CE 387 — SA

Rough / Midterm 18/01

Flat plate

Reinforced concrete slab that sits directly on column with no internal beams.

Minimum thickness 5" (no drop panel)

Flat slab

Flat plate but with thickened section at col slab junction — known as drop panel, col capital.

Increases shearing carrying capacity.

Flat plate + drop panel, or
col capital, or
drop panel + col capital
} flat slab

Neither has internal beam but may have edge beam

On two-way slab, reinf of shorter dir is placed below reinf of longer dir

flat plate, reinf of longer dir is placed below reinf of shorter dir

because — total static moment

$$M_u = \frac{(q_u l_2) l_n^2}{8}$$

so longer dir has more moment

placing reinf at btm increases "d".

$M \propto d$

Design

Considerations

- Place steel at bottom layer for the direction in which moment is lower.
- Minimum reinf = that of shrinkage & temp crack control
  - → $0.0018\,bh$ for grade 60 deformed bars
    $0.0020\,bh$ " " 40, 50 " "
    $\frac{0.0018 \times 60}{f_y}$ " " > 60
    } Table 12.2
- Maximum spacing = $2h$
- Moments are considered const within the bound of col/middle strip

Eff. beam

Broad strip of slab centered on the col lines in each dir serving as the func of beam when beams are omitted

Direct Design Method for col supported slabs

Criteria,

1. Min three consequtive spans
2. Rectangular w/ dimensions ratio ≤ 2
3. Consequtive span len must not differ by more than 1/3 of longer span
4. Columns may not offset from centerline by mor than 10% of span len
5. Loads due to gravity only; and LL $\leq$ 2 DL (unfactored)
6. If internal beam present, relative stiffness must be between 0.2 — 0.5

Procedure:

1. DDM applicability
2. slab thickness → eff. depth
   from table
   min thickness = 5"
3. Factored ld
4. Shear check
   i) Punching shear → $v_c = \min\left[4\lambda\sqrt{f'_c}\, b_o d,\ \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'_c}\, b_o d,\ \left(\frac{\alpha_s d}{b}+2\right)\lambda\sqrt{f'_c}\, b_o d\right]$
   ii) Beam shear → $v_c = 2\lambda\sqrt{f'_c}\, b d$

   Perimeter of critical section
   40, 30, 20 int, ext, con
   col dim ratio
5. Moment → reinf calc (both shorter & longer dir)
   Total factored static mmnt → neg & pos mmnt
   → distribute to col/middle strip
6. d-check

Prob 1

For a floor structure with typical 24 ft by 26 ft **interior** panel sizes, the superimposed dead loads are 20 psf and the superimposed live loads are 100 psf. As a preliminary design, select an appropriate flat slab thickness (without drop panels) and calculate reinforcement. Assume f'_c 4,000 psi and f_y 60,000 psi. The concrete is normal weight with a unit weight of 150 pcf. Assume that the column size is 20 inch x 20 inch.

Soln ;

The design supports DDM and is analyzed following such.

Slab thickness determination :

$$\text{Clear span, } l_n = \left(26 - \frac{20}{12}\right) \times 12 = 292''$$

$$h_{interior} = \frac{l_n}{33} = \frac{292}{33} = 8.85'' \simeq 9''$$

$$h_{exterior} = \frac{l_n}{30} = \frac{292}{30} = 9.73'' \simeq 10''$$

$\therefore$ Thickness, $h = 10''$

$$\text{Eff. depth, } d = 10'' - 0.75'' - \frac{4}{8}'' - \frac{4}{2\times 8}'' = 8.5''$$

Factored load calculation:

DL = self wt = $150 \times \frac{10}{12} = 125$ psf

Superimposed DL = 20 psf

LL = 100 psf

∴ Factored ld, $q_u = 1.2DL + 1.6LL$

$= 1.2(125+20) + 1.6 \times 100$ psf

$= 0.334$ ksf

Shear check:

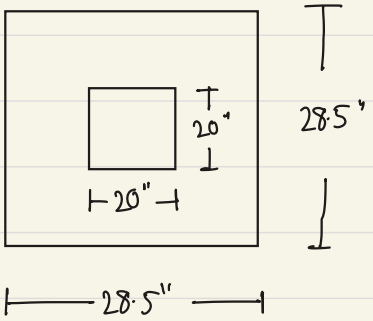
i) Punching shear

tributary area – area defined by critical shear perimeter

$V_u = q_u A$

$= 0.334 \times \left[26 \times 24 - \frac{28.5 \times 28.5}{144}\right]$ kip

$= 207$ kip



$V_c = \min \begin{cases} 4\lambda\sqrt{f_c'}\, b_o d \\ \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_c'}\, b_o d \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f_c'}\, b_o d \end{cases}$

$= \min(245, 368, 305)$

$= 245$ kip

$b_o = 28.5 \times 4 = 114"$

$\beta = \frac{20}{20} = 1$

$\alpha_s = 40$

∴ $\phi V_c = 0.75 \times 245 = 184 < (V_u = 207)$ [not OK]

drop panel to be provided.

ii) Beam shear

$V_{u1} = q_u A_1 = 0.334 \times 275 = 92$ kip

$V_{u2} = q_u A_2 = 0.334 \times 272 = 91$ kip

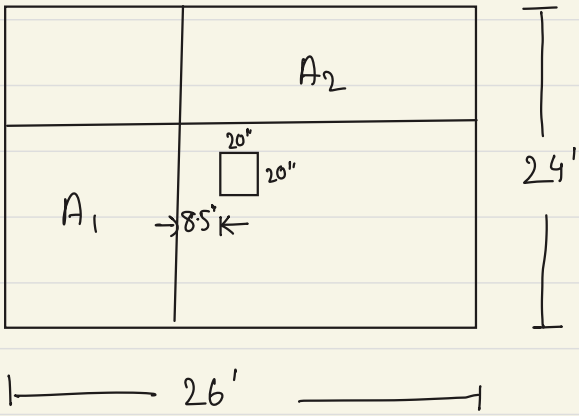
$V_c = 2\lambda\sqrt{f'_c}\, bd$

$= 2\sqrt{4000} \times 10.46 \times 12 \times 8.5$ lb

$= 135$ kip

$\phi V_c = 0.75 \times 135 = 101 > V_{u1}, V_{u2}$

[OK]



$A_1 = \left(13 - \frac{10}{12} - \frac{8.5}{12}\right) \times 24$ ft^2

$= 275$ ft^2

$A_2 = \left(12 - \frac{10}{12} - \frac{8.5}{12}\right) \times 26$ ft^2

$= 272$ ft^2

$b_{least} = 12 - \frac{10}{12} - \frac{8.5}{12}$ ft

$= 10.46$ ft

Reinforcement calculation (longer dir)

Total factored static moment, $M_0 = \frac{q_u l_2 l_n^2}{8} = \frac{0.334 \times 24 \times [26 - \frac{20}{12}]^2}{8}$

$= 593.3$ K-ft

Negative factored moment $= 0.65\ M_0 = 385.6$ Kip-ft

Positive " " $= 0.35\ M_0 = 207.7$ K-ft

	Col strip		Middle strip	
Description	Pos	Neg	Pos	Neg
Mmnt distribution	0.60	0.75	0.40	0.25
m_u (K-ft)	124.6	289.2	83.1	96.4
Strip width, b (in)	144	144	144	144
d (in)	8.5	8.5	8.5	8.5
$R = \frac{M_u}{\phi b d^2} \times 12000$				
ρ	0.0027	0.0065	0.0018	0.0021
$A_{s,min} = 0.0018\ bd$ (in^2)	2.2	2.2	2.2	2.2
$A_s = \rho bd$	3.3	7.96	2.2	2.57
Bar selection	17#4	19#6	11#4	13#4
Spacing $\leq$ 2h or 18"	9	8	14	11

Reinforcement calculation (shorter dir):

$$M_0 = \frac{q_u l_2 l_n^2}{8} = \frac{0.334 \times 26 \times \left[24 - \frac{20}{12}\right]^2}{8} = 541.4 \text{ k-ft}$$

Neg factored moment = $0.65\, M_0 = 351.9$ k-ft

Pos " " = $0.35\, M_0 = 189.5$ k-ft

	Col strip		Middle strip	
Description	Pos	Neg	Pos	Neg
Mmnt distribution	0.60	0.75	0.40	0.25
m_u (k-ft)	113.7	263.9	75.8	88
strip width, b (in)	144	144	168	168
d (in)	8.5	8.5	8.5	8.5
$R = \frac{m_u}{\phi b d^2} \times 12000$				
ρ	0.0025	0.006	0.0014	0.0016
$A_{s,min} = 0.0018\, bd$ (in^2)	2.2	2.2	2.57	2.57
$A_s = \rho bd$	3.06	7.34	2 → 2.57	2.28 → 2.57
Bar selection	16#4	17#6	13#4	13#4
spacing ≤ 2h or 18"	9	9	14	14

d. check :

Max neg moment, $M_{u,neg} = 289.2$ k-ft

$$M_{u,neg} = \phi \rho b d^2 f_y \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)$$

$$\Rightarrow 289.2 \times 12 = 0.9 \times 0.0065 \times 144 \times d^2 \times 60 \left(1 - 0.59 \frac{0.0065 \times 60}{4}\right)$$

$$\Rightarrow d^2 = 72.85 \text{ in}^2$$

$$\therefore d = 8.54 \approx (d_{pro} = 8.5")$$

Drop panel :

$$\text{Thickness} = \frac{1}{4} \text{ slab thickness}$$

$$= \frac{1}{4} \times 10" = 2.5"$$

Extension,

On longer dir $= \frac{1}{6} \times 26' = 4' - 4"$

On shorter dir $= \frac{1}{6} \times 24' = 4'$

Prob 01

16 in sqn interior col

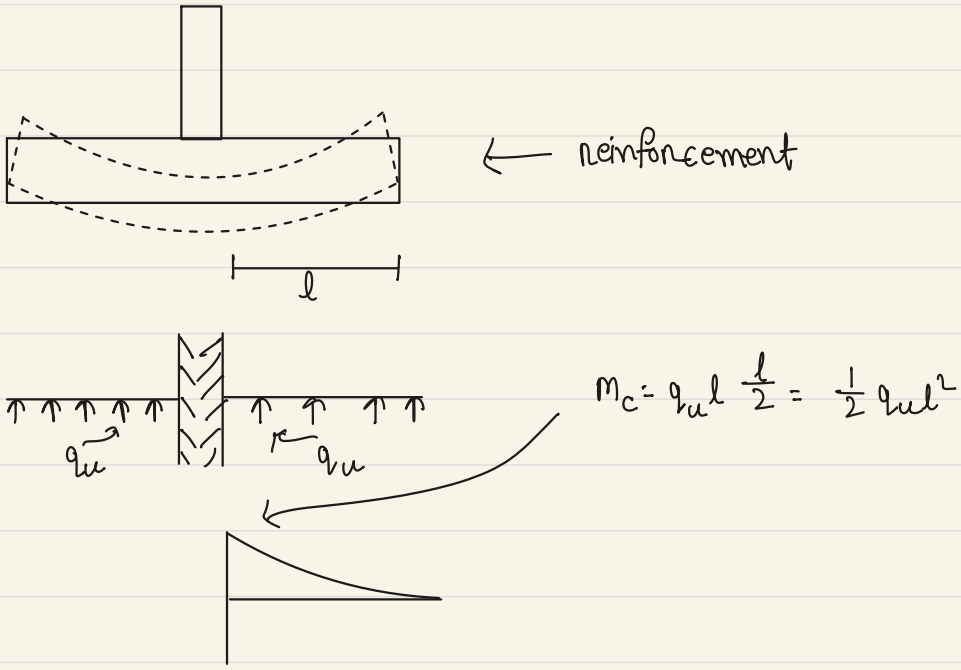
DL = 200K, LL = 80K, LR = 20K

q_a = Allowable bearing capacity ← Soil test report

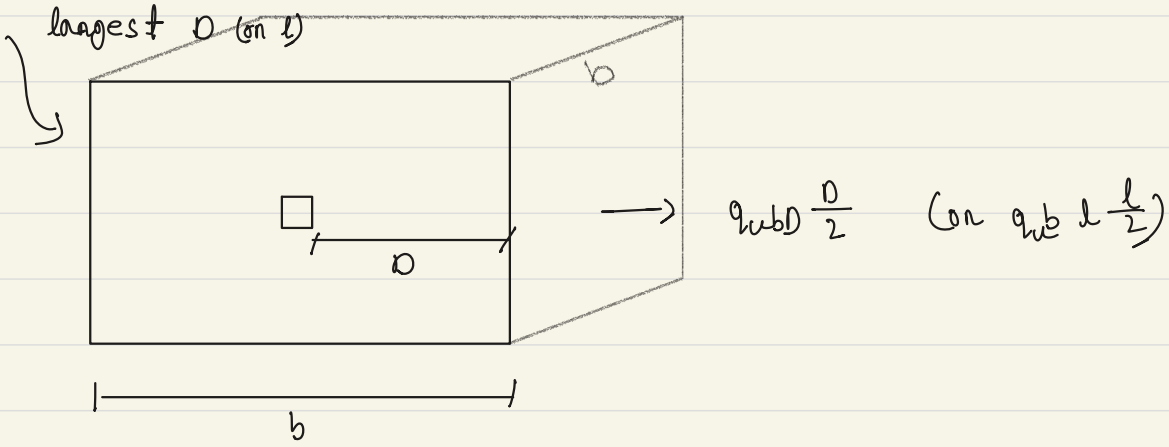
q_s = Reactive force on footing from soil

q_u = Factored ld → check one-way shear
two-way shear

Det^n of moment in footing at critical section



on next, consider



$q_u = 4.34$ ksf $b = 9'-4" = 112"$

$D = \frac{112-16}{2} = 48"$

$m_u = q_u b D \frac{D}{2} = 4.34 \times \frac{112}{12} \times \frac{(48 \div 12)^2}{2}$

$= 324$ k-ft

$R = \frac{m_u}{\phi b d^2} = \frac{324 \times 12 \times 1000}{0.9 \times 112 \times (20)^2} = 96.13$

$P = 0.0017$ $A_s = 0.0017 \times 112 \times 20 = 3.8$ in^2

$A_{s,min} = 0.0018\, b h = 4.84$ in^2 $>$ 3.8 in

$\therefore A_{s,req} = 4.84$ in^2 ← for one dir

col & footing joint, impo for EQ

ensures ld transfer from col to footing

col concrete stren > footing

bez ld is concentrated on col while spread out on footing

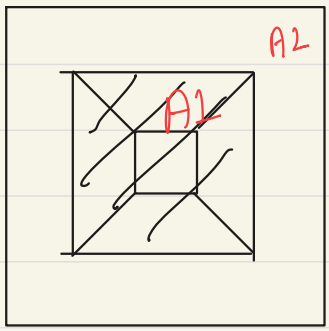
Development length (Table A.10, A.11)

Minimum required → 2', 12", ---

D - cover (3") > l_d

$A_s = 4.84\ in^2$ (#7 = 5.40 in^2)

Dowel bar
Bearing cap of col at base } not for exam



A2 > A1

min 4 bars, 3–4'

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3 ques from design → flat plate, col, footing

Tables be given incl flexural reinf

Assignment : Full math → tuesday

Flat plate

Panel : 24" x 26" Column = 20" x 20"

$w_D = 20$ psf ; $w_{LL} = 100$ psf

$f_c' = 4$ ksi ; $f_y = 60$ ksi

The provided info supports DDM analysis and is done below:

1. Slab thickness :

Exterior panel, $h = \frac{l_n}{30} = \frac{26 \times 12 - 20}{30} = 9.73 \approx 10"$

Interior panel, $h = \frac{l_n}{33} = \frac{26 \times 12 - 20}{33} = 8.85 \approx 9"$

$\therefore h = 10"$

$\therefore d = 10" - 0.75" - \frac{4"}{8} - \frac{4"}{2 \times 8} = 8.5"$

2 Factored load :

Self wt $= 150 \times \frac{10}{12} = 125 = 0.125$ ksf

$\therefore q_u = 1.2 w_D + 1.6 w_L = 1.2(20 + 125) + 1.6(100) = 334$ psf

$= 0.334$ ksf

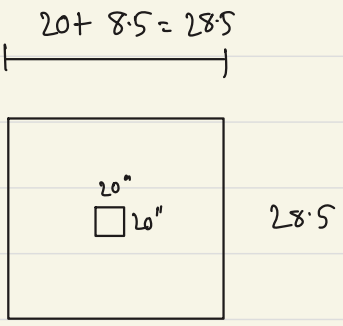
3. Shear check!

Punching shear:

$$V_u = q_u (A_t - A_c)$$

$$= 0.334 \left(24 \times 26 - \frac{28.5^2}{144}\right)$$

$$= 206.53 \text{ kip}$$



$$\sqrt{f_c'}\, b_o d = \sqrt{4000} \times 114 \times 8.5$$

$$= 61.28 \times 10^3$$

$b_o = 28.5 \times 4 = 114$ in

$\beta = 20/20 = 1$

$$\therefore V_n = \min\left(4\lambda\sqrt{f_c'} b_o d,\ \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_c'} b_o d,\ \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'} b_o d\right)$$

$$= \min\left(4 \times 61.28, (2+4) \times 61.28, \left(2 + \frac{40 \times 8.5}{114}\right) \times 61.28\right) \times 10^3$$

$$= \min(245.12,\ 367.68,\ 305.32) \times 10^3$$

$$= 245.12 \times 10^3 \text{ lb} \quad = 245.12 \text{ kip}$$

$$\therefore \phi V_n = 0.75 \times 245.12 = 183.84 \ < \ (V_u = 206.53)$$

$\therefore$ Drop-panel to be provided

Beam shear check:

$V_u = q_u \max(A_\ell, A_s)$

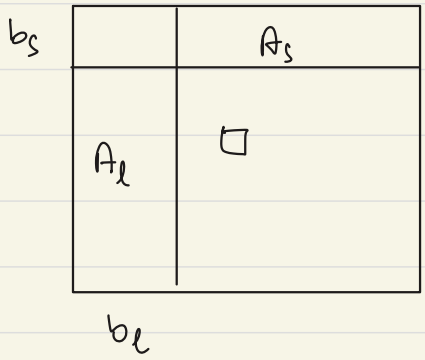
$= 0.334 \times 275 = 91.85$ kip

$V_n = 2\lambda\sqrt{f'_c}\, b_w d$

$= 2 \times \sqrt{4000} \times 10.46 \times 12 \times 8.5$

$= 134.96$ kip

$\therefore \phi V_n = 101.22$ kip [OK]



$b_\ell = \frac{26}{2} - \frac{10}{12} - \frac{8.5}{12} = 11.46$ in

$b_s = \frac{24}{2} - \frac{10}{12} - \frac{8.5}{12} = 10.46$ in

$A_\ell = b_\ell \times l_s = 11.46 \times 24 = 275$ ft^2

$A_s = b_s \times l_\ell = 10.46 \times 26 = 271.96$ ft^2

4. Reinforcement: (Longer dir)

Factored static moment, $M_o = \frac{q_u l_2 l_n^2}{8} = \frac{0.333 \times 24 \times (26 - \frac{20}{12})^2}{8}$

$= 593.3$ k-ft

Neg. moment, $M_{o(neg)} = 0.65 M_o = 385.6$ k-ft

Pos moment $M_{o(pos)} = 0.35 M_o = 207.7$ k-ft

Component	Column strip		Middle strip	
	Pos	Neg	Pos	Neg
Moment distribution	0.60	0.75	0.40	0.25
Moment	124.62	289.2	83.08	96.4
b	144	144	144	144
d	8.5	8.5	8.5	8.5
ρ	0.00273	0.00656	0.00180	0.00210
$A_{s,min} = 0.0018bd$	2.2	2.2	2.2	2.2
$A_s = \rho b d \geq A_{s,min}$	3.34	8.03	2.20	2.57
$S_{max} = 2h$ or 18"	20"	20"	20"	20"
Bar selection	17 #4	19 #6	11 #4	13 #4
S	9	8	14	12

4. Reinf. (shorten din)

5. d-check:

$$M_u = 0.9\,\rho b d^2 f_y \left(1 - 0.59\,\frac{\rho f_y}{f_c'}\right)$$

$\Rightarrow 289.2 \times 12 = 0.9 \ldots (\text{Q})$

$\Rightarrow d = 8.45 < 8.5''$

6. Drop panel

$$\text{thickness} = \frac{h}{4} = 2.5 \text{ in}$$

Extend $= 1/6 \times 26' = 4.33'$

$1/6 \times 24' = 4'$

Design of column:

Diffenence between shont column and long column:

1. Shont column: Ratio of effective length to its least lateral dimension is less than on equal to 12.

   Long column: Said ratio is greaten than 12.

2. Shont column: Ratio of effective length to least radius of gynation is less than on equal to 40.

   Long column: Said ratio is greaten than 40.

3. Shont column: Buckling tendency is low.

   Long column: Buckling tendency is high.

4. Shont column: Load carrying capacity is high compared to long column of same cross sectional area.

   Long column: Load carrying capacity is less than shont column of same cross section area.

5. Shont column: Genenally fails by crushing.

   Long column: Grenerally fails by buckling.

Theory:

$$f_s = n f_c ; \quad n = \frac{E_s}{E_c}$$

$$P = f_c' A_c + f_y A_{st}$$

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

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

0.65 for tied

0.85 for spinal 0.75 " spinal

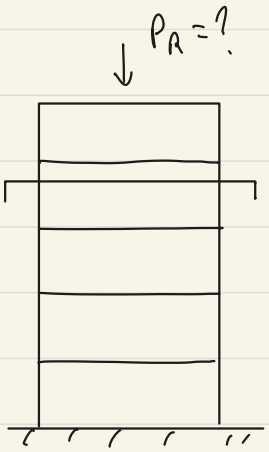
tie → #3 or larger for #10 & smaller longitudinal

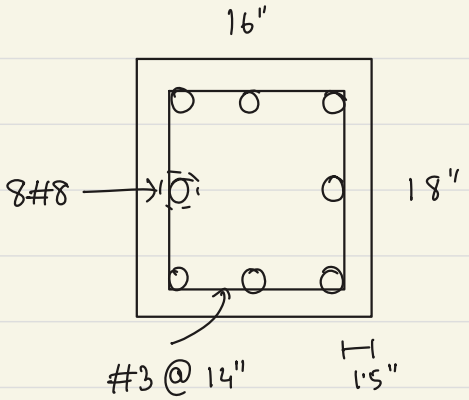
#4 " " " ≥ #11 bars

A_{st} 1% to 8% of A_g (pref ≤ 4%)

Assignment 06

$f_c' = 3$ ksi ; $f_y = 60$ ksi





1. Reinf. ratio check:

$$\rho = \frac{A_{st}}{A_g} = \frac{0.79 \times 8}{16 \times 18} = 2.19\% \qquad [\text{OK}]$$

2. Num of spacing

From table [ ],

For $h = 14"$, #8 bar = 12 > 8 [OK]

3. Max factored load:

$$P_R = \phi P_n = 0.80 \times 0.65 \left[0.85 f_c' (A_g - A_{st}) + f_y (A_{st})\right]$$

$$= 0.80 \times 0.65 \left[0.85 \times 3 \times (16 \times 18 - 8 \times 0.79) + 60 \times 0.79 \times 8\right]$$

$$= 570.7 \text{ kip}$$

4. Check spacing of ties:

$$s_{max} = \min(16 d_b, 48 d_t, b_{min})$$

$$= \min(16 \times 2'', 48 \times 3/8'', 16'')$$

$$= 16'' > 14'' \quad [OK]$$

5. Check spacing of longitudinal bars (max 6"):

$$s = \frac{18'' - 1.5 \times 2 - 2 \times 3/8 - 3 \times 8/8}{2}$$

$$= 5.62'' < 6''$$

Assignment 7.1

1. Tributary area $= \left(\frac{30}{2} + \frac{30}{2}\right)\left(\frac{30}{2} \times \frac{30}{2}\right) = 900\ ft^2$

2. Factored load:

Dead load:

Slab wt	$= 0.150 \times \frac{6}{12}$	$= 0.075$ ksf
Mechanical & electrical	$=$	0.005 ksf
Partition wall	$=$	0.015 ksf
		0.095 ksf

$\therefore P_{D,f} = 0.095 \times 900 + 0.150\left[3 \times (15+15) \times \frac{24}{12} \times \frac{16}{12}\right] + 0.150 \times \left[1 \times (15+15) \times \frac{24}{12} \times \frac{16}{12}\right)$

$= 133.5$ kip

$P_{LL,f} = 0.050$ ksf $\times$ 900 sft $= 45$ kip

Roof:

Slab wt	$= 0.075$ ksf
Roofing & insulation	$= 0.005$ ksf
	0.080 ksf

$\therefore P_{D,roof} = 0.080 \times 900 + 4 \times 0.150\left[30 \times \frac{24 \times 16}{144}\right] = 120$ kip

$P_{L,roof} = 0.030 \times 900 = 27$ kip

∴ Load on ground level:

$$P_D = 2 \times P_{D,floor} + P_{D,roof}$$

$$= 2 \times 133.5 + 120 = 387 \text{ kip}$$

$$P_L = 2 \times P_{L,floor}$$

$$= 2 \times 45 = 90 \text{ kip}$$

$$P_{Ln} = 27 \text{ kip}$$

$$1.83 P_{Ln} = 49 < P_L$$

$$\therefore P_u = 1.2 P_D + 1.6 P_L + 0.5 P_{u,Ln}$$

$$= 621.9 \text{ kip}$$

3. Reinforcement:

$$P_u \leq \phi P_n \qquad A_g = 16 \times 16 = 256 \text{ in}^2$$

$$\Rightarrow 0.65 \times 0.80 \left[0.85 f_c' (256 - A_{st}) + A_{st} \times 60\right] = 621.9$$

$$\Rightarrow A_{st} = 5.75 \text{ in}^2 \qquad [8 \#8]$$

$$\rho = \frac{8 \times 0.79}{A_g} = 2.27\% \qquad [OK]$$

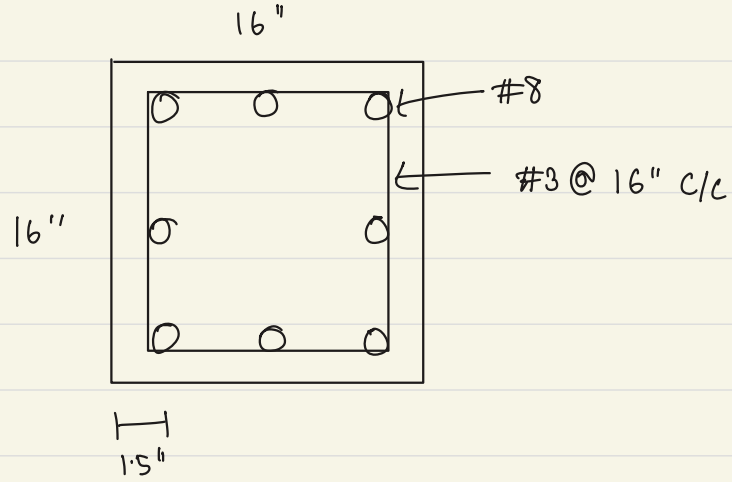
4. Design ties: #3

$$S_{max} = \min(16 d_b, 48 d_t, b_{min})$$

$$= \min(16 \times 2, 48 \times 3/8, 16")$$

$$= 16"$$

$\therefore$ #3 @ 16" c/c



Check arrangement:

$$\text{clear spacing} = \frac{16 - 2 \times 1.5 - 2 \times 3/8 - 3 \times 1}{2} = 4.63" < 6"$$

Recap: Design of interior columns:

1. Tributary area
2. Unfactored load: $P_{D,floor}$, $P_{L,floor}$, $P_{D,roof}$, $P_{L,roof}$
3. Factored load: $P_u = 1.2P_D + 1.6P_L + 0.5P_{Ln}$
4. Reinforcement
5. Steel ratio
6. Design ties
7. Detailing
8. Check arrangement

Design of exterior column:

Assignment 8:

$L_D = 222$ kips
$L_L = 297$ kips
$M_D = 136$ k-ft
$M_L = 194$ k-ft

$f_c' = 4$ ksi
$f_y = 60$ ksi

(a) Factored load: $P_u = 1.2L_D + 1.6L_L$

$= 741.6$ kips

Factored moment: $M_u = 1.2M_D + 1.6M_L$

$= 473.6$ k-ft

Size of column: 20" x 20"

Clear cover = 2.5"

Steel ratio: $\gamma h = (20 - 2.5 \times 2)$

$\Rightarrow \gamma = \frac{15}{20} = 0.75$

$$K_n = \frac{P_u}{\phi f_c' A_g} = \frac{741.6}{0.65 \times 4 \times 20 \times 20} = 0.713$$

$$R_n = \frac{M_u}{\phi f_c' A_g h} = \frac{473.6 \times 12}{0.65 \times 4 \times (20 \times 20) \times 20} = 0.273$$

$P_{(\gamma=70)} = 0.066$
$P_{(\gamma=80)} = 0.057$
} 0.0615

$\therefore A_{st} = 0.0615 \times 20 \times 20$

$= 24.6 \text{ in}^2$ [16#11]

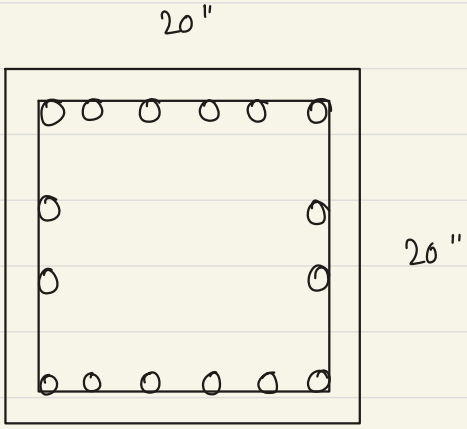
(b) when no live ld on roof :

. . .

Design ties : #4

$$S_{max} = \min(16 d_b, 48 d_t, b_{min})$$

= . . .



$$\text{Clear spacing} = \frac{20 - 2\times 1.5 - 2\times \frac{4}{8} - 6\times \frac{11}{8}}{5} = 1.55"$$

$$\text{Clear spacing} = \frac{20 - \ldots}{3} = \ldots$$

Design of footing

Assignment 10 : 16" sqr interior col

$L_D = 200^K$, $L_L = 80^K$, $L_{Ln} = 20^K$

$q_a = 35$ ksf

f_c' of col = 4 ksi, f_c' of footing = 3 ksi

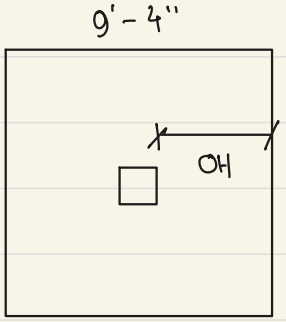
$f_y = 60$ ksi

1. Footing size:

$$P_T = L_L + L_D + L_{Ln} = 300 \text{ kip}$$

$$\therefore A = \frac{P_T}{q_a} = \frac{300}{3.5} = 85.71 \text{ ft}^2$$

$$\therefore b = \sqrt{85.71} = 9.3' \approx 9'-4"$$



2. Thickness:

$$h = 1/2(OH) = 24" > 12"$$
$$= 24"$$

$$OH = \frac{9 \times 12 + 4 - 16}{2}$$

$$= 48 \text{ in}$$

3. Factored load:

$$P_u = 1.2 L_D + 1.6 L_L + 0.5 L_{Ln}$$

$$= 378 \text{ kip}$$

$$1.89 P_{Ln} = 36.6 < 80$$

$$\therefore q_u = \frac{378}{(9+\frac{4}{12})^2} = 4.34 \text{ ksf} > q_a$$

but S.F is high ...

4. Shear check:

h - cover - bar dia

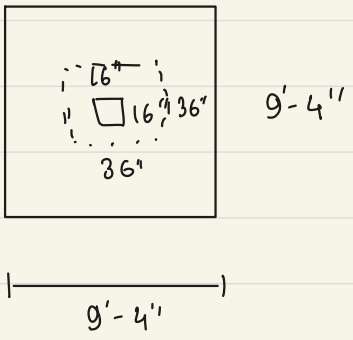
$d = 24'' - 3'' - 1'' = 20''$

Punching shear:

$V_u = q_u (A - A_c)$

$= 4.34 \left[\left(9 + \frac{4}{12}\right)^2 - \left(\frac{36}{12}\right)^2 \right]$

$= 339 \text{ kip}$



$\lambda \sqrt{f_c'}\, b_o d = \left(2\sqrt{3000} \times 144 \times 20\right) \frac{1}{1000}$

$= 157.7$

$\beta = \frac{16}{16} = 1$

$b_o = 36 \times 4 = 144 \text{ in}$

$\therefore V_n = \min\left[4\lambda\sqrt{f_c'}\, b_o d,\ \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_c'}\, b_o d,\ \left(2 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'}\, b_o d \right]$

$= \min[630.8,\ 946,\ 1192]$

$= 630.8 \text{ kip}$

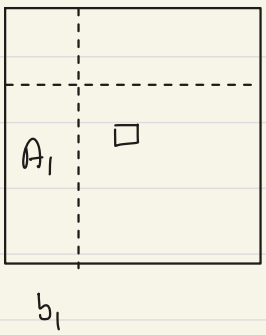
$\phi V_n = 0.75 \times 630.8 = 473.1 \text{ kip} > V_u$

Beam shear:

$V_u = q_u (A_1)$

$= 4.34 \times \left[2.33 \times \left(9 + \frac{4}{12}\right)\right]$

$= 94.38^k$



$V_n = 2\lambda\sqrt{f_c}\, b_w d$

$= 2\sqrt{3000} \times (9 \times 12 + 4) \times 20 = 245.38^k$

$\therefore \phi V_n = 184 > V_u$

$b_1 = \left(9 + \frac{4}{12}\right)\frac{1}{2} - \frac{16}{2 \times 12} - \frac{20}{12}$

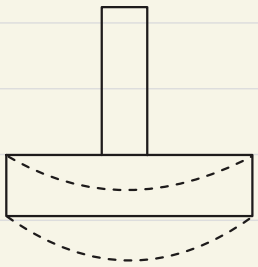
$= 2.33'$

5. Reinforcement :

$$M_u = \frac{q_u \times l \times D^2}{2}$$

$$= \frac{4.34 \times (9 + \frac{4}{12}) \times 4^2}{2}$$

$$= 324 \text{ k-ft}$$



$$D = \left(9 + \frac{4}{12}\right)\frac{1}{2} - \frac{16}{2 \times 12} = 4'$$

$$M_u = \phi \rho b d^2 f_y \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)$$

$$\Rightarrow 324 \times 12 = 0.9 \rho \times (9 \times 12 + 4) \times 20^2 \times 60 \left(1 - 0.59 \frac{\rho \times 60}{3}\right)$$

$$\Rightarrow \rho = 0.00164$$

$$\therefore A_s = \rho b d = 0.00164 \times (9 \times 12 + 4) \times 20 = 3.67 \text{ in}^2$$

$$A_{s,min} = 0.0018 b h = 0.0018 (9 \times 12 + 4)(24) = 4.84 \text{ in}^2$$

$$\therefore A_s = 4.84 \text{ in}^2$$

Recap: Footing

1. Detn area → size of footing $\left[A = \frac{q_u}{P_T}\right]$
2. Det^n thickness → 0.5 (D H)
3. Det^n factored load → $q_u = 1.2L_D + 1.6L_L + 0.5L_{Ln}$
4. Check shear [for beam shear, $b_w = b$]
5. Det^n reinf $\left[M_u = \frac{q_u l D^2}{2}\right]$