COMPUTATION OF THE STRESSES AND DESIGN

OF THE LOGAN STREET VIADUCT

LANSING, MICHIGAN.

THESIS FOR DEGREE OF B. S.

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1928

Computation of the Stresses and Design
of the Logan Street Viaduct
Lansing, Michigan.

A Thesis Submitted to
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Design of the Logan Street Viaduct

This viaduct which is to be built in the near future consists of an arch section and a beam and girder section three circular arches being used to cross the Grand River and the Viaduct portion making a grade separation over the Grand Trunk and Michigan Central Railroads. The total length will be 1,180 feet and the cost will range from 450,000 to a half million dollars making it one of the greatest projects in this section of the country at the present time.

It is to be built of reinforced concrete or steel encased in concrete.

Because of the nature of this thesis the material consists mainly of computations and a complete set of computations of the stresses in the North Arch are given and results from the other two arches while in the beam and girder sections sample computations are given and results are shown. In the design of abutments and piers the maximum stresses producing maximum thrusts were used and the base was assumed and a graphical solution was used. In the piers because of the number of times the computations were repeated the final results are given.

In order that the reader be more familiar with the project and the prospective design an architect's sketch is shown drawn by R.F.Rey, City Structural Engineer of Lansing.

There will be a 39 foot roadway and a walk on the west side of 7'9" and a curb on the east of 3'3". It being necessary to construct in this manner because of property lines and clearances.

The Oldsmobile Company on the last will have an under pass and entrance onto the viaduct portion just below Isaac Street.

The Grand Trunk Railroad Company will lower its siding to the Olds. 4' in order to facilitate clearance while the Olds have complied by agreeing to lower their platforms.

The wearing surface of the roadway will be 3" brick or a ½ inch sand cushion.

The abutments are of a semi-gravity type but reinforced for expansion cracks. The piers are of elastic type reinforced as shown.

Carnegie I beams will be used in the Steel Spans of which there are 3 at about 42 feet.

In designing the Arches the method of Unit Loading or Influence line method was used and the outline followed was that given in Hool's Volume III of Reinforced Concrete Construction. This being suitable for open spandrel construction.

Reference books used are Hool's Reinforced Concrete Construction-All three volumes, O Roukes Concrete Practice, Ketchum's Handbook Steel Construction, Spoffard's Theory of Structures, and Boyd's Strength of Materials.

An attempt was made to maintain all specification set forth by the Joint Committee at their last meeting although 650# concrete was assumed which is below and only 16,000# steel stresses assumed.

For other specifications those issued by the Michigan State Highway Department were used.

The Arch Section is composed of three arches. First the South arch with a span of 61', the Central arch with a span of 76', and the North arch with a span of 90'.

The grade over the arches is 5%. In fact this is the maximum grade on the structure.

The arches are circular segmental and reinforcing steel is used in top and bottom with extra steel being used at the branch and springing.

This viaduct is in a portion of the city where a bridge of shapely contour is necessary being bounded on the South by the beautiful Moores Park and Shady River Drive, and on the North by a commercial district composed of the Oldsmobile factories. So as an arch gives this desired effect it was thought advisable to use it merging into the beam and girder section where it crosses the railroad tracks.

The following sheets contain the

actual final computations in the design of

the

Logan Street Viaduct

Loading

Loadings on the Structure consist of a 20 ton truck pulling one 15 ton trailer assuming the wheels at 12 feet apart and distributing the load over $\frac{1}{4}$ the road way, that is, assuming 4 trucks side by side on the road way. At points, this load for convenience was transformed into a uniform live load of 212 lbs. per lineal foot.

The sidewalk was designed with a front and rear wheel load of the truck resting on the sidewalk.

Impact was taken as 25% of the total load which is in accordance with specification for a structure of this length.

Computation of Stresses in the

NORTH ARCH

Logan Street Viaduct

Sta.	Elevation Grade	Elevation Extrados	Elevation Intrados	Rise Intra.	Ratio Spg. to Cr.t	Radius Ext.	Radius Intrados
South Arch	Span 61'		Cr. 15''	Spg. 2.5' perpendicular to ring			
203	141.65	132.11	129.00	$\frac{11.3}{61} = .185$	2	53.991'	46.81
233.5	143.175	141.55	140.30				
264	144.70	132.11	129.00				
Central Arch	Span 76'		Cr. 17.5''	Spg. 3.091' or 37.1" perpendicular to ring			
270.00	145.000	132.87	129.00	$\frac{14.79}{76} = .194$	$2\frac{1}{8}$	64.509	56.21
308.00	146.900	145.25	143.79				
346.00	148.800	132.87	129.00				
North Arch	Span 90'		Cr. 20"	Spg. 3.71' perpendicular to arch ring			
353.00	149.15	133.70	129.00	$\frac{19.06}{90} = .212$	$2\frac{1}{4}$	71.177'	62.25
398.00	151.40	149.73	148.06				
443.00	153.65	133.70	129.00				
450.00	154.00						

Loadings

Dead Loads - Arch Ring shown on Plate

Wt. of slab and beams(Computed) = 1080# /

1st Col. -(wt. Of)

(base)
1.25 x 11 x 150 = 2060#

Wt. of 6.33' of slab and beam = 6840#

Total 8,900#
Distributed over 3.5 arch ring = 2,540# per ft.

2nd Col.

1.25 x 7 x 150 = 1310#
6840#
8150 ÷ 3.5 = 2,330#

3rd Col.

1.25 x 4.25 x 150 = 800#
6840
7640 ÷ 3.5 = 2,180#

4th Col.

1.25 x .25 x 150 = 47
6840
6887 ÷ 3.5 = 1,975#

5th (no Col.)

6.72' beam and girder = 7250 ÷ 3.5 = 2,070#

6th (No Col.)

7.23' beam and girder = 7,810 ÷ 3.5 = 2,230#

7th
3.25 x 125 x 150 - 610#
7810
8420 ÷ 3.5 = 2,400#

8th Col.

$\frac{1.25 \times 5.00 \times 150 + 7810}{3.5}$ = 2500#

9th Col.

$\frac{1.25 \times 9 \times 150 + 7810}{3.5}$ = 2,510#

10th Col.

$\frac{1.25 \times 14.0 \times 150 + 7810}{3.5}$ = 2980#

Division to make $\frac{S}{I}$ Constant

Measured L of $\frac{1}{2}$ Axis 51.3'
Axi

$P_o = \frac{1.76}{12 \times 20} = .00733$ d' = .201

$A_s = \frac{1.76}{144} = .0122$ sq. ft.

Rein. Rods $\frac{3}{4}$"ϕ 3" c-c

$15A_s = .183$ sq. ft.

Steel Area = 4 x .44 = 1.76 sq. in.

Distance along axis	t	$I_c = \frac{t^3}{12}$	$\frac{t}{2}$	$\frac{t}{2} - d'$	I_s	I(total)	Div.	L. of Div. to make $\frac{s}{I}$ constant
5'	3.35	3.13	1.67	1.47	.40	3.53	1	15.3
10'	3.00	2.25	1.50	1.30	.31	2.56	2	7.75
15'	2.70	1.64	1.35	1.15	.24	1.88	3	5.35
20'	2.45	1.22	1.22	1.02	.19	1.41	4	4.30
25'	2.25	.95	1.12	.92	.16	1.11	5	3.50
30'	2.05	.72	1.02	.82	.12	.84	6	3.10
35'	1.90	.57	.95	.75	.10	.67	7	2.80
40'	1.80	.49	.90	.70	.9	.58	8	2.50
45'	1.70	.41	[illegible]	.65	.8	.49	9	n 2.40
50'	1.69	.40	.84	.64	.075	.475	10	2.20
51.3'	1.68	.40	.84	.64	.075	.475	11	2.10
Spg.	3.71	4.25	1.85	1.65	.49	4.74		

$\frac{I}{S} = .27$

$\frac{S}{I} = 3$

Determination of moments and thrusts at the Crown

	Unit Loads at the Columns					
		Unit Load at L_4			Unit at L_5	
Pt.	m	mx	my	m	mx	my
1	25.9	1051.5	357.4	33.7	1368.2	465.1
2	16.15	498.2	122.7	23.95	738.9	182.0
3	10.3	257.5	50.5	18.1	452.2	88.7
4	5.7	116.2	18.5	13.5	275.4	43.9
5	1.95	32.5	4.2	9.75	162.4	21.0
6				6.5	87.1	9.1
7				3.6	37.8	2.9
8				.9	7.0	.45
9						
10						
11						
Sums	~~28.65~~ 60.1	1955.9	553.3	110.0	3129.0	813.1

North Arch
Determination of Moments and Thrusts at the Crown
Unit Load at Columns

Pt.	x	y	x^2	y^2	Unit at L_1 m	mx	my	Unit at L_2 m	mx	my	Unit at L_3 m	mx	my
L_1	38.2												
L_2	30.4												
L_3	22.6												
L_4	14.7												
L_5	6.9												
1	40.6	13.80	1650	190.5	2.4	97.44	33.1	10.2	414.2	140.8	18.0	730.8	248.4
2	30.85	7.6	955	57.9				.45	13.88	3.42	8.25	254.5	62.7
3	25.0	4.9	625	24.0							2.4	60.0	11.8
4	20.4	3.25	416	10.60									
5	16.65	2.15	277	4.64									
6	13.40	1.4	180	1.96									
7	10.50	.80	110	.64									
8	7.80	.50	61	.25									
9	5.4	.20	28	.04									
10	2.1	.05	4	.00									
11	1.0	.00	1	.00									
Sums		34.65	430.7	290.53	2.4	97.4	33.1	10.65	428.08	144.2	28.6	1045.3	322.9

Unit Load at L_1

$$H_c = \frac{11(33.12)-(2.40)(34.7)}{2(11(290.5)-(34.7)(34.7))} = .0708$$

$$V_c = \frac{97.44}{2(4307.0)} = .0113$$

$$M_c = \frac{2.40-2(.0708)(34.7)}{2(11)} = -.114$$

$$X_o = \frac{-.114}{.0113} = -1.01$$

Unit Load at L_2

$$H_c = \frac{11(144.22)-10.65(34.7)}{3971} = .306$$

$$V_c = \frac{428.08}{8614} = .0497$$

$$M_c = \frac{10.65-2(.306)(34.7)}{22} = -.48$$

$$X_o = \frac{-.48}{.306} = -1.57$$

Unit Load at L_3

$$H_c = \frac{11(322.9)-28.65(34.7)}{3971} = .643$$

$$V_c = \frac{1045.31}{8614} = .121$$

$$M_c = \frac{28.65-2(.643)(34.7)}{22} = -.725$$

$$X_o = \frac{-.725}{.643} = -1.12$$

Unit Load at L_4

$$H_c = \frac{11(553.3)-60.1(34.7)}{3971} = 1.005$$

$$V_c = \frac{1955.9}{8614} = .227$$

$$M_c = \frac{60.1 - 2(1.005)(34.7)}{22} = -.422$$

$$X_o = \frac{-.422}{1.005} = -.419$$

Unit Load at L_5

$$H_c = \frac{11(813.1 - 110(34.7)}{3971} = 1.29$$

$$V_c = \frac{3129.0}{8614} = .363$$

$$M_c = \frac{110 - 2(1.29)(34.7)}{22} = +1.39$$

$$X_o = \frac{+1.39}{1.29} = +1.075$$

Moments and Thrusts

Unit Load @ L_1

$H_c = .0708$ $V_c = .0113$ $M_c = -.114$

	Left Half			Right Half		
Pt.	V_d	$M = H_c V_d$	N	V_d	$H_c V_d$	N
Cr.	-1.01	-.0715	.075			
Spg.	-84.0	-5.95	.76	+8.50	+.602	.645
1	-13.85	- .975	.65	+4.75	+.336	.522
2	+12.50	+ .885	.515	+0.50	+.0354	.385
3	+ 8.70	+ .615	.425	-1.05	-.0744	.330
4	+ 6.15	+ .435	.350	-1.80	-.127	.230
5	+ 4.35	+ .308	.320	-2.15	-.152	.170
6	+ 2.95	+ .209	.270	-2.25	-.159	.120
7	+ 1.85	+ .131	.225	-2.20	-.156	.060
8	+ 1.00	+.0708	.200	-2.05	-.145	.012
9	+ 0.30	+.0212	.160	-1.85	-.131	.035
10	- 0.35	-.0248	.120	-1.50	-.106	.040
11	- 0.00	-.0565	.075	-1.20	-.085	.060

Moments and Thrusts

Unit Load @ L_2

$H_c = .306$ $V_c = +.0497$ $M_o = -.48$

	Left Half			Right Half		
Pt.	V_d	$M = H_cV_d$	N	V_d	H_cV_d	N
Cr.	-1.570	-.480	.310			
Spg.	-27.50	-8.40	.90	+9.5	+ 2.91	.455
1	-14.85	-4.55	.82	+5.5	+ 1.68	.310
2	+ 9.60	+2.94	.71	+1.3	+ .398	.160
3	+ 7.30	+2.23	.63	- .80	- .245	.100
4	+ 4.90	+1.50	.565	- 1.75	- .535	.010
5	+ 3.20	+ .98	.54	- 2.2	- .673	.070
6	+ 1.95	+ .596	.495	- 2.4	- .735	.120
7	+ .95	+ .291	.45	- 2.35	- .72	.180
8	+ .15	+ .045	.42	- 2.3	- .705	.225
9	- .45	- .138	.39	- 2.25	- .687	.275
10	-1.00	- .306	.36	- 2.0	- .612	.280
11	-1.40	- .429	.31	- 1.7	- .520	.203

Moments and Thrusts

Unit Load at L_3

H_c , .643 V_c , .121 M_c , .725

Pt.	Left Half V_d	$M = H_cV_d$	N	Right Half V_d	HcVd	N
Cr.	- 1.12	- .72	.650			
Spg.	- 10.65	- 6.85	1.08	+ 8.70	+ 5.56	.165
1	- 7.85	- 5.05	1.045	+ 4.90	+ 3.13	.050
2	- .65	- .414	.975	+ .60	+ .384	.175
3	+ 4.65	+ 2.98	.920	/-.95	- .607	.240
4	+ 5.90	+ 3.78	.870	- 1.80	- 1.15	.350
5	+ 4.10	+ 2.62	.845	- 2.15	- 1.38	.420
6	+ 2.75	+ 1.76	.810	- 2.30	- 1.47	.465
7	+ 1.70	+ 1.09	.775	- 2.25	- 1.44	.525
8	+ .80	+ .511	.755	- 2.15	- 1.38	.570
9	+ .10	+ .064	.720	- 1.95	- 1.25	.615
10	- .45	- .288	.695	- 1.65	- 1.06	.620
11	- .95	- .607	.650	- 1.30	- .83	.635

Moments and Thrusts

Unit Load at L_4

H_c , 1.005 V_c , .227 M_c , -.419

Left Half **Right Half**

Pt.	V_d	$M = H_cV_d$	N	V_d	H_cV_d	N
Cr.	- .419	-.420	1.00			
Spg.	- 2.80	-2.81	1.25	+ 7.75	+ 7.76	.155
1	- 3.35	-3.37	1.26	+ 4.15	+ 4.16	.350
2	- 2.00	-2.05	1.24	+ .15	+ .16	.540
3	- .15	-.16	1.215	- 1.25	- 1.26	.610
4	+ 1.70	+1.70	1.180	- 1.90	- 1.91	.725
5	+ 3.55	+3.56	1.165	- 2.05	- 2.06	.790
6	+ 3.96	+3.96	1.140	- 2.10	- 2.11	.835
7	+ 2.75	+2.76	1.100	- 2.00	- 2.01	.890
8	+ 1.80	+1.81	1.090	- 1.75	- 1.76	.930
9	+ .95	+.96	1.070	- 1.45	- 1.46	.975
10	+ .30	+.32	1.045	-1.05	- 1.06	.975
11	- .20	- .21	1.000	- .65	- .66	.990

Moments and Thrusts

Unit Load at L_5

$H_c = 1.29$ $V_c = .363$ $M_c = +1.39$

Pt.	Left Half V_d	$M = H_c V_d$	N	Right Half V_d	$H_c V_d$	N
Cr.	+1.075	+1.38	1.290			
Spg.	+2.25	+2.90	1.35	+6.20	+8.00	.46
1	+ .20	+ .258	1.41	+2.95	+3.80	.665
2	-1.20	- .154	1.39	- .35	- .450	.865
3	-1.00	-1.29	1.42	-1.35	-1.74	.935
4	- .35	- .454	1.42	-1.65	-2.13	1.045
5	+ .40	+ .516	1.41	-1.65	-2.13	1.100
6	+1.35	+1.74	1.39	-1.45	-1.87	1.145
7	+2.15	+2.77	1.375	-1.10	-1.42	1.195
8	+3.10	+4.00	1.360	- .70	- .901	1.230
9	+2.85	+3.67	1.340	- .25	- .322	1.270
10	+2.00	+2.58	1.325	+ .30	+ .387	1.270
11	+1.35	+1.74	1.290	+ .80	+1.03	1.280

Data for Influence Lines

Values of NK & NK' Informulas $f_o = \frac{Nk}{bt}$ & $f_c' = \frac{NK'}{bt}$

Unit Load@ L_1

Pt.	t	Left Half P_o	$\frac{X_o}{t}$	K	NK	K'	NK'	Right Half $\frac{X_o}{t}$	K	NK	K'	NK'
Cr.	1.66	.00733	-.574	3.62	-.272	1.80	+.131					
Spg.	3.71	.00329	-2.13	12.6	-9.55	10.75	+8.15	+.252	2.33	+1.50	.3	-.193
I	3.15	.00387	-.476	3.5	-2.27	1.60	+1.04	+.204	2.17	+1.13	.1	-.052
2	2.50	.00488	+.688	4.5	+2.32	2.70	-1.39	+.037	1.20	+.461	-.8	-.308
3	2.20	.00555	+.657	4.25	+1.81	2.40	-1.02	-.103	1.50	-.495	-.3	-+.099
4	2.00	.00610	+.623	4.15	+1.45	2.25	-.785	-.276	2.40	-.552	.5	+.115
5	1.90	.00643	+.505	3.55	+1.14	1.60	-.513	-.470	3.40	-.578	1.5	+.255
6	1.85	.00660	+.418	3.05	+.825	1.20	-.324	-.715	4.55	-.546	2.6	+.312
7	1.75	.00697	+.333	2.60	+.585	.80	-.180	-1.49	8.35	-.500	6.4	+.384
8	1.75	.00697	+.204	2.0	+.400	.1	-.02	-6.90	35.4	-.425	33.3	+.400
9	1.73	.00704	+.076	1.28	+.205	-.6	-.096	-2.16	12.6	-.441	9.8	+.344
10	1.70	.00718	-.121	1.51	-.181	-.3	+.048	-1.55	8.65	-+.346	6.7	+.258
11	1.68	.00725	-.448	3.15	-.236	1.30	+.097	-.843	5.10	-.308	3.3	+.198

Underlined Figures Denote Compression

+ Indicates upper Fiber

- Indicates Lower Fiber

Date for Influence Lines

Values of NK & NK' Informulas $f_c = \frac{NK}{bt}$ & $f'_c = \frac{NK'}{bt}$

Unit Load @ L_2

Pt.	t	Left Half P_o	$\frac{X_o}{t}$	K	NK	K'	NK'	Right Half $\frac{X_o}{t}$	K	NK	K'	NK'
Cr.	1.66	.00733	-.933	5.50	-1.70	3.60	+1.11					
Spg.	3.71	.00329	-2.51	14.70	-13.2	12.8	+11.5	+1.72	10.40	+4.74	8.6	-3.92
I	3.15	.00387	-1.76	10.35	-8.47	8.4	+6.88	+1.72	10.35	+3.20	8.4	-2.61
2	2.50	.00488	+1.65	9.80	+6.96	7.7	- 5.48	+.994	6.15	+.985	4.2	-.671
3	2.20	.00555	+1.60	9.60	+6.05	7.4	-4.65	-1.11	6.70	-.67	4.8	+.48
4	2.00	.00610	+1.32	7.80	+4.40	6.0	-3.39	-27.80	142.9	-1.42	141.1	+1.41
5	1.90	.00643	+.955	5.80	+ 3.13	3.8	-2.05	-5.06	26.60	-1.85	24.9	+1.74
6	1.85	.00660	+.651	4.22	+2.09	2.25	-1.11	-3.31	17.6	-2.11	16.2	+1.94
7	1.75	.00697	+.369	2.80	+1.26	.90	-.405	-2.28	12.5	-2.25	10.7	+1.93
8	1.75	.00697	+.060	1.20	+.504	-.7	-<u>.294</u>	- 1.79	10.0	-2.25	8.0	+1.8
9	1.73	.00704	-.202	2.00	-.780	.10	+ .39	- 1.43	8.15	-2.24	6.3	+1.73
10	1.70	.00718	-.500	3.40	-1.19	1.60	+ .576	- 1.28	7.35	-2.05	5.5	+1.54
11	1.68	.00725	- .829	5.10	-1.58	3.20	+ .99	- 1.06	6.20	+1.82	4.3	+1.26

Underlined Figures Denote Compression
+ Indicates Upper Fiber
- Indicates Lower Fiber

Data for Influence Lines
Values of NK and NK' in formulas for f_c & f_c'

Unit ℄ L_3

Pt.	Left Half t	B_o	$\frac{X_o}{t}$	K	NK	K'	NK'	Right Half $\frac{X_o}{t}$	K	NK	K'	NK'
Cr.	1.66	.00733	-.668	4.15	-2.70	2.30	+1.50					
Spg.	3.71	.00329	-1.71	10.20	-11.0	8.50	+9.19	+9.10	50.96	-+8.40	49.04	-8.10
1	3.15	.00387	-1.53	9.20	-9.61	7.30	+7.63	+19.8	109.4	+5.44	107.7	-5.34
2	2.50	.00488	-.170	1.80	-1.75	000	+00	+.890	5.60	+.98	3.70	-.658
3	2.20	.00555	+1.47	8.70	+8.00	6.70	-6.16	-1.15	6.80	-1.63	5.00	+1.20
4	2.00	.00610	-+2.17	12.30	+10.7	10.0	-8.70	-1.64	9.40	-3.29	7.60	+2.68
5	1.90	.00643	+1.63	9.20	+7.76	7.30	-6.25	-1.73	9.80	-4.12	8.00	+3.36
6	1.85	.00660	+1.17	6.70	+5.43	4.80	-3.89	-1.71	9.40	-4.36	7.80	+3.63
7	1.75	.00697	+.806	4.1	+3.80	3.00	-2.33	-1.57	8.60	-4.52	6.90	+3.62
8	1.75	.00697	+.387	2.7	+2.04	.900	-.68	-1.38	7.70	-3.99	6.00	+3.42
9	1.73	.00704	+.0514	1.2	+.834	-.70	-.487	-1.17	6.60	-4.06	4.80	+2.95
10	1.73	.00718	-.244	2.2	-1.53	.30	+.208	-1.00	6.00	-3.72	4.00	+2.48
11	1.70	.00725	-.555	3.7	-2.40	1.80	+1.17	-.783	4.70	-2.99	2.80	+1.78

Underlined Figures Denote Compression
+ Indicates Upper Fiber
- Indicates Lower Fiber

Data for Influence Lines

Values of NK and NK' in formulas for f_c and f_c'

Unit @ L4

		Left Half							Right Half			
Pt.	t	P_o	$\frac{X_o}{t}$	K	NK	K'	NK'	$\frac{X_o}{t}$	K	NK	K'	NK'
Cr.	1.66	.00733	-.257	2.25	-2.25	.3	+ .3					
Spg.	3.71	.00329	-.606	4.30	-5.37	2.30	+2.88	+13.5	74.94	+11.55	73.06	-11.30
1	3.15	.00387	-.850	5.50	-6.93	3.60	+4.54	+ 3.78	21.60	+ 7.55	20.0	- 7.00
2	2.50	.00488	-.661	4.30	-5.34	2.50	+3.10	+ .064	1.20	+ .647	-.7	- <u>.378</u>
3	2.20	.00555	-.060	1.20	-1.41	-.70	+ <u>.85</u>	- .940	6.00	- 4.35	4.10	+ 2.97
4	2.00	.00610	+.720	4.60	+5.43	2.80	-3.30	- 1.32	7.80	- 6.16	6.00	+ 4.74
5	1.90	.00643	+1.60	9.00	+10.45	7.40	-8.61	- 1.37	[illegible].00	- 6.31	6.10	+ 4.82
6	1.85	.00660	+1.88	10.40	+11.80	8.50	-9.68	- 1.36	7.90	- 6.70	6.00	+ 5.01
7	1.75	.00697	+1.43	8.10	+8.93	6.30	-7.05	- 1.29	7.30	- 6.50	5.50	+ 4.90
8	1.75	.00697	+.95	5.70	+6.21	3.80	-4.14	- 1.08	6.30	- 5.85	4.40	+ 4.09
9	1.73	.00704	+.519	3.50	+3.75	1.60	-1.71	- .855	5.30	- 5.27	3.30	+ 3.22
10	1.70	.00718	+.180	1.80	-+1.88	.00	-000	- .620	4.20	- 4.10	2.20	+ 2.15
11	1.68	.00725	-.125	1.60	-1.60	-.30	+ <u>.3</u>	- .337	2.80	- 2.77	1.00	+ .99

Underlined Figures Denote Compression

+ Indicates Upper Fiber

- Indicates Lower Fiber

Data for Influence Lines
Values of NK and NK' in formulas for f_c and f'_c

Unit # L_5

	Left Half							Right Half				
Pt.	t	P_o	$\frac{X_o}{t}$	K	NK	K'	NK'	$\frac{X_o}{t}$	K	NK	K'	NK'
Cr.	1.66	.00733	+.645	4.10	+5.29	2.30	-2.97	-				
Spg.	3.71	.00329	+.595	4.20	+5.67	2.30	-3.10	+4.69	26.6	-12.25	24.70	-11.20
1	3.15	.00387	+.058	1.25	+1.76	-.7	- .99	+1.81	10.7	- 7.10	8.80	-5.55
2	2.50	.00488	-.045	1.20	-1.67	-.75	+1.04	-.208	2.10	- 1.82	.20	+.177
3	2.20	.00555	-.403	3.10	-4.40	1.20	+1.70	-.744	5.30	- 4.91	3.60	+ 7.37
4	2.00	.00510	-.159	1.70	-2.41	-.1	+.142	-.959	6.00	- 6.27	4.00	+4.18
5	1.90	.00643	+.193	1.90	+2.68	.05	-.705	-1.02	6.20	- 6.82	4.20	+4.60
6	1.85	.00660	+.677	4.30	+5.98	2.40	-3.34	-.884	5.40	- 6.20	3.40	+ 7.00
7	1.75	.00697	+1.15	6.65	+9.15	4.80	-6.60	-.677	4.30	- 5.25	2.20	+2.67
8	1.75	.00697	+1.68	9.30	+12.60	7.40	-10.05	-.420	3.20	- 3.94	1.20	+1.47
9	1.75	.00704	+1.58	8.70	+11.65	6.90	-9.23	-.145	1.80	- 2.29	-.2	+.254
10	1.70	.00718	+1.14	6.50	+8.60	4.70	-6.21	+.179	1.90	- 2.41	-.1	- .127
11	1.68	.00725	+.804	5.10	+6.57	3.10	-4.00	+.479	3.25	- 4.16	1.40	- 1.79

Underlined Figures Denote Compression
+ Indicates Upper Fiber
- Indicates Lower Fiber

No. 1 Loading

Point	Fiber	∠1 Dead Load=10,690# Comp.	Tension	Live L.=15,380# Comp.	Tension	∠2 Dead Load=6,050# Comp.	Tension	Live L.=10,550# Comp.	Tension
							1.54		
10	Upper	.048	.258				.576		
	Lower	.181				2.05			
		.346				1.19			
			.097				.99		
11	Upper		.198				1.26		
	Lower	.308				1.58			
		.236				1.82			

No. I Loading

Point	Fiber	Deadload=10,690# $\angle_1$ Comp.	Tension	Live L.=15,380# Comp.	Tension	Deadload=6,050# $\angle_2$ Comp.	Tension	Live L.=10,550# Comp.	Tension
Cr.	Upper		2(131)				2(1.11)		
	Lower	2(.272)				2(1.70)			
Spg.	Upper		8.15	1.50			1.15	4.74	
	Lower	9.55			.193	13.2			3.92
I	Upper		1.04	1.13			6.88	3.20	
	Lower	2.27			.0522	8.47			2.61
2	Upper			.461 2.32				.935 6.96	
	Lower			.308	1.39				5.48 .671
3	Upper	.099		1.81			.48	6.05	
	Lower	.495			1.02	.67			4.65
4	Upper		.115	1.45	1.45		1.41	4.40	
	Lower	.552			.785	1.42			3.39
5	Upper		.255	1.14			1.74	3.13	
	Lower	.578			.513	1.85			2.05
6	Upper		.312	.825			1.94	2.09	
	Lower	.546			.324	2.11			1.11
7	Upper		.384	.585			1.93	1.26	
	Lower	.500			.180	2.25			.405
8	Upper		.400	.400			1.80		
	Lower	.425			.02	2.25		.504 .294	
9	Upper		.384	.205			1.73 .39		
	Lower	.441		.096		2.24 .780			
10	Upper	.048	.258				1.54 .576		

No. I Loading

Point	Fiber	Deadload=4,980# Comp.	Tension	L2 Live L.=9,320# Comp.	Tension	Deadload=4,370# Comp.	Tension	L4 Live L.=8,490# Comp.	Tension
Cr.	Upper		2(1.50)				2(.3)		
	Lower	2(2.70)				2(2.25)			
Spg.	Upper		9.19	8.40			2.83	11.55	
	Lower	11.0			8.10	5.37			11.30
I	Upper		7.63	5.44			4.54	7.53	
	Lower	9.61			5.34	6.93			7.00
2	Upper			.98			3.10	.647	
	Lower	1.75			.648	5.34 2.97 .85 1.41 4.35		.378	
3	Upper		1.20	8.00					
	Lower	1.63			6.16				
4	Upper		2.68	10.70			4.74	5.43	
	Lower	3.29			8.70	6.16			3.30
5	Upper		3.36	7.76			4.82	10.45	
	Lower	4.12			6.35	6.31			8.61
6	Upper		3.63	5.43			5.01	11.80	
	Lower	4.36			3.39	6.70			9.68
7	Upper		3.62	3.80			4.90	8.93	
	Lower	4.52			2.33	6.50			7.05
8	Upper		3.42	2.04			4.09	6.21	
	Lower	3.99			1.38	5.85			4.14
9	Upper		2.95	.834			3.22	3.75	
	Lower	4.06		m.487		5.27			1.71

No. I Loading

Point	Fiber	Deadload=4,980# ∠3		Live L.=9,320#		Deadload=4,370# ∠4		Live L.=8,490#	
		Comp.	Tension	Comp.	Tension	Comp.	Tension	Comp.	Tension
			2.48						
10	Upper	-	.208				2.15	1.88	
	Lower	.3.72				4.10			
		1.53	1.78						
11	Upper		1.17			-.3	.99		
	Lower	2.99				1.60			
		2.40				2.77			

No. 1 Loading

45

Point	Fiber	Deadload=4,240# Comp.	Deadload=4,240# Tension	Live L.=8,610# Comp.	Live L.=8,610# Tension
Cr.	Upper	2(		2(5.29)	
	Lower				2(2.97)
				12.25	
Spg.	Upper			5.67	
	Lower				11.20
				7.10	3.10
I	Upper			1.76	
	Lower			.99	5.85
2	Upper	1.04	1.73		
	Lower	1.67			
		1.82	1.70		
3	Upper		2.37		
	Lower	4.40			
		4.91			
4	Upper	.142	4.18		
	Lower	2.41			
		6.27			
5	Upper		4.62	2.68	
	Lower	6.82			.705
6	Upper		3.90	5.98	
	Lower	6.20			3.34
7	Upper		[illegible].63	9.15	
	Lower	5.25			6.60
8	Upper		1.47	12.60	
	Lower	3.94			10.05
9	Upper	.254		11.65	
	Lower	2.29			9.23

No. I Loading

Point	Fiber	Deadload=4,240# Comp.	Deadload=4,240# Tension	Live L.=8,610# Comp.	Live L.=8,610# Tension
10	Upper			(2.41+8.60)	
	Lower			.127	6.21
				4.16	
11	Upper			6.57	
	Lower				1.79
					4.00

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Crown

No. 1 Loading

Upper $-2(.131 \times 10690) - 2(1.11 \times 6050) - 2(1.5 \times 4980) - 2(.3 \times 4370) + 2(5.29 \times 8610)$

$f_c = \dfrac{2(28,650)}{1.66 \times 144} = 240$, # COmp.

Lower $2(.272 \times 10690) + 2(1.7 \times 6050) + 2(2.7 \times 4980) + 2(2.25 \times 4370) - 2(2.97 \times 8610)$

$f'_c = \dfrac{2(11,068)}{239} = 92$ # Comp.

No. 2 Loading

Upper $-2(.131 \times 15380) - 2(1.11 \times 10550) - 2(1.5 \times 9320) - 2(.3 \times 8490) + 2(5.29 \times 4240)$

$f_c = \dfrac{-2(7,822)}{239} = 65$ # Tension

Lower $2(.272 \times 15380) + 2(1.7 \times 10550) + 2(2.7 \times 9320) + 2(2.25 \times 8490) - 2(2.97 \times 4240)$

$f'_c = \dfrac{2(53792)}{239} = 447$ # Comp.

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Springing

No. 1 Loading

Upper $-8.15(10690)+1.5(15380)-1.15(6050)+4.74(10550)-9.14(4980)+8.4(9320)$
$-2.88(4370)+11.55(8490)+17.92(8610)-11.2(8610)$

$$f_c = \frac{154,550}{3\ .71 \times 144} = 287\ \#\ \text{Compression}$$

Lower $9.55(10690)-.193(15380)+13.2(6050)-3.92(10550)+11(4980)-8.1(9320)$
$+5.37(4370)-11.3(8490)-3.1(8610)$

$$f'_c = \frac{-35,430}{535} = 62\ \#\ \text{Tension}$$

No. 2 Loading

Upper $-8.15(15380)+1.5(10690)-11.5(10559)+4.74(6050)-9.19(9320)+8.4(4980)$
$-2.88(8490)+11.55(4370)+6.72(4240)$

$$f_c = \frac{-81,600}{535} = 346\ \#\ \text{Tension}$$

Lower $9.55(15380)-.193(10690)+13.2(10550)-3.92(6050)+11(9320)-8.1(4980)$
$+5.37(8490)-11.3(4370)-3.1(4240)$

$$f'_c = \frac{305,240}{535} = 570\ \#\ \text{Comp.}$$

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Point 1

No. 1 Loading

Upper $-1.04(10690)+1.13(15380)-6.88(6050)+3.2(10550)-7.63(4980)+5.44(9320)$
$-4.54(4370)+7.55(8490)+8.86(8610)$

$$f_c = \frac{130,750}{3.15 \times 144} = 288 \text{ \# Comp.}$$

Lower $2.27(10690)-.0522(15380)+8.47(6050)-2.61(10550)+9.61(4980)$
$-5.34(9320)+6.93(4370)-7(8490)+.988610)-5.85(8610)$

$$f'_c = \frac{-25,540}{454} = 51 \text{ \# Tension}$$

No. 2 Loading

Upper $-1.04(15380)+1.13(10690)-6.88(10550)+3.2(6050)-7.63(9320)+5.44(4980)$
$-4.54(8490)+7.55(4370)+8.86(4240)$

$$f'_c = \frac{-67,950}{454} = 149 \text{ \# Tension}$$

Lower $2.27(15380)-.0522(10690)+8.47(10550)-2.61(6050)+9.61(9320)-5.34(4980)$
$+6.94(4980)+ \cancel{77} -7(4370)+.99(4240)-5.85(4240)$

$$f'_c = \frac{178,240}{454} = 393 \text{ \# Tension}$$

Maximum Stresses at Point Due to Loading

Assuming case 1 to apply at all sections

Point 2

No. 1 Loading

Upper $2.8(15380)+7.945(10550)+.98(9320)-3.1(4370)+.647(8490)+1.04(4240)-1.73(4240)$

$$f_c = \frac{124,600}{2.5 \times 144} = 346 \text{ \# Comp.}$$

Lower $.308(15380)-1.39(15380)-6.15(10550)+1.75(4980)-.648(9320)+5.34(4370)+.378(8490)+3.49(4240)$

$$f'_c = \frac{5,340}{360} = 15 \text{ \# Comp.}$$

No. 2 Loading

Upper $2.8(10690)+7.945(15380)+.98(4980)-3.1(9320)+6.47(4370)+1.04(8610)-1.73(8610)$

$$f_c = \frac{124,770}{360} = 346 \text{ \# Comp.}$$

Lower $3.08(10690)-1.39(10690)-6.15(6050)+1.75(9320)-.648(4980)+5.34(8490)+.378(4370)+3.49(4240)$

$$f'_c = \frac{65,570}{360} = 182 \text{ \# Comp.}$$

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Point 3

No. 1 Loading

Upper $.099(10,690)+181(15,380)-.48(6,050)+6.05(10,550)-1.2(4,980)$
$+.8.0(9,320)+3.82(4,370)-5.07(4,240)$

$f_c = \frac{163\ 370}{2.2\times 144} = 515$ # Comp.

Lower $.495(10,690)-1.02(15,380)+.67(6,050)-4.65(10,550)+1.63(4,980)$
$-6.16(9,320)+5.76(4370)+9.31(4240)$

$f'_c = -\frac{40,040}{317} = 126$ # Tension

No. 2 Loading

Upper $.099(15380)+1.81(10690)-.48(10550)+6.05(6050)-1.2(9320)$
$+8.0(4980)+3.82(8490)-5.07(8610)$

$f_c = \frac{84,240}{317} = 266$ # Comp.

Lower $.495(15380)-1.02(10690)+.67(10550)-4.65(6050)+1.63(9320)$
$-6.16(4980)+5.76(8470)+9.31(8610)$

$f'_c = \frac{89,460}{317} = 282$ # COMP.

Assuming Case 1   Maximum Stresses Due to Loading
A

Point 4

No. 1 Loading

Upper $-.115(10,690)+1.45(15,380)-1.41(6,050)+4.40(10,550)-2.68(4,980)$
$+10.70(9,320)-4.74(4,370)+5.43(8,490)+.142(4,240)-4.18(4,240)$

$f_c = \dfrac{153,800}{2.0 \times 144} = 533$ # Comp.

Lower $.552(10,690)-.785(15,380)+1.42(6,050)-3.39(10,550)+3.29(4,980)$
$-8.70(9,320)+6.16(4,370)-3.30(8,490)+8.68(4,240)$

$f'_c = \dfrac{-62,490}{288} = 216$ # Tension

NO. 2 Loading

Upper $-.115(15,380)+1.45(10,690)-1.41(10,550)+4.41(6,050)-2.68(9,320)$
$+10.70(4,980)-4.74(8,490)+5.39(4,370)+.142(4,240)-4.18(4,240)$

$f_c = \dfrac{2,450}{288} = 8$ # COmp.

Lower $.552(15,380)-.785(10,690)+1.42(10,550)-3.39(6,050)+3.29(9,320)$
$-8.70(4,980)+6.16(8,490)-3.30(4,370)+8.68(8,610)$

$f'_c = \dfrac{94,270}{288} = 327$ # COmp.

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Point 5

Upper $-.255(10,690)+1.14(15,380)-1.74(6,050)+3.13(10,550)-3.36(4,980)$
$+7.76(9320)-4.82(4,370)+10.45(8,490)-4.62(4,240)+2.68(8,610)$

No. 1 Loading

$f_c = \dfrac{164,280}{1.9 \times 144} = 598$ # Comp.

Lower $.578(10,690)-.513(15,380)+1.85(6,050)-2.05(10,550)+4.12(4,980)$
$-6.24(9,320)+6.31(4,370)-8.61(8,490)+6.82(4,240)-.705(8,610)$

$f_c' = \dfrac{-72,190}{274} = 263$ # Tension

Upper $-.255(15,380)+1.14(10,690)-1.74(10,550)+3.13(6,050)-3.36(9,320)$
$+7.76(4,980)-4.82(8,490)+10.45(4,370)-4.62(8,610)+2.68(4,240)$

$f_c = \dfrac{-7,420}{274} = 27$ # Tension

Lower $.578(15,380)-.513(10,690)+1.85(10,550)-2.05(6,050)+4.12(9,320)$
$-6.24(4,980)+6.31(8,490)-8.61(4,370)+6.42(8,610).705(4,240)$

$f_c' = \dfrac{89,020}{274} = 326$ # Comp.

Maximum Stresses Due to Loading
Assuming Case 1 to Apply at all Sections

Point 6

Upper $-.312(10690)+.825(15380)-1.94(6050)+2.09(10550)-3.63(4,980)$
$+5.43(9,320)-5.0(4,370)+11.8(8,490)-3.9(4,240)+5.98(8,610)$

No. 1 Loading

$f_c = \dfrac{165,110}{1.85 x 144} = 620$ # Comp.

Lower $.546(10,690)-.324(15,380)+2.11(6,050)-1.11(10,550)+4.36(4,980)$
$-3.89(9,320)+4,370)6.70-9.68(8,490)+6.2(4,240)-3.34(8,610)$

$f_c' = \dfrac{-67,330}{266} = 252$ # ~~Comp.~~ Tension

Upper $-.312(15,380)+.825(10,690)-1.94(10,550)+2.09(6,050)-3.63(9,320)$
$+5.43(4,980)-5.0(8,490)+11.8(4,370)-3.9(8,610)+5.98(4,240)$

No. 2 Loading

$f_c = \dfrac{-9,360}{266} = 26$ # Tension

Lower $.546915,380)-.324(10,690)+2.1(10,550)-1.11(6,050)+4.36(9,320)$
$-3.89(4,980)+6.7(8,490)-9.68(4,370)+6.2(8,610)-3.34(4,240)$

$f_c' = \dfrac{96,180}{266} = 261$ # Comp.

Maximum Stresses Due to Loading
Assuming Case I to apply at all Sections

Point 7

No. I Loading

Upper $-.384(10,690)+.585(15,380)-1.93(6,050)+1.26(10,550)-3.62(4,980)$
$+3.80(9,320)-4.90(4,370)+8.93(8,490)-2.63(4,240)+9.15(8,610)$

$$f_c = \frac{145,850}{1.75\times 144} = 577 \text{ \# Comp.}$$

Lower $.500(10,690)-.186(15,380)+2.25(6,050)-.405(10,550)+4.52(4,980)$
$-2.33(9,320)+6.50(4,370)-7.05(8,490)+5.25(4,240)-6.60(8,610)$

$$f'_c = \frac{-52,300}{252} = 207 \text{ \# Tension}$$

No. 2 Loading

Upper $-.384(15,380)+.585(10,690)-1.93(10,550)+1.26(6,050)-3.62(9,320)$
$+3.80(4,980)-4.90(8,490)+8.93(4,370)-2.63(8,610)+9.15(4,240)$

$$f_c = \frac{-13,440}{252} = 53 \text{ \# Tension}$$

Lower $.500(15,380)-.80(10,690)+2.25(10,550)-.405(6,050)+4.52(9,320)$
$-2.33(4,980)+6.50(8,490)-7.05(4,370)+5.25(8,610)-6.60(4,240)$

$$f'_c = \frac{98,860}{252} = 361 \text{ \# Comp.}$$

Maximum Stresses Due to Loadings
Assuming Case I to apply at all Sections

Point 8

No. I Loading

Upper $-.4(10,690)+.4(15,380)-1.8(6,050)+.504(10,550)-3.42(4,980)+2.04(9320)$
$-4.09(4,370)+6.21(8,490)-1.41(4,240)+12.60(8,610)$

$$f_c = \frac{135,470}{1.75x144} = 536 \text{ \# Comp.}$$

Lower $.425(10,690)-.02(15,380)+2.25(6,050)+2.94(10,550)+3.99(4,980)$
$-1.38(9,320)+5.85(4,370)-4.14(8,490)+3.94(4,240)-10.05(8,610)$

$$f'_c = \frac{-51,367}{252} = 203 \text{ \# Tension}$$

No. 2 Loading

Upper $-.4(15,380)+.4(10,690)-1.8(10,550)+.504(6,050)-3.42(9,320)+2.04(4,980)$
$-4.09(8,490)+6.21(4,370)-1.41(8,610)+12.60(4,240)$

$$f_c = \frac{-5,840}{252} = 23 \text{ \# Tension}$$

Lower $.425(15,380)-.02(10,690)+2.25(10,550)+.294(6,050)+3.99(9,320)$
$-1.38(4,980)+5.85(8,490)-4.14(4,370)+3.94(8,610)-10.05(4,240)$

$$f'_c = \frac{84,917}{252} = 337 \text{ \# Comp.}$$

Maximum Stresses Due to Loading
Assuming Case I to apply at all sections.

Point 9

No. I Loading

Upper $-.384(10{,}690)+.205(15{,}380)-2.12(6{,}050)-2.95(4{,}980)+.834(9320)$
$-3.22(4{,}370)+3.75(8{,}490)+.254(4{,}240)+11.65(8.610)$

$$f_c = \frac{98{,}585}{1.73 \times 144} = 395 \ \# \text{ Comp.}$$

Lower $.441(10{,}690)+.096(15{,}380)+3.02(6{,}050)+4.06(4{,}980)+.487(9{,}320)$
$+5.27(4{,}370)-1.71(8{,}490)+2.29(4{,}240)-9.23(8{,}610)$

$$f'_c = \frac{-11{,}985}{249} = 47 \ \# \text{ Tension}$$

No. 2 Loading

Upper $0.384(15{,}380)+.205(10{,}690)-2.12(10{,}550)-2.95(9{,}320)+8.34(4{,}980)$
$-3.22(8{,}490)+3.75(4{,}370)+.254(8{,}610)+11.65(4240)$

$$f_c = \frac{-8{,}760}{249} = 35 \ \# \text{ Tension}$$

Lower $.441(15{,}380)+.096(10{,}690)+3.02(10{,}550)+4.06(9{,}320)+.487(4{,}980)$
$+5.27(8{,}490)-1.71(4{,}370)+2.29(8{,}610)-9.23(4{,}240)$

$$f'_c = \frac{98{,}515-1{,}000}{249} = 391 \ \# \text{ Comp.}$$

Maximum Due to Loading

Assuming Case I to apply at all sections

Point 10

No. I Loading

Upper $.048(10{,}690)-.258(10{,}690)-2.116(6{,}050)-2.608(4{,}980)-2.15(4{,}370)+1.88(8{,}490)+11.01(8{,}610)$

$$f_c = \frac{78{,}210}{1.70\times144} = 320\# \text{ Comp.}$$

Lower $.527(10{,}690)+3.24(6{,}050)+5.25\ (4{,}980)+4.10(4{,}370)+.121(8610)-6.21(8{,}610)$

$$f'_c = \frac{16{,}750}{245} = 68\# \text{ Comp.}$$

No. 2 Loading

Upper $.048(15{,}380)-.258(15{,}380)-2.116(10{,}550)-2.608(9{,}320)-2.15(8{,}490)+1.88(4{,}370)+11.011(4{,}240)$

$$f_c = \frac{-13{,}233}{245} = 54\ \# \text{ Tension}$$

Lower $.527(15{,}380+\ 3.24(10{,}550)+5.25(9{,}320)+4.10(8{,}490)+.121(4{,}240)-6.21(4{,}240)$

$$f'_c = \frac{100{,}112}{245} = 410\ \# \text{ Comp.}$$

Maximum Stresses Due to Loading

Point 11

No. 1 Loading

Upper $= -.295(10{,}690) - 2.25(6050) - 2.95(4980) + .3(4370) - .99(4370) + 10.73(8610)$

$$f_c = \frac{+58{,}140}{1.68 \times 144} = 240\,\#\ \text{Comp.}$$

Lower $.544(10690) + 3.40(6050) + 5.39(4980) + 4.37(4370) - 5.79(8610)$

$$f'_c = \frac{22{,}700}{242} = 94\,\#\ \text{Comp.}$$

No. 2 Loading

Upper $-.295(15{,}380) - 2.25(10{,}550) - 2.95(9{,}320) + .3(8490) - .99(8490) + 10.73(4240)$

$$f_c = \frac{-16{,}100}{242} = 66\,\#\ \text{Tension}$$

Lower $.544(15{,}380) + 3.40(10{,}550) + 5.39(9{,}320) + 4.37(8{,}490) - 5.79(4{,}240)$

$$f'_c = \frac{107{,}050}{242} = 441\,\#\ \text{Comp.}$$

North Arch

Maximum StressDue to Loading

Point	No. 1 Loading Upper	No. 1 Loading Lower	No. 2 Loading Upper	No. 2 Loading Lower
Cr	240	92	-65	447
Spg.	287	-62	-346	570
1	288	-51	-149	393
2	346	15	346	182
3	515	-126	266	282
4	533	-216	8	327
5	598	-263	-27	326
6	620	-252	-26	261
7	577	-207	-53	361
8	536	-203	-23	337
9	395	-47	-35	391
10	320	68	-54	410
11	240	94	-66	441

+ Compression

\- Tension

North Arch

Total Maximum Stress
Assuming Case 1 to apply to all sections

Loading (a)- No.1 Loading, fall of temp. and rib shortening

Loading (b)- No.2 Loading, rise of temp. and rib shortening

Loading (c)- No.2 Loading, fall of temp. and rib shortening

Loading (d)- No.2 Loading, rise of temp. and rib shortening

Temperature (fall)

$$H_c = -\frac{(.27)(.000006)(20)(92.7)(11)(2000000)144}{2(11(290.53)-34.65 \times 34.65)}$$

$$= -\frac{9,515,030}{3971} = -2,390\#$$

$$M_c = \frac{2390 \times 34.65}{11} = 7,526\#$$

Rib Shorting

$$C_a \text{ (At crown)} = \frac{.0708(10690 + 15380) + .306(6050 + 10550) + (4980 + 9320) + 1.005(4370 + 8490) + 1.29(4240 + 8610)}{1.68 + .183}$$

$$= 24,600$$

$$C_a \text{ (at point 4)} = \frac{(.35 + .23)(36070) + (.565 + .01)(16600) + (.87 + .35)(14300) + (1.180 + .725) + (1.42 + 1.05)(12850)}{2 \times 2 \times 1 + 2 \times .183}$$

$$= 23,900$$

$$C_a \text{ (at Springing)} = \frac{1}{2} \cdot \frac{(.76+.645)(36070)+(.9+.455)(16600)+(1.08+.165)(14300)+(1.25+.155)(12860)+(1.35+.46)(12850)}{3.71+.183}$$

$$= 17{,}000$$

$$C(\text{avg.}) = 3 \ \underline{/\ 65{,}700}$$

$$= 21{,}900 \text{ lb. per sq. ft.}$$

$$H_c = -\frac{(.27)(21900)(92.7)(11)}{3971}$$

$$= -1515$$

$$\text{Ratio OF} \quad \frac{H_c \text{ for Rib Shortening}}{H_c \text{ for temperature}} = \frac{1515}{2390}$$

$$= .635$$

North Arch

Assuming case 1 to apply at all sections

Rise of Temp.& Rib Shortening $\frac{-1 + .635}{1 + .635} = -.223$			Total maximum stresses case1								SECTIONS & LOADINGS at which ~~case~~ case 1 does not apply
point	stress in		(a) 10ading		(b) loading		(c) loading		(d) loading		
	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	
Cr.	-31	+37	377	-75	209	129	72	280	-96	484	
Spg.	42	-40	99	116	329	-102	-534	648	-304	530	c& d
1	36	-34	125	100	324	-85	-312	544	-113	359	c& d
2	22	-21	246	108	368	-6	246	275	368	161	
3	14	-9	453	-85	529	-135	204	323	280	173	
4	2	-3	523	-231	535	-219	-2	312	10	324	
5	-4	10	617	-216	594	-253	-8	373	-31	336	a & b worst
6	-12	18	673	-334	608	-234	27	179	-38	279	
7	-18	26	659	-322	559	-181	29	146	-71	387	
8	-23	29	638	-333	513	-174	79	207	-46	366	
9	-26	33	512	-195	369	-14	82	243	-61	424	
10	-14	22	385	-30	306	90	11	322	-68	432	
11	-22	29	339	-37	218	123	33	310	-88	470	

North Arch

Total Maximum Stresses

Fall of Temperature and Rib Shortening

$f_c = \frac{NK}{bt}$ $\qquad f_s = \frac{NK'}{bt}$

Point	M	N	$\frac{x_0}{t}$	K	K'	Stress in Upper	Stress in Lower
Cr.	+12250	-3900	-1.89	10.20	8.40	+137	-167
Spg.	-66400	-2710	+660	37.20	35.2	-188	+178
1	-41700	-3140	+4.22	23.60	21.8	-163	+151
2	-17400	-3460	+201	11.50	9.7	-100	+93
3	-6870	-3580	+.875	5.50	3.6	-62	+41
4	-410	-3710	+.055	1.20	-.8	-10	-15
5	+3480	-3770	-.485	3.40	1.4	+19	+47
6	+6800	-3820	-.937	5.70	3.70	+53	-82
7	+9150	-3850	-1.28	7.50	5.40	+82	-115
8	+10310	-3890	-1.61	8.40	6.60	+102	-130
9	+11470	-3890	-1.70	9.50	7.50	+117	-148
10	+12030	-3910	-1.81	6.15	4.10	+65	-98
11	+12250	-3910	-1.86	8.10	6.15	+99	-131

North Arch

Total Maximum Stresses

Assuming case 1 to apply at all sections

Point	Y	T	P_o	Temp. fall of 20° $H_c y$	Temp. fall of 20° M	Temp. fall of 20° N	Rib Shorting M	Rib Shorting N
Cr.		1.66	.00733		+7500	-2390	+4750	-1510
Spg.	19.9	3.71	.00329	-47500	-40000	-1660	-26400	-1050
1	13.8	3.15	.00387	-33000	-25500	-1920	-16200	-1220
2	7.60	2.50	.00488	-18500	-20200	-2120	-6750	-1340
3	4.90	2.20	.00555	-11700	-4400	-2190	-2670	-1390
4	3.25	2.00	.00610	-7750	-250	-22160	-160	-1440
5	2.15	1.90	.00643	-5370	+2130	-2310	+1350	-1460
6	1.40	1.85	.00660	-3340	+4160	-2340	+2640	-1480
7	.80	1.75	.00697	-1900	+5600	-2360	+3559	-1490
8	.50	1.75	.00697	-1190	+6310	-2380	+4000	-1510
9	.20	1.73	.00704	-480	+7020	-2380	+4450	-1510
10	.05	1.70	.00718	-120	+7380	-2390	+4650	-1520
11	.00	1.68	.00725	0	+7500	-2390	+4750	-1520

North Arch

Total maximum stress for case 11 conditions

point	t	type of loading	Loading No.1 M	Loading No.1 N	Loading No.2 M	Loading No.2 N	Fall of Temo. & Rib Short. M	Fall of Temo. & Rib Short. N	Rise of Temp. & Rib Short. M	Rise of Temp. & Rib Short. N	Totals M	Totals N
Sog.	3.71	(c)			-133440	+60688	-66400	-2710			-199840	+57978
6	1.85	(a)	-34180	+46735			+6800	-3820			-27380	+ 42915
11	1.68	(d)			-21219	+48575			-2750	+870	-23969	+49645
1		(c)			-72480	+57200	-41700	-3140			-114180	+54060

Total Maximum Stresses

$\frac{x_o}{t}$	K	L	f_c	f_s
.925	.28	.084	850 will be taken care in base	850
.352	.62	.1165	475	less
.237	.74	.1175	102	less

Total Maximum Stress for Case 11 Conditions
Moments and Thrusts

Point	t	P_o	Type Of Loading	Loading No.1 Loading M	Loading No.1 Loading N	No,2 Loading M	No,2 Loading N	Fall of Temp. & Rib Short. M	Fall of Temp. & Rib Short. N	Rise of Temp. & Rib Short.	Total M	Total N
Spg.	2.5	.00488	(c)			-70750	47650	-26330	-2246		-97,080	45,404
1	2.2	.00555	(c)			-42790	44101	-18380	-2420		-61,170	41,681
5	1.40	.00873	(d)			+11172	43544	-634	+810		+10,538	44,354

Total Maximum Stresses

Point	$\frac{x}{t}$	K	L	f_c	f_s
Spg.	.86	.33	.100	825	7,700
1	.685	.39	.105	780	7,000
5	.170	Case 1 Holds			

South Arch

Loadings

Arch Ring   Considering the weight concentrated at the Column points

Col.1	2.55' x 8.6 x 150	equals	3,290 #
Col.2	1.97 x5.85 x150	"	1,730 #
Col.3	1.65 x 5.55 x 150	"	1,370 #
Col.4	1.37 x 5.55 x 150	"	1,140 #
Col.5	1.27 x 5.55 x 150	"	1,050 #

Total Dead Loads

Col.1	5,160 #
Col.2	3,550 #
Col.3	3,090 #
Col.4	2,725 #
Col.5	2,730 #

Dead + Live Loads

Col.1	8,420 #
Col.2	7,030 #
Col.3	6,570 #
Col.4	6,245 #
Col.5	6,280 #

South Arch

Loadings Left Half

Dead Loads

Wt. of beam, girder, and slab = 1080# /lin. foot

1st. Col.(wt.of)

1.36 x 5.5 x 150 = 1,120#

Wt. of 5' of B. and G.= 5400
Total 6520
Distributed over 3.5' of Arch ring= 1,870"concentrated at 1st Col.

2nd Col.

1.36 x 2.9 x 150 = 590#

Wt. of 5.35' B.&G.=5770
6360 ÷ 3.5 = 1820# @ COL .2

3rd Col.

1.36 x 1.20 x 150 = 245
5.35'B.&G.5770
6,015 ÷ 3.5 = 1720# @ Col.3

4th Col.

(No Col.)
5.35' B.& G.= 5770 ÷ 3.5 = 1625# @ Col.4

5th Col.

(No Col.)
5.45' B.& G. = 5890 ÷ 3.5 = 1680# @ Col.5

Live Loads 2280# /lin.foot

(1) 5 x 2280 ÷ 3.5 = 3260#

(2) 5.35x2280 ÷ 3.5 = 3480#

(3) 5.35x2280 ÷ 3.5 = 3480#

(4) 5.35x2280 ÷ 3.5 = 3480#

(5) 5.45x2280 ÷ 3.5 = 3550#

Division to make $\frac{S}{I}$ Constant

Measured Length of 1/2 Arch Axis 33.85' — $P_o = 1.76/(12 \times 15) = .00976$

Reinforcing Rods 3/4 " round Sp. 6" o-c — $a_s = 1.76 / 144 = .0122$ sq. ft.

Steel Area 4x .44 = 1.76 sq. in. — $15\ a_s = .183$ sq. ft.

Dist. along Axis (ft.)	t ft.	I_c	t/2	t/2 -d'	$15\ I_s$	I	Div.	Length of Div.
Spg.	2.5	1.3	1.25	1.05	.201	1.501	1	9.41
5'	2.17	.85	1.08	.88	.141	.999	2	5.40
10	1.90	.57	.95	.75	.104	.674	3	3.55
15	1.65	.375	.825	.624	.071	.446	4	2.85
20	1.47	.265	.375	.534	.052	.317	5	2.60
25	1.35	.205	.675	.474	.041	.246	6	2.21
30	1.27	.171	.635	.434	.034	.205	7	2.10
33.83	1.25	.162	.625	.424	.033	.195	8	2.00
							9	1.85
							10	1.80

$I/S = .1094$

$S/I = 9.14$

Division to make S/I Constant

Measured Length of 1/2 of Axis 33.85
Reinforcing Rods 3/4" sq. 6"c-c
Steel Area 1.76 sq. in.

d' .201 in.

$P_o = \frac{1.76}{12 \times 15} = .00976$

a_s .0122 sq. ft.

$15a_s = .183$ sq. ft.

By graphical method the following values are found

Div.	L to make S/I Constant ft.
1	9.41
2	5.40
3	3.55
4	2.85
5	2.60
6	2.21
7	2.10
8	2.00
9	1.85
10	1.80

I/S = .1094

S/I = 9.14

Cont.

South Arch
Determination of moments and thrusts at the Crown Section
Unit Load at the Columns

Pt.	Unit Load @ L_4			Unit Load @ L_5		
	m	mx	my	m	mx	my
1	17.6	484.00	146.08	22.9	629.75	190.07
2	11.1	233.1	52.2	16.4	344.4	77.08
3	6.9	115.92	18.56	12.2	204.96	35.38
4	3.7	50.32	7.03	9.0	1[illegible]2.4	17.10
5	1.2	13.32	1.56	6.5	72.15	8.45
6				2.1	18.27	1.68
7				1.9	12.235	.95
8						
9						
10						
Sum	40.5	896.6	225.4	71.0	1404.28	330.71

South Arch

Determination of moments and thrusts at the crown

Unit Load at the Columns

Pt.	X	Y	X^2	Y^2	Unit Load @ L_1 m	mx	my	Unit Load @ L_2 m	mx	my	Unit Load @ L_3 m	mx	my
L_1	26.0												
L_2	20.6												
L_3	15.2												
L_4	9.9												
L_5	4.6												
1	27.5	8.3	756.3	68.89	1.5	41.25	12.45	6.9	189.7	57.3	12.3	338.3	102.1
2	21.0	4.7	441.0	22.09				.4	8.4	1.88	5.8	121.8	27.3
3	16.8	2.9	282.2	8.41							1.6	26.88	4.64
4	13.6	1.9	185.0	3.61									
5	11.1	1.3	123.2	1.69									
6	8.7	.8	75.7	.64									
7	6.5	.5	42.3	.25									
8	4.5	.2	20.2	.04									
9	2.6	.1	6.8	.01									
10	.8	.0	.6	.00									
Sums		20.7	1933.3	105.63	1.5	41.25	12.45	7.3	198.2	59.2	19.7	486.9	133.99

No.I Loading

Point	Fiber	L_1 Deadload=5,160# Comp.	L_1 Deadload=5,160# Tension	L_1 Live L.= 8420# Comp.	L_1 Live L.= 8420# Tension	L_2 Deadload=3,550# Comp.	L_2 Deadload=3,550# Tension	L_2 Live L.= 7070# Comp.	L_2 Live L.= 7070# Tension
Cr.	Upper		2(.226)				2(1.01)		
	Lower	2(.371)				2(1.69)			
Spg.	Upper		8.44	1.50			10.75	.73	
	Lower	9.78			.37	12.5			.06
1	Upper		1.41	1.17			6.26	1.56	
	Lower	2.61			.19	7.86			1.06
				.57				.57	
2	Upper			1.70				5.85	
	Lower			.13	.75				.38
									4.44
3	Upper	.21		1.34			.59	5.28	
	Lower	.33			.55	.61		-	4.03
4	Upper		[illegible]	.79			1.22	3.90	
	Lower	.41			.13	1.33			2.72
5	Upper		.14	.49			1.78	2.73	
	Lower	[illegible]		.09		2.01			1.65
6	Upper		.25	.30			1.69	1.71	
	Lower	.46		.17		2.02			.71
7	Upper		.31	.14			1.77	.91	
	Lower	.44		.09		2.17			.01
8	Upper	.06	.36			.21	1.61		
	Lower	.25				.61			
		.40	.07			2.10	1.32		
9	Upper		.28				.39		
	Lower	.30				1.17			
		.32	.25			1.39	1.11		
10	Upper		.19				.85		
	Lower	.75				1.53			
		.35				1.77			

No. I Loading

Point	Fiber	L_3 Dead Load=3090# Comp.	L_3 Dead Load=3090# Tension	L_3 Live L.= 6570# Comp.	L_3 Live L.= 6570# Tension	L_4 Dead Load= 2725# Comp.	L_4 Dead Load= 2725# Tension	L_4 Live L.= 6245# Comp.	L_4 Live L.= 6245# Tension
Cr.	Upper		2(1.32)				2(.111)		
	Lower	2(2.77)				2(2.23)			
Spg.	Upper		8.10	7.55			3.44	9.85	
	Lower	10.25			7.50	6.04			9.15
1	Upper		6.43	.35			3.43	5.76	
	Lower	8.58			.14	7.02			4.85
2	Upper	.53		1.25			2.64		
	Lower	1.47			.71	1.26 5.20			
3	Upper		1.16	7.55		.98	2.77		
	Lower	1.91			5.64	1.50 4.27			
4	Upper		1.24	9.46			4.05	5.85	
	Lower	2.12			7.65	5.66			3.36
5	Upper		2.74	6.88			4.55	9.95	
	Lower	3.74			5.14	6.28			7.55
6	Upper		3.11	4.84			4.70	10.21	
	Lower	4.19			3.12	6.50			7.86
7	Upper		3.20	3.12			3.98	7.38	
	Lower	4.37			1.48	5.90			5.08
8	Upper		2.64	1.49 .08			3.28	4.85	
	Lower	3.90				5.17			2.60
9	Upper	.10	2.10				2.15	2.54	
	Lower	1.38 3.43				4.19			.35
10	Upper		1.53 1.00			.54	.71		
	Lower	2.43 2.92				1.57 2.80			

No. 1 Loading

L_5

Point	Fiber	Dead Load=2730# Comp.	Tension	Live L.=6280# Comp.	Tension
Cr.	Upper			2(3.34)	
	Lower				2(.507)
				11.10	
Spg.	Upper			3.33	
	Lower				9.80
					.38
1	Upper	.39		6.31	
	Lower	2.56			4.71
			2.20		
2	Upper		2.95		
	Lower	2.14			
		5.69	3.50		
3	Upper		3.00		
	Lower	5.64			
		6.00	4.70		
4	Upper		1.25		
	Lower	5.36			
		6.99			
5	Upper	1.17	4.79		
	Lower	1.79			
		7.18			
6	Upper		4.51	3.85	
	Lower	6.94			.91
7	Upper		3.35	6.81	
	Lower	5.92			3.94
8	Upper		2.07	10.5	
	Lower	4.62			7.50
9	Upper	.11		7.31	
	Lower	2.54			4.44
				4.38	
10	Upper			2.59	1.61
	Lower			.10	

NO. 1 LOADING

From Use of Influence Lines the Following Values for NK and NK' Are Found and Tabulated.

Total Maximum Stresses
Assuming Case 1 to apply at all sections

Rise of Temperature and Rib Shortening

$= -\frac{1+.561}{1+.561} = .217$

Pt.	Stress in Upper Fib.	Stress in Lower Fib	(a) Loading		(b) Loading		(c) Loading		(d) Loading		Sections & Loadings @ which Case 1 does not apply.
Cr.	-23	+31	239	174	108	246	5	368	-126	+540	
Spg.	+34	-32	127	58	310	-121	-495	271	-302	+ 92	c & d
1	+30	-28	76	203	248	48	-384	635	-195	480	c & d
2	+19	-15	131	71	238	- 16	- 49	355	58	260	
3	+10	- 7	225	- 22	343	- 63	-103	449	- 48	408	
4	+ 4	+ 2	475	-159	497	-148	- 96	422	-74	433	
5	- 5	+11	482	-194	455	-132	- 87	423	-115	485	c & d
6	-12	+17	526	-147	460	- 48	- 86	402	-171	501	
7	-16	+23	496	-157	407	- 29	- 76	393	-175	519	
8	-20	+27	508	-184	395	- 33	- 61	329	-174	480	
9	-21	+29	330	- 38	209	123	24	344	- 97	503	
10	-24	+30	250	+ 49	116	220	22	344	-102	515	

Total Maximum Stresses
Assuming Case 1 to apply at all sections

Pt.	y	t	Po	Temp. Fall of 20 degrees Moments & Thrusts for rise of 20 degrees of opposite sign. H_c = -1,895 M_c = +3922 Hcy	M	N scaled	Rib Shortening H_c = -1065 M_c = 2200 M	N	Fall of temperature and rib shortening $f_c = \frac{NK}{bt}$ $f_d = \frac{NK'}{bt}$ M	N	X/t	K	K'	Stress in Upper	Lower
Cr.		1.25	.00976		+3922	-1895	+2200	-1065	+6122	-2960	-1.65	3.60	6.60	+108	-141
Spg.	11.0	2.50	.00438	-10,850	-16930	-1440	-9400	-806	-26330	-2246	+4.68	25.50	23.60	-159	+147
1	8.3	2.20	.00555	-15,700	-11780	-1580	-6600	-870	-18380	-2420	+3.43	18.60	16.70	-142	+127
2	4.7	1.86	.00656	-7900	-4930	-1700	-2900	-952	-7780	-2652	+1.60	9.00	7.10	-88	+70
3	2.9	1.57	.00779	-5500	-1570	-1775	-895	-995	-2465	-2770	+.565	3.70	2.80	-45	+34
4	1.9	1.45	.00843	-3600	+380	-1810	+213	-1015	+593	-2825	-.145	.70	1.30	-18	-9
5	1.3	1.40	.00873	-2480	+1440	-1840	+806	-1030	+2246	-2870	-.559	3.60	1.60	+23	-51
6	.8	1.35	.00905	-1500	+2420	-1860	+1355	-1040	+3775	-2900	-.96	5.50	3.60	+54	-82
7	.5	1.30	.00940	-950	+2970	-1880	+1665	-1055	+4635	-2935	-1.21	6.60	4.70	+73	-103
8	.2	1.28	.00955	-380	+3540	-1890	+1985	-1060	+5531	-2950	-1.46	7.80	5.80	+93	-124
9	.1	1.27	.00961	-189	+3721	-1895	+2090	-1060	+5811	-2955	-1.55	8.00	6.70	+100	-132
10	0	1.25	.00976	0	+3922	-1895	+2200	-1060	+6122	-2955	-1.65	8.60	6.70	+110	-141

Total Maximum Stresses
Assuming Case1 to apply at all sections

Loading (a) - No.1 Loading, fall of temp. and rib shortening
Loading (b) - No.1 Loading, rise of temp. and rib shortening
Loading (c) - No.2 Loading, fall of temp. and rib shortening
Loading (d) - No.2 Loading, riseof temp. and rib shortening

Temperature (fall)

$$H_c = \frac{-.1094(.000006)(20)(62.6)(2{,}000{,}000)144}{2(10(105.63)) - (20.7)(20.7)} = \frac{236682.1}{1245.6} = -1895$$

$$M_c = + \frac{(1895)(20.7)}{10} = +3{,}922.6$$

Rib Shortening

$$C_a \text{ (at crown)} = \frac{.0744(13580)+.35(10580)+.742(9660)+1.13(8970)+1.46(9010)}{1(1.25)+ .183} = 24{,}500$$

C

$$C_a \text{ (at point 4)} = \frac{1}{2}\left(\frac{(.35+.22)(13580)+(.61+.06)(10580)+(.97+.46)(9660)+(1.285+.845)(8970)+(1.563+1.205)(9010)}{1(1.45)+ .183}\right) = 19{,}500$$

$$C_a \text{ (at Spg.)} = \frac{1}{2}\,\frac{(.702+.59)(13580)+(.89+.35)(10580)+(1.14+.005)(9660)+(1.342+.345)(8970)+(1.513)+.68)(9010)}{1(2.5) + .183} = 14{,}250$$

$$H_c = -\frac{.1094(19413)(62.6)10}{1245.6} = -1065$$

Ca avg.
Ca avg. = 19,413#/sq.ft.

$$\begin{array}{r} 24{,}500 \\ 19{,}500 \\ 14{,}250 \\ \hline 3\,)\,58250 \\ \hline 19413 \end{array}$$

$$\text{Ratio of } \frac{H_c \text{ for Rib Shortening}}{H_c \text{ for Temperature}} = +\frac{1065}{1895} = .561$$

Maximum Stresses

Due to Loading

Point	No.1 Loading		No.2 Loading	
	Upper	Lower	Upper	Lower
Cr.	131#	215#	-103	509
Spg.	286	-89	-336	124
1	218	76	-225	508
2	219	-1	39	285
3	313	-56	-58	415
4	493	-150	-78	431
5	459	-143	-110	474
6	472	-65	-159	484
7	423	-52	-159	496
8	415	-60	-154	453
9	230	94	-76	476
10	140	190	-88	485

Central Arch Of Viaduct

The stresses in this arch may be considered porportional to the North Arch and to the South Arch using values for the thickness of the ring in porportion to the other two.

To show that this is feasible the arch relations will be given

Span Lengths

South Arch	61 feet		
		Diff.	15 feet
Central Arch	76 feet		
		"	14 feet
North Arch	90 feet		

Thickness of Arch Rings

South Arch	Cr.	15 in.	Spg.	2.5 ft.
Central Arch	Cr.	17.5"	Spg.	3.095 ft.
North Arch	Cr.	20 in.	Spg.	3.71 ft.

Ratio of Crown to Springing

South Arch	2.0
Central Arch	2.25
North Arch	2.5

All are the same type with porportional column spacing and due to these facts it was assumed that the stresses would not exceed the values in the other arches.

Arch Axis Spans

Large Arch	Crown Thick	Thickness @ Spg.
92.7'	20"	3.71'
Central Arch		
78.15'	17½"	3.11'
Small Arch		
62.6'	15"	2.5'

Maximum Thrusts

Large North Arch	X_o	∠ with vertical
57,978#	3.35'	40 degrees
Central Arch		
51,692#	2.73'	43 degrees
South Arch		
45,404#	2.1'	46 degrees

Design of South Pier

Division for Constant $\frac{S}{I}$

Div.	L to make $\frac{S}{I}$ constant
1	2'
2	3'
3	3.4
4	3.9
5	4.4
6	5.1
7	6.3
8	8.0

$$\frac{S}{I} = .604$$

Design of South Pier

Summation $X_R = 974.6$ Summation $M_R = 3,146,180$ $C_R = 6.01$ $M_R = 11$

" $Y_R = 301.1$ " $M_R X_R = 1,361,870,000$

" $X_R Y_R = 11738$ " $M_R Y_R = 264,580,000$

" $X_R^2 = 46229$

" $Y_R^2 = 4170$

Summation $X_L = 724.6$ Summation $M_L = 1,087,530$ $C_L = 9.14$ $M_L = 10$

" $Y_L = 209.1$ " $M_L X_L = 366,360,000$

" $X_L Y_L = 6165$ " $M_L Y_L = 67,811,000$

" $X_L^2 = 27,325$

" $Y_L^2 = 2216$

Summation $Y_P = 111.9$ $C_P = 6.04$ $M_P = 8$

" $Y_P^2 = 2373$

Design of South Pier

Six Simultaneous Equations

(1) $1911M_1 - 20254H_1 + 56348V_1 - 619,792,000 = 1809M_2 + 25061H_2 - 70545V_2 + 159,012$

(2) $724.6M_1 - 6165H_1 + 27,325V_1 - 366,360,000 = 0$

(3) $974.6M_2 - 11,738H_2 + 46229V_2 - 1361,870,000 = 0$

(4) $91.4M_1 - 1911H_1 + 6,622V_1 - 9,940,024 = 66.1M_2 - 1809H_2 + 5,857V_2 - 18,908,541$

(5) $675.8(M_1 - M_2) + 14,332(H_1 - H_2) = 1911M_1 - 20254H_1 + 56,348V_1 - 619,729,000$

(6) $483.2(M_1 - M_2) + 675.8(H_1 - H_2) = 91.4M_1 - 1911H_1 + 6,622V_1 - 9,940,024$

These equations were solved by the method of least squares.

Design of South Pier

Values of Unknowns by Means of Least Squares

$M_1 = 5{,}580'\#$ $\qquad$ $M_2 = 64{,}400'\#$

$H_1 = 3{,}900\#$ $\qquad$ $H_2 = 21{,}500\#$

$V_1 = 1240\#$ $\qquad$ $V_2 = 20{,}370$

$X_o = \frac{5580}{1240} = 4.5'$ $\qquad$ $X_o = \frac{64{,}400}{20{,}370} = 3.15'$

Design of South Pier

$$P = \frac{92,000}{18}\left(1 \pm \frac{3.4 \times 6}{18}\right) = 5,100(1 \pm 1.13)$$

P = 11,000# per sq.ft. compression on Left

P' = 660# Tension on Right

Steel in Stem Wall

M = 17,400 x 31 x12 = 6,352,800 in.lbs.

d = 7' = 84" - 3" for covering = 81" to steel

p = .0077

$$bd^2 = \frac{6,352,800}{107.4} = 59,000$$

$12d^2 = 59,000$ $\quad d^2 = 4,920$

$d = 70"$

$A_s = pbd$

.0077 x 12 x 70 = 6.4 sq.in. steel

Design of North Pier

Division for Constant $\frac{S}{I}$

Div.	L to make $\frac{S}{I}$ constant
1	4
2	4
3	4
4	4
5	4
6	4
7	4

$$\frac{S}{I} = \frac{4}{32.6} = .123$$

Design of North Pier

Summation	$X_R = 1057$	Summation	$M_R = 4,350,800$	$C_R = 3.7$	$N_R = 11$
"	$Y_R = 396$	"	$M_R X_R = 2,014,593,000$		
"	$X_R Y_R = 18173$	"	$M_R Y_R = 559,946,000$		
"	$X_R^2 = 57048$				
"	$Y_R^2 = 7152$				

Summation	$X_L = 974.6$	Summation	$M_L = 1,706,110$	$C_L = 6.01$	$N_L = 11$
"	$Y_L = 301.1$	"	$M_L X_L = 680,935,000$		
"	$X_L Y_L = 11,738$	"	$M_R Y_R = 132,290,000$		
"	$X_L^2 = 46,229$				
"	$Y_L^2 = 4,170$				

Summation	$Y_P = 98$	$C_P = .123$	$N_P = 7$
"	$Y_P^2 = 1821$		

Design of North Pier

Design of North Pier

Six Simultaneous Equations

Loading.

(1) $6.01(301.\ M_1 - 4170\ H_1 + 11738V_1 - 132{,}290{,}000) = -3.7(396M_2 - 7152H_2 + 18{,}173V_2 - 559{,}946{,}000)$

(2) $975\ M_1 - 11{,}738H_1 + 46{,}229V_1 - 680{,}935{,}000 = 0$

(3) $1057\ M_2 - 18{,}173H_2 + 57{,}048V_2 - 2{,}014{,}593{,}000 = 0$

(4) $6.01(11\ M_1 - 301H_1 + 975V_1 - 1{,}706{,}110 = -3.7(11M_2 - 396H_2 + 1057V_2 - 4{,}350{,}800)$

(5) $.123(98M_1 - 98M_2 + 1821H_1 - 1821H_2) = 6.01(301M_1 - 4170H_1 + 11{,}738V_1 - 132{,}290{,}000)$

(6) $.123(7M_1 - 7M_2 + 98H_1 - 98H_2) = 6.01(11M_1 - 301H_1 + 975V_1 - 1{,}706{,}110)$

Design of North Pier

Values of Unknowns

$M_1 = -12,500'\#$	$M_2 = 72,800'\#$
$H_1 = 161\#$	$H_2 = 4,800\#$
$V_1 = 11,200\#$	$V_2 = 33,200\#$
$X_1' = 1.1$ ft. to left	$X_2 = 2.19$ ft. to left

North Pier

C.G. of Section

$28 \times 7 \times 150 = 29,400\#$

$12 \times \frac{2}{2} \times 150 = 1,800\#$

$19 \times 5 \times 150 = 14,250\#$

$$\frac{29,400 \times 12 + 1800 \times 7.8 + 14,250 \times 9.5}{45,450} = 11'$$

$$p = \frac{100,450 \times \frac{7}{12.5}}{18} \left(1 \pm \frac{6 \times 5.3}{18}\right)$$

$= 3120(1 \pm 1.76)$

$= 8,600\#$ per sq. ft.

$p_2 = -2,350\#$

At Base of Stem Wall

$$p = 48,200\left(1 \pm \frac{6 \times 4.8}{9}\right)$$

$p_1 = 153\#$ per sq. in. compression

$$v = \frac{72000 \times 7}{5 \times 12 \times 12 \times 12.5} = 56\#$$

A_s = square root of $\frac{48200 \times 4.8 \times 12}{12.12}$ .0077 x 12 = 1.25 sq.in.

Use $\frac{7}{8}$" ϕ @ 12" and $\frac{7}{8}$" ϕ between for

A_s moment varies as cube of dist. below top

@ 14' the reinforcement necessary = $\frac{1}{2}$ that at

the base. So carry for 14' up.

Viaduct Section

This viaduct section consists of beams and girders of reinforced concrete with the exception of 3 steel spans over the Grand Trunk Railway.

The lengths of these spans and the angle of the bents with the roadway are shown on the layout diagram.

Sample computations are given and the other beams have been worked out in a similar manner, so it is unnecessary to show all of the computations.

These computations have been checked twice so they are assumed to be the correct results.

The depth of the girders are limited because of clearance over the Grand Trunk tracks as shown.

Slab Design

$d_c = 8\frac{1}{4}$ in.

$d_s = 7$ in.

Use $\frac{1}{4}$" sq.bars 4" c.

This design governs all parts as all other spans are shorter, and in order to keep a uniform slab $8\frac{1}{4}$ inches will be used.

Span 37'-0" (for intermediate spans)

D.L. slab $= 7 \times 150 = 1050\#$
beam $= 2.21 \times 2 \times 150 = 663$
$= \frac{1}{3} \times \frac{1}{3} \times 150 = \underline{17}$
$1730\#$

L.L. $= \frac{7}{38}$ of 4 Trucks $= .737$ Of One Truck

$.737 \times 28{,}000 \times \frac{5}{4} = 25{,}000\#$

$.737 \times 20{,}000 \times \frac{5}{4} = 18{,}000\#$

M Truck $= (30{,}500 \times 6.5 + 12{,}500 \times 12).8 = \quad 278{,}000\#$

M D.L. $= \frac{wl^2}{10} = 1703 \times \overline{37}^2 = \quad \underline{233{,}000\#}$
$511{,}000\#$

Shear

$R_1 = 45{,}130$
D.L. $= 29{,}000\#$ $\quad b = 6'$
$V = 74{,}130\#$ $\quad$ Use $d_s = 29"$
$U = \frac{74{,}130}{24 \times \frac{7}{8} \times 29} = 121\#$ ---Use stirrups

$Z_o = \frac{74{,}130}{100 \times \frac{7}{8} \times 29} = 29.2"$

$K = \frac{1}{1 + \frac{15{,}000}{15.650}} = .378$

$A_s = \frac{M}{f_s\,(d_s - \frac{1}{2}t)} = \frac{511{,}000 \times 12}{16{,}000(29-4.25)} = 15.5$ sq.in.

Use 12-1$\frac{1}{8}$" sq.br = 15.18 sq.in.

$K = .36 \qquad j = .892 \qquad P = .00622$

$f_s = \frac{511{,}000 \times 12}{15.18 \times .892 \times 29} = 15{,}650\# \qquad \frac{t}{d} = .293$

$f_c = \frac{15{,}650 \times .36}{15(1 - .36)} = 586\#$

Investigation at Support

$\frac{d'}{d} = \frac{4}{29} = .138$ $\quad$ $p' = p = \frac{15.18}{24 \times 29} = .0218 = p$

$k = .428 \quad j = .855$

$$f_s = \frac{511,000 \times 12}{15.18 \times .855 \times 29} = 16,250$$

$$f_c = \frac{16,250 \times .428}{15(1 - .428)} = 809\#$$

$$f_s' = \frac{16,250(.428 - .138)}{.572} = 8,240\#$$

12 bars in Top at Support

12 bars in Bottom at Support

Span 37'-0"

Stirrups

Use $\frac{41}{2}$ " ϕ = .392

Concrete will stand V_c = vbjd = 40x24x875x29 = 24,400

Sp. at support = $\frac{A_v f_s j d}{V}$

$$= \frac{4 \times .1963 \times 16000 \times .875 \times 29}{74,130 - 24,400} = 6.32"$$

Max stirrup spacing = .45d = 13.1"

$$S @ 7' = \frac{4 \times .1963 \times 16000 \times 29}{49,730 - 7 \times 1730 - 4200} = 9.4"$$

$$S @ 10' = \frac{313,000}{49,730 - 10 \times 1730 - 6,500} = 12.1"$$

Max bent up bar spacing

$$\frac{45 \times 31}{45 + 10} = 25.4'$$

Spa @ 7" for 3'

9" for 3'

13" to center

37' Span Outside

b_f = 10% 37' = 44"

d_o = 46"

$$\begin{aligned}
\text{D.L. slab} &= 4.75 \times .75 \times 150 = 535\# \\
\text{r} &= \tfrac{1}{2} \times \tfrac{1}{4} \times 150 = 19 \\
\text{beam} &= 2 \times 3.833 \times 150 = 1150 \\
&\quad 1 \times 2.12 \times 150 = 318 \\
\text{slab} &= 1.25 \times 1.25 \times 150 = 234 \\
\text{rail} &= \underline{550} \\
& \qquad 2{,}806\#
\end{aligned}$$

R_1 = 21,250#

$$M\ \text{Truck} = (21{,}250 \times 6\tfrac{1}{2} + 8{,}750 \times 12).8 = 195{,}000 \text{ ft.lbs.}$$

$$M\ \text{D.L.} = \frac{w\,l^2}{10} = \frac{2806 \times 37^2}{10} = 384{,}000 \text{ ft.lbs.}$$

$$\text{Mull. walk } 100\#/\text{ft.} = \frac{6 \times 100 \times 37^2}{10} = 82{,}000 \text{ ft.lbs.}$$

$$\begin{aligned}
&579{,}000 \text{ ft.lbs.} \\
&\underline{\times 12} \\
&6{,}948{,}000''\#
\end{aligned}$$

$$R_1 = 17500 + 12500 \times \frac{22}{34} + 12500 \times \frac{10}{34} = 29{,}200\#$$

R_1 = 29,200#

D.L. = 2806 x 17 = <u>47,700</u>

$V_{D.L.}$ = 76,900#

T-beam - Outside 37' - Span

Investigation at the Center

$$A_s = \frac{M}{f_s(d_s - \frac{t}{2})} = \frac{6{,}948{,}000}{16{,}000(42-6)} = 12.00 \text{ sq.in.}$$

Use 12 - 1" = 12 sq.in.

$$\frac{t}{d} = \frac{12}{42} = .285 \qquad p = \frac{A_s}{b_f d} = \frac{12.00}{44 \times 42} = .0065$$

Diagram 6

$$K = .360$$

$$Kd = .36 \times 42 = 15.1"$$

$$f_s = \frac{M}{A_s jd} = \frac{6{,}948{,}000}{12 \times .89 \times 42} = 15{,}500\#$$

$$f_c = \frac{f_s K}{15(1-K)} = \frac{15{,}500 \times .36}{15(1-.36)} = 580\#$$

Concrete takes $V_c = vbjd = 40 \times 24 \times .88 \times 42 = 35{,}500$

Stirrups

$$S \text{ @ support} = \frac{A_s f_s jd}{V_s} = \frac{4 \times .1963 \times 16000 \times .88 \times 42}{76{,}900 - 35{,}500} = 11.2" \text{ or } 12"$$

Max. stirrup spacing $= .45d = .45 \times 42 = 18.9"$

Max. bent up bar spacing $= \frac{.45d}{45 + 10} = \frac{45.44}{55} = 35"$ say 36"

$$\frac{S}{8 ft.} = \frac{4 \times .1963 \times 16000 \times .88 \times 42}{41{,}400 - 8 \times 2856} = 20"$$

3' from support, Spa @ 18"

T-beam Outside

Investigation at Supports

Bending up Six Bars

$\frac{d'}{d} = \frac{4}{42} = .095$ $\qquad$ $p' = p = \frac{A_s}{bd} = \frac{6}{24.42} = .01180$

Diagram 8 $\qquad$ $K = .365$ $\qquad$ $j = .886$

$f_s = \frac{M}{A_s j d} = \frac{6,948,000}{12x.886x42} = 15,650\#$

$f_c = \frac{f_s K}{n(1-K)} = \frac{15,650 \times .365}{15(1-.365)} = 600\#$

6 bars may be bent up from each side

Lap bars @ intermediate supports so as to

provide 12 sq.in. Top

12 sq.in. Bottom

T-beam Outside
37' - Span Negative Moment at the end

$M = \frac{w\,l^2}{16}$ for dead load on fully restrained section.

M truck = $(21{,}250 \times 6\tfrac{1}{2} + 8750 \times 12).5 = 121{,}500$ ft.lbs.

$M_{D.L.} = \frac{w\,l^2}{16} = \frac{2806 \times \overline{37}^2}{16} =$ 240,000 ft.lbs.

361,500 ft.lbs.

12

4,340,000"#

$A_s = \frac{4{,}340{,}000}{16000(42-6)} = 7.53$ sq.in.

Bend up 81" - sq.br. = 8 sq.in.

$\frac{d'}{d} = \frac{4}{42} = .095$ $p' = \frac{8\ p}{12} = \frac{A_s}{bd} = \frac{8}{24 \times 42} = .00793$

Diagram 8

K = .391 j = 880

$f_s = \frac{4{,}340{,}000}{8 \times 88 \times 42} = 14{,}650\#$

$f_c = \frac{14{,}650 \times .391}{15(1-.391)} = 625\#$

$f_s' = \frac{14{,}650(.391-.095)}{.609} = 7{,}100\#$

T-beam

Span 37' - 0" Negative M @ End Supports

$M = \frac{w\,l^2}{16}$ for dead load

L.L. $= \frac{7}{38}$ of 4 Trucks = .737 of 1 Truck

$.737 \times 28000 \times \frac{5}{4} = 25{,}000\#$

$.737 \times 20000 \times \frac{5}{4} = 18{,}000\#$

$M_{Truck} = (30{,}500 \times 6\tfrac{1}{2} + 12500 \times 12).5 =$ 174,000 ft.lbs.

$M_{D.L.} = \frac{w\,l^2}{16} = \frac{1703 \times 37^2}{16}$ = $\frac{145{,}000 \text{ ft.lbs.}}{319{,}000 \text{ ft.lbs.}}$

$A_s = \frac{M}{f_s(d-\frac{1}{2}t)} = \frac{319{,}000 \times 12}{16000(29-4.25)} = 9.66$ sq.in. necessary

Bending up 8 bars = 10.12 sq.in. if 12 bars are used in bottom.

Investigation over the support

$\frac{d'}{d} = \frac{4}{29} = .138$ $p' = \frac{10.12}{15.18}$ $p' = \frac{10.12}{24 \times 29} = .0145$

$p = \frac{15.18}{24 \times 29} = .0218$

From diagrams

K = .461 j = .848

$f_s = \frac{319{,}000 \times 12}{10.12 \times .848 \times 29} = 15{,}300\#/\text{sq.in.}$

$f_c = \frac{15{,}300 \times .461}{15(1-.461)} = 870\#$ ----Is allowed because of extra concrete in knee.

$f_s' = \frac{15{,}300(.461-.138)}{.539} = 6{,}340\#$

Tabulation of Beams according to Span Lengths

Reinforced Concrete T-Beams

Inside - The interior beams

8.4' Span - Over Arch

$b_f = 25"$ $d_c = 18\frac{1}{2}"$ $d_s = 16"$

Use 4 1" ø bars -

Stirrups - Sp. 6" uniformly

20' Span

$b_f = 5' - 0"$ $d_s = 20"$ $d_c = 24"$

Use 10 1"sq.bars

Stirrups - 6" progressing to 11"

22' Span

$b_f = 5' - 0"$ $d_s = 21"$ $d_c = 25"$

Use 10 1"sq.bars

Stirrups - 6" - 11" at center

29' Span

$b_f = 6'-0"$ $d_s = 23"$ $d_c = 27"$

Use 12 1"sq.bars

Stirrups - 6" - 10"

31' Span

$b_f = 6'-0"$ $d_s = 24"$ $d_c = 28"$

Use 12 $1\frac{1}{8}"$ sq.bars

Stirrups - 6" - 10"

34' Span

$b_f = 6'-0"$ $d_s = 27"$ $d_c = 31"$

Use 12 $1\frac{1}{8}"$ sq.bars

Stirrups - 7" - 13" at center

37' Span - See Sample Computations

Outside Spans - Sidewalk Side

8.4' Span

$d_c = 32"$ $d_s = 29"$

Use 4 $\frac{3}{4}"$ round bars in Top and Bottom

Stirrup - 13"

20' Span

$d_c = 37"$ $d_s = 33"$

Use 6 1"sq.bars

Stirrup - 15" uniformly

22' Span

$d_c = 38"$ $d_s = 34"$

Use 6 1"sq.bars

Stirrup - 15" uniformly

29' Span

$d_c = 40"$ $d_s = 36"$

Use 10 1"sq.bars

Stirrup - 13.9" to 16" at Center

31' Span

$d_c = 41"$ $d_s = 37"$

Use 12 1"sq.bars

Stirrup - 12" - 16"

34' Span

$d_c = 45"$ $d_s = 41"$

Use 12 1"sq.bars

Stirrup - 12" - 18"

37' Span - See Sample Computations

Curb Spans T-Beam

20' Span

$b_f = 24"$ $d_c = 37"$ $d_s = 34\frac{1}{2}"$

Use 4 1" sq.bars

Stirrups Max. Sp. 15.5"

22' Span

$b_f = 26"$ $d_c = 38"$ $d_s = 35.5"$

Use 6 1" round bars

Stirrup Max. 16"

29' Span

$d_c = 40"$ $d_s = 37.5"$

Use 7 1" sq.bars

Stirrups $16\frac{1}{2}"$ Sp.

31' Span

$d_c = 41"$ $d_s = 38.5"$

Use 10 1" round bars

Stirrup Sp. 17.3"

37' Span - See Sample Computations

40' Steel Span Outside

D.L. slab =	4.75 x .75 x 150	= 534#
beam =	2 x 3.92 x 150	= 1175
V =		19
I-beam =		= 160
rail =		= 550
		2438#

$M_{D.L.} = \frac{w\,l^2}{8} = \frac{2438 \times 40^2}{8} =$ 488,000 ft.lbs.

Mull. $\frac{600 \times 40^2}{8} =$ 120,000 ft.lbs.

$M_{L.L.} = \frac{Pl}{4} = \frac{17,500}{4} \times 40 =$ 175,000 ft.lbs.

663,000 ft.lbs.

Use Carnegie C.B. 272 27" 160# Beam

M = 675,150 ft.lbs.

Max Shear at end section = 208,570#

Allow. Uni. Load	Fixed	Free
	135,000#	95,000#

(2438 + 600)40 = 121,000 ---Fix ends

Top Flange stayed laterally at intervals of about 15' or 17', if at all.

40' Steel Span Outside

D.L. slab = 600#
beam(c) = 488#
I beam = 130
1218#

L.L. = 90% of wheel loading
= 15,750# & 11,250#

$M_{D.L.} = \frac{w\,l^2}{8} = \frac{1218 \times 40^2}{8} = 244{,}000'\#$

$M_{truck} = 19{,}125 \times 8 + 7875 \times 12 = 247500'\#$

M total 491,500'#

Section Modulus

$S = \frac{491{,}500 \times 12}{18000} = 328$

Use Carnegie C.B. 244 24" 130# S = 334

M = 500,400'#

Shear End R = 159,180#

Allow Uni. L = Fixed 100,000# Free 70,000#

Shear

$R_1 = 15750 + 11250 \times \frac{28}{40} + 11250 \times \frac{16}{40}$

= 28,100#

D.L. = 1218 x 20 = 24,760

V = 52,860#

Suggest 24" C.B. @ 120# Providing 301.9

Steel Spans

40' Span Outside

Use Carnegie C.B. 272 27" 160#

40' Span Inside

C.B. 244 24" 130#

42' Span Outside

C.B. 272 27" 175#

42' Span Inside

C.B. 244 24" 130#

44' Span Outside

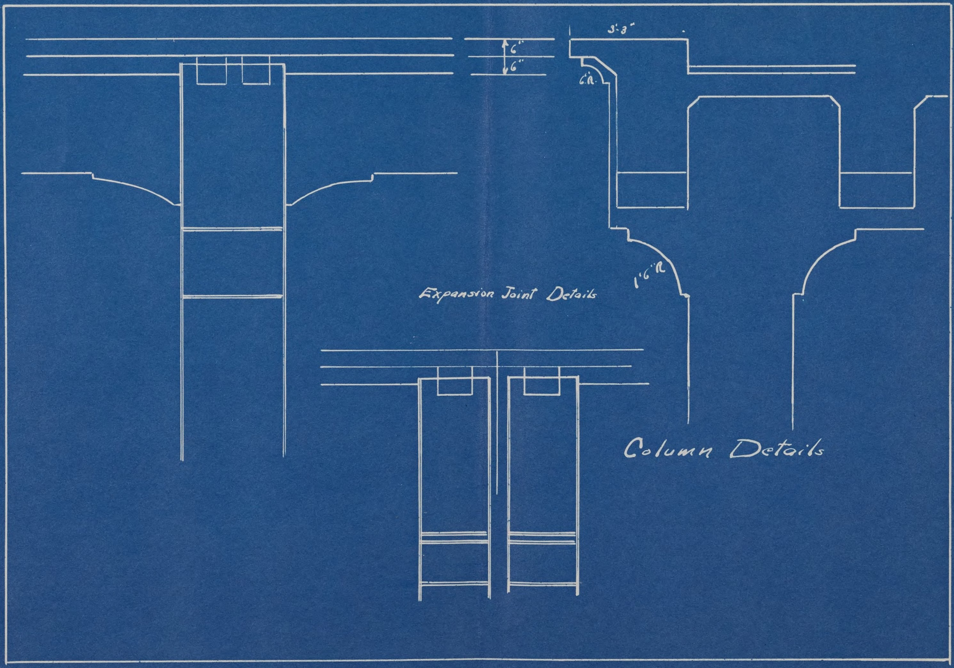
C.B. 272 27" @ 190#

44' Span Inside

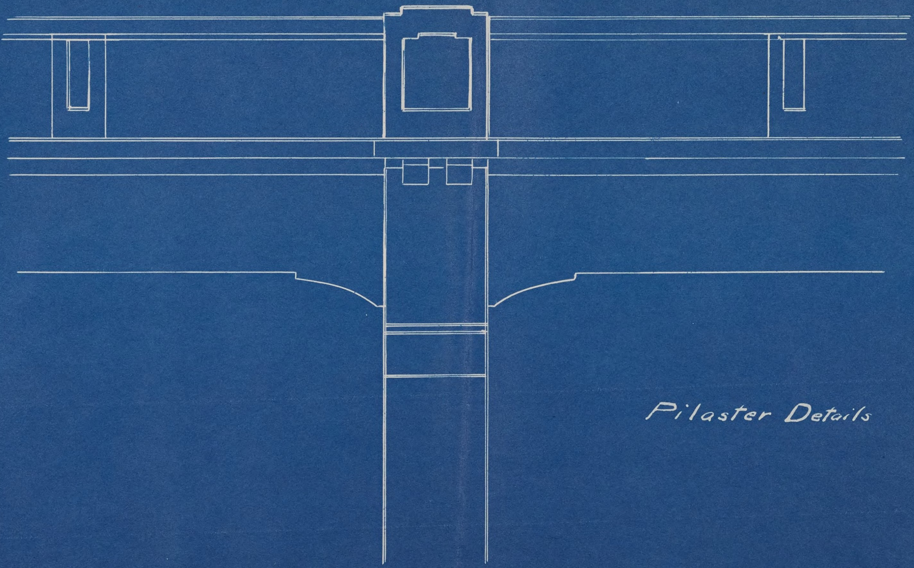
C.B. 244 24" @ 140#

[redacted]

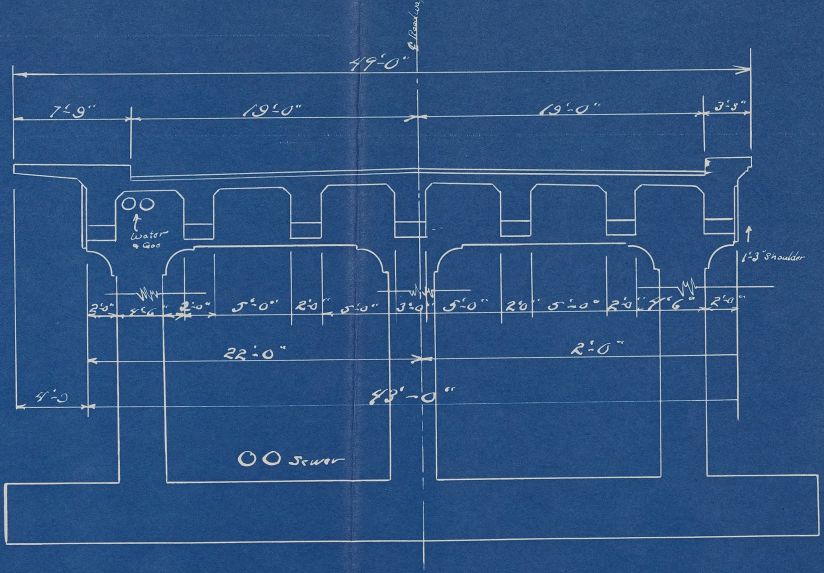
[redacted]



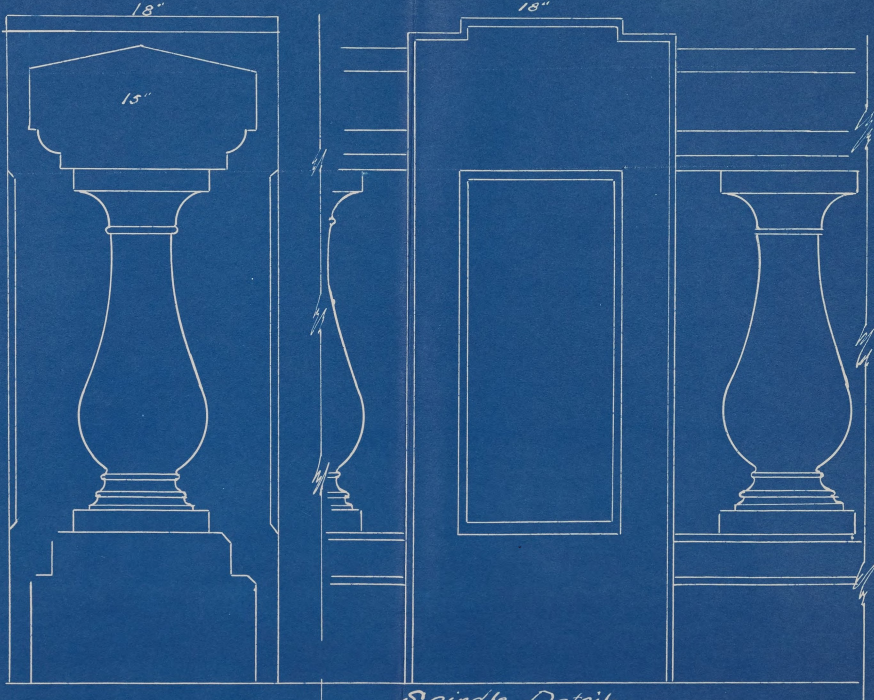
6"
6"
3'-3"
6"R.
1'6"R
Expansion Joint Details
Column Details



Pilaster Details

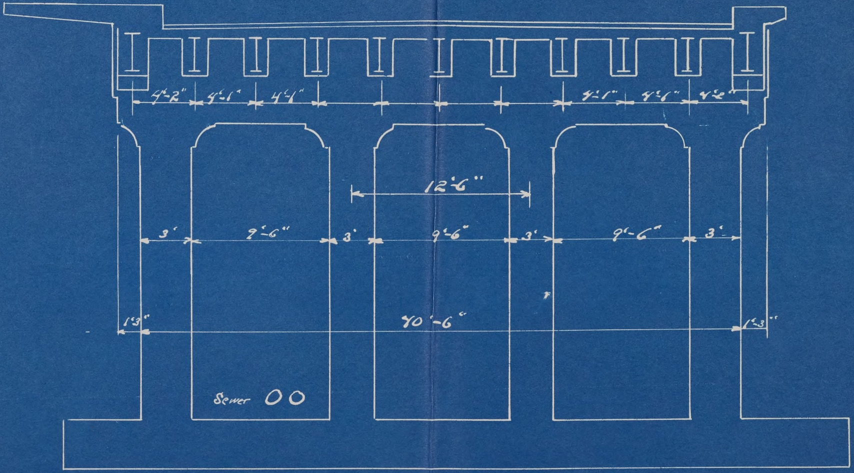


TYPICAL CROSS-SECTION
of
CONCRETE BEAMS

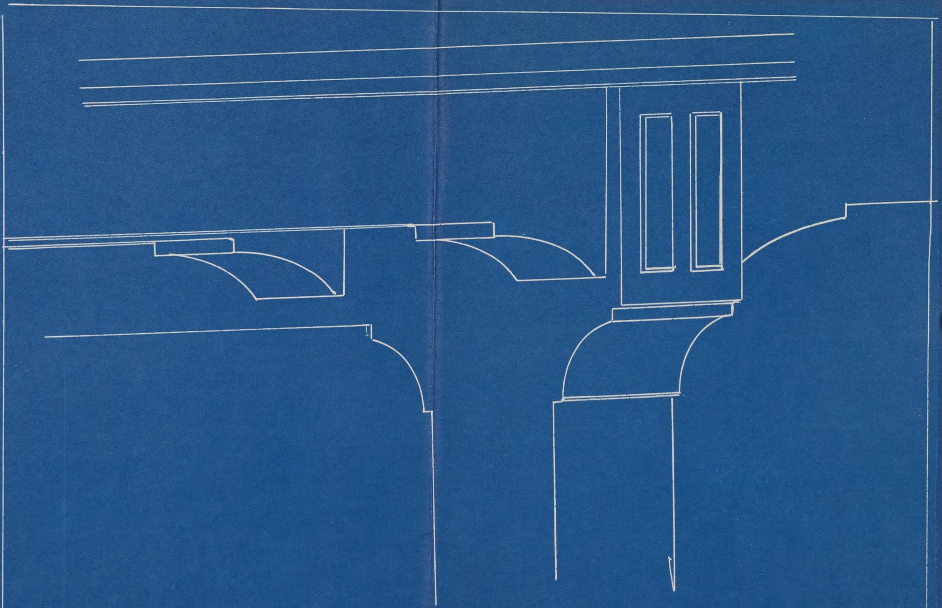


Spindle Details

LOGAN STEET VIADUCT
LANSING



TYPICAL CROSS SECTION
STEEL SPANS



Perspective View of Post - Girder & Beams
Massive effect will be noted

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