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A PROPOSED RETAINING WALL
TO PROVIDE STORAGE FOR
FIVE THOUSAND TONS OF COAL

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
W. D. Strait H. H. Post
1948

A Proposed Retaining Wall
To Provide Storage
For Five Thousand Tons of Coal

A Thesis Submitted to

The Faculty of
MICHIGAN STATE COLLEGE
of
AGRICULTURE AND APPLIED SCIENCE

by

W. D. Strait

H. H. Post

Candidates for the Degree of

Bachelor of Science

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We wish to express our gratitude to
W. A. Bradley for his cooperation and guidance.

H. H. Post

W. D. Strait

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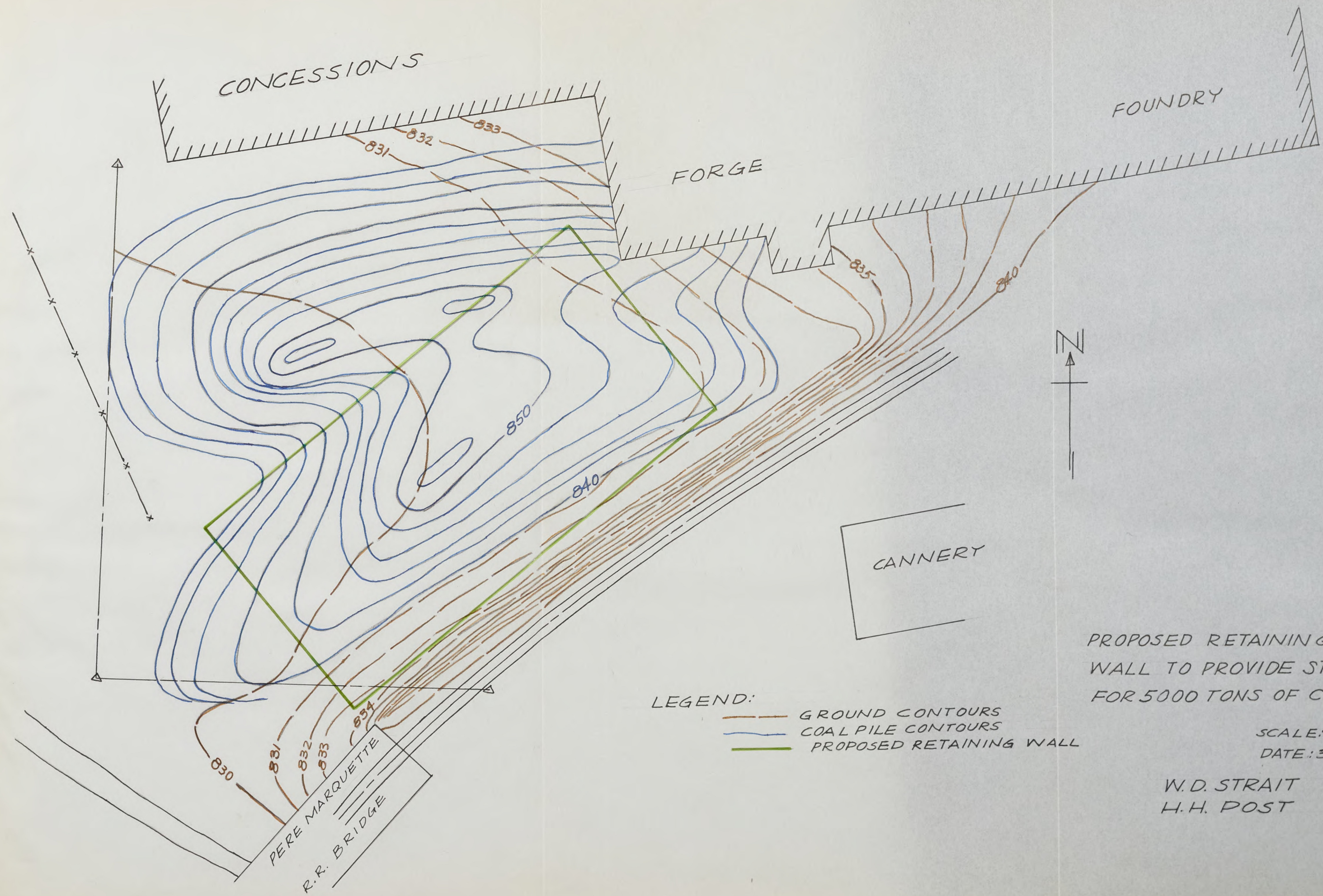
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INTRODUCTION

Mr. Rinney of the Building and Grounds Department expressed a desire for a storage bin for a reserve stock of coal. A reserve stock is now in existence between the Forge Building and the Red Cedar River. This stock pile is unsatisfactory because it is unsightly, the quantity of coal cannot be estimated accurately, and removal of coal by clamshell is difficult.

Five thousand tons of coal is the quantity desired in the proposed storage bin. A bin incorporating the design of retaining walls would alleviate the present difficulties.

The height of wall required for such a bin is about twenty feet. This height brings up the question of whether a cantilever or counterfort wall should be used. Designs are shown for both cases with cost estimates included.



LEGEND:

- GROUND CONTOURS
- COAL PILE CONTOURS
- PROPOSED RETAINING WALL

PROPOSED RETAINING
WALL TO PROVIDE STORAGE
FOR 5000 TONS OF COAL.

SCALE: 1"=20'
DATE: 3-29-48

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COMPUTATIONS FOR SIZE OF BIN

Amount of coal to be stored = 5000 Tons.

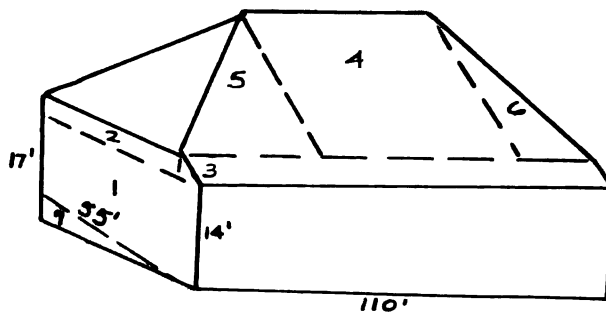
$5000 \text{ T} \times 2000 \text{ lb/T} = 10,000,000 \text{ lb. of coal}$

Coal weighs 70 lb./ft^3

Cu. ft. req'd = $\frac{10,000,000}{70} = 143,000 \text{ ft}^3$

Size of bin = $55' \times 110' = 6050 \text{ ft}^2$

Average height of bin = $\frac{143,000}{6050} = 23.6' \text{ (if piled flat)}$



Volumes:

① Bottom $14 \times 55 \times 110 = 91,600 \text{ ft}^3$

② $2 \times 53 \times 110 = 11,600 \text{ ft}^3$

③ $2 \times 110 = 220 \text{ ft}^3$

④ $53 \times 57 \times 13.25 = 40,000 \text{ ft}^3$

⑤ + ⑥ $53^2 \times \frac{26.5}{3} = 23,800 \text{ ft}^3$

⑦ $\frac{1}{2} \times 55 \times 110 = 3,030 \text{ ft}^3$

$167,250 + 3000 = 170,250$

$\frac{170,250}{2000} \times 70 = 5960 \text{ Tons}$

$\frac{170,250}{2000} \times 60 = 5110 \text{ Tons}$

$\frac{170,250}{2000} \times 50 = 4260 \text{ Tons}$

Allowing for variation in wt. of coal
from 70 lb/ft^3 to 50 lb/ft^3

This bin seems to be of adequate size

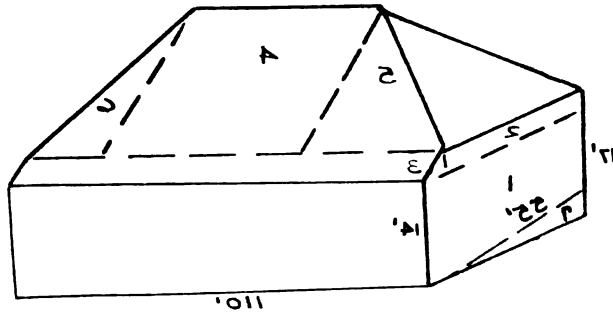
COMPUTATIONS FOR SIZE OF BIN

Amount of coal to be stored = 2000 Tons.
 $2000 \times 2000 \text{ lb/T} = 10,000,000 \text{ lb of coal}$
 Coal weighs 70 lb/ft³

$$\text{Cu. ft. req'd} = \frac{10,000,000}{70} = 143,000 \text{ ft}^3$$

$$\text{Size of bin} = 22' \times 110' = 2420 \text{ ft}^2$$

$$\text{Average height of bin} = \frac{143,000}{2420} = 59' \text{ (if piled flat)}$$



Volumes:

$$\text{① Bottom } 14 \times 22 \times 110 = 33,880 \text{ ft}^3$$

$$\text{② } 22 \times 22 \times 110 = 53,800 \text{ ft}^3$$

$$\text{③ } 22 \times 110 = 2,420 \text{ ft}^2$$

$$\text{④ } 22 \times 22 \times 13.52 = 66,000 \text{ ft}^3$$

$$\text{⑤ } 22 \times 22 \times 2 = 968 \text{ ft}^3$$

$$\text{⑥ } 11 \times 22 \times 110 = 26,620 \text{ ft}^3$$

$$167,520 + 2000 = 170,520$$

$$\frac{170,520}{2000} \times 20 = 1705 \text{ Tons}$$

$$\frac{170,520}{2000} \times 60 = 5115 \text{ Tons}$$

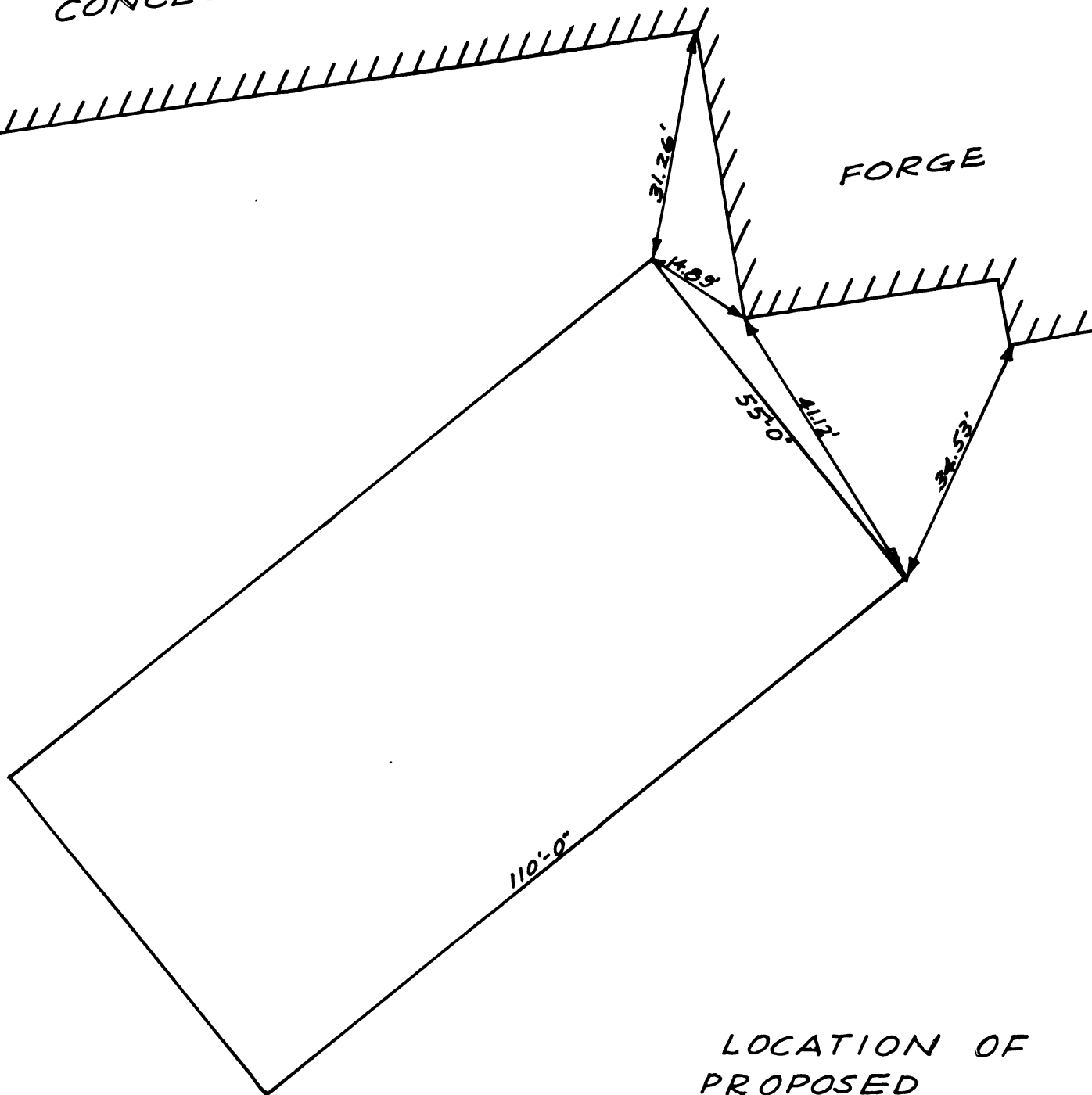
$$\frac{170,520}{2000} \times 20 = 1705 \text{ Tons}$$

Allowing for variation in wt. of coal
 from 70 lb/ft³ to 20 lb/ft³
 This bin seems to be of adequate size

CONCESSIONS



FORGE



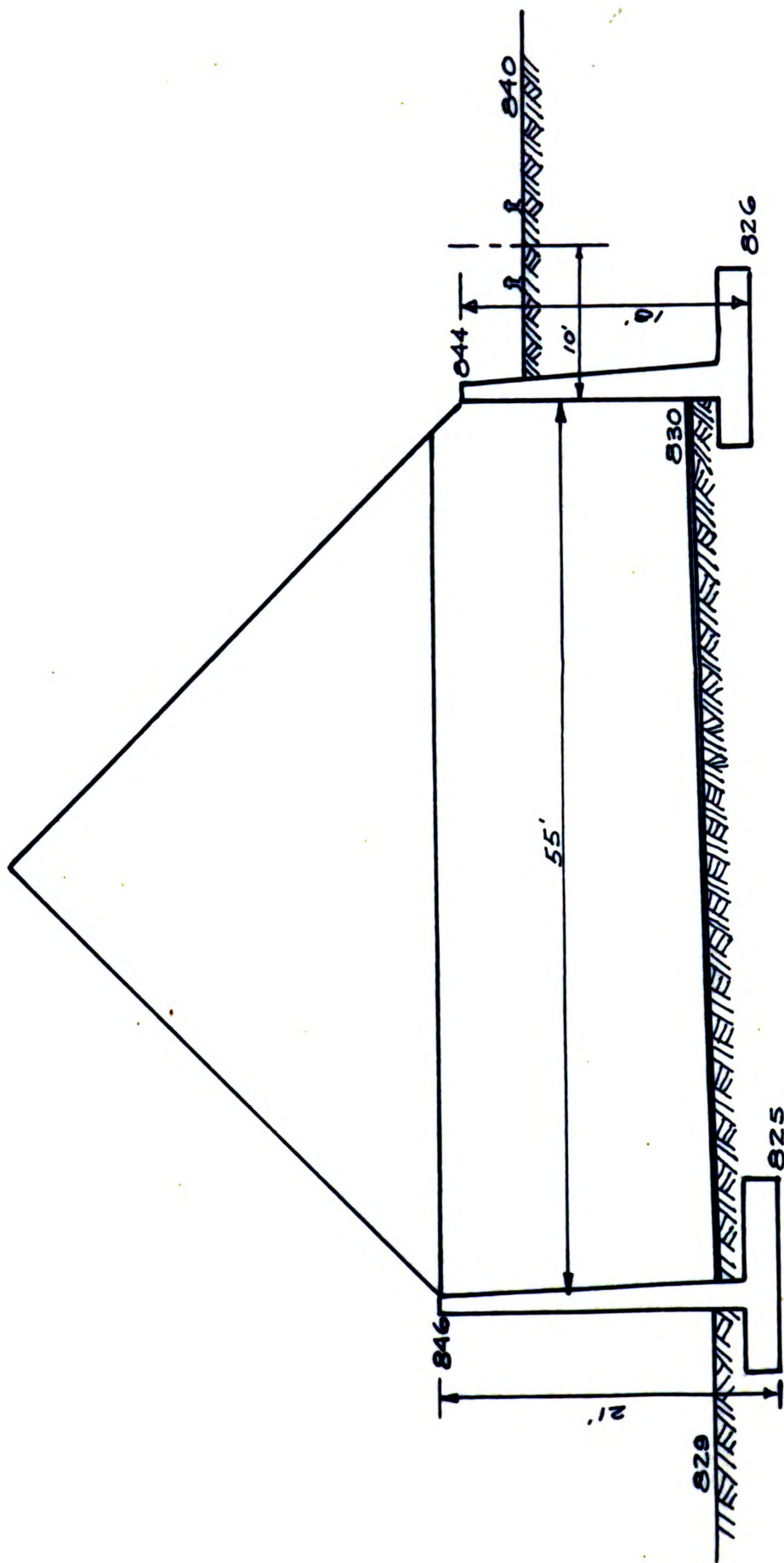
LOCATION OF
PROPOSED
RETAINING WALL

SCALE: 1"=20'

DATE: 4-7-48

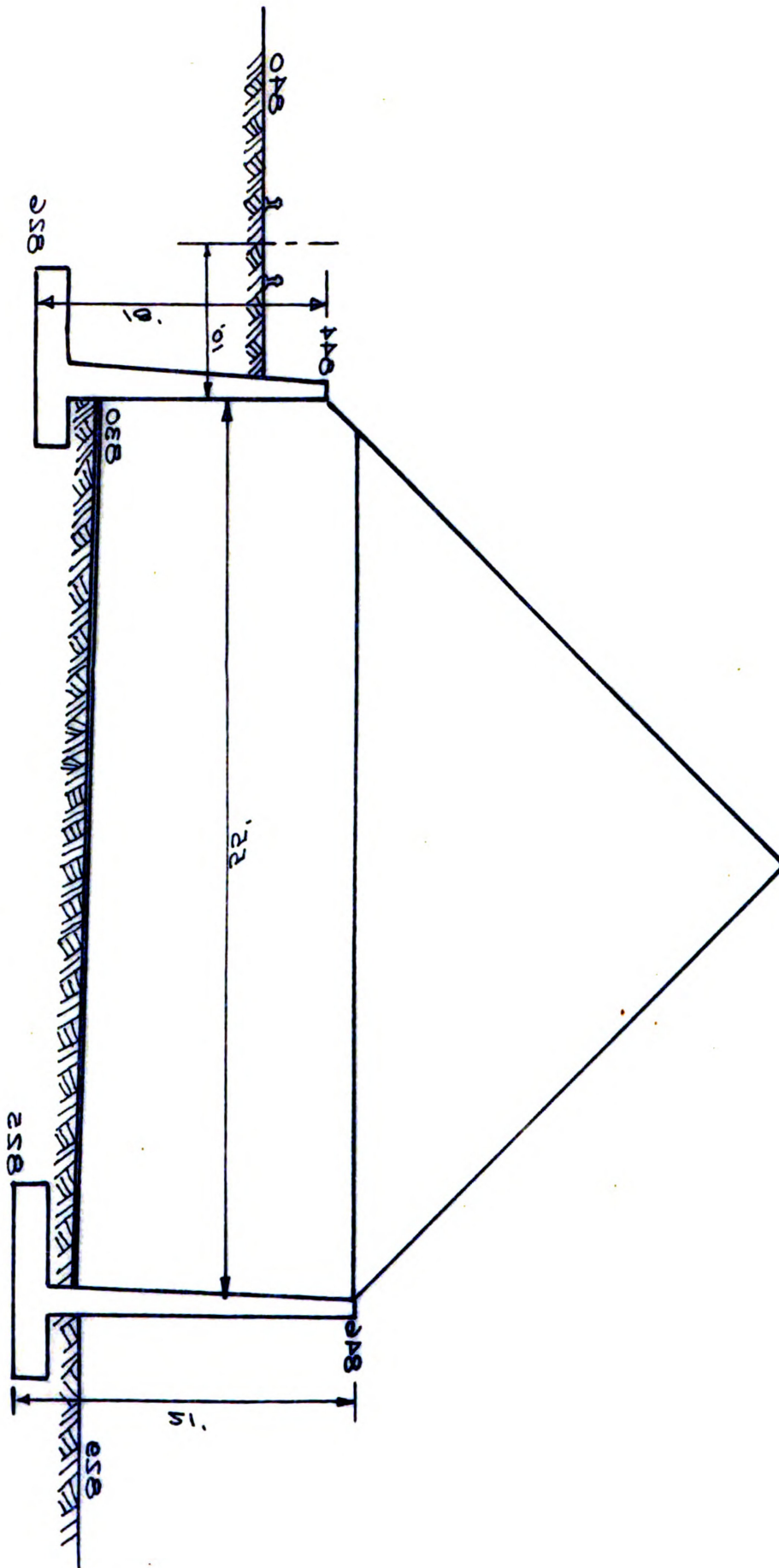
W.D. STRAIT

H. H. POST



Cross Section Looking North
Scale: 1" = 10'

Scale: 1" = 10'
 Cross Section Looking North



ASSUMPTIONS

Angle of Repose for coal = 45° *

Angle of Friction, coal on concrete = 30° *

Wt of coal for design purposes = 70 lb/ft^3

Wt of earth fill = 100 lb/ft^3

Angle of repose, earth, $\phi = 34^\circ$

Angle of friction, earth on concrete = $\delta = 25^\circ$

Concrete weighs 150 lb/ft^3

28 day strength of concrete = $2000 \text{ psi} = f'_c$

Design strength of concrete = $800 \text{ psi} = f_c$

Concrete Shear = $v_c = 40 \text{ psi}$ (60 psi with Special

Concrete Bond = $u = 100 \text{ psi}$ Anchorage)

Steel Strength = $f_s = 18,000 \text{ psi}$

Modular Ratio $\frac{E_s}{E_c} = n = 15$

$$R = 140 = \frac{M}{bd^2}$$

Steel ratio $p = .009$

Design constants: $j = 7/8$
 $k = 3/8$

* Taken from "Walls, Bins, and Grain Elevators" by Milo S. Ketchum.

ASSUMPTIONS

- * Design constants:
 - Steel ratio $\rho = .009$
 - $R = 140 = \frac{M}{\rho d^2}$
 - Modular Ratio $\frac{E_s}{E_c} = n = 12$
 - Steel strength $f_s = 18,000 \text{ psi}$
 - Concrete Bond $\mu = 100 \text{ psi}$
 - Concrete shear $\mu = 40 \text{ psi}$ (60 psi with special Anchorage)
 - Design strength of concrete $= 800 \text{ psi} = f_c$
 - 28 day strength of concrete $= 2000 \text{ psi} = f_c'$
 - Concrete weight 150 lb/ft^3
 - Angle of friction, earth on concrete $= \phi = 25^\circ$
 - Angle of repose, earth, $\phi = 34^\circ$
 - Wt of earth fill $= 100 \text{ lb/ft}^3$
 - Wt of coal for design purposes $= 70 \text{ lb/ft}^3$
 - Angle of Friction, coal on concrete $= 30^\circ$ *
 - Angle of Repose for coal $= 42^\circ$ *

* Taken from "Walls, Bins, and Grain Elevators" by Milo S. Ketchum.

CASE I:

Level Surcharge, Inert Prism, Omit Back Wall Friction.

E-72 Loading, allow 15% for impact.

$$\text{Equivalent Surcharge} = \frac{72,000}{5 \times 18} \times 1.15 = 920 \text{ psf or } 9.2'$$

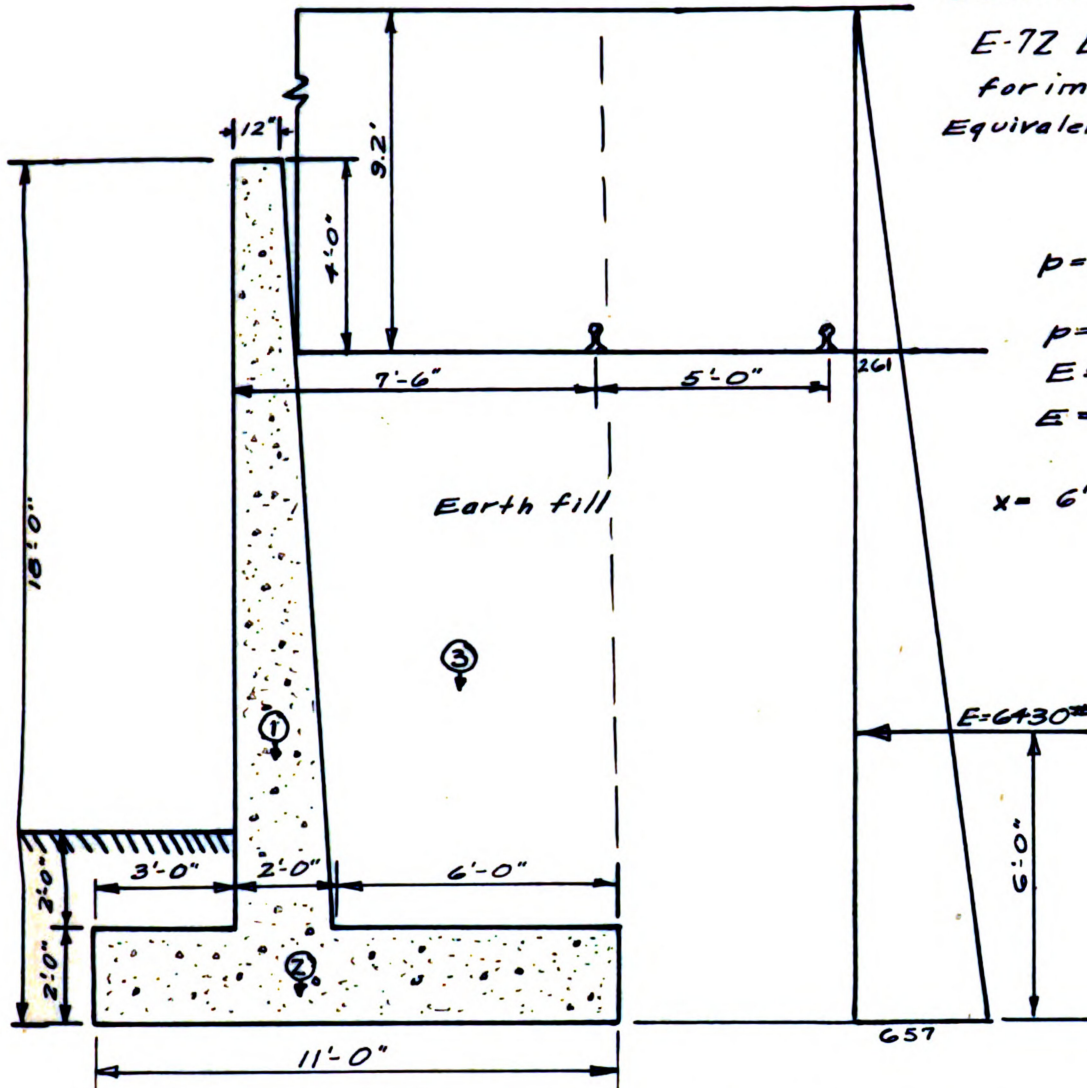
$$p = wh \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)$$

$$p = 28.3 \text{ h}$$

$$E = \frac{657 + 261}{2} \times 14$$

$$E = 6430 \#$$

$$x = 6'$$



Base Pressures: Moments about Toe

$$W_1 = 3300 \text{ lbs.} \quad 5.5' \quad = \quad 18,200 \text{ lb. ft.}$$

$$W_2 = 3600 \quad 3.75' \quad = \quad 13,500$$

$$W_3 = \frac{13900}{20,800} \quad 8.0' \quad = \quad \frac{111,200}{20,800}$$

$$M_R = 142,900$$

$$6430 \times 6 = M_0 = 38,580$$

$$20,800 \quad 104,320$$

$$5.02' \text{ from toe}$$

$$e = 5.5 - 5.02$$

$$e = .48'$$

Toe & Heel Pressures:

$$p = \frac{20,800}{11} \left(1 \pm \frac{6 \times .48}{11} \right)$$

$$p = 1890 \pm 495$$

$$p_t = 2385 \text{ psf}$$

$$p_h = 1395 \text{ psf}$$

Overturning:

$$\frac{M_R}{M_0} = \frac{142,900}{38,580} = 3.7$$

Sliding:

$$\frac{20,800 \times .466}{6430} = 1.51$$

Must Anchor

Railroad Cantilever Wall

Must Anchor

Sliding: $\frac{20,800 \times 4.66}{6430} = 1.21$

Overturning: $\frac{M_o}{M_r} = \frac{38,280}{145,900} = 0.27$

$e = 4.8'$
 $e = 2.2 - 2.02$
 $2.02'$ from toe

$20,800 / 104,350$
 $6430 \times e = M_o = 38,280$
 $M_r = 145,900$

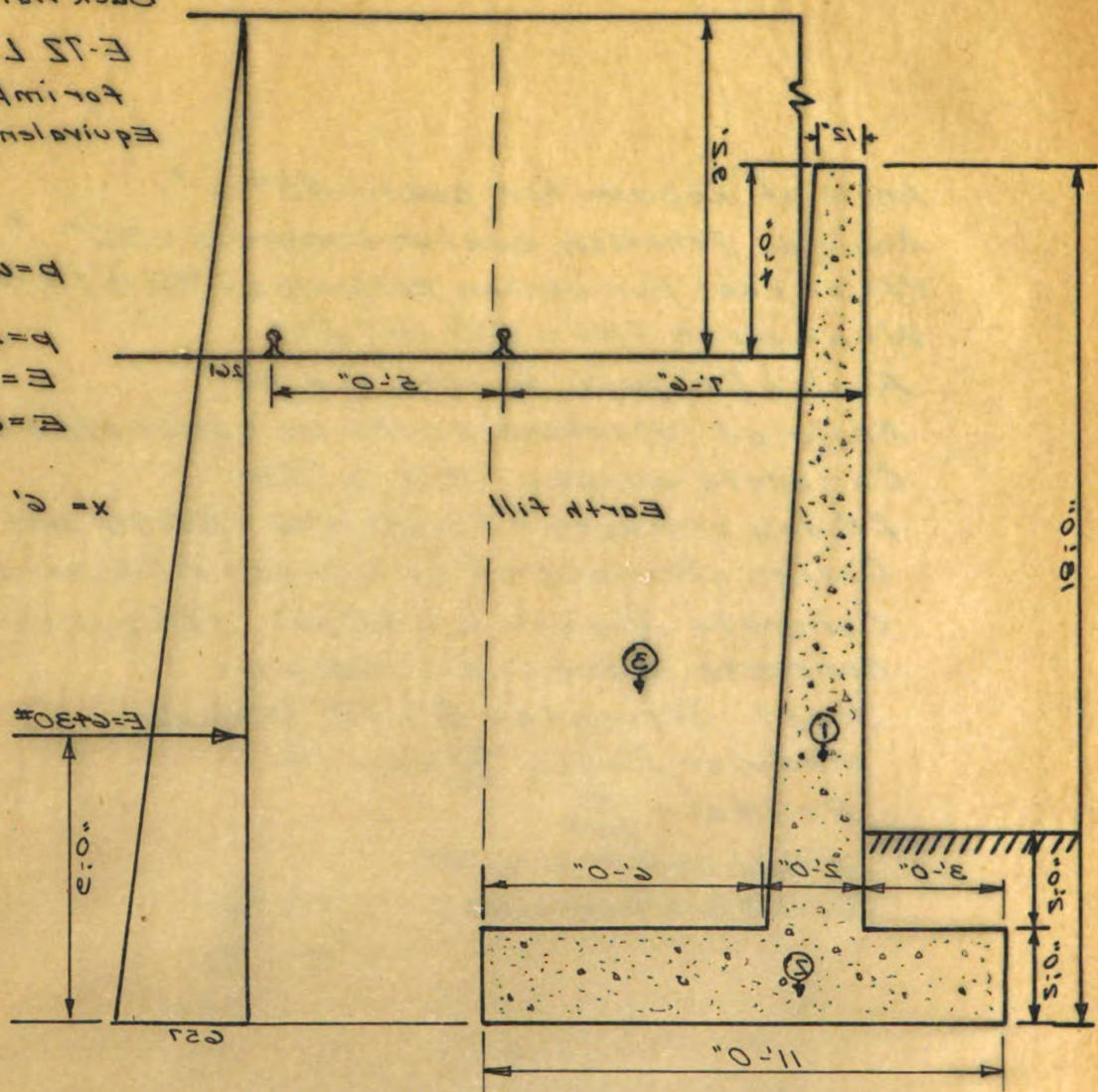
Mom		
18,500 lb-ft	=	2.2'
13,500	=	3.75'
111,500	=	8.0'
<u>145,900</u>		

Toe & Heel Pressures:
 $p = \frac{20,800}{11} \left(1 \pm \frac{e \times 4.8}{11} \right)$

$p = 1890 \pm 492$
 $p_t = 2382 \text{ psf}$
 $p_h = 1392 \text{ psf}$

$W_1 = 3300 \text{ lbs}$	
$W_2 = 3600$	
$W_3 = 13900$	
<u>20,800</u>	

Base Pressures: Moments about Toe



$E = 2430\#$
 $E = \frac{627 + 251}{2} \times 14$
 $p = 28.34$

$p = wh \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)$
 $= 920 \text{ psf} \times 0.92$
 $\frac{72,000}{2 \times 18} \times 1.12$
 $\text{Equivalent surcharge} =$
 for impact
 $E-75 \text{ Loading, allow } 12\%$

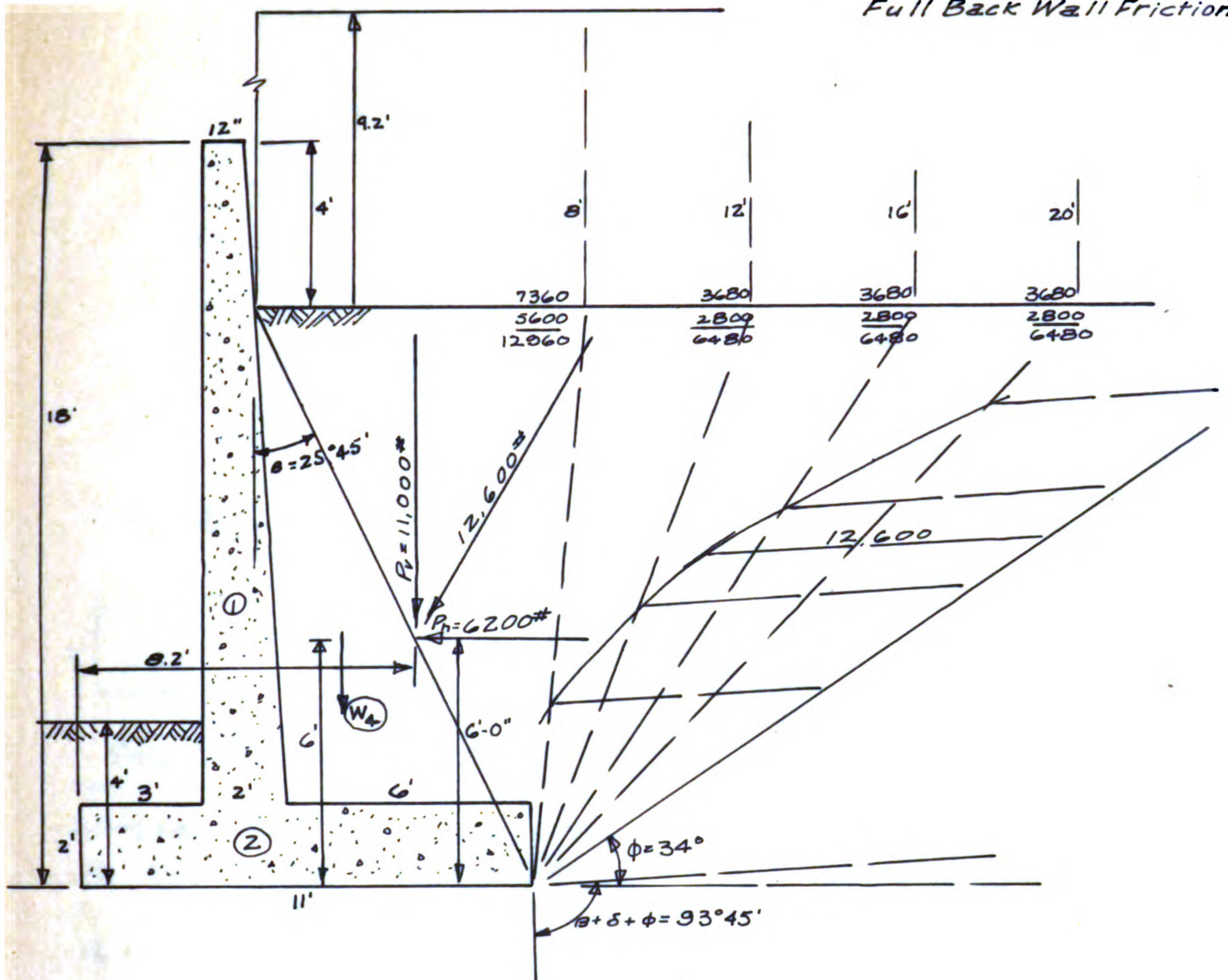
Back Wall Friction.
 Level Surcharge, Inert Prism, Omit

CASE I:

CASE II :

Level / Surcharge - Inert Wedge -

Full Back Wall Friction



Base Pressures: Moments about Toe

	W	arm	Mom.
$W_1 + W_2 =$	6900 lb		31,700 lb.ft.
$W_4 = 5.15 \times \frac{1}{2} \times 100$	3090	6.45	19,900
$P_v =$	11,000	8.2	90,200
	21,090		$M_R = 141,800$ lb.ft.
$P_h =$	6200	6.0	$M_o = 37,200$
	21,090		21,090 / 104,600

Toe & Heel Pressures:

$$p = \frac{21,090}{11} \left(1 \pm \frac{6 \times 0.5}{11} \right)$$

$$p = 1900 (1 \pm .272)$$

$$p_t = 2420 \text{ psf}$$

$$p_h = 1380 \text{ psf}$$

5.0' from toe
 $e = 5.5 - 5.0$
 $e = 0.5'$

Sliding: $\frac{21,090 \times 4.66}{6,200} = 1.59$

Overturn: $\frac{M_R}{M_o} = \frac{141,800}{37,200} = 3.71$ Must Anchor

Railroad Cantilever Wall

Overturn: $\frac{M_r}{M_o} = \frac{141,800}{37,500} = 3.71$

Sliding: $\frac{21,030 \times 4.66}{6,500} = 1.29$

$c = 0.2'$
 $e = 2.2 - 2.0$
 $e = 2.0'$

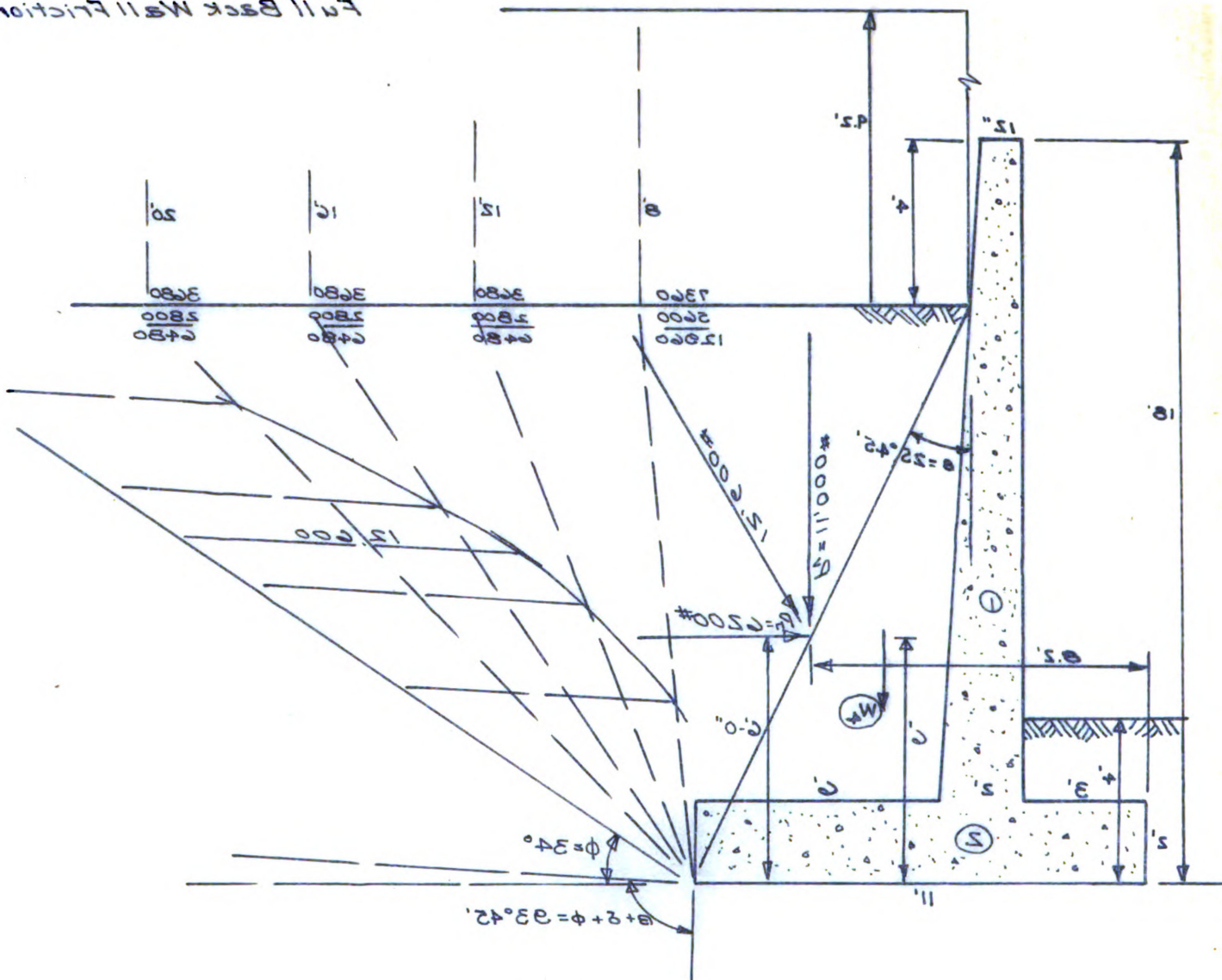
$21,030 / 104,600$
 $6.0 = M_o = 37,500$
 $M_r = 141,800$

Mem. $31,500$ lb-ft
 $19,900$
 $20,500$
 $141,800$ lb-ft

Base Pressures: Moments about ToC

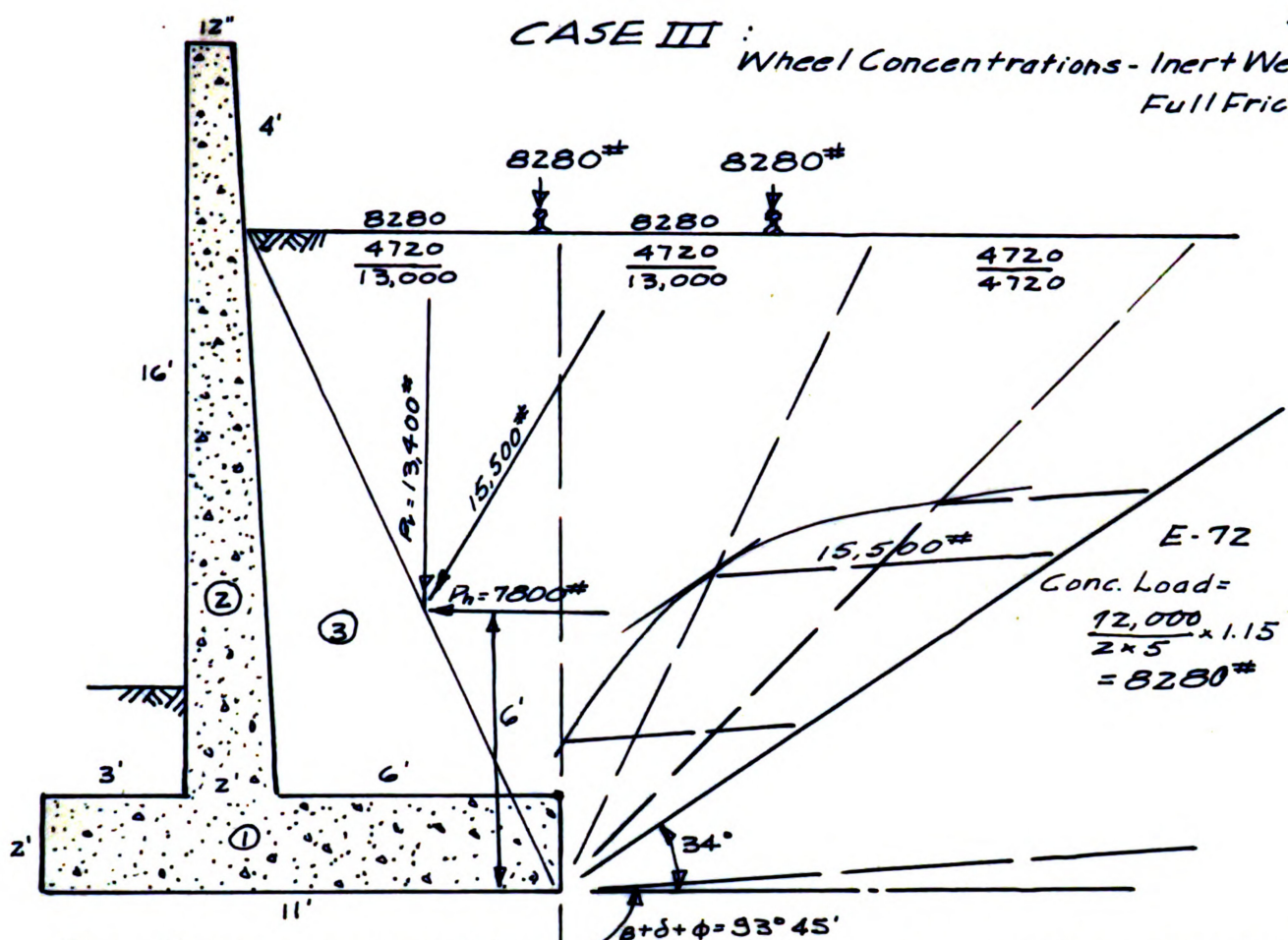
$P_h = 2500$
 $P_v = 11,000$
 $W = 3090$
 $W + W_s = 6900$
 $M_1 + M_2 = 21,030$
 $M_1 = 2.12 \times \frac{1}{2} \times 100 \times 3090 \times 0.42$
 $M_2 = 11,000 \times 8.5$

ToC & Heel Pressures:
 $p = \frac{21,030}{11} \left(1 \pm \frac{e \times 0.2}{11} \right)$
 $p = 1900$ (1.25)
 $p = 2450$ psf
 $p_h = 1380$ psf



CASE III :

Wheel Concentrations - Inert Wedge - Full Friction

Base Pressures: $\Sigma M_{abt. toe}$

Wt.

$$W_1 + W_2 + W_3 = 10,090$$

Mom.

$$51,600$$

$$P_v = \frac{13,400 \times 8.2}{23,390} = 110,000$$

$$M_R = 162,600 \text{ lb.ft.}$$

$$P_h = 7800 \times 6 = M_0 = \frac{46,800}{23,390} = 115,800$$

$$4.95' \text{ from toe}$$

$$e = 5.5 - 4.95$$

$$e = 0.55'$$

Toe and Heel Pressures:

$$p = \frac{23,390}{11} (1 \pm \frac{6 \times 0.55}{11})$$

$$p = 2125 \pm 701$$

$$p_t = 2826 \text{ psf}$$

$$p_h = 1424 \text{ psf}$$

Sliding:

$$\frac{23,390 \times 4.66}{7800} = 1.4$$

Must Anchor

Overturn:

$$\frac{M_R}{M_0} = \frac{162,600}{46,800} = 3.5$$

SUMMARY OF CASES

	P_{toe}	P_{heel}	Hor. Thrust	M_R/M_0	Sliding Factor
I	2835	1395	6430	3.7	1.51
II	2420	1380	6200	3.71	1.59
III	2826	1424	7800	3.5	1.4

Use Case III for design purposes
as it presents the most severe
conditions.

Use Case III for design purposes as it presents the most severe conditions.

Pressure	Result	Hor Thrust	Mr/Mo	Sliding Factor
I 5832	1392	6430	3.7	1.21
II 5450	1380	6500	3.71	1.24
III 5856	1454	7800	3.2	1.4

SUMMARY OF CASES

Overturn: $\frac{M_r}{M_o} = \frac{165,600}{46,800} = 3.2$

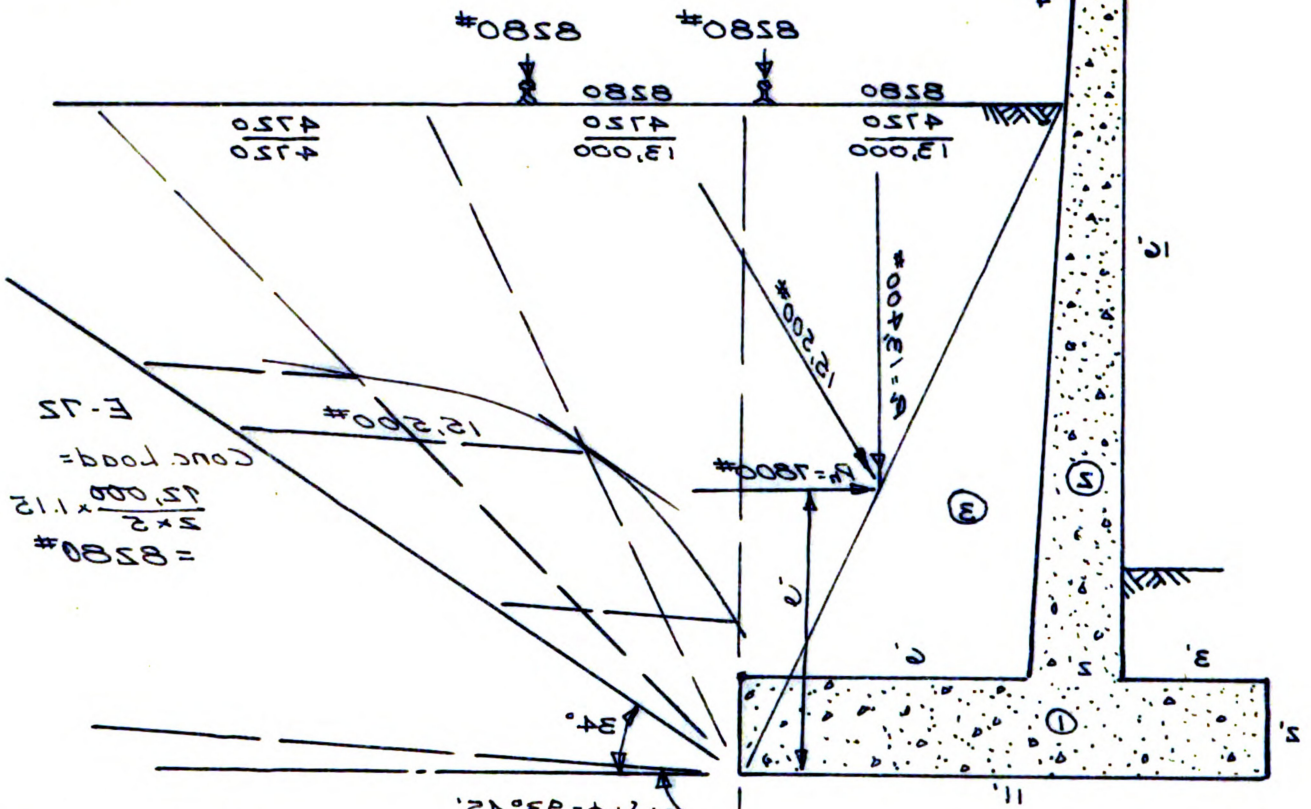
Sliding: $\frac{23,390 \times 4.66}{1800} = 1.4$

Base Pressure: $\leq$ Map. toe: $\frac{23,390}{112,800} = 0.21$
 $\frac{1800 \times 6}{112,800} = 0.097$
 $\frac{13,400}{23,390} \times 8.5 = 110,000$
 $\frac{21,600}{165,600} = 12.7\%$
 $\frac{21,600}{112,800} = 19.2\%$
 $\frac{21,600}{165,600} = 12.7\%$

$p_v = 1454 \text{ psf}$
 $p_t = 5856 \text{ psf}$
 $p = 5152 \pm 101$

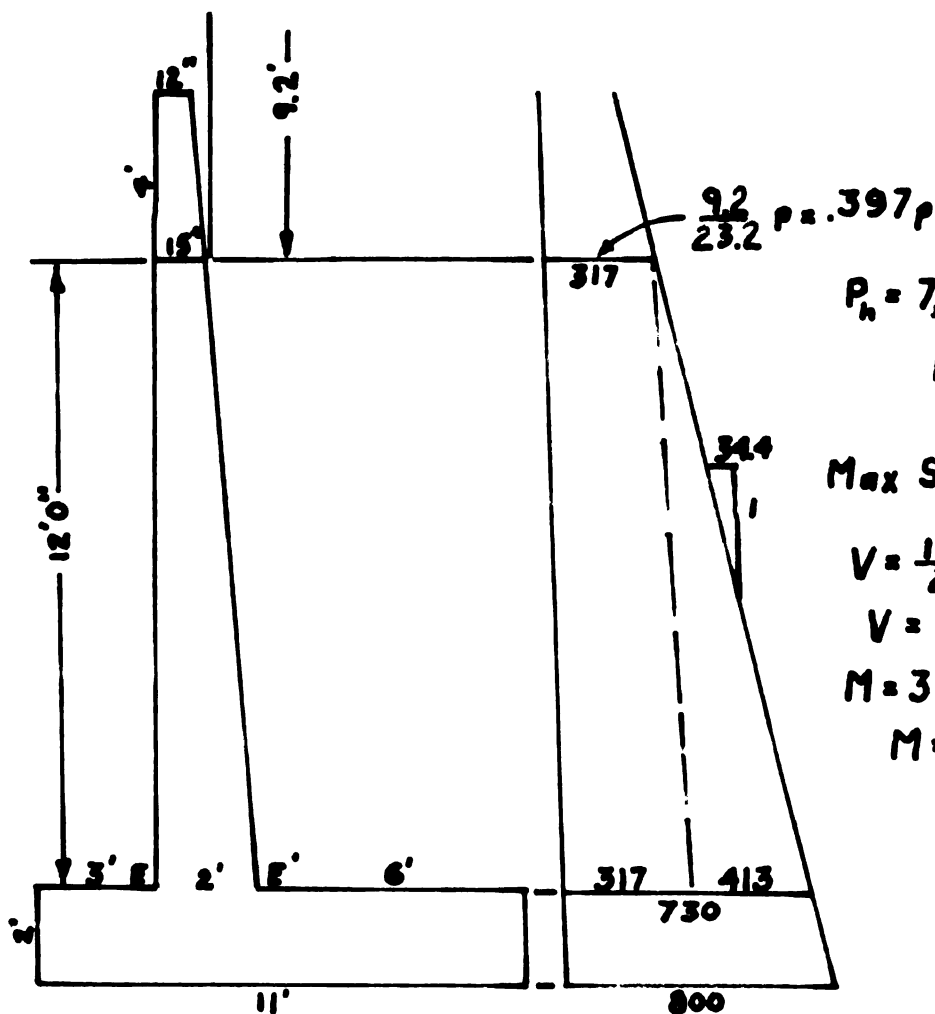
Toe and Heel Pressures: $p = \frac{53,390(1 \pm 6 \times 22)}{11} = 53,390$

$\text{Conc. Load} = \frac{15,000}{5 \times 2} \times 1.12 = 8580 \#$
 $E-75$



CASE III: Wheel Concentrations - Inert Wedge - Full Friction

STEM DESIGN



$$P_h = 7800 = \frac{1}{2} (p + .397p)$$

$$p = 800 \text{ p.s.f.}$$

Max Shear & B.M. on Sect. EE'

$$V = \frac{1}{2} (730 + 317)$$

$$V = 7329^*$$

$$M = 317 \cdot 12 \cdot 6 + 413 \cdot 6 \cdot 4$$

$$M = 32,700^{**}$$

$$d = \frac{V}{b j v} = \frac{7329}{12 \cdot .78 \cdot 40} = 17.4"$$

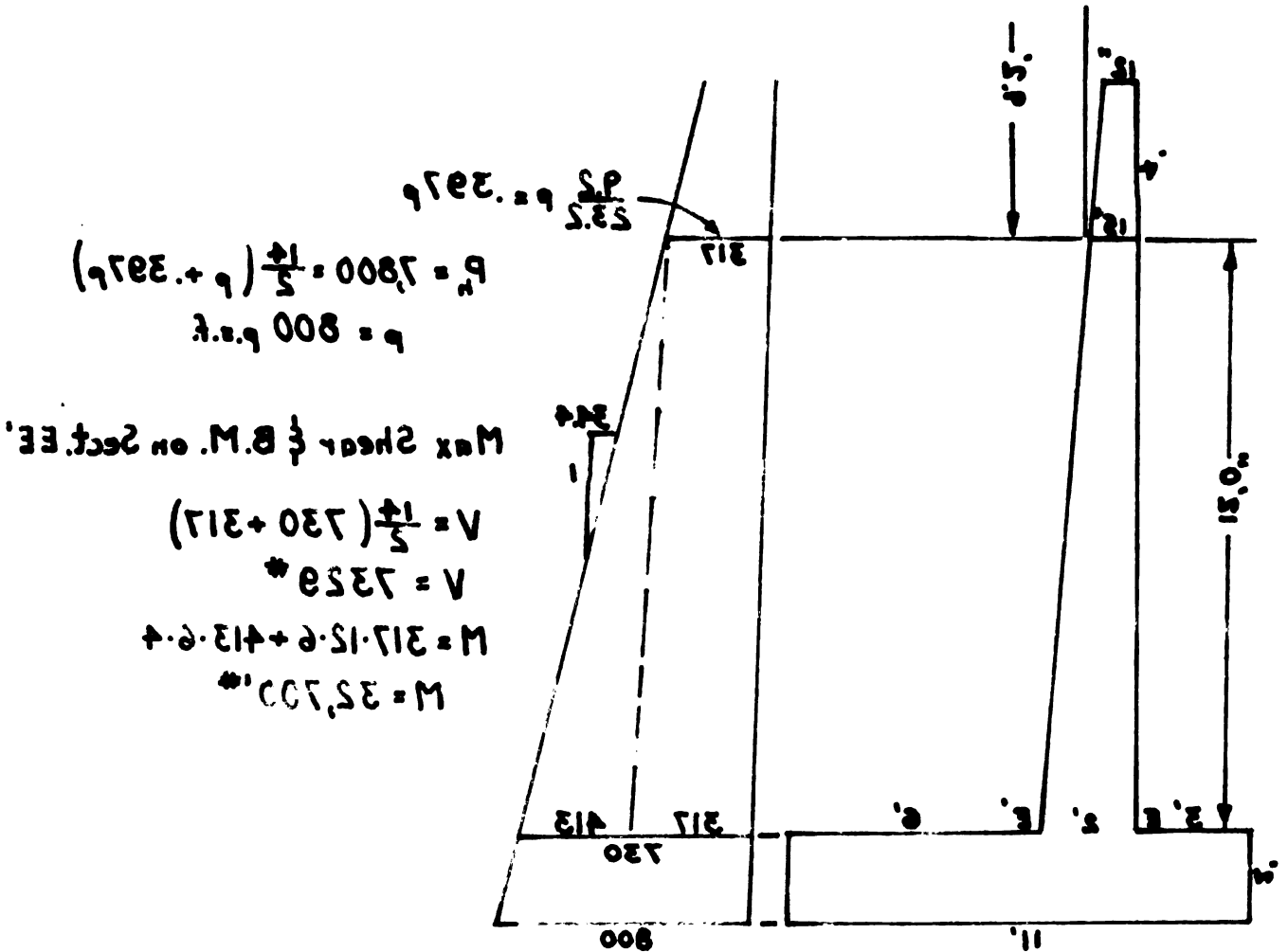
$$d = \sqrt{\frac{M}{b R}} = \sqrt{\frac{32,700}{140}} = 15.3"$$

Use Stem 24" Thick at bottom

Use Stem 54" Thick at Bottom

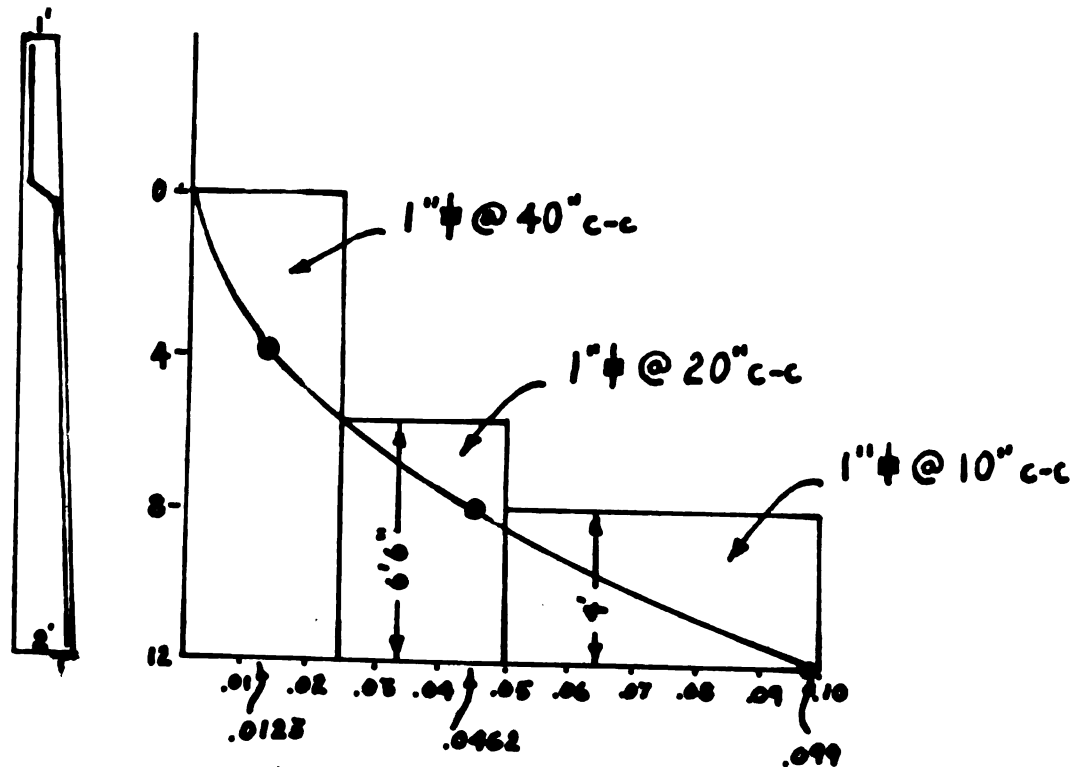
$$q = p_j v = \frac{V}{15.18 \cdot 40} = \frac{7353}{15.18 \cdot 40} = 12.4"$$

$$q = \sqrt{\frac{M}{p R}} = \sqrt{\frac{35700}{140}} = 15.7"$$



STEM DESIGN

STEM DESIGN



At Any Level:

$$V_x = 317 \cdot h + \frac{34.4}{2} h^2 \quad M_x = \frac{317 \cdot h^2}{2} + \frac{17.2 \cdot h^3}{3}$$

STEM SCHEDULE

h	V	d	v	M	R	A _s	Rods
4	1545	15"	9.83	2907	10.7	.0123	1" @ 40"
8	3640	18"	19.3	13,100	40	.0462	1" @ 20"
12	7329	21"	33.3	32,700	57	.099	1" @ 10"

$$\mu = \frac{v \cdot b}{\Sigma_o} = \frac{33.3 \cdot 10}{4} = 83 \text{ p.s.i.} < 100$$

$$L = \frac{f_y \cdot d}{4 \cdot \mu} = \frac{18000 \cdot 1}{4 \cdot 100} = 45" \text{ (Hook Bottom)}$$

$$\text{Temp. Steel: } .002 \cdot 12 \cdot 18 = .44"$$

Temp. Steel: $0.05 \cdot 15 \cdot 18 = .44^\circ$

$$r = \frac{t_p}{4} = \frac{18000 \cdot 1}{4 \cdot 100} = .45" \text{ (Hook Bottom)}$$

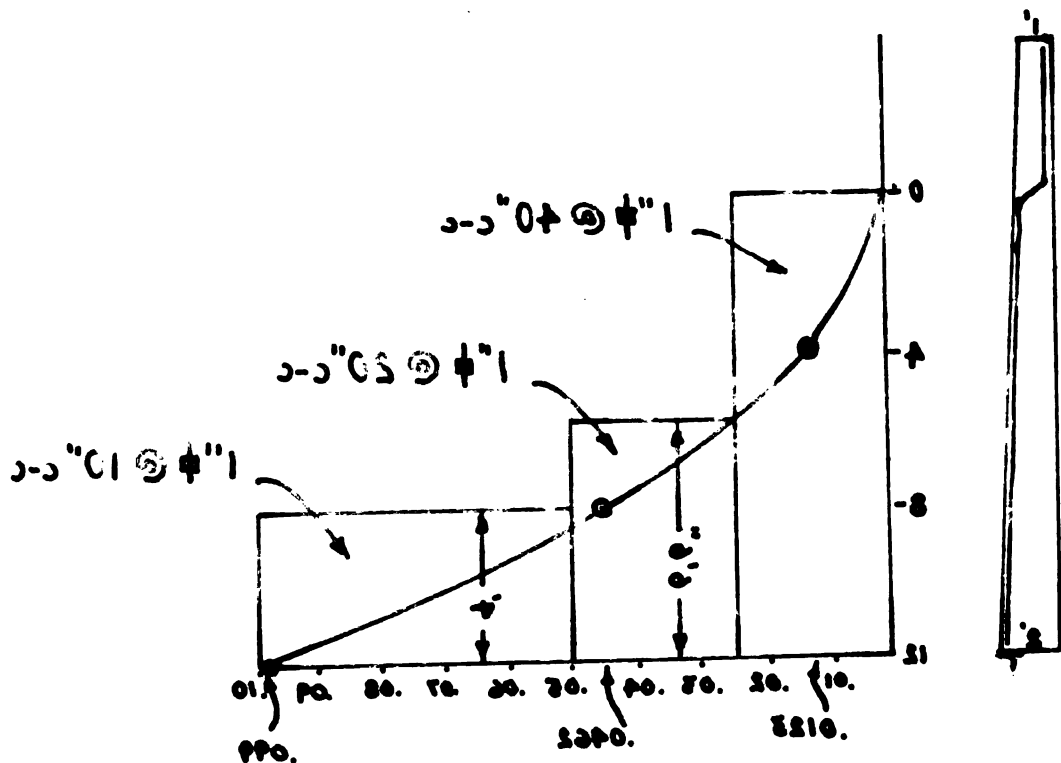
$$v = \frac{v_p}{2} = \frac{333 \cdot 10}{4} = 83 \text{ p.s.i.} < 100$$

h	V	q	v	M	R	A _s	Rods
4	1242	12"	283	5307	107	.0153	1" @ 40"
8	3640	18"	133	13100	47	.0465	1" @ 20"
15	7352	21"	333	35700	21	.099	1" @ 10"

STEM SCHEDULE

$$M^* = 317 \cdot h + \frac{347}{5} \cdot h^2 \quad M^* = \frac{317 \cdot h^2}{5} + \frac{153 \cdot h^3}{3}$$

At Any Level:



STEM DESIGN

Diagram of a beam cross-section showing the calculation of the centroidal moment of inertia. The cross-section is an L-shape with a top flange of width 3' and height 8, and a vertical web of width 4 and height 12. The centroid is marked with a dot and labeled 'C'. The distance from the top-left corner to the centroid is 3' horizontally and 4' vertically. The distance from the bottom-left corner to the centroid is 6' horizontally and 6' vertically. The moment of inertia about the centroidal axis is calculated as $I = 7902$.

$$y = \frac{3}{3} \left(\frac{2142 + 2 \cdot 2526}{2526 + 2142} \right) = 1.54'$$

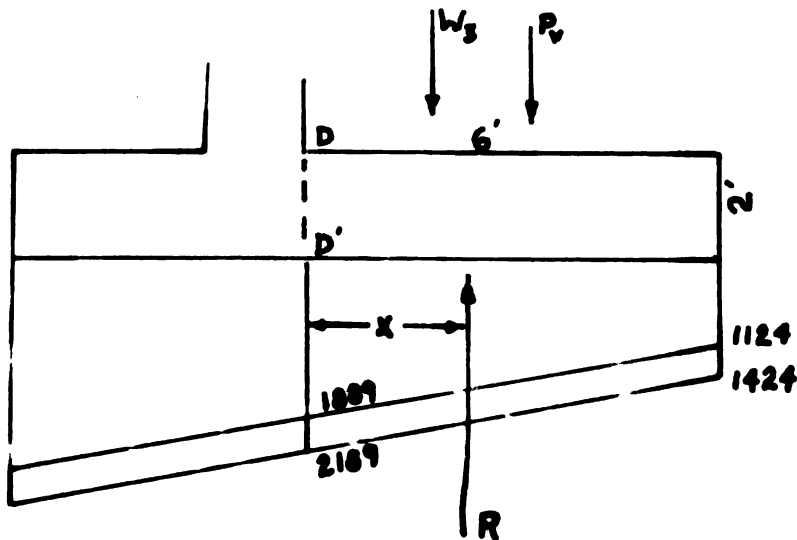
$$V = 7,902^*$$

$$v = \frac{V}{b_j d} = \frac{7902}{12 \cdot 78 \cdot 21} = 35.8 \text{ p.s.i.}$$

$$A_s = \frac{M}{f_s j d} = \frac{12,200}{18,000 \cdot .7 \cdot 21} = .0368 \text{ "}$$

$$\mu = \frac{vb}{\Sigma o} = \frac{40 \cdot 6}{2} = 120 \text{ p.s.i. (Spec. Anch.)}$$

HEEL DESIGN



$$(1124 \cdot 6 \cdot \frac{6}{2}) + (765 \cdot \frac{6}{2} \cdot \frac{6}{3}) = x(1124 \cdot 6 + 765 \cdot 3)$$

$$x = 2.75'$$

$$R = 9045^{**}$$

wt.	arm	Mem.
$P_v = -13,400^{**}$	$3.2'$	$-42,900^{**}$
$W_3 = -3,090$	$1.45'$	$-4,480$
$-16,490$		$-48,380$
$R = +9,045$	$2.75'$	$+24,800$
$V = 7,445$	$M =$	$23,580$

$$d = \frac{V}{v_j b} = \frac{7445}{40 \cdot \frac{7}{8} \cdot 12} = 17.7''$$

$$d = \sqrt{\frac{M}{R b}} = \sqrt{\frac{23580}{140}} = 13''$$

Keep Heel 24" Thick.

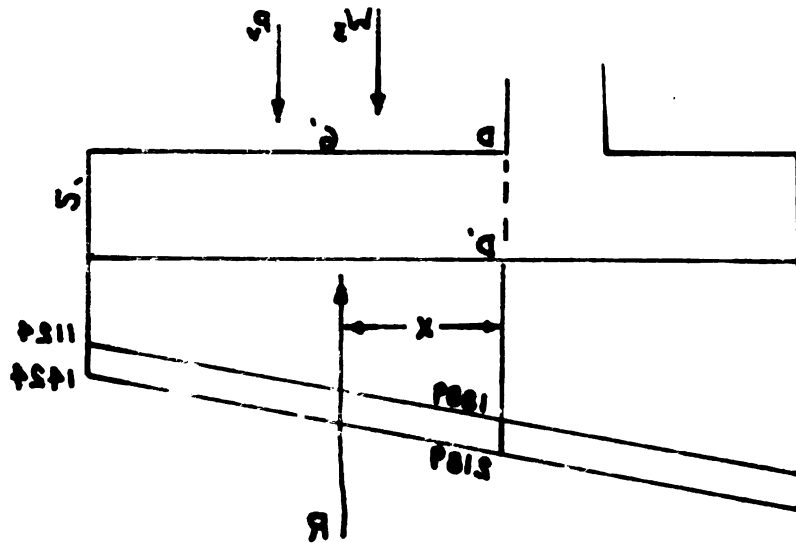
$$R = \frac{M}{b d^2} = 53.4 < 140 \therefore f_c \text{ is low.}$$

$$A_s = \frac{23,580}{18000 \cdot \frac{7}{8} \cdot 21} = .0713 \text{ } \frac{\text{in}^2}{\text{in}} \text{ Top Use } 1'' \phi \text{ at } 11'' \text{ c-c gives } .0718 \text{ } \frac{\text{in}^2}{\text{in}}$$

$$p = \frac{x b}{\Sigma o} = \frac{90 \cdot 11}{\pi} = 118 \text{ p.s.i.} > 100 \therefore \text{Spec. Anch.}$$

Railroad Cantilever Wall

HEEL DESIGN



$$(1154 \cdot \frac{1}{2}) + (202 \cdot \frac{1}{2}) = (1154 \cdot 0 + 202 \cdot 0)$$

$$\begin{aligned} 27.5 &= x \\ 2400 &= R \end{aligned}$$

$ \begin{array}{r} V = 5,442 \\ R = + 2,042 \\ \hline 16,480 \\ W_2 = - 3,020 \\ \hline B = - 13,400 \end{array} $	$ \begin{array}{r} M = 53,280 \\ 5.72 = + 54,800 \\ \hline - 18,380 \\ 1.42 = - 4,480 \\ \hline 3.5 = - 45,200 \end{array} $	$ \begin{array}{r} W_1 = 100,540 \\ 3.5 = 350 \\ \hline 100,890 \end{array} $
--	--	--

$$q = \sqrt{\frac{M}{R_p}} = \sqrt{\frac{140}{53280}} = 13''$$

Keep Heat Seal Thick.

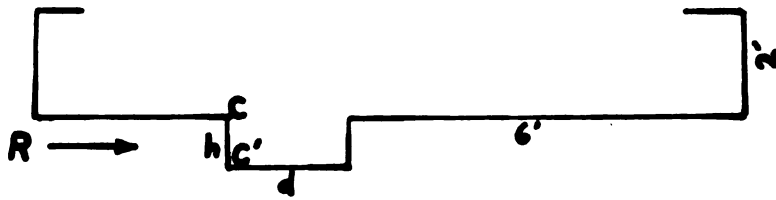
$$R = \frac{M}{P_{qf}} = 23.4 < 140 \therefore f, \text{ is low.}$$

$$A = \frac{0.013 \text{ "}^2 \times 170}{18000 \times \sqrt{15.5}} = 2.87 \text{ ES} = 2.87 \text{ A}$$

$$\therefore \text{Spec. Anch.} = 11.0 \frac{\text{mm}}{\text{mm}} = \frac{11.0}{0.3} = 36.67$$

Railroad Cantilever Wall

CUT OFF WALL



$$\text{Passive Earth Pressure} = wh \left(\frac{1 + \sin 34^\circ}{1 - \sin 34^\circ} \right)$$

$$\text{at C-C'} \quad wh = 2444 \text{ p.s.f.}$$

Case III

$$\text{Possible Earth Resistance} = 3.54 wh = 8650 \text{ p.s.f.}$$

$$\begin{aligned} \text{To make Sliding Factor} = 2 \quad R &= 2 \cdot 7,800 = .466 \cdot 23,390 \\ &= 4,700^{\text{lb}} \text{ req'd} \end{aligned}$$

$$\text{Min. } h = \frac{4700}{8650} = .54' \quad \text{Use } h = 9''$$

$$d = \frac{4700}{9 \cdot \frac{7}{8} \cdot 40} = 15''$$

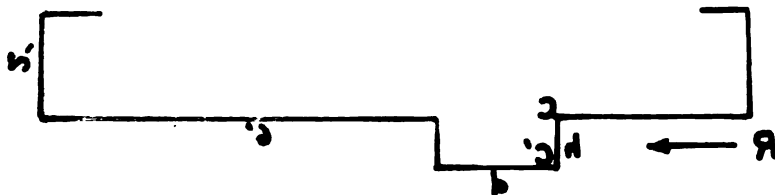
$$\begin{aligned} \text{on C-C'} \quad V &= 4700^{\text{lb}} \\ M &= V \frac{h}{2} = 21,200^{\text{lb-in}} \end{aligned}$$

IF No Tension Reinforcement

$$d = \sqrt{\frac{21,200 \cdot 45}{12 \cdot 60}} = 13.3''$$

$$\text{Use } d = 2'$$

CUT OFF WALL



$$\text{Passive Earth Pressure} = wh \left(\frac{1 + 2 \sin 34^\circ}{1 - 2 \sin 34^\circ} \right)$$

$$\text{at C-C, } wh = 5444 \pm \text{p.s.f.}$$

Case III

$$\text{Possible Earth Resistance: } 324 wh = 8620 \text{ p.s.f.}$$

$$\text{To make Sliding Factor} = 2 \quad R = 5.7800 = 466.53330$$

$$= 4700 \text{ p.s.f.}$$

$$\text{Min. } h = \frac{4700}{8620} = .24' \quad \text{Use } h = 2'$$

$$b = \frac{4700}{2.78 \cdot 40} = 12'$$

$$\text{on C-C, } V = 4700'$$

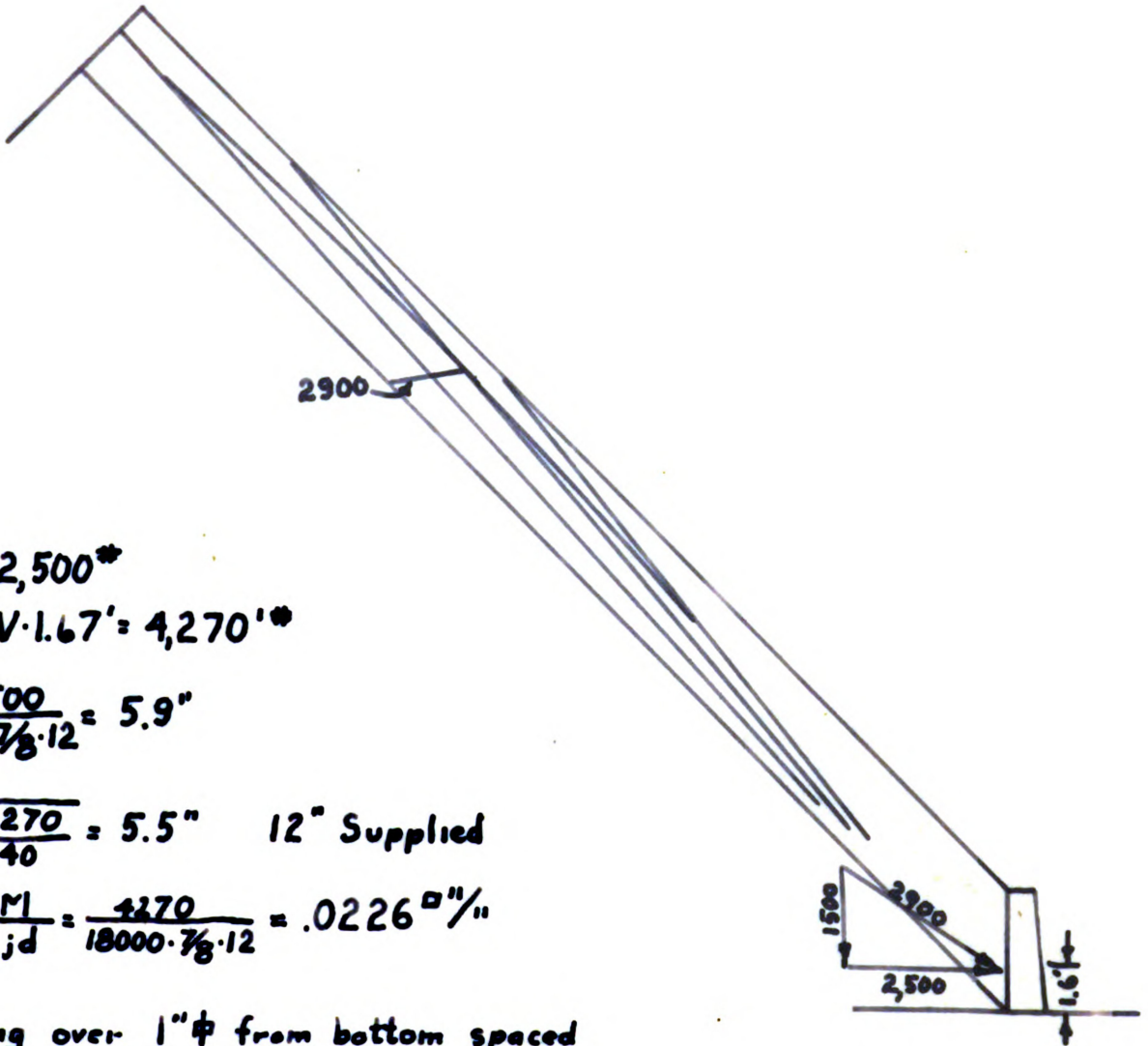
$$M = V \cdot \frac{h}{2} = 51,500'$$

If No Tension Reinforcement

$$q = \sqrt{\frac{51500 \cdot 42}{15 \cdot 60}} = 13.2'$$

$$\text{Use } q = 5'$$

DESIGN OF TOP OF R.R. WALL



$$V = 2,500^*$$

$$M = V \cdot 1.67' = 4,270'*$$

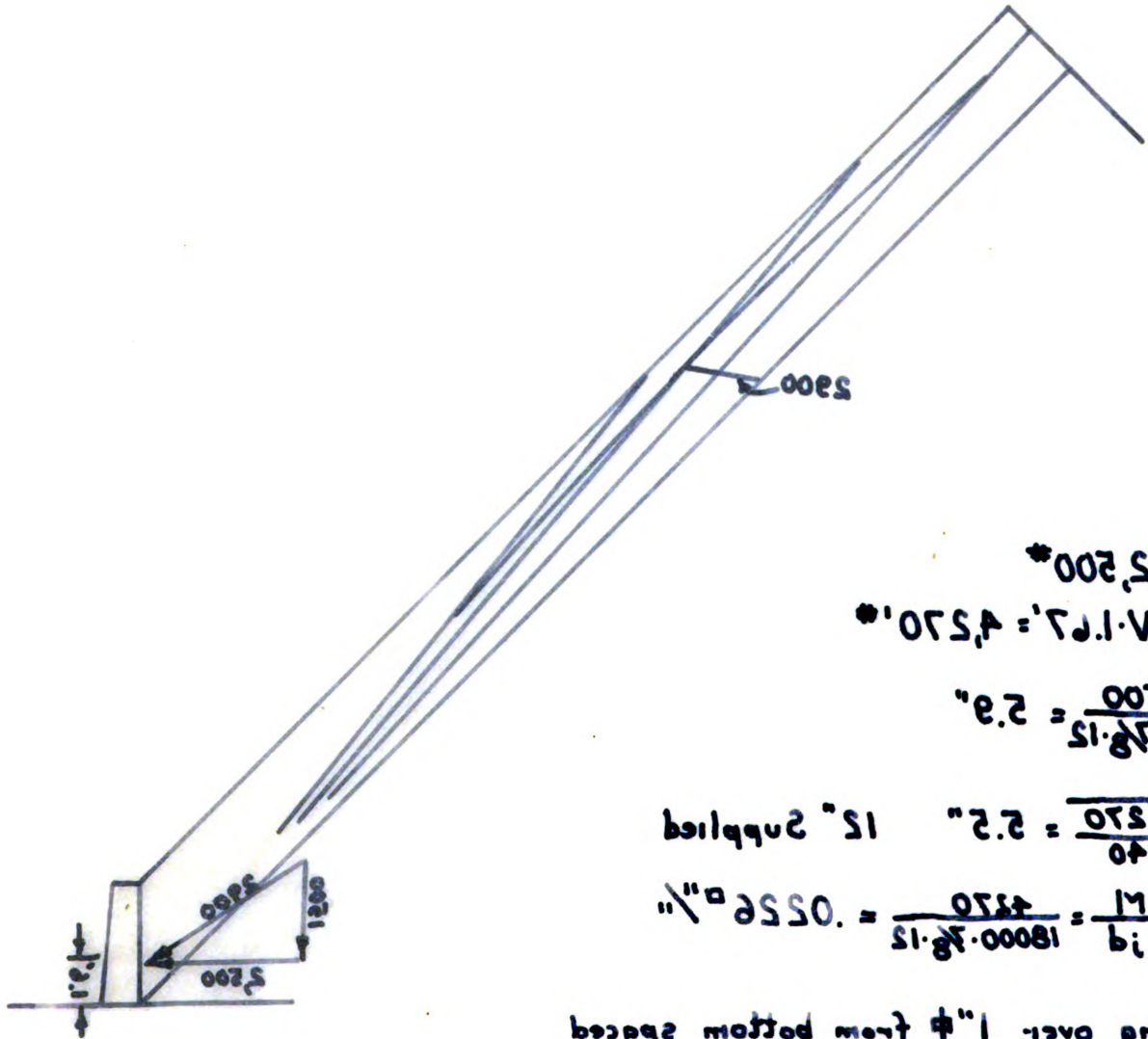
$$d = \frac{2500}{40 \cdot \frac{7}{8} \cdot 12} = 5.9"$$

$$d = \sqrt{\frac{4270}{140}} = 5.5" \quad 12" \text{ Supplied}$$

$$A_s = \frac{M}{f_s j d} = \frac{4270}{18000 \cdot \frac{7}{8} \cdot 12} = .0226 \text{ sq in.}$$

Bending over 1" ϕ from bottom spaced
at 40" supplies .025 sq in.

DESIGN OF TOP OF R.R. WALL



at 40" supplies .052 $\frac{b}{s}$
Bending over 1" from bottom spaced

$$A_s = \frac{f_y A}{f_y} = \frac{18000 \cdot 38 \cdot 15}{4520} = .0556 \frac{b}{s}$$

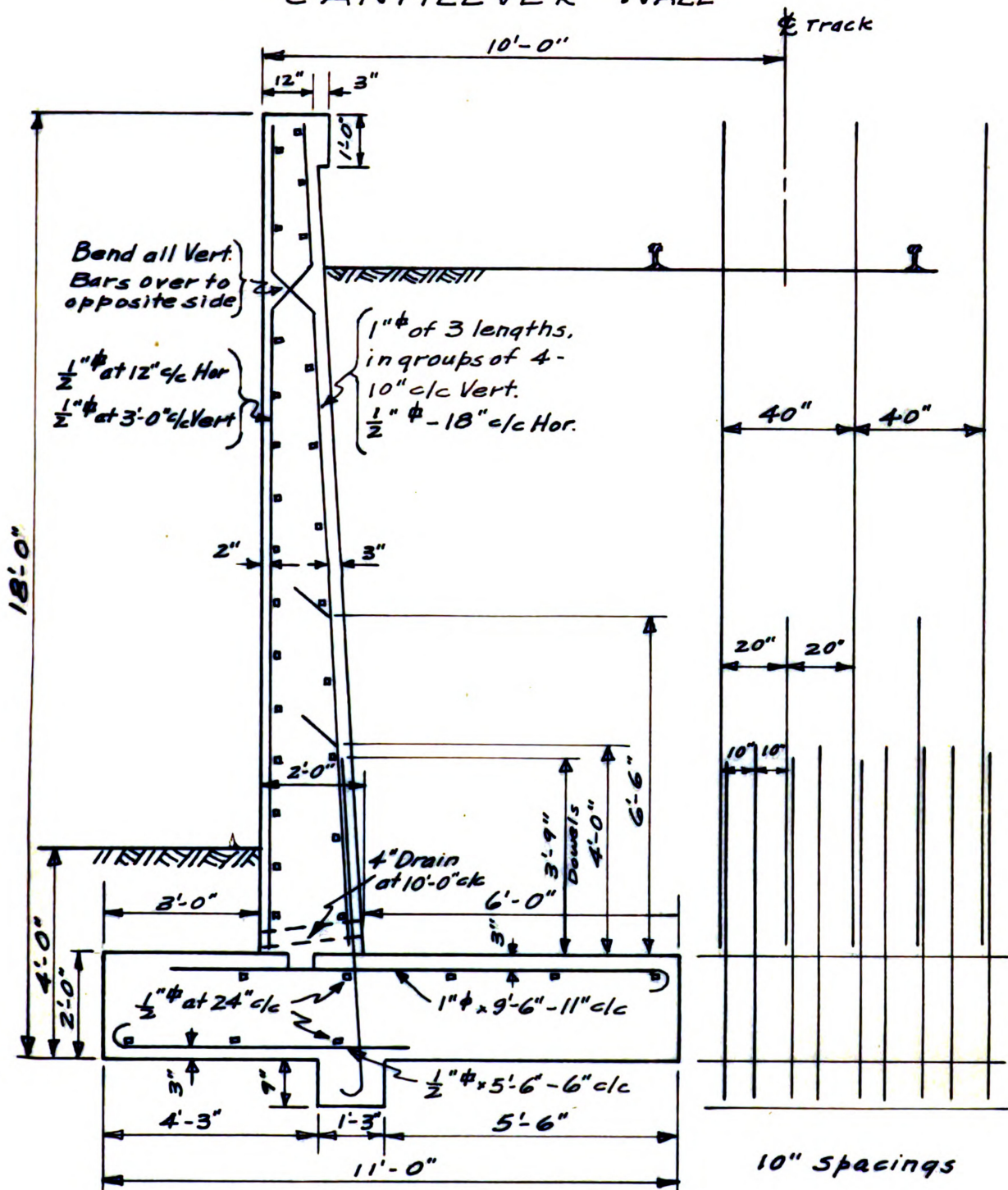
$$q = \sqrt{\frac{4520}{140}} = 2.2" \quad 15" \text{ supplied}$$

$$q = \frac{40 \cdot 38 \cdot 15}{5200} = 2.3"$$

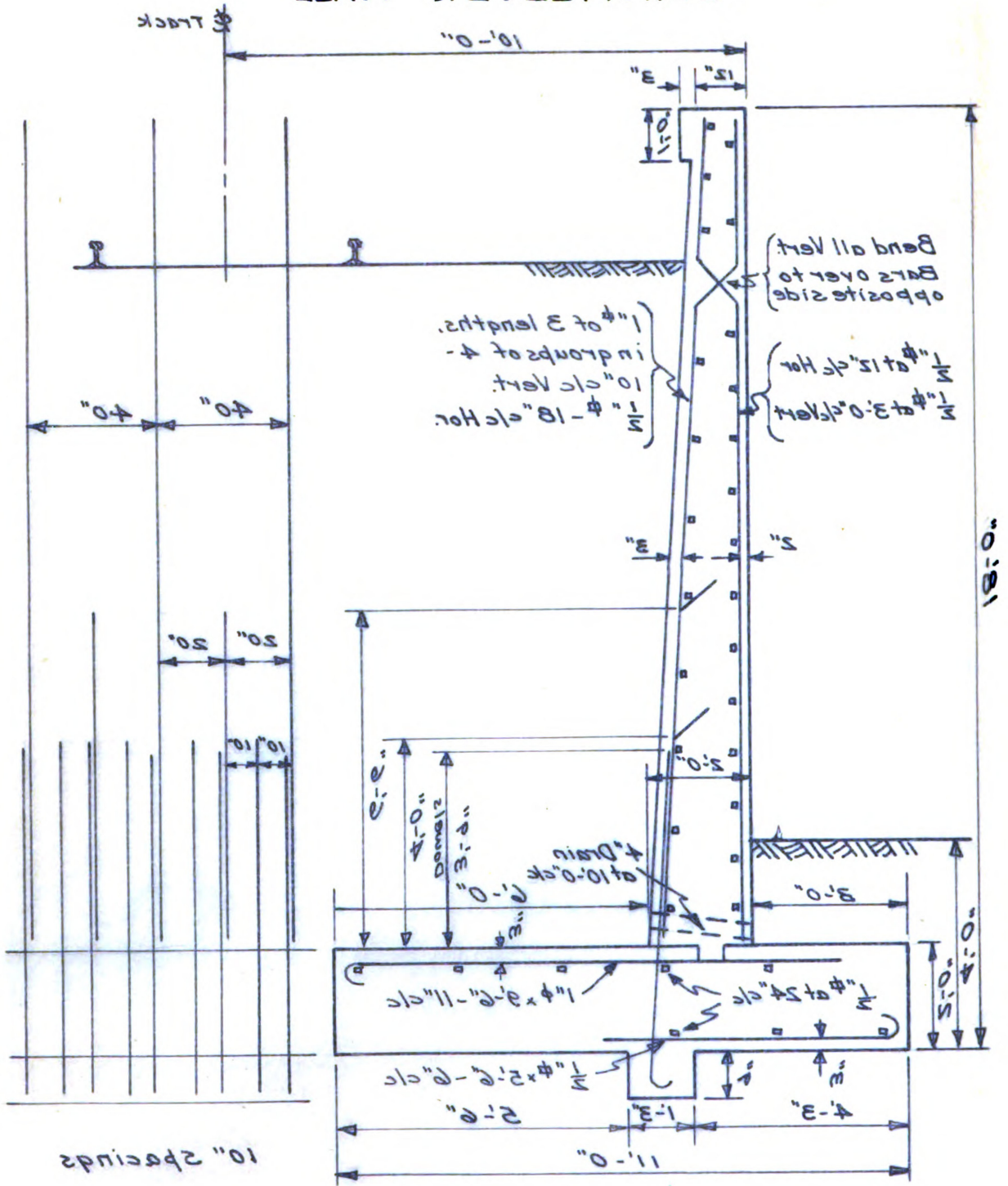
$$M = V \cdot l \cdot 2 = 4520'$$

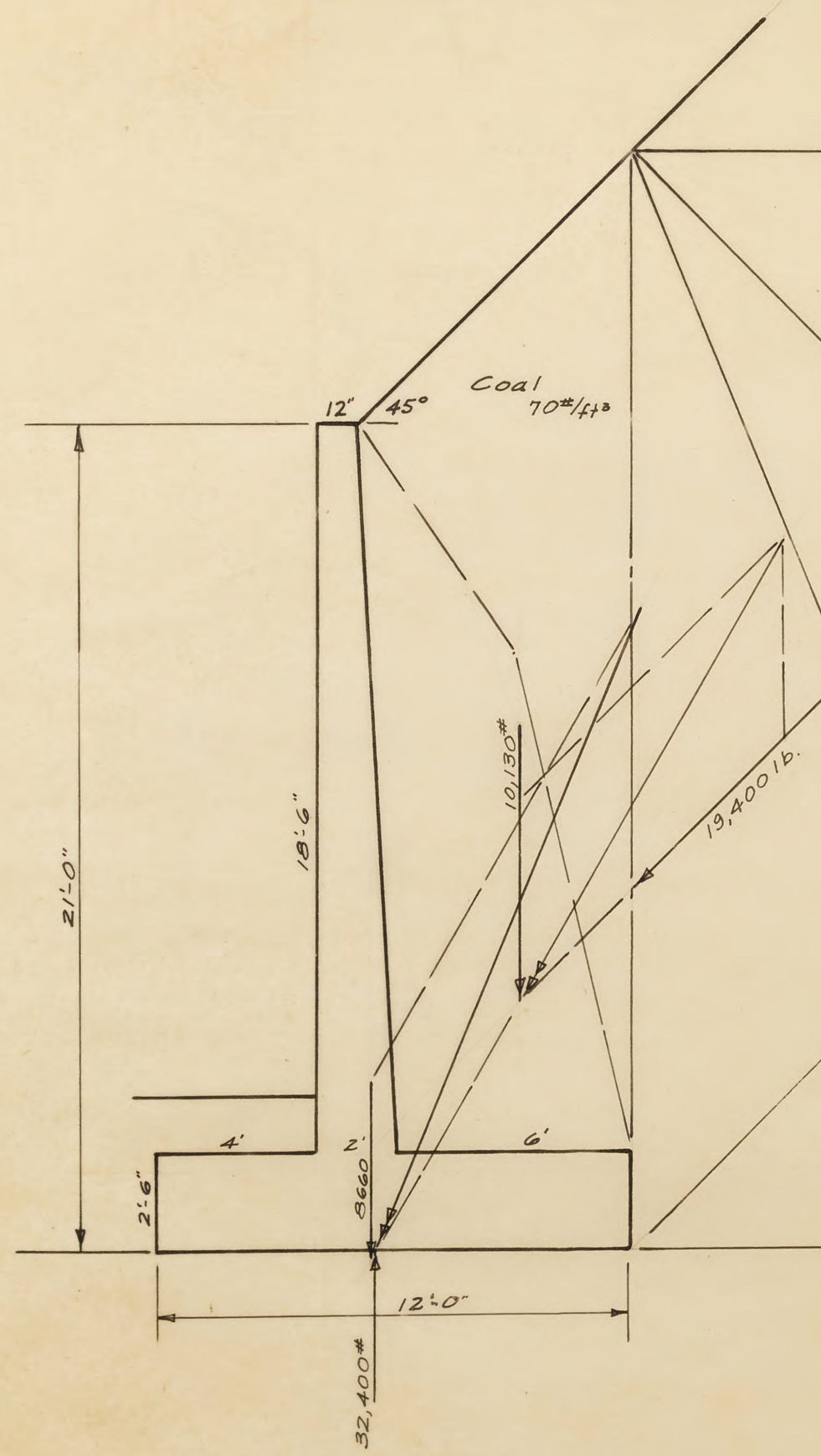
$$V = 5200'$$

DETAILS OF RAILROAD CANTILEVER WALL



DETAILS OF RAILROAD CANTILEVER WALL





Graphical Construction
from "Walls, Bins and
Grain Elevators" by
Ketchum.

STEM DESIGN, - Inert Prism - Full Friction

Shear: $H = \frac{1}{2} wh^2 \cos^2 \phi$
at $h = 18.5'$ $H = 6000 \text{ lb.}$
at $h = 12'$ $H = 2520 \text{ lb.}$
at $h = 6'$ $H = 630 \text{ lb.}$

Bending Moments: H acting at $\frac{1}{3} h$.
at $h = 18.5'$ $M = 37,000 \text{ lb. ft.}$
at $h = 12'$ $M = 10,080 \text{ lb. ft.}$
at $h = 6'$ $M = 1,260 \text{ lb. ft.}$

for Moment, $d = \sqrt{\frac{M}{Rb}}$
at $h = 18.5'$ $d = 19.1''$
at $h = 12'$ $d = 9.87''$
at $h = 6'$ $d = 3.52''$

for Shear
 $d = \frac{V}{bjv}$
at $h = 18.5'$ $d = 14.3''$
Moment controls

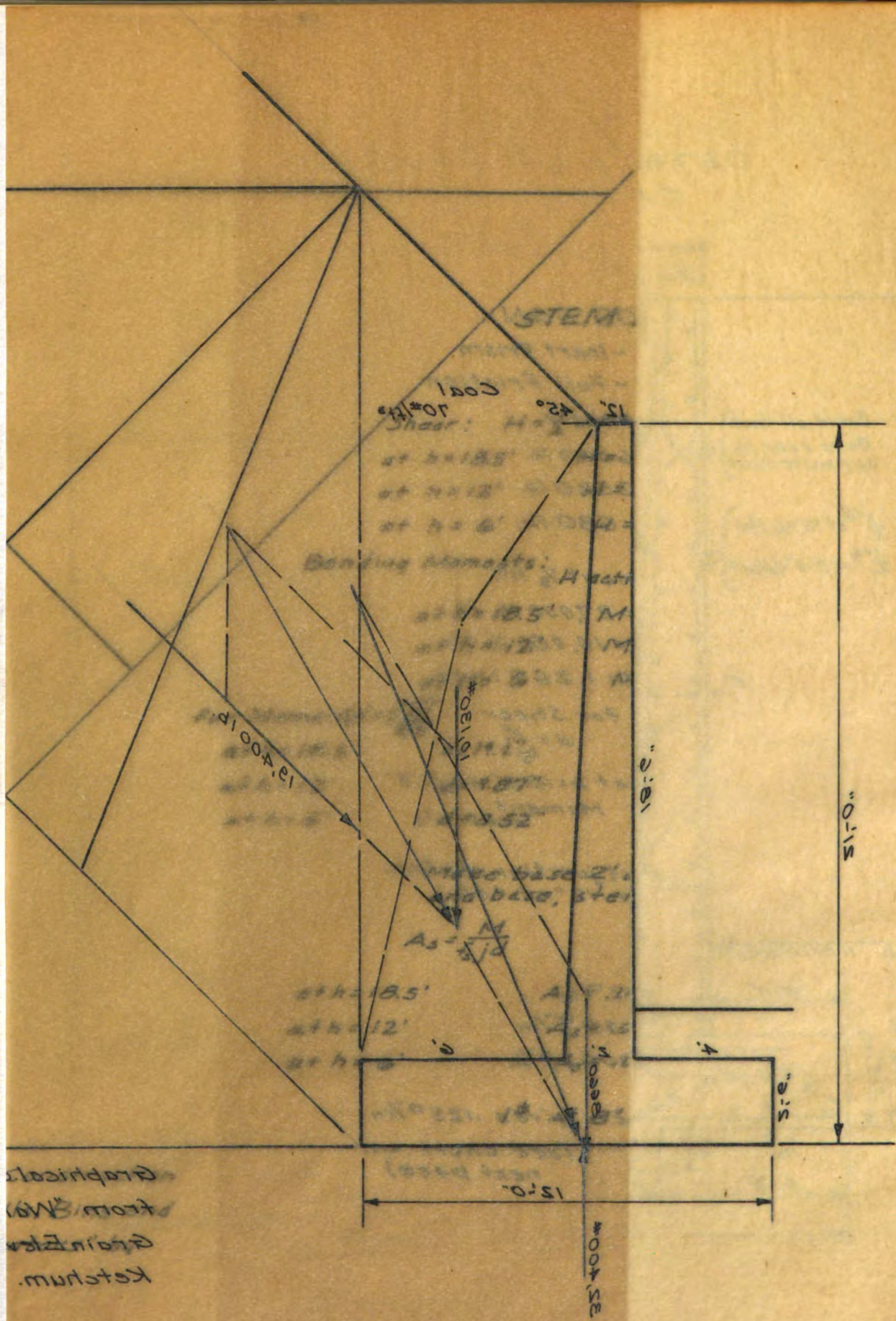
Make base 2' at junction of stem
and base, stem 12" at top.

$$A_s = \frac{M}{f_s j d}$$

at $h = 18.5'$ $A_s = .112''/\text{in}$
at $h = 12'$ $A_s = .038''/\text{in}$
at $h = 6'$ $A_s = .0062''/\text{in}$

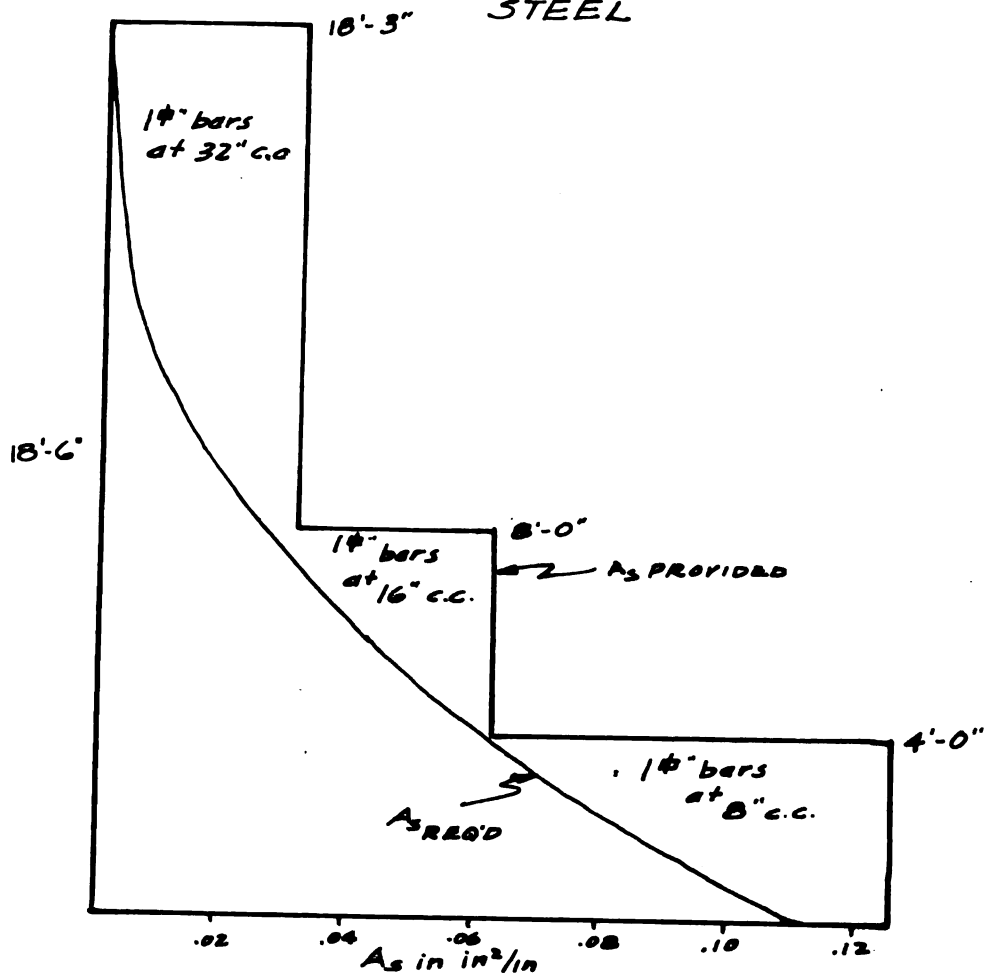
1# at 8" c.c. gives .125''/in
(see chart on
next page)

Cantilever Outside Wall



Cantilever Outside Wall

STEM SCHEDULE FOR STEEL



Bond:

$$u = \frac{V}{\Sigma o j d} = \frac{6000}{4 \times \frac{7}{8} \times 21} = 81.6 \text{ psi}$$

Anchorage:

$$L = \frac{f_y d}{4u} = \frac{18,000 \times 1}{4 \times 100} = 45"$$

Hooks Req'd.

Temperature Steel:

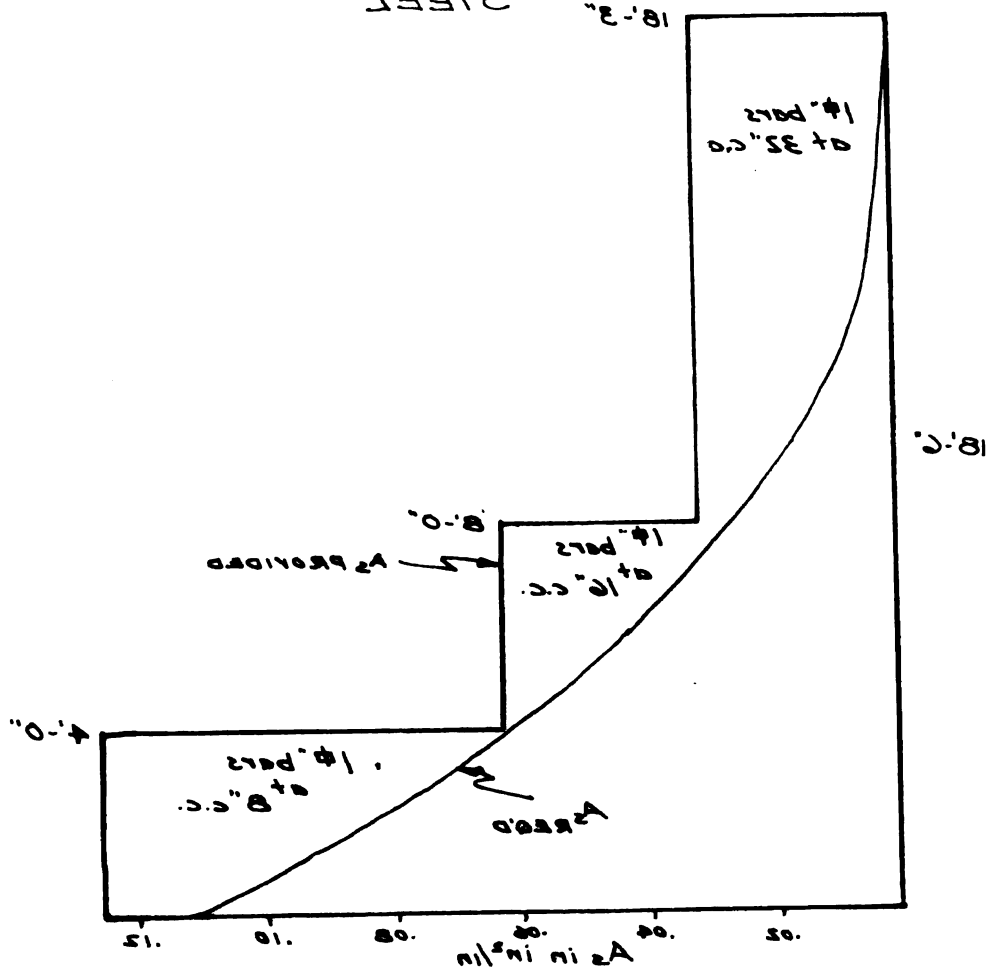
	Front of Stem	Back of Stem
Hor.	$\frac{1}{2}$ " at 12" 18 bars	$\frac{1}{2}$ " at 24" 9 bars
Vert.	$\frac{1}{2}$ " at 30"	

Cantilever Outside Wall.

Vert. $\frac{1}{2}$ " at 30"
 Hor. $\frac{1}{2}$ " at 15" 18 bars
 Front of stem
 Temperature steel:
 Back of stem
 Hooks Red'd.

Anchorage: $L = \frac{f_y d}{4 \times 100} = \frac{18,000 \times 1}{4 \times 100} = 45"$

Bond: $N = \frac{V}{\phi f_y} = \frac{6000}{4 \times 8 \times 51} = 81.2 \text{ psi}$



STEEL
 STEEL SCHEDULE FOR

FOOTING DESIGN

$$P' = \frac{w_h^2}{2} \cos \phi = \frac{1}{2} \cdot 70 \cdot (28)^2 \cdot .707 = 19,400^*$$

acting at $\frac{1}{3} \cdot 28 = 9.34'$ up.

P' combined graphically with fill weight = 10,130^{*} and wall weight = 8,660^{*} to give resultant base pressure in vertical direction = 32,400^{*}.

Resultant cuts base .5' from center of wall.

Toe & Heel pressures:

$$p = \frac{32,400}{12} \pm \frac{32,400 \cdot 6 \cdot .5}{144}$$

$$p_s = 3375 \text{ p.s.f.}$$

$$p_h = 2025 \text{ p.s.f.}$$

Heel Design:

$$\text{Max. Shear} = \text{weight of fill above heel} = V = 9440^*$$

$$v = \frac{V}{b_j d} = \frac{9440}{12 \cdot \frac{7}{8} \cdot 27} = 33.3 \text{ p.s.i. (allowable 40)}$$

$$\text{B.M.} = 9440 \cdot 3.1 = 29,300^*$$

$$A_s = \frac{M}{f_s j d} = \frac{29,300}{18,000 \cdot \frac{7}{8} \cdot 27} = .069 = \%$$

Use $\frac{3}{4}" \phi$ bars at 8" c-c gives .0703 = %

$$v = \frac{V}{2 \cdot b_j d} = \frac{9440}{4 \cdot \frac{7}{8} \cdot 27} = 99 \text{ p.s.i. (allowable 100)}$$

Cantilever Outside Wall

FOOTING DESIGN

$$P = \frac{W}{2} \cos \phi = \frac{1}{2} \cdot 70 \cdot (59)^{\circ} \cdot 707 = 12,400'$$

$$\text{acting at } \frac{1}{3} \cdot 58 = 2.34' \text{ up}$$

P, combined graphically with fill weight = 10,130' and wall weight = 8,660' to give resultant base pressure in vertical direction = 35,400'.
Resultant cuts base 2' from center of wall.

Toe & Heel pressures:

$$P = \frac{35,400}{15} \pm \frac{35,400 \cdot 2}{144}$$

$$P_t = 3372 \text{ p.s.f.}$$

$$P_h = 5052 \text{ p.s.f.}$$

Heel Design:

Max. shear = weight of fill above heel = V = 2440'

$$v = \frac{V}{b \cdot d} = \frac{2440}{15 \cdot 28 \cdot 5.5} = 33.3 \text{ p.s.i. (allowable 40)}$$

$$B.M. = 2440 \cdot 3.1 = 53,300'$$

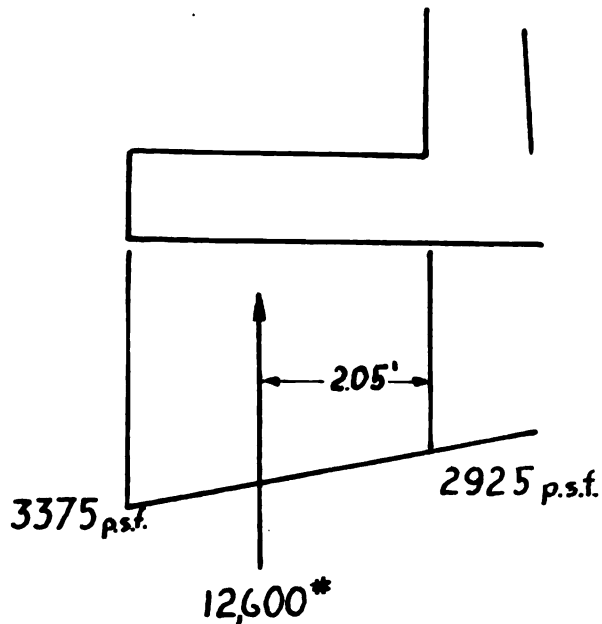
$$A_s = \frac{M}{f_y \cdot b \cdot d^2} = \frac{53,300}{18,000 \cdot 15 \cdot 28^2} = .069 = 1\%$$

Use $\frac{3}{4}" \phi$ bars at 8" c-c gives .0703 %

$$v = \frac{V}{b \cdot d} = \frac{2440}{15 \cdot 28 \cdot 5.5} = 33 \text{ p.s.i. (allowable 100)}$$

Counterforce Outside Wall

TOE DESIGN



Pressure on bottom of
toe = 12,600* = V

$$v = \frac{V}{b_j d} = \frac{12,600}{12 \cdot \frac{7}{8} \cdot 27} = 43.7 \text{ p.s.i.}$$

(allowable = 60 p.s.i.)

$$\text{B.M. at toe} = 12,600 \cdot 2.05' = 25,900' \cdot \text{p.s.f.}$$

$$d = \frac{V}{b_j v} = \frac{12,600}{12 \cdot \frac{7}{8} \cdot 60} = 20''$$

$$d = \sqrt{\frac{M}{b R}} = \sqrt{\frac{25,900}{140}} = 13.6'' \quad \text{Toe } 2\frac{1}{2}' \text{ thick}$$

$$A_s = \frac{M}{f_s j d} = \frac{25,900}{18,000 \cdot \frac{7}{8} \cdot 27} = .0693 \text{ " "}$$

Use $\frac{3}{4}$ " ϕ bars at 8" c-c gives .0703 " "

$$\text{Bond: } \mu = \frac{V}{\Sigma o_j d} = \frac{12,600}{4 \cdot \frac{7}{8} \cdot 27} = 133 \text{ p.s.i. Spec. Anch.}$$

Cantilever Outside Wall

Can't Over Outside Wall

$$\text{Bond: } \gamma = \frac{V}{b \cdot j} = \frac{15600}{4.8 \cdot .57} = 133 \text{ p.s.i. } 2 \text{ sec. Anch.}$$

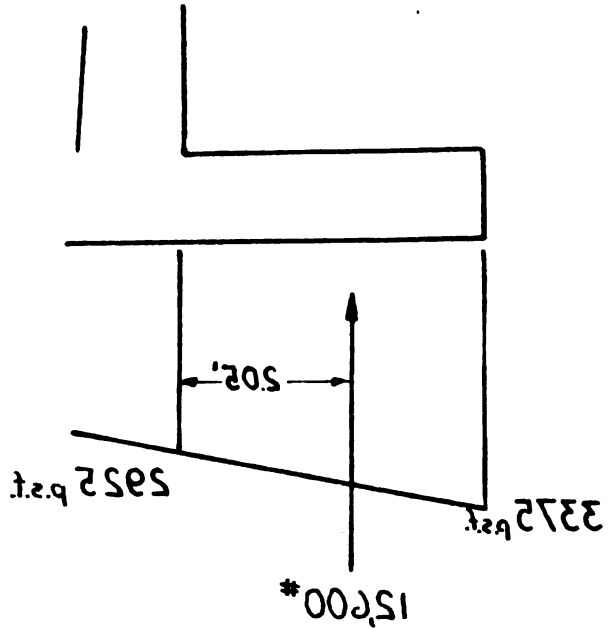
Use 3/4" ϕ bars at 8" c-c gives .0703 in²

$$A_s = \frac{M}{f_s j} = \frac{52200}{18000 \cdot .57} = .0623 \text{ in}^2$$

$$d = \sqrt{\frac{M}{P_R}} = \sqrt{\frac{52200}{140}} = 13.6" \quad \text{Toe 5 1/2" thick}$$

$$q = \frac{V}{P_j} = \frac{15600}{15.8 \cdot .60} = 50"$$

$$\text{B.M. at toe} = 15600 \cdot 5.02 = 52200 \text{ in}^2$$



$$\begin{aligned} & \text{Pressure on bottom of} \\ & \text{toe} = 15600 \text{ p.s.i.} \\ & \gamma = \frac{V}{P_j} = \frac{15600}{15.8 \cdot .57} = 133 \text{ p.s.i.} \\ & (\text{allowable} = 60 \text{ p.s.i.}) \end{aligned}$$

TOE DESIGN

CUT OFF WALL

$$\text{Passive Earth Pressure} = wh \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right) = wh(3.5)$$

At front of cut-off $wh = 2925 \text{ psf}$

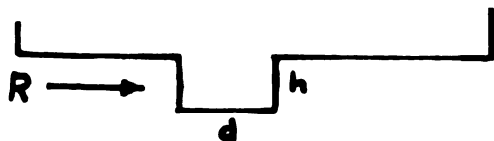
$$\text{Possible Resistance} = 2925 \cdot 3.5 = 10,250 \text{ psf}$$

$$\text{Make S.F.} = 2 \quad R = 2 \cdot 13,700 = .466 \cdot 32,400$$

$$R = 12,300$$

$$\text{Min. } h = \frac{12,300}{10,250} = 1.2' \text{ deep} \quad \text{Use } h = 1'3''$$

$$V = 10,250^*$$



$$d = \frac{10,250}{12 \cdot \frac{7}{8} \cdot 40} = 24.3'' \quad \text{Use } d = 2'$$

Overturn:

Resultant strikes in middle $\frac{1}{3}$ rd. therefor wall is safe.

Sliding:

$$P_h = .707 \cdot 19,400 = 13,700^*$$

$$\text{S.F.} = \frac{32,400 \cdot \tan 25^\circ}{13,700} = 1.10$$

Cut off wall as above needed

Cantilever Outside Wall

Cut off wall as above needed

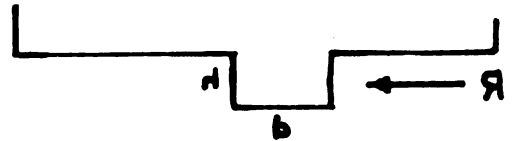
$$\frac{13,700}{32} = 428.125 \text{ ft} = 11.0$$

$$P_u = 107 \cdot 13,700 = 1,466,900 \text{ #}$$

Sliding:

Resultant strike in middle $\frac{1}{2}$ in. therefore wall is safe.
Overtun:

$$q = \frac{10,520}{15 \cdot 38 \cdot 40} = 5.4 \text{ " } 12 \text{ " } 1 = 5$$



$$V = 10,520 \text{ #}$$

$$\text{Min. } h = \frac{10,520}{15,300} = 1.5 \text{ ' } 9 \text{ " } U \text{ se } h = 1.3 \text{ "}$$

$$R = 15,300$$

$$\text{Make S.F.} = 5 \quad R = 5 \cdot 13,700 = 409,250$$

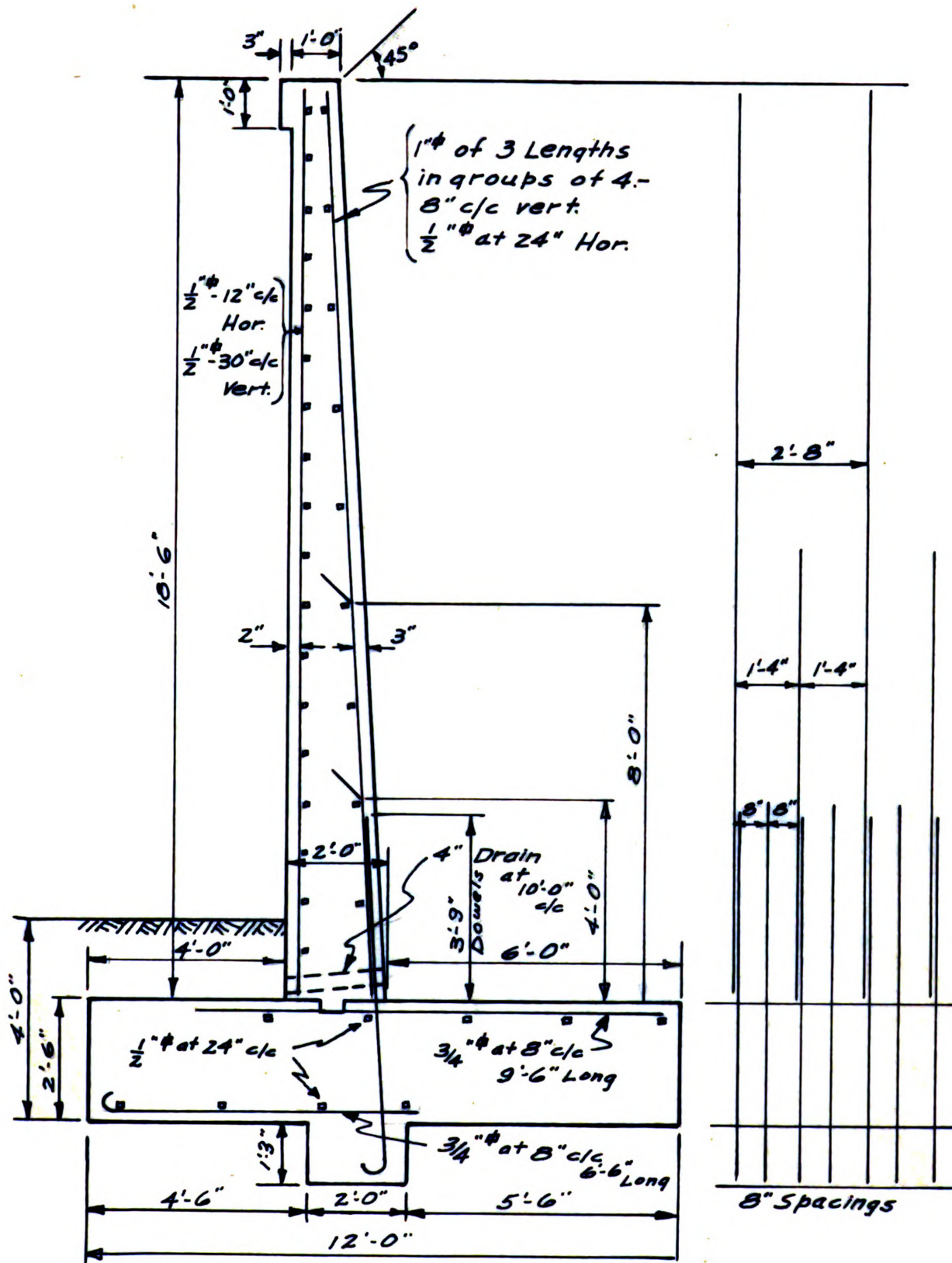
$$\text{Possible Resistance} = 5252 \cdot 3.2 = 10,520 \text{ #}$$

$$\text{At front of cut-off } w_h = 5252 \text{ #}$$

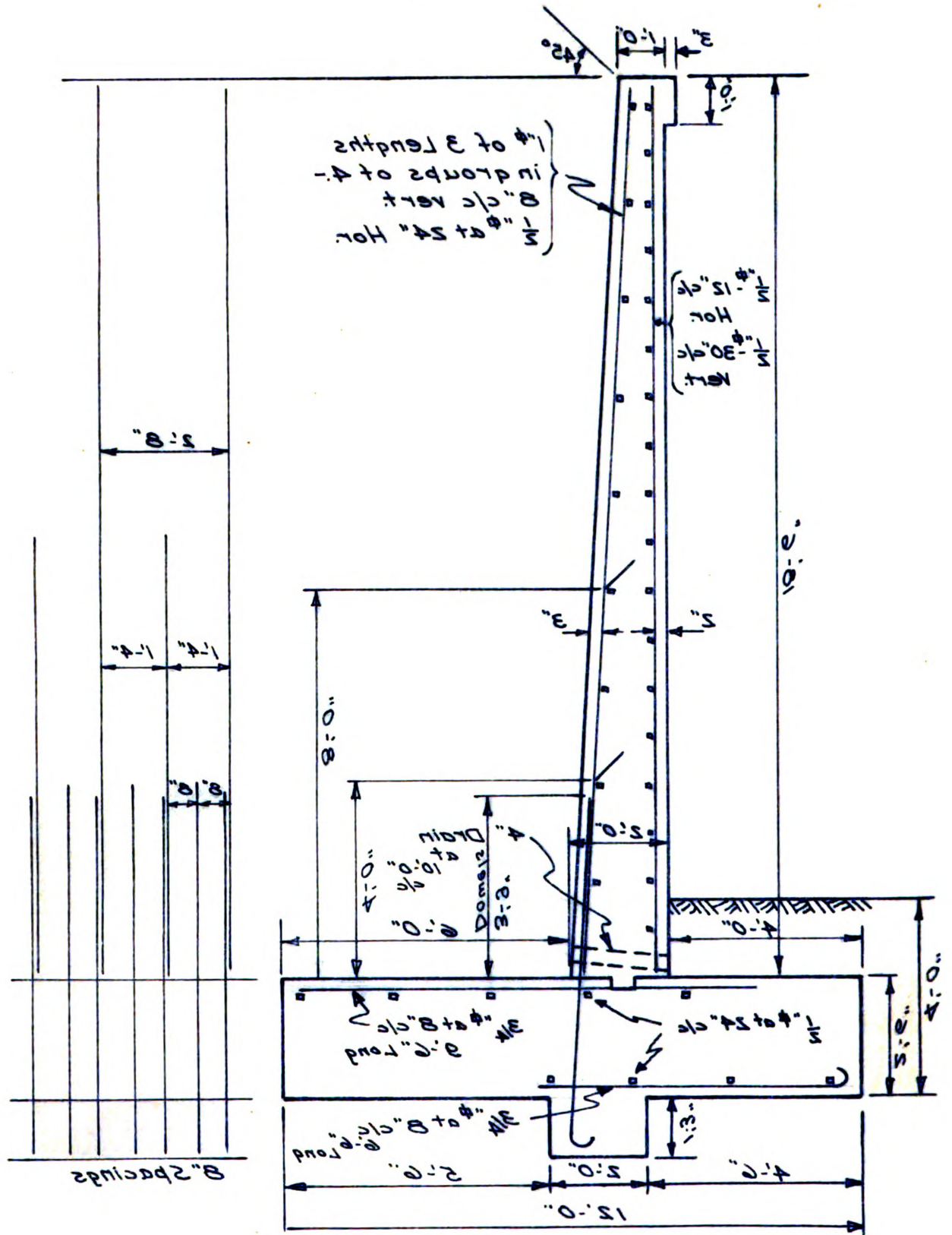
$$\text{Passive Earth Pressure} = w_h \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right) = w_h (3.2)$$

CUT OFF WALL

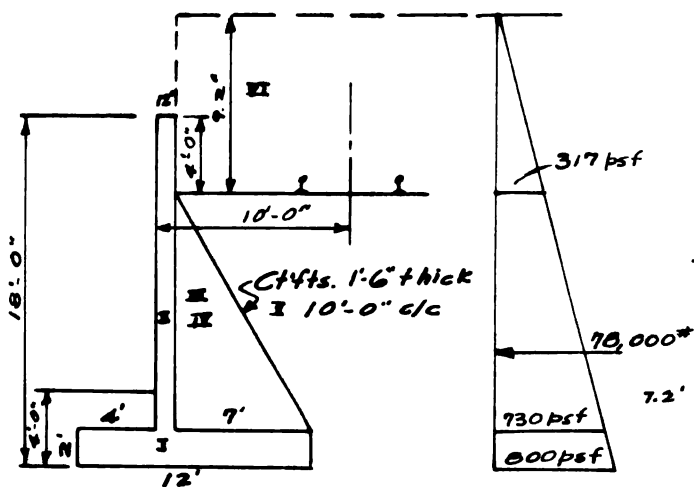
DETAILS OF CANTILEVER OUTSIDE WALL



DETAILS OF CANTILEVER OUTSIDE WALL



DESIGN OF VERTICAL SLAB



from case III (Cont.) $P = 7800^*$

$$7800 = \frac{1}{2} (p + 396p), p = 800 \text{ psf}$$

$$P \text{ for } 10' \text{ of wall} = 78,000^*$$

$$M_o = 78,000 \times 7.2 = 562,000^* \text{ ft.}$$

WT	W	arm	Mom.
I $2 \times 12 \times 150 \times 10$	36,000	6'	216,000
II $16 \times 1 \times 150 \times 10$	24,000	4.5'	108,000
III $7 \times 8.5 \times 14 \times 100$	95,000	8.5'	807,000
IV $7 \times \frac{1}{2} \times 1.5 \times 150$	11,000	7.33'	80,700
V $1.5 \times \frac{1}{2} \times 7 \times 100$	8,400	4.67'	81,100
VI $5.2 \times 7 \times 100 \times 10$	36,000	8.5'	306,000
	<u>210,400</u>		<u>1,538,800 = M_R</u>
			<u>562,000 = M_o</u>

Top and Heel Pressures:

$$P = \frac{2,120,000}{12 \times 10} \left(1 \pm \frac{6 \times 1.07}{12} \right)$$

$$P_t = 2709 \text{ psf}$$

$$P_h = 821 \text{ psf}$$

Overturb:

$$\frac{M_R}{M_o} = \frac{1,538,000}{562,000} = 2.85$$

Slide:

$$\frac{210,000 \times 4.66}{78,000} = 1.26 \text{ Must Anchor}$$

$$\text{Shear: } d = \frac{V}{b \cdot v} = \frac{3100}{12 \times \frac{7}{8} \times 40} = 7.4"$$

$$V = \frac{w \cdot l}{2} = 3100^*$$

Moment:

$$\text{Int. } M = \frac{1}{2} w l^2 = \frac{730 \times 8.5^2}{12} = 4400^* \text{ ft.}$$

$$\text{Ext. } M = \frac{1}{10} w l^2 = \frac{730 \times 8.5^2}{10} = 5270^* \text{ ft.}$$

$$d = \sqrt{\frac{M}{R_b}} \quad d = \sqrt{\frac{4400}{140}} = 5.6"$$

$$d = \sqrt{\frac{5270}{140}} = 6.14"$$

Steel:

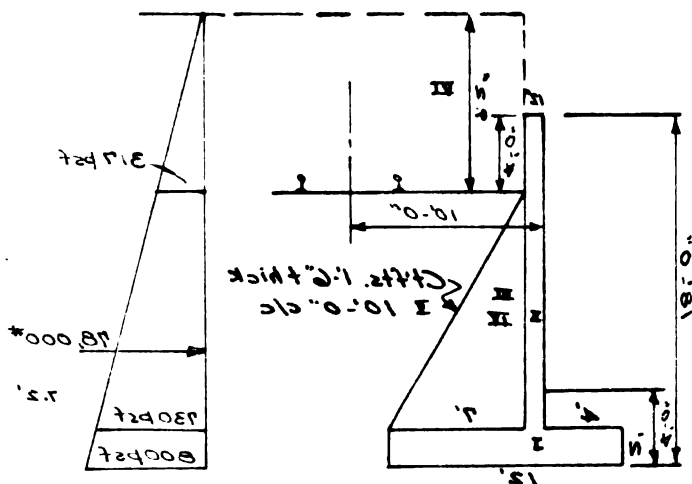
$$A_s = \frac{M}{f_s \cdot j \cdot d}$$

$$\text{Int. } A_s = \frac{4400}{18,000 \times \frac{7}{8} \times 9} = .0311^* \text{ in}$$

$$\text{Ext. } A_s = \frac{5270}{18,000 \times \frac{7}{8} \times 9} = .0418^* \text{ in}$$

DESIGN OF VERTICAL SLAB

$\frac{1}{2} \times 1000000 = 500000$
 $500000 \times 2.5 = 1250000$
 $1250000 \times 1.1 = 1375000$
 $1375000 \times 1.05 = 1443750$
 $1443750 \times 1.05 = 1515937.5$
 $1515937.5 \times 1.05 = 1591734.375$
 $1591734.375 \times 1.05 = 1671321.09375$
 $1671321.09375 \times 1.05 = 1754887.1484375$
 $1754887.1484375 \times 1.05 = 1842631.505859375$
 $1842631.505859375 \times 1.05 = 1934763.08115234375$
 $1934763.08115234375 \times 1.05 = 2031501.235210061$
 $2031501.235210061 \times 1.05 = 2133076.296970564$
 $2133076.296970564 \times 1.05 = 2239730.111819092$
 $2239730.111819092 \times 1.05 = 2351716.617410047$
 $2351716.617410047 \times 1.05 = 2469302.448280549$
 $2469302.448280549 \times 1.05 = 2592767.570694577$
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 $4433365.779562173 \times 1.05 = 4653034.068540281$
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 $5131970.06056566 \times 1.05 = 5388568.563593943$
 $5388568.563593943 \times 1.05 = 5657996.99177263$
 $5657996.99177263 \times 1.05 = 5940896.841361262$
 $5940896.841361262 \times 1.05 = 6237941.683429325$
 $6237941.683429325 \times 1.05 = 6550838.767600791$
 $6550838.767600791 \times 1.05 = 6878380.706080831$
 $6878380.706080831 \times 1.05 = 7222300.741384872$
 $7222300.741384872 \times 1.05 = 7583415.778454116$
 $7583415.778454116 \times 1.05 = 7992586.567376822$
 $7992586.567376822 \times 1.05 = 8432215.895745663$
 $8432215.895745663 \times 1.05 = 8893826.690532946$
 $8893826.690532946 \times 1.05 = 9378518.025059593$
 $9378518.025059593 \times 1.05 = 9847443.926312572$
 $9847443.926312572 \times 1.05 = 10349816.1226282$
 $10349816.1226282 \times 1.05 = 10887306.92875961$
 $10887306.92875961 \times 1.05 = 11461672.27519759$
 $11461672.27519759 \times 1.05 = 12074755.88895747$
 $12074755.88895747 \times 1.05 = 12728493.68340534$
 $12728493.68340534 \times 1.05 = 13424918.36757561$
 $13424918.36757561 \times 1.05 = 14166164.28605439$
 $14166164.28605439 \times 1.05 = 14954572.50035711$
 $14954572.50035711 \times 1.05 = 15792301.12537497$
 $15792301.12537497 \times 1.05 = 16681916.18164371$
 $16681916.18164371 \times 1.05 = 17626012.0907259$
 $17626012.0907259 \times 1.05 = 18627312.6952622$
 $18627312.6952622 \times 1.05 = 19688678.33002531$
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 $27316479.79348482 \times 1.05 = 28882303.78315906$
 $28882303.78315906 \times 1.05 = 30506419.07231701$
 $30506419.07231701 \times 1.05 = 32191740.02593286$
 $32191740.02593286 \times 1.05 = 33941327.0272195$
 $33941327.0272195 \times 1.05 = 35758393.37858048$
 $35758393.37858048 \times 1.05 = 37646312.04750951$
 $37646312.04750951 \times 1.05 = 39608627.64988498$
 $39608627.64988498 \times 1.05 = 41649059.03237923$
 $41649059.03237923 \times 1.05 = 43771511.98399819$
 $43771511.98399819 \times 1.05 = 45980087.5831981$
 $45980087.5831981 \times 1.05 = 48279092.06135791$
 $48279092.06135791 \times 1.05 = 50673046.66442581$
 $50673046.66442581 \times 1.05 = 53156709.00764709$
 $53156709.00764709 \times 1.05 = 55734544.45802945$
 $55734544.45802945 \times 1.05 = 58401271.68093092$
 $58401271.68093092 \times 1.05 = 61161335.26497747$
 $61161335.26497747 \times 1.05 = 64009402.02822734$
 $64009402.02822734 \times 1.05 = 66949872.12963921$
 $66949872.12963921 \times 1.05 = 70087365.73612117$
 $70087365.73612117 \times 1.05 =$



$ \begin{array}{r} 1.00M \\ 000,015 \\ 000,001 \\ 000,500 \\ 005,00 \\ 001,10 \\ \hline 300,003 \\ 1.00M = 000,002,1 \\ 0.00M = 000,002 \end{array} $	$ \begin{array}{r} 0.70 \\ 0 \\ 0.2 \\ 0.0 \\ 0.03 \\ 0.01 \\ 0.2 \\ \hline 0.2 \end{array} $	$ \begin{array}{r} W \\ 000,00 \\ 000,05 \\ 000,20 \\ 000,11 \\ 004,00 \\ \hline 000,00 \\ 004,00 \end{array} $	$ \begin{array}{r} TW \\ 5 \times 15 \times 10 \\ 1 \times 120 \times 10 \\ 1 \times 41 \times 2.8 \times 100 \\ 2 \times 1.2 \times \frac{14}{2} \\ 1 \times 2.1 \times \frac{14}{2} \\ 2.5 \times 7 \times 5.2 \times 10 \end{array} $
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$$p = \frac{5,000}{15,410} (1 + \frac{0.10}{12})^{12}$$

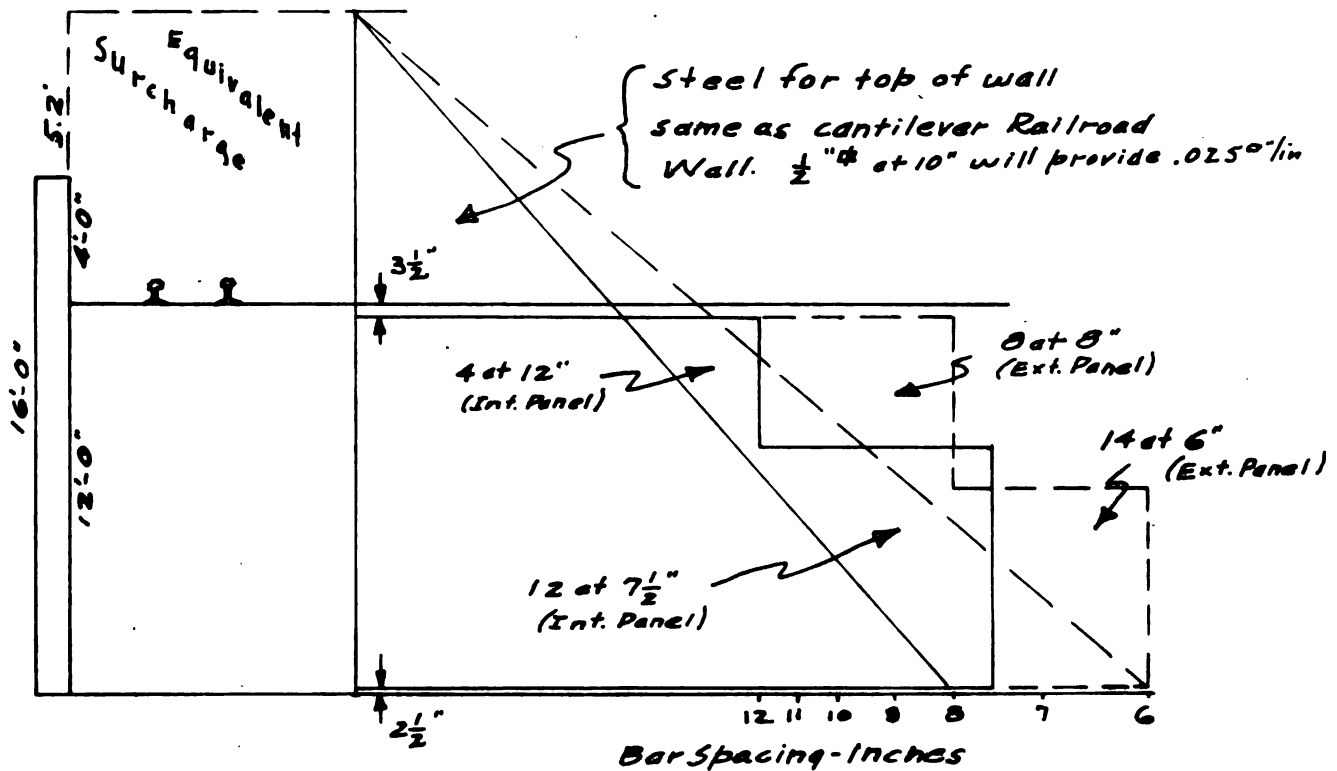
$$p = 1.01$$

$10.1 = 5$
 $28.5 = \frac{000892.1}{500202} = \frac{8M}{0M}$

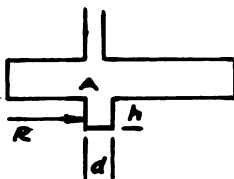
$\therefore \frac{200}{15} = 13.33$

Moment:
 Int. $M = \frac{1}{15} w l^2 = \frac{2.0 \times 30 \times 30}{15} = 120 \text{ ft}$
 Ext. $M = \frac{1}{30} w l^2 = \frac{2.0 \times 30 \times 30}{30} = 60 \text{ ft}$
 $q = \sqrt{\frac{M}{50}}$
 Int. $A^2 = \frac{M}{120} = 1$
 Int. $A^2 = \frac{120000 \times 1}{120000 \times 1} = 0.311 \text{ in}^2$
 Ext. $A^2 = \frac{6000}{180000 \times 1} = 0.0418 \text{ in}^2$
 $q = \sqrt{\frac{140}{2550}} = 0.14$
 $q = \sqrt{\frac{140}{440}} = 0.2$

DESIGN OF STEEL IN VERTICAL SLAB.



DESIGN OF CUT-OFF WALL



$$\text{Passive Earth Pressure} = \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right) wh = 3.5wh$$

$$\text{At A, } wh = 2180 \text{ psf}$$

$$\text{possible resistance} = 3.5 \times 2180 = 7630 \text{ psf}$$

$$\text{If S.F.} = 2 \quad R = 2 \times 7630 - 466 \times 21,040$$

$$R = 15,600 - 9770$$

$$R = 5830^*$$

$$h = \frac{5830}{7630} = .764$$

$$\text{make } h = 1'-0"$$

for Shear:

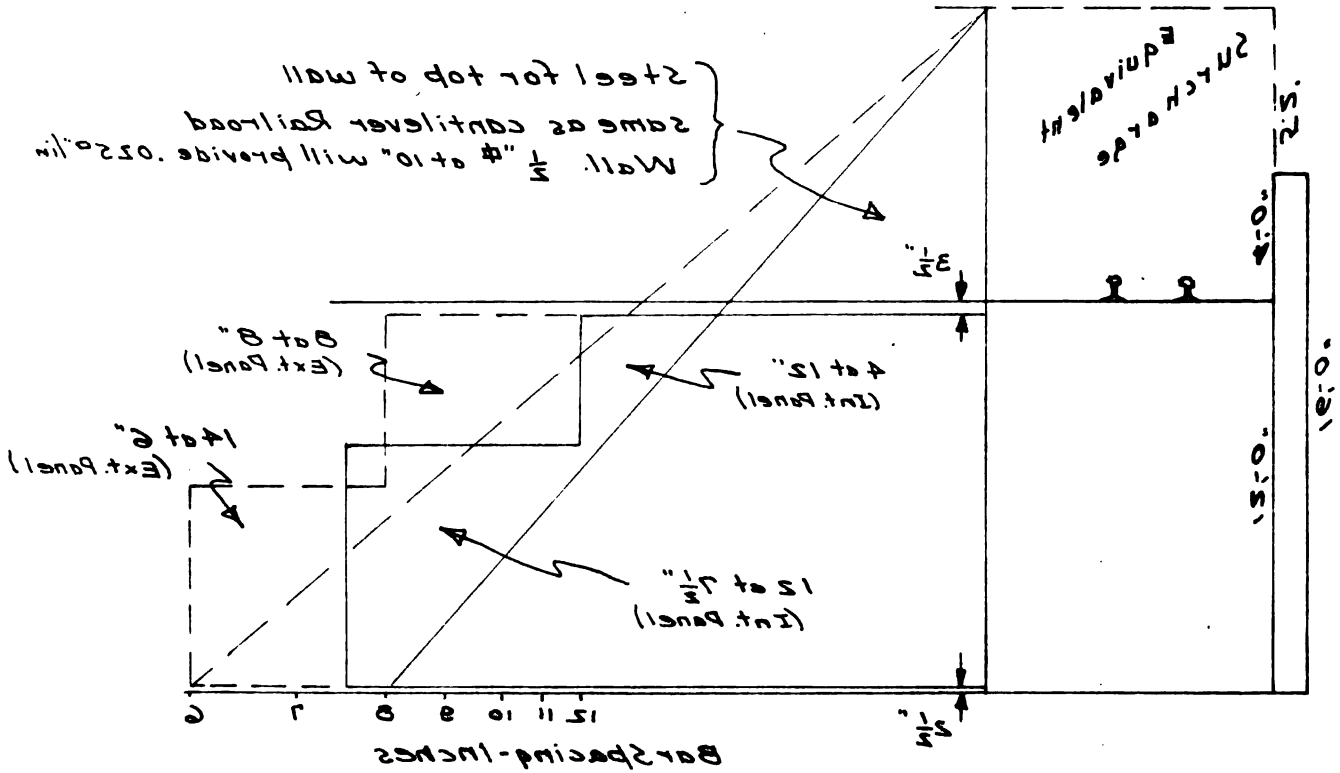
$$d = \frac{V}{b j v} = \frac{5830}{12 \times \frac{2}{3} \times 40} = 13.9"$$

$$d = \sqrt{\frac{6M}{b f}} = \sqrt{\frac{36 \times 5830}{12 \times 60}} = \sqrt{292}$$

$$d = 17.1"$$

$$\text{make } d = 18"$$

DESIGN OF STEEL IN VERTICAL SLAB.



DESIGN OF CUT-OFF WALL

Positive Earth Pressure = $(1.2 \sin \phi) w h = 3.2 w h$

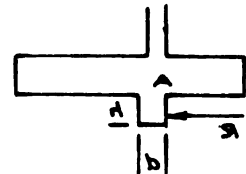
At A, $w h = 5180 \text{ psf}$

Possible resistance = $3.2 \times 5180 = 16576 \text{ psf}$

If 2.F. = 5 $R = 5 \times 16576 = 82880$

$R = 12,000 - 3720$

$R = 2830$



for shear:

$V = \frac{2830}{1.0} = 2830$

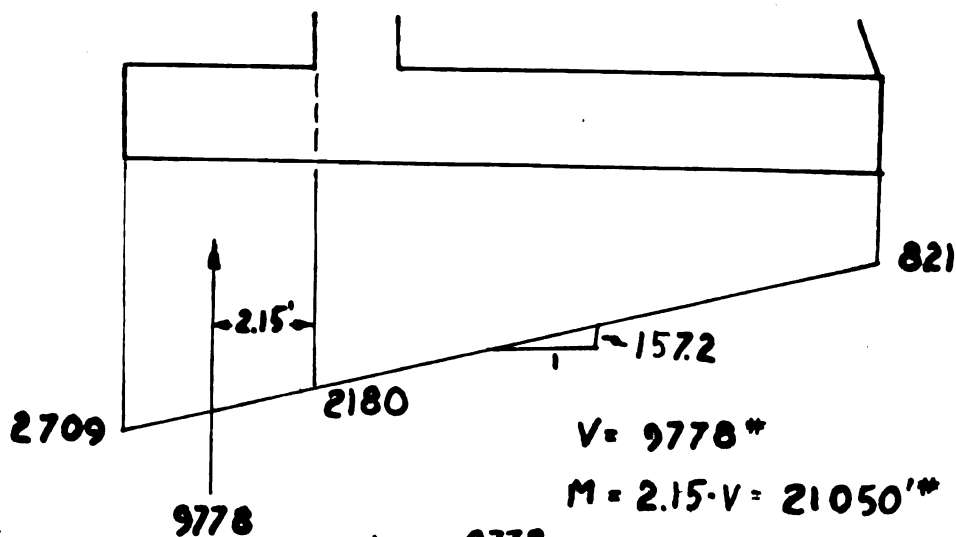
Make $V = 1.0$

$q = \frac{V}{p \times d} = \frac{2830}{15 \times \frac{1}{2} \times 40} = 13.4$

$q = \sqrt{\frac{C \times 2830}{15 \times 60}} = \sqrt{\frac{30 \times 2830}{900}} = \sqrt{95.6} = 9.77$

Make $q = 18$

TOE DESIGN



$$V = 9778^*$$

$$M = 2.15 \cdot V = 21050^*$$

$$d = \frac{V}{b_j c} = \frac{9778}{12 \cdot \frac{7}{8} \cdot 60} = 15.6"$$

$$d = \sqrt{\frac{M}{R b}} = \sqrt{\frac{21050}{140}} = 12.3" \quad A_s = \frac{M}{f_s j d} = \frac{21050}{18000 \cdot \frac{7}{8} \cdot 21} = .0635\%$$

Use 1" ϕ at 12" c-c gives .0655%

HEEL DESIGN

Weight of earth + surcharge above 1' section end of heel = 2420*

$$V = \frac{1}{2} w l = \frac{1}{2} \cdot 1520 \cdot 8.5 = 6460^*$$

$$M = \frac{1}{2} w l^2 = \frac{1}{2} \cdot 1520 \cdot (8.5)^2 = 9150^*$$

$$M = \frac{1}{10} w l^2 = \frac{1}{10} \cdot 1520 \cdot (8.5)^2 = 11000^*$$

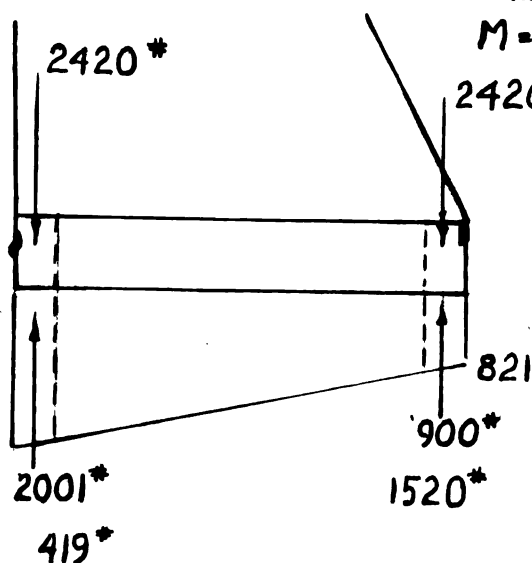
$$d = \sqrt{\frac{11000}{140}} = 8.1"$$

$$d = \frac{V}{b_j c} = \frac{6460}{12 \cdot \frac{7}{8} \cdot 40} = 15.4"$$

Keep heel 24" thick.

$$A_s = \frac{9150}{18000 \cdot \frac{7}{8} \cdot 21} = .0236\% \text{ Int.}$$

$$A_s = \frac{11000}{18000 \cdot \frac{7}{8} \cdot 21} = .0332\% \text{ Ext.}$$



Counterfort Railroad Wall

$$A_2 = \frac{11000}{18000 \cdot 8 \cdot 51} = .0335 \text{ Int.}$$

$$A_2 = \frac{2120}{18000 \cdot 8 \cdot 51} = .0216 \text{ Int.}$$

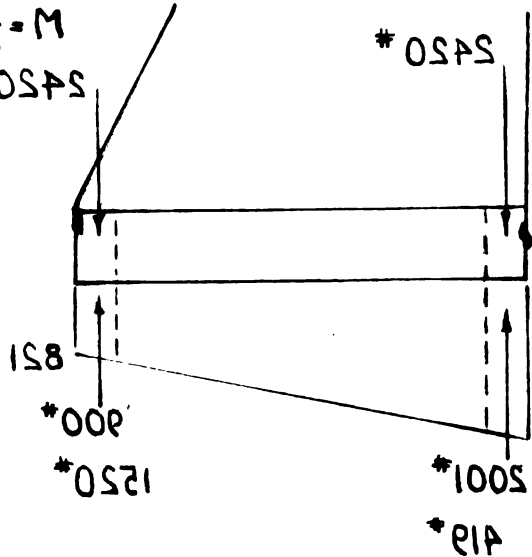
Keep heel 24" thick.

$$q = \frac{V}{P \cdot V} = \frac{6460}{15 \cdot 8 \cdot 40} = 12.4$$

$$q = \sqrt{\frac{11000}{140}} = 8.1$$

$$M = \frac{1}{10} w l^2 = \frac{1}{10} (2.8) \cdot 1250 \cdot (8.2)^2 = 11000$$

$$M = \frac{1}{15} w l^2 = \frac{1}{15} (2.8) \cdot 1250 \cdot (8.2)^2 = 9120$$



Weight of earth + surcharge above 1' section end of heel = 2450
 $V = \frac{1}{2} w l = \frac{1}{2} \cdot 2.8 \cdot 1250 \cdot 8.2 = 6460$

HEEL DESIGN

Use 1" of 15" c-c gives .0216

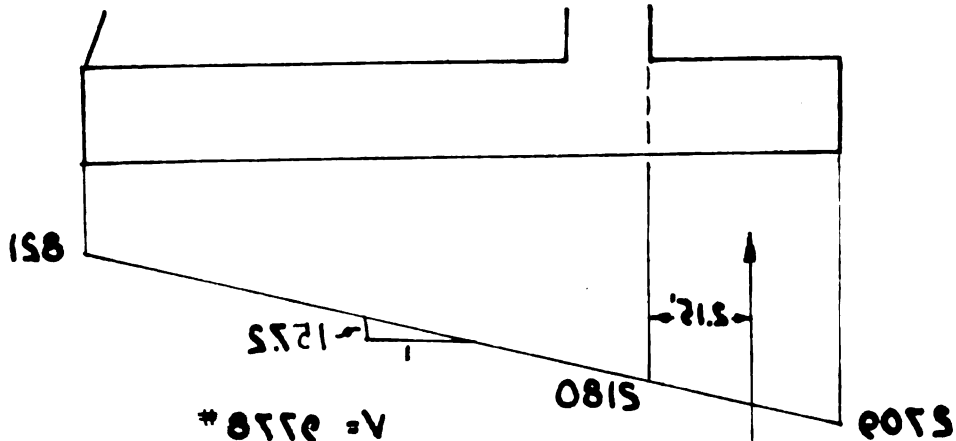
$$q = \sqrt{\frac{M}{R \cdot P}} = \sqrt{\frac{9120}{140}} = 15.3$$

$$A_2 = \frac{M}{f^2 \cdot P} = \frac{9120}{18000 \cdot 8 \cdot 51} = .0216$$

$$q = \frac{V}{P \cdot V} = \frac{6460}{15 \cdot 8 \cdot 40} = 12.4$$

$$M = 5.12 \cdot V = 51020$$

$$V = 9778$$



TOE DESIGN

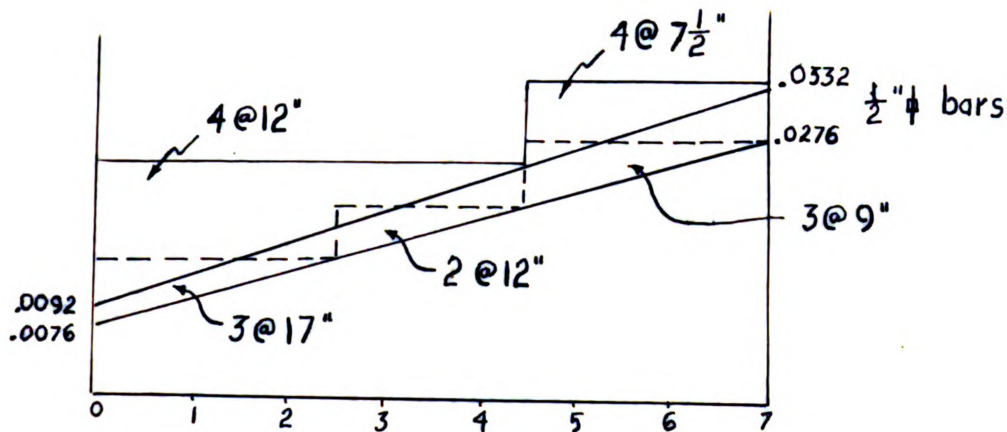
HEEL DESIGN

$$M = \frac{1}{12} 419 \cdot (8.5)^2 = 2520 \text{ 'ft Int.} \quad M = \frac{1}{10} 419 \cdot (8.5)^2 = 3020 \text{ 'ft Ext.}$$

$$V = \frac{1}{2} \cdot 915 \cdot 8.5 = 1890 \text{ 'ft}$$

$$d = \frac{1890}{12 \cdot \frac{7}{8} \cdot 40} = 4.5"$$

$$A_s = \frac{2520}{18000 \cdot \frac{7}{8} \cdot 21} = .0076 \% \text{ Int.} \quad A_s = \frac{3020}{18000 \cdot \frac{7}{8} \cdot 21} = .0092 \% \text{ Ext.}$$



TENSION STEEL

$$\text{Heel } 6460 \cdot 2 = 12,920 \text{ 'ft}$$

$$A_s = 12,920 \div 18,000 = .718 \%$$

$$A_b = \frac{.718 \cdot 9}{2 \cdot 12} = .269 \%/ \text{bar}$$

$$L = \frac{18000 \cdot 5}{4 \cdot 100 \cdot 8} = 29" \text{ long}$$

Hook $\frac{5}{8}$ " ϕ bar every one

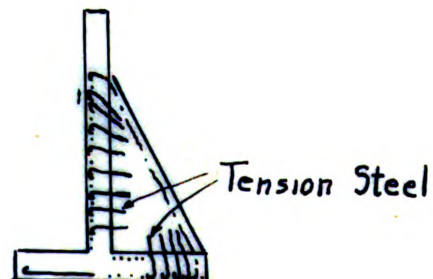
$$\text{Vertical Slab } 730 \cdot 8.5 = 7060$$

$$A_s = 7060 \div 18000 = .392 \%$$

$$A_b = \frac{.392 \cdot 8}{2 \cdot 12} = .131$$

$$L = 18000 \cdot 5 \div 400 = 22.5"$$

Hook $\frac{5}{8}$ " ϕ bar every other one



Counterfort Railroad Wall

HEEL DESIGN

$$M = \frac{1}{15} \cdot 410 \cdot (8.2)^2 = 5250 \text{ in}^2 \cdot \text{ft}$$

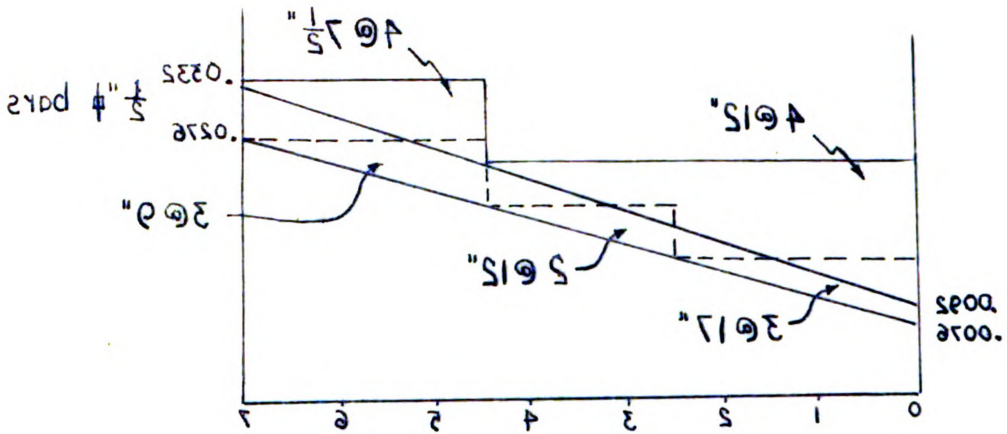
$$V = \frac{1}{2} \cdot 212 \cdot 8.2 = 1820 \text{ #}$$

$$A_2 = \frac{5250}{18000 \cdot \frac{1}{8} \cdot 51} = .0026 \text{ in}^2 \cdot \text{ft}$$

$$A_2 = \frac{18000 \cdot \frac{1}{8} \cdot 51}{3050} = .0025 \text{ in}^2 \cdot \text{ft}$$

$$q = \frac{1820}{15 \cdot \frac{1}{8} \cdot 40} = 4.2 \text{ in}$$

$$M = \frac{1}{10} \cdot 410 \cdot (8.2)^2 = 3050 \text{ in}^2 \cdot \text{ft}$$



TENSION STEEL

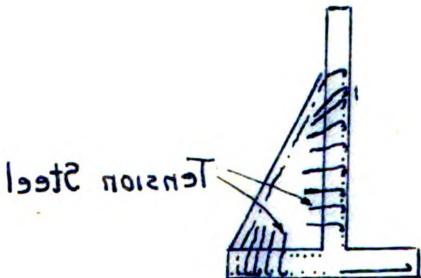
HEEL $6400 \cdot 5 = 15250 \text{ #}$

$$A_2 = 15250 \div 18000 = .718 \text{ in}^2$$

$$A_2 = \frac{.718 \cdot 2}{5 \cdot 15} = .259 \text{ in}^2 \cdot \text{par}$$

$$f = \frac{4 \cdot 100 \cdot 8}{18000 \cdot 2} = 50 \text{ in}^2 \cdot \text{load}$$

Hook $\frac{7}{8}$ in ϕ bar every one



Vertical 2/par $720 \cdot 8.2 = 2060$

$$A_2 = 2060 \div 18000 = .325 \text{ in}^2$$

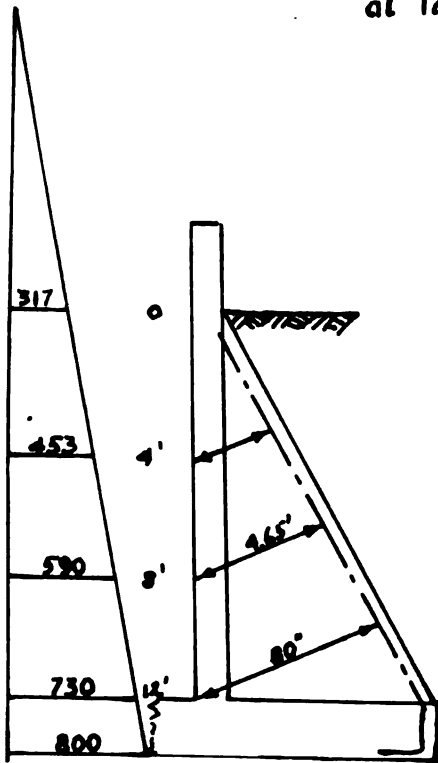
$$A_2 = \frac{.325 \cdot 8}{5 \cdot 15} = .131$$

$$f = 18000 \cdot 2 \div 400 = 55.2$$

Hook $\frac{7}{8}$ in ϕ bar every other one

Counterfort Railroad Wall

COUNTERFORT DESIGN



at 12' $V = \frac{1}{2}(730 + 317) \cdot 12 \cdot 10 = 62,800^*$

$M = 62,800 \cdot 5.2 = 327,300^*$

$A_s = \frac{327,300 \cdot 12}{18,000 \cdot \frac{7}{8} \cdot 80} = 3.12^{\text{in}}$

Use 4 1" ϕ bars

$\mu = \frac{62,800}{16 \cdot \frac{7}{8} \cdot 80} = 56 \text{ p.s.i.}$

at 8'

$V = \frac{1}{2}(590 + 317) \cdot 8 \cdot 10 = 36,280^*$

$M = 36,280 \cdot 3.61 = 131,000^*$

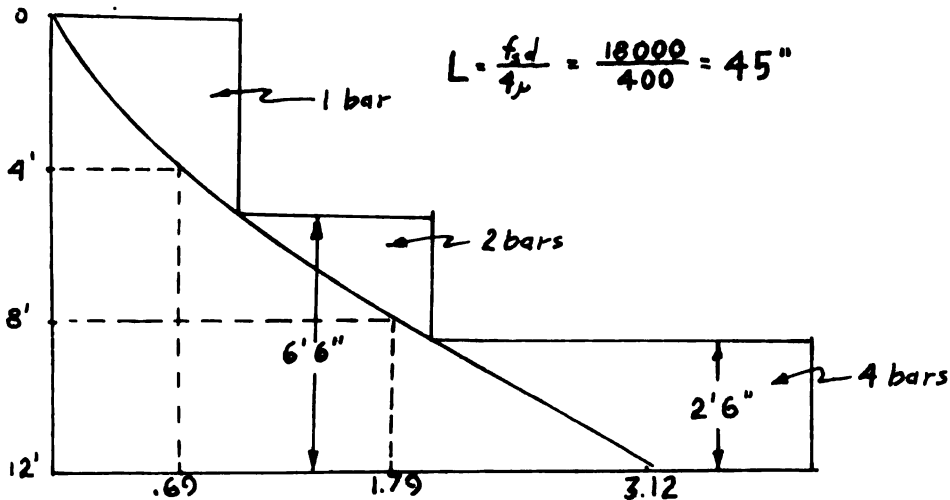
$A_s = \frac{131,000}{18,000 \cdot \frac{7}{8} \cdot 4.65} = 1.79^{\text{in}}$

at 4'

$V = \frac{1}{2}(453 + 317) \cdot 4 \cdot 10 = 15,400^*$

$M = 15,400 \cdot 1.84 = 28,300^*$

$A_s = \frac{28,300}{18,000 \cdot \frac{7}{8} \cdot 2.6} = .69^{\text{in}}$



$L = \frac{f_y d}{4\mu} = \frac{18,000}{400} = 45^{\text{in}}$

Use 1" ϕ bars cut off as shown.

Counterfort Railroad Wall

COUNTERFORT DESIGN

at 15'

$$V = \frac{1}{2} (230 + 317) \cdot 15 \cdot 10 = 65800 \text{ lb}$$

$$M = 65800 \cdot 7.5 = 357300 \text{ lb-ft}$$

$$A_2 = \frac{357300}{18000 \cdot \frac{1}{8} \cdot 80} = 3.15 \text{ in}^2$$

Use 4 # bars

$$r = \frac{65800}{16 \cdot \frac{1}{8} \cdot 80} = 26.9 \text{ in}$$

at 8'

$$V = \frac{1}{2} (20 + 317) \cdot 8 \cdot 10 = 26580 \text{ lb}$$

$$M = 26580 \cdot 3.17 = 131000 \text{ lb-ft}$$

$$A_2 = \frac{131000}{18000 \cdot \frac{1}{8} \cdot 4.02} = 1.79 \text{ in}^2$$

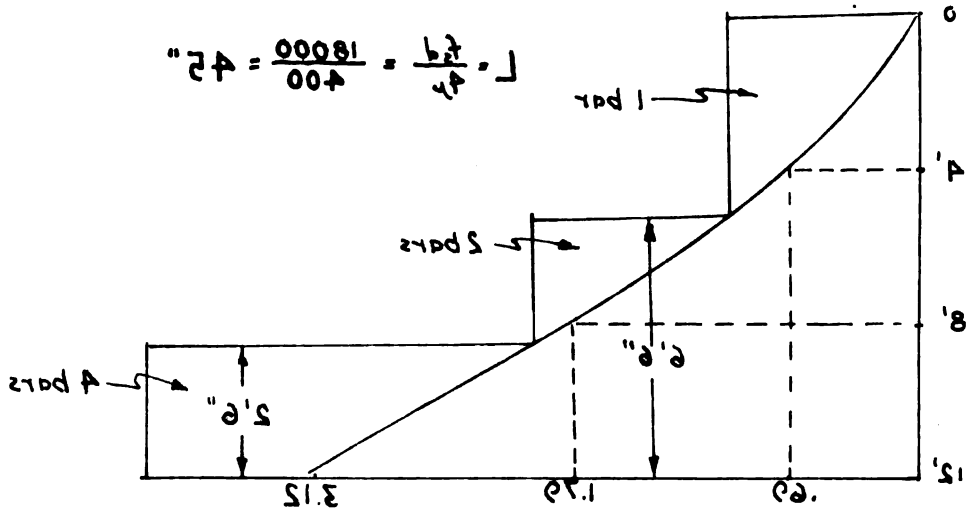
at 4'

$$V = \frac{1}{2} (423 + 317) \cdot 4 \cdot 10 = 12400 \text{ lb}$$

$$M = 12400 \cdot 1.84 = 58300 \text{ lb-ft}$$

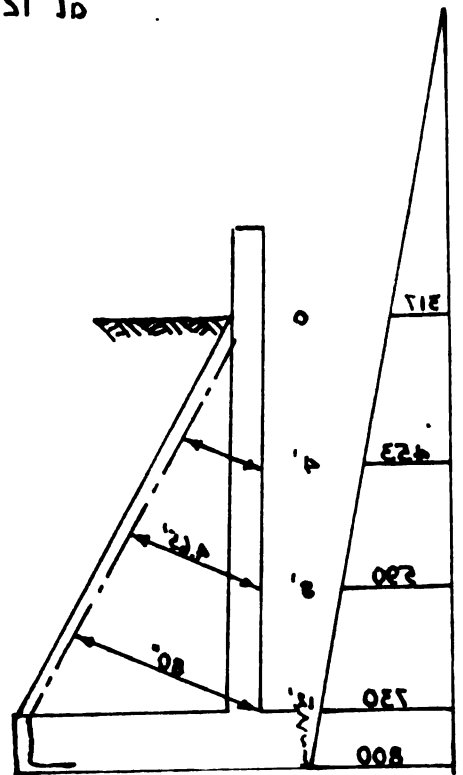
$$A_2 = \frac{58300}{18000 \cdot \frac{1}{8} \cdot 5.2} = .92 \text{ in}^2$$

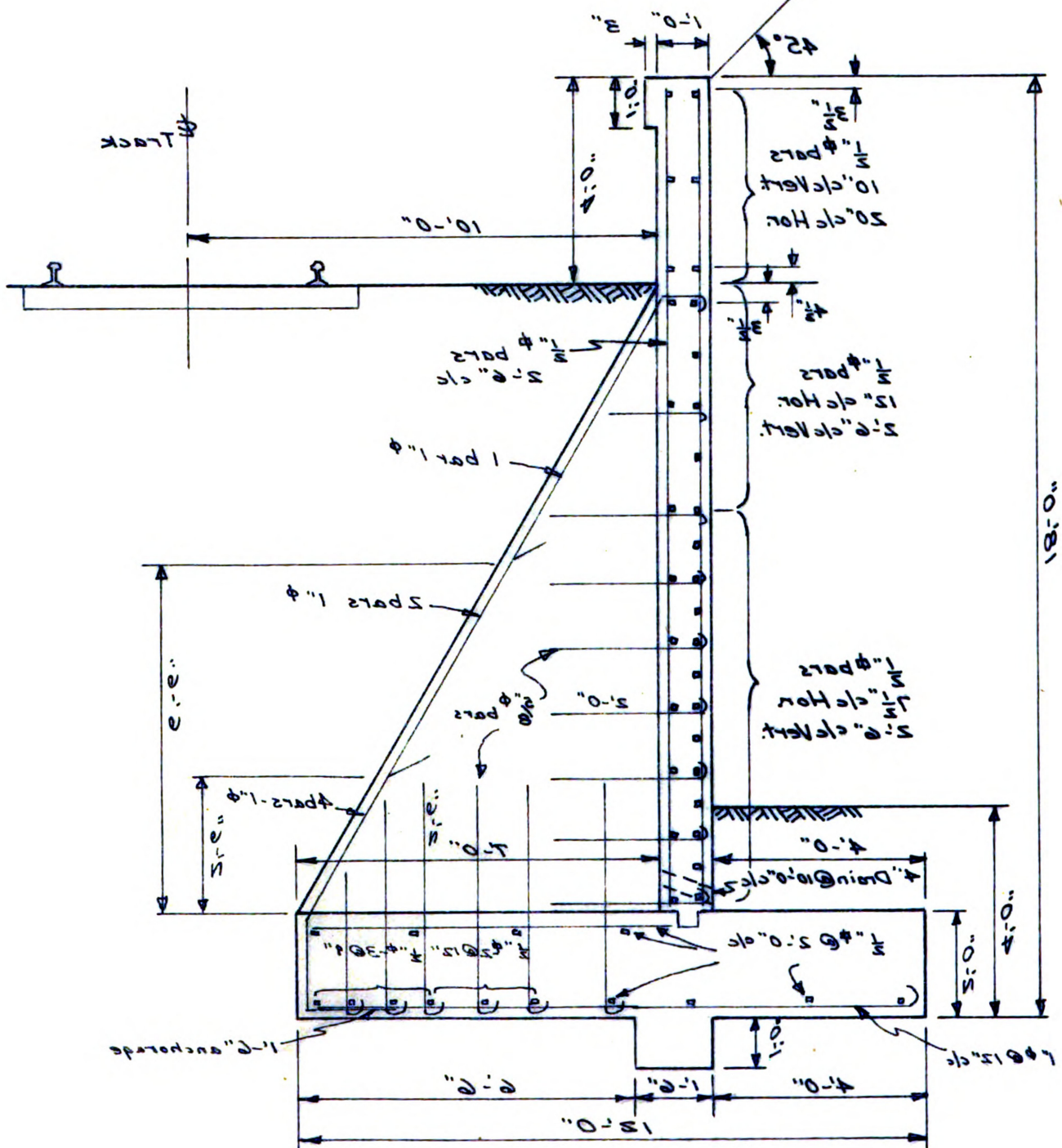
$$r = \frac{58300}{400 \cdot \frac{1}{4} \cdot 4.0} = 42 \text{ in}$$



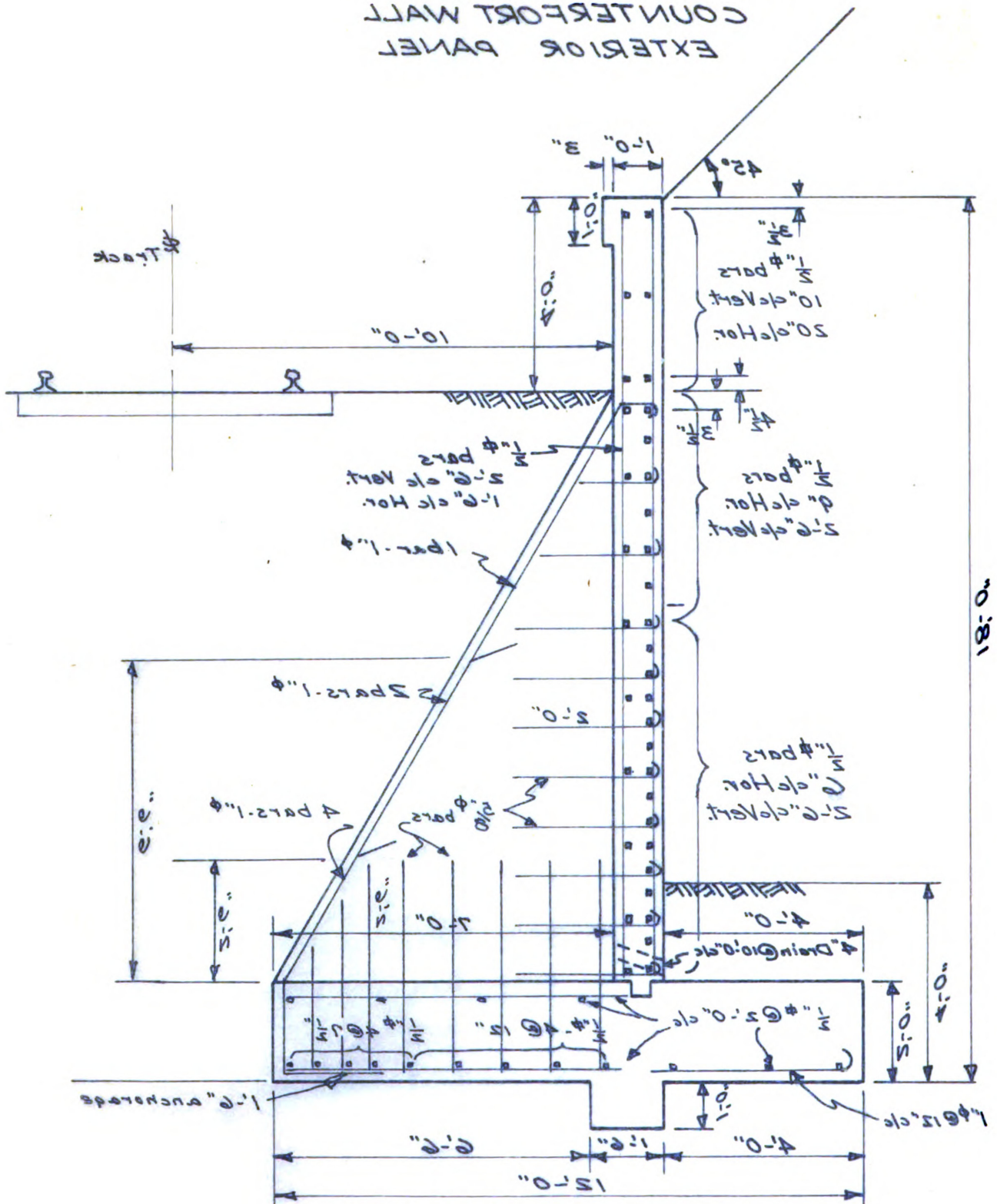
Use 1" bars cut off as shown

Counterfort Railroad Wall





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DESIGN OF VERTICAL SLAB (CONT.)

For Moment: $d = \sqrt{\frac{M}{R_b}}$ Int. Panel, $d = \sqrt{\frac{74,100}{140 \times 12}} = 6.65"$

Ext. Panel $d = \sqrt{\frac{89,000}{140 \times 12}} = 7.28"$

For Shear: $d = \frac{V}{b_j v} = \frac{4120}{12 \times \frac{7}{8} \times 60} = 6.54"$ (Spec. Anch. req'd)

Steel Req'd: $A_s = \frac{M}{f_s j d}$

Int. Panel

Ext. Panel

at 18.5' $A_s = .0436 \text{ in.}^2$

$A_s = .0524 \text{ in.}^2$

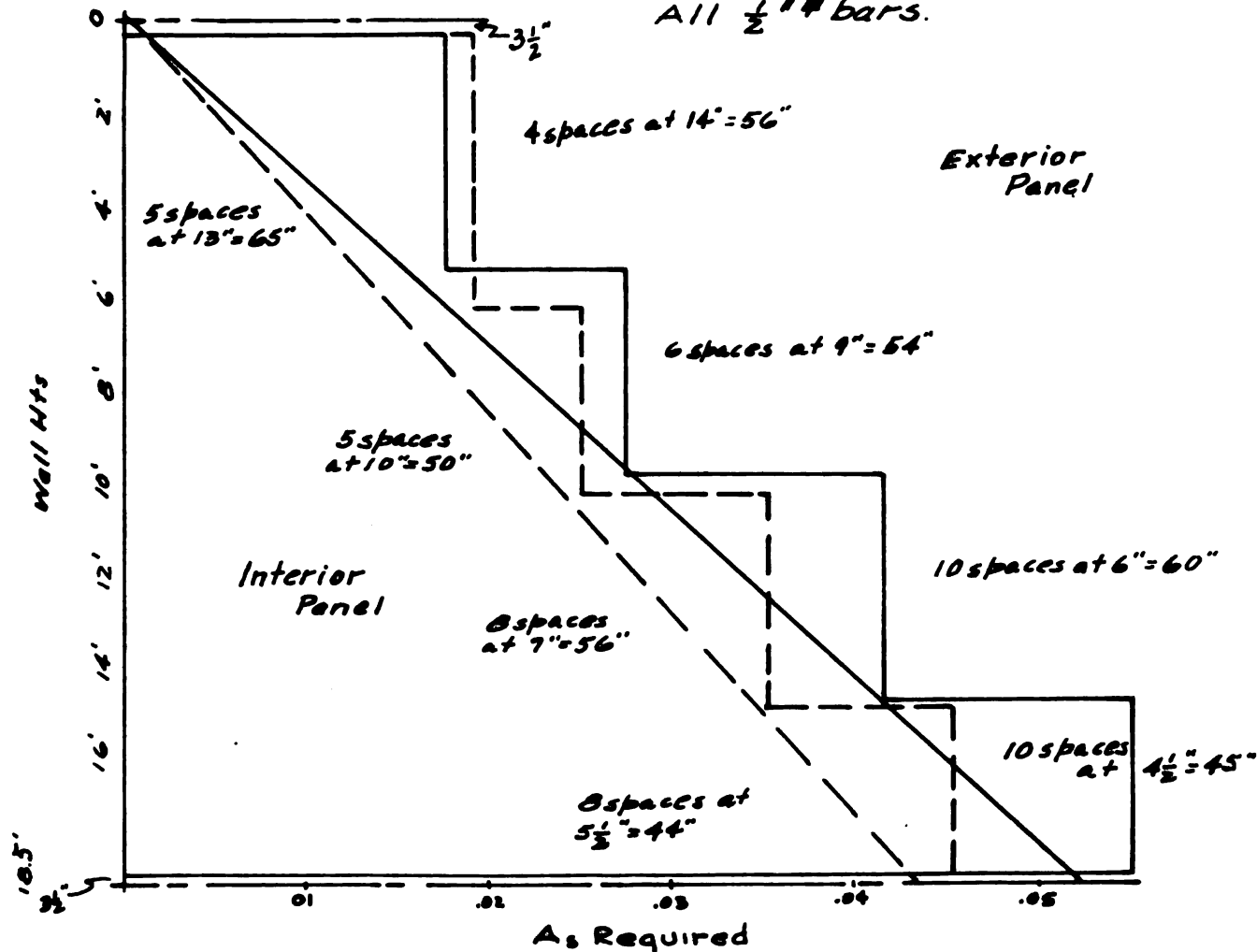
at 12' $A_s = .0283 \text{ in.}^2$

$A_s = .034 \text{ in.}^2$

at 6' $A_s = .0142 \text{ in.}^2$

$A_s = .017 \text{ in.}^2$

All $\frac{1}{2}" \#$ bars.



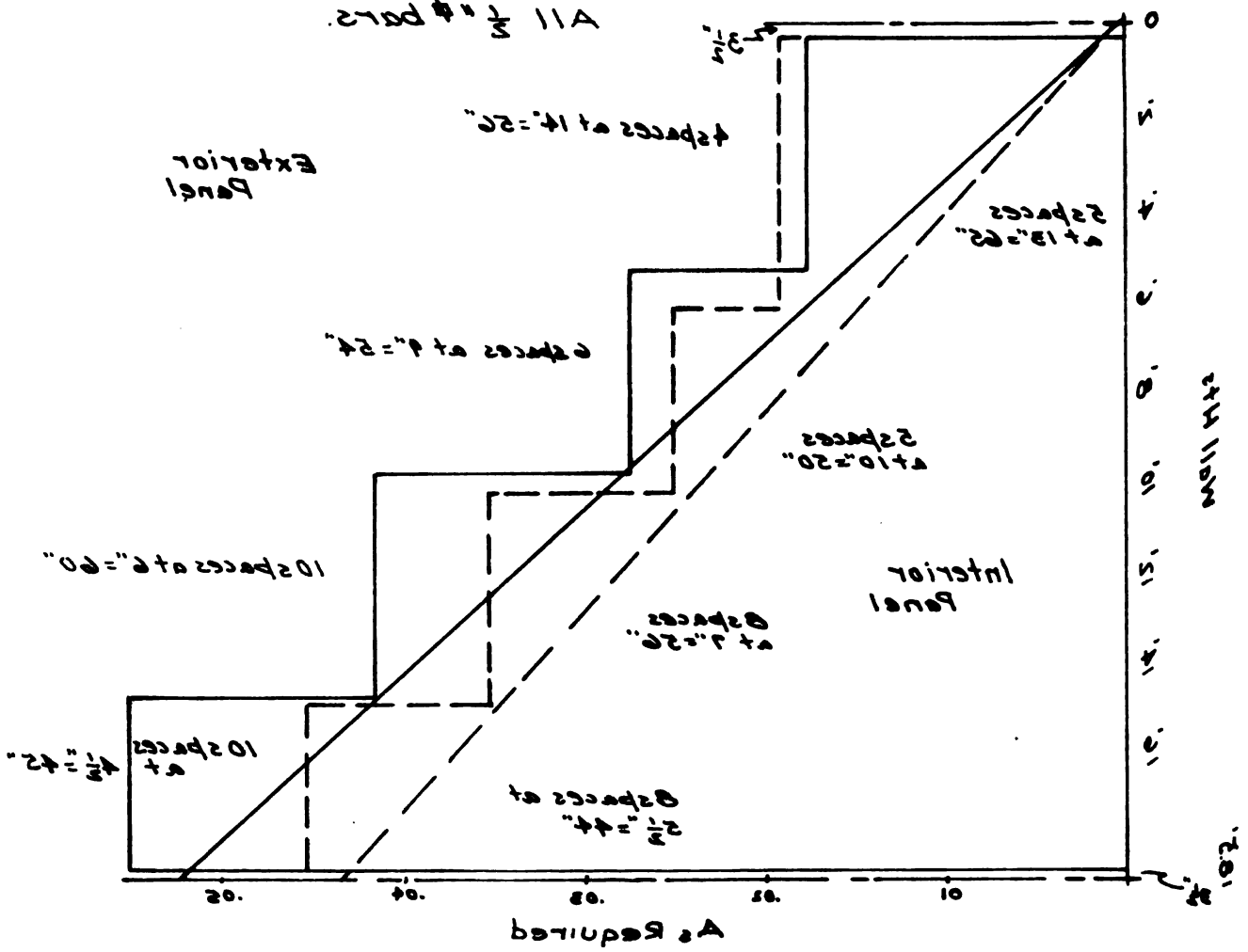
Bond: $\mu = \frac{V}{\sum o_j d} = \frac{4120}{8 \times \frac{7}{8} \times 9} = 65.6 \text{ psi}$

DESIGN OF VERTICAL SLAB (cont.)

For Moment: $d = \sqrt{\frac{M}{Kp}}$
 Int. Panel $d = \sqrt{\frac{74,100}{140 \times 12}} = 25.2"$
 Ext. Panel $d = \sqrt{\frac{89,000}{140 \times 12}} = 27.8"$
 For Shear: $d = \frac{V}{p/v} = \frac{4150}{15 \times \frac{3}{8} \times 60} = 62.4"$ (Spec. Anch. Red'd)
 Steel Red'd: $A_s = \frac{M}{f_y d}$

	Int. Panel	Ext. Panel
at 0'	$A_s = 0.145 \text{ in.}^2$	$A_s = 0.17 \text{ in.}^2$
at 12'	$A_s = 0.258 \text{ in.}^2$	$A_s = 0.34 \text{ in.}^2$
at 18.2'	$A_s = 0.436 \text{ in.}^2$	$A_s = 0.524 \text{ in.}^2$

All $\frac{1}{2}$ # bars



Bond: $n = \frac{V}{8 \times \frac{3}{8} \times d} = \frac{4150}{8 \times \frac{3}{8} \times 60} = 0.20 \text{ psi}$

HEEL DESIGN

Sliding: $\frac{330,000 \cdot 466}{194,000 \cdot 707} = 1.12$ Must Anchor

Overturn: R Strikes within middle 3rd, wall is safe.

Heel design:

Weight of coal above 1' section at end of heel

Coal $25.5 \cdot 70 = 1785^*$

Concrete $= 375^*$

Total $= 2,160^*$ on end of heel (at point of overturn)

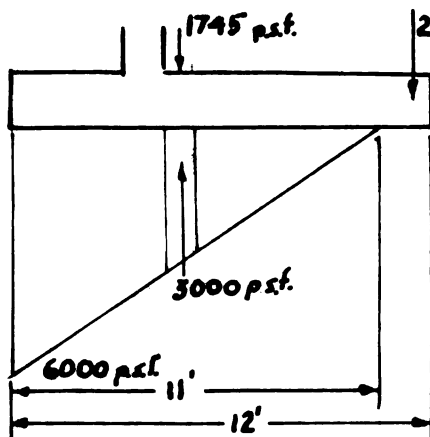
$M = \frac{1}{12} w l^2 = \frac{1}{12} \cdot 2160 \cdot (9)^2 = 14,550^*$ Interior Panel

$M = \frac{1}{10} w l^2 = \frac{1}{10} \cdot 2160 \cdot (9)^2 = 17,500^*$ Exterior Panel

$V = \frac{1}{2} w l = \frac{1}{2} \cdot 2160 \cdot 9 = 9,700^*$

For M $d = \sqrt{\frac{17,500}{140}} = 11.2''$

For Shear $d = \frac{9700}{12 \cdot \frac{7}{8} \cdot 40} = 23.1''$ 27" Supplied



$A_s \text{ Ext.} = \frac{17,500}{18,000 \cdot \frac{7}{8} \cdot 27} = .0417\%$

$A_s \text{ Int.} = \frac{14,550}{18,000 \cdot \frac{7}{8} \cdot 27} = .0342\%$

At point of overturn

Counterfort Outside Wall

HEEL DESIGN

Sliding: $\frac{194,000 \cdot 707}{330,000 \cdot 466} = 1.15$ Must Anchor

Overturm: R Strikes within middle 3rd, wall is safe.

Heel design:

Weight of coal above 1' section at end of heel

Coal $52.2 \cdot 70 = 1782^*$

Concrete 272^*

Total = 5160^* on end of heel (at point

of overturm)

$M = \frac{1}{15} w l^2 = \frac{1}{15} \cdot 5160 \cdot (9)^2 = 14,220^*$ Interior Panel

$M = \frac{1}{10} w l^2 = \frac{1}{10} \cdot 5160 \cdot (9)^2 = 17,200^*$ Exterior Panel

$V = \frac{1}{2} w l = \frac{1}{2} \cdot 5160 \cdot 9 = 2700^*$

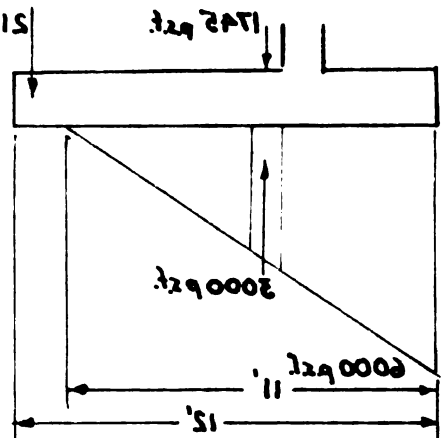
For M $q = \sqrt{\frac{17,200}{140}} = 11.5"$

For shear $q = \frac{2700}{15 \cdot \sqrt{8 \cdot 40}} = 53.1"$ 57" supplied

$A_2 \text{ Ext.} = \frac{17,200}{18,000 \cdot \sqrt{8 \cdot 57}} = .0417 \text{ in}^2$

$A_2 \text{ Int.} = \frac{14,220}{18,000 \cdot \sqrt{8 \cdot 57}} = .0345 \text{ in}^2$

At point of overturm



Counterfort Outside Wall

HEEL DESIGN

At base of heel $V = 3000 - 1745 = 1255$ p.s.f. acting up.

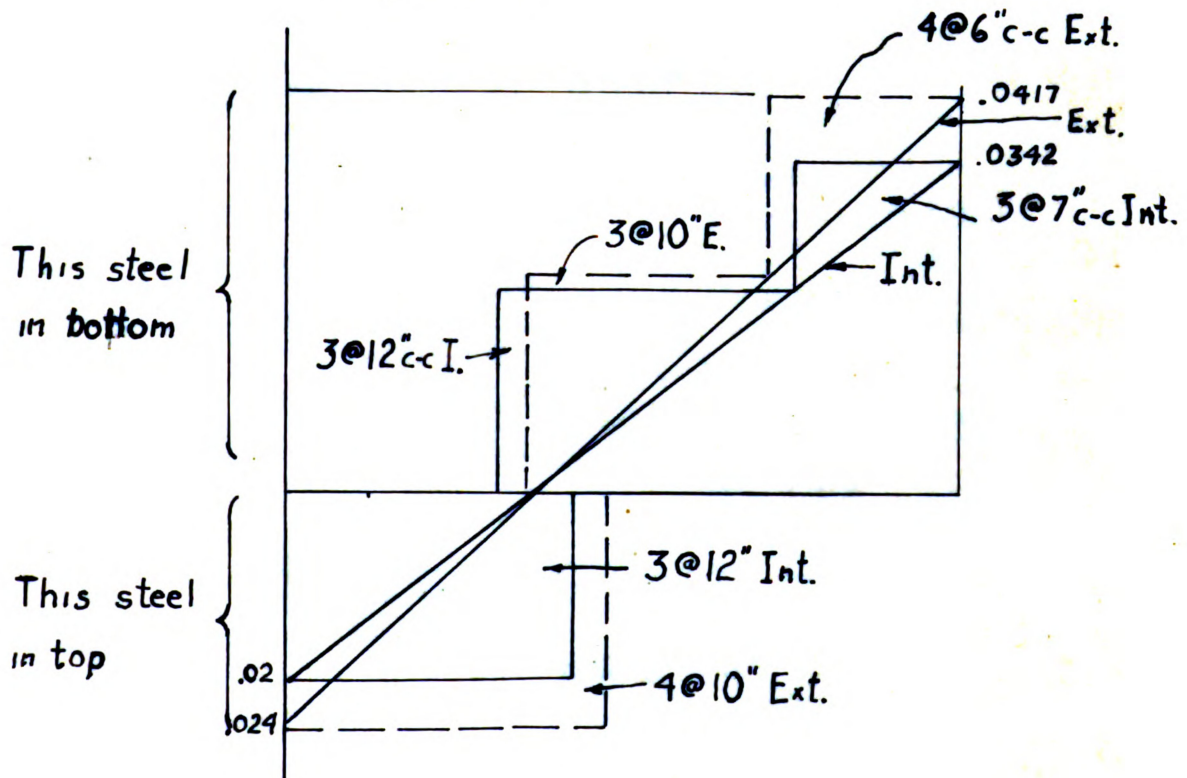
$$M = \frac{1}{12} \cdot 1255 \cdot (9)^2 = 8500' \text{#} \quad \text{Interior Panel}$$

$$M = \frac{1}{10} \cdot 1255 \cdot (9)^2 = 10,180' \text{#} \quad \text{Exterior Panel}$$

$$A_s = \frac{8500}{18000 \cdot \frac{7}{8} \cdot 27} = .02 \text{"} \quad \text{Int.}$$

$$A_s = \frac{10180}{18000 \cdot \frac{7}{8} \cdot 27} = .024 \text{"} \quad \text{Ext.}$$

Heel Steel:
Use $\frac{1}{2}$ " ϕ bars.



$$\mu = \frac{9700}{4 \cdot \frac{7}{8} \cdot 27} = 102 \text{ p.s.i.}$$

Counterfort Outside Wall

HEEL DESIGN

At base of heel $V = 3000 - 1742 = 1258$ p.s.f. acting up.

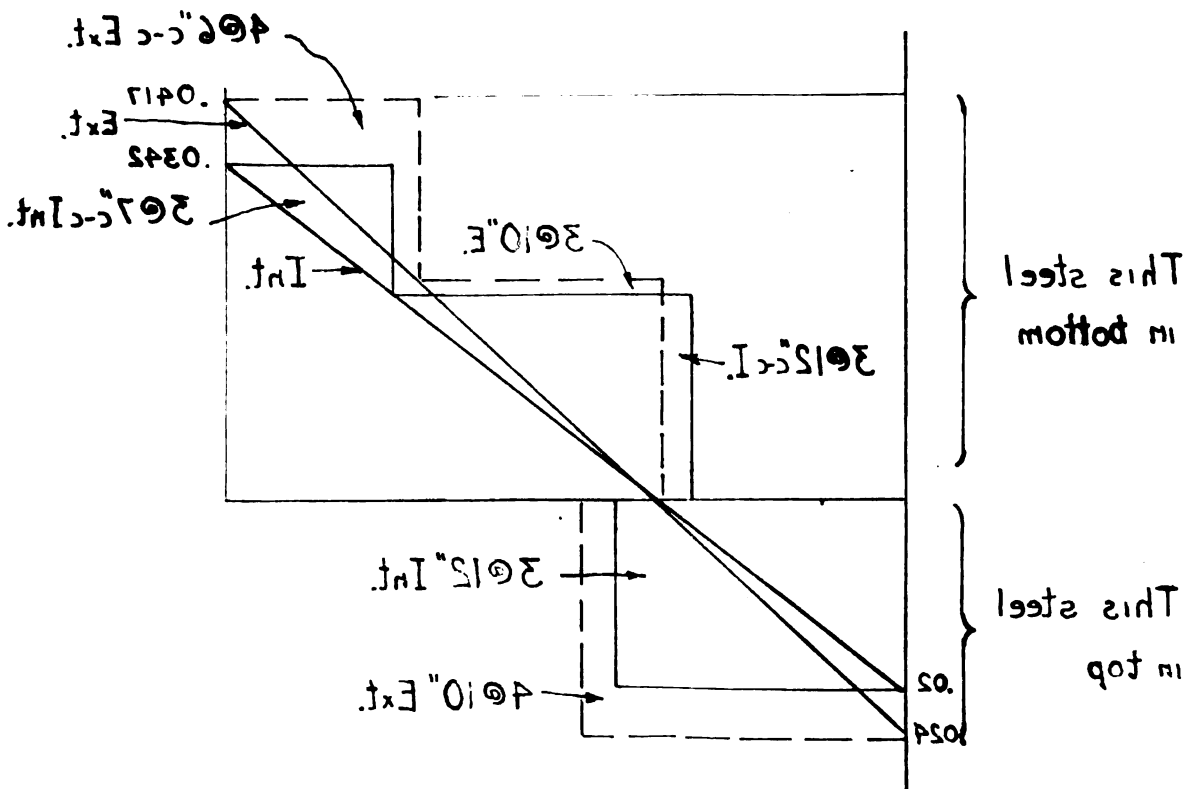
$$M = \frac{1}{12} \cdot 1258 \cdot (9)^2 = 8200 \text{ " Interior Panel}$$

$$M = \frac{1}{10} \cdot 1258 \cdot (9)^2 = 10180 \text{ " Exterior Panel}$$

$$A_s = \frac{8200}{18000 \cdot 8 \cdot 57} = .05 \text{ " Int}$$

$$A_s = \frac{10180}{18000 \cdot 8 \cdot 57} = .054 \text{ " Ext.}$$

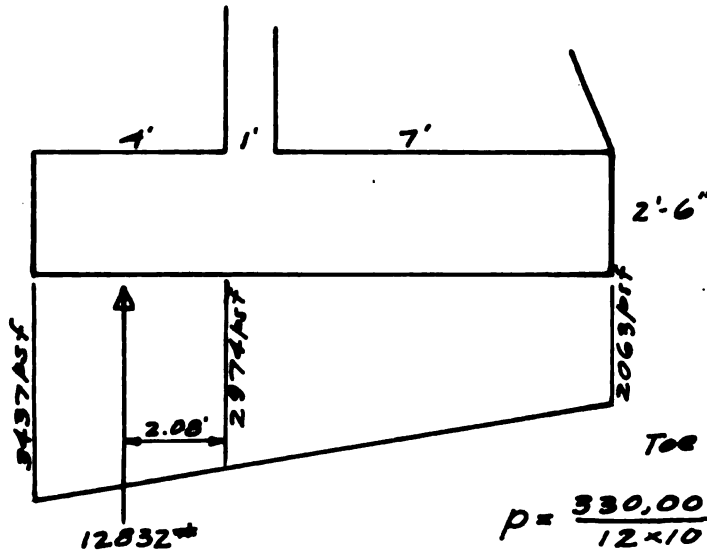
Use $\frac{1}{2}$ " bars
Heel steel:



$$n = \frac{2700}{4 \cdot 28 \cdot 57} = 105 \text{ p.s.f.}$$

Counterfort Outside Wall

DESIGN OF TOE



Toe & Heel Pressures

$$p = \frac{330,000}{12 \times 10} \pm \frac{6 \times 330,000 \times .5}{1440}$$

$$p = 2750 \pm 687$$

$$p_t = 3437 \text{ psf.}$$

$$p_h = 2063 \text{ psf}$$

Shear & Moment

$$V = \frac{3437 + 2974}{2} \times 4$$

$$V = 12,832 \text{ lbs.}$$

$$M = 12,832 \times 2.08$$

$$M = 26,700 \text{ lb.-ft.}$$

For Shear:

$$d = \frac{V}{b_j \tau} = \frac{12,832}{12 \times \frac{7}{8} \times 60}$$

$$d = 20.4'' \text{ (Spec. Anch.)}$$

For Moment:

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{26,700}{140 \times 1}}$$

$$d = 13.8''$$

$$A_s = \frac{M}{f_s j d} = \frac{26,700}{18,000 \times \frac{7}{8} \times 27} = .0628 \text{ in}^2/\text{in}$$

Use $\frac{3}{4}'' \phi$ at 9" c.c.
Bottom

Use $\frac{3}{4}$ " ϕ at d" C.C.
Bottom

$$A_s^2 = \frac{M}{f_y q} = \frac{507,700}{18,000 \times \frac{3}{8} \times 51} = 0.0258 \text{ in}^2$$

$$q = 13.8''$$

$$\text{For Moment: } q = \sqrt{\frac{M}{Kd}} = \sqrt{\frac{507,700}{140 \times 1}}$$

$$q = 50.4'' \text{ (Spec. Anch.)}$$

$$\text{For Shear: } q = \frac{V}{d} = \frac{15,835}{15 \times \frac{3}{8} \times 90}$$

$$\begin{aligned} M &= 507,700 \text{ lb.-ft.} \\ M &= 15,835 \times 5.08 \\ V &= 15,835 \text{ lbs.} \\ V &= \frac{3437 + 5174}{2} \times 4 \end{aligned}$$

Shear & Moment

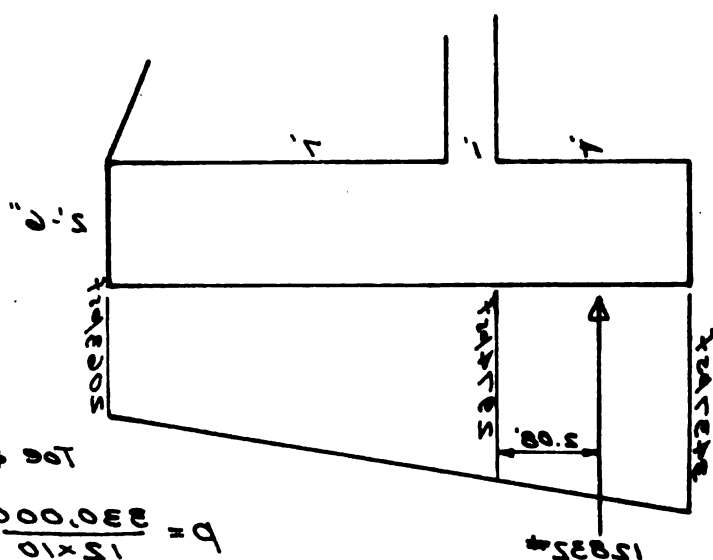
$$p_u = 50.23 \text{ psi}$$

$$p_t = 34.37 \text{ psi}$$

$$p = 51.20 \pm 0.81$$

$$p = \frac{330,000}{15 \times 10} \pm \frac{6 \times 350,000 \times 0.2}{140}$$

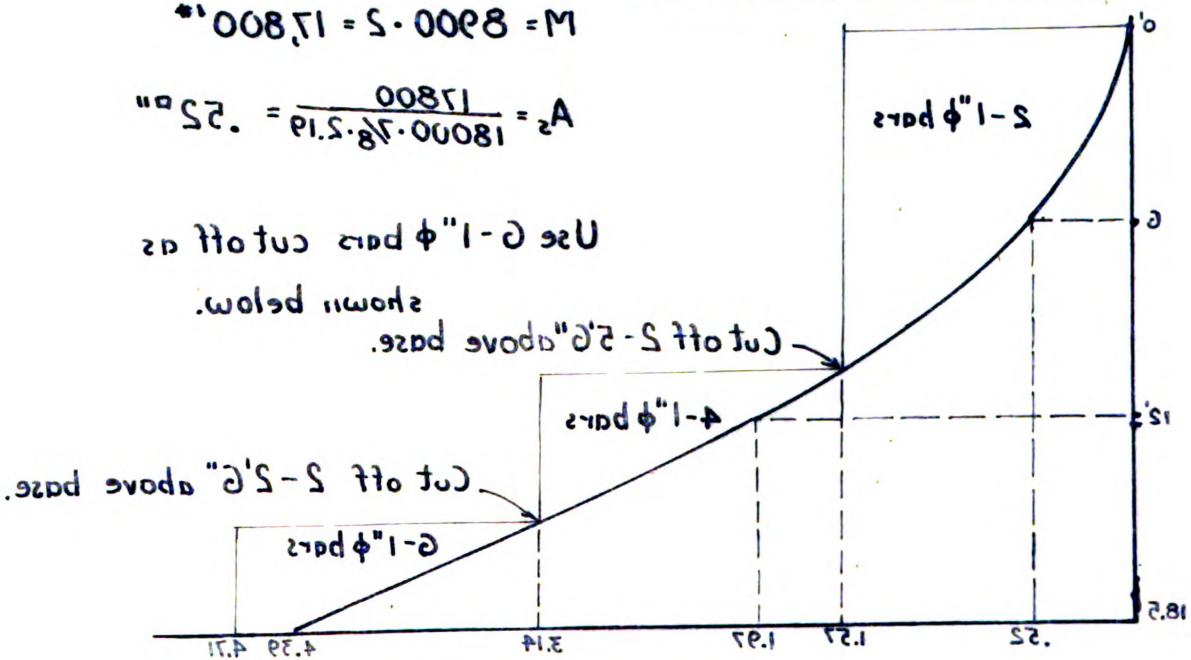
Toe & Heel Pressure



DESIGN OF TOE

Counterfort Outside Wall

Counterfort Outside Wall



Use 6-1" phi bars cut off as shown below.

$$A_2 = \frac{18000 \cdot 5.14 \cdot 5.14}{17800} = 25.0$$

$$M = 8200 \cdot 5 = 17800$$

$$V = \frac{521 \cdot 10 \cdot 6}{5} = 8200$$

At 6'

$$A_2 = \frac{18000 \cdot 7.8 \cdot 4.6}{145000} = 1.27$$

$$M = 32200 \cdot 4 = 145000$$

$$V = \frac{224 \cdot 10 \cdot 15}{5} = 32200$$

At 15'

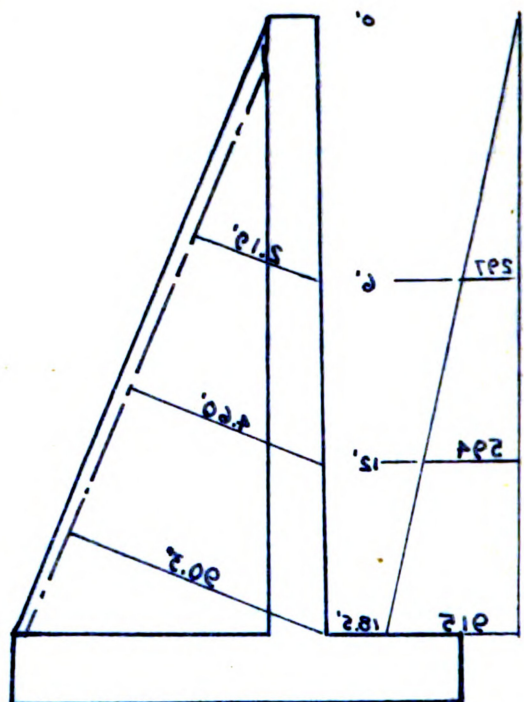
$$A_2 = \frac{18000 \cdot 8 \cdot 20.3}{250000} = 4.32$$

$$M = 84200 \cdot \frac{18.2}{2} = 250000$$

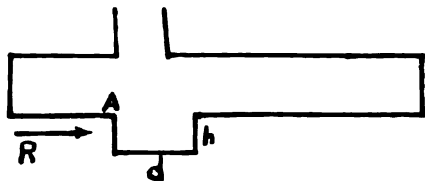
$$V = \frac{212 \cdot 10 \cdot 18.2}{5} = 84200$$

At 18.2'

COUNTERFORT DESIGN



CUT OFF WALL



Passive earth pressure = $3.5 wh$

At "A" $wh = 2950$

Possible resistance = $3.5 \cdot 2950 = 10,300 \text{ psf}$

If S.F. = 2

$$R = 2 \cdot 19400 \cdot 0.707 - 466 \cdot 33000$$

$$R = 12,000$$

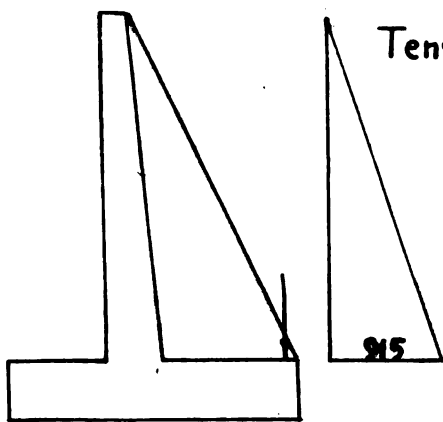
$$h = \frac{12000}{10300} = 1.17' \text{ Say } 1'3" = h$$

$$V = 12,000^* \quad M = 12000 \cdot 7.5 = 90,000^*$$

$$d = \frac{12000}{12 \cdot \frac{1}{8} \cdot 40} = 28.6"$$

$$d = \sqrt{\frac{6 \cdot 90000}{12 \cdot 60}} = 27.4"$$

Make $d = 30"$



Tension Steel:

Vertical Slab

$$P = 915 \cdot 9 = 8220^*$$

$$A_s = \frac{8220}{18000} = .456^* \text{"/ft. (bottom strip)}$$

$$A_b = \frac{.456 \cdot 5}{12 \cdot 2} = .095^* \text{"/bar}$$

Use $\frac{1}{2}" \phi = .20^*$ hook on every other bar.

$$L = \frac{f_y d}{4 \mu} = \frac{18000 \cdot \frac{1}{2}}{4 \cdot 100} = 22.5"$$

Heel Slab

$$P = 9700 \cdot 2 = 19,400^* \quad A_s = \frac{19400}{18,000} = 1.18^* \text{"/ft.}$$

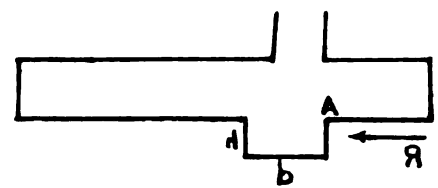
$$A_b = \frac{1.18 \cdot 6}{12 \cdot 2} = .27^* \text{"/bar} \quad L = \frac{f_y d}{4 \mu} = 29"$$

Use $\frac{5}{8}" \phi$ hook on every bar

Counterfort Outside Wall

CUT OFF WALL

Passive earth pressure = 3.2 m
 At A, $w_h = 5250$
 Possible resistance = $3.2 \cdot 5250 = 16800$ lbs
 It 2.7 = 5
 $R = 5 \cdot 16400 \cdot 707 - 426 \cdot 33000$
 $R = 15000$
 $H = \frac{15000}{10300} = 1.17$ say 1.3 = H
 $V = 15000 \cdot M = 15000 \cdot 2.2 = 33000$

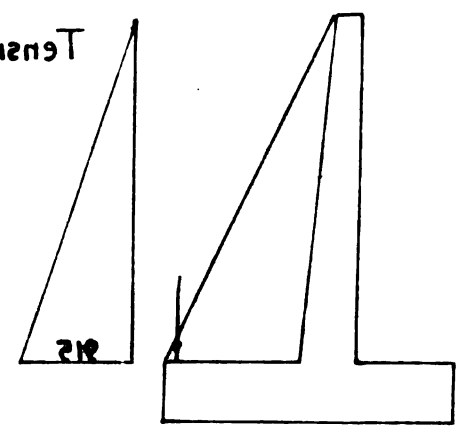


Make $q = 30$ "
 $q = \frac{15000}{15 \cdot 8 \cdot 40} = 58.6$ "
 $1 = \sqrt{\frac{e \cdot 30000}{15 \cdot 60}} = 57.4$ "

Tension Steel:

Vertical Slab

$P = 212 \cdot 2 = 8550$
 $A_s = \frac{8550}{18000} = 0.475$ (from table)
 $A_s = \frac{15 \cdot 5 \cdot 2}{2 \cdot 200} = 0.075$ per
 Use $\frac{1}{2}$ " $\phi = .50$ " hook on every other bar
 $L = \frac{4 \cdot 2 \cdot 18000}{4 \cdot 100} = 55.2$ "

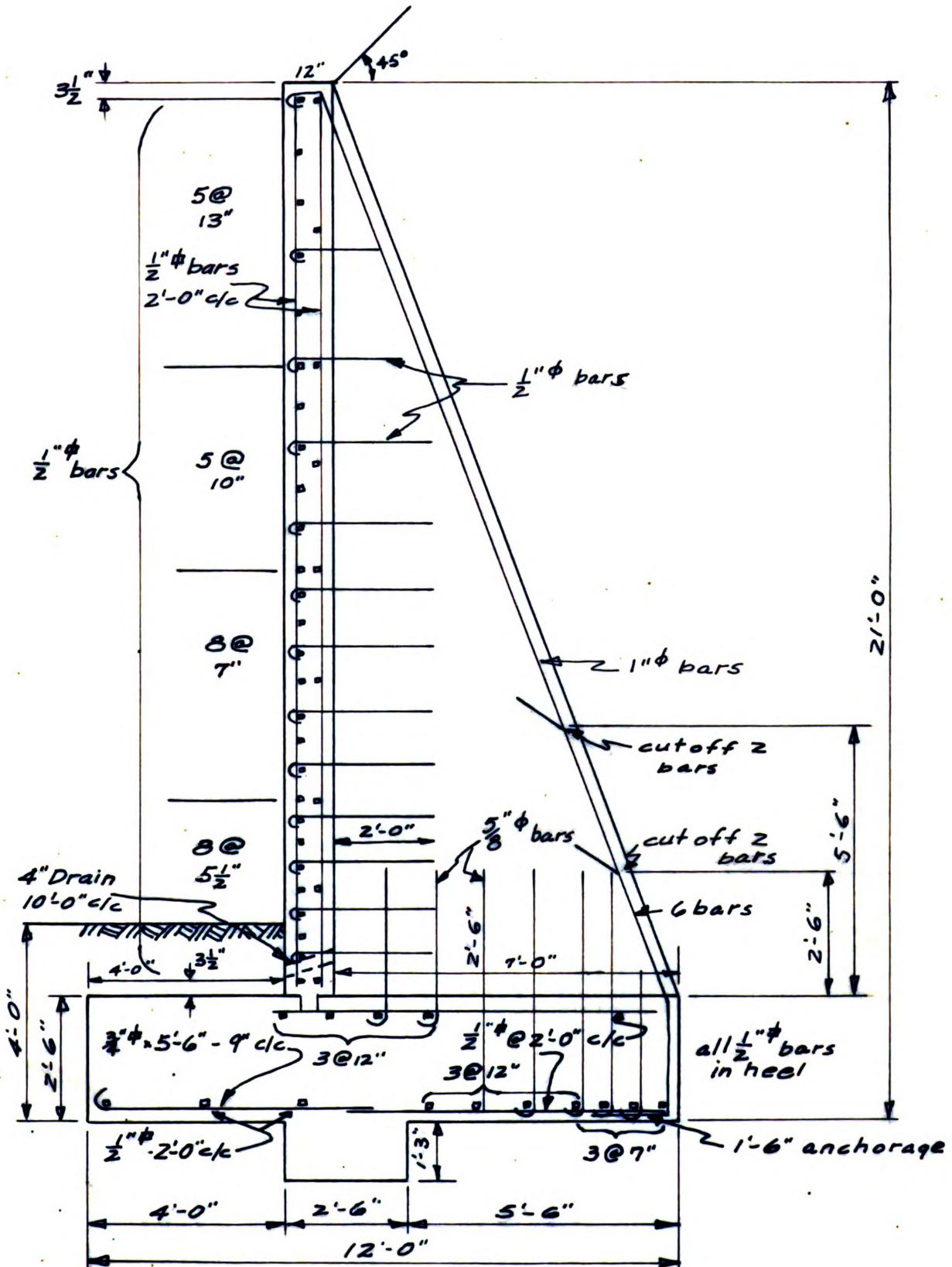


Horizontal Slab

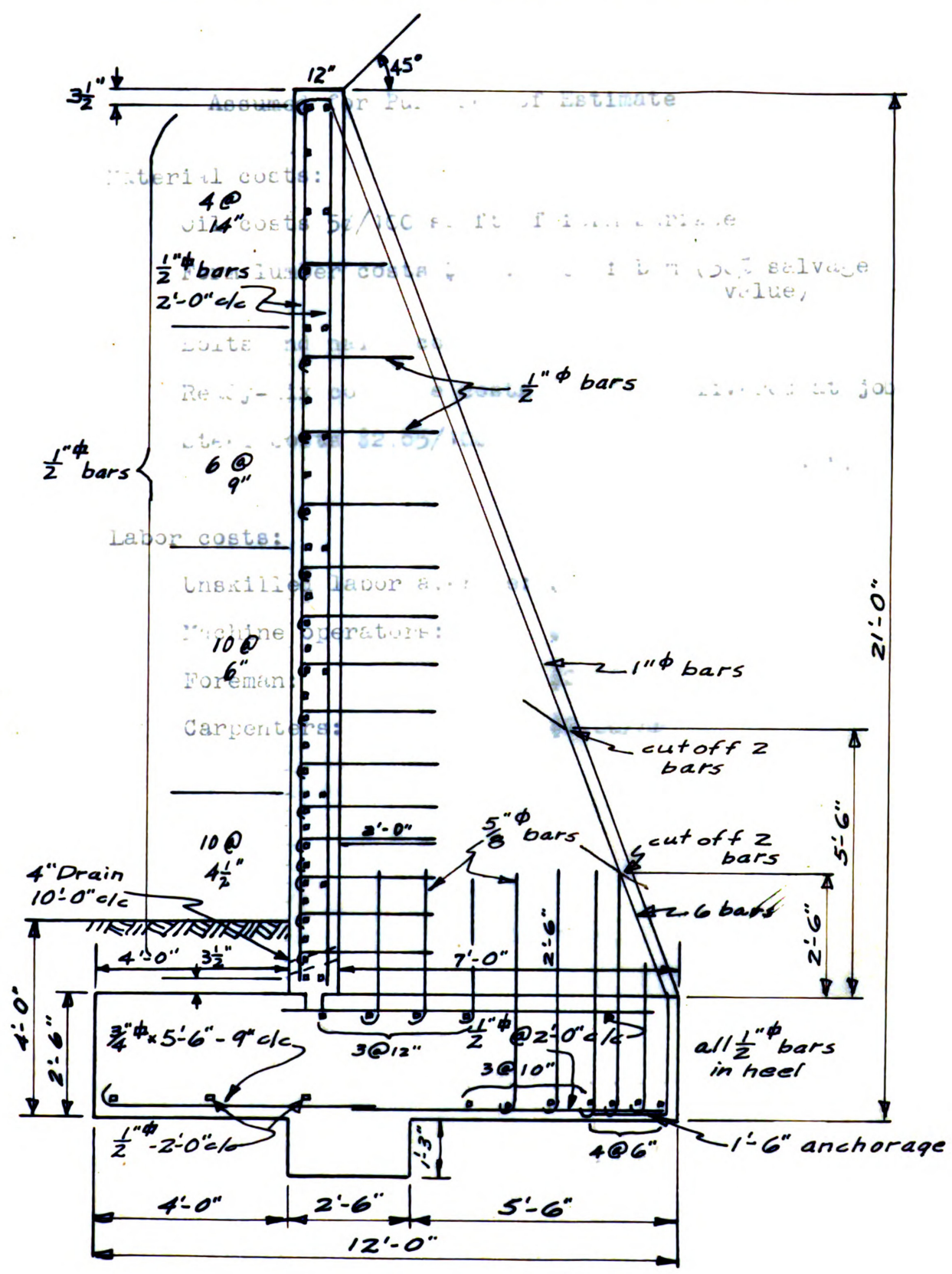
$P = 2700 \cdot 5 = 13500$
 $A_s = \frac{13500}{18000} = 0.75$
 $L = \frac{4 \cdot 2 \cdot 18000}{4 \cdot 100} = 55.2$ "
 Use $\frac{3}{8}$ " ϕ hook on every bar

Counterfort Outside Wall

DETAILS OF COUNTERFORT OUTSIDE WALL - INTERIOR PANEL



DETAILS OF COUNTERFORT OUTSIDE WALL - EXTERIOR PANEL



Assumed for P.C. Estimate

Material costs:

4 @ 14" costs 58/100 ft. 14" bars

1/2" bars 2'-0" c/c

6 @ 9" 1/2" bars

2 1" bars

cut off 2 bars

5'-6"

2'-6"

4" Drain 10'-0" c/c

4'-0" 3 1/2"

3/4" x 5'-6" - 9" c/c

3 @ 12"

5/8" bars

cut off 2 bars

2'-6" bars

7'-0"

2'-0"

10 @ 4 1/2"

1/2" - 2'-0" c/c

1'-6" anchorage

4 @ 6"

3 @ 10"

1/2" @ 2'-0" c/c

all 1/2" bars in heel

4'-0" 2'-6" 5'-6" 12'-0"

1'-3"

Labor costs:

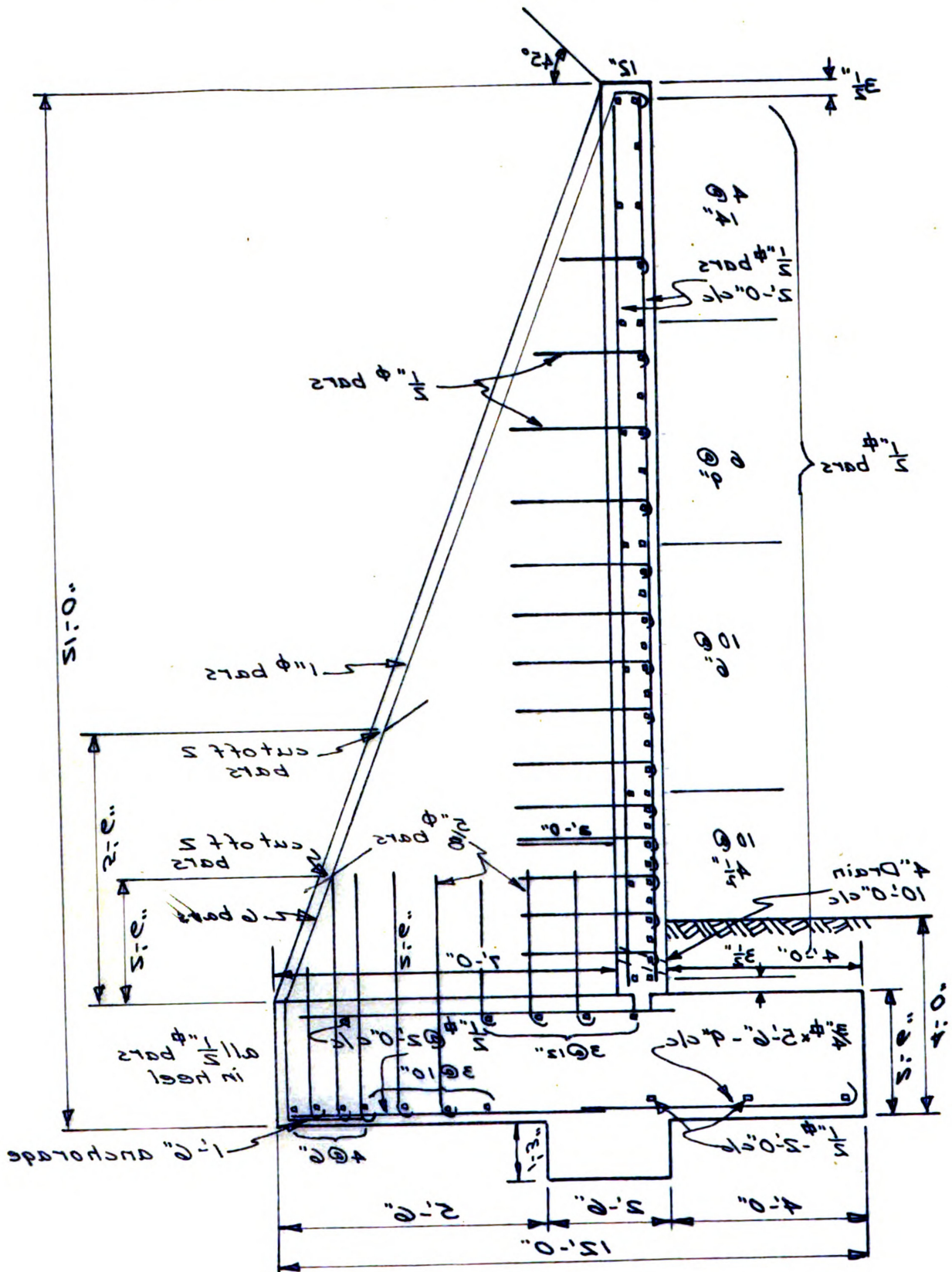
Unskilled labor 8.00

Machine operators: 10 @ 6"

Foreman: 6"

Carpenters:

DETAILS OF COUNTERFORT OUTSIDE WALL - EXTERIOR PANEL



Assumed for Purposes of Estimate

Material costs:

Oil costs 5¢/100 sq ft of form surface

Form lumber costs \$100./1000 f b m (50% salvage value)

Bolts and nails cost 6¢/lb

Ready-mix concrete costs \$10./yd delivered at job

Steel costs \$2.65/100 lb (incl. chairs, spacers and ties,

Labor costs:

Unskilled labor average: \$1.20/hr

Machine operators: \$1.50/hr

Foreman: \$2.00/hr

Carpenters: \$2.00/hr

Cost Estimate for Cantilever Wall

1. Materials:

(a). Forms

R.R. Wall 16x110x2 or 3530 sq ft of wall
 35.3x250 fbm/100 sq ft or 8820 fbm reqd.
 35.3x3 lbs/100 sq ft or 282 lbs nails reqd.
 Lumber @ \$50./1000, 3.82x50 or \$441.00
 Bolts @ 6¢/lb 282x.06 or \$ 16.80
 Oil @ 5¢/100 35.3x.05 or \$ 1.80
 \$459.60

Outside Wall 13.5x2x220 or 8140 sq ft of forms
 81.4x250 or 20350 fbm reqd.
 81.4x3 or 650 lbs reqd.
 Lumber @ \$50./1000, 20.35x50 or \$1017.50
 Nails @ 6¢/lb 650x.06 or \$ 39.00
 Oil @ 5¢/100 81.4x.05 or \$ 4.07
 \$1060.57

Total for forms \$1520.17

(b) Concrete

192 yds in R.R. Wall
510 yds in Outside Wall
 702 yds total (387 in base, 315 in stem)
 \$10.00x702 or \$7020.00

(c) Steel in 10' of wall

1" rd bars (2.67 lb/ft)

3.75 bars, 18.5' long or 69.4'

7.5 bars, 9.0' long or 67.5'

12.5 bars, 7.0' long or 87.5'

224.4'x2.67 or 598 lbs

1/2" sq bars (.85 lb/ft)

4 bars, 18.5' long or 74.0'

36 bars, 10.0' long or 360.0'

434.0'x .85 or 369 lbs

3/4" sq bars (1.21 lb/ft)200.0'x1.91 or 382 lbs.

1349 lbs/10'

1349x33 or 44500 lb total

Total for Steel - 445x2.65 or 1180.00Total Material Costs \$9720.17

2. Labor

(a) Excavation

1 Foreman @ \$2.00/hr.x64 hrs \$ 128.00

2 Trimmers @ \$1.00/hr.x16x2 \$ 32.00

1 Cat. Oper. @ \$1.50/hr.x64 hrs \$ 96.00

1 B'hoe. Op. @ \$1.50/hr.x16 hrs \$ 24.00\$280.00

(b) Forms

11670 sq ft of form surface

15 labor hrs/100 sq ft

Unit cost 30¢/sq ft

Total for forms 11670x.30 or \$3501.00

(c) Concrete

Base: 337 yds @ 1hr/12 yds or 32 hrs

7 men x 32hrs x \$1.20 or \$269.00

Stem: 315 yds @ 1hr/2 yds or 158 man hrs

7 men take 4 days to place

7 x 8 x 4 x \$1.20 or \$269.00

Total for concrete \$538.00

(d) Steel

25 hooks/10' 33x25 or 825 hooks

9 bends/10' 33x 9 or 297 bends

Bending: 2 hrs/100 bends x 297 or 7 hrs

Hooking: 3 hrs/100 hooks x 825 or 25 hrs

Placing:

1/2" bars 25x33 or 826 under 10'

@ 5 hr/100 or 42 hrs

45x33 or 1850 under 20'

@ 6 hr/100 or 90 hrs

1" bars 3.75x33 or 124 under 20'

@ 8.5 hr/100 or 11 hrs

18.5x33 or 610 under 10'

@ 6.5 hr/100 or 40 hrs

215 hrs

Total for steel 215x\$1.20

\$258.40Total Labor Costs \$4577.40

3. Plant

(a) Excavation

Remove 150' R.R. Track @ \$1.50/lin.ft. \$195.00

Reinstall 150' R.R. Track @ \$3.00/lin.ft. \$390.00

Strip off 1' over bin floor

55x110/27 or 224 yds

Excavate along R.R.

15x12x130/27 or 650 yds

2374 yds

Use 2 Bulldozers @ 600 yds/day

2374 or 3 days
1200

\$37.00/day x 2 x 3 or \$512.00

Dig for base

306x4x12/27 or 545 yds

Backhoe @ 400 yds/day

545 or 2 days x \$37.00 or \$74.00
400Total for excavation \$1171.00

(b) Forms

1 power saw and hand tools-stay 20.00

(c) Concrete

5 baggies @ 20¢ ea./day x 4 days or \$4.00

1 Vibrator @ 60¢/day x 8 days or \$4.80

Misc. small tools \$10.00\$18.80

(d) Steel

Bending and Cutting machines

\$100/day x 4 days or \$4.00Total Plant Costs \$1213.80

Total cost of Materials, Labor and Plant	\$15511.37
4. Overhead	
15% of total cost	<u>\$ 2326.73</u>
	\$17838.10
5. Profit	
8% of total (incl. overhead)	<u>\$ 1427.85</u>
GRAND TOTAL	\$19265.15

Cost Estimate for Counterfort Wall

1. Materials

(a) Forms

Outside Wall

20 ct'fts x 7 x 12.5 or 2000 sq ft

26x250 fbm/100 sq ft or 6500 fbm

Nails etc. 26x8 or 208 lbs

Lumber @ \$50./1000, 6.5x50 or \$325.00

Nails @ 6¢/lb, 208x.06 or \$ 12.48

Oil @ 5¢/100, 26x.05 or \$ 1.30

(ct'fts alone) \$338.78

Wall \$1060.57

\$1399.27

R.R. Wall

10 ct'fts x 7 x 12 or 840 sq ft

8.4x250 fbm/100sq ft or 2100 fbm

Nails etc. 8.4x3 or 67lbs

Lumber @ \$50./1000, 2.1x50 or \$105.00

Nails @ 6¢/lb, 67x.06 or \$ 4.02

Oil @ 5¢/100, 8.4x.05 or \$.42

(ct'fts alone) \$109.44

Wall \$459.60

\$568.04

Total for forms \$1968.35

(b) Concrete

102 yds in R.R. Wall

462 yds in Outside Wall and ct'fts

657 yds @ \$10./yd or

\$6570.00

(c) Steel

1" rd bars

2 bars, 26.5' long or 53'

10.5 bars, 2' long or 21'

7.5 bars, 2' long or 15'

89'x2.67 or 238 lbs

1/2" sq bars

39 bars, 10' long or 390'

13 bars, 5' long or 65'

29.5 bars, 10' long or 295'

750'x .35 or 633 lbs

3/4" sq bars

13 bars, 5.5' long or 73.3'

73.3'x1.91 or 140 lbs

1/2 rd bars

24 bars 2.5' long or 60'

24 bars 5.0' long or 120'180'x.67 or 121 lbs

1137 lbs/10'

1137x33 or 37500 lbs

Total for Steel - 375x2.65 or 984.00Total Material Costs \$9512.25

2. Labor

(a) Excavation

Same as for Cantilever \$ 280.00

(b) Forms

15070 sq ft of form surface

15 labor hrs/100 sq ft or 307/sq ft

Total for Forms 15070x.30 or \$4521.00

(c) Concrete

Base: 406 yds @ 1hr/12 yds or 36 hrs

7 men * 36 hrs x \$1.20 or \$320.00

Stem: 251 yds @ 1hr/2 yds or 125 man hrs

7 men take 3 days to place

7 x 8 x 3 x \$1.20 or \$202.00Total for concrete \$522.00

(d) Steel

47 hooks/ct'ft x 30 or 1410 1/2" rd hooks

13.5 x 33 or 445 3/4" sq hooks

2.25hrs/100 x 445 or 10 hrs

2 hrs/100 x 1410 or 29 hrs

11 x 30 or 340 bends

2 hrs/100 x 340 or 7 hrs

Placing Bars

10 hrs/100 x 30 or 3 hrs

8 hrs/100 x 120 or 10 hrs.

6.5 hrs/100 x 120 or 8 hrs

6 hrs/100 x 3175 or 135 hrs

4 hrs/100 x 432 or 17 hrs

269 hrs

Total for Steel 269 x \$1.20 or \$322.80Total Labor Costs \$5645.80

3. Plant

(a) Excavation

Same as Cantilever \$1171.00

(b) Forms

Same as Cantilever \$ 20.00

(c) Concrete

5 buggies @ \$1./day

3 x \$1.00 or \$3.00

1 Vibrator @ 60¢/day

7 x .60 or \$4.20

Misc. tools \$10.00\$ 17.20

(d) Steel

Bending and Cutting Machines

\$ \$4.00/day x 6 days or \$ 6.00Total Plant Costs \$1214.20Total cost of Materials, Labor and Plant \$16372.35

4. Overhead

15% of total cost \$ 2455.85\$18828.20

5. Profit

8% of total (incl. overhead) \$ 1506.25GRAND TOTAL \$20334.45

SUMMARY OF COST ESTIMATES

	Cantilever	Counterfort
<u>Materials</u>		
Forms	\$ 1520.17	\$ 1969.35
Concrete	\$ 7020.00	\$ 6570.00
Steel	<u>\$ 1120.00</u>	<u>\$ 974.00</u>
	\$ 9720.17	\$ 9512.35
<u>Labor</u>		
Excavation	\$ 220.00	\$ 220.00
Forms	\$ 3501.00	\$ 4521.00
Concrete	\$ 532.00	\$ 522.00
Steel	<u>\$ 253.40</u>	<u>\$ 322.80</u>
	\$ 4577.40	\$ 5645.80
<u>Plant</u>		
Excavation	\$ 1171.00	\$ 1171.00
Forms	\$ 20.00	\$ 20.00
Concrete	\$ 18.80	\$ 17.20
Steel	<u>\$ 4.00</u>	<u>\$ 6.00</u>
	\$ 1213.80	\$ 1214.20
	\$ 15511.37	\$ 16372.35
<u>Overhead</u>		
15%	<u>\$ 2326.73</u>	<u>\$ 2455.85</u>
<u>Profit</u>		
8%	<u>\$ 1427.05</u>	<u>\$ 1506.25</u>
GRAND TOTAL	\$ 19265.15	\$ 20334.45

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