THESIS

Analysis of the Mich. C. R. Bridge
at Bay City, Michigan

E. B. SCOTT. M. A. CHAMBERS.

1914

THESIS

-:-:-

AN ANALYSIS

of

THE MICHIGAN CENTRAL RAILWAY BRIDGE

over

THE SAGINAW RIVER

at

BAY CITY, MICHIGAN

-:-:-

PRESENTED IN PARTIAL FULFILLMENT FOR A DEGREE OF

BACHELOR OF SCIENCE

to

THE CIVIL ENGINEERING DEPARTMENT

MICHIGAN AGRICULTURAL COLLEGE.

-: 1914 :-

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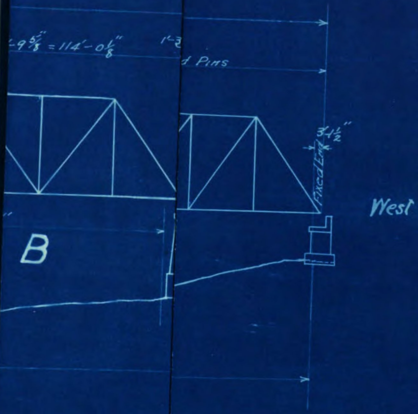
THESIS

PREFACE

The object of this thesis is to analyze the bridge and to become more familiar with the make-up of the different members, and the general construction of a modern railway bridge.


-9 5/8" = 114'-0 1/8"
d Pins
3 1/2"
Fixed End
West
B

The bridge runs over the Saginaw River, connecting East Bay City and West Bay City, its direction being east and West. The bridge contains three simple truss spans and a draw span of the Rim Bearing Swing type.

An inspection was first made of the bridge in which all different parts of the make up of the bridge were measured and many pictures taken. Some of these pictures will be referred to from time to time and they can be found inclosed herewith.

The two trusses on the west side of the draw are of the Through Pratt type, with rivet connected joints, built by the American Bridge Co. in 1905. The draw span was built at the same time and is of the same general construction. The other simple span, on the east side of the draw, is a pin connected truss of the Through Pratt type. It was built by the Detroit Bridge & Iron Works in 1888 and has been moved to its present location from the site for which it was originally built.

The rivet connected simple trusses are made up of seven panels @ 20'-10", having a depth between centers of chords of 30'-00" and a width of 17'-00". These trusses seem to be of heavy construction having end portals made of solid sheets of metal as can be seen from photograph No.1.

The old Pin Connected Pratt truss, shown in Photos Nos. 2 and 3, contains 4 panels at 20'-00", and 2 end panels at 20'-8½". The depth is 24'-00", center to center of pins, and the width of roadway 17'-00", center to center of chords. The truss is much lighter than the others and is made of wrought iron.

The draw span is of the Rim Bearing Swing type, containing five panels at 20'-9 5/8" on each side of the center, and a center panel of 17'-00". The tower is 33'-8" from the center of the lower chord up to the center of the pin which holds the eye-bars connecting the tower with the extending truss. The upper and lower chords of this draw span are all parallel as can be seen by photo No. 4.

The dead loads were computed by using the Cambria Handbook in connection with the measurements taken. For the old wrought iron truss a handbook edited in 1886 by the Phoenix Iron Works, Phoenixville, Pa., was secured and used.

For live loads Cooper's E-50 loading was used, altho this bridge should be strong enough to carry E-55 or E-60 to be consistent with modern heavy loadings.

WEIGHTS OF SPANS.

The bridge includes three entirely different spans, for each of which a table such as Plate No. 1 was filled out. This table is made up of the measurements taken on the bridge, and the first calculations to be made. For example: the calculations were made as follows,

We found the member U1U2 to be made up of four angles

3½" x 3½" x 5/8"	@ 3.98 sq. in.	=	15.92 sq. in.
and two plates			
18" x ½"	@ 9.00 sq. in.	=	18.00 sq. in.
	Gross Area		33.92 sq. in.

As this member is an upper chord and known to be always in

SPAN C

MEMBER	MAKE UP	LACING	SIZE OF RIVET	LENGTH C TO C	WIDTH b TO b WEB	GROSS AREA	HOLES DEDUCTED	NET AREA	ESTIMATED WEIGHT	ACTUAL WEIGHT	% DET.	I_{1-1} (Inches)^4	I_{2-2} (Inches)^4
L_0L_2	4∠s 6x6x$\frac{1}{2}$"	3 PLs. 9"x$\frac{5}{8}$"x12$\frac{1}{2}$	$\frac{7}{8}$"	41'-8"	12$\frac{3}{4}$"	23.00□"	8	19.2□"	3260#	3440#	5$\frac{1}{2}$%		
L_2L_3	4∠s 6x6x$\frac{3}{4}$"	2 PLs. 9"x$\frac{3}{8}$x12$\frac{1}{2}$	$\frac{7}{8}$"	20'-10"	12$\frac{3}{4}$"	33.76	8	28.1□"	23 90	2443	2.2		
L_3L_4	4∠s 6x6x$\frac{3}{4}$" 1 PLATE 12$\frac{1}{2}$x$\frac{5}{8}$"		$\frac{7}{8}$"	20'-10"	12$\frac{3}{4}$"	41.56	8	34.3□"	2950	4238	43.5		

SPAN A

Member	Make Up	Lacing	Size of Rivet	Length c to c	Width b to b Web (inches)	Gross Area	Holes Deducted	Net Area (sq inches)	Estimated Weight	Actual Weight	% Det.	I_{1-1} (Inches⁴)	I_{2-2} (Inches⁴)
L_0L_1	2 Eye Bars 6 x 1 1/8" 1 Pl. 16 1/2 x 3/8			20'-8 1/2"	16 1/2	19.7 □"		13.5	1360 #	1644.0 #	20.9		
L_1L_2	2 Eye Bars 6 x 1 1/8 1 Pl. 19 3/4 x 3/8			20'-0"	19 3/4	20.9		13.5	1310	1586.0	21.0		
L_2L_3	2 Eye Bars 6 x 1 1/2			20'-0"	16 1/2	18.0			1120	1400.0	25.0		
U_1U_2	4 Ls 2 1/2 x 2 1/2 x 1/2 2 Pls. 16 x 1/2	2 1/2 x 3/8 x 25	3/4	20'-0"	13 3/8	25.44			1695	2231.0	31.6	856.0	1275.0
U_2U_3	4 Ls 2 1/2 x 2 1/2 x 1/2 2 Pls 16 x 5/8	2 1/2 x 3/8 x 25	3/4	20'-0"	13 1/8	29.44			1965	2231.0	31.6	942.0	1553.0

compression, there are no rivet holes to be deducted to get the net area.

The estimated weight is found by multiplying the gross section area, in square feet, by the length, center to center, in feet, the product being multiplied by the weight per cubic foot of steel

$$33.92/144 \times 20.833 \times 490\# = \qquad 2400\#.$$

In order to get the actual weight it is found necessary to add the following:-

Lattice- 26 bars - 4½" x ½" x 26"	@ 16.6#	=	433#.
Tie plates- 2 plates-21" x 3/8" x1'-10"	@49#	=	98#.
Rivet Heads- 378 (3/4")	@ 22#/C	=	83#.
	Weight to be added		614#.
	Estimated weight		2400#.
	Actual Weight		3014#.

The ratio of the amount to be added to the estimated weight is found, in this instance, to be

$$614/\ 2400 = 0.256 = 25.6\%.$$

The moment of inertia is found about the axis parallel and perpendicular to the plates thru the center as follows:-

About axis 1-1 Angles.

$$I_{1-1} = I_g + Ad^2.$$

$$= 4\ \ 4.33 + \ 3.98 \times (8.02)^2 \qquad = 1020.$$

Plates.

$$I_{1-1} = \frac{1}{12}\ bh^3.$$

$$= 2 \times 1/12 \times \tfrac{1}{2} \times 18^3 \qquad = 486.$$

Total 1506.

About axis 2-2 Angles.

$$I_{2-2} = 4\ 4.33 + 3.98 \times (8.60)^2 = 1198.$$

Plates.

$$I_{2-2} = 2 \times (18 \times \tfrac{1}{2}) \times (7.25)^2 = \underline{948.}$$

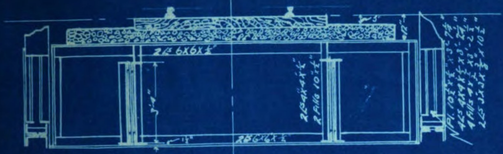
Total 2146.

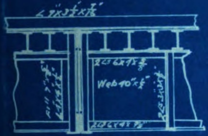
The moments of inertia about these two axes should be about the same for the best design but on account of making riveted connections with other members this is not always possible.

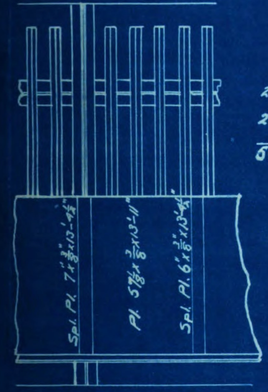
The above calculations were carried thru for each different member of the truss and the weights of all members totaled. To this sum is added the weights of the sway bracing and the end portals which are also calculated on Plate No. 1. The sum of the weights of the trusses, sway bracing and end portals give the total weight of the bridge above the floor.

The remainder of the dead load is in the floor and in the girders supporting the floor. A section of one is shown on Plate No. 2, on which all of the weights are tabulated for one panel. The weight of the floor is found for the seven panels. On this same plate is summed up the total weight of the span. The weights of the gusset plates were added in the last. This should have been included on Plate No. 1, which gives the total weight of steel above the floor.

One half of the total weight is placed on each truss and divided among the panel points. The weight of the truss was divided equally between the upper and lower panel points, the weight of the floor taken entirely by the lower panel points. The calculations were made so as to bring the load







WEIGHT OF FLOOR FOR ONE PANEL

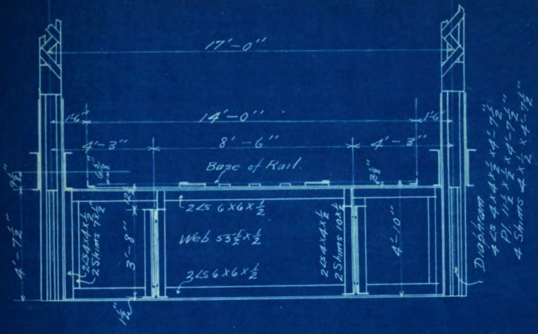
4400#	Floor Pl. $13'\text{-}11'' \times \frac{3}{8}''$	
544	Spl. Pls. $13'\text{-}4\frac{1}{2}'' \times \frac{3}{8}''$	
8060	19 I Beams 10" @ 30# per ft.	
2840	2 Web Pls. $40'' \times \frac{1}{2}'' \times 20'\text{-}10''$	(Girders)
3330	8∠s $6'' \times 4'' \times \frac{5}{8}'' \times 20'\text{-}10''$	(Flanges)
187	8∠s $3 \times 3 \times \frac{3}{8} \times 3'\text{-}3''$	(Stiffeners)
624	2∠s $7 \times 3\frac{1}{2} \times \frac{7}{16} \times 20'\text{-}10''$	
137	4∠s $4 \times 4 \times \frac{1}{2} \times 2'\text{-}8''$	
192	8 Fills $7 \times \frac{3}{8} \times 2'\text{-}8''$	
1215	4∠s $6 \times 6 \times \frac{1}{2} \times 15'\text{-}6''$	
1360	Web $51\frac{1}{2} \times \frac{1}{2} \times 15'\text{-}6''$	
439	8∠s $4 \times 4 \times \frac{1}{2} \times 4'\text{-}3\frac{1}{2}''$	
119	4 Fills $7'' \times \frac{1}{2} \times 2'\text{-}6''$	
170	4 Fills $10'' \times \frac{1}{2} \times 2'\text{-}6''$	
259	4 sp. Pls. $11'' \times \frac{1}{2}'' \times 3'\text{-}5\frac{1}{2}''$	
8	2 sp. Pls. $6'' \times \frac{3}{8}'' \times 6''$	
23884	Total	
2470	Rails 3 @ 65# 2 @ 80#	
23100	Broken stone on floor 184.58 cu ft @ 125#	
1383	Ties 6"x8"x8' @ 40# per cu ft.	
50837	Total weight of floor for one panel	

7 x 50837 = 355855# for span

166379	wt. of 2 Trusses
522234	Total wt. of span.

$\frac{522234}{2} = 261117$# to each Truss

1002	Pls. for Sway Bracing
2898	Upper Gusset Pls.
3676	Lower Gusset Pls.
268693#	Weight on each Truss.



WEIGHT OF FLOOR FOR ONE PANEL.

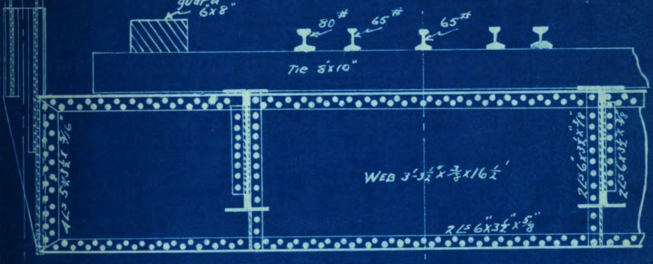
4810 lbs.	Floor P/s. 13'-11" x 3/8"
513	Spl. P/s 13'-4½ x 3/8
8420	21 I Beams 10" @ 30#/ft.
3410	2 Web P/s (girder) 44 x ½ x 22'-9 5/8"
3640	8 ∠s 6x4 x 5/8 (flanges)
205	8 ∠s 3x3 x 3/8 (stiff.)
379	2 ∠s 3½ x 3½ x 3/8
179	4 ∠s 4x4 x ½ x 3'-6"
357	8 Shims 7 x 5/8 x 3
474	8 ∠s 4x4 x ½ x 5'-5½"
1215	4 ∠s 6x6 x ½ x 15'-6"
1490	Web 55½ x ½ x 15'-6"
204	4 Shims 10 x ½ x 3'-8"
143	4 " 7 x ½ x 3'-8"
25698	Total.
2700	Rails 3 @ 65# and 2 @ 85#
9140	Stone (broken) 72.8 cu. ft @ 125#/cu.ft.
67	Guard Rail P/s.
1960	Rail P/s.
39565	Total for One Panel

39565 ÷ 20.833 = 18994# wt./ft. of bridge

18994 x 245 = 4653553#

4653553	
205325	Wt of extra floor beams and end mach.
4858878	Total wt of floor of Swing Span.
2740056	Total wt. of Trusses
29700	Wt of Gusset P/s.
2950	" " Top Lat ".
7925584	Total wt. of Span.

SECTION OF FLOOR (SPAN A)



WEIGHT OF FLOOR FOR ONE PANEL

812# Web 3'-3½" x 3/8" x 16½'
1236 4∠s 6x3½" x 5/8" x 16½'
112 4∠s 6x3½ x 3/8 x 30"
146 4∠s 6x3½ x 3/8 x 39"
119 4∠s 5x3½ x 5/16 x 39"
89 254 Rivets Heads @ 17.5/100
2514# Weight of 1 floor beam
1500 2 Webs 30" x 3/8" x 20'
2400 8∠s 6"x3½" x ½" x 20',
180 2∠s 3"x3" x 3/8" x 12½
326 (are) 2 Rods 1 3/8 x 1¾ (first panel) 1½x1½ (second) ¾ x ¾ (third)
4906# Weight of floor girder (1 panel)
6300 Ties 18 (8x10x14') yellow pine @ 45#/ft.
600 guard-rail 2 (6"x8"x20')
6900# Weight of timber in one panel.
200 added for rivets in Girders and rod fasteners
14020# Total wt. of floor (one panel)
2370 5 steel Rails
16390# 16390 x 6 = 98340# floor for six panels

98340
1110 Wt. added for extra 8½" on each end.
99450# Wt. of floor for Span.
2514 extra floor Beam on end.
2350 Pins
82308 Wt. of 2 Trusses.
186622# Total weight of span

$\frac{186622}{2}$ = 93311# to each Truss.

on the end panel points equal to one half of that on the other lower panel points as shown under the division of weights on Plate No. 3.

The dead load stresses for span "C" are shown on Plate No. 4. The shear was first found for all panels. This value, when multiplied by the secant of the angle made by the diagonal with the vertical gives the stress in each web member. The shear times the tangent of this angle gives the chord increment which is added to each chord to give the stress in the succeeding one. The stresses were calculated in the above manner and tabulated, for spans "C" and "A", on Plate No. 4.

The dead load stresses for span "B", the swing span, were computed for both swinging and closed position. Plate No. 5 shows the graphical solution by which the stresses wsre computed when the span was in the swinging position. The results are tabulated on Plate No. $10\text{-}b_1$.

In order to compute the dead load stresses in the swing span when closed, the reaction at the end must be found from which the stresses can be figured by the ordinary method. This reaction is caused by a force applied at the ends in an upward direction and should be great enough to take out the deflection of the end when swinging or at least enough to counter-act any negative reaction, or tendency to lift off from the support that may be caused by a moving load on the other end. To be on the safe side, about 10,000# should be added to these negative reactions. There did not seem to any provision made for taking care of the negative reactions

DISTRIBUTION OF DEAD LOAD

	Weight of Trusses. (in pounds)	Weight of Floor. (in pounds)	Total (in pounds)	Weight to One Truss. (in pounds)	Load to Upper and Lower Panel Points $\left(\frac{\text{Wt of One Truss}}{\text{Total Panel Pts} - 1}\right)$ (in pounds)	Load to Lower Panel Points. $\left(\frac{\text{Wt of Floor}}{\text{Total Lower Panel Points} - 1}\right)$ (in pounds)
SPAN A	84658	101964	186622	93311	3848	8497
SPAN B	306706	485878	792584	396292	7303	22085
SPAN C	181531	355855	537386	268693	6982	25418

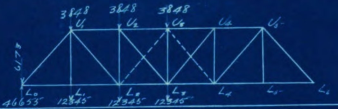
PLATE 4

DEAD LOAD STRESSES (SPAN A)

End Panels $\begin{cases} \sec\theta = 1.3208 \\ \tan\theta = 0.863 \end{cases}$

Other Panels $\begin{cases} \sec\theta = 1.3018 \\ \tan\theta = 0.8333 \end{cases}$

Diagonals	Shears (V)	Stresses ($V\sec\theta$)	Verticals	Stress in Verticals	Chord Increment ($V\tan\theta$)	Chord Stress	Chords
L_0U_1	40482	53470	L_1U_1	12345	34936	34936	L_0L_1 & L_1L_2
L_2U_1	24289	31620	L_2U_2	11944	20961	55897	U_1U_2 & L_2L_3
L_3U_2	8096	10540	L_3U_3	3848	6987	62884	U_2U_3

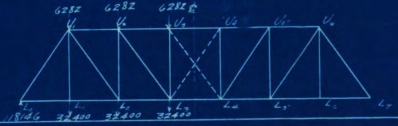


DEAD LOAD STRESSES (SPAN C)

$\sec\theta = 1.2177$

$\tan\theta = 0.6944$

Diagonals	Shears (V)	Stresses ($V\sec\theta$)	Verticals	Stresses Verticals	Chord Increment ($V\tan\theta$)	Chord Stresses	Chords
L_0U_1	118146	143866	L_1U_1	32400	82040	82040	L_0L_1 & L_1L_2
U_1L_2	78764	95910	L_2U_2	46364	54693	136733	U_1U_2 & L_2L_3
L_3U_2	39382	47955	L_3U_3	6282	27347	164080	U_2U_3 & L_3L_4 U_3U_4



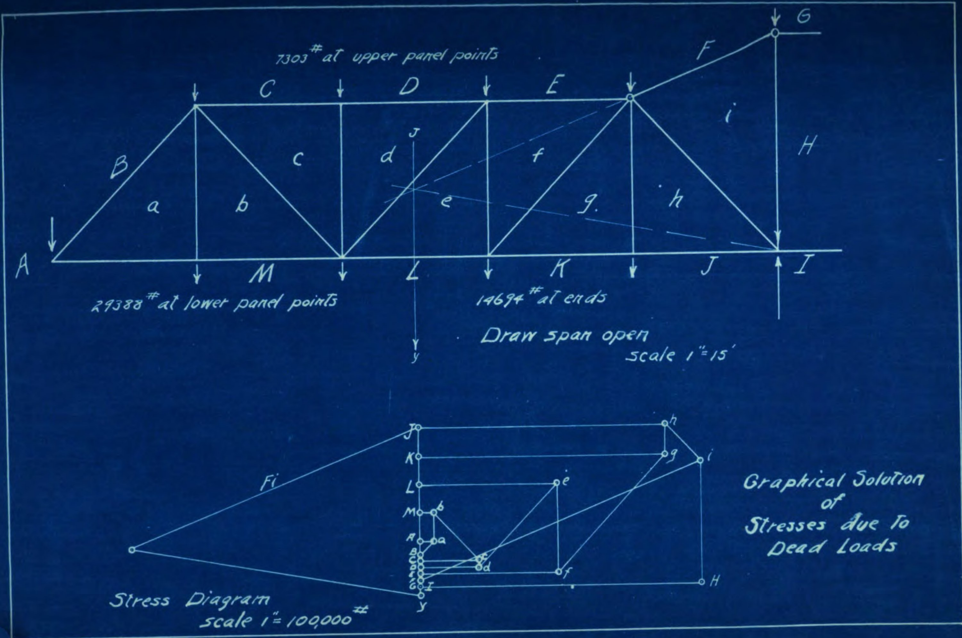
DEAD LOAD STRESSES (SPAN B)
(SWING SPAN CLOSED)

$\sec\theta = 1.3614$
$\tan\theta = 0.9244$

Diagonals	Shears (V) (kips)	Stresses ($V\sec\theta$) (kips)	Verticals	Stress in Verticals (kips)	Moments (kip-ft.)	Chord Stress (kips)	Chords
L_0U_1	57.4	70.0	L_1U_1	29.4	1148.0	46.50	L_0L_1 & L_1
U_1L_2	14.7	20.0	L_2U_2	-7.3	1504.0	61.0	U_1U_2 & U_2U_3
L_2U_3	-22.0	30.0	L_3U_3	29.3	1014.0	41.2	L_2L_3 & U_3U_4
L_3U_4	-58.7	80.0	L_4U_4	29.4	5642	90.0	U_4L_5-
L_5-U_4	(See Chords)				336	13.6	L_3L_4 & L_4L_5

(SWING SPAN OPEN)

Diagonals	Shears (V)	Stresses ($V\sec\theta$)	Verticals	Stress in Verticals	Moments	Chord Stresses	Chords
L_0U_1	14.7	20.0	L_1U_1	29.4	338.0	13.6	L_0L_1 + L_1L_2
U_1L_2	57.4	70.0	L_2U_2	23.0	1507.0	61.5	U_1U_2 + U_2U_3
L_2U_3	88.1	120.0	L_3U_3	80.8	3512.0	142.0	L_2L_3 + U_3U_4
L_3U_4	124.8	169.8	L_4U_4	29.4	6360.0	258.0	L_3L_4 + L_4L_5
					3764.0	60.0	U_4L_5-



Graphical Solution of Stresses due to Dead Loads in this bridge, because when a load came on one end, the other end was seen to jump up and down considerably so only enough reaction to take out the deflection will be used in our case.

Innorder to find the force necessary to overcome the deflection, the deflection must first be found. This was done by considering a truss as a simple cantilever beam and h the formula used to find the deflection due to each panel point load was

$$f = PL^3(2 - 3k + k^3) / 6 EI.$$

The computations are shown on Plate No. 6.

The live load stresses in web members for span "C" and span "A" are tabulated on Plate No. 7. All stresses were found by placing the wheel loads in that position which would cause the maximum stress.

LIVE LOAD STRESS IN L_0U_1

This is a web member and the criterion is followed out that;- The shear in any panel of a bridge is largest when the load on the panel is equal to the total weight of the live load on the bridge divided by the number of panels, or

$$V = 1/m \times W$$

where: V = shear.

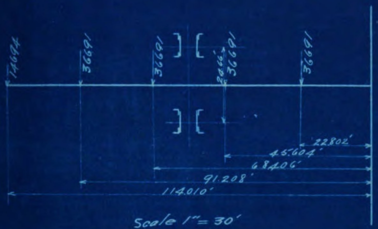
m = number of panels.

W = total weight of live load.

Wheel No. 2 was placed at the right hand end of the panel or at L_1 and the load on the panel is found to vary from 12.5 to 37.5 kips by moving the wheel a very small distance in either direction. The total load on the bridge is 415 kips and since "n" equals 7, the shear should be

PLATE 6.

COMPUTATIONS FOR REACTIONS SWING SPAN. (CLOSED)



Scale 1" = 30'

$f = \frac{Pl^3}{6EI}(2-3k+k^3)$.

kl = the distance of the load from left end.
f = deflection in inches
P = load in lbs
l = length in inches
E = modulus of elasticity (30,000,000)
I = moment of inertia of section

$6EI = 6 \times 30{,}000{,}000 \times 1125245$
$= 202544100000000$
$Pl^3 = 36691 \times 2560100000$
$= 93932629100000$
$\Sigma(2-3k+k^3) = 2.800$
$Pl^3[\Sigma(2-3k+k^3)] = 263011361480000$
$2Pl^3$ (for load at end) 75236218800000
338247580280000 = Total

$f = \frac{338247580280000}{202544100000000} = 1.67''$

Av. area (top ch.) = 25.0 □"
" " (bottom ch) = 26.5 □"
I_9 (top ch.) = 1125.2 inches⁴
I_9 (bottom ch) = 1224.6 "

$\frac{26.5}{25.0} = \frac{x}{24.66 - x}$
$653 - 26.5x = 25x$
$x = 12.66'$
(Top ch) $I_z = I_9 + Fd^2$
$= 1125.2 + 25.0 \times (12.66 \times 12)^2$
$= 574517$ inches^4
(Bottom ch.) $I_z = 1224.6 + [26.5 \times (12.0 \times 12)^2]$
$= 550728$ inches^4
Total $I_z = 1125245$ "

Then - knowing deflection (f) solve back for the force necessary to raise the end of the bridge to horizontal.

$f = 1.67 = \frac{338247580280000}{202544100000000} = \frac{P \times 2 \times 2560100000}{202544100000000}$

P = End Reaction = 66061 lbs.

WEB STRESSES (Live Load)

Spans A and C

Panel	Wheel at Right End of Panel	Moment at Right Support (kip-feet)	Moment at Right End of Panel (kip-feet)	Shear (kips)	Stress (kips)	Web Member
SPAN A						
L_0L_1	4	24016	600.0	168.0	222.5	L_0U_1
					84.25	U_1L_1
L_1L_2	3	15277.0	287.5	111.5	145.0	U_1L_2
L_2L_3	2	8320.5	100.0	63.5	− 63.5	U_2L_2
					+ 82.5	U_2L_3
L_3L_4	2	5612.5	100.0	32.0	− 32.0	U_3L_3
					+ 41.6	U_3L_4
SPAN C						
L_0L_1	4	33970	600.0	204.0	248.0	L_0U_1
					84.25	U_1L_1
L_1L_2	3	23437	287.5	146.5	178.0	U_1L_2
L_2L_3	3	16233	287.5	97.0	−97.0	L_2U_2
					+118.0	U_2L_3
L_3L_4	2	8745	100.0	58.2	− 58.2	L_3U_3
					+70.8	U_3L_4

CHORD STRESSES

DUE TO LIVE LOAD

SPANS A AND C.

Section	Wheel at Section	Length of Train on	Moment at Right Support	Moment at Section	Bending Moment	Stress	Chords
			SPAN A				
U_1L_1	4	9.7'	24017	600	3517	+146.5	L_0L_1 & L_1L_2
U_2L_2	6	3.7'	21782	2050	5250	±219.0	L_2L_3 U_1U_2
U_3L_3	10	7.7'	23255	5790	5865	±244.0	L_3L_4 U_2U_3
	11	15.7	24794	6510	5865	±244.0	U_3U_4
			SPAN C				
U_1L_1	4	34	33970	600	4250	+141.5	L_0L_1 & L_1L_2
U_2L_2	6	27.16	31016	2050	6810	±228	L_2L_3 + U_1U_2
U_3L_3	9	22.33	28995	4370	8060	±269	L_3L_4 U_2U_3
	10	30.33	32350	5790	8060	±269	U_3U_4

415/7 or 59.3. As this value does not lie between 12.5 and 37.5 the condition mentioned above does not satisfy the criterion. Wheel No. 3 was then tried. The load on the panel varies from 37.5 to 62.5 and the total load is 427.5. We found that 427.5/7 = 61.2 lies between the required limits, satisfying the cruterion and the shear due to this loading is found. Wheel No. 4 also satisfied the criterion but whe No. 5 did not. The largest shear of those caused by wheels No. 3 and 4 at L_1 was used which gives the stress when multiplied by the secant of the angle, the same as in the case of dead loads. This method was followed out for all of the web members.

For the chord members, the criterion

$$P' = n'/m \times W$$

where P' = sum of loads on left side of panel thru which section is taken.

N' = number of panels on the left side of the center of moments.

m = number of panels in truss.

W = total weight on bridge.

was used to find the wheel position which gives the maximum chord stress.

The wheels were moved so as to satisfy this criterion and the stress found by taking moments about the center of moments for each chord.

For L_2L_3 the center of moments is taken at U_2. Wheel No. 6 was tried at L_2 and found to satisfy the criterion, as P' varies from 112.5 to 128.75 kips and W = 121 kips, which falls between the required limits. With this load the mom-

ent about the left support is 31016, which, divided by the length of the span, 145.833', gives the left reaction as 213 kips. The moment about U_2 of this reaction is then

$$213 \times 41.6 = {}^{+}8900.$$

The moments of the loads on the left about $U_2 = {}^{-}2050$ and the difference gives the moment about the center of moments which is resisted by L_2L_3.

$$8900 - 2050 = 6850, \quad 6850/30 = 228 \text{ kips}$$

which is the live load stress for L_2L_3. This method is used for finding all of the chord stresses. The stress in the verticals is the maximum shear found in the section. The end verticals, as U_1L_1, were found by placing the wheels so as to satisfy the criterion

$$P = 2P'$$

where P = the load in both panels, L_0L_3

P' = the load in L_0L_1

and then substituting values in the formula

$$R_b = (M_c - 2M_b)/P$$

where R_b = reaction at L_1 or the stress in U_1L_1

M_c = moment of the loads about L_2

M_b = moment of the loads about L_1

P = panel length.

The live load stesses in the swing span were found graphically as is shown on Plate No. 11. Stresses were found for all of the chord members and shears for all of the web members for a load of unity at each panel point and the influence line drawn for each member. The loads are then placed so as to cut the maximum ordinates, the sum of these ordinates, each multiplied by its wheel load gives the stress or dhear desired. In getting moments and shears for these

diagrams, a book, The Designing of Draw Spans, by Wright was used. As the members in the center panels are very light the case was taken of no shear transferred across the ce center panel. Coefficients for reactions for this case can be taken from this book, page 111.Take for example member U_1U_2:- Take center of moments as L_2. Now for unit load at L_1 the left hand reaction is 0.760. Taking moments about L_2 for a load of unity at L_1

$$M_{L_2} = (0.760 \times 41.66) - (1 \times 20.8) - U_1U_2 \times 24.66$$

$$U_1U_2 = \frac{(0.760 \times 41.66) - 20.8}{24.66} = 0.473$$

which is laid off to some scale under L_1 where the load was. Points are plotted in the same way for a load ay each point and a straight line joining each of these points gives the stress influence line for the member U_1U_2. The same is calculated for each member. The stresses are found and tabulated. The diagrams give shear for the web members, this must be multiplied by the secant of the angle to give the stress. In the panel next to the center there are two diagonal members and the method of moments had to be used. From the shear influence line for this panel it is found that the maximum shear occurs when wheel No. 4 is at L_4 with the wheels moving toward the center. Take moments for loading about the same points "a", the intersection of the other two members, as shown on Plate 10.

By similar triangles

$$cU_5 : U_4c = U_5L_5 : aL_5$$

$$9 : 22.8 = 33.66 : aL_5$$

$aL_5 = 85.5'$ distance to center of moments.

GRAPHICAL SOLUTION
OF DRAW SPAN
FOR LIVE LOAD

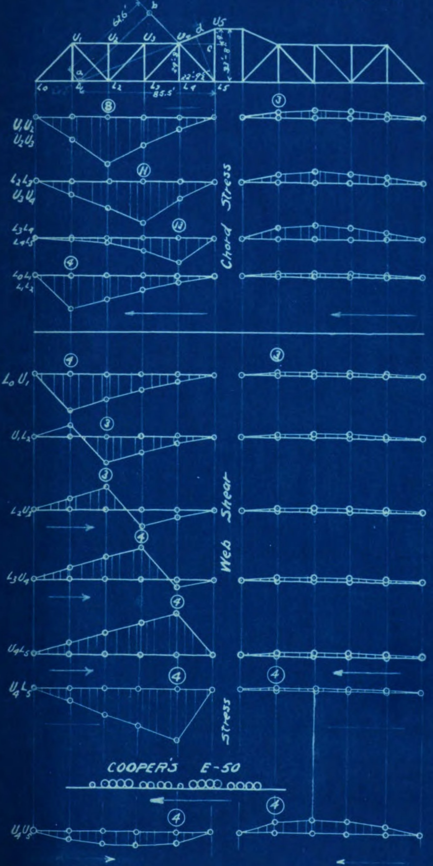


PLATE 9

Scales
1" = 60'
1" = 2#

Also

$$U_4L_4 : U_4L_5 = ab : aL_5$$

$$24.66 : 33.59 = ab : 85.5$$

ab = 62.5' gives the distance at which U_4L_5 acts.

With these distances known, an influence line can be drawn for both U_4U_5 and U_4L_5, always taking the moments of the loads to the left of the section.

After all the stresses due to live load, dead load and impact were calculated a stress sheet such as plate 10 was made out for each different span. These stresses were summed up and the unit stress computed. A column of allowable stresses was calculated and placed alongside. These two results were then compared.

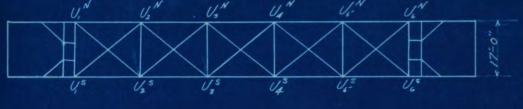
This is far from a complete analysis but it has served the purpose for which it was intended, namely to go a little more into construction details than we have had time for in the regular course.

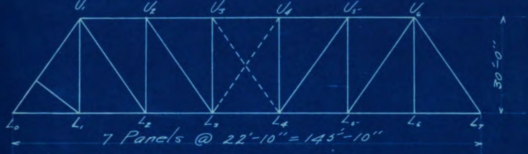
Conclusions

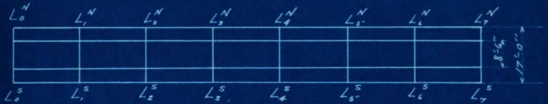
We found that the old truss is stressed in some members nearly to the elastic members. This is not in keeping with the best modern practice. In time a permanent set will probably take place.

The results were not altogether unexpected. It was thought that the bridge was in this condition on account of certain unverified reports that its use was to be discontinued. It has carried heavy loads for some time and will probably continue to do the same in the future, but we would advise replacing the old span with a more modern structure.

STRESS SHEET SPAN C.

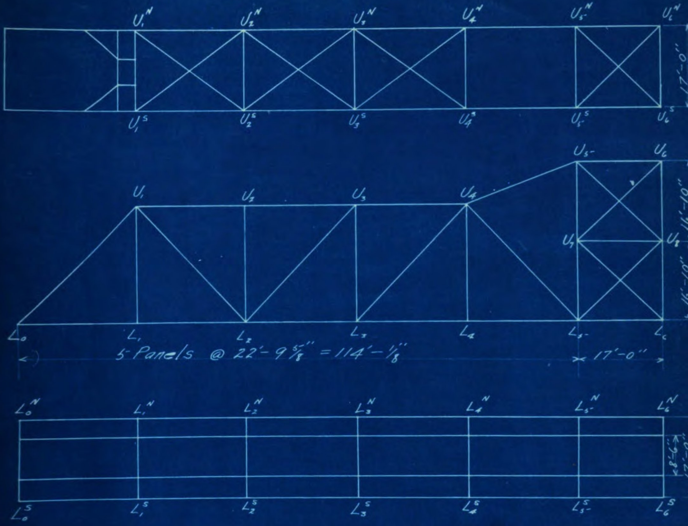






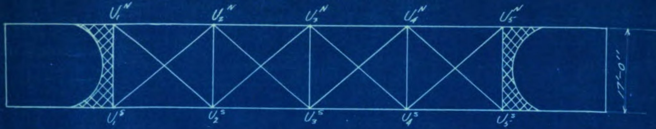
Member	Stress Dead	Stress Live	Stress Impact	Allowable Unit Stress	Stress Unit	Actual Area	(Length)² l^2	Least Radius Gyr.²
L_0U_1	-143.87	-248.0	-168.0	11.3	11.4	54.0□"	192173	44.2
U_1L_2	+95.91	+178.0	+128.0	15.0	16.5	24.3		
U_2L_3	+47.96	+118.0	+90.0	15.0	14.8	17.3		
U_3L_4	0	+70.8	+56.6	15.0	16.3	7.8		
L_0L_1	+82.04	+141.5	+97.1	15.0	16.7	19.2		
L_1L_2	+82.04	+141.5	+97.1	15.0	16.7	19.2		
L_2L_3	+136.73	+228.0	+156.8	15.0	18.6	28.1		
L_3L_4	+164.08	+269.0	+186.7	15.0	18.0	34.3		
U_1U_2	-136.73	-228.0	-156.8	13.3	15.4	34.0	75076	44.3
U_2U_3	-164.08	-262.0	-186.7	13.3	15.4	39.9	75076	43.6
U_3U_4	-164.08	-262.0	-186.7	13.3	15.4	39.9	75076	43.6
U_1L_1	+32.40	+89.3	+57.1	15.0	14.8	11.7		
U_2L_2	-46.36	-97.0	-70.0	9.7	10.2	20.6	129600	17.3
U_3L_3	-62.82	-58.2	-43.6	10.1	11.2	14.7	129600	19.6

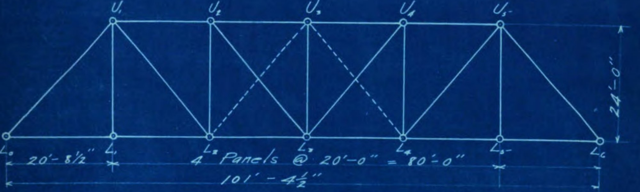
STRESS SHEET SPAN B

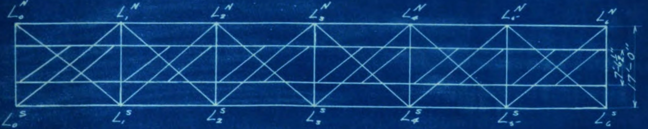


Member	Stress Dead	Stress Live	Stress Impact	Allowable Unit Stress	Stress Unit	Actual Area	(Length)^2 / r^2	Least Radius Gyr^2
L_0U_1	-70.0	-183.0	-134.2	11800	11700	36.1	162000	44.0
U_1L_2	-20.0	-103.9	-75.9	11100	11100	18.0	162000	34.8
L_2U_3	+30.0	+68.0	+53.6	15000	10060	15.0		
L_3U_4	+80.0	+139.0	+116.3	15000	13800	24.3		
U_4L_5	-154.5	-212.0	-155.0	12000	11350	45.8	162000	49.3
L_0L_1	+46.5	+123.45	+90.5	15000	13900	18.7		
L_1L_2	+46.5	+123.45	+90.5	15000	13900	18.7		
U_1U_2	-61.0	-161.8	-117.9	13000	13600	25.0	76500	49.0
U_2U_3	-61.0	-161.8	-117.9	13000	13600	25.0	76500	49.0
L_2L_3	+41.2	+150.75	+110.5	15000	14400	21.2		
U_3U_4	-41.2	-150.75	-110.5	11900	10600	28.7	76500	44.8
L_3L_4	-258.0	+72.0	+54.5	11900	9000	28.7	76500	44.8
L_4L_5	-258.0	+72.0	+54.5	11900	9000	28.7	76500	44.8

STRESS SHEET SPAN A







Member	Stress Dead	Stress Live	Stress Impact	Allowable Unit Stress	Stress Unit	Actual Area (sq. in.)	(Length)² / ²	Least Radius Gyr. ²
L_0U_1	-53.47	-222.5	-161.0	11.15	13.9	31.44	145000	31.2
U_1L_2	+31.62	+145.0	+110.8	15.0	25.0	11.25		
U_2L_3	+10.54	+82.5	+67.1	15.0	22.85	7.00		
U_3L_4	0	+41.6	+35.8	15.0	25.6	3.13		
L_0L_1	+34.94	+146.5	+23.2	15.0	15.15	13.50		
L_1L_2	+34.94	+146.5	+23.2	15.0	15.15	13.50		
L_3L_4	+55.90	+219.0	+61.7	15.0	18.7	18.00		
U_1U_2	-55.90	-219.0	-61.7	12.7	13.2	25.44	57600	33.6
U_2U_3	-62.88	-244.0	-68.5	13.4	12.7	29.44	57600	31.9
U_1L_1	+12.35	+84.25	+68.5	15.0	20.4	8.10		
U_2L_2	-11.94	-63.5	-57.6	10.2	13.85	9.60	83000	13.25
U_3L_3	-3.85	-32.0	-27.54	10.2	6.6	9.60	83000	13.25
L_2L_3	+55.90	+219.0	+61.7	15.0	18.67	18.00		

Member	Stress Dead	Stress Live	Stress Impact	Allowable Unit Stress	Stress Unit	Actual Area	$(\text{Length})^2 / r^2$	Least Radius Gyr
U_1L_1	29.4	112.2	81.5	15000	16800	13.3		
U_2L_2	7.3	0	0	15000	600	12.2		
U_3L_3	29.3	77.9	63.0	11000	9700	17.6	88000	18.4
U_4L_4	29.4	112.2	81.5	15000	16800	13.3		

Graphical results from Plate 5

L_0U_1 = 20000

U_1U_2 = 60000

U_2U_3 = 60000

U_3U_4 = 145000

U_4U_5 = 320 000

U_1L_2 = 70000

L_2U_3 = 120000

L_3U_4 = 167000

U_4L_5 = 55000

U_1L_1 = 30000

U_2L_2 = 8000

U_3L_3 = 95000

U_4L_4 = 30000

U_5L_5 = 128000

L_0L_1 = 15000

L_1L_2 = 15000

L_2L_3 = 143000

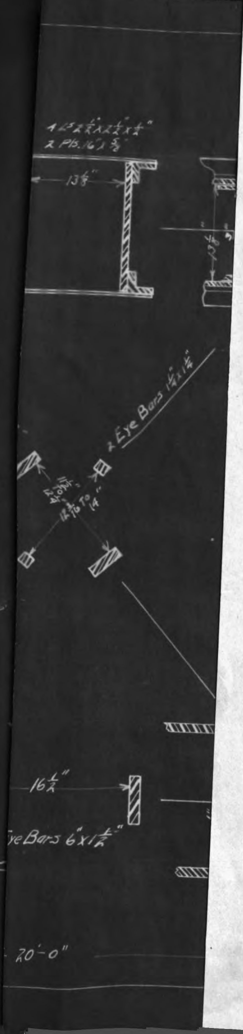
L_3L_4 = 258000

L_4L_5 = 258000

If this was done, the bridge would be in fair condition, as the remainder has a factor of safety of about 4 throughout.

On account of this bridge being on a branch and not on a main line, Cooper's E-50 loading was used, but a bridge of the present time should be safe under E-55 or E-60 loading when there is any possibility of the heaviest engines of the road passing over it.

There is a rule requiring all trains to cross the bridge a speed not greater than six miles per hour. If this rule is lived up to, it may make the life of the bridge much longer by reducing the stress caused by impact.



2 PIs. 16" x 5/8"
13 5/8"
2 Eye Bars 1 1/4" x 1 1/4"
16 1/2"
Eye Bars 6" x 1 1/2"
20'-0"

[redacted]

[redacted]

Pocket
has:
14 Photos

10 post cards

24

SUPPLEME
MATE

128
848
TtS
post
card
10

12
848
Tt
Ph