slab is safe in shear.

STEP 7: CHECK FOR DEVELOPMENT LENGTH

cl. 26.2.1

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}}$$

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

$$= 902.62\,mm$$

MODULE 4

DESIGN OF COLUMNS

A member carrying direct axial load is called as column, the effective length of which exceeds 3 times of its lateral dimensions.

- If the compression member carrying the load is inclined or horizontal it is termed as strut
- The columns may be various shapes such as circular, rectangular, square etc.
- The longitudinal reinforcement bars in the columns are tied by laterally ties of suitable columns are tied by laterally ties or stirrups of suitable intervals, so that the bars does not buckle.

FUNCTIONS OF LONGITUDINAL REINFORCEMENT

- To share the vertical compressive load
- To provide ductility to the column.
- To resist the tensile stresses due to eccentric load, moment or transverse load.
- To reduce the effect of creep and shrinkage.
- To prevent brittle failure.

FUNCTIONS OF TRANSVERSE REINFORCEMENT

- To prevent brittle failure
- To provide ductility to column
- To confine the concrete thereby preventing longitudinal splitting.
- To resist diagonal tension caused due to transverse shear.
- To prevent longitudinal buckling of the longitudinal reinforcement.

SHORT COLUMNS AND LONG COLUMNS

A compression member is considered as short when the slenderness ratio (l/d) is less than 12 and if the slenderness ratio (l/d)

is greaterthan 12, is called as long columns.

TYPES OF COLUMNS

1. Short axially loaded columns in compression
2. Columns subjected to combined axial load & unaxial bending.
3. Columns subjected to combined axial load & biaxial bending

CLASSIFICATION BASED ON THE TYPES OF REINFORCEMENT PROVIDED

1. Columns with longitudinal steel and lateral ties.
2. Columns with longitudinal steel and helical reinforcement or spiral.

Q: A concrete column is reinforced with four bars of 20mm dia. Determine the ultimate load capacity of the column using M20 concrete and Fe 415 steel, If the size of the column is 450mm x 450mm, what will be the allowable service load in the columns.

Given:

A_{sc} = 4 Nos 20mm ϕ

$A_{sc} = 4 \times \pi/4 \times 20^2$

$P_u = ?$ $\quad$ $P = ?$

$f_{ck} = 20 N/mm^2$ $\quad$ $D = 450mm$

$f_y = 415 N/mm^2$

Cl. 39.3 of IS 456:2000

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

Min eccentricity =

$$\frac{l}{500} + \frac{D}{30}$$

Assume min. e = 20mm

$0.05D = 0.05 \times 450$

$= 22.5mm$

$\therefore$ min. e $< 0.05 D$

$\therefore$ The ultimate load can be computed from,

Area of concrete = Gross area − Area of steel

$A_c = A_G - A_{sc}$

$A_G = 450 \times 450$

$= 202500 mm^2$

$A_{sc} = 4 \times \pi/4 \times 20^2$

$= 1256.637 mm^2$

$A_c = A_G - A_{sc}$

$= 201243.363 mm^2$

$\therefore P_u = 0.4 \times 20 \times 201243.3\ldots$

$+ 0.67 \times 415 \times 1256.63$

$= 1959352.87 N$

$= 1959.352 kN$

Allowable service load,

$$P_a = \frac{P_u}{1.5}$$

$$= \frac{1959.35}{1.5}$$

$= 13.2 kN$

Q. Design a ... loaded squar... 500 x 500mm... Service load... Use M20 con... Fe 415 steel.

Soln: given

D = 500mm

$f_{ck} = 20 N/...$

$f_y = 415$

P = 200...

P_u = 30...

Min eccentric...

Assume min...

0.05D ...

$\therefore$ min e ...

$\therefore$ the colu... designed w... cl. 39.3 of IS...

$P_u = 0.4 f_{ck} A_c$...

Area of conc... Gross area −...

$A_c = 500 \times 5$...

$P_u = 0.4 \times 2$...

$0.67 \times$...

$3000 \times 10^3 = 2$...

$8 A_s$...

$3000 \times 10^3 = 2$...

$A_{sc} = 3$...

Q. Design a short axially loaded square column 500x 500mm for a Service load of 2000kN Use M20 concrete & Fe 415 steel.

Soln: given

$D = 500mm$

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

$f_y = 415\ N/mm^2$

$P = 2000\ kN$

$P_u = 3000\ kN$

Min eccentricity = $\frac{l}{500} + \frac{D}{30}$

Assume min e = 20mm

$0.05D = 0.05 \times 500$

$= 25mm$

∴ min e < 0.05 D

∴ the column can be designed using cl. 39.3 of IS 456 : 2000

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

Area of concrete = Gross area − Area of steel

$A_c = 500 \times 500 - A_s$

$P_u = 0.4 \times 20 [250000 - A_{sc}] + 0.67 \times 415\ A_{sc}$

$3000 \times 10^3 = 2000000 - 8 A_{sc} + 278.05 A_{sc}$

$3000 \times 10^3 = 2000000 + 270.05 A_{sc}$

$A_{sc} = 3703.01\ mm^2$

$A_{sc\,min} = \frac{0.8 A_G}{100}$

[cl. 26.5.3.1.a of IS 456:2000]

$= \frac{0.8 \times 500 \times 500}{100}$

$= 2000mm^2$

$A_{sc\,MAX} = \frac{6}{100} \times A_G$

$= \frac{6}{100} \times 500 \times 500$

$= 15000mm^2$

$A_{sc\,min} < A_{sc} < A_{sc\,max}$

∴ the section is safe

No. of bars = $\frac{A_{sc}}{\text{Area of 1 bar}}$

Assume a φ of 25mm for longitudinal bars

$= \frac{3703.01}{\frac{\pi}{4} \times 25^2} = 7.54$

≈ 8 bars

Transverse reinforcement:

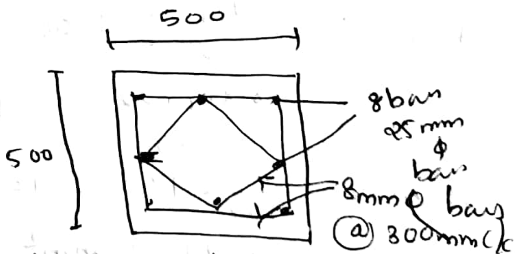
Pitch of lateral ties

[cl. 26.5.3.2.c]

(i) 500mm

(ii) 16 x 25 = 400mm

(iii) 300 mm



Provide 8mm φ bars @ 300mm c/c.

Q: Design a circular column of axial load 1000 kN use M_{20} concrete & Fe 415 steel

Soln: $P = 1000\,kN$

$P_u = 1000 \times 1.5 = 1500\,kN$

$f_{ck} = 20\,N/mm^2 \quad f_y = 415\,N/mm^2$

Min eccentricity $= \frac{l}{500} + \frac{D}{30}$

Assume min eccen $< 0.05D$

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

Assume % of steel,

$A_{sc} = 1\%\ A_G$

$\boxed{A_{sc} = 0.01\ A_G}$

$A_c = A_G - A_{sc}$

$= A_G - 0.01\ A_G$

$\boxed{A_c = 0.99\ A_G}$

rectangle - $0.05D = B$

$0.05 D \times D = A_g$

$1500 \times 10^3 = 0.4 \times 20 \times 0.99\ A_G + 0.67 \times 415 \times 0.01\ A_G$

$1500 \times 10^3 = 7.92\ A_G + 2.78\ A_G$

$1500 \times 10^3 = 10.7\ A_G$

$A_G = 140180.36\,mm^2$

$\pi/4\ D^2 = 140180.36$

$D = 422.57 \approx 430\,mm$

$A_G = \pi/4 \times 430^2 = 145220.120\,mm^2$

$A_{sc} = 0.01 \times 145220.120$

$= 1452.20\,mm^2$

$A_{sc\,min} = \frac{0.8\ A_G}{100}$ [cl 26.5.3.1.a]

$= \frac{0.8 \times 145220.12}{100} = 1161.76\,mm^2$

$A_{sc\,max} = \frac{6}{100} \times 145220.12$

$= 8713.20\,mm^2$

$A_{sc\,min} < A_{sc} < A_{sc\,max}$

∴ The section is safe

No. of bars $= \frac{A_{sc}}{\text{Area of 1 bar}}$

Assume a ϕ of ~~8mm~~ 16mm for longitudinal bars

$\frac{1452.20}{\frac{\pi}{4} \times 16^2} = 7.22 \approx$ 8

8 Nos bars.

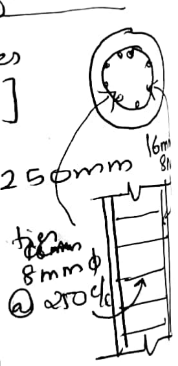
Transverse Reinforcement

Pitch of lateral ties [cl. 26.5.3.2.c]

(i) 430mm

(ii) $16 \times 16 = 256 \approx 250\,mm$

(iii) 300mm



Provide 8mm ϕ bars @ 250mm c/c

ASSIGNMENT

1) Design a rectangular column of axial load of 1500kN. Use M_{20} concrete and Fe 500 steel.

2) Design a square column of axial load of 800 kN. Use M20 Concrete & Fe 500 steel

1.19 JAY Q: Design ... column 4.6m ... in position ... ends and re... rotation at ... carry a ax... 1200 kN and ... is restricted ... M_{20} ...

unsupported length $\approx$ direct apply

Soln: $l = 4.6$ ...

$P = 12$...

$D = 4$...

$f_{ck} = $ ...

$f_y = $ ...

For the give... the effecti... the compre... is

$l_{eff} = 0.$ ...

[Tab...

$= 0.8$ ...

$= 3.$ ...

$= 36$ ...

• eccentricity

$e_{min} = $ ...

[from ...

$e_{min} = 3$ ...

$=$ ...

$0.05D =$ ...

e_{min} ...

22. ...

$\ldots c < A_{sc\,max}$

...s is safe

$\frac{A_{sc}}{\text{Area of 1 bar}}$

...of 16mm

...al bars

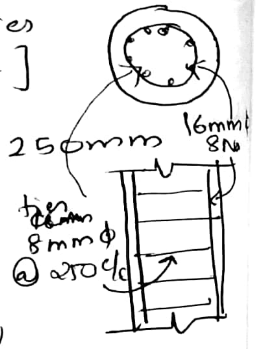
$= 7.22 \approx$

...ars.

...forcement

...es

]



...angular

...ial load of

M20 concrete

...teel.

...uare column

...d of 800 kN

...crete &

11.19 MONDAY

Q: Design a circular column 4.6m high held in position at both ends and restrains against rotation at one end to carry a axial load of 1200 kN and its diameter is restricted to 450mm. M20 concret Fe415 steel

Soln: $l = 4.6\,m$

$P = 1200\,kN$

$D = 450\,mm$

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

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

using factored load & length apply

for the given end conditions the effective length of the compression member is

$l_{eff} = 0.80\,l$

[Table 28 of IS 456 : 2000]

$= 0.8 \times 4.6$

$= 3.68\,m$

$= 3680\,mm$

• eccentricity min

$e_{min} = \frac{l}{500} + \frac{D}{30}$ min 20

[from cl. 25.4]

$e_{min} = \frac{3680}{500} + \frac{450}{30}$

$= 22.36\,mm > 20$

$0.05D = 0.05 \times 450$

$= 22.50$

$e_{min} < 0.05D$

$22.36 < 22.5$

$e_{min} < 0.05D$ ∴ the column can be designed using cl 39.3.

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

~~1200×10~~ $P_u = 1200 \times 1.5 \times 10^3$

$= 1800 \times 10^3\,N$

$\therefore A_c = [A_g - A_{sc}]$

$1800 \times 10^3 = 0.4 \times 20\,[A_g - A_{sc}] + 0.67 \times 415 \times A_{sc}$

$1800 \times 10^3 = 0.4 \times 20 \left[\frac{\pi}{4} \times 450^2 - A_{sc}\right] + 0.67 \times 415 \times A_{sc}$

$A_{sc} = 1953.915\,mm^2$

Check: {cl ~~26.5~~. 26.5.3}

$A_{sc\,min} = 0.8\%$ of c/s Area

$= \frac{0.8}{100} \times \frac{\pi}{4} \times 450^2$

$A_{sc} = 1272\,mm^2$

$A_{sc\,max} = 6\%$ of c/s A

$= \frac{6}{100} \times \frac{\pi}{4} \times 450^2$

$= 9542\,mm^2$

$A_{sc\,min} < A_{sc} < A_{sc\,max}$

No. of bars $= \frac{A_{sc}}{\text{Area of 1 bar}}$

Assume 20mm φ bar

No. of bars $= \frac{1953.95}{\pi/4 \times 20^2}$

$= 6.2$

≈ 7 No.s

Provide 7 No.s 20mm φ bars as longitudinal reinforcement

Scanned by CamScanner

Transverse reinforcement pg: 49

Pitch of lateral ties

[cl 26.5 .3.2.bc]

Assume 8mm ϕ lateral ties

Pitch :-

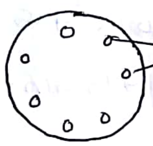
(i) least latural dimension

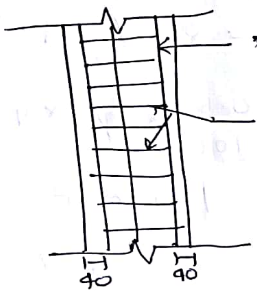
(i) 450 mm

(ii) $16\phi = 16 \times 20 = 320$

(iii) 300 mm

Provide 8mm ϕ stirrup 300mm c/c





~~Provide 4 Nos, 20mm ϕ bars as longitudinal reinforcement~~

Q: Design a circular column with helical reinforcement to carry a axial load of 1000 kN use M20 concrete Fe 415 steel

Soln: Given

$P = 1000$ kN

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

$f_y = 415$ N/mm^2

Assume e_{min} is less than 0.05D

NOTE

Strength of compression member with helical reinforcement is 1.05 times the strength of similar member with lateral ties.

There fore

~~$P_u = 1000 \times 1.5$~~

$$P_u = \frac{P \times 1.5}{1.05}$$

$$= \frac{1000 \times 1.5}{1.05}$$

$= 1428.57$ kN

$= 1428.57 \times 10^3$ N

From cl. 39.3

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

Assume % of steel provide is 1% of gross cross sectional area

$\therefore$ $A_{sc} = \frac{1}{100} \times A_G$

$A_{sc} = 0.01\ A_G$

Area of concrete.

$= A_G - A_{sc}$

$= A_G - 0.01 A_G$

$A_c = 0.99\ A_G$

~~$P_u = 0.4$~~

$1428.57 \times 10^3 =$

$0.4 \times 20 \times 0.99\ A_G +$

$0.67 \times 415 \times 0.01\ A_G$

$\therefore A_G = 133504.974$ mm^2

$\frac{\pi}{4} D^2 = A_G$

$133504.974 = \pi/4\ D^2$

D = 412.29

$\therefore$ Actual

$A_G = \frac{\pi}{4}$

= 13

$A_{sc} = 0.01$

= 0.01 ×

= 1385

Check

$A_{sc_{min}} = 0.8\%$

= 0.8 ×

= 1108

$A_{sc_{max}} =$ 0.6

= 831

$\therefore$ No. of bar

Assume 16m

= 1385

π

= 6

≈ 4

Provide 7 No

bars as lon

reinforcemen

Transverse r

mpression
helical
is
the strength
ember with

s

-5

5

57 kN

57 $\times 10^3$ N

-0.67 f_y A_{sc}

eel provide
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G

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1 AG

AG

AG +
.01 AG
504.974 mm^2

$\pi/4\ D^2$

$D = 412.29 \approx 420\ mm$

∴ Actual Gross Area

$A_G = \frac{\pi}{4} \times 420^2$

$= 138544.236\ mm^2$

$A_{sc} = 0.01\ A_G$

$= 0.01 \times 138544.236$

$= 1385.44\ mm^2$

Check

$A_{sc\,min} = 0.8\%$ of ~~13854~~ A_G

$= \frac{0.8 \times 138544.236}{100}$

$= 1108.35\ mm^2$

$A_{sc\,max} = \frac{0.6 \times 138544.236}{100}$

$= 8312.64\ mm^2$

∴ No. of bars = $\frac{A_{sc}}{\text{Area of 1 bar}}$

Assume 16mm ϕ bars

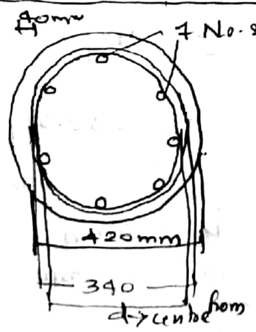
$= \frac{1385.44}{\pi/4 \times 16^2}$

$= 6.89$

≈ 7 bars.

Provide 7 Nos of 16mm ϕ bars as longitudinal reinforcement.

Transverse reinforcement



Diameter of core to the outside of the helix

Outside diameter of helix =

$420 - 2 \times 40$

$= 340\ mm$

Area of core $= \frac{\pi}{4} \times 340^2$

$= 90792.02\ mm^2$

Area of core excluding longitudinal bars:-

$A_k = \frac{\pi}{4} \times 340^2 - 7 \times \frac{\pi}{4} \times 16^2$

$= 89384.6\ mm^2$

Vol. of the core measured to 1mm ~~a~~ A_k length

$= A_k \times 1$

$= 89384.6\ mm^3$

Vol. of helix per mm of column

$= \boxed{\underbrace{\left(\frac{\pi d}{s}\right)}_{\text{height}} \underbrace{\left(\frac{\pi}{4} \times \phi_s^2\right)}_{\text{Area}}}$ ϕ_s

diameter of the helix to the centere.

$= 340 - 8 = 332\ mm$

{Assume ϕ of helix = 8mm}

Volume of helix =

$\frac{\pi \times 332}{s}\left(\frac{\pi}{4} \times 8^2\right)$

From Cl. 39.4.1 (71 page)

$\frac{\text{Vol. of helical reinfor}}{\text{Vol. of core}} = \frac{0.36\left(\frac{A_g}{A_k} - 1\right) f_{ck}}{f_y}$

$\frac{\frac{\pi \times 332}{s} \times \frac{\pi}{4} \times 8^2}{89384.6} = \frac{0.36\left[\frac{138544.2}{89384.6} - 1\right] \times 20}{415}$

$S = 61.469mm$ hoop → Stirrup (Ring, Square, circular)

Pitch

cl. 26.5.3.2.(d)

(i) $S < 75mm$ ✓

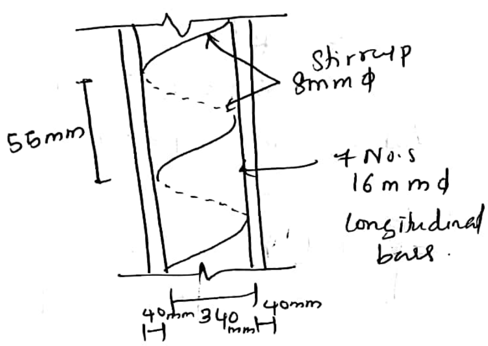
(ii) $S < \frac{1}{6} \times 340 = 56.6$ ↑max. × so we can take 61.469

$(S > \frac{1}{6} \times 340)$

(iii) $S > 25mm$ ✓

(iv) $S > 3 \times \phi_s = 3 \times 8 = 24$ ✓

provide helical reinforcement of 8mm ϕ @ 55 mm c/c



18.11.19 MONDAY

Q: Design a rectangular column of 4.5m unsupported length restrain a position & direction of both ends. To carry a axial load of 1200kN. Use M20 concrete & Fe 415 steel

Soln: Given:

Unsupported length = 4.5m

$P = 1200 kN$

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

$f_y = 415 N/mm^2$

$P_u = 1200 \times 1.5$

$= 1800 kN$

Assume

$e_{min} = \frac{l}{500} + \frac{D}{30} > 20$ (e_{min} min 20) lateral dimension

Assume $e_{min} < 0.05D$

[cl. 39.3 of IS 456:2000]

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

Assume 1% of reinforcement as 1% Gross area.

$A_{sc} = \frac{1}{100} A_G$

$A_c = A_G - A_{sc}$

$= A_G - 0.01 A_G$

$A_c = 0.99 A_G$

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

$1800 \times 10^3 =$

$0.4 \times 20 \times 0.99 A_G + 0.67 \times 415 \times 0.01$

$A_G = 168216.4385$

Rectangular $= B \times D = A_G$

$[B = 0.5 \text{ to } 0.67] D$

Assume $B = \frac{D}{2}$

$\frac{D}{2} \times D = A_G$

$\frac{D^2}{2} = A_G$

$D = 580.02$

$\approx 600 mm$

$D = 600mm$

$B = 300mm$

Actual gross

$A_G = 60$

$= 18$

$A_{sc} = 0.$

$= 1$

[c] 26.

$A_{sc_{min}} = 0.$

1

$= 1$

$A_{sc_{max}} =$

$=$

Assume

No. of

$=$

unsupported length as assumed effective length assumed

check f

ratio :-

Table

>12 long column

$l_{eff} =$

$=$

<12 Short columns

$\frac{l_{eff}}{D} =$

$\frac{l_{eff}}{B} =$

1.5

kN

e_{min} min 20

$+ \frac{D}{30} > 20$

mm $< 0.05D$ lateral dimension

S 456:2000]

$+ 0.67 f_y A_{sc}$

reinforcement area.

sc

01 A_G

A_G

$0.67 f_y A_{sc}$

$A_G + 0.67 \times 415 \times 0.01$

16.4385

5 to 0.67] D

$\frac{D}{2}$

A_G

G

580.02

600 mm

mm

00 mm

Actual gross area = $B \times D$

$A_G = 600 \times 300$

$= 180000 \ mm^2$

$A_{sc} = 0.01 \times 180000$

$= 1800 \ mm^2$

[Cl 26.

$A_{sc_{min}} = \frac{0.8}{100} \times 600 \times 300$

$= 1440$

$A_{sc_{max}} = \frac{6}{100} \times 600 \times 300$

$= 10800$

Assume 20mm ϕ bars

No. of bars = $\frac{A_{sc}}{\text{Area of 1 bar}}$

$= \frac{1800}{\pi/4 \times 20^2} = 5.7$

unsupported length or effective length

≈ 6 Nos

Check for slendernous ratio :-

Table 28 (Cl E-3) Pg.99

$\frac{l}{d} > 12$ ⇓ long column

$l_{eff} = 0.65 \, l$

$\frac{l}{d} < 12$ ⇓ Short column

$= 0.65 \times 4.5$

$= 2.925$ mm

$= 2925$ m

$\frac{l_{eff}}{D} = \frac{2.925}{600} = 4.875 \times 10^{-3}$

$\frac{l_{eff}}{B} = \frac{2.925}{300} = 9.75 \times 10^{-3}$

$\frac{l_{eff}}{D} < 12$

$\frac{l_{eff}}{B} < 12$

∴ It is a short column.

Check for eccentricity :-

In x direction

$\frac{l}{500} + \frac{B}{30} = \frac{2925}{500} + \frac{300}{30}$

$= 15.85$

$e_{min} (x \ dir) = 20$

$0.05B = 0.05 \times 300$

$= 15$

$e_{min} > 0.05B$

In y direction ⇒

$\frac{l}{500} + \frac{D}{30} = \frac{2925}{500} + \frac{600}{30}$

$= 25.85$

$0.05D = 0.05 \times 600 = 30$

$e_{min} < 0.05D$

Since the equation is not strictly applicable the column is to be redesigned we have,

$0.05B = 20$

$B = 400$ mm

$B \times D = 180000$

$D = 450$ mm

Transverse Reinforcement
Pitch

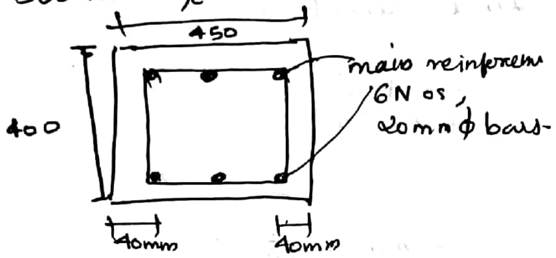
cl. 26.5.3.2

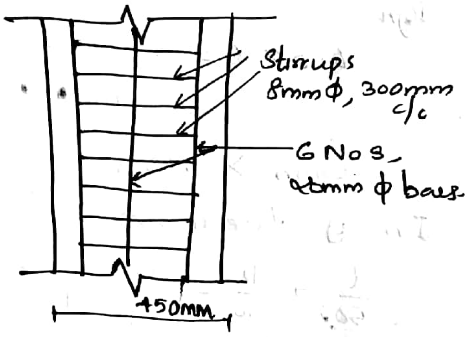
(i) least lateral dimension = 400

(ii) $16\phi = 16 \times 20 = 320$

(iii) 300mm

provide 8mm ϕ bars @ 300mm c/c





MODULE 4

ISOLATED FOOTING

ISOLATED FOOTING FOR AXIALLY LOADED COLUMNS

ISOLATED FOOTING FOR UNIFORM DEPTH FOR RCC COLUMN

Design a isolated footing of uniform thickness for a RCC column having a vertical load of 600kN and having a base of size 500 x 300mm. The safe bearing capacity of soil is 120 kN/m². Use M_{20} concrete & Fe 415 steel.

$W = 600kN$ $\quad q_0 = 120\ kN/m^2$

$b = 500mm$, $d = 500mm$ } column.

$f_{ck} = 20 N/mm^2$ $\quad f_y = 415 N/mm^2$

Step 1: Dimension of the section

Let w' be the selfweight of the column 10% of Super imposed load.

$W' = 10\% \cdot W$

$= 600 \times \frac{10}{100} = 60kN$

Total load $= 600 + 60 = 660kN$

Area $= \frac{\text{Load}}{\text{Pressure}}$

$= \frac{660}{120}$

$= 5.5m^2$

So provide a square column of size $B^2 = 5.5$

$B = 2.34$

$B = 2.4m$

Provide a square footing of $2.4 \times 2.4m$

Net upward pressure $= \frac{\text{Actual load}}{\text{Area}} = \frac{600}{2.4 \times 2.4}$

$= 104.17\ kN/m^2$

Design of the

1. Depth on

bending co

The max b

act at th

column

Moment

$M = P_0 B$ (

lo

$\boxed{M = }$

$M = 10$

$= 112$

$M_u = M$

$= 1$

=

Calculate 'd'

$M_{ulimit} = 0.36$

$169.2 \times 10^6 = 0.36 \times$

$d = 159.85$