REINFORCED CONCRETE - II

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A Hand-note On

REINFORCED CONCRETE - II

CE 317

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Topics

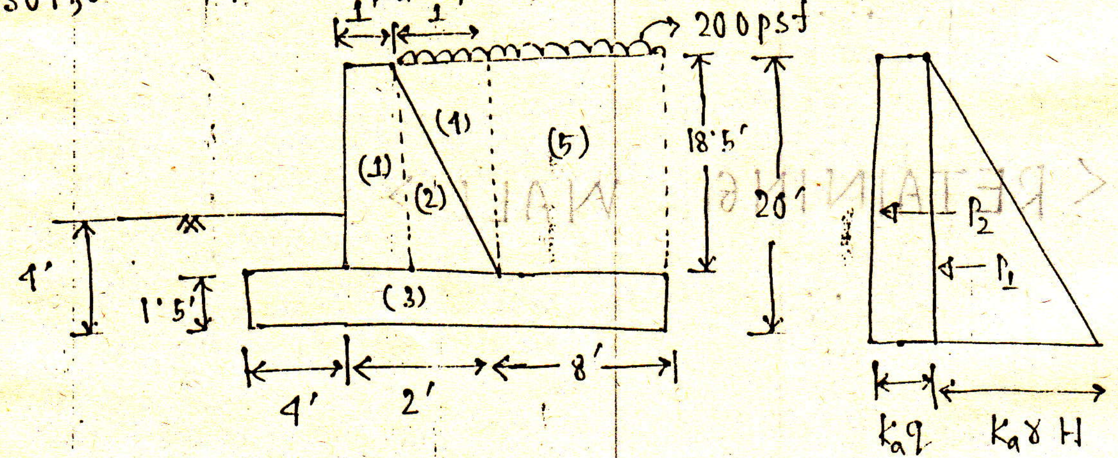
Retaining wall
Two way slab

< RETAINING WALL >

Retaining wall.

2007.

8.(b) Design the heel of retaining wall shown in figure below. Given, $\phi = 31°$, $f = 0.5$, unit weight of soil, $\gamma = 100$ psf, $q_a = 3$ ksf, $f_c' = 3$ ksi, $f_y = 60$ ksi.



Total overturning moment, $M_o = M_{o_1} + M_{o_2}$ ——— (1).

$M_{o_1} = P_1 \times \frac{H}{3} = \frac{1}{2} K_a \gamma H^2 \times \frac{H}{3}$

$= 37.33$ k-ft.

$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

$= 0.28$.

$M_{o_2} = P_2 \times \frac{H}{2} = K_a q \times H \times \frac{H}{2} = 11.2$ k-ft.

$M_o = 48.53$ k-ft.

$P = P_1 + P_2 = \frac{1}{2} \times K_a \gamma H^2 + K_a q H = 6720$ lb.

$= 6.72$ kip.

Resisting moment, M_R

Section	W (kip)	moment arm (ft)	moment (kip-ft)
1	$1 \times 18.5 \times .150$	4.5	12.49
2	$\frac{1}{2} \times 1 \times 18.5 \times .15$	$5 + \frac{1}{3}$	7.4
3	$14 \times 1.5 \times 0.15$	7	22.05
4	$\frac{1}{2} \times 1 \times 18.5 \times 0.1$	$5 + \frac{2 \times 1}{3}$	5.24
5	$8 \times 18.5 \times 0.1$	$6 + \frac{8}{2}$	1.48

$\therefore \Sigma W = 23.04$ kip.

$M_R = 195.18$ kip-ft.

Stability check:

(i) $F.S \text{ against sliding} = \frac{\Sigma W \times f}{P} = \frac{23.04 \times 0.5}{6.72} = 1.71 > 1.5$ (OK)

(ii) $F.S \text{ against overturning} = \frac{M_R}{M_o} = \frac{195.18}{~~53.84~~\ 48.53} = 4.02 > 1.5$ (OK).

(iii) Check against soil pressure:

$$P_1 = \frac{\Sigma W}{A} + \frac{Mc}{I} \quad \text{——— (i)}$$

$$P_2 = \frac{\Sigma W}{A} - \frac{Mc}{I} \quad \text{——— (ii)}$$

$$M = \Sigma W \times e.$$

$$e = \frac{b}{2} - a.$$

$$= \frac{b}{2} - \left(\frac{M_R - M_o}{\Sigma W}\right)$$

$$= \frac{14}{2} - \left(\frac{195.18 - 48.53}{23.04}\right)$$

$$= 0.63.$$

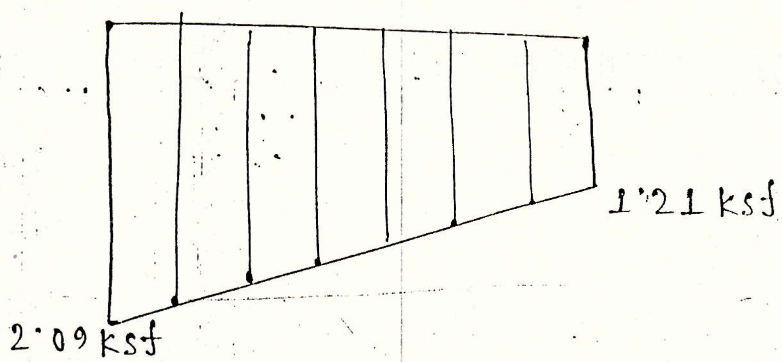
$\therefore M = \Sigma W \times e = 23.04 \times 0.63 = 14.515$ kip-ft.

$A = b \times 1 = 14 \times 1 = 14\ ft^2$

$I = \frac{1 \times 14^3}{12} = 228.67\ ft^4$. $\quad c = \frac{b}{2} = 7$ ft

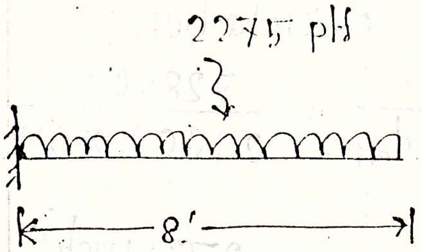
$$P_1 = \frac{23.04}{14} + \frac{14.515 \times 7}{228.67} = 1.65 + 0.44 = 2.09\ ksf < 3\ ksf$$

$$P_2 = 1.65 - 0.44 = 1.21\ ksf. < 3\ ksf$$



pressure diagram.

Design of Heel:



Load calculation:

(i) Self weight = $1.5 \times 150 = 225$ psf

(ii) soil weight = 18.5×100 = ~~2775 psf~~ 1850 psf.

(iii) Surcharge = 200 psf.

For safety purpose, upward soil pressure is neglected.

$W = 225 + \sout{2775}\ 1850 + 200 = 2275 \text{ psf} \times 1 \text{ ft} = 2275$ plf.

$M_{max} = \frac{WL^2}{2} = \frac{2275 \times 8^2}{2} = 72800$ lbft

$V_{max} = \frac{WL}{2} = \frac{2275 \times 8}{2} = 9100$ lb.

Depth check:

$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{72800 \times 12}{205 \times 12}}$

$= 18.88''$.

$d_{effective} = 1.5 \times 12 - 3$.

$= 15'' < d$

More thickness should be provided.

$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{3000}}$

$= 9.28 \approx 9$.

$r = \frac{f_s}{f_c} = \frac{24}{0.45 \times 3} = 17.78$.

$k = \frac{n}{n+r} = 0.34$.

$J = 0.89$.

$R = \frac{1}{2} f_c J k = 204.26$ psi

≈ 205 psi.

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d_{eff}} = \frac{72800 \times 12}{24000 \times 0.89 \times 15}$$

$$= 2.73 \text{ inch}^2.$$

Using #9 bar, $S = \frac{1 \times 12}{2.73} = 4.39 \approx 4.25''$ c-c.

$(A_s)_{min} = 0.0018\,bt = 0.0018 \times 12 \times 18 = 0.39$ inch^2.

#5, spacing $= \frac{0.31 \times 12}{0.39} = 9.53 = 9.5''$ c-c.

Shear check

$$V_{all} = \frac{V_{max}}{bd} = \frac{9100}{12 \times 15} = 50.56 \text{ psi}.$$

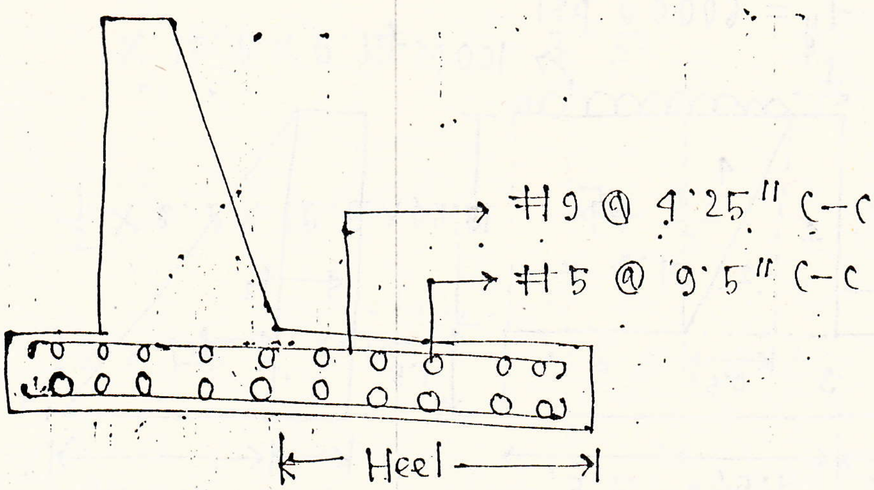
$$V_c = 1.1\sqrt{3000} = 60.25 \text{ psi} > V_u \text{ (OK)}.$$

Development length:

$$L_d = \frac{\phi f_s}{4 u_d}$$

$$u_d = \frac{V_{max}}{\Sigma_o j d} = \frac{9100}{\pi \times \frac{9}{8} \times \frac{12}{4.5} \times 0.89 \times 15} = 72.32 \text{ psi}$$

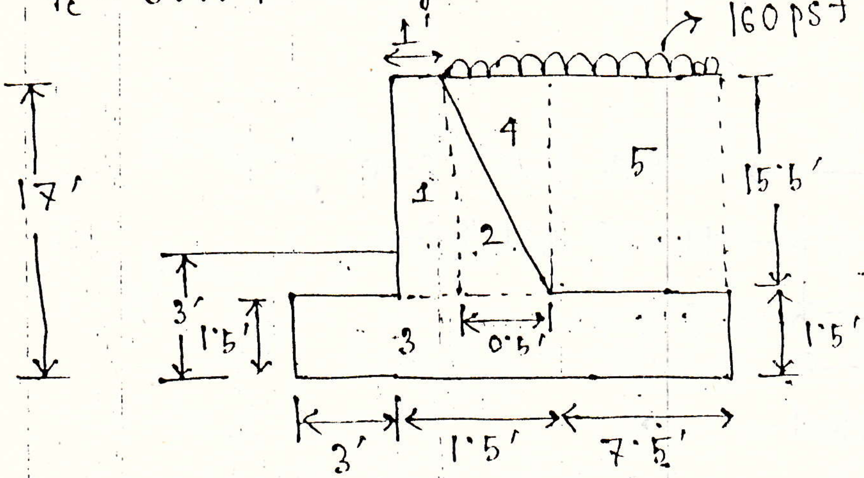
$$L_d = \frac{24000 \times \frac{9}{8}}{4 \times 72.32} = 93.34''$$

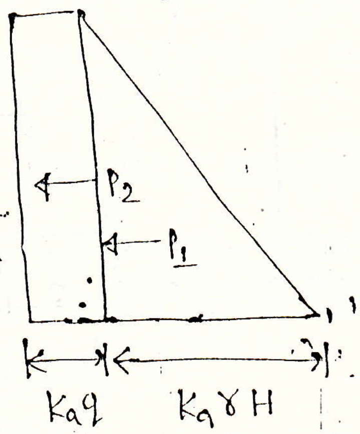


#9 @ 4.25" C-C
#5 @ 9.5" C-C
Heel

2006

6 b Design the stem of the retaining wall shown in figure below. $\phi = 32°$, $f = 0.4$, $\gamma_{soil} = 110$ psf, $f'_c = 3000$ psi, $f_y = 60000$ psi.





stability check: same as before.

$$M_o = M_{o_1} + M_{o_2} \quad \text{——— (1)}$$

$$M_{o_1} = P_1 \times \frac{H}{3} = \frac{1}{2} K_a \gamma H^2 \times \frac{H}{3} = 27.92 \text{ K-ft.}$$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.31$$

$$M_{o_2} = P_2 \times \frac{H}{2} = K_a q H \times \frac{H}{2} = ~~4.78 \text{ K-ft}~~ \; 7.16 \text{ Kft}$$

$$M_o = ~~32.7 \text{ Kft}~~ \; 35.09 \text{ Kft.}$$

$$P = P_1 + P_2 = K_a q H + \frac{1}{2} K_a \gamma H \times H = 5.77 \text{ kip.}$$

Resisting moment:

Section	weight (kip)	Moment arm (ft)	moment (kip-ft)
1	$1 \times 15.5 \times 0.15$	3.5	8.14
2	$\frac{1}{2} \times 0.5 \times 15.5 \times 0.15$	$4 + \frac{0.5}{3} = 4.17$	2.42
3	$12 \times 1.5 \times 0.15$	6	16.2
4	$\frac{1}{2} \times 0.5 \times 15.5 \times 0.11$	$4 + \frac{2 \times 0.5}{3} = 4.33$	1.86
5	$15.5 \times 7.5 \times 0.11$	$4.5 + \frac{7.5}{2} = 8.25$	105.50
	$\Sigma W = 18.82$ kip		$M_R = 134.12$ kip-ft.

Stability check:

(i) F.S. against sliding $= \frac{\Sigma W \times f}{P} = \sout{1.63}\ 1.30 < 1.5$ (Not ok).

∴ key must be provided.

(ii) F.S against overturning $= \frac{M_R}{M_O} = 3.82 > 1.5$ (OK)

Check against soil pressure:

$$P_1 = \frac{\sum W}{A} + \frac{MC}{I}$$

$$P_2 = \frac{\sum W}{A} - \frac{MC}{I}$$

$$M = \sum W \times e$$

$$e = \frac{b}{2} - a$$

$$= \frac{b}{2} - \left(\frac{M_R - M_o}{\sum W}\right)$$

$$= \frac{12}{2} - \left(\frac{134.12 - 35.09}{18.82}\right)$$

$$= 0.74$$

$$\therefore M = 18.82 \times 0.74 = 13.89 \text{ kip ft}$$

$$c = \frac{12}{2} = 6'$$

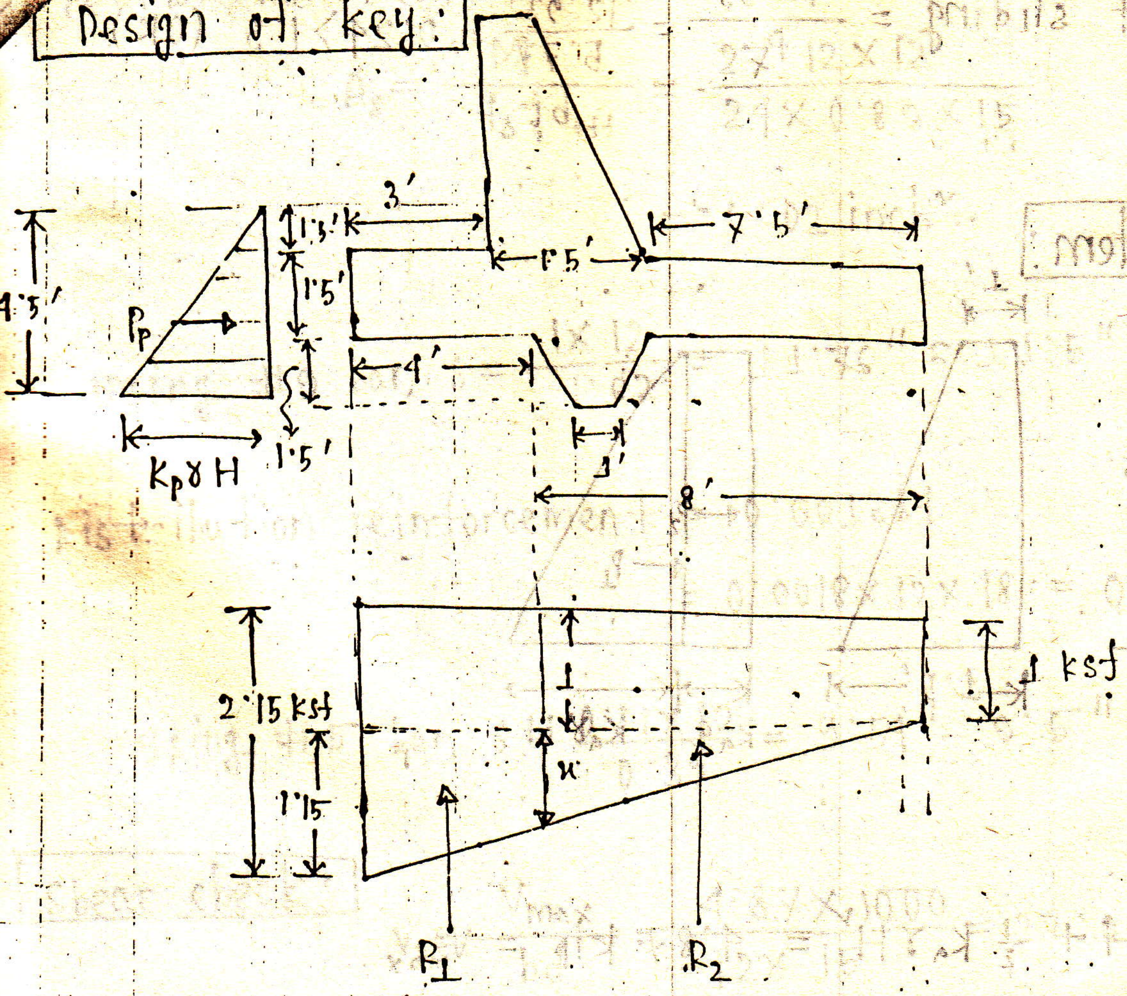
$$I = \frac{1 \times 12^3}{12} = 144 \text{ ft}^4$$

$$A = 1 \times 12 = 12 \text{ ft}^2$$

$$P_1 = \frac{18.82}{12} + \frac{13.89 \times 6}{144} = 1.57 + 0.58 = 2.15 \text{ ksf}$$

$$P_2 = 1.57 - 0.58 = 1 \text{ ksf}$$

Design of key:



$$\frac{x}{8} = \frac{1.15}{12}$$

$$\therefore x = 0.77$$

$$R_1 = \frac{1}{2} \times (2.15 + 1.77) \times 4 = 7.84 \text{ klf}$$

$$R_2 = \frac{1}{2} \times (1 + 1.77) \times 8 = 11.08 \text{ klf}$$

$$P_p = \frac{1}{2} K_p \gamma H^2 = \frac{1}{2K_a} \gamma H^2 = \frac{1}{2 \times 0.31} \times 0.11 \times 4.5^2$$

$$= 3.60 \text{ klf}$$

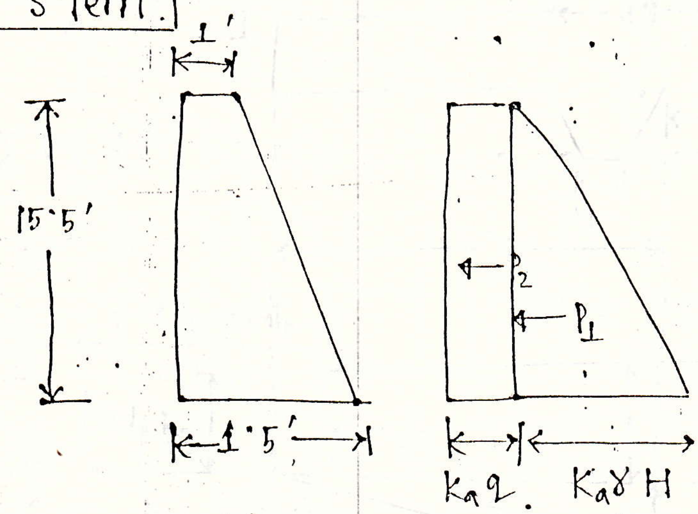
$$\text{Sliding resistance} = (R_1 \times \tan\varphi + R_2 \times f) + P_p$$

$$= (7.84 \times \tan 32 + 11.08 \times 0.4) + 3.60$$

$$= 12.93 \text{ klf}$$

$$\text{F.S against sliding} = \frac{12.93}{P} = \frac{12.93}{5.77} = 2.24 > 1.5 \text{ (OK)}$$

Design of stem:



$$P = P_1 + P_2$$

$$= K_a q H + \frac{1}{2} K_a \gamma H^2 = 4.87 \text{ kip} = V_{max}$$

$$M = K_a q H \times \frac{H}{2} + \frac{1}{2} K_a \gamma H^2 \times \frac{H}{3} = 27.12 \text{ k-ft}$$

Depth check, $d = \sqrt{\frac{M}{Rb}}$

$$\Rightarrow d = \sqrt{\frac{27.12 \times 12}{0.205 \times 12}}$$

$$\therefore d = 11.50''$$

$d_{eff} = 1.5 \times 12 - 3 = 15'' > d$.

Check is ok.

$R = \frac{1}{2} f_c j k$.

$n = \frac{E_s}{E_c} = 9$.

$r = \frac{f_s}{f_c} = 17.78$.

$K = 0.34$.

$J = 0.89$.

$R = 204.255 \text{ psi} = 205$.

Reinforcement calculation.

$$A_s = \frac{M}{f_s j d_{eff}} = \frac{27.12 \times 12}{24 \times 0.89 \times 15}$$

$$= 1.02 \text{ inch}^2.$$

Using #9 bar, $s = \frac{1 \times 12}{1.02} = 11.76'' \approx 11.5''$ c-c.

Distribution reinforcement $= 0.0018bt$

$= 0.0018 \times 12 \times 18 = 0.39 \text{ inch}^2$

Using #5 bar, $s = \frac{0.31 \times 12}{0.39} = 9.54'' \approx 9.5''$ c-c.

Shear check:

$$V_d = \frac{V_{max}}{bd} = \frac{4.87 \times 1000}{12 \times 15} = 27.06 \text{ psi}.$$

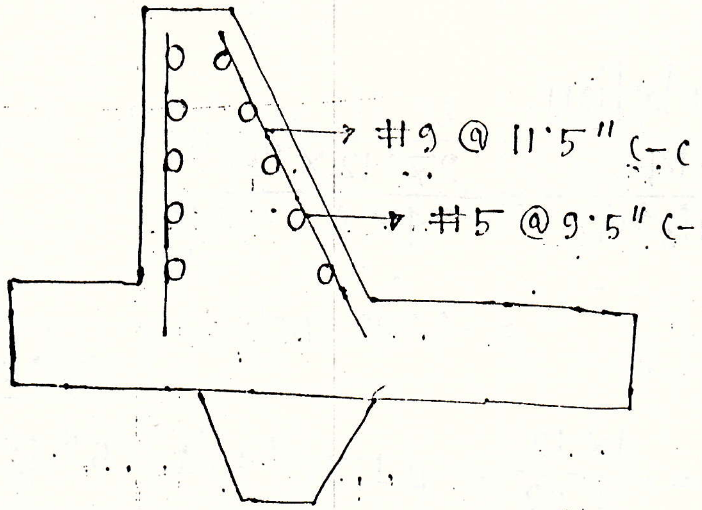
$$V_c = 1.1\sqrt{f_c'} = 60.25 \text{ psi}.$$

$V_c > V_d$ (OK).

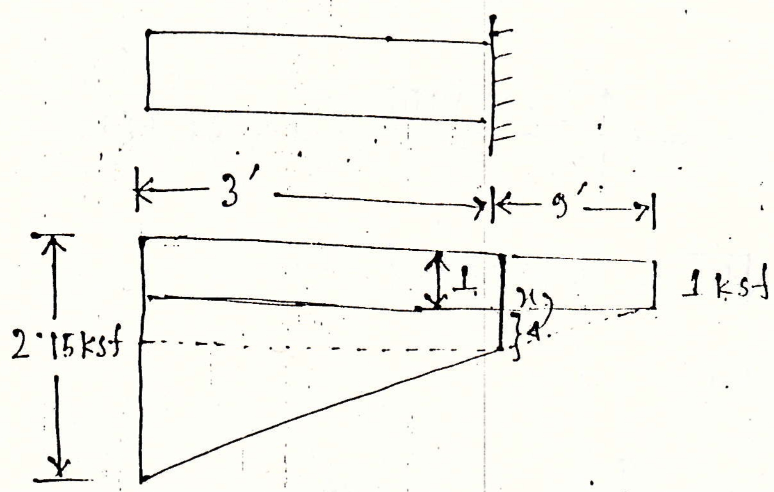
Development length: $L_d = \frac{\varphi f_s}{4 U_d}$ —— (1).

$$U_d = \frac{V_{max}}{\Sigma_0 j d} = \frac{4.87 \times 1000}{\pi \times \frac{9}{8} \times \frac{12}{9.5} \times 0.89 \times 15} = 81.17 \text{ psi}.$$

$$L_d = \frac{\frac{9}{8} \times 24{,}000}{4 \times 81.17} = 83.16''$$



Toe Design: For safety purpose, weight of soil and self weight of toe is neglected.



$$\frac{x}{9} = \frac{1 \cdot 15}{12}$$

$\therefore x = 0 \cdot 86$

$P = \frac{1}{2}(2 \cdot 15 + 1 \cdot 86) \times 3 = 6 \cdot 02 \text{ kip} = V_{max}$

$M = 1 \cdot 86 \times 3 \times \frac{3}{2} + \frac{1}{2} \times 3 \times (2 \cdot 15 - 1 \cdot 86) \times 2 \times \frac{3}{3}$

$= 8 \cdot 37 + 0 \cdot 87 = 9 \cdot 24 \text{ k-ft}$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{9.24 \times 12}{0.205 \times 12}} = 6.71''$$

$$d_{eff} = 18 - 3 = 15'' > d \text{ (ok)}.$$

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d} = \frac{9.24 \times 12}{24 \times 0.89 \times 15} = 0.35 \text{ inch}^2.$$

$$\#6, \quad S = \frac{0.44 \times 12}{0.35} = 15.08 \simeq 15'' \text{ c-c}.$$

Distribution reinforcement $= 0.0018bt$.

$$= 0.39 \text{ inch}^2.$$

$$\#5 \text{ bar}, S = \frac{0.31 \times 12}{0.39} = 9.54'' \simeq 9.5''.$$

Shear check:

$$V_d = \frac{6.02 \times 1000}{12 \times 15} = 33.44 \text{ psi}.$$

$$V_c = 1.1\sqrt{f_c'} = 60.25 \text{ psi}.$$

$$V_c > V_d \text{ (OK)}$$

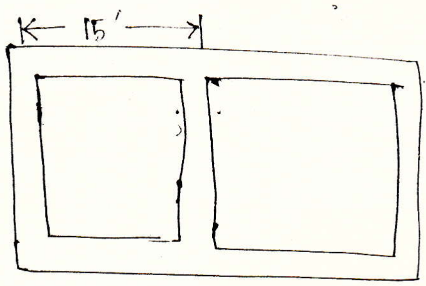
Development length:

Working diagram:

USD | SLAB

RON 090001

☐ A reinforced concrete slab is built integrally with its support consists of two equal span of 15 ft. Service live load is 100 psf. $f_c' = 4000$ psi, $f_y = 60{,}000$ psi. Design the slab.



Solve Load calculation:

By ACI CODE, thickness, $t = \frac{L}{28} = \frac{15 \times 12}{28} = 6.43$ inch.

thickness is taken as $= 6.5$ inch.

Total load $= D.L \times 1.2 + L.L \times 1.6$.

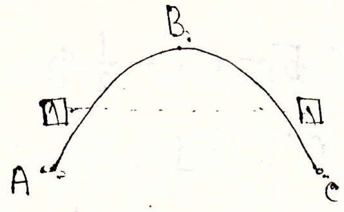
$= \frac{6.5}{12} \times 150 \times 1.2 + 100 \times 1.6$.

$= 97.5 + 160$.

$= 257.5$ psf.

$= 0.258$ kip ft^{-2}.

Factored moment



$W = 0.258 \text{ kip ft}^{-2}$

$L = 15 \text{ ft}$.

At exterior support, $M_A = \frac{WL^2}{24} = 2.41$ kip ft.

At midspan, $M_B = \frac{WL^2}{14} = 4.13$ kip ft.

At interior support, $M_C = \frac{WL^2}{9} = 6.43$ kip ft.

So, ultimate moment, $M_u = 6.43$ kip ft.

Depth check

$M_u = \phi R b d^2$

$\Rightarrow d = \sqrt{\frac{M_u}{\phi R b}}$ ——— (i)

Where, $R = \rho_{max} f_y \left(1 - 0.59 \frac{f_y}{f_c'} \cdot \rho_{max}\right)$ ——— (ii)

$\rho_{max} = 0.85 \beta_1 \cdot \frac{f_c'}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}$

$= 0.85 \times 0.85 \times \frac{4}{60} \times \frac{0.003}{0.003 + 0.004}$

$= 0.021$.

$R = 0.021 \times 60 \times \left[1 - 0.59 \times \frac{60}{4} \times 0.021\right]$

$\therefore R = 1.025823$ ksi.

$\therefore d = \sqrt{\frac{6.43 \times 12}{0.90 \times 1.025823 \times 12}} = 2.64$ inch.

$d_{actual} = 6.5 - c.c - \frac{1}{2}\phi$

$= 6.5 - 1$

$= 5.5'' > d$

Reinforcement calculation

$$a = \frac{A_s f_y}{0.85 f'_c . b} = \frac{A_s \times 60}{0.85 \times 4 \times 12} = 1.47 A_s$$

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

$$\Rightarrow A_s = \frac{6.43 \times 12}{0.9 \times 60 \times (5.5 - 0.74 A_s)}$$

$$\Rightarrow 54 A_s (5.5 - 0.74 A_s) = 77.16$$

$$\Rightarrow 297 A_s - 39.96 A_s^2 = 77.16$$

$$\Rightarrow 39.96 A_s^2 - 297 A_s + 77.16 = 0$$

$$\therefore A_s = 0.27 \text{ inch}^2$$

$a = 1.47 A_s = 0.4$ inch

At, interior support, $A_s = 0.27$ inch^2

At mid span, $A_s = \frac{M}{\phi f_y (d - \frac{a}{2})} = \frac{4.13 \times 12}{0.9 \times 60 \times (5.5 - \frac{0.4}{2})}$

$= 0.17$ inch^2

At exterior support, $A_s = \frac{2.41 \times 12}{0.9 \times 60 \times (5.5 - \frac{0.4}{2})} = 0.10$ inch^2

$(A_s)_{min} = 0.0018\,bt = 0.0018 \times 12 \times 6.5 = 0.14$ inch^2.

so, reinforcement has to be revised at exterior support:

Extra bar at exterior $= 0.14 - 0.10 = 0.04 \approx 1\#3$.

Shear check:

$V_{ud} = 1.15 \times \frac{WL}{2} - \frac{Wd}{12}$ | $W = 257$.

$= 2100$ lb. | $L = 15$. $d = 5.5$.

Design strength of concrete slab

$= 2\phi\sqrt{f_c'}\, b_w d$.

$= 6260$ lb. $[\phi = 0.75]$

Shear check is ok, no shear reinforcement is required

Working diagram: রুমন ২৫৭.

Spacing at interior (#3) $= \frac{0.11 \times 12}{0.27} = 4.88$ inch c-c.

" " midspan (#3) $= \frac{0.11 \times 12}{0.17} = 7.76''$ c-c

" " exterior (#3) $= \frac{0.11 \times 12}{0.10} = 13.2''$ c-c.

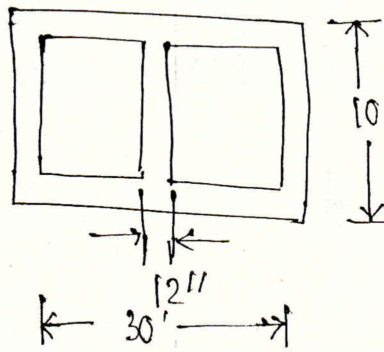
Bond check:

$U_d = \frac{V_d}{\Sigma_o (d - \frac{a}{2})}$. | $V_d = 1.15 \frac{WL}{2}$

$U_{all} = \frac{6.7\sqrt{f_c'}}{D}$. | $\Sigma_o = \frac{\pi\phi 12}{\text{spacing}}$

WSD

1. Design a isolated slab 10′×30′ supported on brickw... on four sides. The slab should be design to carry a minimum live load of 40 psf. Design the slab using $f_c' = 3000$ psi and $f_s = 24000$ psi.



Solve For simple supported beam:

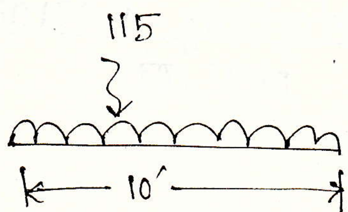
thickness, $t = \frac{10 \times 12}{20} = 6$ inch. (shortest distance)

[1] Load calculation:

Dead load $= \frac{t}{12} \times 150 = 75$ psf.

Live load $= 40$ psf.

Total load $= 115$ psf.



$$M_{max} = \frac{WL^2}{8} = \frac{115 \times 10^2}{8} = 1437.5 \text{ lb ft}.$$

$$V_{max} = \frac{WL}{2} = 575 \text{ lb}.$$

Depth check, $d = \sqrt{\frac{M}{Rb}}$.

$= \sqrt{\frac{1437.5 \times 12}{205 \times 12}}$

$= 2.65$ inch.

$R = \frac{1}{2} f_c j k$.

$n = 9.28 \approx 9$.

$r = 17.78$.

$k = 0.34$.

$j = 0.89$.

$f_c = 1350$.

$R = 205$

$b = 12$

$d_{actual} = 6 - c.c - \frac{1}{2}\phi$.

$= 5$ inch $> d$

d_{actual} is to be provided

[c] Reinforcement calculation:

$A_s = \frac{M}{f_s j d_{ac}} = \frac{1437.5 \times 12}{24000 \times 0.89 \times 5} = 0.16 \text{ inch}^2/\text{ft}$.

Using #3 bars. spacing $= \frac{0.11 \times 12}{0.16} = 8.25''$ c-c.

Max. spacing. $= 3t = 18$ inch.

$(A_s)_{min} = 0.0018\,bt = 0.0018 \times 12 \times 6 = 0.13 \text{ inch}^2$.

[d] Distribution reinforcement:

$A_s = 0.0018\,bt = 0.13 \text{ inch}^2$ per feet.

Using no. 3 bar, spacing $= \frac{0.11 \times 12}{0.13} = 10.15$ inch.

Max. spacing $= 5t = 30$ inch.

Use no. 3 bar at $10''$ spacing c-c.

Shear check:

$V_{allowable} = 2\phi\sqrt{f_c'}\, b_w d$.

$= 2 \times 0.75 \times \sqrt{3000} \times 12 \times 5$

$= 4929.50$ psi.

Shear at a distance d, $V_{ud} = 1.15 \times \frac{wl}{2} - \frac{wd}{12}$

$= 1.15 \times 575 - \frac{573 \times 5}{12}$

$= 661.25 - 238.75$.

$= 422.5$ psi

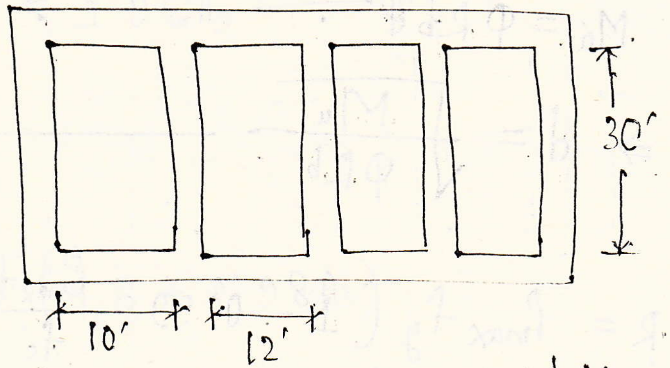
So shear check is ok.

Working diagram:

2008

Q4

$f_c' = 3$ ksi.
$f_y = 50$ ksi.
L.L = 60 psf.



Beam width 10"

Solve:

Support condition = Both end continuous.

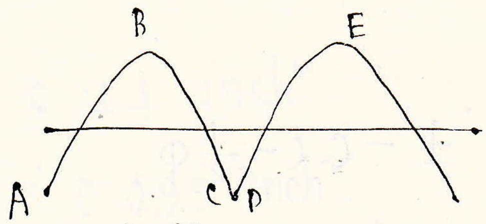
$$t = \frac{L}{28} = \frac{12 \times 12}{28} = 5.14 \approx 5.5''$$

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

$$= 1.2 \times \frac{5.5}{12} \times 150 + 1.6 \times 60.$$

$$= 82.5 + 96 = 178.5 \text{ psf} = 0.18 \text{ ksf}$$

Moment diagram:



L = 10.

$$M_A = \frac{WL^2}{24} = 0.75 \text{ kft}$$

$$M_B = \frac{WL^2}{14} = 1.29 \text{ kft}$$

$$M_C = \frac{WL^2}{10} = 1.8 \text{ kft}$$

L = 12;

$$M_D = \frac{WL^2}{11} = 2.36 \text{ kft}$$

$$M_E = \frac{WL^2}{16} = 1.62 \text{ kft}$$

Ultimate moment, $M_u = 2.36$ k-ft.

$$M_u = \phi R b d^2$$

$$\Rightarrow d = \sqrt{\frac{M_u}{\phi R b}} \text{ ——— (i)}$$

$$R = \rho_{max} f_y \left(1 - 0.59 \frac{\rho_{max} f_y}{f_c'}\right) \text{ ——— (ii)}$$

$$\rho_{max} = 0.85 \beta_1 \frac{f_c'}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}$$

$$= 0.85^2 \frac{3}{50} \cdot \frac{0.003}{0.003 + 0.004} = 0.0186$$

$$R = 0.7599.$$

$$d = \sqrt{\frac{2.36 \times 12}{0.90 \times 0.7599 \times 10}} \quad [b = 10]$$

$$\therefore d = 2.03''$$

$$d_{actual} = t - c.c - \frac{1}{2}\phi$$

$$= 5.5 - 1$$

$$= 4.5'' > 2.03'' \quad \text{(OK)}$$

so, d_{actual} is to be provided.

Reinforcement calculation:

$$a = \frac{A_s f_y}{0.85 f_c' b} = 1.96 A_s \quad \text{——} ①$$

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

$$\Rightarrow 2.36 \times 12 = 0.9 \times A_s \times 50 \left(4.5 - 0.98 A_s\right)$$

$$\Rightarrow 28.32 = 202.5 A_s - 44.1 A_s^2$$

$$\Rightarrow 44.1 A_s^2 - 202.5 A_s + 28.32 = 0$$

$A_s = 0.14 \text{ inch}^2$ for D point.

$a = 0.27 \text{ inch}^2$.

For A point, $A_s = \dfrac{M_a}{\phi f_y \left(d - \frac{a}{2}\right)} = 0.045 \text{ inch}^2$

For B point, $A_s = 0.079 \text{ inch}^2$.

For C point, $A_s = 0.11 \text{ inch}^2$.

For E point, $A_s = 0.099 \text{ inch}^2$.

$(A_s)_{min} = 0.0020 bt = 0.11 \text{ inch}^2$

So, $(A_s)_{min}$ is to be provided at A, B & E point.

Using #3 bars, spacing,

at A, $s = \frac{0.11 \times 12}{0.11} = 12''$

at B, $s = \frac{0.11 \times 12}{0.11} = 12''$

at C, $s = \frac{0.11 \times 12}{0.11} = 12''$

at D, $s = \frac{0.11 \times 12}{0.14} = 9.43 \approx 9.5''$

at E, $s = 12''$

Maximum spacing = 3t = 16.5 inch.

Max. spacing for distrution reinforcement = 5t = 27.5 inch.

Check Shear check:

$V_{ud} = 1.15 \frac{WL}{2} - \frac{Wd}{12}$ | W = 178.5 psf. L = 12 d = 4.5.

$= 1164.71$ lb

$V_{all} = 2\phi\sqrt{f_c'}\, b_w d.$

$= 2 \times 0.75\sqrt{3000} \times 10 \times 4.5$

$= 3697.13$ lb.

so, no shear reinforcement is required.

Bond check:

$$V_d = 1.15 \times \frac{WL}{2} = 1231.65$$

$$U_d = \frac{V_d}{\Sigma_0 (d - \frac{a}{2})} = \frac{1231.65}{1.18(4.5 - \frac{0.27}{2})}$$

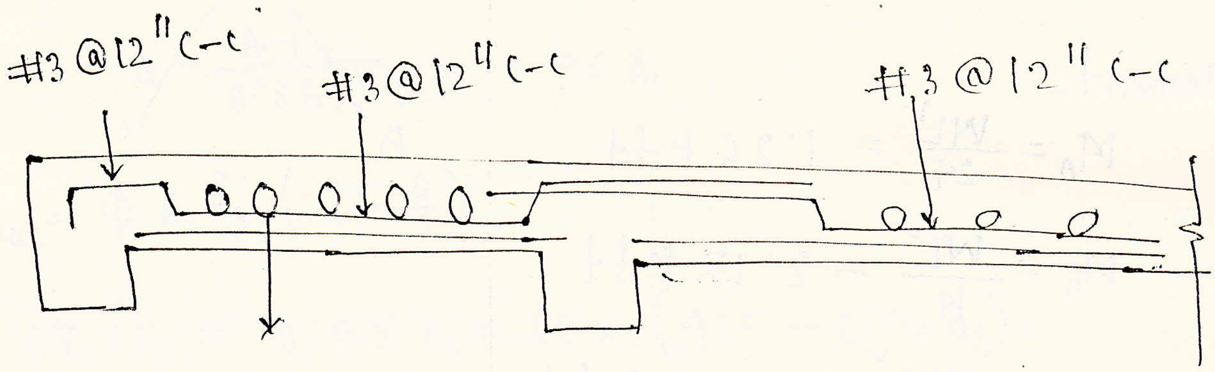
$$\Sigma_0 = \frac{\pi \times (\frac{3}{8}) \times 12}{12} = 1.18.$$

$$\therefore U_d = 239.122 \text{ psi}$$

$$U_{all} = \frac{6.7\sqrt{f_c'}}{D} = 978.59 \text{ psi}.$$

so, bond check is okh.

Working diagram:



2007

2 b Given,

clear span = 12'

Live load = 80 psf.

$f_c' = 3000$ psi.

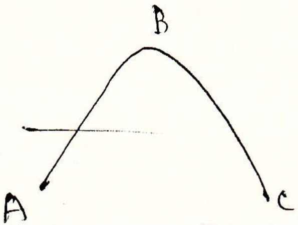
$f_y = 60{,}000$ psi.

Thickness, $t = \frac{12 \times 12}{28} = 5.5''$

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

$= 1.2 \times \frac{5.5}{12} \times 150 + 1.6 \times 80$

$= 210.5$ psf.



Moment:

$M_A = \frac{WL^2}{24} = 1.26$ kft.

$M_B = \frac{WL^2}{14} = 2.17$ kft

$M_C = \frac{WL^2}{9} = 3.37$ kft.

$M_u = 3.37$ kft.

$M_u = \phi R b d^2$

$\Rightarrow d = \sqrt{\frac{M_u}{\phi R b}}$ ——— (1).

$$R = \rho_{max} f_y \left(1 - 0.59 \frac{\rho_{max} f_y}{f_c'}\right) \quad \text{——— (1)}$$

$$\rho_{max} = 0.85 \beta_1 \frac{f_c'}{f_y} \cdot \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} = 0.0155$$

$R = 0.7599$ ksi

$$d = \sqrt{\frac{3.37 \times 12}{0.90 \times 0.7599 \times 12}} = 2.22''$$

$d_{actual} = t - c.c - \frac{1}{2}\phi = 4.5'' > d$

d_{actual} is to be provided.

Reinforcement calculation:

$$a = \frac{A_s f_y}{0.85 f_c' b} = 1.96 A_s$$

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

$\Rightarrow 3.37 \times 12 = 0.9 \times A_s \times 60 \times (4.5 - 0.98 A_s)$

$\Rightarrow 52.92 A_s^2 - 243 A_s + 40.44 = 0$

$\therefore A_s = 0.17$ inch^2 (provided at c point).

$\therefore a = 0.333$ inch.

At A point, $A_s = \frac{M_A}{\phi f_y (d - \frac{a}{2})} = 0.065 \text{ inch}^2$

At B point, $A_s = 0.11 \text{ inch}^2$.

$(A_s)_{min} = 0.0018\, bt = 0.12 \text{ inch}^2$.

So, $(A_s)_{min}$ is to be provided at A & B point.

Spacing for #3 bar.

At A, $S = \frac{0.11 \times 12}{0.12} = 11''$

At, B, $S = 11''$.

At, C, $S = \frac{0.11 \times 12}{0.17} = 7.76'' \approx 8''$

Max. spacing, $= 3t = 16.5$ inch.

Max. spacing for distribution reinforcement

$= 5t = 27.5''$.

Shear check:

$$V_{ad} = 1.15\frac{WL}{2} - \frac{Wd}{12}$$

$= 1373.5$ lb.

$W = 210.5$ psf.
$L = 12'$
$d = 4.5''$

$V_{allowable} = 2\varphi\sqrt{f_c'}\,bd = 4436.55$ lb.

so, no shear reinforcement is required.

Bond check:

$$U_d = \frac{V_d}{\Sigma_o\left(d - \frac{a}{2}\right)}$$

$\therefore U_d = 259.82$ psi.

$V_d = 1452.45$

$$\Sigma_o = \frac{\pi \times \left(\frac{3}{8}\right) \times 12}{11}$$

$= 1.29$

$a = 0.833$

$$U_{all} = \frac{6.7\sqrt{f_c'}}{D} = 978.59 \text{ psi (OK)}.$$

Working diagram: