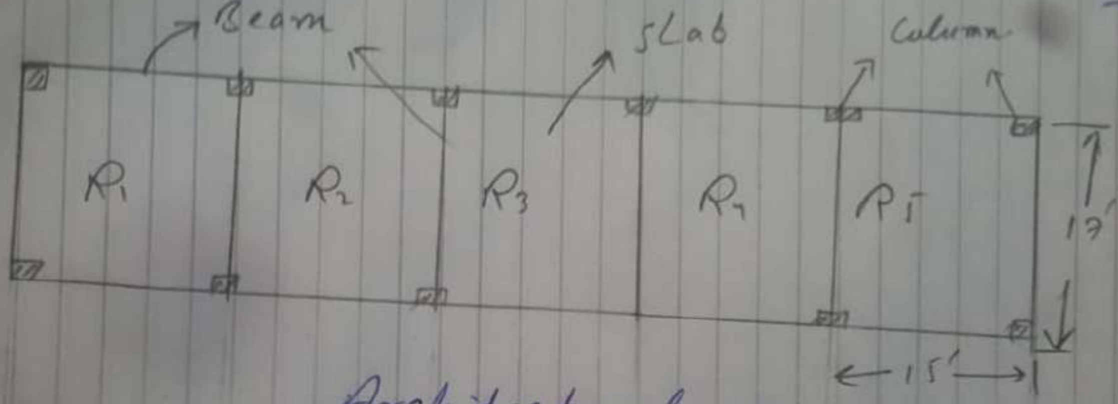


Architectural Drawing.

Area $= 5 \times (17 \times 15) =$

Area $= 1275 \simeq$ 5 Marl

ID 14390

ID 14390 ⑧

Solu. **Slab design:**

⇒ Minimum Thickness of continous

Assume data

$f_c' = 3000\,psi$
$f_y = 4000\,psi$

⇒ Dead load of 4in thick mud Flooring.
2inch thick brick flooring.

⇒ Live load = 40 psf.

Behaviour of slab

⇒ $l_b/l_a = \frac{17}{15} = 1.133 < 2$

So it will act as a two-way slab with bending in four directions.

⇒ **Depth of Slab:**

moments
Negative moments.

⇒ Depth of Slab

⇒ Minimum depth of two way slab is given by formula

$h_{min} = \text{perimeter}/180$

$h_{min} = 2\times(17+15)\times 12/180 = 4.26\,in$

we Assume $h = \cancel{6in}\ 5in$

Loads

Materials	Thickness	γ (kcf)	Load = $\gamma \times$ tick
Slab	5	0.15	$0.15\times(5/12) = 0.0625$
Mud	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 $= 0.0625 + 0.04 + 0.02$

$S.D.L = 0.1225\,ksf$

$S.L = 40psf = 0.04ksf$

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

$W_u = 1\times2\times1.225 + 1.6\times0.04 = 0.211ksf$

$M_A(+ve)$ L

$\phi M_n(min) = 16$

Computation of Moments cofficients

$$m = \frac{l_a}{l_b} = \frac{15}{17} = 0.8823$$

End Panel

=> Cofficients for Negative moments in slabs

$C_{a,neg} = 0.060$
$C_{b,neg} = 0.040$

=> Cofficients for Dead load positive moments in slabs

$C_{a,dl} = 0.033$
$C_{b,dl} = 0.022$

=> Cofficients for live load positive moments in slab

$C_{a,ll} = 0.055$
$C_{b,ll} = 0.016$

$\phi M_n(\ldots)$ 16.44 in

$\phi M_n(min) =$

$M_A(tre)$.

is greater than

less than

ID 14390 (5)

$\Rightarrow M_{a,(-ve)} = C_{a,-ve} \times W_u \times l_a^2$

$M_{a,(-ve)} = 0.060 \times 0.211 \times 15^2 = 2.84$ ft-k

$M_{a,(-ve)} = 34.182$ in-k

$\Rightarrow M_{b,(-ve)} = 0.040 \times 0.211 \times 17^2 = 29.269$ in-k

$\Rightarrow M_{a,+ve,dl} = C_{a,+ve} \times W_{u(dl)} \times l_a^2$

$\Rightarrow M_{a,+ve,dl} = 0.033 \times 0.147 \times 15^2 = 13.09$ in-k

$\Rightarrow M_{b^+,dl} = 0.022 \times 0.147 \times 17^2 = 11.21$ in-k

$\Rightarrow M_{a^+,L.L} = C_{a^+,LL} \times W_{u(LL)} \times l_a^2$

$\Rightarrow M_{a^+,LL} = 0.055 \times 0.064 \times 15^2 = 9.504$ in-k

$\Rightarrow M_{b^+,LL} = 0.016 \times 0.064 \times 17^2 = 0.295$ ft-k

$\Rightarrow M_{b^+,LL} = 3.55$ in-k.

Finally we have

$\Rightarrow M_a(-ve) = 34.182$ in-k

$\Rightarrow M_b(+ve) = 29.269$ in-k

$\Rightarrow M_a,(+ve)(D.L) = 13.09$ in-k

$\Rightarrow M_b,(+ve)(D.L) = 11.21$ in-k

$\Rightarrow M_a(+ve)(L.L) = 9.504$ ink

$\Rightarrow M_b(+ve)(L.L) = 3.55$ in-k

$\Rightarrow$ Design

$As_{min} = 0.002\, b \times h_f = 0.002 \times 12 \times 5 = 0.12$

$a = As_{min}\, f_y / (0.85 f'_c \times b)$

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

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

$\phi M_n(min) = 0.9 \times 0.12 \times 40\,(4 - 0.156/2)$

$\phi M_n(min) = 16.94$ in-k

$\phi M_n(min)$ is greater than positive moments but less than Negative moments.

=> For All Four positive Moments $A_s min$ governs.

Therefores $A_s min = 0.12 in^2$

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

Spacing $= \left(0.11/_{0.12}\right) \times 12 = 11''$

=> Maximum spacing According to (ACI 13.3.2) for two way slab is:

→ @ $2h_f = 2 \times 5 = 10$

Therefore maximum spacing of 10" governs.

=> Finally use #3 @ 9" c/c.

=> Provide #3 @ 9" c/c for all positive moments.

14390 (8)

⇒ For Ma, Negative = 34.182 in-k

let $a = 0.2d = 0.2 \times 4 = 0.8$

$As = 34.182 / (0.9 \times 40 \times (4 - 0.8/2))$

$As = 0.263 in^2$

$a = 0.26 \times 40 / (0.85 \times 3 \times 12) = 0.339$

$As = 34.182 / (0.9 \times 40 \times 4 - \frac{0.339}{2})$

$As = 0.237 in^2$

$a = 0.23 \times 40 / (0.85 \times 3 \times 12) = 0.30 in.$

$As = 34.182 / (0.9 \times 40 \times (4 - 0.30/2)) = 0.23 in^2$

⇒ Using 3/8" ϕ (#3) with bar Area

$Ab = 0.11 in^2$

$spacing = (0.11 \times 12 / 0.23) = 5.7" = 5" c/c$

Finally use #3 @ 5" c/c

14390 ⑤

For $M_B = 29.269$ in k.

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

$A_s = 29.269 / \left(0.9 \times 40 \times \left(40 - \frac{0.8}{2}\right)\right) = 0.24 in^2$

$a = 0.24 \times 40 / (0.85 \times 3 \times 12) = 0.31$ in

$A_s = 29.6296 \left(0.9 \times 40 \left(40 - \frac{0.31}{2}\right)\right) = 0.22 in^2$

$a = 0.22 \times 40 / (0.85 \times 3 \times 12) = 0.28 in^2$

$\Rightarrow$ Using 3/8" ϕ (#3) bars

Spacing $= 0.11 \times 12 / 0.28 = 6"$

Finally use #3 @ 5" c/c

Middle Rooms Slab Panel Design.

Moments Coefficients.

$C_{a}(-ve) = 0.068$
$C_{b}(-ve) = 0.025$

$C_{a}, dl = 0.026$
$C_{b}, dl = 0.015$

$C_{a}, L.L = 0.031$
$C_{b}, L.L = 0.016$

$\Rightarrow M_{a}(-ve) = 0.068 \times 0.211 \times 15^2 = 38.73$ in-k

$\Rightarrow M_{b}(-ve) = 0.025 \times 0.211 \times 17^2 = 18.2987$ in-k

$M_{a}(+ve)\ D.L = 0.026 \times 0.147 \times 15^2 = 10.3155$

$M_{b}(+ve)\ D.L = 0.015 \times 0.147 \times 17^2 = 7.64$ in-k

$M_{a}(+ve)\ L.L = 0.031 \times 0.064 \times 15^2 = 5.35$ in-k

$M_{b}(+ve)\ D.L = 0.16 \times 0.064 \times 17^2 = 3.55$ in-k

14390 (11)

As $As_{min} = 0.12$ and $\phi M_{n(min)} = 16.94$ in-k

So All Positive Moments are governed by $As_{(min)}$ =

$M_a (Neg) = 38.739$ in-k

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

$As = 38.739 / (0.9 \times 40 (4 - 0.8/2))$

$As = 0.29\ in^2$

$a = 0.29 \times 40 / (0.85 \times 3 \times 12)$

$a = 0.37$ in

$As = 38.739 / 0.9 \times 40 (4 - \frac{0.37}{2})$

$As = 0.2827$

$a = 0.282 \times 40 / (0.85 \times 3 \times 12)$

$a = 0.368$ in.

$As = 38.339 / (0.9 \times 40 (4 - \frac{0.368}{2}))$

$As = 0.281$ ok

Try using #3 bars

Spacing $= 0.11 \times 12 / 0.281 = 4.69$ in

Finally use #3 @ 4" c/c

For M_x & $M_y = 18.29$ in-k

Let $a = 0.2 \times d = 0.8$

$As = 18.29 / 0.9 \times 40 \times (4 - \frac{0.8}{2})$

$As = 0.141 \; in^2$

$a = 0.141 \times 40 / (0.85 \times 3 \times 12)$

$a = 0.184$ in

14392.17

$As = 18.29 / 0.9 \times 40 \times \left(4 - \frac{0.184}{2}\right)$

$a = 0.13$

$As = 18.29 / 0.9 \times 40 \left(4 - \frac{0.13}{2}\right) = 0.128$

$a = 0.129$ in

$As = 18.29 / 0.9 \times 40 \left(4 - \frac{0.121}{2}\right) = 0.128$

Try #3 bar

spacing $= \frac{0.11 \times 12}{0.128} = 10.3$ in

Finally use #3 @ ~~10~~ 9" c/c.

14390 (14)

For End span

c/c distance = 15'

Clear span = ~~15~~ 14'

For End span and continuous span

$l = 14 + 15.5 = 15.5 > c/c$

so $l = 15'$

For End span

$$h_{min} = l/18.5 \times \left(0.4 + \frac{f_y}{100000}\right)$$

$h_{min} = 7.78\,in$

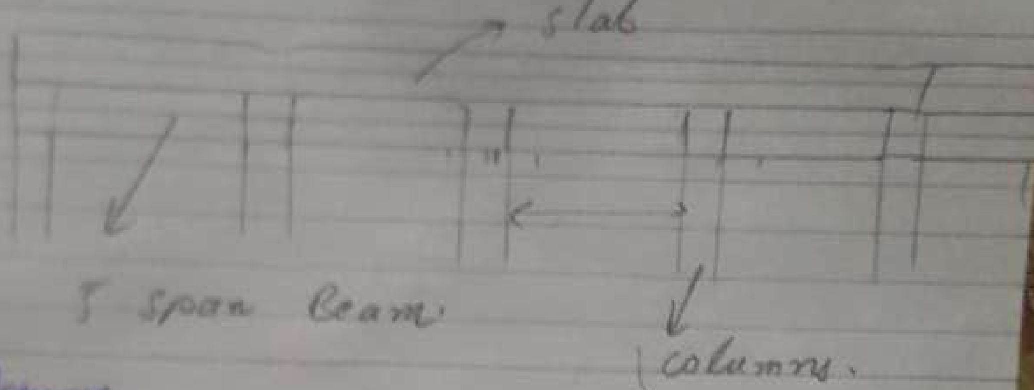
For Continuous span

$$h_{min} = \frac{l}{21}\left(0.4 + \frac{f_y}{100000}\right)$$

$h_{min} = 6.8571$

Therefore we assume. $h'' = 18\,inch$

Beam Design



Assume.

Column = 12" x 12"

Beam width = 12"

According to ACI 9.5.2.1 Table 9.5(a)

Minimum thickness of beam with one end continuous = $h_{min} = l/18.5$

l = clear span (l_n) + depth of member (beam) $\leq$ c/c distance between supports

For Interior span $h_{min} = l/21$

14340 (15)

Loads From Slab

Service Dead load = 0.1225 ksf

Service live load = 0.04 ksf.

Beam is supporting ~~..~~ 7.5' slab per running foot

So

Service Dead load from slab
= 0.1225 × 7.5 = 0.918 k/ft

Self weight of Beam

$= \left(\frac{15 \times 12}{144}\right) \times 0.15 = 0.187$ k/ft.

Total Dead Load = 0.918 + 0.187 = 1.11 k/ft

Service live load per running feet

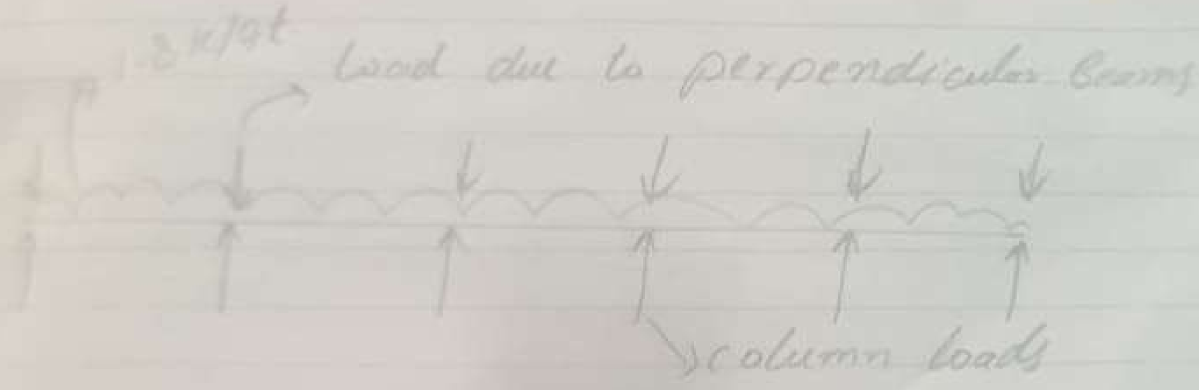
= 0.04 × 7.5 = 0.3 k/ft

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

$W_u = 1.2 \times 1.1 + 1.6 \times 0.3$

$W_u = 1.8$ k/ft.

14390



Shear Forces at Supports:

$$V_{Exterior} = \frac{W_u l_n}{2} - d^2 = \frac{1.8 \times 14}{2} - \left(\frac{12.5}{12}\right)^2$$

$V_{Exterior} = 11.51$ k

$$V_{interior} = \frac{1.15 W_u l_n}{2} - d^2$$

$$V_{interior} = \frac{1.15 \times 1.8 \times 14}{2} - \left(\frac{12.5}{12}\right)^2$$

$V_{interior} = 13.409$ k

Moments

Ends span From ACI Code Table 12.1 (Nelson Book)

$$M_{+ve} = \frac{1}{14} \times W_u l_n^2 = \frac{1.8}{14} \times 14^2 = 25.2 \text{ ft-k}$$

$M_{+ve} = 302.4$ in-k

⇒ Interior Spans
Negative Moment at exterior Face
of First Interior support

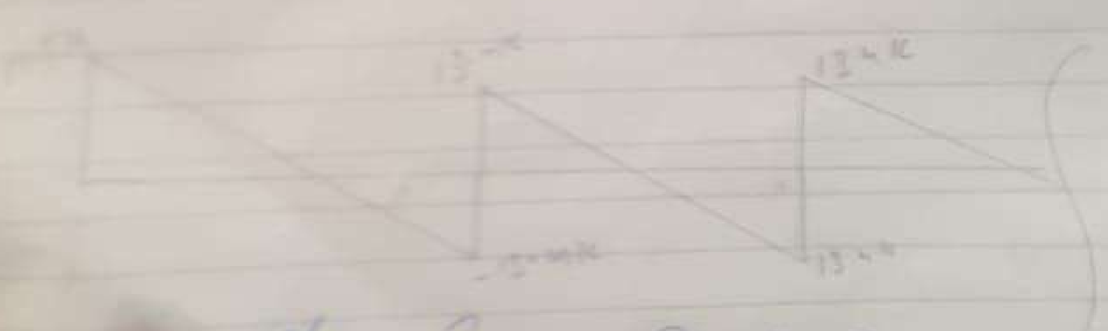
$$= \frac{1}{10} W_u l_n^2 = \frac{1.8 \times 14^2 \times 12}{10} = 421.76$$

⇒ Negative moment at others
Faces of Interior supports.

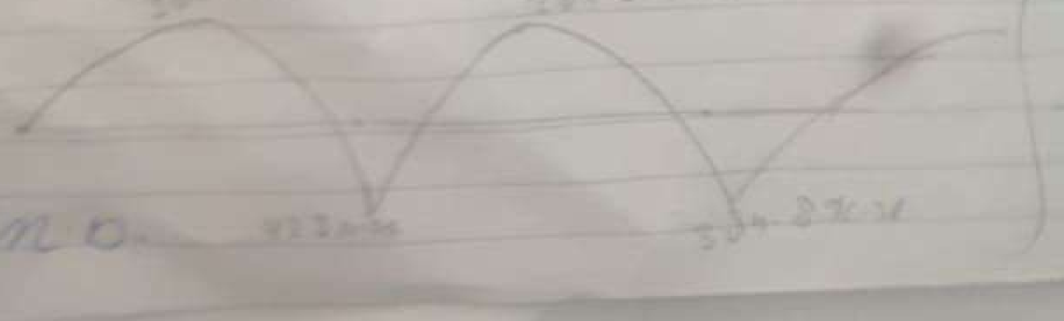
$$= \frac{1}{11} W_u l_n^2 = \frac{1.8 \times 14^2 \times 12}{11} = 384.8 kN$$

⇒ Positive moment at Interior
Supports

$$= \frac{1}{16} W_u l_n^2 = \frac{1.8 \times 14^2 \times 12}{16} = 264.6 kN$$



Shear Force Diagram.



B.M.D.

Flexural Design for Positive moment

According to ACI 8.10, beff for L-Beam

(i) $6h_f + b_w = 6(5) + 12 = 42''$

(ii) $b_w + \frac{l}{12} = 12 + \frac{(14) \times 12}{12} = 26''$

So b effective = 26"

Check if beam is to be designed as Rectangular Beam or L shaped Beam

$\Rightarrow$ $\boxed{\text{+ve Moments at End span}}$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{302}{0.9 \times 40 \left(12 - \frac{5}{2}\right)}$$

$A_s = 0.883$

$a = A_s f_y / 0.85 f'_c b_{eff}$

$$a = \frac{0.883 \times 40}{0.85 \times 3 \times 26} = 0.53 < h_f$$

Hence, it is a Rectangular Beam

Designing as a Rectangular Beam

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

$$A_s = \frac{302}{0.9 \times 40 (12 - \frac{0.73}{2})} = 0.71 in^2$$

$$a = \frac{0.71 \times 40}{0.85 \times 3 \times 26} = 0.431 in$$

$$A_s = \frac{302}{0.9 \times 40 (12 - \frac{0.431}{2})} = 0.711 in \ ok$$

Check for maximum and minimum Reinforcement

$A_{s\,min} = \rho_{min} \times b_w \times d = 0.005 \times 12 \times 12 = 0.72$

As As < As min
So As(min) Governs

Trying 4 #4 bars $A_s = 0.78 in^2$

As positive Moment at Interior Supports is less than +ve moment at Interior Support

So we provide here Asmin) also

So Trying 4 #4 bars.

Flexural Designs For Negative Moments.

$M_{u(-ve)} = 423\ k\text{-}ft.$

$$A_s = \frac{423}{0.9 \times 40 \left(12 - \frac{a}{2}\right)} = 1.23 in^2$$

$$a = \frac{1.23 \times 40}{0.85 \times 3 \times 26} = 0.7425$$

So It also Acts as a Rectangular Beam

$$A_s = \frac{423}{0.9 \times 40 \left(12 - \frac{0.74}{2}\right)} = 1.00 in^2$$

14390

$$a = \frac{1.1 \times 40}{0.85 \times 3 \times 20} = 0.603$$

$$A_s = \frac{423}{0.9 \times 40 \left(12 - \frac{0.603}{2}\right)} = 1.006\ in^2 = ok$$

Trying 5 # 4 bars

Shear Design of Beams:

Vu Exterior = 11.8k

Vu Interior = 13.4k

$$\phi v_c = \phi 2 \sqrt{f_c'} \times b_w \times d = \frac{0.75 \times 2 \sqrt{5000} \times 12}{1000}$$

ϕ Vc = 13.62k > Vu Exterior and Vu Interior

Theoritically no shear Reinforcement is required but minimul will be provided.

14.2.90

(i) $\frac{A_v f_y}{50 b_w} = \frac{0.22 \times 40000}{50 \times 12} = 14.67''$

(ii) $d/2 = \frac{12}{2} = 6''$ c/c

(iii) $24''$ c/c

(iv) $A_v f_y / 0.75 \times \sqrt{f'_c} \times b_w = \frac{0.22 \times 40000}{0.75 \times \sqrt{3000} \times 12}$

$= 17.85''$ c/c

$\boxed{\text{So, } s = 6'' \text{ c/c}}$

Check for spacing under "Maximum spacing requirement"

$\phi V_s \leq \frac{\phi 4 \sqrt{f'_c} \times b_w \times d}{1000}$ (ACI 11.5.4.3)

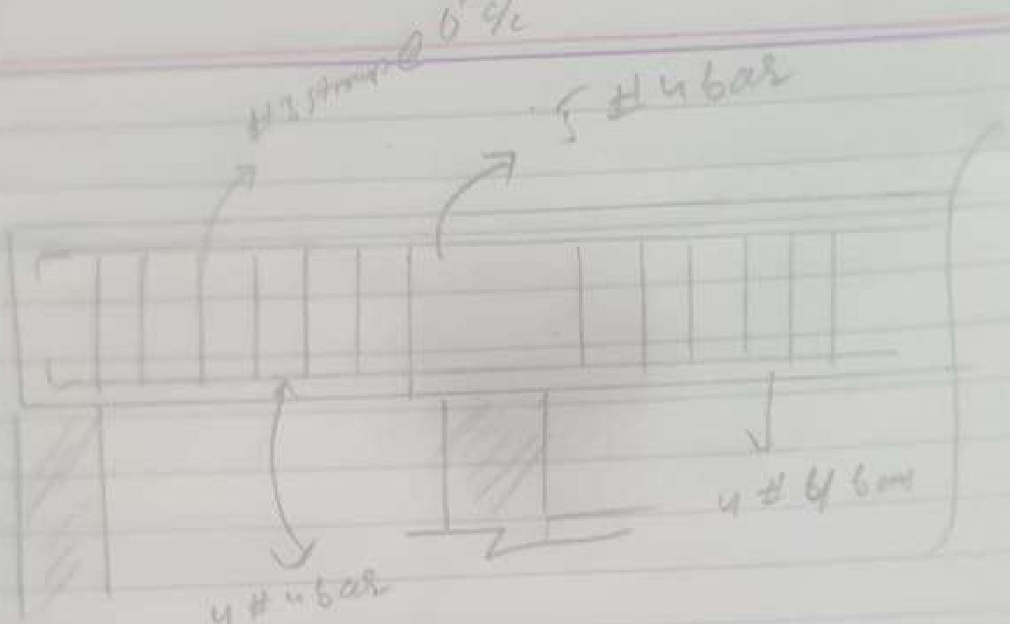
$\phi 4 \sqrt{f'_c}\, b_w \times d = \frac{0.75 \times 4 \times \sqrt{3000} \times 12 \times 12}{1000}$

$= 23.54$ k

$\phi V_s = \frac{\phi A_v f_y d}{s} = \frac{0.75 \times 0.22 \times 40 \times 12}{6}$

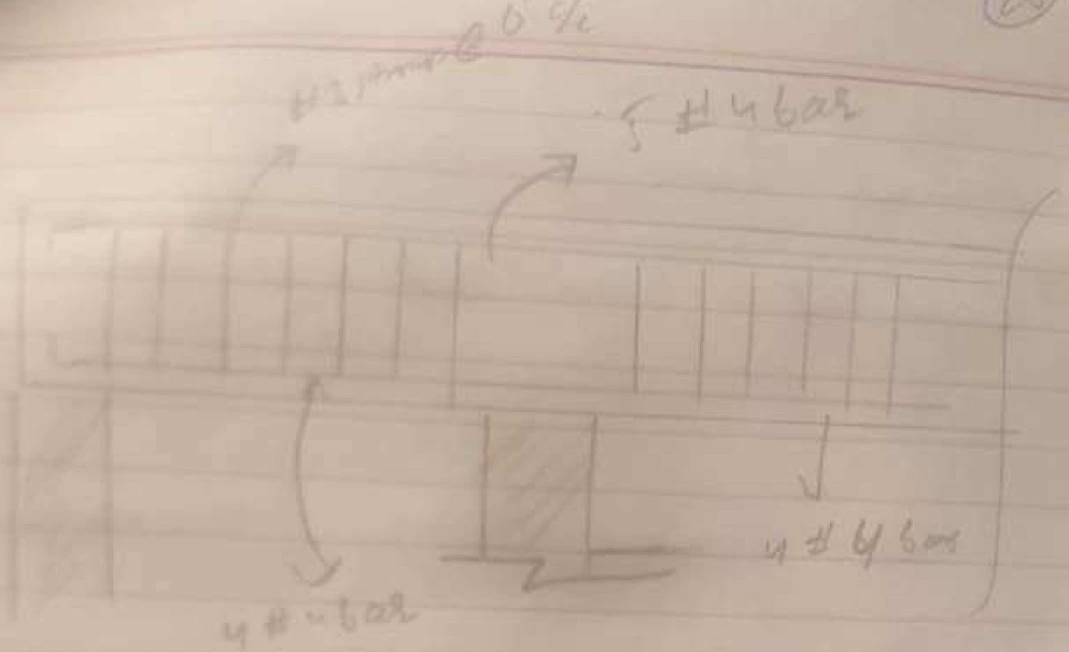
$13.2 k < 23.6$

14330



Structural Designing Drawings

Q. Beam



Beam. Structural Designing Drawings

Colum Designing

From shear force diagram

We have shear value of 11.5k at End support and 13.4k at All External supports.

Calculating values of self weight of perpendicular beams.

$w = \frac{15 \times 12 \times 0.155}{144} = \cancel{\quad}$ 0.158 k/ft

Each column will take half of this load.

14390

Axial loaded colum Design

Load at End columns = 11.5 + 0.198 =
= 11.698

We have design formula
for tied columns

$\phi P_n (max) = 0.80 \phi [0.85 f_c' (A_g - A_{st}) + f_y A_{st}$

$11.698K = 0.65 \times 0.80 [0.85) 3 (A_g - 0.02 A_g)$
$+ 60 (0.02 A_g)$

$A_g = 14.31 in^2$

Substituting into colum equation
with A_g and A_{st}
we obtain

$11.698K = (0.65)(0.85)(4)(196 - A_{st})$ [illegible]

$A_{st} = 3.31 in^2$

Use 6 #7 bars ($3.6 in^2$)

Design of Ties

Spacing

(a) $48 \text{ in} \times \frac{3}{8} = 18 \text{ in}$

(b) $16 \times \frac{7}{8} \text{ in} = 14 \text{ in}$

(c) Least dimention = 14 in

So, Use #3 ties @ 14 in