E N G I N E E R I N G

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T H E S I S

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R E I N F O R C E D - C O N C R E T E

WATER RESERVOIR

PART I

B.S.C.E

WADI KAYSAR ZAHKA

1944 - 1945

WATER RESERVOIR

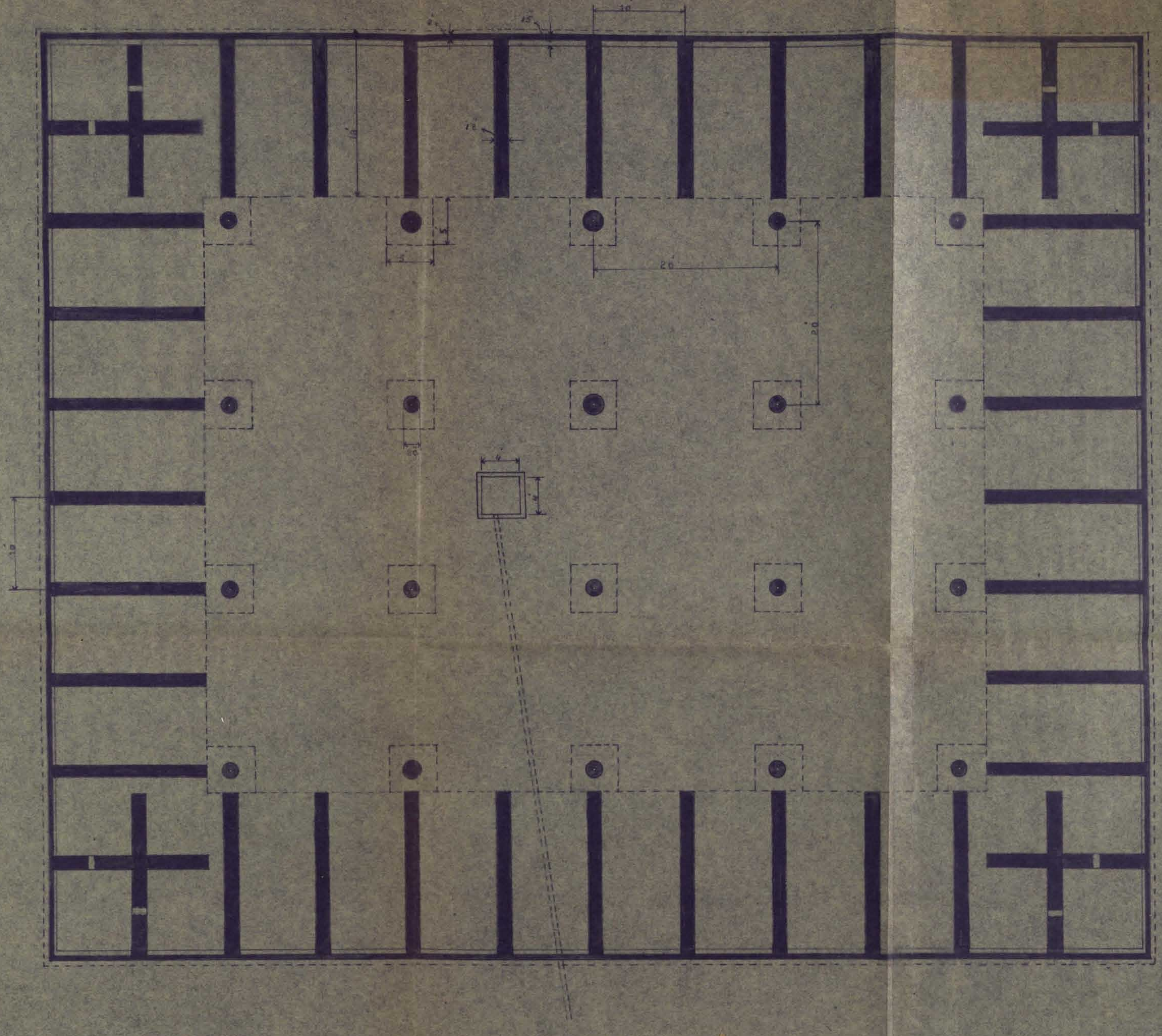
INTRODUCTION

A reinforced concrete water reservoir of I20 feet by IOO feet by 25 feet is composed of a flat slab with drop panels supported on columns with capitals. The exterior walls are hold by buttresses.

The reservoir is covered and surrounded by earth in order to preserve the water from the high changes in temperature.

An opening at the top is made to permit inspection. A manhole at the bottom connected to a pipe IO inches in diameter is used for the distribution of water.

Ventilation of the water is realized through aeration pipes.



Scale 1" = 8'

PLAN OF THE WATER RESERVOIR

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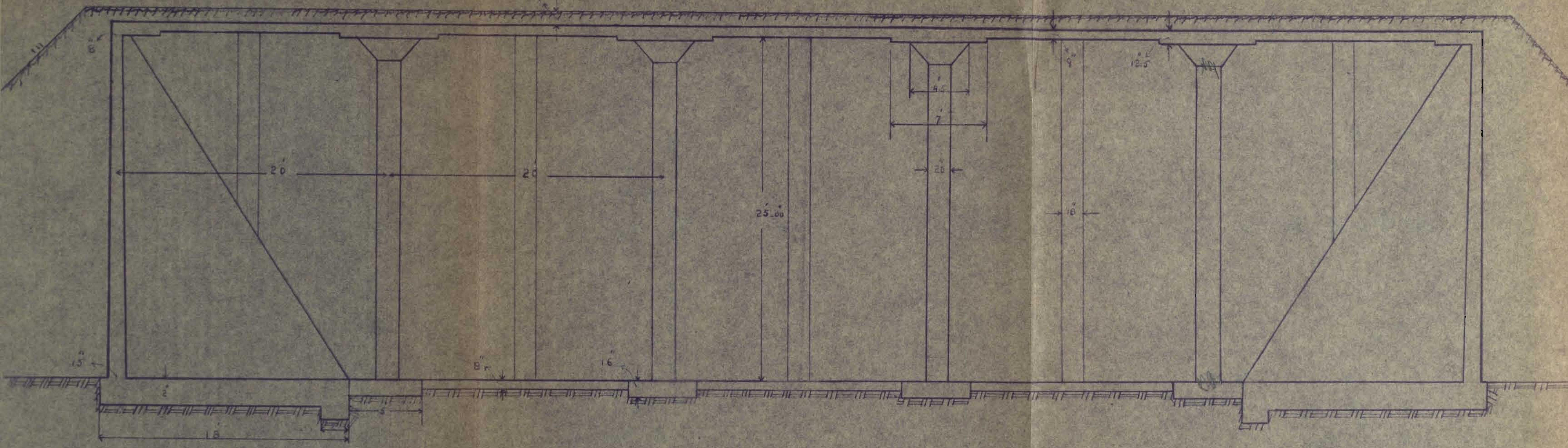

SECTION OF THE WATER RESERVOIR

Scale 1" = 5'

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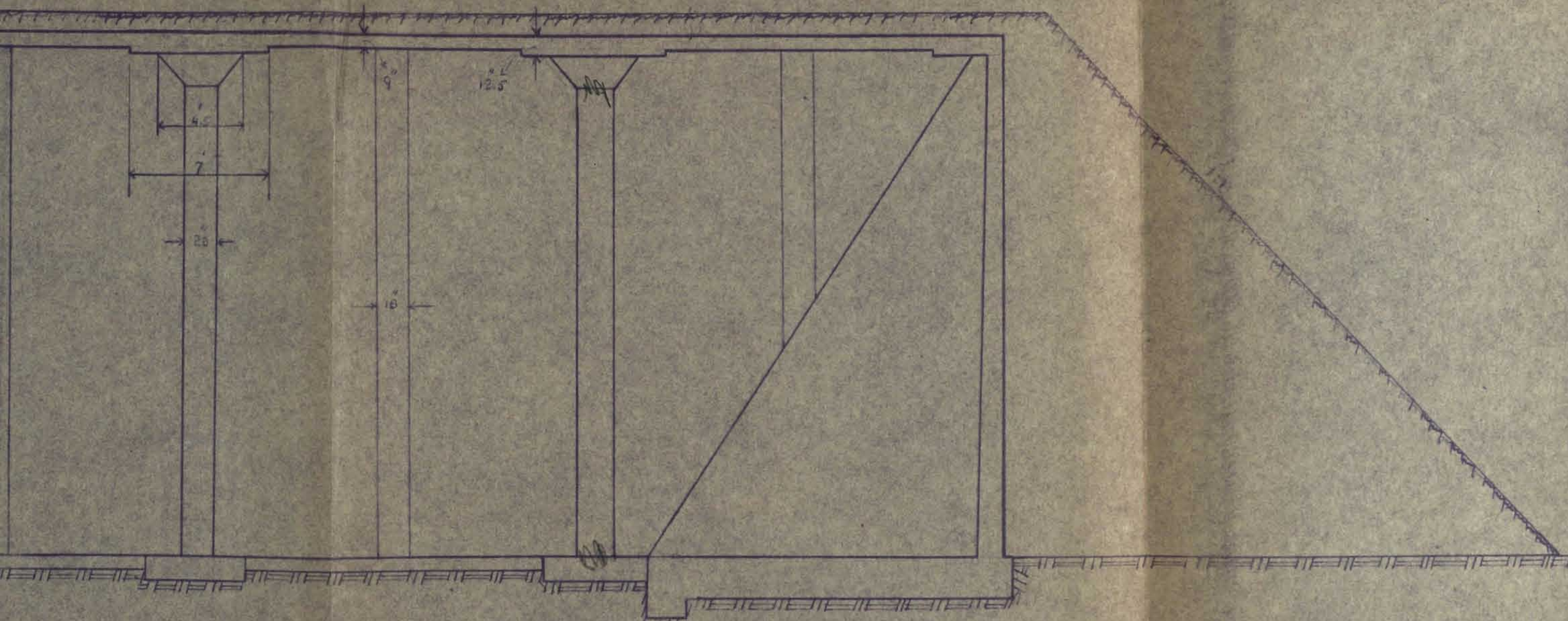


WATER RESERVOIR

Wadi Zahka

D E S I G N   O F   F L A T   S L A B

INTERIOR PANEL

DATA:

Two way flat slab with drop panels and capitals.

Live Load:

Earth 1 ft of depth = $\frac{12}{12} \times 100 = 100$

Live load	= 50	
Total Live Load	150	lbs/ft Sq.

Panel: 20 ft sq. center to center of columns.

Unit stresses: $f_s = 18000$ lbs/in. sq.

$f_c' = 2000$ lbs/in. sq.

$f_c = 0.4 \times 2000 = 800$ lbs/in.sq.

$n = 15$; $v = 40$ lbs/in.sq.

$$K = \frac{1}{1 + \frac{fs}{n.fc}} = \frac{1}{1 + \frac{18000}{15x800}} = 0.40$$

$$j = 1 - 1/3\ K = 0.867$$

Calculations:

Loads:

Live Load	= 150	
Dead Load=9/12 x 150	= 113	(assumed)
w	= 263	lbs/ft. sq.

Thickness of main slab:

For slabs with dropped panels, the total thickness in inches at points beyond the dropped panel shall be not less than that given by:

$$t_2 = 0.02\ L \sqrt{w} + 1 \qquad L = 20 \text{ ft}$$

$$t_2 = 0.02 \times 20 \sqrt{263} + 1 = 7.5 \text{ in.}$$

Use t_2 = 9 in

Minimum allowable = $L/32 = \frac{20 \times 12}{32} = 7.5$ in. $<$ 9 in. O.K.

Size of Column Capital:

Minimum Dimension $C = 0.225\ l = 0.225 \times 20 = 4.5$ ft

Use Capital's Diameter at the top 4.5 ft

Size of drop panel:

Minimum $b = l/3 = 20/3 = 6.67$ ft.

Use 7 ft drop panel.

Thickness of Drop Panel:

Max. thickness $t_1 = 1.5\ t_2 = 1.5 \times 9 = 13.5$ in.

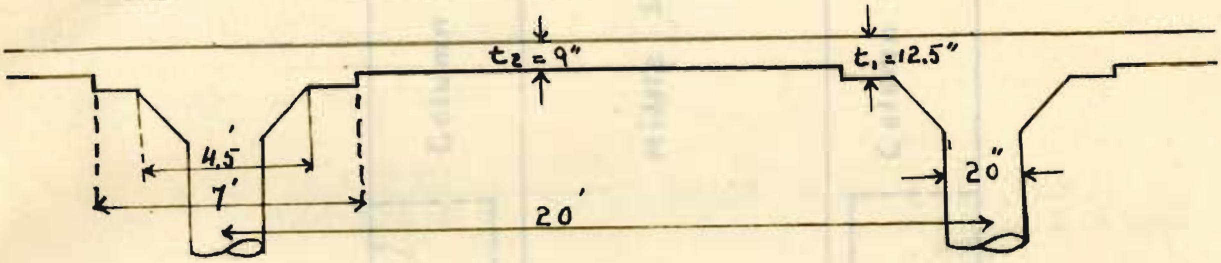
Min. thickness $t_1 = 0.038\ (1 - 1.44\ C/l)\ l\sqrt{Rwl/b} + 1.5$

where: C, l, w, b are the same mentioned above.

R = Ratio of negative moment in the two column strips to M_0 (it will be seen later). From tables R = 0.5

Therefore $t_1 = 0.038\ (1 - 1.44 \times 4.5/20) \times 20\sqrt{0.5 \times 263 \times 20/7} + 1.5 = 11.5$in

Use 12.5 in. as thickness of Drop Panel.



Moments:

The numerical sum of the positive and negative moments in the direction of either side of the panel for which tension reinforcement must be provided, shall be assumed as not less than that given by:

$$M_0 = 0.09\ Wl\ (1 - 2/3\ C/l)^2$$

where:

M_0 = sum of positive and nagative bending moments in either rectangular direction at the principal design sections of a panel of a flat slab.

W = Total dead and live load uniformly distributed over a single panel area.

C & l are the same mentioned above.

$W = wl^2 = 263 \times \overline{20}^2 = 105{,}200$ lbs.

$M_0 = 0.09 \times 105{,}200 \times 20\ (1 - 2/3 \times 4.5/20)^2$
$= 137{,}000$ ft.lb. $= 1{,}650{,}000$ in.lb.

Moments in Principal Moment Strips

Two Column Strips

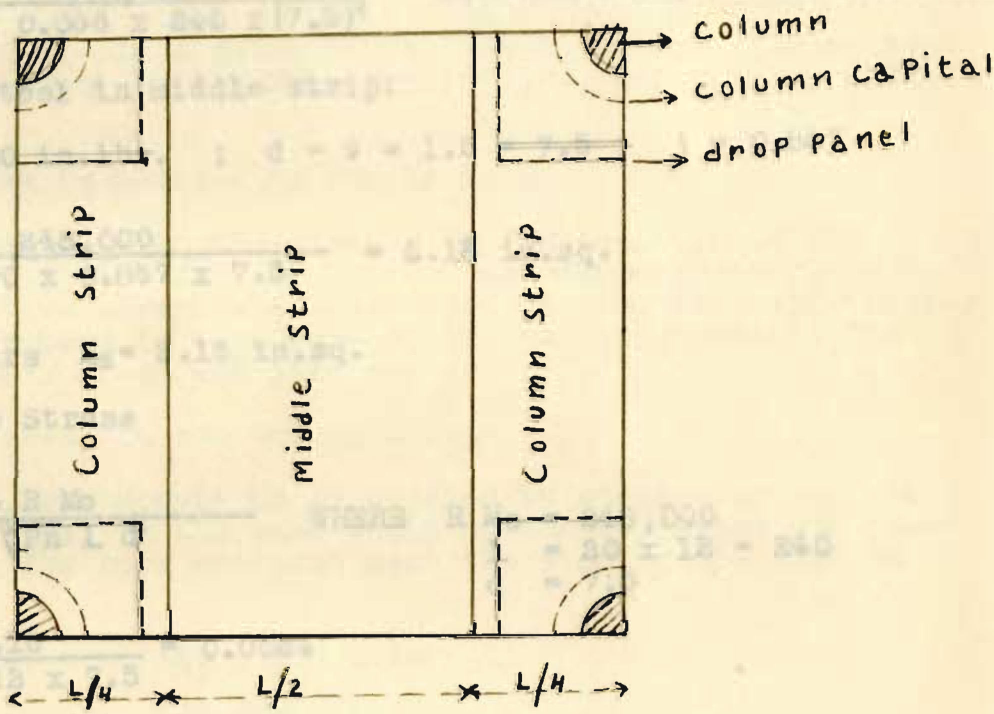
Negative moment = $- 0.5\ M_o$ 0.5 is taken from tables
$= - 0.5 \times 1{,}650{,}000 = - 825{,}000$ in.lbs.

Positive moment = $0.2\ M_o = 0.2 \times 1{,}650{,}000 = + 330{,}000$ in.lbs.

Middle Strip

Negative moment $= -0.15\ M_o = - 0.15 \times 1{,}650{,}000 = - 248{,}000$ in.lbs.

Positive moment = $0.15\ M_o = 0.15 \times 1{,}650{,}000 = + 248{,}000$ in.lbs.



Design of Principal Moment Strips:

Positive Moment steel in two Column Strips:

M = + 330,000 in.lbs. Moment to be carried by one layer of steel near bottom of slab. Steel to have one inch of cover. Assume $\frac{1}{2}$ in. bars placed $1\frac{1}{2}$ in. from bottom of slab.

$d = 9 - 1\frac{1}{2} = 7.5$ in. ; $j = 0.867$

$$A_s = \frac{M}{f_s\, jd} = \frac{330{,}000}{18{,}000 \times 0.867 \times 7.5} = 2.82 \text{ in.sq.}$$

Use 15-$\frac{1}{2}$ in. Φ where $A_s = 2.94$ in.sq.

Check Compressive stress in Concrete.

The compressive stress in the concrete in flat slabs shall be taken as not less than that computed by:

$$f_c = \frac{6\ R M_o}{0.67\ \sqrt[3]{P\ n}\ 1\ d^2}$$

For compression due to positive moment, R M_o, in the two column strips, or negative or positive moment in the middle strip:

$$R\,M_o = 330{,}000$$

$$p = \frac{2.94}{10 \times 12 \times 7.5} = 0.00326$$

$$p\,n = 0.00326 \times 15 = 0.049 \quad \text{where } n = 15$$

$$\sqrt[3]{p\,n} = \sqrt[3]{0.049} = 0.366$$

$$l = 20 \times 12 = 240 \;;\; d = 7.5$$

$$f_c = \frac{6 \times 330{,}000}{0.67 \times 0.366 \times 240 \times (7.5)^2} = 650 \text{ lbs/in.sq.} < 800 \text{ Allowable}$$

Positive moment steel in middle strip:

$$M = 248{,}000 \text{ in.lbs.} \;;\; d = 9 - 1.5 = 7.5 \;;\; j = 0.867$$

$$A_s = \frac{248{,}000}{18{,}000 \times 0.867 \times 7.5} = 2.12 \text{ in.sq.}$$

Use 11-½ in. ⌀ bars A_s = 2.16 in.sq.

Check Compressive Stress

$$f_c = \frac{6\,R\,M_o}{0.67\,\sqrt[3]{Pn}\,l\,d^2} \quad \text{WHERE } R\,M_o = 248{,}000,\; l = 20 \times 12 = 240,\; d = 7.5$$

$$P = \frac{2.16}{10 \times 12 \times 7.5} = 0.0024$$

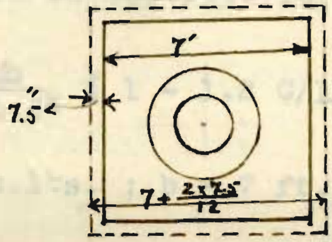
$$P\,n = 0.0024 \times 15 = 0.036$$

$$\sqrt[3]{P\,n} = \sqrt[3]{0.036} = 0.33$$

$$f_c = \frac{6 \times 248{,}000}{0.67 \times 0.33 \times 240 \times (7.5)^2} = 500 \text{ lbs/in.sq.} < 800$$

Shearing stresses in slab:

Shearing stresses in slab to be taken on a section $t_2 - 1.5 = 9 - 1.5 = 7.5$ in. from the edge of drop panel.



$$\text{Load causing shear} = V = 263\left[20 \times 20 - \left(7 + \frac{2 \times 7.5}{12}\right)^2\right]$$

$$= 263\,(400 - 68) = 87{,}500 \text{ lbs.}$$

$$\text{Shear Area} = \frac{7}{8} (t_2 - 1.5) \times 4 \times \left[b + 2 (t_2 - 1.5) \right]$$

$$= \frac{7}{8} (9 - 1.5) \times 4 \left[7 \times 12 + 2 (9 - 1.5) \right] = 2{,}600 \text{ in.sq.}$$

$$v = \frac{V}{\text{Shear Area}} = \frac{87{,}500}{2{,}600} = 33.8 \text{ lbs/in.sq.} < 40$$

Since the Compressive and shearung stresses in the slab are within the allowable limits, the assumed slab is satisfactory.

Negative Moment steel in middle strip:

M = - 248,000 in.lbs. Moment to be carried by one layer of steel near top of slab.

$d = 9 - 1.5 = 7.5$ in. ; $j = 0.867$

Since the absolute value of the moment is the same as that of the positive moment in the same strip; d, j, and f_s, being also the same,

Use as before 11-½ in φ bars ; $A_s = 2.16$ in.sq.

Compressive stress is also the same: $f_c = 500$ lbs/in.sq. < 800

Six bars of positive moment steel are bent up from ~~each~~ side and another five bars from the other side making a total of ~~eleven~~ bars to take care of the negative moment. The remaining ~~five~~ bars from each side are cut off.

Negative moment steel in two Column Strips

M = - 825,000 in.lbs. Moment to be carried by steel bent up from positive moment steel in the same strip on the two sides and some additional bars. The bars are bent near the stop of the slab at the drop panel.

$d = 12.5 - 1.5 = 11$ in. ; $j = 0.867$

$$A_s = \frac{M}{f_s\, j\, d} = \frac{825{,}000}{18{,}000 \times 0.867 \times 11} = 4.80 \text{ in.sq.}$$

Use 25-½ in. φ bars ; $A_s = 4.9$ in.sq.
9 bars from 15 are bent up for negative moment on each side making a total of 9 x 2 = 18 bars. 7 bars are also added and placed over the drop panel; the total number being now 25 bars.

The ratio of reinforcement for negative moment steel in the Column Strip shall not exceed 0.01. For the adopted arrangement:

$$p = \frac{4.9}{7 \times 12 \times 11} = 0.0053$$

Check Compressive Stress in Concrete for the negative moment in the two Column Strips:

$$f_c = \frac{3.5\, R\, M_o}{0.67 \sqrt[3]{p\, n}\;\; b\, d^2} (1 - 1.2\, C/L)$$

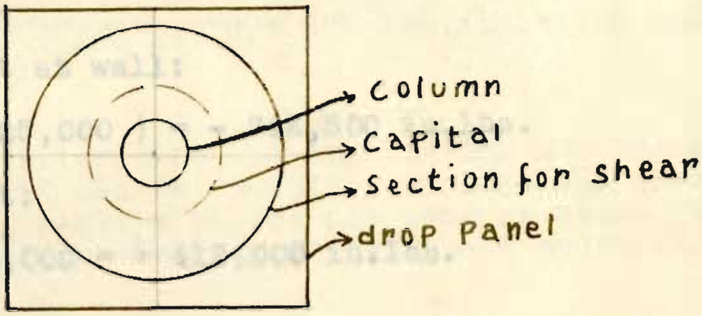
where $R\,M_o = 825{,}000$ in.lbs. ; b = 7 ft. = 84 in. ; d = 11 in.

$C/L = 4.5/20 = 0.225 \quad ; \quad P = 0.0053 \quad ; \quad n = 15$

$$f_c = \frac{3.5 \times 825,000}{0.67 \times \sqrt[3]{0.0053 \times 15} \times 84 \times (11)^2} \times (1 - 1.2 \times 0.225) = 720 \text{ lbs/in.sq.}$$

$$720 < 800$$

Shearing stresses in Drop Panel.



$$\text{Load causing shear} = V = 263\left\{ \overline{20}^2 - \pi/4 \left[C + 2(t_1 - 1.5)\right]^2 \right\}$$

$$V = 263 \left\{ \overline{20}^2 - \pi/4\left[4.5 + 2(12.5 - 1.5)\right]^2 \right\}$$

$$V = 263 (400 - 31.6) = 97,000 \text{ lbs.}$$

$$\text{Shear Area} = 7/8 (t_1 - 1.5) \pi \left[C + 2 (t_1 - 1.5) \right]$$

$$= 7/8 (12.5 - 1.5) \pi \left[4.5 \times 12 + 2(12.5 - 1.5)\right]$$

$$= 7/8 \times 11 \times 3.14 \times 76 = 2,300 \text{ in.sq.}$$

$v = V/\text{Shear Area} = 97,000/2,300 = 42$ lbs./in.sq.

The allowable shearing stress is given by:

$$\text{Allowable Shear} = 0.02 f_c' (1 + r)$$

where r = proportional amount of negative reinforcement within the column strip, crossing the column capital. In order to be on the safe side, use the minimum value of r which is 0.25.

$$v = 40 (1 + 0.25) = 50 \text{ lbs./in.sq.}$$

Since $42 < 50$, the shearing stress is satisfactory and the design of the interior panel will be also satisfactory.

N.B.: The point of inflection shall be assumed to lie at a distance from the center of the span equal to 0.25 of the span. Provision shall be made for possible shifting of the point of inflection by carrying all bars each side of a section of critical moment to points at least 20 diameters beyond the point of inflection.

EXTERIOR PANEL

The specifications of the American Concrete Institute make a definite recommendation regarding the moments to be used in an exterior panel. No increase in moments is recommended for moment sections on the first interior row of columns. Moments in the section half-way between this line of columns and the discontinuous edge of the panel are to be increased 25 % over similar normal interior sections. At the wall, or discontinuous edge of the panel, it is recommended that the negative moment in the column strip be taken as not less than 90 % and in the middle strip not less than 62.5 % of the corresponding moments for a normal interior panel. This recommendation will be adopted in the design under consideration.

Two Column Strips:

Negative Moment at wall:

M = 0.90 (- 825,000) = - 742,500 in.lbs.

Positive Moment:

M = 1.25 x 330,000 = + 412,000 in.lbs.

Middle Strip:

Negative Moment at wall:

M = 0.625 (- 248,000) = - 155,000 in.lbs.

Positive Moment:

M = 1.25 x 248,000 = + 310,000 in.lbs.

Design of the Principal Moments Strips:

Positive Moment in two Column Strips:

M = + 412,000 in.lbs. The slab conditions of the exterior panel being the same as for interior panels, the amount of steel needed is found by:

A_s = 1.25 x 2.82 = 3.52 in.sq.

Use 18-½ in. bars where A_s = 3.52

Check Compressive Stress in Concrete

$$f_c = \frac{6 \; R \; M_o}{0.67 \sqrt[3]{P \; n} \;\; l \; d^2}$$

Where R M_o = 412,000
l = 20 x 12 = 240 in.
d = 9 - 1.5 = 7.5 in.

$$P = \frac{3.52}{10 \times 12 \times 7.5} = 0.0039$$

$$f_c = \frac{6 \times 412,000}{0.67 \sqrt[3]{0.0039 \times 15} \times 240 \times (7.5)^2} = 705 \text{ lbs./in.sq.} < 800$$

Positive Moment Steel in Middle Strip:

M = 310,000 in.lbs.

A_s = 1.25 x 2.16 = 2.70 in.sq.

Use 14-½ in. bars where A_s = 2.75 in.sq.

Check Compressive Stress in Concrete

$$f_c = \frac{6\ R\ M_o}{0.67 \sqrt[3]{P\ n}\ L\ d^2}$$

$$P = \frac{2.75}{10 \times 12 \times 7.5} = 0.00305$$

$$f_c = \frac{6 \times 310,000}{0.67 \times \sqrt[3]{0.00305 \times 15} \times 240 \times (7.5)^2} = 575 \text{ lbs./in.sq.} < 800$$

Shearing Stress in Slab:

Since the exterior slab and its loading are the same as for the interior panel, it can be readily seen that the shearing stresses in the exterior panel slab are the same as those for the interior panel since half vertical shear is spread over half area shear.

Negative Moment in the middle strip:

$M = -155,000$

$A_s = 0.625 \times 2.12 = 1.33$ in.sq.

Use 7-½ inbars where $A_s = 1.37$ in.sq.

7 bars from positive moment are bent and continued to the wall to take care of the negative moment. 7 bars are continued at the bottom to the wall.

Checking Compressive Stress in Concrete:

$$f_c = \frac{6\ R\ M_o}{0.67 \sqrt[3]{P\ n}\ \ L\ d^2} \qquad R\ M_o = -155,000 \text{ in.lbs.}$$

$$P = \frac{1.37}{10 \times 12 \times 7.5} = 0.00152$$

$$f_c = \frac{6 \times 155,000}{0.67 \sqrt[3]{0.00152 \times 15} \times 240 \times (7.5)^2} = 362 \text{ lbs./in.sq.} < 800$$

Negative Moment in two Column Strips:

$M = -742,500$ in.lbs.

$A_s = 0.9 \times 4.80 = 4.32$ in.sq.

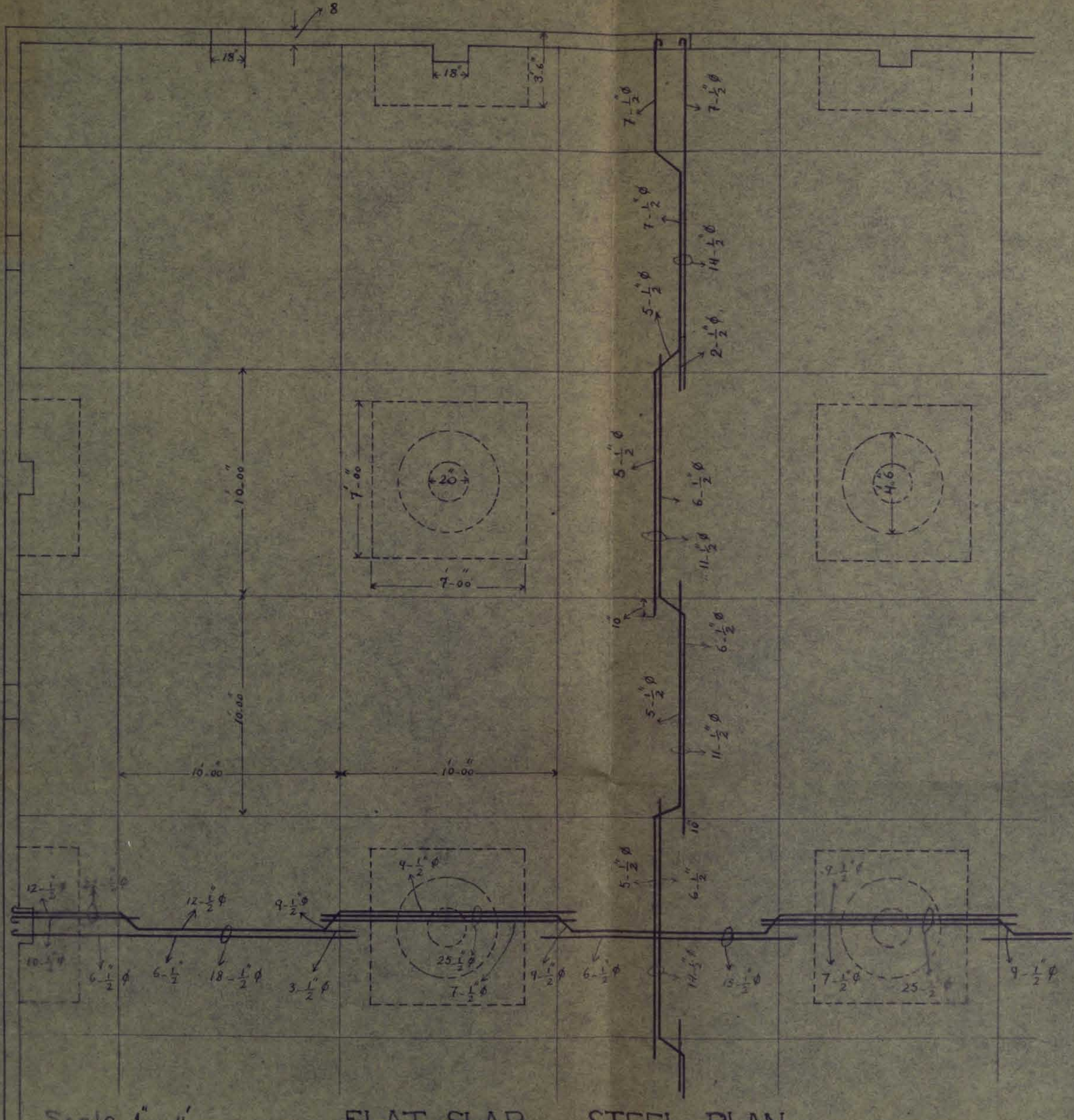
Use 22-½ in. φ bars where $A_s = 4.31$ in.sq.

12 bars of 18 of the positive moment in the two Column Strips are bent up with additional 10-½ in. φ bars to take care of the negative moment, making a total of 22 bars. The six remaining bars of the positive moments are continued directly to the wall.

Check Compressive in Concrete.

$$f_c = \frac{3.5\ R\ M_o}{0.67 \sqrt[3]{P\ n}\ \ b\ d^2} (1 - 1.2\ C/L)$$

$R\ M_o = -742,500$ in.lbs.
$b = 7 \times 12 = 84$ in.
$d = 11$ in.
$C/L = 0.225$



Scale 1" = 4'

FLAT SLAB _ STEEL PLAN

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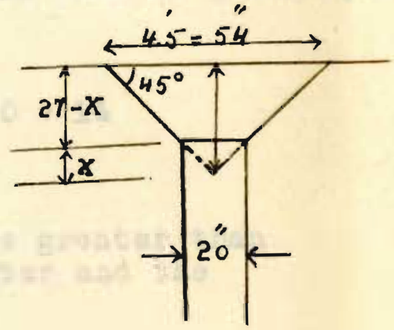
$P = \frac{4.31}{10 \times 12 \times 11} = 0.00465$; $\sqrt[3]{P\,n} = \sqrt[3]{0.00465 \times 15} = 0.413$

$$f_c = \frac{3.5 \times 742{,}500}{0.67 \times 0.413 \times 84 \times (11)^2} \times 0.73 = 675 \text{ lbs./in.sq.} < 800$$

DESIGN OF COLUMN



The diameter of the column is assumed to be 20 in.

$x/27 = 10/27$

$x = 10$ in.

Height of Capital = 27 - x = 27 - 10 = 17 in.

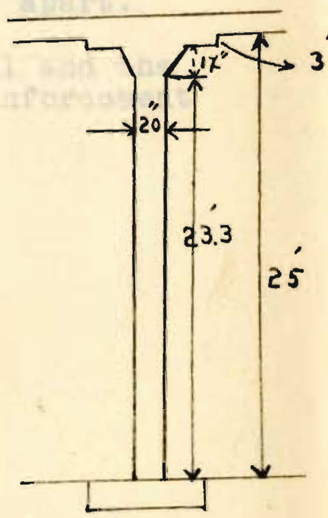
Volume of Capital = $27/3 \times \pi \times 27^2 - 10/3 \times \pi \times 10^2 = 19{,}600$ in.cu.

Weight of Capital = $\frac{150 \times 19{,}600}{12 \times 12 \times 12} = 1{,}700$ lbs.

Weight of Drop panel = $7 \times 7 \times 3.5/12 \times 150 = 2{,}150$ lbs.

Weight of slab and live load: = $20 \times 20 \times 263 = 105{,}200$ lbs.

Weight of Column:



Unsupported length = $25 - \frac{17+3.5}{12} = 23.3$ Ft.

Weight of round Column: = $\frac{\pi \times 10^2 \times 23.3 \times 150}{12 \times 12} = 7{,}650$ lbs.

Total Axial Load.... 116,700 lbs.

The amount of longitudinal reinforcement shall be considered 0.5 % of the total area of the column. The longitudinal reinforcement shall consist of four round bars of ¾ (three quarters) in. in diameter, placed with a clear distance of two inches from the face of the column.

$A_s = 0.005 \times \pi \times 10^2 = 1.57$ in.sq.

Use 4-3/4 in. φ bars where $A_s = 1.77$ in.sq.

For short columns, the safe axial load on columns reinforced with longitudinal bars and seperate lateral ties shall be not greater than that determined by the following formula:

$$P = (A_c + A_s\,n)\,f_c$$

Where A_c' = net area of concrete in the column = $\pi \times 10^2 - 1.77$ =
= 312.40 in.sq.

$A_s = 1.77$; $n = 15$

f_c = Permissible compressive stress in concrete
= $0.225\ f_c'$ = 0.225 x 2,000 = 450 lbs./in.sq.

P' = (312.40 + 1.77 x 15) 450 = 152,550 lbs.

For long columns, as in this case, the safe axial load will be given by:

$P = P'\ (1.3 - 0.03\ L/D)$

L = 23.3 x 12 = 280 in. ; D = 20 in. ; L/D = 280/20 = 14

P = 152,550 (1.3 - 0.03 x 14) = 134,250 lbs.

Since 134,250 lbs. (Allowable axial load is greater than 116,700 lbs., actual axial load,), the assumed diameter and the longitudinal reinforcement are satisfactory.

Lateral Ties:

Use lateral ties of 1/4 in. φ bars, 8 in. apart.

N.B. The longitudinal bars are extended to the capital and the footing; four extra bars of ½ in. are added to the reinforcement of the capital.

DESIGN OF FOOTING

Total axial load of the column = 116,700 lbs.

Assuming weight of footing = 8,300 lbs.

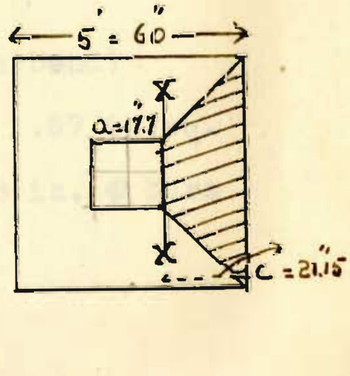
Total Load on Foundation = 125,000 lbs.

Bearing Power of Soil = 8,000 lbs./ft.sq.

Area of Footing = 125,000/8,000 = 15.6 ft.sq.

In order to be on the safe side, use a square footing of 5 ft.

Actual bearing power w = 125,000/25 = 5,000 lbs./ft.sq.



The square column equivalent to 20 in. round one is $\sqrt{\pi r^2} = \sqrt{\pi \times 10^2}$ = 17.7 in.

$M_{xx} = w/2\ (a + 1.2\ C)\ C^2$

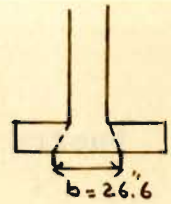
$= 5,000/2(17.7/12 + 1.2 \times 21.15/12)(21.15/12)^2 \times 12$

= 337,000 in.lbs.

The effective width of bearing is assumed to be about 1.5 times the width of the equivalent square column i.e. b = 1.5 x 17.7 = 26.6

$R = f_c/2 \times K\, j = 800/2 \times 0.4 \times 0.867$

$R = 138 \quad ; \quad b = 26.6$



$d = \sqrt{M/R\,b} = \sqrt{\frac{337,000}{138 \times 26.6}} = 9.6$ in.

Use d = 13 in. in order to take care of Diagonal Tension and specially bond stress because the bond resistance is one of the most important factors governing the strength of column footings.

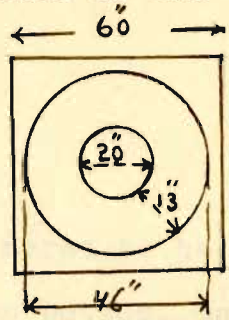
Using 3 in. for covering, the depth of the footing will be 13 + 3 = 16 in.

Check for weight of footing:

Weight of Footing = 16/12 x 25 x 150 = 5,000 lbs. < 8,300 (assumed)

Check for Diagonal Tension:

The critical Section for diagonal tension is a circular vertical section at a distance of d = 13 in. from the perimeter of the actual column.



$v = V/b\,j\,d$

$V = \left[25 - \pi/4(46/12)^2\right] 5,000 = 70,000$ lbs.

$b = \pi \times 46 = 145$ in. ; $d = 13$ in. ; $j = 0.867$

$v = \frac{70,000}{145 \times 0.867 \times 13} = 42.8$ lbs./in.sq. < 60

Use hooked bars, the allowable unit shear = $0.03\, f_c' = 0.03 \times 2,000$
= 60 lbs./ in.sq.

Design for Steel:

$A_s = M/f_s\,j\,d = \frac{337,000}{18,000 \times 0.867 \times 13} = 1.67$ in.sq.(for each band)

Design for Bond:

Bars are hooked at both ends, the allowable bond stress given by the Joint Committee for a $f_c' = 2,000$ is $u = 112.5$ lbs./in.sq.

$\Sigma_o = V/u\,j\,d$

$V = 1/4 \times 5,000 \left[25 - \pi/4(20/12)^2\right] = 28,600$ lbs.(for each band)

$\Sigma o = \frac{28,600}{112.5 \times 0.867 \times 13} = 22.5$ in. & A_s as calculated = 1.67 in.sq.

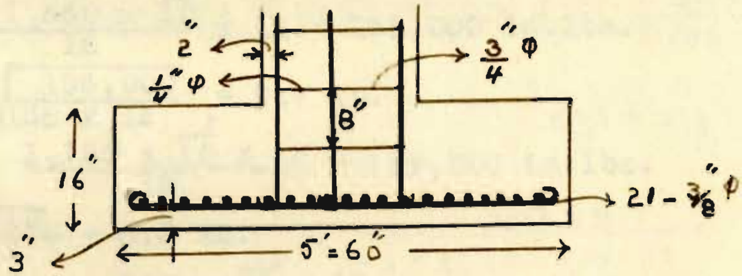
These requirements are satisfied by using 19-3/8 in. φ bars hooked at both ends.

The spacing of bars = 53/19 = 2.88 in.

One additional bar is placed at each side. Allowing 3 in. from each side for covering the spacing of bars:

$$= \frac{60 - 6}{19 + 2} = \frac{54}{21} = \underline{2.5 \text{ in.}}$$

The reinforcement is to be two-way and the bars will extend to 3 in. from the edge of the column.



D E S I G N   O F   W A L L S

I - 1st. Case.: Water is acting alone.

Buttresses are 10 ft. apart
Height of wall = 25 ft.
1 Cu. ft. of water weighs 62.5 lbs.
The thickness of wall at the top is considered to be 8 in.

In order to calculate its thickness at the bottom, consider a horizontal strip 1 ft. high, 25 ft. down from the top of the wall, the pressure per linear foot:

$$w_a = 62.5 \times 25 = 1,560 \text{ lbs.}$$

Since shear governs in this case as will be shown,

$$d = V/v\, b\, j$$

$$V = w_a l/2 = 1,560 \times 10 / 2 = 7,800 \text{ lbs.}$$

$b = 12$ in. ; $j = 0.867$; $v = 60$ lbs.(with special anchorage)

$$\text{Hence } d = \frac{7,800}{60 \times 12 \times 0.867} = 12.5 \text{ in.}$$

Use d = 13 in.

Allowing 2 in. for covering the total thickness at the bottom = 13 + 2 = 15 in.

Pressures at sections a, b, c, & d:

$w_a = 62.5 \times 25 = 1,560$ lbs./ft.sq.

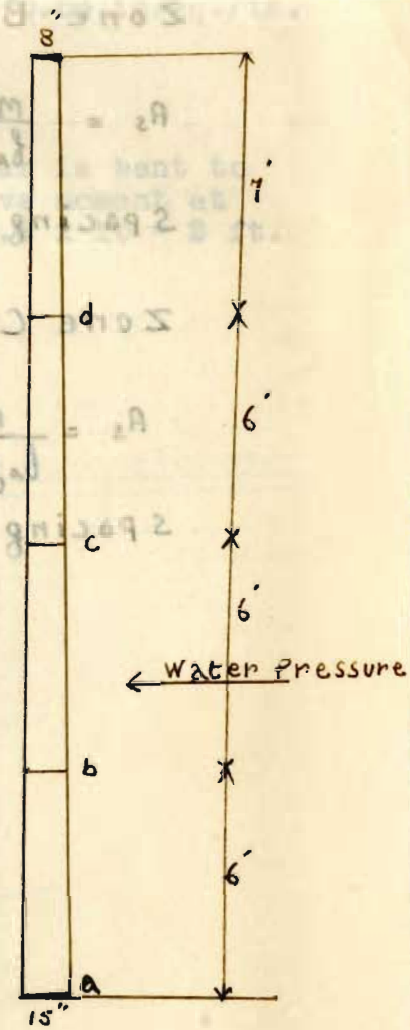
$w_b = 62.5 \times 19 = 1,190$ lbs./ft.sq.

$w_c = 62.5 \times 13 = 825$ lbs./ft.sq.

$w_d = 62.5 \times 7 = 440$ lbs./ft.sq.

Moments at the same sections:

$L = 10$ ft. (Distance between buttresses)



$M_a = w_a L^2/12 = \frac{1,560 \times \overline{10}^2}{12} \times 12 = 156,000$ in.lbs.

$d_a = \sqrt{\frac{M_a}{R\ b}} = \sqrt{\frac{156,000}{138 \times 12}} = 9.7$ in.

$M_b = \frac{w_b \times L^2}{12} = \frac{1,190 \times \overline{10}^2 \times 12}{12} = 119,000$ in.lbs.

$d_b = \sqrt{\frac{119,000}{138 \times 12}} = 8.5$ in.

$M_c = \frac{w_c L^2}{12} = \frac{825 \times \overline{10}^2 \times 12}{12} = 82,500$ in.lbs.

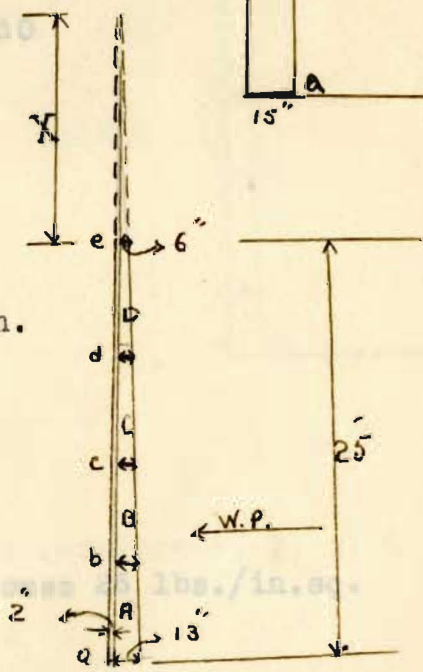
$d_c = \sqrt{\frac{82,500}{138 \times 12}} = 7.2$ in.

$M_d = w_d \times L^2 / 12 = \frac{440 \times \overline{10}^2 \times 12}{12} = 44,000$ in.lbs.

$d_d = \sqrt{\frac{44,000}{138 \times 12}} = 5.2$ in.

Actual d for sections a, b, c, & d :

Allowing 2 in. for covering, the actual d for sections will be considered as follows:



$d_a = \underline{13}$ in. $>$ 9.7 in. (found by moment)

d_b : Similarity of triangles

$$\frac{x}{6} = \frac{25 + x}{13} \qquad x = 21.4 \text{ ft.}$$

$$\frac{d_b}{13} = \frac{25 - 6 + 21.4}{25 + 21.4} \qquad \text{Hence } d_b = 11.3 \text{ in.}$$

d_c is found to be 9.6 in.

d_d = 8.15 in.

Design for Steel :

Zone A 6 ft. high.

$$A_s = \frac{M_a}{f_s\ j\ d_a} = \frac{156,000}{18,000 \times 0.867 \times 13} = 0.77 \text{ in.sq./ft. of height} = 0.0642 \text{ in.sq./in.}$$

Use 3/8 in. φ bars $A_s = 0.11$

Spacing = 0.11/0.0642 = 1.7 in. center to center to a height of 6 ft.

Read on the back please

Zone **D**

$A_s = \frac{M_d}{f_s\ j\ d_d} = \frac{44,000}{18,000 \times 0.867 \times 8.15} = 0.346$ in.sq./ft. = 0.03 in.sq./in.

Spacing = 0,11/0.03 = 3.7 in. to a height of 7 ft.

N.B. : In the above design for steel, every second bar is bent to the opposite face of the slab to provide for the negative moment at the supports at a distance from the buttress equal to 0.2 x 10 = 2 ft.

Shear stresses at sections a, b, c, & d :

Zone A. The critical shear is at section a.

$$v_a = V_a / b\ j\ d_a$$

$V_a = \frac{1,560 \times 10}{2} = 7,800$ lbs. ; b = 12 in. ; d_a = 13 in.

$v_a = \frac{7,800}{12 \times 0.867 \times 13} = 57.7$ lbs./in.sq. < 60

Use hooked bars in Zone A.

Zone B.

$V_b = \frac{1,190 \times 10}{2} = 5,950$; d_b = 11.3 in.

$v_b = \frac{5,950}{12 \times 0.867 \times 11.3} =$ 51 lbs./in.sq. < 60

Use also hooked bars in Zone B.

Zone C.

$V_c = \frac{825 \times 10}{2} = 4,125$; d_c = 9.6 in.

$v_c = \frac{4,125}{12 \times 0.867 \times 9.6} = 41.5$ lbs.

No need for hooked bars.

Zone D.

In similar way, the unit shear becomes 26 lbs./in.sq.

Bond Stress.

Check for the critical section at a.

$u = V_a / \Sigma o\ j\ d_a$ $V_a = 7,800$; d_a = 13 in. ; $\Sigma o = 8.25$

$u = \frac{7,800}{8.25 \times 0.867 \times 13} = 84$ lbs./in. < 112.5

Temperature Reinforcement:

A_s = 0.0025 x 13 x 12 = 0.39 in.sq./ft. = 0.0325 in.sq./in

Use 3/8 in. φ bars. A_s = 0.11

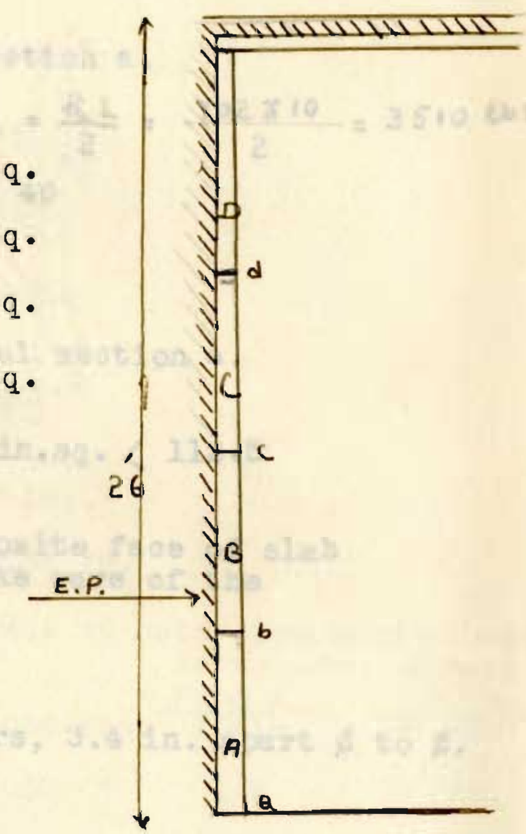
Spacing = 0.11/0.0325 = 3.4 in. center to center.

These vertical bars are embedded at the bottom in the floor and at the top in the flat slab.

II. Snd Case.: Pressure due to earth alone (Reservoir is empty)

Data:

1 cubic foot of earth weighs 100 lbs.
Angle of repose = 35°, K = 0.27



Pressures:

P_a = K w h = 0.27 x 100 x 26 = 702 lbs./ft.sq.

P_b = 0.27 x 100 x 20 = 540 lbs./ft.sq.

P_c = 0.27 x 100 x 14 = 378 lbs./ft.sq.

P_d = 0.27 x 100 x 8 = 216 lbs./ft.sq.

Moments.

$$M_a = \frac{P_a l^2}{12} = \frac{702 \times \overline{10}^2 \times 12}{12} = 70{,}200 \text{ in.lbs.}$$

$$M_b = \frac{P_b l^2}{12} = \frac{540 \times \overline{10}^2 \times 12}{12} = 54{,}000 \text{ in.lbs.}$$

$$M_c = \frac{P_c l^2}{12} = \frac{378 \times \overline{10}^2 \times 12}{12} = 37{,}800 \text{ in.lbs.}$$

$$M_d = \frac{P_d l^2}{12} = \frac{216 \times \overline{10}^2 \times 12}{12} = 21{,}600 \text{ in.lbs.}$$

Design for Steel.

Allowing 2 in. for covering, d at sections a, b, c, & d is considered as before.

Zone A.

$$A_s = \frac{70{,}200}{18{,}000 \times 0.867 \times 13} = 0.346 \text{ in.sq./ft.} = 0.0288 \text{ in.sq./in.}$$

Use 3/8 in. φ bars. A_s = 0.11

Spacing = 0.11/0.0288 = 3.8 in. center to center to a height of 6 ft.

Zone B.

$$A_s = \frac{54,000}{18,000 \times 0.867 \times 11.3} = 0.306 \text{ in.sq./ft.} = 0.0255 \text{ in.sq./in.}$$

Spacing = 0.11/0.0255 = 4.3 in. ∅ to ∅ to a height of 6 ft.

X

Zone D.

$$A_s = \frac{21,600}{18,000 \times 0.867 \times 8.15} = 0.17 \text{ in.sq./ft.} = 0.0142 \text{in.sq./in.}$$

Spacing = 0.11/0.0142 = 7.75 in. ∅ to ∅ to a height of 7 ft.

Check for shear at the critical section a.

$v_a = V_a / b\ j\ d_a$ $\qquad V_a = \frac{P_a L}{2} = \frac{702 \times 10}{2} = 3510$ lbs.

$$v_a = \frac{3,510}{12 \times 0.867 \times 13} = 26 \text{ lbs./in.sq.} < 40$$

No need for hooked bars

Check for bond strdss at the critical section a.

$$u = \frac{V_a}{\Sigma o\ j\ d_a} = \frac{3,510}{3.53 \times 0.867 \times 13} = 88 \text{ lbs./in.sq.} < 112.5$$

Every second bar is bent to the opposite face of slab at a distance of 2 ft. from buttress to take care of the negative moment there.

Temperature Reinforcement:

Use as before:vertical 3/8 in. φ bars, 3.4 in. apart ∅ to ∅.

D E S I G N O F B U T T R E S S

I - 1st. case (Pressure due to water alone).

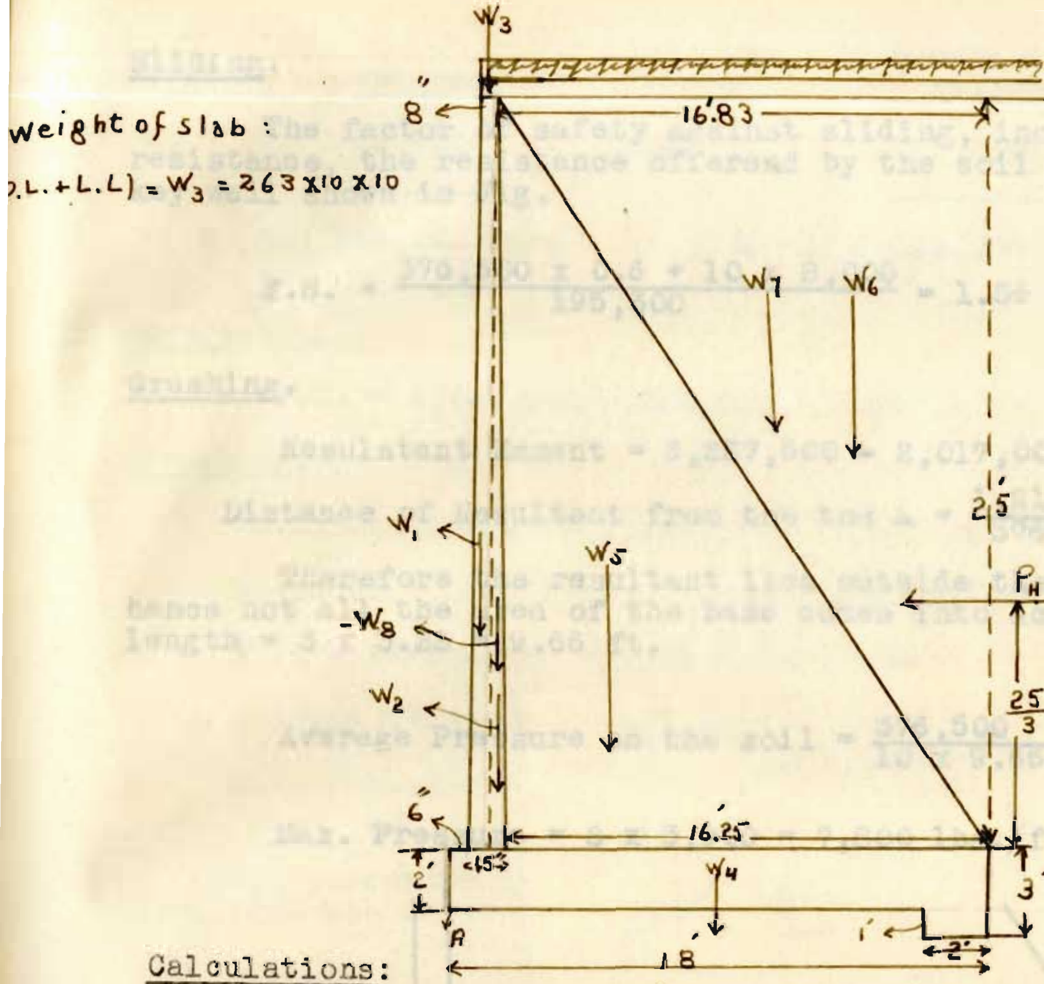
DATA:

Spacing between buttresses = 10 ft. Height = 25 ft.
Thickness of Buttress = 18 in. Base = 18 ft.(assumed)
All other dimensions are shown in Fig. on the following page.
Bearing power of soil = 8,000 lbs./ft.sq.
Coefficient of sliding = 0.6
Allowable unit stresses as before.

X : Zone C

$$A_s = \frac{37,800}{18,000 \times 0.867 \times 9.6} = 0.252 \text{ in.sq./ft.} = 0.021 \text{ in.sq./in}$$

$$\text{Spacing} = \frac{0.11}{0.0212} = 5 \text{ in. ¢ to ¢ to a height of 6 ft.}$$



Calculations:

$P_H = w\,h^2/2 \times 10 = \dfrac{62.5 \times \overline{25}^2}{2} \times 10 = 195,300$ lbs.

Its moment arm from A = 2 + 25/3 = 10.33 ft.

M_A = 195,300 x 10.33 = 2,017,000 ft.lbs. (due to water pressure alone)

		lever arm from A	Moment in.lbs.
w_1 = 10 x 8/12 x 25 x 150	= 25,000 lbs.	10/12	20,800
w_2 = 10 x 7/12 x 25/2 x 150	= 11,000 "	4/3	14,700
w_3 = 2,630 x 10	= 26,300 "	10/12	22,000
w_4 = 10 x 18 x 2 x 150	= 54,000 "	9	486,000
w_5 = 16.25/2 x 25 x 18/12 x 150	= 45,000 "	7.15	325,000
w_6 = 16.83/2 x 25 x 18/12 x 62.5	= 19,700 "	12.40	244,000
w_7 = 16.83 x 25 x 8.5 x 62.5	= 223,700 "	9.58	2,140,000
$-w_8$ =-7/12 x 25/2 x 8.5 x 62.5	= - 3,900 "	4/3	- 5,200
W	= 376,500 " ;	M =	3,227,500 'lbs.

Overturning

Factor of Safety = $\dfrac{3,227,500}{2,017,000}$ = 1.6

Sliding.

The factor of safety against sliding, including in the sliding resistance, the resistance offerend by the soil in front of 1 ft. key wall shown in Fig.

$$F.S. = \frac{376,500 \times 0.6 + 10 \times 8,000}{195,300} = 1.56$$

Crushing.

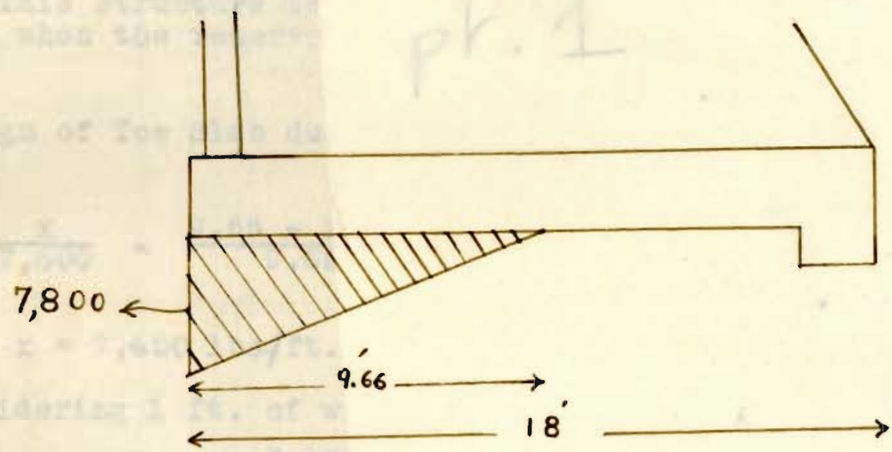
Resulatant Moment = 3,227,500 - 2,017,000 = 1,210,500 ft.lbs.

$$\text{Distance of Resultant from the toe A} = \frac{1,210,500}{376,500} = 3.22 \text{ ft.}$$

Therefore the resultant lies outside the middle third and hence not all the area of the base comes into action. The effective length = 3 x 3.22 = 9.66 ft.

$$\text{Average Pressure on the soil} = \frac{376,500}{10 \times 9.66} = 3,900 \text{ lbs./ft.sq.}$$

Max. Pressure = 2 x 3,900 = 7,800 lbs./ft.sq. $<$ 8,000



II - 2nd Case. (Pressure due to earth alone; Reservoir is empty)

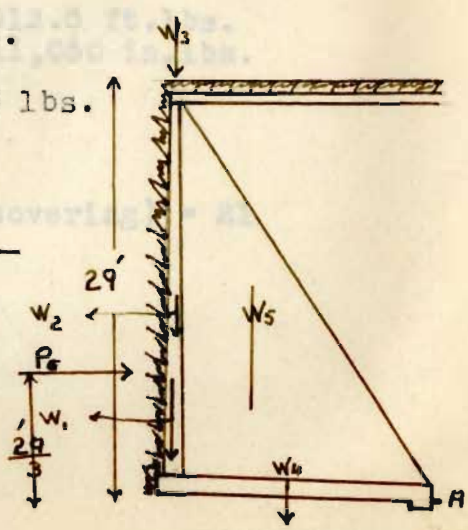
Height of earth = 25 + 2 + 1 + 9/12 = 28.75 ft almost 29 ft.

K = 0.27 ; Weight of 1 cu. ft. = 100 lbs.

$$P_E = \frac{K\,w\,h^2}{2} \times 10 = \frac{0.27 \times 100 \times 29^2}{2} \times 10 = 114,000 \text{ lbs.}$$

M_A = 114,000 x 29/3 = 1,100,000 ft.lbs.

Symbols	Weights	Liver arm from A	Moment in ft.lbs.
W_1	25,000	17 1/6	430,000
W_2	11,000	16 2/3	183,000
W_3	26,300	17 1/6	452,000
W_4	54,000	9	486,000



Symbols	Weights	Liver arm from A	Moment in ft.lbs.
w_5	45,700	10.85	495,000

W = 162,000 lbs. M = 2,046,000 ft.lbs.

Overturning.

F.S. = 2,046,000/1,100,000 = 1.86

Crushing.

Resultant Moment = 2,046,000 - 1,100,000 = 946,000 ft.lbs.

Distance of Resultant from A = 946,000/162,000 = 5.84 ft.

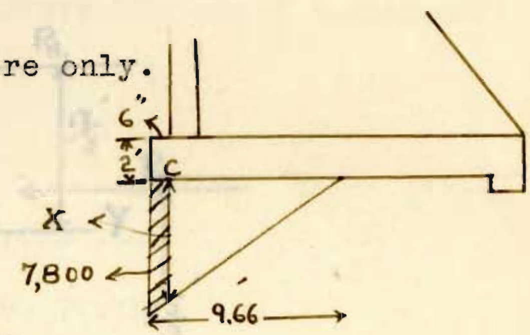
Effective length = 5.83 x 3 = 17.52

$$\text{Average Pressure} = \frac{162,000}{10 \times 17.52} = 925 \text{ lbs./ft.sq.}$$

Max. Pressure = 925 x 2 = 1,850 lbs./ft.sq.

This structure is safe against earth pressure even when the reservoir is empty.

Design of Toe Slab due to water pressure only.



$$\frac{x}{7,800} = \frac{9.66 - 0.5}{9.66}$$

x = 7,400 lbs./ft.sq.

Considering 1 ft. of width of Toe Slab:

$$\text{Upward pressure} = \frac{7,800 + 7,400}{2} \times 0.5 \times 1 = 3,800 \text{ lbs.}$$

Center of Gravity of Area is approximately at 0.25 ft. from C.

M_c = 3,800 x 0.25 - 2 x 0.5 x 1 x 150 x 0.25 = 912.5 ft.lbs.
= 11,050 in.lbs.

$$A_s = \frac{11,050}{18,000 \times 0.867 \times 21} = 0.034 \text{ in.sq./ft.}$$

d = 24 - 3 (for covering) = 21

Shear Stress.

V = 3,800 - 150 = 3,650 lbs.

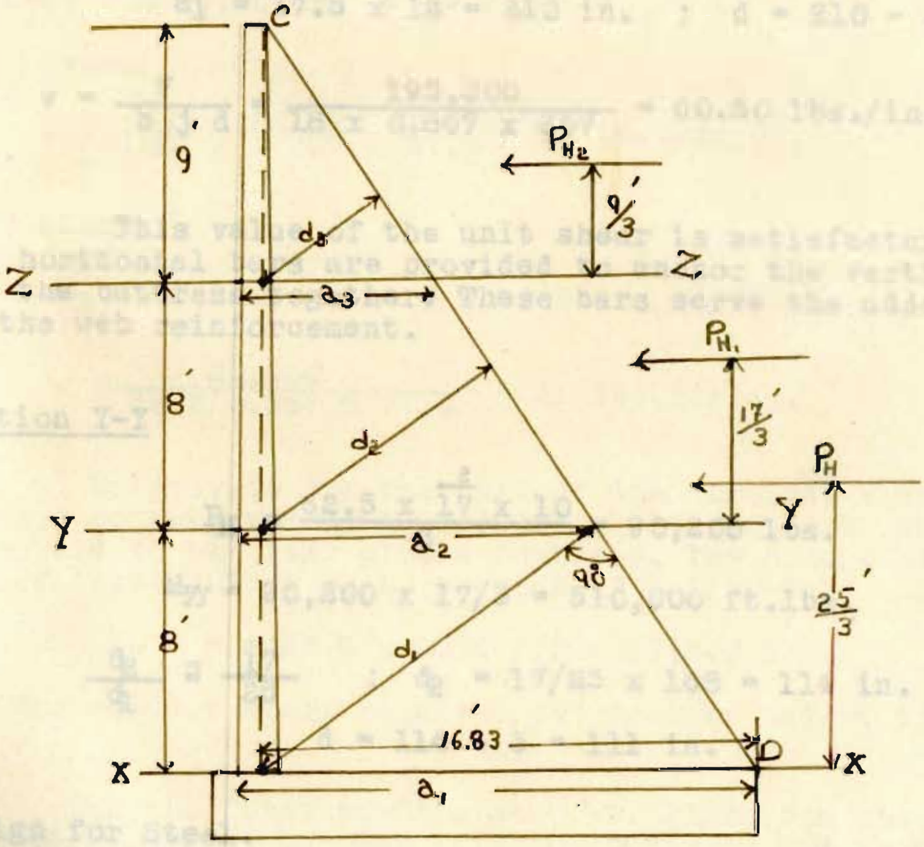
$$v = \frac{3,650}{12 \times 0.867 \times 21} = 16.8 \text{ lbs./in.sq;}$$

Bond Stress.

$$\Sigma o = \frac{V}{u\ j\ d} = \frac{3,650}{112.5 \times 0.867 \times 21} = 1.79 \text{ in.}$$

These requirements are satisfied by using 2-3/8 in. φ bars per foot of width. These bars are continued to the heel, to furnish transverse reinforcement in the heel slab.

DESIGN OF BUTTRESS (itself, due to water pressure)



Section x-x

$$M_{xx} = 195,300 \times 25/3 = 1,630,000 \text{ ft.lbs.}$$

$$\text{Length of CD} = \sqrt{25^2 + 16.83^2} = 30.10 \text{ ft.}$$

$$\frac{d_1}{16.83} = \frac{25}{30.10} \qquad d_1 = 14 \text{ ft.} = 168 \text{ in.}$$

Allowing 3 in. for covering: d = 168 - 3 = 165 in.

Design for Steel.

$$A_s = \frac{1,630,000 \times 12}{18,000 \times 0.867 \times 165} = 7.60 \text{ in.sq.}$$

Use 10-1 in. ϕ bars where Σo = 31.42 in.

Bond Stress.

$$u = \frac{V}{\Sigma o\ j\ d} = \frac{195,300}{31.42 \times 0.867 \times 165} = 43.5 \text{ lbs./in.sq.}$$

Shearing Stress.

The effective depth to be used in determining the unit shear stress on the base of the buttress is equal to the horizontal distance to the reinforcing steel.

$$a_1 = 17.5 \times 12 = 210 \text{ in.} \quad ; \quad d = 210 - 3 = 207 \text{ in.}$$

$$v = \frac{V}{b\ j\ d} = \frac{195,300}{18 \times 0.867 \times 207} = 60.50 \text{ lbs./in.sq. (where b=18")}$$

This value of the unit shear is satisfactory because the horizontal bars are provided to anchor the vertical slab and the buttress together. These bars serve the added function of the web reinforcement.

Section Y-Y

$$R_{M_1} = \frac{62.5 \times 17^2 \times 10}{2} = 90,200 \text{ lbs.}$$

$$M_{yy} = 90,200 \times 17/3 = 510,000 \text{ ft.lbs.}$$

$$\frac{d_2}{d_1} = \frac{17}{25} \quad ; \quad d_2 = 17/25 \times 168 = 114 \text{ in.}$$

$$d = 114 - 3 = 111 \text{ in.}$$

Design for Steel.

$$A_s = \frac{510,000 \times 12}{18,000 \times 0.867 \times 111} = 3.54 \text{ in.sq.}$$

Use 5-1" ϕ bars where Σo = 15.71 in.

Bond Stress.

$$u = \frac{90,200}{15.71 \times 0.867 \times 111} = 60 \text{ lbs./in.sq.}$$

Shear Stress.

$$a_2 = 145 \text{ in.} \; ; \; d = 145 - 3 = 142$$

$$v = \frac{90,200}{18 \times 0.867 \times 142} = 40.70 \text{ lbs./in.sq.}$$

Section Z-Z.

$$P_{H_2} = \frac{62.5 \times \overline{9}^2 \times 10}{2} = 25,300 \text{ lbs.}$$

$$M_{ZZ} = 25,300 \times 9/3 = 75,900 \text{ ft.lbs.}$$

d_3 = 168 x 9/25 = 60.5 in. therefore d = 60.5 - 3 = 57.5 in.

Design for Steel.

$$A_s = \frac{75,900 \times 12}{18,000 \times 0.867 \times 57.5} = 1.01 \text{ in.sq.}$$

Use 2-1" φ bars, where Σo = 6.28 in.

Bond Stress.

$$u = \frac{25,300}{6.28 \times 0.867 \times 57.5} = 81 \text{ lbs./in.sq.}$$

Shear Stress.

a_3 = 80.5 in. d = 80.5 - 3 = 77.5 in.

$$v = \frac{25,300}{18 \times 0.867 \times 77.5} = 21 \text{ lbs./in.sq.}$$

To provide for the pull of the vertical slab, horizontal bars are so arranged as to hook over the vertical slab reinforcement and extend to the rear of the buttress. The amount of pull at the base of the vertical slab per foot of height is:

$$P = 62.5 \times 25 \times 8.5 = 13,300 \text{ lbs.}$$

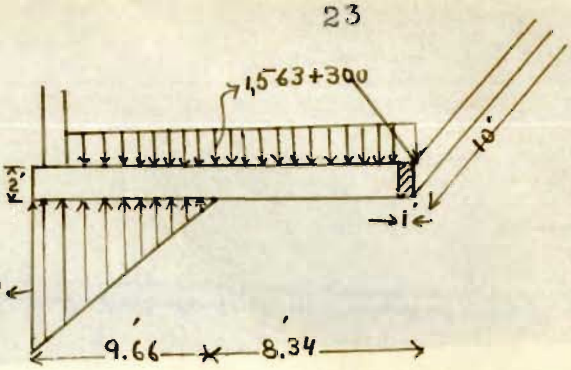
The area of steel required per foot of height is $\frac{13,300}{18,000}$ = 0.74 in.sq.

For the first section at 8 ft. high from the base, 1-1"φbar per foot of height is bent into the buttress and anchored to the vertical slab. This procedure is fulfilled in the Zone between 4' & 8' of height. 3 extra bars of the same dimension are added in the same way at the bottom. For the other sections, the procedure is the same but the spacing of bars will be greater.

Design of the Base:

The rear portion of the base is designed as a simple beam supported by the buttress. The load on the slab is equal to the difference between the downward load of the water and the upward soil pressure at the rear part of the heel slab, downward load of the water and the weight of the slab itself will be only considered.

The load distribution is shown in Fig.



Considering a strip of 1' wide:

Load of water = w h = 62.5 x 25 = 1,563 lbs./ft.length.

Weight of beam = 1 x 2 x 1 x 1.50= 300

Total weight = 1,863 lbs./ft.

$$M = \frac{1{,}863 \times \overline{8.5}^{2}}{8} = 16{,}800 \text{ ft.lbs.} \qquad L = 8.5 \text{ ft.}$$

$$V = \frac{1{,}863 \times 8.5}{2} = 7{,}920 \text{ lbs.}$$

The depth required is governed in this case by the shear.

$$d = \frac{7{,}920}{40 \times 0.867 \times 12} = 19 \text{ in.} \qquad \text{Use } d = 20 \text{ in.}$$

Allowing 4" for covering, the total depth = 20 + 4 = 24" or 2 ft. as assumed.

Design for Steel.

$$A_s = \frac{16{,}800 \times 12}{18{,}000 \times 0.867 \times 20} = 0.645 \text{ in.sq./ft.} = 0.0537 \text{ in.sq./in.}$$

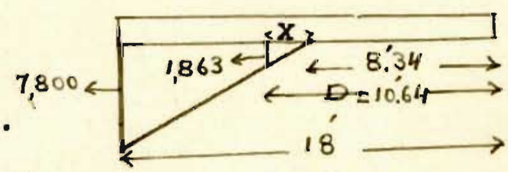
Using ½" φ bars, A_s = 0.20

Spacing = 0.20/0.0537 = 3.7 in. ⌀ to ⌀

The steel must be placed at the bottom of the slab.

The point where the upward pressure becomes equal to the downward load is located by:

$$\frac{x}{1{,}863} = \frac{9.66}{7{,}800} \; ; \quad x = 2.3 \text{ ft.}$$



D = 2.30 + 8.34 = 10.64 ft. from heel.

The heel slab steel in front of this point is required at the top of the slab.

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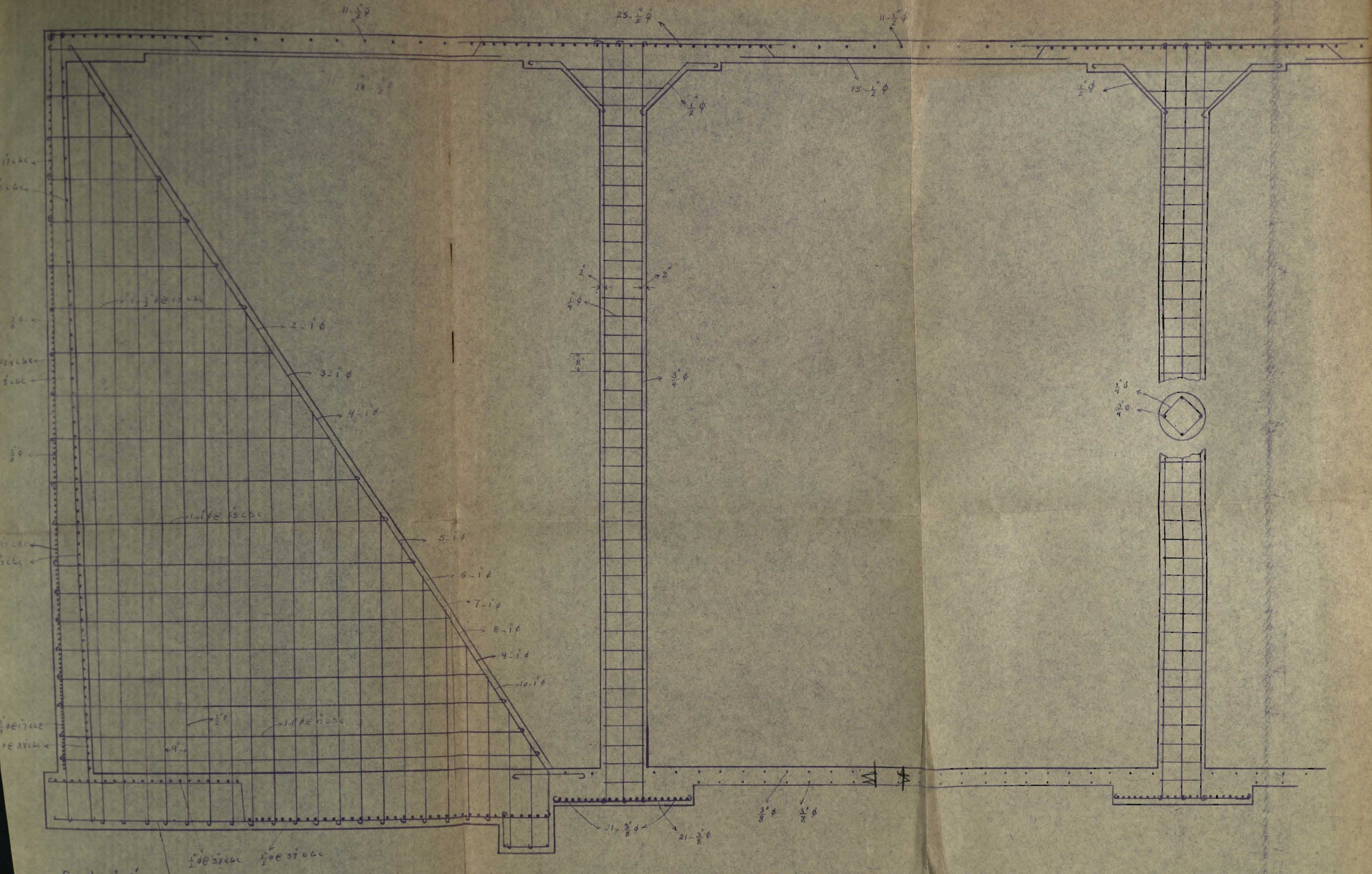
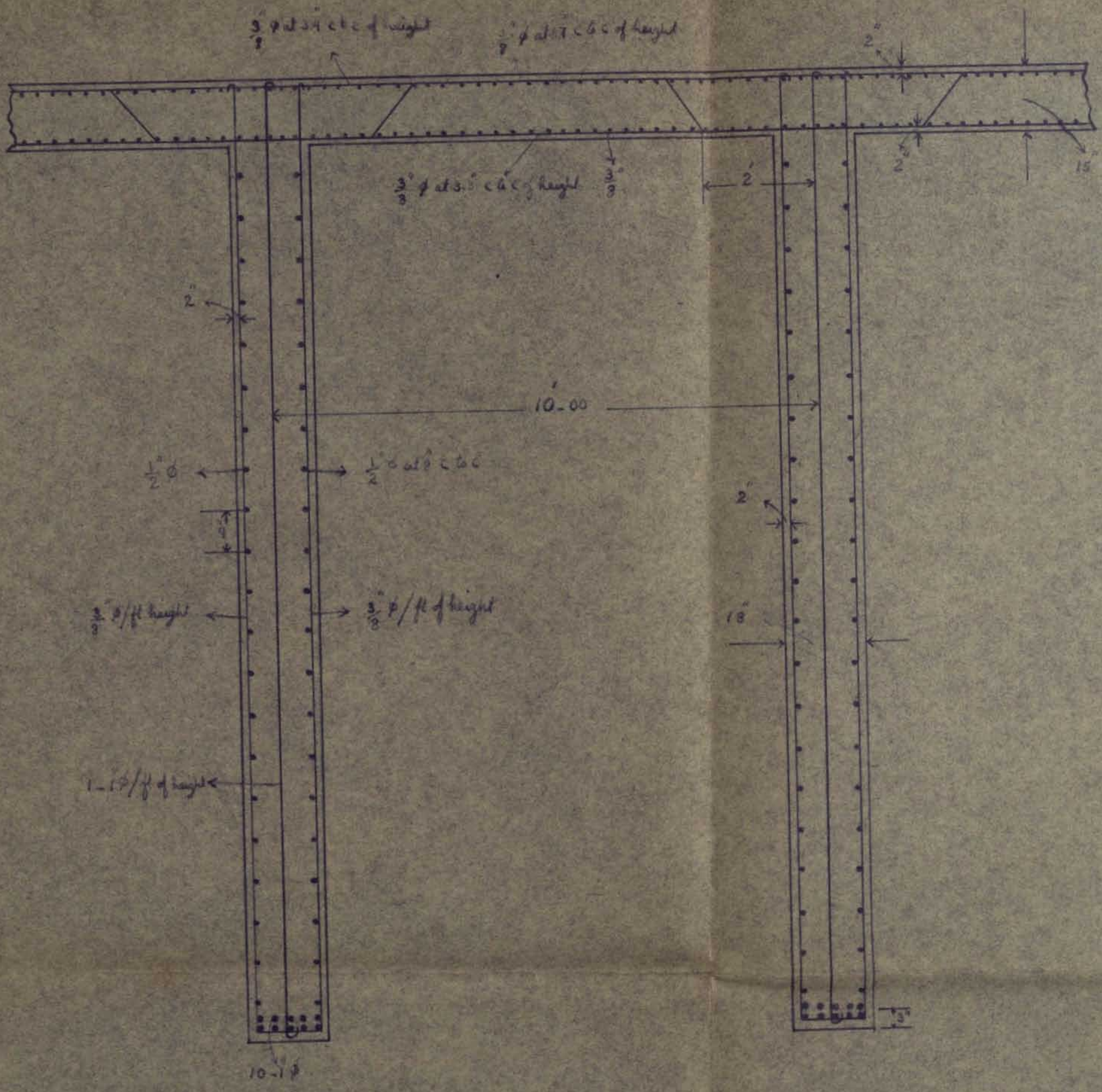


Scale 1" = 2'

SECTION THRU TWO COLUMN STRIPS

Wadi Zahka



Scale 1"=2'   HORIZONTAL SECTION THRU BUTTRESSES AND EXTERIOR WALL

Wadi Zahka