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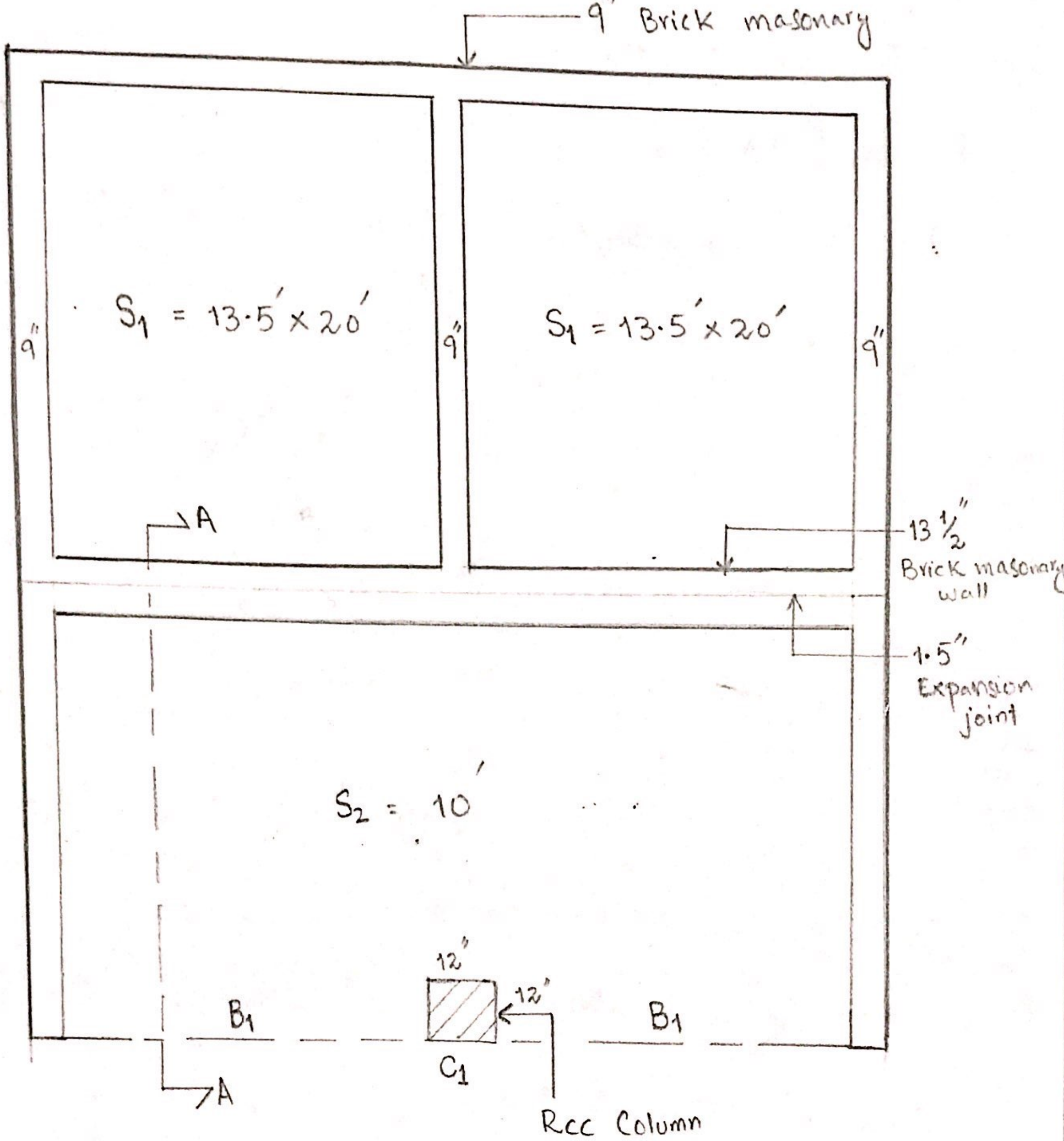
Name : Shah Rukh Khan
Subject :- Advance Conc design.

Question: Dimensions of five Marla plot :

Live load = 40 psf



Solution :-

Concrete Compressive Strength (f_c') = 3ksi

Steel yield Strength (f_y) = 40ksi

Load on Slab :

4" thick mud.

2" thick brick tile.

(1) Design of slab "S_2"

(a) Sizes l_b/l_a = $27.75/9$ = 3.083 > 2

"Oneway Slab" Assume 5" Slab.

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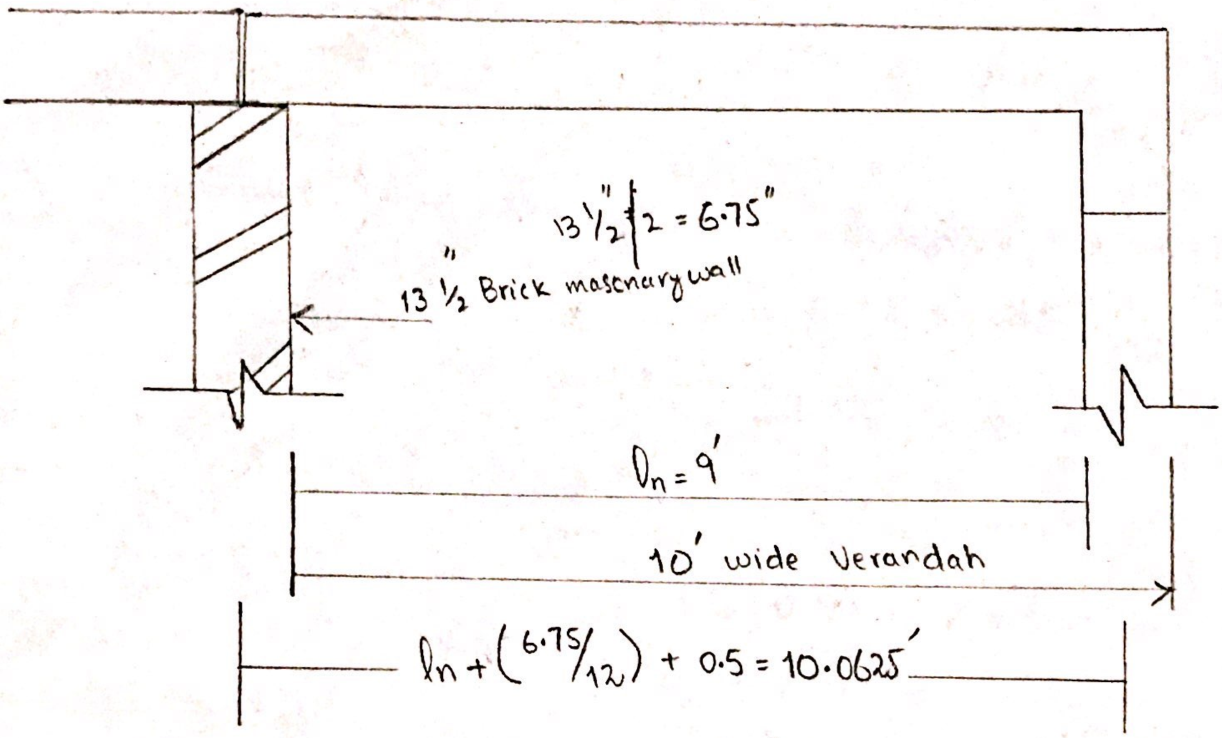
(2.)

Span length for end span according to ACI 8.7 is minimum of : (i) $L = L_n + h_f = 9 + (5/12) = 9.41'$

(ii) c/c distance between supports =



Section A-A (See fig 1 above)

Therefore $l = 9.41'$

Slab thickness $(h_f) = (l/20) \times (0.4 + f_y/100000)$ [For $f_y < 60000$ psi]

$= (9.41/20) \times (0.4 + 40000/100000) \times 12$

$= 4.5168''$ (Minimum requirement of ACI 9.5.2.1)

Therefore take $h_f = 5''$

$d = 5 - 0.75 - (3/8)/2 = 4.08$

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Table 1.1 Dead load

Material	Thickness(in)	γ (kcf)	Load = $\gamma \times$ thickness (ksf)
Slab	5	0.15	$0.15 \times (5/12) = 0.0625$
Mub	4	0.12	$0.12 \times (4/12) = 0.04$
Brick Tile	2	0.12	$0.12 \times (2/12) = 0.02$

Service Dead load (D.L) = $0.0625 + 0.04 + 0.02$

$= 0.1225$ ksf

Service live load = 40 psf 0.04 ksf

factored load (W_u) = $1.2 D.L + 1.6 L.L$

$= 1.2(0.1225) + 1.6(0.04)$

$= 0.211$ ksf

Step #03 Analysis

$M_u = W_u l^2/8$ (l = Span length of slab)

$M_u = 0.211 \times (9.41)^2/8 = 2.33$ ft-k/ft $= 27.96$ in-k/ft

Step #04 Design

$As_{min} = 0.002\, b h_f$ (for $f_y = 40$ ksi, ACI)(10.5.4)

$= 0.002 \times 12 \times 5 = 0.12 \text{ in}^2$ $\quad a = As_{min} f_y / 0.85 f'_c b$

$= 0.12 \times 40 / (0.85 \times 3 \times 12) = 0.156$ in

$\phi M_{n(min)} = \phi As_{min} f_y (d - a/2)$

$= 0.9 \times 0.12 \times 40 \times (4 - 0.156/2)$

$= 16.94$ in-k $< M_u$

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Therefore;

* $A_s = M_u / \phi f_y (d - a/2)$

Take $a = 0.2d$ here $M_u =$

$A_s = 22.33 / 0.9 \times 40 \times (4(0.2 \times 4)/2$

$= 0.179 \text{ in}^2$

* $a = 0.179 \times 40 / (0.85 \times 3 \times 12) = 0.224 \text{ in}$

$A_s = 22.33 / \{0.9 \times 40 \times (4 - (0.224/2)\}$

$= 0.155 \text{ in}^2$

* $a = 0.155 \times 40 / (0.85 \times 3 \times 12) = 0.202 \text{ in}$

$A_s = 22.33 / \{0.9 \times 40 \times 4 - (0.202/2)\}$

$= 0.155 \text{ in}^2$ OK

Using $1/2'' \phi$ (#4) {#13, 13 mm} with bar Area $A_b = 0.20 \text{ in}^2$

Spacing = Area of one bar $(A_b) / A_s$

$= [0.20 (\text{in}^2) / 0.155 \text{ in}^2/\text{ft}] \times 12 = 15.48 \text{ in}$

Using $3/8'' \phi$ (#3) {#10, 10 mm}, with bar Area

$A_b = 0.11 \text{ in}^2$

Spacing = Area of one bar $(A_b) / A_s$

$= [0.11 / 0.155] \times 12 = 7.516 \approx 6''$

Finally use #3 @ 6" c/c (#10 @ 150 mm c/c)

Shrinkage Steel or temperature Steel (A_{st}):

$$A_{st} = 0.002\, b h_f$$

$$A_{st} = 0.002 \times 12 \times 5 = 0.12 \text{ in}^2$$

Using 3/8" ϕ (#3) { # 10, 10mm }, with bar area $A_b = 0.11 \text{ in}^2$

Spacing = Area of one bar (A_b) / A_{smin}

$= (0.11/0.12) \times 12 = 11''$ c/c

finally Use #3 @ 9" c/c (#10 @ 225 mm c/c)

* Max. Spacing for main Steel in one way slab according to ACI 7.6.5 is minimum of:

(i) $3h_f = 3 \times 5 = 15''$

(ii) 18"

Therefore 6" Spacing is OK.

* Max. Spacing for Shrinkage Steel in one way slab according to ACI 7.12.2 is mini. of.

(i) $5h_f = 5 \times 5 = 25''$

(ii) 18"

There fore 9" Spacing ok.

(2) Design of Slab "S_1" :-

(a) Step No 1: Sizes $l_b/l_a = \frac{20}{13.5} = 1.48 < 2''$

Minimum depth of two way Slab is given by

$$h_{min} = \frac{\text{Perimeter}}{180} = 2 \times (20 \times 13.5) \times 12 / 180 = 36$$

Assume ⇒ $h = 5''$

Step #02:- Loads

Factored load = W_u = $W_{u,dl} + W_{u,ll}$

$W_u = 1.2D.L + 1.6L.L$

$W_u = 1.2(0.1225) + 1.6(0.04)$

$= 0.211$ ksf

Step #03:- Analysis

$M_{a,neg} = C_{a,neg} W_u l_a^2$

$M_{a,neg} = C_{a,neg} W_u l_a^2$

where, C_a, C_b = tabulated moment coefficient as given in Appendix A

W_u = Ultimate uniform load, Psf l_a, l_b = length of clear spans in short and long direction respectively.

Therefore, for the design problem under discussion, $m = l_a / l_b = 13.5/20 = 0.67 \approx 0.65$

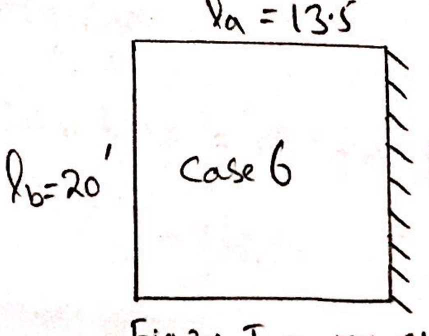


Fig 3: Two way slab (S_2)

Table 1.2: Moment coefficient for Slab

coefficient for neg. moments in slab		case #6 [m = 0.65] coefficient for dead load positive moment in slab		coefficient for live load positive moment in slabs	
$C_{a,neg}$	$C_{b,neg}$	$C_{a,dl}$	$C_{b,dl}$	$C_{a,ll}$	$C_{b,ll}$
0.093	0.000	0.054	0.007	0.064	0.010

$M_{a,neg} = C_{a,neg} \times w_u \times l_a^2$

$= 0.088 \times 0.211 \times 13.5^2 = 3.384 \text{ ft-k} = 40.60 \text{ in-k}$

$M_{b,neg} = C_{b,neg} \times w_u \times l_b^2 = 0 \times 0.211 \times 20^2 = 0 \text{ ft-k}$

$M_{a,pos,dl} = C_{a,pos,dl} \times w_{u,dl} \times l_a^2$

$= 0.054 \times 0.147 \times 13.5^2 = 1.44 \text{ ft-k} = 17.28 \text{ in-k}$

$M_{b,pos,dl} = C_{b,pos,dl} \times w_{u,dl} \times l_b^2$

$= 0.007 \times 0.147 \times 20^2 = 0.411$

$M_{a,pos,ll} = C_{a,pos,ll} \times w_{u,dl} \times l_a^2$

$= 0.064 \times 0.064 \times 13.5^2 = 0.746$

$M_{b,pos,ll} = C_{b,pos,ll} \times w_{u,ll} \times l_b^2$

$= 0.010 \times 0.064 \times 20^2 = 0.256 \text{ ft-k} = 3.072 \text{ in-k}$

$M_{a,neg} = 40.60 \text{ in-k}, \ M_{b,neg} = 0, \ M_{a,pos(dl+ll)} = 17.28 + 0.746 = 18.026 \text{ in-k}$

$M_{b,pos,(dl+ll)} = 0.411 + 3.072 = 3.483 \text{ in-k}.$

<u>Step #04</u>:: Designing.

$A_{s\,min} = 0.002\, b h_f = 0.002 \times 12 \times 5 = 0.12 \text{ in}^2$

$a = A_{s\,min} f_y / (0.85 f_c' b) = 0.12 \times 40 / (0.85 f_c' b) = \frac{0.12 \times 40}{(0.85 \times 3 \times 12)}$

$= 0.156 \text{ in}$

$\phi M_{n(min)} = \phi A_{s\,min} f_y (d - a/2) = 0.9 \times 0.12 \times 40 \times (4 - 0.156/2)$

$= 16.941 \text{ in-k}$ (capacity provided)

$\phi M_{n\,min} = \phi A_{s\,min} f_y (d - a/2)$

$= 0.9 \times 0.12 \times 40 \times (4 - 0.156/2) = 16.94$ in-k

$\phi M_n(min)$ is greater than $M_{b,pos,(dl+ll)}$ but less

$M_{a,pos,(dl+ll)} = 3.483$ in-k $< \phi M_n(min)$

Therefore, $A_{s_{min}} = 0.92$ in^2 governs

Using 3/8" ϕ (#3) {#10, 10mm}, with bar Area $A_b = 0.11$ in^2

Spacing $= (0.11/0.12) \times 12 = 11"$

Max. Spacing according to ACI 13.3.2 for two way slab is, $2h_f = 2 \times 5 = 10"$

Therefor max. spacing of 10" governs

finally use #3 @ 9" c/c (#10 @ 225mm c/c)

Provide #3 @ 9" c/c as neg. reinforcement along the longer direction.

$M_{a,pos,(dl+ll)} = 18.02$ in-k $> \phi$

M_n Let $a = 0.2d = 0.2 \times 4 = 0.8$ in

$A_s = 1.53 \times 12 / 0.9 \times 40 \times (4 - 0.8/2)$

$= 0.146$ in^2, $a = \dfrac{0.146 \times 40}{0.85 \times 3 \times 12}$

$= 0.191$ in

$A_s = 1.53 \times 12 / \{0.9 \times 40 \times 4 - 0.191/2\} = 0.131$ in^2

$a = 0.131 \times 40 / 0.85 \times 3 \times 12 = 0.171$ in

$A_s = 1.53 \times 12 / 0.9 \times 40 \times (4 - 0.30/2) = 0.131$ in^2

Ok.

Using 3/8" ϕ (#3) {#10, 10mm} with bar Area

$A_b = 0.11 \text{ in}^2$ Spacing $= 0.11 \times 12/0.131 = 10.07"$

$\simeq 9"$ c/c

finally use #3 @ 9" c/c (#10 @ 225mm c/c)

$M_{a,neg} = 40.60$ in-k $= 3.384$

Let $a = 0.2d = 0.2 \times 4 = 0.8$ in

$A_s = 3.384 \times 12 / 0.9 \times 40 \times (4 - 0.8/2)$

$= 0.285 \text{ in}^2$ $a = 0.28 \times 40 / 0.85 \times 3 \times 12 = 0.33$ in

$A_s = 3.38 \times 12 / 0.9 \times 40 \times 4 - 0.33/2 = 0.26 \text{ in}^2$

ok

Using 3/8" ϕ (#3) {#10, 10mm} with bar

Area $A_b = 0.11$, Spacing $= 0.11 \times 12 / 0.26$

$= 5.9" \simeq 4.5"$ c/c

Finally use #3 @ 4.5 c/c (#10 @ 110mm c/c)

③ Beam Design (2 span contineous):

Exterrior Support = 9" BMW

$f_c' = 3$ ksi

$f_y = 40$ ksi

Column = 12" × 12

Step # 01: Sizes

$h_{min} = L / 18.5 \, l =$

End Span $= 12.375 - (13.5/12)/2 = 11.815$

Let depth = 18" of Beam

l_n + depth of Beam = $11.875 + 18/12 = 13.375'$

c/c distance b/w beam support = 12.375

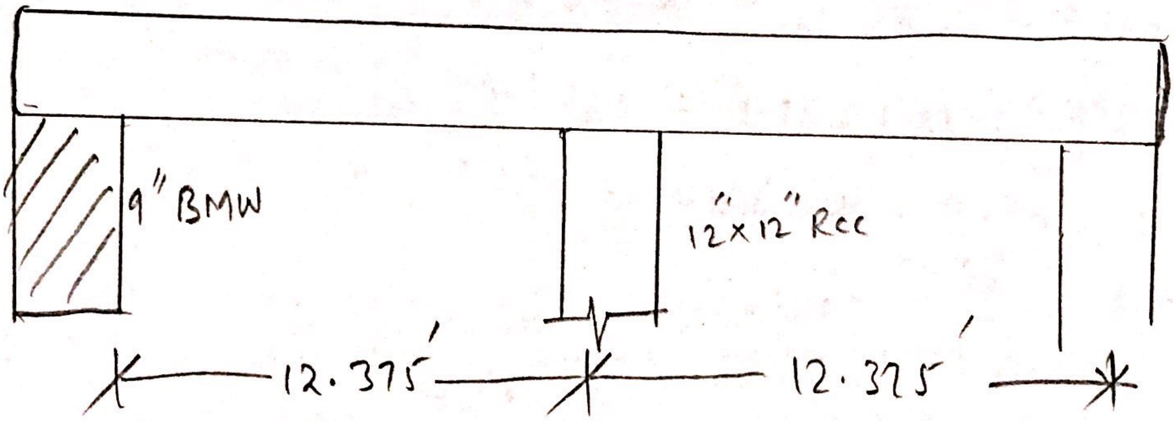
therefore = $l = 12.75'$ + $(4.5/12) = 12.75'$

Depth (h) = $(12.75/18.5) \times (0.4 + 40000/100000) \times 12$

= 6.62" (Min req of)

Take h = 1.5' = 18"

d = h − 3 = 15"



Step # 02 :: Loads

D.L = 0.625 + 0.04 + 0.02 = 0.1225 ksf

L.L = 40 psf or 0.04 Ksf

D.L from Slab = 0.1225 × 5 = 0.6125 k/ft

self weight = $h_w b_w \gamma_c$ = $\left(\frac{13 \times 12}{144}\right) \times 0.15$ = 0.1625 k/ft

Total d.L = 0.775 k/ft

L.L = 0.2 k/f . w_u = 1.2 D.L + 1.6 L.L

= 1.2(0.775) + 1.6(0.20) = 1.25 k/ft

Step #03 :- Analysis

① At interior Support :-

M_{neg} = coefficient × $w_u l_n^2$

= $(1/9) \times \{1.25 \times (11.875)^2\}$

= 19.59 ft.k = 235.08 in-k

② At Mid Span :-

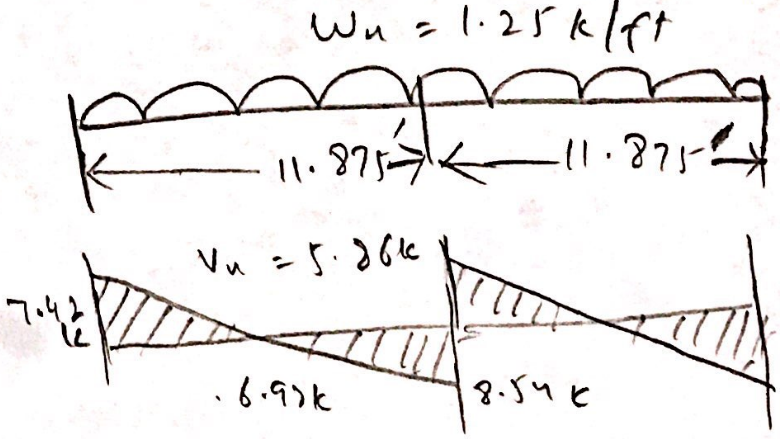
M_{pos} = Co × $w_u l_n^2$

= $(1/11) \times \{1.25 (11.875)^2\}$

= 16.02 ft.k = 192.24 in.k

V_{in} = $1.15\, w_u l_n/2$ = $1.15 \times 1.25 \times 11.875/2$

= 8.54 k V_u (int) = 8.54 − 1.25 × 1.25 = 6.97 k



(4) Design of Column:-

(i) Load on column :

$P_u = 2 V_{int} = 2 \times 8.54 = 17.08k$

Gross area of column cross section (A_g)

$= 12 \times 12 = 144 \ in^2$, $f'_c = 3ksi$

$f_y = 40ksi$

(ii) Design :-

Nominal Strength (ϕP_n) of axially loaded col.

Let $A_{st} = 1\%$ of A_g, $\phi P_n = 0.80 \times 0.65 \times \{0.85 \times 3 (144 - 0.01 \times 144) + 0.01 \times 144 \times 40\}$

$= 218.98k > (P_u = 17.08k)$ ok

$A_{st} = 0.01 \times 144 = 1.44 \ in^2$

Using 3/4" ϕ (#6) {#19, 19mm)

$A_b = 0.44 in^2$

No. of bars $= A_s / A_b$

$= 1.44 / 0.44 = 3.27 \approx 4$ bars use

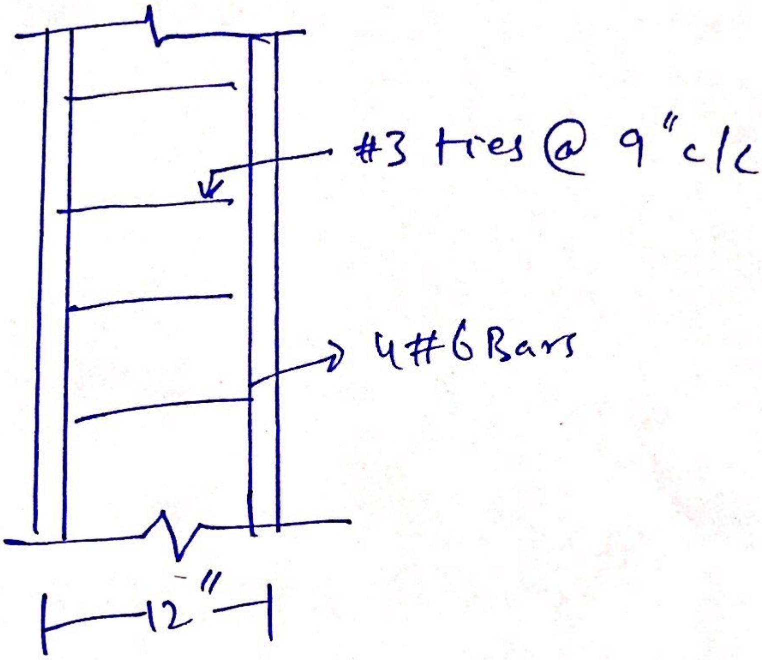
4 #6 bars {4 #19 bars, 19mm}

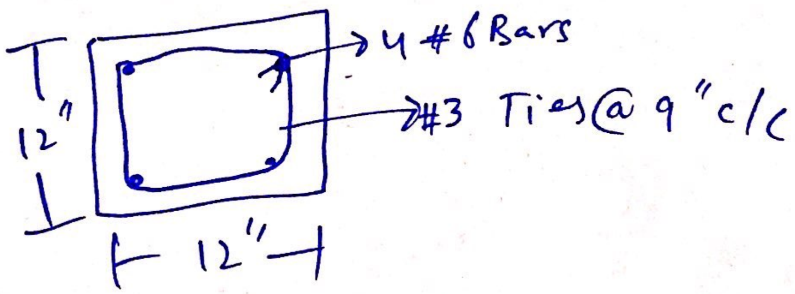
Tie bar:-

3/8" ϕ (#3) {#10, 10mm} 3/8" ϕ

(#6) {#19, 19mm} main bar

(c) Column :-





(5) Footing :-