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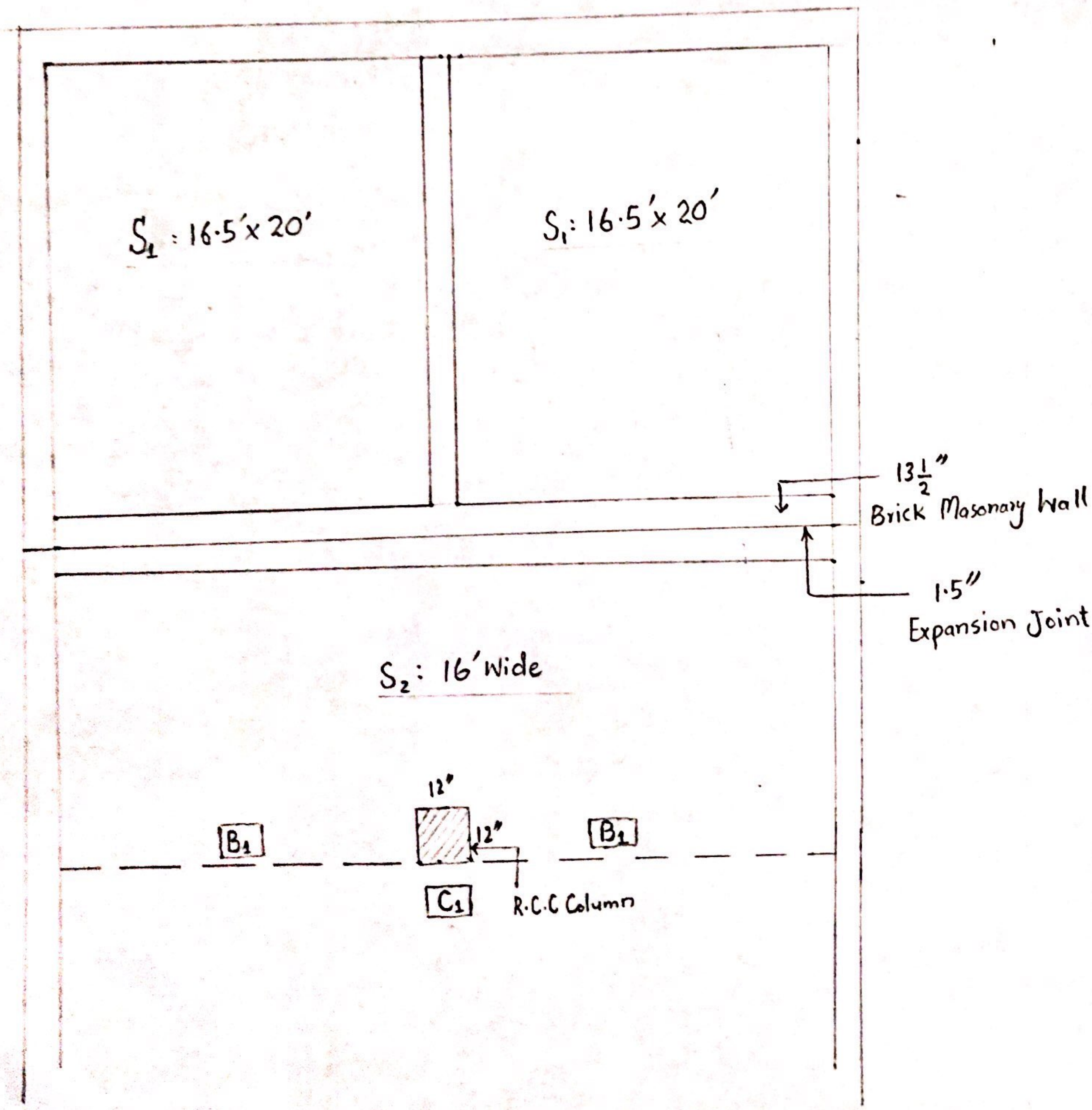
Dimension of 10 Marla plot

1. Considering 1 Marla = 272.25 sq.ft

Plot Dimension of 10 marla will be 36'x 75'

(36' Width , 75' length)



Solution :

Concrete Compressive strength (f_c') = 3 ksi

Steel yield strength (f_y) = 40 ksi

Load on slab :

4" Thick mud .

2" Thick brick tile .

(1) Design of slab "S_2"

(a) size $l_b/l_a = \frac{33.75}{15} = 2.25$

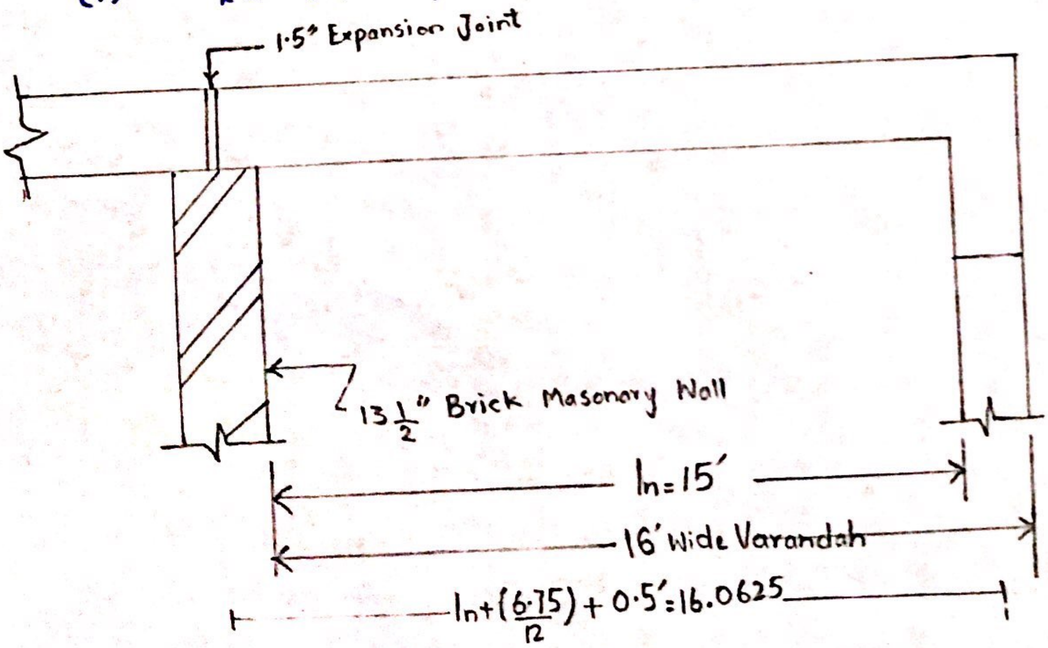
$2.25 > 2$

" one way slab "

Assume 5" slab .

Span length for end span according to ACI 8.7 is minimum of

(i) $l = l_n + h_f = 15 + (5/12) = 15.41'$



Section A-A (see fig 1 above)

(ii) c/c distance between supports = 16.0625'

Therefore $l = 15.41'$

Slab Thickness $(h_f) = (l/20) \times (0.4 + f_y/100{,}000)$ [for $f_y < 60000$ psi]

$$= \left(\frac{15.41}{20}\right) \times \left(0.4 + \frac{40000}{100{,}000}\right) \times 12$$

$$h_f = 7.4''$$

take $h_f = 7.5''$

$$d = h_f - 0.75 - (3/15)/2$$

$$= 7.5'' - 0.75 - (3/15)/2$$

$$= 6.65''$$

Step no 2 : Loading

Table 1.1 Dead Loads			
Material	Thickness (in)	γ (kcf)	Load = γ x Thickness (ksf)
Slab	7.5	0.15	$0.15 \times (7.5/12) = 0.09375$
Mud	5	0.12	$0.12 \times (5/12) = 0.04$
Brick Tile	2	0.12	$0.12 \times (2/12) = 0.02$

Service Dead load (D.L) $= 0.09375 + 0.04 + 0.02$

$= 0.15375$ ksf

Service Live load (L.L) = 40 psf or 0.04 ksf

factored load $(w_u) = 1.2\,D.L + 1.6\,L.L$

$= 1.2 * 0.15375 + 1.6 * 0.04$

$= 0.2485$ ksf

Step no 3 Analysis

$Mu = Wu l^2/8$ (l = span length of slab)

$= 0.2485 \times \frac{(15.41)^2}{8}$

$= 7.37$ ft.k/ft $= 88.51$ in-k/ft

Step no 4 Design

$As_{min} = 0.002 bhf$ (for fy 40 ksi, ACI 10.5.4)

$= 0.002 b hf$

$= 0.002(12)(7.5)$

$= 0.18 \ in^2$

$a = As_{min} fy / 0.85 f_c' b$

$= 0.18 \times 40 / 0.85 \times 3 \times 12$

$= 0.235$ in

$\phi Mn(min) = \phi As_{min} fy (d - a/2)$

$= 0.9(0.18)(40)(4 - \frac{0.235}{2})$

$= 25.1586$ in.k $< Mu$

Therefore

$As = Mu / \{\phi fy (d - a/2)\}$

Take $a = 0.2d$

$As = 88.51 / \{0.9 \times 40 (6.65 - \frac{0.235}{2})\}$

$As = 0.376 \ in^2$

$a = 0.376 \times 40 / (0.85 \times 3 \times 12)$

$a = 0.491 \ in^2$

$A_s = 88.51 / (0.9 \times 40 \times (6.65 - (\frac{0.491}{2})))$

$= 0.383 \ in^2$

$a = 0.383 \times 40 / (0.85 \times 3 \times 12)$

$= 0.383 \ in^2$

O.K

using $\frac{1}{2}''$ φ (#4) {#13, 13mm} with bar Area $A_b = 0.40 \ in^2$

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

$= [0.40 (in^2) / 0.383 (in^2/ft)] \times 12$

$= 12.5$ in

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

$A_b = 0.11 in^2$

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

$= \left[\frac{0.11 \ in^2}{0.383 (in^2/ft)} \right] \times 12$

$= 3.44'' \approx 3.5''$

finally used #3@ 3.5" c/c (#10@ 150 mm c/c)

Shrinkage steel or temperature steel (Ast):

$A_{st} = 0.002 \ b h_f$

$= 0.002 \times 12 \times 7.5$

$= 0.18 \ in^2$

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

Spacing area of one bar $= A_b / A_{smin}$

$= (0.11/0.18) \times 12$

$= 7.3''$ c/c

finally used #3 @ 7" c/c (#10@225 mm c/c)

- Maximum spacing for main steel in one way slab according to ACI 7.6.5 is minimum of :

  i) $3h_f = 3 \times 7.5 = 22.5''$

  ii) $18''$

  Therefore 3.5" spacing Is ok.

- Maximum spacing for shrinkage steel is one way slab according to ACI 7.12.2 is minimum of :

  i) $5h_f = 5 \times 7.5 = 22.5$

  ii) $18''$

  Therefore 7" Spacing is ok.

Design of Slab "S_1" :

Step no-1 Size's

$$l_b / l_a = \frac{20}{16.5} = 1.21 < 2$$

" Two way slab ".

Minimum depth of two way slab is given by formula , h_{min} = perimeter / 180

$$= \frac{2 \times (16.5 + 20) \times 12}{180}$$

$$= 4.86 \text{ in}$$

Assume $h = 5''$

Step no-2 load's

Factored loads (W_u) = $W_{u,dl} / W_{u,ll}$

$$W_u = 1.2\,DL + 1.6\,L.L$$

$$= 1.2(0.15375) + 1.6(0.04) = 0.1845 + 0.064$$

$$= 0.2485 \text{ ksf}$$

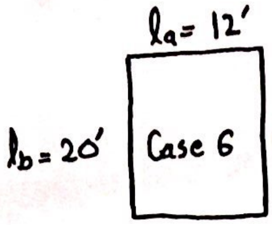
Step no-3 Analysis

W_u = ultimate uniform loads psf

l_a, l_b = length of clear span in short and long direction respectively.

Therefore , for the design problem under discussion

$$m = l_a / l_b = \frac{16.5}{20} = 0.825$$



Two way slab (S_2)

Table 1.2 : Moment Coefficient for slab					
Case # 6 ($m = 0.80$)					
Coefficient for -ve moments in slab		Coefficient for Dead load moment in slab		Coefficient for live load +ve moments in slab.	
$C_{a\,neg}$	$C_{b\,neg}$	$C_{a\,dl}$	$C_{b\,dl}$	$C_{a\,ll}$	$C_{b\,ll}$
0.086	0	0.051	0.019	0.051	0.019
Refer to table 12.3 to 12.6 of nilson 12th Ed					

1. $M_{a\,neg} = C_{a,neg} \times W_u \times l_{a2}$

$= 0.086 \times 0.2485 \times (16.5)^2$

$= 5.81$ ft.K

$= 5.81 \times 12 = 69.72$ in.k

2. $M_{b\,neg} = C_{b\,neg} \times W_u \times l_b^2$

$= 0 \times (0.2485) \times (20)^2$

$= \boxed{0}$

3. $M_{a\,pos\,dl} = C_b\ pos\,dl \times W_u\,dl \times l_a^2$

$= 0.019 \times (0.1845) \times (16.5)^2$

$= 0.095$ ftk $= 1.145$ in.k

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

$= 0.019 \times 0.1845 \times (20)^2$

$= 1.40$ ft.K $= 16.56$ in.K

5. $M_{a\,pos\,ll} = C_{a\,pos\,ll} \times W_{u\,ll} \times l_a^2$

$= 0.058 \times 0.064 \times (16.5)^2$

$= 0.88$ ft K $= 10.6$ in.K

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

$= 0.019 \times 0.064 \times 20^2$

$= 0.4864$ ft K

$= 5.83$ in.K

Therefore finally we have

1) $M_{a,neg} = 69.72$ in.k

2) $M_{b\,neg} = 0$

3) $M_{a\,pos(dl+ll)} = 1.145 + 16.56$

$= 17.705$ in.k

4) $M_{b\,pos(dl+ll)} = 16.56 + 5.83$

$= 22.39$ in.k

Step # 04 Design

$A_{s\,min} = 0.002\ bh_f$

$= 0.002\,(12)(7.5)$

$= 0.18\ in^2$

$a = \dfrac{A_{s\,min}\,f_y}{0.85\,f_c'\,b} = \dfrac{0.18 \times 40}{0.85 \times 3 \times 12}$

$= 0.235\ in$

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

$= 0.9 \times 0.18 \times 40 \times (4 - 0.235/2)$

$\phi M_{n(min)} = 25.1586\ in.k$

(capacity provided by As min)

using 3/8" ϕ (#3) (#10) 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 in $2h_f$

$= 2\,(7.5) = 15"$

therefor maximum spacing of 15" governs.

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

Provide #3 @ 9" c/c as negative reinforcement along the long Direction.

Ma pos (dl+ll) = 17.05 in.K > Φ

Mn let a = 0.2d = 0.2 x 6.65

a = 1.33 in

As = 0.131 in² (OK)

using 3/8" Φ (#3) (#10)

with bar Area

A_b = 0.11 in²

Spacing = 0.11x12/0.131 = 10.07" ≈ 9 c/c

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

Beam Design

Given Data :

Exterior Support = 9" brick masonory wall

$f_c' = 3$ ksi

$f_y = 40$ ksi

Column Dimension = 12" x 12"

Step no 1 Sizes :

According to ACI 9.5.2.1 Table 9.5 (a)

Minimum Thickness of beam with one end continous

$h_{min} = l/18.5$

l = Clear Span (l_n) + depth of member

$\leq$ c/c distance b/w supports

Table 1.3 clear Span of Beam.

Case	Clear Span
end Spain (one end Contineous)	$12.375-(12/12)/2 = 11.875$

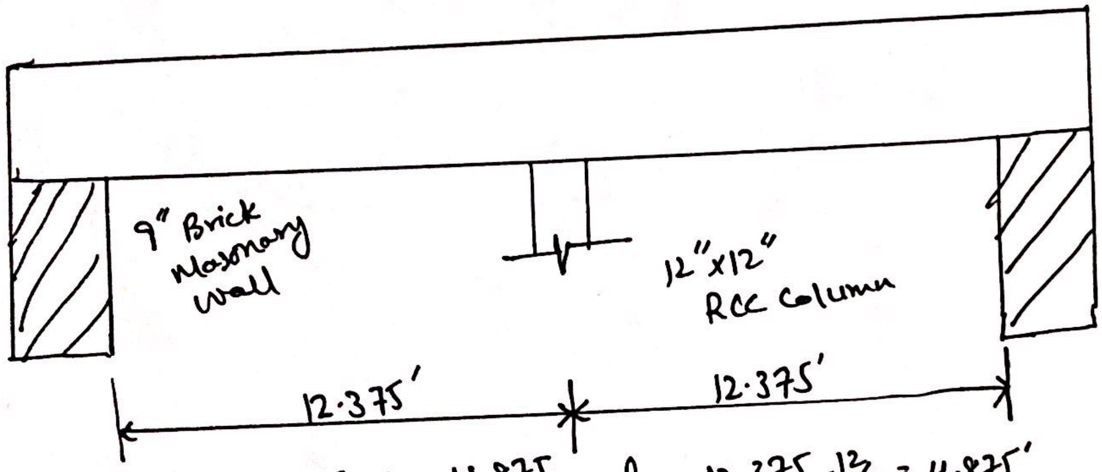
Let Depth of Beam = 18"

Depth(h') = 6.62"

Minimum Requirement of ACI

take h = 1.5' = 18"

d = h - 3 = 15"



c/c Distance & clear Span

Service D.L = 0.0625 + 0.04 + 0.02
= 0.1225 ksf

Service L.L = 40 or 0.04 ksf

Service Dead load = 0.15375 x 5
= 0.76875 K/ft

Service D.L from Beam Selfweight =

$h_w \times b_w \gamma_w$ = (13 x 12/144) x 0.15
= 0.1625 K/ft

Total D.L = 0.76875 + 0.1625
= 0.93125

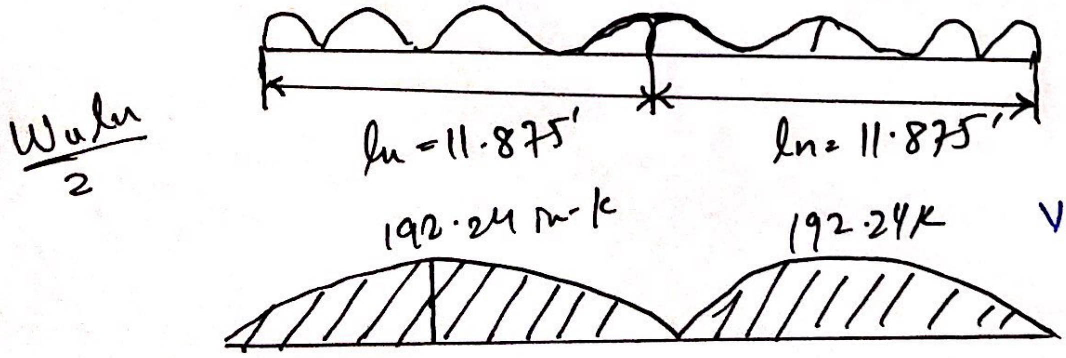
Step # 03 Analysis

1) At interior Support

Negative moment (M_{neg}) = Coefficient × $(W_u l_n^2)$
$= (1/9) \times (1.25 \times (11.875)^2)$
= 19.5 ft.K
= 235.08 in.k

2) At mid Spain.

Positive moment (M_{pos}) = Coefficient × $(W_u l_n^2)$
$= (1/11) \times (1.25 \times (11.875)^2)$
= 16.02 ft.K = 192.24 in.K



Vext = $W_u l_n / 2$
$= 1.25 \times \frac{11.875}{2}$
= 7.42 K

(4) Design Column

Load of Column

$P_u = 2V_{int} = 2 \cdot 8.54 = 17.4K$

$A_g = 12 \times 12 = 144\ in^2$

$f_c' = 3ksi$

$f_y = 40ksi$

Design :

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

$= 0.80 \times 0.65 (0.85 \times 3 \times (144 - 0.01 \times 144) + 0.01 \times 144 \times 40)$

$= 218.98^a > P_u\ 17.08\ ok$

$A_{st}\quad 0.01 \times 144 = 0.144\ in^2$

using 3/4" ϕ (#6)(#19) with bar Area

$A_b = 0.44 in^2$

No of bar $= A_s / A_b = 1.44/0.44$

$= 3.27 \approx 4$ bars

Tie bars :·

using 3/8 ϕ (#3) (#10) tie bar
3/4" ϕ (#6) {#19} main bar,

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