THE DESIGN OF A CONCRETE-THREE HINGED-ARCH AEROPLANE HANGAR

Thesis for the Degree of B. S.

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Aeroplanes

Title: Aeroplane hangar

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Aeroplane Hangar

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THESIS

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Bibliography

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Preface

The last few years has seen some of the greatest strides in the development of the aeroplane industry. Heavier-than-air travel has been extended to nearly all parts of the civilized world. With the building of hundreds of new planes there arises the problem of adequate hangar facilities, thus, new and original designs for aeroplane hangars are always in demand.

Up to the present time most hangars have been of steel construction. It has been the conviction of the writer that reinforced concrete could be used to good advantage in this field.

There are certain advantages in this form of construction over the old steel type. Generally speaking, it is more fire-proof. It has considerably more clearance. Finally, it is more economical to build where cement and aggregate is cheap.

This hangar is designed with a span of one hundred and fifty feet, and a length of two hundred and sixty-four feet, with a clear distance at center of span of fifty-three and a half feet. The main doors at either end are thirty- five feet high and ninety feet wide. They are in three thirty-foot sections of the rolling-metal type.

The roof span is thirty-two feet and O-T joists are used. It is designed so that the bottom of the joist is on the same elevation as the bottom of the arch rib. This last

is so constructed that with a bottom cord extension, metal lath may be fastened to the bottom of the joist and plastered, giving a smooth ceiling. Sixteen and a half inches of the arch rib will project on the outside of the roof slab, which, with the pillars will give a pleasing and modernistic appearance.

Windows will cover the area between the pillars from four feet above the ground to the roof, and also at each end beside and above the doors.

The floor slab is of the usual reinforced concrete type.

Inasmuch as this hangar will probably be located in a large airport, the heating plant, office facilities, and accomodations would be situated in another building.

Design of Roof Slab.

Snow load - 25#/ sq. ft. of horizontal projection.

Assume live load - 25#/ sq. ft. of roof.

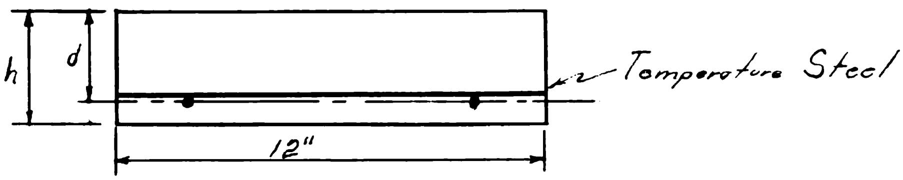
Assume slab - t = $3\frac{1}{2}$".

Assume - f'c = 2000#/ sq. in.

Wt./ sq. ft. = $\frac{3.5}{12}$ x 1x1x150 = 43.8#/ sq. ft. of roof - dead load

25. #/ sq. ft. of roof- live load

W = 68.8#/ sq. ft. of roof.



Cross section through slab unit.

Maximum positive bending moment $Mp = \frac{wl2}{12}$

Maximum negative bending moment $Mn = \frac{wl^2}{12}$

Will design for positive bending moment.

Having selected O-T Joist maximum distance between joist is 30".

Assume clear span as 30".

$Mp = \frac{68.6x\,(2Y5)^2}{12} = 35.7ft.\# = 429$ in. #.

With fc = 0.4x2,000 = 800#/ Sq. In.

fs = 20,000 #/ sq. in. (from A.C.I. 306-307

$k = \frac{1}{1 + \frac{fs}{nfc}} = \frac{1}{1 + \frac{20,000}{15x800}} = 3/8$

$j = 1 - \frac{k}{3} = 7/8$

$Mp = \frac{fc}{2}\, jkbd^2 = \frac{800}{2} \times 7/8 \times 3/8 \times 12x\, d^2 = 429$

$d = \left(\frac{429x2x8x8}{800x7x3x12}\right)^{\frac{1}{2}}$

$d = .273$ in.

Estimate bars = $\frac{1}{4}$" round $1\frac{1}{2}$" covering.

$h = d + 1\frac{1}{2} + 1/8 = .27 + 1\frac{1}{2} + 1/8 = .27 + 1.5 + .125$

$h = 1.875$"

Must assume at least a $3\frac{1}{2}$" slab.

Commercial slab = $3\frac{1}{2}$"

$$d = 3.5 - 1.625 = 1.875$$

Steel required.

$$A_p = \frac{M_p}{f_s j d} = \frac{429}{20{,}000 \times .875 \times 1.875} = .01335 \text{ sq. in./ft. of slab.}$$

Use $\frac{1}{4}$ in. bar spaced at 12 in. C-C.

The maximum negative bending moment Mn at the support is also numerically = 429"#. The tension side is now on top of the slab and the protecting covering need be only one inch. The positive will be bent up to be served as negative steel.

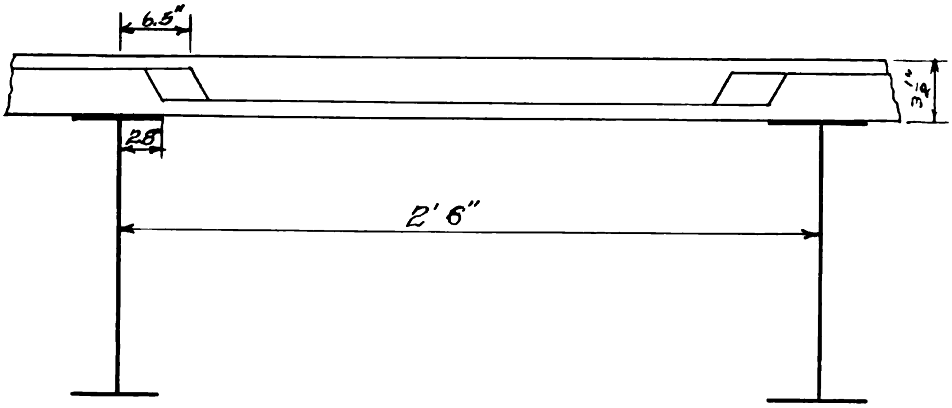
The commercial $d = 3.5-1-.125 = 2.375$"

The minimum are An will be less than Ap but due to efficiency and size, (smallest size that is easily handled) the same steel will be used over the support.

Points of Inflection.

Mp for positive steel $9\frac{1}{2}$ % clear span = 2.85"

Mn for negative steel $21\frac{1}{2}$ % clear span = 6.45"



Temperature Steel.

p = .002 for slabs with deformed bars.

As = pbd = .002 x 12 x 1.875 = .045 sq. in./ lin. ft.

Use $\frac{1}{4}$" round bars spaced at 10" C-C.

Selection of Joists.

Truscon "O-T" Open truss Steel Joist.

Clear Span - 32 ft.

Joist type - 167.

Total safe load pounds - 5860

Total safe loads in pounds per sq. ft. for joist spacing of 30" = 73# / sq. f.

O.K. have only 68.8#/ sq. ft.

Wt. per. ft. of joist = 9.6#

Design of Arch.

Wt. transmitted to the arch by one joist.

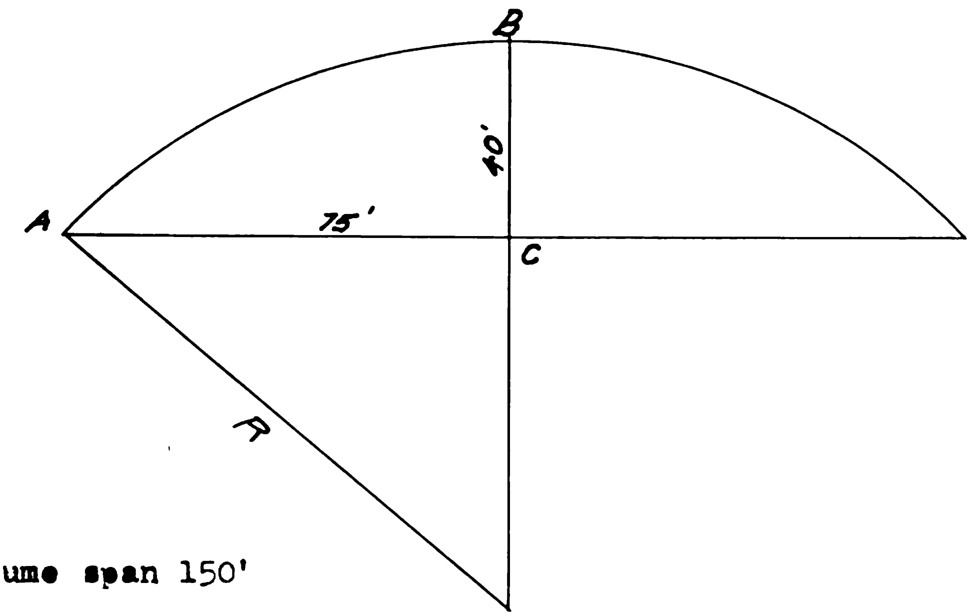
Wt. of joist - $\frac{32}{2}$ x 9.6 = 153.6#

Wt. of roof slab $\frac{32}{2}$ x 25 x 43.8 = 1752.0

Total wt. 1905.6#/ each end of joist

Wt. of snow = 1000#/ each joist end

Find size of Arch.



Assume span 150'

rise 40'

$$R = \frac{\overline{BC}^2 + \overline{AC}^2}{2\ BC}$$

$$= \frac{\overline{40}^2 + \overline{75}^2}{2 \times 40}$$

$$= \frac{1600 + 5625}{80}$$

$$= \frac{722.5}{80}$$

R = 90.3125' radius of arch axis.

Size of Arch.

Assume width 1'

depth 3'

Wt. of arch / joist connection (every 2.5')

W. = 3 x 1 x 2.5 x 150 = 1125#/ $2\frac{1}{2}$' of arch.

Stress Diagram.

Wt. per joist connection (every 2.5')

Dead Load

Roof and joist 2 x 1905.6 $=$ 3811.2
Arch 1125.0
4936.2#/pt. app.

Snow Load

1000 x 2 $=$ 2,000#/pt. app.

Wind Load

Duchemin's Formulae $Pn = P \frac{2 \sin. A}{1 + \sin 2 A}$

Assume $P = 30$ $\sin A = 40/85$

P = pressure / sq. ft. of vertical projection.

$Pn = 30 \frac{2 \times 40/85}{1 + (40/85)} = 30 \frac{.442}{1.22} = 27.6$ #/ sq.ft. of roof.

27.6 x 32 x 2.5 $=$ 22; 0#/ pt. app.

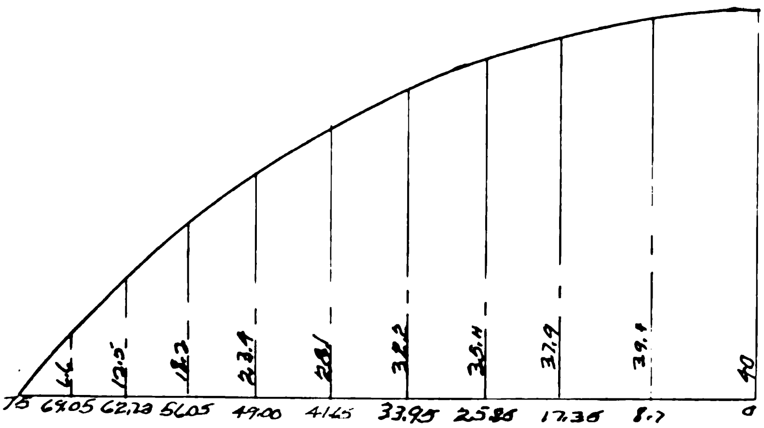
A segment of a circle was such that funicular polygon was about one foot from the arch axis which would call for a 6' arch to have funicular polygon within the middle 1/3.

A parabolic curve was used as the shape of the arch which gave a depth of 3'.

Test the size of the arch.

The arch was divided into ten equal parts and the bending moment is computed for each section.

Sketch shows horizontal distances to points from crown and vertical distances from pillar.



Bending moment at any section is equal to the summation of moments to either side of the section.

Reactions at Pillar.

Summation of V = 0

$$V = 35 \times 6936.2$$

$$V = 242{,}767.0\#$$

Summation of M = 0

$$- 40 H + 2{,}427{,}670 \times 75 + 6936.2 (1.25 + 3.75 + 6.25 + 8.75 + 11.20 + 16.10 + 18.55 + 21.00 + 23.40 + 25.75 + 28.1 + 30.4 + 32.65 + 34.90 + 37.10 + 39.30 + 41.45 + 43.60 + 45.70 + 47.80 + 49.85 + 51.85 + 53.85 + 55.85 + 57.80 + 59.70 + 61.60 + 63.45 + 65.25 + 67.05 + 68.80 + 70.50 + 72.20 + 73.85)$$

$$- 40 H = 9{,}726{,}286.4 - 18{,}207{,}525.0$$

$$H = \frac{8{,}481{,}239}{40}$$

$$H = 212{,}030.9\#$$

Bending moment at the different sections. First section from left - All moments to left.

$$BM_1 = 6936.2 (.25 + 1.95 + 3.65 + 5.30) + 212{,}030.9 \times 6.6 - 242{,}767 \times 5.95$$

$$BM_1 = 6936.2 (11.15) + 212{,}030.9 \times 6.6 - 242{,}767 \times 5.95/$$

$$BM_1 = 77338.63 + 1{,}399{,}403.9 - 1{,}444{,}463.6$$

$$BM_1 = 32{,}278.9'\#$$

$$BM_2 = 6936.2 (1.2 + 3.0 + 4.8 + 6.55 + 8.25 + 9.95$$

+ 11.60) +212030.9 x 12.5 - 242,767 x 12.27.

BM$_2$ = 6936.2 (45.35) + 212030.9 x 12.5 - 242767 x 12.27.

BM$_2$ = 314,556.6 + 2,650,386.0 - 2,978,751.0.

BM$_2$ = 13,808'#

BM$_3$ = 6936.2 (.3 + 2.25 + 4.15 + 6.05 + 7.90 +9.70 + 11.50 + 13.25 + 14.95 + 16.65 + 18.30) + 212,030.9 x 18.2 - 242,767.0 x 18.95.

BM$_3$ = 6936.2 (105.00 + 212030.9 x 18.2 - 242,767 x 18.95.

BM$_3$ = 728,301 + 3,858,962.38 - 4,600,436.6

BM$_3$ = - 13,177'#

BM$_4$ = 6936.2 (1.35 + 3.35 + 5.35 + 7.35 + 9.30 + 11.20 + 13.10 + 14.95 + 16.75 + 18.55 + 20.3 + 22.0 + 23.7 + 25.35) + 212,030.9 x 23.4 - 242767 x 26.

BM$_4$ = 1,335,912 + 4,961,523 - 6,311,942

BM$_4$ = 14,507'#

BM$_5$ = 6936.2 (0.3 + 2.45 + 4.55 + 6.65 + 8.70 + 10.70 + 12.70 + 14.70 + 16.65 + 18.55 + 20.45 + 22.30 + 24.10 + 25.90 + 27.65 + 29.36 + 31.05 + 32.70 + 212,030.9 x 28.1 - 212,767 x 33.35.

BM$_5$ = 6936.2 (309.45) + 212,030.9 x 28.1 - 242,767 x 33.35.

BM$_5$ = 2,146,407 + 5,958,068.29 - 8,096, 279.4.

BM$_5$ = 7,195.9'#

BM$_6$ = 6936.2 (1.45 + 3.65 + 5.8 + 8.00 + 10.15 + 12.25 + 14.35 + 16.40 + 18.40 + 20.40 + 22.40 + 24.35 + 26.25 + 28.15 + 30.0 + 31.80 + 33.60 + 35. 35 + 37.05 + 38.75 + 40.40) + 212,030.9 x 32.2 - 242,767. x 41.05.

BM6 = 6936.2 (459.00 + 212,030.9 x 32.2 - 242,767 x 41.05.

BM6 = 3,183,715.8 + 6,827,394.98 - 9,965,585.3

BM$_6$ = 45,525.4'#

BM$_7$ = 6936.2 (.35 + 2.70 + 5.00 + 7.25 + 9.50 + 11.70 + 13.90 + 16.05 + 18.20 + 20.30 + 22.40 + 24.45 + 26.45 + 28.45 + 30.45 + 32.40 + 34.30 + 36.20 + 38.05 + 39.85 + 41.65 + 43.40 + 45.10 -+ 46.80 + 48.45 + 212,030.9 x 35.4 - 242,767 x 49.15.

$BM_7 = 6936.2\ (633,35) + 212,030.9 \times 35.4 - 242,767 \times 49.15.$

$BM_7 = 4,413,042.3 + 7,505,893.8 - 11,931,998,05.$

$BM_7 = 13,061.9'\#$

$BM_8 = 6936.2\ (1.6 + 4.05 + 6.45 + 8.80 + 11.15 + 13.45 + 15.70$
$+ 17.95 + 20.15 + 22.35 + 24.50 + 26.65 + 28.75 + 39.85$
$+ 32.90 + 34.90 + 36.90 + 38.90 + 40.85 + 42.75 + 44.65$
$+ 46.50 + 48.30 + 50.10 + 51.85 + 53.55 + 55.25 + 56.90)$
$+ 212039.9 \times 37.9 - 242767 \times 57.65.$

$BM_8 = 6936.2\ (866.69) + 212030.9 \times 37.9 - 242,767 \times 57.65.$

$BM_8 = 6,001,535.2 + 8,015,971 - 13,995,517.5$

$BM_8 = 21,988.7'\#$

$BM_9 = 6936.2\ (.55 + 3.00 + 5.45 + 7.90 + 10.35 + 12.80 + 15.20$
$+ 17.55 + 19.90 + 22.20 + 24.45 + 26.70 + 28.90 + 31.10 +$
$33.25 + 35.40 + 37.50 + 39.60 + 41.65 + 43.65 + 54.65 +$
$47.65 + 49.60 + 51.50 + 53.40 + 55.25 + 57.05 + 58.85$
$+ 60.60 + 62.30 + 64.00 -+ 65.65) + 212030.9 \times 39.4 -$
$242,767 \times 66.3$

$BM_9 = 6936.2\ (1,128,60) + 212030.9 \times 39.4 - 242,767 \times 66.3.$

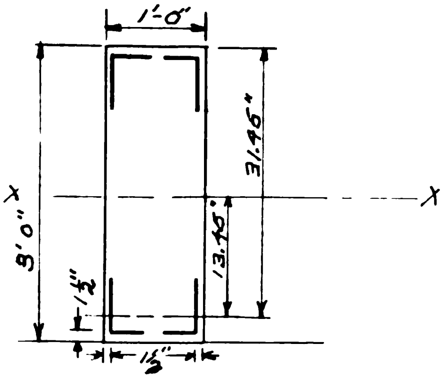
$BM_9 = 7,808,195.3 + 8314017.4 - 16,095,452.1.$

$BM_9 = 36,760.68'\#$

Reinforcing **Steel**.

Assume 4-8" x 4" x 1" angles.

Test for unit stress



Moment of Inertia of transformed area of angles about xx.

$$I = 4 \times 15\,(1/12 \times 11 \times 1^3 + 11 \times \overline{13.45}^2)$$

$$= 119{,}451''4$$

$$fe = \frac{P}{A} + \frac{MC}{I}$$

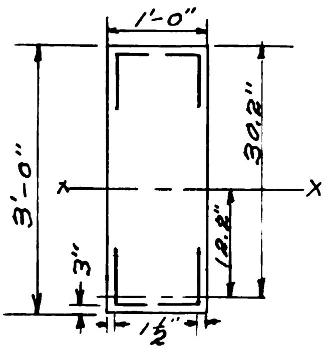
$$= \frac{\frac{45{,}525 \times 12}{1.0}}{36 \times 12 + 4 \times 1 \times 15} + \frac{45{,}525 \times 12 \times 18}{1/12 \times 12 \times 36^3 + 119451}$$

$$= 502 + 59.4$$

$= 561.4$ or 442.6 #/□" too small.

Assume 4-8" x 4" x ½" angles.

Moment of inertia of transformed area of angles of axisxx



$$I = 4 \times 15\,(1/12 \times 5.75 \times 1^3 + 5.75 \times \overline{12.2}^2)$$

$I = 13{,}402''^4$

$$fc = \frac{\frac{45525 \times 12}{1.0}}{36 \times 12 \times 4 \times 5.75 \times 15} + \frac{45525 \times 12 \times 18}{1/12 \times 12 \times 36^3 + 13{,}402}$$

fc = 714 I 164

Fc = 878 or 550#/sq."

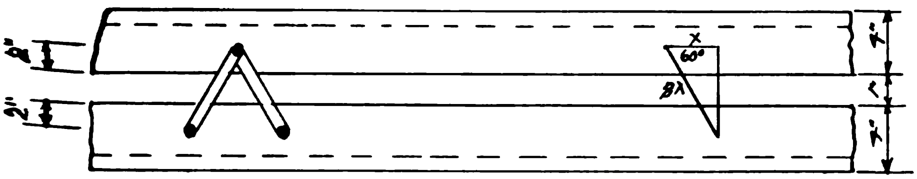
$$Fs = \frac{15 \times 12.2 \times 8.78}{18}$$

Fs = 8,928#/sq."

Design of top and bottom lacing.

Compression $S = 15000 - 50 \frac{l}{r}$

Lacing stress $= 50 \frac{l}{r}$



Lacing Load = .025 x 45.525 = 1138#

Use single lacing at 60° with length of angle.

Straps - 1½" x ½"

$$K = \sqrt{\frac{I}{A}} = \left(\frac{1/12 \times 1.5 \times .5^3}{1.5 \times .5}\right)^{\frac{1}{2}}$$

$K = .144$

$2 + 2 + 1 = 5"$

$X = 5 \times \tan 30^\circ = 5\,(.57735) = 2.88675$

Specifications.

Center 3/4" rivet hole not less than $1\frac{1}{4}"$ from sheared edge.

Lacing Length = 2X = 5.77" + $1\frac{1}{4}"$ at each end = $7\frac{1}{4}"$

$\frac{L}{V} = \frac{5}{.144} = 34.7$ O.K. allowable = 170

1138# for each single bar.

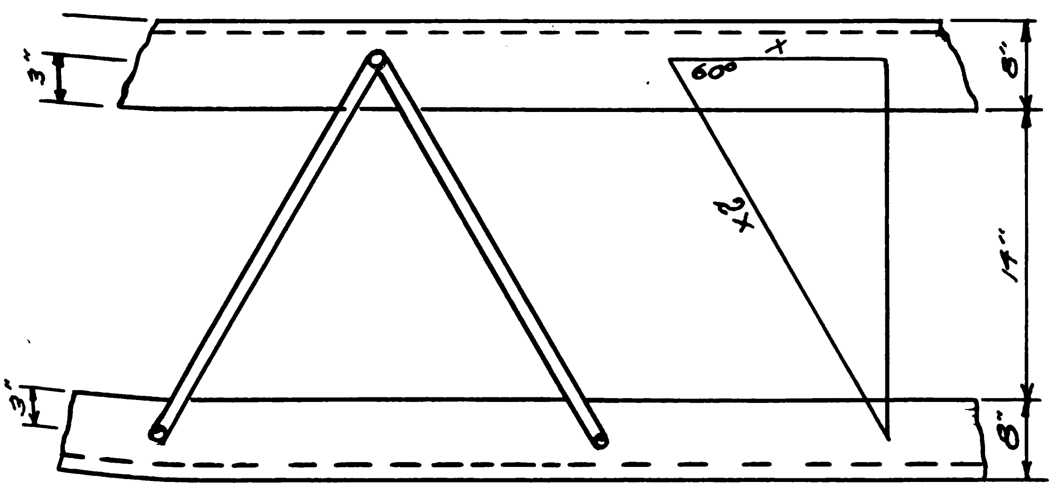
$\frac{1138}{.866} = 1311$# for each single lacing.

$S = \frac{1311}{.5 \times 1.5} = 1750$#/ sq" stress.

Allowable - $S = 15000 - 50\frac{1}{r}$

$= 15000 - 50\,(34.7) = 13{,}265$#/ sq."

Design of Side Lacing.



$3 + 3 = 20"$

$X = 20 \tan 30^\circ = 20 \times .57735 = 11.54"$

Lacing length = 2x = 23.09, say 23.1

Overall length = 23.1 + $1\frac{1}{4}$ at each end = 25.5"

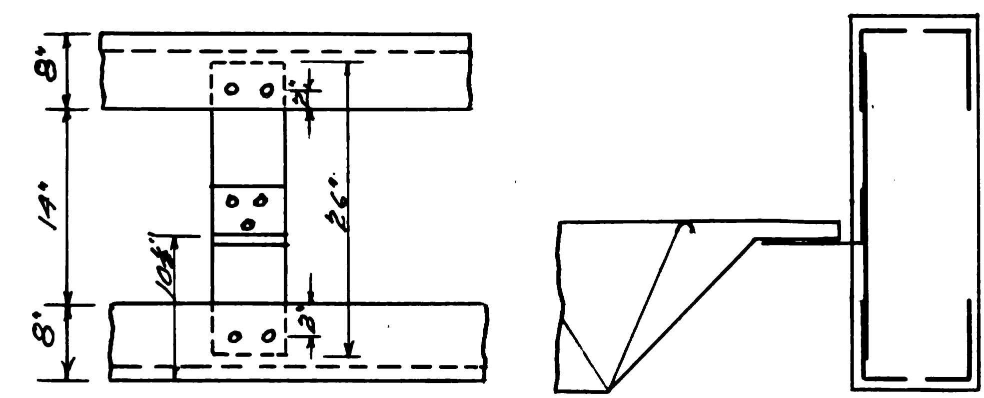
$$\frac{l}{r} = \frac{23.1}{.144} = 165. \text{ O.K. Allowable} = 170$$

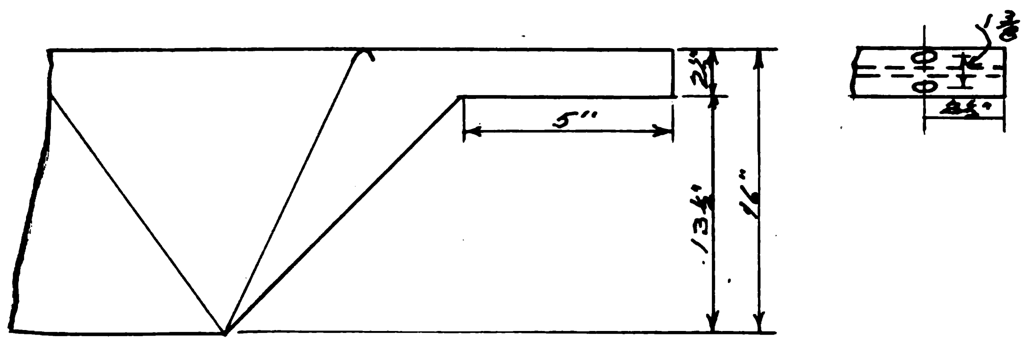
$$\frac{1138}{.866} = 1311\# \text{ for each lacing.}$$

$$S = \frac{1311}{.5 \times 1.5} = 1750\#/\text{ sq." stress.}$$

Allowable $S = 15000 - 50\,(165) = 6750\#/$ sq. in.

Design of plate to hold bearing angle.





Assume Plate.

$2\frac{1}{2}" \times 26" \times \frac{1}{2}"$

Shear in rivets.

S = 1752 + 153.6 + 1000 = 2905.6   O.K. for sheer.

Allowable 4420#/ 3/4" rivets.

The rivets should be 2-3/4" both bottom and top.
Tension in bottom rivets due to the bending moment.

$$\frac{2905.6\ (2.5 + \frac{1}{4} + 1\frac{1}{2} - 3/4)}{10.86} = 1340\#$$ Stress in rivets.

$$\frac{1340}{.4418 \times 2} = 1462\ \#/\ sq."$$ Allowable tension = 16,000#/sq.'

Design bearing Angle.

Test for shear in rivets.

$$\text{No. of rivets} = \frac{2905.6}{4420} = .658$$ Use 3 rivets.

Test for tension in top rivets.

$$\frac{2905.6\ (2.5 + \frac{1}{4} + 1\frac{1}{2} + 3/4)}{3.89} = 3738\#$$

$$\frac{3738}{.4418 \times 2} = 4230\#/\ sq.\ in.$$ Allowable = 16,000#/$\square$"

Test for bending moment.

$$S = \frac{MC}{I} \qquad S = \frac{2905.6 \times 2\ 3/4 \times \frac{1}{4}}{1/12 \times 2.5 \times \frac{1}{2}^3} = 76,700\#/\ sq.\ in.$$

Not safe.

Assume 8" x 4" x 1" angle.

Let S = 16000 and solve for length of angle.

$$16000 = \frac{2905.6 \times 2\,3/4 \times \frac{1}{2}}{1 \cdot \times .13}$$

$$X = 2.99"$$

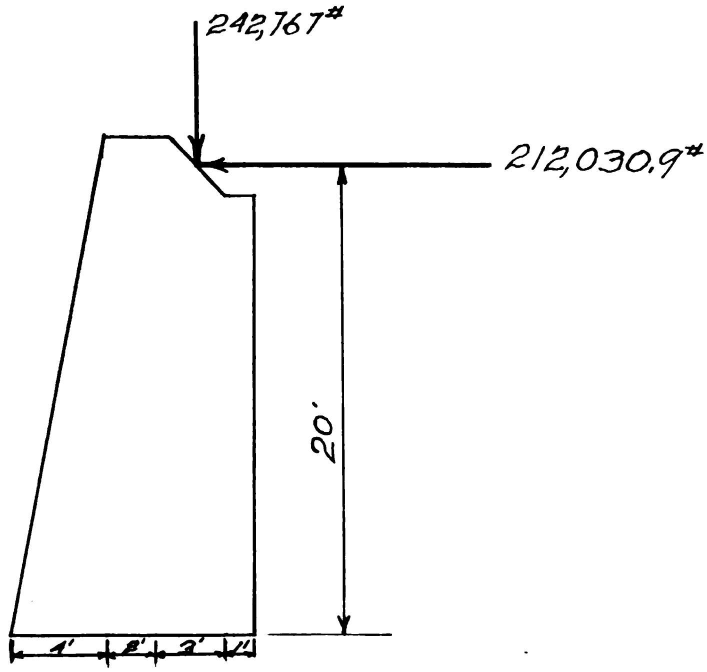
Use 3 " Length of angle.

Bearing Angle - 8"x 4" x 1" x 3".

New size of bearing plate
26" x ½" x 3".

Design of Pillar

Use approximate dimensions on first trials.

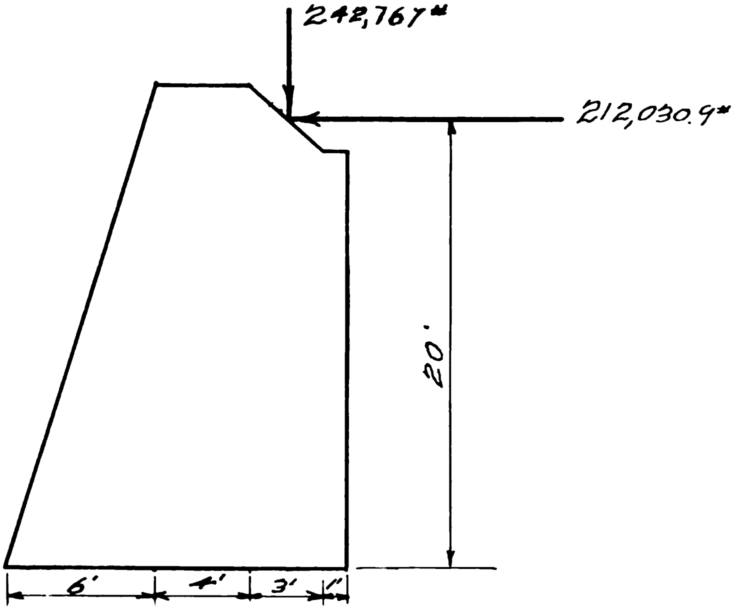


Assume pillar 1' thick.

$$B.M._o = 212030.9 \times 20 + 242{,}767 \times 7\tfrac{1}{2} + 18 \times 1 \times 150 \times 9.5 + 20 \times 3 \times 150 \times 7.5 + 2 \times 22 \times 150 \times 5 + 2 \times 22 \times 150 \times 8/3$$

$$B.M._o = - 4{,}240{,}600 + 1{,}820{,}752 + 25{,}650 + 67{,}000 + 33{,}000 + 17{,}600$$

B.M.$_o$ = − 2,275,598'#. Very unsafe.



Assume thickness of pillar as 3'

B.M.$_o$ = − 212030 x 20 + 242767 x 12 + 1 x 18.5 x 3 x 150 x 13.5 + 3 x 3 x 20.5 x 11.5 x 150 + 4 x 3 x 22.5 x 8 x 150 + 3 x 22.5 x 150 x 3 x 4.

B.M.$_o$ = − 4,240,600 + 2,913,204 + 112,387 + 318,300 + 324,000 + 121,500.

B.M.$_o$ = 451,209'# Unsafe.

Make base 2' longer on outside.

B.M.$_o$ = 212030 x 20 + 242767 x 14 + 18.5 x 3 x 150 x 15.5 + 3 x 3 x 20.5 x 13.5 x 150 + 4 x 3 x 22.5 x 10 x 150 + 3 x 22.5 x 150 x 3 x 5 1/3.

B.M.$_o$ = 4,240,600 + 3,398,738 + 129,037 + 373,650 + 236,250 + 162,000.

B.M.$_o$ = + 59,075'#

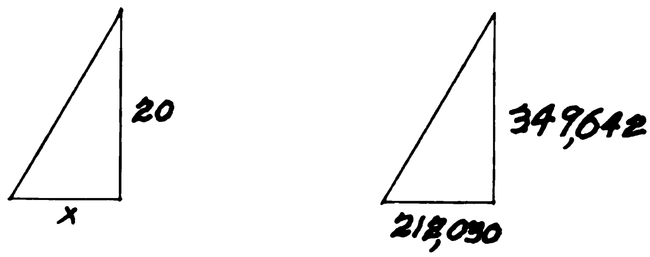
Point Applications of vertical forces.
Wt. of pillar.

150 x 3 (1 x 18.5 + 20.5 + 4 x 27.5 + 3 x 22.5) = 106,875#

$$\frac{129{,}037 + 373{,}650 + 236{,}250 + 162{,}000 + 3{,}398{,}738}{106{,}875 + 242{,}767} =$$

$$\frac{4{,}299{,}675}{349.642} = 12.31 \text{ ft. from toe}$$

From similar triangle.

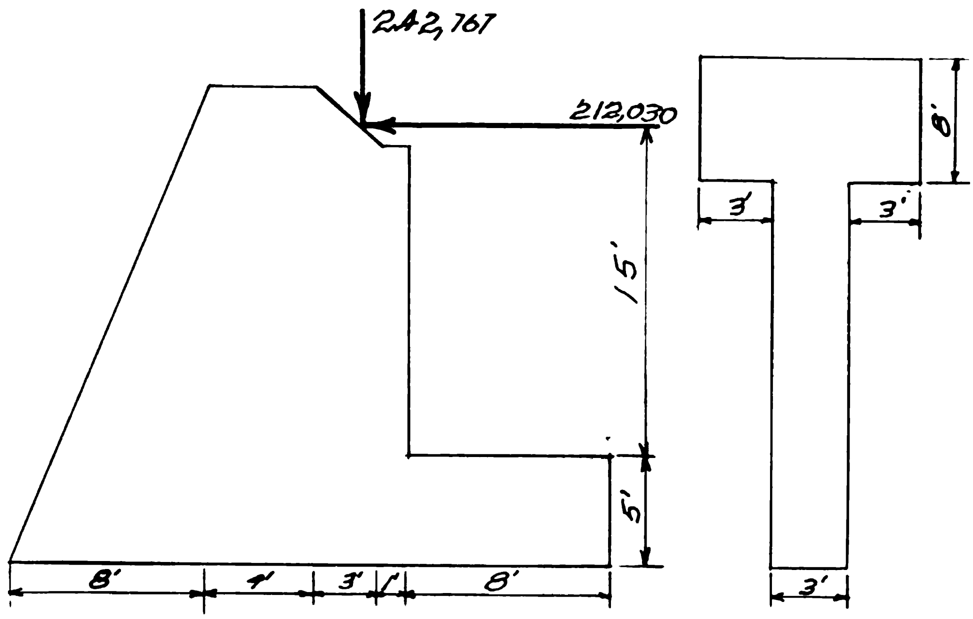


$$\frac{X}{212030} = \frac{20}{349,642}$$

$$X = \frac{212030 \times 20}{349642} = \frac{4240600}{349642}$$

X = 12.15' Where resultant force hits base.
Way outside middle 1/3.

Assume new design.
Have section of floor slab as part of pillar.
Bottom of pillar 5' below surface of ground.



Wt. of Pillar.

3 x .50 (4 x 22.5 + 4 x 22.5 + 3 x 20.5 + 1 x 18.5) +
8 x 5 x 9 x 150 = 180,000#

Pt. app. V forces.

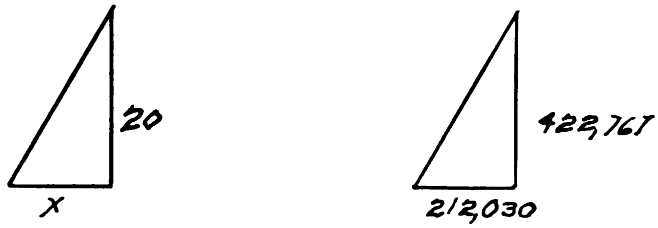
$$X \text{ from toe} = \frac{54{,}000 \times 20 + 8{,}325 \times 15.5 + 27{,}675 \times 13.5 + 40500 \times 10}{422767}$$

$$+ \frac{40{,}500 \times 16/3 + 242767 \times 14}{422{,}767}$$

$$X = \frac{1{,}080{,}000 + 29{,}000 + 373{,}600 + 405{,}000 + 216{,}500 + 3\ 398.\ 738}{422{,}767}$$

$$X = \frac{5782338}{422{,}767} = 13.66' \text{ from toe}$$

From similar triangles.



$$\frac{X}{212030} = \frac{20}{422767}$$

$$X = \frac{20 \times 212030}{422767} = 10.05'$$

$Z = 13.66 - 10.5 = 3.61$

$C = 12 - 3.6 = 8.39'$ Middle 1/3 = 2'

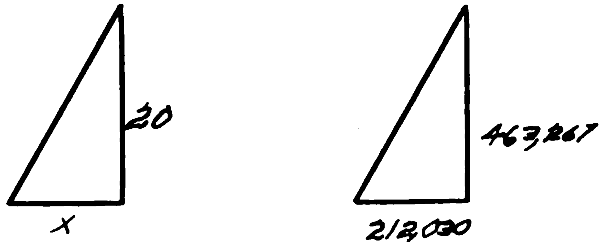
Assume toe section 8' longer on bottom.
Pt. App. of V. Forces.

$$X = \frac{63{,}000 \times 28 + 8325 \times 23.5 + 27{,}675 \times 21.5 + 40500 \times 18}{463267}$$

$$+ \frac{2 \times 40500 \times 3\ 2/3 + 242767 \times 22}{463267}$$

$$X = \frac{8488483}{463267} = 18.35' \text{ from the toe}$$

From similar triangles.



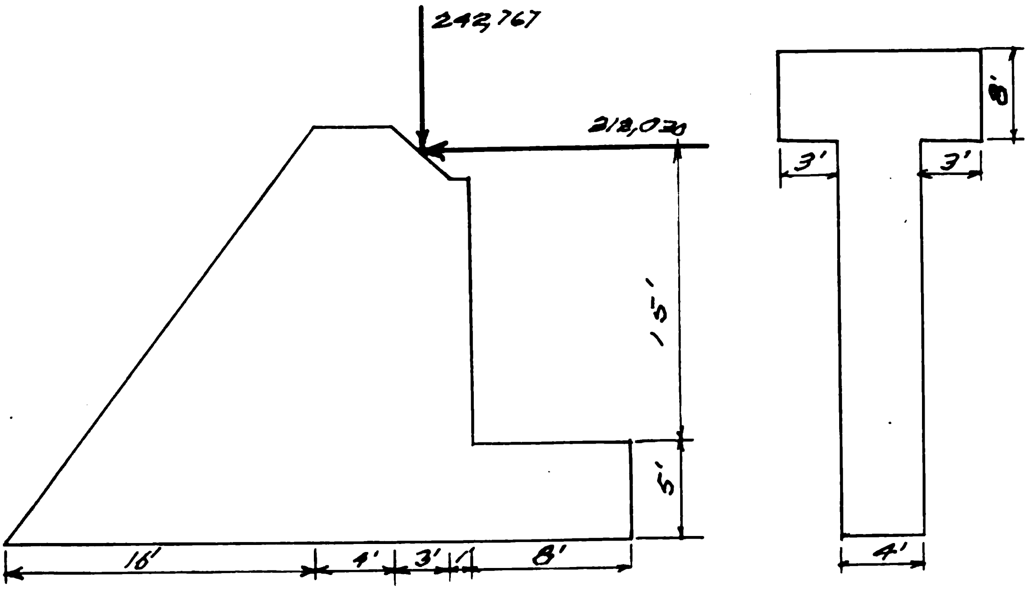
$$X = \frac{20 \times 212030}{463,267}$$

$X = 9.16'$

$Z = 18.35 - 9.16' = 9.19$

$C = 16 - 9.19 = 6.81'$ Middle $1/3 = 5.33'$

Assume Pillar, 4' wide other dimensions the same.



Wt. of Pillar.

$8 \times 5 \times 10 \times 150 + 18.5 \times 1 \times 4 \times 150 + 20.5 \times 3 \times 4 \times 150$
$+ 4 \times 4 \times 22.5 \times 150 + 8 \times 22.5 \times 4 \times 150$
$= 60,000 + 11,100 + 40,800 + 54,240 + 108,000 = 274,140\#$

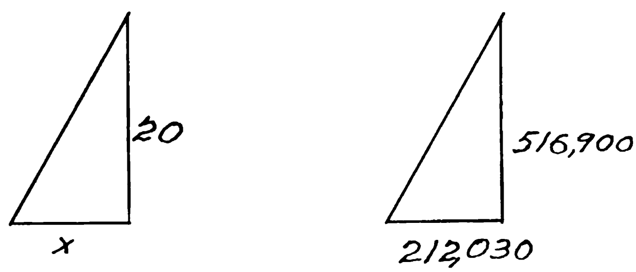
Pt. App. V. Forces.

$$X = \frac{60,000 \times 28 + 11,100 \times 23.5 + 40800 \times 21.5 + 54,240 \times 18}{516900}$$

$$+ \frac{108000 \times 3\ 2/3 + 242767 \times 22}{5169000}$$

$$X = \frac{10,291,874}{516,900} = 19.9'$$

From similar triangles.

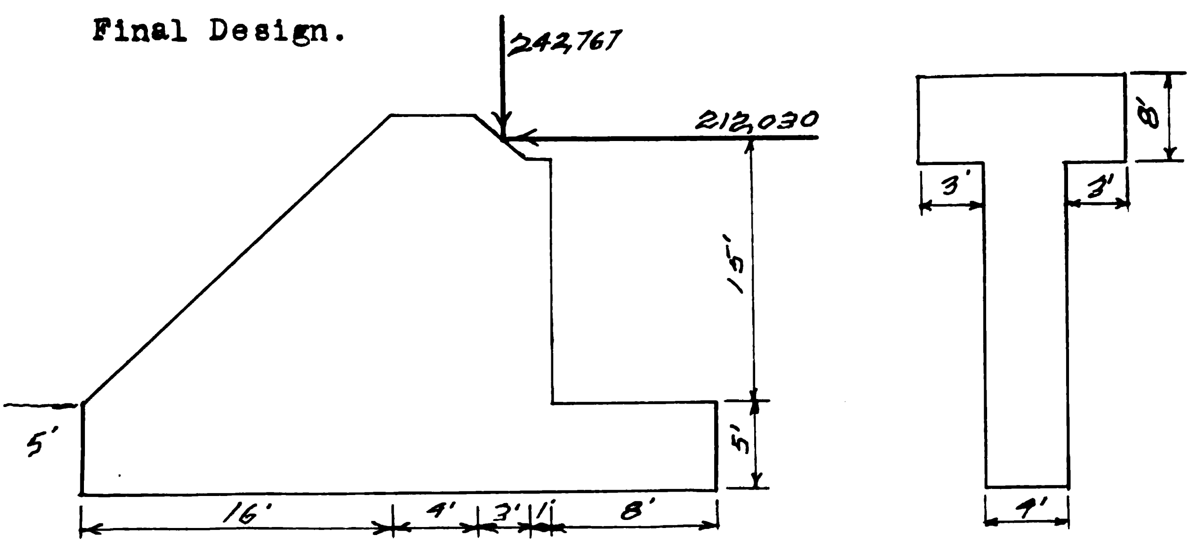


$$X = \frac{20 \times 212030}{516,900} = 8.22$$

$$Z = 19.9 - 8.22 = 11.68$$

$C = 16 - 11.68 = 4.32'$ **Middle** $1/3 = 5.33'$
Within middle 1/3

Final Design.



Wt. of Pillar.

$$= 8 \times 5 \times 10 \times 150 + 4 \times 24 \times 5 \times 150 + 1 \times 13.5 \times 4 \times 150 + 3 \times 15.5 \times 4 \times 150 + 4 \times 17.5 \times 4 \times 150 + 8 \times 17.5 \times 4 \times 150.$$

$$W = 60{,}000 + 72{,}000 + 8{,}100 + 27{,}900 + 42{,}000 + 84{,}000.$$

$$W = 294{,}000$$

$$W = 294000$$

Pt. App. V Forces.

$$X = \frac{60{,}000 \times 28 + 72{,}000 \times 12 + 8{,}100 \times 23.5 + 27{,}900 \times 21.5 + 42{,}000 \times 18}{536767}$$

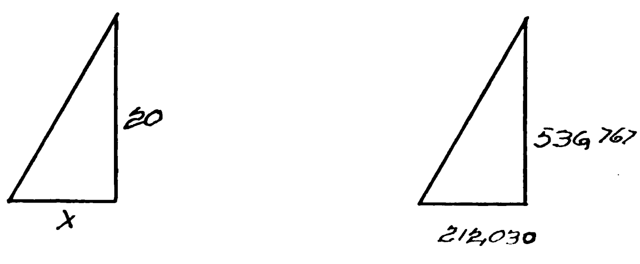
$$+ \frac{84000 \times 3\ 2/3 + 242767 \times 22}{536{,}767}$$

$$X = \frac{1{,}680{,}000 + 864{,}000 + 190{,}000 + 600{,}000 + 756{,}000 + 897{,}000}{536767}$$

$$+ \frac{5{,}340{,}874}{536767}$$

$$X = \frac{10{,}327{,}874}{536767} = 19.3' \text{ from toe.}$$

From similar triangles.



$$X = \frac{20 \times 212030}{536{,}767} = 7.9'$$

$$Z = 19.3 - 7.9' = 11.4$$

$$C = 16 - 11.4 = 4.6' \quad \text{Middle } 1/3 = 5.33'$$

With in middle 1/3.

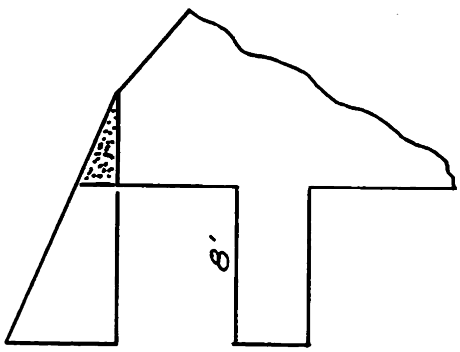
Sliding.

$$F = \frac{536767 \times .4}{212030} = 1.015 \text{ Not too safe}$$

Use key to increase F.

Passive earth pressure.

$$C'e = \text{Cos. } \theta \ \frac{\text{Cos. } \theta + (\text{Cos}^2\ \theta - \text{Cos}^2\ \phi)^{\frac{1}{2}}}{\text{Cos.}\theta\ (\text{Cos.}2\ \theta - \text{Cos.}2\ \phi)^{\frac{1}{2}}}$$



$$P_p = Ce' \ \frac{wh^2}{2} = \frac{3.7 \times 100 \times 169 \times 4}{2} = 125{,}000\#$$

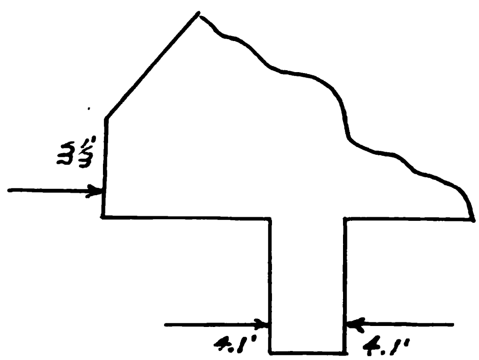
$$= \frac{3.7 \times 100 \times 25 \times 4}{2} = 4{,}625\#$$

Passive earth pressure on key - 120,375#/4'

Sliding.

$$F = \frac{536767 \times .4 + 120{,}375}{212030} = 1.6$$

Pt. app. of forces on key.



$$B.M._e = 125{,}000 \times \frac{13 \times 2}{3} = 4625 \times \frac{5 \times 2}{3} + 120{,}000 \cdot x$$

$$X = \frac{1,074,600}{120375} = 8.9'$$

Active pressure on stem.

126000# of concrete.

$$\frac{126000}{16 \times 4 \times 150} = 13.1 \text{ ft. of concrete.}$$

$$\frac{13.1}{X} = \frac{100}{150}$$

$$X = \frac{150 \times 13.1}{100} = 19.71$$ equivelent height of dirt.

$$P_{27} = \frac{.27\ 100 \times 27^2}{2} = 10550\#$$

$$P_{29} = \frac{.22\ 100 \times 19^2}{2} = 4.860$$

Active pressure on stem = 5690#/'

22,760#/4'

Pt. of App.

$$10550 \times 27 \times 2/3 = 4860 \times 19\ 2/3 + 5690 \cdot x$$

$$x = 22.9'$$

$$\text{B.M. (max.)} = \frac{(120375 - 22760)}{4} 3.9 = 69149.6'\#$$

$$bd^2 = \frac{M}{K}$$

$$d = \frac{69149.6 \times 12}{164 \times 12} = 20.5 \text{ say } 21'' \quad D = 24''$$

Shear.

$$V = \frac{120,375 - 22760}{4} = 24615$$

$$v = \frac{V}{bjd} = \frac{24615}{12 \times 7/8 \times 21} = 111.5$$ too large.

Make d = 47"

$v = \frac{24615}{12 \times 7/8 \times 47} = 49.8\#/\square''$ O.K. Allowable = 50

Steel =

$A_s = pbd = .0094 \times 12 \times 47 = 5.3$ sq."

Use $1\frac{1}{4}$" sq. rods @ $3\frac{1}{2}$ C-C Area = $5.36\square'$

Bond

$U = \frac{V}{\Sigma o j d} = \frac{24615}{5 \times 12/3.5 \times 7/8 \times 47} = 34.9$ O.K.

Allowable = 100
125

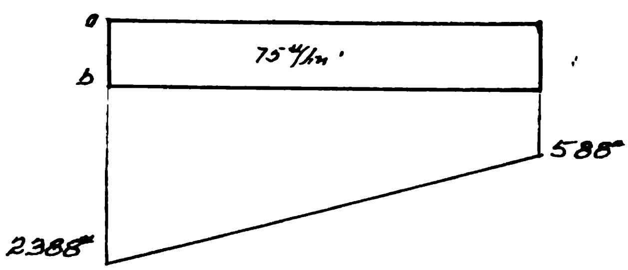
Soil Pressure.

$P = \frac{V}{b}\left(1 \pm \frac{6c}{b}\right)$

$P = \frac{\frac{536767}{4}}{32}\left\{1 + \frac{6 \times 4.6}{32}\right\} = \frac{134192}{32}(1 \pm .86)$

P = 7,800#, 588#/ sq.'
Allowable on good clay and gravel.
6 tons / sq' = 12,000#/ sq.'

Test of heel slabs which composes part of interior floor.



$588 + \left(\frac{7{,}800 - 588}{32}\right) = 2388\#$

$B.M._{ab} = -750 \times 8 \times 4 + 588 \times 8 \times 4 + \frac{2388}{2} \times 8 \times 8/3$

B.M.ab $= 24000 + 18,816 + 25,472$

B.M.ab $= 20,288$'#

$$d = \left(\frac{20288 \times 12}{164 \times 12}\right)^{\frac{1}{2}} = (123.5)^{\frac{1}{2}}$$

$d = 11.1$ say 12"

Steel.

$A_s = pbd = .0094 \times 12 \times 12$
$= 1.354$ sq."

Use 7/8" round @ 5" area $= 1.44$ sq."

Shear.

$$V = 750 \times 8 + 588 \times 8 + \frac{2388}{2} \times 8$$
$$= -6000 + 4704 + 9552$$
$$= 8256\#$$

$$v = \frac{V}{bjd} = \frac{8256}{12 \times 7/8 \times 57} \quad 13.8\#/\text{ sq.}"$$ Allowable $= 50.$

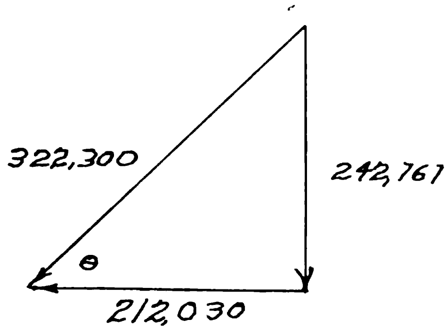
Bond.

$$u = \frac{V}{\Sigma ojd} = \frac{8256}{2.75 \times 12/5 \times 7/8 \times 12} = 111.5$$ Allowable $= 125$

Deformed bars.

Design of Hinge and Pin at Pillar.

Angle resultant makes with horizontal at pillar.



$$\text{Tan.}\,\theta = \frac{242767}{212030} = 1.14497$$

$\theta = 48^\circ\ 52'$

$\text{Sin.}\ \theta = .75318$

$\text{Cos.}\ \theta = .65781$

Assume a 6" pin.

Total bearing pressure = 322,300#

Allowing bearing stress in a cast steel pin = 99,600#/sq."

Bearing surface = 2 surfaces @ 2"

Bearing area = 6 x 2 x 2 = 24 sq."

$\frac{322,300}{24}$ = 13,420#/ sq. " O.K.

Shear.

$\frac{322300}{\pi \times 32}$ = 11,390#/ sq." Allowable 44,000, Single shear.

Allowable 88,000, double shear.

Hinge in pillar.

Allowable bearing stress in.

Concrete = 350#/ sq."

Size of Base.

$\frac{322,300}{350}$ = 922 sq."

Select 3' x 3' base.

Area = 1296 sq."

Design of Hinge and Pin at crown.

Assume a 4" pin.

Bearing area = 4 x 4 = 16 sq."

Force on pin = 212,030#

$\frac{212030}{16}$ = 13,260 #/ sq." O.K.

Shear.

$\frac{212030}{\pi \times 2}$ = 16850#/ sq." O.K.

Hinge connection to the structural steel at the crown and pillar. Use 2-8" x 4 x 3/4 angles to go vertical between structural steel.

The hinge will be held in place by the angles but will also bear directly on the structural steel.

[redacted]

[redacted]

SUPPLEMENTARY
MATERIAL