Project - (Reinforced Concrete Design)

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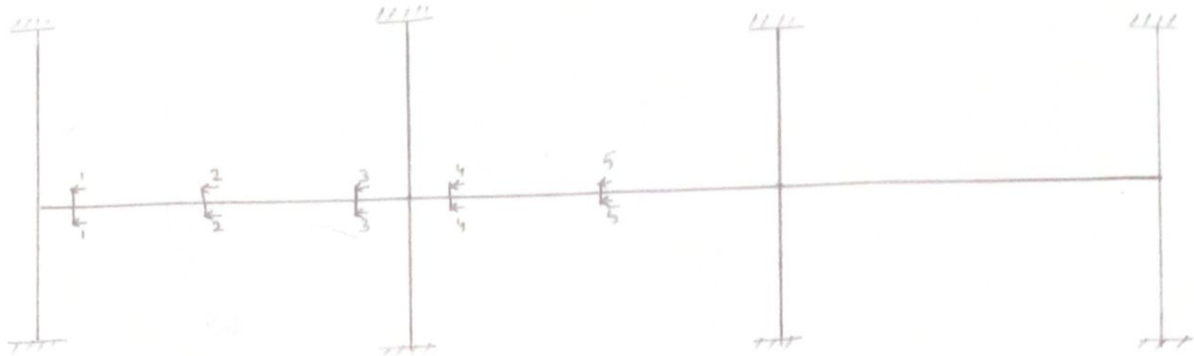
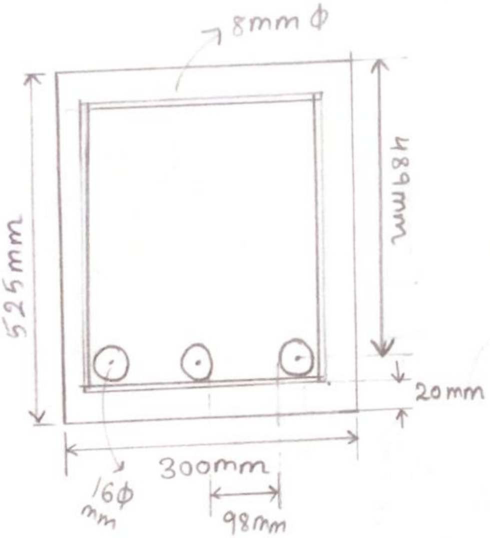
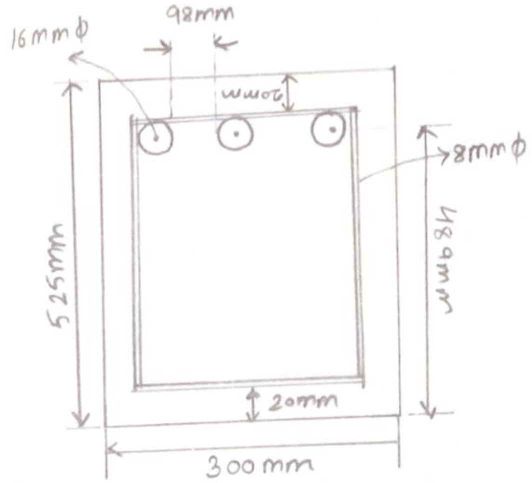


Beam cross-sections :-

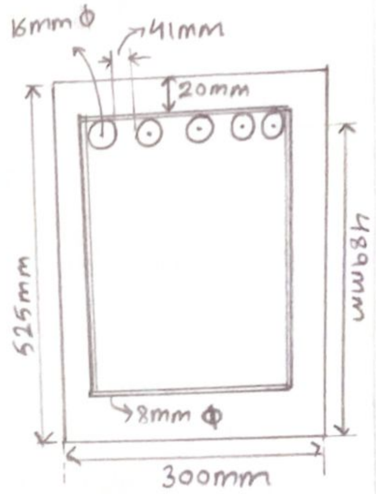
(i) Section :- 2-2



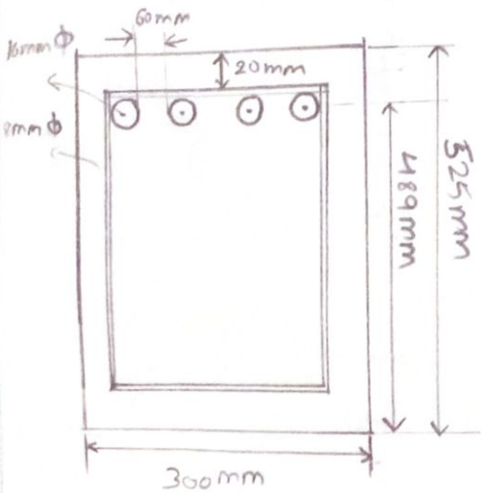
(ii) Section :- 1-1 :-



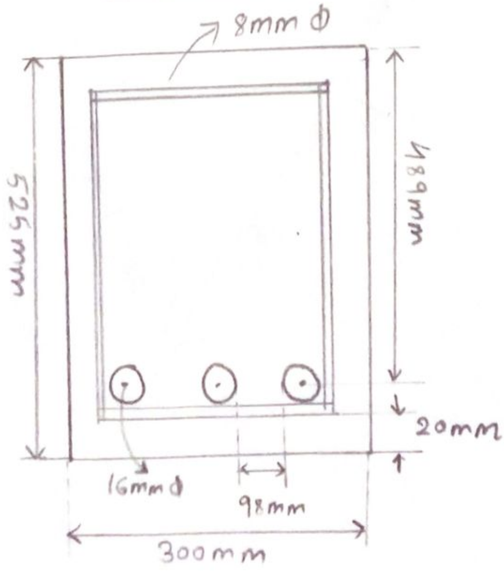
(iii) Section :- 3-3



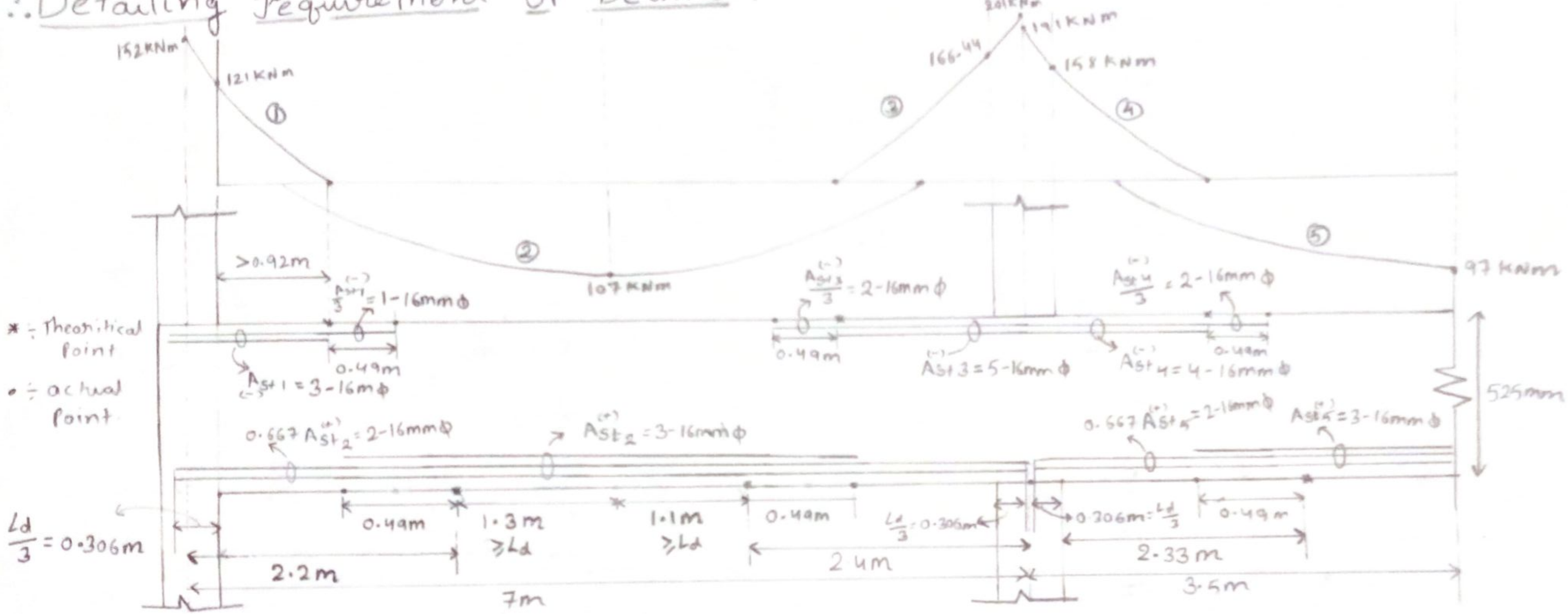
(iv) Section: 4-4 :-



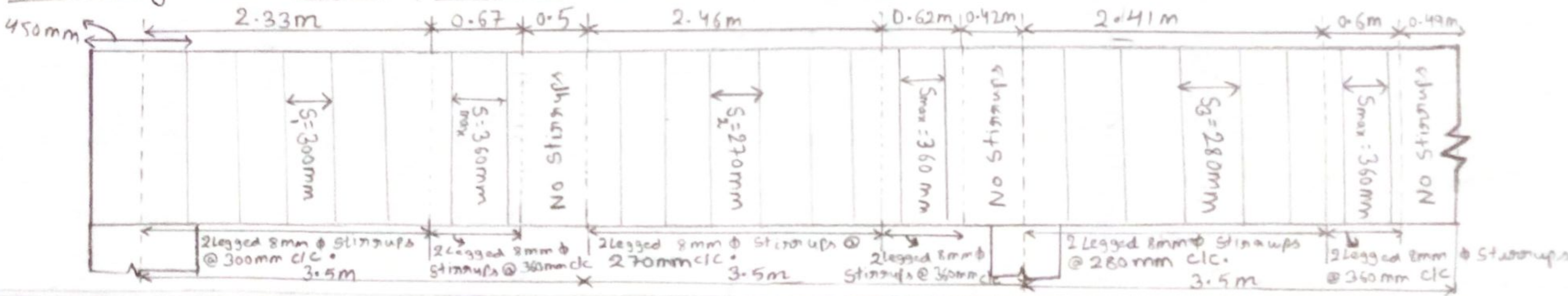
(v) Section :- 5-5 :-



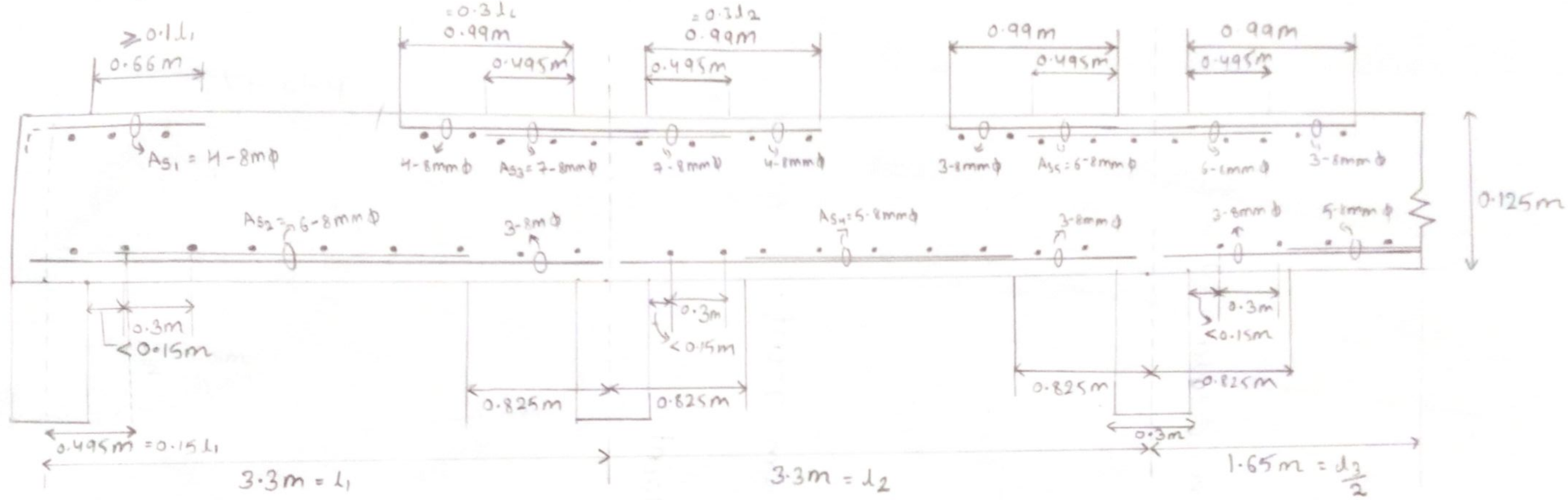
∴ Detailing requirement of beams ÷



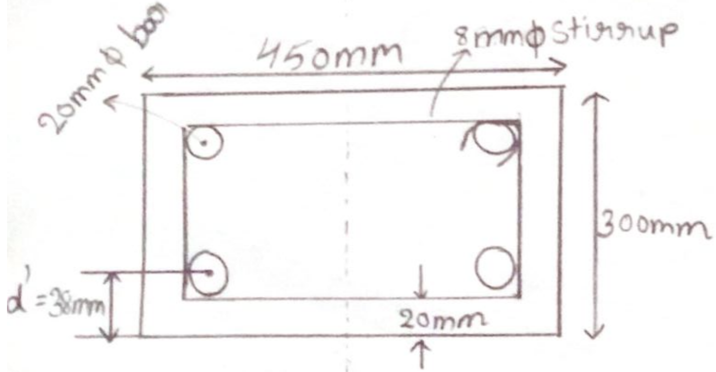
∴ Detailing of Shear Reinforcement of Beams ÷



Detailing requirement of one way Slabs :-

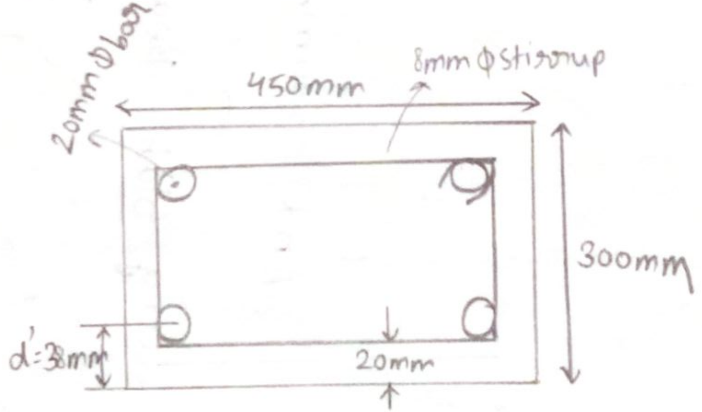


Coloumn Cross Sections :- (Symmetric about 1-1)



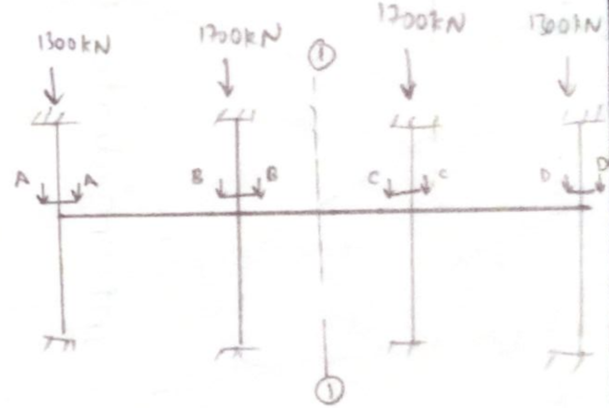
- End coloumns

(Section A-A, D-D)

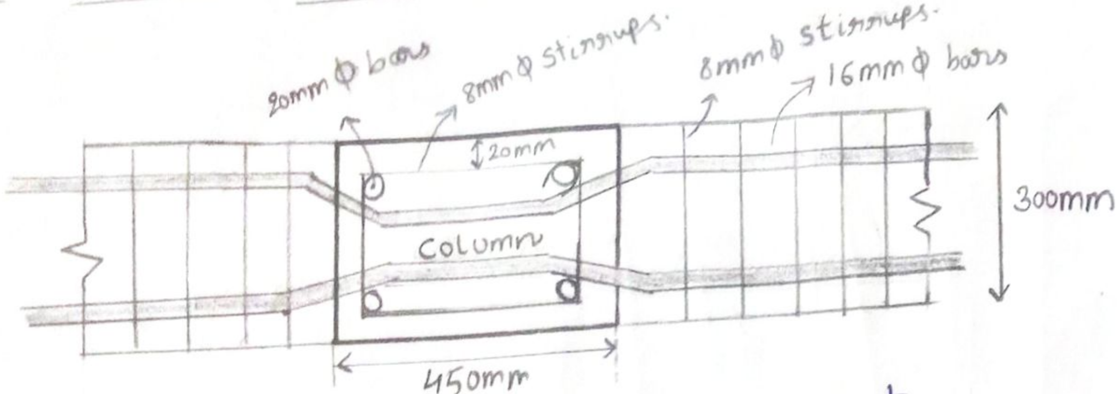


- Intermediate coloumns

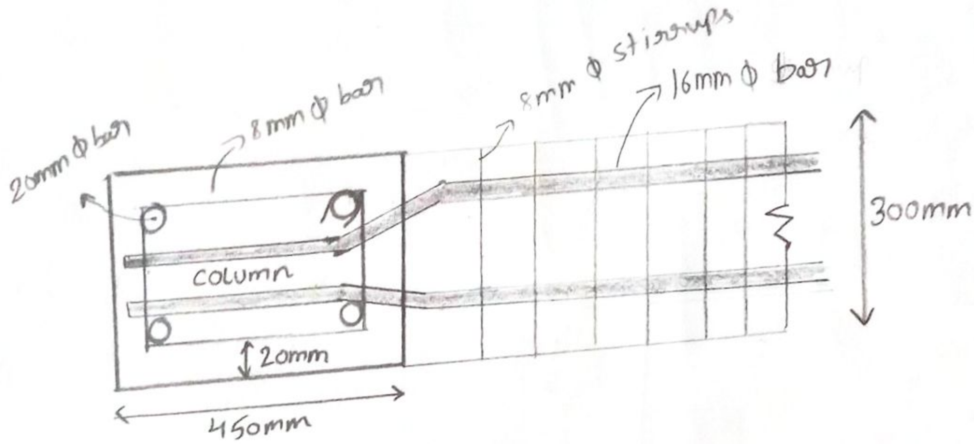
(Sections B-B, C-C)



∴ Beam-Coloumn Joints :

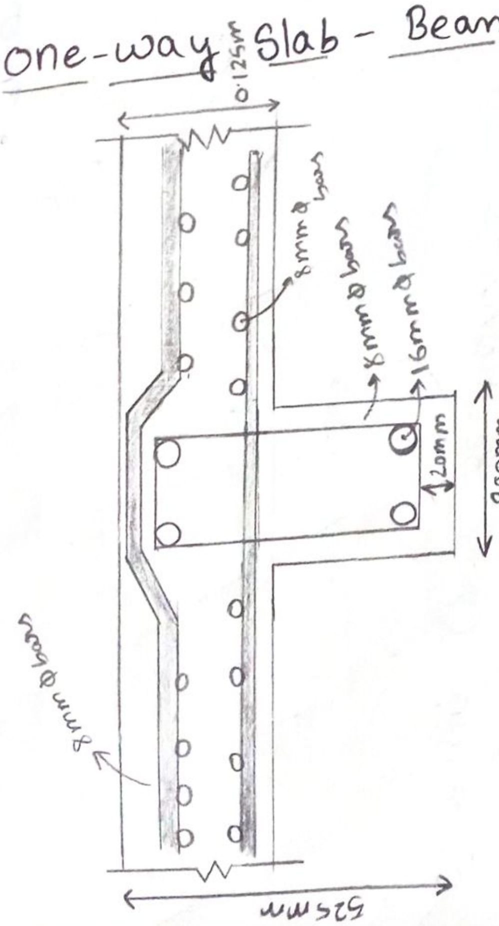


Intermediate Beam Coloumn Joint

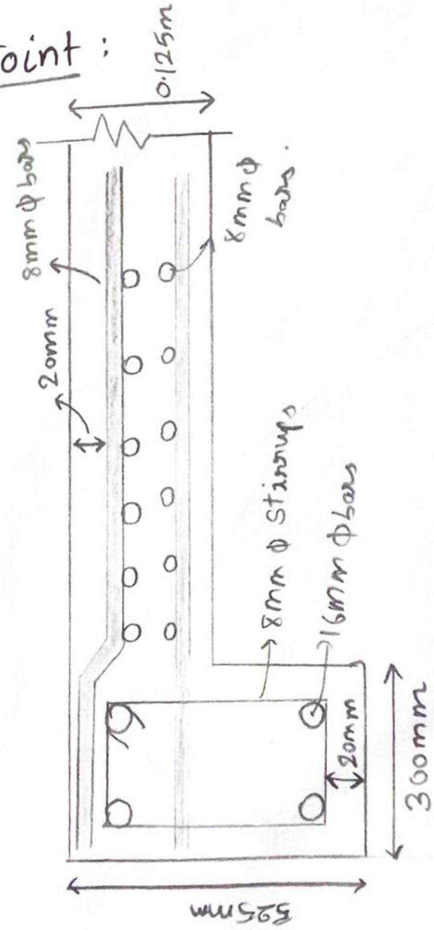


End coloumn - Beam Joint.

∴ One-way Slab - Beam Joint :



Intermediate



At End

Assumptions:

Concrete - M45 { $f_{ck} = 45\,MPa$ }

Steel Rebar - Fe500 { $f_y = 500\,MPa$ }

Consider Mild Exposure - 20mm

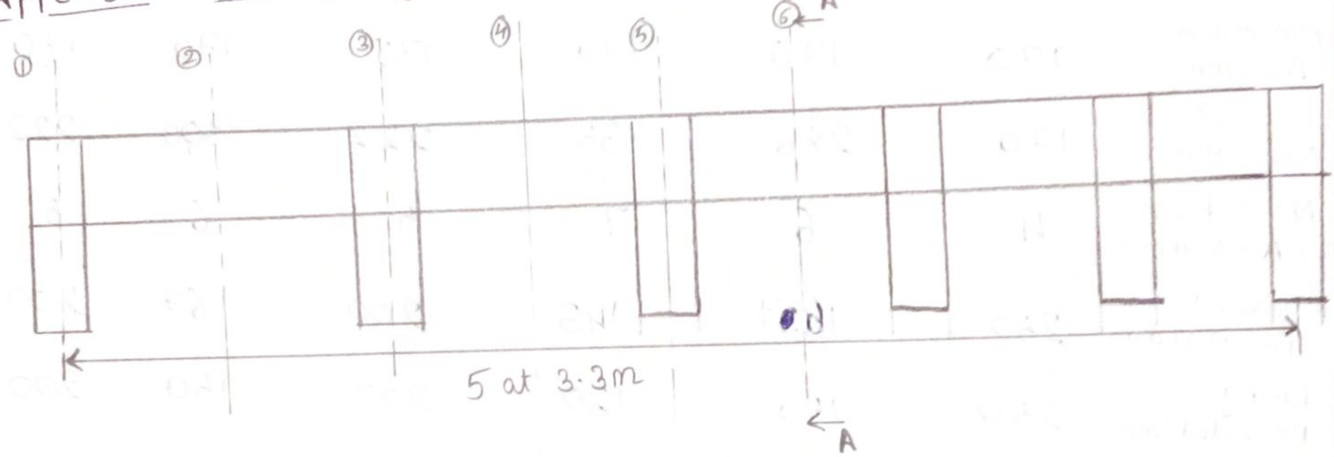
Diameter of Stirrup = 8mm

Diameter of Beam bars (flexure bars) = 16mm

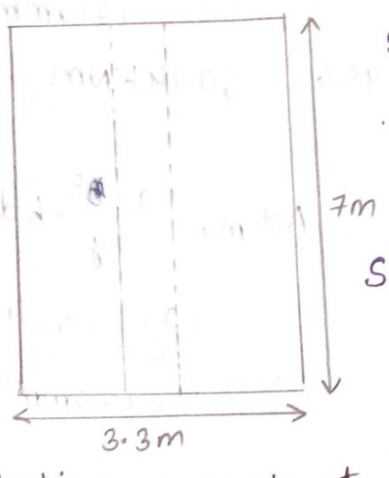
} Beams.

Diameter of slab bars = 8mm → Slab

Appendix for design of one-way slab:



{ Slab is Symmetric about Section A - A }



Since;

$\therefore \boxed{l_y > 2 l_x}$

$(\because 7 > 2 \times 3.3)$

So, The slab is one-way slab

$W_{self} = 25 \times 0.125 = 3.2\ KN/m^2$

$W_{u,DL} = 1.5(2 + 3.2) = 7.8\ KN/m^2$

$W_{u,LL} = 1.5(3) = 4.5\ KN/m^2$

$(\because LL = 3\,KN/m^2 ;\ DL = 2\,KN/m^2)$

Finding Moment at all Sections:- (by using Table-12)

(1) $M_{u(1)} = -\left(\dfrac{W_{u,DL}}{24} + \dfrac{W_{u,LL}}{24}\right)L^2 = -5.6\ KNm$

(2) $M_{u(2)} = \left(\dfrac{W_{u,DL}}{12} + \dfrac{W_{u,LL}}{10}\right)L^2 = 12\ KNm$

(3) $M_{u(3)} = -\left(\dfrac{W_{u,DL}}{10} + \dfrac{W_{u,LL}}{9}\right)L^2 = -14\ KNm$

4) $M_{u(4)} = +\left(\dfrac{W_{u,DL}}{16} + \dfrac{W_{u,LL}}{12}\right)L^2 = 9.4\ KNm$

5) $M_{u(5)} = -\left(\dfrac{W_{u,DL}}{12} + \dfrac{W_{u,LL}}{9}\right)L^2 = -12.6\ KNm$

6) $M_{u(6)} = +\left(\dfrac{W_{u,DL}}{16} + \dfrac{W_{u,LL}}{12}\right)L^2 = 9.4\ KNm$

∴ Since, Slab is Symmetric about section A-A, so we design half of the slab and rest is the replica of it.

Given :-

Slab thickness $(D) = 125mm$; cover $= 20mm$; $b = 1000mm$

So,
Effective depth $(d) = 125 - 20 - \frac{8}{2} = 101 \approx \underline{100mm}$

Location	1	2	3	4	5	6
M_u (kNm)	-5.6	12	14	9.4	-12.6	9.4
Required Area (mm^2)	131	286	335	222	300	222
Minimum Ast (mm^2)	170	170	170	170	170	170
Provided Area (mm^2)	170	286	335	222	300	222
No. of Bars ($A = 50mm^2$)	4	6	7	5	6	5
Required Spacing (mm)	250	167	143	200	167	200
Spacing Provided (mm)	250	150	120	200	150	200

Sample calculations :-

$P_{t,lim} = 41.61 \times \left(\frac{45}{500}\right) \times 0.456 = 1.71\%$; $A_{st,lim} = \frac{1.71 \times 1000 \times 100}{100}$
$= 1710 mm^2$

$M_{u,lim} = 0.87 \times 500 \times 1710 \times (100 - 0.42 \times 100 \times 0.456) = \underline{60.14 kNm}$

So,
$x_u = \frac{0.87 \times f_y \times A_{st}}{0.362 \times f_{ck} \times b} = \frac{0.87 \times 500 \times A_{st}}{0.362 \times 45 \times 1000} = 0.027 A_{st}$; $A_{st,min} = \frac{0.85}{f_y} \times b d$

$= \frac{0.85}{500} \times 1000 \times 1000$

$\Rightarrow \underline{170mm^2}$

$M_{uR} = M$

$0.87 \times f_y \times A_{st} \times (d - 0.42 \times x_u) = M \times 10^6$

$\therefore \boxed{4.93 A_{st}^2 - 43500 A_{st} = M \times 10^6}$ for :- $M = 5.6 kNm \Rightarrow \boxed{A_{st} = 131 mm^2}$
$< A_{st\,min} = 170mm^2$

No. of bars :- $\frac{170}{50} \approx 4$ bars

Required Spacing :- $\frac{1000}{n} = \frac{1000}{4} = \underline{\underline{250mm}}$

Secondary Reinforcement :- (Shrinkage Steel)

$A_{st,min} = 170\ mm^2$

$$\text{Spacing } (S_{max}) = \max \begin{cases} 3d = 3\times100 = 300 \\ 300 \end{cases} \Rightarrow \underline{300\,mm}$$

Appendix for flexure design of beam :

Assumptions :-

Total depth of Beam $(D) = 400 + 125 = 525\,mm$

Effective depth of Beam $(d) = 525 - 20 - 8 - \frac{16}{2} = 489\,mm$

for Fe 500 ; $\frac{x_{u,max}}{d} = 0.456$; $x_{u,max} = 0.456 \times 489 \simeq 223\,mm$

$$P_{t,lim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{u,max}}{d}\right)$$

$$= 41.61\left(\frac{45}{500}\right)(0.456) \simeq 1.71\%$$

$$A_{st,lim} = \frac{1.71 \times b \times d}{100} = \frac{1.71\times300\times489}{100}$$

$$\simeq \underline{2509\,mm^2}$$

$$M_{u,lim} = 0.87 f_y A_{st,lim} \times (d - 0.42\,x_{u,max})$$

$$= 0.87\times500\times2509 \times (489 - 0.42\times223) \simeq \underline{432\,KNm}$$

So, Now,

$\therefore$ Clause 22.6.1, says that for monolithic construction, the moments computed at the face of the supports shall be used in the design of the members.

from Bending moment envelope : (Using STADD PRO)

M_u at Section 1 (at face of support) $\Rightarrow$ 121 KNM

M_u at Section 2 $\Rightarrow$ 107 KNM

M_u at Section 3 (at face of support) $\Rightarrow$ 166.44 KNM

M_u at Section 4 (at face of Support) $\Rightarrow$ 158 KNM

Obtained from STADD.

M_u at Section 5 $\Rightarrow$ 97 KNM

$\Rightarrow$ Since, Bending moment envelope is symmetric at mid way so we design half and rest is the replica of it.

$x_u = \frac{0.87 f_y A_{st}}{0.362 f_{ck} b} = \frac{0.87 \times 500 \times A_{st}}{0.362 \times 45 \times 300} \Rightarrow 0.09 A_{st}$

so, $M_R = 0.87 \times f_y \times A_{st} (d - 0.42 x_u)$ $(\because M_R = M_u)$

so, $\boxed{16.443 A_{st}^2 - 212715 A_{st} + M \times 10^6 = 0}$

for section - 1 :-

$16.443 A_{st}^2 - 212715 A_{st} + 121 \times 10^6 = 0$

$\boxed{A_{st_1} = 597 \text{ mm}^2}$

- No. of bars $= \frac{597 \times 4}{\pi (16^2)} \approx$ 3 bars
- spacing ~~provided~~ = 98mm

for Section - 2 :-

$16.443 A_{st}^2 - 212715 A_{st} + 525 \times 10^6 = 0$

$\boxed{A_{st_2} = 525 \text{ mm}^2}$

- No. of bars $= \frac{597 \times 4}{\pi \times 16^2} \approx$ 3 bars
- Spacing Provided = 98mm

for section - 3 :-

$16.443 A_{st}^2 - 212715 A_{st} + 167 \times 10^6 = 0$

$\boxed{A_{st_3} = 840 \text{ mm}^2}$

- No. of bars $= \frac{840 \times 4}{\pi \times 16^2} \approx$ 5 bars
- Spacing Provided = 41mm

for section - 4 :-

$16.443 \times A_{st}^2 - 212715 A_{st} + 158 \times 10^6 = 0$

$\boxed{A_{st_4} = 792 \text{ mm}^2}$

- No. of bars $= \frac{792 \times 4}{\pi \times 16^2} \approx$ 4 bars
- Spacing Provided = 60mm

for section - 5 :-

$16.443 A_{st}^2 - 212715 A_{st} + 97 \times 10^6 = 0$

$\boxed{A_{st_5} = 474 \text{ mm}^2}$

- No. of bars $= \frac{474 \times 4}{\pi \times 16^2} \approx$ 3 bars
- Spacing Provided = 98mm

$\therefore$ Minimum Horizontal Spacing:

Clause (26.3.2)

min spacing $= \max \begin{cases} \text{dia. of bar} = 16\text{mm} \\ \text{Nominal size of aggregate} + 5\text{mm} = 20+5 = 25 \\ \to 20\text{mm} \end{cases}$

Take - max(16, 25) $\Rightarrow$ $\boxed{\text{Min H.S} = 25\text{mm}}$

~~Appendix for Shear design of Beam :-~~

Appendix for curtailment of the Beam :- (Clause : 26.2.3)

Section - 1 :- $P_{t_1} = \frac{A_{st_1}}{bd} \times 100 = \frac{597}{300 \times 489} \times 100 \Rightarrow 0.407\%$

Section - 2 :- $P_{t_2} = \frac{A_{st_2}}{bd} \times 100 = \frac{525}{300 \times 489} \times 100 \Rightarrow 0.36\%$

Section :- 3 :- $P_{t_3} = \frac{A_{st_3}}{bd} \times 100 = \frac{840}{300 \times 489} \times 100 \Rightarrow 0.57\%$

Section :- 4 :- $P_{t_4} = \frac{A_{st_4}}{bd} \times 100 = \frac{792}{300 \times 489} \times 100 \Rightarrow 0.54\%$

Section :- 5 :- $P_{t_5} = \frac{A_{st_5}}{bd} \times 100 = \frac{474}{300 \times 489} \times 100 \Rightarrow 0.32\%$

Development Length (L_d) = $\frac{\phi \sigma_s}{4\tau_{bd}}$ = $\frac{16\times0.87\times500}{4\times1.9}$ ≈ 916 mm

for M45 concrete τ_{bd} = 1.9 MPa. So, Take $\boxed{L_d = 920\,mm}$

Note: Section 1, 3, 4 cannot be curtailed, since the required development length will not be available after curtailment. So, we only curtail $\frac{2}{3}$rds of the steel in section 2 & 5.

Section-2: (curtailment)

so, $\frac{2}{3}\times A_{st2} \Rightarrow \frac{2}{3}\times525 = 350\,mm^2$ ⇒ No. of bars = $\frac{350\times4}{\pi\times16^2}$ ≈ 2 bars

so, The moment carrying capacity dropped to:

$M_R = -16.443\,A_{st}^2 + 212715\,A_{st}$

$= (212715\times350 - 16.443\times350^2)\,10^{-6}$

$\Rightarrow 72.435 \approx 73\,KN$

Section-5: (curtailment)

so, $\frac{2}{3}\times A_{st5} = \frac{2}{3}\times474 = 316\,mm^2$ ⇒ No. of bars = $\frac{316\times4}{\pi\times16}$ ≈ 2 bars

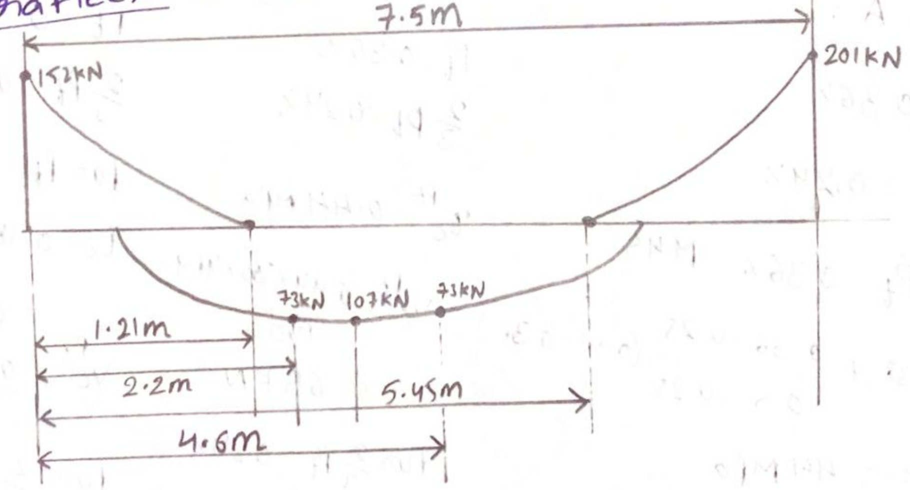
The moment carrying capacity dropped to:

$M_R = (-16.443\times316^2 + 212715\times350)\times10^{-6} = 66\,KN$

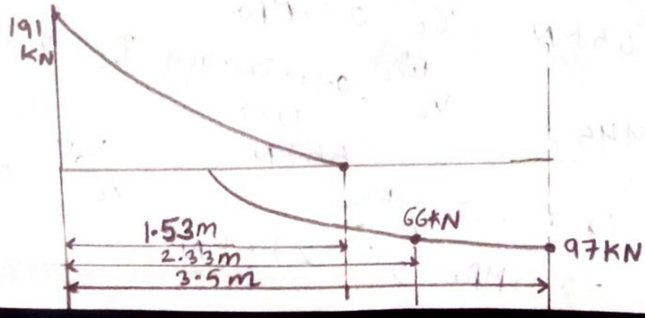
Now find contraflecture Points and theoritical cutoff-Points:

Using STADD:

Span-1:



Span-2:
(only half)



Finding actual cut-off Points:

→ from the theoritical cut-off Point we should extend L_d distance.

so, $L_d = \max\{d \text{ or } 12\phi\} = \max\{489mm, 12\times16\} = \max\{489mm, 192mm\}$

so, $\boxed{L_d = 489mm \approx 0.49m}$

→ 26.2.3.4 : For Negative moment, Steel should be extended beyond Point of contraflexure by distance L_a :

so, $L_a = \max\begin{cases} d = 489mm \\ 12\phi = 12\times16 = 192\ mm \\ \text{clear span}/16 = \frac{7000}{16} = 437.5 \end{cases} = 489mm \approx \underline{0.49m}$

Appendix For Shear design of Beam:

Assumptions:

$b = 300mm$ $\qquad A_{st} = \frac{\pi\times8^2}{4} \approx 50mm^2$

$d = 489mm$ $\qquad A_{sv} = 2\times50 = 100mm^2$

for minimum Shear reinforcement:

$$S_{v,max} = \frac{2.175\times A_{sv}\times f_y}{b} = \frac{2.175\times100\times500}{300} = 362.5mm \approx \underline{360mm}$$

for - A:

$P_t = 0.36\%$

$\frac{2}{3}P_t = 0.24\%$

for; $P_t = 0.36\%$, M45 :

$\tau_c = 0.38 + \frac{0.36-0.25}{0.5-0.25}(0.51-0.38)$

$\tau_c = 0.44\ MPa$

$V_c^{Pt} = \frac{0.44\times300\times489}{1000} \approx 65KN$

for; $\frac{2}{3}P_t = 0.24\%$, M45;

$\tau_c = 0.3 + \frac{(0.24-0.15)}{(0.25-0.15)}(0.38-0.3)$

$= 0.37MPa$

for: B :

$P_t = 0.36\%$

$\frac{2}{3}P_t = 0.24\%$

$\tau_c^{Pt} = 0.44\ MPa$

$V_c^{Pt} = \frac{0.44\times300\times489}{1000}$

$\approx \underline{65KN}$

for $\frac{2}{3}P_t = 0.24\%$

$\tau_c^{\frac{2}{3}Pt} = 0.37MPa$

$V_c^{(Pt\frac{2}{3})} = \frac{0.37\times300\times489}{1000}$

$\approx \underline{55KN}$

$\frac{V_c^{(Pt\frac{2}{3})}}{2} = \underline{27.5KN}$

for - C :

$P_t = 0.32\%$

$\frac{2}{3}P_t = 0.213\%$

for $P_t = 0.32$; M45

$\tau_c = 0.38 + \frac{0.32-0.25}{0.5-0.25}(0.51-0.38)$

$= 0.411MPa$

$V_c^{Pt} = \frac{0.411\times300\times489}{1000} \approx 60KN$

for; $\frac{2}{3}P_t = 0.213\%$, M45:

$\tau_c = 0.3 + \frac{(0.213-0.15)}{(0.25-0.15)}(0.38-0.3)$

$= 0.35MPa$

$V_c^{\frac{2}{3}Pt} = \frac{0.35\times300\times489}{1000} \Rightarrow \underline{52KN}$

for - A :

$$V_c^{\frac{2}{3}Pt} = \frac{0.37 \times 300 \times 489}{1000}$$

$\Rightarrow 55KN$

$$\frac{V_c^{\frac{2}{3}Pt}}{2} = \frac{55}{2} = 27.5\ KN$$

for - B :

$V_c^{\frac{2}{3}Pt} = 55KN$

$$\frac{V_c^{\frac{2}{3}Pt}}{2} = \frac{55}{2} = 27.5KN$$

for - c :

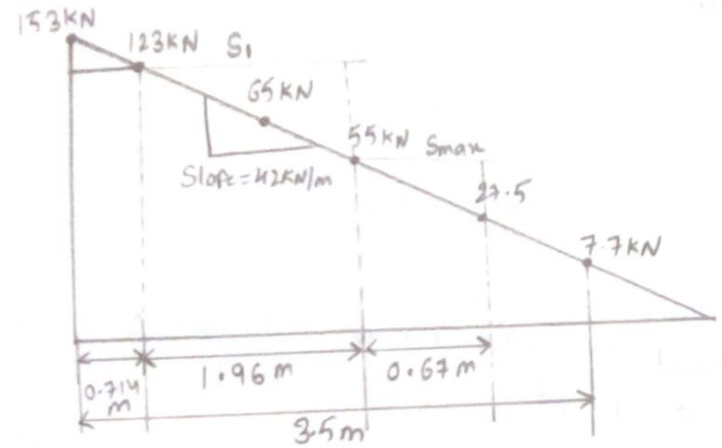
$V_c^{\frac{2}{3}Pt} = 52KN$

$$\frac{V_c^{\frac{2}{3}Pt}}{2} = 26\ KN$$

$\Rightarrow$ At ends, due to compression from supports the shear demand reduces, we should consider shear at a distance of 'd' from face of the support.

so, $x = d + \frac{450}{2} = 0.489 + 0.225 = \underline{\underline{0.714m}}$

for - A :



(i) $y = 153 - 0.714 \times 42$
$= 123KN$

(ii) $x = \frac{153-55}{42} = 2.33m$

(iii) $x = \frac{153-27.5}{42} = 3m$

$$S_1 = \frac{0.87 \times 500 \times 100 \times 489}{(123-55) \times 10^3}$$

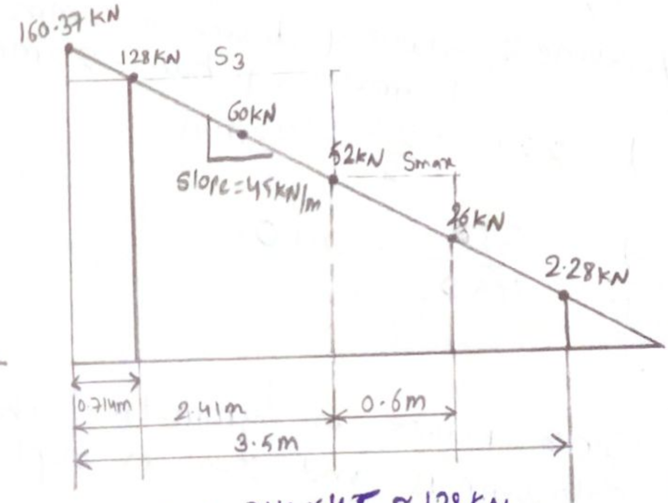
$312.8mm \approx 300mm$

{It should not cross 300mm)

$\boxed{S_{max} = 360mm}$

$$Slope = \frac{153 - 7.7}{3.5} \approx \underline{42KN/m}$$

for - C :



(i) $y = 160.37 - 0.714 \times 45 \approx 128KN$

(ii) $x = \frac{160.37 - 52}{45} = 2.41m$

(iii) $x = \frac{160.37 - 26}{45} = 3.01m$

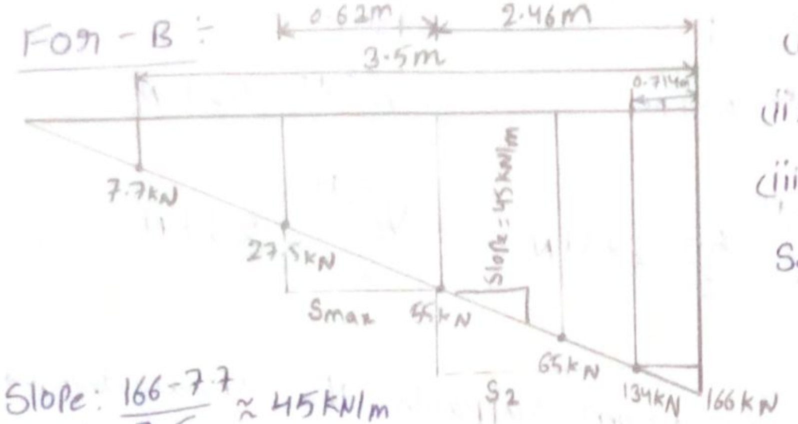
$$S_3 = \frac{0.87 \times 500 \times 100 \times 489}{(128-52) \times 10^3}$$

$\Rightarrow \cancel{268}\ 279.8 \approx 280mm$

$\boxed{S_{max} = 360mm}$

$$\therefore Slope = \frac{160.37 - 2.28}{3.5} \approx \underline{45KN/m}$$

For - B :



Slope: $\frac{166-7.7}{3.5} \approx 45\,kN/m$

(i) $y = 166 - (0.714 \times 45) \approx 134\,kN$

(ii) $x = (166-55)/45 \approx 2.46\,m$

(iii) $x = (166-27.5)/45 \approx 3.08\,m$

$S_2 = \frac{0.87 \times 500 \times 100 \times 419}{134-55} \approx 270\,mm$

$\boxed{S_{max} = 360\,mm}$

Appendix for column design :

Assumptions :

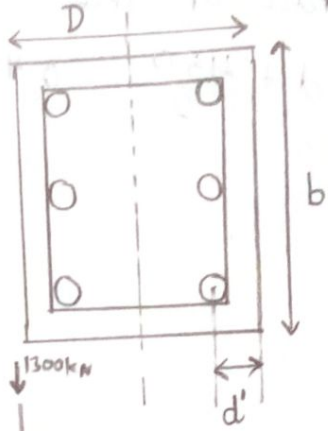
$f_y = 500\,MPa$; $D = 450\,mm$

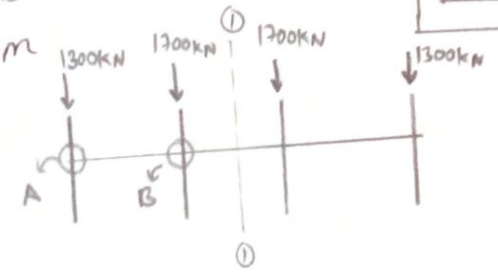
$f_{ck} = 45\,MPa$; $b = 300\,mm$

Assume : Stirrup diameter = 8mm
Diameter of bar = 20mm

$d' = 20 + 8 + \frac{20}{2} = 38\,mm$

$\frac{d'}{D} = \frac{38}{450} \approx 0.10$





→ Since, figure is symmetric about 1-1, so design coloum A & coloumn B and rest is replica of it.

End coloumn (A) :

$P_u = 1300 + 153 = 1453\,KN$

$M_u = 152\,KN$

$\frac{P_u}{f_{ck} b D} = \frac{1453 \times 10^3}{45 \times 300 \times 450} \approx 0.24$

$\frac{M_u}{f_{ck} b D^2} = \frac{152 \times 10^6}{45 \times 300 \times 450^2} \approx 0.06$

from SP-16 graphs : $\frac{P}{f_{ck}} = 0.02 \Rightarrow P = 0.9\%$

so, $A_s = \frac{0.9 \times 300 \times 450}{100} \Rightarrow 1215\,mm^2$

No. of bars (n) $= \frac{1215 \times 4}{\pi \times 20^2} \approx 4$ bars

Spacing Provided : $(300 - 40 - 16) - 2 \times 20 = 204\,mm$

(Place the above bars symmetrically in column)

for intermediate coloumn (B) :

$P_u = 1700 + 160 + 166 = 2026\,KN$

$M_u = 201 - 191 = 10\,KN$

$\frac{P_u}{f_{ck} b D} = \frac{2026 \times 10^3}{45 \times 300 \times 450} \approx 0.33$

$\frac{M_u}{f_{ck} b D^2} = \frac{10 \times 10^6}{45 \times 300 \times 450^2} \approx 0.004$

from SP-16 : $\frac{P}{f_{ck}} = 0.02 \Rightarrow P = 0.9\%$

so, $A_s = \frac{0.9 \times 300 \times 450}{100} \Rightarrow 1215\,mm^2$

No. of bars (n) $= \frac{1215 \times 4}{\pi \times 20^2} \approx 4$ bars

Spacing : 204mm

(Place bars symmetrically)