Epsn: 49

R E I N F O R C E D   C O N C R E T E

A R C H   B R I D G E

A T   A L H I R I   V A L L E Y

Designed by FUAD ABD-EL BAKI
B.S.C.E

-1947-
?

- C O N T E N T S -

PART III

TABLES :

1.- P R E F A C E

During summer 1946 The Lebanese public works sent a party of engineers to study the possibility of enlarging and shortening the present 5 meters road passing over the "Alhiri valley" in Al chouf district and joining Kaferhiem - Deir-El Kamer and Baakline.

The engineers confronted two possibilities.

1.- to keep the present road and widen it by three meters.

2.- to change location of this road beginning from Kafer-heim and construct the road at higher countours - This requires the construction of a bridge over the valley having a span a about 40 m. and by this way the road will be shortened by about 1500 meters;

Sentence construction?

Due to great expenses and certain political considerations.

The second possibility was rejected and the first possibility is being performed.

According to my knowledge and study the second possibility, though more expensive due to the erection of this large bridge, and the new cutting of road - ~~it~~ is more economical, useful and practical from all ~~the~~ we points of view, and I am optimistic that the second possibility will be adapted in the future.

Poor English

In my thesis I designed the bridge trying to let it meet strength economy and easthetic. ?

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2. - F O R E W ~~A~~ O R D :

~~Here~~ This is the thesis required ~~from~~ of a final year student in Engineering at the American University of Beirut.

Course number	525-526
Thesis supervisor	Prof. J.R. Osborn
Current year	1946 - 47
Presented by	F'uad-Abd-El Baki B.A.

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3.- N O T A T I O N :

A: area of sections - Left springing point

a: radial thickness

B: right springing point

b: width of rectangular beam or width of flange of T beam

b':width of stem of T beam

C: Crown point - Distance between center of wheels

c: ratio

D.L.: Dead load

D: Deflection

d: effective thickness of slab, or distance.

d': depth from compression face to center of compressed reinforcement.

E : effective width of one wheel.

E : modulus of elasticity.

e : $\frac{M}{N}$

fc: compressive unit stress.

fs: tensil unit stress. .

f'c: ultimate unit stress for concrete.

h : height of Bridge rib - or retaining wall

I : moment of inertia

L : arch rib span

l : span, of beams or slab

M : moment at any section

N : Thrust at any section

n : $\frac{Es}{Ec}$

o : sum of perimeter of bars in one set

ps: ratio of steel to effective concrete area

S : spacing of stirrups or beams spacing

T : tire width

t : thickness

u : bond stress

v : shearing unit stress

V : Shear

v': Total shear on any section after that carried by concrete.

yo: Ordinate of crown from elastic center of arch

y1: Ordinate of any point on arch axis reffered to elastic center.

w : Coefficient of expansion.

4.- References:

Cap. names

a - "Concrete plain and Reinforced" by Taylor Thompson and Smulski - Volume II, 4th edition.

b - "Theory of continuous structures and arches" by Spofford

c - "Masoury structure" by Spadling, Hyde and Robinson (second Edit.).

d - "Design of concrete structures" by Urquhart and O'Rourke

e - "Reinforced concrete Bridges" - Scott (3th edition).

f - "Theory of structure" by Timoshenko and Young.

- I. PRELIMINARY PART -

1.- Statement of the problem:

Design of a reinforced Concrete arch whose

Clear span	=	118.4'
width	=	32"
Bituminuous macadam	=	2"
Crowing ?	=	2"

Specifications

Joint code (1936) ~~specification~~ A.C.I.

H-10 Loading

fs = 18000 p.s.i.

Temperature:

fall 30° Fahr

rise 40° Fahr

coefficient of expansion : 0.000,006

2.- Choice of problem :

Having the specific site on which to construct the bridge thus leaves me a self control on the choice of:

A - appearence and details

B - allowable stress and specification

C - system of loading

D - method of Design.

A.- Appearence and details of the Bridge:

F from the contour map, a ~~visiting~~ ~~of~~ to site, we see that the bridge is to be between two roçky cliffs constituted of cretecious lime-stone and over an intermittant river (dry during summer) the problem is to chose the right kind of bridge.

a - Classification of arch bridges

We have the following types

1 - one hinged arch (hinged at crown)

2 - two hinged arch

3 - three hinged arch

4 - Fixed or hingless arches

The difference between these four types is not only of construction but also in method of Design.

According to their construction, these types of Bridges are divided into:

1 - open spandrill arches

a'- barrel type

b'- ribbed type

2 - filled spandrill arches.

Sentence construction very bad so far.

Due to economical reasons

1 - Dead load reduced by omitting the fill

2 - arch do not need to be made the full widlh of Bridge.

3 - narrow ribs coast reduced.

Very Poor grammar

Due to architectural reasons

1 - Deep valley.

2 - pittoresque of the site

picturesque

3 - aesthetic appearence.

I decide to design the bridge as a "fixed-end open spandrill, ribbed type arch bridge".

b - form of arch axis

An arch with a parabolic form and loaded with fixed load would require no steel reinforcement because the line of pressure will coincide with the arch axis - for these reas ons, it is usually to make axis conforme to dead load equilibrun polygone thru, crown and springing.

In this case I shall use the following formulea given in "Urquhart" text book. complete title

$$y = \frac{8\ rl}{6 + 5v}\ (3\ C2 + IO\ C^4r)$$ for the form of the arch axis.

c - <u>proportionning of the arch - rise</u>.

Rise is a function of the height of the roadway, span of the bridge, and water-way level - the rise in the case of my bridge is independant of water-way level therefore it is advantageous from an economical stand point to have the $r = \frac{h}{L}$ varie between $\frac{1}{4}$ and $\frac{1}{8}$ - Let us take it as $r = \frac{1}{6}$

d - <u>Bracing of arch ribs</u>

When ribs are narrow they require lateral bracing not only to increase the unsupported length of ribs but also to resist yielding to wind pressure.

e -<u>Reinforcement of arch ribs</u>

A good range of longitudinal steel reinforcement is between 0.5% and 1% of cross-section of the arch rib. Source?

These reinforcement are placed symetrically about arch axis and half of that steel area is used near each face.

f - <u>Vertical supports - colums</u>:

Live load and Dead load comming from the superstructure are transmitted to arch ribs by colums - These colums must carry the loads properly and uniformely to the arch ribs.

In the case where a row of colums is used per rib it should be placed in the center of the rib - or as the colums are poured separally from ribs, dowels should be provided in arches of same number and size used for colums. Sentence Construction Very Poor

g - Sides walk

A good side walk must have a width of 5 feet, in the case of this bridge, it is a cantiliver side walk without a bracket support and extends to the outside edge of the spandrill column. ?

h - Road-way :

The American general practice uses 9' feet per lane of track - for practical reason and specific for the site considered above the "alhiri" valley the road will be designed for 2 lanes traffic with a clear width per lane of 11 feet; therefore the distance from railing to railing is 32' feet. Punctuation

B.- Allowable stresses

a - concrete f'_c = 3000 p.s.i.

n = 10

flexure extreme fiber in compression	1200 p.s.i.
use	1000 p.s.i.

Shear

1 - Beam with no web reinforcement

bar with special anchorage	90 p.s.i.
use	80 p.s.i.
bar without special anchorage	60 p.s.i.

2 - Beam with web reinforcement

bars with special anchorage	300 p.s.i.
bars without special anchorage	I80 p.s.i.
~~used~~ use	I50 p.s.i.

Bond :

use	I20 p.s.i.

Compression :

explanatory note

Bending and compression $e > \frac{1}{6} a$	
use $0.315 f'c$	950 p.s.i.
Bent column $e < \frac{1}{6} a$	
2 use $0.265 f'c$	800 p.s.i.
Concentric colum $fc = 0.225 f'c$	675 p.s.i.
b - Fs - steel	I8000p.s.i.

[Taylor: page 453
Urquhart: " 5I5: Appendix B.]

C - System of Loading :

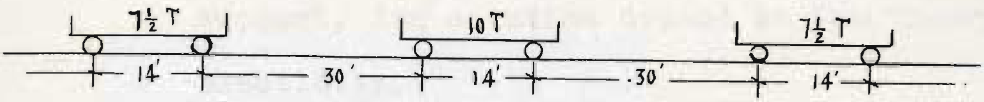
a - H-10 Loading used

The bridge to be constructed will be subjected to a midum traffic. for that the cooper H-10 loading was used.

Two systemsof loads will haveto be considered.

1 - for loaded length of 60' and more a uniforme live load of 320 # per linear foot per lane plus a concentrated load of 9000 for moment and 13000 # for shear will compose this system.

2 - for a loaded length less than 60' a system of trucks as indicated below is considered.



b- D distribution of loads.

1. When a concentrated load is placed on a Reinforced concrete slab, the load is distributed over an area larger than the actual area of contact. *Source in foot note

Case I main reinforcement parallel to direction of traffic

$E = 0.7\ L + T$ with $E = 7'$ for maximum.

Case II main reinforcement perpendicular to direction of traffic

$E = 0.7\ (2\ D + T)$.

2. Distribution of loads to longitudinal beam for bridges designed for two lanes of traffic each interior beam sustain 5? $\frac{S}{4.5}$ wheel loads. ? Source

D - - DESIGN METHOD -

For slab - beams- girders and cantiliver side walk their Design depends on the governing shear and moment - no need to explain here the theory.

For the arch bridge the folling must be determined:

Not consistent with caps

1 - the type of construction is selected: open spandrill - ribed arch -

2 - preliminary dimensions

3 - Curvature of the arch axis

4 - with fixed end arch bridge which is statically

indeterminate the 6 unkowns quantities 3 at each support, its solution depend on the theory of elasticity.

Grammar ?

4. Outline of the problem :

Having the necessary data :

1. strength of materials

2. specification of the problem.

The problem becomes simple designing and detailing of:

1 - slab system

2 - Columns

3 - Arch ribs

4 - Abutments

5 - Drainage - parapet and expansion Joints.

5. - ANALYSIS OF THE PROBLEM -

The final analysis should include :

1 - Dead load, including the rib shortining

2 - Live load

3 - Effect of change in temperature

4 - Effect of shrinkage equivalent to 10° fall.

A resume of the brief arch analysis by the elastic theory is given below.

Deflection at Crown

$$\sum_{C}^{A} \Delta x = - \sum_{C}^{B} \Delta x = \sum_{C}^{A} \frac{My \Delta s}{EI}$$

$$\sum_{C}^{A} \Delta y = \sum_{C}^{B} \Delta y = \sum_{C}^{A} \frac{Mx \Delta s}{EI}$$

Effect of temperature

$$Ho = \mp \frac{wtL}{2 \sum \frac{y_1^2}{I}} \times \frac{E}{\Delta s}$$

$$M = Ho\ y_1$$

+ for fall in temperature

- for rise in temperature

Effect of shortening:

$$Ho = + \frac{CaL}{2 \Delta s \sum \frac{y_1^2}{I}}$$

Knowing the following formulea and the method of Designing so let us devide the arch into equal divisions (20) and compute the constants of the section and other quantities.

5. Reliability of theory and result :

The results which are based upon the elastic theory are the most reliable in arch design.

Having assumed the following four conditions :

1 - arch ribs of a definit form and symmetrical about vertical axis

2 - Length of span remains unchanged

3 - inclination of arch axis at abutment is constant

4 - Level of support A and B remain constant

The accuracy of the calculation will approximate very closely the actually produced in the work, if care is taken by us to ensure the above conditions of beeing realized.

Use complete sentences

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P A R T II

D E S I G N

A - Floor system

B - Supporting column

C - Arch ribs

D - Abutment

E - Drainage and expansion Joint.

A. Design of Floor system is

Divided into the following :—

a - slab design

b - cross-beams

c - girders

d - side walks

e - ~~sides~~ side beams

a - Slab of road-way

SOURCE ?

Moment of end span = $M = 1/10\ wL^2$ *

assuming 7" 1/2" thick. of slab with 2"

of wearing surface

Dead Load per foot $= 7\ 1/2 \times \frac{I50}{12} = 90$

allowing 10 Lb for future covering $= \underline{10}$

100 Lb

Dead Load Moment $= 1/10 \times 100 \times 5^2 \times 12 = 3000$ in-Lb

effective width of one wheel :

$E = 0.7\ (2D+T) = 0.7\ (5.67 + 0.83\ \ ) = 4'.55$

Concentrated load at center of 1' strip

$\frac{8000}{4.55} = I775$ Lb

SOURCE ?

Coefficient of impact = 1/3 *

Live load moment :

$(\frac{I775}{2} \times \frac{5}{2} \times 12) \times I.35 = 35500$ in - Lb.

Total moment = 38500 in - Lb

fs = I8000 Units?

fc = I000

N.B.

n = I0

K = I57

k = 0.357

J = 0.88I

$$d^2 = \frac{38500}{12x157} = 20.2$$

d = 4.5 in; take it as 4.5 inch

with 1" insulation

Total thickness 4.5 + 1" + 2" = 7.5 inch as assumed

$$As = \frac{38500}{18000x0.88Ix4.5} = 0.502 \text{ sq-in}$$

use 2 - 1/2 sq-bars 6 in center to center

(area = 0.5 sq-in).

Each alternating bar is bent up over the support all bar are enclosed at 90°. ? Temperature and distribution of stress in direction of the span are provided by placing 1/2" ф 8" center to center in the botton and top of the slab : Crowded

Result : d = 4.5 inch

1/2 square-bars 6" center to center

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b - <u>Design cross - T. beam.</u>

Let us design the longest of them - 12 feet and owing to the little difference between them we will have approximatly the same results and dimensions for the other.

Moment formula used: = 1/10 wL^2

Impact $= \dfrac{50}{12+125} = 0.36$ units, if any

Distribution of wheel load $= \dfrac{S}{4.5} = \dfrac{5}{4.5} = 1.12$ ft. ?

Dead Load from slab = 5.67 x 100 = 567

Assumed stem section: 8" x 12" 1/2 = $\underline{103}$

670 Lb/. ?

1 - <u>Moment</u> :

D.L.M. = $1/10 \times \overline{670} \times \overline{12}^2 \times 12$ = 102,300 in-lb

Maximum L.L moment occurs when

the 8000 lb wheel is in the middle of span of beam

L.L.M = $1.12 \times \dfrac{8000}{2} \times \dfrac{12}{2} \times 12$ = 322,560 in-lb

I.M. = 422560 x 0.36 = $\underline{116.120}$ in-lb

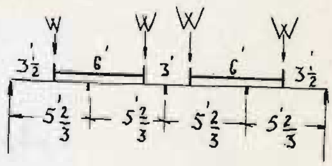
Total moment = 540.980 in-lb

2 - <u>Shear</u>

End Dead Load shear = 670 x 12/2 = 4020 Lb

with the load so placed (fig.1) the middle

beam sustains $\dfrac{4.17 + 4.17}{5.67}$ = 1.47 wheel load. Small



max end shear occurs with wheel at end of beam

L.L. shear = 1.47 x 8000 = $\underline{11760}$

15780 lb.

The required b'd for T beam is

$$b'd = \frac{15780}{7/8 \times 150} = 120 \text{ sq-in}$$

b' = 8 inch

$$d = \frac{I20}{8} = I5 \text{ inch}$$

Total depth considering two rows of steel

2" center to center and 2" insulation.

= I5 + 2 + I = I8 inch.

depth of stem = I8- 5 I/2 = I2 I/2 inch

Required steel area

$$As = \frac{540980}{I8000\ (15-\frac{4.5}{2})} = 2.I85 \text{ sq-in.}$$

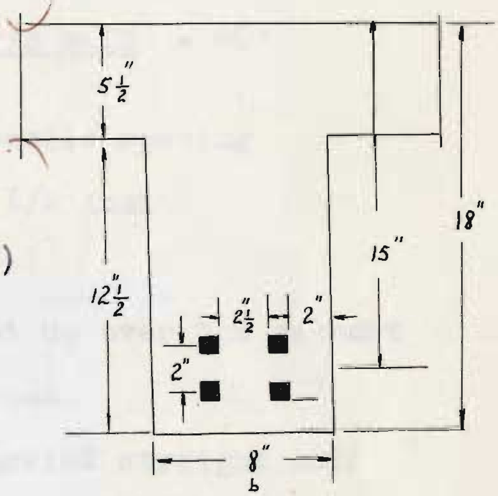
<u>Result</u> use :

d = 15 inch

b'= 8 in

4 - 3/4 sq - bars (A

(As = 2.298) sq-in)



<u>Web reinforcement</u>

<u>Bond</u>.

perimeter required for bond stress

$$\sum o = \frac{I5780}{I20 \times 7/8 \times 15} = 10 \text{ inch}$$

furnish by 4 - 3/4" $\sum o$ = I2 inch

<u>Shear</u>

unit shear at support

$$v = \frac{I5780}{8 \times 15 \times 7/8} = I50 \text{ p.s.i.}$$

Allowable 80 p.s.i.

Concrete take care of

80 x 7/8 x 8 x 15 = 8400 Lb

The remainder I5780 - 8400 = 7380 Lb

is left for stirrups and bent up bars

Spacing of bars bent at 45° beginning at girder

$$S = \frac{0.5625 \times 18000 \times 7/8 \times 15}{0.7 \times 7380} = 25 \text{ inch}$$

or max s = d = . 15 inch

Distance over which web reinforcement is needed

$$\frac{80 \times 6}{150} = 3.2 \text{ feet}$$

Spacing of U stirrups $\frac{3}{8}$"

$$\frac{2 \times 0.1104 \times 18000 \times 7/8 \times 15}{92050 - 8400} = 60"$$

according to Joint Code, 6 max allowable spacing

ls - d/2 = 1/2 x 15 = 7 I/2 inch

Result :

2 bars from each beam are bent up over the support to the third point of adjoining span.

The two remaining bars are carried straight into adjoining span far enough to develop their strength in bond, therefore, we have 4 bars, two from each beam.

Bend up 2 bars at 15" interval from end, then use 3/8" U stirrup at 8" interval tell the 5 feet from left then the remainder U- stirrup at an interval of 12".

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c - <u>Girder - Design</u> Spelling ?

see figer (1) Source ?

Moment at center - $1/8\ wL^2$

moment at end - $1/16\ wL^2$

Impact $= \dfrac{50}{20+I25} = 0.34$

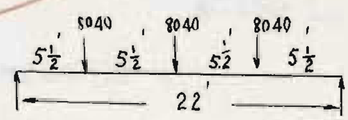
assumed stem 12 x 20"; = 250 Lb per foot

a) - <u>Moment</u>

Dead load transmetted to quater ?

point by T beam

670 x 12 = 8040 Lb



(fig-1-)

Dead Load reaction (see figure 1)

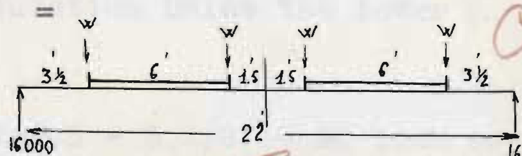
8040 x 3/2 = I2000

1.- <u>D.L. Moment</u> = I2000 x 11 x 12 = I,59I,920 in-lb

Moment due to its own weight : I8I,500 in-lb

$I/8 \times 250 \times 22^2 \times 12 =$

Crowded



2.- <u>Live Load moment</u>

Occurs as a maximum with the two truch symmetrical ? (fig.2)

with respect to center of girder (fig.2)

M = 16000 x 11 - (11-3 1/2) 8000- 800 (1.5) 12 =

= I248000 in-Lb

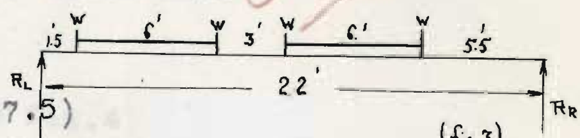
impact = M x 0.34 = 424320

3,445,740 in-lb

b)- <u>Shear</u>

1 - D.Load end shear = 8040 x 3/2 = I2000 Lb Crowded

2 - Ll end shear (fig.3) =



(fig.3)

$$I.34 \left(\frac{W}{22}\right) \Big[(22-1.5) + (22-7.5) + (22-10.5) + (22-16.5)\Big]$$

= 8000 x 52 x 1.34 = 25340 Lb

3.- End shear due to uniform dead load of stem

= 250 x 22/2 = 2750 Lb

Total 40I50 Lb

The least allowable section area is :

$$b'd = \frac{40I50}{7/8 \times 180} = 244 \text{ sq-in.}$$

b' = 12 in

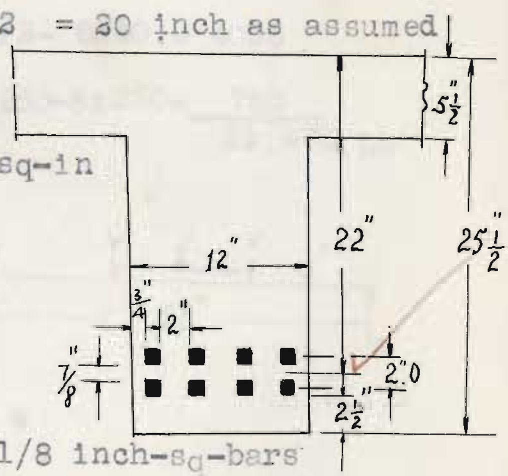
d = 20.4 use d = 22 inch

depth of stem equal considering 2 rows of steel 2" c.to c. and 2 1/2 insulation below the lower row to bottin of girder

depth of stem := 22+1+2.5 - 5 1/2 = 20 inch as assumed

<u>Required steel area</u>

$$As = \frac{3,445,740}{18000\ \left(\frac{20-5.5}{2}\right)} = I0.64 \text{ sq-in}$$



<u>Result</u> use :

stem depth - 20 in

b' - 12 in

As' - 8- 1 1/8 inch-sq-bars

c)- Web-reinforcement

1 - Shear at end = 40150 Lb

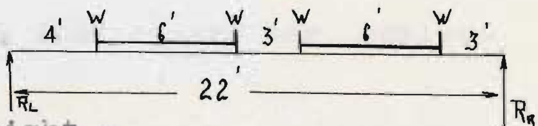
2- shear at 4 ft from end

a - Live load shear : see fig.4

$V = R_L = 1.34\ (18 + 12 + 9 + 3)\ \frac{W}{22}$

$V = 1.34 \times 15270$ 20500 Lb

b - Dead Load shear



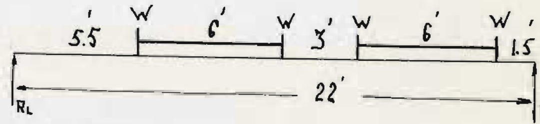
— fig.4 —

concentrated load and stem weight =

$8040 \times 3/2 + 22/2 \times 250 - 4 \times 250 = 14310$

Total 34800 Lb

3 - Shear at 8 ft from end :



—.fig.5.—

a - Live load see fig (5)

$V = RL = \frac{W \times 1.34}{22}\ (1.5 + 7.5 + 10.5 + 16.5)$

$= W \times 1.34 \times \frac{36}{22} =$ 17500

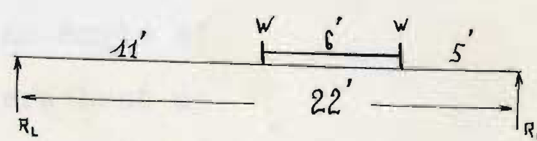
b - Dead Load shear

1 concentrated loads = $8040 \times 3/2 - 8040 = 4025$

2 girder stem weight = $22/2 \times 250 - 8 \times 250 =$ 750

Total 22,275 Lb

4 - Shear at middle of girder :



—. fig.6.—

a - Live load shear fig 6

one truck governs :

$V = RL = \frac{W}{22} \times 11 + \frac{W}{22} \times 5 = \frac{16}{22} W$

$V = \frac{16}{22} \times 8000 \times 1.34$ = 7825 Lb

b - dead load shear 000

Total 7825 Lb

Concrete takes care of : $80 \times 7/8 \times 12 \times 22 = 18500$ Lb

The remainder : $40150 - 18500 = 21650$ Lb

is left for bent bars and U - stirrups

Bond

perimeter required for bond stress is

$$\sum_o = \frac{40150}{120 \times 7/8 \times 22} = 19 \text{ inch}$$

which is furnish by 5 - 1 1/8 inch square bar, perimeter = 22 inches.

Thus leaves 3 - 1 1/8 sq - bar to be bent up to provide negative Moment.

Shear :

Unit shear at support is :

$$V = \frac{40150}{12 \times 22 \times 7/8} = 175 \text{ p.s.i}$$

and the unit shear at the other point is as follow:

$V_4 = 150$ p.s.i, $v_8 = 97$ p.s.i, $v_{11} = 34$ p.s.i

Required distance over which web reinforcement is needed :

$$4 + 4 + 3/63 \times 17 = 8 \text{ ft.} 9 \text{ inch}$$

Spacing of bent up bars is : 20 in according to Joint code at an angle of 45° or the 3 bars of girders bent up at 45° will take care of reinforcing the web a distance of 60 inch.

The remaining distance is left for 3/8 round bars U stirrups.

Extra shear at the 5 ft point over that concrete can take is:

$V' = 31650 - 18500 = 12150$ Lb

therefore distance that 3/8 in-round bar U.stirrup can take care is :

$$S = \frac{2 \times 0.1406 \times 18000 \times 22 \times 0.875}{12150} = 7.5 \text{ inch}$$

Resumé:

Five of the eight I I/8 inch - square bar are needed for bond stress. The 3 other I.I/8 in-sq-bar are bent at 20 inch interval from middle of support, and these bars are prolonged to the outside of the girder to reinforce the brackets under the side walks used for architectural effect.

Then 3/8 inch round U stirrups are used at 6 inch interval till point 7 ft and beyong that use 3/8 inch bars at 8 inch intervals.

-=-=-=-=-=-=-=-=-=-=-=-=-

d - Side walk

1) Live load = 80 Lb-sq-ft ?

units

2) Dead Load

Fill = 8/I2 x I20 = 80

wearing surface = 2 x 10 = 20

slab = 6/I2xI50 = 75

steel parapet = 45

220 Lb per-sq.-ft.

Total 300 Lb per-sq-ft.

Moment :

D.Load and L.Load moment = 300 x 5 x 5/2 = 3750 ft-lb

effective depth $= \frac{3750 \times 12}{12 \times 157} = 23.75$

d = 4.8 inch

take d as 5 inches

Total depth with 1" insulation = 5+I = 6 inches

Required steel area :

$$As = \frac{3750 \times 12}{18000 \times 7/8 \times 5} = 0.575 \text{ sq-inch.}$$

Result : total thickness of slab - 6 inch

use

1 - 1/2" ϕ at 12 " interval $As_1 = 0.25$

2 - 1/2" ϕ at 4" interval $As_2 = 0.392$

0.642 sq-in.

The odd bars of the deck slab will be bent up and carried thrue the cantiliver slab between these bars will be inserted 2- 1/2" ϕ @ 4" interval.

e - <u>Sides beams</u>

End span moment formulea = $1/10\ WL^2$

Intermediate span ... = $I/12\ wL^2$ (Why?)

Let us design all sides beam as end span

1 - Dead Load + Live load from cantilive = 300x5 = I500

2 - Dead Load from curb-stone = 100x5/2= 250

I750 lb-

3 - Stem of beam 200

I950 lb-per-ft

4 - Distribution of truck live load to side beam

$\frac{S}{4.5} = \frac{5.I}{4.5} = 1.12\ W$

or it is at the middle and 1!5 away from side beam

therefore $\frac{1.12\ W \times 3.5}{5} = 0.8\ W$ Wheel load

<u>Moment</u> :

Dead load moment= I/I0xI950xI2x12x12 = 335960

Live load moment with impact:

$1.26 \times 0.8 \times \frac{8000}{2} \times 6 \times 12$ = 3I3,400

649360 in-lb

I will design it as rectangular beam

$bd^2 = \frac{649360}{157} = 4040$

b = 10"

d = 20"

$As = \frac{649360}{18000 \times 7/8 \times 20} = 2,06$ sq-in

<u>Result</u> use : total depth = 20" + 3 = 23"

b = 10"

4 - 3"/4 As= 2.248 sq-inches

web reinforcement

shear

1 - shear at end : D.L. = I950 x 12/2 = II700

L.L. = 0.8 x 8000 = 6400

impact= 6700x 0.28 = I700

I9800 Lb

shear taken by concrete is

$$V = 10 \times 20 \times 80 \times 0.875 = 14000 \text{ Lb}$$

Bond

Required perimeter for bond stress

$$\sum o = \frac{19800}{0.875 \times 120 \times 20} = 9.5 \text{ inche}$$

therefore the 4 - 3/4 inch sq-bars are needed for bond and web reinforcement is needed over a distance of:

$$\frac{19800 - 14000}{19800} \times 6' \times 12 = 21 \text{ inch.}$$

therefore use 4 - 3/8 inch square bar U. stirrup at 7 inch interval and all thru use the same kind of bars at an interval of 10 inch.

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B - Design of supporting columns.

1 - Side beam reaction to column = I950 x 12 = 23450

Dead load of column = $I9.8 \times \frac{15}{12} \times \frac{30}{12} \times I50$ = 9300

32450 Lb

2 - Girder reaction introducing an eccentricity: = I2060

25340

(?) 2750

72600 Lb.

3 - eccentric moment = I/2 x 3445740 = I,722870 in-Lb

allowable load per suqare-inch is 945 p.s.i

assuming a section of 30 x 15 inch we get a unit compressive stress of :

$$s = \frac{72600}{450} + \frac{1722870 \times 12}{15 \times 30 \times 30 \times 30} = I60 + 760 = 920 \text{ p.s.i}$$

The assumed section is satisfactory ;

Let us take 2.25 ~~% steel which is equal to~~ why(?)

450 x 0.02 = 10,15 sq-inchs.

Result :

use :

1 - column section - 30x15 inch.

2 - steel - 8-1 inch square bar As_1 = 8-sq-in

- produce 2-1 1/8 inch square bar from girde to reach arch rib As_2 = 2.53 sq-in

I0.53 sq-in

3 - ties - Lateral ties 3/8 inch 15 inch ctoc.

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

C.- - Design of arch rib -

1 - Span

clear span of ribs is taken as II8.4 feet to give a net opening of 116 feet

2 - Columns distances

the spacing of columns is taken as follow for architectural reasons - Location?

from retaining wall to - C1	=	12 feet	
	C1 - C2	=	12
	C2 - C3	=	11
	C3 - C4	=	11
	C4 - C5	=	11
	C5 - crown	=	5 feet

3 - Rise

h is taken as 1/6 of span = $\frac{II8.4}{6}$ = I9.72 feet

4 - Thickness of ribs

Crown thickness 32 inch

Springing thickness 56 inch

See table I for variation in thickness along the arch axis taking from page 438 after Urquhart Title?

5 - Dead Load

Dead Load transmitted to designed point of arch rib

C1 - span 12 ft.

Girder reaction	= I2xIIxI00 + I2x2xI03+250xII	=	I7770
Side walk	= 300 x 5 x 12	=	I7230
Side beam	= 200 x I2	=	2300
Column C1	= I9.8 x $\frac{30}{I2}$ x $\frac{15}{I2}$ x 150	=	9300
			46,600 Lb

Calc. OK

C2 - Span 11.5 ft.

Girder reaction = II.5 x II x I00 + II.5x2xI03+250xII = I7770

side walk = 300 x 5 x II.5 = I7230

side beam = 200 x II.5 = 2300

column C2 = $13.8 \times \frac{30}{12} \times \frac{15}{12} \times 150$ = 6400

tie bracing between $C2_L - C2_R$ = $\frac{20}{12} \times \frac{15}{12} \times 150 \times 11$ = 3450

47I50 Lb

C3 - Span 11 ft.

Girder reaction = IIxIIxI00 + II x 2 x I03 +250 xII = I7I20

Side walk = 300 x 5 x II = I6500

Side beam = 200 x II = 2200

Column 3 = $9.8 \times \frac{30}{12} \times \frac{15}{12} \times 150$ = 4580

Tie bracing between $C3_L - C3_R$ = $\frac{20}{12} \times \frac{15}{12} \times 150 \times 11$ = 3450

43850 Lb

C4 - Span 11 ft:

Girder Reaction = II x I00 x II + IIx2 x I03+250xII = I7I20

Side walk = 300 x 5 x II = I6500

Side beam = 200 x II = 2200

Column C_4 = $7.5 \times \frac{30}{12} \times \frac{15}{12} \times 150$ = 3530

Tie bracing between $C_4L - C_4R$ = $\frac{18}{12} \times \frac{15}{12} \times 150 \times 11$ = 3I00

42450 Lb

C5 - Span 10.5 ft:

Girder reaction = IIx I00 x I0.5+ I0.5x2 x I03+250xII= I6780

Side walk = 300 x 5 x I0.5 = I5770

Side beam = 200 x 10.5 = 2I00

Column C5 = $6.28 \times \frac{30}{12} \times \frac{15}{12} \times 150$ = 2950

tie bracing between C5L-C5-R = 3I00

40.400 Lb

<u>Summarizing the result</u> :

C1 = 46600#

C2 = 47I50

C3 = 43850

C4 = 42450

C5 = 40400#

6 - <u>Live load</u> :

The H - 10 Loading is used

it is composed of an uniform~~e~~ load of 320# per foot

of lane, a concentrated load of 9000# for moment;

a concentrated load of I3000# for shear

Impact $= \dfrac{50}{116+125} = 0.2$

uni~~f~~orm~~e~~ live load from roadway = 320xI.2= 4I0 per f.of lane

uniform~~e~~ load from side walk = 5x80x1.2=480 " " " "

890 Lb p.f.of lane

Live Load concentrated at Co: for moment :
890 x 6 + 9000 = I4400#

Live Load concentrated at C1: for moment :
890 x I2 + 9000 = I9700

Live load concentrated at C2= for moment :
890 x 11.5 + 9000 = I9300

Live load concentrated at C3: for moment :
890 x II + 9000 = I8800 Lb

Live load concentrated at C4: for moment :
890 x II + 9000 = I8800

Live load concentrated at C5 : for moment:
890 x 10.5 + 9000 = I8300#

+ SOURCE?

7 - <u>Form of arch axis</u>

See table II

It is a curve of the form $y = \frac{8rL}{6 + 5r}\ (3C2 + 10C^4 r)$

8 - <u>Radius of neutral arch axis</u>

Three centered curves are to be used for neutral axis - the larger radius in each case being from quater point to crown

$$R_1 = \frac{29.585^2 + 4.48^2}{2 \times 4.48} = 99.95 = 100 \text{ feet.}$$

$$\theta_1 = 17°2 \quad \sin\theta = \frac{29.58}{100} = 0.2958 \quad \cos\theta = 0.9553$$

$$R_2 = 1/2 \quad \frac{29.582^2 + 15.34^2}{15.34 \times 0.9553 - 29.58 \times 0.2958} = 95 \text{ feet}$$

Exterior radius =

R1 = 103 feet R2 = 107 feet

Interior radius :

R1 = 97 feet R2 = 83 feet

9 - <u>Length of neutral axis</u>

Tangent to axis at springing

$$tg\theta = \frac{8r}{6+5r}\ (3 + 5r) = 0.748 \qquad \theta = 36.8$$

from page 438 Urquhart text book

length of arch axis = 0.535 x 118.34x2 = 128 feet.

10 - <u>Arch properties and constants</u>:

Dividing the arch axis into ten equal divisions - the length of each division is 6!4. Laying of the centers of each division with origin of ordinate at crown - The coordinates are scaled and put in tabular form - see table III. We shift the origin of coordinate so that $\sum \frac{y}{I} = 0$

now $y_o = \sum \frac{yc}{I} \div \sum \frac{1}{I} = 4.35$ feet

see table IV.

taking: from springing to pt 3, $ps = 0.012$

from pt 3 to point 7 $ps = 0.01$

from 7 to Crown ; $ps = 0.008$

So that at Crown $As = 8208$ sq.inch

at quator point $As = 10.752$ sq-inch

at springing $As = 20.504$ sq-inch

11 - Ribs tie bracing :

The ties between $C_2L - C_2R - C_3L - C_3R$ have the following dimensions 16 x 20 inches and reinforce with 4-1 inch-square bars their main function is to act as horizontal columns. The ties between $C_4L - C_4R$ and $C_5L - C_5R$ have the following dimensions 15 x 18 inches and reinforced also with 4-1 inch square bars.

12.- Dead Load Ho-Vo, Mo at Crown

Having the results of tables III, IV, V, the formulea for Mo, Ho, Vo are :

$$M_o = \sum m/I \div 2 \sum 1/1$$

$$H_o = \sum m y_1/I \div 2 \sum y_1^2/1$$

$$V_o = \sum mx/I \div 2 \sum x^2/1$$

with the axis transfered to elastic center: yo-x

The results are put in table VI

13 - Ordinates of influence line at Crown

$M_c = M_o + 4.35\ H_o$

$N_c = H_o$

$V_c = V_o$

making a unit load travel along the arch we get

The results which are put in table VII.

I4.- Ordinates of influence line at quarter point

Having the result of table VI we can easely get, M 1/4 N 1/2 due to any load on the rib after developping the folowing formulea :

A unit load from C to quarter point:

$$M\ 1/4 = MO - 0.65\ Ho + 31.4\ Vo - (31.4 - a)$$

$$N\ 1/4 = Ho \cos 1/4 - (1-Vo) \sin 1/4$$

A unit load from A to quarter point or from B to crown

$$M\ 1/4 = Mo - 0.65\ Ho + 31.4\ Vo$$

$$N\ 1/4 = Ho \cos 1/4 + Vo \sin 1/4$$

or $\sin 1/4 = 0.3195$, $\cos 1/4 = 0.9475$ $\angle 1/4 = 18^{\circ}-40'$

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

I5 - Ordinates of influence line at springing A.

Having the values of Ho, Vo and Mo at crown for a unit load travelling along the arch and with respect to the elastic center as origin of coordinate.We can get Ms and Ns after developping the following formulea :

A unit load from C to A

$$Ms = Mo - 15.37\ Ho + 59.2\ Vo - (59.2 - a)$$

$$Ns = Ho \cos s - (1-Vo) \sin s$$

A Unit load from C to B

$$Ms = Mo - 15.37\ Ho + 59.2\ Vo$$

$$Ns = Ho \cos s + Vo \sin s$$

$\cos s = 0.8$ $\quad \sin s = 0.6$ $\quad \angle s = 36^{\circ}-48'$

Making a unit load travel along the arch rib the results are the ordinates of the influence line curve for springing and results are put in table IX.

I6. - Dead load and Live Load moment and Thrust :

All computations are put in tabular from in table X

I7.- Temperature and shrinkage effect :

fall - 30°

rise 40° fahr

shrinkage effect equal 10° fall

E = 288000,000 Lb - feet-sq.

at Crown

$$M_o = - 4.35 \ H_o$$

$$H_o = \mp \frac{wtL}{2 \sum \frac{y^2}{I}} \times \frac{E}{ds}$$

$$= \mp \frac{0.000006 \times 119.2}{2 \times 84.I} \times \frac{288,000.000}{6.4} \times t$$

$$= \mp \ I90 \ t$$

Rise; 40° Ho = - I90 x 40 = (- 7600 Lb

Shrink;fall40° Ho = + I90 x 40 = + 7600 Lb

Rise 40° M = 4.35 x 7600 = - 33000 ft-Lb

Shrink;fall;40° = M = + 33000 ft-Lb

I/4 point

$$M \ 1/4 = M_o - 0.65 \ H_o$$

$$N \ 1/4 = H_o \cos 1/4$$

Rise 40°Fah M 1/4= - 33000-0.65x-7600 = - 28000 ft-Lb

N 1/4= - 7600 x 0.95 = - 7250 Lb

Shrinkage + fall=40° Fah

M 1/4 = + 33000 - 5000 = + 28000 ft-Lb

N 1/4 = + 7600 x 0.95 = - 7250 Lb

at Springing

Ms = M - I5.37 Ho

Ns = Ho cos s

Rise 40° Fah

Ms = - 33000 + 15.37 + 7600 = + 83800 ft-Lb

Ns = - 7600 x 0.8 = - 6080 Lb

Shrinkage and fall;40° fahr

Ms = + 33000 - I5.37 x 7600 = - 83800 ft-Lb

Ns = + 7600 x 0.8 = + 6080 Lb

I8.- Combined moments and Thrusts:

Results are put in table XI

I9.- Fiber stress

Final results of moments,thrusts and fiber stresses are put in in table XII.The formulea and diagram used are those of Turneare (Diagram 13-22).

All the stresses, crown, quarter point,springing are good and within reasonable limits.

D.- - DESIGN OF ABUTMENT -

Retaining wall under girder

Surcharge

The two trucks are considered and their weight is uniformely distributed and equal to :

$$\frac{2 \times 20.000}{32 \times 62} = 21\text{Lb-ft}2$$

which is negligable

widlh of relaining wall = 32 ft.

depth -h- = 28 ft.

Dead Load at top of R.W.

from girder $\frac{8040}{2} \times 3$ = 12060

250×24 = 6000

side walk = $300 \times 6 \times 2$ = 3600

21660

: 21660 ___ 32 = 700 Lb-ft.

Live load:

16000 x 2 ___ 32 = 1000 Lb-ft.

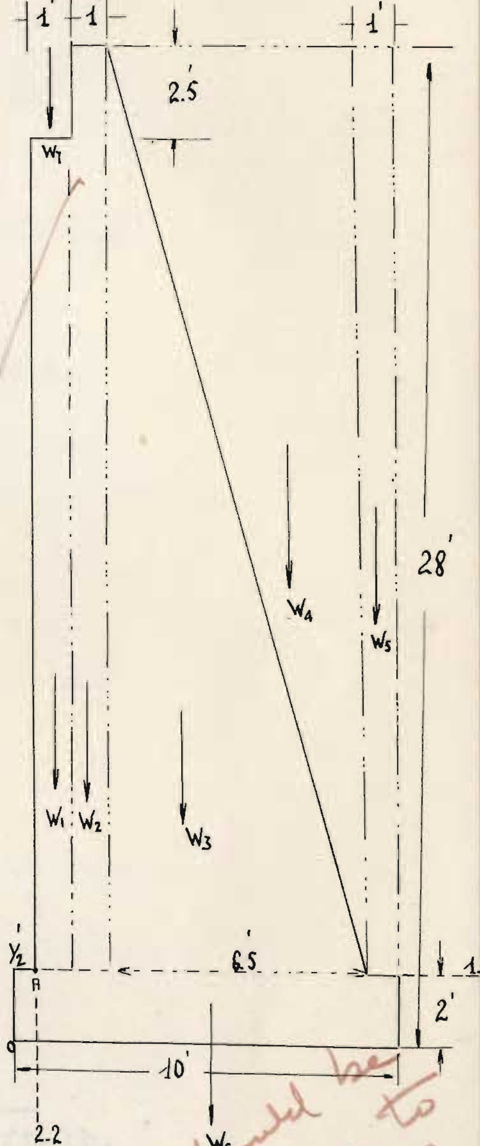
Coefficient of friction μ = 0.60

angle of friction = 0.45°

K = 0.18

$$: P_1 = K w \frac{h1^2}{2} = 0.18 \times 120 \times \frac{\overline{26}^2}{2} = 7300 \text{ Lb}$$

$$P_2 = K w \frac{h^2}{2} = 0.18 \times 120 \times \frac{\overline{28}^2}{2} = 8450 \text{ Lb}$$



The vertical forces

W1 = I x 23.5 x 120 = 2820 Lb

W2 = I x 26 x I20 = 3I20 Lb

W3 = 6.5x26x1/2x120 = IOIOO Lb

W4 = 6 I/2x26x1/2x120=IOIOO Lb

W5 = I x 26 x 120 = 3I20 Lb

W6 = IO x 120 = I200 Lb

W7 = = 700 Lb

R = 3I,I60 Lb

CROWDED

Study of section 1-1

a.- Stabilizating moment :

M.A	2820 x 1/2	= I4IO
	3I20 x 1 1/2	= 4680
	IOIOO x 4.I7	= 42IIO
	IOIOO x 6.35	= 64I40
	3I20 x 9	= 28080
	700 x 1/2	= 350
		I40770 ft-lb.

Should be drawn to better scale

b.- Overturning

$M_A = 7300 \times \frac{26}{3} = 63200$ ft-lb

factor of safety against overturning =

$$\frac{I40770}{63200} = \underline{\underline{2.2}}$$

Sliding :

N = 3II60 - I200 - 3I20 = 27840 Lb

Nƒ= 27840 x 0.6 = I6704 Lb

factor of sliding = $\frac{16704}{7300}$ = 2.2

Crushing

$$M_A = N x$$

$$x = \frac{I40770 - 63200}{27840} = 2'.78 \text{ from A.}$$

which is within the middle Third.

Study of foundation section

a.- Stabilizating moment:

Mo	=	2820 x 1	= 2820
		3I20 x 2	= 6240
		I0I00 x 4.67	= 47I70
		I0I00 x 6.85	= 69I90
		3I20 x 9.5	= 29640
		700 x I	= 700
		I200 x 5	= 6000
			I6I,760 ft-lb.

Overturning:

$$Mo = 8450 \times \frac{28}{3} = 78900 \text{ f.lb.}$$

$$\text{factor against overturning} = \frac{I61760}{78900} = 2.$$

Sliding :

$$N = R = 31160 \text{ lb}$$

$$N f = 0.6 \times 3I160 = I8696 \text{ lb.}$$

$$\text{factor against sliding} = \frac{I8696}{8450} = 2.2I$$

Crushing :

place where resultant cuts the foundation

Mo = Nx

$$X = \frac{161760 - 78900}{31160} = 2.65 \text{ from O.}$$

Resultant acting outside the middle third.

Max pressure :

$$\frac{2\ N}{3x2.65} = \frac{62320}{7.95} = \underline{7790}\ \text{lb-ft}^2.$$

or foundation is of solid rock having an allowable bearing value of 10 T/ft^2

Shearing a long section 2-2

1 - The forces are: total pressure upward from

left to 2-2 $= \frac{7790 + 7270}{2} = 7530$

weigh of maconry up to section 2-2

$= 1/2 \times 2 \times 1 \times 120 = 120$ lb

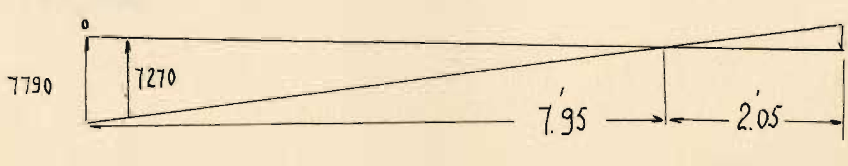
therefore: $\frac{7410}{1x12x2x12} = 25$ p.s.i

within the 40 $lb\text{-}in^2$ allowable.

2nd - Side relaining wall

depth h - 29'

$$P_1 = \frac{Kwh^2}{2} = 0.18x120x\frac{29^2}{2} = 9080 \text{ lb-ft.}$$



Vertical forces:

W1 = 27 x 1 x 120 = 3240 lb

W2 = 26 x 1 x 120 = 3120 lb

W3 = 6.5x1 x 26x1/2x120
= 10100 lb

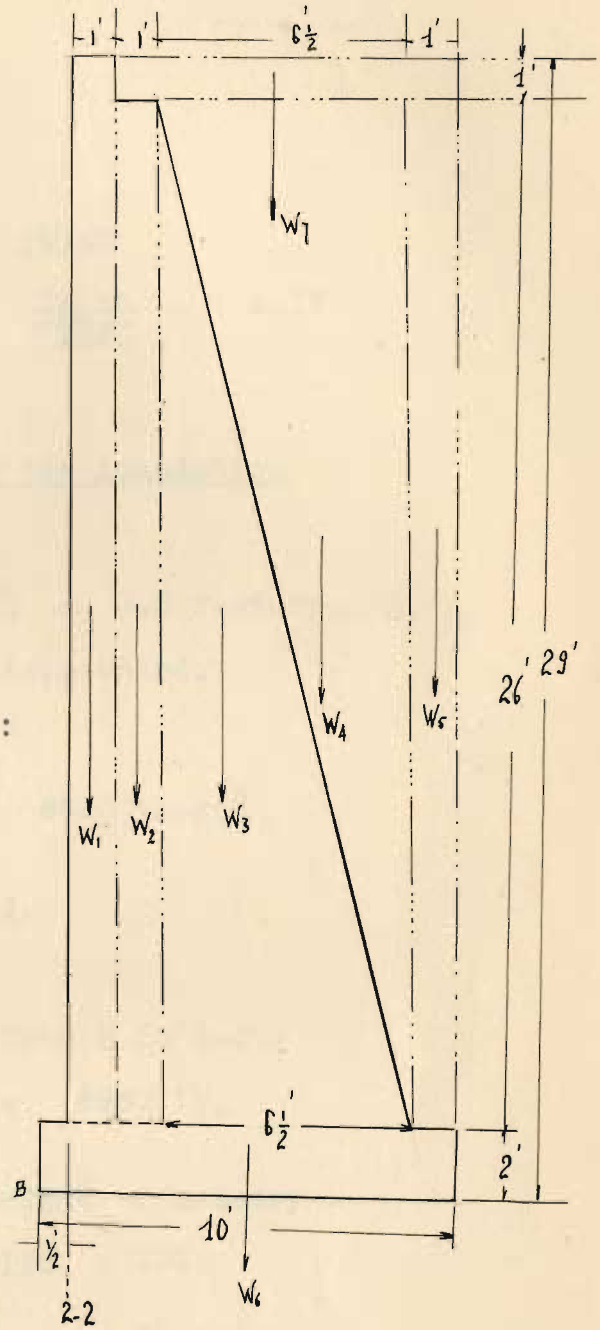
W4 = 6.5x26x1/2x120= 10100 lb

W5 = 1 x 26 x 120 = 3120 lb

W6 = 10 x 1 x 120 = 1200 lb

W7 = 1 x 8.5 x 120 = 1020 lb

R = 31900 Lb



Study of foundation section :

Stabilizating moment

Stabilizing

MB = 3240 x 1 = 3240

3120 x 2 = 6240

10100 x 4.67= 47170

10100 x 6.85= 69190

3120 x 9.5 = 29640

1200 x 5 = 6000

1020 x 5.75= 5860

167,340 ft-lb

Overturning:

$M^B = 9080 \times \frac{29}{3} = 87,700$ ft-lb

factor of overturning :

$\frac{167,340}{87,700} = 1.90$

Sliding :

$N = R = 31.900$

$N = 31900 \times 0.6 = 19140$

factor against sliding : $\frac{19140}{9080} = 2.17$

Crushing

Place where resultant cuts the foundation

$M_B = Nx$

$X = \frac{167340 - 87700}{31900} = 2.5$ feet from B.

Resultant acts outside middle third.

Max pressure

$\frac{2\ N}{3 \times 2.5} = \frac{2 \times 31900}{7.5} = 8520$ lb-ft^2

Shearing along section 2-2.

Forces acting are:

1 - pressure up wards from B to 2-2 =

$\frac{8520 + 7980}{2} = 8250$ lb.

2 - down ward force: weight of maconry =

$1/2 \times 2 \times 1 \times 120 = 120$

therefore $S_s = \frac{8370}{1 \times 12 \times 2 \times 12} = 29$ p.s.i.

which within the allowable limit 40 p.s.i.

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3 - Abutment :

See plate -   for graphical solution.

A - Dead Load Case:

The loads are :

1 - R = 20850 + 200 + 375 + 43950 + I500 + 2I00

+ 270 + 2850 + 790 = 72855 lb.

Location of Resultant from point B.

$$M_B = \begin{array}{r} 452445 \\ 4175 \\ 7825 \\ 404340 \\ I1280 \\ 7350 \\ 380 \\ 3990 \\ I470 \\ \hline 893,255 \end{array} \text{ ft-lb}$$

therefore: $\frac{M_B}{R}$ = I2!28

2 - P = 604I65 / 6 = I00;690 lb ; e + 0.5I ft.

B - Max positive moment case:

1 - R the same; d = I2!28 ft

2 - P = 7I2580 / 6 = I18760 lb   e + I.47 ft.

C - Max negative M case:

I - R the same d = I2.28 ft.

2 - P = 679400 / 6 = I13230 lb   e = - 0.423 ft.

Earth pressure :

the resultant of the combined forces fall in the three case in the middle third threfore the max. and min.earth pressure are given by the formules :

$$p_1 = (4L - 6a)\ F/L^2$$
$$p_2 = (6a - 2L)\ F/L^2 \quad \text{(Urquhart page 400).}$$

Case A :

$F = 155650$

$a = 6.85$ ft

$L = 18.4$ ft

$$p_1 = \frac{4 \times 18.4 - 6 \times 6.85}{18.4^2} \times 155650 = 14900 \text{ lb.p.ft}^2$$

$$p_2 = \frac{6 \times 6.85 - 2 \times 18.4}{18.4^2} \times 155650 = 1970 \text{ lb-p.ft}^2$$

B.- Max- positive moment Case:

$F = 172800$ lb $\quad a = 6.15$ ft $\quad L = 18.15$ feet

$$p_1 = \frac{4 \times 18.15 - 6 \times 615}{18.15^2} \times 172800 = 18{,}580 \text{ lb.per.ft-sq.}$$

$$p_2 = \frac{6 \times 6.15 - 2 \times 1815}{18.15^2} \times 172800 = 312 \text{ lb-per.ft-sq.}$$

C.- Max-negative moment case:

$F = 184500$, $a = 7.35$ ft $\quad L = 18.05$ ft.

$$p_1 = \frac{4 \times 18.05 - 6 \times 7.35}{18.15^2} \times 184500 = 15950 \text{ lb-p.ft}^2$$

$$p_2 = \frac{6 \times 7.35 - 2 \times 18.05}{18.05^2} \times 184500 = 4540 \text{ lb.p.ft}^2$$

All the earth pressure are within allowable limit : 20,000 p.s.f.

Sentence construction?

E.- Expansion Joint, drainage, parapet.

Sliding joint:

Under the end girder and on the top of retaining wall four thin rectangular plâtes of metal: steel and copper not attached to the beam above neither the wall below are used.

The thickness of plates is 1 inch for copper and steel

Expansion and contraction movements taking place after completion of the structure causes the two upper plate to slide over the lower copper plate.

Between the retaining wall, and the slab a space of 2" is filled with bitumen to allow every movement.

Drainage:

water on the bridge superstructure is disposed of ~~it~~ quickly, by crowing the road about 3" all thru the 32 ft and pitching gutters to drain into inlets - The inlets drain are cast iron scuppers in the gutter (see drawing details) and are designed so as to prevent water touching the concrete.

Parapet:

The parapet is formed of steel pipes, the horizontal members are 2" in diameter and the vertical are 3".5 in d.

F.- Details:

For details see plates and drawings.

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T A B L E S

Form of arch axis I						
Origin of ordinate taken at crown.						
C	CL=x ft	C^4	Ioc^4r	C^2	$3c^2+IOc^4r$	y ft
0.05	5.9I7	0.000006	0.0000I	0.0025	0.0075	0.I73
0.I	II.834	0.000I	0.000I6	0.0I	0.03I6	0.7I6
0.I5	I7.75I	0.000506	0.00084	0.0225	0.0689	I.573
0.20	23.668	0.00I6	0.0026	0.04	0.I226	2.806
0.25	29.585	0.0039	0.0065	0.0625	0.I94	4.480
0.30	35.502	0.008I	0.00I35	0.09	0.283	6.5I0
0.35	41.4I9	0.0I5	0.025	0.I225	0.392	9.0I
0.40	47.336	0.0256	0.042	0.I6	0.522	I2.00
0.45	53.253	0.04I	0.068	0.2025	0.675	I5.53
0.50	59.I7	0.0625	0.I04	0.25	0.854	I9.72

- Variation of arch Thickness- II			
$\frac{Sx}{S} = v$	$u = \frac{ts}{tc}$	Thickness ft	1/2Thick. a/2 ft
0	I.000	2.66	I.33
0.05	I.006	2.667	I.3335
0.I5	I.0I8	2.687	I.343
0.25	I.030	2.7I9	1.359
0.35	1.042	2.75	I.375
0.45	I.054	2.782	I.39I
0.55	1.072	2.830	I.415
0.65	I.I25	2.935	1.467
0.75	1.225	3.235	2.6I8
0.85	I.393	3.677	I.838
0.95	I.62I	4.28	2.I4
I	I.750	4.66	2.33

- Property of arch ring - III

d'= 2".8 (1-3); d'=2" (3-10) width 32"

Points	a in ft	Ic = a3b/12	$\frac{a}{2}$ - d'	$(\frac{a}{2} - d)^2$	area of arching	area of 1' width
I	4.28	6.55	I.9I	3.65	I0.7	4.280
2	3.68	4.I4	I.6I	2.60	9.2	3.68
3	3.26	2.98	I.42	2.0I	8.I5	3.26
4	2.96	2.I0	I.32	I.693	7.40	2.96
5	2.83	I.88	I.25	I.562	7.075	2.83
6	2.78	I.80	I.225	I.495	6.95	2.78
7	2.75	I.73	I.209	I.452	6.875	2.75
8	2.72	I.67	I.I95	I.423	6.8	2.72
9	2.69	I.60	I.I79	I.385	6.725	2.69
I0	2.66	I.55	I.I65	I.345	6.65	2.66
Crown	2.66	I.53	I.I65	I.335	6.65	2.66
1/4 point	2.80	I.84	I.234	I.524	7	2.80
Springing	4.66	8.32	2.I64	4.644	II.65	4.66
					sq-ft	sq-ft

- Properties of arch ring - IV

ps = 0.008 (10-7) ; ps = 0.0I (7-4): ps=0.0I2(4-1)

point	$2A_s^{'} \left(\frac{a}{2}-d'\right)^2$ = Is	(n-1)Is	It	yc	$\frac{yc}{It}$	$\frac{1}{It}$
1	0.2055	I.85	8.50	I7.85	2.08	0.II7
2	0.I22	I.I0	5.2	I4.35	2.60	0.I90
3	0.072	0.65	3.65	II.20	2.82	0.273
4	0.0488	0.44	2.54	8.40	3.33	0.400
5	0.044	0.397	2.277	6.05	2.66	0.430
6	0.04I6	0.375	2.I78	3.95	I.82	0.460
7	0.040	0.360	2.09	2.35	I.I2	0.478
8	0.0308	0.278	I.96	I.I5	0.58	0.5I3
9	0.0296	0.267	I.867	0.35	0.20	0.54
I0	0.0284	0.256	I.802	0.04	0.02	0.550

$$\sum \frac{yc}{It} = 17.23$$

$$\sum \frac{1}{It} = 3.951$$

$$yo = \frac{17.23}{3.951} = 4.35$$

Properties of arc ring. V

Coordinates with respect to elastic center

Point	x	y_1 (y_o-x)	x^2	y^2	$\frac{x^2}{I_t}$	$\frac{y^2}{I_t}$
I	56.7	-I3.5	32I5	I82.3	376.2	21.32
2	5I.4	-I0.00	2642	I00	50I.8	I9.00
3	45.8	- 6.85	2098	47	548.75	I2.80
4	40.2	- 4.05	I6I6	I6.5	646.5	6.60
5	34.2	- I.70	II70	2.9	503.0	I.25
6	28.3	+ 0.40	80I	0.I6	368.0	0.07
7	22.2	+ 2.00	493	4.00	235.75	2.00
8	I6.0	+ 3.20	256	I0.25	I3I.25	5.25
9	9.60	+ 4.00	93	I6.00	50.50	8.65
10	3.20	+ 4.3I	1025	18.58	5.50	I0.20
Crown	0.00	+ 4.35				
1/4 point	31.4	- 0.65				
Springing	59.2	-15.37				
C5	5'	+ 4.28				
C4	16'	+ 3.20				
C3	27'	+ 1.00				
C2	38'	- 3.I5				
C1	50	- 9.I5				

$\sum \frac{x^2}{I_t}$ = 336725

$\sum \frac{y^2}{I_t}$ = 87.I4

Mo, Vo, Ho, at crown section. - VI

	Unit load at I				2				C1				3			
section	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I
I	"	"	"	"	5.3	0.6I	34.6	-8.2	6.7	0.77	43.7	-I0.4	I0.9	I.25	70.9	-I6.9
2	/	"	"	"	"	"	"	"	I.4	0.25	I2.8	-3.4	5.6	I.00	5I.4	-I0
3	"				"	"	"	"	"	"	"	"	"	"	"	"
4																
$\sum$	"	"	"	"		0.6I	+34.6	-8.24		I.02	56.5	-I3.8		2.25	I2.23	-26.9
Mo		0				0.078				0.I3				0.28		
Vo			0				0.005				0.008				0.0I8	
Ho				0				-0.048				-0.080				-0.I59

$$Vo = \frac{\sum mx/I}{2 \sum (1/I)}$$

$$Ho \doteq \frac{\sum my/I}{2 \sum (y2/I)}$$

$$Mo = \frac{\sum m/I}{2 \sum 1/I}$$

S = constant

$$\sum \frac{y}{I} = o$$

Mo, Vo, Ho, at crown section. - VI

	Unit load at 4				C2				5				6			
section	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I
I	16.5	I.9	107.7	-25.6	I8.7	2.I5	I22	-29	22.5	2.60	147	-35.I	28.4	3.25	I8.5	-43.9
2	II.2	2.0	102.8	-20	13.4	2.35	I2I	-23.5	I7.2	3.05	I56.7	-30.5	23.I	4.08	209.7	-40.8
3	5.6	1.4	64.2	-9.6	2.2	0.88	35.4	-3.55	6.	2.4	96.5	-9.7	I9.7	4.76	I9I.3	-I9.3
4	"	"	"	"	7.8	I.9	87	-I3.02	II.6	2.9	I32.8	-I9.85	I7.5	4.4	202	-30.I5
5	"	"	"	"	"	"	"	"	"	"	"	"	5.9	2.55	87.2	-4.35
6	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
7																
8																
9																
IO																
$\sum$		5.3	2747	-55.2		7.28	365	-69.12		10.95	+534	-95.I		I9.05	864	-138.5
Mo		0.677				0.93				1.4				2.43		
Vo			0.04I				0.055				0.081				0.13I	
Ho				-0.328				-0.4I				- 0.565				-0.823

This table is crowded

Mo, Vo, HO, at crown section. - VI

	Unit load at C_3				7				8-C_4			
Section	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I
I	29.7	3.40	I92.7	-45.9	34.5	3.95	226	-54	40.7	4.6	26I	-62.I
2	24.4	4.32	222	-43.2	29.2	5.I5	264.7	-5I.5	35.4	6.25	32I.2	-62.5
3	I8.8	4.7	2I5.2	-32.4	23.6	5.9	275	-4I.I	29.8	7.45	34I.2	-5I.05
4	I3.2	5.28	2I2.2	-2I.4	I8.	7.2	289.4	-29.6	24.2	9.68	388.	-38.8
5	7.2	3.I	I05.	-5.25	I3.2	5.25	I79.5	-8.95	I8.4	7.9	270.2	-I3.43
6	I.3	0.6	I7.0	+0.25	6.I	2.8	79.25	I.I2	I2.3	5.65	I58.6	+2.25
7	"	"	"	"	"	"	"	"	6.2	3.00	66.6	6.00
8	"	"	"	"	"	"	"	"	"	"	"	"
9												
IO												
$\sum$		2I.4	964.2	-I48.		30.25	I333.6	-I84.I		44.55	I807	-2I9.6
Mo		2.73				3.87				5.69		
Vo			0.I45				0.204				0.274	
Ho				-0.880				-I.095				-I.305

Mo, Vo, Ho, at crown section.

	Unit load at 9				C5				10			
Section	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I	m	m/I	mx/I	my/I
I	47.I	5.4	306.2	-72.9	5I.7	5.95	340.	-8I.4	53.5	6.I5	348.7	-83
2	41.8	7.4	380.5	-74	46.4	8.2	42I.3	- 82	48.2	8.55	439.5	-85.5
3	36.2	9.05	4I2.2	-6I.6	40.8	I0.0	458.0	-68.5	42.6	I0.65	487.8	-72.95
4	30.6	I2.25	490.	-49	35.2	I4.I	566.7	-57.I	37.0	I4.8	594.9	-59.25
5	24.8	I0.65	364.2	-I8.I	29.2	I2.6	430.9	-2I.4	3I.2	I3.4	458.3	-22.8
6	I8.7	8.6	243.4	+3.45	23.3	I0.7	302.8	4.3	25.I	I1.5	325.45	+4.6
7	I2.6	6.05	I34.3	+I2.I	I7.2	8.25	I83.2	+I6.5	I9.0	9.I2	202.4	+I8.25
8	6.4	3.25	52	+I0.4	II.0	5.65	90.4	+I8.I	I2.8	6.5	I04.	+20.8
9	"	"	"	"	4.6	2.5	24.0	+I0.	6.4	3.5	33.06	+I4.0
I0	"	"	"	"	"	"	"	"	"	"	"	"
$\sum$		62.65	2330	-245.7		77.95	28I7.3	-260.5		84.I7	2974.7	-265.8
MO		8.0I				9.968				I0.76		
Vo			0.354				+0.428				+0.452	
Ho				-I.46				-I.548				-1.580

Table VII

Ordinates of Influence line for crown-section

$x = o$
$y = 4.35'$

$M_c = M_o + 4.35' H_o$

$N_c = H_o$

Sect.	H_o	$H_o\ y$	V_o	$V_o x$	m	M_o	M_c	N_c
I	.	.	.	.	.	.	-0.0I5	-0.0I
2	-0.0489	-0.2I3	0.0052	.	.	0.078	-0.I35	-0.05
C_I	-0.0802	-0.348	0.0086	.	.	0.I3	-0.2I8	-0.08
3	-0.I59	-0.696	0.0I86	.	.	0.287	-0.4I0	-0.I6
4	-0.328	-I.426	0.04I	.	.	0.677	-0.758	-0.33
C_2	-0.4I0	-I.783	0.055	.	.	0.93	-0.853	-0.4I
5	-0.565	-2.457	0.08I	.	.	I.4	-I0.57	-0.57
6	-0.823	-3.58	0.I3I	.	.	2.43	-I.I40	-0.825
C_3	-0.880	-3.828	0.145	.	.	2.736	-I.09	-0.88
7	-I.095	-4.78	0.203	.	.	3.87	-0.9	-I.I
8-C_4	-I.305	-5.67	0.275	.	.	5.69	+0.02	-I.3I
9	-I.46	-6.35	0.354	.	.	8.0I	+I.66	-I.46
C_5	-I.548	-6.742	0.428	.	.	9.968	+3.226	-I.55
I0	-1.580	-6.87	0.452	.	.	I0.76	+3.89	-I.60

Ordinates of influence line for quarter point - Table VIII

	$X\frac{1}{4} = 31.4$	$Y\frac{1}{4} = 0.65$	$M\frac{1}{4} = Mo - 0.65\,Ho + 31.4\,Vo - m;\ N\frac{1}{4} = Hocos\frac{1}{4} - (1-Vo)\,sinl/4$									
Loaded point	Mo	Ho	-0.65 Ho	Vo	31.4 Vo	-m = -(31-a)	Hocos 1/4	(1-Vo)	-(1-Vo) sinl/4	Vo sinl/4	$M\frac{1}{4}$	$N\frac{1}{4}$
A-1	.	.	.	.	.	.	.	.	.	.	+0.13	0.0I
2	0.078	-0.05	+0.033	0.0052	0.I57	.	-0.0475			+0.00I7	+0.268	-0.046
C1	0.13	-0.08	+0.052	0.0086	0.27	.	-0.076			+0.0028	+0.452	-0.075
3	0.287	-0.I6	+0.I04	0.0I86	0.58	.	-0.152			+0.0060	+0.97I	-0.I46
4	0.677	-0.33	+0.215	0.04I	I.256	.	-0.315			+0.0I3	+2.I48	-0.302
C2	0.93	-0.4I	+0.266	0.055	I.728	.	-0.389			+0.0I8	+2.925	-0.372
5	I.4	-0.57	+0.370	0.08I	2.512	.	-0.542			+0.026	+4.285	-0.5I4
6	2.43	-0.825	+0.535	0.I3I	4.082	-3.I	-0.784	0.869	-0.278		+3.95	-I.062
C3	2.736	-0.88	+0.575	0.145	4.553	-4.4	-0.835	0.855	-0.274		+3.45	-I.I09
7	3.87	-1.1	+0.7I5	0.202	6.30	-9.2	-I.05	0.798	-0.255		+I.69	-I.305
8-C4	5.69	-1.3I	+0.850	0.274	8.60	-15.4	-I.245	0.726	-0.233		-0.26	-I.478
9	8.0I	-I.46	+0.942	0.354	II.I2	-21.8	-1.378	0.646	-0.207		-I.73	-1.585
C5	9.968	-1.55	+I.008	0.428	13.50	-26.4	-1.473	0.572	-0.I83		-I.95	-I.656
E-10	10.76	-I.6	+I.04	0.452	14.2	28.2	-1.520	0.548	-0.I75		-2.20	-I.695

Ordinates of influence line for quarter point: Table VIII

	$M\frac{1}{4} = Mo - 0.65\ Ho + 31.4\ Vo$					$N\frac{1}{4} = 0.95\ Ho + 0.32\ Vo$				
Load at Section	Mo	Ho	− 0.65 Ho	Vo	+31.4 Vo	0.95 Ho	0.32 Vo		$M\frac{1}{4}$	$N\frac{1}{4}$
⊏-10′	10.76	-1.6	1.04	-0.452	-14.20	-1.52	-0.145		- 2.40	-1.665
C5	9.968	-1.55	1.008	-0.428	-13.50	-1.472	-0.137		- 2.53	-1.610
9′	8.01	-1.46	0.942	-0.354	-11.12	-1.378	-0.113		- 2.17	-1.490
8′-C4	5.69	-1.31	0.850	-0.274	-8.60	-1.245	-0.088		- 2.05	-1.333
7′	3.87	-1.1	0.715	-0.202	-6.30	-1.050	-0.065		- 1.72	-1.115
C3	2.736	-0.88	0.575	-0.145	-4.55	-0.835	-0.0465		- 1.235	-0.880
6′	2.43	-0.825	0.535	-0.131	-4.08	-0.784	-0.042		- 1.12	-0.826
5′	1.4	-0.57	0.370	-0.081	-2.512	-0.542	-0.026		- 0.75	-0.568
C2	0.93	-0.41	0.266	-0.055	-1.728	-0.389	-0.018		- 0.532	-0.408
4′	0.677	-0.33	0.215	-0.041	-1.256	-0.315	-0.013		-0.364	-0.328
3′	0.287	-0.16	0.104	-0.0186	-0.580	-0.152	-0.006		-0.190	-0.158
C1	0.13	-0.08	0.052	-0.0086	-0.270	-0.076	-0.0028		-0.085	-0.0788
2′	0.078	-0.05	0.033	-0.0052	-0.157	-0.0475	-0.0017		-0.056	-0.0492
⊟-1′	•	•	•	•	•	•	•		-0.02	-0.01

Ordinates of Influence line for Springing Section (table IX)

$M_s = M_o - 15.37\ H_o + 59.2\ V_o - (59.2-a)$

$N_s = H_o \cos. s - (1-V_o) \sin.s.$ $\sin.s=0.6,\ \cos.s.= 0.8$

Section	Mo	Vo	Ho	-I5.37 Ho	+59.2 Vo	-59.2+a m=-(59.2-a)	0.8 Ho	0.6 (1-Vo)	Ms	Ns
I	0.	0'	0	0	0	-2.5	0	-0.6	-2.5	-0.6
2	0.078	0.0052	-0.05	+ 0.77	+ 0.307	- 7.8	- 0.04	-0.597	- 6.65	- 0.64
C1	0.I3	0.0086	-0.08	+I.229	+ 0.475	- 9.2	-0.064	-0.595	- 7.36	- 0.66
3	0.287	0.0I86	-0.I6	+2.46	+ I.065	-I3.4	-0.I28	-0.588	- 9.58	-0.7I6
4	0.677	0.04I	-0.33	+ 5.I2	+ 2.38	-I9.0	-0.266	-0.576	-I0.83	-0.842
C2	0.93	0.055	-0.4I	+ 6.I6	+ 3.25	-2I/0	-0.328	-0.568	-I0.65	-0.896
5	I.4	0.08I	-0.57	+ 9.20	+ 4.85	-25.0	-0.456	-0.552	-9.55	-I.008
6	2.43	0.I3I	-0.825	+I2.70	+ 7.80	-30.9	-0.6600	-0.52	-8.00	-I.I8
C3	2.736	0.I45	-0.88	+I3.55	+ 8.55	-32.2	-0.704	-0.5I6	-7.40	-I.220
7	3.87	0.202	-I.I	+I6.90	+ II.83	-37.	-0.88	-0.48	-4.40	-I.36
8	5.69	0.274	-I.3I	+20.02	+ I6.24	-43.2	-I.048	-0.44	-I.25	-I.488
9	8.0I	0.354	-I.46	+22.35	+2I.30	-49.2	-I.I68	-0.387	+2.46	-I.555
C5	9.968	0.428	-I.55	+23.90	+25.34	-54.2	-I.240	-0.345	+5.00	-I.585
I0	I0.76	0.452	-I.6	+24.64	+27.I4	-56.	-I.28	-0.33	+6.54	-I.6I

Ordinates of Influence line for Springing Section.

Section	Mo	Vo	Ho	-I5.37 Ho	59.2 Vo	0.8 Ho	+0.6 Vo		Ms	Ns
10'	I0.76	-0.452	-1.6	+24.64	-27.14	- I.28	-0.2712		+ 8.26	-1.55I2
C'5	9.965	-0.425	-1.55	+23.90	-25.34	I.24	-0.255		+ 8.55	-1.49.5
9'	8.01	-0.354	-1.96	+22.35	-21.30	- 1.168	-0.2I24		+ 9.06	-I.38
8'-C'	5.69	-0.274	-1.31	+20.02	-16.24	- 1.05	-0.1644		+ 9.47	-I.2I5
7'	3.87	-0.202	-1.1	+16.90	-11.83	- 0.88	-0.1212		+ 8.95	-I.00
C'3	2.736	-0.145	-0.88	+I3.55	-8.55	-0.704	-0.087		+ 7.75	-0.79
6-'	2.43	-0.131	-0.825	+12.70	-7.80	-0.66	-0.078		+ 7.33	-0.738
5'	I.4	-0.08I	-0.57	+9.20	-4.85	-0.46	-0.0486		+ 5.75	-0.51
C'2	0.93	-0.055	-0.4I	+6.I6	-3.25	-0.328	-0.033		+ 3.84	-0.34
4'	0.677	-0.041	-0.33	+5.12	-2.38	-0.266	-0.024		+ 2.08	-0.29
3'	0.287	-0.0186	-0.16	+2.46	-1.065	-0.128	-0.0I08		+ 0.88	-0.138
C'1	0.I3	-0.0086	-0.08	+I.23	-0.475	-0.064	-0.0048		+ 0.55	-0.069
2	0.078	-0.0052	-0.05	+0.77	-0.307	-0.04	-0.003		+ 0.53	-0.043
									+ 0.12	-0.008

Dead-Load, Live Load Moments and Thrusts X

	Section	Weight Ax 6.5x150	Live Load.	CROWN SECTION ordinate Mc	DL.Mc	ordinate Nc	DL.Nc	Upper fiber LL.Mc	LL.Nc
A	I	I2350		-0.0I5	- I86	-0.0I	- I24		
	2	I0590		-0.I35	- I430	-0.05	- 530		
	Cl	46600	I9700	-0.2I8	-I0300	-0.08	- 3720		
	3	9220		-0.4I0	- 3780	-0.I6	- I480		
	4	8450		-0.758	- 6530	-0.33	- 2820		
	C2	47I50	I9300	-0.853	-40220	-0.4I	-I9330		
	5	8I60		-I.05	- 8550	-0.57	- 4650		
	6	8000		-I.I4	- 9I20	-0.825	- 6600		
	C3	43850	I8800	-I.09	-47800	-0.88	-42060		
	7	7900		-0.9	- 7II0	-I.I0	- 8690		
	8-C4	50275	I8800	+0.002	+ I005	-I.3I	-65360	376	- 24700
	9	7700		+I.66	+I2890	-I.46	-II250	58950	- 28365
	C5	40400	I8300	+3.226	+I30500	-I.55	-62620		
C	I0	7650		+3.89	+29800	-I.66	-I2700		
	I0'	7650							
	C5'	40400	I8300					58950	- 28365
	9'	7700							
	8'-C4'	50275	I8800					376	- 27700
	7'	7900							
	C3'	43850	I8800						
	6'	8000							
	5'	8I60							
	C2'	47I50	I9300						
	7'	8450							
	3'	9220							
	CI'	46600	I9700						
	2'	I0590							
B.	I'	I2350							
					+784I0ft-lb		-483870lb	+II8652 ft-lb	-I06I30 lb

Crown-Section		Quarter point				Section		
Lower fiber		ordinate		ordinate		Upper fiber		Lower
LL.Mc	LL.Mc	M1/4	DL.M$\frac{1}{4}$	N 1/4	DL.N1/4	LL.M$\frac{1}{4}$	LL.N $\frac{1}{4}$	LL.M $\frac{1}{4}$
		+ 0.13	+ 1600	- 0.01	- 124			
		+ 0.268	+ 2860	- 0.046	- 487			
- 4350	-1600	+ 0.452	+20970	- 0.075	-3995	+8900	-1477	
		+ 0.971	+ 8950	- 0.146	-1350			
		+ 2.148	+18175	- 0.302	-2535			
- 4700	-7800	+ 2.925	+138150	- 0.372	-17540	+56450	-7183	
		+ 4.285	+34925	- 0.514	-4240			
		+ 3.95	+31600	- 1.062	-8495			
-20500	-16500	+ 3.45	+151290	- 1.109	-48670	+64860	-20870	
		+ 1.69	+13350	- 1.305	-10270			
		- 0.26	-13070	- 1.478	-74300			4700
		- 1.73	-13320	- 1.585	-12200			
		- 1.95	-78780	- 1.656	-66900			35680
		- 2.20	-16830	- 1.695	-12960			
		- 2.40	-18360	- 1.665	-12740			
		- 2.53	-102210	- 1.610	-65044			46300
		- 2.17	-16710	- 1.49	-11550			
		- 2.05	-103060	- 1.33	-67030			38540
		- 1.72	-13480	- 1.115	- 8800			
-20500	-16500	- 1.235	-54120	- 0.88	-38600			23500
		- 1.12	-8960	- 0.826	-6610			
		- 0.75	-6120	- 0.568	-4640			
-14700	- 7800	- 0.532	-25140	- 0.408	-19330			10230
		-0.364	-3075	- 0.328	-2790			
		- 0.190	-1750	- 0.158	-1450			
- 4350	- 1600	- 0.085	-3960	- 0.078	-3670			1580
		- 0.056	- 595	- 0.049	- 530			
		- 0.02	- 250	- 0.01	- 125			
-79,100 ft-lb	-53,800 lb		57,930 ft-lb		-504,175 lb	130,310 ft-lb	-29,530 lb	160,450 ft-lb

$\frac{1}{4}$pt.sect.	SPRINGING SECTION							
fiber	ordinate		ordinate		Upper fiber		Lower fiber	
LL.Nl/4	Ms+	DL.Ms	Ns	DL.Ns	LL.Ms	LL.Ns	LL.Ms	LL.Ns
	- 2.5	- 30875	- 0.6	- 7410				
	- 6.65	- 70400	- 0.64	- 6785				
	- 7.35	-342500	- 0.66	-30756			-144800	-13000
	- 9.58	- 88300	-0.716	- 6600				
	-10.83	- 91225	-0.842	- 7100				
	-10.65	-502150	-0.896	-42200			-205550	-17350
	- 9.55	-77930	-1.008	- 8181				
	- 8.00	- 64000	-1.118	- 8944				
	- 7.40	-324500	-1.220	-53500			-139120	-22936
	- 4.40	- 34760	-1.36	-10744				
-27825	- 1.25	- 62850	-1.488	-74400			-23500	-28014
	+ 2.46	+18940	-1.555	-11973				
-30200	+ 5.00	+202000	-1.585	-63850	+ 91500	-29005		
	+ 6.54	+ 50030	-1.61	-12316				
	+ 8.26	+ 63190	-1.551	-11857				
-30375	+ 8.55	+345400	-1.495	-60600	+156460	-27450		
	+ 9.06	+ 69760	-1.38	-10626				
-28700	+ 9.47	+476100	-1.215	-57550	+178030	-22936		
	+ 8.95	+ 70700	-1.00	-50275				
-16550	+ 7.75	+339850	-0.79	-35080	+145700	-15014		
	+ 7.33	+ 58640	-0.738	- 5904				
	+ 5.75	+ 46920	-0.51	- 4176				
- 7910	+ 3.84	+181000	-0.34	-15700	+ 74110	- 6562		
	+ 2.08	+ 17570	-0.29	- 2535				
	+ 0.88	+ 8110	-0.138	- 1290				
- 1560	+ 0.55	+ 25630	-0.069	- 3262	10830	1379		
	+ 0.53	+ 5610	-0.043	- 455				
	+ 0.12	+ 1480	-0.008	- 98				
143,115 lb		309640 ft-lb		-604165 lb	+656630 ft-lb	102346 lb	-512970 ft-lb	-81320 lb

	Fiber Stresses - Table XII					
	Crown Sect.		Quarter point-Sect		Springing section.	
	+ M	- M	+ M	- M	+ M	- M
moment in ft-lb	230,040	-33690	I00,290	246370	I,050,070	-287,130
thrust in,lb	-582,400	-545270	-526455	-654540	- 7I2580	-679,400
e	0.395	0.06I	0.I9	0.377	I.47	0.423
a	2.66	2.66	2.8	2.8	4.66	4.66
e/a	0.I48	0.024	0.068	0.I35	0.3I	0.09
a/e	.	.	.	.	0.3I9	I0.9
ps	0.008	0.008	0.0I	0.0I	0.0I2	0.0I2
pn	0.08	0.08	0.I	0.I	0.I2	0.I2
d'/a	0.062	0.062	0.057	0.057	0.05	0.05
Case	I	I	I	I	II	I
C	I.5	0.975	I.I3	I.40	6.5	I.I5
fc/lb-in2	860	530	560	865	850	470
fs/Lb-in2	9850	6890	7000	9800	9520	5930

XI. Combined moment and Thrust.

			Dead Load	Live Load	Short. and Temp	T. and S effect	Total
Crown section	Positive moment	+Mc	+ 78410	+ 118650	fall	+ 33000 I/	+230,040 '/#
		Nc	- 483870	-106130	fall	+7600	-582,400 #
	negative moment	+Mc	+ 78410	-79100	rise	- 33000	-33,690 '/#
		Nc	-483870	-53800	rise	- 7600	-545,270 #
quarter point	Positive moment	$M \frac{1}{4}$	- 57920	+130210	fall	+ 28000	+100,290 '/#
		$N \frac{1}{4}$	- 504175	- 29530	fall	+ 7250	-526,455 #
	N.moment	$M \frac{1}{4}$	-57920	160450	rise	- 28000	-246,370 '/#
		$N \frac{1}{4}$	-504175	143115	rise	- 7250	-654,540 #
Sprin-ging section	Positive moment	Ms	+309640	+656630	rise	+ 83800	+1,050,070
		Ns	- 604165	-102346	rise	- 6080	-712,580 #
	negative moment	Ms	+ 309640	-512970	fall	- 83800	- 387,130 '/#
		Ns	- 604165	-81320	fall	+ 6080	-679,400 #