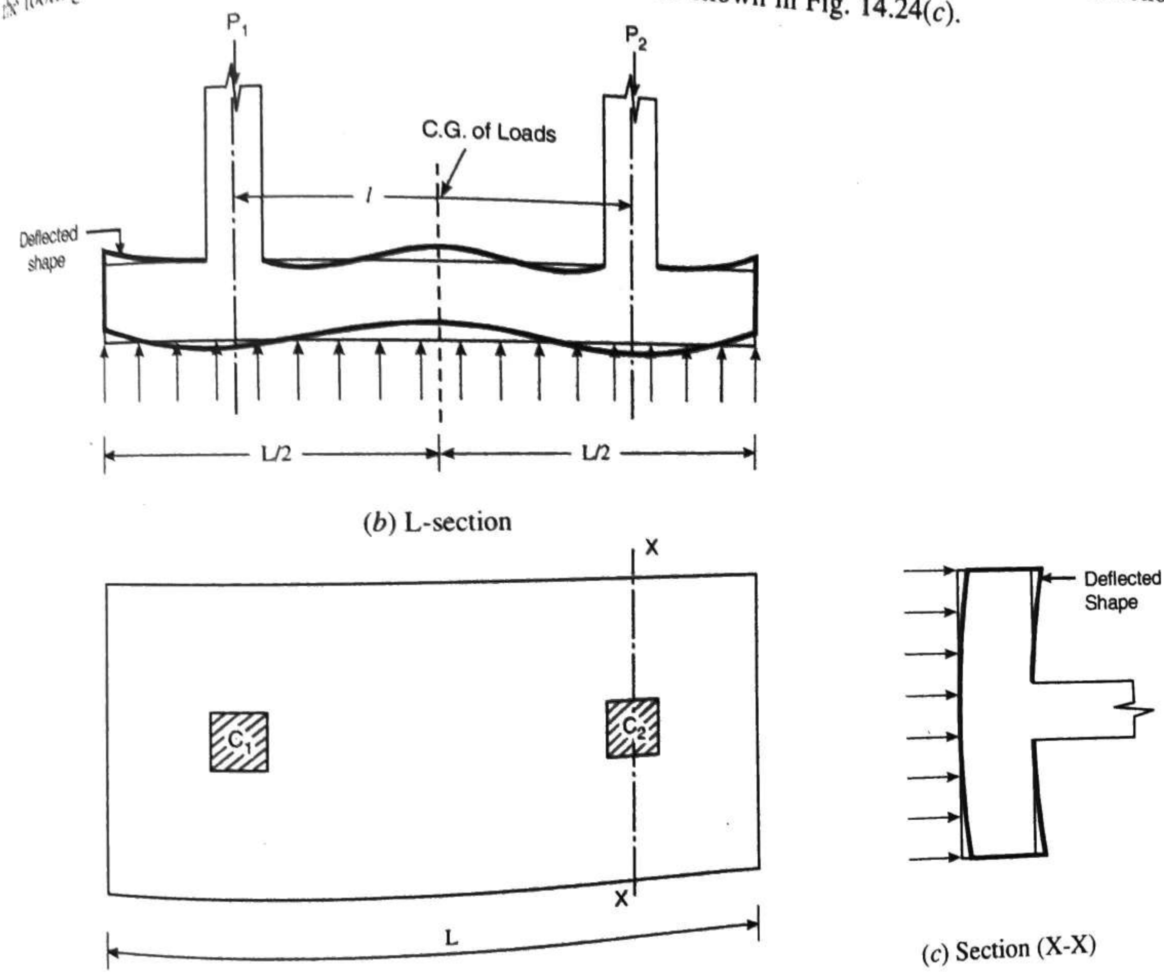
= 8.904 > 5.54

no need for dowl bars

(1) Rectangular combined footing
(2) Trapezoidal combined footing
(3) Beam and slab type footing
(4) Strap footing.

14.6. ANALYSIS OF A COMBINED FOOTING

Consider the combined footing shown in Fig. 14.24. The footing carries an upward base pressure and acts like a beam supported on two columns in the longitudinal direction. In the transverse direction the footing acts like a cantilever supported at columns as shown in Fig. 14.24(*c*).



(*b*) L-section

(*a*) Plan

(*c*) Section (X-X)

Fig. 14.24. Rectangular combined footing

Let the column C_1 carries a load P_1 and the column C_2 carries a load P_2. The footing is proportioned in such a way that the CG of the load system coincides with the CG of the footing.

Distance of the resultant load $(P_1 + P_2)$ from column C_1 : (Refer Fig. 14.25)

$$\bar{x} = \frac{P_2 . l}{P_1 + P_2}$$

∴ The point G should be the CG of the footing

$$x' + \bar{x} = \frac{L}{2}$$

$$\Rightarrow \quad L = 2(x' + \bar{x})$$

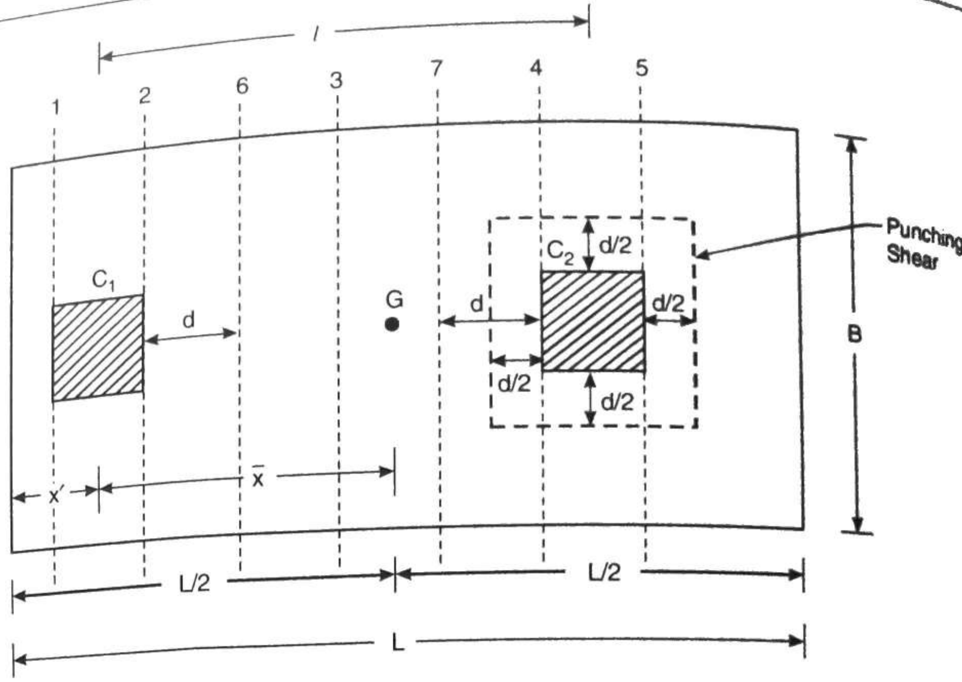


Fig. 14.25.

In the longitudinal direction, the footing is designed by considering it to be a beam of width B and the maximum values of bending moment and shear force is calculated at the critical sections shown in Fig. 14.25. In the transverse direction, the footing is considered as cantilever projecting from the column supports. In this case, the bending moment is calculated at the face of the column.

For designing a combined footing: (Refer Fig. 14.25)

- The critical sections for moment are at the face of the column and near the mid span (Sections 1-1, 2-2, 3-3, 4-4, 5-5).
- The critical section for one way shear is at a distance '*d*' from the face of the column (Section 6-6, 7-7).
- The heavier column is checked for punching shear at a distance *d*/2 around the periphery of the column.

The following example will illustrate the design of combined footing:

Example 14.4. Design a combined footing for two columns 500 mm × 500 mm each, 5 m apart carrying a load of 1600 kN. Available width restriction is 2.4 m. The safe bearing capacity is 200 kN/m². Use M25 concrete and Fe 415 steel. [MDU 201...]

Solution. Column size: 500 mm × 500 mm

$$P_1 = P_2 = 1600 \text{ kN}$$
$$l = 5 \text{ m}$$
$$q_0 = 200 \text{ kN/m}^2$$

- **Area of footing**

$$\text{Total column load} = 2 \times 1600 = 3200 \text{ kN}$$

Assuming self weight of footing as 10% of total weight = 320 kN

$$\text{Total load, } W = 3520 \text{ kN}$$

$$\text{Area of footing required} = \frac{W}{q_0} = \frac{3520}{200} = 17.6 \text{ m}^2$$

Available width $= 2.4$ m

$$\text{Length} = \frac{17.6}{2.4} = 7.33 \text{ m}$$

Hence adopting a length of 7.5 m such that the CG of the load system coincides with the CG of the footing as shown in Fig. 14.26.

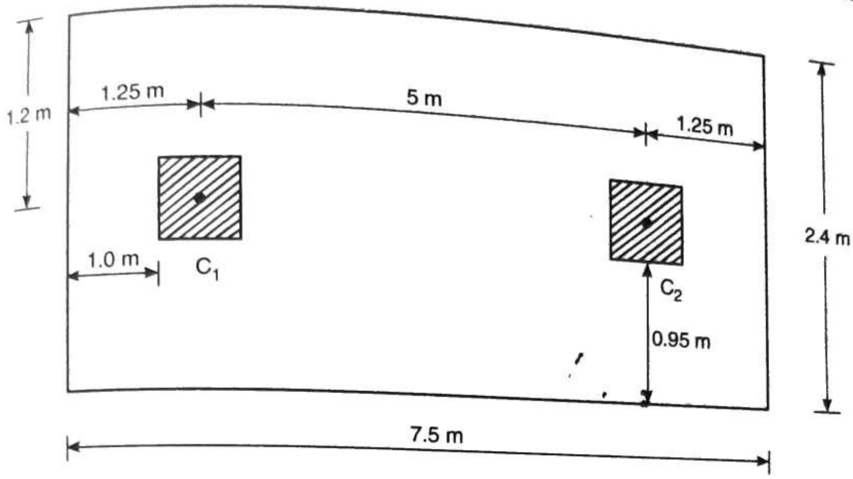


Fig. 14.26

$$\text{Upward soil pressure} = \frac{\text{Total load}}{\text{Area of footing}}$$

$$= \frac{1600 \times 2}{7.5 \times 2.4}$$

$$= 177.8 \text{ kN/m}^2$$

Factored soil pressure $= 1.5 \times 177.8 = 266.7$ kN/m^2

Design in the longitudinal direction

Upward soil pressure per unit length $= 266.7 \times 2.4 = 640$ kN/m

1. Calculation of maximum bending moment and shear force (Fig. 14.27)

***Shear Force Distribution*:**

Shear force at $C_1 = -640 \times 1.25 = -800$ kN [just left of centre]

Shear force at $C_1 = -800 + 1.5 \times 1600 = 1600$ kN [just right of centre]

Similarly,

Shear force at $C_2 = +800$ kN [just right of centre]

Shear force at $C_2 = +800 - 1600 \times 1.5 = -1600$ kN [just left of centre]

***Bending Moment Distribution*:**

$$\text{Bending moment at } C_1 \text{ or } C_2 = 640 \times \frac{1.25^2}{2} = 500 \text{ kNm}$$

$$\text{Bending moment at midspan} = 640 \times \frac{3.75^2}{2} - 2400 \times 2.5$$

$$= -1500 \text{ kNm}$$

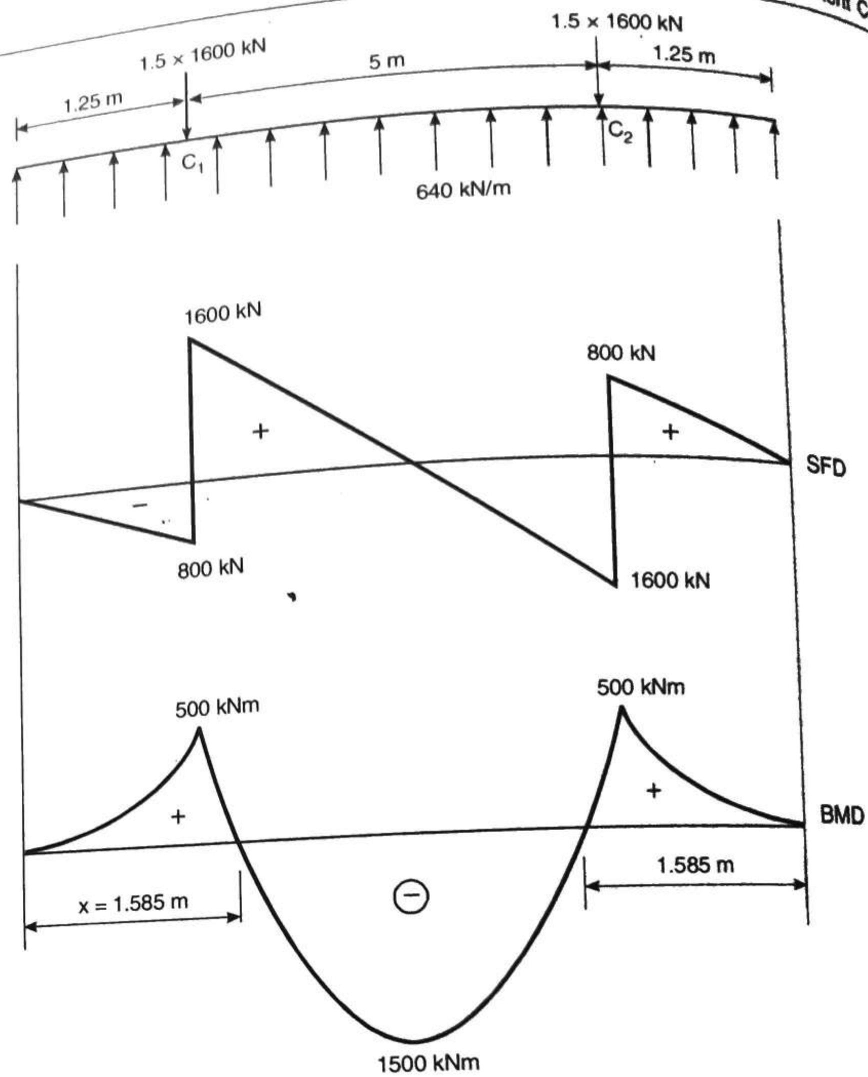


Fig. 14.27

Point of zero moment or contraflexure:

$$M_x = 640 \cdot \frac{x^2}{2} - 2400(x - 1.25) = 0$$

$$x^2 - 7.5x + 9.375 = 0$$

which gives $\qquad x = 1.585$ m

(*ii*) Depth of footing

(*a*) *From BM consideration:*

$$M_u = 1500 \times 10^6 \text{ Nmm}$$

$$d_{\text{reqd}} = \sqrt{\frac{M_u}{R_u \cdot b}}$$

For M25 and Fe 415 steel,

$$R_u = 0.36 f_{ck} \frac{x_{u\max}}{d}\left[1 - 0.416 \frac{x_{u\max}}{d}\right]$$

$$= 0.36 \times 25 \times 0.48[1 - 0.416 \times 0.48]$$

$$\boldsymbol{R_u = 3.45}$$

$$d_{reqd} = \sqrt{\frac{1500 \times 10^6}{3.45 \times 2400}}$$

$$= 425 \text{ mm}$$

(b) From one way shear consideration: (at a distance '*d*' from face of column)

$$V_u = \left[1600 - 640\left[0.25 + \frac{d}{1000}\right]\right] \times 10^3 \text{ N}$$

Assuming 0.2% steel, $\tau_c = 0.32$ N/mm^2 from Table 5.1

$$\frac{V_u}{bd} < 0.32$$

$$\frac{\left[2400 - 640\left[0.25 + \frac{d}{1000}\right]\right] \times 10^3}{2400 \times d} < 0.32$$

On solving we get, $d = 1023$ mm

Hence adopting total depth of 1100 mm and effective depth, $d = 1100 - 60 = 1040$ mm.

(c) **Checking depth for two way shear**

The critical section for two way or punching shear is at a distance '$\frac{d}{2}$' from the face of column.

$\therefore$ Area resisting punching shear $= b_0.d = 4(500 + 1040) + 1040$

Shear force at critical section $= 2400 - 266.7(0.5 + 1.04)^2$

$$V_u = 1768 \text{ kN}$$

$$\tau_v = \frac{V_u}{b_0.d} = \frac{1768 \times 10^3}{4(500 + 1040) \times 1040}$$

$$\tau_v = 0.28 \text{ N/mm}^2$$

Permanent shear stress, $\tau_c = 0.25\sqrt{f_{ck}}$

$$= 0.25\sqrt{25}$$

$\tau_c = 1.25$ N/mm$^2 > \tau_v$. Hence OK

Longitudinal reinforcement

(i) Negative moment or hogging moment reinforcement

$$M_u = 1500 \text{ kNm}$$

$$1500 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1040\left[1 - \frac{415 A_{st}}{25 \times 2400 \times 1040}\right]$$

On solving we get

$$A_{st} = 4052 \text{ mm}^2$$

Using 16 mm diameter bars,

$$\text{Spacing required} = \frac{201 \times 2400}{4052}$$

$$= 119 \text{ mm}$$

$$\text{Number of bars} = \frac{4052}{201} \text{ @ } 21 \text{ bars}$$

or

Hence provide 16 mm diameter bars @ 110 mm c/c or 21 bars at top as hogging moment reinforcement.

$$A_{st\ min} = 0.12\% \text{ of X-sectional area}$$

$$= \frac{0.12}{100} \times 2400 \times 1100$$

$$= 3168 \text{ mm}^2$$

$$\text{Number of 16 mm dia. bars} = \frac{3168}{201} \cong 16 \text{ bars}$$

Hence out of 21 bars 5 bars can be curtailed at the point of contraflexure, say 1.6 m from both edges.

$$L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.92} = 753 \text{ mm. Hence OK}$$

(*ii*) Positive Moment Reinforcement

$$M_u = 500 \text{ kNm}$$

$$500 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1040 \left(1 - \frac{415 A_{st}}{25 \times 2400 \times 1040}\right)$$

$$A_{st} = 1338 \text{ mm}^2 < A_{st\ min}$$

Hence providing 16-16 mm diameter bars under columns C_1 and C_2 as +ve moment reinforcement.

■ **Transverse reinforcement**

In the transverse direction, the footing is designed as cantilever supported on the columns. The transverse reinforcement is provided under each column within a band having a width equal to the width of the column plus two times the effective depth of foundation. (Refer Fig. 14.28)

Bandwidth under column C_1 or C_2 = 0.5 + 1.04 + 1.0

= 2.54 m [on the outer side only 1.0 m length is available]

$$\text{Upward pressure} = \frac{1.5 \times 1600}{2.4} = 1000 \text{ kN/m}$$

Bending moment at the face of the column in transverse direction:

$$= \frac{1000 \times 0.95^2}{2}$$

$$= 451.25 \text{ kNm}$$

Hence providing $A_{st\ min} = \dfrac{0.12}{100} \times 2540 \times 1100$

$$= 3353 \text{ mm}^2$$

Spacing of 16 mm diameter bars

$$= \frac{201 \times 2540}{3353}$$

$$= 152 \text{ mm}$$

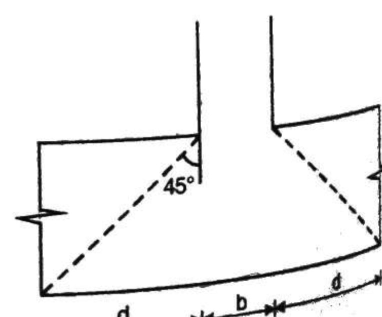
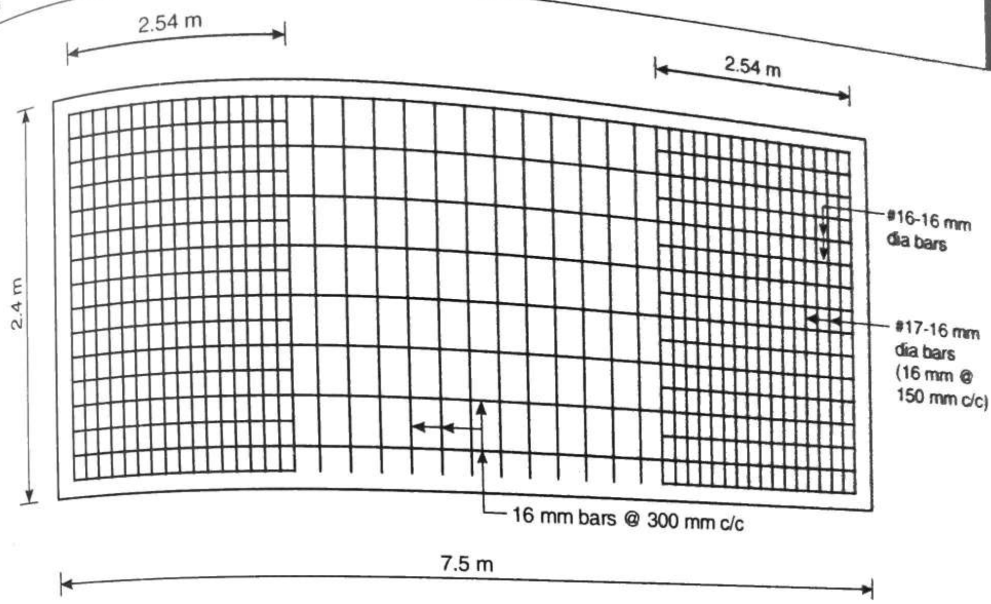
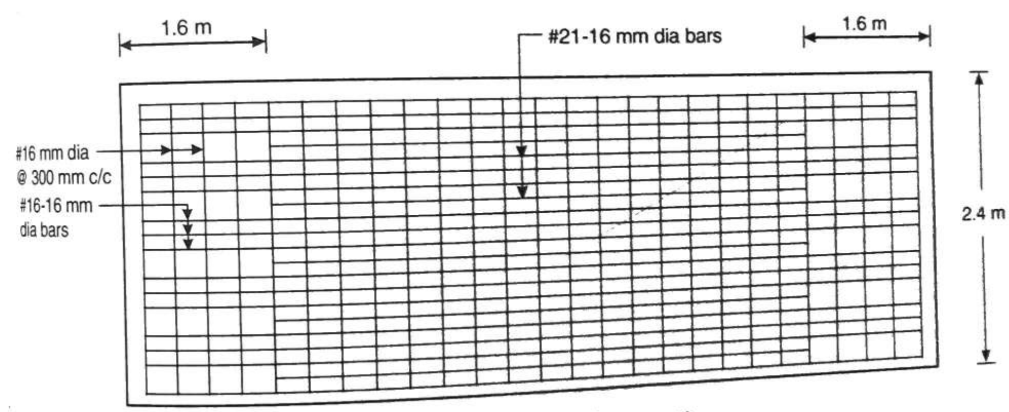


Fig. 14.28

Hence provide **16 mm diameter bars** @ 150 mm c/c (17 bars) under columns C_1 and C_2 in the width 2.54 m and in rest of the portion 16 mm bars @ 300 mm c/c.



Plan of Bottom reinforcement



Plan of Top reinforcement

Fig. 14.29

Example 14.5. Design a combined footing for two columns, C_1 and C_2, 400 mm × 400 mm and 500 mm × 500 mm in size and carrying 500 kN and 800 kN of load respectively. The smaller column is 0.4 away from the property line. The columns are 4 apart the bearing capacity of the soil is 140 kN/m^2. Use M20 concrete and Fe 415 steel.

Solution. Given:

$$P_1 = 500 \text{ kN}, P_2 = 800 \text{ kN}$$
$$l = 4.0 \text{ m}$$
$$q_0 = 4.0 \text{ m}$$
$$x' = 0.4 \text{ m}$$
$$f_y = 415 \text{ N/mm}^2, f_{ck} = 20 \text{ N/mm}^2$$

■ **Position of resultant load ($\bar{x}$) from column C_1**

$$\bar{x} = \frac{P_2 . l}{(P_1 + P_2)} = \frac{600 \times 4}{500 + 800}$$

$$\bar{x} = 2.46 \text{ m}$$

$$\text{Length of footing} = 2(\bar{x} + x') = 2(2.46 + 0.4)$$

$$l = 5.72 \text{ m}$$

■ **Width of footing (*B*)**

$$\text{Assuming self weight of footing} = 10\% \text{ of total load}$$

$$= \frac{10}{100}(500 + 800) = 130 \text{ kN}$$

$$\text{Total load } (P) = 1300 + 130 = 1430 \text{ kN}$$

$$\text{Area of footing} = \frac{P}{q_0} = \frac{1430}{140} = 10.21 \text{ m}^2$$

$$\text{Width of footing} = \frac{10.21}{5.72} = 1.78 \text{ m}$$

Adopt a width of 2.0 m.

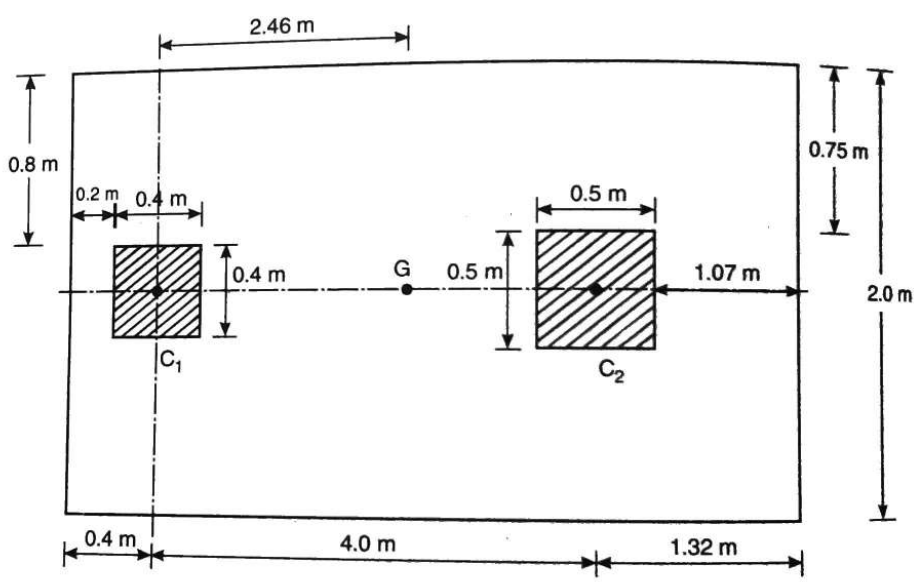


Fig. 14.30. Plan of Footing

■ $$\text{Upward soil pressure} = \frac{\text{Total load}}{\text{Area of footing}} = \frac{1300}{5.72 \times 2}$$

$$= 113.64 \text{ kN/m}^2$$

$$\text{Factored pressure} = 1.5 \times 113.64$$

$$= 170.5 \text{ kN/m}^2$$

Upward soil pressure per unit length

$$P_0 = 170.5 \times 2 = 341 \text{ kN/m}$$

■ **Calculation of Moments and Shear Forces:** The Fig. 14.31 shows the loading, SFD and BMD for the slab footing.

$$\text{S.F. at } C_2 = +341 \times 1.32 = +450.12 \text{ kN just right of } C_2$$

$$= +450.12 - 1.5 \times 800$$

$$= -749.88 \text{ kN just left of } C_2$$

S.F. at C_1 $= -341 \times 0.4 = 136.4$ kN just left of C_1

$= -136.4 + 1.5 \times 500 = +613.6$ kN just right of C_1

Point of zero shear force from column C_2

$$\frac{749.88}{x} = \frac{613.6}{4 - x}$$

$$x = 2.2 \text{ m}$$

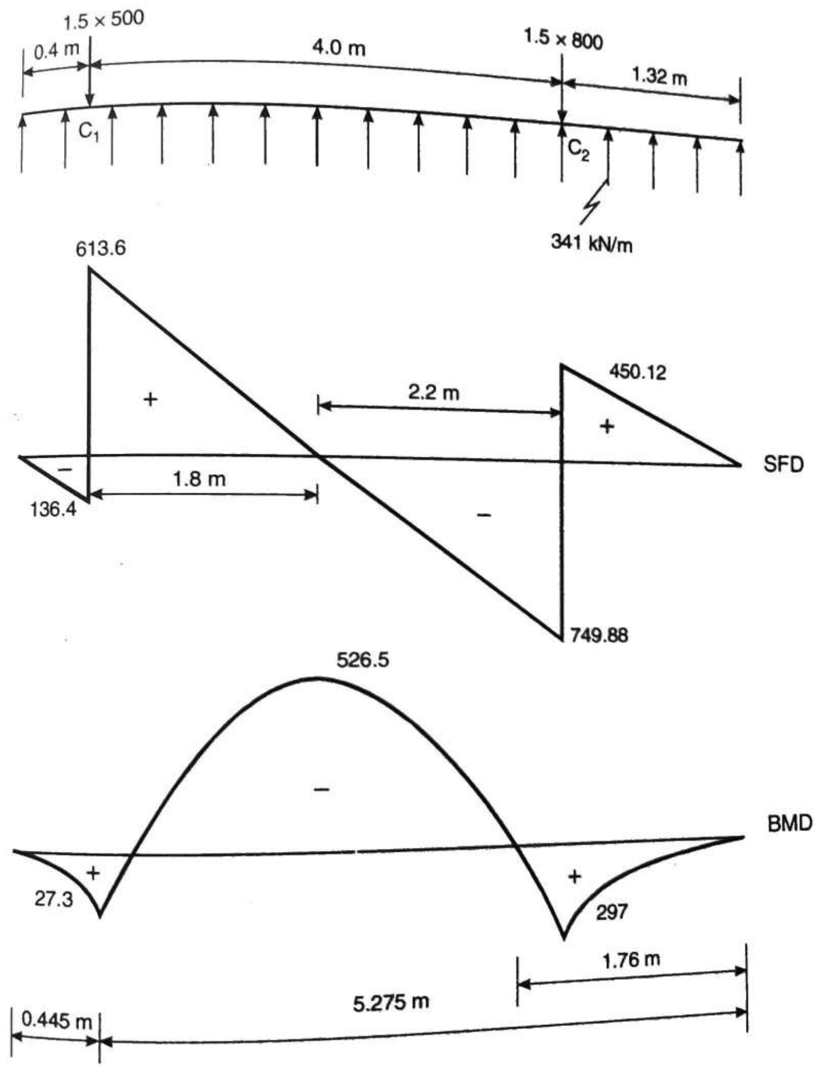


Fig. 14.31.

BMD:

$$\text{Moment at } C_1 = 341 \times \frac{0.4^2}{2} = 27.3 \text{ kNm}$$

$$\text{Moment at } C_2 = 341 \times \frac{1.32^2}{2} = 297 \text{ kNm}$$

Maximum moment occurs at the point of minimum (zero) shear force.

$$M_{max} = 341 \times \frac{3.52^2}{2} - 1.5 \times 800 \times 2.2$$

$$= -526.5 \text{ kNm}$$

Points of zero moment or contraflexure.

$$M_x = 341\frac{(x)^2}{2} - 1.5\times 800(x-1.32)$$

$$M_x = 0$$

$$170.5x^2 - 1200x + 1584 = 0$$

On solving, we get

$$x = 1.76 \text{ m and } 5.275$$

■ **Depth required from moment consideration**

$$M_u = 526.5\times 10^6 \text{ Nmm}$$

$$d_{reqd} = \sqrt{\frac{526.5\times 10^6}{2.76\times 2000}}$$

$$= 310 \text{ mm}$$

■ **One way shear at column C_2 (heavier load)**

Shear at distance 'd' from face of column

$$V_u = \left(749.88 - 341\left(0.25+\frac{d}{1000}\right)\right)\times 10^3 \text{ N}$$

Assuming percentage of tensile steel as 0.2%

$$\therefore \quad \tau_c = 0.32 \text{ N/mm}^2 \text{ from Table 5.1}$$

$$\frac{V_u}{bd} < 0.32$$

$$\therefore \quad \frac{664.63\times 10^3 - 341d}{2000\times d} < 0.32$$

$$\therefore \quad d > \frac{664.63\times 10^3}{981}$$

$$d > 678 \text{ mm}$$

Hence adopting a total depth of 750 mm and effective depth as 690 mm and checking for two way shear

■ **Two way shear**

The critical section is at $\frac{d}{2}$ from the face of the column.

(*a*) Column C_2

Area resisting punching shear $= b_0.d = 4(500+690)\times 690$

Shear force at $d/2$

$$V_u = 1200 - 170.5\,(0.5+0.69)^2$$

$$V_u = 959.3 \text{ kN}$$

$$\tau_v = \frac{V_u}{b_0 d} = \frac{959.3\times 10^3}{4(500+690)\times 690}$$

$$\tau_v = 0.29 \text{ N/mm}^2$$

$$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20}$$
$$= 1.11 \text{ N/mm}^2 > \tau_v. \text{ Hence safe.}$$

(b) Column C_1

Here

$$b_0 = 2(400 + 690) + 2\left(400 + \frac{690}{2} + 200\right)$$
$$b_0 = 4070 \text{ mm}$$
$$V_u = 750 - 170.5(0.4 + 0.69)\left(0.4 + \frac{0.69}{2} + 0.2\right)$$
$$V_u = 575 \text{ kN}$$
$$\tau_v = \frac{575 \times 10^3}{4070 \times 690}$$
$$= 0.20 \text{ N/mm}^2 < \tau_c$$

Hence safe.

Longitudinal reinforcement

(1) Negative moment reinforcement

$$M_u = 526.5 \text{ kNm}$$
$$M_u = 0.87 f_y A_{st}\, d\left(1 - \frac{A_{st} f_y}{f_{ck} bd}\right)$$
$$526.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times 690\left(1 - \frac{415 A_{st}}{20 \times 2000 \times 690}\right)$$

Area of steel required, $A_{st} = 2186 \text{ mm}^2$

Choosing 16 mm diameter bars

$$A_\phi = 201 \text{ mm}^2$$
$$\text{Spacing required} = \frac{201 \times 2000}{2186} = 183 \text{ mm}$$

Hence provide #16@ 180 mm c/c or 11 number of bars at top near the mid span

Development length for 16 mm bars

$$= 47\phi = 47 \times 16$$
$$= 752 \text{ mm (available on both sides of section)}$$

Out of these 11 bars 5 bars can be curtailed after the outer edge of the each column.

$$A_{st\,\min} = 0.12\% = \frac{0.12}{100} \times 2000 \times 750$$
$$= 1800 \text{ mm}^2 \quad \text{Hence OK}$$

(2) Positive moment reinforcement

$$M_u = 297 \text{ kNm}$$
$$297 \times 10^6 = 0.87 \times 415 \times A_{st} \times 690\left(1 - \frac{415 A_{st}}{20 \times 2000 \times 690}\right)$$

$$A_{st} = 1215 \text{ mm}^2 < A_{st\,min}$$
$$A_{st} = 1800 \text{ mm}^2$$

$\therefore$

$$\text{Spacing required} = \frac{201 \times 2000}{1800} = 223 \text{ mm}$$

Hence provide #16 @ 220 mm c/c or 10 bars at bottom under each column and out of these 5 bars can be curtailed at a distance of 800 mm from inside edge of column.

■ **Transverse reinforcement:** The transverse reinforcement is provided under each column within a band having a width equal to the width of the column plus two times the effective depth of the foundation.

$$\text{Bandwidth for column } C_1 = 0.4 + 0.69 + 0.2 \quad \text{[On the outer side only 0.2 m length is available]}$$
$$= 1.29 \text{ m}$$

$$\text{Upward pressure under column } C_1 = \frac{750}{2.0} = 375 \text{ kN/m}$$

$$\text{BM at the face of the column} = 375 \times \frac{0.8^2}{2}$$
$$= 120 \text{ kNm}$$

Area of steel required

$$120 \times 10^6 = 0.87 \times 415 \times A_{st} \times 676 \times \left(1 - \frac{415 A_{st}}{20 \times 1290 \times 676}\right)$$

$$(d = 750 - 50 - 16 - 8 = 676 \text{ mm})$$

$$A_{st} = 495 \text{ mm}^2$$
$$A_{st\,min} = \frac{0.12}{100} \times 1290 \times 750$$
$$A_{st} = 1161 \text{ mm}^2$$

Spacing of 16 mm diameter bars

$$\text{Spacing required} = \frac{201 \times 1290}{1161} = 223 \text{ mm}$$

Hence provide 16 mm diameter bars @ 220 mm c/c under column C_1 or 6 bars under column C_1 in the width 1.29 m.

■ **Transverse reinforcement**

Column C_2

$$\text{Bandwidth} = 0.5 + 0.69 + 0.69$$
$$= 1.88 \text{ m}$$

$$\text{Upward pressure} = \frac{1200}{2.0} = 600 \text{ kN/m}$$

Negative moment at face of the column C_2

$$= 600 \times \frac{0.75^2}{2} = 168.75 \text{ kNm}$$

$$A_{st\,min} = \frac{0.12 \times 1880 \times 750}{100}$$
$$= 1692 \text{ mm}^2$$

$$\text{Spacing required} = \frac{201 \times 1880}{1692}$$

$$= 223 \text{ mm} \quad \text{or} \quad 6 \text{ bars under column } C_2 \text{ in the width 1.88 m.}$$

Hence provide #16@220 mm c/c under the columns and #16@300 mm c/c in the middle portion. Figure 14.32 shows the reinforcement detailing of the footing.

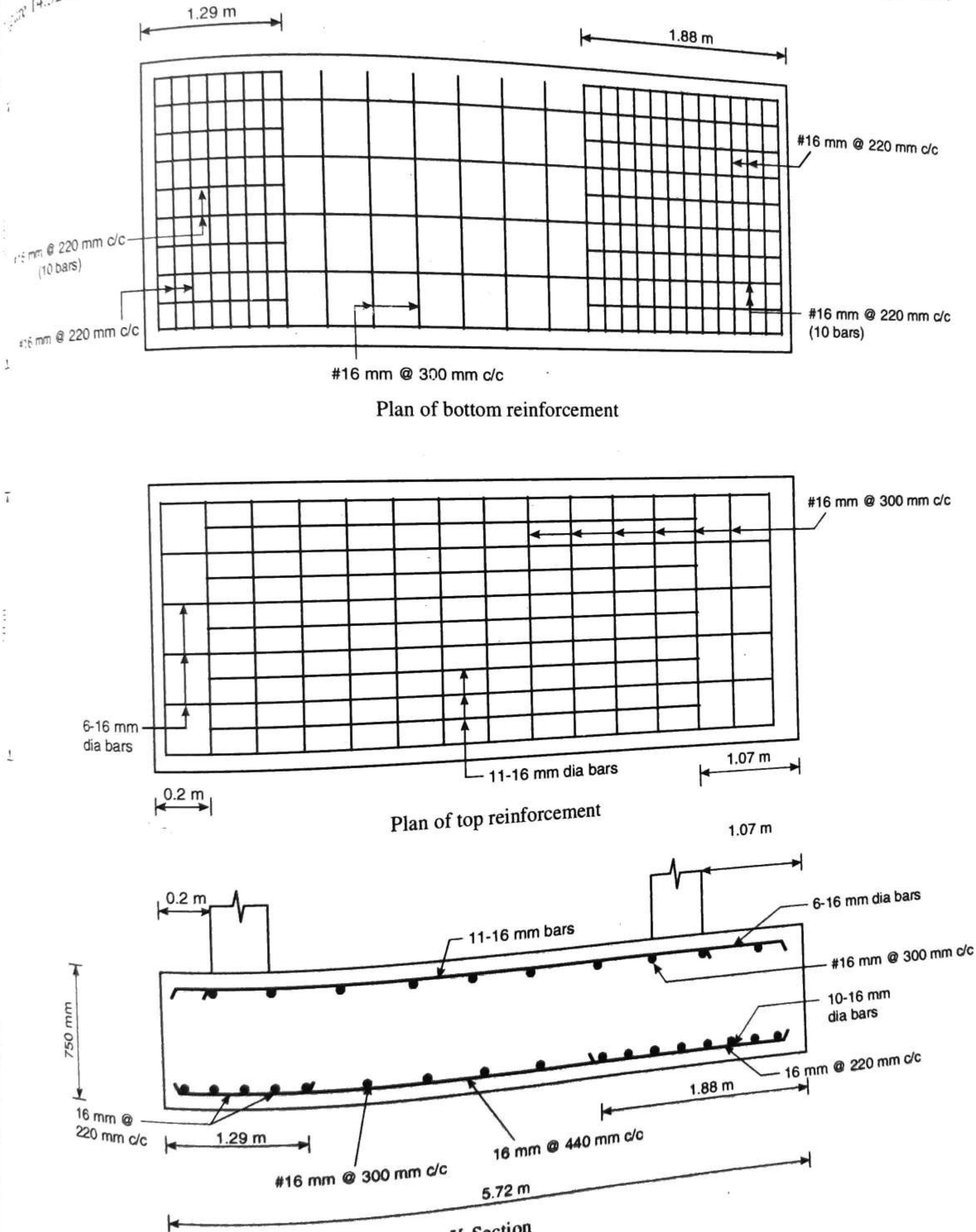
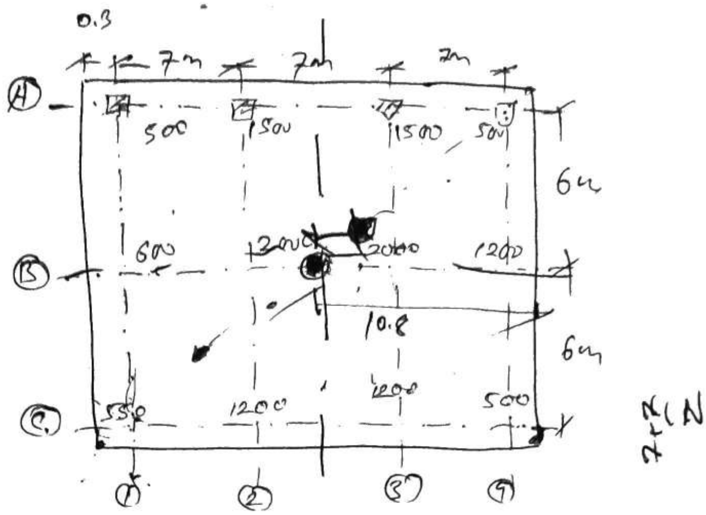


Fig. 14.32. Plan of Bottom reinforcement

Raft foundation

Design a raft foundation for a layout as fig. Net bearing capacity of soil is 65 kN/m² and column size of is 30×30 cm, use M_{30} concrete grade Fe500 steel



soln

$$\sigma = \frac{P}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

$M_y = P * e_y$

$M_x = P * e_x$

$\bar{x}$ from grid 1

$$\bar{x} = \frac{0*1650 + 7*2900 + 14*3700 + 21*2200}{1650+2900+3700+2200}$$

$= \cancel{11.32}\ 10.94$ m

$\bar{y} = \cancel{5.1674}\ 6.25$

$e_x = \frac{21}{2} - \bar{x} = -0.44$ m,

$e_y = \frac{12}{2} - \bar{y} = -0.25$ m,

$M_x = 5836.$

$M_y = 3312.5$

$A = 21.6 * 12.6 = 272.16$ m²

$I_x = \frac{21.6 * 12.6^3}{12} = 3600.67$ m³

$I_y = \frac{21.6^3 * 12.6}{12} = 10581.58$ m³

Stress at / ...

−+	++
−×	+−

$\sigma_{A-4\,comb} = \frac{13250}{27216} + \frac{33125}{10581.58} \times 108 + \frac{5830}{3600.68} \times 6.2$

$= \cancel{92.69}\ 62.27\ KN/m^2$

$\sigma_{A-1} =$ − + $= 55.5$

$\sigma_{B-4} =$ + − $= 35.10$

$\sigma_{C-1} =$ − − $= 41.86$

$\sigma_{B-1} =$ − $\times 0 = 45.30$

$\sigma_{B-4} =$ + $\times 0 = 52.05$

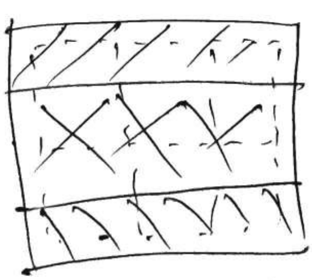
< Bearing capacity

Let divide three strip in x direction

① Beam A-A with 3.3 m width and soil pressure = 62.27 (max of 62.27 and Avg of 62.27, 55.5)

② Beam B-B with 6.0 m width soil pressure = 57.16 KN/m² (max of 55, Avg 55, 62, Avg ...)

③ Beam C-C with 3.3 m width soil pressure = 46.96 KN/m² (max 41.86 Avg 55, 41.86)



For length take maximum

Now moments: (continuous beam)

Strip A-A $= \frac{wl^2}{10} = \frac{62.27 \times 7^2}{10} = 305.12$

Strip B-B $= \frac{wl^2}{10} = \frac{57.16 \times 7^2}{10} = 280.084$

Strip C-C $= \frac{wl^2}{10} = \frac{41.86 \times 7^2}{10} = 205.14$

Now

Strip moment 1-1, 2-2, 3-3, max load = 62.27 on top

$$M = \frac{62.27 \times 7}{10} = \cancel{214.172}\ 43.96 \text{ kN-m}$$

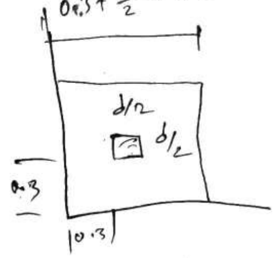
Transverse moment $= \frac{wl}{8}$

Pg 23

Depth calculation

(C-1)

$0.3 + \frac{0.3}{2} + d/2 = 0.45 + d/2$



C-1

nominal shear stress

$$\tau_v = \frac{V_u}{b_o d} \qquad b_o = (0.45 + d/2) * 2$$

$$= \frac{1.5 * 550 * 10^3}{\left(0.45 + \frac{d}{2}\right) * 2 * d} \quad \text{— (i)}$$

modified permissible shear stress $\qquad k_s = 0.5 + \frac{0.3}{0.3} = 1.5 \not> 1.0$

$$\tau_c' = k_s * \tau_c$$

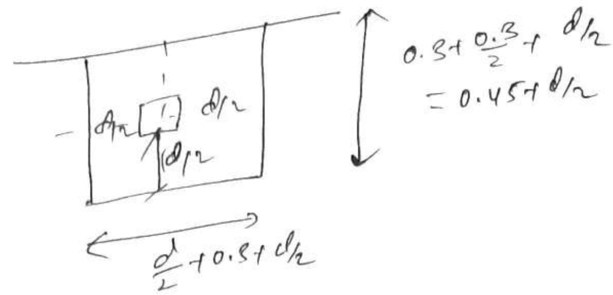
$$= \left(\cancel{0.5 + \frac{0.3}{0.3}}\right)^{1} * 0.25\sqrt{30}$$

$$= \cancel{2.054}\ 1.3693 \quad \text{— (ii)}$$

equating (i) and (ii)

$$d = \cancel{778 \text{ mm}}\ 447 \text{ mm}$$

at A-2



$= 0.3 + d$

$$\tau_v = \frac{1.5 * 1500 * 10^3}{\left[0.3 + d + 0.45 + d/2 + 0.45 + d/2\right) * d} \quad N/mm^2$$

$$= \frac{2250 \times 10^3}{(1.2 + 2d)\, d} = \frac{2250 \times 10^3}{(1200 + 2d)\, d}$$

$\tau_c' = 1.3693 \quad N/mm^2$

solving $d = 654.$ mm $\simeq 660$ mm

$D = d + 50 + 20/2 = 720$ mm

$\frac{wl}{10} =$

Steel calculation

Max^m BM $= 0.87 f_y A_s \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$

$$43.96 * 10^6 = 0.87 * 500 * A_{st} \left(660 - \frac{500\, A_{st}}{30 \times 1200} \right)$$

$A_{st} =$

min steel $= 0.12 * bd.$

Torsion

$$K_T = \frac{T}{\theta} = \frac{GJ}{L}$$

Equivalent shear

(pg 42)

$$V_e = V_u + 1.6\frac{T_u}{b}$$

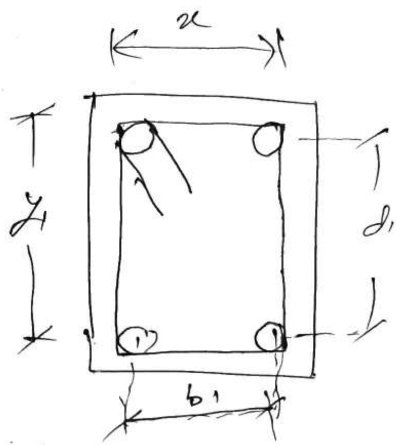
V_e = equivalent shear ~~failure~~ force

V_u = factored shear force

~~V_u~~ T_u = factored torsional moment

$$\tau_e = \frac{V_e}{bd}$$

Torsion Reinforcement



$$M_{e1} = M_u + M_t$$

$$M_t = T_u\left(\frac{1 + D/b}{1.7}\right)$$

if $M_t > M_u$

$$M_{e2} = M_t - M_u$$

Transverse Reinforcement

$$0.87 f_y \frac{A_{sv}}{x} = \frac{T_u}{b_1 d_1} + \frac{V_u}{2.5 d_1}$$

$x < x_1$

$x < (x_1 + y)/4$

$x < 300mm$

But

$$0.87 f_y \frac{A_{sv}}{x} < (\tau_e - \tau_c)\, b$$

Design a section of ring beam 50cm wide to 70cm deep subjected to a bending moment of 200 kN-m and twisting moment of 15 kN-m, shear force of 150 kN at ultimate

(pg. 42)

we have

$V_u = 150 \, kN$

$T_u = 15 \, kN$

$M_u = 200 \, kN$

$b = 500 \, mm$

$D = 700 mm$

and $f_{ck} = 20 N/mm^2$ (Assume)

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

$M_{e1} = M_u + M_t$

$M_t = T_u \left(\frac{1 + D/b}{1.7} \right)$

$= 15 \times \left(\frac{1 + 0.7/0.5}{1.7} \right)$

$= 21.17$

$d = D - 50$

$= 700 - 50$

$= 650 mm$

$M_u = 200 \, kN\text{-}m$

$M_{e1} = 221.17 \, kN\text{-}m$

for L-bar $M_{e1} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$

$A_{st} = 1007.189 \, mm^2$

provide

Shear

equivalent shear force : $V_e = V_u + 1.6 \frac{T_u}{b}$

$= 150 + 1.6 \times \frac{15}{0.5}$

$= 198$ kN

nominal shear stress $(\tau_{ve}) = \frac{V_e}{bd} = \frac{198 \times 10^3}{500 \times 650}$

$= 0.609$ N/mm

shear strength of M_{20} concrete $(\tau_c) = 0.42$ N/mm^2

$\tau_{ve} > \tau_c$ Here shear reinforcement should provide.

Let us 8mm ϕ 2-legged vertical stirrups

$\therefore A_{sv} = \frac{T_u \times s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$

$500 \times 700 = \frac{15 \times x}{}$ / min $A_{sv} = \frac{(\tau_{ve} - \tau_c) \times b x}{0.87 f_y}$

$x = 240$ mm / $\Rightarrow m = 394$

min $A_{sv} = \tau$

$x_1 = b - 25 - 25 + 8 + 8 = 466$

$y_1 = D - 25 - 25 + 8 + 8 =$

$\frac{x_1 + y_1}{4} = 283 < 300$

provide 8mm @ $240 \simeq 230$ mm c/c

(26-5-1-7) (26-5-1-3)

Side reinforcement Bars

Area of side reinforcement bar $= 0.1\%$ of bd

$$= \frac{0.1}{100} \times 500 \times 650$$

$$= 325 \text{ mm}^2$$

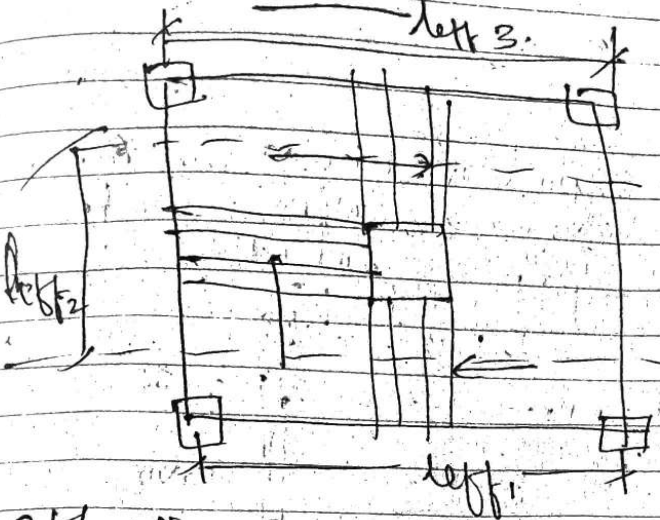
provide $2-16\text{mm}\,\phi$ $(A = 402)$

$$\frac{700 - 25 - 25}{2} = 325 \text{mm}$$

provide $4-12\text{mm}\,\phi$

Staircase

2075/01/09



1st staircase → c/c of supports

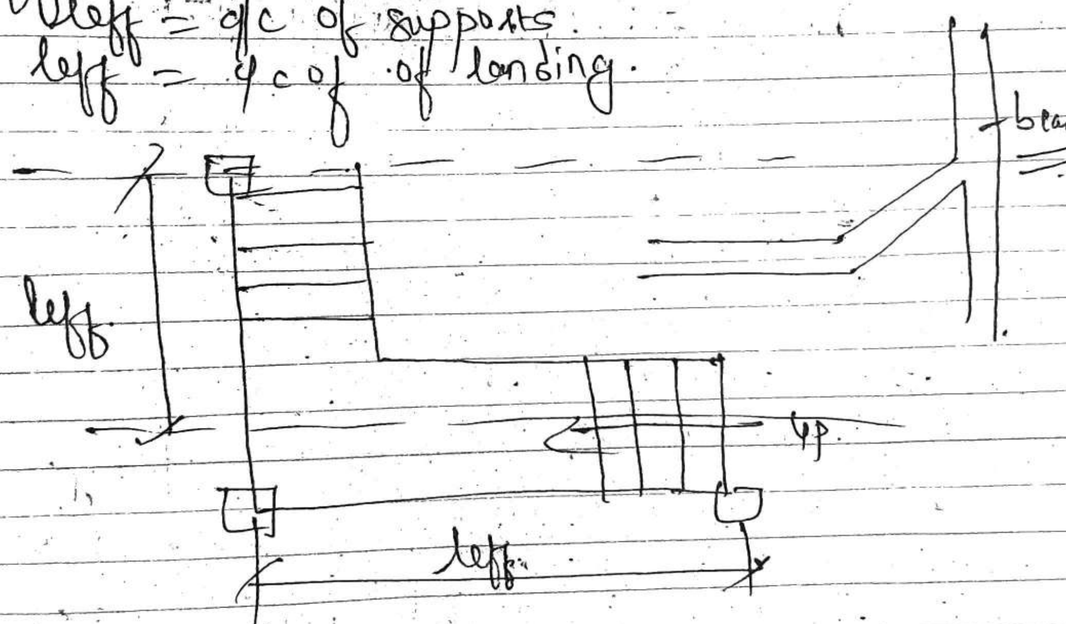
2nd stair → c/c of landing

Effective span:

① leff = c/c of supports.

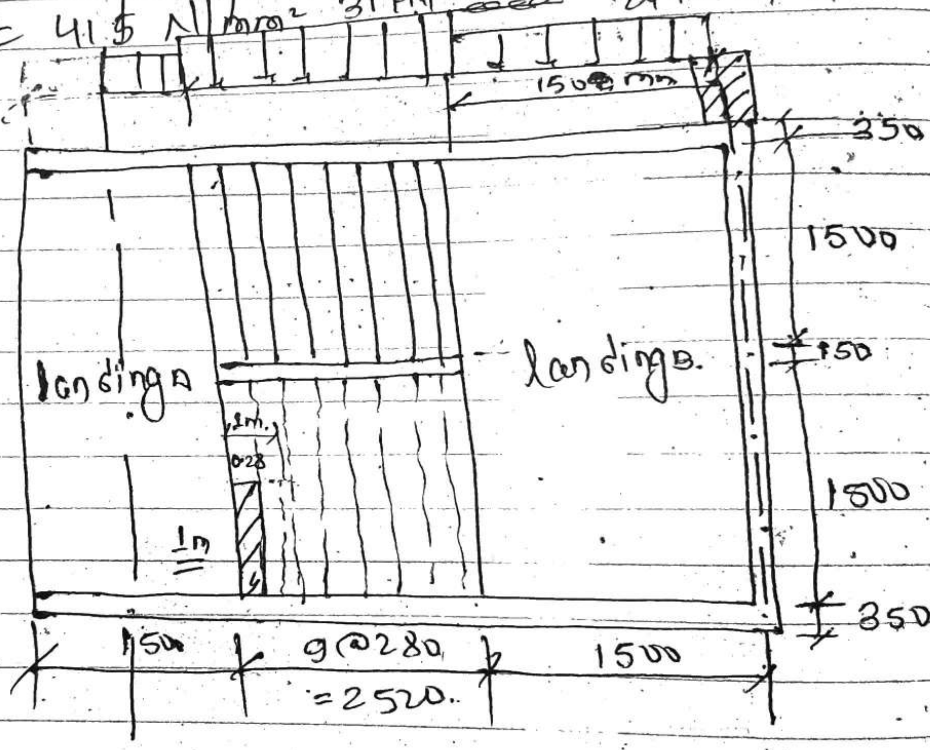
② leff = c/c of landing.

③



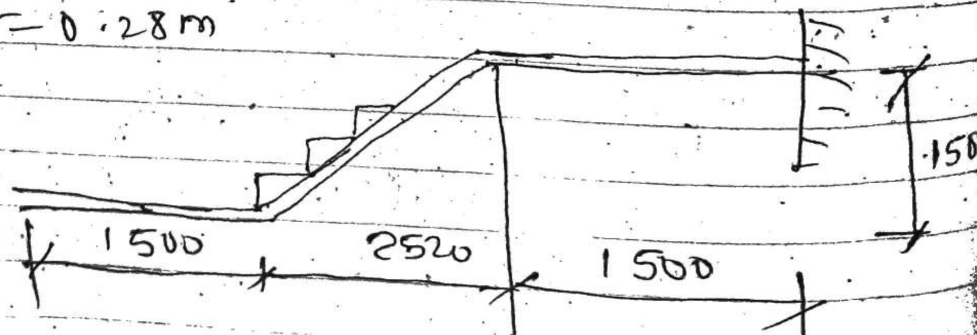
page no. 028   33.1

Design a stair flight and landing for staircase. Take superimposed load as 5 kN/m² and $f_{ck} = 20$ N/mm², $f_y = 41.5$ N/mm²



$R = 0.15$ m

$T = 0.28$ m



Step 1: Given:

soln slab thickness = $\frac{l_{eff}}{28}$ (preliminary design)

we have; f_{ck} = 20 N/mm²

f_y = 415 N/mm²

let, overall thickness of waist slab = 200 mm,

Step 2: Load and moment calculation flight:

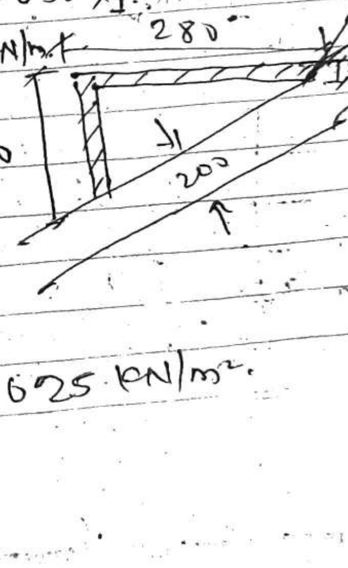
Step section = $\frac{1}{2} \times R \times T = \frac{1}{2} \times 0.15 \times 0.28 = 0.021 m$

inclined slab = $\sqrt{R^2 + T^2} \times D = \sqrt{0.15^2 + 0.28^2} \times 0.2$
$= 0.064 m^2$

finishing = $(R+T) \times 0.030 = (0.15 + 0.28) \times 0.030$
$= 0.013 m^2$

Total area = 0.097 m²

DL of 1 step in 1.0m width = $25 \times 0.098 \times 1$
= 2.45 kN/m



DL/m² in plan = $\frac{2.45 \times 1000}{280}$
= 8.75 kN/m²

LL/m² " " = 5.0 kN/m²

Total load = 13.75 kN/m²

factored load = 1.5×13.73 = 20.625 kN/m².

Total factored load on 1.5 m width = 20.625×1.5
= 31 kN/m.

load on landing

Self wt. of slab = 25×0.2 = 5 kN/m²

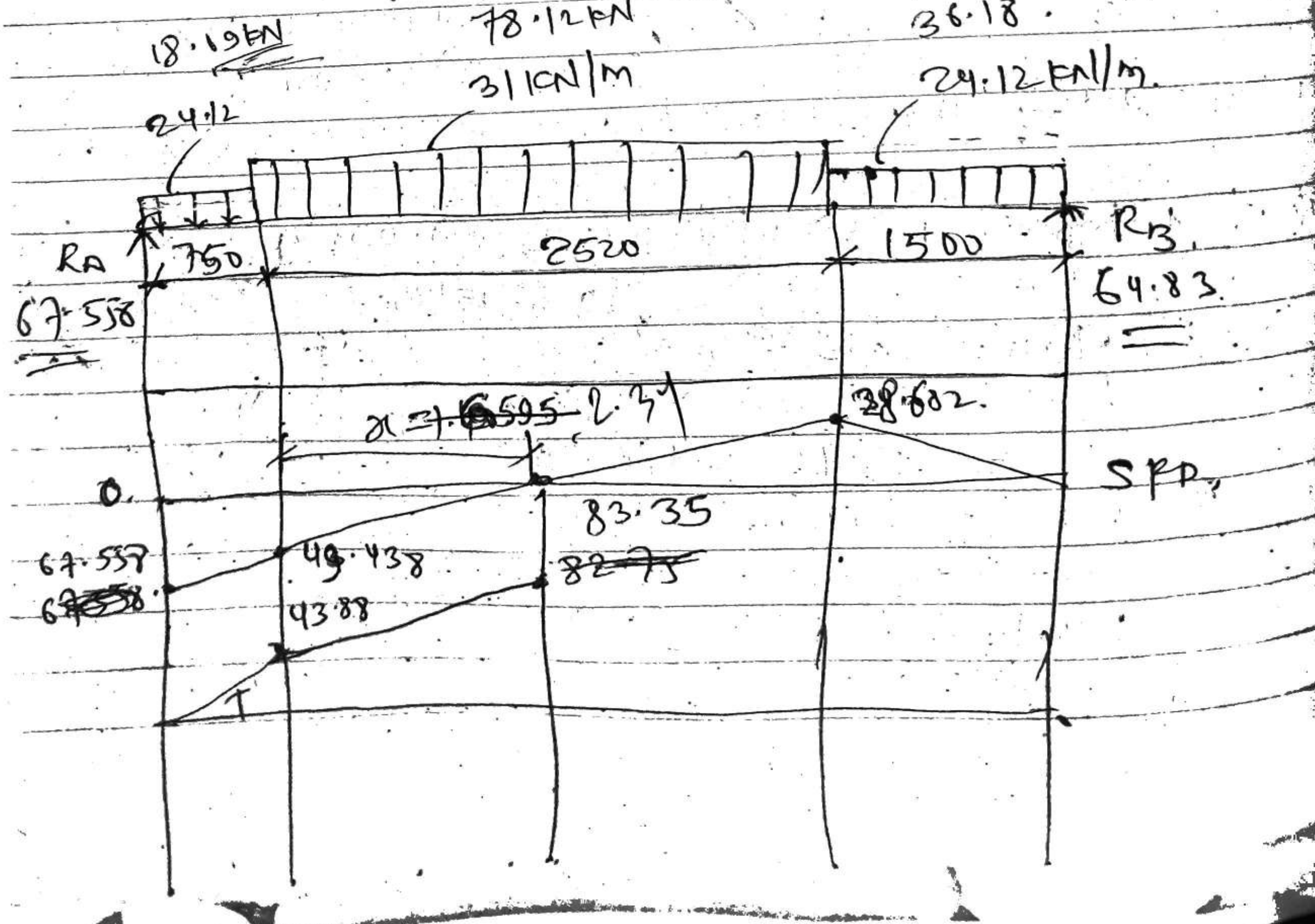
floor finishing = 0.03×24 = 0.72 kN/m²

live load = 5 kN/m²

Total load = 10.72 kN/m²

factored load = 1.5×10.72 = 16.08 kN/m²

Total factored load on 1.5 m width = 1.5×16.08
= 24.12 kN/m



max^m BM = 83.35 kN-m.

let; $M_{lim} = 0.138 f_{ck} b d^2$

$83.35 \times 10^6 = 0.138 \times 20 \times 1500 \times d^2$

$d = 141 mm \simeq 145 mm$

$D = d + 20 + \frac{20}{2} = 175 mm$

~~Again,~~ for steel;

$M_{lim} = 0.87 A_{st} f_y \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$

$83.35 \times 10^6 = 0.87 \times A_{st} \times 415 \left(145 - \frac{415 \times A_{st}}{20 \times 1500} \right)$

$A_{st} = 1957.76 mm^2$

now, spacing $(S) = \frac{1500 A_{st\phi}}{A_{st} req.}$

$= \frac{1500 \times 201.1}{1957.76}$

$= 154.1 mm > 100 mm.$

$\therefore$ provide 16 mm ϕ bar at 150 mm c/c.

A_{st} provided $= \frac{1500 \times 201.1}{150}$

$= 2011 mm^2.$

for distribution bar (8 mm ϕ)

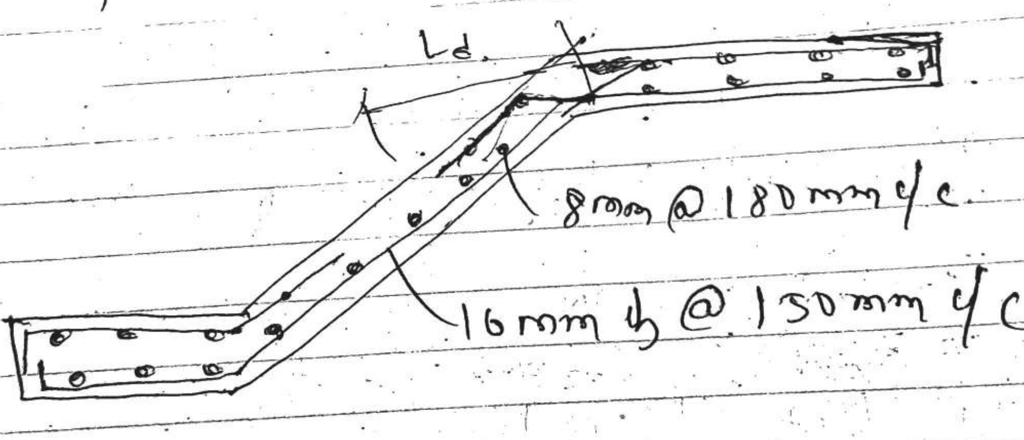
$A_D = 0.12\%$ of bD

$= 315\ mm^2$

$s = \frac{1500 \times 50.26}{315} = 239.33\ mm$

but $s < 300\ mm$

$\therefore$ provide 8 mm ϕ bars at 180 mm c/c.



Prob ① Design on RCC column to the following particulars :-

(P_u) Ultimate Axial load = 1250 KN

(M_{2x}) Ultimate moment at top about x-axis = 40 KNm

(M_{2y}) Ultimate moment at top " y-axis = 15 KNm

(M_{1x}) " at bottom " x-axis = 25 KNm

(M_{1y}) " " " " y-axis = 15 KNm

Unsupported length of the column = 6m

(l_{ex}) Eff. length about x-axis = 4.75m

(l_{ey}) " " y-axis = 4.50m

(b) width of column = 300 mm

Use M20 & Fe 415 steel.

The column is braced and it bends in single curvature.

Soln :- Let column size = $b \times D$ = 300 × 400 mm

Let us provide 8 bars of 20 mm diameter distributed on all sides.

$A_g = 300 \times 400 = 120000\ mm^2$

$A_{sc} = 8 \times 314 = 2512\ mm^2$

$A_c = 120000 - 2512 = 117488\ mm^2$

$\therefore$ % p of steel = $\frac{2512}{300 \times 400} \times 100 = 2.1\%$

Now,

$\frac{l_{ex}}{D} = \frac{4.75 \times 1000}{400} = 11.875 < 12$

$\frac{l_{ey}}{b} = \frac{4.50 \times 1000}{300} = 15 > 12$

Hence, The column is a long column about y-a tial Moments (M_i) (for single curvature)

$M_{ix} = 0.6 M_{2x} + 0.4 M_{1x} = 0.6 * 40 + 0.4 * 25$

$= 34$ kNm

$> 0.4 M_{2x}$ $\boxed{OK}$!

$M_{iy} = 0.6 M_{2y} + 0.4 M_{1y}$

$= 0.6 * 15 + 0.4 * 15$

$= 15$ kNm

$> 0.4 M_{2y}$ $\boxed{OK}$!

inimum moments

$e_{x min} = \frac{L}{500} + \frac{D}{30} = \frac{6000}{500} + \frac{400}{30} = 25.33$ mm

$e_{y min} = \frac{L}{500} + \frac{b}{30} = \frac{6000}{500} + \frac{300}{30} = 22$ mm

$\therefore M_{ux min} = P_u * e_{x min} = 1250 * \frac{25.33}{1000} = 31.67$ kNm

$M_{uy min} = P_u * e_{y min} = 1250 * \frac{22}{1000} = 27.5$ kNm

ince,

$M_{ix} > M_{ux min}$ $\therefore$ Take $M_{ix} = 34$ kNm

$M_{iy} < M_{uy min}$ $\therefore$ Take $M_{iy} = 27.5$ knm

Additional Moments (for bending about y-axis)

$M_{ay} = \frac{P_u b}{2000} \left(\frac{l_{ey}}{b} \right)^2 * K_y$

where,

$K_y = \frac{P_{uz} - P_b}{P_{uz} - P_b}$

where, $P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

for P_b → SP-16, page 171

$$P_b = \left(K_{1y} + \frac{K_{2y}\,P}{f_{ck}}\right) f_{ck}\, bD$$

$$\therefore \frac{d'}{b} = \frac{50}{300} = 0.167$$

Refering to table,

$$\frac{d'}{b} = 0.167, \quad K_{1y} = 0.192 \; \& \; K_{2y} = 0.1435$$

$$\therefore P_b = \left(0.192 + \frac{0.1435 * 2.1}{20}\right) * 20 * 300 * 400$$

$$= 496.84 \text{ KN}$$

$$\therefore K_y = \frac{P_{uz} - P_u}{P_{uz} - P_b} = \frac{1839.25 - 496.84}{1839.25 - 496.84} = 0.439$$

$$\therefore M_{ay} = \frac{1250}{2000} * \frac{300}{1000} * 15^2 * 0.439 = 18.52 \text{ KNm}$$

$$\therefore M_{ux} = M_{ix} + M_{ax} = 34 + 0 = 34 \text{ KNm}$$

$$M_{uy} = M_{iy} + M_{ay} = 27.5 + 18.52 = 33.52 \text{ KNm}$$

So, Column should be designed for following loads moments.

$$P_u = 1250 \text{ KN}$$

$$M_{ux} = \cancel{40}\ 34 \text{ KNm} \quad \& \quad M_{uy} = 33.52 \text{ KNm}$$

④ Check for bi-axial bending

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1.0$$

Bending about x-axis

$$\frac{d'}{D} = \frac{50}{400} = 0.125 \quad , \quad \frac{p}{f_{ck}} = \frac{2.1}{20} = 0.104$$

$$\frac{P_u}{f_{ck} b D} = \frac{1250 \times 10^3}{20 \times 300 \times 400} = 0.521$$

Chart	$\frac{d'}{D}$	P/f_{ck}	$P_u/f_{ck}bD$	$\frac{M_{ux_1}}{f_{ck}bD^2}$
35 ←	0.10	0.104	0.521	0.095
	0.125	0.104	0.521	~~0.087~~ ~~0.090~~ 0.090
←	0.15	0.104	0.521	~~0.090~~ 0.085

$$\therefore M_{ux_1} = 0.090 \times 20 \times 300 \times 400^2 = 86.4 \text{ kNm}$$

Bending about y-axis

$$\frac{d'}{b} = \frac{50}{300} = 0.167$$

Chart	$\frac{d'}{b}$	P/f_{ck}	$P_u/f_{ck}bD$	$\frac{M_{uy_1}}{f_{ck}bD^2}$
←	0.15	0.104	0.521	0.085
←	0.167	0.104	0.521	0.082
←	0.20	0.104	0.521	0.077

$$\therefore M_{uy_1} = 0.082 \times f_{ck} \times b^2 D = 59.04 \text{ kNm}$$

$$\frac{P_u}{P_{uz}} = \frac{1250}{1839.282} = 0.68$$

$$\therefore \alpha_n = 1.80$$

$$\left(\frac{40}{86.4}\right)^{1.80} + \left(\frac{33.52}{59.04}\right)^{1.80} = 0.25 + 0.36 = 0.61 \leq 1.0 \ \boxed{OK}\ !$$

$$= H = \omega \cdot R \left[\frac{\cos\theta + \cos\theta - 1}{1 + \cos\theta} \right] \quad -(11)$$

INTZE TANK

- special type of OHT with top dome & bottom conical as well as circular dome.
- aesthetically good & economic.

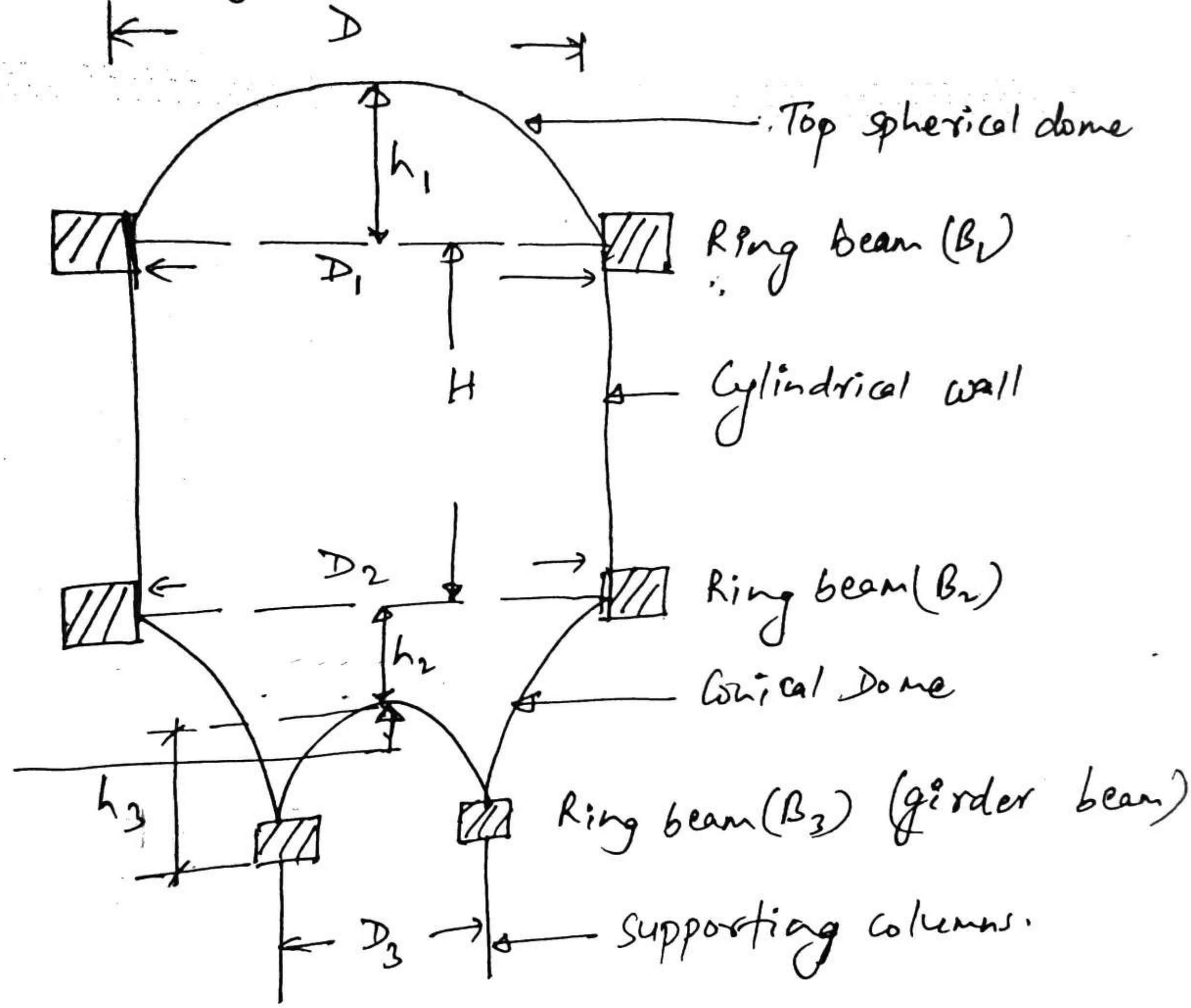


Fig:- Intze tank

Height of cylindrical portion $\boxed{H = 0.6D}$

By Reynold's Model,

Volume of Intze tank $(V) = 0.585\,D^3$

Rise of top dome $= h_1 = \frac{D}{7}$

Rise of conical dome $(h_2) = 0.2D$

Height of bottom spherical dome $(h_3) = \frac{D}{7}$

Diameter of ring beam $(D_3) = 0.6D$

<u>Calculation of radius of top dome (R)</u>

$$\left(\frac{D}{2}\right)^2 = (2R_1 - h_1)\,h_1$$

<u>Calculation of radius of bottom dome (R_3)</u>

$$\left(\frac{D_3}{2}\right)^2 = (2R_3 - h_3)\,h_3$$

Design an Intze tank for the following data :-

Capacity = 500000 litres

Height of tank floor above GL = 15m

~~Safe bearing Capacity of Soil = 100 kN/m²~~

Use M25 concrete & Fe415 steel.

Height of cylindrical wall = H = 8m

LL = 2.0 kN/m²

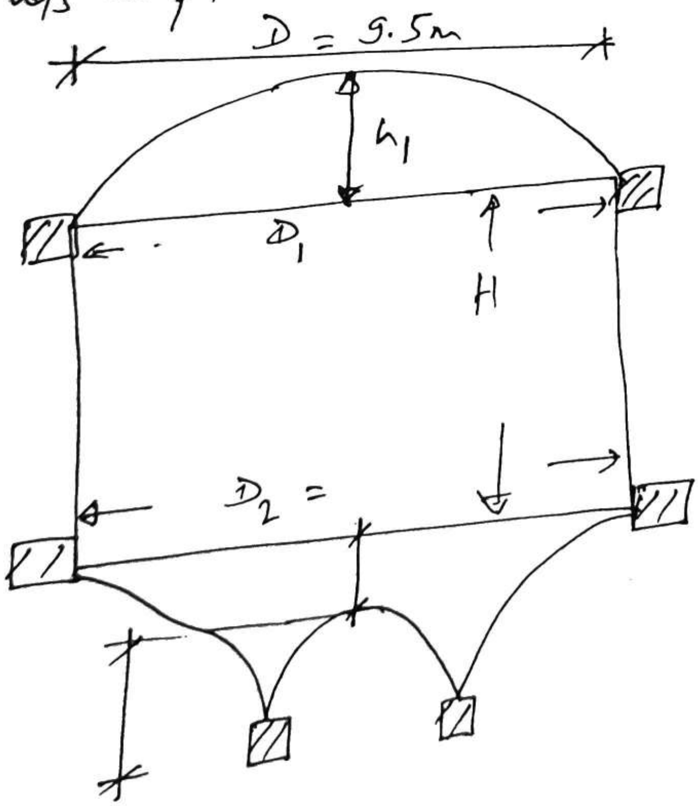
Estimation of diameter, D_1 & height 'H' as suggested by Reynold's.

$$V = 0.585 D^3$$

$$\text{or, } \frac{500000}{1000} m^3 = 0.585 * D^3$$

$$\therefore D = 9.49 m$$

Then, lets adopt D = 9.5 m



Let's adopt

thickness of dome at top = 100 mm

" " " at bottom = 300 mm

Now,

$H = 8m$

other required dimensions

$$h_1 = \frac{D}{7} = \frac{9.5}{7} = 1.35\,m$$

$$h_3 = \frac{D}{7} = 1.35\,m$$

$$h_2 = 0.2D = 0.2 \times 9.5 = 1.9m$$

∴ Dia. of bottom girder $(D_3) = 0.6D = 0.6 \times 9.5 = 7.5m$

2.) Estimation of dome radius & angle

$$h_1(2R_1 - h_1) = \left(\frac{D}{2}\right)^2$$

or, $1.35(2R_1 - 1.35) = \left(\frac{9.5}{2}\right)^2$

∴ $R_1 = 9.03\,m$

Inclination of dome near wall can be determined by -

$$\cos\theta = \frac{R_1 - h_1}{R_1} = \frac{9.03 - 1.35}{9.03} = 0.85$$

∴ $\theta = 31.7° < 51.8°$ → Thus, entire dome is in hoop compression.

calculations

ead load of dome = $0.10 * 25 = 2.5\ KN/m^2$

LL on dome = $2.0\ KN/m^2$

Total load (ω) = $4.5\ KN/m^2$

ermination of Meridional thrust (T) & compressive stress (Ho).

$$T = \frac{\omega R}{1+\cos\theta} = \frac{4.5 * 9.05}{1+0.85} = 22.01\ KN$$

$$\cancel{f_c = \frac{H\theta}{bd} = 22.01 \times}$$

Also,

$$H = \omega \cdot R\left[\frac{\cos^2\theta + \cos\theta - 1}{1+\cos\theta}\right]$$

$$= 4.5 * 9.05\left[\frac{0.85^2 + 0.85 - 1}{1+0.85}\right]$$

$$=$$

9.5

m

US provide $P_t\% = 0.24\%$ in both directions in single layer at mid-height of the dome.

$$A_{sc} = \frac{0.24\, bt}{100} = \frac{0.24}{100} * 1000 * 100 = 240\ mm^2$$

$$\boxed{\text{Use } \phi - 8\,mm @ 200\,mm\ c/c \text{ giving}}$$

A_{st}provided = $251\ mm^2$

(5) Design of Ring beam

Horizontal component of (radial) meridional thrust $= T_R = T\cos\theta = 22.01 * 0.85 = 18.71$ KN

Hoop tension in ring beam $= T_{RH} = T_R * \frac{D}{2}$

$= 18.71 * \frac{9.5}{2}$

$= 88.9$ KN

$\therefore$ Area of hoop reinforcement $= A_{st} = \frac{T_{RH}}{f_{st}}$

$= \frac{88.9 * 1000}{130}$

$= 684$ mm^2

$\boxed{\text{Provide } 4-\phi\ 16\text{mm bars}}$

(6) Design of vertical wall

(7) Design of ring beam (B_2)

(8) Design of conical dome