

Blair ~ An analysis of the stresses in the Mich. ave. two ringed

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**SUPPLEMENTARY
MATERIAL**
IN BACK OF BOOK

Thesis

1912

A. Blair

C. W. Parsons.



THESIS

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AN ANALYSIS OF THE STRUCKS IN THE BRITISH AVENUE
TWO HUNDRED AND EIGHTY-ONE OVER THE GREAT
RIVER AT LANSING, MICHIGAN

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JUN. - 1912

This thesis was contributed by

Mr. D. A. Blair.

under the date indicated by the department
stamp, to replace the original which was de-
stroyed in the fire of March 5, 1916.

SUPPLEMENTARY
MATERIAL
IN BACK OF BOOK

THESIS

In taking this as a subject for our thesis we hope to gain more knowledge of the theory of arches and some practical knowledge of bridge analysis and design.

This bridge was built in 1894 at a cost of \$60,000.00 by the Wrought Iron Bridge Company of Canton, Ohio, now amalgamated with the American Bridge Company. (Eng. News 11-14-95). It was a subject of interest to bridge engineers at that time because of its extreme width and unusually heavy floor system. The plans were made by Mr. E. J. Lander, M. A. S. C. E., at that time connected with the bridge company. Newland & Fraser of Lansing, built the substructure, which consists of abutments and piers of Ohio sandstone laid on a footing of Portland cement concrete deposited on piling driven into hard, blue clay.

The bridge is 115½ feet wide and has two spans each 110 feet, center to center of piers, built on six two-hinged steel arch ribs spaced 13 feet apart and having a rise of 13 feet. The floor system is made up of built-up I beams, 2 feet, 3½ inches in depth, placed 11 feet apart and supported by columns which carry the load to the arch ribs. The stringers are 12 inch I beams, placed 6 feet apart between which are masonry arches to support the pavement. These arches consist of one layer of brick upon which is laid a stone concrete filling varying in depth from 16 inches to 10 inches at the crown. Upon this concrete is laid the pavement which forms the roadway of the bridge. (See general drawings of bridge.)

The single track of the Michigan United Traction Company's line is situated in the center of the bridge over which much of the city's traffic passes as well as the heavy interurban cars of the Lansing - Oshtemo division. As half of the live wheel loads which may come upon this track is carried by each of the two middle Arch ribs, and as this wheel load is probably the heaviest live load which comes upon the bridge, we have confined our analysis to one of these middle ribs.

Not being able to obtain specifications of the M. U. T. Company's cars for the live wheel loads, two cars were taken which were standard and also compared satisfactorily with the cars crossing the bridge at the present time; one with a total loaded weight of 46,000 pounds on two axles 10 feet center to center, the other a total loaded weight of 90,000 pounds on two axles 35 feet center to center. (See Elec. Railway Journal 4-22-11, page 705). A standard uniform live load of 100 pounds per square foot was considered as covering the bridge except for a twelve foot strip occupied by the car track.

The following is an estimate of the concentrated dead load at the several columns supported by the steel arch ribs. (This was taken from the plans of the bridge obtained from the City Engineer of Lansing. See blue print of general plans in pocket.)

Dead load due to road bed per linear foot of girder:		
18 sq.ft. brick pavement (one layer wearing surface)		
	6.75 cu.ft. at 140',	946
18 " " " (one layer arches)	6.75 cu.ft. at 140'	946
Concrete (average depth 12")	17.15 cu.ft. at 150	2570

1 tie- 20 F. B. M. at 4",	90
2' - 12" channel at 20",	40
2' - 12" I beams at 30",	64
1' Nail and fastenings at 20",	20
Total per lin. ft. of arch,	<u>4676</u>

Dead panel load at each column = $4676 \times 11 = 51,436$

Estimation of weight of Floor Beams (2'x8")-4'x8"-4'x8".
(See blue print in pocket.)

Web Pl. 27" x 1/2" x 17'- 11 1/2" at 10.51' per sq.ft.,	425
2 angles 3 x 3 x 1/2" x 17'- 11 1/2" at 4.9 per ft.,	170
2 " 3 x 3 x 1/2" x 16'- 6 1/2" " 4.9 " " ,	164
234 rivet heads at 24.29 per 100,	54
Total,	<u>819</u>

Estimate of weight of column supporting Floor Beams, 2'x8".

1 Pl. 14" x 1/2" x 5'- 11 1/2" at 10.51' per sq. ft.,	73
2 angles 4 x 3 x 5/16 x 6'- 11 3/4" at 7.2 per ft.,	131
2 " 4 x 3 x 5/16 x 7'- 11" at 7.2 per ft.,	103
136 rivet heads at 24.29 per 100,	33
2 Pl. 7 1/2" x 1/2" x 2'- 7 3/4" at 10.51 ,	30
2 angles & fillers,	25
Total,	<u>375</u>

Estimated weight of columns supporting Floor Beams 2'x8".

2 Pl. 7 1/2" x 1/2" x 3'- 4 1/8" at 10.51' per sq. ft.,	44
1 " 14 x 1/2" x 15 5/8" at 10.51 per sq. ft.,	16
2 angles 4 x 3 x 5/16 x 5'- 1 1/8" at 7.2 per ft.,	40
2 " 4 x 3 x 5/16 x 5'- 5 1/2" at 7.2 per ft.,	30
90 rivet heads at 24.29 per 100,	22
2 angles & 2 fillers,	25
Total,	<u>207</u>

Estimated weight of Floor Beam C₍₁₋₂₎ = C₍₁₋₂₎. (No lost).

1 web pl. 27" x 1/2" x 13'- 2 1/2" at 10.51 per sq. ft.,	313
2 angles 3 x 3 x 1/2 x 17'- 11 1/2" at 4.9 per ft.,	176
2 " 3 x 3 x 1/2 x 17'- 2 1/2" at 4.9 per ft.,	100
4 fl. 6" x 1/2" x 21 1/2" at 10.51 per sq. ft.,	33
4 angles 3 x 3 x 1/2 x 2'- 3" at 4.9 per ft.,	44
6 Filler fl. & Spl. fl.,	30
2 Plates, (brackets),	170
392 rivet heads at 24.20 per 100,	949
Total,	<u>1036</u>

Estimated weight of Floor Beam D₍₁₋₂₎ = C₍₁₋₂₎. (No lost.)

1 web pl. 27" x 1/2" x 17'- 9 1/2" at 10.51 per sq. ft.,	420
2 angles 3 x 3 x 1/2 x 17'- 3" at 4.9 per ft.,	100
2 " 3 x 3 x 1/2 x 17'- 3" at " " " ,	100
216 rivet heads at 24.20 per 100,	522
Total,	<u>612</u>

Posts supporting A' & A". Estimated weight.

fl. 14" x 1/2" x 2'- 11 1/2" at 10.51 per sq. ft.,	160
2 fl. 7 1/2" x 1/2" x 2'- 7 1/2" at 10.51 per sq. ft.,	75
2 angles 4 x 3 x 5/16 x 5'- 1 1/2" at 7.2 per ft.,	74
2 " 4 x 3 x 5/16 x 4'- 11 1/2" at 7.2 per ft.,	70
100 rivet heads at 24.20 per 100,	2420
Plates, angles and fillers,	
Total,	<u>2580</u>

Weight of one panel of Arch Rib C₁.

2 top angles 6" x 6" at 10.6, 11'- 7" long,	480
2 bot. " 6 x 6 at 10.6, 11' - 7" long,	480
Web pl. 48 1/2" x 3/8" x 11'- 7" at 10.75 per sq. ft.,	730
6 angles 3 1/2" x 3" x 5/8" x 3'- 11 1/2" at 7.2 per ft.,	180
6 fillers 3 1/2" x 1/2" x 3' - 0" at 20.4 per sq. ft.,	187

442 rivet heads at 24.20 per 100, 107'
 4 cover pl. 14" x 3/8" x 11'- 7" at 17.95 per sq. ft., 207
 Total, 2241

Dead Load Concentrations.

At A.

Road Bed, 51430'
 Floor Beams, 810
 Column, 208
 Arch Rib, 2241
 Total, 54764'

At B.

Road Bed, 51430'
 Floor Beam, 1036
 Arch Rib, 2241
 Total, 54713'

At C, B, E, F.

Road Bed 51430'
 Floor Beam, 810
 Arch Rib, 2241
 Total, 54487'

At G.

Road Bed, 51430'
 Floor Beam, 1036
 Arch Rib, 2241
 Total, 54713'

At H.

Road Bed, 51430'
 Floor Beam, 810
 Column, 208
 Arch Rib, 2241
 Total, 54701'

As the concentrated dead load at the columns varied only 0.8%, a uniform dead panel load of 54,600 pounds was adopted in the calculations which follow.

To obtain the stresses in the arch girder due to the above loading, Howo's analytical theory of the two hinged arch was used. (See "A Treatise on Arches" by H. A. Howo.) Twenty radial sections of the Arch Rib were selected and the value of the arch properties used in the formulae for H_L (see plate 1.) at each section are tabulated in table A of plate No. 2. In tables B (plate 2.) and C and D (plate 3.) these properties are combined so as to form the terms of the formulae for H_L . The values of H_L due to a load of 1000# at each panel point are also given in table E. These values are given both when thrust is considered and when it is neglected for the purpose of comparison.

From the values of H_L (including thrust) and the corresponding values of V_L which are found as we would find the reactions for a simple beam of span L, the diagram of Thrust, Radial Shear and Bending Moments (plate No. 4) was drawn, from which the total thrust, shears and moments due to a 1000# load concentrated at the different posts, i. e., floor beams, were scaled. These values were tabulated in table E, (plate No. 3). From these results were plotted the influence lines for Thrust (plate No. 5), for Shear (plate No. 6) and for Moments (plate No. 7). The columns ^{are located} on a line shown as the abscissa and the values of the Thrusts, Shears and Moments due to 1000# at any column are plotted as the ordinates over that column. By using a slider in connection with these influence lines, on which is marked the wheel

spacing to the same scale as the horizontal scale of the influence lines, the positions of loading to give the Maximum Moments, Thrusts and Shears at any section were found. By multiplying the stress due to a 1000' load at any section by the actual load at that section in thousands of pounds, the actual Maximum Moments, Shears and Thrusts were obtained and are tabulated in table D (plate No. 5).

From table D it may easily be seen what live load combined with the given dead load gives the Maximum Thrust, Shear or Moment at any section. The uniform live panel load as well as the dead load was considered as a concentrated load at the column.

Thus it was found that the large interurban car gave the largest stress in every case and diagrams showing the positions of the live load on the bridge for maximum stresses in each section are shown in table C, (plate No. 9).

--- Conclusion. ---

Upon referring to table C, (plate No. 9) it can be seen that the shear at any section is very small, the maximum being 1210' per sq. in. at section 2, giving an ultimate factor of safety of about $50,000/1210 = 41.3$ which is a great deal on the safe side. Then referring to the stress due to moments and thrusts, it is seen that the elastic limit, about 30,000' per sq. in., is very nearly attained. In using the stress in section 8 of 24,140' per sq. in., it gives an approximate ultimate factor of safety of $60,000/24,140 = 2.48$ which is entirely too small, as it should be to agree with general

practice, about four.

By comparing the moments in table F, it can be seen that the heavy floor system causes a very large part of this stress, and thus indicates that the lightest floor system consistent with strength should be used in designing such a structure. Also from the results above it would seem that the bridge was not designed for such a heavy car loading.

With a double track on the bridge as proposed for a not far distant future, the dead load would remain practically the same, the increase due to additional rails and ties being very small compared to the other dead loads and being practically offset by the road material displaced. Considering two inter-urban cars on the bridge, for the case giving the maximum stress for one car, we would have, -

Dead load moment=	346,835.0'
Mom. for 35 ft. car + 40% impact =	505,949.4
Uniform live load moment + 40% impact =	<u>143,744.4</u>
Total,	997,529.4'

$$S = \frac{997,529.4 \times 2.0000}{1.143 \times 144} = 12,502' \text{ per sq. in.}$$

Dead load thrust =	550,256.0'
Uniform live load thrust + 40% impact =	53,592.0
35' car Thrust + 40% impact =	<u>121,007.0</u>
Total,	725,057.0'

$$S = \frac{725,057.0}{56.18} = 13,150' \text{ per sq. in. due to Thrust.}$$

Stress in top flange = 12502 + 13150 = 25,652' per sq. in.
compression.
Stress in bottom flange = 13150 - 12502 = 648 per sq. in.
compression.

The above figures show that if the single track which is now on the bridge were replaced by a double track, it would be possible to load the bridge so as to strain the structure almost to its elastic limit. It is especially important to note here that the material in the structure is thin, and deterioration due to rust will considerably increase the units found above. Also the modern tendency is towards heavier all-steel cars and it would not seem advisable to place an extra track on this structure.

Theoretical Analysis Of The Two Hinged Arch.

Fundamental Formulae of Elastic Arch:

In a fixed arch, symmetrical, the following conditions are assumed:

1. The Radial Lines at the Springing remain unchanged in direction. Thus

summing up $d\phi$ for all sections between Springing Points,

$$\sum_0^L d\phi = \sum_0^L \frac{M \Delta s}{E I} = 0. \quad [\text{see figure below}]$$

2. The supports are fixed an immovable distance apart i.e. are rigid.

$$\text{Thus } \sum_0^L dx = \sum_0^L \delta x - \sum_0^L \frac{T \Delta x}{A E} + \sum_0^L \frac{M \cdot y \Delta s}{E I} = 0. \text{ since } \sum_0^L \frac{y \cdot M \Delta s}{E I}$$

$$\text{becomes } 0, \sum_0^L \frac{M \Delta s}{E I} = 0.$$

3. The supports are relatively immovable in Elevation.

$$\text{Then } \sum_0^L dy = \sum_0^L \delta y - \sum_0^L \frac{T \Delta y}{A E} - \sum_0^L \frac{M \cdot x \Delta s}{E I} = 0, \text{ since the term } \sum_0^L \frac{M \Delta s}{E I}$$

$$\text{becomes } 0, \sum_0^L \frac{M \Delta s}{E I} = 0 \text{ and since } \sum_0^L \frac{M \Delta s}{E I} = 0$$

In the Two Hinged Arch, a small unequal settlement δy will not cause any stress in the Arch since it is free to turn about each hinge. Also the total change in ϕ or $\delta\phi$, cannot be known, but it certainly is not equal to 0, since the Arch is free to turn on its hinges. This leaves only the 2nd Equation available for determining stresses. Thus $\sum_0^L d\phi = \sum_0^L \delta\phi - \sum_0^L \frac{T \Delta x}{A E} + \sum_0^L \frac{M \cdot y \Delta s}{E I} = 0$.

From figure below, $T = H_L \cos \phi + V_L \sin \phi + W_L \cos \phi$, $M = V_L x - H_L y - W_L(x-a) - W_L(y-b)$
 But by Moments about right hinge, $V_L = W_L \frac{L-a}{L} - H_L \frac{b}{L}$. Substituting this value of V_L in the above

values of T & M , they become $T = H_L \cos \phi + W_L \left[\frac{L-a}{L} \sin \phi - \sin \phi \right] - W_L \left[\frac{b}{L} \sin \phi - \cos \phi \right]$

$$M = -H_L y + W_L \left[\frac{L-a}{L} x - (x-a) \right] - W_L \left[\frac{b}{L} x + (y-b) \right]$$

$$\text{Substituting these values, the above equation becomes, } H_L \left[\sum_0^L \frac{y^2 \Delta s}{I} + \sum_0^L \frac{\Delta x \cos \phi}{A} \right] = E \left[T \cdot L + W_L \left[\frac{L-a}{L} \sum_0^L \frac{x \cdot y \Delta s}{I} - \sum_0^L \frac{(x-a) y \Delta s}{I} - \frac{b}{L} \sum_0^L \frac{\Delta x \sin \phi}{A} + \sum_0^L \frac{\Delta x \sin \phi}{A} \right] - W_L \left[\frac{b}{L} \sum_0^L \frac{x \cdot y \Delta s}{I} + \sum_0^L \frac{(y-b) y \Delta s}{I} - \frac{b}{L} \sum_0^L \frac{\Delta x \sin \phi}{A} + \sum_0^L \frac{\Delta x \cos \phi}{A} \right] \right]$$

$$\text{For Vertical Loads Only } H_L = V_L \left[\frac{\sum_0^L \frac{x \cdot y \Delta s}{I} - \frac{1}{L} \frac{\sum_0^L (x-a) y \Delta s}{I}}{\sum_0^L \frac{y^2 \Delta s}{I}} \right] \text{ Neglecting Thrust}$$

$$\text{For Horizontal Loads Only } H_L = V_L \left[\frac{\sum_0^L \frac{x \cdot y \Delta s}{I} + \frac{b}{L} \frac{\sum_0^L (y-b) y \Delta s}{I}}{\sum_0^L \frac{y^2 \Delta s}{I}} \right] \text{ Neglecting Thrust.}$$

$$\text{For Temperature Only } H_L = \frac{E \cdot \alpha \cdot L}{\sum_0^L \frac{y^2 \Delta s}{I}} \text{ Neglecting Thrust.}$$

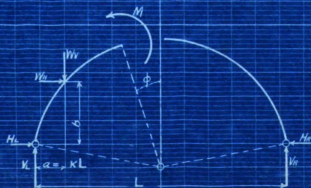


PLATE I.

TABLE

Point	x	K	y	ϕ	$\sin \phi$
1	+5.5		+2.586	+23°45'35"	+4029
2	11.0	.100	4.850	+20 59 22	+3582
3	16.5		6.812	+18 16 00	+3134
4	22.0	.200	8.486	+15 35 00	+2686
5	27.5		9.884	+12 56 00	+2238
6	33.0	.300	11.015	+10 19 02	+1791
7	38.5		11.888	+7 43 06	+1343
8	44.0	.400	12.508	+5 08 16	+895
9	49.5		12.878	+2 34 00	+0428
10	55.0	.500	13.002	0	0
9'	60.5		12.878	-2 34 00	-0428
8'	66.0	.600	12.508	-5 08 16	-0895
7'	71.5		11.888	-7 43 06	-1343
6'	77.0	.700	11.015	-10 19 02	-1791
5'	82.5		9.884	-12 56 00	-2238
4'	88.0	.800	8.486	-15 35 00	-2686
3'	93.5		6.812	-18 16 00	-3134
2'	99.0	.900	4.850	-20 59 22	-3582
1'	104.5		2.586	-23 45 35	-4029

T

Point	$\frac{1.5}{1}$	$\frac{1.45}{1}$	$\frac{1.45}{1}$
1	5.2598	28.9289	13.602
2	5.1557	56.7127	25.005
3	5.0691	83.6401	34.531
4	4.9965	109.9230	42.400
5	4.9379	135.7922	48.806
6	4.8915	161.4195	53.880
7	4.8565	186.9752	57.734
8	4.8320	212.6080	60.439
9	4.8171	238.4464	62.035
10	4.8127	264.6985	62.575
9'	4.8171	291.4345	62.035
8'	4.8320	318.9120	60.439
7'	4.8565	347.2397	57.734
6'	4.8915	376.6455	53.880
5'	4.9379	407.3767	48.806
4'	4.9965	439.6920	42.400
3'	5.0691	473.9608	34.531
2'	5.1557	510.4143	25.005
1'	5.2598	549.6491	13.602

TA

Station	1000' at 2 $\frac{(x-a)y_{05}}{I}$	1000' at 4 $\frac{(x-a)y_{05}}{I}$	1000' at 6 $\frac{(x-a)y_{05}}{I}$	1000' at 8 $\frac{(x-a)y_{05}}{I}$
1				
2				
3	189.921			
4	466.400			
5	805.299	268.153		
6	1185.360	592.680		
7	1587.685	952.611	317.537	
8	1994.187	1329.658	664.829	
9	2388.347	1705.963	1023.577	34
10	2753.300	2064.975	1376.650	68
9'	3070.733	2388.347	1705.962	102
8'	3324.145	2659.316	1994.187	132
7'	3492.907	2857.533	2222.759	158
6'	3556.080	2963.400	2370.720	177
5'	3489.629	2952.763	2415.897	187
4'	3264.800	2798.400	2332.000	186
3'	2848.807	2468.966	2089.125	170
2'	2200.440	1925.385	1650.330	137
1'	1271.787	1122.165	972.543	82
Σ	37890.127	29050.835	21136.416	1440

TA

Load at	$\frac{L-a \Sigma x y_{05}}{L}$	$\Sigma \frac{(x-a)y_{05}}{I}$
10	23634.530	8988.319
8'	18907.624	4946.760
6'	14180.718	2228.759
4'	9453.812	689.408
2'	4726.906	74.811

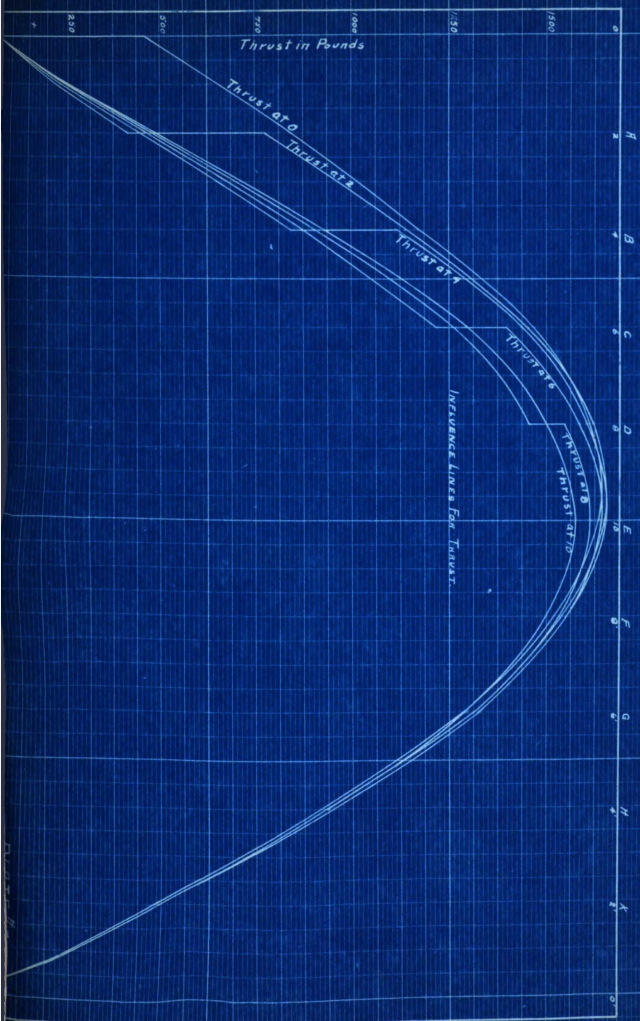
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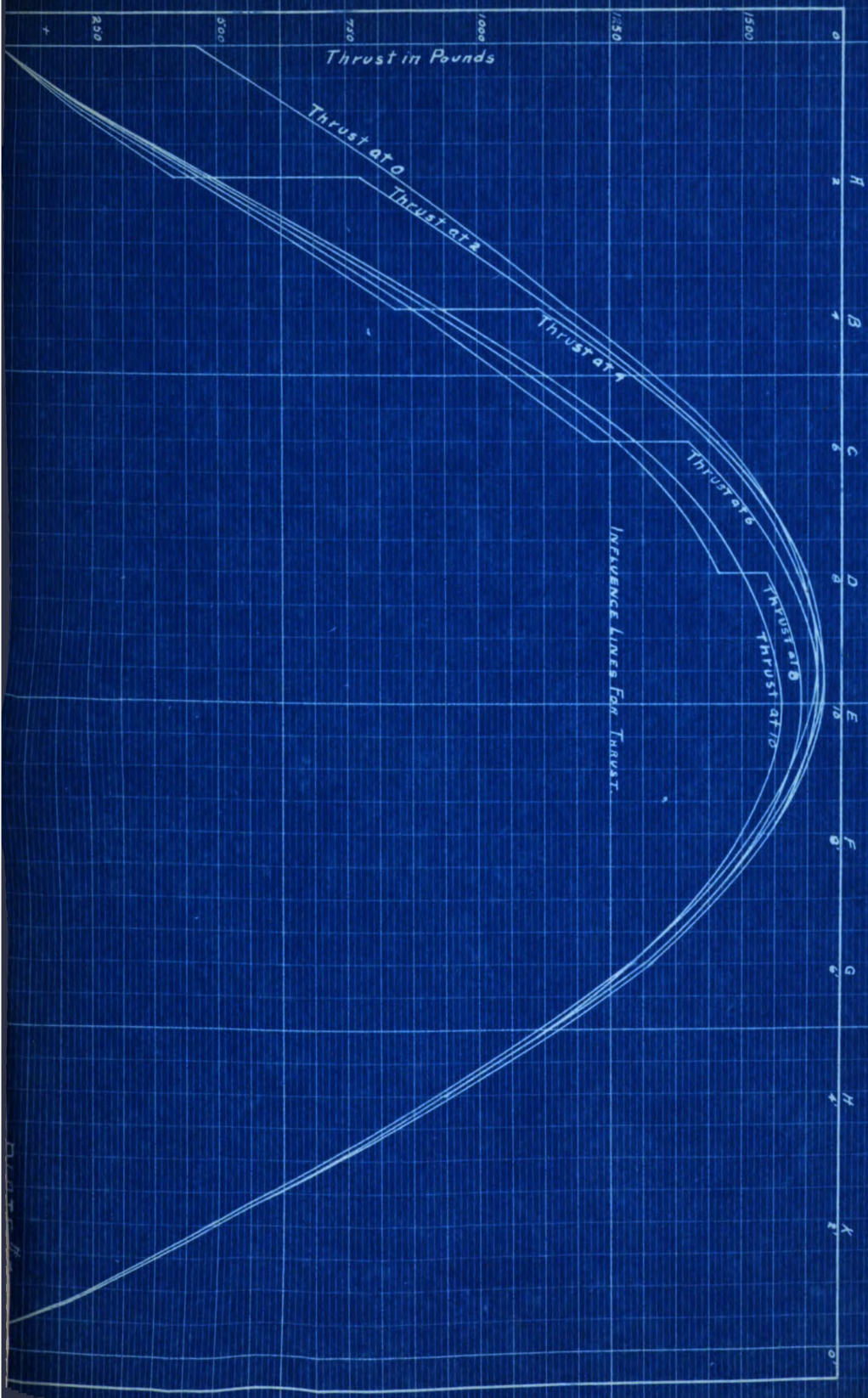
Stress At	Load at 10			Load at 8'		
	T	S	M	T	S	M
0	+1632	-263	0	+1520	-318	0
2	1648	-100	-22092	1542	-165	-30
4	1652	+55	-25248	1550	-20	-41
6	1639	+209	-10257	1548	+123	-35
8	1612	+355	+22092	1530	+262	-13
10	1578	+500	+62432	1500	+400	+24
8'	1612	-355	+22092	1545	-463	+76
6'	1639	-209	-10257	1583	-323	+31
4'	1652	-55	-25248	1606	-178	+30
2'	1648	+100	-22092	1617	-28	-82
0'	1632	+263	0	1609	+133	0

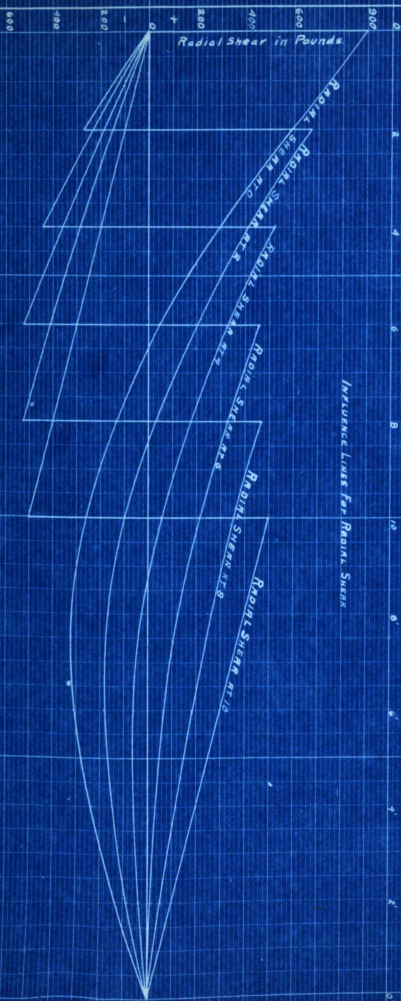
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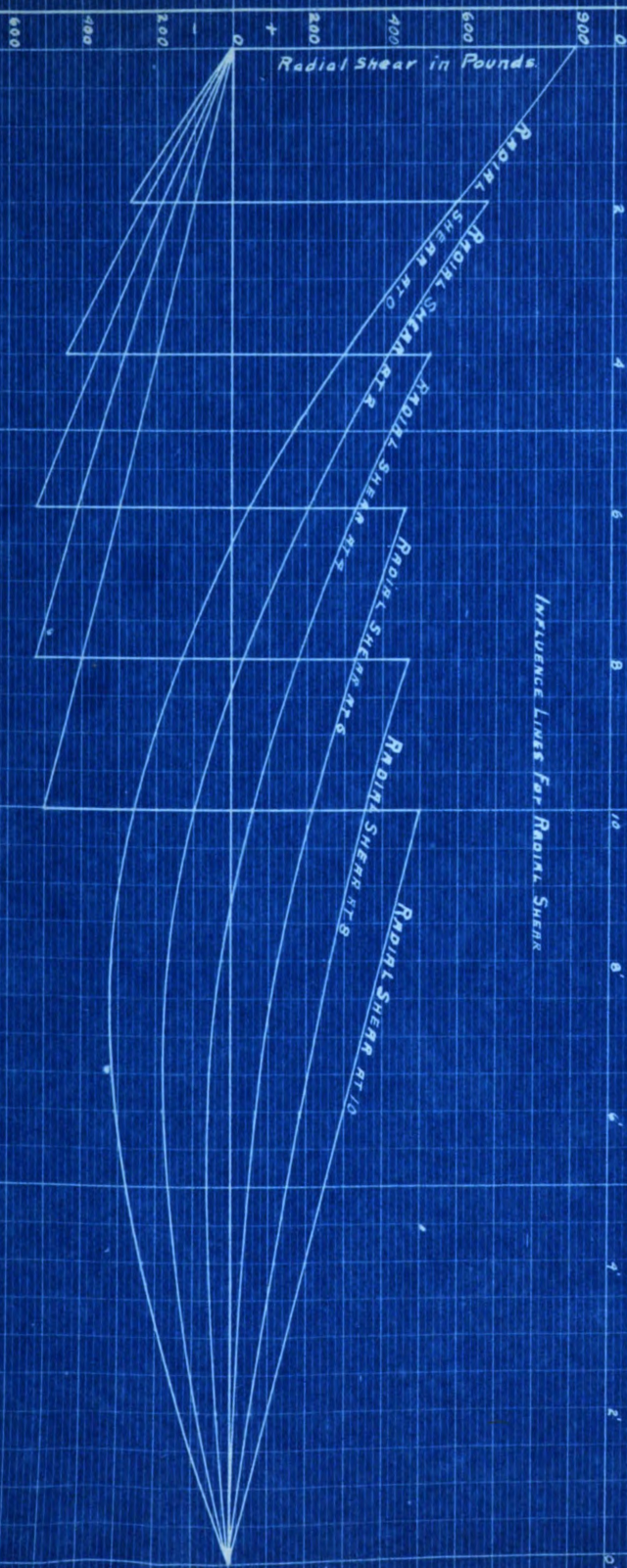






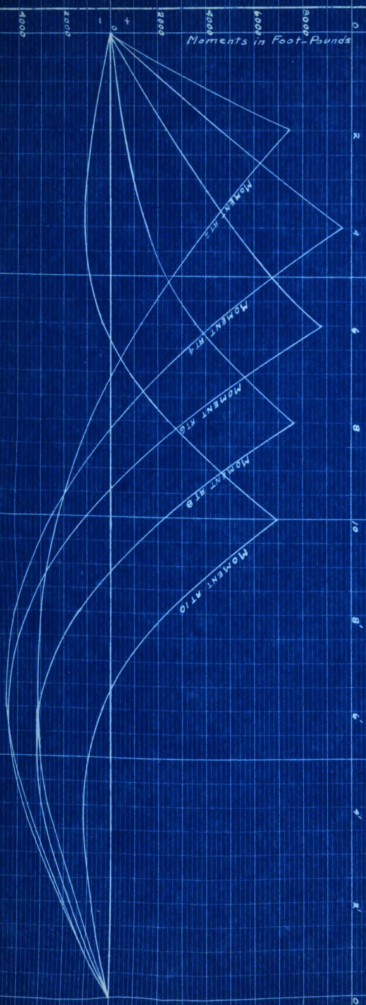


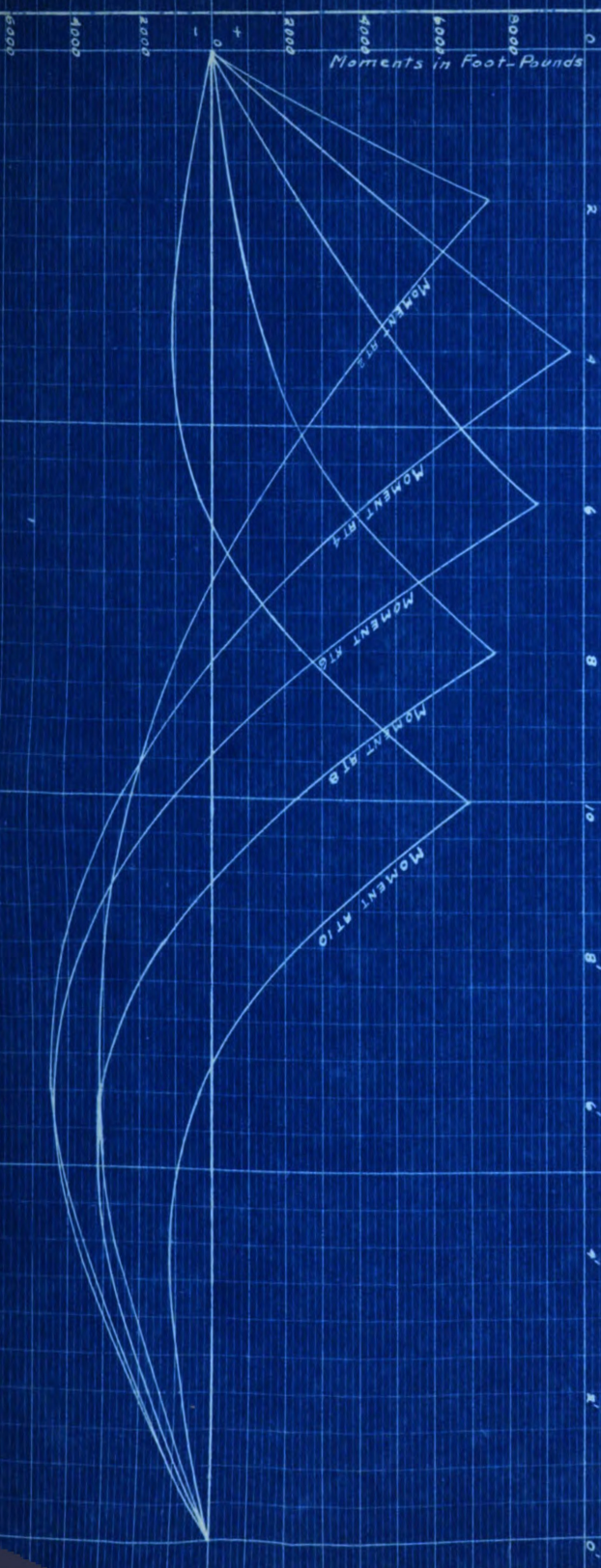




INFLUENCE LINES FOR MOMENTS

Moments in Foot-Pounds





INFLUENCE LINES FOR MOMENTS

	D+L+I	D+L+I	D = DEAD LOAD - L = LIVE LOAD - I = IMPACT			
SECTION	MAXIMUM MOMENT	CORRESP'G THRUST	FLANGE			
			STRESS PER SQ. IN.	STRESS DUE TO IMPACT % TO LIVE LOAD	TOTAL STRES. PER SQ. IN.	POSITION OF (90000 ⁺) CHR AND UNI- FORM LIVE LOAD (100 ⁺ PER SQ. FT.).
2	+321127.0 ^{1/4}	659151.6	TOP +15936.0	+2142.0	+18098.0	88.5 LIN. FT. U.L.L. CHR
			BOT. + 7910.0	-1242.0	+ 6668.0	98.5 LIN. FT. U.L.L.
4	+576968.3	662455.2	TOP +19221.0	+2949.0	+22170.0	79.5 LIN. FT. U.L.L. CHR
			BOT. + 4763.0	-1709.0	+ 3054.0	49.5 LIN. FT. U.L.L.
6	+691430.8	649396.0	TOP +20418.0	+2899.0	+23317.0	49.5 LIN. FT. U.L.L. CHR
			BOT. +3092.0	-1621.0	+ 1471.0	49.5 LIN. FT. U.L.L.
8	+732238.0	670321.0	TOP +21309.0	+2840.0	+24149.0	60.5 LIN. FT. U.L.L. CHR
			BOT. +2959.0	-1022.0	+ 1937.0	60.5 LIN. FT. U.L.L.
10	+671482.0	660707.0	TOP +20373.0	+2275.0	+22646.0	39 LIN. FT. U.L.L. CHR
			BOT. + 3547.0	- 607.0	+ 2940.0	39 LIN. FT. U.L.L.
SECTION	TOTAL SHEAR	SHEAR PER SQ. IN.	POSITION OF CHR AND UNIFORM LIVE LOAD ON BRIDGE TO GIVE MAX. SHEAR.			
0	-54133.3 ^{1/4}		71.5 LIN. FT. U.L.L. CHR			60.5 LIN. FT. U.L.L.
2	+66881.6	1210 ^{1/4}	49.5 LIN. FT. U.L.L. CHR			60.5 LIN. FT. U.L.L.
4	+64434.0	1166	44 LIN. FT. U.L.L. CHR			60.5 LIN. FT. U.L.L.
6	+66011.6	1195	82.5 LIN. FT. UNIFORM L.L. CHR			60.5 LIN. FT. U.L.L.
8	+65681.8	1189	71.5 LIN. FT. U.L.L. CHR			60.5 LIN. FT. U.L.L.
10	+635140.0	686	60.5 LIN. FT. U.L.L.			60.5 LIN. FT. U.L.L.



TABLES C and H
PLATE # 9

	D+L+I	D+L+I	D = DEAD LOAD - L = LIVE LOAD - I = IMPACT				
SECTION	MAXIMUM MOMENT	CORRESP'G THRUST	FLANGE	STRESS PER SQ. IN.	STRESS DUE TO IMPACT 40% LIVE LOAD	TOTAL STRS. PER SQ. IN.	POSITION OF (90,000 [#]) CHR AND UNIFORM LIVE LOAD (100 [#] PER SQ. FT.).
2	+321127.0 [#]	659151.6	TOP BOT.	+15956.0 +7910.0	+2142.0 -1242.0	+18098.0 +6668.0	38.5 LIN. FT. U.L.L. CHR 38.5 LIN. FT. U.L.L.
4	+576968.3	662455.2	TOP BOT.	+19221.0 +4763.0	+2949.0 -1709.0	+22170.0 +3054.0	49.5 LIN. FT. U.L.L. CHR 49.5 LIN. FT. U.L.L.
6	+691430.8	649396.0	TOP BOT.	+20418.0 +3092.0	+2899.0 -1621.0	+23317.0 +1471.0	49.5 LIN. FT. U.L.L. CHR 49.5 LIN. FT. U.L.L.
8	+732238.0	670321.0	TOP BOT.	+21309.0 +2959.0	+2840.0 -1022.0	+24149.0 +1937.0	60.5 LIN. FT. U.L.L. CHR 60.5 LIN. FT. U.L.L.
10	+671482.0	660707.0	TOP BOT.	+20373.0 +3547.0	+2273.0 -607.0	+22646.0 +2940.0	33 LIN. FT. U.L.L. CHR 33 LIN. FT. U.L.L.
SECTION	TOTAL SHEAR	SHEAR PER SQ. IN.	POSITION OF CHR AND UNIFORM LIVE LOAD ON BRIDGE TO GIVE MAX. SHEAR.				
0	-54133.3 [#]		71.5 LIN. FT. U.L.L. CHR 71.5 LIN. FT. U.L.L.				
2	+66881.6	12.10 [#]	49.5 LIN. FT. U.L.L. CHR 49.5 LIN. FT. U.L.L.				
4	+69434.0	11.66	44 LIN. FT. U.L.L. CHR 44 LIN. FT. U.L.L.				
6	+66011.6	11.95	82.5 LIN. FT. UNIFORM L.L. CHR 82.5 LIN. FT. UNIFORM L.L.				
8	+65681.8	11.89	71.5 LIN. FT. U.L.L. CHR 71.5 LIN. FT. U.L.L.				
10	+35140.0	6.36	60.5 LIN. FT. U.L.L. CHR 60.5 LIN. FT. U.L.L.				

0

2

4

6

8

10

8

6

4

2

0

30

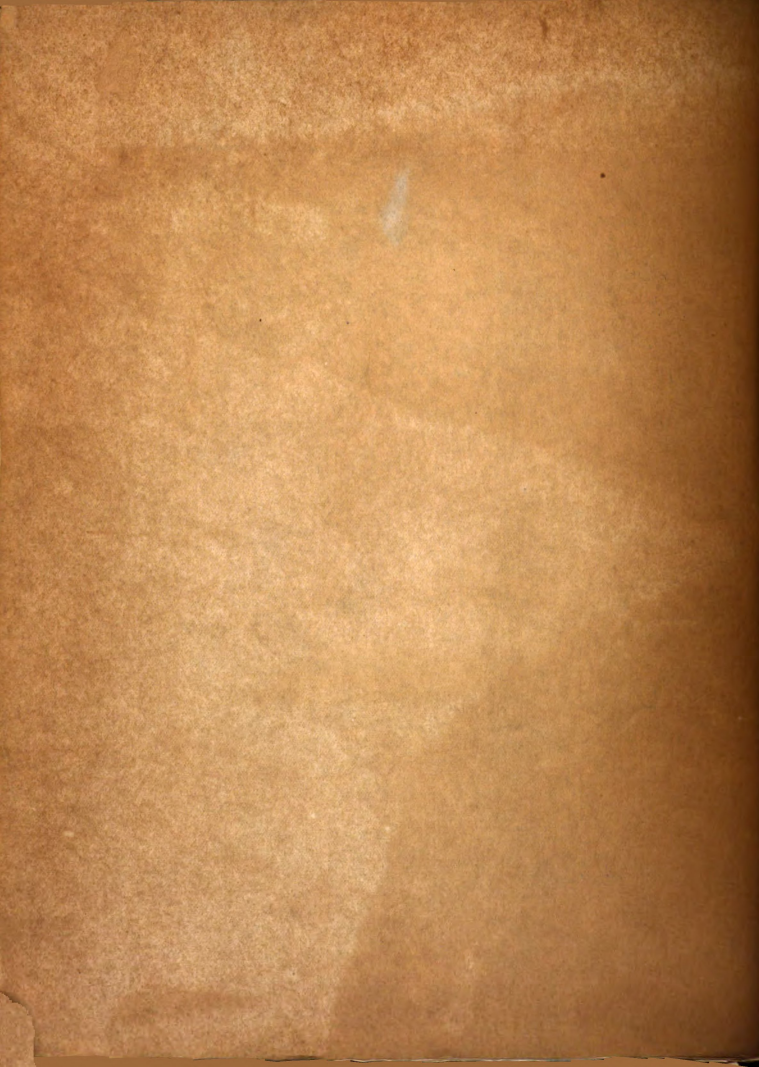
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TABLES C and H
PLATE # 9



TABLES C and H
PLATE # 9



ROOM USE ONLY

ROOM USE ONLY

ROOM USE ONLY

ROOM USE ONLY

Pocket Mus:

1 plate (plate 10)

101

476

THS

Plate 10

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3 1293 02920 8240

SUPPLEMENTARY
MATERIAL

Pocket no. 1

1 plate (plate 10)

101

476

THS

Plate 10

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SUPPLEMENTARY
MATERIAL

MICHIGAN STATE UNIVERSITY LIBRARIES



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