DESIGN

OF A

REINGORCED CONCRETE HIGHWAY THROUGH ARCH BRIDGE

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SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF
B.S. IN CIVIL ENGINEERING FROM THE
ENGINEERING SCHOOL OF ROBERT COLLEGE.
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I N D E X

PAGE

I N T R O D U C T I O N

The Highway Arch Bridge in question is designed for a particular location at the outside of the city of Manisa . The new structure is to replace the old wooden bridge . The two abutments of massive masonary will be left as they are , for they are still in good condition and strong enough to support the new structure . For the sake of appearance they will only be faced with cement mortar .

The bridge will be of bowstring type with two hinges , one of them being a roller , as it is best suited for the foundation conditions . The clear span of the bridge is 150 feet with a rise of 25 feet .

The roadway was designed for a double lane traffic and is 18 feet clear from curb to curb . There are two 5 feet sidewalks , both cantilivered out .

Throughout the design the "Standard Specification for Steel Highway Bridges" of the United States department of agriculture (Department Bulletin No. 1259) were followed . The H-15 standard truck loading was adopted for both the floor and the supporting rib .

Impact of 30% was used for the floor system and the following formula for the arch rib :

$$I\ (\%) = \frac{L + 250}{10\ L + 500}$$

where L = Total loaded length

The 1928 joint Standard Building Code was used for the concrete and steel stresses . A 25 % increase of the allowable unit stresses is permitted for a combination of dead , live and impact stresses . The temperature stresses are completely neglected for the reason that is discussed in the theory part .

Both the preliminary and final designs of the arch rib were done by the method of least work .

T H E O R Y

An arch is any structure which develops horizontal reactions under vertical loads . Arches may be classi - fied as hinged and hingless arches . Hingless arches are those having fixed ends while hinged arches are those having hinges at the two ends or at the crown . The advantage of hinged arches over the hingless arches is that m the arch rib is better analyzed and thus makes possible a saving of material .

Arches may also be classified as deck arches and m through or bowstring arches . In the former the roadway is over the arch and in the latter the roadway is hanged from the arch ribs by means of verticals .

Many engineers have considered the bowstring-girder type in reinforced concrete to be a somewhat questionable type of construction on account of the greater difficulty in constructing the forms , in depositing the concrete m and in making connections in the reinforcement ; and also because some members are subjected to direct tensile stress .

In spite of these disadvantages , however , there are many cases where such types are more economical than solid webbed girders . They are , of course , never preferred to the arch and their use is limited to those cases where conditions make arch construction impracticable on account of limited underclearance over waterways , highways or railways or where unstable foundation conditions exist . In short the through or bowstring girder type of arch bridge is used at level ground where the bed rack is far and where there is a possibility of lateral movement of the abutments ; because when a bowstring - girder type of arch is used with one reaction fixed and the other rollers , the abutments at both sides take only vertical loads . Since there is not any hori - zontal reaction , there won't be any force to cause lateral movement of the abutments .

In through arches with one end fixed and the other on rollers , the stresses due to temperature are always neglected . Because it is a well known fact that the influence of heat upon large sectional areas is less than upon smaller areas ; the low conductivity of concrete is a quality which has not been taken sufficiently into consideration . A concrete bridge is heated principally on its exterrior surface. On account of the low conductivity of concrete this heating does not apply to the whole body and the lower interrior temperatures are changed but little , if any . Hence the steel which is a good conductor may not be affected at all .

Furthermore one reaction of the bridge being on rollers and the other fixed , for the same rise in temperature in the roadway and arch rib , they both expand the same amount laterally , so that the arch rib does not change its shape to have stresses caused in it . There are small temperature stresses only when the arch rib and the roadway is heated unequally . But this being very rare and the concrete being a poor conductor small stresses due to temperature were not taken into consideration .

P A R T I

DESIGN OF THE FLOOR SYSTEM :

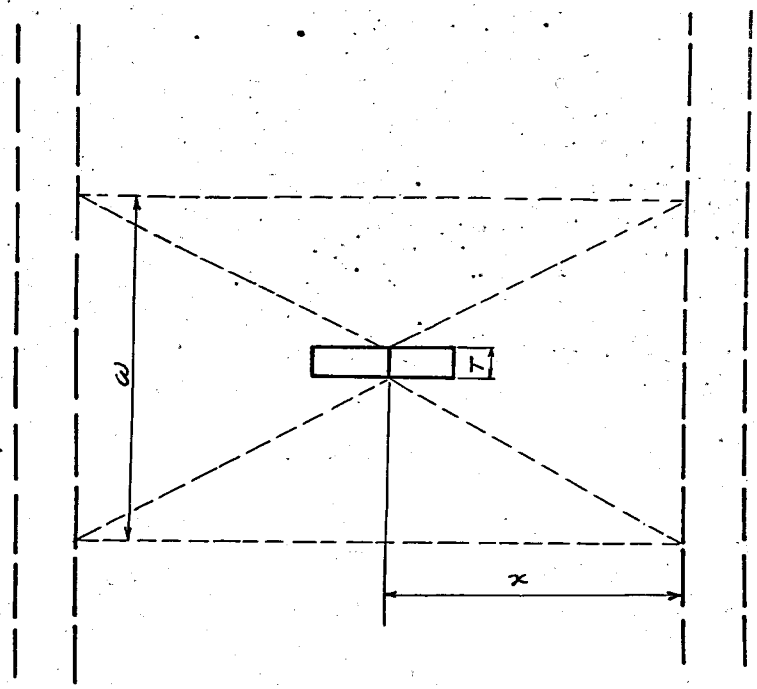
Before the final design of the floor system , four initial designs of the system were made to determine the distance between the floor beams to give the most economical design . For this purpose the square foot of form work was taken as 30 cents and a cubic yard of reinforced concrete as 25 dollars .

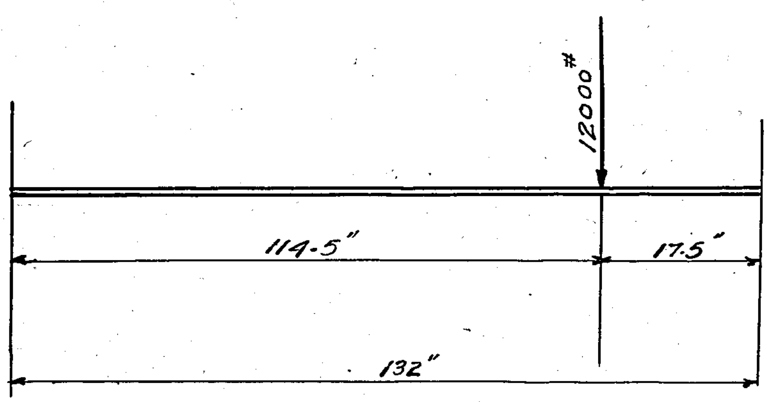
The results obtained are in tabulated form :

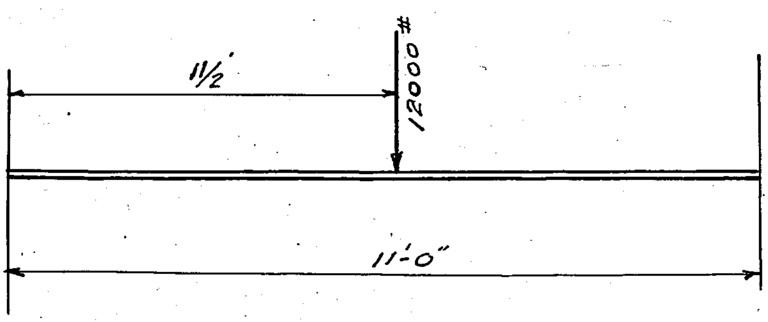
Number of Trials	Ist.	2nd.	3rd.	4th.
Distance between the centers of the floor beams	10'-0"	12'-6"	15'-0"	13'-0" (14'-3") at sides
Number of floor beams	16	13	11	12
Thickness of slab	7½"	8"	9"	8½"
Dimensions of floor beams .	16"x32"	18"x32"	18"x34"	18"x33"
Cost of form-work per lineal foot along the axis of the bridge	$ 32.1	$ 30.5	$29.38	$ 29.85
Cost of reinforced concrete per lineal foot along the axis of the bridge	$ 63.6	$ 64.35	$67.9	$ 66.20
Total Cost	$ 95.7	$ 94.85	$97.28	$ 96.05

From a comparison of the total costs in the above table we see that a distance of 12 feet 6" between the centers of the floor beams give the most economical design (or a clear distance of 11 feet between beams) .

Another design was made to see whether the use of stringers would give a more economical design than by using only floor beams without any stringers . It was found out that a design without the stringers is more economical .


w
T
x


12000#
114.5"
17.5"
132"


1½'
12000#
11'-0"

FINAL DESIGN OF THE FLOOR SYSTEM :

Roadway Slab :

Clear Span = 11' - 0"

Assume 8 inches slab .

$$\text{Weight of slab per foot} = \frac{8}{12} \times 150 = 100 \text{ \#/ft.}$$

$$\text{Weight of bituminous wearing} = 20 \text{ \#/ft.}$$

$$\text{TOTAL WEIGHT} = 120 \text{ \#/ft.}$$

Moment :

w = width over which wheel load is distributed

$$w = \frac{4}{3} x + T$$ (See the figure on opposite page)

$$w = \frac{4}{3} \times \frac{11}{2} + 1.25 = 8.58'$$

Use 6' as the effective width .

$$\text{Moment L.L.} = \frac{12000}{6} \times \frac{1}{2} \times \frac{11}{2} \times 12 \times \frac{8}{12} = 44000 \text{ in-\#}$$

$$30\ \%\ \text{Impact} = 44000 \times .30 = 13200 \text{ ""}$$

$$M_{d.l.} = \frac{1}{12} w l^2 = 120 \times 11^2 \times \frac{12}{12} = 14500 \text{ ""}$$

$$\text{TOTAL M} = 71700 \text{ "-\#}$$

$$d_M = \sqrt{\frac{71700}{131 \times 12}} = 6.75 \text{ "}$$

Use 8" slab (d = 7")

Shear :

I - Method - Punching Shear

$$2.5 \times 7 = 17.5''$$

$$w = \frac{4}{3} x + T \qquad \text{Where } x = 2.5\, d .$$

$$w = \frac{4}{3} \times 17.5 + 15'' = 38'' = 3.16'$$

Take $w = 3'$

For punching shear $v = 120$

$$S_{l.l.} = \frac{12000}{3} \times \frac{114.5}{132} = 3470 \;\#$$

$$S_{Impact} = 3470 \times .30 = 1040 \;\#$$

$$S_{d.l.} = \tfrac{1}{2} \times 120 \times 11 = 660 \;\#$$

TOTAL S = 5170

$$d_v = \frac{5170}{120 \times 7/8 \times 12} = 4.1''$$

Punching Shear does not govern .

II Method - D??agonal Tension :

taking $w = 6'$ (w of moment)

$$S_{l.l.} = \frac{12000}{6 \times 2} = 1000 \;\#$$

$$S_{Impact} = .30 \times 1000 = 300 \;\#$$

$$S_{d.l.} = \; = 660 \;\#$$

TOTAL S 1960 #

v for diagonal tension $= 40 \;\#/in^2$

$$d_v = \frac{1960}{40 \times 7/8 \times 12} = 4.67 ''$$

Diagonal tension does not govern .

Use 8" slab for roadway (d=7")

<u>Design of Steel :</u>

$$A_S = \frac{M}{f_s j\ d} = \frac{71700}{2000 \times 7/8 \times 7} = .586 \text{ sq."/ft of slab}$$

$$\text{Steel required per inch of slab} = \frac{.586}{12} = .0488 \text{ sq.in.}$$

Area of ½ ∅ bar = 0.1963 $in.^2$

$$\text{Spacing} = \frac{.1963}{.0488} = 4.02 \quad \text{Use 4" Spacing}$$

Alternate bars bent up at fifth points and run through to quarter points .

<u>Check for Bond :</u>

$$u = \frac{V}{\Sigma_o j\ d} = \frac{4}{12} \times \frac{1900}{1.57 \times 7 \times 7/8} = 65.8 \ \#/i$$

$$u = 65.8 \ \#/in.^2$$

O.K.

<u>Temperature Reinforcement :</u>

$$A_{St.} = x\ y\ .0025 = 8 \times 12 \times .0025 = .24 \text{ sq."/ft. of slab}$$

$$A = \frac{.24}{12} = .02 \ in^2/in. \text{ of slab}$$

Use 3/8" ∅ at 5" c.c.

DESIGN OF SIDEWALK SLAB :

Clear Span = 5' - 0" Assume 4" slab .

Weight of slab = 50 #/ft

Weight of bituminous

Wearing = 20 #/ft

TOTAL w = 70 #/ft l.l. w = 100 #/sq.ft

Shear :

$$S_{l.l.} = \tfrac{1}{2} w l = \tfrac{1}{2} \times 100 \times 5 = 250 \#$$

$$S_{d.l.} = \tfrac{1}{2} \times 70 \times 5 = 175 \#$$

TOTAL S = 425 #

$$d_v = \frac{425}{40 \times 7/8 \times 12} = 1.01"$$

Moment :

$$M = 1/8\ w\ l^2 = 1/8(70+100)\ 5^2 \times 12 = 6370 \text{ "}\#$$

$$d_m = \sqrt{\frac{6370}{131 \times 12}} = 2.01"$$

Use 4" slab (d = 2.5") ~~ 3"

DESIGN OF REINFORCEMENT :

$$A_s = \frac{.6370}{20000 \times 7/8 \times 2.5} = .146 \text{ sq. in./ft. of slab}$$

$$A_S = \frac{.146}{12} = .01215 \text{ sq. in / in. of slab}$$

Use 3/8" ∅ bars each ~~9"~~ 8" c.c.

TEMPERATURE REINFORCEMENT :

$$A_{St.} = 4 \times 12 \times .0025 = .12 \text{ sq. in. / ft. of slab}$$

$$A = \frac{.12}{12} = .01 \text{ sq. in / in. of slab}$$

Use 3/8" ⌀ bars each 11" c.c.

DESIGN OF BEAMS :

Outer Sidewalk Beams :

Assuming b = 12" of cantiliver beam

Clear Span = 12' - 6" - 12" = 11' - 6' = 11.5'

w = 170 #/sq.ft.

w = 170 x 5/2 = 425 #/ft

railing = 20 #/ft

beam(6" x 12") = 75

TOTAL W = 520 #/ft.

Moment :

$$M = 1/10\ w\ l^2 = 1/10 \times 520 \times \overline{11.5}^2 \times 12 = 82500\ ''\ \#$$

$$d_{-m} = \sqrt{\frac{82500}{157 \times 6}} = 9.35\ '' \quad \text{O.K.}$$

Shear :

$$S = \tfrac{1}{2} \times 520 \times 11.5 = 2990\ \#$$

$$d_v = \frac{2990}{60 \times 7/8 \times 6} = 9.5'' \quad \text{O.K.}$$

Use 6" x 12" beam with special reinforcement .

(d = 10)

Design of Steel :

$$A_S = \frac{82500}{20000 \times 7/8 \times 10} = .471$$

Use 2 - ½" square bars for reinforcement with special anchorage .

No stirrups are necessary .

Design of Cantilevers :

Clear Span = 5' - 3" = 5.25'

D.L. Railing and concentration = 200 #

Outer side slab l.l. concentration 513 x 12.5 = 6420 #

Total P = 200 + 6420 = 6620 #

M = 6620 x 5.25 x 12 = 417,000 " #

S = 6620 (At the end) = 6620 #

$M_{(d.l.)}$-the cantilivered end $= \frac{1}{2} w l^2$ =

$= \frac{1}{2} \times 250 \times 5.25^2 \times 12$ = 41,300 " #

$S_{d.l.}$ = 250 x 5.25 = 1310 #

Total Moment = 417000 + 41300 = 458300 " #

Total Shear = 6620 + 1310 = 7930 #

b = 12"

At the Cantilivered End :

$$d_m = \sqrt{\frac{458300}{131 \times 12}} = 17.1"$$

$$d_v = \sqrt{\frac{7930}{60 \times 7/8 \times 12}} = 12.6"$$

O.K. Since d = 30 at the cantilivered section.

At the End :

d is governed by shear only since moment is zero at the end .

$$S = 6620 \text{ lbs.}$$

$$d_V = \frac{6620}{60 \times 7/8 \times 12} = 10.5 \text{ inches}$$

Use 12" x 12"

Use 12" x 12" beam at the end .

12" x 28" beam at the cantilivered side .

Use special anchorage . No stirrups are necessary .

Use ½" Ø stirrups at 12" spacing .

Design of Steel :

$$A_S = \frac{458300}{20000 \times 7/8 \times 27} = .97 \text{ square in.}$$

Use	2 - 5/8" Ø	A = .61 sq.in.
	1 - 3/4" Ø	A = .44 sq.in.
		1.05 square inches

DESIGN OF FLOOR BEAMS :

Clear Span = 18' + 2 x 8" = 19' - 4" = 19.33'

Assume 18 x 32 beam

Weight of Slab 120 x 12.5 = 1500 #/ft

Stem $\frac{22}{12} \times \frac{18}{12} \times 150$ = 412 #/ft

TOTAL 1912 #/ft

Moment :

$M_{d.l.} = 1/8 \times 1912 \times \overline{19.33}^2 \times 12 = 1,073,000$ " #

$M_{l.l.} = 24(8.17 - 3)\ 12000 = 1,490,000$ " #

$M_{imp.} = 447,000$ " #

T O T A L M = 3,010,000 " #

$$d_M = \sqrt{\frac{3,010,000}{131 \times 9.6 \times 12}} = 14.07 \text{ inches}$$

Shear :

$S_{d.l.} = \frac{1}{2}\ w\ l = \frac{1}{2} \times 1912 \times 19.33 = 18500$ #

$S_{l.l.} = 24000$ #

$S_{Impact} = 7200$ #

49700 #

$$d_v = \frac{49700}{120 \times 7/8 \times 18} = 26.2 \text{ inches}$$

Since there shall be two rows of steel use 18" x 30" beam.

Design of Steel :

$$A_S = \frac{3,010,000}{20000 \times 7/8 \times 26} = 6.61 \text{ sq.in.}$$

Use 4 - 1" ∅ A = 3.14 sq.in.

4 - 1" sq. A = 4.00 sq.in.

7.14 sq.in.

Check For Bond :

$$w = \frac{49700}{28.75 \times 7/8 \times 26} = 76.5 \text{ \#/in}^2 \quad 7120$$

O.K.

Use special anchorage . ?

Distance from cenmter line , where to bend 2 - 1" square bars

$$x = \sqrt{\frac{9.75^2 \times 2}{7.14}} = 5.16' \text{ or } 5' - 2"$$

Distance from center line of beam where to bend the other 2 - 1" square bars

$$y = \sqrt{\frac{9.75^2 \times 4}{7.14}} = 7.3' \text{ or } 7' - 3"$$

To find whether stirrups are necessary at the bent up bars

9.75' - 7.25' = 19"

$$\frac{77 \times 98}{117} = 64.5 \text{ \#/in}^2$$

64.5 x 3/4 x 18 x 22 2 = 27100 #

Stress in one bar = $\frac{27100}{2}$ = 13,550 #

13550 is smaller than 16000

Therefore stirrups are not necessar at bend up bars.

Minimum spacing of stirrups = 4"

Maximum spacing of stirrups = 15"

Use :

1	@	2"	=	2"
2	@	4"	=	8"
1	@	19"	=	19"
1	@	5"	=	5"
1	@	21"	=	21"
2	@	8"	=	16"
3	@	11"	=	33"
1	@	13"	=	13"
	S U M		=	117"

O.K.

Use 5/8" ∅ stirrups

DEAD LOAD STRESSES IN VERTICALS :

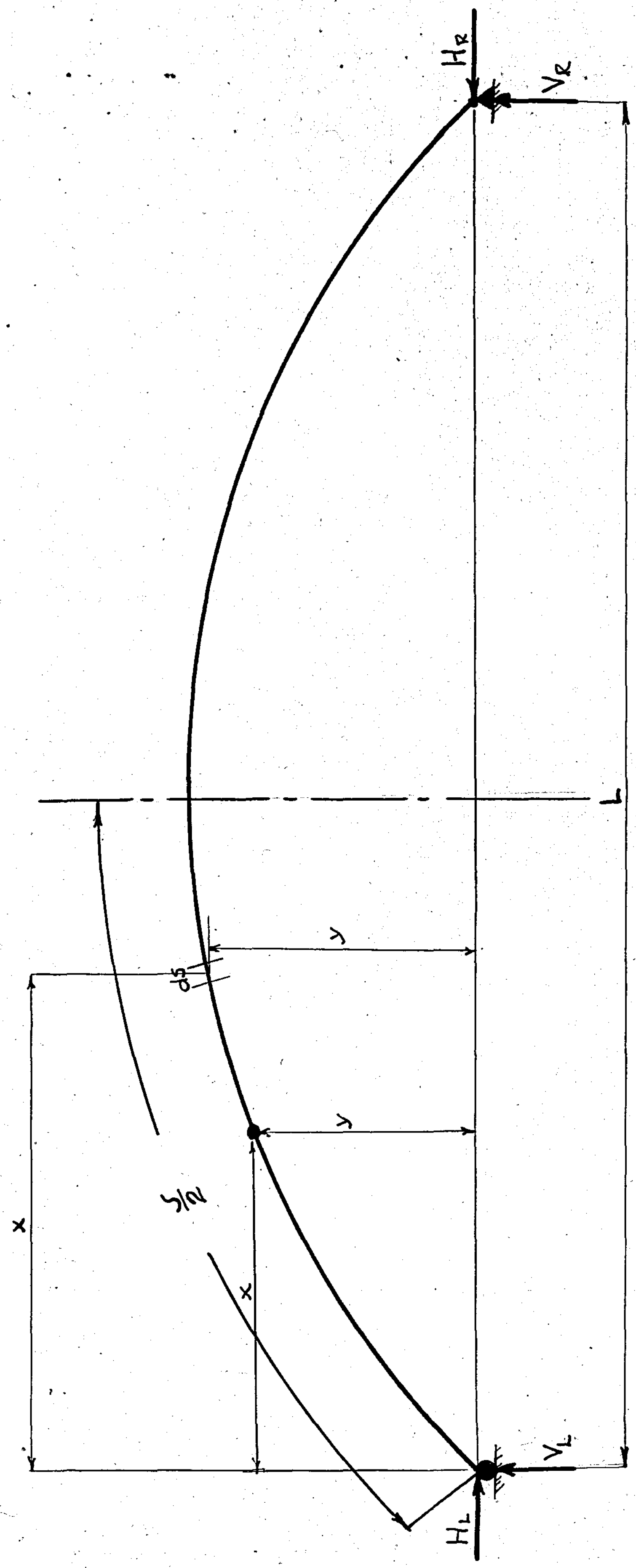
End reaction of floor beam		=	18900	#
Cantilever beam	6.5 x 21/12 x 12/12 x 150	=	1710	#
Outer beam	12.5 x 12/12 x 6/12 x 150	=	940	#
Railing	12.5 x 40	=	500	#
Slab(Side walk)	70 x 5 x 11	=	3850	#
Tension chord	Curb tension verticals	=	4000	#
	T O T A L T	=	29900	#

Take d.l. T = 30,000 #

Live Load stresses in verticals (For the design of arch rib) :

Weight of 15 ton truck = 24,000 #

Live Load on side walk slab = 5,500 #


H_R
V_R
L
y
ds
y
x
S/2
x
V_L
H_L

Derivation of the Least Work Formula Used in the Desin of the Arch Rib :

(See figure on the opposite page)

Let m_L = numarical value of moment at any point on left half of arch due to down ward loads applied between that point and springing assuming left half of arch to act as a cantilever beam fixed at the crown .

m_R = similar value for right half

$H_L = H_R = H \qquad E_T = E\,K$

$M_L = V_L\,x - H\,y - m_L$

$M_R = V_R\,x - H\,y - m_R$

d_s = length of one segment

General Case :

D.L. + L.L. on one side(this is quite general for any loading) .

$$W = \int_0^{s/2} \frac{M_L^2\,d\,s}{2\,E\,I} + \int_0^{s/2} \frac{M_R^2\,d\,s}{2\,E\,I} + 2\int_0^{s/2} \frac{H^2\,d\,s}{2\,A\,E} + \frac{H^2\,L}{2\,A_T\,E_T}$$

$$\frac{\partial w}{\partial H} = 0 = \int_0^{s/2} \frac{M_L\,d\,s}{E\,I} \cdot \frac{\partial M_L}{\partial H} + \int_0^{s/2} \frac{M_R\,d\,s}{E\,I} \cdot \frac{\partial M_R}{\partial H} + 2\int_0^{s/2} \frac{H\,d\,s}{A\,E} + \frac{H\,L}{A_T\,E_T}$$

$$0 = \int_0^{s/2} \frac{(V_L x - H y - m_l)}{E I}(-y)\, d s + \int_0^{s/2} \frac{(V x - Hy - m_R)}{E I}(-y)\, ds$$

$$+ 2 \int \frac{H d s}{A E} + \frac{H L}{A_T E_T}$$

$$H = \frac{\int \frac{V_L x y}{E I} d s + \int \frac{V x y}{E I} d s - \int \frac{m_l y}{E I} d s - \int \frac{m_R y}{EI} d s}{2 \int \frac{y^2 d s}{E I} + 2 \int \frac{d s}{A E} + \frac{L}{A_T E_T}}$$

$$H = \frac{\Sigma \frac{V_L xy}{I} + \Sigma \frac{V_R xy}{I} - \Sigma \frac{m_L y}{I} - \Sigma \frac{m_R y}{I}}{2 \Sigma \frac{y^2}{I} + 2 \Sigma \frac{I}{A} + \frac{L}{A_T K d s}}$$

(When all summations are for one half of the arch)

$$H = \frac{(V_L \;\; V_R) \Sigma \frac{xy}{I} - \Sigma \frac{m_L y}{I} - \Sigma \frac{m_R y}{I}}{2 \Sigma \frac{y^2}{I} + 2 \Sigma \frac{I}{A} + \frac{L}{A_T K d s}}$$

DESIGN OF THE ARCH RIB :

Assume 28" x 52"

Assume 28 - $1\frac{1}{4}$" square bars .

For the purpose of design the arch was drawn to scale on a milimeter paper and the values of x and y were measured to scale .

The arch rib is divided into 15 equal sections as shown on the sketch on the opposite page . The I was assumed to be constant .

$$I \text{ of concrete } \frac{1}{12} b h^3 = \frac{1}{12} \times 2.33 \times 4.33^3 = 15.80 \text{ ft}^4$$

$$I \text{ of steel } \frac{14 \times 14 \times 1.5625}{144} \times \frac{23.5^2}{144} = 8.17 \text{ ft}^4$$

$$I \text{ of '' } \frac{14 \times 14 \times 1.5625}{144} \times \frac{19.75^2}{144} = 5.76 \text{ ft}^4$$

$$\text{TOTAL I} = 29.73 \text{ ft}^4$$

$$A \text{ of concrete } = \frac{28}{12} \times \frac{52}{12} = 10.12$$

$$A \text{ of steel } = \frac{14 \times 28 \times 1.5625}{144} = 4.26$$

TOTAL A = 10.12 + 4.26 = 14.38 feet square

$$\frac{1}{A} = \frac{1}{14.38} = .0695 \ 1/\text{ft}^2$$

d s = 5.4 feet

$$W \text{ of each section} = 5.4 \times \frac{28}{12} \times \frac{52}{12} \times 150 = 8.2 \text{ kipd}$$

TABLE SHOWING THE VALUES OF Y , X AND THE TERMS THAT ARE NECESSARY IN THE LEAST WORK FORMULA

$$I = 29.73 \ ft^4$$

$$\frac{1}{A} = .0695 \ 1/ft^2 \qquad \frac{1}{A} = 15 \times .0695 = 1.043$$

$$= 1.043 \ 1/ft^2$$

SECTION	Y ft.	X ft.	XY ft^2	Y^2 ft^2	Y^2/I $1/ft^2$	XY/I $1/ft^2$	Y/I $1/ft^3$
1	25.0	73.0	1825	625	21.03	62.3	.854
2	24.5	66.5	1630	600	20.20	55.7	.836
3	24.0	60.5	1453	576	19.40	49.6	.82
4	23.0	54.5	1253	529	17.80	42.8	.785
5	22.0	48.5	1067	484	16.30	36.4	.751
6	20.5	43.0	882	420	14.13	29.67	.700
7	19.0	37.5	713	361	12.15	24.00	.648
8	17.5	32.5	569	307	10.33	19.14	.597
9	15.5	27.0	418	241	8.11	14.10	.529
10	13.0	22.0	286	169	5.69	9.62	.444
11	10.5	17.5	184	105	3.53	6.28	.358
12	8.5	13.5	115	72	2.42	3.93	.2905
13	6.0	9.5	57	36	1.21	1.92	.205
14	4.0	5.5	22	16	.54	.75	.137
15	1.5	2.0	3	2	.07	.10	.051
					152.91	356.31	

TABLE SHOWING THE VALUES OF $\frac{Y}{I} m_L$ & $\frac{Y}{I} m_R$

$$\frac{Y}{I} m_L = \frac{Y}{I} m_R$$

SECTION	$m'_1 = m'_R$ ft-kips	$m''_1 = m''_R$ ft-kips	$m_1 = m_R$	$\frac{Y}{I}$	$\frac{Y}{I} m_R$
15	0	0	0	.051	0
14	38	0	38	.137	5.21
13	107	0	107	.205	21.95
12	219	72	291	.2905	84.5
11	386	225	611	.358	219
10	592	375	967	.444	430
9	828	666	1494	.529	790
8	1125	975	2100	.597	1253
7	1470	1372.5	2843	.648	1845
6	1867	1858.5	3726	.700	2610
5	2305	2370	4675	.751	3510
4	2760	2970	5730	.785	4500
3	3290	3618	6908	.82	5670
2	3870	4400	8270	.836	6910
1	4480	5210	9690	.854	8275
					36123.66

DETERMINATION OF H DUE TO DEAD LOAD ONLY :

$$\frac{L}{A_T K ds} = \frac{151.5}{15 \times 5.4 A_T} = \frac{1.87}{A_T} = \frac{1.87 \times 144}{30} = 9$$

(Assuming 30 1"sq. bars A = 30)

$$H = \frac{133253}{316.9} \doteq 421,000 \ \#$$

V = 288000 (Under Dead Loads only)

TABLE SHOWING THE VALUE m_R WHEN UNIT LOAD IS AT DIFFERENT POINTS

When 1 kip load is at	F	E	D	C	B	A
SECTION	m_1.	m_1.	m_1.	m_1.	m_1.	m_1.
15	0	0	0	0	0	0
14	0	0	0	0	0	0
13	0	0	0	0	0	0
12	2.4	0	0	0	0	0
11	7.5	0	0	0	0	0
10	12.5	0	0	0	0	0
9	17.35	4.85	0	0	0	0
8	22.50	10.00	0	0	0	0
7	27.75	15.25	2.75	0	0	0
6	33.15	20.65	8.15	0	0	0
5	38.50	26.00	13.5	1.0	0	0
4	43.50	31.00	18.5	6.0	0	0
3	48.90	36.40	23.9	11.4	0	0
2	53.35	41.85	29.35	16.85	4.35	0
1	59.75	47.25	34.75	22.25	9.75	0

TABLE SHOWING THE VALUES OF $M_R \frac{Y}{I}$ DUE TO UNIT LOAD

SECTION	$\frac{Y}{I}$	F $M_R \frac{Y}{I}$	E $M_R \frac{Y}{I}$	D $M_R \frac{Y}{I}$	C $M_R \frac{Y}{I}$	B $M_R \frac{Y}{I}$
12	.2905	.698	0	0	0	0
11	.358	2.685	0	0	0	0
10	.444	5.55	0	0	0	0
9	.529	9.17	2.56	0	0	0
8	.597	13.43	5.97	0	0	0
7	.648	18.00	9.9	1.78	0	0
6	.700	23.20	14.47	5.71	0	0
5	.751	28.90	19.53	10.14	.751	0
4	.785	34.20	24.35	14.53	4.72	0
3	.82	40.10	29.84	19.60	9.35	0
2	.836	44.60	35.00	24.53	14.08	3.64
1	.854	51.06	40.30	29.66	19.00	8.33
SUMMATION		272.593	181.92	105.95	47.901	11.97

INFLUENCE TABLE FOR $\bar{V}$:

When load is

at F $\quad V_L = \dfrac{138.25}{151.5} = .913 \qquad V_R = .087$

E $\quad V_L = \dfrac{125.75}{151.5} = .830 \qquad V_R = .170$

D $\quad V_L = \dfrac{113.25}{151.5} = .748 \qquad V_R = .252$

C $\quad V_L = \dfrac{100.75}{151.5} = .665 \qquad V_R = .335$

B $\quad V_L = \dfrac{88.25}{151.5} = .582 \qquad V_R = .418$

A $\quad V_L = \dfrac{75.75}{151.5} = .500 \qquad V_R = .500$

VALUES OF H WHEN UNIT LOAD IS :

AT F $\quad H = \dfrac{356.31 - 272.59}{316.9} = \dfrac{83.72}{316.9} = .264$

E $\quad H = \dfrac{356.31 - 181.92}{316.9} = \dfrac{174.39}{316.9} = .551$

D $\quad H = \dfrac{356.31 - 105.95}{316.9} = \dfrac{250.36}{316.9} = .788$

C $\quad H = \dfrac{356.31 - 47.901}{316.9} = \dfrac{308.409}{316.9} = .975$

B $\quad H = \dfrac{356.31 - 11.97}{316.9} = \dfrac{344.34}{316.9} = 1.088$

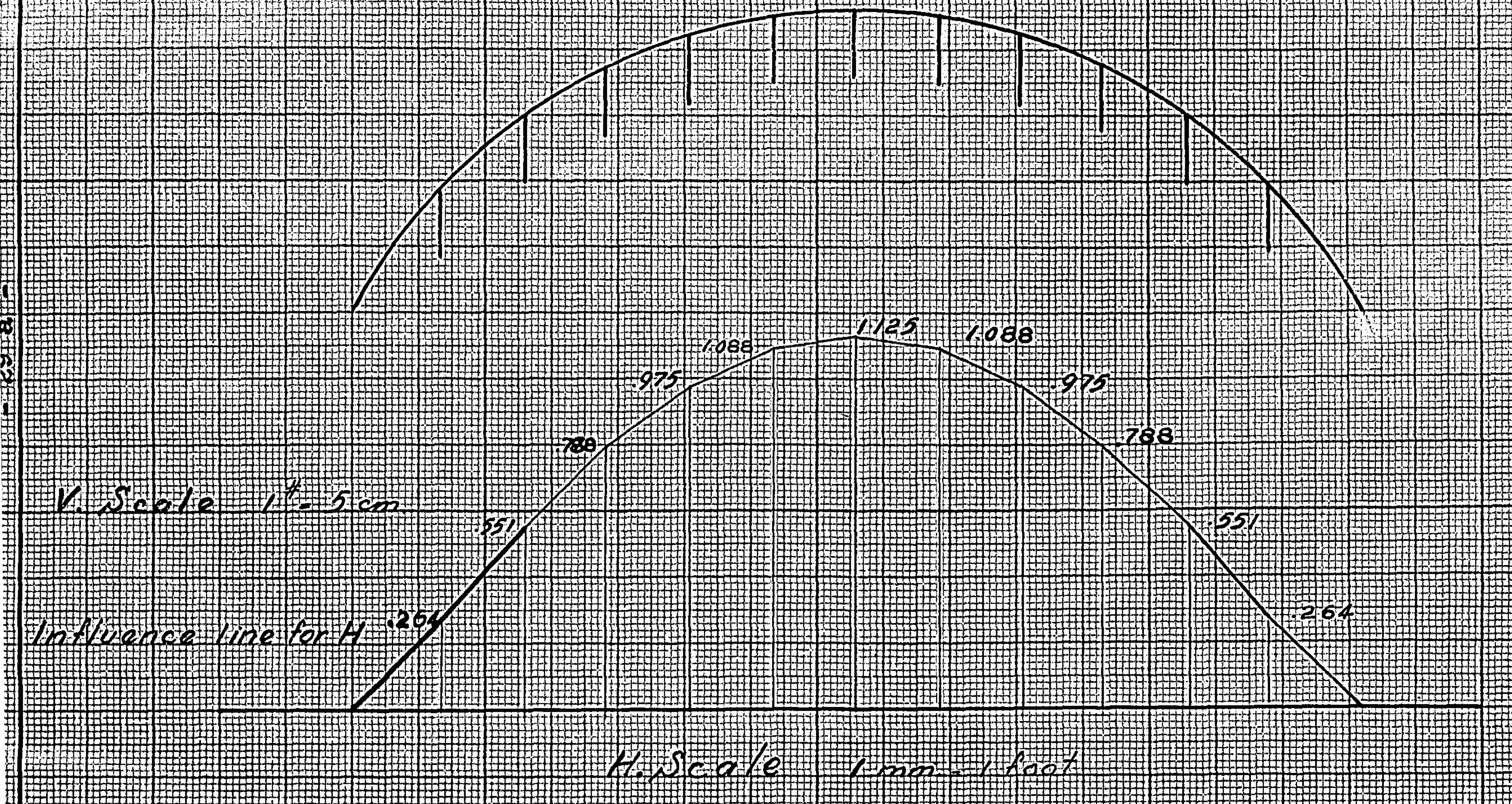
A $\quad H = \dfrac{356.31}{316.9} = \dfrac{356.31}{316.9} = 1.125$

INFLUENCE TABLE

FOR

H , V_L , V_R , $M_{\frac{1}{4}}$, $M_{\frac{1}{2}}$, $M_{\frac{3}{4}}$, M_c

	F	E	D	C	B	A	B'	C'	D'	E'	F'
H	.264	.551	.788	.975	1.088	1.125	1.088	.975	.788	.551	.264
V_L	.913	.830	.748	.665	.582	.5	.418	.335	.252	.170	.087
V_R	.087	.170	.252	.335	.418	.5	.582	.665	.748	.830	.913
$M_{\frac{1}{4}}$	8.42	9.25	4.94	1.15	-1.76	-3.75	-4.85	-5.10	-4.48	-3.26	-1.45
$M_{\frac{1}{2}}$	4.74	8.58	12.97	6.18	0.85	-2.95	-5.35	-6.32	-5.83	-4.30	-1.86
$M_{\frac{3}{4}}$	1.97	3.00	5.20	8.60	7.30	1.70	-1.45	-4.10	-4.38	-3.44	-1.38
M_c	-0.01	-0.9	-0.6	+0.99	3.90	9.73	3.90	0.99	-0.6	-0.9	-0.01



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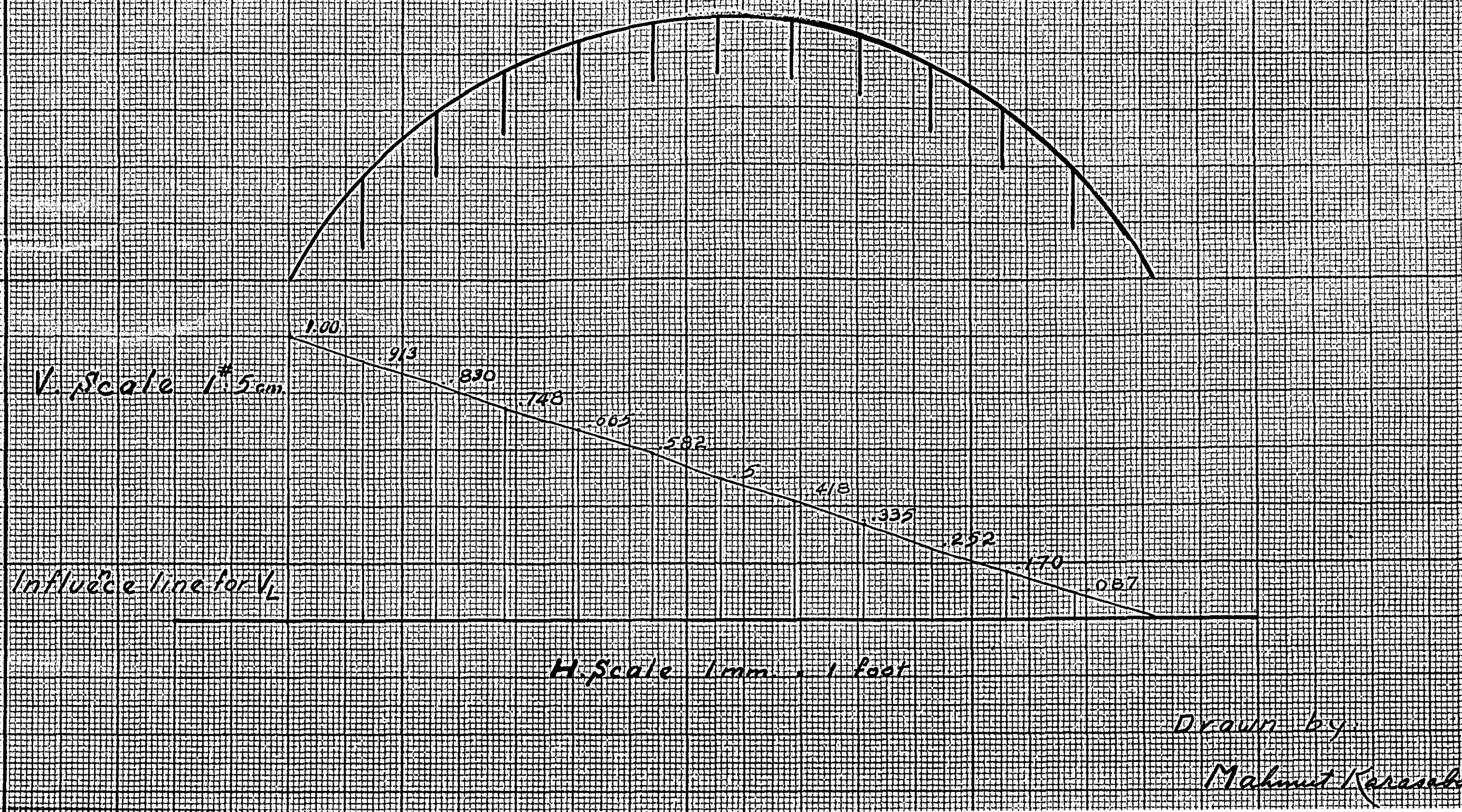
Mahmut Karasabey


V. Scale 1# 5cm.
Influence line for V_L
1.00
.913
.830
.748
.665
.582
.5
.418
.335
.252
.170
.087
H. Scale 1mm = 1 foot
Drawn by:
Mahmut Karasah

INVESTIGATION OF STRESS AT SPRINGING :

Maximum V and H occur when all the bridge is loaded . This can be seen from the influence lines of H and V .

d.l. H = 421000#

d.l. V = 288000 #

l.l. H = 8.457 (24 + 5.5) = 203 + 46.5 = 249.5 k

$$\text{H Impact} = 203 \times \frac{(151.5 + 250)}{1515 + 500} = 203 \times \frac{410.5}{2015} = 41.5 \text{ k}$$

TOTAL H = 421 + 249.5 + 41.5 = 912 kips.

l.l. V = $\frac{1}{2}$(24 + 5.5) = 132 + 30.3 = 162.3 kips

$$\text{V Impact} = 132 \times \frac{410.5}{2015} = 26.9$$

TOTAL V = 288 + 162.3 + 26.9 = 477.2 Kips

$$\text{Max. Thrust} = T_S = \sqrt{477.2^2 + 912^2} = 1070 \text{ kips}$$

$$f = \frac{P}{A}$$ since moment is zero

$$f = \frac{1{,}070{,}000}{14.38 \times 144} = 518 \ \#/\text{in}^2$$

O.K.

INVESTIGATION OF STRESS AT 1/4 POINT :

For this purpose we must , by means of influence lines , determine the maximum moment at 1/4 point.

$$\text{Value of } x = \frac{151.5}{8} = 18.95'$$

$$\text{Value of } y = \frac{4.7}{10} \times 25 = 11.75'$$

Influence Line for Moment at 1/4 Point :

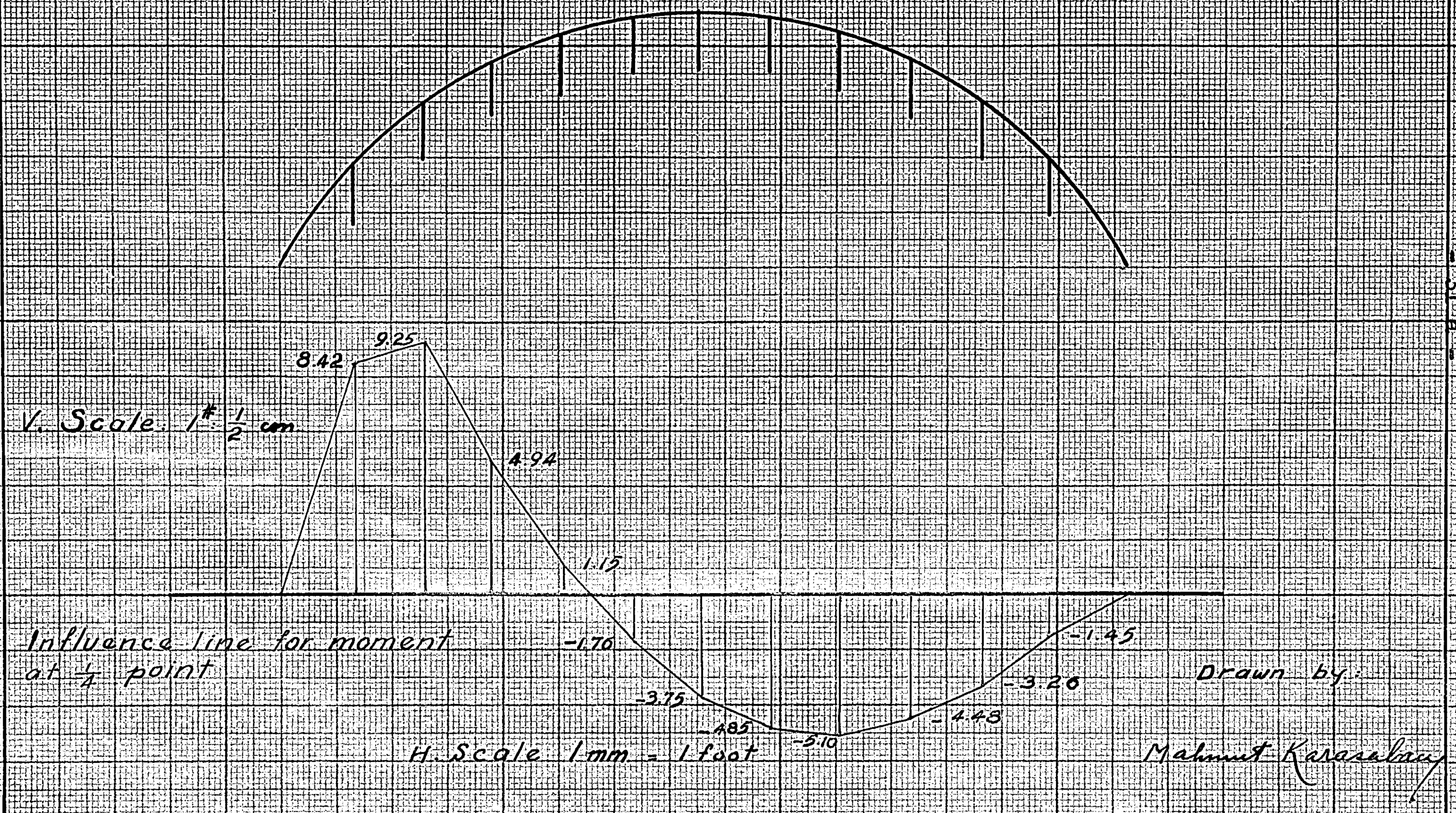
When unit load is at:

F	M = 132.5 x .087 - 11.75 x .264 = 11.52 - 3.1 = 8.4
E	M = 18.95 x .830 - 11.75 x .551 = 15.73 - 6.48 = 9.25
D	M = 18.95 x .748 - 11.75 x .788 = 14.20 - 9.26 = 4.94
C	M = 18.95 x .665 - 11.75 x .975 = 12.60 -11.45 = 1.15
B	M = 18.95 x .582 - 11.75 x 1.088 = 11.02 -12.78 = -1.76
A	M = 18.95 x .5 - 11.75 x 1.125 = 9.48 - 13.23 = -3.75
B'	M = 18.95 x .418 - 11.75 x 1.088 = 7.93 -12.78 = -4.85
C'	M = 18.95 x .335 - 11.75 x .975 = 6.35 -11.45 = -5.10
D'	M = 18.95 x .252 - 11.75 x .788 = 4.78 - 9.26 = -4.45
E'	M = 18.95 x .170 - 11.75 x .551 = 3.22 - 6.48 = -3.26
F'	M = 18.95 x .087 - 11.75 x .264 = 1.65 - 3.1 = -1.45

Sum of minus ordinates = - 24.65

Sum of plus ordinates = 23.76

Max. negative point = -24.65(24 + 5.5) = 592 + 135.5 = 727.5 ft-k.



8.42
9.25
V. Scale: 1#: 1/2 cm
4.94
1.15
Influence line for moment
at 1/4 point
-1.76
-3.75
-4.85
-5.10
-4.48
-3.20
-1.45
Drawn by:
H. Scale 1mm = 1 foot
Mahmut Karasalray

$$M.\ Impact = \frac{592(89+250)}{890+500} = \frac{592 \times 339}{1390} = 144$$

TOTAL Moment = 144 + 727.5 = 871.5 ft-k. or 871,500 ft-lbs.

The values of H and V when maximum moment occurs are :

$H_{l.l.} = 5.879(24 + 5.5) = 141 + 32.4 = 173.4$ kips

$$H\ Impact = \frac{141 \times 339}{1390} = 34.4$$

$H_{d.l.} = 421$ kips

TOTAL H = 628.8 Kips

$V_{l.l.} = 2.344(24 + 5.5) = 56.2\ \ 12.9 = 69.1$

$$V\ Impact = 56.2 \times \frac{339}{1390} = 13.7$$

$V_{d.l.} = 288$ kips

d.l. shear at 1/4 point = 288 - 30 - 5 x 8.2 = 217

TOTAL V = 69.1 + 13.7 + 217 = 399.8 kips or 400 kips

$$T \text{ at } 1/4 \text{ section} = \sqrt{\overline{400}^2 + \overline{629}^2} = 745000$$

$$C_s = 21.88 \times 15 \times \left(\frac{x - 4.37}{x}\right) f_c = 328\ f_c \left(\frac{x - 4.37}{x}\right)$$

$$C_c = \tfrac{1}{2}\ f_c \times 28\ x = 14\ x\ f_c$$

$$T = 21.88 \times 14 \times \left(\frac{52 - x}{x}\right) f_c = 306\ f_c \left(\frac{52 - x}{x}\right)$$

$$C_s \alpha = 328\ f_c \left(\frac{x - 4.37}{x}\right) \times 9.93$$

$$C_c \alpha = 14\ x\ f_c \left(14.3 - \frac{x}{3}\right)$$

$$T_a = 306\ f_c \left(\frac{52 - x}{x}\right) 33.33$$

$$325\ f_c \left(\frac{x - 4.37}{x}\right) + 200\ f_c\ x - \frac{14\ x^2\ f_c}{3} + 10200\ f_c \left(\frac{52 - x}{x}\right) = 0$$

$$-14\ \frac{x^3}{3} + 200\ x^2 - 9875\ x + 528580 = 0$$

$$x^3 - 43\ x^2 + 2120\ x - 113.500 = 0$$

$$x = 48$$

$$C_s = 328\ f_c \left(\frac{43.63}{48}\right) = 298\ f_c$$

$$C_c = 14 \times 48\ f_c = 672\ f_c$$

$$T = 306\ f_c \left(\frac{4}{48}\right) = 25.5\ f_c$$

$$745{,}000 - 298\ f_c - 672\ f_c + 25.5\ f_c = 0$$

$$f_c = \frac{745000}{944.5} = 788\ \#/in^2$$

O.K.

INVESTIGATION OF STRESS AT 1/2 POINT :

For this purpose we must , by drawing influence lines , determine the maximum moment at 1/2 point .

$$\text{Value of } x = \frac{151.5}{4} = 37.9$$

$$\text{Value of } y = \frac{7.8}{10} \times 25 = 19.5$$

Influence Line Moment at 1/2 Point :

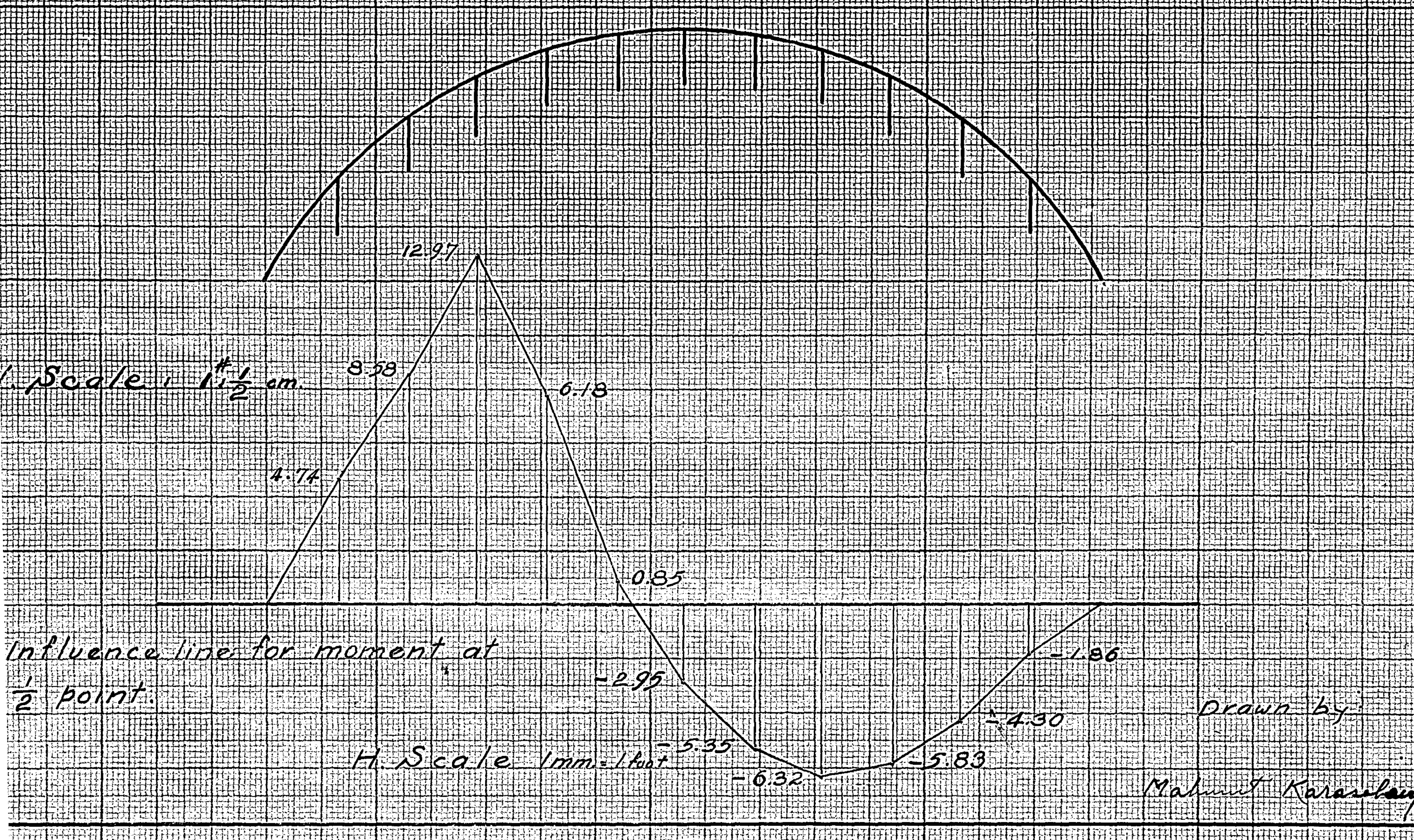
When unit load is

at : F	M	= .087 x 113.625 - .264 x 19.5 = 9.89 - 5.15	= 4.74
E	M	= .170 x 113.625 - .551 x 19.5 = 19.32 -10.74	= 8.58
D	M	= .748 x 37.875 - .788 x 19.5 = 28.35 -15.38	= 12.97
C	M	= .665 x 37.875 - .975 x 19.5 = 25.20 -19.02	= 6.18
B	M	= .582 x 37.875 -1.088 x 19.5 = 22.05 -21.2	= 0.85
A	M	= .5 x 37.875 -1.125 x 19.5 = 18.95 -21.93	= -2.98
B'	M	= .418 x 37.875 -1.088 x 19.5 = 15.85 -21.2	= -5.35
C'	M	= .335 x 37.875 - .975 x 19.5 = 12.70 -19.02	= -6.32
D'	M	= .252 x 37.875 - .788 x 19.5 = 9.55 -15.38	= -5.83
E'	M	= .170 x 37.875 - .551 x 19.5 = 6.44 -10.74	= -4.30
F'	M	= .087 x 37.875 - .264 x 19.5 = 3.29 - 5.15	= -1.86

Sum of minus ordinates = - 26.64

Sum of plus ordinates = 33.32

loaded distance = 63.5'


12.97
8.58
6.18
4.74
0.85
-2.95
-5.35
-6.32
-5.83
-4.30
-1.86
V. Scale : 1# = 1/2 cm.
Influence line for moment at 1/2 point.
H. Scale 1mm = 1 foot
Drawn by:
Mahmut Karaselay

Positive moment $= 33.32(24+5.5) = 800+180.3 = 980.3$ ft-kips

Moment Impact $= 800 \times \frac{(63.5+250)}{635+500} = \frac{800 \times 313.5}{1135} = 221$ ft-kip

TOTAL M $= 980.3+221 = 1201.3$ ft.-kips

The value of H and V when maximum moment occurs :

$H_{l.l.} = 3.666(24+5.5) = 88+20.2 = 108.2$ kips

H Impact $= 88 \times \frac{313.5}{1135} = 24.3$ kips

$H_{d.l.} = 421$ kips

TOTAL H $= 421+108.2+24.3 = 553.5$ Kips .

$V_{l.l.} = 3.728(24+5.5) = 89.5 \quad 20.5 = 110$ kips

V Impact $= \frac{89.5 \times 313.5}{1135} = 24.7$ kips

$V_{d.l.} = 288$ kips

d.l. shear at 1/2 point $= 288 - 2 \times 30 - 8.2 \times 8 = 168$ k.

l.l. shear at 1/2 point $= 134.7 - 2(24+5.5) = 75.7$ k

TOTAL SHEAR $= 243.7$ Kips

Thrust at 1/2 point $= \sqrt{553.5^2 + 243.7^2} = 605$ kips

$$\frac{M}{P} = e = \frac{1201.3}{605} = 2 \text{ feet}$$

$$C_s = 21.88 \times 15 \left(\frac{x - 4.37}{x}\right) f_c = 328 f_c \left(\frac{x - 4.37}{x}\right)$$

$$C_c = \tfrac{1}{2} f_c\, 28\, x = 14 f_c\, x$$

$$T = 21.88 \times 14 \left(\frac{52 - x}{x}\right) = 306 f_c \left(\frac{52 - x}{x}\right)$$

$$C_s a = 328 f_c \left(\frac{x - 4.37}{x}\right) 2.17 = 712 f_c \left(\frac{x - 4.37}{x}\right)$$

$$C_c a = 14 f_c\, x \left(\frac{x}{3} - 2.2\right)$$

$$T_a = 306 f_c \left(\frac{52 - x}{x}\right) 45.43 = 14000 f_c \left(\frac{52 - x}{x}\right)$$

$$712 \left(\frac{x - 4.37}{x}\right) 14\, x \left(\frac{x}{3} - 2.2\right) - 14000 \left(\frac{52 - x}{x}\right) = 0$$

$$712\, x - 3120 \frac{14x^3}{3} - 30.8\, x^2 - 152000 = 0$$

$$x^3 - 6.6\, x^2\, 3230\, x - 152000 = 0$$

$$x = 35.6$$

$$C_s = 328 f_c \left(\frac{x - 4.37}{x}\right) = 288 \; f_c$$

$$C_c = 14 \times 35.6 \; f_c \qquad = \underline{500}$$

$$788 \; f_c$$

$$= 306 \left(\frac{52 - x}{x}\right) f_c \qquad = \underline{141}$$

$$647$$

$$f_c = \frac{60500}{647} = 933 \;\#/in^2$$

O.K. since a variation of 25% of stress is allowable .

INVESTIGATION OF STRESS AT 3/4 POINT :

For this purpose we must , by drawing influence lines , determine the maximum moment at 3/4 point .

Value of x = 3/8 x 151.5 = 56.8

$$\text{Value of } y = \frac{9.5}{10} \times 25 = 23.75$$

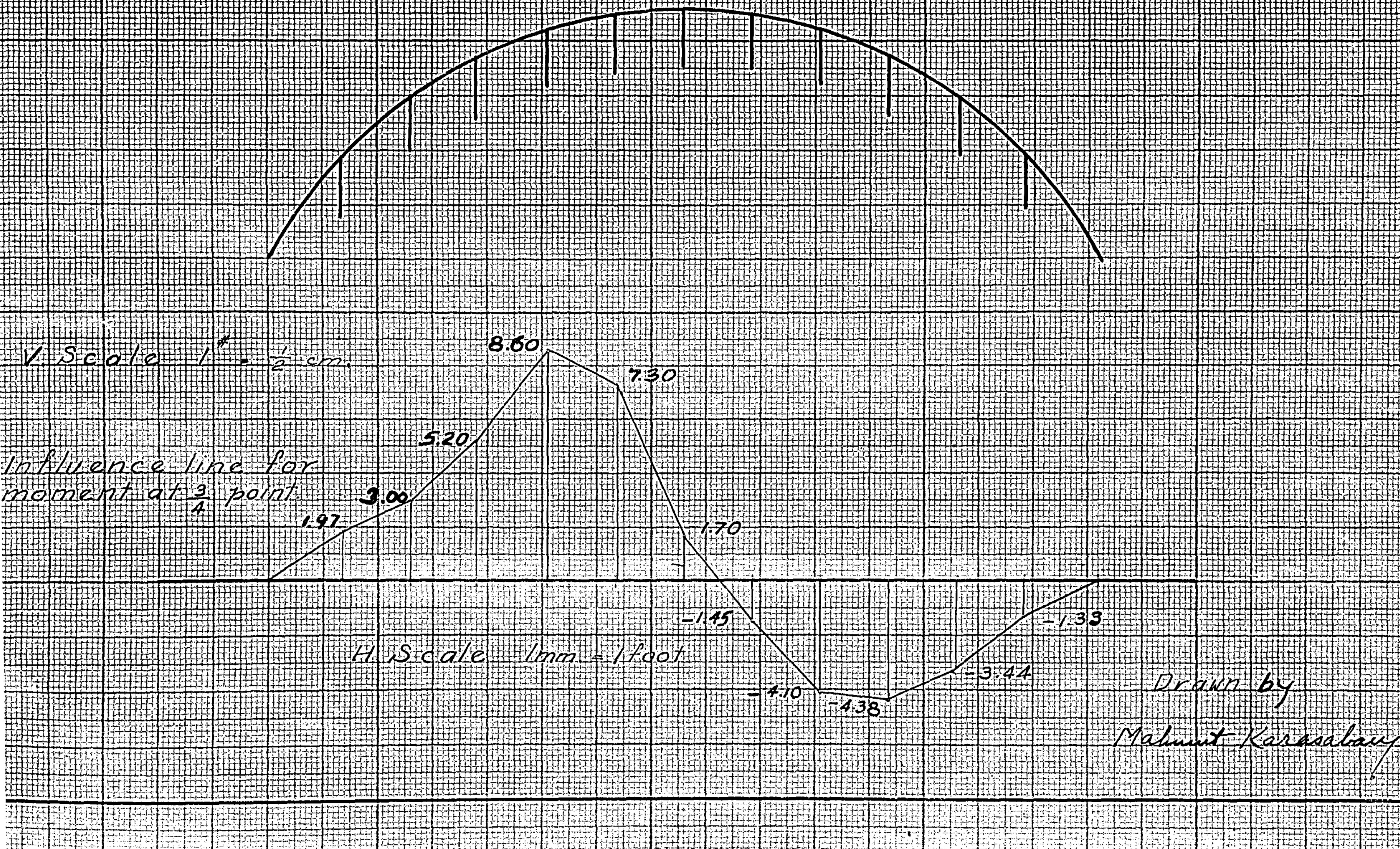
Influence Line for Moment at 3/4 Point :

When unit load is

at : F	M = .087 x 94.7 - 23.75 x .264	= 8.24 - 6.27	=	1.97
E	M = .170 x 94.7 - 23.75 x .551	= 16.10 - 13.10	=	3.00
D	M = .252 x 94.7 - 23.75 x .788	= 23.90 - 18.70	=	5.20
C	M = .335 x 94.7 - 23.75 x .975	= 31.75 - 23.15	=	8.60
B	M = .582 x 56.8 - 23.75 x 1.088	= 33.10 - 25.8	=	7.30
A	M = .5 x 56.8 - 23.75 x 1.125	= 28.40 - 26.7	=	1.70
B'	M = .418 x 56.8 - 23.75 x 1.088	= 24.35 - 25.8	=	-1.45
C'	M = .335 x 56.8 - 23.75 x .975	= 19.05 - 23.15	=	-4.10
D'	M = .252 x 56.8 - 23.75 x .788	= 14.32 - 18.70	=	-4.38
E'	M = .170 x 56.8 - 23.75 x .551	= 9.66 - 13.10	=	-3.44
F'	M = .087 x 56.8 - 23.75 x .264	= 4.94 - 6.27	=	-1.33

Sum of minus ordinates = - 14.70

Sum of plus ordinates = 27.77


V. Scale 1# = ½ cm.
Influence line for
moment at 3/4 point.
8.60
7.30
5.20
3.00
1.97
1.70
-1.45
-4.10
-4.38
-3.44
-1.33
H. Scale 1mm = 1 foot
Drawn by
Mahmut Karasabay

Since moment and shear at $\frac{3}{4}$ point will be less than the moment and shear at $\frac{1}{2}$ point this section won't govern, therefore investigation of f_c was omitted .

INVESTIGATION OF STRESSES AT CROWN :

INFLUENCE LINE WHEN UNIT LOAD IS

AT	F	= .087 x 75.75 - .264 x 25 = 6.59 - 6.60 =	- 0.01
	E	= .170 x 75.75 - .551 x 25 = 12.89 - 13.79 =	- 0.90
	D	= .252 x 75.75 - .788 x 25 = 19.1 - 19.7 =	- 0.60
	C	= .335 x 75.75 - .975 x 25 = 25.39 - 24.40 =	0.99
	B	= .410 x 75.75 - 1.088 x 25 = 31.1 - 27.2 =	3.90
	A	= .5 x 75.75 - 1.125 x 25 = 37.91 - 28.18 =	9.73

Sum of positive ordinates = 19.51

Loaded distance = 5 x 12.5 = 62.5

$M_{l.l.} = 19.51(24 + 5.5) = 469 + 107.5 = 576.5$ ft-kips

$$M_{imp.} = 469 \times \frac{62.5 + 250}{62.5 + 500} = 469 \times \frac{312.5}{1125} = 130$$

TOTAL M = 577 + 130 = 707 ft-kips = 707000 ft/lbs

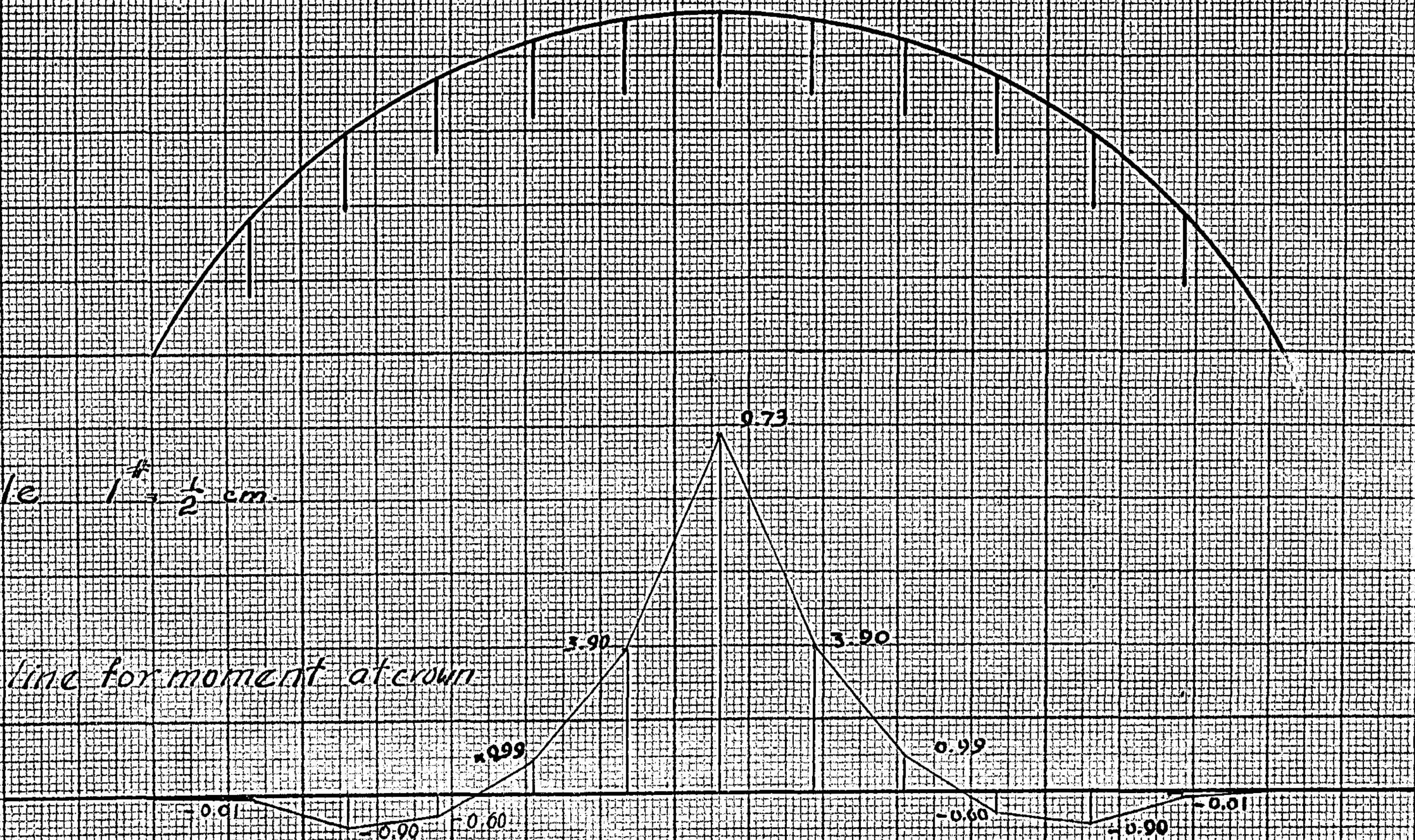
Maximum H = 912000

$$e = \frac{707}{912} = .88 \text{ ft.}$$

$$f = \frac{912000}{14.38 \times 144} + \frac{707000 \times 12 \times 26}{29.73 \times 144 \times 144}$$

$$= 440 + 358 = 798 \ \#/in^2$$

O.K.



V. Scale 1# = 1/2 cm.

Influence line for moment at crown

H. Scale 1mm = 1 foot

Drawn by:

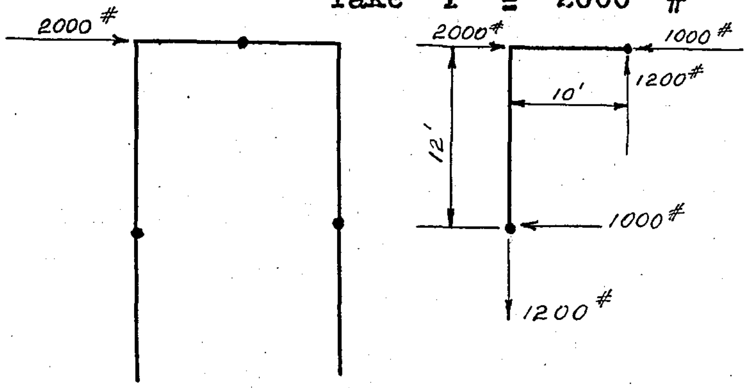
Mahmut Karasatay

DESIGN OF WIND BRACING :

By Portal method (Assuming points of counter-flexure at mid - points and shear taken equally by the verticals) .

Concentrated wind Load $= 15 \times 30 \times \dfrac{52}{12} = 1950$ #

Take P = 2000 #



$$\frac{1000 \times 12}{10} = 1200$$

The bracing members should be designed for :

$$M = 1200 \times 10 = 12000 \text{ ft/lbs.}$$

Direct Stress = 1000 feet/lbs.

Shear = 1200 lbs.

Moment Dead Load assume 12" x 24"

$$M = \frac{300}{10} \times 20^2 = 12000 \text{ ft/lbs.}$$

Total M = 12000+12000 = 24000 ft/lbs.

$$d = \sqrt{\frac{24000 \times 12}{131 \times 12}} = 13.6 \text{ "}$$

Use 12" x 24" beam

$$A_S = \frac{24000 \times 12 \times 8}{20000 \times 7 \times 22} = .75 \text{ in.}^2$$

Use 3 ½" square bar.

<u>Check for Direct Stress</u> :

$$A = \frac{S}{f_c} = \frac{1000}{450} = 2.22 \,\#/\text{in}^2$$

O.K.

<u>Check for Stirrups</u> :

$$V = 12000$$

$$W = \frac{V}{b\ j\ d} = \frac{1200 \times 8}{7 \times 12 \times 24} = \frac{100}{21} = 4.26$$

Stirrups are not needed.

Use five wind bracings.

I at 3 rd vertical.

I at 5 th vertical.

I at 6 th vertical.(center of arch)

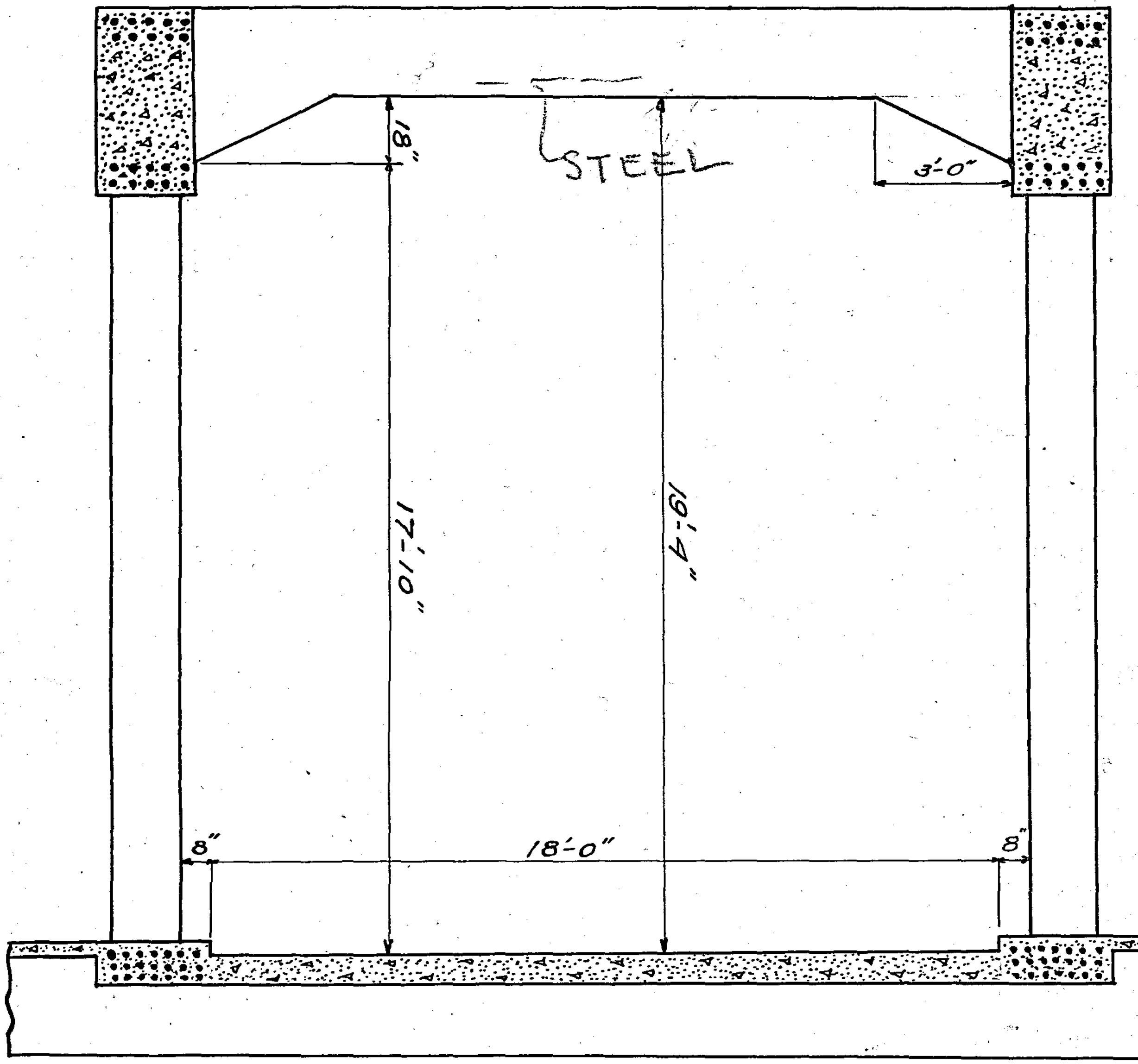
I at 7 th vertical.

I at 9 th vertical.

Clearance diagram under the wind bracing of verticals 3 and 9 is shown at the next page.

Clearance Diagram

WHICH ONE?



Scale 1/4 in = 1 ft.

DESIGN OF VERTICALS : (There are II verticals.)

Live Load	=	24000	#
Live Load Impact 30%	=	7200	#
Live Load on road - way slab	=	5500	#
Dead Load	=	30000	#
Wind Stress	=	1200	#
T O T A L T		68200	#

$$A_S = \frac{68200}{18000} = 3.78 \text{ sq.in.}$$

Use 18 - 1" ∅ bars.

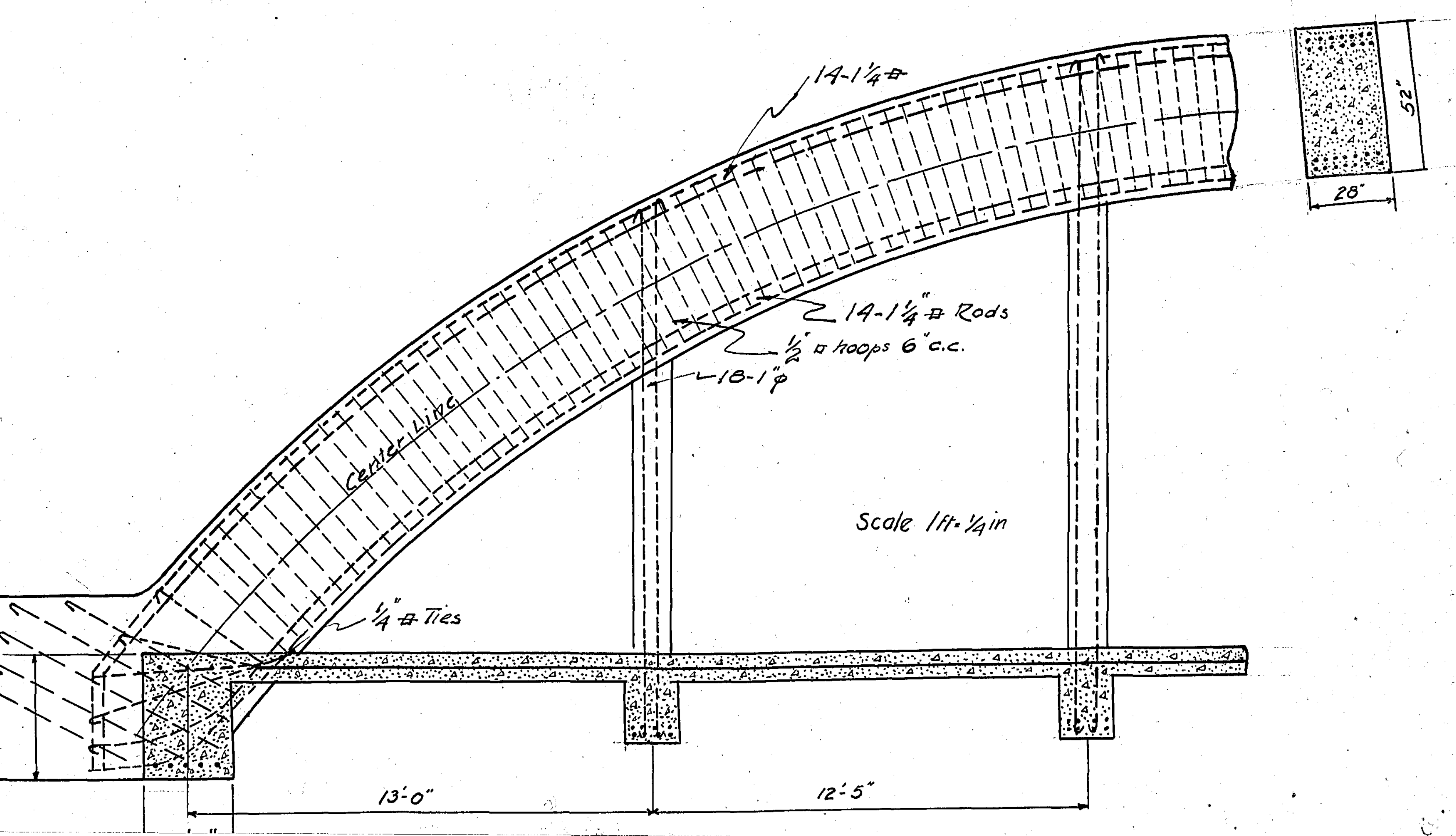
DESIGN OF BOTTOM CHORD :

Maximum D.L.	H	=	421000	#
Maximum L.L.	H	=	491000	#
T O T A L	H	=	912000	#

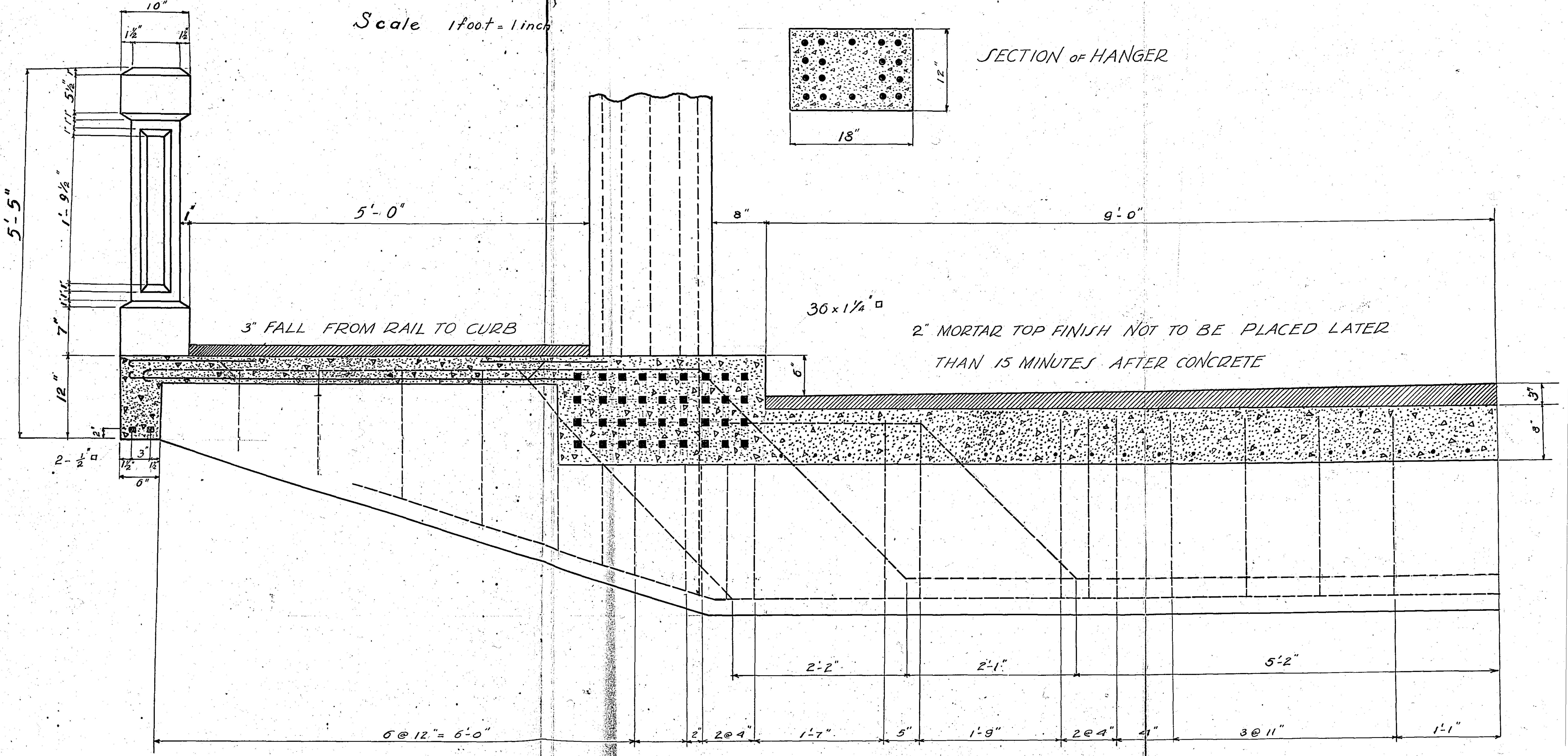
$$A_S = \frac{912000}{18000} \doteq 50.6$$

Use 34 x 1 1/4" square bars.

SECTION SHOWING REINFORCEMENT IN ARCH



Scale 1 foot = 1 inch



TOP WIND BRACING