Square Panel, No drop, No column head

Example → Design an interior panel of a flat slab of size 5m × 5m without providing drop and column head. Size of columns is 500mm × 500mm and the live load on the panel is 4 kN/m². Take floor finish load as 1 kN/m². Use M20 concrete and Fe 415 steel. Height of column is 4m.

Solution →

Step-1 → Calculation of depth of slab →

Because drops are not provided so $\frac{l}{d}$ ratio should be multiplied by 0.9. [clause no → 31.2.1 Page no 53]

$l = 5m$, $d = ?$

$$\frac{l}{d} = 26 \times 0.9 \times M.F.$$

$$\frac{5000\ mm}{M.F. \times 26 \times 0.9} = d \text{ in mm}$$

$$d = \frac{213.675\ mm}{M.F.} \underset{\text{should be}}{>} 125\ mm$$ [clause no 31.2.1, Page no 53]

$f_y = 415\ N/mm^2$

See Page no (38)

$$f_s = 0.58 f_y \frac{\text{Area of C/S of steel required}}{\text{Area of C/S of steel provided}}$$

Because we don't know about the reinforcement so assume →

Area of C/S of steel required = Area of C/S of steel provided

$f_s = 0.58 \times 415 \times 1$

$f_s = 240.7\ N/mm^2$

Because r/f is not provided till now so assume Min. Ast is provided $= 0.12\%$ of c/s Area

From Fig 4 Page no 38 , $f_s = 240.7$, $p_t = 0.12\%$

$M.F. = 1.7$

$$d = \frac{213.675}{M.F.} = \frac{213.675}{1.7} = 125.69\ mm$$

$d > 125\ mm$ safe

so provide $d > 126\ mm$

for safety purpose we would provide

$d = 150\ mm$

because two layer of r/f has to be provide

so $D = d + \phi +$ effective cover

$D = 150 + 20 + 30$

$D = 200\ mm$

Step-2 → Loading Calculation -

Self weight of flat slab $= 0.2 \times 25^{kN/m^3} = 5\ kN/m^2$

Live load $= 4\ kN/m^2$

Floor finish $= 1\ kN/m^2$

Total load $= 5+4+1 = 10\ kN/m^2$

Factored load $= 10 \times 1.5 = 15\ kN/m^2$

Step-3 → Check for 2-way shear →

critical section for shear is at a distance of $\frac{d}{2}$ from the face of the column.

$d = 150\ mm$

periphery of critical section

$= 4\left(500 + \frac{150}{2} + \frac{150}{2}\right)$

$= 1300\ mm$

c/s area availiable $= 5000 \times 5000 - 650 \times 650$
$= 24577500\ mm^2$

Total given load $= \frac{15 \times 1000 \times 24577500}{1000 \times 1000}$

$= 368662.5\ N$

$$\tau_v = \frac{V_u}{bd} = \frac{368662.5}{1300 \times 150} = 1.89\ \frac{N}{mm^2}$$

$\tau_c' = k_s \tau_c$ (Page no - 59)

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

$k_s = 0.5 + \beta_c$

$\beta_c = \frac{\text{short side of column}}{\text{long side of column}}$ max $k_s = 1$

$\beta_c = \frac{5m}{5m} = 1$

$k_s = 0.5 + 1 = 1.5$, $\because k_s \not> 1$

So $k_s = 1$

$\tau_c = 0.25\sqrt{20} = 1.118$

$\tau_c' = 1 \times 1.118\ \frac{N}{mm^2} = 1.118\ \frac{N}{mm^2}$

because $1.5\,\tau_c' < \tau_v$
$1.5 \times 1.118 < 1.89$
$1.677 < 1.89$

so the slab should be redesigned.

$\frac{l}{d} = 32 \times 0.9$

$\frac{5000}{32 \times 0.9} = d = 173.611$ mm

provide $d = 175$ mm

$D = 175 + 15 + 30 = 220$ mm

Loading Calculation →

Self weight of flat slab $= 0.22 \times 25 \frac{kN}{m^2} = 5.5 \frac{kN}{m^2}$

Live load $= 4\ kN/m^2$

Floor finish $= 1\ kN/m^2$

Total load $= 5.5 + 4 + 1 = 10.5\ kN/m^2$

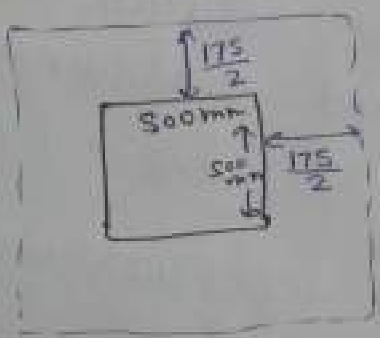
Factored load $= 1.5 \times 10.5 = 15.75\ kN/m^2$

check for 2-way shear →

periphery of critical section

$= 4\left(500 + \frac{175}{2} + \frac{175}{2}\right)$

$= 2700$ mm



$$V_u = \frac{15}{1000}\left[5000 \times 5000 - 675 \times 675\right]$$

$$= 368165.625 \text{ N}$$

$$\tau_v = \frac{V_u}{bd} = \frac{368165.625}{2700 \times 175} = 0.779 \text{ N/mm}^2$$

and $\tau_c' = 1.118 \frac{N}{mm^2}$

$\because \tau_c' > \tau_v$

So no shear reinforcement is required.

Step-4 → Calculation of Stiffness and α_c -

or

Correction for pattern loading →

(not required if length of column is not given)

K_c = Flexural stiffness of the columns meeting at the joint.

$$K_c = \frac{4EI}{l_{eff}}$$

Page no - 94, Table no - 28

Because the column is effectively held in position and restrained against ration at one end so $l_{eff} = 1.20 l$

$l_{eff} = 1.20 \times 4000 \text{ mm} = 4800 \text{ mm}$

$$I = \frac{bd^3}{12} = \frac{500 \times (500)^3}{12} = 5208333333 \text{ mm}^4$$

$$K_c = \frac{4E \times 520833333}{4800} = 4E \times 108506.944$$

K_s = Flexural stiffness of the slab, expressed as moment per unit rotation.

$l_{eff} = 5000\,mm$

$$I = \frac{bd^3}{12} = \frac{5000 \times (220)^3}{12}$$

$$K_s = \frac{4E \times 5000 \times (220)^3}{5000 \times 12} = 4E \times 887333.33$$

$$\alpha_c = \frac{\Sigma K_c}{K_s}$$ (Page no - 55)

$$\alpha_c = \frac{2 \times 4E \times 108506.944}{4E \times 887333.33} = 2.44$$

Check for α_{cmin}

$$\frac{\text{Imposed load}}{\text{Dead load}} = \frac{4}{6.5} = 0.615$$

From table no - 17, Page no - 56

value 0.615 exists between 0.5 to 1.0

at $\frac{I.L}{D.L.}$	α_{cmin}
0.5 $\longrightarrow$	0
1.0 $\longrightarrow$	0.6

at 0.615 $\alpha_{cmin} = \frac{0.115 \times 0.6}{0.5} = 0.138$

$\alpha_{cmin} < \alpha_c \rightarrow$ Safe

Ratio $\frac{l_2}{l_1} = \frac{5}{5} = 1$ Table no (17) Page no 56

at $\frac{l_2}{l_1} = 1$, $\alpha_{cmin} = 0.7$

$\alpha > \alpha_{cmin} \rightarrow$ Safe

no correction required here for pattern loading.

<u>Step-5 → Calculate length and size of column strip and middle strip.</u>

$l_1 = l_2 = 5000$ mm

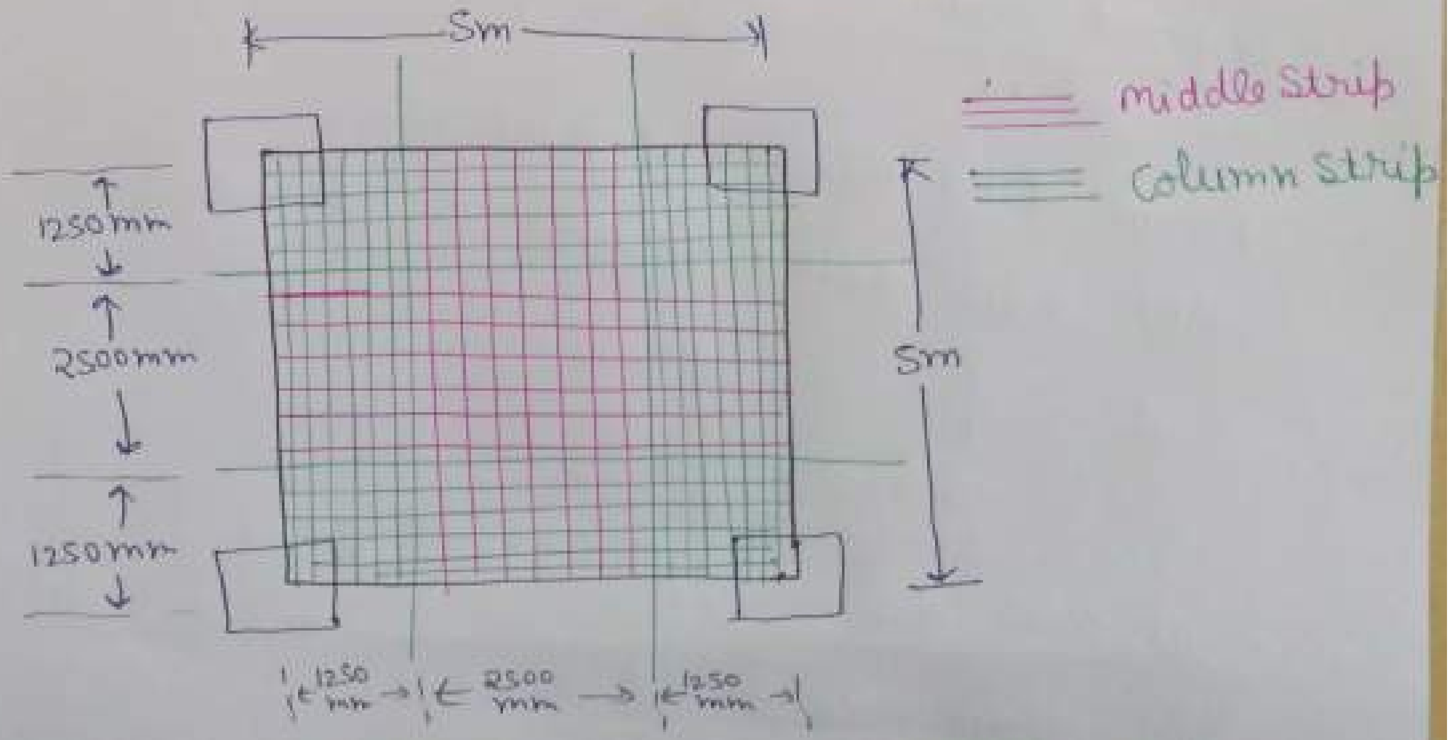
Column strip thickness = $0.25\, l_1$ or $0.25\, l_2$ whichever is less

Page no-53, clause no - 31.1.1 (a)

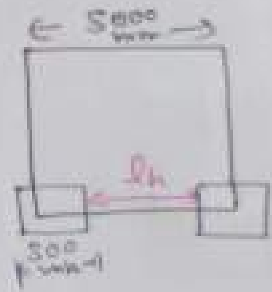
$= 0.25 \times 5000 = 1250$ mm

middle strip thickness = $5000 - (1250 + 1250)$

$= 2500$ mm



Step-6 → Calculate moments in all strips →



$l_n = 5000 - \frac{500}{2} - \frac{500}{2}$

$l_n = 4500$ mm

here $l_1 = l_2 = 5000$ mm

$0.65\, l_1 = 0.65 \times 5000$

$= 3250$ mm

$l_n > 0.65\, l_1 \rightarrow$ OK

$W = w\, l_2\, l_n = \frac{15.75}{1000} \times 5000 \times 4500 = 354375$ N

Page no-55

$M_o = \frac{W l_n}{8} = \frac{354375 \times 4500}{8} = 199335937.5$ Nmm

Total -ve design moment $= 0.65 M_o$ Page no 55 clause no-31.4.3.2

$= 0.65 \times 199335937.5$

$= 129568359.4$ Nmm

Total +ve design moment $= 0.35 M_o$

$= 0.35 \times 199335937.5$

$= 69767578.13$ Nmm

Column Strip-

-ve moment $= 0.75 \times$ Total -ve moment

Page no 57, clause no- 31.5.5.1

$= 0.75 \times 129568359.4$

$= 97176269.55$ Nmm

+ve moment $= 0.60 \times$ Total +ve moment

Page no 57, clause no 31.5.5.3

$= 0.60 \times 69767578.13$

$= 41860546.88$ Nmm

Middle strip →

$-ve$ moment $= 129568359.4 - 97176269.55$

$= 32392089.85$ Nmm

$+ve$ moment $= 69767578.13 - 41860546.88$

$= 27907031.25$ Nmm

$M_{ulim} = k b d^2$

$= 2.76 \times 2500 \times (175)^2$

$= 211312500$ Nmm

for column strip

$b = 2500$ mm

$d = 175$ mm

for middle strip

$b = 2500$ mm

$d = 175$ mm

for both strip

$M_{ulim} > \begin{matrix} +M \\ -M \end{matrix}$

So depth provided is safe in Bending moment criteria.

Step-7 → Reinforcement along longer and shorter direction

because $l_1 = l_2 = 5$ m so same r/f would be provided in both the directions.

r/f for $-ve$ M in Column Strip –

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

$A_{st\,min} = \frac{0.12}{100} \times bD$

$= \frac{0.12}{100} \times 2500 \times 220$

$= 660 \text{ mm}^2$

$$9176269.55 = 0.87 \times 415 \times Ast \times 175 \left(1 - \frac{415 \times Ast}{20 \times 2500 \times 175}\right)$$

$$\frac{9176269.55}{6318.375} = Ast\left(1 - 4.74 \times 10^{-5}\, Ast\right)$$

$$1537.99 = Ast - 4.47 \times 10^{-5}\, Ast^2$$

$$4.47 \times 10^{-5}\, Ast^2 - Ast + 1537.99 = 0$$

$Ast_1 = 20709.99\ mm^2$ (rejected due to spacing criteria)

$Ast_2 = 1661.36\ mm^2$ (accepted)

Ast > Astmin OK

take dia of bars = 12 mm

$$\text{spacing} = \frac{\frac{\pi}{4}(\phi)^2 \times b}{Ast}$$

$$= \frac{\frac{\pi}{4} \times (12)^2 \times 2500}{1661.36} = 170.187$$

provide 12mm dia bars at 170 mm c/c spacing.

r/f for +ve M in column strip -

$$Mu = 0.87\, fy\, Ast\, d\left(1 - \frac{fy\, Ast}{fck\, b\, d}\right)$$

$$41860546.88 = 0.87 \times 415 \times Ast \times 175\left(1 - \frac{415 \times Ast}{20 \times 2500 \times 175}\right)$$

$$662.52 = Ast - 4.47 \times 10^{-5}\, Ast^2$$

$$4.47 \times 10^{-5}\, Ast^2 - Ast + 662.52 = 0$$

$Ast_1 = 21687.97\ mm^2$ (reject)

$Ast_2 = 683.396\ mm^2$ > Astmin OK

Take dia of bars = 8mm

$$\text{spacing} = \frac{\frac{\pi}{4} \times (8)^2 \times 2500}{683.396} = 183.88$$

provide 8mm dia bars @ 170 mm c/c

<u>r/f for -ve M in middle strip →</u>

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

$$32392089.85 = 0.87 \times 415 \times A_{st} \times 175 \left(1 - \frac{A_{st}\ 415}{20 \times 2500 \times 175}\right)$$

$$512.665 = A_{st} - 4.47 \times 10^{-5} A_{st}^2$$

$$4.47 \times 10^{-5} A_{st}^2 - A_{st} + 512.665 = 0$$

$A_{st1} = 21846.38$ (rejected)

$A_{st2} = 524.98$ (accepted) but < $A_{st\,min}$

so provide $A_{st\,min}$

$A_{st} = 660\ mm^2$

Take dia of bars = 8 mm

$$\text{spacing} = \frac{\frac{\pi}{4} \times (8)^2 \times 2500}{660} = 190\ mm$$

provide 8mm dia bars @ 180 mm c/c

r/f for +ve M in midde strip →

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

$$27907031.25 = 0.87 \times 415\, A_{st} \times 175 \left(1 - \frac{A_{st} \times 415}{20 \times 2500 \times}\right.$$

$$4.47 \times 10^{-5} A_{st}^2 - A_{st} + 441.68 = 0$$

$A_{st_1} = 21920.6$ (reject)

$A_{st_2} = 450.76\ mm^2$ (Accept) but < $A_{st\,min}$

so provide $A_{st\,min}$

$A_{st} = 660\ mm^2$

Take dia of bars = 8 mm

$$\text{Spacing} = \frac{\frac{\pi}{4} \times (8)^2 \times 2500}{660} = 190\ mm$$

provide 8mm dia bars @ 180 mm c/c

→ Spacing max = 2 × Slab thickness   Page no - 59 Clause no 31.7.1

= 2×220

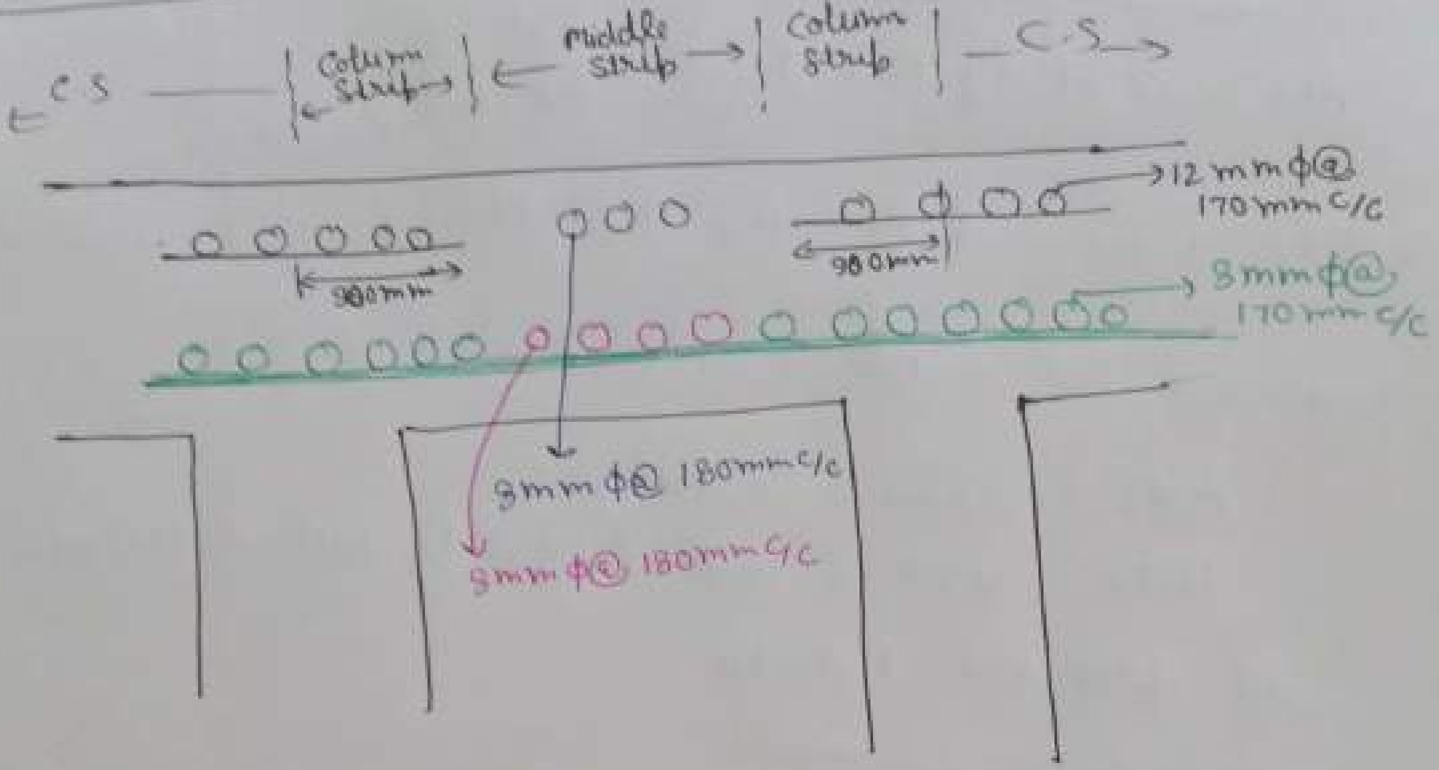
= 440 mm

all spacing are within it. → OK

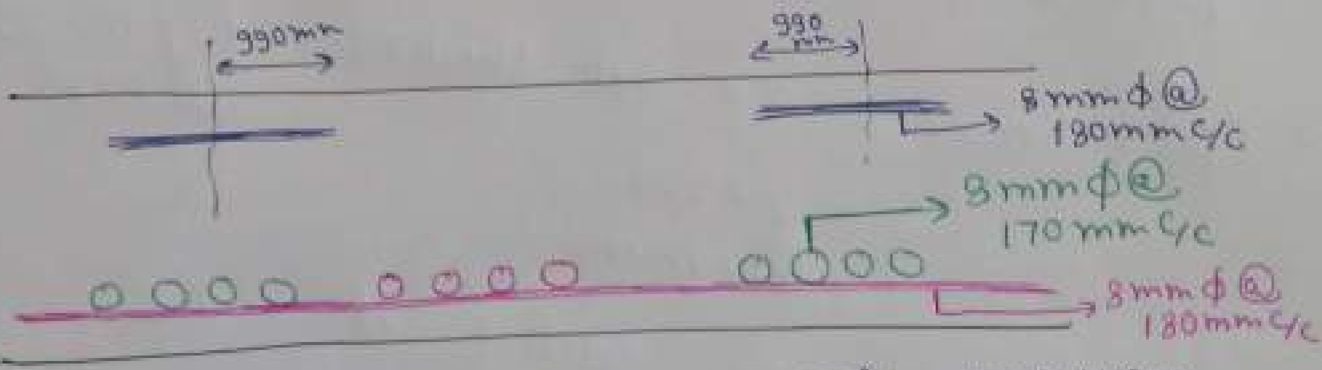
Step - 8 → Detailing diagram →

Scanned with CamScanner

Column Strip –



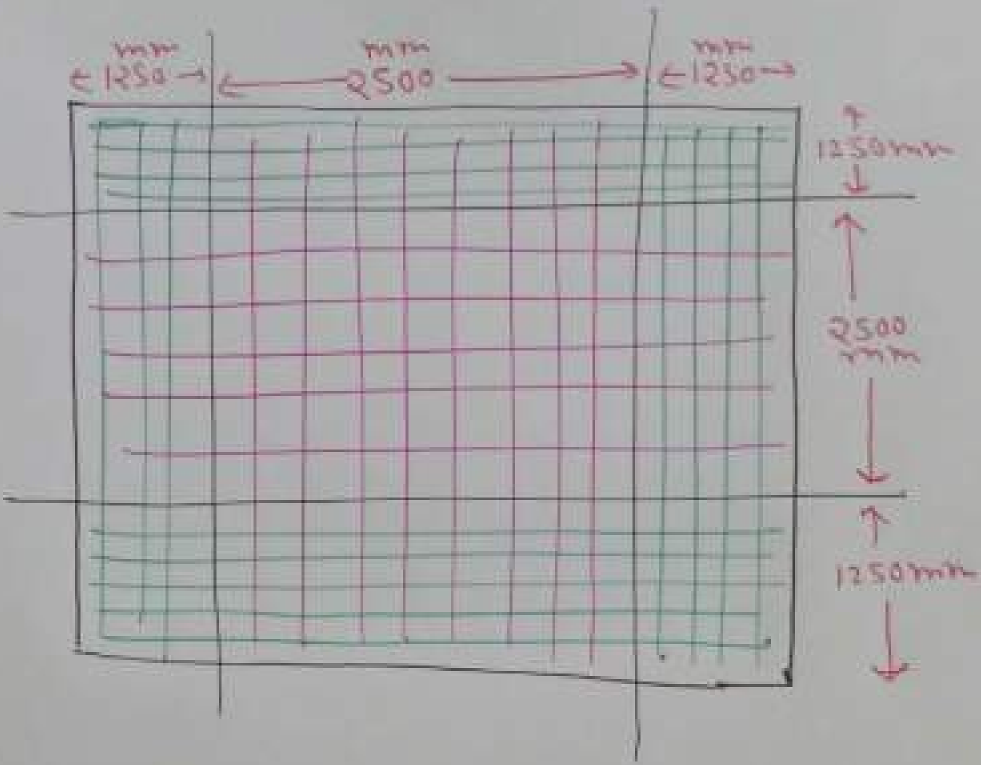
Middle Strip –



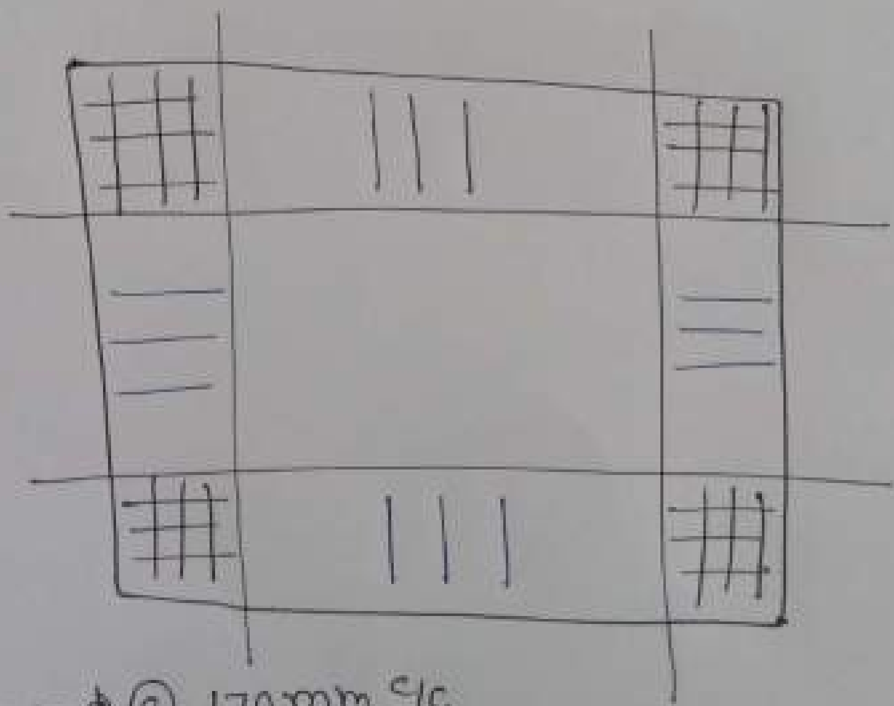
for –ve BM r/f column strip distance = $0.20 l_n = 0.20 \times 4500$
$= 900$ mm

for –ve BM r/f middle strip distance = $0.22 l_n = 0.22 \times 4500$
$= 990$ mm

Down side of panel →



up side of panel →



12 mm ϕ @ 170mm c/c

8 mm ϕ @ 180 mm c/c

8 mm ϕ @ 170 mm c/c

8 mm ϕ @ 180 mm c/c