D E S I G N of R A Y M O N D T H E A T R E

Pasadena California.

in
Reinforced Concrete

T H E S I S
by
Morris Goldsmith.

dead wt. of Roofing Material = 10 lb./sq. ft.

$\tan\theta = \frac{4}{50} = .08$

$\theta = 0°\text{-}30'$

Normal wind Pressure at P = 30 lb. per foot = 5 lbs. Ketchum. Pg. 6.

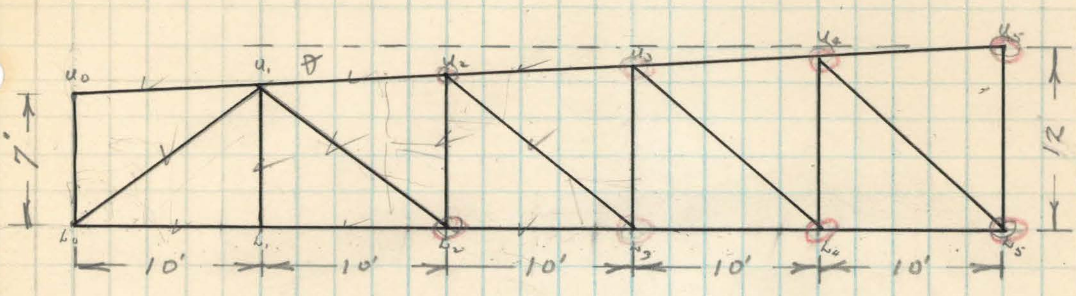
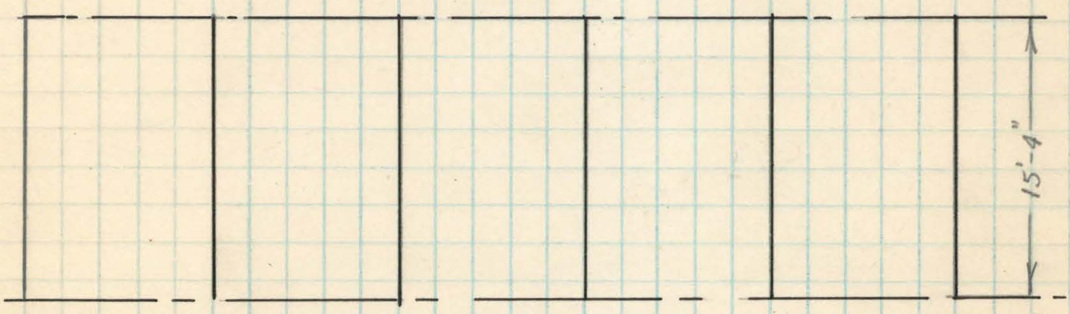
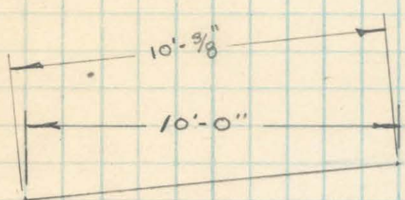


Fig. 1





assume dead load of Slab	= 50	lb. sq. ft.
Roofing "	= 10	" " "
Wind "	= 5	" " "
2" of Rain water	= 10	" " "
	75	" " "

Since this slope is very small this slab can be computated similar to a horizontal floor slab.

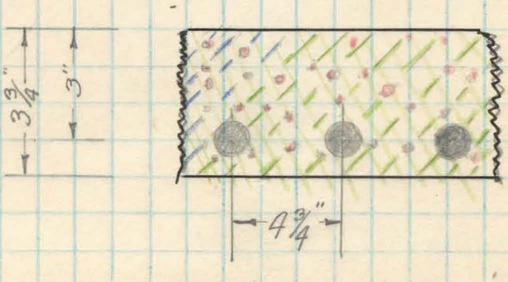
$M = \frac{75 \times 100 \times 12}{12} = 7500$ in-lb. $f_s = 16{,}000$ $f_c = 650$ $n = 15$.

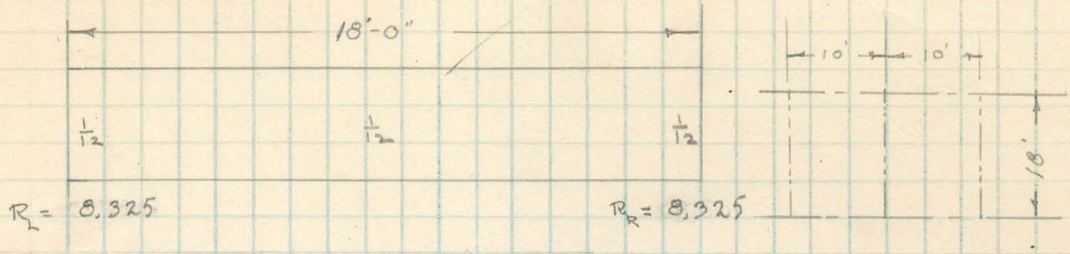
take d = 3"

Area of steel per foot of breadth = .0077 x 3 x 12 = .277 sq. in.

use 3/8" round rods spaced $4\frac{3}{4}$" c.c. Lefax.

$\frac{3.75 \times 12}{12 \times 12} \times 150 = 47$ lb./□





Total Load at 75 lb. per sq. foot. = 180 x 75 = 13,500 lb.
3,150
16,650

Uniform Load = $\frac{13,500}{18}$ = 750 lb. per foot.
Dead weight = 175 " "
Max. M = $\frac{925 \; 18 \times 18 \times 12}{12}$ = 300,000 in-lb.

f_s = 16,000
f_c = 650
p = .0077

Coeff.	Center	Left.	Right.
M.	300,000	300,000	300,000
R = M/bd²	107.4		
bd²	2790		
b	9"		
d	17.75		
Steel	4 - 5/8 □ rods	4 - 5/8" □ rods	4 - 5/8" □ rods
"		1 - 3/8" □ stirr.	1. 3/8" □ stirr.
Shear #/sq. in.		60#/sq. in	60#/sq. in.
Shear carried by steel.		3250 lb.	3250 lb.
		1 - 1/2" □ stirrup.	1 - 1/2" □ stirrup.

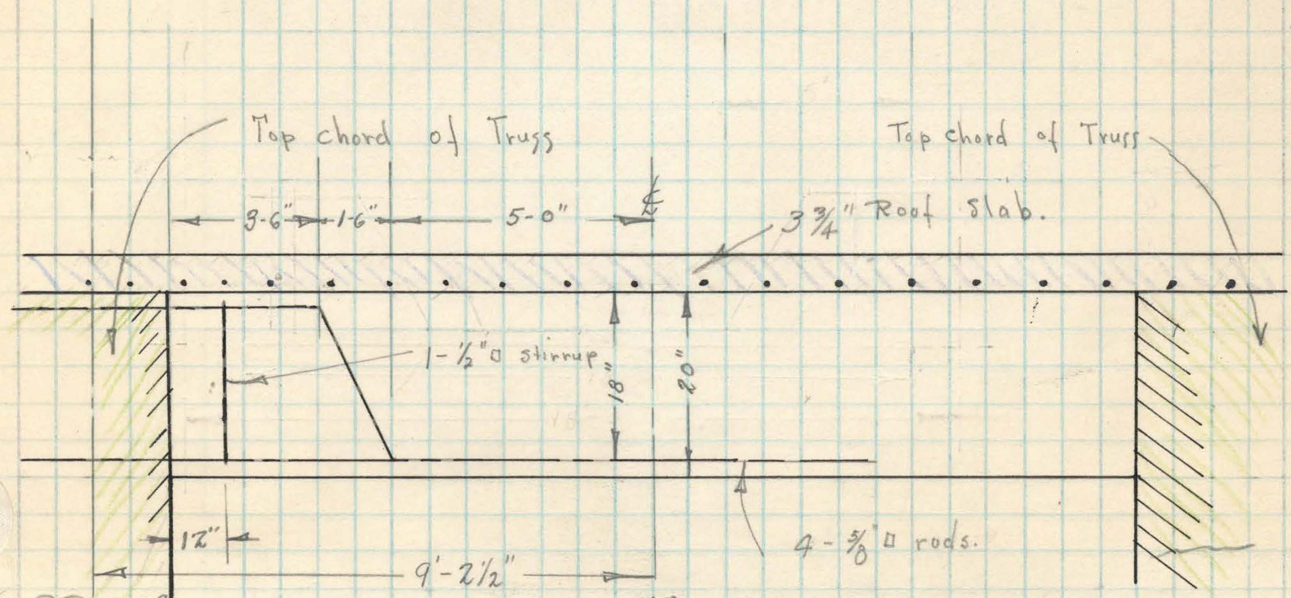
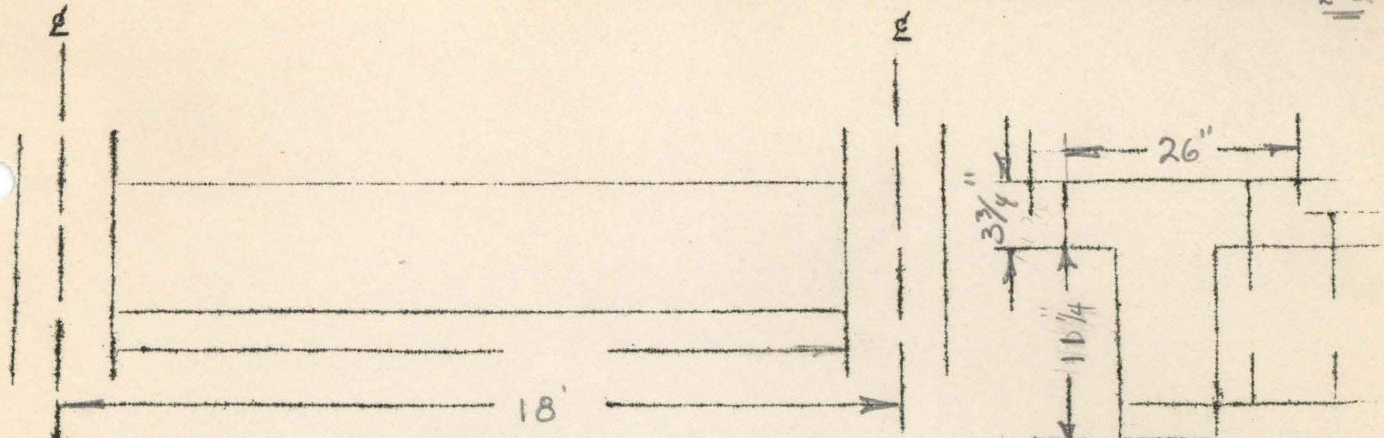


Fig. 2.



R_L ______ R_R ______

Uniform Load per Foot
Slab =
Stem =

Max. Unit Shear=
Stirrups Ns =
1' =
Spacing =

Total Unif.Load= = x
Conc. Loads =
Total Load =
Equiv. Load =

CENTER	SUPPORT	Right	Left
M Coef.= 1/12	M Coef. 1/12		
M = 300,000 "#	M at ℄ of support	"#	"#
M/bd2 = M/29x14² = 52.5	M at face support	"#	"#
b = 26 "	M/bd^2	M/	M/
d = 14 "	Steel top	sq."	sq."
t/d = 2.68	p		
f_c = 450 #/sq."	p'		
jd = .91x14= "	M/bd^2 allowable		
As = 12.7 sq."	f'steel	#/sq."	#/sq."
Steel = 4-5/8"⌀ = 1.57 sq."	Length to develop	"	"
	Bond Stress	#/sq."	#/sq."
p =	Steel bottom	sq."	sq."

$$A_s = \frac{M}{jd\,f_s}$$

$$= \frac{300,000}{12.7 \times 16,000}$$

= 1.48 sq. in.

MARK ______

Computed	Date
Checked	Detailed
Work Order	Page No.

Notes on the computation of dead weight of Roof Truss

At the present time it being impossible to obtain resonably exact estimates on the dead weight of a reinforced concrete truss the following method will be tried ; With the known loads from the roof, and the span given, a beam of reinforced concrete will be designed to carry these loads under the given conditions. The dead weight of this beam will be computated with its proper amount of steel.

The dead weight of this beam will then be assumed to be an approximate weight of the reinforced concrete truss to be designed. The assumtion being that the excess in steel in the reinforced concrete truss, over the steel in the reinforced concrete beam, will compensate for the excess of concrete in the reinforced concrete beam, over the concrete in the reinforced concrete truss'

In using this method the design necessitated a T-Beam. so as to be able to mantain the depth about 12 foot. Under the given loadings it would take a T-Beam weighing about 5000 lbs per lieal foot. This weight seems too high to assume as the dead weight of the reinforced concrete roof truss to be designed.

The metod now to be used will be to compare the weight of the roof trusses in Grauman's Metropalitan Theater Los Angeles, Calif. As these trusses are of Reinforced concrete they will be a a good basis for estimating the ~~weig~~ weight of the trusses under question.

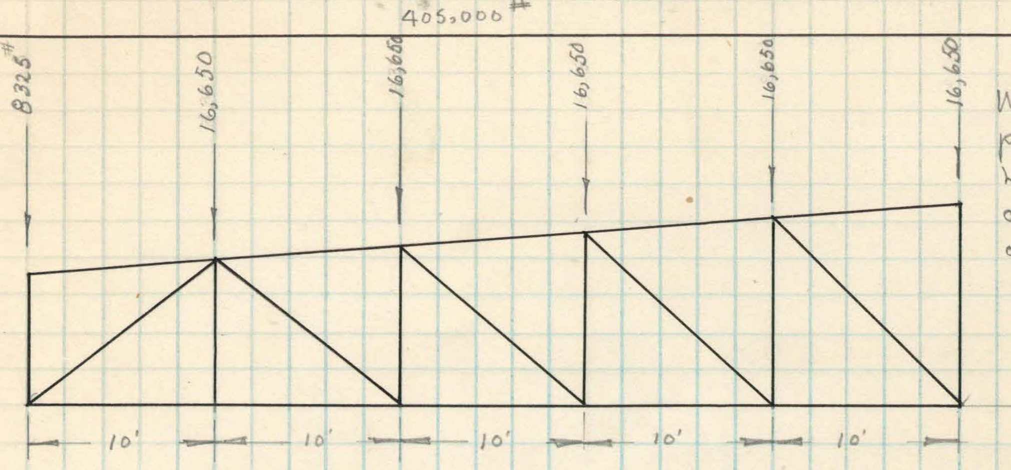
The dead weight of a roof truss in this ~~this~~ theater is composed of 61 tons of reinforcing steel, 231 cu. yd. of concrete and carries a load of 750 tons. The dead weight is

6237 cu, ft. @ 150 #/cu. ft.=	935,550 lbs.
61 tons of steel @ 20000#=	122,000
	1057,550 lbs.

This is about 8400 lbs./ lin' ft.
The load carried is 1,500,000 lbs. or 19000 lbs. /lin. ft.

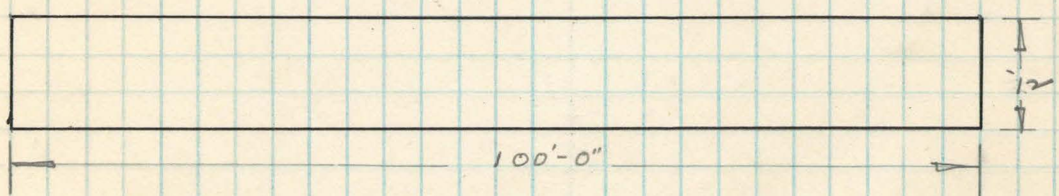
The load per foot in the Raymond Theater, Pasadena, to be carried by the roof trusses is 1665 lbs. The ratio of loads to be carried, in the two theaters, will give a dead load of about 750 lbs. per lineal foot for the truss in the Raymond theater.

The weight of a steel truss for such conditions as exist in the Raymond Theater Would be about 30,000# as calculated by Ketchum's Formula. This would be 300 lbs. per. foot. This multiplied by 3.33(the ratio of allowable stress in steel and concrete, to their weight respective weight) would give about 1000 lbs. per liN. ft.



Weight per panel point due to Live roof load and dead load of Roof slab = 8325 lbs.

See page 2.a· for assumptions made in design.



16,650 lb. per Panel point. Total load = 10 × 16,650 = 160,650 lb.

$\frac{160,650}{100}$ = 1606.5 lb. per lineal foot. 1610 lb. will be used.

Assume the width of Beam to be 2.5 ft.

2. × 11 × 150 = 3300 lb. per lineal foot = dead weight of Beam.

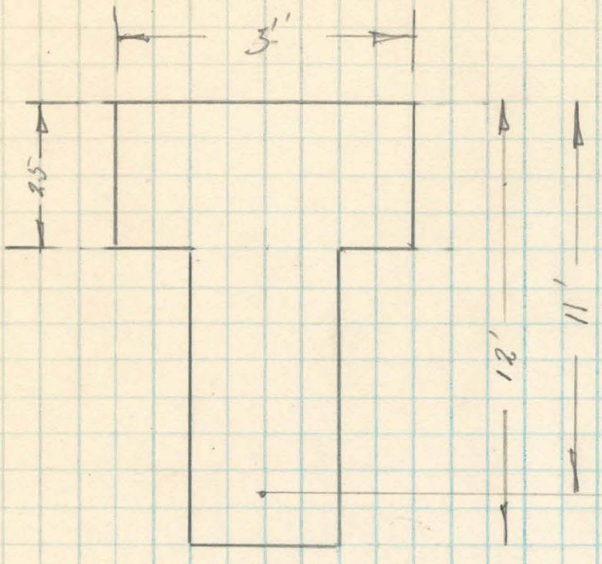
Weight of Floor slab on top chord = 2. × 1 × 75 = 150 lb./lineal ft.

Total load per lineal foot = 1610
3300
150
5060 lb. 5,000 lb. will be used.

$M = \frac{1}{8} w l^2 = \frac{5,000 \times 100 \times 100 \times 12}{8} = 62,575,000 =$ in-lb.

$M = a_s d \times f_s j \qquad a_s = \frac{M}{d f_s j} = \frac{62,575,000}{16,000 \times 10 \times 12 \times .87}$

$a_s = 37.23$ sq in.



$d = 11'$ $t = 2.5'$

$\frac{t}{d} = 2.27$

$k = .335$ $p = .05$ Hool P. 234.

$j = .9$

$\frac{M}{bd^2} = 98$ Hool 283

$\frac{62,575,000}{b \times (11 \times 12)^2} = 98$

$$\frac{62,575,000}{(11 \times 12)^2 \times 98} = b = \frac{62,575,000}{14,610 \times 98} = 43.75''.$$

$b = 4$ ft.

$$a_s = \frac{M}{j \times d \times f_s} = \frac{62,575,000}{.9 \times 121 \times 16,000} = 36 \text{ sq in steel.}$$

$$M_c = \tfrac{1}{2} f_c b t (d - \tfrac{1}{2} t) = .5 \times 650 \times 48 \times 30 \times 106 = 62,500,000 \text{ in-lb}$$

use 37-1" ◻ rods.

Dead weight of this beam will be about 4900 lbs. per lineal foot.

See page no. 2 for explanation of proceeding methods used.

Dead Weight of Truss = 1500 lb. per. lineal foot

Live Load = 16,500 lb. per Panel.

Pg. 3.

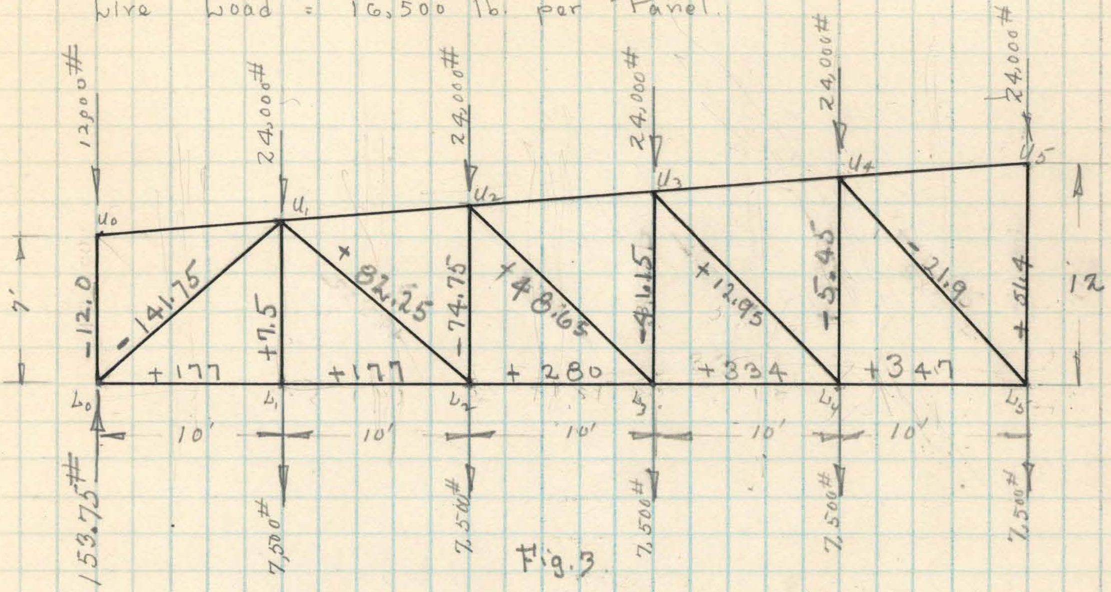


Fig. 3

66.2

$H.C.\ U_4U_5 = 377$, $V.C\ U_4U_5 = 377 \times \frac{1}{10} = 37.7$

$H.C.\ U_3U_4 = 343$ $V.C\ U_3U_4 = 343 \times \frac{1}{10} = 34.3$

$H.C.\ U_2U_3 = 330.7$ $V.C\ U_2U_3 = 330.7 \times \frac{1}{10} = 30.1$

$H.C.\ U_1U_2 = 280$ $V.C.\ U_1U_2 = 280 \times \frac{1}{10} = 28$

$H.C.\ U_0U_1 = 0$

$.8 L_0L_1 - 153.75 + 12.0 = 0$

$L_0L_1 = \frac{141.75}{.8} = +177$

$.9 L_2L_3 - 153.75 \times 2 + 12.0 \times 2 + 31.5 \times 1 = 0$

Stress in members due Wind on side of Building.

30#/sq. ft. Horiz
540#/lin. ft.

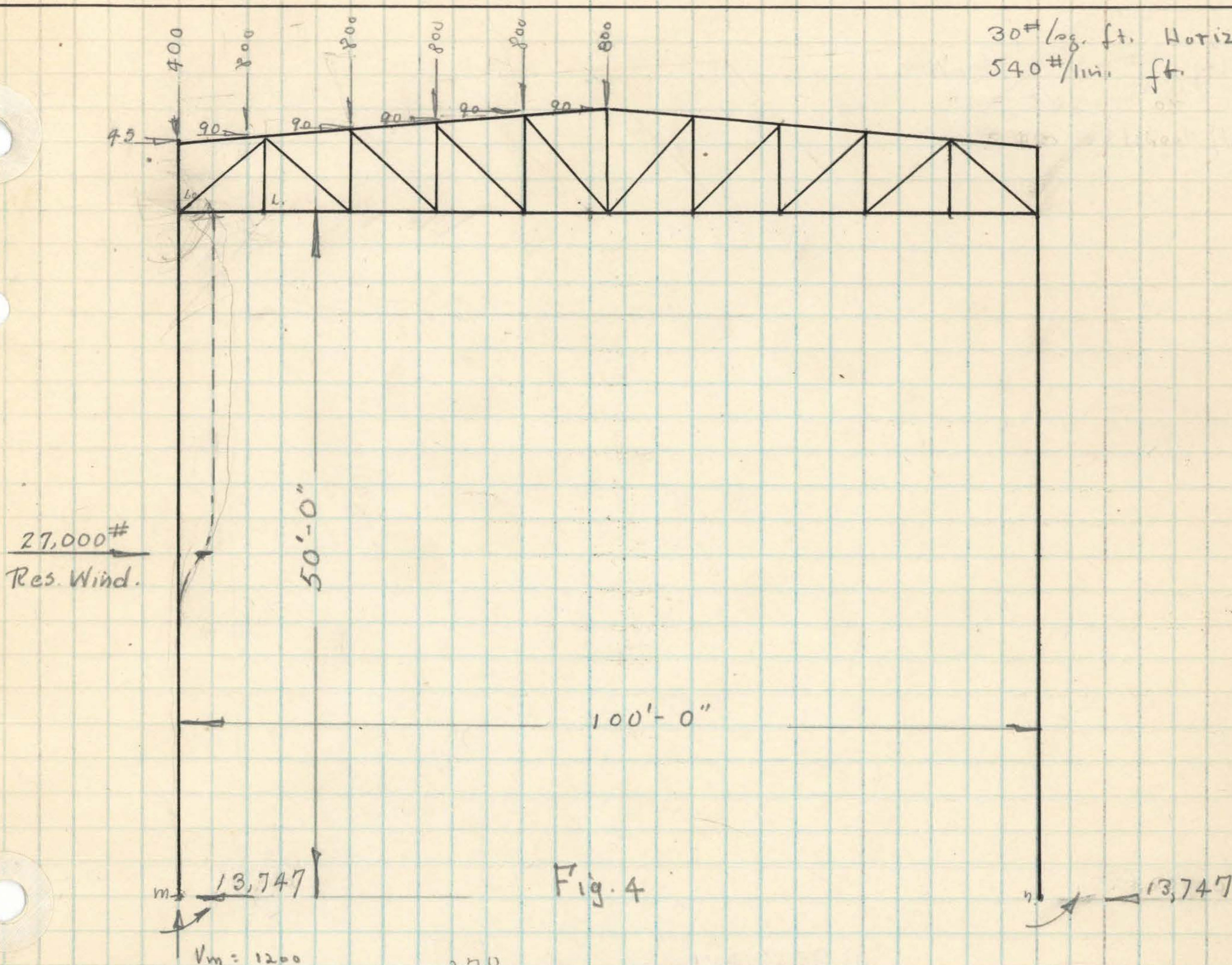


Fig. 4

$$M_m = 13{,}747 \times 25 - \overset{270}{\cancel{540}} \times 25 \times \frac{25}{2} = 174{,}925 \text{ ft-lb.}$$

$$M_n = 13{,}747 \times 25 = 343{,}675 \text{ ft. lb.}$$

$$\overset{-174{,}925}{\cancel{-9{,}175}} + V_m \times 100 + 27{,}000 \times 25 + 495 \times 59.5 - 4400 \times 75 - 343{,}675 = 0$$

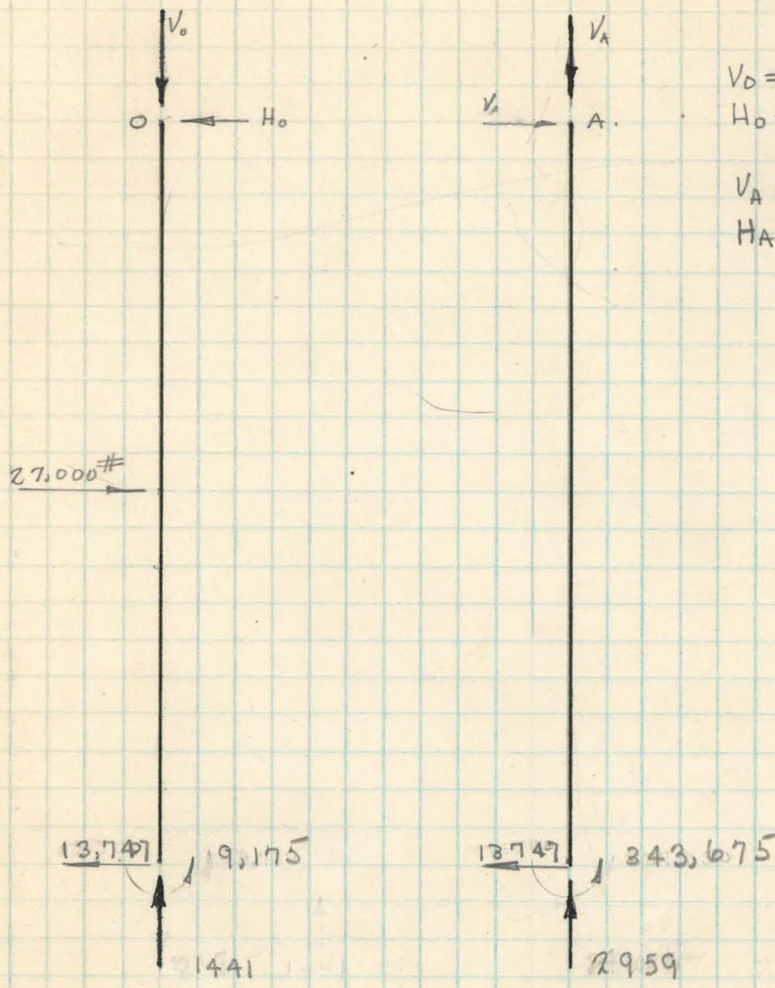
$$-9{,}175 + 100V_m + 675{,}000 + 29{,}452 - 330{,}000 - 343{,}675$$

$$100V_m = \cancel{-21{,}550} + 144{,}148$$

$$V_m = \cancel{-215} \text{ lb.} + 1441.$$

$$\Sigma V = 0 = 4400 - 1441 - V_n = 0$$

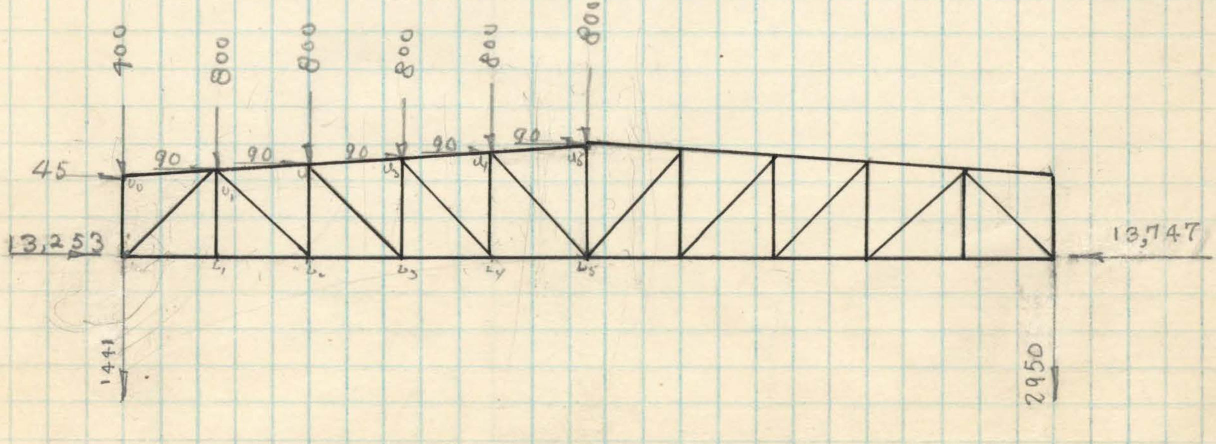
$$V_n = 2950 \text{ lb.}$$



$V_O = +1441$
$H_O = 27,000 - 13,747 = 13,253$

$V_A = +2959$
$H_A = +13,747$

Fig. 5.



Stress in Hundreds of Pounds. - V.C.

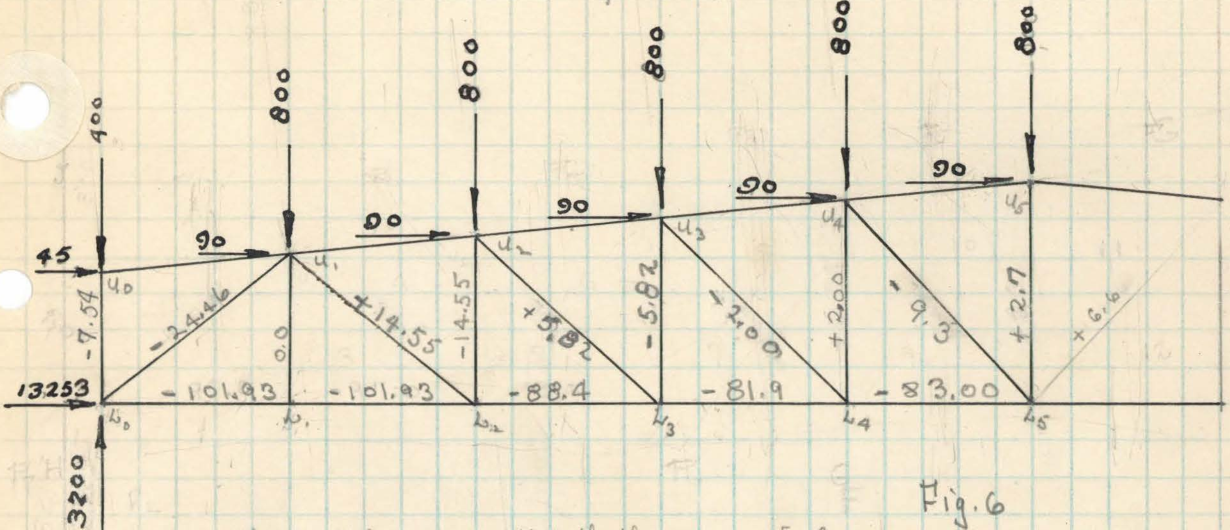
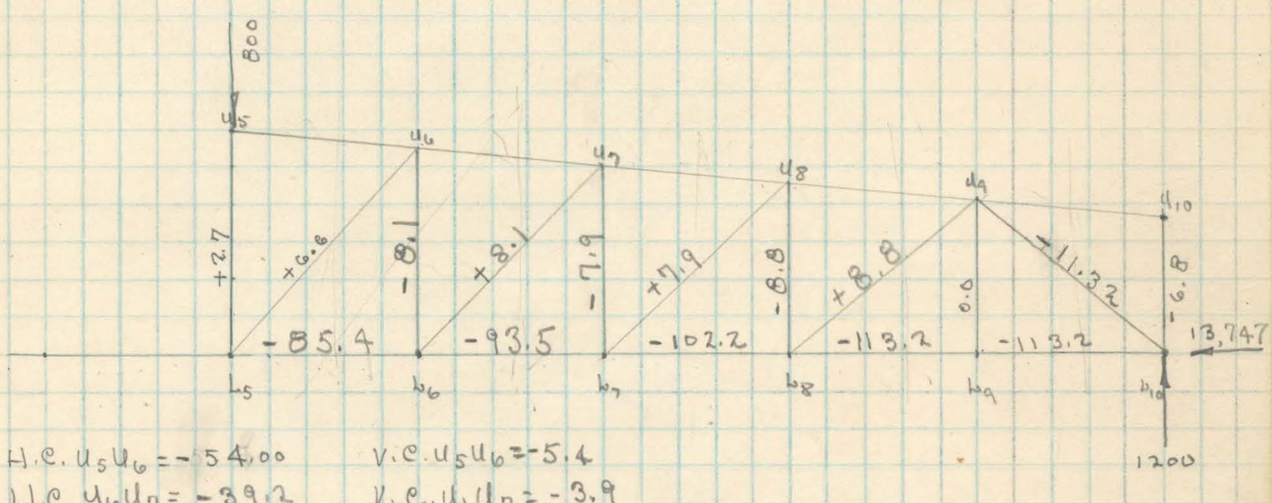


Fig. 6

H.C. U_5U_6 = -5400 V.C. U_5U_6 = -5.4
H.C. U_4U_5 = -5300 V.C. U_4U_5 = -5.3
H.C. U_3U_4 = -6070 V.C. U_3U_4 = -6.00
H.C. U_2U_3 = -618. V.C. U_2U_3 = -6.18
H.C. U_1U_2 = -54.5 V.C. U_1U_2 = -5.45
H.C. U_0U_1 = -35.4 V.C. U_0U_1 = -3.54

H.C. L_0L_1 = - 8[illegible]



H.C. U_5U_6 = -54.00 V.C. U_5U_6 = -5.4
H.C. U_6U_7 = -39.2 V.C. U_6U_7 = -3.9
H.C. U_7U_8 = -40.8 V.C. U_7U_8 = -4.08
H.C. U_8U_9 = -32 V.C. U_8U_9 = -3.2
H.C. U_9U_{10} = -6.8 V.C. U_9U_{10} = -.68

Live, dead, wind and critical stress in members of Roof truss.

In Thousands of Pounds

Member	Length	Dead - Live Stress H.C.	Dead - Live Stress V.C.	Wind Stress H.C.	Wind Stress V.C	Max. Stress.
$U_0 U_1$	10.04'	0	0	- 3.54		- 3.68
$U_1 U_2$	10.04'	- 280		- 5.45		-296.0
$U_2 U_3$	10.04'	- 330.7		- 6.18		-350.0
$U_3 U_4$	10.04	- 343.0		- 6.07		-363.0
$U_4 U_5$	10.04	- 377.0		- 5.3		-398.0
$U_5 U_6$	10.04	- 377.0		- 5.4		-398.0
$U_6 U_7$	10.04	- 343.0		- 3.92		-361.0
$U_7 U_8$	10.04	- 330.7		- 4.08		-348.0
$U_8 U_9$	10.04	- 280.0		- 3.2		-294.0
$U_9 U_{10}$	10.04	0		- .68		- .71
$L_0 L_1$	10.00'	+177.0		- 10.2		+177.0
$L_1 L_2$	"	+177.0		- 10.2		+177.0
$L_2 L_3$	"	+ 280		- 8.84		+ 280.0
$L_3 L_4$	"	+ 334		- 8.2		+ 334.0
$L_4 L_5$	"	+ 347		- 8.3		+ 347.0
$L_5 L_6$	"	+ 347		- 8.54		+ 347.0
$L_6 L_7$	"	+ 334		- 9.35		+ 334.0
$L_7 L_8$	"	+ 280		- 10.2		+ 280.0
$L_8 L_9$	"	+ 177.		- 11.3		+ 177.0
$L_9 L_{10}$	"	+ 177		- 11.3		+ 177.0
$L_0 U_0$	7.0'		- 12.0		- .754	- 12.75
$L_1 U_1$	8.0'		+ 7.5		0.0	+ 7.5
$L_2 U_2$	9.0'		- 74.75		- 1.45	- 76.2
$L_3 U_3$	10.0'		- 41.15		- .582	- 41.73
$L_4 U_4$	11.0'		- 5.45		+ .2	- 5.45
$L_5 U_5$	12.0		+ 51.4		+ .27	+ 51.67
$L_6 U_6$	11.0		- 5.45		- .81	- 6.26
$L_7 U_7$	10.0		- 41.15		- .79	- 41.94
$L_8 U_8$	9.0		- 74.75		- .88	- 75.63
$L_9 U_9$	8.0		+ 7.5		0.0	+ 7.5
$L_{10} U_{10}$	7.0'		- 12.0		- .68	- 12.68
$L_0 U_1$	12.8'		- 141.75		- 2.45	- 231.0
$U_1 L_2$	12.8		+ 82.25		+ 1.45	+ 134.0
$U_2 L_3$	13.45		+ 48.65		+ .58	+ 73.5
$U_3 L_4$	14.14		+ 12.95		- .2	+ 18.3
$U_4 L_5$	14.87		- 21.9		- .93	- 30.9
$U_6 L_5$	14.87		- 21.9		+ .66	- 29.4
$U_7 L_6$	14.14		+ 12.95		+ .81	+ 17.7
$U_8 L_7$	13.45		+ 48.65		+ .79	+ 73.7
$U_9 L_8$	12.8		+ 82.25		+ .88	+ 133.0
$U_9 L_{10}$	12.8		- 141.75		- 1.13	- 228.0

$U_4 U_5$. Stress due to dead, live, and wind 398.000 lb.

length 10.04 ft.

assume a section 20x24" = 480 sq. in. Area.

$wt = \frac{480}{144} \times 150 = 500$ lb./ft.

Moment Due to Dead wt.

$$M = \frac{1}{8} Wl = \frac{500 \times 10 \times 10 \times 12}{8} = 750{,}000 \text{ in.-lb.}$$

eccentricity

$$x_o = \frac{750{,}000}{398{,}000} = 1.88 \text{ in.}$$

Diagram 6 U.D.

$$\frac{x_o}{t} = \frac{1.88}{24} = .0785 \qquad K = \qquad p_o =$$

$$f_c = \frac{WK}{bt} = \frac{398{,}000}{480} K = 550$$

$$K = .665$$

solve for p.

$$K = \left[\frac{1}{1+15p} + \frac{x_o}{t} \cdot \frac{6}{1+28.8p}\right]$$

$$.665 = \left[\frac{1}{1+15p} + .0785 \frac{6}{1+28.8p}\right] = \frac{1}{1+15p} + \frac{.4710}{1+28.8p}$$

$$.665\left[(1+15p)(1+28.8p)\right] = \underline{1+28.8p + .4710(1+15p)}$$

$$.665\left[1+43.8p+434p^2\right] = 1+28.8p + .4710 + 7.06p$$

$$.665 + 29.2p + 288p^2 = 1.471 + 35.06p$$

$$-.806 - 5.8p + 288p^2 = 0$$

$$p = \frac{+5.8 \pm \sqrt{5.8^2 - 4 \times -.806 \times 288}}{576} = \frac{+5.8 \pm \sqrt{33.5 + 930}}{576}$$

$$p = \frac{+5.8 \pm 31.03}{576} = \frac{36.8}{576} = .064$$

Value of p

$p = 6.4\%$ $\quad a_s = 480 \times .064 = 30.7$ sq. in.

use 31 - 1" □ rods.

Superseded.
Too much Steel.

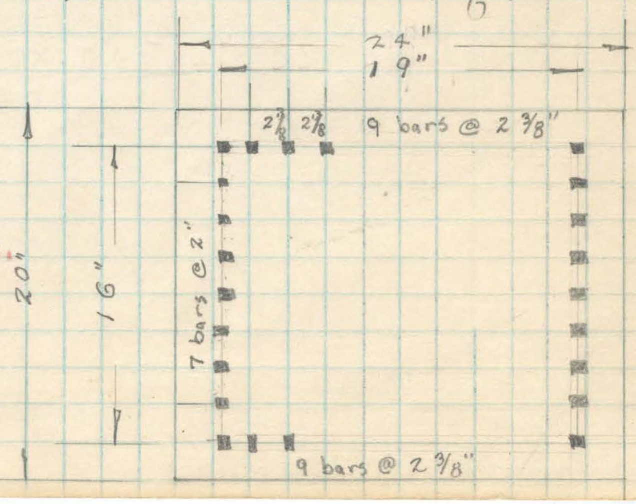


Fig. 7.

U_4U_5 — Stress due to Live, Wind, and Dead $= -398{,}000$ lb.

assume a section $= 24 \times 36 = 864$ sq. in.

wt. $\frac{864}{144} \times 150 = 900$ lb./ft.

Moment due to dead wt. — $M = \frac{1}{10} Wl = \frac{900 \times 10 \times 10 \times 12}{10} = 108{,}000$ in-lb.

$x_0 = \frac{108{,}000}{398{,}000} = .271$ in

$\frac{x_0}{t} = \frac{.271}{36} = .0075$.

Diagram. 13 Hool. — $K = .85$ for $p_0 = 1.5\%$

fc. — $f_c = \frac{WK}{bt} = \frac{398{,}000 \times .85}{24 \times 36} = 392$ #/sq. in.

As. — $a_s = 864 \times .015 = 13$ sq. in.

fs. — $f_s = 15 \times 392 = 5880$ #/sq. in.

Rods. — Use 24 - 3/4" □ rods.

length for Bond. — $50 \times \frac{3}{4} \times \frac{5880}{16000} = 14$ inches.

This spacing superceded

8 - 3/4" □ rods @ 2 5/8" o.c.

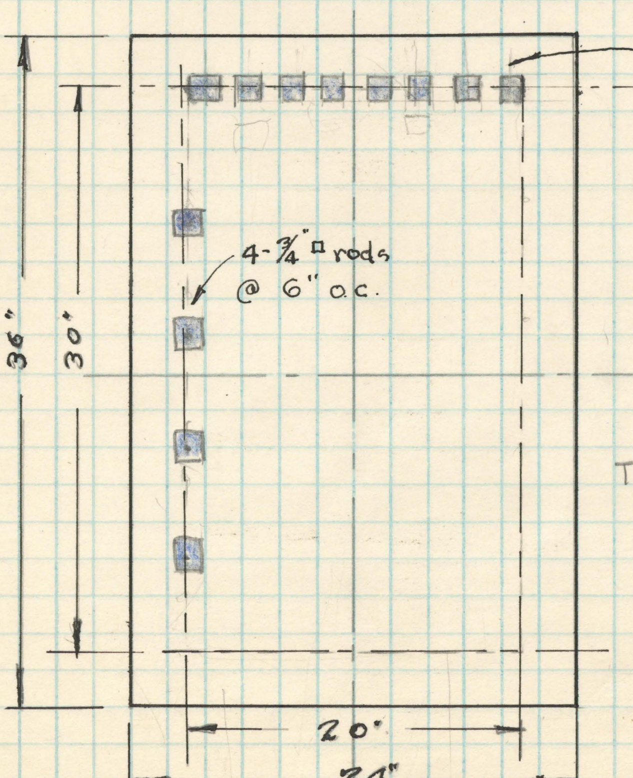


Fig. 8

Wt. of Member.

Concrete.

$\frac{36 \times 24 - 13.5}{144} \times 150 \times 10 = 8875$ lb.

steel.

$13.5 \times 10 \times 3.4 = 459$ lb.

Total wt $= 9334$ lb.

Spacing of Rods in U_4U_5.

Use 5 - 1 1/8" □ rods. = 6.32 sq. in.
4 - 1 1/4" □ rods = 6.25 sq. in.

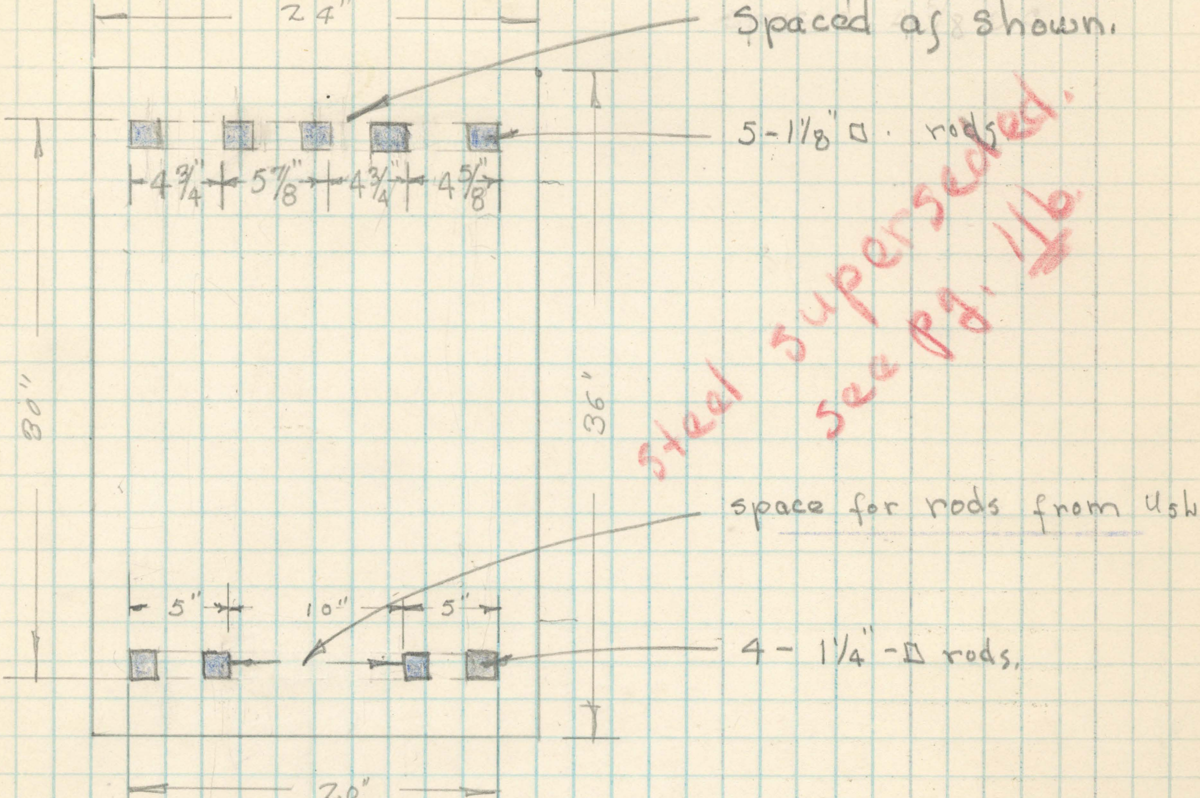


Fig. 9

steel superseded. see pg. 11b

stress in U_4U_5 acting as a Beam

$M = 108,000$ in. lbs.

Beam 20" = b. d = 29.56"

$a_s = 5 \times 1.266 = 6.328$ in.

$p = \frac{6.328}{20 \times 29.56} = .0107.$

Hool. Dia 2.

$j = .855$

Allowable M.

$M = a_s \times 16,000 \times jd =$

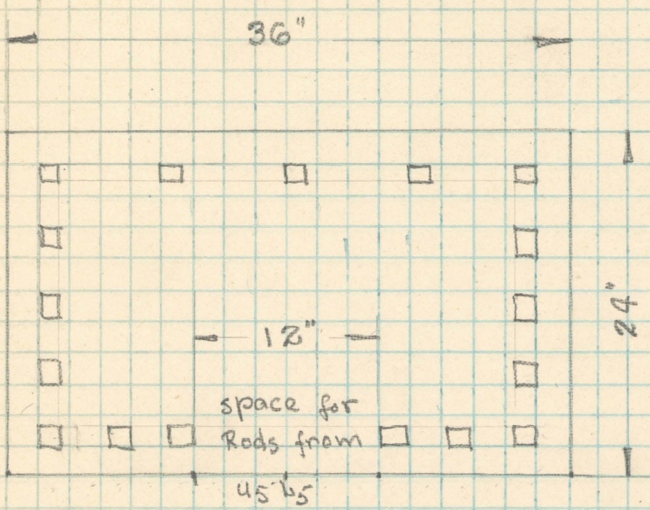
$M = 6.328 \times 16,000 \times .855 \times 29.56$

$M = 2,506,000$ in - lb.

This beam will stand stresses caused by Bending alone.

Total A_s = 13 sq. in.

use 17 - 7/8" ɸ bars.



U_5L_5.

Wind, Live, Dead Stress + 51,670 lb.

This being a tension Member steel alone will be used to take Tension, while enough Concrete will be used to give sufficient Bond.

$$\frac{51{,}670}{16{,}000} = 3.25 \text{ sq. in. of Steel.}$$

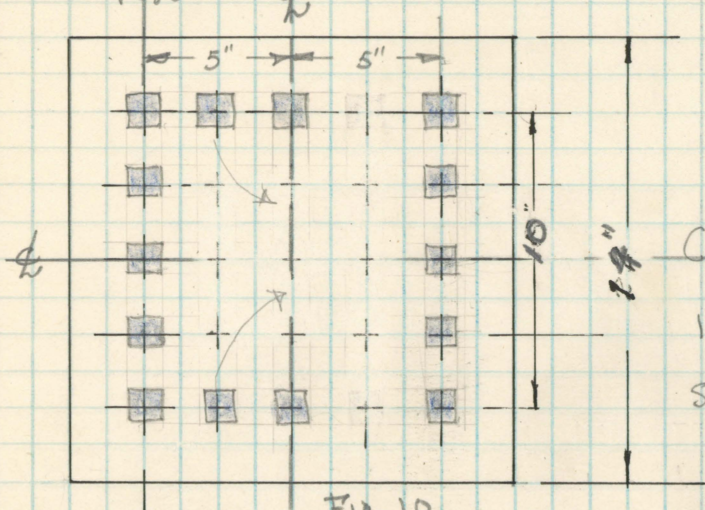


Fig. 10

Use 14 - 1/2" □ rods.

Wt. of Member.

Concrete. $\frac{144 - 35}{144} \times 150 = 113$ #/ft.

$12 \times 113 = 1360$ lb.

Steel. 12 ft. @ 12 #/ft = 144 lb.

Wt. of U_5L_5.

Total Wt = 1360 + 144 = 1500 lb.

L_4L_5.

Wind, Live, Dead stress = + 347,000 lbs

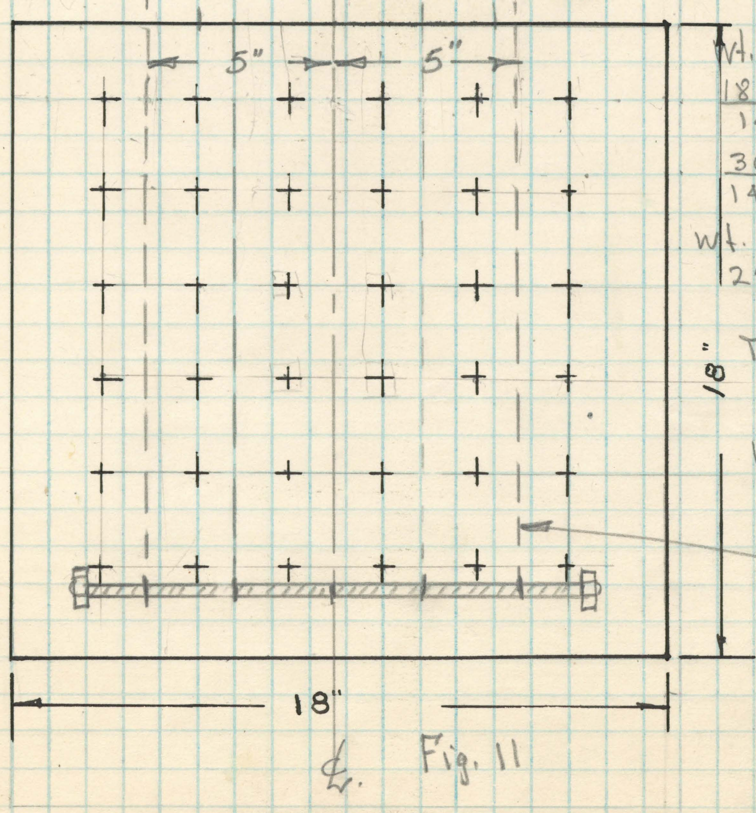
As.

$$\frac{347{,}000}{16{,}000} = 21.7 \text{ sq. in.}$$

use 34 - 3/4" □ rods. - 2 - 1" □ rods.

Area = 22.2 □"

spaced 3" o.c.



Bars from U_5L_5 Hooked on Bolt.

Fig. 11

Wt. of Concrete =

$\frac{18 \times 18 - 23.2}{144} = \frac{324 - 23.2}{144} \times 150$

$\frac{300}{144} \times 150 = 312$ #/ft.

wt. of Steel =

$23.2 \times 3.4 = 79$ #/ft.

Total wt. per ft = 391 #

Wt. of Member = 3910 #

Size of Bolt.

4-bars from U_5L_5 will be hooked to 1-bolt.
each bar is $\frac{1}{2}''\square$ – hence Load in each bar is

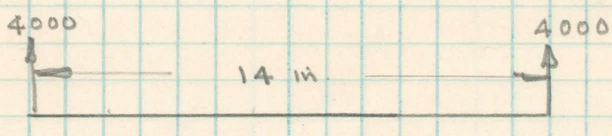
$$\frac{16{,}000}{4} = 4{,}000 \text{ lb.}$$

By Bearing on Concrete.

Total load carried by bolt is 16,000 lb.
useing $f_c = 550$, and Length of Bolt = 14 inches

$$\frac{16{,}000}{550 \times 14} = 2.08 \text{ in. Dia. of Bolt.}$$

By Bending Moment.



Max. Bending Moment will occur on Bolt with two Bars.

This is similar to a uniform load of 8000 # being distributed over the entire 14 in. of Bolt.
or $\frac{8000}{14} = 570$ lb./in.

$$M = 4000 \times 7 - 570 \times 7 \times \tfrac{7}{2} = 14{,}000 \text{ in-lb.}$$

$f_s = 20{,}000$ lb./sq. in. $\qquad S = \frac{Mc}{I}$

$$20{,}000 = \frac{14{,}000 \times \frac{D}{2}}{.0491\, D^4}$$

$$20{,}000 = \frac{14{,}000}{.098\, D^3}$$

$$D^3 = \frac{14{,}000}{20{,}000 \times .098} = \frac{.7}{.098}$$

$$D^3 = 7.15$$

$$D = 1.92$$

Use a 2" Bolt.

Wt. per ft. = 391 lb.

$$M = \frac{391 \times 10 \times 10 \times 12}{10} = 46,750 \text{ in-lb.}$$

Compute f_s and f_c similar to a Rectangular Beam. $b = 18''$ $d = 15.25$ in.

Moment. $M = 16,000\, a_s \times jd.$

$46,750 = 16,000\, a_s \times .87 \times 15.25$

As. $$a_s = \frac{46,750}{16,000 \times 15.25 \times .87}$$

$a_s = .224$ sq. in. This amount of Steel Can be taken care of By the excess steel in Member itself.

f_c computed as in a rectangular Beam.

$K = .1$ $\frac{Kd \times b}{2} f_c = a_s f_s.$

$$f_c = \frac{16,000 \times .224 \times 2}{.1 \times 15.25 \times \cancel{18}\,9} = \frac{3,600}{13.75} = 262 \text{ \#/sq. in.}$$

This stress is allowable. Also since there is Considerable steel in upper portion of Beam some of the Compressive action will be taken by this steel. Also since this steel is in tension, the compressive stress will not be prohibitive for the steel.

U_4L_5.

Live, Dead, and Wind Stress $- 30,900$ lb. Length. 14.87 ft.

Assume Section $12 \times 12 = 144$ sq. in.

$wt = \frac{144}{144} \times 150 = 150$ lb/ft. Total wt. $= 150 \times 14.87 = 2230$ lb.

$M = 1115 \times 5 - 1115 \times 2.5 = 1115 \times 2.5$ ft. lb.

$M = 1115 \times 2.5 \times 12 = 33,500$ in.-lb.

$X_0 = \frac{33,500}{30,900} = 1.12$ in. Superceded

$\frac{X_0}{t}$, $\frac{X_0}{t} = \frac{1.12}{12} = .93$ $p_o = .5\%$ Not Economical.

Diagram. 14 Hool. Dia. 15. $K = .31$ $a_s = p_o\, bt = .005 \times 144 = .72$ sq. in.

$L = .1$

$f_c = \frac{M}{Lbt^2} = \frac{33,500}{.1 \times 12 \times 144} = \frac{33,500}{144 \times 1.2} = 195^{\#}/\text{sq. in.}$

$f'_s = nf_c\left(1 - \frac{d'}{kt}\right) = 15 \times 195\left(1 - \frac{2}{.31 \times 12}\right) = 2920 \times .465 = 1360\ ^{\#}/\text{sq. in}$

$f_s = n \times f_c\left(\frac{d}{kt} - 1\right) = 2920 \times \left(\frac{10}{3.72} - 1\right) = 2920\,(3.1 - 1) = 2920 \times 2.1$

$f_s = 5900$ lb./sq. in.

U_4L_5. Live Dead and Wind Stress $= -30,900$ lb.

Assume Section $8'' \times 12'' = 96$ sq. in.

wt. $= \frac{96}{144} \times 150 = 100$ lb./ft. Total wt. $= 100 \times 14.87 = 1487$ lb for Concr.

$M = 743 \times 5 - 743 \times 2.5 = 1860$ ft. lb $= 1860 \times 12 = 22,320$ in-lb.

$X_0 = \frac{22,320}{30,900} = .723$ in. $\frac{X_0}{t} = .0602$ use $p_o = .5\%$.

$K = 1.25$ $f_c = \frac{W}{bt}K = \frac{30,900}{8 \times 12} \times 1.25 = 403$ lb/sq. in.

$a_s = .5 \times 8 \times 12 = .48$ sq. in.

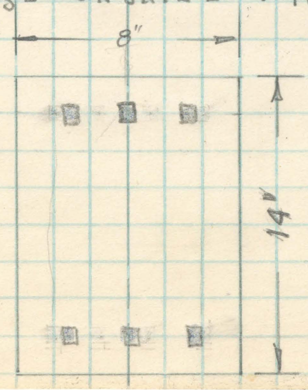


Fig. 12

use 6 - 3/8" □ rods.

wt. of steel $= 1.7 \times 10 = 17$ lb.

Total wt $= 1504$ lb

U_4L_4

Live, Dead, Wind Stress = $-6,260$ lb.

section 8" x 8". wt. of Member. $\frac{64}{144} \times 150 \times 11 = \underline{667^{\#}}$.

use steel - 4 1/4" □ - rods.

In this member, it would be possible for the concrete alone to take the entire Load, but due to the fact of eccentric Loading having effect, the above steel will be used.

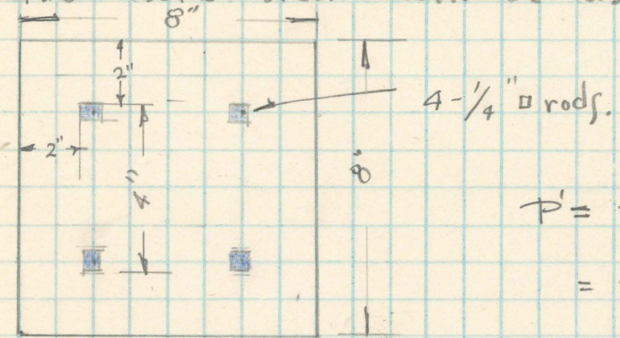


Fig. 13

$$P' = P(1.6L/25D)$$

$$= 550\left(\frac{1.6 \times 11 \times 12}{200}\right) = 550 \times .967$$

$P' = 533^{\#}$/sq. in allowable unit stress.

Specif. Pg. 77.

$$f_c = \frac{6260}{16} = 391^{\#}/\text{sq. in.}$$

Wt. of Members in Panel U_4U_5. As Designed.

pg. 9

Wt.	of	U_4U_5 =	9334 lb.
"	"	U_5L_5 =	1500
"	"	L_4L_5 =	3910
"	"	U_4L_5 =	1504
"	"	U_4L_4 =	667
			16,915 lb.

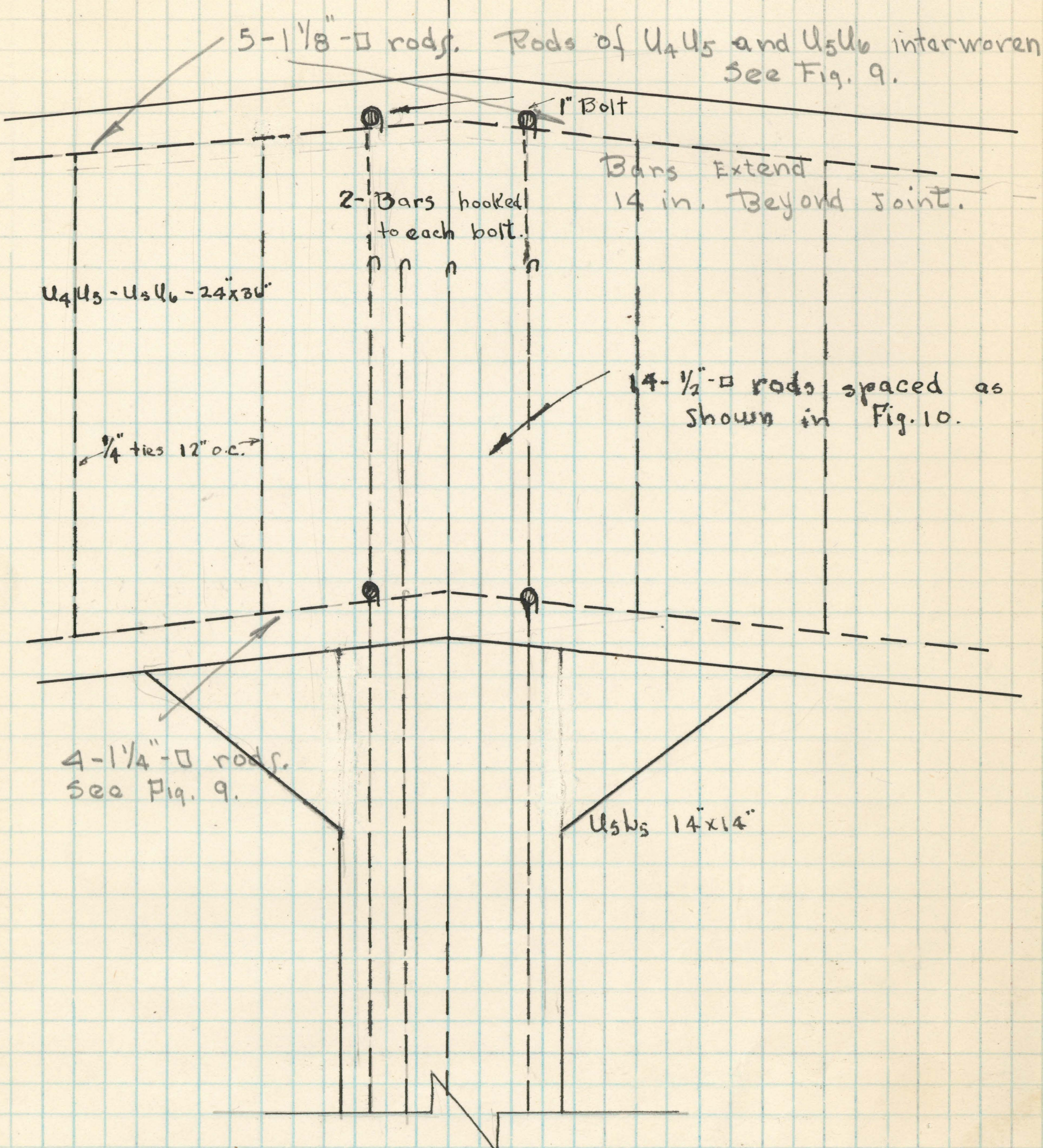


Fig. 14.

As this Butt Plate carries No Stress to Chords from U_5W_5, it may not be considered, but it will be made by sloping out concrete on 45° Angle at a point 18" up U_5W_5.

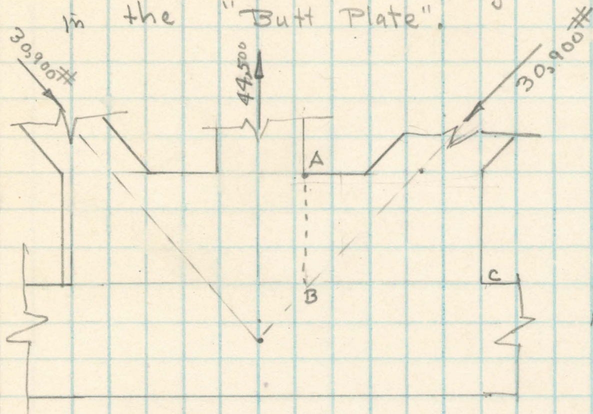
Steel.

4 - ⅜" □ stirrups Vertical.
8 - ⅜" □ bars horizontal.

Joint L_5.

concrete at junction of U_4L_5, L_5U_5, and L_4L_5, should be designed to act as a Butt Plate. i.e. to carry thru the stress from U_4L_5 to Bottom chord. The amount of Stress to be carried, will be taken as equal to the amount carried by U_4L_5.

This stress causing both Bending Moment and shear in the "Butt Plate".



Shear across section AB will equal vertical Comp. of 30,900 or
$30,900 \times .7 = 21,600$ lb.

Assume section AB 14"x18"

BC =

Shear lb./□"

$\frac{21,600}{18 \times 14} = 86$ #/□" shear on section AB. $18" + 8.5 = 26.5"$

amount tobe taken by stirrups = $46 \times 14 \times 18 = 11,610$ lb.

$\frac{11,610}{16,00} = a_s = .726$ sq. in.

use 3 - 3/8" □ stirrups.

For Moment consider Butt Plate as Beam, with Loads of 30,900 acting where shown.

$M = 21,600 \times 18 = 390,000$ in.-lb.

$\frac{M}{bd^2} = \frac{390,000}{14 \times 16 \times 16} = 109.$ $\quad p = .008$

$a_s = 14 \times 16 \times .008 = 1.79$ sq. in. use 3 - 3/4" □ rods.

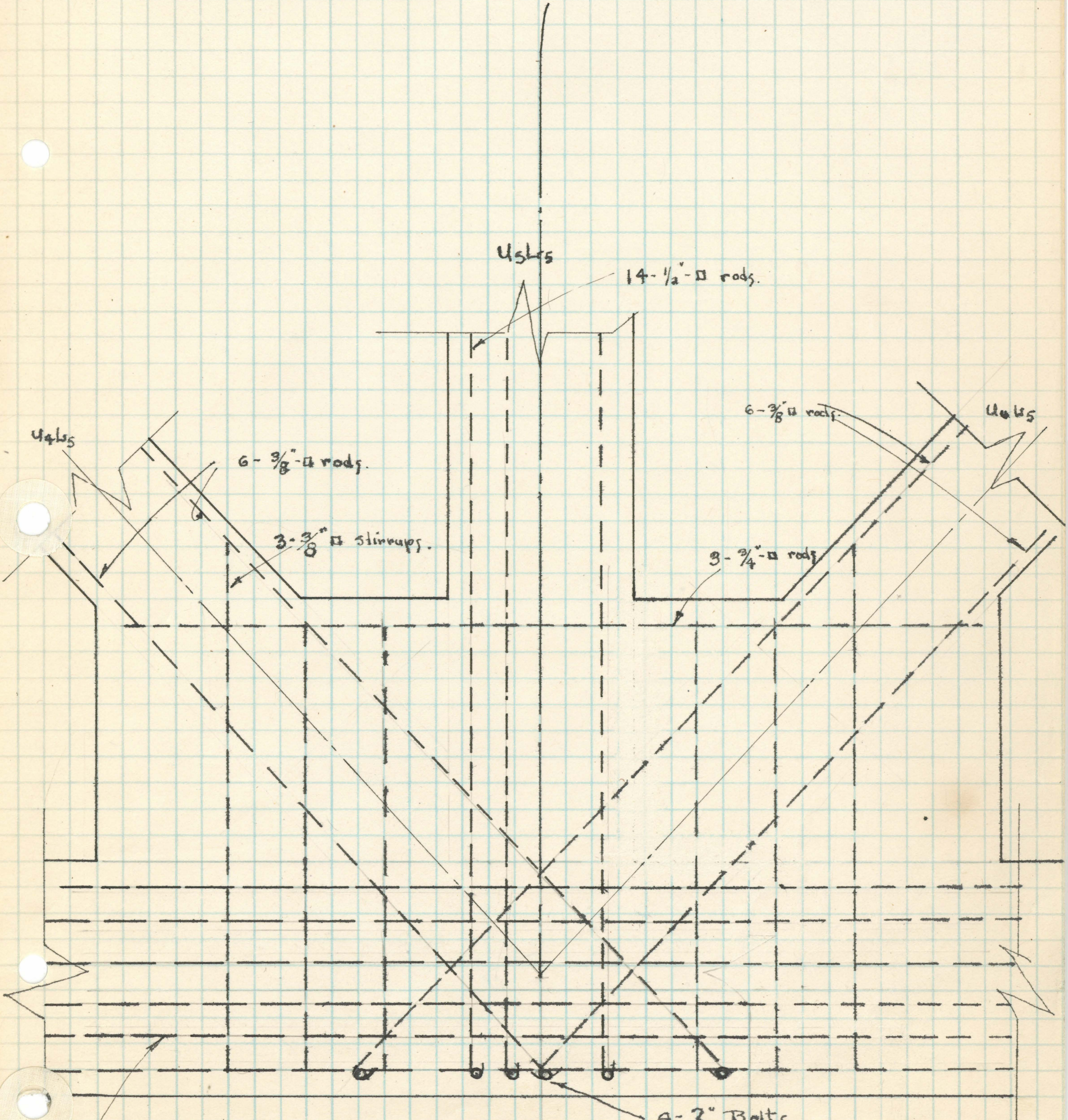


Fig. 15.

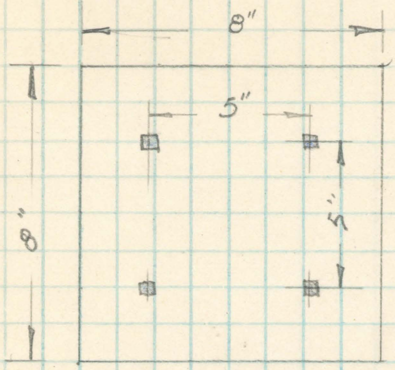
34-3/4"-□ rods
4-1"-□ rods
See Fig. 11.

This steel will also be carried thru L3L4 and 36" Beyond L3

$U_3 L_4$.

Stress = + 18,300 lb.

$\frac{18,300}{16,000}$ = 1.14 sq. in. = a_s. Use - 4 - 5/8" - □ - rods = 1.56 sq. in.



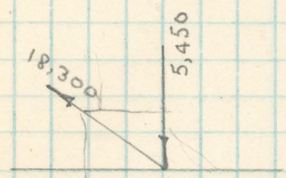
$14.14 \times \frac{8 \times 8}{144} \times 150$ = 942 lb. wt. of Concrete.

16 X 5.31 = 85 lb. wt. of steel.

Total Weight = 1027 lb.

L_4.

Butt Plate Concrete.



This Butt Plate will be made 8" thick the same as $U_3 L_4$ and $U_4 L_4$

18,300 x .7 = 12,800 lb. shear.

Make plate 8" X 18".

Shear #/□"

$\frac{12,800}{8 \times 18}$ = .89#/sq. in. $a_s = \frac{49 \times 8 \times 18}{16,000}$ = .45 sq. in.

Use 2 - 3/8" □ stirrups.

For Moment consider as a Cantilever Beam.

M = 12,800 x 18 = 216,000 in-lbs.

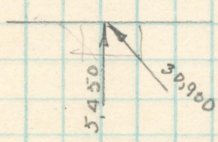
$\frac{M}{bd^2} = \frac{216,000}{8 \times 16 \times 16}$ = 105 p = .0077 as = 8 x 16 x .0077 = .985

Use - 1 - 5/8" □ - 1 - 3/4" □ rods

Steel Placed in Bottom of Plate.

U_4

"Butt Plate" Concrete.



[8" X 18"]

Total Maximum Shear = 5,450 + 30,900 x .7 = 27,000

Shear.

$\frac{27,000}{120}$ = 225 sq.-in. Make plate 12" x 18"

Stress taken by Stirrups = 216 X 80 = 17,300 lb

$\frac{17,300}{1600}$ = 1.08 5 - 3/8" □ Stirrups.

For M use 6 - 7/8" □ rods placed 2" from Top.

10 5/8" □ rods Horizontal thru out.

Diagram of Joint L_4.

Butt Plate Horizontal Steel.

$$\frac{18{,}300 \times .7}{16{,}000} = .8 \text{ sq. in.}$$

Use 6 - 3/8" □ rods.

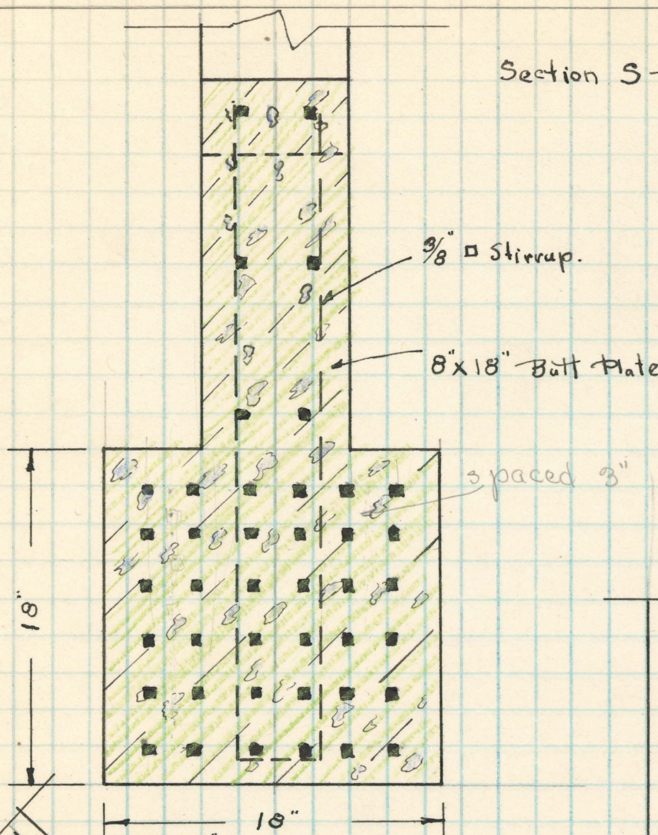


Fig. 16a.

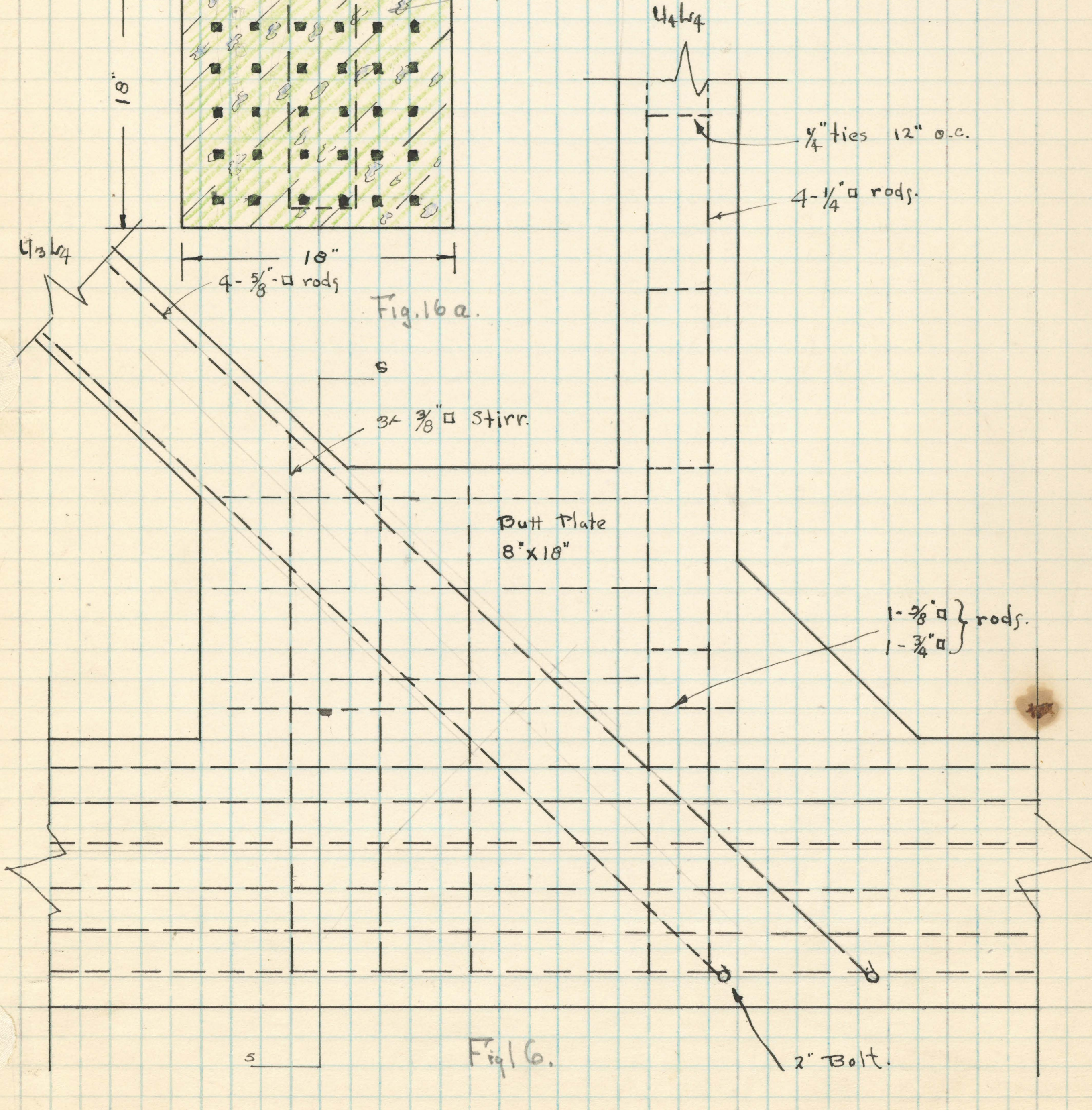


Fig. 16.

Total Stress to transmitt = - 41,730 lb.

Allowable f_c.

$P' = p(1.6 - \frac{L}{25D})$ $L = 120"$

$P' = 550(1.6 - \frac{120}{25 \times 7}) = 550(1.6 - .69) = 550 \times .91$

$P' = 500$ lb./sq. in.

With Section 10" x 10". Eff. Dia. = 7"

$P = 500 \times 7 \times 7 = 24,500$ lb.

Hool Pg. 169

$\frac{P}{P_1} = 1 + (n-1)p$

$\frac{41.7}{24.5} = 1 + 14p$

$1.7 = 1 + 14p$

$p = \frac{.7}{14} = .05$ or 5%. This is too high.

$A_s = 49 \times .05 = 2.45$ sq. in.

Use 6 - 3/8" □ bars. 3/16" - ties every 7"

Weight of Member.

Wt. of Mem.

Concrete = $\frac{10 \times 10}{144} \times 10 \times 150 = 1040$. lb.

Steel = $7.9 \times 10 = 79$

1119 lb.

Weight = 1120 lb.

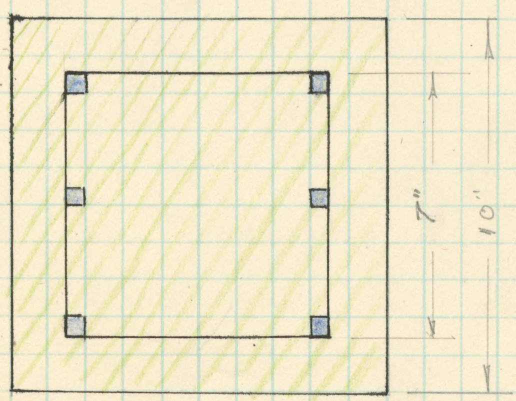


Fig. 17.
Member L_3U_4
6 - 3/8" □ bars
3/16" - ties every 7"

Steel

Total Stress = + 280,000 lb.

$\frac{280,000}{16,000}$ = 17.5 sq. in. of Steel.

Use 12 - $1\frac{1}{4}$" ▯ rods = 18.1 sq.-in.

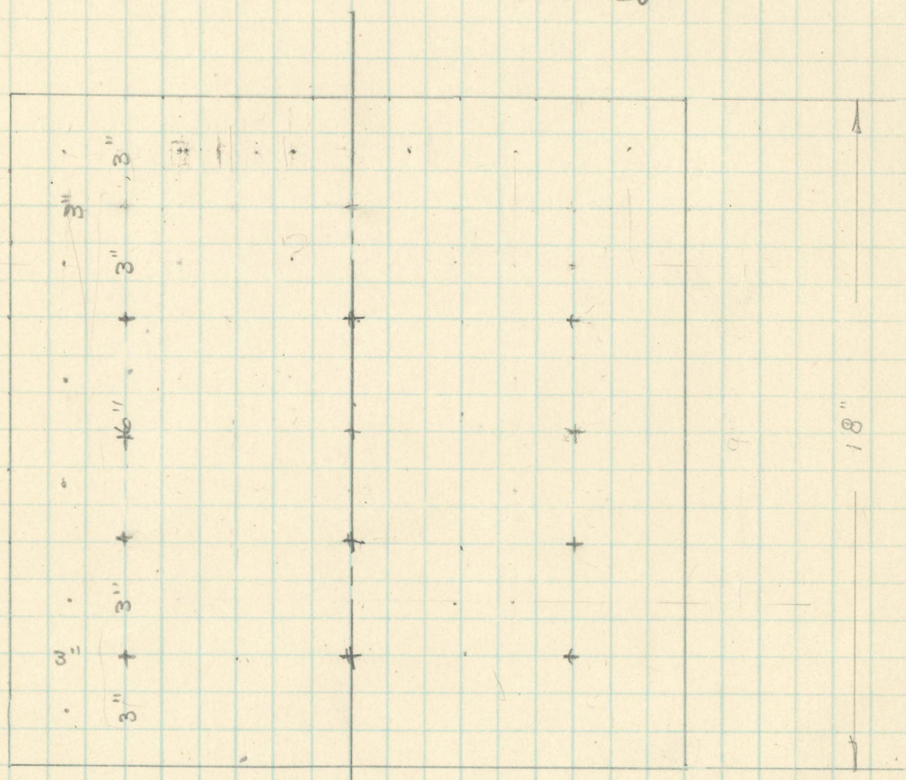


Fig. 18

These bars are arranged as above so as to fit into the bars of Member L_3L_4 and still not interfere with the bars coming into joint L_3 from the Web Members U_3L_3 and U_2L_3.

Wt. of Conc. = $\frac{18 \times 18}{144} \times 10 \times 150 =$ 3340

" of Steel = 63.75×10 = 637.5

3,977

This Member is 18"x18" - 40" past joint L_3. From here on it is 15"x15" square.

Member U_2L_3

Total Stress = + 73,500 lb.

$\frac{73,500}{16,000}$ = 4.58 sq. of steel.

use 6 - 7/8"-□ rods. = 4.59 sq-in.

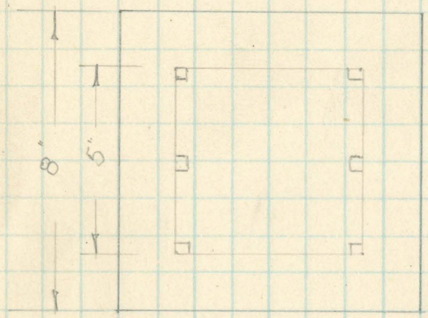


Fig. 19.

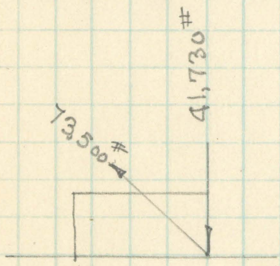
wt. of Concrete = $\frac{8 \times 8}{144} \times 13.45 \times 150$ = 895 lb

wt. of Steel = 18.22×13.45 = 245 lb

Bars Hooked on 1½" Bolts.

Joint L_3

Butt Plate.



For Shear. VP = 24,500 [pg 21]

$\frac{24,500}{120}$ = 204 sq. in. $\frac{204}{12}$ = 17 inches.

For Shear.

make Butt Plate 12"x18" = 216 sq. in.

Stress taken by steel = 80 X 216 = 17280

As.

$\frac{17,280}{16,000}$ = 1.08 sq. in.

Stirrups.

Use 5 - 3/8" □ Stirrups.

Moment.

Moment = 24,500 x 18 = 440,000 in-lb

d = 16" b = 12 $\frac{M}{bd^2} = \frac{440,000}{12 \times 16^2}$ = 143

p = .002

As = 12 X 18 X .002 = 4.3 sq. in

Steel

6 - 7/8" □ rods. Placed 2" from Top of Plate.

11 - 5/8"-□ rods Distributed uniformly in Vertical Direction.

Steel for Horizontal Tension

$H.C = 73{,}500 \times .7 = 51{,}500$ lb.

$\frac{51{,}500}{16{,}000} = 3.22$ sq. in $= A_s$ Use 14 ½"-□ bars.

The reason so many bars are used is to allow the bars to be distributed thruout the Butt Plate.

Wt. of Butt Plate $= \frac{12 \times 36}{144} \times 3 \times 150 = 1040$ lb. Conc.
steel. $6 \times 4.78 + 3 \times 11.9 = 64$
1104 lb.

Butt Plate ~~12"x36"~~ 12x18"

For Changes See [Page 23]

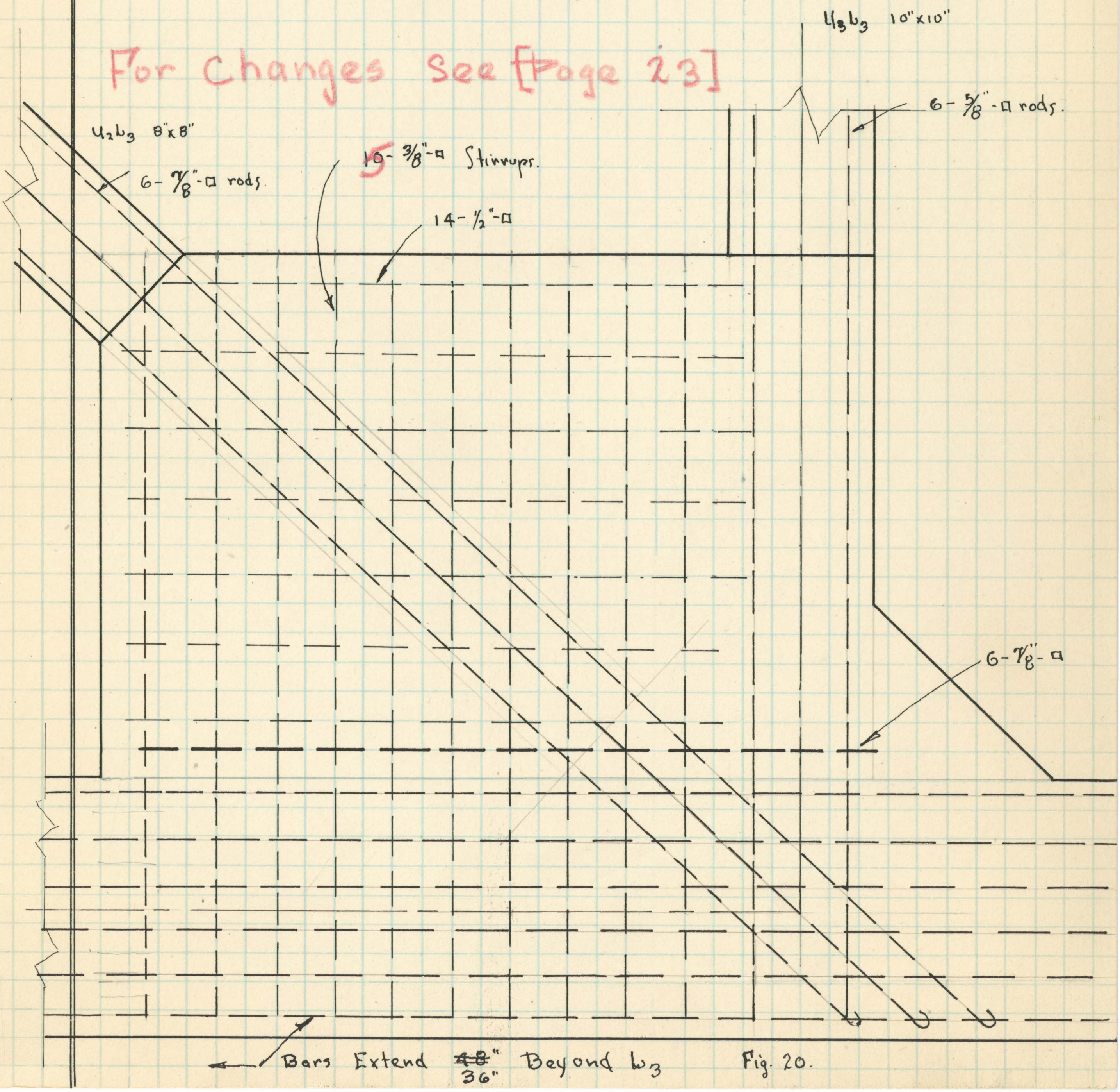


Fig. 20.

U_2L_2

Stress = −76,200 lb

Section 12"x12" Eff. Dia. = 9"

$f_c = 550$ #/sq.-in.

$P = 9 \times 9 \times 550 = 44,500$ lb.

$\frac{P}{P'} = \frac{76,200}{44,500} = 1.72 = 1 + 14p$

$p = \frac{.72}{14} = .05$

$A_s = 81 \times .05 = 4.05$ sq. in.

use − 6 − 7/8" − □ bars. = 4.5 sq. in. 3/16" − ties − 10" c.-c.

U_2L_2 12"x12"

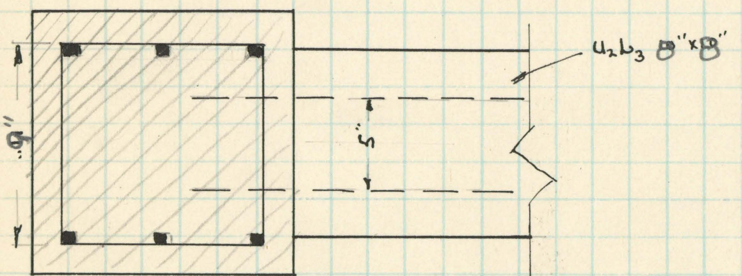


Fig. 21.

Wt. of Concrete = $\frac{144}{144} \times 9 \times 150$

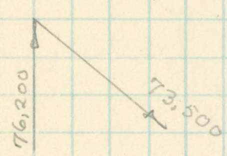
" " " = 1350

Wt. of Steel = $9 \times 15.62 = 140.58$

Total Wt. = 1490 lb.

U_2 Butt Plate Concrete. Joint U_2.

Shear.



$P = 44,500$ [Pg. 25]

$\frac{44,500}{120} = 370$"

Size 12"x30" Butt Plate.

$\frac{44,500}{16,000} = 2.77$ sq in

Steel. 10 3/8" □ Stirrups. Vertical.

3.2 sq. in for Horizontal Steel.

use 13 − 1/2" □ Bars. 1/2 of thes number placed on each side of Butt Plate.

$M = 44.500 \times 28 = 1,250,000$

$\frac{M}{bd^2} = 133$ $p = 1.5\%$ $A_s = 12 \times 28 \times .015 = 5.1$ in. of Steel.

5-1" □ Bars. d = 28"

Design of $L_1 L_2$.

L_1L_2 Stress.

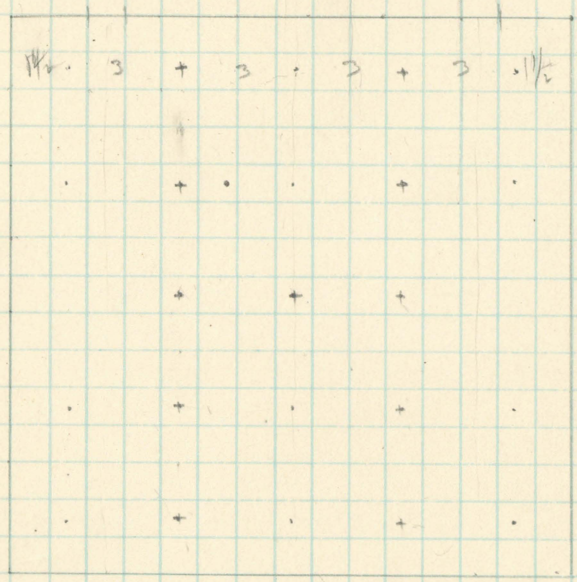
Total Stress = +177,000 lb.

$\frac{177,000}{16,000}$ = 11.05 sq.-in.

Area of Steel.

use 11 - 1" ⌀ rods = 11 sq.-in.

Bar Arrangement.



L_1L_2 is 15"x15" - 48" past joint L_2. then 12"x15" from there to the Reaction.

U_1L_1

Stress = + 7,500 lb.

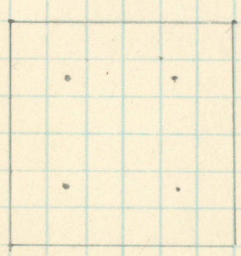
$\frac{7,500}{16,000}$ = .467 sq.-in.

Steel.

use 4 - 3/8" ▫ rods. Area = .562 sq.-in.

6"x6". Rods. 3" O.C.

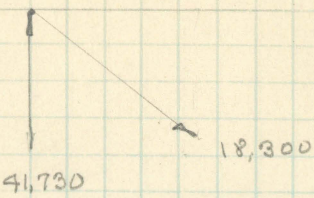
Hooked on 1" Bolts at each end.



Joint U_3

Shear in Butt Plate will be equal to P in U_3L_3

P = 24,500 lb. [Pg. 21]



Shear

$\frac{24,500}{120} = 204$ sq. in.

$\frac{204}{12} = 17''$. Make Plate 12x18".

Tension.

stress taken by Stirrups = 80x216 = 17,280

$\frac{17,280}{16000} = 1.08$ sq. in. Use 5 - 3/8" □ Stirrups.

Use 8 - 3/8" φ bars Horizontal Steel.

Moment.

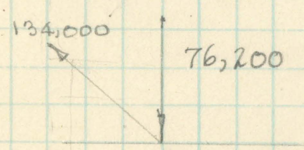
d = 16"

24,500x18 = 442,000 $\frac{M}{bd^2} = \frac{442,000}{12 \times 16^2} = 144$

p = .02 A_s = 12x16x.02 = 4.3 sq. in

6 - 7/8" □ rods Placed 2" from top.
10 - 5/8" □ rods Horizontal througout Plate.

Joint L_2.



U_1L_2 is stressed to 134,000 in Tension this is carried by steel bars in U_1L_2 hence Delivered by the bars and not the butt Plate.

The Load Carried by Butt Plate concrete is Stress in Concrete of U_2L_2.

fc = 550. P = 550x9x9 = 44,500 [Pg. 25]

Shear.

$\frac{44500}{120} = 370$ sq. in. $\frac{370}{15} = 25''$

Make Butt Plate 15"x25". Use 10 - 1/2" φ bars Horiz. steel.

Steel.

370x80 = 29,600 lb. $\frac{29,000}{16000} = 1.85$ sq. in.

Use 7 - 3/8" □ Stirrupes. Vertical Steel.

Moment

44,500x25" = 1,120,000 in. lbs. $\frac{M}{bd^2} = 140$

p = .017. A_s = 15x23x.017 = 5.82 sq. in.
use - 6 - 1" □ bars.

U_1U_2

Stress = − 296,000 lb.

Assume a section 24" x 24"

$$Wt = \frac{24 \times 24}{144} \times 10.04 \times 150 = 6025 \text{ lb}$$

Moment Due to Dead wt.

$$M = \frac{6025 \times 10.4 \times 12}{8} = 90,800 \text{ in.-lb.}$$

$$x_0 = \frac{90,8000}{296,000} = .307 \text{ inches.}$$

$$\frac{x_0}{t} = \frac{.307}{24} = .0128$$

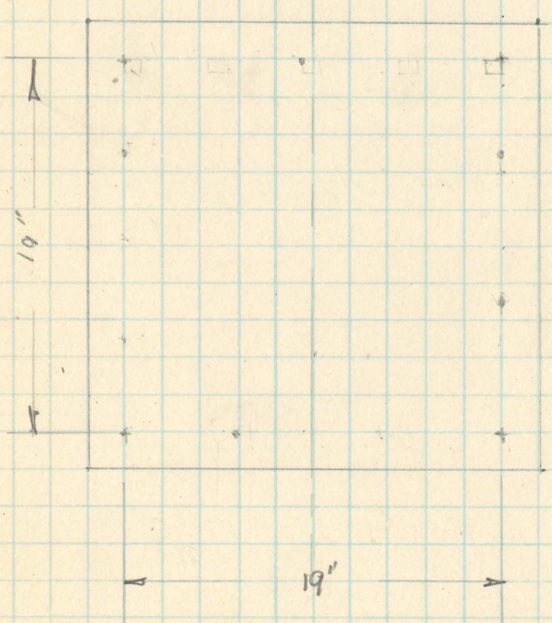
Dia. 13 Hool.

$K = .87$ $p = .015$ or 1.5%.

$$f_c = \frac{WK}{bt} = \frac{296,000 \times .85}{24 \times 24} = 437 \text{ lb./sq. in.}$$

$a_s = 24 \times 24 \times .015 = 8.6$ sq. in.

use 9 - 1" □ rods.



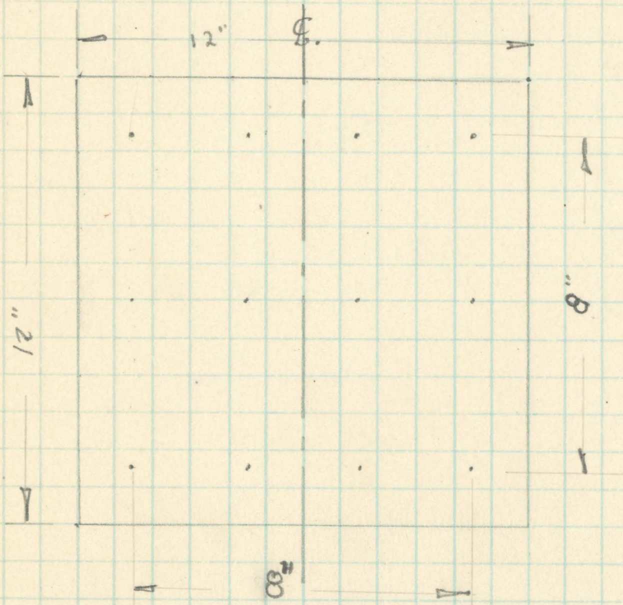
This Member is 24" x 36" for 3 feet past joint U_2, then it is 24" x 24" from there to the Reaction.

U_1L_2

Tension Member.

Stress. Stress = + 134,000 lb $\quad A_s = \frac{134,000}{16,000} = 8.35$ sq. in.

use - 11 - 7/8" □ Bars.



U_1L_0

Stress = - 231,000 lb.

Assume section 18 x 18" $\quad$ wt. = $\frac{18 \times 18}{144}$ 12.8 x 150 = 4330 lb

$M = \frac{1}{8} \times 4330 \times 10 \times 12 = 66,500$ in-lbs.

$x_0 = \frac{66,500}{231,000} = .288$

$\frac{x_0}{t} = \frac{.288}{18}$ .0155 $\quad$ K = .57 $\quad$ p = .05

$f_c = \frac{WK}{bt} = \frac{231,000 \times .57}{18 \times 18} = 405$ lb/sq. in.

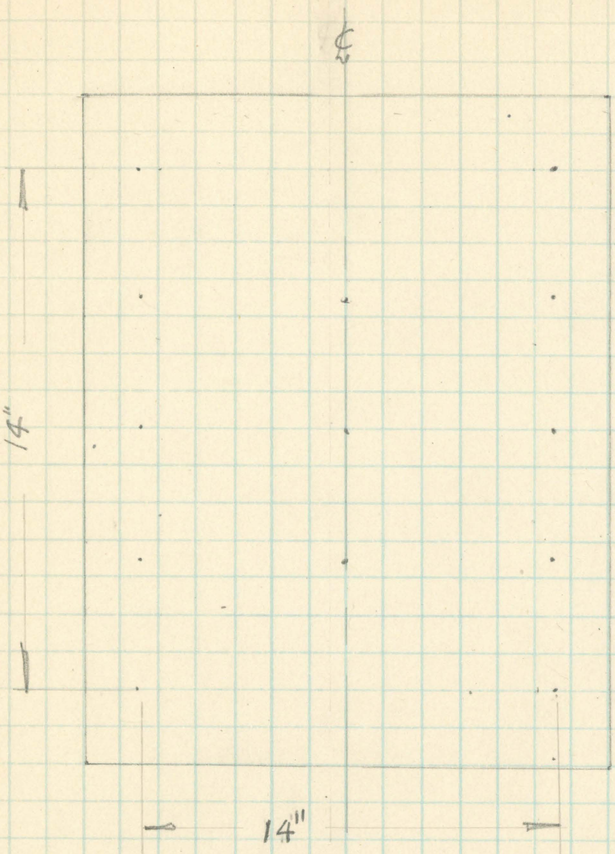
$A_s = 18 \times 18 \times .05 = 16.2$ sq in

use 13 - 1 1/8" □ rods.

P = 14 x 14 x 550 = 108,000 lb.

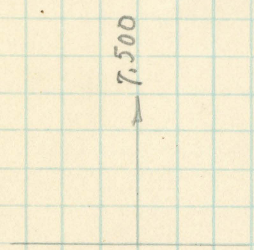
U_1L_0 18" x 18".

13 - 1 1/8" □ rods [see Pg. 29]



This steel is to extend Down into column, within hooping, for 48".

Joint L_1



Joint L_1 Butt Plate.

Rods in U_1L_1 transmitt all stress in U_1L_1.

∴ there must be sufficient concrete to take bond.

$u = 120\#/\text{sq. in.}$

each Bar needs enough concrete to transmitt its stress.

each Bar transmitts $\frac{7.500}{4} = 1875$ lb.

$\frac{3}{8} \times 4 \times 120 . L = 1875$

$L = \frac{1875}{\frac{3}{2} \times 120} = 10.4$ inches or 11 inches.

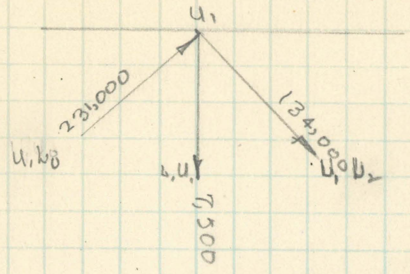
This Distance is supplied by Bottom Chord Member.

No Butt Plate will be used.

Joint U_1

Left of U_1

[Pg. 29]



P of concrete in L_0U_1 = 108,000 lb.

Stress Transmitted by steel in L_0U_1 =

15 × 550 × 1.266 × 16 = 167,000 lb.

$\frac{108,000 \times .7}{120}$ = 630 sq. in. cross Section of Butt Plate.

make Plate 24" × 26" = 625 sq. in.

$\frac{625 \times 80}{16,000}$ = 3.12 sq. in of Stirrup Steel.

use 11 - 3/8" ▫ Stirrups to left of Joint U_1

22 - 3/8" ▫ Bars Horizontal steel.

Right of U_1.

U_1U_2 transmits its stress by steel Bars hence Butt Plate must aid in providing concrete for Bond.

11 × 7/8 × 4 × 120 L = 134,000

$L = \frac{134,000}{3.5 \times 120 \times 11} = 29"$.

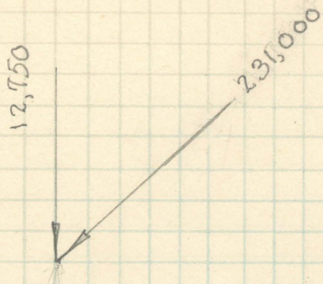
This Butt Plate will Provide sufficient Length for Bond.

Moment

134,000 × .7 × 50" = 4,600,000 in-lb.

$\frac{M}{bd^2} = \frac{4,600,000}{18 \times 16 \times 16} =$

Design of Butt Plate at L_0



Member U_1L_0 has 13 - 1 1/8" □ bars which must transmitt their stress to the bottom chord by means of Butt Plate. The Length of Butt Plate must be such as to provide sufficient surface for bars to transmitt bond stress.

fe.

$f_c = 405$ in. L_0U_1 [Pg. 29] $A_s = 16.45$ sq. in.

$f_s = 405 \times 15 = 6100$ lb.

stress carried by rods $= 6100 \times 16.45 = 100{,}000$ lb.

This 100,000 lb. must be distributed thru Butt Pate.

Shear.

$.7 \times \frac{100{,}000}{120} = 580$ sq. in. cross section of Butt Plate.

Make plate <u>18" x 32"</u>

Bond.

$\frac{100{,}000}{80 \times 13[4 \times 1.125]} = \frac{100{,}000}{60{,}80} = 16.5$ in of Length

This length will be supplied by Butt Plate.

Vertical Steel.

$\frac{100{,}000 \times .7}{16{,}000} = 4.38$ sq. in.

<u>use 9 - 1/2" □ stirrupes.</u>

Horizontal steel.

$\frac{100{,}000 \times .7}{16{,}000} = 4.38$ sq. in.

<u>use 16 - 3/8" □ rods.</u>

Moment.

$231{,}000 \times 32 = 7{,}380{,}000$ in-lbs.

$A_s = \frac{7{,}380{,}000}{16{,}000 \times .87 \times 30} = 17.6$ sq. in. $p = \frac{17.6}{18 \times 30} = .033$

$f_c = \frac{7{,}380{,}000}{.87 \times .615 \times 18 \times 30 \times 30} = \underline{510}$ #/sq. in

Use 14 - 1 1/8" □ Bars in <u>two</u> <u>layers</u>.

Mem.	Refer. Pg.	Size	Steel.	Weight.		Total Weight.	
				Concrete	Steel		
U_5L_5	12	14x14	14-½"□	1360	144	1500	
U_4U_5	11,12	24x36	5-1⅛"□, 4-1¼"□	8875	427	9302	
L_4L_5	12	18x18	34-¾"□, 2-1"□	3120	790	3910	
U_4L_5	14	8"x14"	6-⅜"□	1000	43	1043	
U_4L_4	15	8"x8"	4-¼"□	730	9	740	
U_3L_4	19	8"x8"	4-⅝"□	940	85	1025	
L_3L_4	Same as L_4L_5			3120	790	3910	
U_3U_4	Same as U_4U_5			8875	427	9302	
L_3U_3	21	10"x10"	6-⅝"□	1040	79	1200	
L_2L_3	22	18"x18"	12-1¼"□	2705	900	3605	
U_2L_3	23	8"x8"	6-⅞"□	895	245	1140	
U_2L_2	25	12"x12"	6-⅞"□	1350	140	1490	
L_1L_2	26	15"x15"	11-1"□	2100	630	2730	
L_0L_1	Same as L_1L_2			1870	374	2245	
U_1L_1	26	6"x6"	4-⅜"□	300	16	316	
U_2U_3	Same as U_4U_5			8875	427	9302	
U_1U_2	28	24"x24"	9-1"□	6000	306	6310	
U_1U_0	Same as U_1U_2			6000	306	6310	
U_1L_2	29	12"x12"	11-⅞"□	1920	366	2286	
U_1L_0	29	18"x18"	13-1⅛"□	4330	715	5045	

steel

144.00 -
427.00 -
790.00 -
43.00 -
9.00 -
85.00 -
790.00 -
427.00 -
79.00 -
900.00 -
245.00 -
140.00 -
630.00 -
374.00 -
16.00 -
427.00 -
306.00 -
306.00 -
366.00 -
715.00 -

7.945.11 S

1,360.00 -
8,875.00 -
3,120.00 -
1,000.00 -
730.00 -
940.00 -
3,120.00 -
8,875.00 -
1,040.00 -
2,705.00 -
895.00 -
1,350.00 -
2,100.00 -
1,870.00 -
300.00 -
8,875.00 -
6,000.00 -
6,000.00 -
1,920.00 -
4,330.00 -

73,350.11 S

Total Wt.

93
31
62

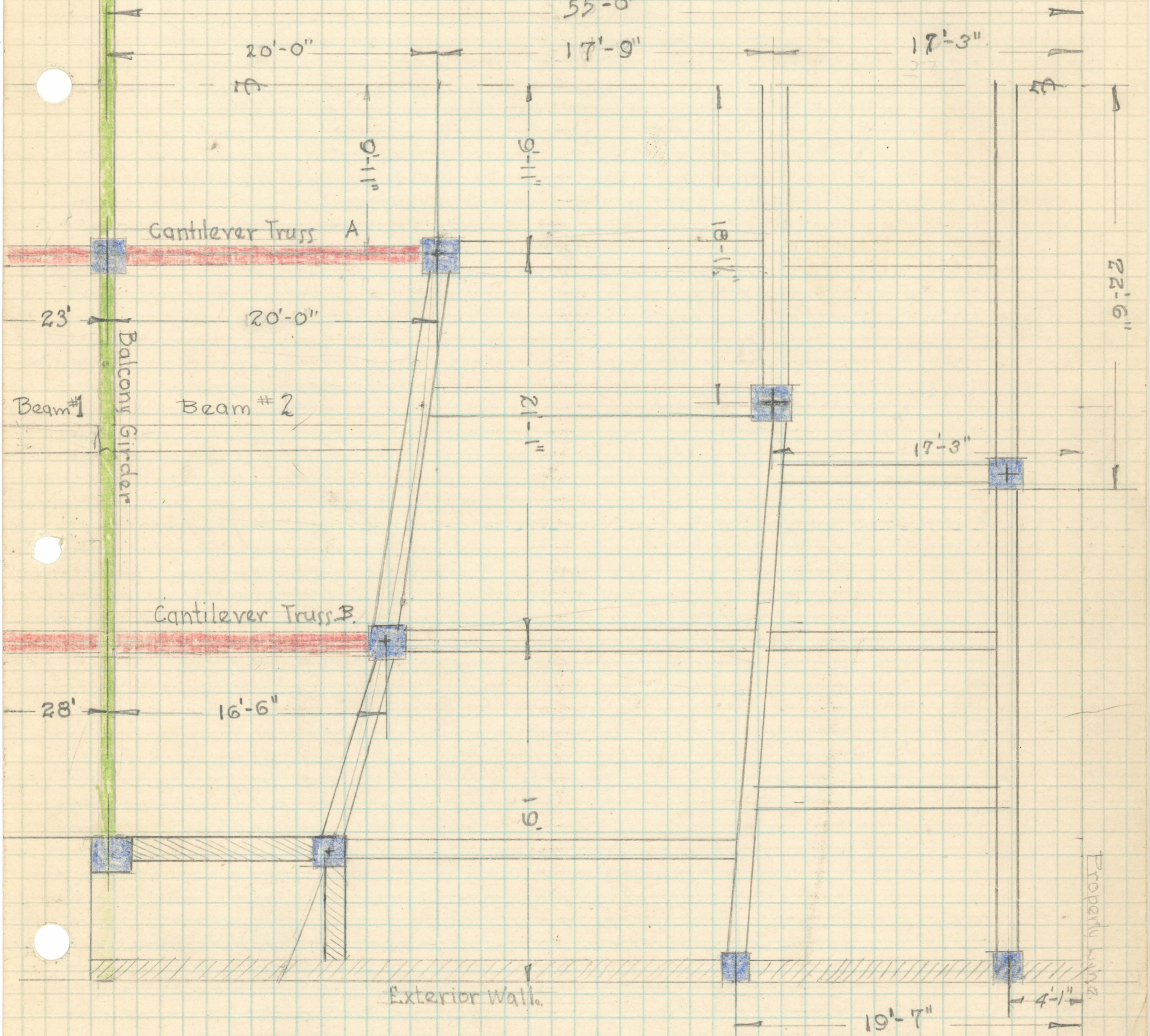
1~~4~~5.00
93.02
39.10
10.43
7.40
10.25
39.10
93.02
12.00
36.05
11.40
14.90
27.30
22.45
3.16
93.02
63.10
63.10
22.86
50.45

72~~6~~7.11 s

72,711

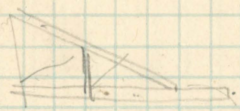
Cantilever, Girder, and Beam Supports for.

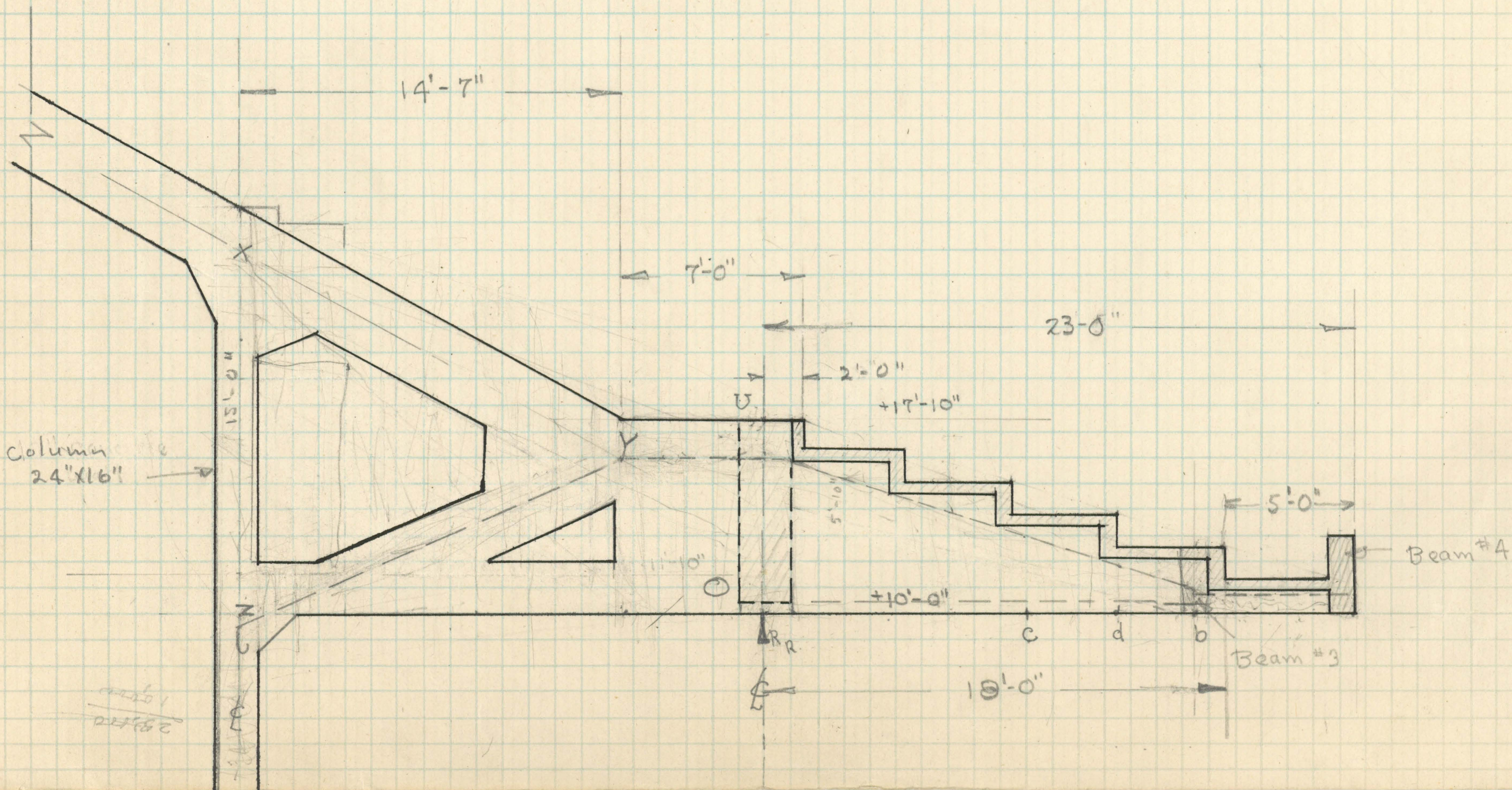
Balcony.



General Plan of Cantilever "A"

12' - 20'-1" - 20'-1" - 20'-1" 12'-0"

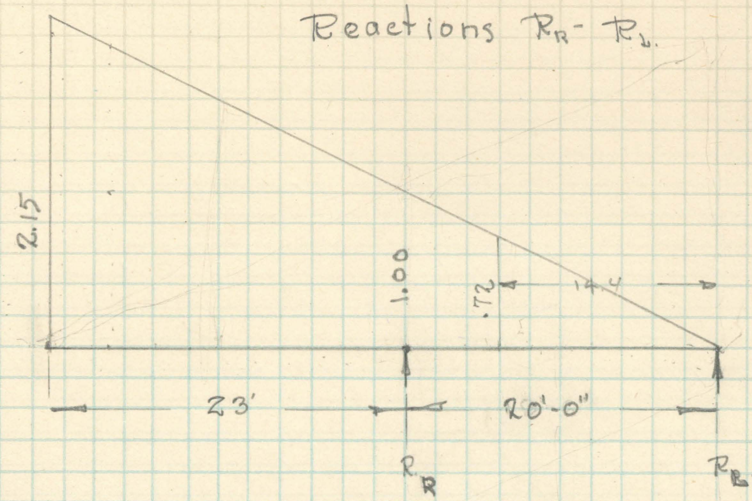




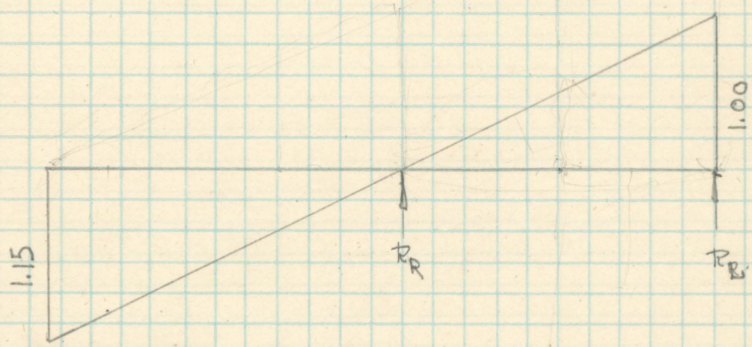
Influence Lines for Cantilever Truss A.

Reactions R_R - R_L.

R_R

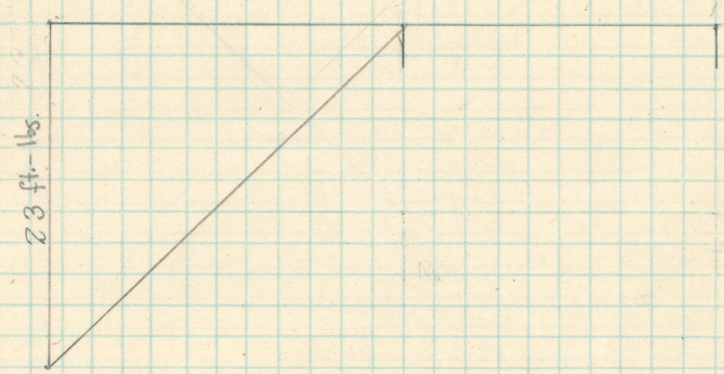


R_L

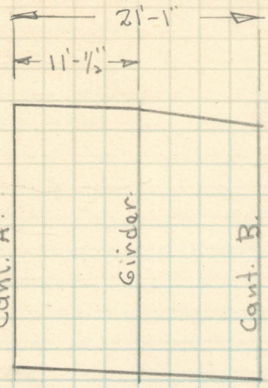


Moments at R_L and R_R.

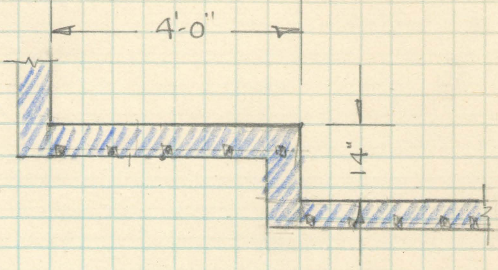
M at R_R.



Design of Balcony slab.



Lodge slab.



Live Load of 75#/sq. ft. =
Assume Dead Load as 100#/ft. } = 175

Moment. $M = \frac{1}{12} Wl = \frac{175 \times 11 \times 11 \times 12}{12} = 21,200$ in-lbs.

$\frac{M}{bd^2} = 107.4$ $p = .0077$. $d = 4$. inches.   Total Depth = 4.5 inch.

$A_s = .0077 \times 12 \times 4 = .37$ sq. in. per foot.

steel. Use - 3/8" ⌀ 4½" c-c.

Load per Sq. foot $= 75 + \frac{12 \times 4.5}{144} \times 150 = 75 + 56.25 = 131.25$

Load per Sq. foot Dead + Live = 135 lb.

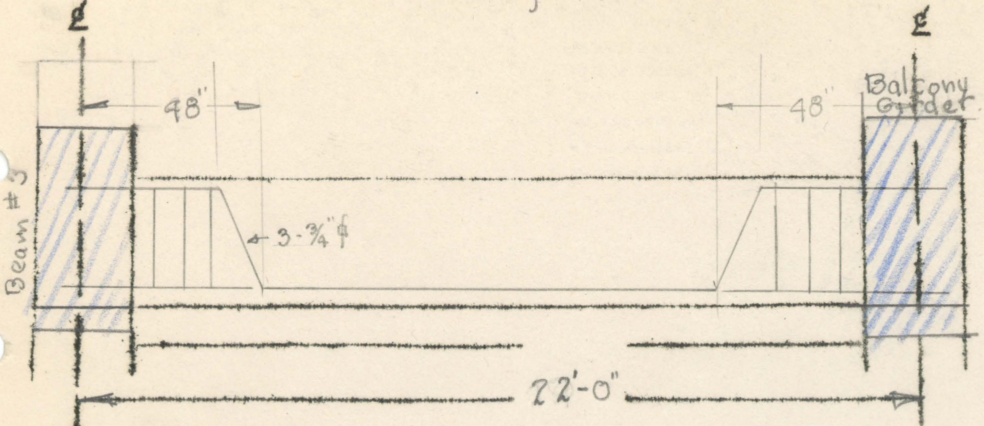
Load Delivered to Beam #1.

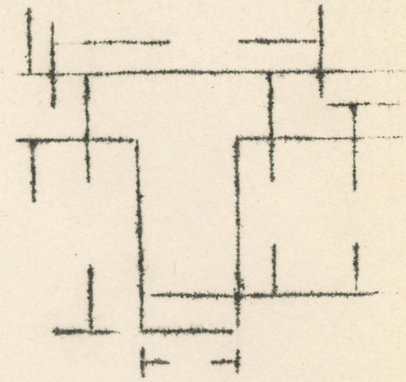
135 x 11 = 1485# per foot on Beam #1.

Load Delivered to Beam #2.

135 x 11 = 1485# per foot on Beam #2.

Balcony Beam #1.





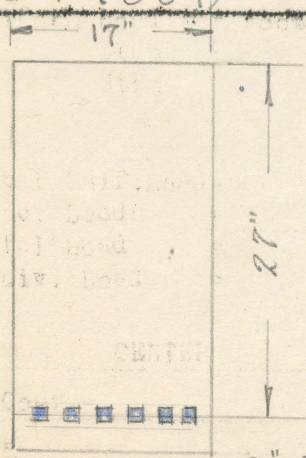
R_L 19,635 R_R 19,635

Uniform Load per Foot Pg. 35 1485
Slab =
Stem = 300

Max. Unit Shear = $\frac{19,635}{17 \times 27}$ = 43#/□"
Stirrups No =
1' =
Spacing = use - 3 - 3/8" ∅ stirrups. spaced @ 3".

Total Unif. Load = 39,270 = 22 x 1785
Conc. Loads =
Total Load =
Equiv. Load =

CENTER		SUPPORT	Right	Left
M Coef. = 1/8		M Coef.		
M = 1,295,000	"#	M at ℄ of support	"#	"#
M/bd2 = M/ 105		M at face support	"#	"#
b = 17	"	M/bd2	M/	M/
d = 27	"	Steel top	sq."	sq."
t/d =		p		
fc = 600	#/sq."	p'		
jd =	"	M/bd2 allowable		
As = 3.44	sq."	f' steel	#/sq."	#/sq."
Steel = 6-3/4∅ = 3.375	sq."	Length to develop	"	"
		Bond Stress	#/sq."	#/sq."
p = .0075		Steel bottom	sq."	sq."



6 - 3/4" ∅ bars.

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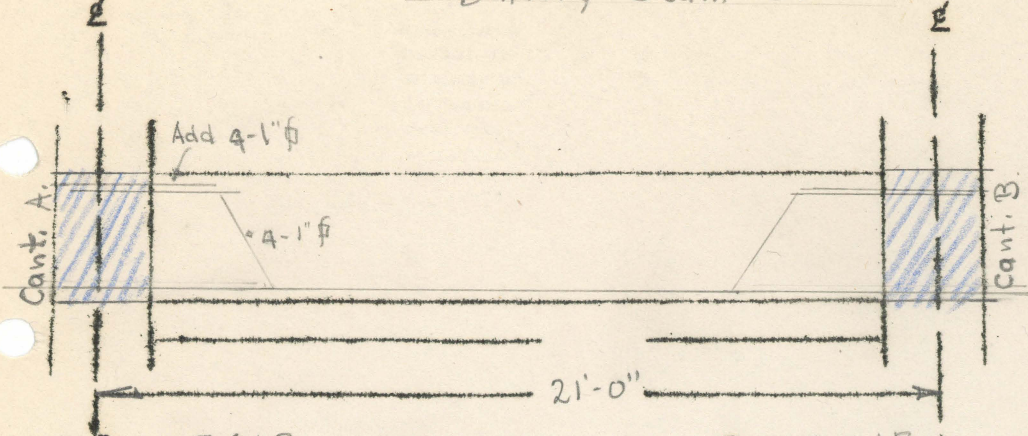
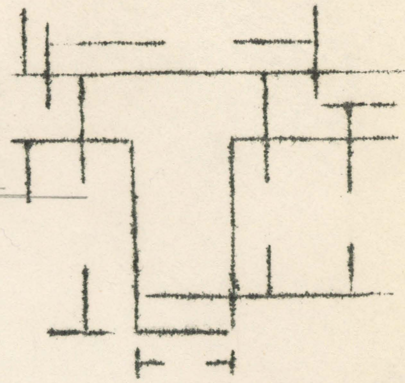
Work Order	Page No.

Balcony Beam #3.



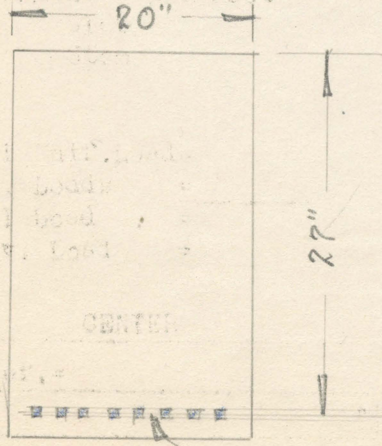


R_L 17417 R_R 17,417

Uniform Load per Foot	225	Max. Unit Shear =	$\frac{17,417}{20 \times 27}$ = 32 #/sq. in.
Slab	=	Stirrups Ns =	
Stem	= 500	1' =	No Stirrups Needed.
		Spacing =	

Total Unif. Load = 15,200 = 21 x 725
Conc. Loads = 19,635
Total Load = 34,835
Equiv. Load =

CENTER		SUPPORT	Right	Left
M Coef. = 1/8 + 1/4		M Coef.	1/10	1/10
M = 1,720,000	"#	M at ℄ of support	"#	"#
M/bd2 = M/ 118		M at face support	"#	"#
b = 20	"	M/bd^2	M/	M/
d = 27	"	Steel top	sq."	sq."
t/d =		p		
fc = 600	#/sq."	p'		
jd =	"	M/bd^2 allowable		
As = 8.1	sq."	f' steel	#/sq."	#/sq."
Steel = 8-1"ф = 8	sq."	Length to develop	"	"
		Bond Stress	#/sq."	#/sq."
p = .0145		Steel bottom	sq."	sq."



8-1"ф bars

Dead wt. of Beam #3 = 13,100#

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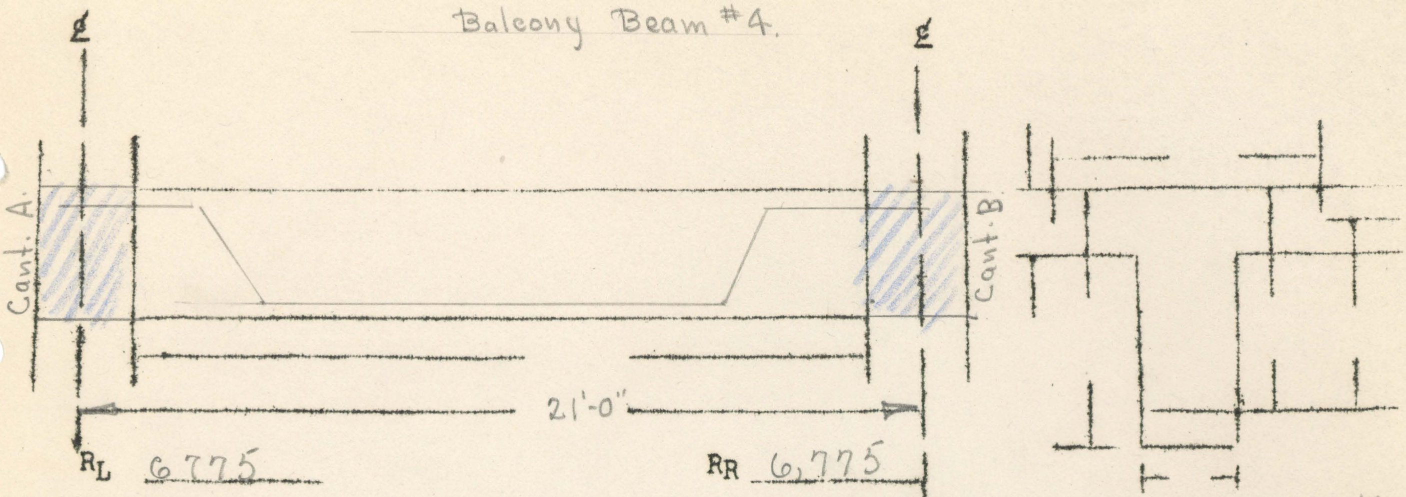
Balcony Beam #4.

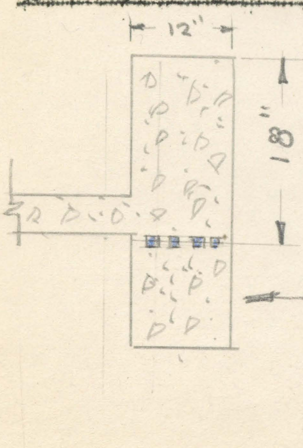


R_L 6775 R_R 6,775

Uniform Load per Foot	225	Max. Unit Shear =	$\frac{6,775}{12 \times 18}$ = 31.9 #/sq. in
Slab	=	Stirrups No	=
Stem	= 420	1'	=
		Spacing	= No stirrups Needed.

Total Unif. Load = 13,550 = 21 x 645
Conc. Loads =
Total Load =
Equiv. Load =

CENTER		SUPPORT	Right	Left
M Coef. = 1/10		M Coef. 1/10	1/10	1/10
M = 341,000	"#	M at ℄ of support	"#	"#
M/bd2 = M/ 87.6		M at face support	"#	"#
b = 12	"	M/bd^2	M/	M/
d = 18	"	Steel top	sq."	sq."
t/d =		p		
fc = 600	#/sq."	p'		
jd =	"	M/bd^2 allowable		
As = 2.26	sq."	f' steel	#/sq."	#/sq."
Steel = 4-3/4" ⌀ = 2.25	sq."	Length to develop	"	"
		Bond Stress	#/sq."	#/sq."
p = .0056		Steel bottom	sq."	sq."



See Page. 33a.

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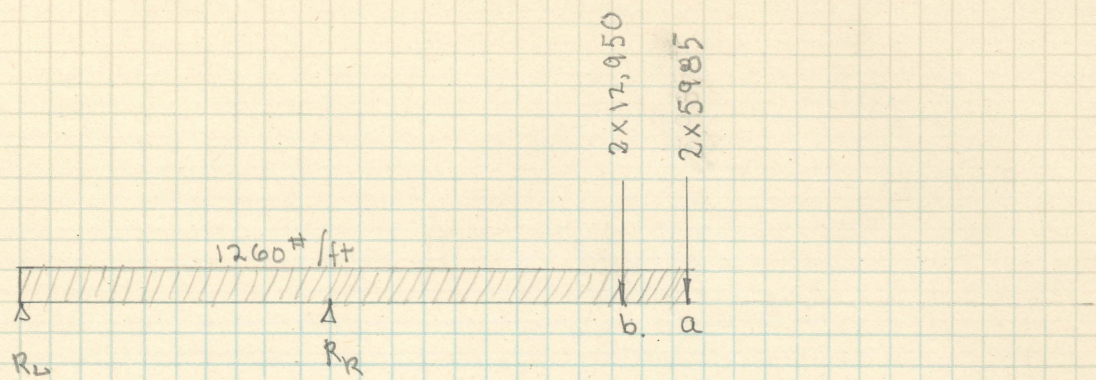


At a.-

Load at a will be 1/4 of wt. of slab between Beam #3 and #4, plus 1/2 wt. of Beam #4.

$$\frac{60\times4\times21}{4} = 1260\# + \frac{12\times36}{144}\times150\times21\times\frac{1}{2} = 4725\ \text{lb}$$

1260

Total Dead wt. of Balcony at a = 5,985 #.

At. b.

Load at b. will be 1/4 of slab between Cant. A. and Beam 1 plus 1/4 of wt. of Beam #1, plus 1/2 of wt. of Beam #3.

slab. $\frac{1}{4}\times60\times11\times22 = 3620$

Beam #1. $\frac{1}{4}\times\frac{17\times30}{144}\times150\times21 = 2,780$

Beam #3. $\frac{1}{2}\times\frac{20\times30}{144}\times21 = 6550$

12,950

2

25,900

There will be a uniform Load Due to Dead weight of slab coming directly upon the Cantilever.

$60\times[9'\text{-}11'' + 11'\text{-}\tfrac{1}{2}''] = 1260\#$ per foot.

Concentrated Loads.

M at b — $11,970 \times 4 = 47,800$ ft.-lbs. $= \underline{\underline{573,600}}$ in.-lb.

M at. R_R — $11,970 \times 23 + 25,900 \times 19 = 275,310 + 492,100 = 767,410$ ft.-lb $= \underline{\underline{9,208,920}}$ in.-lb.

Value of R_R at c. [pg. 34. Influence Line].

$R_R = 2.15 \times 11,970 + 1.95 \times 25,900 = 25,735 + 50,505.$

R_R — $R_R = \underline{\underline{76,240}}$ lb.

Value of R_L.

R_L. — $[1.15 \times 11,970] + [-.95 \times 25,900] = -13,765 + -24,605 = -38,370$ lb.

M at. R_L. — $11,970 \times 43 + 25,900 \times 39 - 76,240 \times 20 = 514,710 + 1,010,100 - 1,524,800 = 0$

Uniform Distributed Loads.

M at. b. — $1260 \times 4 \times \frac{4}{2} = 10,080$ ft. lb. $120,960$ in-b.

M at C. R_R — $1260 \times 23 \times \frac{23}{2} = 332,700 = 399,2400$ in-lb

R_R — Value of $R_R = [1260 \times 23 \times 1.575] + [1260 \times 20 \times .5] = 45,643 + 12,600$

$R_R = \underline{58,243}$ lb.

R_L — Value of $R_L = -[1260 \times 23 \times .575] + [1260 \times 20 \times .5] = -16,663 + 12,600$

$R_L = \underline{\underline{-16,663}}$ lb. and $+12,600$ lb.

Distance to left of R_R where $M = 0$.

$1260 \times 23 [11.5 + x] - 58243x + \overset{630}{1260} x \cdot \frac{x}{2} = 0$

$332,700 + 28,980x - 58,243x + 630x^2 = 0$

$630x^2 - 29,263x + 332,700 = 0$

$x^2 - 46.44x + 528.1 = 0$

$x = 20$ ft. to the left of R_R.

Uniform Distributed Loads [contd].

Moment 3.5 ft. from R_L or Midway between d and e.

$4,063 \times 3.5 \quad 1260 \times 3.5 \times \frac{3.5}{2} = 14,220 + 7,717 = 21,937$ ft.#

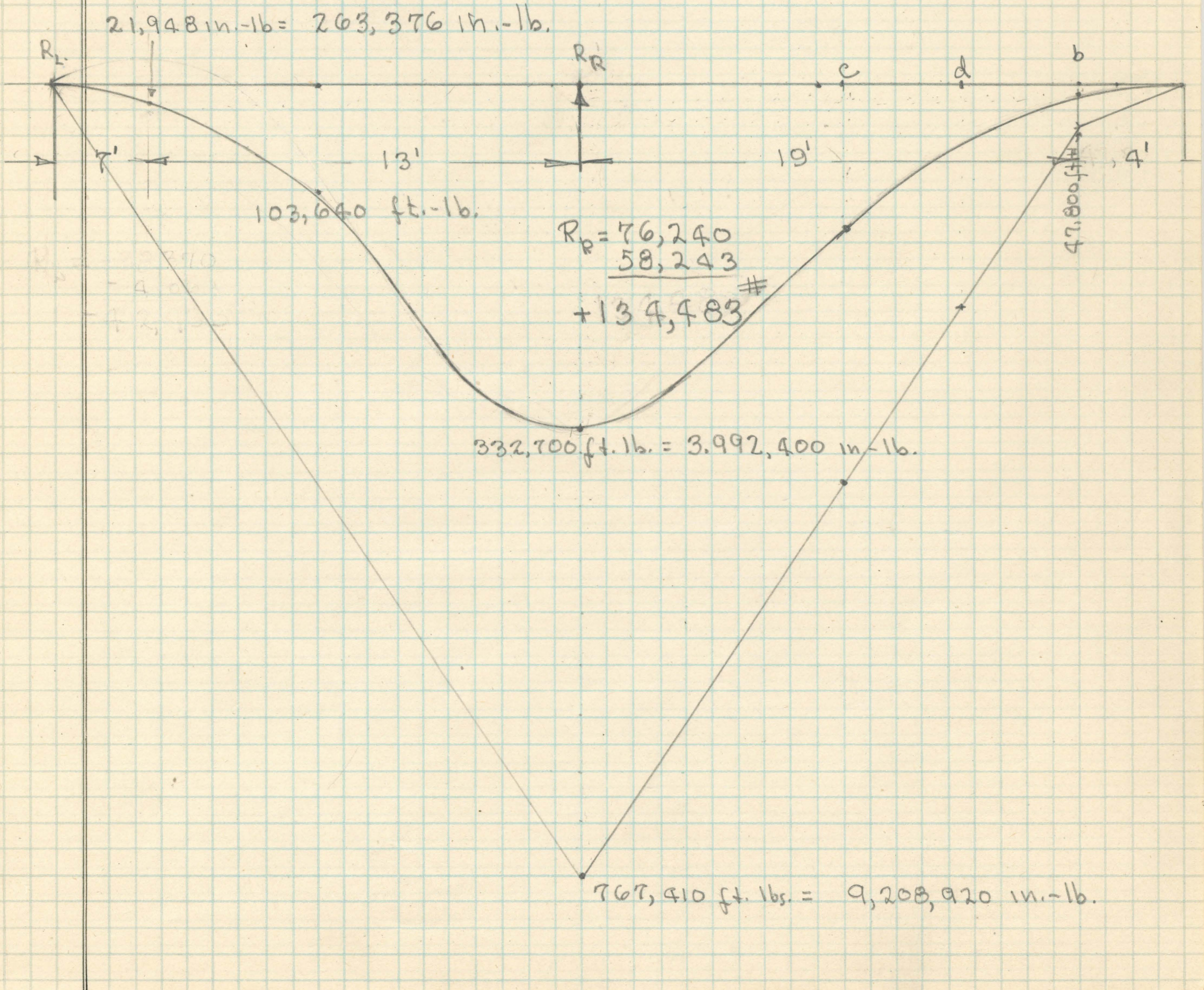
As a check.

$1260 \times 39.5 \times \frac{\cancel{39.5}^{19.75}}{\cancel{2}} - 58,243 \times 16.5 = M$

$M = 982,957 - 961,009$

$M = 21,948$ ft lb. Check.

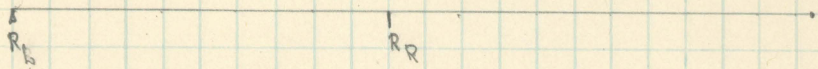
Moment Diagram of Dead Loads, Conc. and Uniformly Distrib.



Live Load = 75 lb. per sq. ft.

Live Load = 75[11+9'-11"] = 1575 lb. per. linear foot.

use 1600 lb. per linear foot.



Values of R_R and R_L under Various Loadings.

R_R. Cant Loaded.

$1600 \times 23 \times \frac{2.15+1}{2} = 1600 \times 23 \times 1.575$

$R_R = +57,960$ lb.

Anchor Loaded.

$1600 \times 20 \times .5 = +16,000$ lb.

Maximum $R_R = \underline{57,960 + 16,000} = +73,960$ lb. for Live Load.

R_L.

Cant Loaded.

$R_L = 1600 \times 23 \times \frac{1.15}{2} = 1600 \times 23 \times .575$

$R_L = -21,160$ lb.

Anchor Loaded.

21,160

$R_L = 1600 \times 20 \times .5 = +16,000$ lb.

For an Estimate assume Cant. to be 30" thick.
This will cause a varying distributed Dead Load.

The depth at Any point of Cantilever is $\frac{7.5}{23}x$
where x is the distance from cantilever end to point
under question.

Weight of Cant and Anchor

The weight of Cantilever to point x $= \frac{\left[\frac{7.5}{23}x\right]x}{2} \times 2.5 \times 150$ lb. =

$150 \times 1.25 \times \frac{7.5}{23}x^2 = 150 \times 1.25 \times .326\,x^2 = 61.13\,x^2$ lb.

Wt. of Cantilever = $61.13 \times 23 \times 23 =$ 32,327 lb.

Wt. of Anchor = $61.13 \times 43 \times 43 - 32,327 = 113,029 - 32,327$
= 80,702 lb.

Reactions Due to Dead Weight of Cantilever.

R_R.

$R_R = 61.13 \times 43 \times 43 \times .72 = +81,380$ lb.

R_b. =

$-61.13 \times 23 \times 23 \times .382 + [61.13 \times 43 \times 43 - 61.13 \times 23 \times 23] \times .5 =$
$-12,288 + 43,579 = +31,291$ lb.

Moment Due to Weight of Cantilever.

at. b.

$61.13 \times 4 \times 4 \times \frac{1}{3} \times 4 = 61.13 \times 16 \times 1.33 =$ 1300 ft. lb. at. b.

at. R_R.

$61.13 \times 23 \times 23 \times \frac{1}{3} \times 23 = 61.13 \times 529 \times 7.66$
$61.13 \times 4052 = 247,698$ ft. lbs.

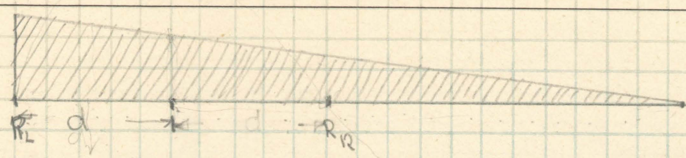
M at. R_R = 2,972,376 in.-lbs.

10 ft. to the right of R_R.

$M_{10} = 61.13 \times 10 \times 10 \times \frac{10}{3} = 20,356$ ft. lbs.

$M_{10} = 244,272$ in-lb. 10 feet right of R_R.

Distance to the Right of R_L where $M = 0$



$$R_L \times d - \left[(61.13\times 43\times 43) - 61.13\times(43-d)^2\right]\frac{d}{2} = 0$$

$$31{,}291d - \left[113{,}029 - 61.13(1849 - 86d + d^2)\right]\frac{d}{2} = 0$$

$$31{,}291d - \left[\cancel{113{,}029} - \cancel{113{,}029} + 5256d - 61.13d^2\right]\frac{d}{2} = 0$$

$$31{,}291d + \left[-5256d + 61.13d^2\right]\frac{d}{2} = 0$$

$$31{,}291d - 2628d^2 + 30.56d^3 = 0$$

$$31{,}291 - 2628d + 30.56d^2 = 0$$

$$d^2 - 86d + 1023.92 = 0$$

$$d = \frac{+86 \pm \sqrt{7396 + 4095.68}}{2}$$

$$d = \frac{+86 \pm \sqrt{3301}}{2} = \frac{+86 \pm 107}{2}$$

This gives a negative value for d. ∴ The moment is not zero Between R_L and R_R.

Moment midway between R_L and R_R

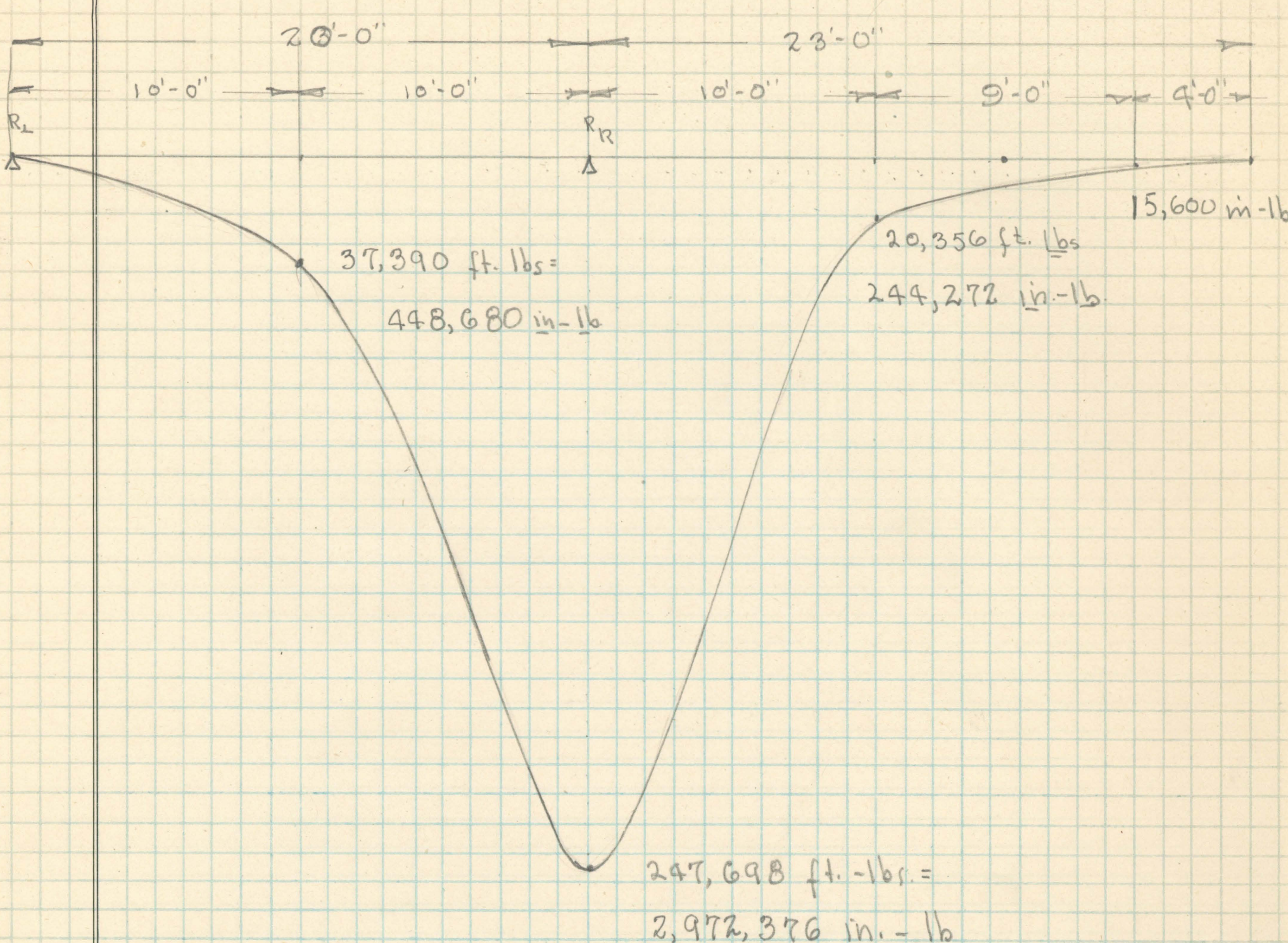
$$R_L \times 10 - \left[61.13\times 43\times 43 - 61.13\times 33\times 33\right]5$$

$$31{,}291\times 10 - \left[113{,}029 - 66{,}570\right]7.54 = M$$

$$312{,}910 - 46{,}459\times 7.54 = 312{,}910 - 350{,}300$$

$M = -37{,}390$ ft. lb. $= -448{,}680$ in-lb.

Moment Diagram for Wt. of Cant.



Moment Due to Uniform Distributed Live Load.

Entire Cant. Loaded.

Liveload = 1600/ft.

M at b. = $1600 \times 4 \times \frac{\cancel{4}^{2}}{\cancel{2}} \times \cancel{12}^{6}$ = 153,600 in.-lb. = 12,800 ft.-lb.

M at R_R = $1600 \times 23 \times \frac{23}{\cancel{2}} \times \cancel{12}^{6}$ = 5,078,400 in-lbs = 423,200 ft.lb.

M, 7 ft. from R_L.

$1600 \times 36 \times \frac{\cancel{36}^{18}}{\cancel{2}} - 73,960 \times 13$ = 1,036,800 − 961,480 = 75,320 ft.lb

M mid way between R_L and R_R.

$1600 \times 33 \times \frac{33}{2} - 73960 \times 10$ = 817,200 − 739,600 = 77,600 ft.lb.

M = 77,600 ft. lb. = 931,200 in-lb.

Cantilever Arm Loaded Only.

M at b = 12,800 ft.-lb. = 153,600 in-lb. [Pg. 45]

M at. R_R = 423,200 ft. lb = 5,078,400 in-lbs. [Pg. 45]

M 10 feet. left. of R_R.

1600 X 23 X 21.5 - 73,960 x 10 = 791,200 - 739,600

M = 51,600 ft. lb. = 619,200 in. lb.

M 10 feet. Right of R_R.

$1600 \times 13 \times \frac{13}{2}$ = 135,200 ft. lb. = 1,622,400 in-lb.

Position where M is zero.

1600 x 23 (11.5 + X) - 73960 X = 0

423,200 + 36,800 X - 73,960 X = 0

X = 11.4 ft. to left. of R_R.

M at. this point.

1600 X 23 X 22.9 - 73,960 X 11.4 = 0

842,720 - 843,144 = 0 Approx.

M. 6 feet from R_L.

1600 X 23 X 25.5 - 73,960 X 14 = M.

M = 938,400 - 1,035,440

M = - 97,040 ft. lbs.

M 4 feet. from R_L.

1600 X 23 X 27.5 - 73,960 X 16 = M

M = 1,012,000 - 1,183,360

M = - 171,360 ft. lb.

Anchor loaded Only.

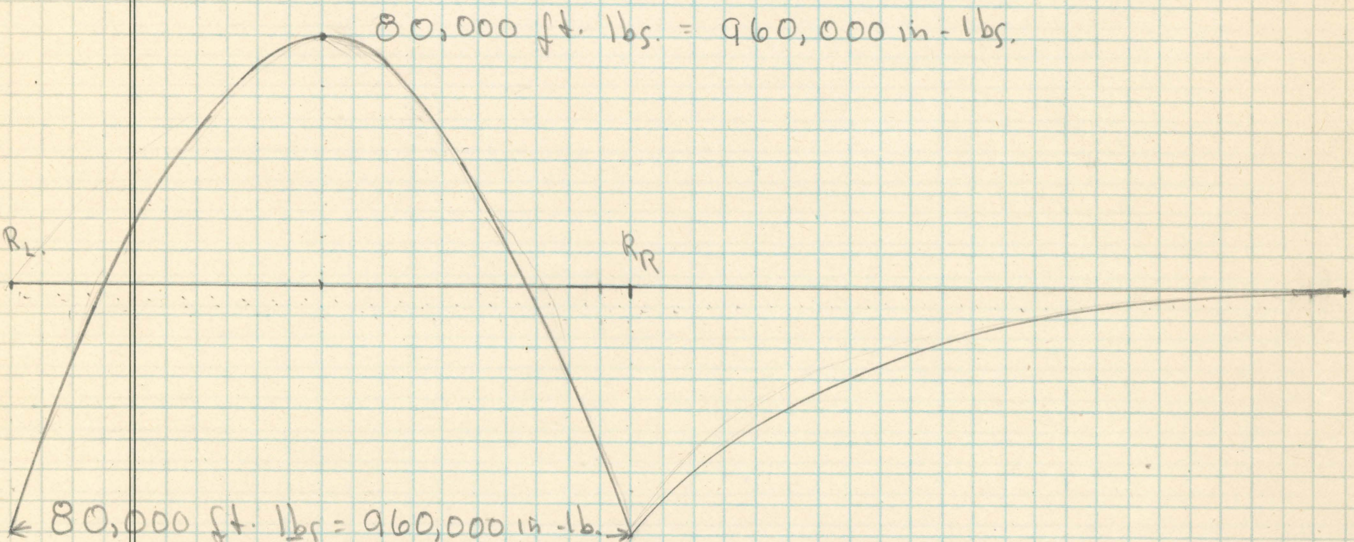
$R_L = R_R = +16,000$ lbs.

M at the center $= \frac{1}{8} W l$.

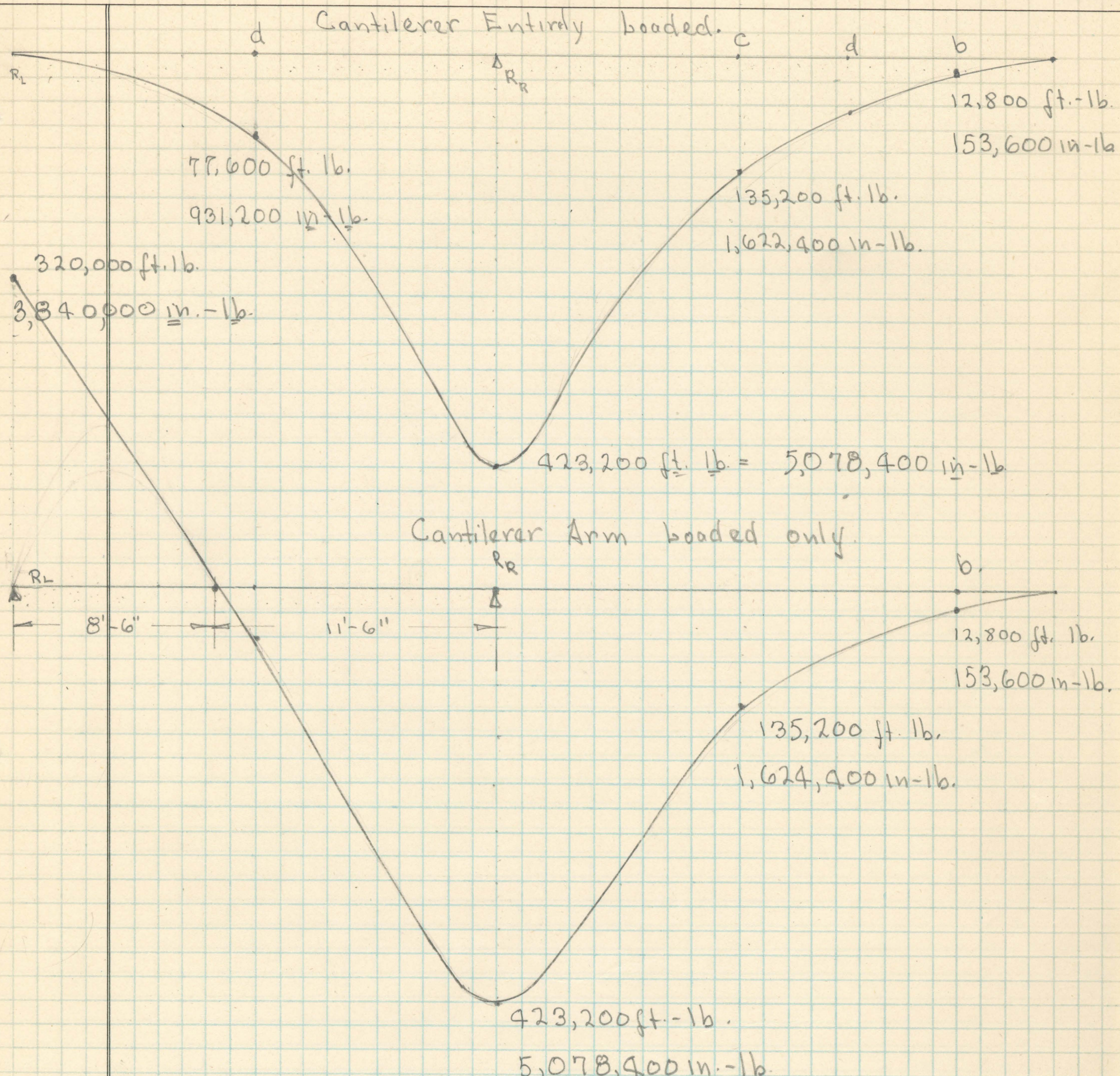
$M_c = \frac{1}{8} \times 32000 \times 20 \times 12 = 32000 \times 20 \times 1.5$

$M_c = 960,000$ - in-lb. $= 80,000$ ft-lb.

$M_{R_R} = M_{R_L} = -80,000$ ft.-lb. $= -960,000$ in-lb.



Moment Diagram. Live Load.



Part Loaded	Dead Loads Concentrated.						Dead Load Uniformly Distributed, Balcony Slab.					
	R_l	R_r	M_{R_l}	M_{R_r}	M_c	M_b	R_l	R_r	M_{R_l}	M_{Rr}	M_c	M_b
	-38,370	+76,240	0	-767,410 ft.#	-375,000 ft#	-47,800 ft#	-4,063	+58,243	0	-332,000 ft.lb.	-137,000 ft.-lb.	-12,500 ft.-lb.
	Pg.40	Pg.40		-9,208,920 "lb.	-4,500000 in.-lb.	-573,600 in.-lb.	Pg.40	Pg.40		-3,992,000 in-lb.	-1,644,000 in.-lb.	-150,000 in.-lb.
				Pg.41	Pg.41	Pg.41				Pg.41	Pg.41	Pg.41

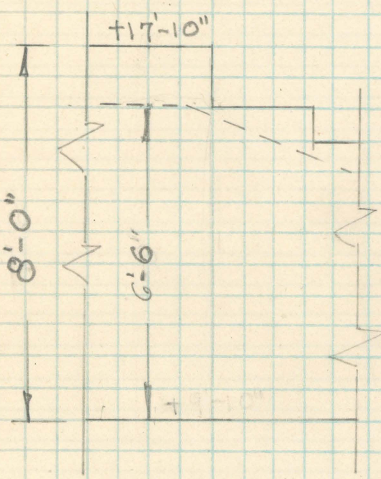
Part Loaded.	Dead Load Varying Uniformly. Wt. of Cantilever. R_l	R_r	M_{Rl}	M_{Rr}	M_c	M_b	Live Load Uniformly Distributed. R_l	R_r	M_{Rl}	M_{Rr}	M_c	M_b
	+31,291	+81,380	0	-247,698 ft.-lb. -2,972,376 in-lb Pg. 43, 45.	-20,356 ft.lb -244,272 in-lb Pg. 43, 45	-15,600 in-lb Pg. 45						
Entire Cant. Loaded.							-5,160 Pg. 42.	+73,960 Pg. 42.	0	-423,000 ft.lb. -5,078,400 in-lb. Pg. 45, 48	-135,200 ft.lb. -1,622,400"# Pg. 48.	-12,800'# -153,000" Pg. 48.
Cantilever Arm Loaded.							-21,160 Pg. 42	+57,960 Pg. 42.	+320,000 ft.lb. +3,840,000 in-lb. Pg. 48.	-423,200 ft.# -5,078,400 in.# Pg. 48	-135,200 ft.lb. -1,622,400 in# Pg. 48	-12,800 ft.# -153,600" Pg. 48.
Anchor Loaded.							+16,000 Pg. 47	+16,000 Pg. 47.	-80,000 ft.lb. -960,000 in# Pg. 47	-80,000 ft.# -960,000 in.# Pg. 47.		

Total M. at R_R. Moment in inch-pounds over R_R.

Concentrated Dead Loads.	−9,208,920 in-lb.
Uniformly Distr. " "	−3,992,000
Varying " " "	−2,972,376
Live Load.	−5,078,400
	−21,251,696 in.-lb.

Moment over R_R = −21,251,696 in.-lb.

Section over R_R.



$d = 78''$ $b = 24''$

$\frac{M}{bd^2} = \frac{21,251,696}{24 \times 5984} = \frac{21,251,696}{143,616} = 148$

p. for $f_s = 16,000$ $f_c = 800$ $p = .011$

This will necessitate compression steel to reduce f_c.

A_s Tension Steel. $A_s = .011 \times 78 \times 24 = 20.6$ sq. $\times 1.07 = 22.04$

use 22 - 1" ⌗ bars in two layers. [Tension Steel.]

Bond. $R_R = 289,823$# $u = \frac{289,823}{4 \times 20 \times 78 \times .855} = \frac{289,823}{5335}$

$u = 54.3$ #/sq.-in.

Comp. Steel A'_s Reduction of f_c of 25% this will necessitate $p' = .0085$.

$A'_s = 78 \times 24 \times .0085 = 15.9$ sq. in.

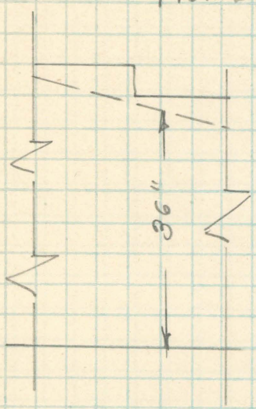
use 16 - 1" ⌗ bars in two layers

Section 10 ft. Right of R_R.

Moment 10 ft. to the right of R_R or M_c

Concentrated Dead Loads.	$-4,500,000$ in.-lb.
Uniformly Distrib. " "	$-1,644,000$
Varying " " "	$-244,000$
Live Load.	$-1,622,400$
	$-8,020,400$ in.-lb.

Moment $M_c = -8,020,400$ in.-lb.



$d = 36''$ $b = 30''$

$$\frac{M}{bd^2} = \frac{8,020,400}{24 \times 1296} = \frac{8,020,400}{38,880} = 206$$

useing same steel as at R_R.

$$p = \frac{20.5}{36 \times 30} = .019 \quad j = .81 \quad K = .52$$

$$M_c = \tfrac{1}{2} f_c K j (bd^2) \qquad f_c = \frac{2 \times M_c}{K j \, bd^2} = \frac{2 \times 8,020,400}{.52 \times .81 \times 31,104}$$

$$f_c = \frac{8,020,400}{31,104 \times .208} = 1240 \ \#/\text{sq. in.}$$

Reduction of 47.5% is necessary by compressive steel A'_s.

p' should be .02

$A'_s = 16$ sq.-in same as at R_R.

$$p' = \frac{16}{36 \times 24} = .019$$

The steel from R_R may be extended clear to M_c and will take care of bending moment.

Moment at Section b. M_b.

Moment at b.

Dead Load Concent.	- 573,600 in.-lb.
" " Uniformly Dist.	- 150,000 in-lb.
" " " Vary.	- 15,600
Live Load " Distrib.	- 153,600
Moment =	- 892,800 in.-lb.

d = 20" b = 30"

$$\frac{M}{bd^2} = \frac{892,800}{24 \times 400} = \frac{892,800}{9600} = 93.$$

p

p = .007. fc = 575 lb sq. in

As

As = 20 x 24 x .007 = 3.36 sq.-in x 1.07 = 3.59

use 4 - 1" ⌀ bars.

Moment at Section d. 4 1/2 feet to the left of c.

Dead Load Concentrated	- 2,544,000 in-lbs.
" " Uniformly Dist.	1,644,000
" " Varying	120,000
Live Load.	720,000
b = 30" d = 36 inch.	5,028,000 in-lbs.

$$\frac{M}{bd^2} = \frac{5,028,000}{24 \times 1296} = \frac{5,028,000}{31,104} = 128$$

$$p = \frac{20.5}{24 \times 36} = .019$$

for fs = 11,000, and fc = 650 p = .015 therefore beyond this point compressive steel is not needed. Extend compressive steel to point "b".

Extend Tension steel to the end of Cantilever Arm.

Shear

Distance to point where shear is 40 #/sq. in.

Shear at a point 19' feet to the Right of R_R or at b.

$4 \times [1260+1600] + 61.13 \times \overline{4}^2 + 25,900$

$11,440 + 978 + 25,900 = 38,318$ lb.

unit shear $= \dfrac{38,318}{24 \times 20} = \dfrac{38318}{480} = 80$ lb.

Shear b.

To the right of Point b.

$\dfrac{38,318 - 25,900}{480} = \dfrac{12,418}{480} = 25.8$ #/sq. in.

Shear 7 feet to the left of R_R.

$[1260+1600]30 + 61.13 \times \overline{23}^2 + 8 \times 2.5 \times 7 \times 150 - 289,823$

Shear 7 ft. left. R_R

$85,800 + 32,337 + 21,000 - 289,823 + 25,900 + 11,960$

$37,860 + 139,137 - 284,823 = 112,826$ lb.

Unit Shear $= \dfrac{112,826}{24 \times 78} = \dfrac{112,826}{1872} = 60.2$ #/sq. in

Shear right of R_R.

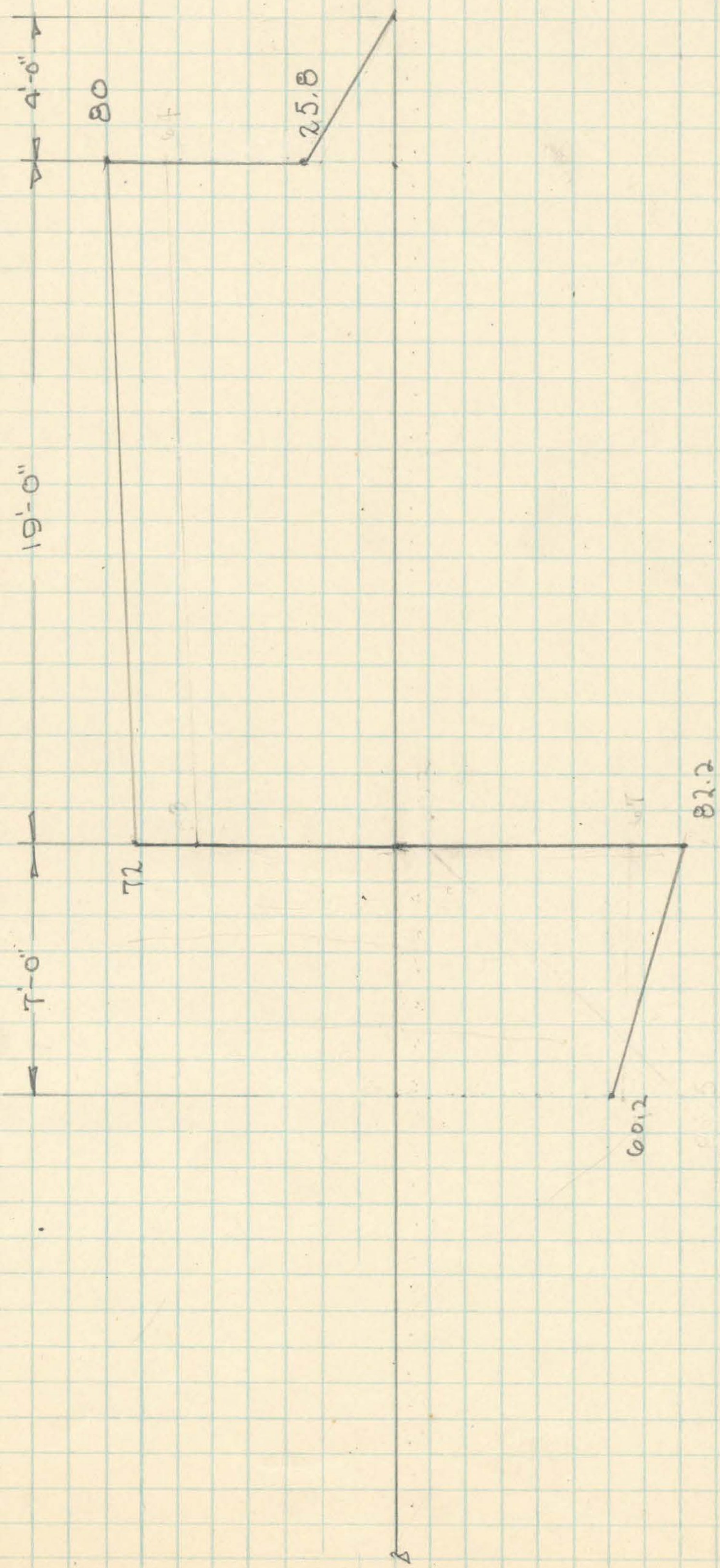
$[1260+1600]23 + 61.13 \times 23 \times 23 + 25900 + 11,960] =$

$65,780 + 32,337 + 37,860 = 135,977$

$\dfrac{135,977}{24 \times 78} = \dfrac{135,977}{24 \times 78} = 72.6$ #/sq. in

Shear left of R_R.

$\dfrac{289,823 - 135,977}{1872} = \dfrac{153,846}{1872} = 82.2$ #/sq. in



Unit Shear in Cantilerer Arm.

Shear from R_R to point b.

$$\left[\frac{32+40}{2} \times \overset{12}{\cancel{24}}\right] \times 19 \times 12 = 72 \times 12 \times 19 \times 12 = 864 \times 228 = 196{,}992$$

Shear — Total Shear 196,992 #

As. — $\frac{196{,}992}{16{,}000} = 12.3$ sq. in.

Stirrups — Use 22 - 3/4" ϕ Stirrups. from R_R to point b.

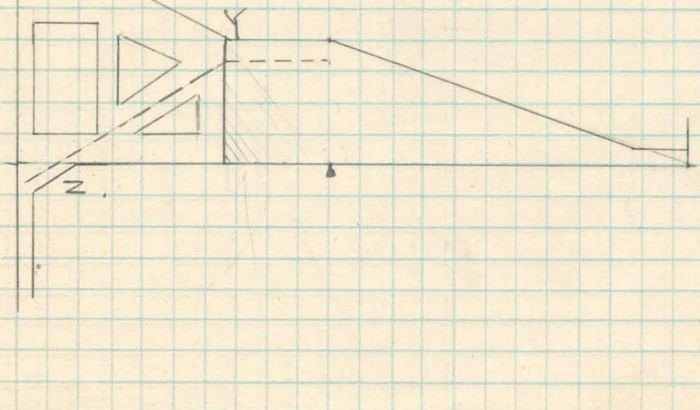
Shear from R_R to point 7 feet to the left.

$$\frac{20+42}{2} \times \overset{12}{\cancel{24}} \times 7 \times 12 = 62 \times 12 \times 84 = 62 \times 1{,}008 =$$

Total Shear = 62,496 lb

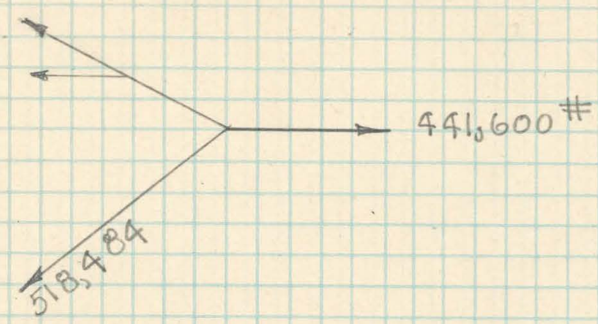
A_s — $\frac{62{,}496}{16{,}000} = 4.06$ sq. in

Stirrups — use 18 - 5/8" ϕ stirrups to a point 7 ft. to left of R_R.



H.C. of Members XY and YZ will be equal to the stress in steel of Cantilever Arm.

Stress in steel = 16,000 × 27.6 = 441,600 #.



H.C. of XY. Taking Moments about point Z.

Positive Moment about Z = 320,000 ft.-lb. [Pg. 48]
Negative " " " = 80,000 " " [Pg. 47]

Superseded

$\frac{320,000}{15} = -21,333$ lb compression.

This thrust occurs only when Cantilever Arm is loaded.

$\frac{80,000}{15} = +5,333$ lb Tension

This stress occurs only when Anchor alone is loaded.

H.C. of ZY. H.C. of ZY = 441,600 + 21,333 = 462,933 #

Stress in ZY = $462,933 \times \frac{\sqrt{6.5^2+13^2}}{13} = 462,933 \times \frac{14.6}{13}$

Stress in ZY = 518,484 lb.

A_s ZY $\frac{518,484}{16,000} = 32.4$ sq in of steel

Use 28-1"ϕ bars which come from Cantilever Arm and 5-1"ϕ bars extra which take this extra stress.

passing a section and taking moments about U.

$M = -21,251,696$ in-lb. [pg.51]

$M = 1,770,970$ ft. lb.

$H.C.\ ZO = \frac{1,770,970}{6.5} = 272,456$ # compression.

This force acts at a point which is $[6.5 - j \times 6.5]$ ft. from Bottom of Cantilever.

$j = .855$

$6.5 - 6.5 \times .855 = 6.5 - 5.5 = 1$ ft. from Bottom.

Consider Member ZO acting as a column

Outside Dimensions 24"x24". inside dimensions 20"x20"

$550 \times 400 = 220,000$ lb. strength of concrete.

p $\frac{P}{P'} = \frac{272}{220} = 1.24$ $p = .017$

As $A_s = .017 \times 400 = 6.8$ sq. in.

Use 12 - 3/4" ϕ rods = 6.75 sq. in.

Extend these bars 40 inches beyond R_R.

and also down into column at point Z.

Use - 3/16" ϕ ties every 12 inches

---

shear. Vertical comp of ZY $= 79,544 \times \frac{6.5}{14.5} = 35,600$ lb.

Superseded.

Dist. where Shear = 40 #/sq in = $\frac{35,600 - 24 \times 24 \times 40}{600} = \frac{35,600 - 23244}{600}$

" " " = 20 feet.

shear = $\frac{35600}{576} = 62$ #/sq in. $22 \times 24 \times 20 \times 12 = 150,172$

$\frac{150,172}{16,000} = 9.4$ sq. in. - Use 12 - 3/8" ϕ Stirrups.

Design of Member ZY.

H.C.

This member's H.C. will be equal to the stress of ZO = 272,456 lb. [Pg 58]

Total Stress = $272,456 \times \frac{14.6}{13}$ = 306,000 lb.

As

$\frac{306,000}{16,000}$ = 19.25 sq. in.

Use 20 - 1" ф bars. These to be brought over by using steel from Cantilever.

Design of Member XY

Tension Steel.

Stress taken by XY in Tension will be equal to the Difference of stresses in the steel in Cantilever and the stress in ZY.

Total stress in Cantilever Steel = 22x16,000
" " " " = 352,000 lb.

H.C. of stress in steel of ZY = 272,456 lb

H.C. of XY = 352,000 - 272,456 = 79,544 lb.

As

$\frac{79,544}{16,000}$ = 4.96 sq. in.

Use 5 - 1" ф bars

Two of These bars may be supplied by extending 2-1" ф bars from Cant. Tension steel, the other 3-1" ф bars must be extra. These extra 3-1" ф bars to extend Beyond point U for 48 inches.

Tension steel.

Design for Direct loading

w. Uniform load = 1260 + 1600 = 2860 lb/ft.

$M = \frac{1}{8} Wl = \frac{2860 \times 14.5 \times 14.5 \times \overset{1.5}{\cancel{12}}}{\cancel{8}} = 2860 \times 315.4$

M. $M = 902{,}044$ in-lb.

Assume Dead wt. $\frac{30 \times 24}{144} \times 150 = 750$ #/ft.

$M = \frac{1}{8} 750 \times 14.5 \times 14.5 \times 12 = 750 \times 315.4 = 236{,}550$ in-lb.

Total M = 1,138,594 in.-lb.

$d = 24" - b = 24"$

$\frac{M}{bd^2} = \frac{1{,}139{,}594}{24 \times 576} = \frac{1{,}139{,}594}{10{,}284} = 110.8$

As. $p = .0084 \quad A_s = 24 \times 24 \times .0084 = 4.9$ sq. in.

steel. Use - 9 - 3/4" ф bars

Shear. $\frac{[2860 + 750] \times 7.25}{576} = \frac{3610 \times 7.25}{576} = \frac{26{,}172}{576} = 45$ #/sq. in

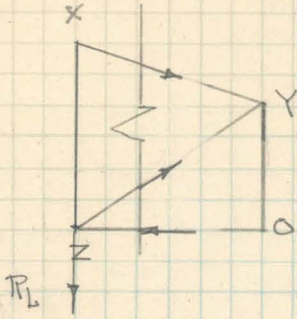
$\frac{14.5}{2} - \frac{40 \times 24 \times 24 \times .87}{3610} = 7.25 - 6.2 = 1.05$ ft.

Shear = $5 \times 24 \times 1.05 \times 12 = 1512.$

Stirrups. use 2 - 3/8" ф stirrups.

Turn up 5 - 3/4" ф bars 3'-4" from end.

Design of Member XZ.



Tension Strength.

with R_L = minus

V.C. of ZY = $272,456 \times \frac{6.5}{14.5} = 122,000$ lb

R_L = $-68,753$ lb

V.C. of XZ = $68,753 - 35,600$

" " " = $33,153$ lb

This causes Tension in XZ.

Superseded.

As.

$\frac{33,153}{16,000} = 2.1$ sq-in.

Steel

Use 6 - 5/8" Square, bars. For Tension Steel

Compression.

V.C. of ZY = 122,000 lb [pg. 58].

Max [+ R_L] = 47,291 lb. [pg. 49 + 50]

V.C. of XZ in this loading = 169,291 lb.
Direct loading = 2860×7.25 = 20 735

Net Section of XZ which consists of 16" x 24"
" " " " 12 X 20 = 160 sq. in.

$P' = 550(1.6 - \frac{15 \times 12}{25 \times 8}) = 550 \times .9 = 495.0$ lb.

$240 \times 495 = 118,800$ lb.

$\frac{P}{P'} = \frac{190,026}{118,000} = 1.61 = 1 + (14)p$

$p = \frac{.61}{14} = .0435$

As. $As = 240 \times .0435 = 10.05$ sq.-in.

steel. Use - 8 - 1 1/8" ⌀ bars with 1/4" ties every 12 inches.

steel at joint X to transmitt stress from XY to XZ.

H.C. of XY = 79,544 lb [Pg. 59]

V.C. of XY = $79,544 \times \frac{7.5}{14.5} = 41,000$ lb.

As

$\frac{41,000}{16,000} = 2.5$ sq. in.

steel. Use 5 - 1/2" ⌀ Stirrups at Joint X and Y.

V.C. of ZY = 122,000 lb [pg. 58]

$\frac{122,000}{120}$ = 1010. sq. in. width at Z = 24 inches.

$\frac{1010}{24}$ = 41.5 inches Deep.

There extended ZO 42 inches Below Toilet floor level at joint Z. Since ZO is 24" Deep, this will extend 18 inches below ZO. Slope this at 45° and make concrete extend 18" from column out on ZO.

shear. 1010 X 80 = 80,800 lb.

As. $\frac{80,800}{16,000}$ = 5 sq. in

steel. use 7 - 5/8" ɸ stirrups spaced 3" apart.

Tension. Design of Column at R_L.

Maximum - R_R = -68,753 lb

As = $\frac{68,753}{16,000}$ = 4.2 sq.-in.

steel use - 8 - 3/4" ɸ bars. 3/16" ties every 12 inches.

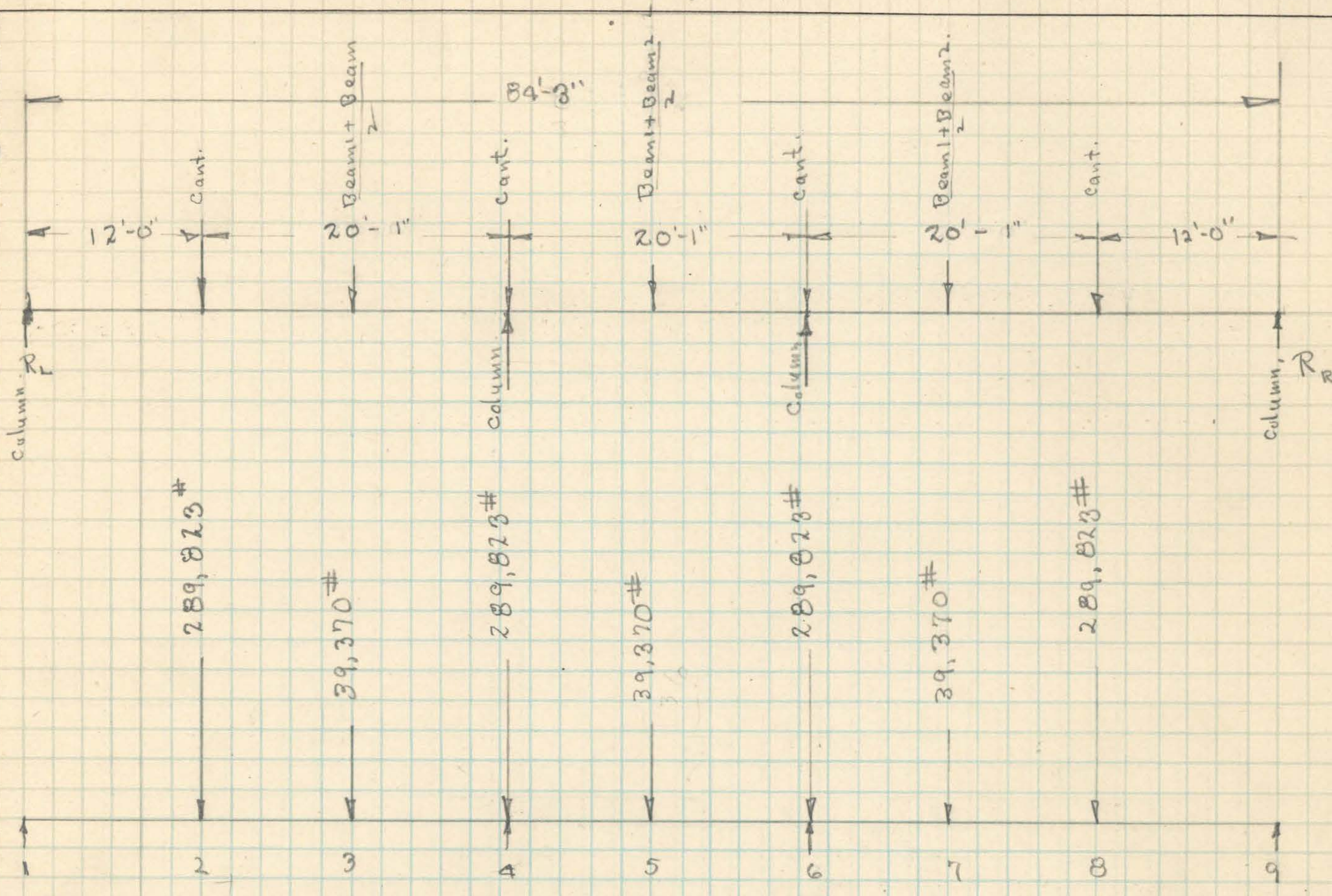
Compression. Max + R_R = 47,291 lb.

Utilize 12 inch. wall for column.

net section 8x20" = 160 sq. in.

P = 160×500 = 80000 lb this will carry the Truss with the steel as specified above.

Loading For Balcony Girder.



Moment Coeff.

+.244 +.156 -.267 +.1 -.267 +.156 +.244

Shear Coeff.

Hool Pg. +.332. will be assumed the same as in Simple beams.

M at 2.

$M = +.244 \times Pl = .244 \times 289,823 \times 32 \times 12$

$M = 93.7 \times 289,823 = +27,156,415$ in-lbs

M at 3.

$M = +.156 \times Pl = .156 \times 39,370 \times 32 \times 12 = +2,358,263$

$M = +.156 \times Pl = +.156 \times 289,823 \times 32 \times 12 = +17,380,105$

M. at 4.

$M = -.267 \times Pl = -.267 \times 289,823 \times 32 \times 12$

$M = 102 \times 289,823 = -29,561,946$ in.-lb.

M at 5

$M = +.1 \times Pl = .1 \times 39,370 \times 20 \times 12$

$M = 24 \times 39,370$ in-lb. $= 944,880$

$M = .1 \times 289,823 \times 240 = +6,955,752$

$\left.\begin{matrix} 944,880 \\ 6,955,752 \end{matrix}\right\} 7,900,632$ in-lb

M at 1.

M at 1 will be taken as 1/2 of M at 2

$M = -13,578,207$ in-lb.

Dead Moment.

Moment Due to Dead wt.

assume Dead wt = $8 \times 3 \times 150 = 3600$ lb. per foot

M at 2.

$M = \frac{wl}{10} = \frac{3600 \times 32 \times 32 \times 12}{10} = +4,420,800$ in-lb.

M at 4.

$M = \frac{wl}{12} = \frac{3600 \times 32 \times 32 \times 12}{12} = -3,686,400$ in-lb

M at. 5

$M = +3,686,400$ in-lb.

M at 1

$M = -\frac{wl}{20} = \frac{3600 \times 32 \times 32 \times 12}{20} - 2,210,400$ in-lb at 1. R_2 = Shear at 1.

Dead wt. $3600 \times \frac{32}{2} = 3600 \times 16 = 57,600$

Concentrated. $289,823 \times \frac{20}{32} = 289,823 \times .625 = 181,139$

$39,370 \times \frac{10}{32} = 39,370 \times .31 = 12,204$

Total R_2 = + 250,943 lb

Shear at 2. Left of Section. +250,943 − 3600×12 = 250,943 − 43200

Left of Section 2 = +207,743 lb

Right of Section 2. +207,743 − 289,823

Right of Section 2 = − 82,080 lb

Shear at 3. Left of Section 3.

− 82,080 − 3600×10 = − 82,080 − 36,000 = − 118,080 lb

Right of Section 3.

−118,080 − 39,370 = −157,450 lb.

Shear at 4 Left of 4.

−157,450 − 3600×10 = −157450 − 36000 = −193,450 lb

Right of 4.

Reaction at 4. = $289,823 + [289,000 - 250,943] + 3600 \times 10 + \frac{39,370}{2}$

Reaction at 4 = 289,823 + 38,057 + 36,000 + 19,685

Reaction at 4 = + 383,566 lb

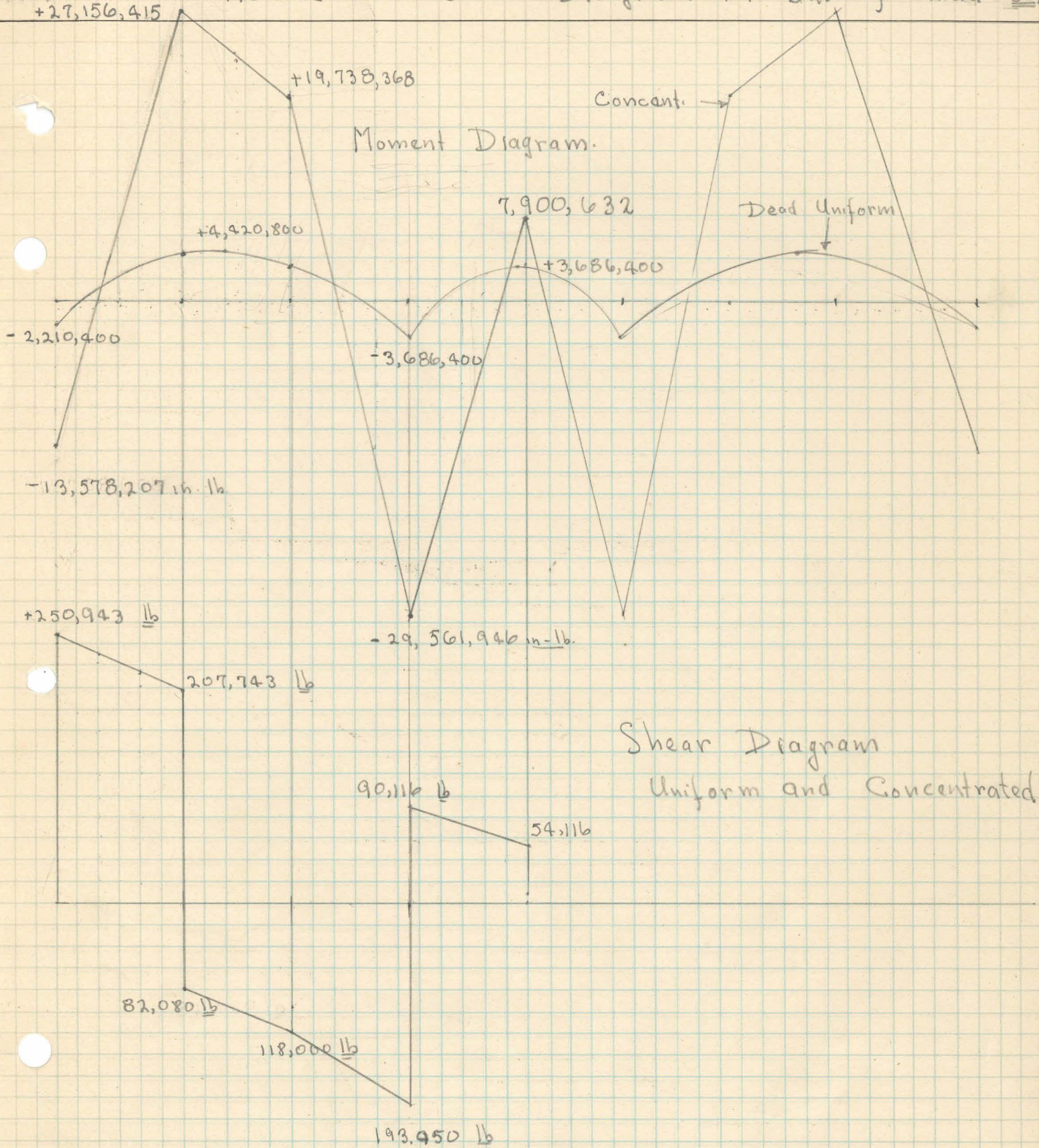
Shear to right of 4.

383,566 − 193,450 = + 90,116 lb

Shear at 5. Left of 5. 90,116 − 3600×10 = 90,116 − 36000 = + 54,116 lb

Right of 5. 54,116 − 39,370 = + 14,746 lb.

Moment and Shear Diagram For Balcony Girder 66.



Section at 2.

$M = +31,577,215$ in-lbs.

$b = 30''$ $d = 92''$

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

$\frac{M}{bd^2} = \frac{31,577,215}{30 \times 92 \times 92} = \frac{31,577,215}{30 \times 8464} = \frac{31,577,215}{253,920} = 124$

$p = .013$

As.

$A_s = .013 \times 92 \times 30 = .013 \times 2760 = \underline{35.88}$ sq in. of steel.

Section at. 4.

$M = -33,248,346$ in-lb.

$\frac{M}{bd^2} = \frac{-33,248,346}{253,920} = 130$

$p = .016$

$A_s = .016 \times 30 \times 92 = .016 \times 2760 = \underline{44.16}$ sq. in. of steel.

Section at 5

$M = +11,587,032$ in-lb

$\frac{M}{bd^2} = \frac{11,587,032}{253,920} = 46.$

$p = .004$

$A_s = 30 \times 92 \times .004 = 2760 \times .004 = \underline{11.04}$ sq in.

Section at 1.

$M = -15,788,607$

$\frac{M}{bd^2} = \frac{15,788,607}{253,920} = \underline{62.1}$

$p = .005$ $A_s = 2760 \times .005 = \underline{13.8}$ sq. in.

Section at 3.

$M = +23,238,368$ in. lb

$\frac{M}{bd^2} = \frac{23,238,368}{253,920} = 91.5$

$p = .0066$ $A_s = .0066 \times 2760 = \underline{18.2}$ sq. in.

Section midway between [illegible]-2.

$M = +8,000,000$ in-lb

$\frac{M}{bd^2} = \frac{8,000,000}{253,920} = 40$

$p = .003$ $A_s = 2760 \times .003 = 8.28$ sq. in

7 - 1 1/8" ⌀ bars

Section 4 ft. left of 2.

$M = 15,250,000$ in-lb.

$\frac{M}{bd^2} = \frac{15,250,000}{253,920} = 60$

$p = .0044$ $A_s = 2760 \times .0044 = 12.14$ sq. in

10 - 1 1/8" ⌀ bars.

Section 5 ft. right of 2.

$M = +27,500,000$ in-lb

$\frac{M}{bd^2} = \frac{27,500,000}{253,920} = 108.3$

$p = .008$ $A_s = 2760 \times .008 = 22.08$ sq. in

use 18 - 1 1/8" ⌀ bars.

Section 3 feet to left of 4

$M = -15,000,000$ in-lb

$p = .0044$ $A_s = 2760 \times .0044 = 12.14$ sq. in

10 - 1 1/8" ⌀ bars.

Section. 5 feet right of 4

$M = -13,250,000$ in-lb

$\frac{M}{bd^2} = \frac{13,250,000}{253,900} = 52.$

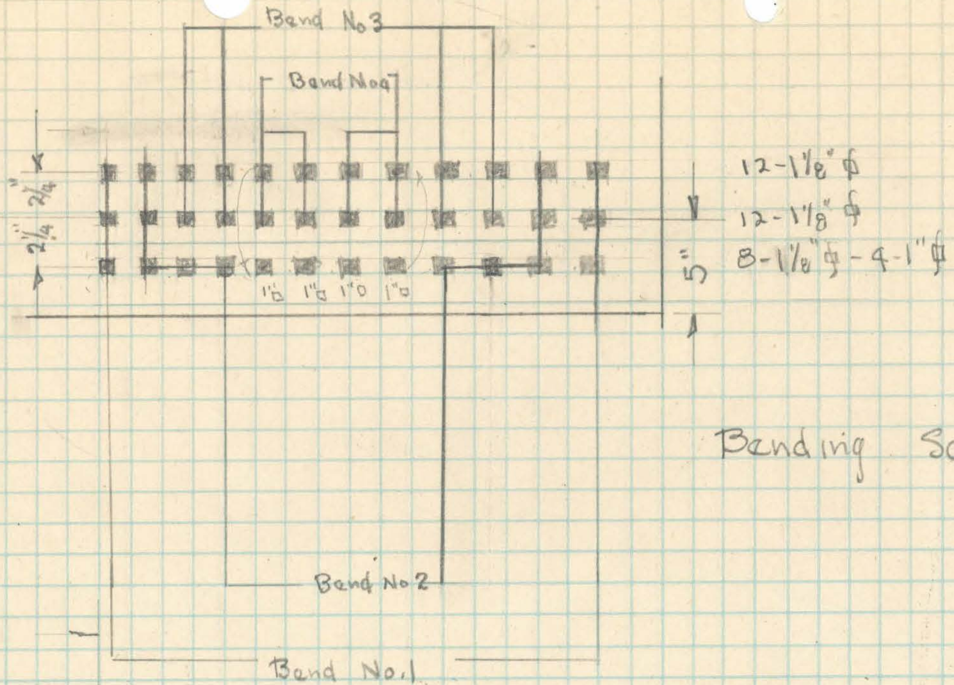
$p = .004$ $A_s = 2760 \times .004 = 11.04$ sq. in

use 9 - 1 1/8" ⌀ bars.

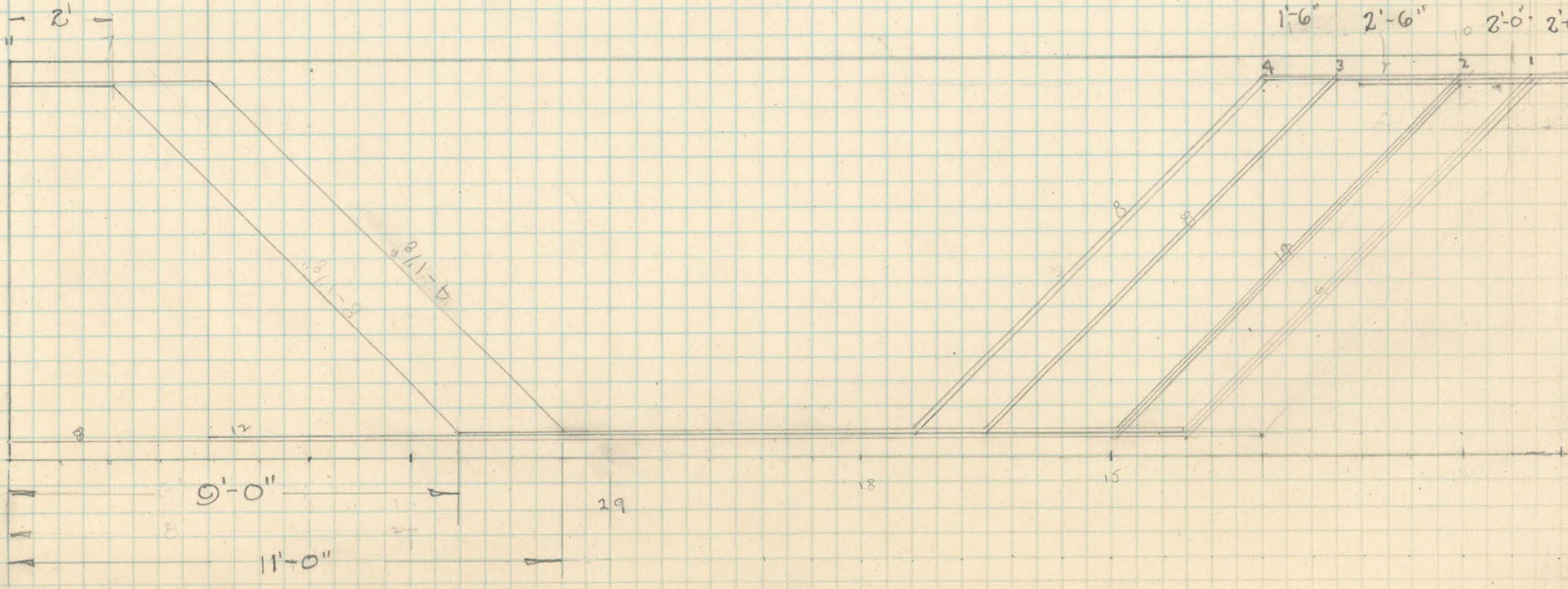
Design of Balcony Girder.

Steel at 4.	$A_s = 44.16$ sq. in. use 33 - $1\frac{1}{8}$" ϕ and 3 - 1" ϕ bars. These bars to be placed in 3 - layers. $2\frac{1}{4}$" c.c.
Steel at 5.	$A_s = 11.04$ in use 9 - $1\frac{1}{8}$" ϕ bars.
Steel at 3	$A_s = 18.2$ in. - use 15 - $1\frac{1}{8}$" ϕ bars.
Steel at 2.	$A_s = 35.88$ in. use 29 - $1\frac{1}{8}$" ϕ bars.
Steel at 1	$A_s = 13.8$ sq. in use 11 - $1\frac{1}{8}$" ϕ bars.

steel over 4.
12-1⅛" ϕ
12-1⅛" ϕ
8-⅛" ϕ · 4-1" ϕ



Bending Schedule for Span No. 1.



Unit shear at 1

$\frac{250,943}{30 \times 92} = 90. \#/sq. in.$

Unit shear at 2.

$\frac{207,743}{30 \times 92} = 75 \#/sq. in.$

Total shear between 1 and 2

$\frac{50+35}{2} \times \overset{15}{30} \times 12 \times 12 = 85 \times 15 \times 144 = 184,000$ lb.

$\frac{184,000}{16000} =$ 11.5 sq. in. of Steel in Stirrups.

Unit shear 4 feet right of 1

$\frac{225,000}{30 \times 92} = 81.5 \#/sq. in$

Unit Shear 8 feet from 1

$\frac{215,000}{30 \times 92} = 77.5 \#/sq. in$

Total shear in section 4 feet from 1

$\frac{50+41.5}{2} \times \overset{15}{30} \times 4 \times 12 = 91.5 \times 15 \times 4 \times 12 = 65,800$ Lb.

$\frac{65,800}{16,000} = 4.1$ sq. in.

use 6 - 5/8" ⌀ Stirrupes from 1 to a point 4 feet to the right of 1. spaced 8" c.c.

Total shear from point 4 feet to 8 feet.

$\frac{41.5+37.5}{2} \times 30 \times 4 \times 12 = 79 \times 15 \times 48 = 56,900$ lb

$\frac{56,900}{16000} = 3.55$ sq. in. of steel.

use 5 - 5/8" ⌀ Stirrupes from 4 feet to 8 feet point.

Spaceing. 9"- 9"- 10"-10"-10".

Shear from 8 feet pt. to 12 feet pt.

$\frac{37.5+35}{2} \times \overset{15}{30} \times 4 \times 12 = 72.5 \times 15 \times 48 = 52,100$ lb

$\frac{52,100}{16000} = 3.27$ sq. in of Steel.

use 4- 5/8" ⌀ Stirrupes. Spaced 12" c.c.

Unit shear at. 2.	$\frac{82,080}{30 \times 92} = 30$ lb / sq. in
unit shear at 3.	$\frac{118,000}{30 \times 92} = 43$ lb/sq. in.
Unit shear at 4.	$\frac{193,450}{30 \times 92} = 70$ lb. / sq. in.
Total shear between 3 and 4.	$\frac{3+30}{2} \times 30 \times 10 \times 12 = 33 \times 15 \times 120 = 59,500$ lb.

$\frac{59,500}{16,000} = 3.72$ sq. in. of Steel.

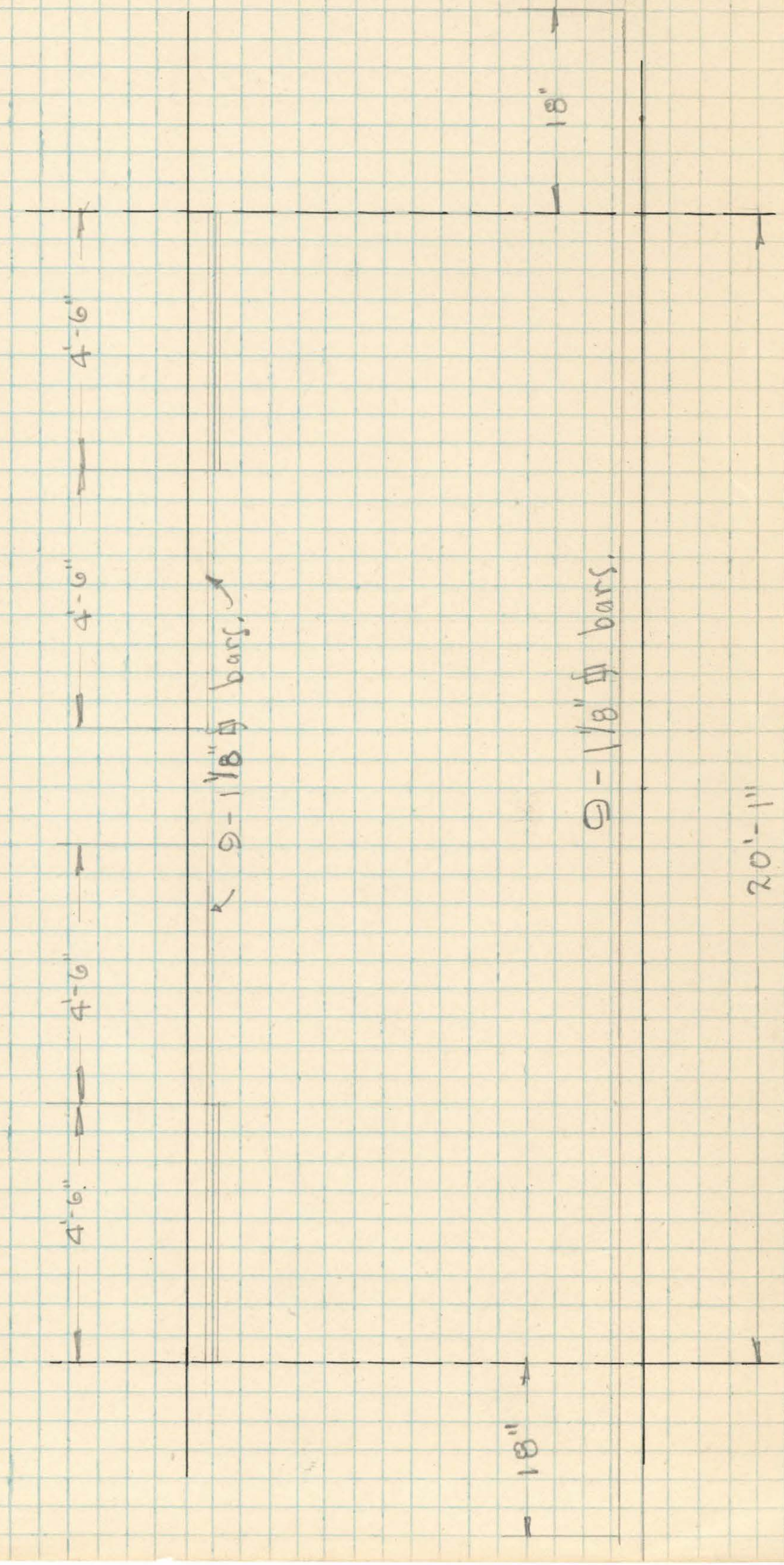
Use 9 - ½" ϕ Stirrupes.

spacing from 4 to the left.

10" - 10" - 12" - 14" - 14" - 14" - 14" - 16" - 16"

Spacing of Stirrupes and bending of Rods for Span No. 3 is similar to that of Span No 1.

Bending Schedule for Span No. 2.



Py.

Dead wt. = 72,711 lb.

Live Load = 120,000 lb.

192,711 lb.

increase this 20% for Bending

Total Load = 231,111 lb

fc

$$fc = 800\left(1.6 - \frac{60 \times 12}{25 \times 24}\right) = 800(1.6 - 1.2) = 800 \times .4$$

$fc = 320$ #/sq. in.

$P' = \pi \times 144 \times 320 = 145,000$ lb.

$$\frac{P}{P'} = \frac{231}{145} = 1.60$$

$$1.60 = 1 + 14p$$

$$p = \frac{.6}{14} = .0428$$

$A_s = \pi \times 144 \times .0428 = 19.4$ sq. in

use 13 - 1¼" ф bars.

$\frac{7}{16}$" ф spiral 2½" Pitch

Outside Dimensions = 28" x 28".