AMERICAN UNIVERSITY OF BEIRUT

BEIRUT LEBANON

Department of Civil Engineering

THESIS

by

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on

Water Tank Design

American University of Beirut

February 1945

Foreword

Water Tanks are extensively used for various purposes. Their construction suppose a great deal of knowledge of Reinforced Concrete Structures. These are the reasons which decided me to study very closely water tanks connected specially with their skeletons.

Very much credit is due Professor Hilu for his supervision and kind assistance. May he find here my warmest thanks.

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Emile Keat

Contents

Notation

A_c	Cross section area of concrete
A_s	Cross section area of steel reinforcement
b	Breadth
c	Diameter of column capital
d	Effective depth of beams
f_c	Unit stress in concrete
f_s	Unit stress in steel reinforcement
jd	Lever arm of tension and compression in beams
kd	Depth of neutral axis
l	Span
M_o	Summation of moments
M	Moment
n	Ratio of moduli of elasticity of steel to concrete
o	Perimeter of one reinforcing bar
P	Load
p	Percentage of reinforcing steel
R	Coefficient $= \frac{1}{2} f_c k j$
s	Spacing of bars
T	Tension
t	Thickness
u	Unit bond strength
V	Shear
v_c	Shear carried by concrete (unit)
v_s	Unit shear carried by steel
W	Total weight
w	Load per square foot
x	Any distance

DESIGN OF A

4,000,000 Gallons

WATER TANK

by Emile Klat

Length = 16 @ 12 = 192 feet

Breadth = 12 @ 12 = 144 feet

Height = 24 feet

Assumptions for the Reinforced Concrete Calculations:

f_s = 16,000 lbs p. sq. inche

f_c = 650 "

f_{s_s} = 14,000 "

v_c = 50 "

n = 15

from which it follows:

k = .379

j = .867

R = 108

I. Flat Slab

a) Interior Panels (24' x 24')

$$\Sigma M = 0.09\, \omega\, l^3 \left(1 - \frac{2c}{3l}\right)^2$$

for 2 way reinforcement without dropped panels the moments to be used are:

Strip	Negative	Positive
2 Columns	0.46 M_o	0.22 M_o
Middle	0.16 M_o	0.16 M_o

Let us assume

$$\omega = 125 + 125 = 250 \text{ lbs/sq.ft}$$

$$c = 6 \text{ ft}$$

Then

$$\Sigma M = 0.09 \times 250 \times 24^3 \left(1 - \frac{2 \times 6}{3 \times 24}\right)^2$$

$$= \underline{216000 \text{ ft lbs}}$$

The negative moment carried by the 2 columns strips is:

$$0.46 \times 216\,000 = 99500 \text{ ft lbs}$$

The width being 12 feet

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{99500}{108 \times 12}} = 8.75 \text{ inches}$$

for covering 1.25 "

Total Depth 10.00 inches

Dead load: $\frac{10}{12} \times 150 = 125$ lbs/sq.ft.

∴ 1st Assumption is correct.

Reinforcement

Negative moment 2 columns strips

$$A_s = \frac{M}{f_s j d} = 0.46 \frac{216000 \times 12}{16000 \times 0.867 \times 8.25} = 0.46 \times 22.6$$

$$= 10.41 \text{ sq. inches} = 24 \phi \, 3/4''$$

Positive moment 2 columns strips

Similarly $A_s = 0.22 \times 22.8 = 5.03 = 12 \phi \, 3/4''$

Negative moment middle strip

$$A_s = 0.16 \times 22.6 = 3.63 = 12 \phi \, 5/8''$$

Positive moment middle strip

$$A_s = 0.16 \times 22.6 = 3.63 = 12 \phi \, 5/8''$$

Points of Inflection

Let us assume them to be on a straight line at a distance from the centre of the span equal to 3/10 the distance between the centres of columns:

$$\frac{3}{10} \times 24 = 7.2 \text{ ft}$$

from the middle of the span

b) Exterior Panels

The Joint Committee does not make a definite recommendation regarding the moments to be used in exterior panels but the American Concrete Institute specifies that moments in the section halfway between the discontinuous edge and the first row of columns are to be increased 25% over similar normal interior sections; at the discontinuous edge the negative moment in the column strip is to be not less than 90% and in the middle strip not less than 62.5% of the corresponding moments for the interior panel.

Following these specifications our moments will become:

Strip	Negative	Positive
2 Columns	$0.46 \times 90 =$ 0.42 M_o	$0.22 \times 125 =$ 0.28 M_o
Middle perpendicular to edge	$0.16 \times 62.5 =$ 0.10 M_o	$0.16 \times 125 =$ 0.20 M_o
Middle parallel to edge	$0.16 \times 125 =$ 0.20 M_o	$0.16 \times 125 =$ 0.20 M_o

As the maximum moment here 0.42 M_o is less than the maximum moment 0.46 M_o in the exterior panel the depth have not to be increased and we will keep it 8.75 inches ($D = 10''$).

Reinforcement

Negative moment 2 columns strips

$$A_s = \frac{M}{f_s jd} = 0.42 \times \frac{216\,000 \times 12}{16000 \times 0.867 \times 8.75} = 0.42 \times 22.6$$

$$= 9.5 \text{ sq. inches} = \underline{\underline{22\ \phi\ 3/4''}}$$

Positive moment 2 columns strips

Similarly $A_s = 0.28 \times 22.6 = 6.33 = \underline{\underline{15\ \phi\ 3/4''}}$

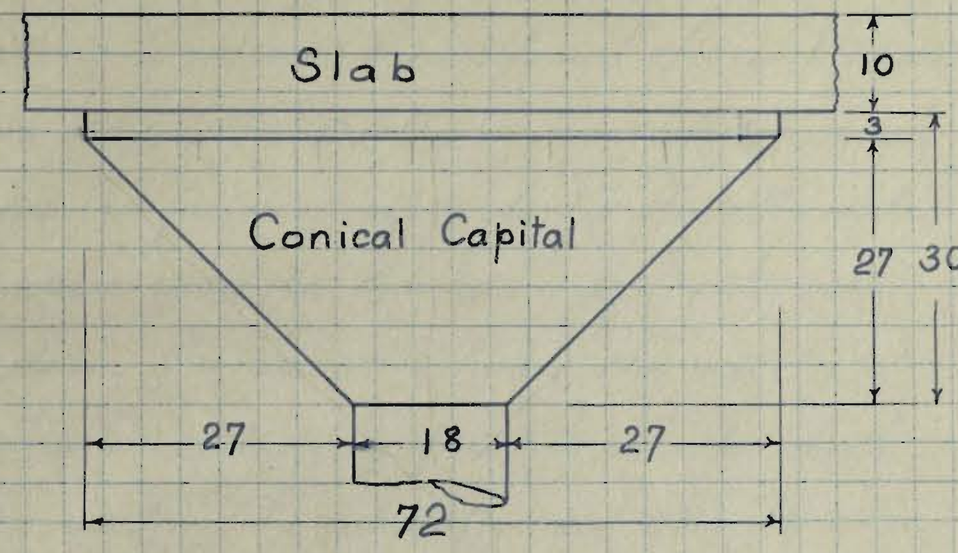
Negative moment middle strip transverse

$$A_s = 0.10 \times 22.6 = 2.26 = \underline{\underline{12\ \phi\ 1/2''}}$$

For all other moments

$$A_s = 0.20 \times 22.6 = 4.52 = \underline{\underline{15\ \phi\ 5/8''}}$$

CAPITAL



Distances in Inches

Assuming these dimensions let us find the stresses.

Shearing stress at circumference of radius $36 + jd$.

$$36 + jd = 36 + .867 \times 8.75 = 43.6'' = 3.64 \text{ ft.}$$

$$\text{Shear} = 250 \times 24^2 - 250 \cdot \pi \cdot 3.64^2 = 134\,000 \text{ lbs}$$

$$v = \frac{V}{bjd} = \frac{134000}{2\pi \cdot 43.6 \times 7.6} = 60 \text{ lbs / sq. inch}$$

The allowable shearing stress in flat slabs is

$$v = v_c (1 + r)$$ from Turneaure & Maurer's page 443

where r is the ratio of cross sectional area of negative reinforcement which crosses entirely over the column capital to the cross sectional area of the negative reinforcement in the two column strips

here $r = \frac{6}{12} = 0.5$

therefore $v = 50 (1 + 0.5) = 75$ lbs / sq. inch

and no need of stirrups; but bars have to be well anchored

Column

As the structure is a water tank the columns have only to support the slab and the earth above and as there is practically no live loads we can decrease our factor of safety to let the allowable compressive stress be the same as for the beams and slabs. On the other hand the ratio n of the modulae of elasticity of steel to concrete shall be decreased in the calculations and this in order to be on the safe side when plastic flow due to dead load occurs. Thus we shall take

$$f_c = 600 \text{ lbs / sq inch}$$

$$n = 12$$

$$A_s = 0.015\ A_c.$$

Using the straight line formula

$$\frac{P}{A} = f_c\left[1+(n-1)p\right]\left(1.33 - \frac{1}{120}\frac{l}{r}\right)$$

let us assume r to be $\frac{D}{4} = \frac{18''}{4} = 4.5''$ or 0.375

the unsupported length is $24 - 2.5 = 21.5$

And our formula becomes

$$\frac{144\,000}{A} = 600\left[1+(12-1)0.015\right]\left(1.33 - \frac{1}{120} \times \frac{21.5}{0.375}\right)$$

$$= 600\,(1 + 0.165)(1.33 - 0.48)$$

$$= 600 \times 1.165 \times 0.85$$

and $A = \frac{144\,000}{600 \times 1.165 \times 0.85} = 242$ sq. inches.

Taking a circular section this will give a diameter of $\sqrt{\frac{4 \times 242}{\pi}} = 17.6$ inches

we will take as we assumed i.e. <u>18 inches.</u>

$A_s = 0.015 \times 242 = 3.63$ sq. inch $= 8\,\phi\,3/4''$

[illegible]

Footings

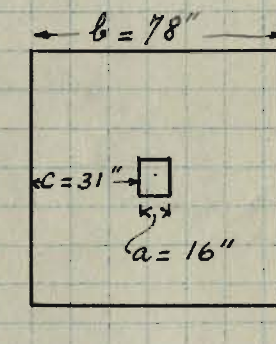
Load coming to column : $250 \times 24^2 = 144000$ lbs

Weight of column $\frac{\pi}{4} \times 1.5^2 \times 150 \times 24 = 6500$ "

Total load 150500 lbs

The bearing power of the soil being 5000 lbs per sq. foot and the pressure of water being 1500 the net bearing power is therefore 3500

$$M = \frac{w}{2}(a + 1.2c)c^2$$



$$b^2 = \frac{150500}{3500} = 43 \text{ sq. feet.}$$

$b = 6.5$ feet or 78 inches

The diameter of the column being 18" our a is $\sqrt{\frac{\pi}{4} \times 18^2} = 16$ inches

$$c = \frac{b - a}{2} = \frac{78 - 16}{2} = \frac{62}{2} = 31 \text{ inches}$$

Thickness

It is determined by shear

Assuming $d = 14$" the critical section is a square of side : $a + 2d = 16 + 2 \times 14 = 44$" whose area is $\frac{44^2}{144} = 13.4$ sq. feet

The shear is : $(43 - 13.4)\,3500 = 104000$ lbs

$$v = \frac{V}{bjd} = \frac{104000}{4 \times 44 \times .867 \times 14} = 48.5 \text{ lbs p. sq. inch}$$

$d = 14$ being sufficient our total depth is therefore $14 + 2 = \underline{16"}$

Reinforcement

$$M = \frac{3500}{2 \times 144}(16 + 1.2 \times 31)\, 31^2 = 621\,000 \text{ inch lbs}$$

$$A_s = \frac{M}{f_s j d} = \frac{621\,000}{16000 \times 0.867 \times 14} = 3.20 \text{ sq. inches}$$

$$= \underline{29\ \phi\ 3/8''}$$

Checking for bond strength

$$u = \frac{V}{\Sigma o j d} \text{ where } V = \frac{w(b^2 - a^2)}{4}$$

$$V = \frac{3500\,(78^2 - 16^2)}{4 \times 144} = 35\,500 \text{ lbs}$$

$$\Sigma o = 29 \cdot \pi \cdot \frac{3}{8} = 34.2 \text{ inches}$$

$$\therefore\ u = \frac{35500}{34.2 \times 0.867 \times 14} = 85.5 \text{ lbs p. sq. inch}$$

Spacing

As this is a two ways reinforcement one depth will be 14" and the other 14 - 3/8". So we will put 30 ϕ 3/8" in both directions having a uniform spacing of 2.5 inches.

Exterior Wall

We take counterforts @ 12′ c. to c.

Therefore the wall may be considered as a slab continuous over its supports equally spaced. Since no moving loads exist the critical moments may be taken as given by the moment equation:

$$M_o = \pm \frac{wl^2}{16} \quad \text{where } w = 63x.$$

We assume width of counterfort to be 2′

the clear span is therefore: $12 - 2 = 10'$

$$\text{and} \quad M_o = \frac{63x \times 10^2}{16} = 394x$$

Section of the wall

At the bottom where $x = 24'$

$$M_o = 394 \times 24 = 9450 \text{ ft lbs}$$

$$\text{the depth } d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{9450 \times 12}{108 \times 12}} = 9.3 \text{ inches}$$

The shear is $5wx = 5 \times 63 \times 24 = 7560$ lbs

The unit shear is $v = \frac{V}{bjd} = \frac{7560}{12 \times 0.867 \times 9.3} = 78$ lbs/□″

which is excessive. The depth of the wall at the bottom is therefore determined by the shear.

$$d = \frac{78}{50} \times 9.3 = 14.5 \text{ inches.}$$

for covering 1.5 ″

TOTAL DEPTH AT BOTTOM = 16 inches

TOTAL DEPTH AT TOP = 6 inches

Study of the Reinforcement

d at bottom .... 14.5 inches

d at top 4.5 "

The effective depth d of the wall at any distance x from the top is therefore:

$$d = 4.5 + \frac{x}{24}(14.5 - 4.5) = 4.5 + \frac{10x}{24} \text{ inches.}$$

The steel required per foot width is:

$$A_s = \frac{M}{f_s j d} = \frac{394\,x \times 12}{16000 \times 7/8 \times 4.5 + \frac{10x}{24}} = \frac{0.81\,x}{10.8 + x}$$

Using 1/2" ϕ whose area is 0.196 □" the formula for spacing at different depthes is

$$s = \frac{0.196}{\frac{0.81\,x}{10.8+x}} = 0.242\left(\frac{10.8+x}{x}\right) \text{ ft}$$

$$= 2.9\left(\frac{10.8+x}{x}\right) \text{ inches}$$

Depth in feet	4	8	12	16	20	~~24~~
1/2" ϕ Spacing inches	10.7	6.8	~~5.5~~	4.8	4.5	4.2

Using 3/4" ϕ having an area of 0.442 □" our spacing would be $s = 6.55\left(\frac{10.8+x}{x}\right)$ inches

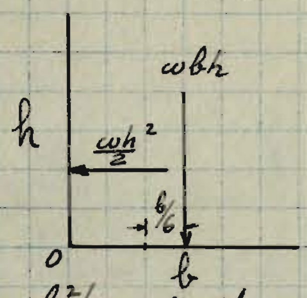
3/4" ϕ Spacing inches	~~24.2~~	~~15.4~~	12.4	11.0	10.1	9.5

To economize workmanship bigger bars are desirable but in order not to have excessive spacing we will use 1/2" ϕ till a depth of 8 feet and beyond that point we will use 3/4" ϕ.

COUNTERFORTS

General Considerations

In order to have the resultant fall in the middle third of the counterfort base we will require a base whose width is equal to the height of the water: if h is the height and b the base the overturning moment is $\frac{wh^2}{2} \times \frac{h}{3} = \frac{wh^3}{6}$



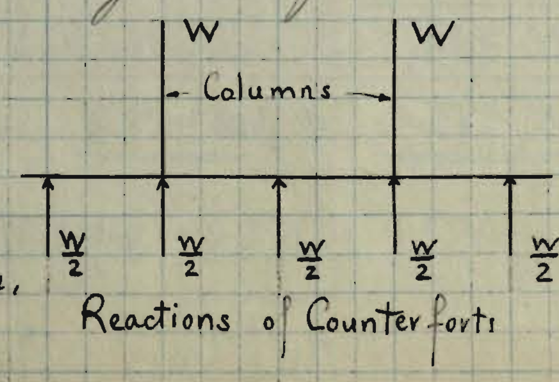
the resisting moment is $wbh \times \frac{b}{2} = \frac{wb^2h}{2}$ if the resultant comes at 0 but we want it to come on the middle third i.e: at a distance $\frac{b}{6}$ from the center of the base therefore the resisting moment around that point is $wbh \times \frac{b}{6} = \frac{wb^2h}{6}$

For equilibrium $\frac{wb^2h}{6} = \frac{wh^3}{6}$ and $b = h$

As we have to make our base 24 feet its end will meet the columns: this advantage will enable us by using the effect of the column to increase considerably our resisting moment.

But as our columns are spaced 24 feet apart while our counterforts are only at 12 feet from each other we will require a beam connecting the columns to the midway counterfort.

Such beam may be designed as concentrately loaded continuous beam. The critical moments being,



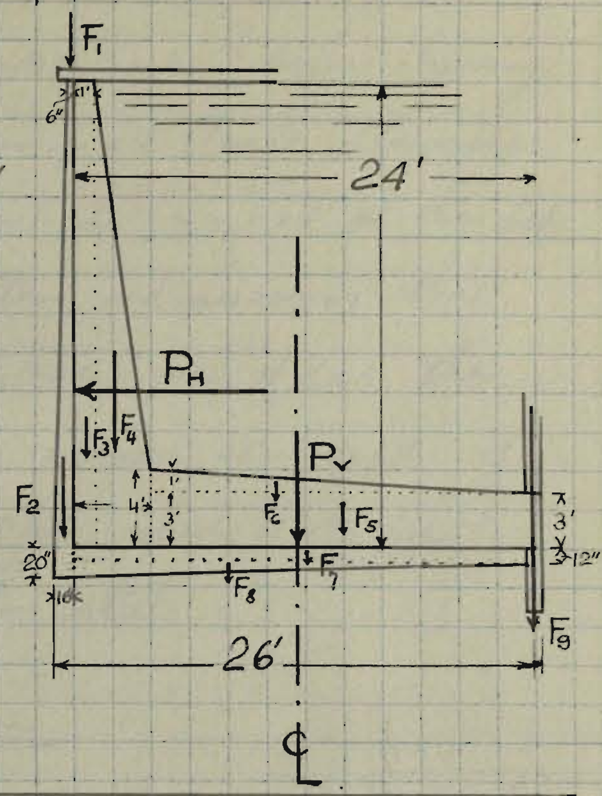
$$\frac{Pl}{4} \times \frac{8}{16} = \frac{Pl}{8}$$

PRESSURE DISTRIBUTION AT THE BASE

Assuming dimensions shown on figure with 2' as width of counterforts, the stresses can be readily computed by the use of the formula:

$$S = \frac{P}{A} \pm \frac{Mc}{I}$$

or $$S = \frac{P}{bd} \pm \frac{6M}{bd^2}$$



Force	Magnitude		Arm	Moment
F_1	$250 \times 12 \times 12$	= 36,000	12	+432,000
F_2	$\frac{1}{2}\left(\frac{6+16}{12}\right) \times 24 \times 12 \times 150$	= 39,600	12.5	+495,000
F_3	$1 \times 2 \times 24 \times 150$	= 7,200	11.5	+ 83,000
F_4	$\frac{4+24}{2} \times 3 \times 2 \times 150$	= 12,600	10	+126,000
F_6	$\frac{1}{2} \times 20 \times 2 \times 150$	= 3,000	1.0	+ 3,000
F_7	$1 \times 24 \times 12 \times 150$	= 43,200	0	Nil
F_8	$\frac{1}{12} \times \frac{8}{2} \times 24 \times 12 \times 150$	= 14,400	4	+ 58,000
P_H	$\frac{1}{2} \times 63 \times 24^2 \times 12$	= 217,000	9	+1,953,000
	Total Clockwise			+3,150,000
P_V	$63 \times 24 \times 24 \times 12$	= 436,000	0	Nil
F_5	$3 \times 20 \times 2 \times 150$	= 18,000	2	- 36,000
F_9	$75300 + 7200$	= 82,500	12.5	-1,030,000
	Total Anticlockwise			−1,066,000

Resultant Vertical force 692,500#; M= 2,084,000'#

$$S = \frac{692500}{26 \times 12} \pm \frac{2084,000 \times 6}{12 \times 26^2} = 2220 \pm 1520$$

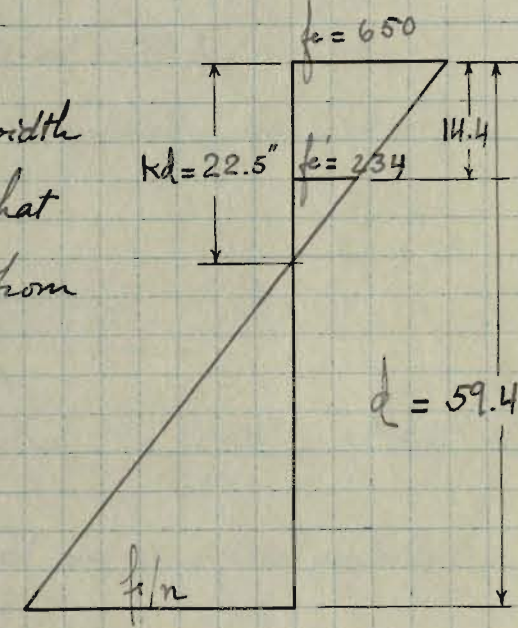
i.e.: 3740 #/□ft & 700 #/□ft

VERTICAL COUNTERFORT

It may be counted as a T beam having the exterior wall forming its flange. Assuming the dimensions of the web to be 1'×2' at the top and 4'×2' at the bottom and assuming the horizontal counterfort to have a web of 4'×2' the maximum moment is the moment at a depth of $24-(4-0.25) = 20.25$ feet which is the depth of the horizontal counterfort steel.

Study of the beam.

We will take a flange width of 6 feet and we assume that the eccentric load coming from the roof slab is supported by the remaining 6 feet without affecting our considering the section of the 6 feet T beam as working on bending only.



Stress Diagram

At a depth of 20.25 ft.

The thickness of the wall is $6 + \frac{10}{24} \times 20.25 = 14.4''$

The effective depth of the web is $4 - 0.25 = 3.75' = 45''$

it follows $d = 59.4''$

$kd = .379 \times 59.4 = 22.5''$

$f_c' = 650 \left(\frac{22.5 - 14.4}{22.5}\right) = 234 \ \#/\square''$

Taking jd as 53 the moment which can be resisted by concrete is

$M_c = \frac{1}{2}(f_c + f_c')\, t\, b\, jd = \frac{1}{2}(650+234)\, 14.4 \times 72 \times 53 \;{}^{"\#} =$

$2,020,000$ ft lbs.

Our actual moment is: $12 \frac{wx^3}{6} = \frac{12 \times 63 \times 20.25^3}{6}$ ft lb

$= 1,050,000$ ft lbs.

Taking the dimensions assumed will therefore under stress our concrete but considering the effect of shearing and considering our will to economise on steel we will take the dimensions assumed and we will calculate the reinforcement according to the value of the moments at different sections noting without computations that the concrete can carry them very safely.

Longitudinal Reinforcement

$A_s = \frac{M}{f_s jd}$ where $M = \frac{12 w x^3}{6} = 126 x^3$ ft lb

As the concrete is understressed we can take safely jd as $d - t/2$

jd at top $= 15 - 6/2 = 12''$

jd at bottom $= 59.4 - 14.4/2 = 52''$

Formula for jd: $jd = 12 + \left(\frac{52-12}{20}\right) x = 12 + 2x$ inch.

$\therefore\ A_s = \frac{126 x^3 \times 12}{16000(12+2x)} = \frac{0.047 x^3}{6+x}\ \square''$

Depth in feet	8	12	15	17.5	20.25
Area Required	1.73	4.53	7.60	10.7	15.5
No of 1" ⌀	2	5	8	11	16

Study of the Shear

Besides shearing stresses we have tensile stresses due to the reaction of the wall on the counterfort: water pressure tends to pull out the exterior wall therefore the bars of the exterior wall must be attached to the bars of the counterfort. In order to have this properly done we will let the stirrups have the same spacing as the bars of the exterior wall, the number of branches of each stirrup varying from section to section according to the value of the shearing and tensile stresses.

The shear at a depth x is $12 \times \frac{1}{2} w x^2 = 378\, x^2$

The unit shear is $v = \frac{V}{b'jd} = \frac{378\, x^2}{24(12+2x)} = 7.88 \frac{x^2}{6+x}$

The shear to be carried by steel is $v - 50$

For a spacing s" the stress for shearing is: $24s(v-50)$

The span of the wall being 10' the force tending to pull out one bar is $10 w x s = 10 \times 63 \times x \times \frac{s}{12} = 52.5\, sx$

The stress to be carried by the stirrup is $52.5\, sx + 24s(v-50)$

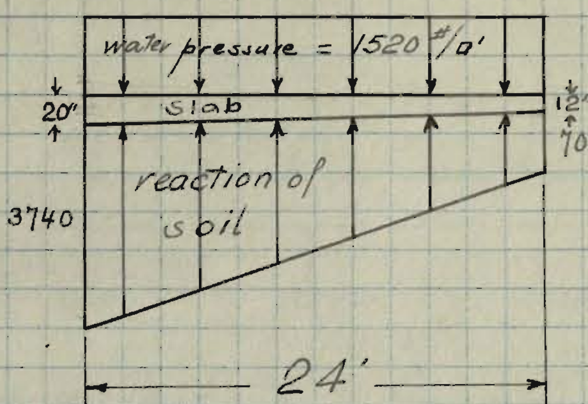
The table giving readily the number of bars:

x = depth in feet	4	8	12	16	20	24
s = spacing in inches	10.7	6.8	12.4	11.0	10.1	9.5
$v = 7.88 \frac{x^2}{6+x}$	12.6	36.1	63.0	91.7	121	Nil
$v_s = v - 50$	–	–	13	41.7	71	–
Shearing Stress = $24s\, v_s$	–	–	3870	11,000	17,200	–
Tensile Stress = $52.5\, sx$	2230	2860	7800	9240	10,600	12,000
Stress	2230	2860	11670	20240	27800	12000
Using 1/2" Φ (fs=14.000)	1	2	5	8	10	5

Base Counterfort Slab

Cross-Section

Here the shearing force governs. Assuming a depth of 20" at the toe and 12" at the heel the dead loads of this slab are respectively 250 #/□' and 150 #/□'



The clear span is 12 - 2 = 10 ft

Max. shear at the toe: 5(3740 - 1520 - 250) = 9850 lb

$$v = \frac{V}{bjd} = \frac{9850}{12 \times 0.867 \times 19} = 49.7 \#/□''$$

Max. shear at the heel: 5(1520 + 150 - 700) = 4850 lbs

$$v = \frac{V}{bjd} = \frac{4850}{12 \times 0.867 \times 11} = 43 \#/□''$$

Assumptions correct

Study of the Reinforcement

At a distance x from the heel

the effective depth $d = 11 + \frac{x}{24} \times 8 = 11 + \frac{x}{3}$ inches

the pressure is $p = 1520 + \left(12 + \frac{x}{3}\right) \cdot \frac{150}{12} - 700 - \frac{x}{24}(3740 - 700)$

$$= 970 - 122.5x \text{ lbs}$$

Steel required per foot: $A_s = \frac{M}{f_s jd} = \frac{(970 - 122.5x)\frac{10^2}{16} \times 12}{16000 \times 0.867 \times (11 + \frac{x}{3})}$

$$= 1.990 \, \frac{7.93 - x}{33 + x}$$

Using 3/4" φ whose area is 0.442 □"

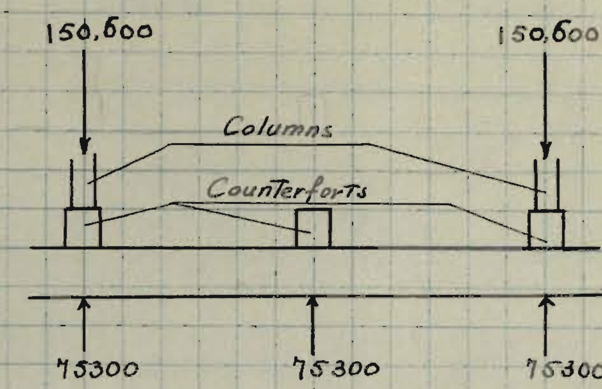
The formula for spacing in inches: $\frac{12 \times 0.442}{1.99 \frac{7.93 - x}{33 + x}} = 2.66 \frac{33 + x}{7.93 - x}$

x	0	4	6	10	12	16	20	24
Spacing	11.1	25	53.6	55.3	29.4	16.1	11.7	9.5

(spacings for x = 4 to 12: not to be used)

From x = 4 to x = 12 we will use 1/2" with a uniform spacing of $25 \times \frac{0.196}{0.442} = 12$ inches.

BEAM CONNECTING COUNTERFORTS



Clear span
$= 24 - 2 = 22'$

Moment due to concentrated load: $\frac{Pl}{4} \times \frac{1}{2}$ because continuous

$= \frac{Pl}{8} = \frac{75300 \times 22}{8} = 207,000$ ft lbs

Moment due to uniform load: $M_o = \frac{wl^2}{16}$ where w equals: weight of beam + pressure of water − resistance of soil

$= 525 + 1520 - 700 = 1345$ if width = 12"

$M_o = \frac{1345 \times 22^2}{16} = 40,000$ ft lbs.

The critical moments are $207,000 - 40,000 = \pm 167,000$ ft lbs

if width = 12" $d^2 = \frac{167000}{108} = 39.4^2$

So Gross section: 42" × 12"

$A_s = \frac{M}{f_s jd} = \frac{167000 \times 12}{16000 \times 0.867 \times 39.4} = 3.66\square'' = 4\phi 1''$

Max shear is at center $= \frac{75300}{2} = 37600$

at supports it is lower but we neglect the difference

$v_c = \frac{V}{bjd} = \frac{37600}{12 \times 0.867 \times 39.4} = 92\ \#/\square''$

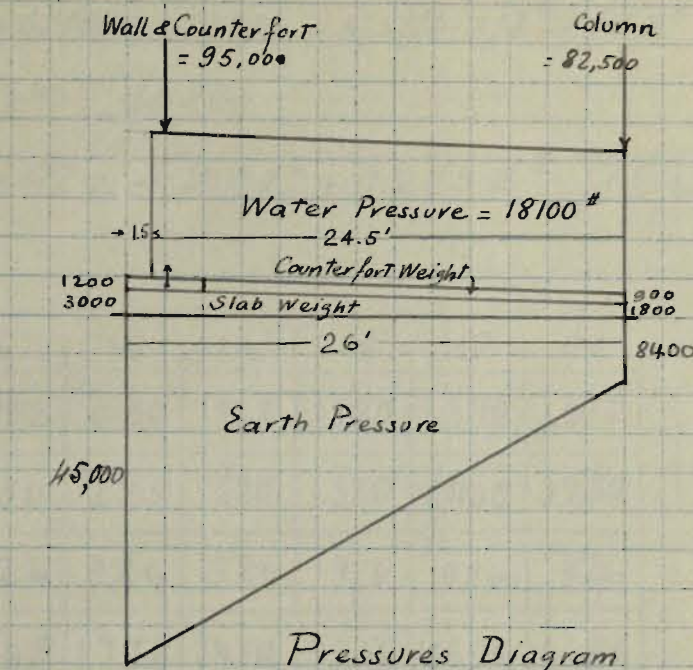
Using $2\phi\ 5/8$ per stirrup the spacing is:

$s = \frac{f_s \times A_s}{v_s \times b} = \frac{14000 \times 0.61}{42 \times 12} = 17$ inches

Check for bond

$u = \frac{V}{\Sigma_o jd} = \frac{37600}{15.71 \times 0.867 \times 39.4} = 70\ \#/\square''$

HORIZONTAL COUNTERFORTS



Pressures Diagram

Earth Pressures:

$700 \times 12 = 8400$ #

& $3740 \times 12 = 45000$ #

Slab weights:

$1 \times 12 \times 150 = 1800$

& $\frac{20}{12} \times 12 \times 150 = 3000$

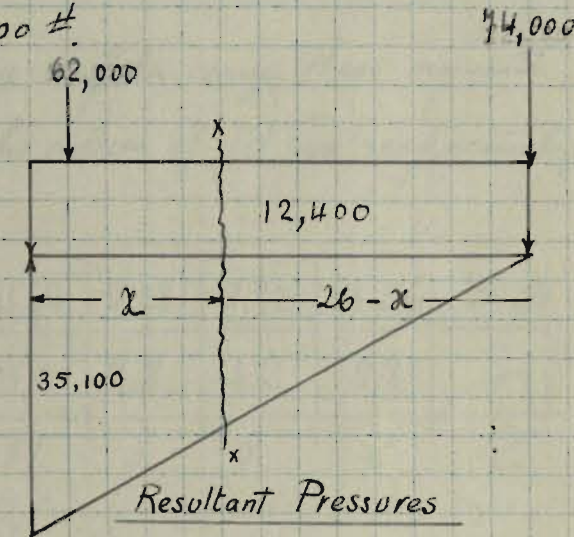
Counterfort weights:

$3 \times 2 \times 150 = 900$

& $4 \times 2 \times 150 = 1200$

Wall & Counterfort weight: $F_1 + F_2 + F_3 + F_4 = 95,000$ #

Column weight $= 82,500$ #



Resultant Pressures

Simplifying we get:

Wall & Counter. $= 95 - 18 \times 1.5 - 1.2 \times 5 = 62$

Column: $82.5 - 8.4 = 74$

Downward pressures:

$18.1 + .9 + 1.8 - 8.4 = 12.4$

& $45 - 18.1 - 1.2 - 3 = 22.7$

$22.7 + 12.4 = 35.1$

Moment Equation

In general $M_{x-x} = M_0 + \Sigma \frac{\omega x^2}{2} + \Sigma P_x + \Sigma k x^3$

In our case $M_0 = 1,050,000$ ft-lbs

$$\Sigma \frac{\omega x^2}{2} = \frac{12400 x^2}{2} = 6200 x^2$$

$$\Sigma P_x = 62000(x-2) = 62000x - 124000$$

$$\Sigma k x^3 = -\left[\left(\frac{26-x}{26}\right) 35100 \frac{x^2}{2} + \left(35100 - \frac{26-x}{26} 35100\right) \frac{x}{2} \cdot \frac{2x}{3}\right]$$

$$= -0.675(26-x)x^2 - 0.450x^3$$

The moment equation thus is:

$$M_{x-x} = 1050000 + 62000x - 124000 - 675(26-x)x^2 - 450x^3 + 6200x^2 \text{ ft lbs}$$

or $$= 926 + 62x - 9.6x^2 + 0.225x^3 \text{ ft kips}$$

x	6	10	14	18	22	26
$62x$	372	620	868	1120	1370	1610
$0.225x^3$	50	225	620	1310	2400	3950
926	926	926	926	926	926	926
Sum	1348	1771	2414	3356	4696	6486
$9.6x^2$	346	960	1880	3100	4650	6480
Value of Moment foot kips	1002	811	534	256	46	—

The concrete can very safely carry these moments

$A_s = \frac{M}{f_s jd}$; the values of jd are respectively

jd feet	5.0	4.64	4.28	3.92	3.56	3.20
A_s = sq. inches	12.8	11.0	7.8	4.2	0.8	—
Number of 1" ⌗	14	11	8	5	2	2

Study of the Shear

d being the distance from the column the shear is: $74000 + 12400d - \frac{d}{26} \cdot 35100 \cdot \frac{d}{2}$

$$V = 74000 + 12400d - 675d^2$$

Near the column we have, besides shearing stresses, tensile stresses because earth pressure being here less than water pressure and weight of slab (See Base Counterfort Slab design). So we will space our stirrups according to the spacing of the slab bars. s being the spacing in inches the tensile force per bar is $T = \frac{s}{12}\left(12400 - \frac{d}{26} \cdot 35100\right) \cdot \frac{10}{12}$ ($\frac{10}{12}$ since the clear span is only 10')

$$T = 93.6\, s\,(9.18 - d) \text{ lbs}$$

d	20	16	12	8	4	0
$12400\,d$	248000	198300	149000	99000	49600	—
$74000 + 12400\,d$	322000	272300	223000	173000	123600	74000
$-675\,d^2$	270000	173000	97000	43200	10800	—
$V = 74000 + 12400\,d - 675\,d^2$	52000	99300	126000	129800	112800	74000
jd in feet	5.0	4.64	4.28	3.92	3.56	3.20
$v = \frac{V}{b'jd}$ ($b' = 24$)	36	74	102	115	110	80
Shear to be carried by steel : $v - 50$	—	24	52	65	60	30
Spacing	11.1	12	12	16.1	11.7	9.5
Stress $= 24^{b'} \times s \times (v - 50)$	—	6900	15000	25100	16850	6840
Tensile stress $T = 93.6\, s\,(9.18 - d)$	—	—	—	1780	5680	8100
Stresses to be carried by stirrups	—	6,900	15,000	26,880	22,530	14,940

Using 3/4" Φ having an area of 0.442 and able to carry a stress of $14000 \times 0.442 = 6200$ lbs the number of branches of each stirrup is respectively

| 2 | 3 | 5 | 4 | 3 |

Now for counterforts between columns, these must be connected to the ground beam by means of $\frac{72000}{16000} = 5$ Φ 1" in order to have the 72000 lbs force operate.

TEMPERATURE STRESSES

In order to take into account temperature stresses in the horizontally reinforced exterior wall and the one way reinforced counterfarts slabs 1/2" ϕ shall be put transversely with a uniform spacing of 12".

This reinforcement in the exterior wall will extend from the top to the bottom and besides temperature stresses will help the wall to resist safely earth pressure adjacent to it.

EXPANSION JOINTS

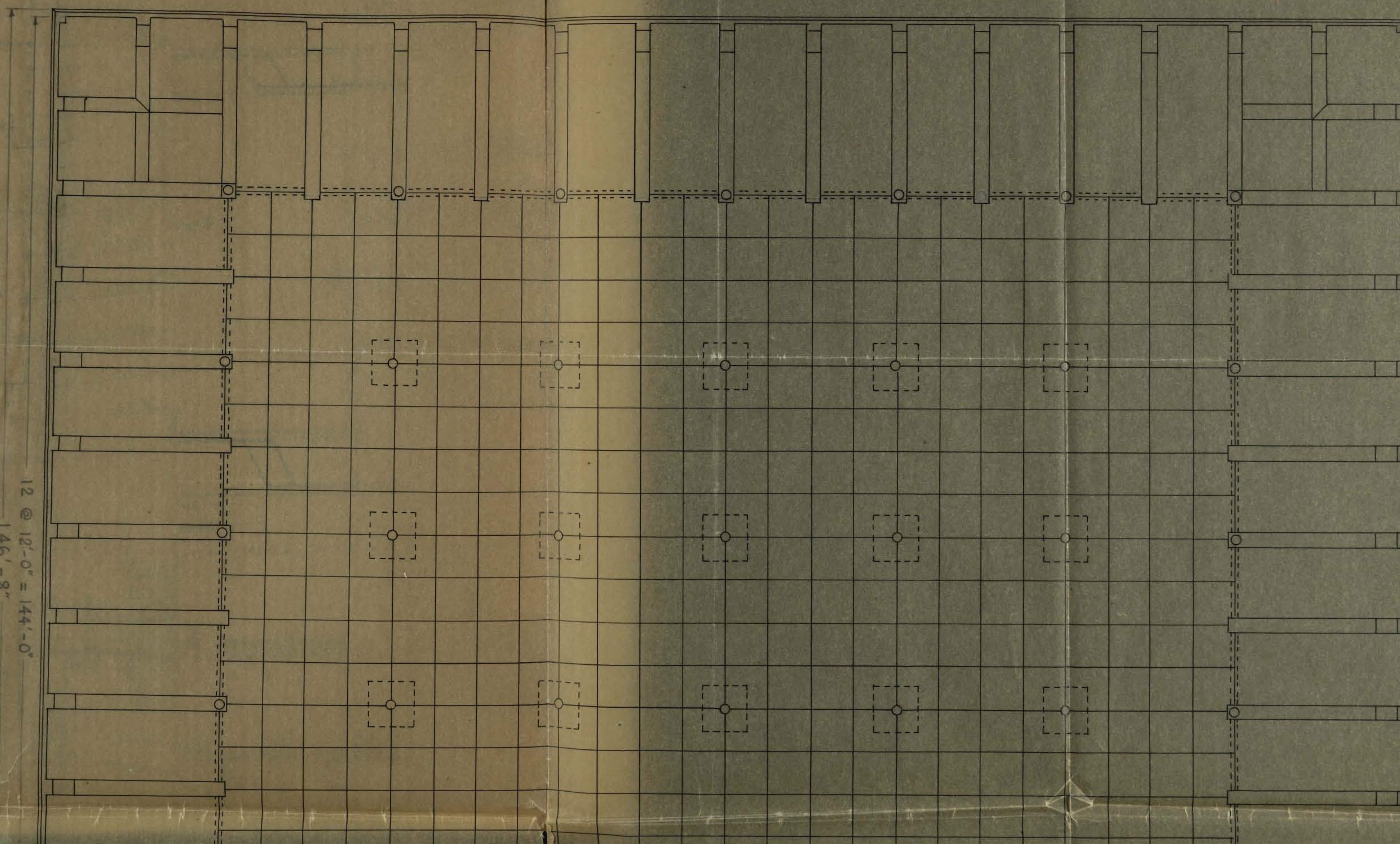
The ground slab shall be 6" thick and shall be constructed of squares having a side of 6 feet.

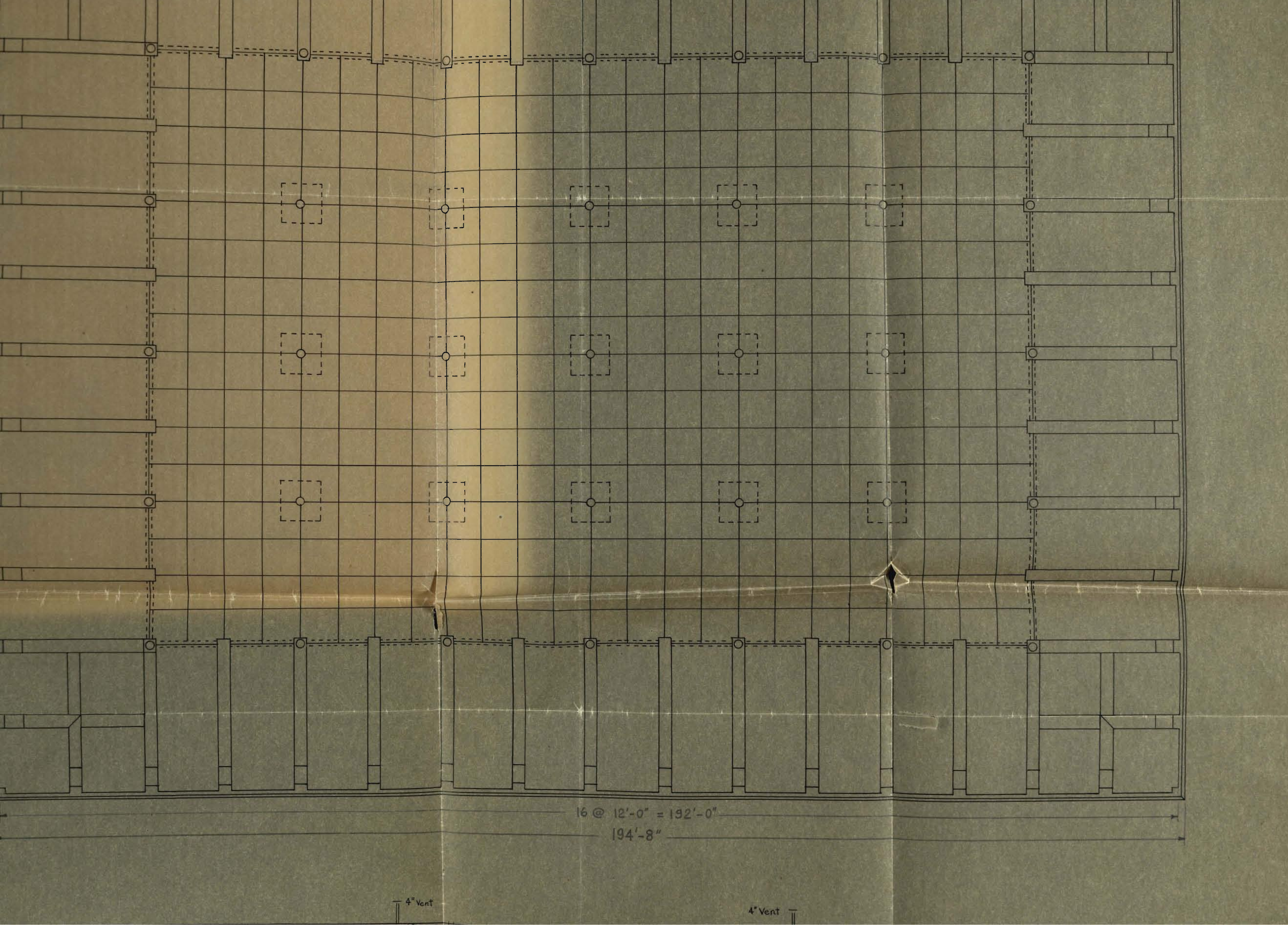
Between each two consecutive tiles thus formed an expansion joint shall be made and filled with an impervious material such as bitumen.

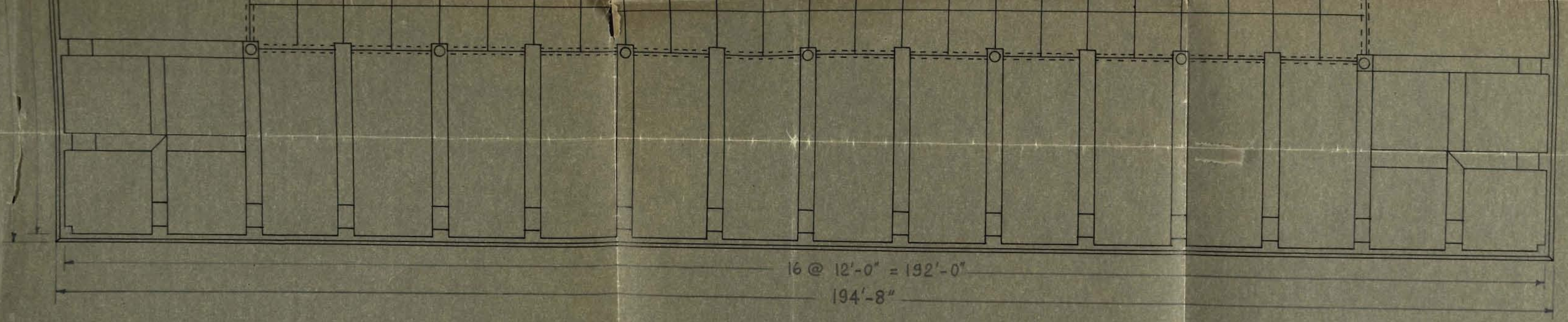
Sheet No 1

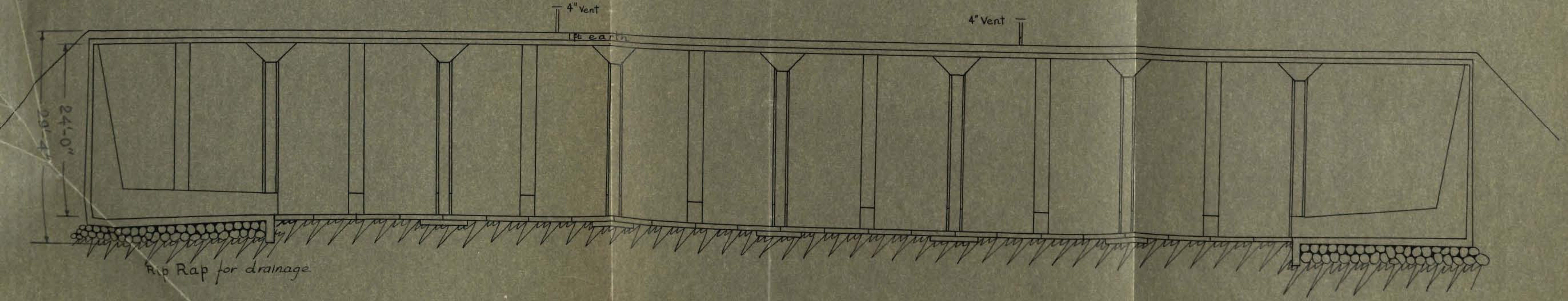
WATER TANK 4,000,000 GALLS.

Plan & Section Scale 1/144

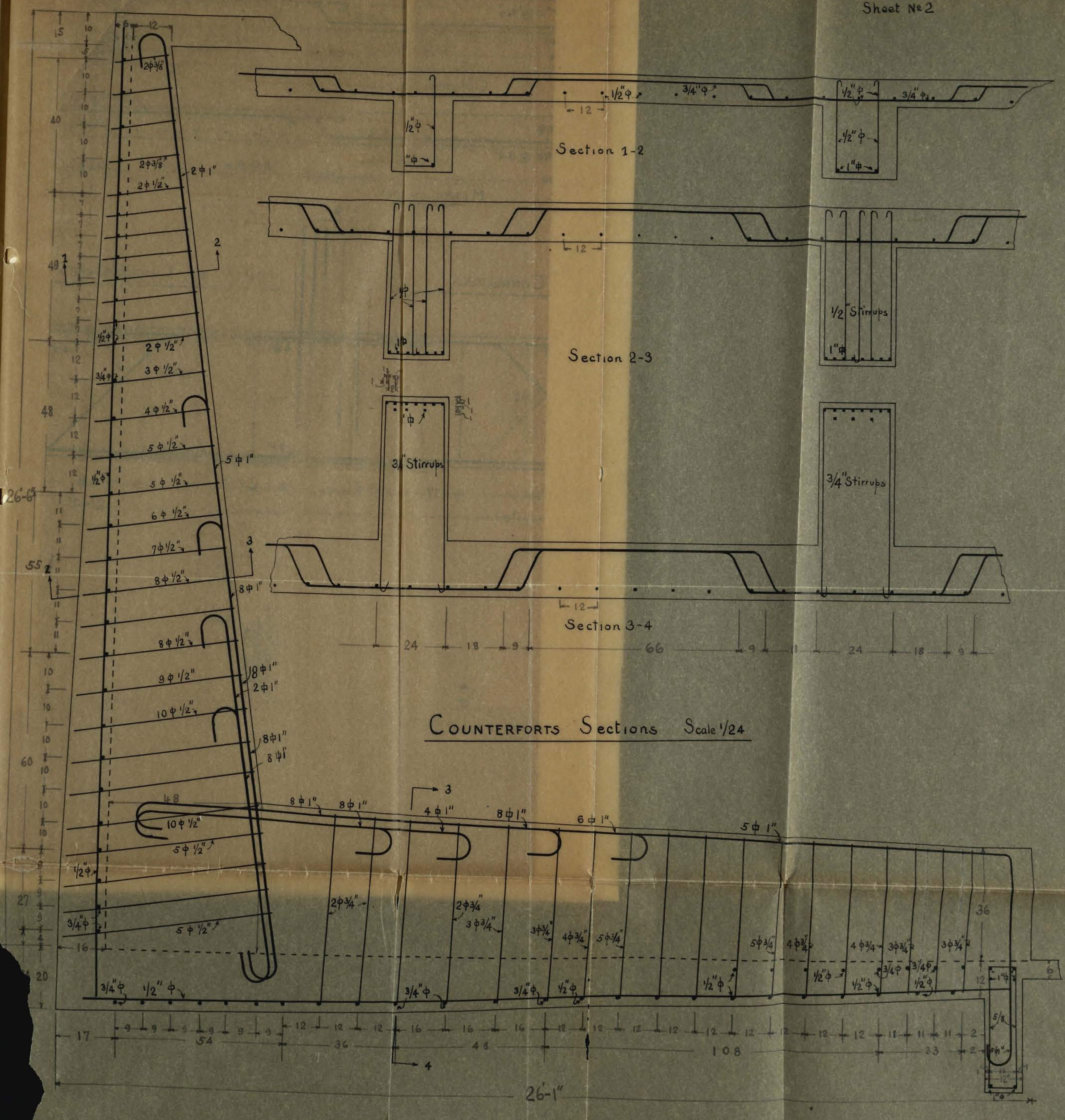



16 @ 12'-0" = 192'-0"
194'-8"
4" Vent
4" Vent

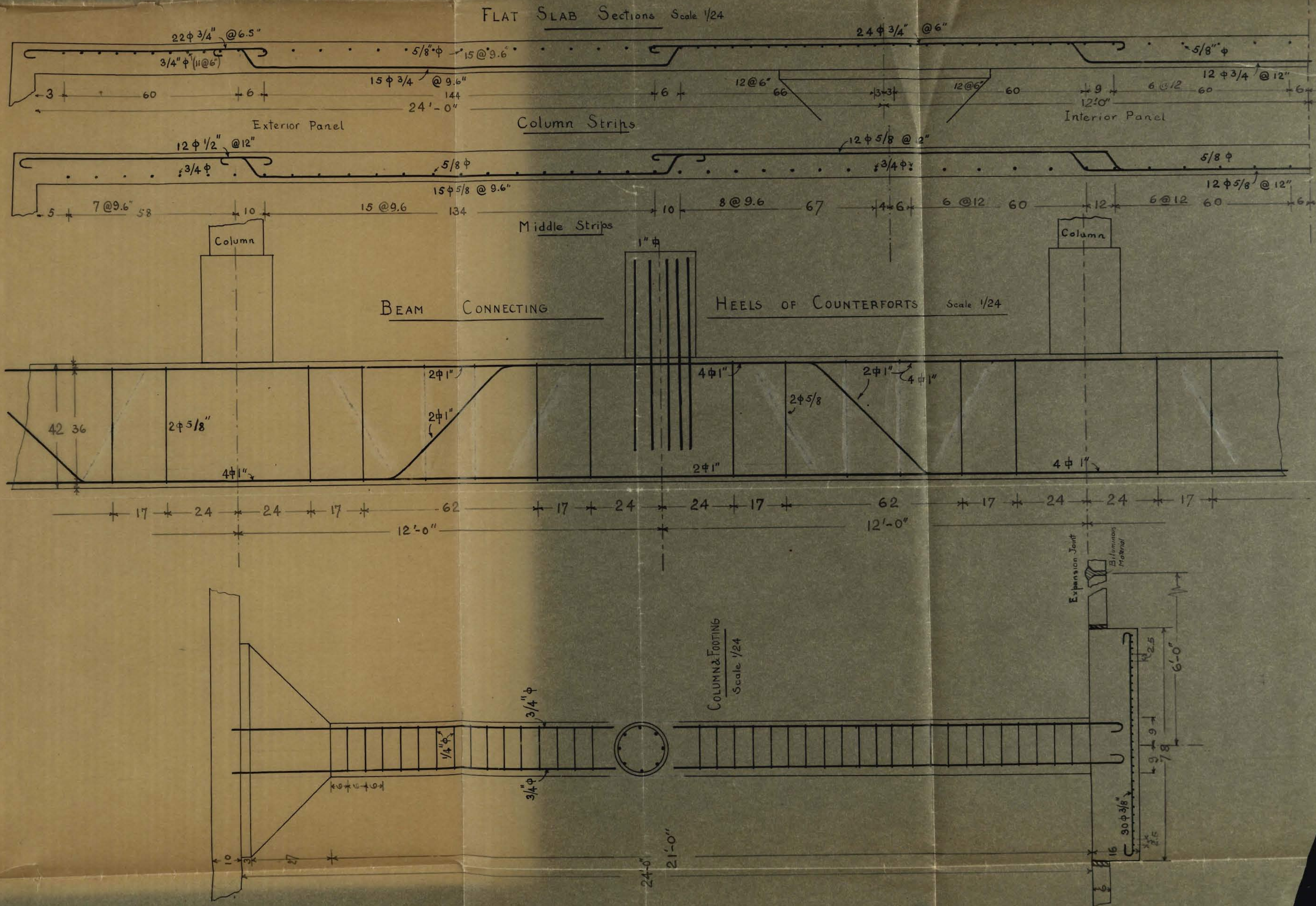

16 @ 12'-0" = 192'-0"
194'-8"


4" Vent
4" Vent
1ft earth
24'-0"
29'-4"
Rip Rap for drainage

Sheet No 2



COUNTERFORTS SECTIONS Scale 1/24


Flat Slab Sections Scale 1/24
22 φ 3/4" @6.5"
3/4" φ (11@6")
5/8" φ
15 @ 9.6"
15 φ 3/4 @ 9.6"
24 φ 3/4" @6"
5/8" φ
12 φ 3/4 @ 12"
12@6"
3 60 6 144 6 66 3 3 60 9 6@12 60 6
24'-0"
12'-0"
Exterior Panel
Column Strips
Interior Panel
12 φ 1/2" @12"
3/4 φ
5/8 φ
15 φ 5/8 @ 9.6"
12 φ 5/8 @ 2"
3/4 φ
5/8 φ
12 φ 5/8 @ 12"
5 7@9.6" 58 10 15 @9.6 134 10 8 @ 9.6 67 4 6 6 @12 60 12 6@12 60 6
Middle Strips
Column
1" φ
Column
Beam Connecting Heels of Counterforts Scale 1/24
2φ1"
2φ1"
4φ1"
2φ1"
4φ1"
2φ5/8
42 36
2φ5/8"
4φ1"
2φ1"
4 φ 1"
17 24 24 17 62 17 24 24 17 62 17 24 24 17
12'-0"
12'-0"
Expansion Joint
Bituminous Material
Column & Footing Scale 1/24
3/4" φ
1/4" φ
3/4" φ
6'-0"
2.5
9
9
78
30 φ 3/8"
16
10
3
27
21'-0"
24'-0"