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Discussion Expansion Joints (2)

Solution

Design of slab S_2

Step No 1 size $l_b/l_a = 24.75/8 =$

3.09 > 2 one way slab" Assume

5" slab

Span length for end span according ACI 8.7.5

minimum of $l = l_n + h_f = 8 + 5(12) = 8.42'$

c/c distince b/w supports = 9.0625'

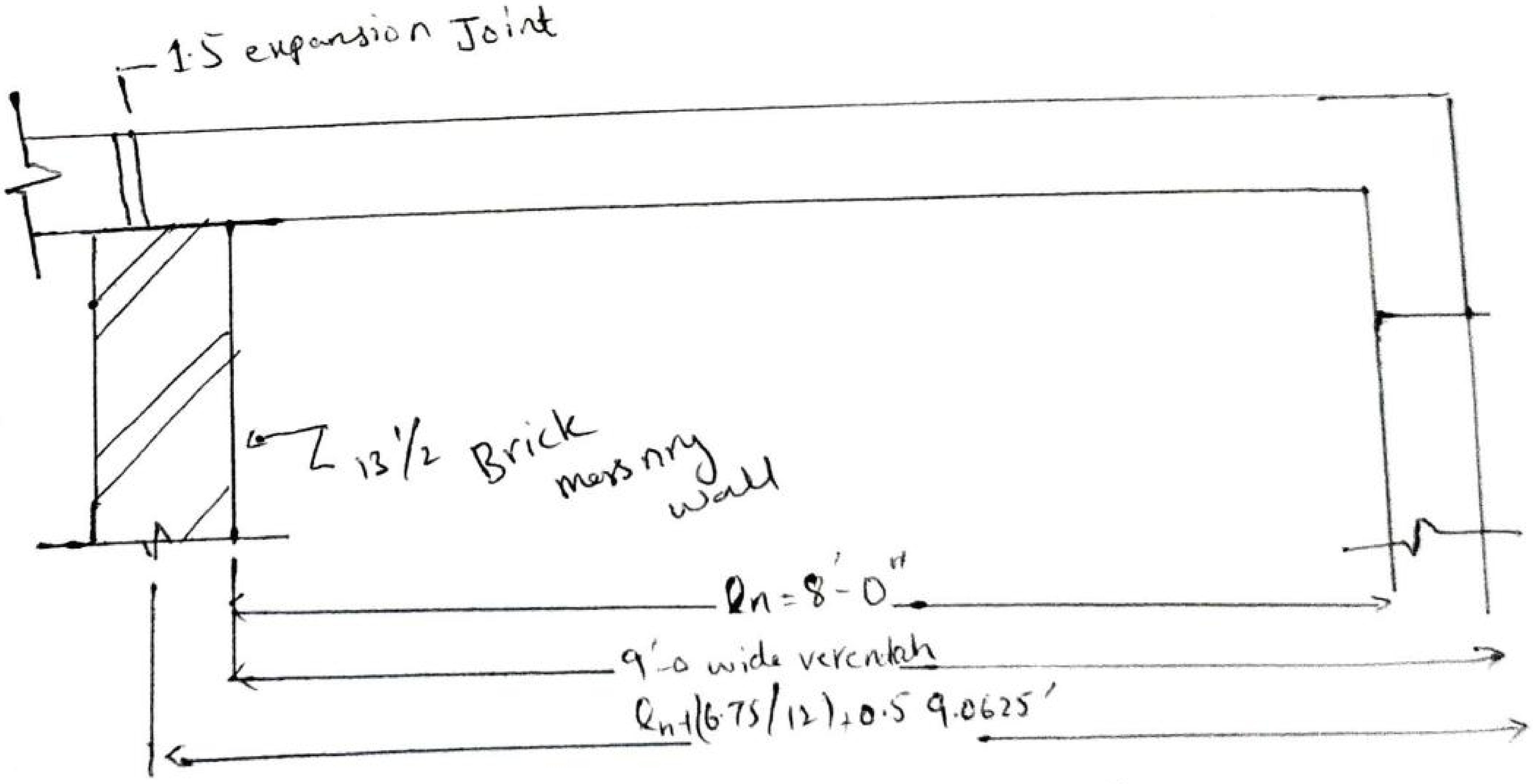

figure 2 Section A-A see figure 1 above

Therefore $l = 8.42'$

Slab thickness (h_f) $(l/20) \times (0.4 + f_y/100000)$ for f_y < 60000

$(8.4/20) \times 0.4 + 40000/100000) \times 12$

... of ACI) 9.5.2.1

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Therefore take hf - 0.75 - (3/8)2 = 4"

Step No 2 loading.

Table 1.1 Dead load			
Material	Thickness (in)	γ (kcf)	Load = γ × thickness (ksf).
Slab	5	0.15	0.15 x(5/12) = 0.0625
Mud	4	0.12	0.12 x (4/12) = 0.04
Brick Tile	2	0.12	0.12 x (2/12) = 0.02

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

0.1225 ksf

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

Factored load (wu) = 1.2 D.L + 1.6 L.L

1.2 x 0.1225 + 1.6 x 0.04

= 0.211 ksf

Step No 3 Analysis.

$M_u = w_u l^2/8$ (l = span length of slab).

$M_u = 0.211 \times (8.42)^2/8$

1.87 ft - k/ft = 22.44 in k/ft

Step No 4 Design

Asmin 0.002 bhf (for fy 40ksi ACI 10.5.4) = 0.002 × 12 × 5 =

0.12 in² a = Asmin fy (0.85 fc' b)

0.12 × 40 / (0.85 × 3 × 12) = 0.156 in

ϕ Mn (min) = ϕ Asmin fy (d - a/2)

0.9 × 0.12 × 40 × (4 - 0.156/2)

16.94 in-k < Mu.

Therefore

As = Mu {ϕ fy (d - a/2)}

Take a = 0.2d

As = 22.44 / {0.9 × 40 × (4 - (0.2 × 4))/2}

As = 0.173 in²

a = 0.173 × 40 / (0.85 × 3 × 12) = 0.226 in

As = 22.44 / {0.9 × 40 × (4 - 0.226/2))}

0.160 in²

a = 0.160 × 40 (0.85 × 3 × 12) = 0.209 in As 22.44 / {0.9 × 40 × (4 - (0.2909/2)

= 0.160 in² OK

Using 1/2" ϕ (#4) (# 13, 13 mm} with bar area

(5)

$A_b = 0.20 \ in^2$

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

$[0.11 \ (in^2) / 0.160 \ (in^2/ft)] \times 12 = 7.5" \approx 6"$

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

Shrinkage steel or temp (A_{st}).

A_{st} 0.002 bh_f

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

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

$A_b = 0.11 in^2$

spacing = Area of one bar $(A_b)/A_{s min}$

$(0.11/0.12 \times 12 = 11"$ c/c

Finally use # 3 @ 9" c/c # 10 @ 225mm c/c

Max. spacing for main steel in one way slab according to ACI 7.6.5 is ~~maximum~~ minimum

i) $5 h_f = 5 \times 5 = 25"$

ii) 18"

Therefore 9" spacing is Ok

Design of slab "S_1"

(6)

Step No 1 Sizes l_b/l_a $18/14 = 1.28$ <2"

This is called Two way Slab

minimum depth of two way slab is given by

$= 2(14 \times 18) \times 12 / 180 = 33.6$

Assume $h = 5''$

Step # 2 loads

Factors load $= W_u = W_u\,dl + W_u\,ll$

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

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

$= 0.211$ ksf

Step 3 Analysis

$M_{a,neg}$ $W_u\,l_a^2$

$M_{a,neg}$ $W_u\,l_a^2$

where $C_{a,b}$ = tabulated moment cofficient as given in Append is A

W_u = Ultimate uniform load, psf $l_a\,l_b$ = length

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Step 4 Design

Asmin

$= 0.002\, bhf = 0.002 \times 12 \times 5$

$= 0.12\, in^2$, $a = As_{min}\, f_y\, (0.85 f_c'\, b)$

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

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

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

Capacity provided by Asmin

ϕmn is greter then Mb pos (dl+ll) but less then Ma neg & Ma (dl+ll) Therfore Asmin 0.12 in² governs

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

Spacing accordng ACI 13.3.2 for two way slab is $2hf = 2 \times 5 = 10"$

Therfore max spacing of 10" governs.

finally use #3 @ 9 c/c as negative reinforcement along the longer direction

Ma pos (dl+ll) = 1.53 ft-k = 18.536 in-k > ϕ

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

$As = 1.53 \times 12 / \{0.9 \times 40\, (0.85 \times 3 \times 12) = 0.~~11~~$ in

~~As = 1.53 × 12 {~~

$(0.85 \times 3 \times 12)$

$0.146\, in^2$ $a = 0.146 \times 40 / (0.85 \times 3 \times 12)$

0.171 in

$As = 1.53 \times 12 / \{0.9 \times 40 \times (4 - 0.30/2) = 0.131\, in^2$ a

M_b pos dl C_b pos dl $\times W_u$ dl $\times l_b^2$ 10

$0.012 \times 0.147 \times 18^2 = 0.57$ ft-k 6.85 in-k

M_a pos ll = C_a pos ll $\times W_u$ ll $\times l_a^2$

$0.055 \times 0.064 \times 14^2 = 0.68$ ft-k

= 8.2 in-k

M_b pos ll $C_b \times W_u$ ll $\times l_b^2$

$0.016 \times 0.064 \times 18^2 = 0.33$ ft-k = 3.98 in-k

Therfore finally we have

$M_a = 43.67$

$M_b = 0$

M_a pos (dl + ll) = 16.59 + 8.2

= 24.79

M_b pos (DL + LL) = 6.85 + 3.98

= 10.83

Using 3/8 ϕ {10mm 10ϕ with bar area Ab =

0.11 in² spacing = 0.11 × 12/0.131 = 10.07 ≃ 9 c/c

finally use #3 9' c/c #10 225 mm c/c

Ma neg 2.67 ft-k 32.04 .

let 0.2d = 0.2 × 4 0.8 in.

2.67 × 12 {0.9 × 40 /0.85 × 3 × 12) = 0.30

= 2.67 × 12 {0.9 × 40 (4 – 0.30/2) 0.23 in² ok

Using 3/8 ϕ #3 10mm with bar area

Ab 0.11 spacing = 0.11 12/0.23 = 5.7 = 4.5

finally use #3 @ 4.5 c/c 10 @ 110 mm c/c

(12)

Beam Design 2 span contiuous

Exterior Supoint = 9 BMW.

$fc' = 3 ksi$

$fy = 40 ksi$

colum = 12" × 12

step # size

$h_{min} = l / 18.5 \; l.$

End span $12.375 - (14/2)/2 = 11.75'$

let depth = 18" of beam

l_n + depth of Beam $11.75 + 18/12 = 13.375$

c/c destine b/w beam suport = 12.375'

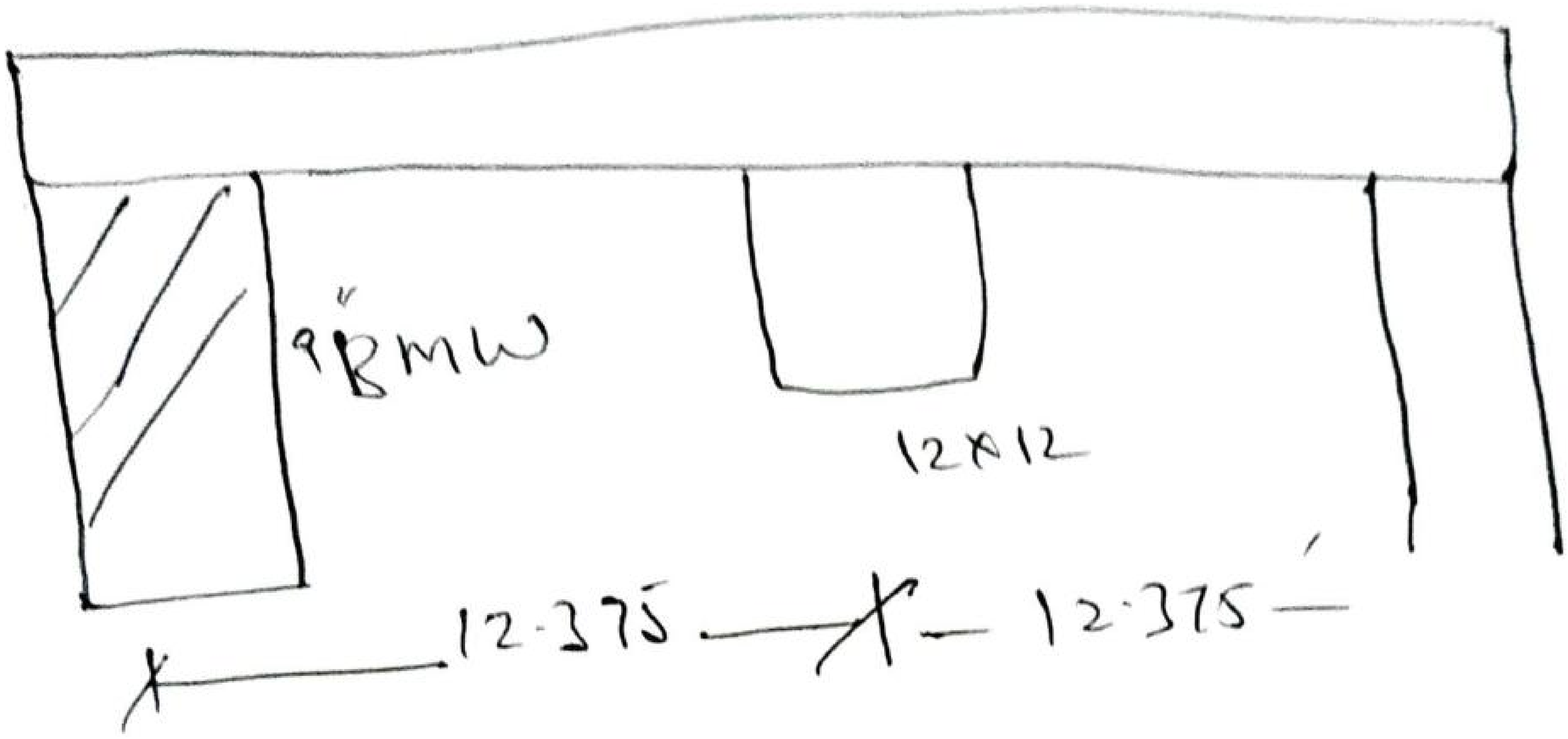
therfore = $l = 12.75$ $(4.5/12) = 12.75'$

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

$= 6.62"$ (min reg of) $\times 12$

Take h = 1.5' = 18" 13

d = h-3 = 15"



Step 2 loads

D.L 0.125 + 0.04 + 0.02 = 0.1225 ksf

L.L 40 psf or 0.04 ksf

D.L form slab = 0.1225 x 5 = 0.6125 k/ft

Self weight = $h_w b_w \gamma_c$ = $\left(\frac{13 \times 12}{144}\right) 0.15$

= 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 LL

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

Step 3 Analysis 17

(1) At interior support

$M_{neg} = \text{Coefficient} \times w_u l_u^2$

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

$= 19.59 \text{ ft.k} = 235.08 \text{ in-k}$

(2) At Mid Span

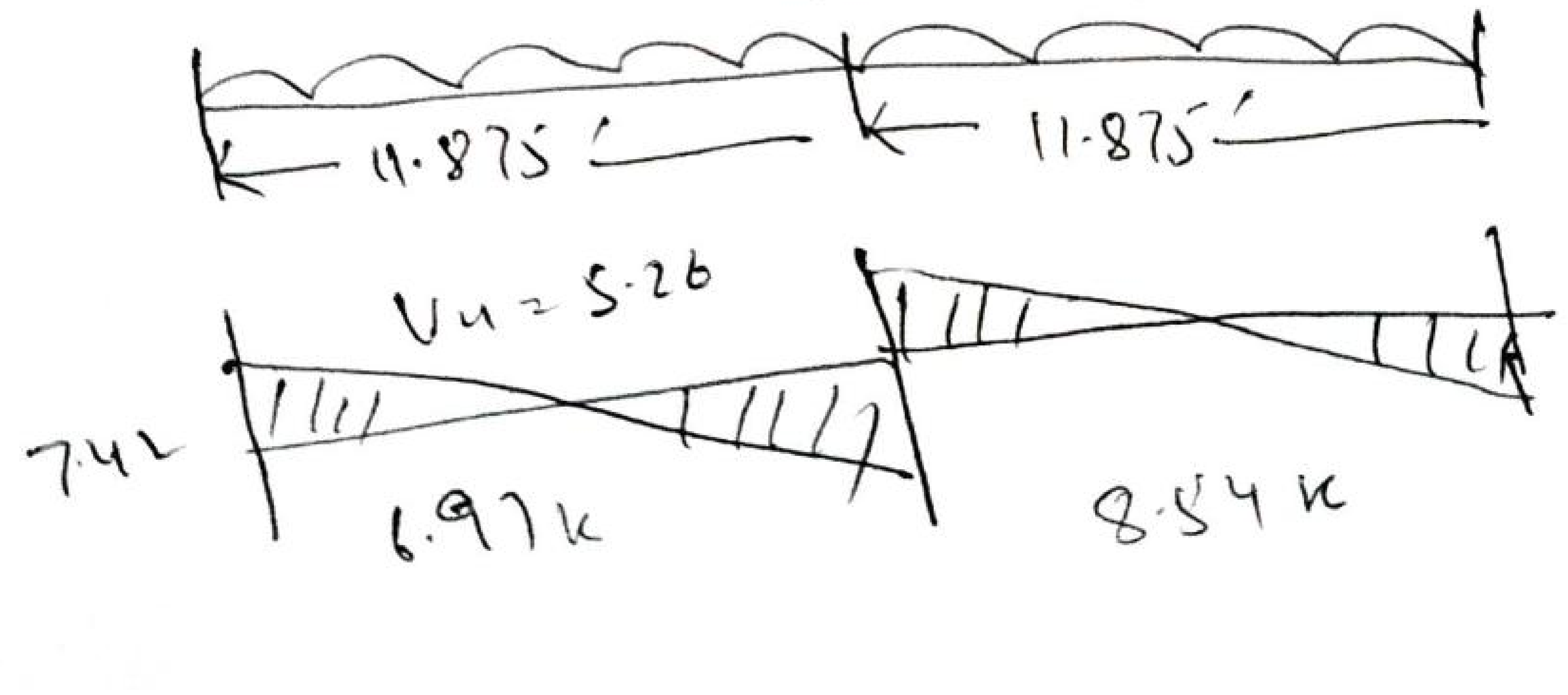
$M_{pos} = C_o \times w_u l_n^2$

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

$= 16.02 \text{ ft.k} = 192.24 \text{ in.k}$

V.in $1.15\, w_u l_u/2$ $\quad 1.15 \times 1.25 \times 11.875/2$

$= 8.54\text{ k}$ $\quad V_u$ (int $= 8.54 - 1.25 \times 1.25 = 6.97\text{k}$



weight combi

Given data D = 20 k/ft

L = 15 k/ft

γ_s = 100 lb/ft³

q_a = 4 ksf = 4000 psf

f_c' = 3 ksi = 3000 psi

f_y = 60 ksi 6000 psi

Design of column

load of column

P = 2 vint = 2 × 8.54 = 17.08 k

Gross area of column cross section

12×12 = 144 in² f_u = 3 ksi

f_y = 40 ksi

Design Nominal Strength ϕP_n of axially loaded

let Ast = 1% of Ag ϕP_n = 0.80 × 0.65 × {0.85×3 (144 − 0.01×144) + 0.01×144×40}

= 218.18 k > (P_u = 17.08 k) ok

Ast 0.01×144 = 1.44 in²

using 3/4" ϕ (#6) {#19, 19mm} (26)

No of bars As/Ay

1.44/0.44 ≃ 3.27 ≃ 4 bars use

4 #6 {bars 4 #9 bars 19mm}

Tie bar 3/8" ϕ (#3) {#10, 10mm}

3/8" ϕ

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

(c) Column