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Subject :- Reinforcement concrete structure -

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Paper :- final term -

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Peshawar -

(15629) Roll-No

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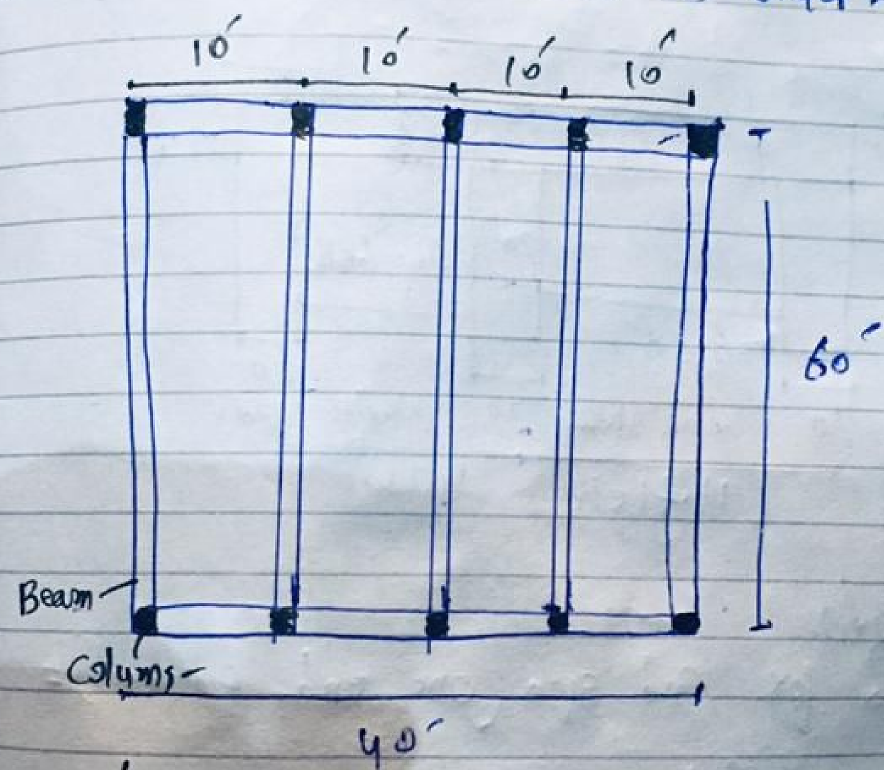
(Q# 1)

design of frame structure:

Sol:

Sir I am going to design a hall (frame structure) as under-



Covered Area-

$$60 \times 40' = \frac{2400}{275.25} = 8.71 \text{ Marla}$$

Specification:-

hall height = 15'

$f_c' = 3$ Ksi

$f_y = 40$ Ksi

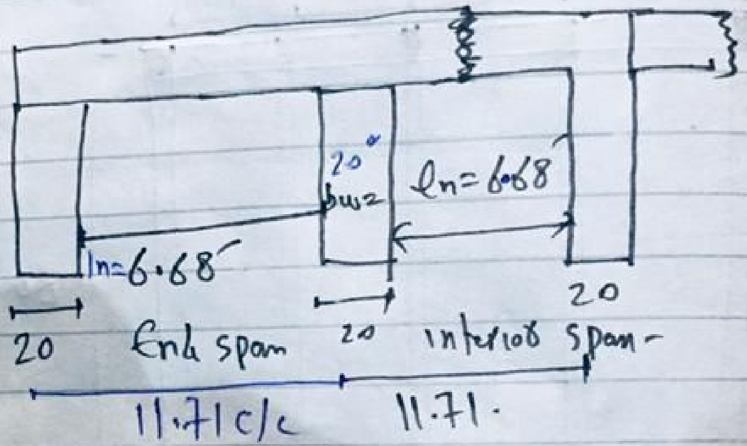
Assumed-

Beams 20"x20" (~~all~~ ~~columns~~)

~~Column~~ = ~~16x16~~

Live Load = 40ksi (As per code).

1) Slab design.



Size:

→ for End span (one end continous) =

$$l/24$$

→ for interior span (Both End continous)

$$l/28$$

Assume 6" slab thickness.

End span = l_n + depth of slab

$= 6.68 + 0.5 = 7.18'$

interior span $= 6.68 + 0.5 = 7.18'$.

Slab thickness :-

End span $= \frac{l_n}{24} \times \left(0.4 + \frac{f_y}{100000}\right)$

$= \frac{7.18}{24} \times \left(0.4 + \frac{4000}{10000}\right)$

$= \frac{7.18}{24} \times 0.44$

$= 0.1316 \times 12 = 1.57''$

interior span :- $\frac{l}{28} \times \left(0.4 + \frac{f_y}{10000}\right)$

$= \frac{7.18}{28} \times 0.44$

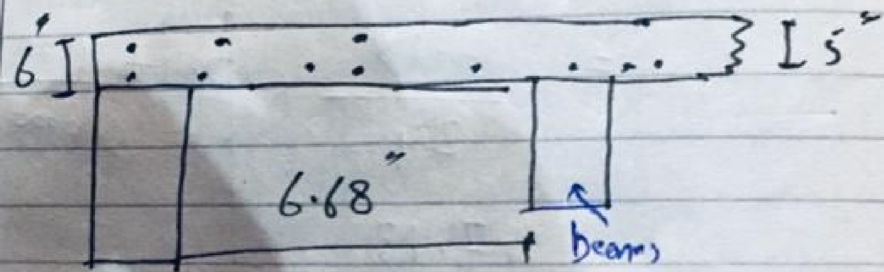
$= 0.112 \times 12$

$= 1.35''$

So as per ACI-9.5.2.1
Minimum thickness = 6"
so

$h_f = 6"$

effective depth = 6" − 1" = $\boxed{5"}$



Loading :

Slab = 6"
tile = 2"

Material	thickness	γ (k/ft)	Load
Slab	6"	0.15	6/12 x 0.15 = 0.0
tiles	2"	0.12	2/12 x 0.12 0.0

Now

$D.L = 0.075 + 0.02 = 0.095\, ksf$ — (A)

Live load $= 0.04$ ksf (code).

Service load $= w_s = D.L + L.L.$
$= 0.125 + 0.04$
$= 0.165$ ksf.

factored load $= 1.2DL + 1.6LL.$
$= 1.2(0.095) + 1.6(0.02)$

$w_u = \boxed{= 0.146\, ksf.}$

Analysis 2.

Our slab system -

$\frac{\text{longer span}}{\text{shorter span}}$

$= \frac{60}{10} = 6$

So the slab is one way slab because the value is greater than 2

→ Exterior ends of slab are discontinuous and unrestrained.

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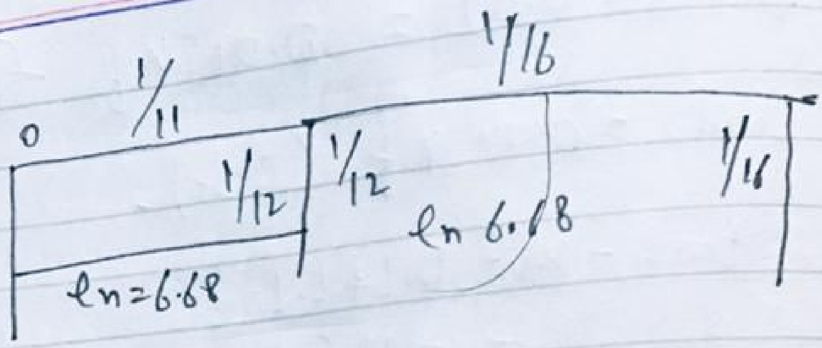

1. at interior support (Left of Support)

$M^{\ominus}_{L}$ = co-effcient x $w_u l_n^2$

$= \frac{1}{12} \times 0.146 \times (6.68)^2$

$= \frac{1}{12} \times 6.51$

$= 0.0833 \times 6.51$

$= 0.5425 \times 12$

$= \boxed{6.51 \text{ ink-ft}}$

at interior support

$M^{\ominus}_{R}$ = co-effcient x $w_u l_n^2$

$= \frac{1}{12} \times 0.146 \times (6.68)^2$

$= \boxed{6.51 \text{ ink-ft}}$

At Exterior Mid span -

$M+ \quad$ coefficient $\times w_u(l_n)^2$

$= 1/11 \times 0.146(6.68)^2$

$= 1/11 \times 6.51$

$\boxed{= 7.10 \text{ in-k/ft}}$

at interior Mid span -

$1/16 \times 0.146(6.68)^2$

$= 1/16 \times 6.51$

$= 0.4063 \times 12$

$\boxed{= 4.83 \text{ in-k/ft}}$

design 2

$A_{s\,min} = 0.002\, b h_f$

$= 0.002 \times 20 \times 6$

$= 0.24 \text{ in}^2/\text{ft}$

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$$\cancel{a = A_{s\,min} f_y \left(d - \frac{a}{2}\right)}$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.244 \times 40}{0.85 \times 3 \times 20}$$

$$a = 0.1913'$$

$$\phi M_n = \phi A_{s\,min} f_y \left(d - \frac{a}{2}\right)$$

$$= 0.9(0.244)(40) \times \left(5 - \frac{0.1913}{2}\right)$$

$$= 8.784 \times 4.90$$

$$= 43.04 \text{ in-k/ft.}$$

Use 3@4 where area is

$$A_b = 0.20 \text{ in}^2$$

Spacing

Area of one bar A_b/A_s.

$= 0.20/0.244 \times 12 = 9.8"$

=

using 3/8" @ #3 $A_b = 0.11$.

$\frac{0.11}{0.24} \times 12$

$= 5.5$ in.

~~using~~ = #3 9" c/c for both neg and positive moment.

shrinkage reinforcement.

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

$A_{st} = 0.002 \times 20 \times 6$

$= 0.244\ in^2/ft$.

$\boxed{A_{st} = A_{smin} = 0.24\ in^2/ft}$

Minumum spacing $= 3h_f = 3 \times 6 = 18"$
so ok

Minumum spacing of temprature steel -
$5h_f = 5 \times 6 = 30"$

Spacing $= 9"$ is ok.

(2) **Beam design:-**

Assume column dimention $= 20" \times 20"$

$f_c' = 3ksi$

$f_y = 40ksi$

Beams spacing $= 10'$

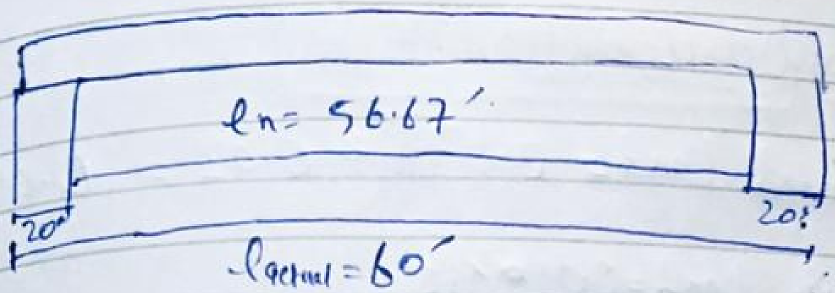
Size:

Minimum thiksne of Beam $=$
$h_{min} = 18"$

depth of beam $5'$ (Assumed -

$l_n = 60' + 5' = 65'$

c/c distan $= 60 - \frac{(20/12)}{2} = 58.33 ft$



$\boxed{l =}$ ~~[illegible]~~ 58.33 c/c.

~~$h_{min} = 18"$~~

also

Simply supported -

$$h = \frac{l}{16}\left(0.4 + \frac{f_y}{100000}\right)$$

$$= \frac{58.33}{16}\left(0.4 + \frac{40000}{100000}\right) \times 12$$

$$3.64\,(0.44)\,(12)$$

$$\boxed{= 19.24"}$$

So we will use the same depth as we Assumed -

$h = 5' = 60"$

$d = h - 3 = 57$

eq (A)

(2) Load calculation 2

dead load = 0.075 + 0.02 = 0.095 k

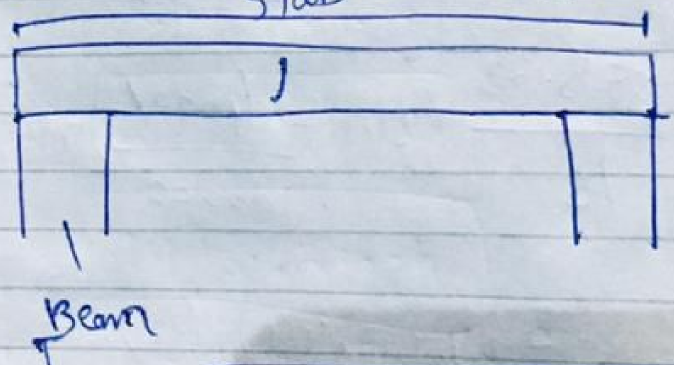
Live Load = 0.040 ksf

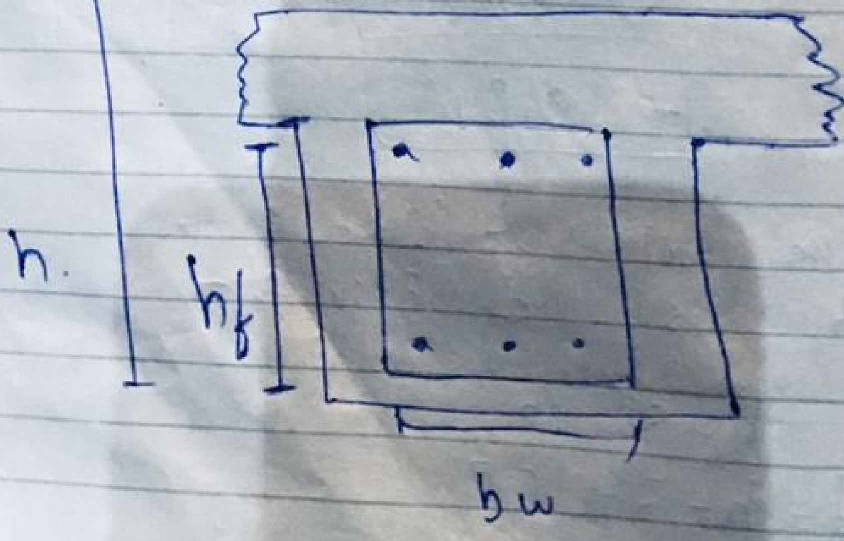
Beam is supporting 10' slab

therefore loads per running foot are as follow.

D.L. from slab ~~per foot~~ = 0.095 X 10

slab = 10' slab each panal -





So

D.L with slab = 0.095 x 10

= 0.95 k/ft.

Beam self weight = $h_w b_w \gamma_c$

$= \frac{54 \times 18}{144} \times 0.15$

= 1.0125 k/ft.

Total D.L = 0.95 + 1.0125

$\boxed{= 1.9625 \text{ k/ft.}}$

Total L.L = 0.04 x 10 = $\boxed{0.4 \text{ k/ft.}}$

$W_s = D.L + L.L$

$W_s = 1.9625 + 0.4$

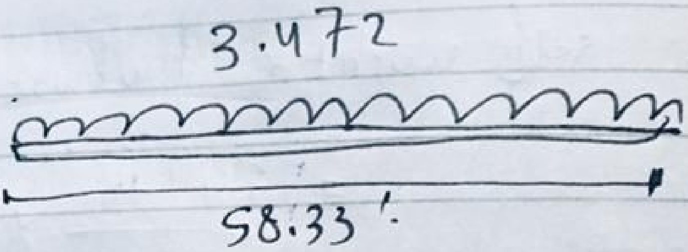
= 2.36 k/ft.

$W_u = 1.2 DL + 1.6 LL$

$W_u = 1.2(2.36) + 1.6(0.4)$

$$\boxed{w_u = 3.472\ k/ft}$$

Analysis 2



$$M_u = \frac{w_u L^2}{8}$$

$$M_u = \frac{3.472(58.33)^2}{8}$$

$$= \boxed{1476.63\ k\text{-}ft}$$

$d = 57'' = 4.75'$

Analysics2

① At Exterior Support2-

Negative

$M\ominus = \text{Co-effs} \times w_u l_n^2$

$= \frac{1}{9} \times 3.472 (58.33)^2$

$= \frac{1}{9} \times 11813.09$

$= 1312$ ft-k.

$= 15750$ in-k.

at Mid span 2

Positive

$\frac{1}{11} \times w_u (l_n)^2$

$\frac{1}{11} \times 3.472 (58.33)^2$

$= (11813.09) \times 0.09$

$= 1073.91 \times 12$

$= 12887.0$ in-k.

Design :

1. Flexural Moment :-

(a) for positive Moment -

1. $16h_f + b_w = 16 \times 6 \times 12 = 108''$

2. $c/c \text{ span}/4 = 58.33/4 = 17$

3. $10 \times 12 = 120''$

Therefore $b_{ff} = \boxed{108''}$

Step #(b)

Check if the beam is Rectangual or T-beam -

Assume $a = h_f = 6$.

$$A_s = \frac{M_u}{\phi f_y (d - a/2)}$$

$$= \frac{12887.0}{(0.90)(40)(57 - 6/2)}$$

$= 6.629 in$

Re-calculate -

~~As Ma~~

$a = A_s f_y / 0.85 f_c' b_{ff}$

$$a = \frac{6.629 \times 40}{0.85 \times 3 \times 108} = 0.96 < h_f$$

there fore beam is Rectangular

Reinforcement 2

$A_{s(min)} = \rho_{max}\, b_w d$

$\rho_{max} = 0.0203$

$A_{smin} = 0.0203 \times 12 \times 57$

$A_{smin} = 13 in^2$

A_{smax} $\rho_{max} \times b_w \times d$ -

$= 0.005 \times 12 \times 57$

$= 3.42 in^2.$

$A_{smin} < A_s < A_{smax}$ ok.

Using #8 (#25, 25mm) with

bar Area $= 0.79 in^2$

No of bar $= A_s/0.79 = \frac{6.629}{0.79}$

$= 8$ bars

Use

8#8 bar

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(i) for interior Negative -

$Mu = 15750$ in-k.

$b_w = 12$

$h = 24''$

$d = 57''$

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

Let $a = 0.2d$ -

$$A_s = \frac{15750}{0.90 \times 40 \times (\cancel{57} - 0.2 \times \cancel{57}/2)}$$

$$= \frac{15750}{36(57 - 5.7)}$$

$$= \boxed{8.52 \text{ in}^2}$$

$$a = \frac{8.52 \times 40}{0.85 \times 3 \times 12}$$

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$\boxed{a = 11.13}$

$$A_s = \frac{15750}{0.90 \times 40 \left(57 - \frac{11.3}{2}\right)}$$

$\boxed{A_s = 8.51996}$

Reinforcement:

Use 6#8 bars; - {6#25 bars

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Size column = 20"x20"   18629

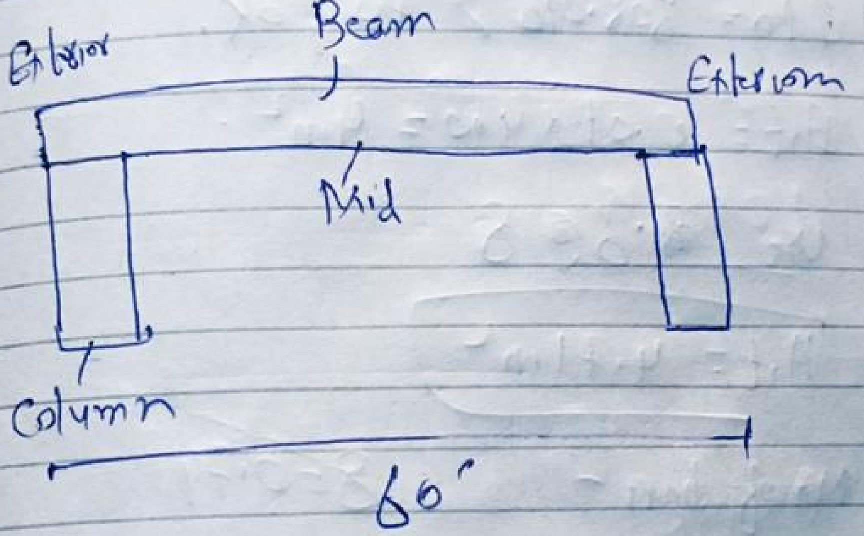
3) Design of Column 2. (for all 10 column).

gross Area of Column = 20"x20" = 400 in^2

$f_c' = 3$ Ksi

$f_y = 40$ Ksi

Load on Column -



~~Load of~~

Pu = 1312 ft-k. $\boxed{Pu = 109.33\ k.}$
Exterior

design

Nominal strength ϕP_n.

$\phi P_n = 0.80 \cdot 0.9 \{0.85 f_c (A_g - A_{st}) + A_{st} f_y\}$

for tied column

let $A_{st} = 1\%$ of A_g

$\phi P_n = 0.80 \times 0.65 \times \{0.85 \times 3 \times (400 - 0.01 \times$

$+ 0.001 \times 3100 \times 40)$.

$\phi P_n = 808.296 > P_u$ / ok.

$A_{st} = 0.01 \times 400 = 4 \, in^2$.

Use 6 @ 8

$\boxed{A_{st} = 4.71 \, in^2}$

No. of bars = $8 = 0.79$

$A_s / A_b = 4 / 0.79$

5.06

≈ 6 bars

Use

6 bars

$\boxed{8 @ 6}$

ties bars-

$3/8''$ @ (#3) ACI

Spacing of tie bars -

$16 \times$ dia of main bar $= 16 \times 3/4 = 12''$ c/c

$= 48 \times$ dia of bar $= 48 \times 3/8 = 18''$ c/c

Least column diameter $20''$ c/c

finaly use #3 tie bar @ 9/c/c

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(4) Design of footing -

Column = $20'' \times 20''$

$f_c' = 3\,ksi$

$f_y = 40\,ksi$

$q_a = 2.204\,k/ft^2$

factored load on column -

$\boxed{109.3k}$

(1) Size

Assume

$h = 20''$

$d_{avg} = h - 3$

$= 17''$

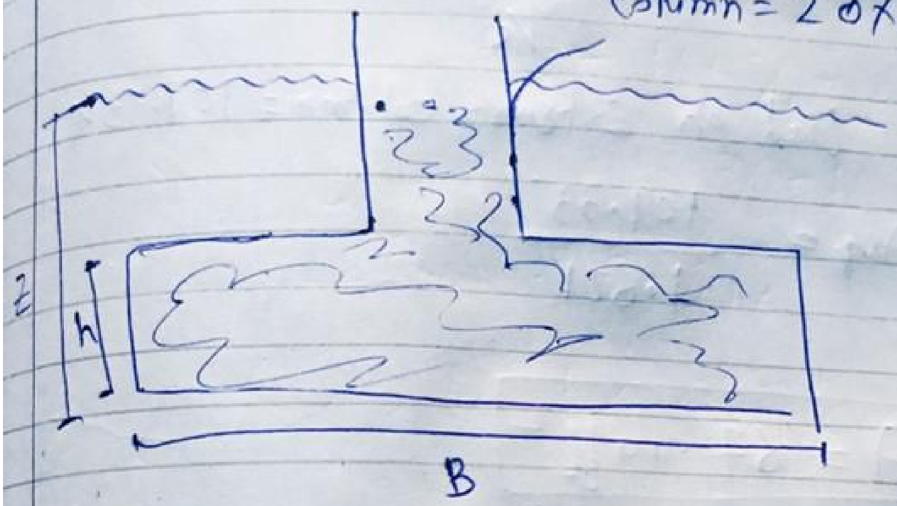
$b_o = 2(c + d_{avg}) + (c + d_{avg})$

$b_o = 2(20 + 17) + (20 + 17)$

$b_o = 148''$

Assume depth of footing from ground level = $5' = Z$

column = $20'' \times 20''$



$w = \gamma_{fill}(Z-h) + \gamma_c h$

$w = 100 \times (5 - 1.66) + 150 \times 1.66$

$w = 100 \times 3.34 + 150 \times 1.66$

$w = 583$ psf

$q_e = ?$

$q_e = q_a - w$

$2204 - 583$

$q_c = 1621$

Area of footing -

$$A_{req} = \frac{\text{Load of column}}{q_c}$$

Load on Exteriors column $= 109.33k$

$$A_{rq} = \frac{109.33}{1.621}$$

$$\boxed{A = 67.44 ft^2}$$

$A_{rq} = 10' \times 10'$ $\boxed{A_{req} = 100 ft^2}$

<u>Load</u>

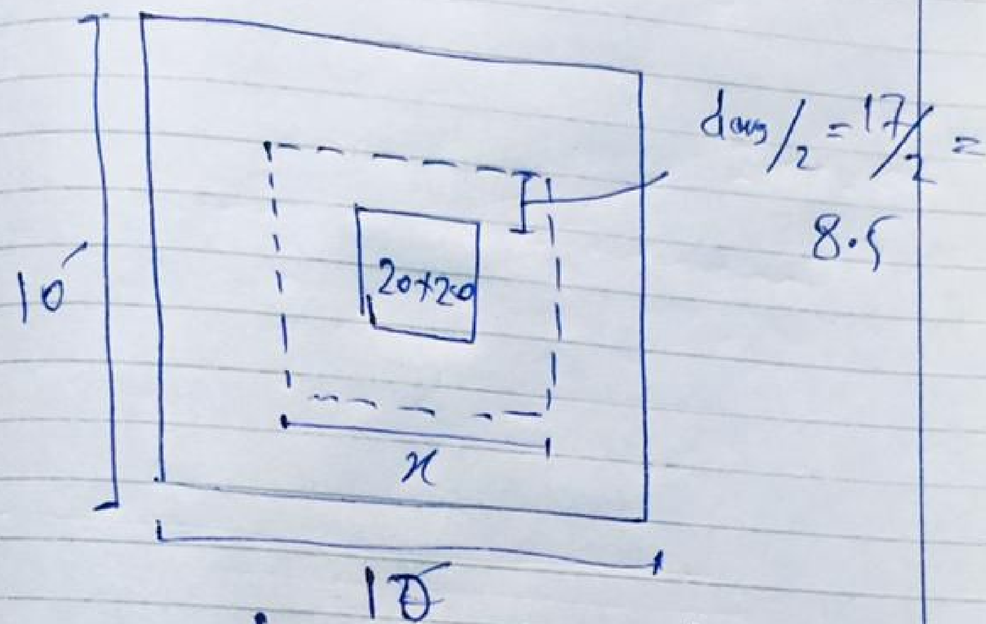
$$q_u = \frac{\text{factored load}}{10 \times 10}$$

$$q_u = \frac{109.5}{100}$$

q_u $1.095 ksf$

Analysis 2

① punching shear -



$$V_{up} = q_u B^2 - q_u (c + d_{avg})^2$$

$$= 1.095(10) - 1.095\left(20 + 17/12\right)$$

$$= \boxed{308.53K}$$

ii) Beam shear -

$$V_{u1} = q_u \{(B-C)/2 - d_{avg}\} B$$

$$= 1.095\left(10 - 20/2 - 17\right)10$$

$$= 108.136K.$$

design 2

→ design of punching.

$V_{up} = 308.53K$

Punching shear capacity $\phi V_{cp}) = \phi 4\sqrt{f_c'}$ $b_o d_{avg}$ -

$\phi V_{cp} = 0.75 \times 4 \times \sqrt{3000} \times 148 \times 17/1000$

$\phi V_{cp} = 413.42 > V_{up}$ / oK.

→ Design of Beam shear -

$V_{ud} = 108.136K$

punching shear capacity

$\phi V_c = \phi 4\sqrt{f_c'}\ b_o d_{avg}$ -

$\phi V_{cp} = 0.75 \times 4 \times \sqrt{3000} \times 148 \times 17/1000$

$= 413.42 > V_{ub}$ / ok

design for moment

Let $a = 0.2 d_{avg} = 0.2 \times 17 =$

$a = 3.4$

$$A_s = \frac{M_u}{\phi f_y (d_{avg} - a/2)}$$

$$= \frac{3085.95 \text{ in-k}}{0.90 \times 40 \times (17 - 3.4/2)}$$

$= 5.602 \text{ in}^2$

$$a = \frac{A_s f_y}{0.85 f_c' B}$$

$$a = \frac{5.602 \times 40}{0.85 \times 3 \times 100}$$

$$\boxed{a = 0.87''}$$

A=

$$A_s = \frac{3085.95}{0.90 \times 40 \times \left(17 - \frac{0.87}{2}\right)}$$

$A_s = 5.1748 \, in^2$ / ok

Checking the Reinforcement Ratio

$As_{min} = \left(3\sqrt{f_c'} / f_y\right) b d_{avg} > 200 / f_y \, B d_{avg}$

$$\frac{3 \times (\sqrt{3000})(100 \times 17)}{40000} > \frac{200}{4000} \times 100 \times 17$$

$6.98 > 8.5$ / Not ok

so

$As_{min} = 10.08 \, in$

Using 1" #8 with bar Area = 0.79

Spacing

#8 11" c/c

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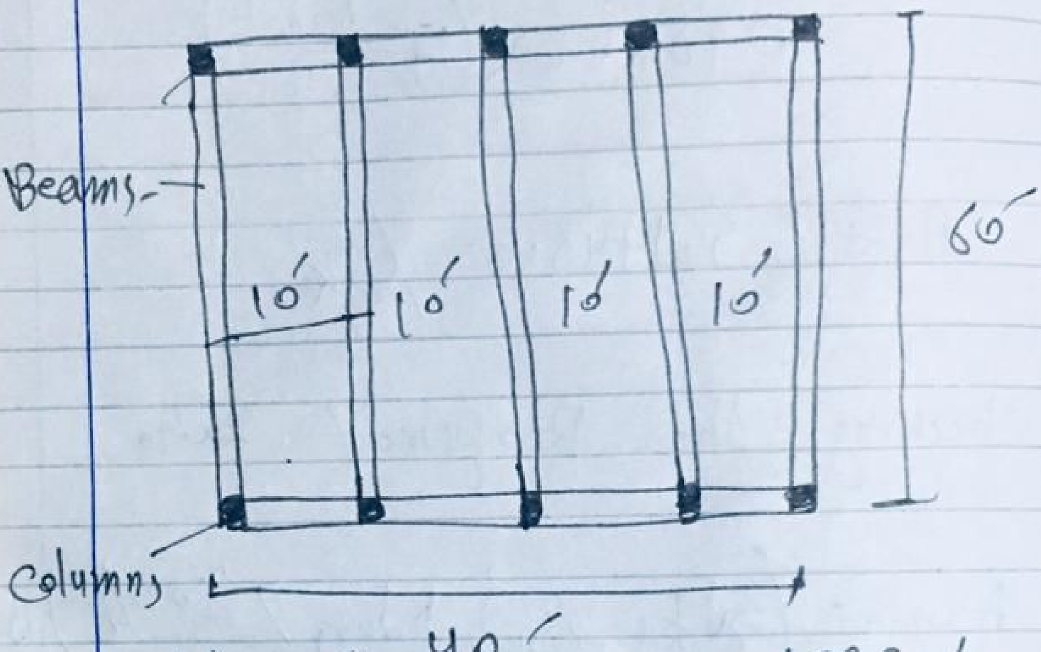
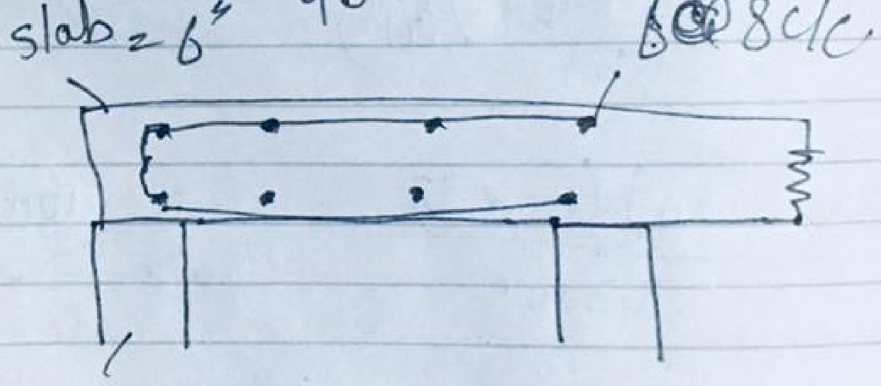
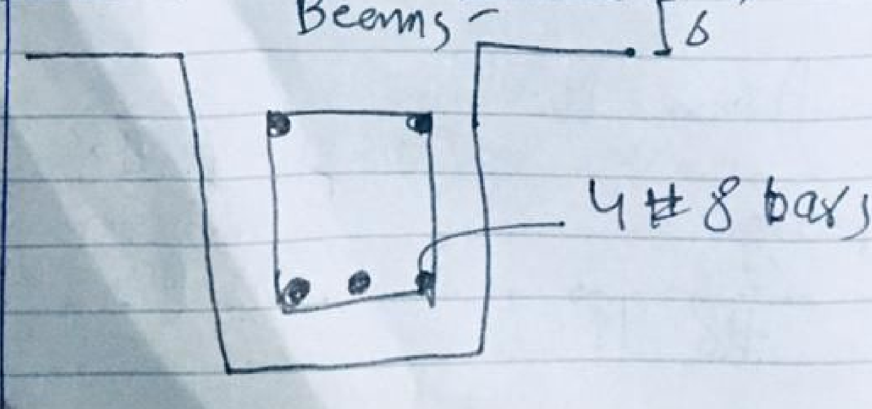
Design diagram

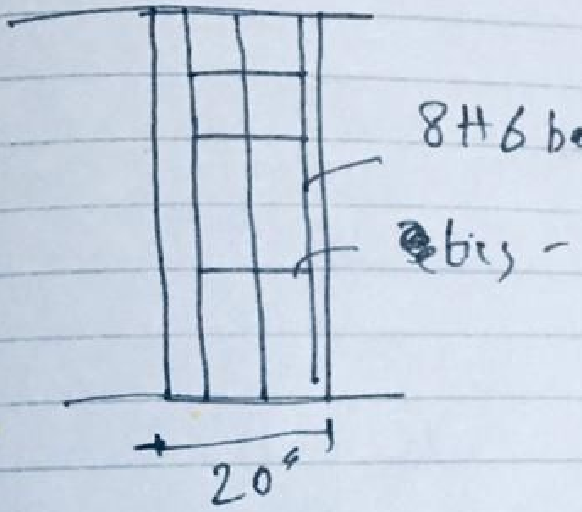
Structure-





Beams-





Column.