REINFORCED CONCRETE ARCH BRIDGE DESIGN

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Reinforced Concrete Arch Bridge Design

By

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-1947-
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Foreword.

This work is the undergraduate thesis for the final year of Engineering in the American University of Beirut .

Course number : 525-526.

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....

Current year : 1945-1946.

Presented by:: Mohammed Munthir Al-Jundi .
B. A. C. E.

Signature .

M. M. Jundi

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References.

I- Principles of Reinforced Concrete by Turneaur and Maurer. Fourth Ed.

II- Masonry Structures by Spalding, Hide and Robinson. Second Ed.

III- Reinforced Concrete Bridge Design by Chettoe and Adams. Second Ed.

IV- Concrete Plain and Reinforced by Taylor, Thompson and Smulsky.

V - Theory of structures by Timoshenko and Young.

VI- Informations from the department professors, Ecole Française des Ingénieurs à Beyrouth, and the Public Works Ministry in Beirut and Damascus.

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- Notations.-

A	=	Area, left springing point.
	=	Angle of tangent with horizontal.
	=	Thickness of wearing surface.
B	=	Right springing point.
	=	Thickness of slab, moment coef. in slab design.
b b'	=	Width of Beam in R.C., width of web in AC
C	=	Crown point
C.L.	=	Crown Load, Center point, refers to concrete and crown
D.	=	Load point
D.L.	=	Dead Load
D.	=	Deflection
d.	=	Thickness of slab in R.C.

d'	=	Depth of steel bars under surface
e	=	M/N
f	=	Direct stress
g	=	$\frac{Ws}{Wc}$
h	=	Heigh of member
I	=	Moment of inertia
L	=	Span length
l	=	Span length
M m	=	Moment
N	=	Normal thrust
n	=	Es/Ec
Q	=	Load
P	=	Load
q	=	I_1/I
S	=	Spacing of stirrups
s	=	Refers to steel and springings
T	=	Thrust
t	=	Thickness
U	=	Bond stress
V	=	Shear
v'	=	Shear taken by stirrups
x	=	Horizontal abcissa
Ws,Wc	=	Spring and crown loads
y'	=	ordinate of center of elastic arch.

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R. C. A. B. D.

I. Preliminary Part.

1 - Statement of the problem :

Design a " Reinforced Concrete Arch bridge where :

Clear span	80 ft.
Width	30 ft.
Bituminous Macadam wearing course	2 in.
Crown for roadway	2 in.

Specifications.

1924 Joint Committee Report

fs	18000 p.s.i.

The French System of Loading.

Temperature :

Fall	60° F.
Rise	20° F.
Coef. of expansion	0.000006

2- Choice of the problem:

This kind of problem leaves to the designer the choice in the following things :

A - Appearance and details of the structure.

B - Allowable stresses and specifications.

C - Method of design

A. Appearance and details of the structure

If the problem were to design a bridge for a specific site, or river, then the nature of the place, its topography, geology and many other factors would determin much of the things under this heading. But since I am given the choice, I'll take the following :

a- Kind of bridge. There are many kinds of R. C. A. B. following a structural classification we have :

1- Three Hinged arches.

2- Two Hinged arches,

3- One Hinged arches.

4- Fixed arches.

The last kind is the most used in reinforced concrete structures. A Fixed arch might be :

1 - Spandrel filled.

2 - Open spandrel.

No. 1 is used in small spans and rises.
No. 2 is used in big spans and rises. Because it reduces much of the dead load (fill). The open spandrel arch might be :

1. Arch ring type

2. Ribbed Arch type

in the first the arch is continuous transversly, where in the second, the arch consists of two or more ribs transversely. This ribbed arch is more economical than the archring type, since it saves the Dedd load of some of the concrete. As in the ribbed slab and Tr beam.

I 'll use the two-Ribbed type. See plates.(2, 3, 4, 5).

b- Arch axis and ratio of rise to span. See pls. (6,2)

The most economical arch axis is that arch which gives a zero moment all through under dead load + ½ L.L. For filled Spandrel arches there are some special indications (Corchrane's.). For our case (cochrane) gives a special formula :

$$y = hz^2 \frac{(1 + 1/6\ (g-1)\ z^2)}{(1 + 1/6\ (g-1)\quad)}$$

where g is $\frac{ws}{wc}$ which is one in our case. So our arch axis is a parabola $y = hz^2$. This would somehow facilitate my calculations and make them more accurate, since the parabola could be solved mathematically for : tan, cos., sin., ds, x, y, etc....

c- Rise.- Usually the rise is governed by the height of the roadway from the surface of ground, and the span of the bridge. That is why some writers say that arches are not of use for shallow bridges. This is not always true. In The ribbed arch the rise might be anything below certain limit (the height of roadway); and in the bow string girder (a kind of two hinged arch) the rise is absolutely independent of the height of roadway above the ground.

I'll take my rise as one forth of the span of the arch.

d- Roadway and Side walks: The American requirement is 9 ft. per lane. Taking two lanes will require 18 ft. Now our width of bridge allows us to have a 20 ft. roadway with a two 5 ft. side walks. All details and specifications in this respect are taken from " General Specifications for the design of steel Highways bridges" given in " Appendix B " in " Structural Design in steel, by Shed. See pls. (4, 5)

B- Allowable Stresses

a- Steel		18 000 psi.
b- Concrete		2500 psi.
Compression in extreme fibers		1000 psi.
Used		900 "
Shearing (Long bars with special anchorage):		
No web reingorcement	3 persents	75 "
Used		70 "
With web reinforcement	12%	300 "
Used		270 "
Bond		100 "
Used		100 "
Compression :		
Bending & compression	36 %	900 "
Colums	25 %	625 "
Bent colums	30 %	750 "

c- The French system of loading :

Two systems of loading will have to be considered :

1 - The roadway is designed to carry a uniform Live load of :

L = span

= (820 - 4L) Kgs/m2 with a minium of 500 Kgs/m2
for L = 80 m.

The sidewalks will also have to carry a uniform L.L.
of 400 Kgs/m2 –

2- The roadway is then designed to carry a system composed of two trucks each having the following characteristics : fig.1

Total load	16 Tons
Rear axle load	12 "
Front axle load	4 "
Total length	10 m.
Total width	2.5 m
Distance center to center of axes	4.0 m
Distance center to center of wheels	1.7 m.
Wheel width	0.30 m.

We will assume, travelling side by side and in the same direction as many of these systems as the width of the roadway permits.

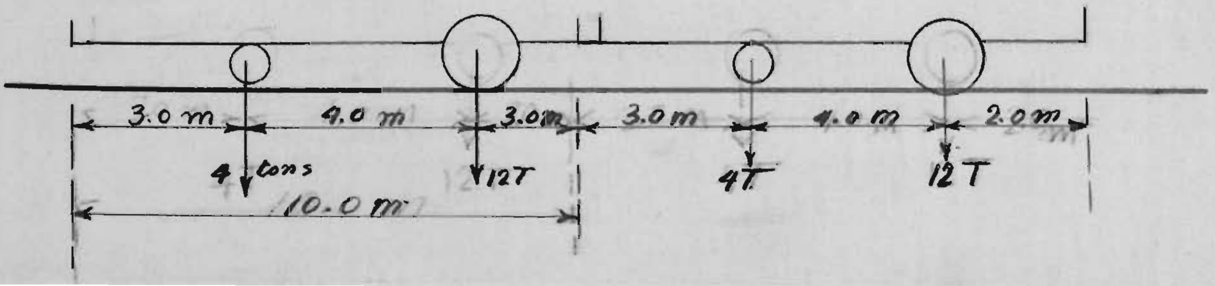
The coefficient of impact is given by :

$$I = \left(1 + \frac{0.4}{1+0.2L} + \frac{0.6}{1+4p/s}\right)$$

L = span , P = Total D. L., S = Total L.L.

The two systems of loading have to be considered, and whichever gives the biggest results will govern the design.

fig. "1"



d- French Formulas for dispersion:

My special inquiries in the Public Works in Beirut, Damascus, and in the French School for Engineers led me to the following result..

The French arrêt ministériel specifies the design to be as follows for a concentrated load P on a point on the slab. See fig. "2"

The slab is designed to support the load P as a uniformely distributed load over a rectangle whose sides are :

1- For reinforcement parallel to (L_1) :

$$A = \alpha + \beta$$

$$B = L_1 \, 1/3$$

2- For reinforcement parallel to (L_2)

$$A = 1/3 \; L_2$$

$$B = \alpha + \beta$$

The same procedure is followed as in the design of ordinary slab in what concerns M_1, M_2 and their coefficients

$$\beta_1 = \frac{1}{1+2\left(\frac{l_1}{l_2}\right)^4}$$

$$\beta_2 = \frac{1}{1+2\left(\frac{l_2}{l_1}\right)^4}$$

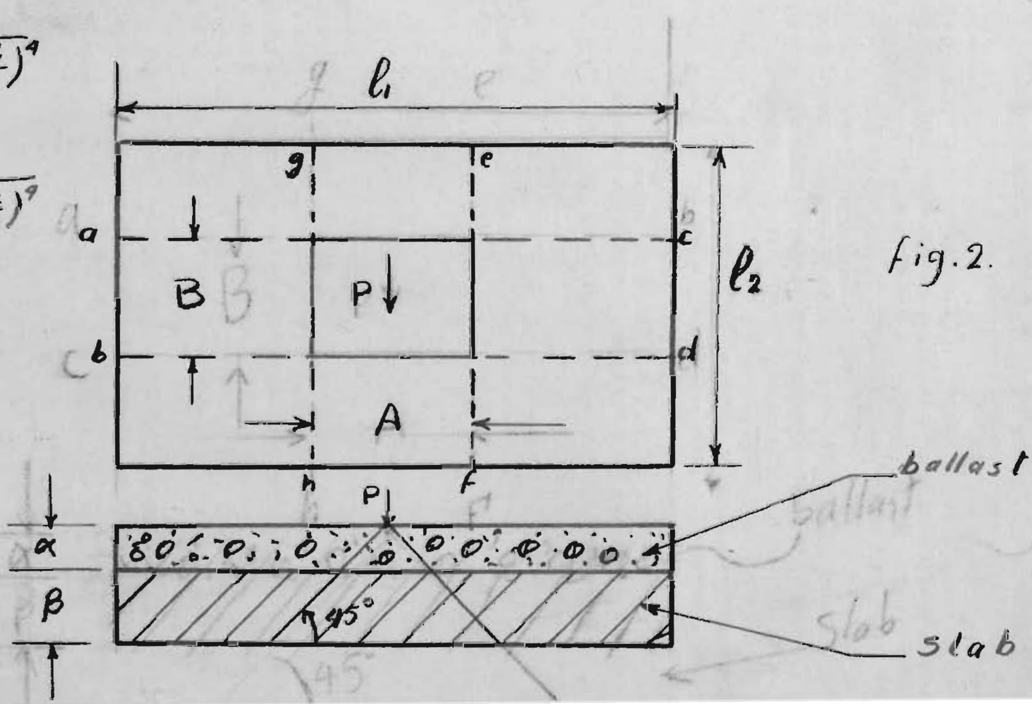


fig.2.

These are used for a uniformly distributed load, which is accurate enough to replace $(L_1/L_2)^3$ in our small range $3/2 > L_1/L_2 > 2/3$.

M_1, M_2 are independent moments taken at beams parallel to L_1 and L_2 respectively. So the Slab is made as if only a b c d and efgh were only present to support the load.

In case we consider the tyres width " W " this width is added to A or B according to direction of traffic.

As usual when L_1/L_2 is $< 2/3$ or $> 3/2$ the slab is designed in one direction only, the shorter.

A comparison between French and American For.

Taking L_1/L_2 very big we design in one direction.

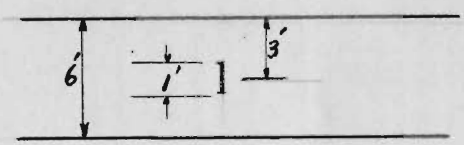


Fig 3

American Formula :

$E = 0.7\ (2\ D + w)$

$E = 0.7\ (2 \times 3 + .83) = 4.8$ feet

French Formula :

$1/3\ L = 1/3 \times 6 = 2$ feet.

That meanst the American method is more economical and more logical, since $E = .7\ L + 7w = .7\ L + 30$ where the French $= .33\ L$

C. Method of Design

In all the design of details, as slab, beams etc. I am following the specifications of the joint Committee 1924 Chapter XI.

The fixed arch is an indeterminate structure of the third degree. Its solution depends on the theory of Elasticity. The theory of Elasticity is applied under one of two forms :

1- The deflection and slope method

2- The Castigliano or least work method

No. (1) is the classical method given in almost all text books on structure, while the second is a special method that facilitates tremendously the solution of almost all indeterminate structures.

I followed the deflection and slope method given in very nicely in (principles of R.C. construction) by Turneaure and Maurer. (Fourth edition, third printing.)

3- Outline of the problem.

After having the specifications, strength of materials, etc. the problem became to design and detail :

A - the Slab System,.

B - The colums.

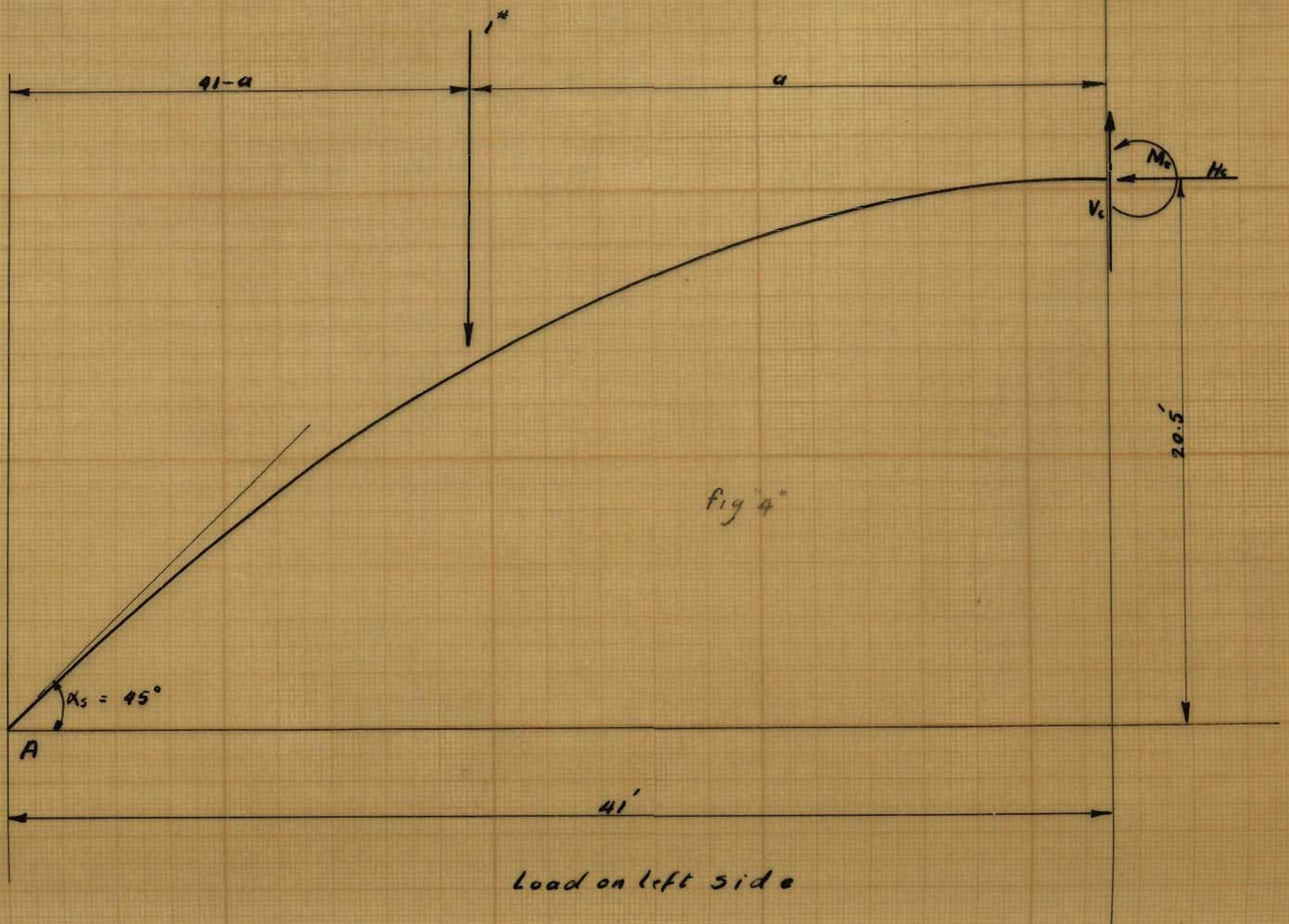
C - The arch.

D - Abutments.

E - Drainage and parapet.

4- Analysis of the problem.

The Analysis and theory of A, B, D and E is known and elementary. Below I'll give a brief analysis of the arch by the deflection and slopemethod : in mechanics of materials Δx, Δy, $\Delta \phi$ are given in terms of M, f, W. Δx, Δy, $\Delta \phi$ represents the deflection along x, y, axis and rotation by angle ϕ.



Load on left side

Terms with M, F, W, represents respectively the effect of moment, Rib shortening, and temperature.

At crown : the condition equations are :

$$\Delta y_L = \Delta y_r \qquad \Delta x_L = -\Delta x_R$$

$$\Delta \phi_L = -\Delta \phi_R$$

At any section (left side)

$$M_x = M_c + H_c y + V_c x + m_L$$

Right side

$$M_x = M_c + H_c y - V_c x + m_R$$

with vertical loads :

$$T_x = H_c \cos\alpha + \left(\sum_C^D P - V_c\right) \sin\alpha$$

Left

$$V_x = H_c \sin\alpha - \left(\sum_C^D p - V_c\right) \cos\alpha$$

Right

$$T_x = H_c \cos\alpha + \left(\sum_C^D P - V_c\right) \sin\alpha$$

$$V_x = -H_c \sin\alpha + \left(\sum_C^D P - V_c\right) \cos\alpha$$

In symmetrical Arches : Assuming Hc = Tc

some termes are negligible.

By transferring the x - axis to the Elastic center the terms are greatly simplified and ~~why~~ yo is such that

$$\sum y_1 q = 0 \qquad y_0 = \frac{\sum y q}{\sum q}$$

All the above will lead to the following Final expressions :

$$M_c = \frac{-\sum_A^C (m_R + m_L) x q}{2 \sum_A^C q} - H_c y_0$$

$$H_c = \frac{-\sum_A^C (m_R + m_L) y_1 q + \frac{w t L E}{d s_1 / I_1}}{2\left[\sum_A^C y_1^2 q + I_1 \sum_A^C \frac{\cos\alpha}{A}\right]}$$

$$V_c = \frac{\sum_A^c (m_R - m_L)\, x\, q}{2 \sum_A^c x^2 q}$$

Defelection at the crown : $D_c = -\frac{ds_1}{EI_1}\Big[(M_c + H_c y_0)\sum_A^c x q + H_c \Big(\sum_A^c x y_1 q - \frac{1}{ds_1/I_1}\sum_A^c \frac{dy}{A}\Big) + \Big(V_c \sum_A^c x^2 q + \sum_A^c m_L x q\Big)\Big] - wth$

So the method is to devide the arch rib into a suitable number of divisions and to compute the constants of the sections and get the other quantities

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II. - Design part

1- Plan of carrying out the work.

we have to choose a certain specific arch, Then we have to see whether it is safe or not. In Turneaur's he is using the Cochrane Tables. So I did.

2- The numerical results given by the elastic theory are the most reliable in the arch design. But they are still not very accurate and no great refinement is required because of the following :

1- We neglected some terms in the theoritical analysis

2- We assumed three ideal conditions.

A- The length of span remains unchanged.

B-- Continuity of the arch axis is maintained and one and does not move vertically with respect to the other.

c- The inclination of the arch axis at each abutment remains unchanged.

So here we neglected the effect of the unavoidable settlement, spreading, rotation of abutment, even though these deformations are to a small scale.

3- The use of the slide rule introduces small errors which accumulates and at the end might be of a big magnitude in the terms expressing moments thrusts etc...

3- Design :

Taking the design in its natural order, I'll take it as follows : See plates (4,9)

A - Design of slab system.

B - Design of supporting Columns

C - Design of arch ribs.

D - Design of abutments.

See the Plates in the tube.

A- Design of slab system : This part might be divided into the following : See pls (4,9).

a- slab of roadway

b- transverse beams

c- side walks

d- side beams.

a) Slab of roadway (Design for end support)

For end spans $M = W L^2/10$, For intermediate spans $M = \frac{WL^2}{12}$

I'll design all as end spans. See Figs. (5, 6, 7), pls (4, 9).

D. L. of slab : 9 x 12 = 108 # lbs /ft^2

wearing surface 3 x 10 =30

D. L. = $\frac{12 \times 9 \times 12 \times 12 \times 144}{144 \times 12}$

= 1300 lbs.

W. S. = 360 "

Total = 1660 lbs.

L. L. = $\frac{12000}{12/3+1}$ = 2400 lbs.

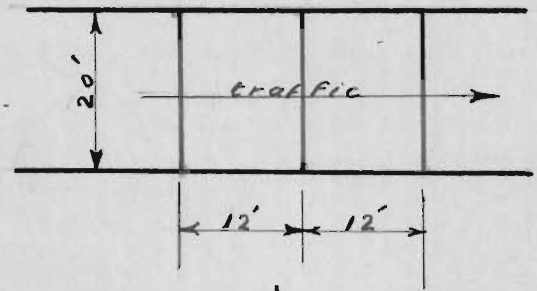

fig 5

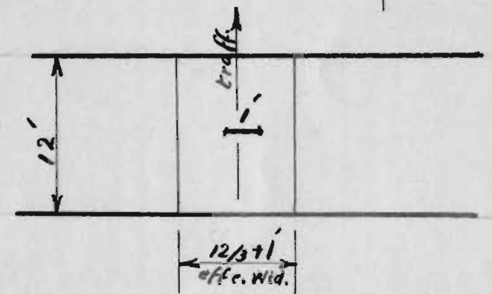

fig 6

in ft. lbs. system :

$I = 1 + \frac{0.4}{1+0.061L} + \frac{0.6}{1+4\frac{P}{S}}$

$\frac{P}{S} = \frac{1660}{2400}$

I = 1 + 0.25 + 0.16 = 1.41

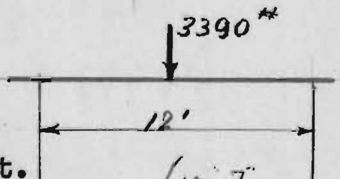

fig 7

L. L. + I = $\frac{12 \times 1.41}{5}$ = 3390 P. per ft.

Max. M. is at the middle.

L. L. M. = 1/5 X 3390 X 12 = 8150 ft. lbs.

D. L. M. = $\frac{138 \times 12 \times 12}{10}$ = 1990

10140 ft. lbs.

For fs = 18000 fc = 900

N = 12

P = 0.0094 R = 148 K = .375 j = .875

$d^2 = \frac{10140 \times 12}{12 \times 148}$ = 68.7

d = 8.3 + .7 = 9 in.

As = 0.0094 x 8.3 x 12 = 1.0 sq. in.

Results : USE : See pl. (9)

$d = 8.3 + .7 = 9$ in.

$\frac{1}{2}$ in @ 3 in. intervals (10)

Note : No need to look for shearing.

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b) Transverse Beams :

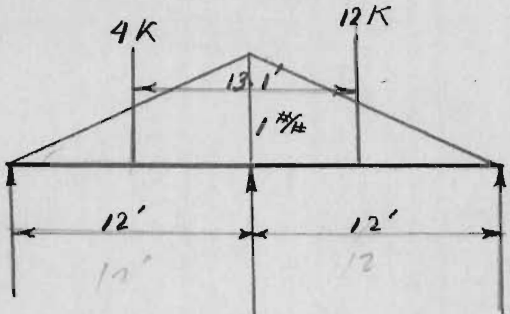


fig "8"

M. at centre is = $+\frac{wL^2}{8}$

M. at end is = $-\frac{wL^2}{16}$

See fig. 8 :

The max. reaction of A is 12 kips.

$I = 1 + \frac{0.4}{1+0.061 \times 20} + \frac{0.60}{1+ 4\,P/S}$

$P = 1.25 \times 144 \times 20 + 12 \times 20 \times (9 \times 12 + 30)$

$= 36800$ lbs.

$S = 4 \times 12000 = 48000$ Lbs.

$4P/S = \frac{368000 \times 4}{48000} = 3.07$

$I = 1 + 0.18 + 0.147 = 1.327 = 1.33$

Every kip becomes = 1330 lbs.

See Fig (9)

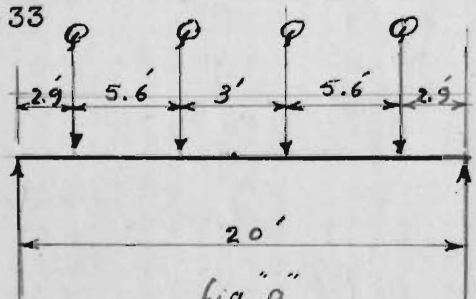


fig "9"

L.L.M :

From symmetry it occurs at the centre.

$M = 20\,Q - 7.1\,Q - 1.5\,Q = 11.4\,Q$

$Q = 12 \times 1330$

$M = 12 \times 1330 \times 11.4 = 182\,000$ ft. lbs.

D. L. M.	Beam	=	11/12 x 144	=	140
	Slab	=	9 x 12 + 3 x 10		
	Total	=	12 (9 x 12 + 30)	=	1660
					1800 lbs

D. L. M. = 1/8 1800 x 20 x 20 = 90200

L. L. M. 182000

272200 ft. lbs.

The uniform L. L. gives M = 90000 ft. lbs.

So the concentrated L. L. still governs.

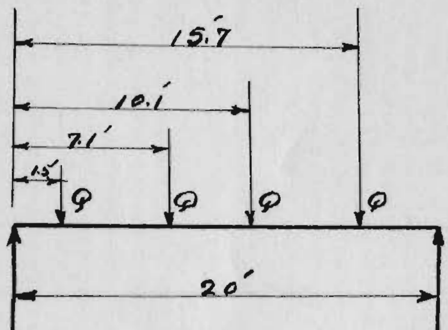


fig. 10

Shearing

D. L. = $\frac{20}{10}$ x 1800 = 18000 lbs.

L. L. see fig (10)

From Fig. (6) we have :

$$L.L.V. = \frac{Q}{20} \left(\begin{array}{l} (1.5 - 20) + (20 - 7.1) \\ +(10.1 - 20) + (20 - 15.7) \end{array} \right)$$

= 2.28 x Q = 2.28 x 1330 x 12

= 36 400 lbs

D. L. = 18 000 "

54 800 lbs.

The least allowable b'd in T - beam is :

$$b'd = \frac{54\,800}{270 \times .9} = 22.6 \text{ sq. in.}$$

Compitations showed that the most economical design of this transverse beam is to design it as a rectangular beam (Diag. 8 in Turneaure's)

As a rectangular beam

$$d^2 = \frac{272200 \times 12}{12 \times 5 \times 148} = 375$$

d = 19.5 in. say 20 in.

b = (20 : 4 = 5 ft.
(8 x 9 x 2/12 = 12 ft. + web.
(12/2 = 6 "

= 5 ft. = 60 in.

From Shear requirement :

$$b' = \frac{226}{20} = 11.3 \text{ say } 12 \text{ in.}$$

As = 0.0094 x 60 x 19.5 = 11.0 sq.in.

Results USE :

d = 20 + 4 = 24"

9 - 1" 1/8 □ (= 11.39 sq.in).

b' = 12 in.

Web reinforcement :

Drawing Shear Diagram :

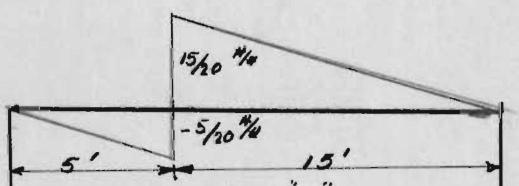


Fig. "11"

V at end : = 54800 lbs.

V at 5 ft. from end:

$$D.L : \frac{20}{2} \times 1800 - 5 \times 1800$$

= 9200 lbs.

L. L. a- two trucks.. Since it is impossible to have a wheel on the 5 ft. point, the max V in this case is when we have a wheel at 1.5 ft. point which gives :

V = R - Q = (2.28 - 1) Q = 1.28 Q

b- One truck:

$$V = R = Q\left(\frac{15}{20} + \frac{9.4}{20}\right)$$

$$= \frac{24.4}{20}\,Q \quad \text{Less than (a)}$$

L. L. = 1.28 x 12 x 1330 = 20 400 lbs.

D. L. = 9 200 "

29 600 "

V at 4.4 ft. from end: is governed by two truck loading

V = R = 4- 2.28) Q = 1.72 Q

= 1.72 x 12 x 1330 = 27 400

D. L. 7 900

35 300 lbs

V at 9.9 ft from end: See Fig. (8)

D. L. = Zero nearly

L. L. a- two trucks :

V = Q (R - 1) = Q (1.72 - 1) = 72 Q

b- One truck :

$$V = R = \left(\frac{10.1}{20} + \frac{4.5}{20}\right) Q =$$

V = .730 Q that means one truck loading governs

L. L. = .73 x 12 x 1330 = 11 650 lbs.

~~D.L.~~ = Say 12 000 "

Drawing M. Diagram

M at middle :

M = 272 200 ft. lbs.

M at end M = zero

R. C. A. B. D.

M at 5 ft from end

L. L. = RL x 5 - Q x 3.5

= 126 000 ft. lbs.

D. L. = RL x 5 - 5 x 1840 x 25

= 69000 ft. lbs.

Total (D.L. + L.L.) M = 195 000 ft. lbs.

The concrete takes care of :

75 x .875 x 12 x 20 = 15.800 lbs.

The remainder 54800 - 15 800 = 39.000 "

is left for stirrups and bent up bars

$$= \frac{54800}{.87 \times 100 \times 20} = 31 \text{ inch.}$$

So don't bend up 5 bars. Bend the four remaining at an angle of 30 degrees.

$$S = \frac{2 \times 45}{3 \times 10 + 30} = 3/4 \ 20 = 15 \text{ in.}$$

$$V' = P \frac{(\cos 30^\circ \ P \ \sin 30^\circ)}{bs} = \frac{18000 \times 1.27 \times 1.367}{12 \times 15}$$

V = bjdv' = 173 x 12 x .875 x 20 = 36400 lbs.

From the shear diagram the extra shear over that taken by concrete is 28 500 - 15800 = 12 700 lbs.

P = V' bs taking s = 6 in.

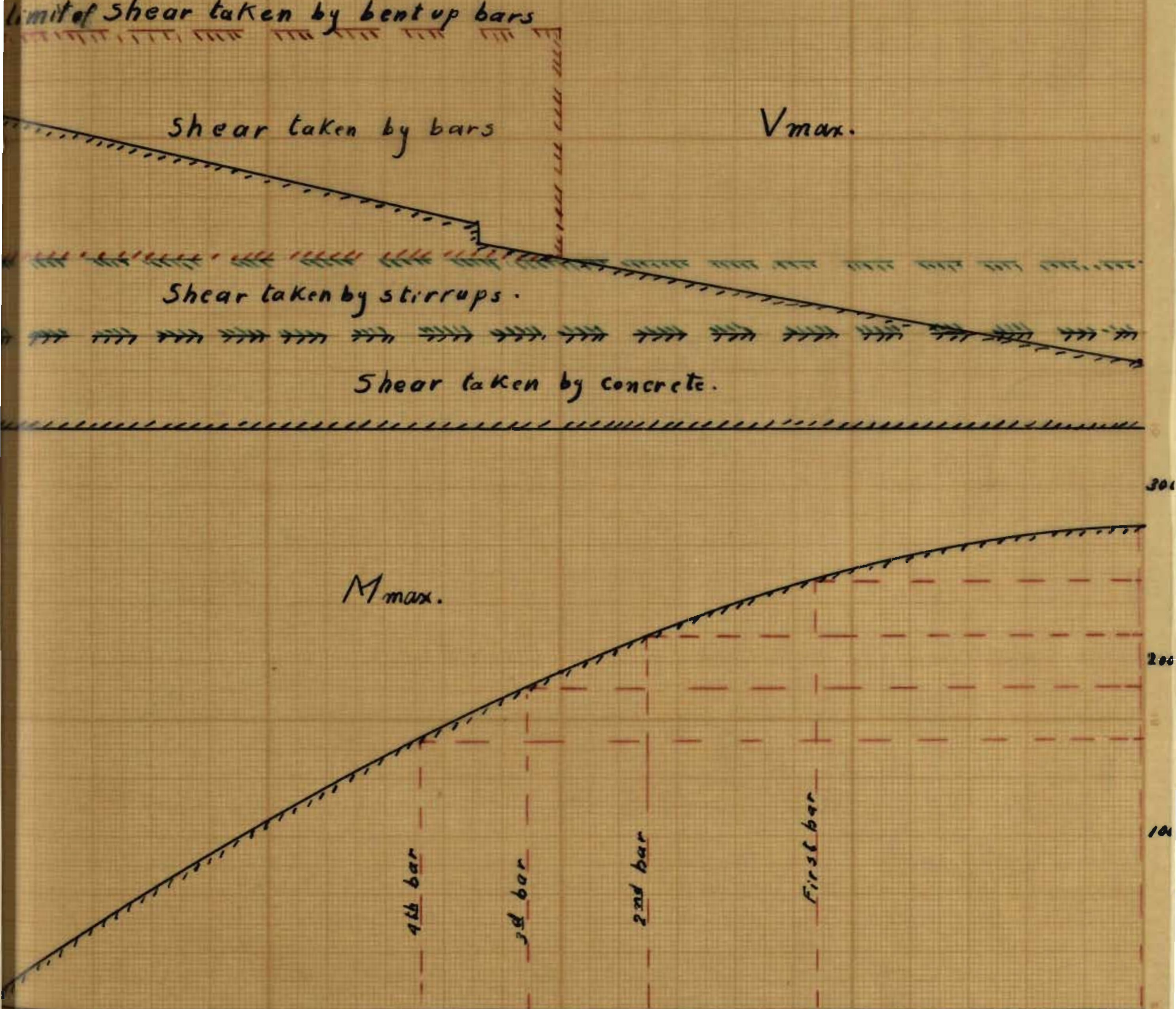
$$P = \frac{12700}{jd} \times s = 4300 \text{ lbs.}$$

4300 : 18000 = 22 sq. in.

USE : U - stirrups (vertical)

3/8" ∅ (0.22 sq. in)

Max. S = 0.45 x 20 = 9 in.



D. I

M. and V. Diagrams for Transverse Beams.

$$V = \frac{Pjd}{s} = \frac{4300 \times .875 \times 20}{9} = 8500 \text{ lbs.}$$

Or 8500 + 15800 = 24300 lbs that occurs at a point 6.5 ft from end.

Results : USE :

Bend up 4 bars at 15" intervals from end, then USE 3/8" in. bar U - stirrups at 9 in. interval all through. USE the same stirrups at points 5, 5.5, and 6 ft. from end.

Note : The moment Diagram allows the bending up of bars at these poinds(at 15 in. intervals.)

Contact area :

54800 : 625 = 87.8 say 90 sq. in. for a width of 12" the depth is 90 : 12 = 7.8 say 8 inches which is available.

c) Side walks.

See Fig. 11 : and plates 4,9.

L. L. = 400 kg/ sq.m.

= 82 Lbs. / ft.sq.

L. L. =		82
Fill =	1x110	110
W.S. =	3x10	30
Slab =	7.5x12	90
Parapet		138
		450 lbs. par ft.

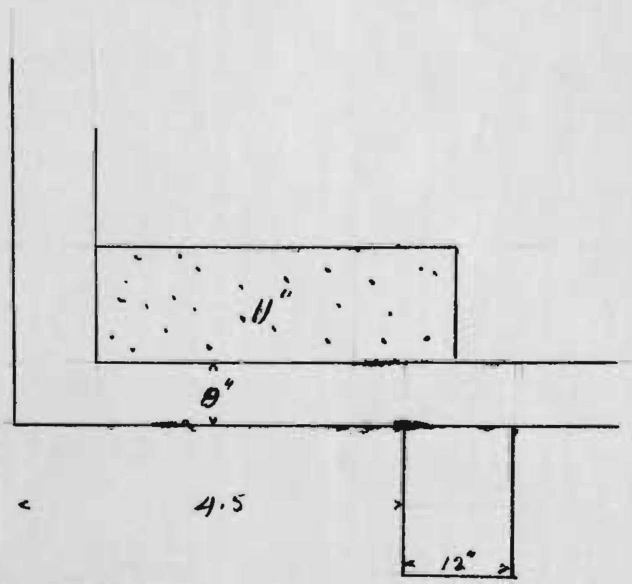


fig. 12

$$M = \frac{5 \times 5 \times 450}{2} = 5630 \text{ ft. lbs.}$$

$$d^2 = \frac{5630 \times 12}{12 \times 148} = 38.0$$

d = 6.16 say 6.5 + 1.5 = 8 in.

As = 0.0094 x 6.16 x 12 = .70 sq. in.

<u>Results :</u> d = 6.5 + 1.5 = 8 in.

$\frac{1}{2}$ in. bars @ 4 in. intervals (.75)

d - <u>Side Beams</u>

End span beams $M = 1/10\ wL^2$

intermediate spans $M = 1/12\ wL^2$

I'll design all as End spans since the beam carries some torsion.

D.L. + L.L. =	450 x 5 =	2250 lbs. per ft.
beam =		162 "
		2412 say 2420 lbs per ft.

$$M = \frac{2420 \times 12 \times 12}{10} = 34900 \text{ ft. lbs.}$$

As a rectangular beam : d= 20 in.

b'= 12 in.

$$R = \frac{M}{bd^2} = \frac{34900 \times 12}{20 \times 20 \times 20} = 87.1$$

P = 0.0055

As = 20 x 12 x .0055 = 1.32 sq. in.

<u>Results:</u> USE :

d = 20 + 4 = 24 in.

b'= 12 in.

4 - 3/4 in. bars (= 1.77) The extra steel is to provide for torsion.

Shearing :

At the end

D. L. = $(2410 - 5 \times 82)\ \frac{12}{2}$	=	12 000
L. L. = $5 \times 82 \times 12/2$	=	= 2 460
		14 460
		Say 14 500 lbs.

Shear taking by concrete is :

$V = 12 \times 20 \times 75 \times .875 =$ 15.800 lbs.

That means no need for web reinforcement. But for practical reasons use 1/4 in. bars. U -stirrups at 1 ft. interval all through.

The Bond requires :

$$= \frac{14500}{875 \times 100 \times 20} = 8.3 \text{ in.}$$

That means bend up two bars only at .2 x 12 = 2.4 ft. from End. Keep two bars through out the beam.

B. Design of supporting Calumns

Floor beam reaction	=	54 800
D. L. of column = 3x150x10	=	4 500
Side walk and beam = 2412 x 12	=	29 000
		88 300 lbs.

Take a section of 24 x 16 in.

$M = 1/16\ wL^2$ or 1/2 Moment of transverse beam

$= 1/2 \times 277000 =$ 138000 ft. lbs.

$e = \frac{138}{88} = 1.57 \qquad h/e = \frac{2}{1.57} = 1.27$

$d'/h = 2/24 = .085$ Say average of .1 and .05

$fc = \frac{C\,M}{bh^2}$ $C = \frac{fc \times bh^2}{M}$

$= \frac{750 \times 16 \times 24 \times 24}{138000 \times 12} = 5.2$

For the given e, d'/h , e/h we have :

$P = 0.0125$

$As = 0.0125 \times 24 \times 16 = 4.8$ sq. in.

Results : USE :

A column of 16 in. x 24 in.

Produce 2 - 1 1/8"squared bars from transverse beams: 2.53

four - 3/4 in. Round bars at corners 1.77

4.30

The total is less, since we are not using symmetrical reinforcement.

C. Design of Arch rib

The arch properties are :

1. The span is 80 ft. But take 82 ft. to give a net span of 80 ft.

2. The type is the open spandrel (ribbed) The columns spans are from center to center 9, 10, 11, 12 feet.

3. Choice of Rise : = $\frac{Rise}{span} = \frac{1}{4}$ Rise = 20.5

4. Dead load at crown and springing

Span 9 ft.

F. beam reaction (140 + 9 (108 + 30) $\frac{20}{2}$ = 13 800

Side walk = (2412 - 5 x 82) x 9 = 18 000

Rib dead load = 2 x 2.5 x 144 x 9 = 4 050

at C3 span 12

F. b. reaction = 18 000

Side walk = 24 000

Column = 11 x 1 x 144 = 1580 = 1 600

Rib = 7.0 x 3 x 4.6 x 1.44 = 13 900

43600 - 31800 = 11 000

: 3 = 3 900

+ 31 800 = 35 700

C1 = 35700

C2 = 39 600

Say = C1 = 35 700

C2 = 39 600

C3 = 43 600

Crown = 31 800

5) Live Loads

A uniform is : 820 - 4L = 820 - 4 x 24.4

= 720 Kg.

720 kg. /m2 = 148 Lbs/ft^2 say 150 lbs/ft^2

Impact = $\frac{50}{80 + 125}$ = 0.244 = .25

1.25 x 150 = 187.5 Lbs/ft^2 say 190 Lbs/ft.^{2}

Every rib supports 10 feet of roadway

Roadway = 10 x 190 = 1900

Side walk = 5 x 82 = 410 = 410

2310 Lbs/ft

L.L. uniform at crown =	2300 x 9	=	20700
C1 =	2300 x 9.5	=	21800
C2 =	2300 x 10.5	=	24200
C3 =	2300 x 12	=	27700

B - L.L. concentrated :

1- Rear axle = 2.25 x 12 x 1.25 = 33800

Sidewalk = 10 x 410 = 4200

38000 #

2- Front axle 2.25 x 4 x 1.25 = 11300

Side walk = 4200

15500

(1) to (2) = 13.1 ft.

(2) to (1) = 19.7 "

6) Form of Arch axis = See Table A

a Parabola : $y = hz^2 = Ah = \frac{4x2}{L^2}$

7) Length of Arch axis = tangent to arch axis at springing

$y = \frac{x2}{L}$ $y' = \frac{2x}{L} = 1$ at x = 41

$\alpha_s = 45°$

$\cos \alpha_s = .707$

From Calculus :

$$S = \int_a^b [1 + y'^2]^{1/2} dx \qquad y' = \frac{2x}{l}$$

After integration and computations :

$S = 41 \times 1.148 = 47$ ft. @ 10 = 4.7 ft. each devision

Table A

Form of Arch Axis

Z	.1	.2	.3	.4	.5	.6	.7	.8	.9	1
A	.01	.04	.09	.16	.25	.36	.49	.64	.81	1
y	.205	.81	1.85	3.28	5.12	7.4	10	13.1	16.6	20.5
x	4.1	8.2	12.3	16.4	20.5	24.6	28.7	32.8	36.9	41.
x 2.05	6.15	10.25	14.35	18.45	22.55	26.65	30.75	34.85	38.95	

Table B

Arch rib Thickness

Axis	0	.05	.15	.25	.35	.45	.55	.65	.75	.85	.95	1
Ratio	1.00	1.007	1.021	1.035	1.049	1.63	1.77	1.095	1.145	1.245	1.406	1.5
Thick.	2.00	2.014	2.042	2.070	2.098	2.126	2154	2.190	2.290	2.490	2.812	3.00
Thick.	2.00	2.028		2.057	2.084	2.112	2.140	2.172	2.240	2.390	2.651	3.00
$\frac{1}{2}$ thick.	1.00	1.014		1.028	1.042	1.056	1.070	1.086	1.120	1.195	1.325	1.5

Width all throug = 2.5 ft.

8. Thickness : See Table B

d_c = 2 ft.

d_s = 3 ft.

Width = 2.5 ft all throughout

The data for the variation in thickness along the arch axis is taken from (Table 24) after cochrane. Turneaure page 390.

9. The arch properties and constants.

Following the method given in turneaure we divided the arch axis into ten equal divisions. We assume that the I, q, A, etc... to be uniform all through each division. The computations are put in a tabular form, Tables C, D, E, F are self expressing. See diagram.

Now y_o = = 5.88 ft.

In table C we have :

P_n = .08 p = .0067

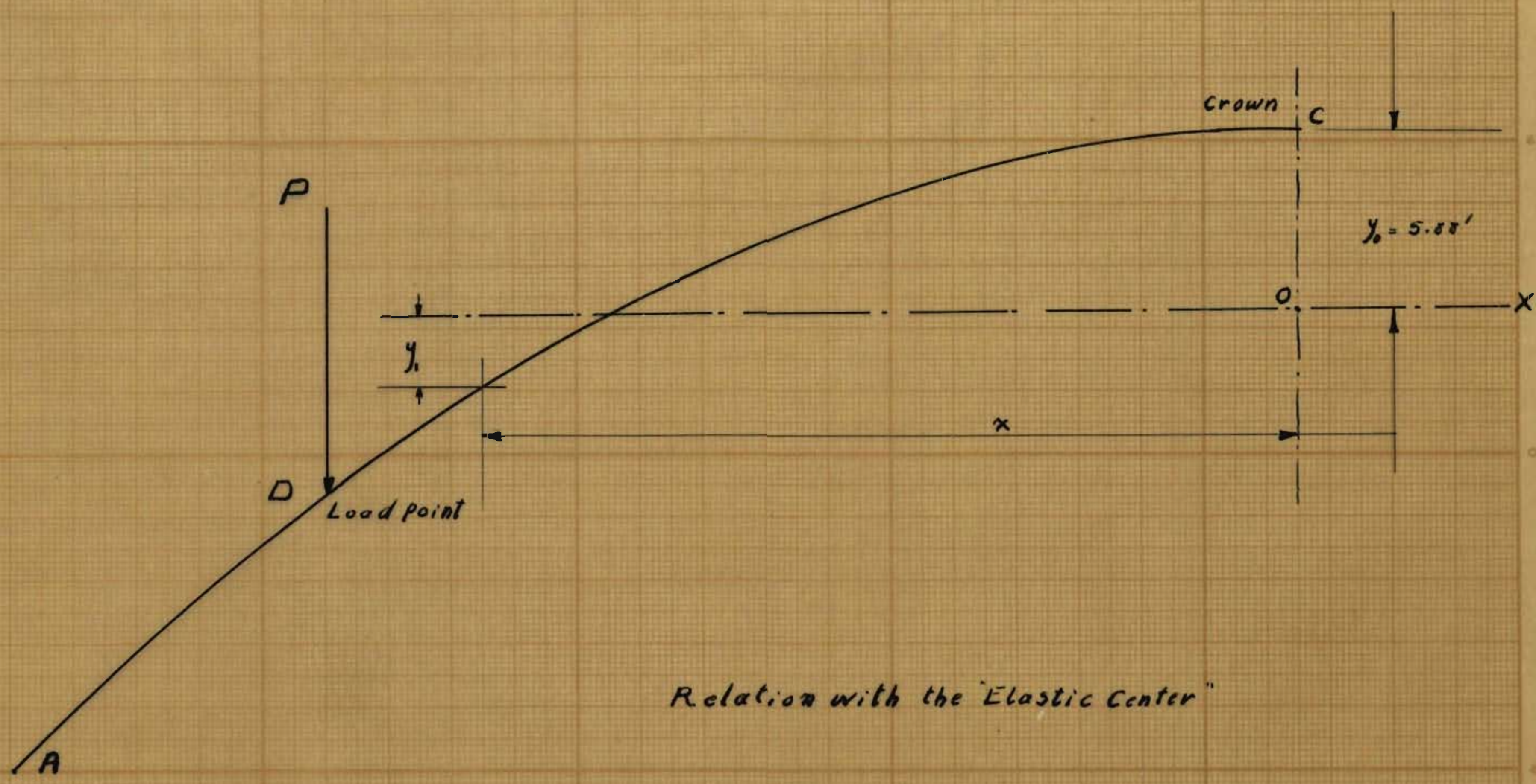
at crown A_s = .0067 x 30 x 24 = 4.8 sq. in.

at springing A_s = 1.5 x 4.8 = 13.2 sq. in.

10. Influence lines at crown.

After having the results of tables C, D, E, F, the formulas for, Mc Hc, Vc became as follows :

$$V_c = \frac{\sum_A^c (m_R - m_L)\, x q}{2 \sum_A^c x^2 q}$$



Relation with the "Elastic Center"

Table C

Properties of the arch ring

End of Section (1)	Radial depth d (2)	d' $d/2 - .15$ (3)	d'^2 (4)	Is $2(n-1)As\, d'^2$ (5)	d^3 (6)	Ic $1/12\, bd^3$ $-.2083\, d^3$ (7)	I (5)+ (7) (8)	A $6d + 2(n-1)\, As$ (9)
Crown	2.00	.850	.722	.265	8.00	1.667	1.932	5.377
1	2.028	.864	.748	.274	8.33	1.735	2.009	5.45
2	2.057	.878	.770	.282	8.70	1.813	2.095	5.51
3	2.084	.892	.795	.292	9.07	1.890	2.182	5.58
4	2.112	.906	.820	.301	9.45	1.970	2.271	5.65
5	2.140	.920	.845	.310	9.80	2.041	2.351	5.72
6	2.172	.936	.879	.484	10.25	2.137	2.621	5.99
7	2.240	.970	.940	.517	11.25	2.342	2.859	6.15
8	2.390	1.045	1.092	.602	13.65	2.842	3.444	6.53
9	2.651	1.175	1.380	.760	18.75	3.910	4.670	7.19
10	3.00	1.350	1.820	1.000	27.00	5.630	6.630	8.05

Table D

Properties of the arch ring

Sec- tion	Ac I	q 1/1	Av. A.	Xn -X_{n-1}	Cos. = 1 $\frac{xn-xn-1}{ds}$	$\frac{Cos\ x}{A}$	Centers
1	1.970	1.000	5.41	4 -70	1.00	.185	1
2	2.052	.962	5.48	4.60	.980	.179	2
3	2.138	.922	5.54	4.50	.957	.173	3
4	2.226	.885	5.61	4.35	.925	.165	4
5	2.311	.853	5.68	4.25	.904	.159	5
6	2.486	.792	5.85	4.12	.875	.149	6
7	2.740	.720	6.07	3.92	.834	.137	7
8	3.151	.625	6.34	3.72	.791	.125	8
9	4.057	.486	6.81	3.52	.748	.110	9
10	5.650	.349	7.62	3.32	.707	.093	10
						1.475	

Table E

Properties of the arch ring

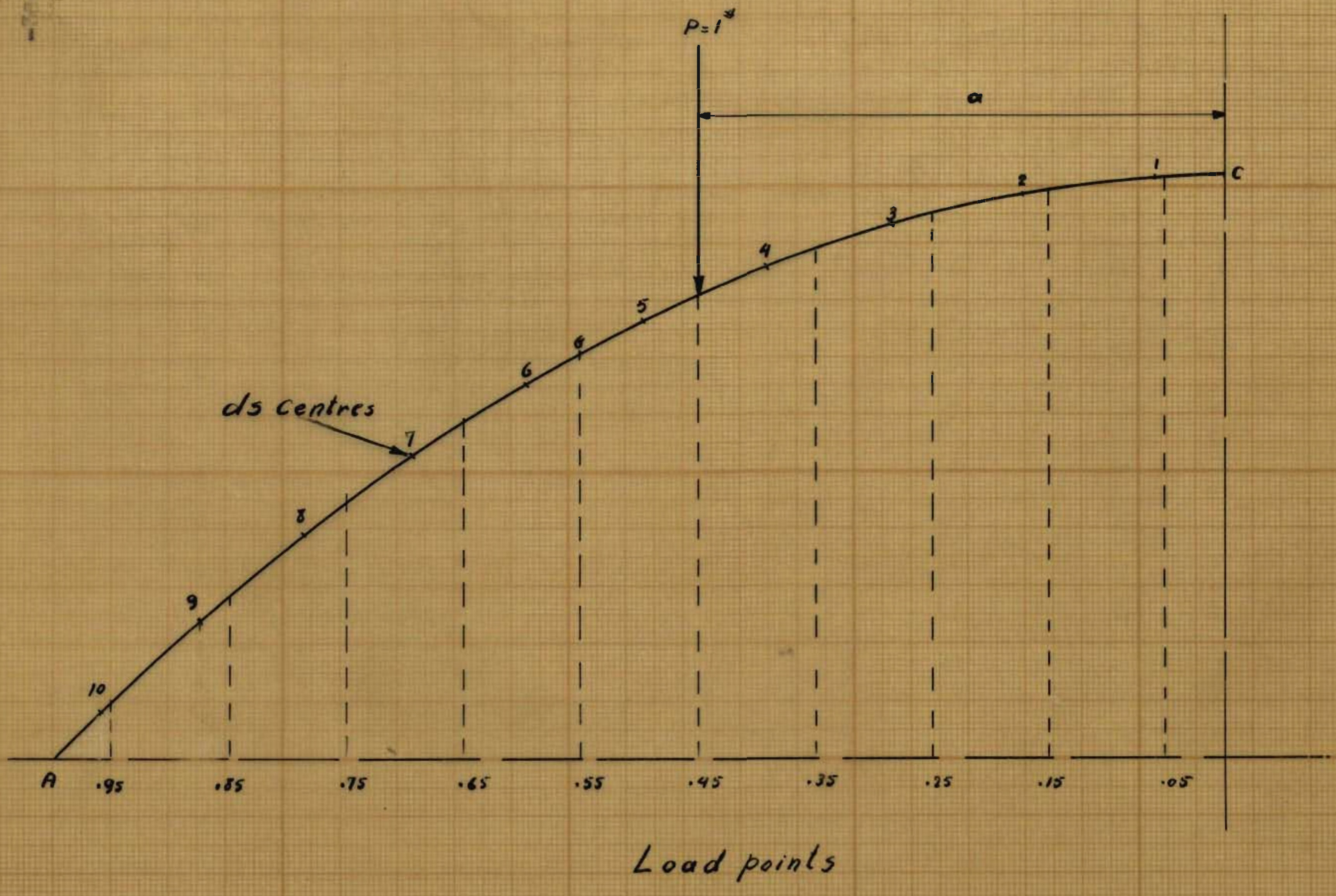
Section	q	q	qy	y_1 yq= y-yo	X	xq	x q	y, q
								17.71
1	1.000	.06	.060	- 5.82	2.35	2.35	5.5	- 5.82
2	.962	. 6	.578	- 5.28	7.0	6.73	47.	- 5.08
3	.922	1.7	1.570	- 4.18	11.6	10.70	124	- 3.86
4	.885	3.2	2.830	- 2.68	16.1	14.23	230	- 2.37
5	.853	5.2	4.440	- 0.68	20.3	17.30	352	- .58
6	.792	7.4	5.850	+ 1.52	24.5	19.40	476	+1.20
7	.720	9.9	7.130	+ 4.02	28.5	20.50	585	+2.89
8	.625	12.7	7.950	+ 6.82	32.3	20.20	655	+4.26
9	.486	15.7	7.650	+ 9.82	36.0	17.50	630	+4.77
10	.349	18.9	6.600	+13.02	39.4	13.80	541	+4.54
	7.594		44.660			142.71	3646	- .05

$$yo = \frac{yq}{q} = \frac{44.66}{7.594} = 5.88 \text{ ft:}$$

Table F'

Properties of the arch ring

Section	$y_1^2\ q$	Xy, q	q	x q	$x^2\ q$	y, q	x y, q	Section
		144.35						
1	34.0	- 13.75	7.594	142.71	3646	-.05	456.85	1
2	26.8	- 35.6	6.594	140.36	3640	+5.77	470.6	2
3	16.2	- 45.0	5.632	133.63	3593	+10.85	506.2	3
4	6.35	- 38.2	4.710	122.93	3469	+14.71	551.2	4
	29							
5	.39	- 11.8	3.825	108.70	3239	+17.08	589.4	5
6	1.82	+ 29.4	2.972	91.40	2887	+17.66	601.2	6
7	11.6	+ 82.3	2.180	72.00	2411	+16.46	571.8	7
8	29.1	+138.	1.460	51.50	1826	+13.57	489.5	8
9	47.0	+172.	.835	31.30	1171	+ 9.31	351.5	9
	51.9							
10	59.1	+179.5	.349	13.80	541	+ 4.54	179.5	10
	232.86	456,85						



P=1#
a
C
1
2
3
4
5
6
6
7
8
9
10
ds centres
A
.95
.85
.75
.65
.55
.45
.35
.25
.15
.05
Load points

Table G

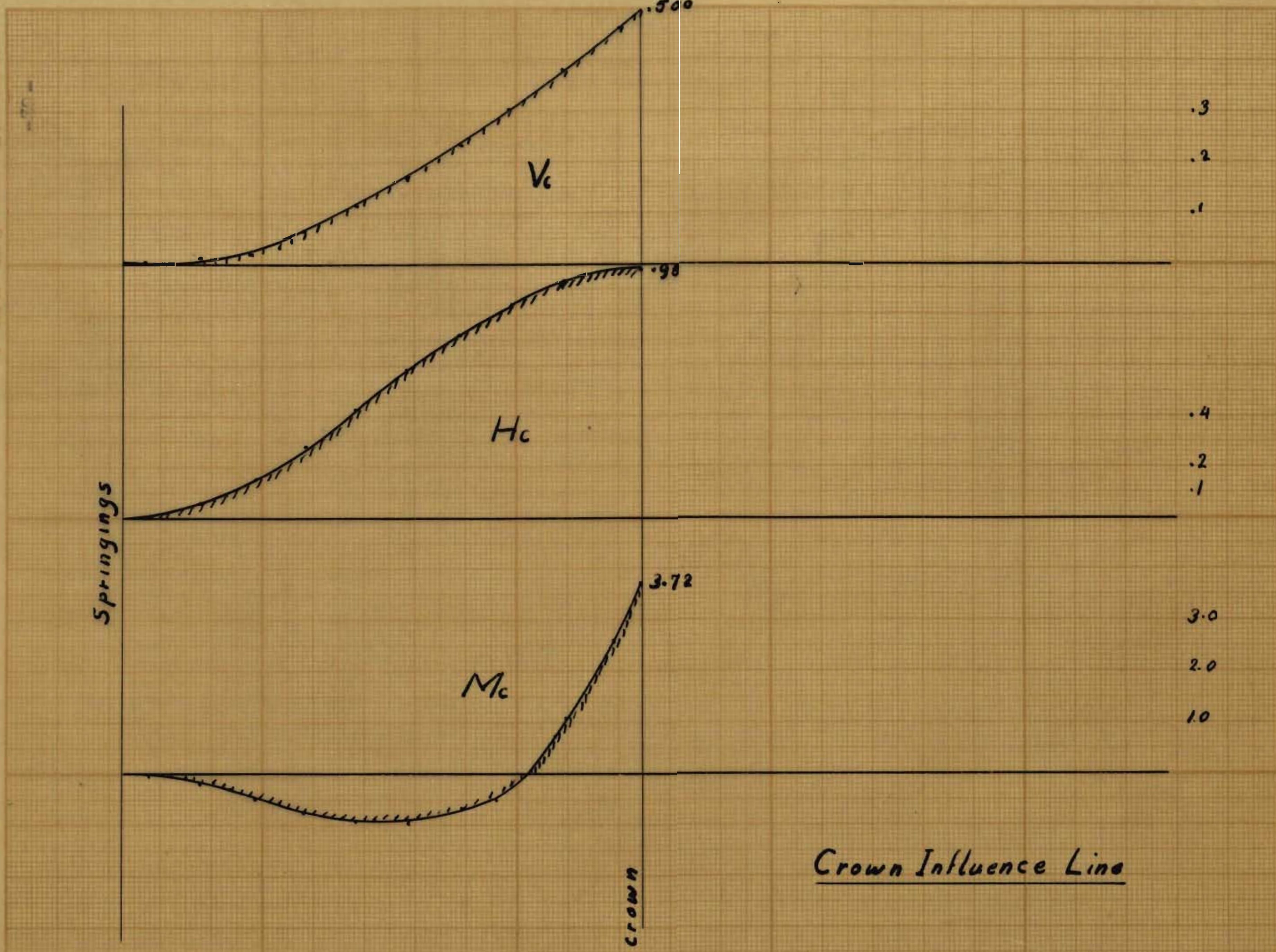
Influence line for crown

Load: $\frac{1}{2}$ a/L: (1)	a (2)	Section included: (3)	$\sum_A^D x y_{q}$ (4)	$a \sum_A^D y_q$ (5)	H_c (6)	$\sum_A^D x q$ (7)	$a \sum_A^d q$ (8)	$\frac{(7)-(8)}{15.1}$	$\frac{(7)\times(8)}{15-1}$
crown	0	1-10	456.85	0.00	.98	142.7	0	9.44	crown
.05	2.05	1-10	456.85	-0.10	98	142.7	15.6	8.41	0.05
.15	6.15	2-10	470.6	35.6	.93	140.4	40.6	6.60	.15
.25	10.25	3-10	506.2	111	.84	133.6	51.6	5.44	.25
.35	14.35	4-10	551.2	211	.72	122.9	67.6	3.65	.35
.45	18.45	5-10	589.4	314	.59	108.7	70.5	2.54	.45
.55	22.55	6-10	601.2	399	.43	91.4	67.0	1.62	.55
.65	26.65	7-10	571.8	439	.28	72.0	58.1	.93	.65
.75	30.75	8-10	489.5	217	.155	51.5	45.0	.44	.75
.85	34.85	9-10	351.5	325	.057	31.30	29.2	.15	.85
.95	38.95	10	179.5	177	.0054	13.80	13.6	.02	.95

Table H

Influence line for crown

Loads a ½ L	a	5.88 Hc (10)	Mc (9)-(10) (11)	A X^2q (12	a x q (13)	VC (12)- (13) / 7292 (14)	D A
Crown	0	5.72	+ 3.72	3646	0.00	.500	1-10
.05	2.05	5.72	+ 2.69	3646	293	.460	1-10
.15	6.15	5.50	+ 1.10	3640	865	.380	2-10
.25	10.25	4.97	- 0.37	3593	1400	.300	3-10
.35	14.35	4.30	- 0.65	3469	1764	.233	4-10
.45	18.45	3.45	- .91	3239	2010	.168	5-10
.55	22.55	2.53	- .91	2887	2060	.113	6-10
.65	26.65	1.67	- .74	2411	1920	.067	7-10
.75	30.75	.92	- .48	1826	1690	.0186	8-10
.85	34.85	.33	P .18	1171	1090	.0111	9-10
.95	38.95	.03	- .01	541	508	.0045	10



Crown Influence Line

$$Mc = \frac{-\sum_A^c (mR - mL) x q}{2\sum_A^c q} - Hc\, y_0$$

$$Hc = -\frac{\sum_A^c (mR + mL) y_1 q + \frac{wtlE}{ds_1/I_1}}{2\left[\sum_A^c y_1^2 q + I_1 \sum_A^c \frac{\cos\alpha}{A}\right]}$$

$$Vc = \frac{\sum_A^c (mR - mL) x q}{7292}$$

$$Hc = -\frac{\sum_A^c (mR + mL) y_1 q}{468}$$

$$Mc = \frac{-\sum_A^c (mR + mL) q}{15.1} - 5.88$$

Influence line Formulas.

$$\sum_A^c (mR - mL) = -\sum_A^D mL \quad . \qquad \text{but } mL = -(x-a)$$

$$\sum_A^c (mR + mL) = -\sum_A^D (x-a)$$

$$V_c = \frac{\sum_A^D (x-a)\, qx}{7292} = \frac{\sum_A^D x^2 q - a\sum_A^D qx}{7292}$$

$$H_c = \frac{\sum_A^D x y_1 q - a\sum_A^D q y_1}{468}$$

$$M_c = \frac{\sum_A^D xq - a\sum_A^D y}{15.1} - 5.88\ ft.$$

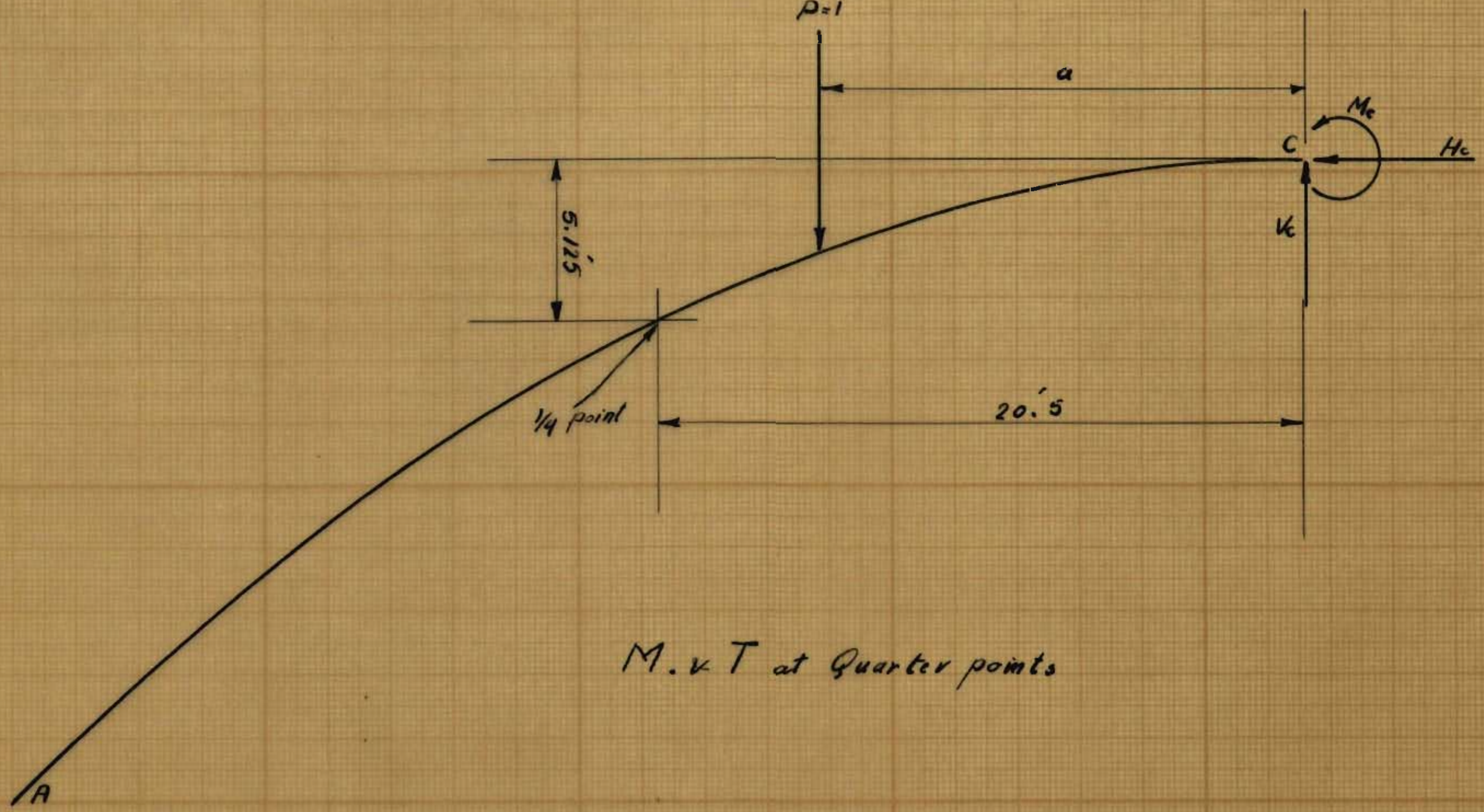
11. Influence lines for quarter points.

The results of the preceding tables with Diagram will help to get the M, H, at the quarter points due to any load on the ring after developing the following formulas.

A unit load from C to D.

$$M'/4 = Mc + 5.13\, Hc + 20.5\, Vc - (20.5 - a)$$

$$T'/4 = Hc \cos\alpha\, 1/4 + (1 - Vc)\sin\alpha\, 1/4$$



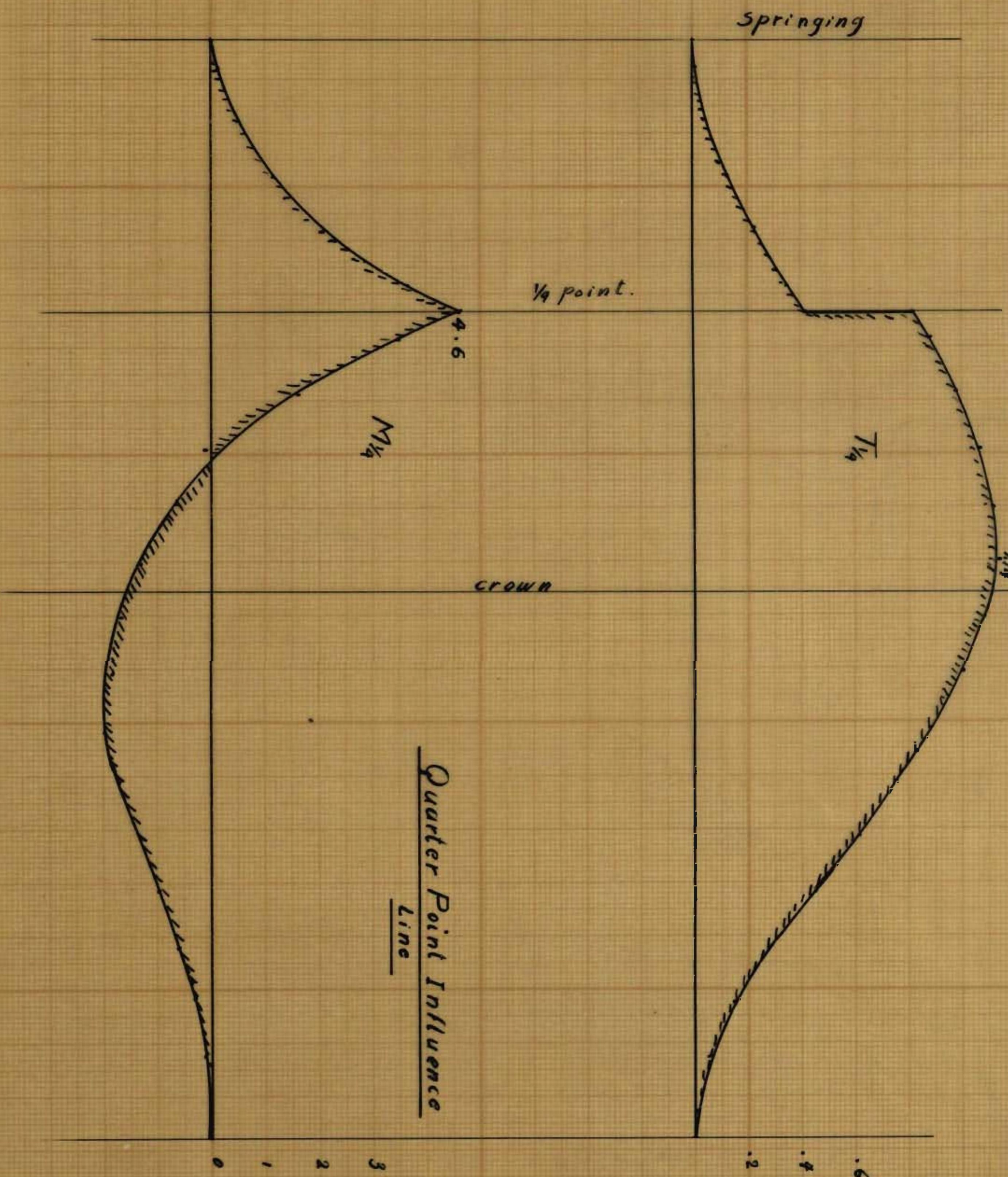
M. v T at Quarter points

Table I

Influence line for quarter paints

	Local paint	Mc	5.13 Hc 5.125	Vc	20.5 Vc	- (20.5 - a)	M 1/4
A	.95	-.01	.023	+. 0045	+ . 092	-- - -	+ .105
	.85	-.18	.293	+. 0111	+ . 228	-- - -	+ .503
	.75	-.48	.800	+. 0186	+ . 381	-- - -	+1.133
	.65	-.74	1.44	+. 067	+ 1.38	-- - -	+2.08
	.55	-.91	2.21	+.113	+ 2.31	-- - -	+3.61
¼ pt	.50	-.91	2.63	+.140	+ 2.86	000	+4.58
	.45	-.91	3.04	+.168	+ 3.45	-3.05	+3.49
	.35	-.65	3.70	+.233	+ 4.77	-6.15	+1.67
	.25	-.37	4.33	+.300	+ 6.15	-10.25	- .14
	.15	+1.12	4.80	+.380	+ 7.80	-14.35	- .63
	.05	+2.69	5.01	+.460	+ 9.401	-18.45	-1.35
C	Crown	+3.72	5.01	+.500	+10.25	-26.5	-1.42
	.05	+2.69	5.01	-.460	-9.40	- - - -	-1.70
	.15	+1.12	4.80	-.380	-7.80	- - - -	-1.88
	.25	- .37	4.33	-.300	-6.15	- - - -	-2.19
	.35	- .65	3.70	-.233	-4.77	- - - -	-1.72
	.45	- .91	3.04	-.168	-3.45	- - - -	-1.32
	.55	- .91	2.21	-.113	-2.31	- - - -	-1.01
	.65	- .74	1.44	-.067	-1.38	- - - -	- .68
	.75	- .48	.800	-.0185	-.381	- - - -	.29
	.85	- .18	.293	-.0111	-.228	- - - -	-.105
B	.95	- .01	.023	-.0045	-.092	- - - -	-.079

Hc Cos x	(1-Vc)	(1-Vc) sinx	Vc sinx	T $\frac{1}{4}$
.0041	----	----	+ .0019	.0022
.0517	----	----	+ .00470	.0470
.140	-----	----	+ .0078	.1322
.254	----	----	+ .0283	.226
.390	----	----	+ .0475	.343
.462	.860	.364	+ .059	(.403
				(.826
.535	.832	.353	----	.888
.651	.767	.324	----	.975
.760	.700	.296	----	1.056
.842	.620	.262	----	1.104
.890	.540	.228	----	1.118
.890	.500	.211	.24	1.101
.890	----	----	-1.194	1.084
.842	----	----	-.160	1.002
.760	----	----	-.127	.887
.651	----	----	-.098	.749
.535	----	----	-.071	.606
.390	----	----	-.0475	.437
.254	----	----	-.0283	.282
.140	----	----	-.0078	.149
.0517	----	----	-.0047	.056
.0041	----	----	-.0019	.006


Springing
1/4 point.
4.6
M1/4
T1/4
crown
Quarter Point Influence
Line
0
1
2
3
.2
.4
.6

A unit load from A to D.

$M\ 1/4 = Mc + 5.13\ Hc + 20.5\ Vc.$

$T\ 1/4 = Hc \cos\alpha\ 1/4 - Vc \sin\alpha\ \frac{1}{4}$

$\cos\alpha\ \frac{1}{4} = .904 \qquad 1/4 = 25°$

Now making the unit load travel all along the arch the results are put in a tabular form in table G.

12) Influence linnes for springing

With diagram and the preceding results we can get Ms, Hs, for any unit load on the arch axis, after deriving the following formulas.

A unit load from C to A

$Ms = Mc + 20.5\ Hc + 41\ Vc - (41 - a)$

$Hs = Hc \cos\alpha_s + (1 - Vc) \sin\alpha_s$

A unit load from C to B.

$Ms = Mc + 20.5\ Hc + 41\ Vc$

$Hs = Hc \cos\alpha_s - Vc \sin\alpha_s$

Now making the unit load travel all along the arch axis the results are put in a tabular form in table H, and in a graphical form in Diagram =

13- Dead Load M. and H.

All computations are put in a tabular form in TableI

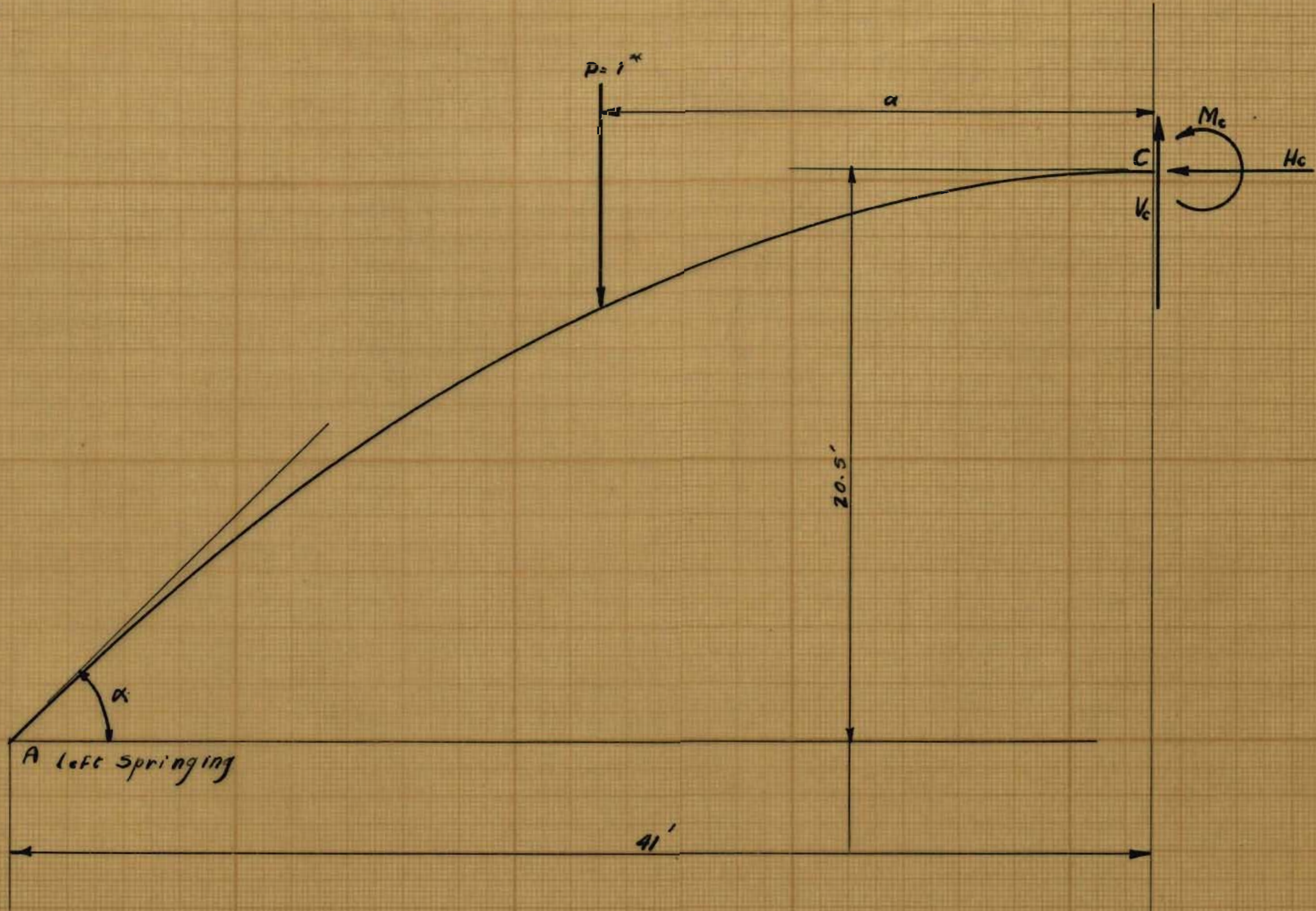
14 - Temperature effett.

Fall 60°

Rise 20°

$W. = 10^{-6} \times 6$

$E = 1/12 \quad 30000000\ lbs/ft.^2$



M_s and T_s

Table J

Influence Line for springings

	Load paint	Mc	205. Hc	Vc	41 Vc	-(41-a)	Ms
A	.95	-.01	.1110	+.0045	.184	- 2.05	-1.76
	.85	-.18	1.17	+.0111	.456	- 6.15	- 4.70
	.48	-.48	3.17	+.0186	.762	-10.25	- 6.80
	.65	-.74	5.75	+.067	2.76	-14.35	- 7.78
	.55	-.91	8.82	+.113	4.65	-18.45	- 5.95
	.45	-.91	12.12	+.168	6.90	-22.55	- 4.44
	.35	-.65	14.80	+.233	9.54	-26.65	- 3.00
	.25	-.53	17.23	+.300	12.30	-30.75	- 1.75
	.15	+.1.12	19.00	+.380	15.60	-34.85	+ 1.00
	.05	+ 2.69	20.10	+.460	18.80	-38.95	+ 2.74
C	Crown	+ 3.72	20.10	+.500	20.50	-41.00	+ 3.32
	.05	+ 2.69	20.10	-.460	-18.80	- - - - -	+ 4.00
	.15	+ 1.12	19.00	-.380	-15.60	- - - - -	+ 4.40
	.25	- .53	17.23	-.300	-12.30	- - - - -	+ 4.50
	.35	- .65	14.80	-.233	- 9.54	- - - - -	+ 4.60
	.45	- .91	12.12	-.168	- 6.90	- - - - -	+ 4.31
	.55	- .91	8.82	-.113	- 4.62	- - - - -	+ 3.29
	.65	- .74	5.75	-.067	- 2.76	- - - -	+ 2.25
	.75	- .48	3.17	-.0186	-.762	- - - -	+ 1.93
	.85	- .18	1.17	-.0111	-.456	- - - - -	+ .53
B	.95	-.015	.111	-.0045	-.184	- - - - -	+ .01

Hc cos x	(1 - Vc)	(1-Vc) sinx	Vc sin x	Ts	
.0038	.996	.704	- - - -	.708	.95
.040	.989	.699	- - - -	.739	.85
.110	.981	.695	- - - -	.805	.75
.198	.933	.660	- - - -	.858	.65
.304	.887	.628	- - - -	.932	.55
.418	.832	.590	- - - -	1.008	.45
.510	.767	.541	- - - -	1.051	.35
.595	.700	.495	- - - -	1.090	.25
.659	.620	.439	- - - -	1.098	.15
.692	.540	.382	- - - -	1.074	.05
.692	.500	.354	. 354	1.046	crown
.692	----	----	-. 325	1.017	.05
.659	----	----	-. 269	.920	.15
.595	----	----	-. 212	.907	.25
.510	----	----	-. 165	.675	.35
.418	----	----	-. 119	.537	.45
.304	----	----	-. 080	.384	.55
.198	----	----	-.0475	.246	.65
.110	----	----	-.0132	.123	.75
.040	----	----	-.0079	.048	.85
.0038	----	----	-.0032	.007	.95

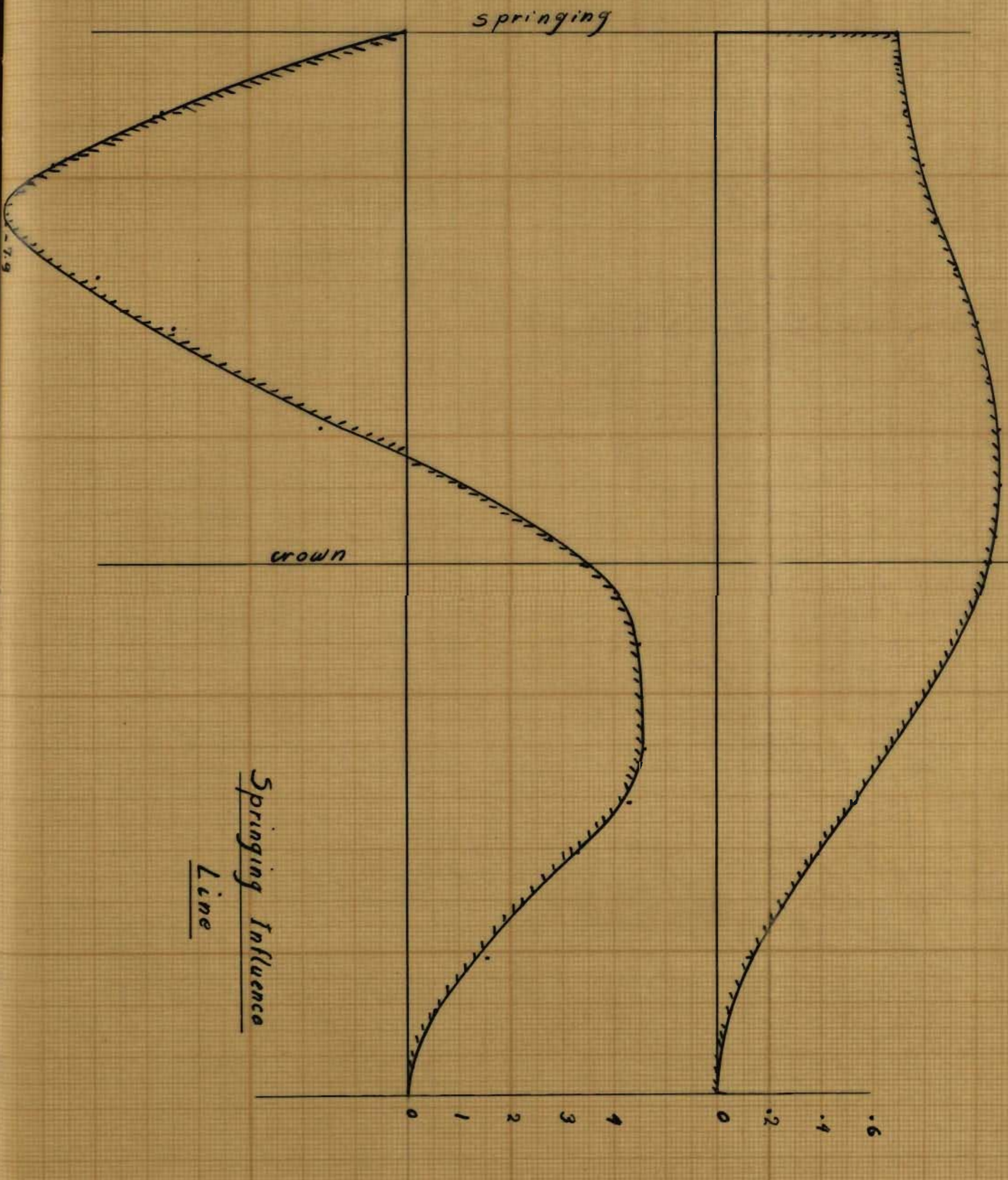

springing
crown
-7.9
0
1
2
3
4
0
.2
.4
.6
Springing Influence Line

Table K

Dead Load Moments and thrusts.

Section		Area	Weigh AX4.7x x140=1000 A	Ordin. Mc	Mc	Crown ord Tc	Tc.	Ord. $M\frac{1}{4}$	$M\frac{1}{4}$
A	10	7,62	7620	-.01	-76.2	.01	76.2	+ .1	+ 762
	9	6.81	6810	-.15	-1020	.04	273	+ .45	+3060
	8	6.34	6340	-.35	-2250	.11	700	+ .90	+5700
C3	C3	----	43600	-.60	-26200	-2	8720	+1.6	+70000
	7	6,07	6070	-.65	- 3650	.21	1280	+1.60	+9700
	6	5,85	5850	-.85	- 5000	.36	2100	+2.80	+16400
	5	5,68	5680	-.95	- 5400	.52	2960	+4.4	+25000
C2	C2	----	39600	-.91	-36100	.59	23500	+3.4	+135000
	4	5,61	5610	-. 8	- 4500	.67	3760	+3.1	+11800
	3	5,59	5540	-. 5	- 2780	.81	4500	+. 3	+ 1660
C1	C1	----	35700	0,00	0,000	.87	31100	-.25	- 9000
	2	5,48	5480	+.75	+ 9100	.92	5050	-.6	- 3300
	1	5,41	5410	+.2.60	+14100	.98	5300	-1.3	- 7100
	Crown	----	31800	+ 3.72	+11900	.98	31300	-1.6	-51000
	1	5,41	5410	----	----	----	----	-1.8	- 9780
	2	5,48	5480	----	----	----	----	-2.0	-11000
C1	C1	----	35700	----	----	----	----	-2.0	-71500
	3	5,54	5540	----	----	-----	-----	-1.90	-10500
	4	5,61	5610	----	----	----	----	-1.6	-900
C2	C_2	----	39600	----	----	-----	----	-1.35	-56700
	5	5,68	5680	----	----	----	----	-1.2	- 6800
	6	5,85	5850	----	----	----	----	-. 8	- 4700
	7	6,07	6070	----	----	----	----	-. 5	- 3040
C3	C_3	----	43600	----	----	----	----	-.41	-17100
	8	6,34	6340	----	----	----	----	-.2	- 1270
	9	6,81	6810	----	----	----	----	-.1	- 680
B	10	7,62	7620	----	----	----	----	-.05	- 380
					-18000#		-210000 #		+11000

Mc = - 18000# Tc = 210 000 # $M\frac{1}{4}$ = 11000#

ord. T ¼	T ¼	Ord. Ms	Ms	Ord.Ts.	Ts (Springing)	
0.00	zero	-1.5	-11400	.708	5400	10
.025	170	-4.0	-27900	.730	4970	9
.085	540	-6.0	-38200	.772	4920	8
.17	7400	-7.5	-328000	.820	35900	C3
.18	1090	-7.8	-47500	.830	5050	7
. 3	1760	-6.9	-40500	.900	5250	6
.83	4720	-5.4	-30800	.975	5550	5
.88	35000	-4.7	-186000	1.00	39600	C2
.95	5350	-3.7	- 20800	1.030	5780	4
1.04	5780	-1.7	- 9420	1.08	6000	3
1.08	38500	-.4	- 14300	1.09	39000	C1
1.10	6000	+.5	+ 2750	1.09	6000	2
1.14	6140	+2.7	+ 14600	1.075	5820	1
1.11	35300	+3.5	+112000	1.05	33500	Crons
1.08	5500	+4.0	+ 21800	1.02	5530	1
.96	5270	+4.5	+ 24600	.90	4940	2
.92	33000	+4.5	+161000	.84	30000	C1
.84	4660	+4.6	+ 25500	.76	4220	3
.70	3940	+4.5	+ 25300	.61	3430	4
.60	23900	+4.2	+167000	.52	20600	C2
.54	3070	+3.9	+22200	.46	2600	5
.37	2170	+2.9	+17000	.32	1940	6
.21	1380	+1.8	+10950	.20	1220	7
.20	8700	+1.7	+74200	.18	7550	C3
.11	700	+1.0	+ 6340	.10	634	8
.04	270	+.4	+ 2700	.03	205	9
0.00	000	+.05	+ 380	.003	23	10
	240000 #		-66000 #		286000 #	

T ¼ 240000# Ms = - 66000# Ts = 286000#

At the crown $\quad Mc = - 5.88\ H_c$

$Hc = \frac{wt/L\ E\ I_1}{468\ ds1} = tK$

$= \frac{wL\ E\ I_1}{ds_1 x\ 468 t}\ t = 152 \times t$

Rise 20° Hc = 152 x 20 = 3040

Fall 60° Hc = - 152 x 60 = -9120 Lbs.

Rise 20° Mc = - 5.88 x 3040 = -17900

Fall 60° Mc = 5.88 x 9100 = 53600 Lbs/ft

at $\frac{1}{4}$ points $\quad M\frac{1}{4} = Mc + 5.13\ H_c$

$H\frac{1}{4} = Hc\ \cos\ \frac{1}{4}$

Rise 20° $M\frac{1}{4}$ = - 2300 Ft/lbs.

$H\frac{1}{4}$ = + 2750 Lbs.

Fall 60° $M\frac{1}{4}$ = 6800 ftlbs.

$H\frac{1}{4}$ = - 8240 Lbs.

At springings :

Ms = Mc + 20.5 Hc

Hs = Hc cos $\quad$ Cos = .707

Rise 20 ° Ms = - 17900 + 20.5 x 3040 = 44500 Lbsft

Hs = + 2150 lbs.

Fall 60° Ms = - 133400 ftlbs.

Hs = - 6450 lbs.

15- Concentrated L.L. M, H.

L.L. = A = 38000 Lbs. = B = 15.500 Lbs.

A to B = 13.1 ft. $\quad$ B to A' = 19.7 ft.

See influence line Diagrams

1- Positive moment : Mc = 3.72 x 38000 = + 141 000

2- Negative moment : Mc = 2(38000 x .91 + 15500 x .6 x 9/12

= - 83200 ftlbs

1'- Hc = .98 x 38000 = 37200 lbs.

2'- Hc = 2 (38000 x .59 + 15500 x .2 9/12) = 49 500 Lbs.

at 1/4 point

1- Positive moment :

Mc = (3.4 x 38000 + 1.6 x 15500 x 9/12) = + 147600 ftlbs

Hc = .88 x 3 8000 x 1.6 x 11 600 = 52 000 lbs.

2- Negative moment : M 1/4 = 2.0 x 38000 + 1.6 x 11600

+ 15500 x .41 = - 90 950 ft lbs.

H 1/4 = 38000 (.92 + 11600 x 1.11 + 15500 x .2) = 51000

At springing

1 - Positive moment Ms = 38000 x 4.2 + 1.7 x 11 600 + 3.5

x 15500 = + 233900 ftlbs.

Hs = 38000 x.52 + 11600 x .18 + 1.05 x 15500 = 38900 lbs.

2- Negative moment

Ms = - (7.5 x 38000 + 11600 x 4.7) = - 339500 ftlbs

Hs = 38000 x .82 + 11600 x 1.00 = 42 600 lbs.

16- Uniform L. L. M. H.

All calculations are put in table J.

17- Combined Moments and thrust

All put in table K.

18- Fibre stresses

All computations are put in table N. The formulas and dia-

Table L

Uniform L. L. Moments and thrusts

Paint	Load	Crown							
		Ord Mc	+ Mc	-Mc	ord.Hc	+Hc	+Hc -Mc	ord.M $\frac{1}{4}$	+ M $\frac{1}{4}$
C 3	27700	-.6	- - -	16600	.2	- - -	5520	+ 1.6	44300
C 2	24200	-.91	- - -	22000	.59	- - -	14300	+ 3.4	82300
C 1	21800	0.00	000	~~55600~~	.87	19000	19000	- .25	- - -
Crown	20700	+3.72	77200	----	.98	20400	----	-1.6	- - -
C 1	21800	000	000	000	.87	19000	1900	-2.0	- - -
C 2	24200	-.91	---	22000	.59	---	14300	-1.35	- - -
C 3	27700	-.6	---	16600	.2	---	5520	-.41	- - -
			+77200 + Mc	-77200 -Mc		58 400 +Mc	77640 -Mc		+12600

							Springing		
$-M\frac{1}{4}$	ord $H\frac{1}{4}$	$+H\frac{1}{4}$ $+M\frac{1}{4}$	$+H\frac{1}{4}$ $-M\frac{1}{4}$	Ord Ms	+Ms	-Ms	ord Hs	+Hs +Ms	+Hs -Hs
----	.17	4720	----	-7.5	---	208000	.82	---	22800
----	.88	21300	---	-4.7	---	114000	1.00	---	24200
54700	1.08	----	23600	-.4	---	8750	1.09	---	23900
33200	1.11	---	23000	+3.5	72700	---	1.05	21800	----
43600	.92	---	20100	+4.5	98000	---	.84	18400	---
32700	.60	---	14500	+4.2	102000	---	.52	12600	---
11400	.20	---	5500	+1.7	47100	---	.18	5000	---
175600		26020	86700		+319800	330750		57800	70900

Table M

Combinued Moments and thrusts.

			Dead Load	N. Load Concr.	N. Load Unifor.	Tempor. condi- tion	Temp. effect.	Total
Crown	Positive Moment	+ Mc Tc	-18 000 +210 000	+141 000 + 37 000	+77 000 +58 400	Fall 60°	+ 53 600 - 9 200	+ 176 600 + 238 000
	Negative Moment	- Mc Tc	-18 000 +210000	- 83 000 + 49 500	-83 000 +77 600	Riser 20°	- 17 900 + 3 100	- 119 000 + 262 600
$\frac{1}{4}$ Points	Positive moment	+M $\frac{1}{4}$ T $\frac{1}{4}$	+11 000 +240000	+147 600 + 52 000	+126600 + 26000	Fall 60°	+ 6 800 - 8 300	+ 165 100 + 283 700
	Negative Moment	-M $\frac{1}{4}$ T $\frac{1}{4}$	+11 000 +240000	- 91 000 + 51 000	-175600 + 86700	Rise 20°	- 2 300 + 2 300	- 167 000 + 329 000
Sprin- ging	Positive Moment	+Ms Ts	-66 000 +286 000	+ 234000 + 39000	+320000 + 57800	Rise 20°	+ 44 500 + 2 200	+ 299 000 + 346 000
	Negative Moment	-Ms Ts	-66 000 +286 000	-340000 + 42600	-331000 + 70900	Fall 60°	-133 400 - 6500	- 539 000 + 322 000

Table N

Fibre Stresses

Section	M.	T.	E	h	e/h	h/e	P	pn	d'/h	Case	C	Fc	Fs.
Crown	+176600	+238000	.743	2	.372	2.68	.67	.08	.083	II	7.4	9.00	9500
¼ point	-167000	+329000	.507	2.2	.222	4.5	.67	.08	.073	II	8.4	750	8300
Springing	-539000	+322000	1.67	3.	.656	1.8	.67	.08	.055	II	7.2	1120	9700

---------- ----------

grams used are those of (Turneaure, Diagrams 12 to 25) we can see that at the assumed steel ration, the steel stresses at crown and quarter points are O.K. but at springings are a little bit high by taking P = 1.22 Pn = .147 we get fc = 890 fs = 9700.

This change will not affect much the properties of the arch and is permissible as (Taylor and Thompson) say in their book (concrete plain and reinforced).

D. Design of Abutment

Surcharge : The surcharge due to trucks is very small since the fill distributes uniformly over a quite big area at the bottom.

$$S = \frac{P}{A} = \frac{48000}{(24-2)x28} = 38 \text{ Lbs/ft2}$$

which means .38 ft of fill which is negligible.

Width of retaining wall =	30 ft.
Depth " " " =	28 ft.

D.L. Reaction at top of wall :

6 x (9 x 12 + 30) + 140 =		970 lbs/ft.
970 x 20	=	19400 lbs.
Side walls = 6 x 2000	=	12000 "
Other side wall	=	12000 "
		43400 lbs say 35 kips

+ surcharge + side walks = 1460 lbs/ft

The L.L. uniform = 1150 lbs/ft.

Concentrated = 2700 lbs/ft.

1st Retaining wall: under transverse beam :

h= 26 ft. total h = 28 ft (See fig.)

K = .22

$P = Kw \frac{h2}{2}$ = .22 x 100 26x26/2 = 67000 ftlbs.

N = 12 fc. 800 fs = 18000 R = 148

$d2 = \frac{67000 \times 12}{12 \times 148}$ = 453 d = 20.5 in = 21"

As = 0.0095 x 20.5 x 12 = 2.34 sq. in.

Results : USE

d = 21 + 3 = 24 in.

1" round bars @ 4" interval (= 2.36)

Shearing :

P = 7750 lbs. $v = \frac{7750}{12x21x.875}$ = 35 psi o.k.

Position of Resultant

Taking a base of 11 ft. (See fig)

$P = .22 \times 100 \times \frac{.28 \times 28}{2}$ = 8640 lbs.

Ma overturning = $8640 \times \frac{28}{3}$ = 80600 ftlbs.

Stabilizing :

1 =	2 x 26 x 144 x 3 =	22500 ft.lbs.
2 -	7 x 26 x 100 x 7.5 =	136000 " "
3)	2 x 11 x 144 x 55 =	17400 " "
4)	1460 x 2.5 =	3600 " "
		179500 ft.lbs.

Vertical forces :

(1) 2 x 26 x 144 = 7500

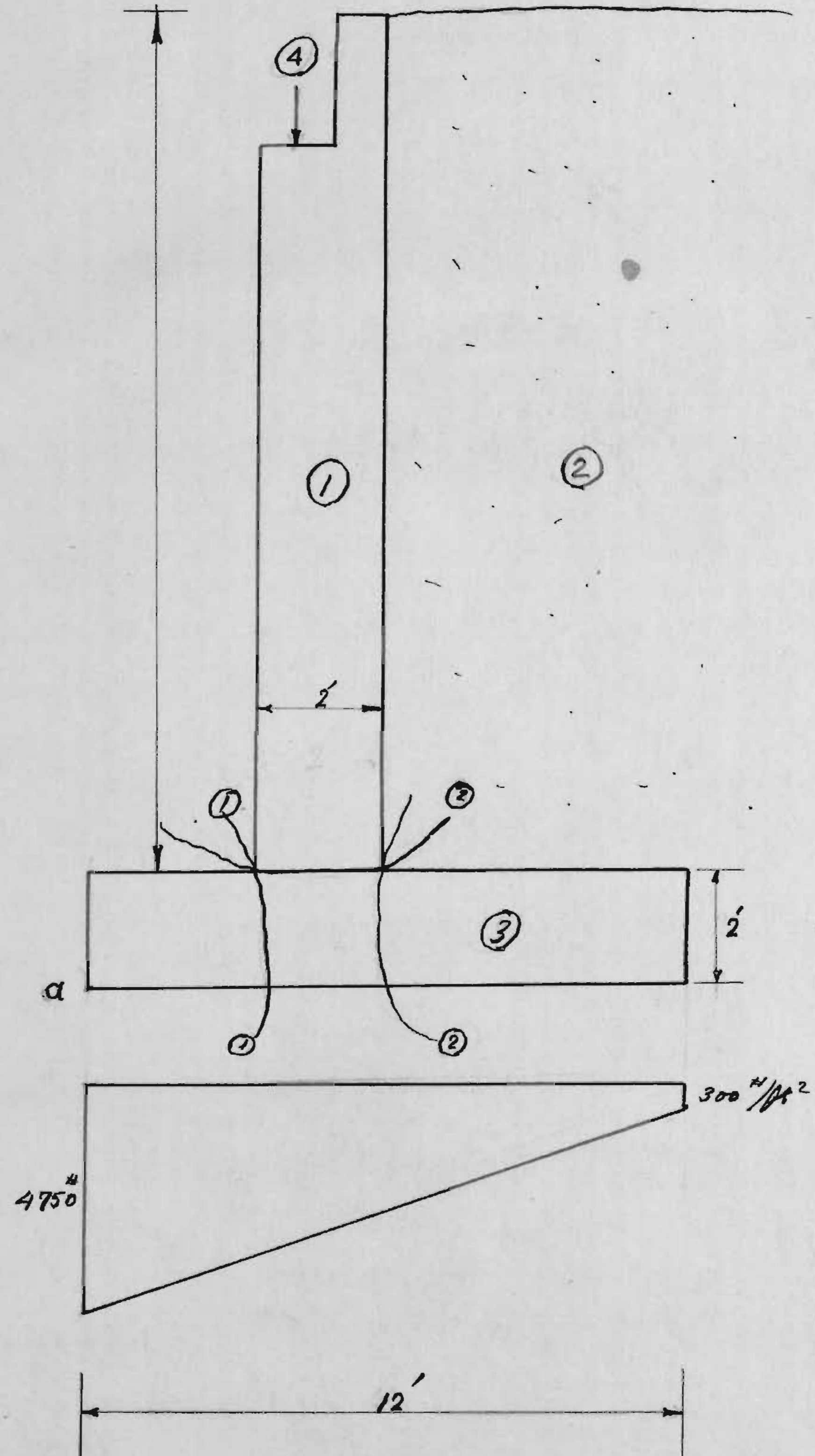


fig.

(2) 7 x 26 x 100 = 18200

(3) 2 x 11 x 144 = 3170

(4) 1460 = 1460

V = 30330 say 30350 lbs.

increasing the base to 12 ft. we have :

Ma = Overturning is the same

Stabilizing is 179560

V x 1 30350

20900 ft. lbs.

Factor of safety : $\frac{209850}{80600}$ = 2.6 o.k.

= $\frac{129000}{30350}$ = 4.25 ft. 12/3 = 4 ft.

That means that the resultant falls in the middle third.

The excentricity e = 6 - 4.25 = 1.75 ft.

Mc at centre = 1.75 x 30350 = 53200 ft. lbs.

S = P/A ± Mc/I P = V = 30350 lbs.

A = 12 sq. ft.

C = 6 ft.

I = $\frac{bd3}{12}$ = 144 ft^4

S = $\frac{30350}{12}$ ± $\frac{35200 \times 6}{144}$ = (+4750 lbs/ft2
)+ 300 " "

Sliding : 30500 x .6 = 18200 lbs.

While P = 8640 lbs.

Section (1) - (1) (see Fig)

$\frac{x}{x + 12} = \frac{300}{4750}$ $\quad x = 2900$ lbs.

$\frac{7.8}{12.8} = \frac{x}{4750}$ $\quad$ or $\quad x = 3600$ lbs.

M (1)- (1) = up 3600 x 3.1 x 1.5 = 16700

1150 x 3/2 x 2/3 x 3 = 3600

20300 ft.lbs

down down = negligible

M (2)-(2) up 300 x 7 x 3.5 = 7350

$\frac{2600}{2}$ x 7 x 7/3 = 21200

28 550 ft.lbs

down 7 x 28 x 100 x 3.5 = - 68.500

28.600

40.000 ft.lbs.

$d2 = \frac{40000 \times 12}{12 \times 148} = 270 \quad d = 16.5$

but for the sake of big shear take d = 20"

As = 0.0063 x 20 x 12 = 1.5 sq. in.

<u>Results :</u> USE :

d = 20 + 4 = 24"

3/4 " round bars @ $3\frac{1}{2}$ " in-terval = (1.51)

(1)-(1) Section : As = 20 x 12 x .0035 = .84 sq.in.

<u>Results :</u> USE : d = 20 + 4 = 24 in.

$\frac{1}{2}$" round bars @ $3\frac{1}{2}$ "interval (=.86)

<u>Shearing :</u> (1) (1)

$v = \frac{13000}{12 \times 20 \times .875} = 62$ psi. safe.

(2)-(2) Section down $28 \times 7 \times 100 =$ 19600

up $\frac{.3000 \times 7}{2} =$ 10500

9000 lbs.

That is safer even.

2nd. retaining wal (see fig.)

h = 22 ft.

L = 22 x .165 = 8.95 ft. Say 9 ft. x = 3 ft

Thickness of wall :

$$M = \frac{.22 \times 100 \times 22 \times 22 \times 22}{6} = 39000 \text{ ft. lbs.}$$

$$R = \frac{M}{bd2} = \frac{39000 \times 12}{12 \times 18 \times 18} = 120$$

P = 0.0076 As = 12 x 18 x 0.0076 = 1.64 sq.in

Results : USE : L = 9 ft. d = 18 in.

7/8" Round bars at 4" interval.

3 d retaining wall (see fig)

h = 18 ft.

L = 18 x .165 = 7.3 ft. say 7.5 ft.

x = 2.5 ft.

Thickness of wall.

$$M = \frac{.22 \times 100 \times 18 \times 18 \times 18}{6} = 21400 \text{ ft. lbs.}$$

R = 148 P = 0.0095

As = 12 x 12 x .0095 = 1.36

Results : USE : d= 12 in. L = 7.5 ft. 3/4" R.B.@ 4" interval.

4th. Abutment (for arch ribbes)

See the graphical solution in plate.

A- Dead load case.

P_1 = 5100 Lbs. P_2 = 11700 Lbs. P 3 7700 Lbs.

P 4 = 10400 lbs. P 5 = 82000 Lbs.

B) Max. negative Moment case.

P1 , P2, P3, P4 the same as before

P5 = 322 : 3.5 = 92000 Lbs. e = -.67

C) Max. positive Moment case.

P1, P2, P3, P4 the same as before

P5 = 346 : 3.5 = 99000 lbs. e = + .87 ft.

D) Max. mommal thrust.

P1, P2, P3,P4 are the same as before

P5 = 415 : 3.5 = 118000 lbs. e = -.18 ft.

Earth pressures: A) Since the resultant passes through the center the pressure is uniform and is equal to $P/A = \frac{102000}{9.5} =$ 10750 Lbs/ft^2

B- Max. negative moment. R = 112 kips e= 1.1 ft.

M = 1.1 x 112 = 123 000

$I = \frac{bd^3}{12} = 71.3$ C = 4.75

$S = P/A \pm \frac{Mc}{I} = \frac{112000}{9.5} \pm \frac{123000 \times 4.75}{71.3}$

= + 3600 Lbs/ft2

\+ 20000 Lbs/ft2

C- Max. positive moment. R = 119 e = 1.1

M = 1.1 x 119 = 131000

$S = \frac{119000}{9.5} \pm \frac{131000 \times 4.75}{71.3} = 12500 \pm 8700$

S = +3800 Lbs/ft.

+21200 " "

D - Max. Thrust R = 138 e = .3

$M = .3 \times 138000 = 41\,400$

$$S = \frac{138000}{9.5} \pm \frac{41000 \times 4.75}{71.3} = \begin{matrix} +11\,800 \text{ ft2} \\ +17300 \text{ "} \end{matrix}$$

E

E- Drainage and parapets

All details and dimensions are given on plates

D- Deflection of the Bridge :

The rise at the crown is:

$$Dc = -\frac{4.7}{1.932 \times 2.5 \times 10^6 \times 144}(176000 + 5.80 \times 2.38000)\,142.7$$

$$+ (238000 \times 457)$$

$$+ \text{negligible} + 10^{-6} \times 6 \times 20 \times 20.5$$

$$Dc = \frac{4.7 \times 116 \times 10^6}{696 \times 10^6} = .78 \text{ ft.} = 9''.5$$

up word

4- Data and details

All put in Detail drawings and Plates.

III. Conclusion

My design was close enough to the specifications. But in the arch I could reduce the steel ratio at springing if I used a ratio of 1 to 2 instead of 1 to 1.5 for crown to springing thickness.

In the design of abutments I was playing always safe since the smallest overstressing ~~the~~ the foundations, would change the stresses much and on the unsafe side.

END