Colums
Slenderness
Sway
Design
Gross Area
Details
Example
Eccentric Loads
Example

— Column Slenderness

A column is short if:

non-sway (braced) frame [10.10.1]

$$\frac{k\,l_u}{r} \leq 34 - 12\left(\frac{m_1}{m_2}\right) \leq 40$$

- assume k (unless specified)

l_u = unbraced

radius of gyration $r \equiv \sqrt{\frac{I_g}{A_g}} = \sqrt{\frac{\frac{1}{12}bh^3}{bh}} = \sqrt{\frac{1}{12}}\,h$

$m_1 \equiv$ minimum moment

$M_2 \equiv$ maximum moment

$\left(\frac{M_1}{M_2}\right) = \begin{matrix}(+) \text{ single curvature} \\ (-) \text{ double curvature}\end{matrix}$

— SWAY (unbraced) [10.10.1]

$$\frac{k\,l_u}{r} \leq 22$$

* The focus in this class is NON-sway Frames with short colums *

Short ≡ Non-Slender

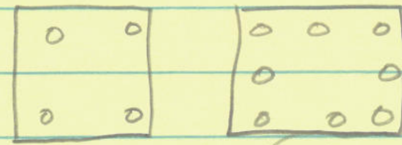
— Column Design

- Structural Analysis
- Estimate column cross section [2-4]
- Analyze using interaction diagram [5-7]
- Iterate
- Detail Column [8-12]

* Focus on tied columns *

- Estimate Gross AREA
  - Assume column is square & has steel on all faces

symmetric



A_g & reinforcement ratio can vary

$$A_g \geq \frac{P_u}{0.45(f'_c + f_y \rho_{TRIAL})}$$

ρ_g = reinforcement ratio

$$\rho_g = \frac{A_s}{A_g} = \frac{A_s}{bh}$$

0.01 - 0.03 typically

$$0.01 \leq \rho_g \leq 0.08 \quad [10.9.1]$$

- Column Details

  TIES [7.10.5]

  longitudinal bars ≤ #10, #3

  " 11, 14, 18, #4

Maximum spacing of ties.

$$\min \begin{cases} 16 d_b \text{ of longitud. bars} \\ 48 d_b \text{ of ties} \\ \min b, h \end{cases}$$

See [7.10.5.4] if near slab

- Shear [11.2.1.2]
  ties act as shear stirrups

- Example Design a rectangular column (tied) with long side equal to two times the short side.

$P_D = 650^k$ $P_L = 400^k$

$f'_c = 3\ ksi$ $f_y = 60ksi$

initially assume $\rho_{gross} = 0.02$

$A_{steel} = \rho A_g$

Select column size

$P_u = 1.2(650^k) + 1.6(400^k) = 1420^k$

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

$1420 = 0.80(0.65)[0.85(3)(A_g - 0.02A_g) + 60(0.02A_g)]$

$A_g = 738.25 in^2$

↳ use 20X40 column ($A_g = 800\ in^2$)

Select reinforcement bars

$1420 = 0.80(0.65)[0.85(3)(800 - A_{st}) + 60(A_{st})]$

$A_{st} = 12.02\ in^2$

use 8 #11 bars ($12.50\ in^2$)

Design Ties

Max spacing = min

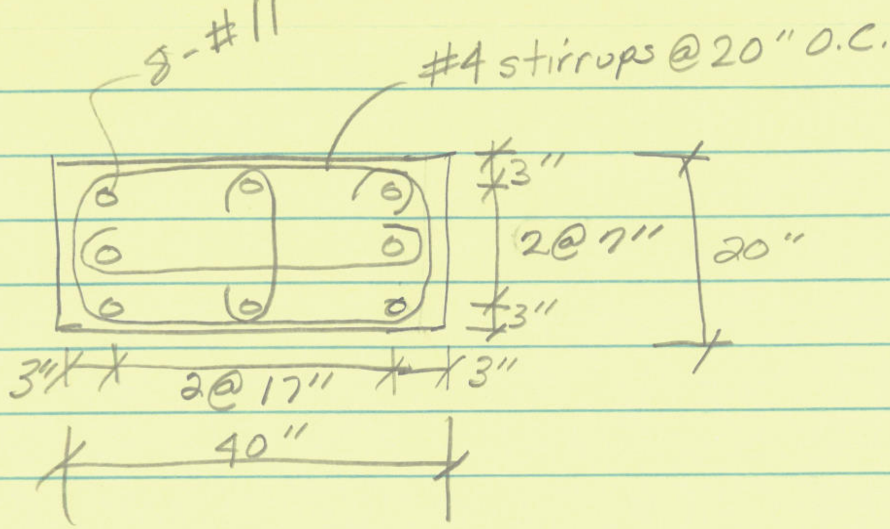
Longitudinal bars

$16d_b = 16(1.41) = 22.56''$

$48\ d_b = 48(0.5) = 24''$

least column dim $= 20''$

use #4 ties @ 20" O.C.



Check code 7.10.5 for maximum spacing of vertical bars

Design of Eccentrically Loaded Columns Using INteraction Diagrams

11/8/10

Column Design Using INteraction Diagrams Examples

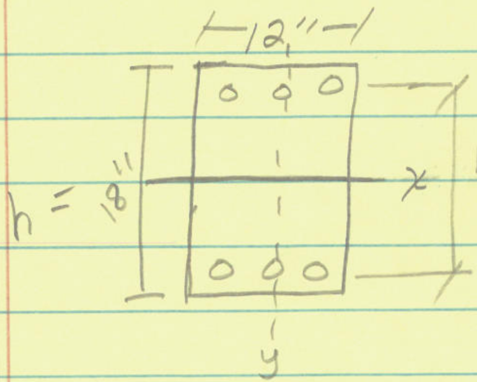
If we prepared them for all combinations there would be too many.

X-axis: $\frac{P_n e}{f'_c A_g h} = R_n$

y-axis: $\frac{P_n}{f'_c A_g} = K_N$

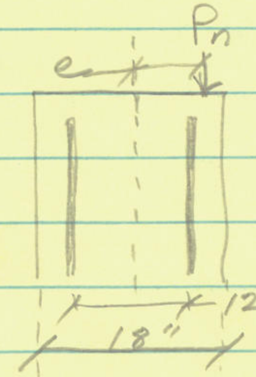
Warning: Be sure the picture agrees with the column being consider'd

Example: Use the interaction curves in Appendix A to select reinforcing $f'_c = 4\text{ksi}$ $f_y = 60\text{ ksi}$



$e_x = 8''$

$P_n = 400^k$



① Structural Analysis (done already)

$P_n = 400^k$

$M_{n_x} = P_n \cdot e_x = (400^k)(8'') = 3200 \text{ in-}k_x$

② Select column cross section

- already done

$\frac{b}{h}$ about bending axis

(3) Analyze Column

$\gamma = \text{ratio} = \frac{12''}{18''} = 0.667$

Left side of chart — $K_n = \frac{P_n}{f'_c A_g} = \frac{400^k}{4\,ksi\,(12'' \times 18'')} = 0.463$

Bottom of chart — $R_n = \frac{P_n e}{f'_c A_g h} = \frac{(400^k)(8'')}{(4\,ksi)(12'')(18'')(18'')} = 0.206$

from chart 0.60 | interpolate | from chart 0.70

γ	0.600	0.667	0.70
ρ_g	0.03	0.026	.024

$A_g = \rho_g b h \qquad = 0.026(12)(18)$

$A_g = 5.62\,in^2$ use 6 #9 ($A_g = 6.0\,in^2$)

Check fit of bars [7.6.3]

Clear Distance: min. $1.5\,d_b$ or $1\frac{1}{2}''$ [7.6.3]; $4/3$ max agg. size (3.3.2)

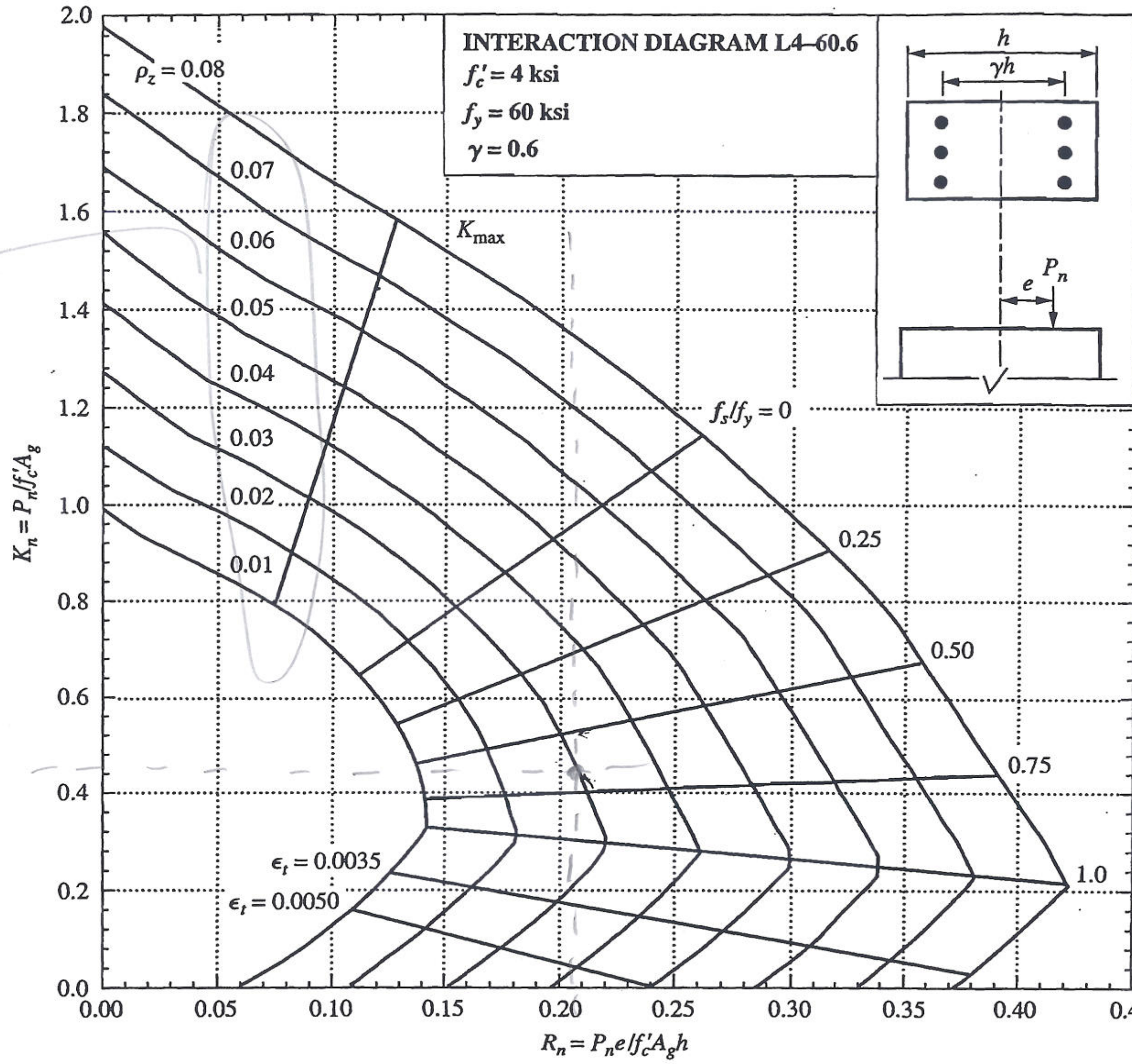
$1.5(1.25) = 1.6875$

$1.5 \longrightarrow 1.5''$

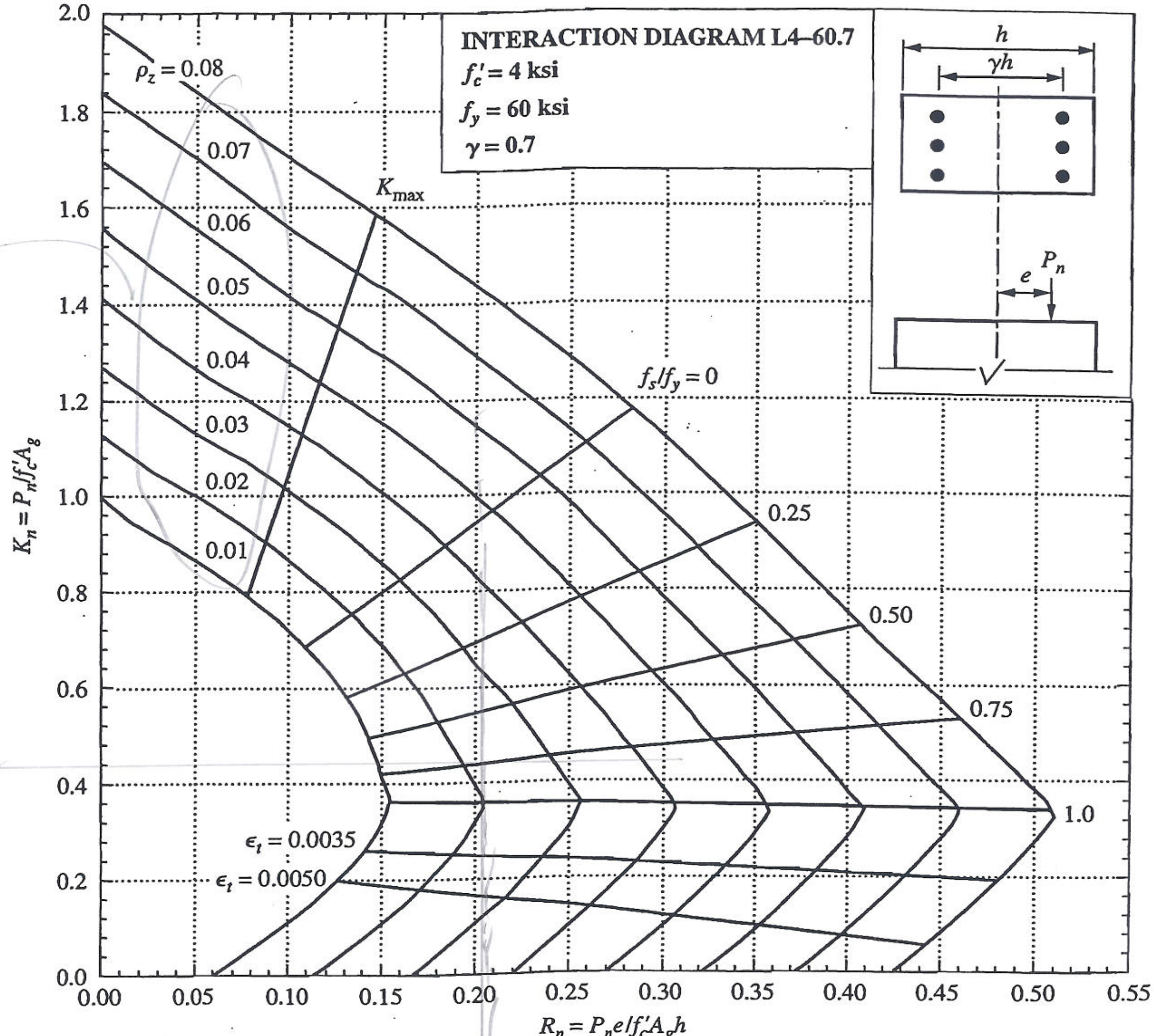
$4/3(1'') = 1.25$

$b_{req} = \underbrace{3(1.125)}_{\text{bars}} + \underbrace{2(3/8)}_{\text{ties}} + \underbrace{2(1.5)}_{\text{Spacing between bars}} + \underbrace{2(1)}_{\text{cover}}$

$= 9.125 < 12''$ ok



Graph 2 Column interaction diagrams for rectangular tied columns with bars on end faces only. (Graphs 2 through 13 are published with the permission of the American Concrete Institute.)



Graph 3 Column interaction diagrams for rectangular tied columns with bars on end faces only.

Ties

#3 bars for #9 longitutinal bars

$$\text{Tie } S_{max} = \min \begin{cases} 16 d_{bar} = 16(1.125) = 18'' \\ 48\, d_{tie} = 48(3/8'') = 18'' \\ b = 12'' \end{cases}$$

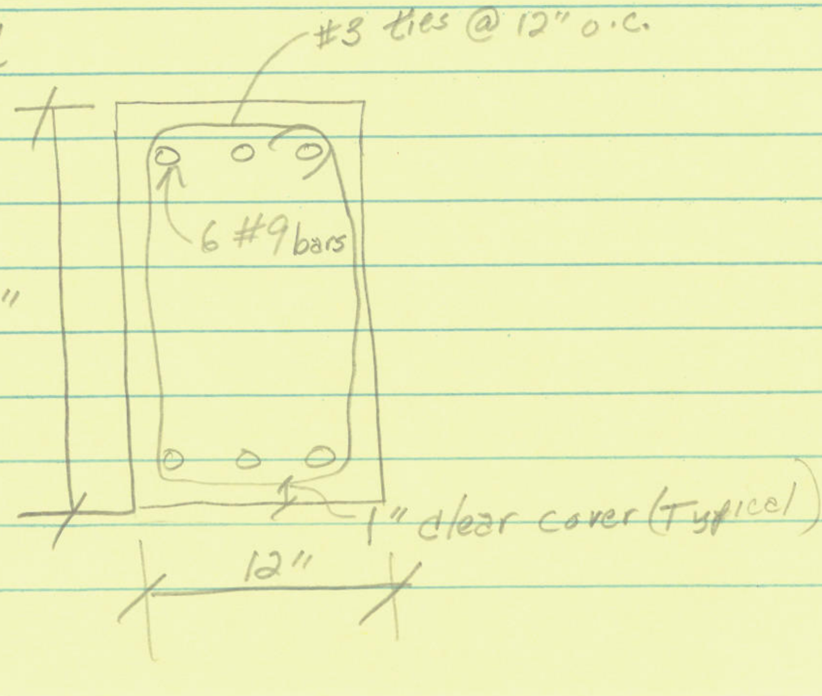
$S_{max} = 12''$

Check Shear

$$V_c = 2\boxed{\left[1 + \frac{N_u}{2000 A_g}\right]}^{*} \sqrt{f'_c}\; b_w\, d \quad [\text{eq } 11\text{-}4]$$

* Accounts for increased capacity due to aggregate interlock

#5 Detail



→ Purpose of foundations

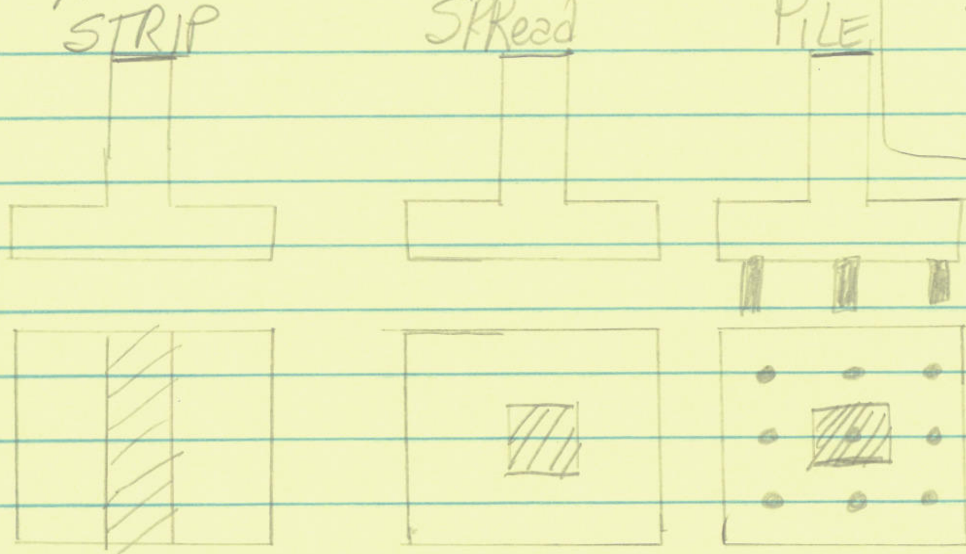
To safely transmit load from the Structure to the ground without unsafe differential settlement

→ What if settlement?

EVEN → Not huge deal (Mexico City)

UNEVEN → BIG DEAL (Los Vegas Casino Leaning Tower of Pizza)

→ Types of Foundation

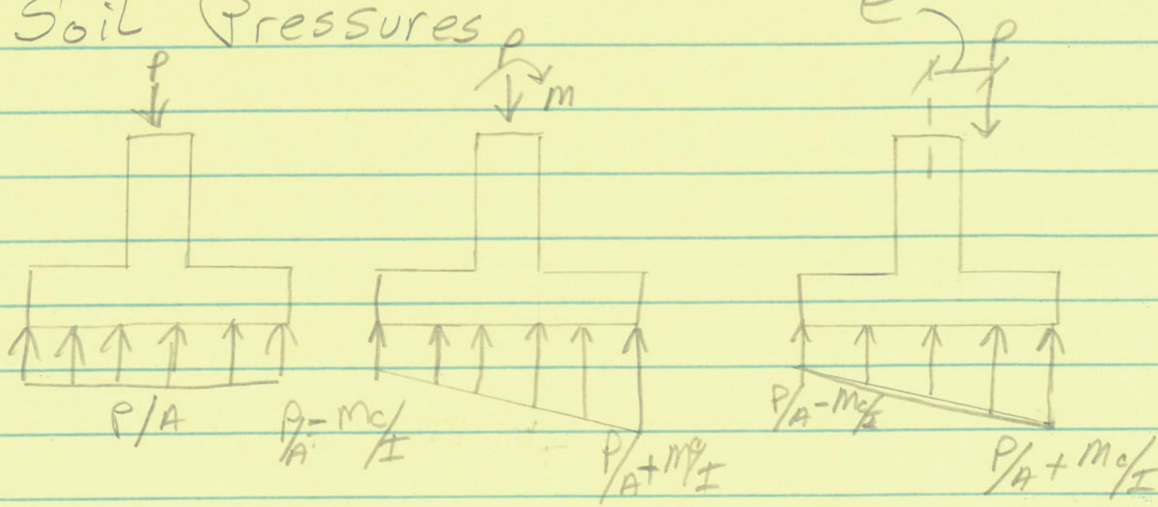


→ Raft or Mat

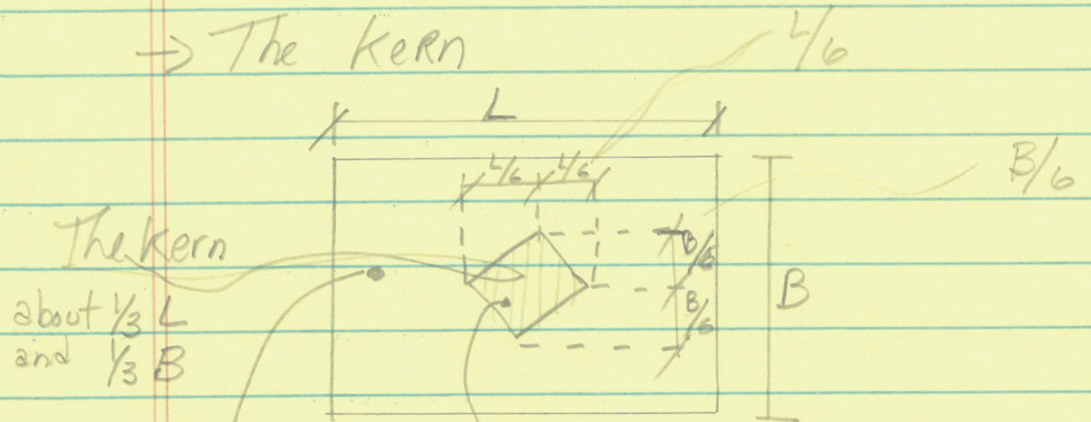
Large foundation under entire structure

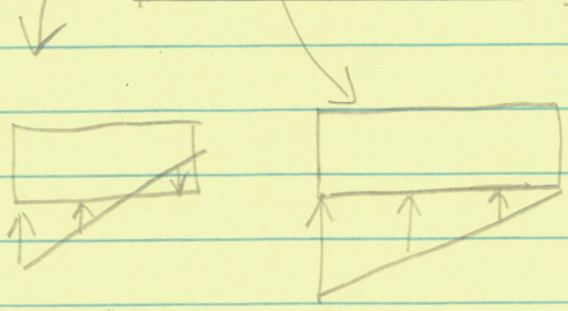
Foundations
Types
Soil Pressures
Kern
Limit States
A required
Design of Footings
Soil Failure
One-Way Slab
2way Shear
Flexure
Bearing/Load Transfer

→ Soil Pressures



→ The Kern





* Load must fall in the kern of the footing

→ Soil Limit

- Soil bearing failure

$q_{act} < q_{allow} = \dfrac{q_{ultimate}}{FS}$

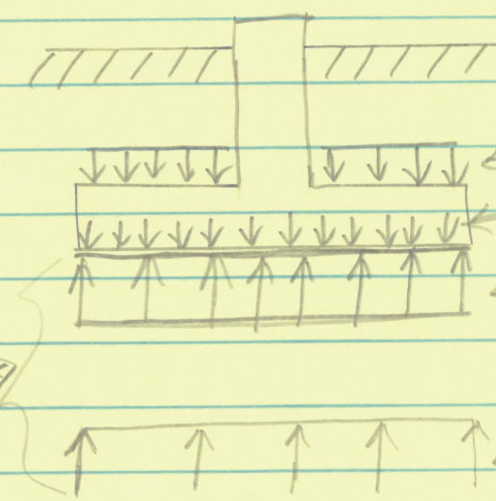
From geo tech

From service loads

FS usually 2.5 – 3.0

- differential settlement
- excessive settlement

→ Calculations of Soil Pressure



→ A required of Spread footing

Soil Capacity $\quad q_{gross} \leq q_{all}$

$$A_{req'd} = \frac{D\begin{Bmatrix}\text{structure}\\ \text{footing}\\ \text{over burden}\end{Bmatrix} + L}{q_{allowed}}$$

service loads

→ other structural elements

$q_{net} = \dfrac{\text{Max factored Load}}{A_{footing}}$

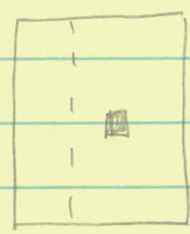
usually $1.2D_L + 1.6_{LL}$

→ Spread Footing Limit State (Things we need to check

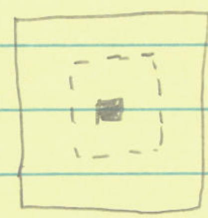
Soil Failure $q_{act} \leq q_{allowed}$

Shear

1 way →



2 way →

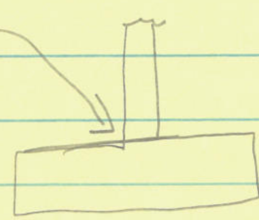


→ Flexure $\phi M_n \geq M_u$

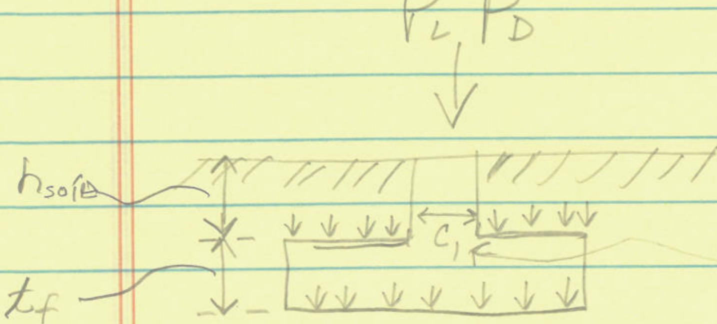
l_d

→ Bearing/Load transfer

@ joint



Soil Failure



h_{soil} = depth of surcharge
t_f = thickness of footing
rule of thumb $\approx 1.5 c_1$

$q_{actual} \leq q_{allowed}$ ASD

$$q_{act} = \gamma_{soil} (x) h_{soil} + \underset{\text{weight of footing}}{W_c} t_f + \frac{P_D + P_L}{A_{required}}$$

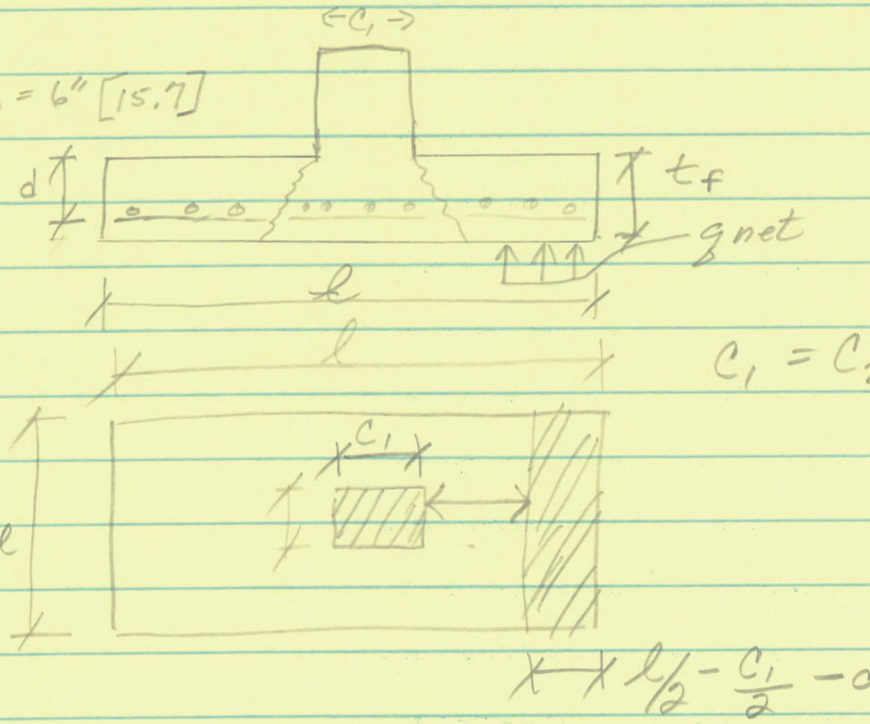
$$\frac{P_D + P_L}{A_{req}} \leq q_{allowed} - \gamma_s h_s - W_c t_f$$

$$\Downarrow$$

$$A_{req} = \frac{P_D + P_L}{\underbrace{q_{allowed}}_{\text{max allowed}} - \underbrace{\gamma_s h_s}_{\text{soil}} - \underbrace{W_c t_f}_{\text{footing}}}$$

One Way Shear

$d_{min} = 6''$ [15.7]



$c_1 = c_2 = c$ for square

$$V_u = l\left(\frac{l}{2} - \frac{c}{2} - d\right) q_{net}$$

$$V_c = 2\sqrt{f'_c}\; l d \qquad \phi = 0.75$$

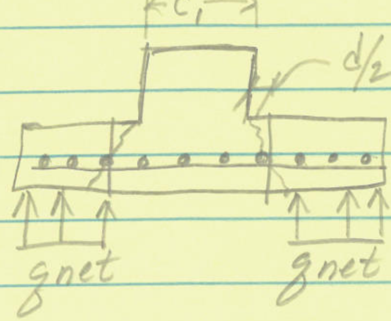
[11.4.5.1]

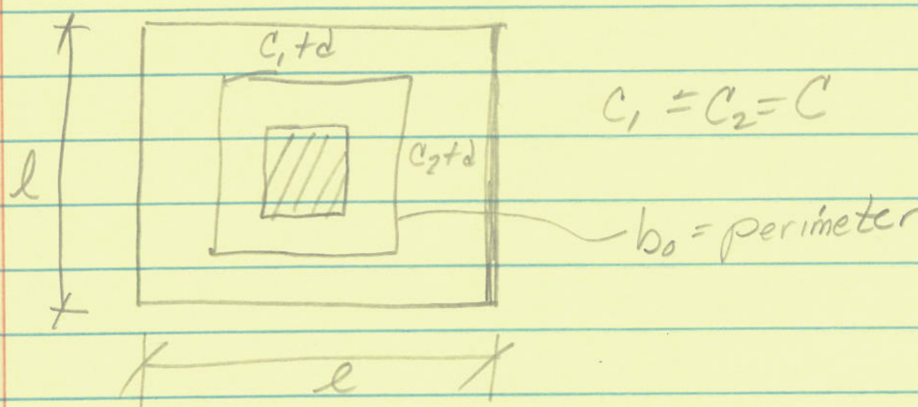
If $\phi V_c > V_u \rightarrow$ No shear required

If Shear problems

- Increase t_F
- Use Shear reinforcing

Two Way Shear





$$V_u = [\ell^2 - (c+d)^2]\ q_{net}$$

$$V_c = \min \begin{cases} \left(2 + \frac{4}{\beta}\right)\sqrt{f'_c}\ b_o d \\[2ex] \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\ b_o d \\[2ex] 4\sqrt{f'_c}\ b_o d \end{cases}$$

β = ratio long side to short side

α_s = 40 interior column
30 edge column
20 corner column

If $\phi V_c \geq V_u$, no shear reinf. needed