SCHERTZ

Investigation of
a Highway Bridge

Civil Engineering

B. S.

1907

INVESTIGATION OF A HIGHWAY BRIDGE

BY

JOSEPH WILLIAM SCHERTZ

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

PRESENTED JUNE, 1907

C O L L E G E   O F   E N G I N E E R I N G

April 30, 1907.

This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

JOSEPH WILLIAM SCHERTZ

entitled INVESTIGATION OF A HIGHWAY BRIDGE

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker.

Head of Department of Civil Engineering

CONTENTS.

Page

I. INTRODUCTION. 1

II. WEIGHT OF METAL AND LUMBER. 4

III. INVESTIGATION AS A SIMPLE TRUSS. 10

Art. 1. Dead Load Stresses. 10

Art. 2. Live Load Stresses. 10

Art. 3. Stresses in the Lateral System. 13

Art. 4. Secondary stresses in the Lower Chord. 14

IV. INVESTIGATION AS A TWO-HINGED ARCH. 17

Art. 5. Upper Chord Considered as a Two-Hinged Arch. 17

Art. 6. Effect of Temperature. 23

Art. 7. Efficiency of Upper Chord. 24

Art. 8. Efficiency of Verticals. 26

Art. 9. Efficiency of the Diagonal Rods. 27

Art. 10. Efficiency of the Lower Chord. 28

Art. 11. Investigation of the Floor System. 30

Art. 12. Investigation of the Lower Chord Casting. 31

Art. 13. Investigation of the Lateral Rods. 32

Art. 14. Bearing and Shearing Stress on the Upper Chord Caused by the Vertical Rods. 34

Art. 15. Investigation of the Sway Bracing. 34

Art. 16. Investigation of the Pedestal. 36

Art. 17. Table of Stresses. 37

V. DETERMINATION OF THE SAFE LOAD. 39

VI. CONCLUSION. 41

INTRODUCTION.

The bridge to be investigated is an old highway bridge built in 1867. It is located on the old state road about one mile south of St. Joseph, Illinois, and spans the Salt Fork Creek. The bridge is of the bow-string type, and the material used is wrought iron. There are two spans, one of one-hundred feet and one of eighty feet. The abutments are of sandstone.

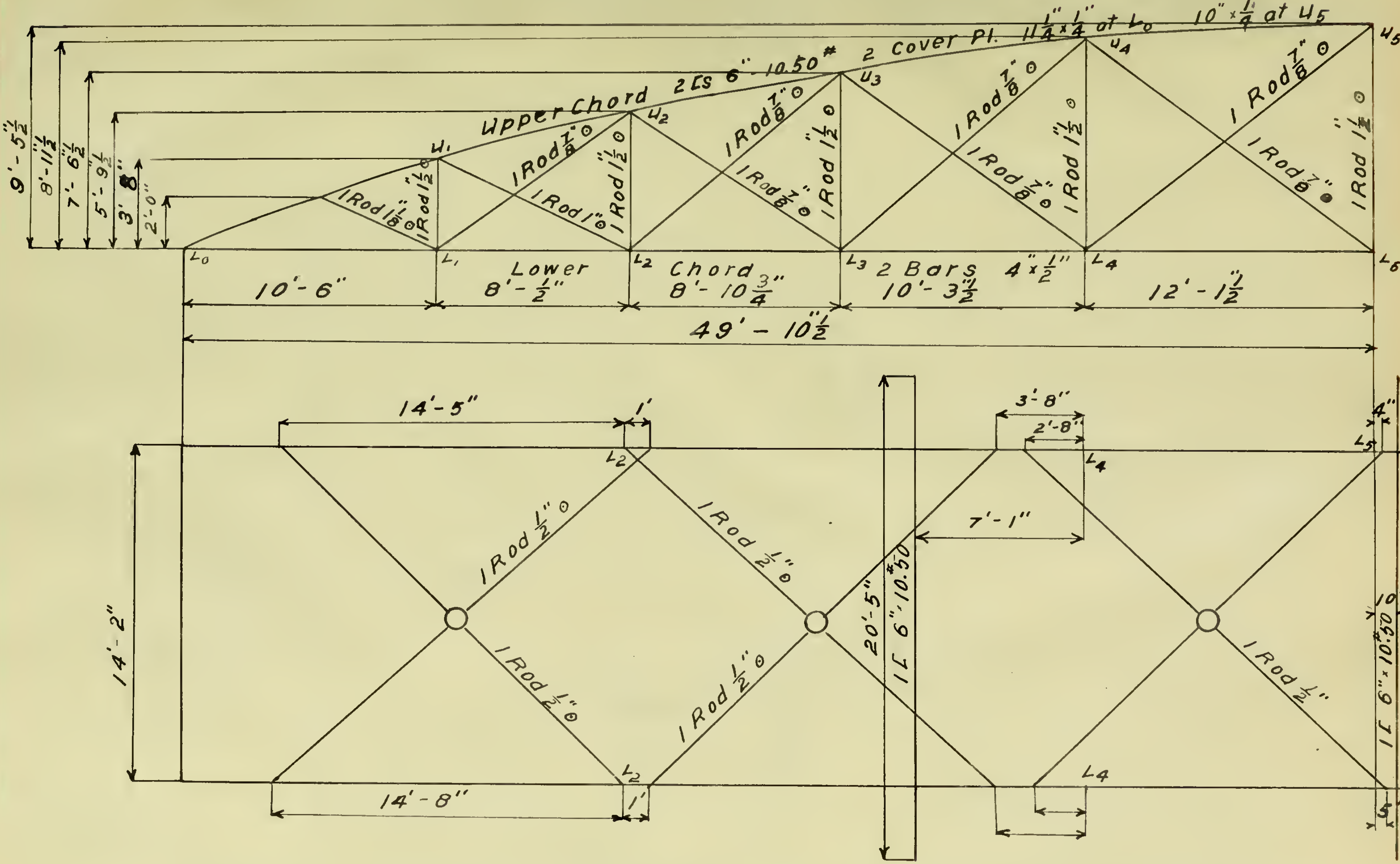
This investigation is to determine the efficiency of the members. From general appearance, an engineer would consider some of the members entirely insufficient to take the stresses which are caused in them. However the fact remains that the bridge still stands and is used for the purposes of all ordinary highway traffic, and also for heavy trac-

tion engines.

The investigation will be limited to the one-hundred foot span. The spans, being similar in structure and many of the members being of the same size, the least efficiency will probably be found in the longer span.

The bridge is investigated according to Cooper's Specifications for Highway Bridges - 1901 - Edition. The class to which this bridge belongs is class D which is required to support 75 pounds per square foot of floor surface for the live load. The general dimensions of the bridge are given in Plate I on page 3.

PLATE I



General Dimensions of Bridge and Composition of Members

WEIGHT OF METAL AND LUMBER.

The weight of the bridge was computed by using 450 pounds per cubic foot of iron, and the lumber was considered weighing 4.5 pounds per board foot. The computations of weights follow and the summary is found in Table II page 8.

TABLE I.

WEIGHT OF METAL.

Ref. No	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Weight Pounds Per. Ft.	Total Weight Main Member	Total Weight Details
1	Top Chord	4					
	[s	2	$6" \times 2" \times \frac{1}{4}"$	51.06	10.50	1075	
	Cover Pl.	2	$10\frac{1}{2}" \times \frac{1}{4}"$	51.06	8.92	913	
	Bat. Pl.	5	$6" \times \frac{1}{4}"$	0.92	5.10		23
	Rivet Hds.	1380	$\frac{1}{2}"$ Rivets	Per 100	5.80		80
	Bat. Pl.	2	$5" \times \frac{3}{8}"$	0.92	6.38		12
	Bat. Pl. (1)	$\frac{1}{4}$	$11" \times \frac{1}{4}"$	3.00	9.35		7
						1988	122
			4 Top Chords			7952	488
2	Lower Ch.	2					
	Bars	2	$4" \times \frac{1}{2}"$	95.00	6.80	1290	
	Upset Ends	4	$1\frac{1}{2}"$ ⊙	2.50	6.01	60	
	Bat. Pl.	12	$4" \times \frac{1}{2}"$	0.92	6.80		77
	Nuts	4	$1\frac{1}{2}"$ Diam.	Per 100	319.0		13
	Rivet Hds.	144	$\frac{3}{4}"$ Diam	Per 100	13.6		20
						1350	110
			2 Lower Chords			2700	220

TABLE I Continued.

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Weight Pounds Per. Ft.	Total Weight Main Member	Total Weight Details
3	Inter. Post	2					
U_5L_5	Bar	1	$1\frac{1}{2}''$ ⊙	10.42	6.01	63	
U_4L_4	Bar	2	$1\frac{1}{2}$ ⊙	9.96	6.01	120	
U_3L_3	Bar	2	$1\frac{1}{2}$ ⊙	7.54	6.01	90	
U_2L_2	Bar	2	$1\frac{1}{2}$ ⊙	6.80	6.01	82	
U_1L_1	Bar	2	$1\frac{1}{2}$ ⊙	4.71	6.01	57	
	Nuts	27		Per 100	319.0		86
	Bar (ring)	9	$\frac{1}{2}'' \times \frac{1}{2}''$	0.52	0.85		4
	Bar (ring)	9	$\frac{1}{2}'' \times \frac{7}{8}''$	0.52	1.49		7
	Clamps	9			20.0		180
						412	277
			2 Inter Posts			824	554
4	Main Ties	4					
L_5U_4	Bar	1	$\frac{3}{4}''$ ⊙	15.16	1.50	23	
	Loop End	1	$\frac{3}{4}''$ ⊙	0.75	1.50	1	
L_4U_3	Bar	1	$\frac{3}{4}''$ ⊙	12.92	1.50	20	
	Loop End	1	$\frac{3}{4}''$ ⊙	0.92	1.50	1	
L_3U_2	Bar	1	$\frac{3}{4}''$ ⊙	10.67	1.50	16	
	Loop End	1	$\frac{3}{4}''$ ⊙	0.75	1.50	1	
L_2U_1	Bar	1	$1''$ ⊙	9.00	2.67	24	
	Loop End	1	$1''$ ⊙	0.85	2.67	2	
	Nuts	3	$\frac{3}{4}''$ Diam.	Per 100	32.0		1
	Nuts	1	$1''$ Diam.	Per 100	68.0		1
	Bar	1	$1\frac{1}{8}''$ ⊙	5.90	3.38	20	
	Nuts	2	$1\frac{1}{8}''$ ⊙	Per 100	103.0		2
						108	4
		4	Main Ties			432	16

TABLE I Continued.

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Weight Pounds Per Ft.	Total Weight Main Members	Total Weight Details
5	Counters	4					
U_6L_4	Bar	1	$\frac{3}{4}$" ⊙	15.42	1.50	23	
	Loop End	1	$\frac{3}{4}$" ⊙	0.75	1.50	1	
U_4L_3	Bar	1	$\frac{3}{4}$" ⊙	13.71	1.50	21	
	Loop End	1	$\frac{3}{4}$" ⊙	0.75	1.50	1	
U_3L_2	Bar	1	$\frac{3}{4}$" ⊙	11.92	1.50	18	
	Loop End	1	$\frac{3}{4}$" ⊙	0.75	1.50	1	
U_2L_1	Bar	1	$\frac{7}{8}$" ⊙	10.04	2.04	21	
	Loop End	1	$\frac{7}{8}$" ⊙	0.85	2.04	2	
	Nuts	3	$\frac{3}{4}$" Diam.	Per 100	32.00		1
	Nuts	1	$\frac{7}{8}$" Diam.	Per 100	63.00		1
		4 Counters				88 332	2 8
6	Floor Beam	1	(Wood)				
	Beam	44	3"×12"	15.5	4.5		
		Given under Weight of Lumber					
7	Joists	1	(Wood)				
	Joist	62.5	3"×4"	16.0	4.5		
		Given under Weight of Lumber					
8	Bottom Laterals	2					
	Bar	1	$\frac{1}{2}$" ⊙	21.06	0.67	14	
	Loop End	2	$\frac{1}{2}$" ⊙	0.35	0.67	1	
	Bar	1	$\frac{1}{2}$" ⊙	20.12	0.67	14	
	Loop End	2	$\frac{1}{2}$" ⊙	0.35	0.67	1	
	Bar	1	$\frac{1}{2}$" ⊙	20.60	0.67	14	
	Loop End	2	$\frac{1}{2}$" ⊙	0.35	0.67	1	
	Bar	1	$\frac{1}{2}$" ⊙	20.80	0.67	14	
	Loop End	2	$\frac{1}{2}$" ⊙	0.35	0.67	1	

TABLE I Continued.

Ref. No	Name of Pieces	No. of Pieces	Cross Section	Length in Feet	Weight Pounds Per Ft.	Total Weight	
						Main Member	Details
	Bar	1	½" ⊙	20.60	0.67	14	
	Loop End	2	½" ⊙	0.35	0.67	1	
	Bar	1	½" ⊙	20.60	0.67	14	
	Loop End	2	½" ⊙	0.35	0.67	1	
	Nuts	12	½ Diam.	Per 100	8.60		1
	Bolts & Nuts	12	½" x 4½"	Per 100	38.4		5
	Rings	12	¼" x 1½"	0.17	1.28		3
	Rings	3	½" x 1½"	2.45	2.55		19
		2 Bottom Laterals				86 172	28 56
9	Sway Bracing 1						
	[s	3	6" x 2" x ¼"	20.42	8.00	492	
	Bars	4	1½" ⊙	7.50	6.01	181	
	Bars (end)	4	1" ⊙	0.67	2.67	7	
	Bars	2	1½" ⊙	9.33	6.01	124	
	Bars (end)	2	1"	0.67	2.67	4	
	Bars (clevis)	6	1¾" x ¼"	1.17	1.49		10
	Rivet Hd.	24	½" rivet	Per 100	5.800		1
	Nuts	12	1" Diam.	Per 100	68.0		8
	Bolts & Nuts	6	½" x 2"	Per 100	25.0		2
		1 Sway Bracing				808	21
10	Pedestal	4					
	Plate	1	10" x ¾"	1.30	25.50	33	
	Plate	1	8" x ¾"	1.30	20.40	27	
	Plate	2	2" x ½"	0.50	3.40	4	
	Plate	2	4" x ¾"	0.83	10.20	17	
		4 Pedestals				81 324	

TABLE I Continued.

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Weight Pounds Per Ft.	Total Weight Main Member	Details
11	Spikes						
	6" spike	940	6" long	No. Per.	Lb. 10		94
	5" spike	2000	5" long	No. Per	Lb. 13		152
							246
12	Flooring	(Wood)					
		100	12"×2½"	14.20	4.5		
		Given under		Weight of		Lumber	
	Total Weight of Metal						
1						7952	488
2						2700	220
3						824	554
4						432	16
5						332	8
8						162	56
9						808	21
10							324
11							246
						13210	1933
			Total				15143

TABLE II.
WEIGHT OF LUMBER.

Ref. No.	Name of Piece	No. of Pieces	Cross Section	Length in Feet	Feet B. M.	Lb. Per Foot B. M.	Weight in Pounds
6	Floor Beam	50	3"×12"	15.5	2325	4.5	10500
7	Joists	75	3"×4"	16.0	1200	4.5	5400
8	Flooring	100	2½"×12"	14.20	3550	4.5	159800
9	Sway Bracing Plank	3	2"×10"	20.0	100	4.5	4.50
				Total			32300

TABLE III.
SUMMARY OF WEIGHTS.

Ref. No.	Name of Piece	Weight Pounds	Total Weight Pounds
	Metal		
1	Top Chord	8440	
2	Lower Chord	2920	
3	Inter. Post	1378	
4	Main Ties	448	
5	Counters	340	
8	Bottom Lat.	218	
9	Sway Bracing	829	
10	Pedestal	324	
11	Spikes	246	
	Total Metal		15 143
	Lumber.		
6	Floor Beam	10 500	
7	Joists	5 400	
12	Flooring	15 980	
9	Sway Bracing	450	
	Total Lumber		32 300
Total Weight of Bridge			47 473

The weight per linear foot of bridge is 231 pounds.

INVESTIGATION AS A SIMPLE TRUSS.

Art. 1. Dead Load Stresses.

The weight of the bridge was considered as being distributed over the entire span, one-third being considered at the upper panel points.

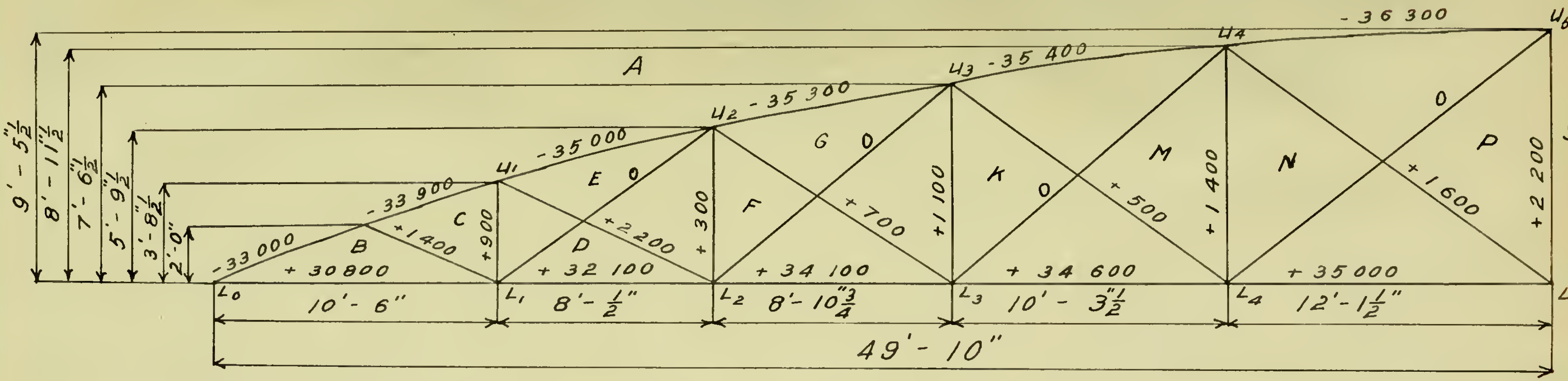
The dead load used was taken as 231 pounds per linear foot of truss, which was the value determined by the computation of weights. The dead load stresses in the members were found by graphic resolution and the results are given in Plate II, page 11.

Art. 2. Live Load Stresses.

The stresses were computed for a uniform live load of 75 pounds per square foot of floor surface. The trusses were considered as simple trusses.

The chord stresses were computed graphically and the web members by algebraic moments. The results are given on Plate III, page 12.

PLATE II

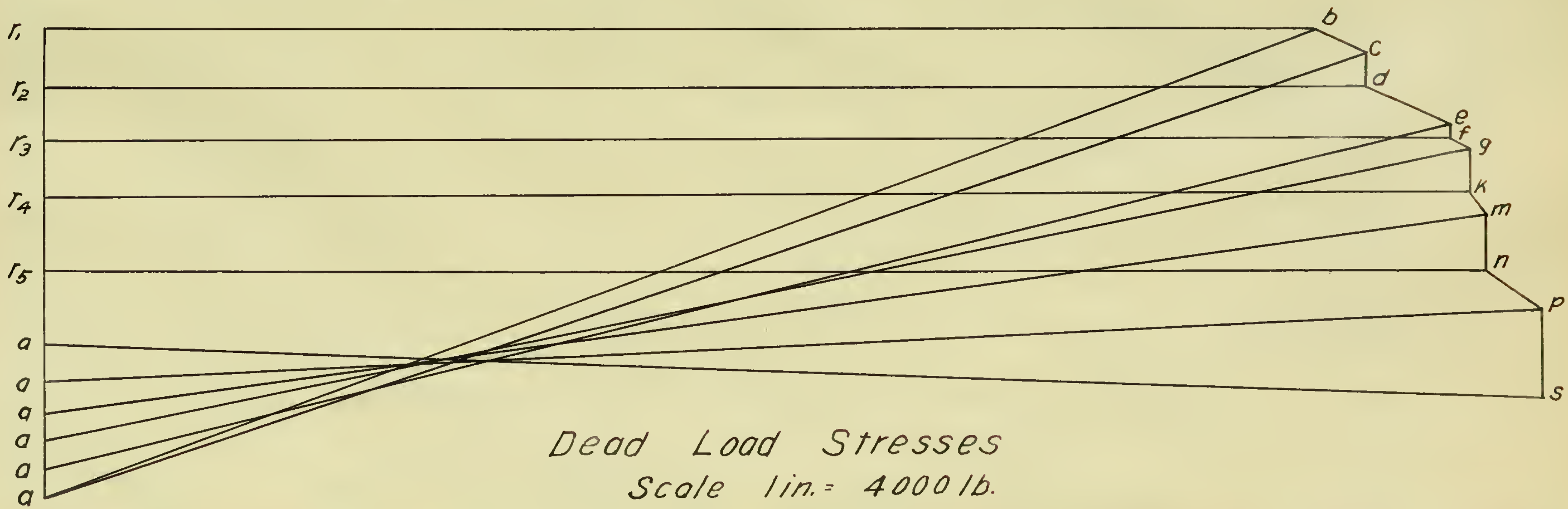


Panel Loads in Kips

$L_1 = 2.14$ $L_3 = 2.21$ $L_5 = 2.80$

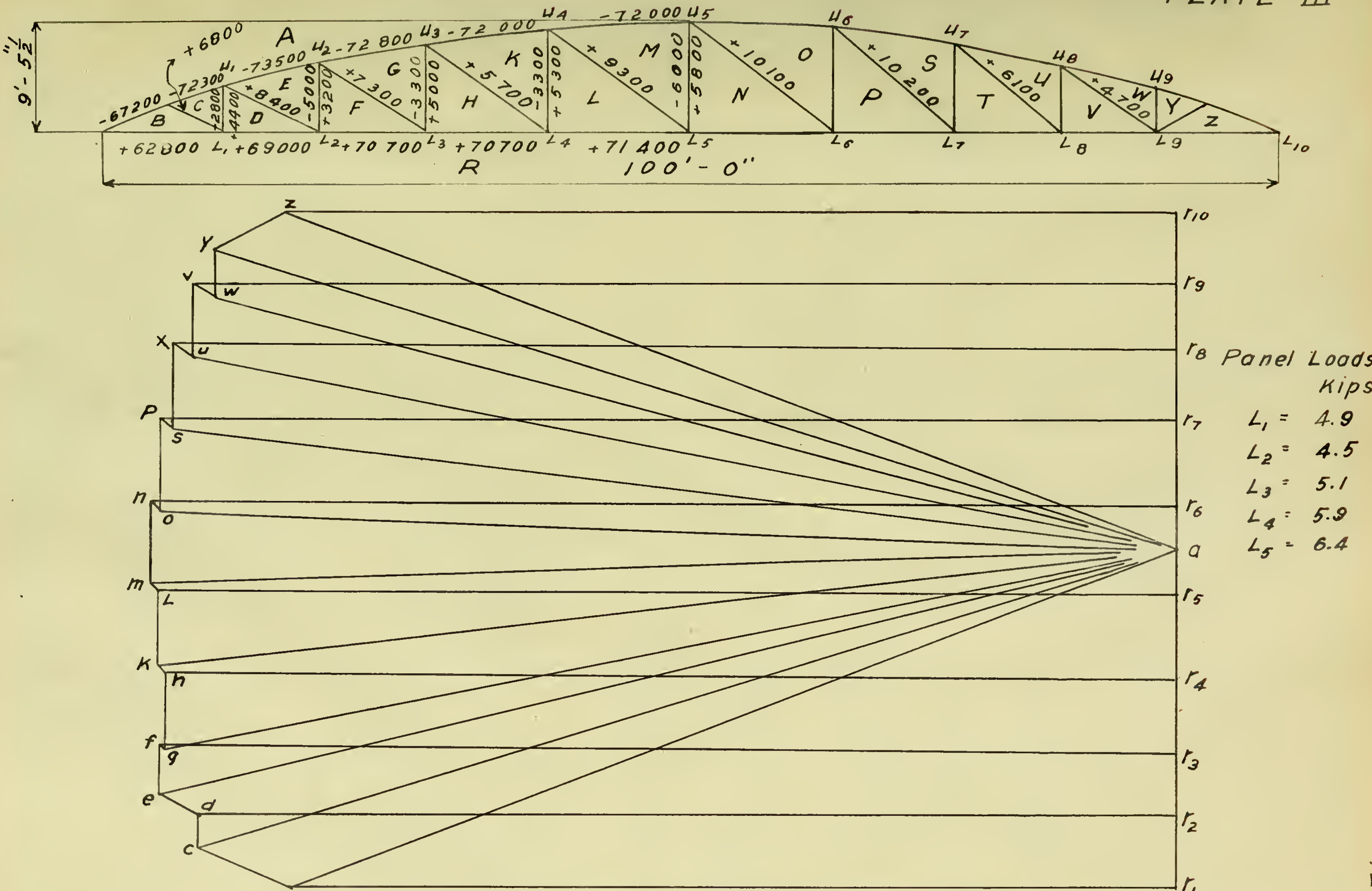
$L_2 = 1.96$ $L_4 = 2.50$

$\frac{1}{3}$ of Load on Upper Chord



Dead Load Stresses
Scale 1in. = 4000 lb.

PLATE III



Stresses Due to Uniform Live Load

Scale 1 in. = 10000 lb.

Art. 3. Stresses in the Lateral System.

The stresses will be computed for a dead load of 150 lb. per lin. ft. of span and a moving load of 150 lb. per lin. ft. of span. The lateral system consists of rods fastened to the lower chord.

There are no cross-struts, but the wooden beams of the floor system are notched over the lower chord. They are not notched very deep, but several beams near the lateral connections, probably act as struts.

The lateral rods do not join the lower chord at the panel points, but as shown on Plate I, page 3. The lateral system does not extend entirely to the ends of the span. Assume one half of the wind load to come upon each truss. The value of the stresses is given on top of page 14, in Fig. 1.

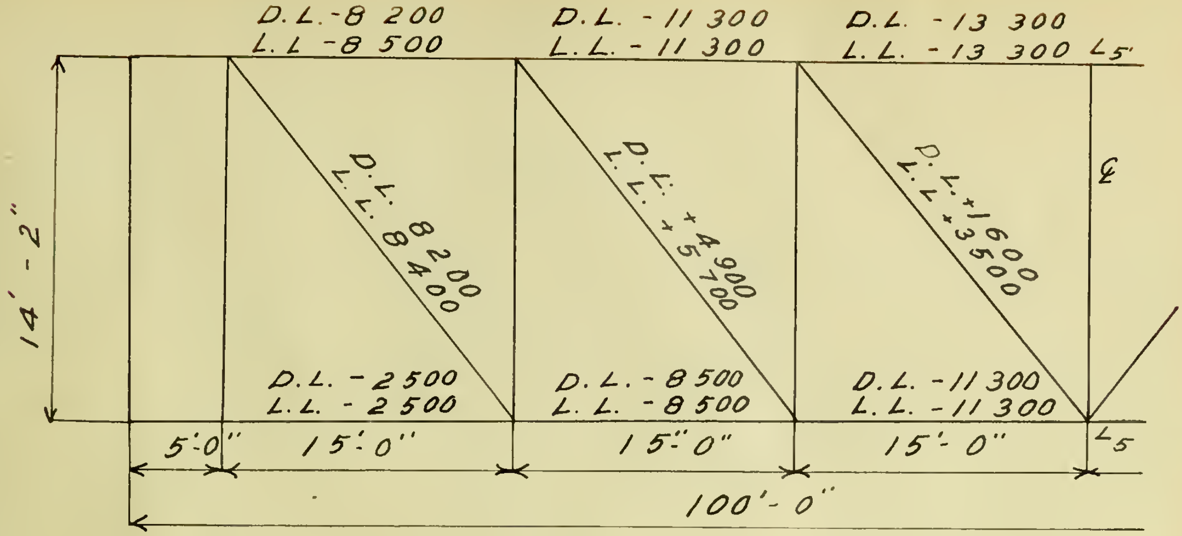


Fig. 1.

Art. 4. Secondary Stresses in the Lower Chord.

The floor beam rests directly upon the lower chord, the spacing being about two feet. This causes a secondary stress in the lower chord. The wind load will also cause a secondary stress, the direction of the wind forces however being at right angle to that of the dead and live loads.

Let S_1 be the unit stress caused by the dead and live loads resting upon the lower chord, and let S_2 be the direct wind stress. The lower chord has a direct ten-

sion, which will be called P.

The moment at any section is equal to $M_1 - Py$ where M_1 = moment of uniform load and y is the deflection of the beam. The deflection in terms of the unit stress and other functions is given in text books of applied mechanics and is $Ky = \frac{S_1 l^2}{Ec}$ where

K = a constant depending upon ends of beam,

E = the modulus of elasticity.

c = the distance from the neutral axis to the remotest fiber, and

l = the length of the beam.

Place this value of y in equation for total moment gives,

$$M = M_1 - P\left(\frac{S_1 l^2}{KEc}\right)$$

$$M = \frac{S_1 I}{c} = M_1 - P\left(\frac{S_1 l^2}{KEc}\right).$$

Solving for S_1 in the above equation gives $S_1 = \frac{M_1 c}{I + \frac{Pl^2}{KE}}$, and in a similar manner the value of $S_2 = \frac{M_1 c}{I + \frac{Pl^2}{KE}}$, where the values of I, c, and M_1 differ from their values in S_1. S_1 and S_2, act at right angles to each other so the total direct unit stress will

be equal to $\sqrt{S_1 + S_2}$.

The maximum stress S_1 will not be at the maximum S_2, because the lateral rods do not join the lower chord at the panel points. But S_1 is much larger than S_2 and the $\sqrt{S_1 + S_2}$ will be a maximum when S_1 is a maximum.

The maximum secondary stresses will occur either at the middle of the truss, due to live load or near the end, where there are no lateral rods, due to wind load. Therefore L_0L_1 and L_4L_5 will be investigated for secondary stresses. Table IV below gives the value of these stresses.

TABLE IV.

Secondary Stresses in Lower Chord.

	Load Per Lin. In.	M_1 In. Lb.	C In.	I	P	l In.	K	E	$\frac{M_1 c}{1+\frac{Pl^2}{KE}}$	Total Unit Stress
L_4L_5 S_1	63.5	164 800	2	5.30	106 400	144	10	25 000 000	23 200	23 300
L_4L_5 S_2	12.5	32 400	$\frac{1}{4}$	0.04	53 200	144	10	25 000 000	2 200	
L_0L_1 S_1	63.5	126 000	2	5.30	93 600	126	10	25 000 000	24 800	44 400
L_0L_1 S_2	12.5	42 700	$\frac{1}{4}$	0.04	46 800	126	10	25 000 000	37 000	

INVESTIGATION AS A TWO HINGED ARCH.

Art. 5. Upper Chord Considerad as a Two Hinged Arch.

Now it will be considered that the top chord acts as a two hinged arch. The curve of the chord is very nearly a true parabola and will be considered as such.

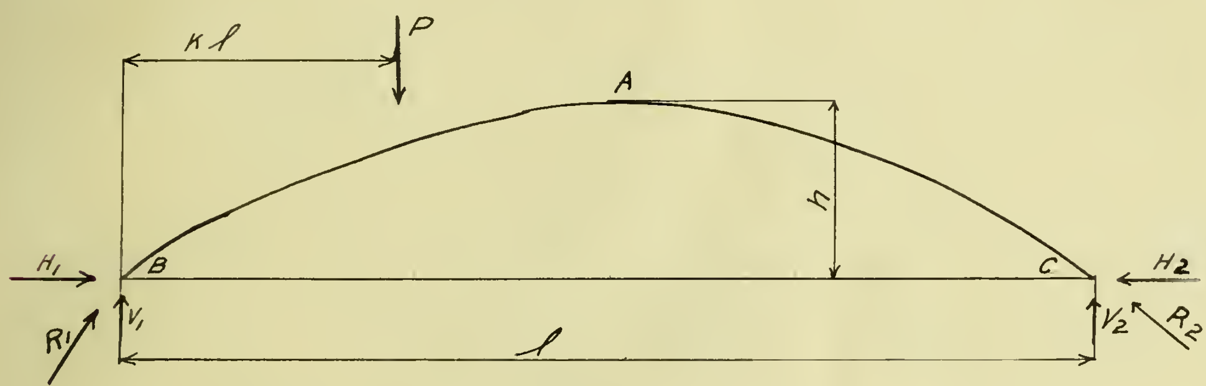


Fig. 2.

A parabola with its origin at A in Fig. 2 would have equation $x^2 = my$ (1). From $x = \frac{l}{2}$ when $y = -h$, we get equation $x^2 = -\frac{l^2}{4h} y$. Transforming the origin to point B, the left reaction, we have $x = x' - \frac{l}{2}$ and $y = y' - h$ which gives equation $y = 4h\left(\frac{x}{l} - \frac{x^2}{l^2}\right)$ (2) as the new equation for the parabola.

Take a load P at some point kl from the left reaction. At the

left reaction the force R_1 will act. This can be replaced by H_1 and V_1. The value of V_1 can be found by taking moments about C. this gives $V_1 = P(1-K)$.

It will now be necessary to find the value of H_1.

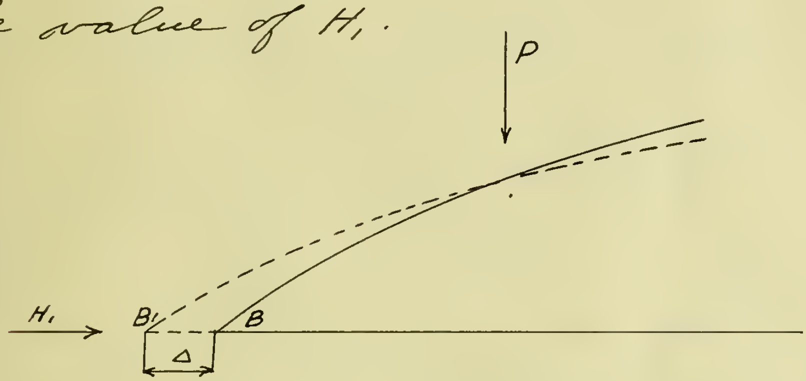


Fig. 3.

In Fig. 3, if there were no H_1, the load at P would cause the arch to move out to B', the motion caused by P being B'B or Δ. Now apply a force H_1 which will move the arch back to its original position, a distance of BB' or Δ_1. The values of Δ and Δ_1 are equal and will be determined.

Apply a horizontal force of unity at B, then the external work done will be $\frac{1}{2}\Delta$, and the internal work will be equal to it. Take a length ds in Fig. 4. The load P will cause it

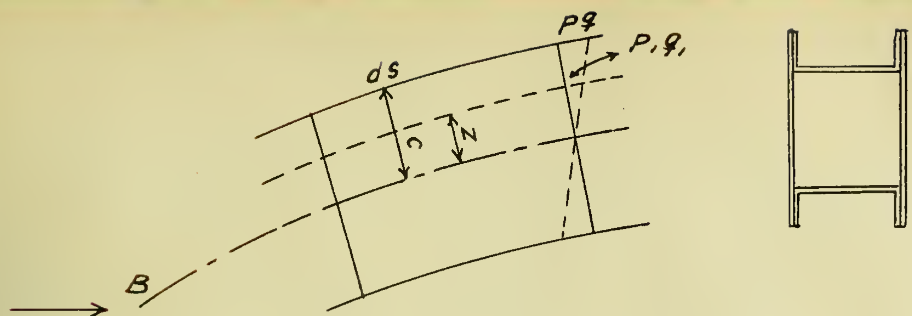


Fig. 4.

to elongate at the remotest fiber to pq, and $pq = \frac{ds\,S}{E}$, E being the modulus of elasticity, and S the unit stress. S will equal $\frac{M_1 c}{I}$, where M_1 is the bending moment caused by a vertical load, I the moment of inertia and c is the distance from the neutral axis to the remotest fiber. Substituting S in pq gives $pq = \frac{M_1 c\,ds}{EI}$ for any fiber distance z from the neutral axis, the elongation will be equal to $\frac{z M_1 ds}{EI}$. Let m equal the moment of the horizontal force of unity at B. The fiber stress at z is $\frac{maz}{I}$, where a is the area of the fiber, and the internal work on a fiber at z is $\frac{1}{2} \times \frac{M_1 z\,ds}{IE} \times \frac{maz}{I} = \frac{M_1 m a z^2 ds}{2EI^2}$. A summation of the internal work over the entire area gives $\int \frac{M_1 m\,ds}{2EI}$, and

placing this equeal to the external work gives $\Delta = \int \frac{M_1 m\, ds}{EI}$ (3).

In a similar manner Δ_1 is found; the external work is $\frac{1}{2}\Delta_1$, M'' the moment caused by H is $-Hm$. A fiber distance z from the neutral axis will have a stress of $\frac{M'' a z}{I}$, and will have an elongation equal to $\frac{M'' z\, ds}{IE}$, the internal work will be $\frac{1}{2} \times \frac{M'' z\, ds}{EI} \times \frac{M'' a z}{I} = \frac{1}{2}\frac{M''^2 a z^2 ds}{EI^2}$, and the internal work over the entire area will be $\int \frac{M''^2 ds}{2EI}$. Substituting the value of M'' and equating the internal work equal to the external work gives $\Delta_1 = H_1 \int \frac{m^2 ds}{EI}$ (4). Placing equation (3) equal to (4) gives

$$H_1 = \frac{\int \frac{M' m\, ds}{EI}}{\int \frac{m^2 ds}{EI}} \quad (5).$$

Apply formula (5) to the parabolic rib. $M' = P(1-K)x$ on left of load and $M' = P(1-K)x - P(x - Kl)$ on the right of the load, $m = y$, $ds = \frac{dx}{\cos\phi} = dx \sec\phi$ where ϕ = the slope of the tangent. Assume I to vary as the secant of ϕ. Substituting these values in formula (5) gives,

$$H_1 = \frac{\int_0^{kl} P(1-k)xy\,dx + \int_{kl}^{l}[P(1-k)x - P(x-kl)]\,y\,dx}{\int_0^l y^2\,dx}, \text{ and}$$

substituting values of y from equation (2) integrating gives

$$H_1 = \frac{5Pl}{8h}(k - 2k^3 + k^4),$$ which is the horizontal thrust of a parabolic arch rib under a load P at a distance kl from the left reaction.

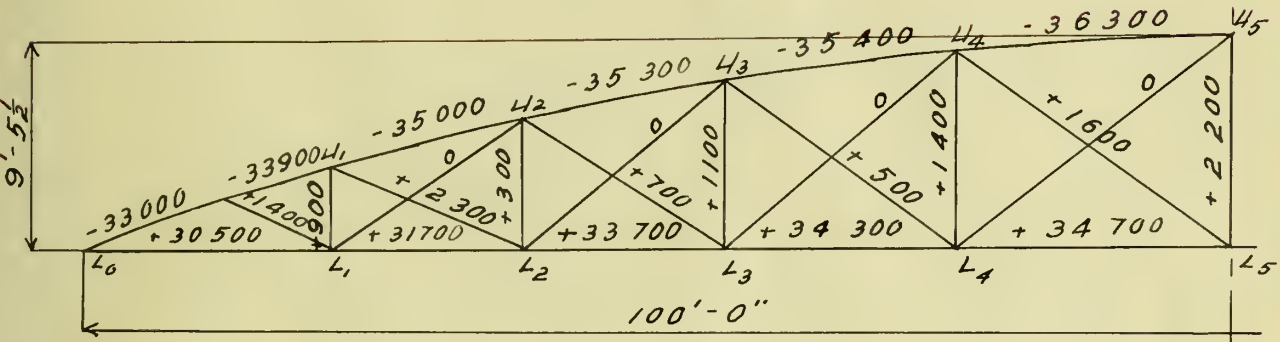
Table V gives the values of the horizontal thrust for panel loads at the various points as indicated.

TABLE V.

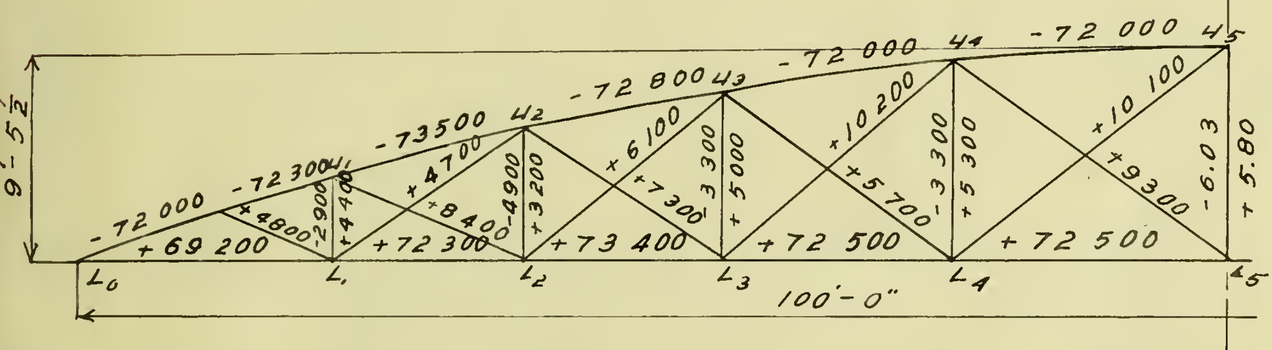
Value of Horizontal Thrust.

Panel Loads at	Panel Load Kips	Value of K	Horizontal Thrust Dead Load Kips	Horizontal Thrust Live Load Kips
L_1	2.14	0.105	1.46	3.33
L_2	1.96	0.185	2.25	5.21
L_3	2.21	0.274	3.51	8.13
L_4	2.59	0.377	4.96	11.30
L_5	2.80	0.500	5.81	13.22
L_6	2.59	0.621	4.96	11.30
L_7	2.21	0.727	3.51	8.13
L_8	1.96	0.816	2.25	5.21
L_9	2.14	0.895	1.46	3.33
Total Thrust			30.19	69.16

The stresses in the truss, when considered as a two hinged arch, are given on Plate IV, page 22.



Dead Load Stresses



Live Load Stresses

Stresses In Main Truss When Considered
As
A Two-Hinged Arch

Art. 6. Effect of Temperature.

The ends of the two-hinged arch are supposed to be fixed. Any contraction or expansion of the metal caused by a change in temperature will stress the truss. This will not change the value of the vertical reaction, but causes a change in the horizontal thrust. If the truss were free to move, then the temperature change would cause the truss to move a distance of Δ. From page 20, $\Delta = H\int\frac{m^2 ds}{EI}$, and $\Delta = \epsilon t l$, where ϵ = the coefficient of expansion, t = the temperature variation, and l = the length of span. Substituting this value of Δ in the above equation gives $\epsilon t l = H\int\frac{m^2 ds}{EI}$. Substitute the following values for members in the above equation, $m = y$, being the moment due to a horizontal force of unity, $ds = dx \sec\phi$, and $I = I_c \sec\phi$ being the moment of inertia of the upper chord at the center of the truss,

gives $e t l = \frac{H}{EI_c} \int_0^l y^2 dx$. Integrating and transposing, $H = \frac{15 EI e t}{8 h^2}$. Solving this formula for H gives a horizontal thrust of less than one kip for the case in hand. In this derivation the ends were considered fixed, but if they are free to move, the lower chord will also change its length and the stress will be smaller than that computed. The temperature stress being so small can be neglected.

Art. 7. Efficiency of Upper Chord.

The upper chord has the greatest stress at the middle. At that point the section is the smallest; and therefore the efficiency of $L_4 L_5$ will be determined. The total stress in the top chord is 108000 lb. The section contains 11.2 sq. in. The stress in lb. per sq. in. is $\frac{108000}{11.2} = 9650$. The least radius of gyration is about the axis a-a and is

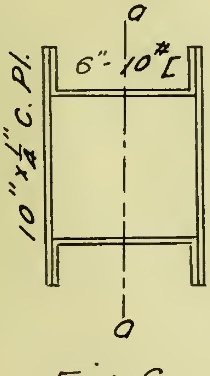


Fig. 6.

equal to 2.81 in. The required area is 9.81 sq. in. and the allowed unit stress is 11000 lb. per sq. in.

The splice plates in the upper chord are 10" × $\frac{1}{4}$", and have only three five-eighths inch rivets. If the splice plates carried all the stress in the cover plate, each rivet would be required to carry 7000 lb. which is over twice the allowed stress. The stresses in the rivets of the channel splice-plates are about the same as in the splices of the cover-plate. The splices in the channel and the cover-plate do not occur at the same points. The required area is 9.81 sq. in. and the effective area, due to the rivets, is only 9.79 sq. in. This gives an efficiency of $\frac{9.79}{9.81} = 0.99$.

Art. 8. Efficiency of Verticals.

The vertical and diagonal rods and their connections at the upper and lower chords are shown in Fig. 7. At the lower

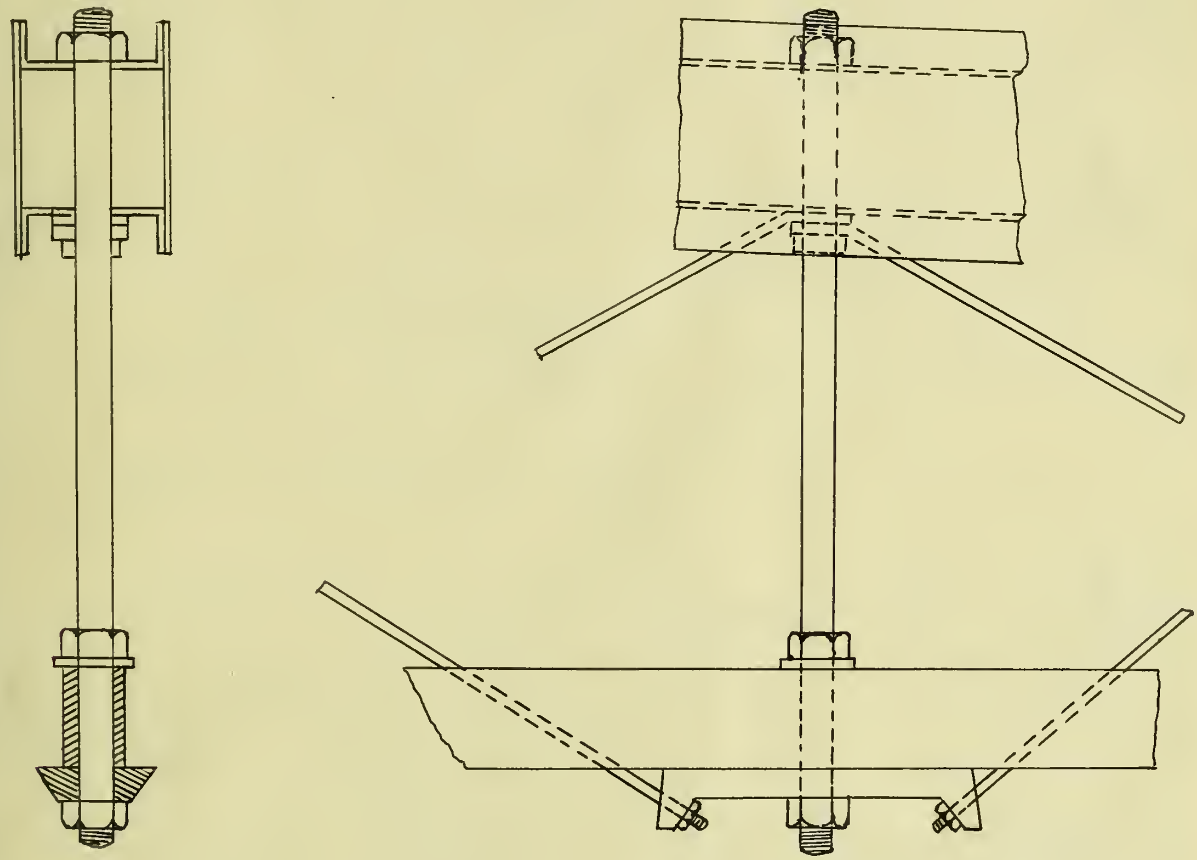


Fig. 7.

chord the vertical is held in place by two nuts, one above the lower chord one below the clamp which sets against the lower chord. At the upper connection there is a nut above, and below there is a ring under

the diagonal rods. The vertical rods are subject to both tensile and compression stresses. The rod at the center having the largest stress, will be investigated. The vertical rods are not designed to resist compression, the ratio $\frac{l}{r}$ being very large. The rod has an area of 1.77 sq. in. and the largest area required for the tensile stress is 0.55 sq. in. This gives the rod an efficiency of $\frac{1.77}{0.55} = 3.21$

Art. 9. Efficiency of the Diagonal Rods.

The diagonal rods are connected as shown in Fig. 7, page 26. The smallest rods and the largest stresses occur near the center of the bridge. Under full load the rods in the center panel would have a unit stress of 24 000 lb. per sq. in. The upper connection of the diagonal rod, to the vertical rod

at the upper chord joint, is a loop which goes around the vertical rod, and a one-half inch ring keeps the diagonal from sliding down the vertical rod. The shear due to this downward pressure is 2 300 lb. per sq. in. The actually area is 0.44 sq. in and the greatest required area is 0.81 sq. in.; therefore the rod has an efficiency of $\frac{0.44}{0.81} = 0.54$.

Art. 10. The Efficiency of the Lower Chord.

The lower chord consists of two bars $4'' \times \frac{1}{2}''$. Near the end these bars are upset to $1\frac{1}{2}''$ rounds. The net section of one bar, owing to one rivet being through the section, is 1.63 sq. in. and the section of the upset end is 1.77 sq. in..

At L_0L_1 the lower chord under full load is subject to a direct stress of 93,600 lb. or a unit stress of 25,400 lb. per. sq. in.. In addition

to this there is a stress due to the fact that the floor system rests direct upon the lower chord. From Table IV this stress is found to be 44,000 lb. per. sq. in., making a total of 69,000 lb. per. sq. in. in. $L_0 L_1$.

Near the middle of the bridge at $L_4 L_5$, the lower chord has a tensile stress of 106,400 lb. which is equal to a unit stress of 29,000 lb. per. sq. in. The unit stress due to direct loading is 23,300 lb. per. sq. in., which makes the total unit stress 52,000 lb. per. sq. in. The lower chord is spliced by two splice plates. There are four rivets in double shear to take the stress. Each rivet is required to take a stress of about 12,000 lb. which is double the amount allowed.

At $L_0 L_1$ the actual stress is 69,000 lb. per. sq. in. and the allowable stress is 14,500 lb. per. sq. in. and at

L_4L_5 the actual unit stress is 52,500 lb. per sq. in. and the allowable is 15,000 lb. per sq. in.. The respective efficiencies are, at L_0L_1 $\frac{14500}{69000} = 0.20$, and at L_4L_5 $\frac{15000}{52000} = 0.29$.

Art. 11. Investigation of the Floor System.

The floor system is shown in Fig. 8. The 12" x 3" beams which

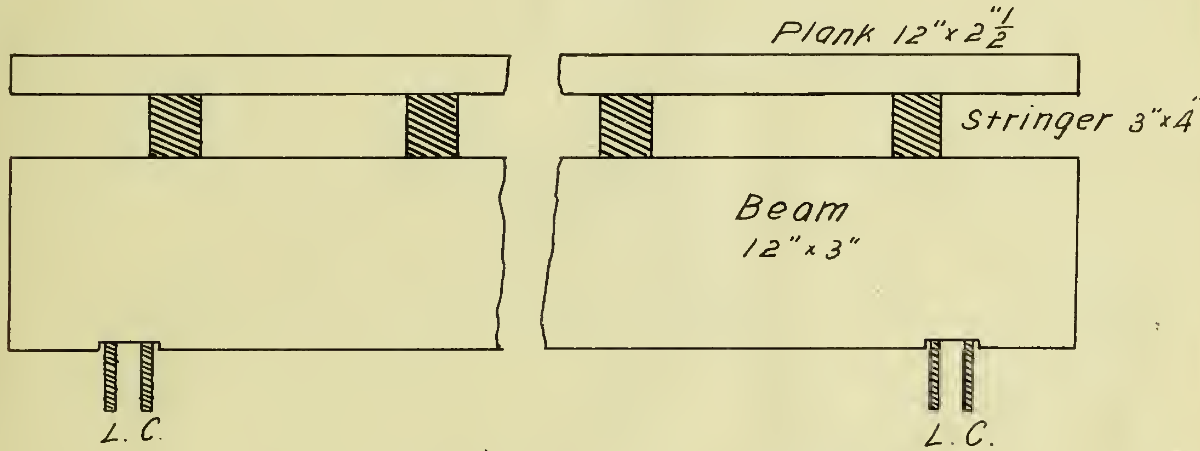


Fig. 8.

are notched over the lower chord, are spaced about two feet apart. On these are placed 3" x 4" stringers, which range from ten to fourteen in number. They are not equally spaced along the beams. Some of the stringers are in poor condition, and are not

of much value.

The floor beams and stringers under a live load of 75 lb. per sq. ft. of surface have a unit stress of 1,050 lb and 300 lb. respectively. This is a safe stress, as a wooden beam should take a stress of 1,000 lb. per sq. in. A traction engine weighing 16,000 lb. with 10,000 lb. on the rear axle causes a unit stress of 2,700 lb. per sq. in. in the beam and 2500 lb. per. sq. in. in the stringer. Under either loading the unit shearing stress is less than 600 lb. per sq. in. and this is entirely safe. The efficiency, for a live load of 75 lb. per sq. ft. of floor surface is $\frac{1000}{1050} = 0.95$, and for the traction engine is $\frac{1000}{2700} = 0.37$.

Art. 12. Investigation of the Lower Chord Casting.

The connection of the lower chord with vertical and diagonal rods is made by means of a casting. Fig. 9 gives a detail of the cast-

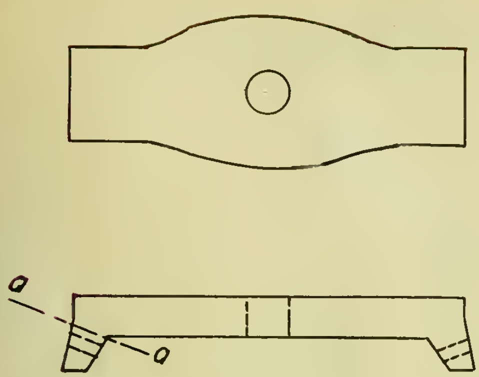


Fig. 9.

ing. The manner of making the connections is shown in Fig. 7, page 26. The weakest part of this casting is at the section a-a, caused by the tension stress in in the diagonal rods. The maximum stress in the diagonal is 10,000 lb., which will produce a moment of 10,000 lb. in. at section a-a. The moment of inertia at this section is about 0.5. The tensile stress in the cast iron is about 12,000 lb. per sq. in., which is high for cast iron. Using an allowable unit stress of 2,500 lb. per sq. in. gives an efficiency of $\frac{2,500}{12,000} = 0.21$.

Art. 13. Investigation of the Lateral Rods.

The lateral rods consist of one-half inch rods. Fig. 10. shows the connections at the end of the rods. The figure on the left

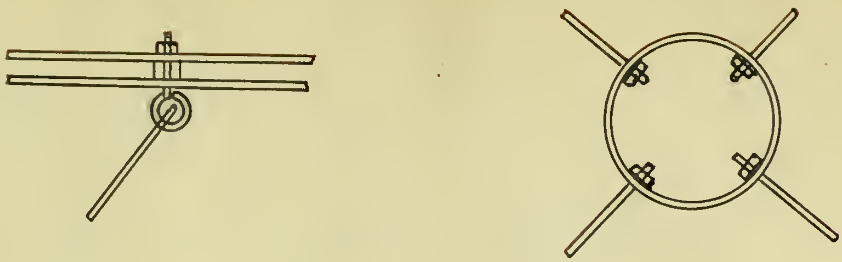


Fig. 10.

shows the lower chord connections, and the one on the right shows the manner in which the intersecting diagonals are joined at the center. The diameter of the iron ring is about one foot, and the cross-section is $\frac{1}{2}'' \times 1''$. The stress in the rod nearest the end is nearly five times the allowed stress. The shear on the one-half inch bolt is nearly 12,000 lb, and the allowed shear is only 3,000 lb. There will also be a bending stress in the bolt at the lower chord connections. The area of the rod is 0.20 sq. in. and the required area is 0.92 sq. in.; therefor the efficiency is $\frac{0.20}{0.92} = 0.22$

Art. 14. Bearing and Shearing Stresses on Upper Chord Caused by Vertical Rods.

The vertical rod causes a shearing in the channel of the upper chord. The maximum stress is 8,000 lb., and there is 3.0 sq. in. of meatal to resist the shearing. This gives a unit stress of 2,700 lb. per sq. in. The efficiency is $\frac{3,000}{2,700} = 1.10$

The horizontal component of the stress in the diagonal, will cause a bearing stress, which will be taken up by the lower channel of the upper chord. The stress will be equeal to 17,000 lb. per sq. in., and the efficiency is $\frac{18,000}{17000} = 1.05$

Art. 15. Investigation of the Sway Bracing.

The sway bracing consists of a channel, which is connected with the upper chord by means of one one-half inch bolt. The channel has a plank bolted to its web, but it is in poor condition, in

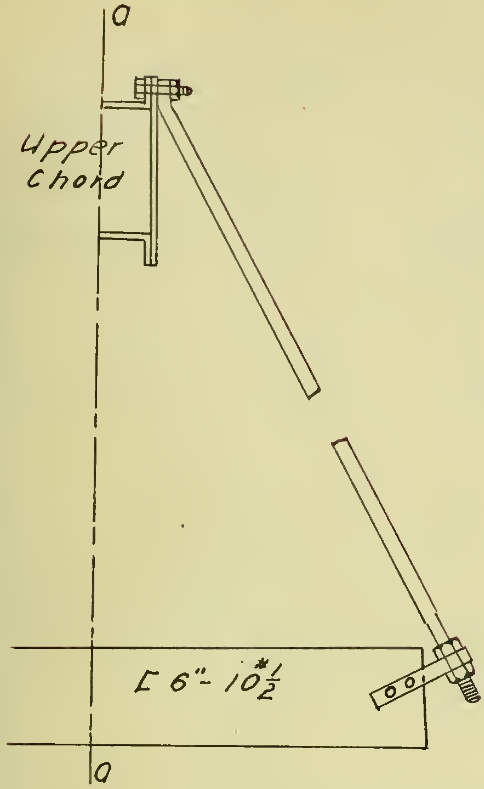


Fig. 11.

some places the plank is entirely loose from the channel.

There are three of these braces on the bridge. A wind stress of 40 lb. per sq. ft. on the upper chord, would cause a stress of 1300 lb. on the bolt which joins the diagonal rod of the bracing to the upper chord. This gives a stress of 3,900 lbs. in the diagonal rod. Take moments about section a-a, of the channel, in Fig. 11. The required value of $\frac{I}{c}$ is found to be 7.2, the actual value of $\frac{I}{c}$ is 5.0. The shear in the rivets which fasten the diagonal rod to the channel is 10,000 lb., and the allowed shearing value is 4700 for each rivet. This gives an efficiency of $\frac{47,000}{10,000} = 0.47$.

Art. 16. Investigation of Pedestal.

The end pedestals consist of a casting, which rests upon the masonry. The upper and lower chord fit into the casting as shown in Fig. 12. The left drawing being a

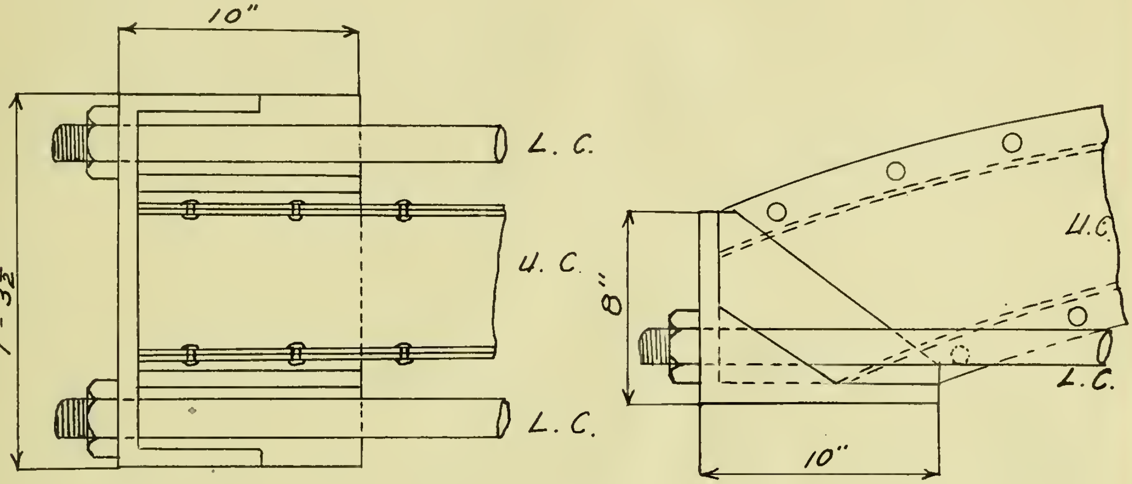


Fig. 12.

plan and the right one is a side elevation. The bottom of the casting which rests upon the masonry has an area of 180 sq. in. The maximum unit stress on the masonry is 200 lb. per sq. in. which is less than the allowed stress. All the metal in the pedestal is three-fourths inch thick.

The stress from the upper

chord is transfered to the pedestal by the upper chord resting against the pedestal. If the entire abutting end of the top chord rests against the pedestal, the bearing stress will not be more than 7,000 lb. per. sq. in. but very likely this is not true and the stress is much higher.

The bearing and shearing stress causes by the lower chord is not greater than 7,000 lb. per. sq. in. The shearing stress should not be greater than 2,500 lb. per sq. in. This gives an efficiency of $\frac{2,500}{7000} = 0.38$

Art 17. Table of Stresses.

On page 42, Table VI gives the dead and live load stresses for the members of the truss when considered as a simple truss and as a two-hinged arch.

TABLE VI.
Stresses For Live Load of 75 Lb. Per Sq. Ft.

Member	Dead Load: Simple Truss lb.	Dead Load: Two-Hinged Arch lb.	Live Load: Simple Truss lb.	Live Load: Two-Hinged Arch lb.	Secondary Stresses lb. per sq. in.	Unit Stress lb. per sq. in: Simple Truss	Unit Stress lb. per sq. in: Two-Hinged Arch	Efficiency: Simple Truss	Efficiency: Two Hinged Arch
L_0U_1	-33,000	-33,000	-72,300	-72,300		-10,200	-10,200	1.08	1.08
U_1U_2	-35,000	-35,000	-73,500	-73,500		-10,500	-10,500	1.04	1.04
U_2U_3	-35,300	-35,300	-72,800	-72,800		-10,600	-10,600	1.03	1.03
U_3U_4	-35,400	-35,400	-72,000	-72 000		-10.900	-10.900	1.01	1.01
U_4U_5	-36,300	-36,300	-72,000	-72,000		-11,500	-11,500	0.99	0.99
U_1L_1	+ 900	+ 900	- 2,800 + 4,400	- 2,800 + 4,400		- 1,070 + 3,000	- 1,070 + 3,000	7.90 4.80	7.90 4.80
U_2L_2	+ 300	+ 300	- 4,970 + 3,200	- 4,970 + 3,200		- 2,600 + 2,000	- 2,600 + 2,000	0.00 6.60	0.00 6.60
U_3L_3	+ 1,100	+ 1,100	- 3,300 + 5,000	- 3,300 + 5,000		- 1,300 + 3,800	- 1,300 + 3,800	0.00 4.01	0.00 4.01
U_4L_4	+ 1,400	+ 1.400	- 3,300 + 5,300	- 3,300 + 5,300		- 1,100 + 3,800	- 1,100 + 3,800	0.00 3.70	0.00 3.70
U_5L_5	+ 2200	+ 2.200	- 6,030 + 5,800	- 6,030 + 5,800		- 2,100 + 4,500	- 2,100 + 4,500	0.00 3.21	0.00 3.21
U_1L_2	+ 2,200	+ 2,200	+ 8.400	+ 8.400		+ 13,400	+ 13,400	1.04	1.04
L_1U_2			+ 4,700	+ 4,700		+ 7,700	+ 7 700	1.62	1.62
U_2L_3	+ 700	+ 700	+ 7,300	+ 7,300		+ 18,200	+ 18,200	0.72	0.72
L_2U_3			+ 6,100	+ 6,100		+ 13,800	+ 13,800	0.91	0.91
L_3U_4			+ 10,200	+ 10,200		+ 23,100	+ 23.100	0.55	0.55
U_3L_4	+ 500	+ 500	+ 5,700	+ 5.700		+ 14,000	+ 14,000	0.90	0.90
L_4U_5			+ 10,100	+ 10,100		+ 23,000	+ 23,000	0.55	0.55
L_5U_4	+ 1,600	+ 1600	+ 9,300	+ 9,300		+ 23,400	+ 23,400	0.54	0.54
$m.L_1$	+ 1.400	+ 1400	+ 6,800	+ 6,800		+ 8,600	+ 8,600	1.57	1.57
L'_0L_1	+ 30,800	+ 30500	+ 62,800	+ 69,000	+ 44,000	+ 69,000	+ 71,000	0.21	0.20
L_1L_2	+ 32,100	+ 31,700	+ 69,000	+ 72,300	+ 16,300	+ 44,000	+ 44,600	0.34	0.31
L_2L_3	+ 34,100	+ 33,700	+ 70,700	+ 73,400	+ 17,700	+ 46,500	+ 47,300	0.34	0.33
L_3L_4	+ 34,600	+ 34,300	+ 70,700	+ 72,500	+ 20,900	+ 50,000	+ 50,300	0.30	0.30
L_4L_5	+ 35,000	+ 34,700	+ 71,400	+ 72,500	+ 23,300	+ 52,600	+ 52,900	0.29	0.28

DETERMINATION OF THE SAFE LOAD.

The safe load per sq. ft. of floor surface will now be determined. The efficiency of the lower chord is the least, so the safe load must be determined in respect to the lower chord. The wind stress will not be considered.

The following expression

$$S = \frac{P}{A} + \frac{M.c}{1 + \frac{Pl^2}{10E}}$$, where

S = the allowed unit stress,

P = the tension in the lower chord,

A = the net area,

M_1 = the moment caused by the direct loading.

c = the distance from the neutral axis to the extreme fiber,

l = the panel length in inches, and

E = the modulus of elasticity, gives the value of the unit stress in the lower chord.

The dead load per linear foot of truss is 231 lb., and the dead load

resting directly upon the lower chord is 160 lb. per linear foot of truss.

Let x equal the live load in lb. per sq. ft. of floor surface, then the total load per linear foot of truss is $(7x+231)$ lb., the distance between trusses being 14 ft. The total load per lin. foot resting upon the lower chord is $(7x+160)$ lb. To find the value of P for L_4L_5 take moments about U_4 and divide by the height of the truss at that point. This makes $P=(7x+231)\,131.5$. The net area A is 3.63 sq. in.,

$M_1 = \frac{1}{8}\frac{(7x+160)}{12} \times 144^2 = (7x+160)\,216$, $I = 5.3$, $c = 2$ in., $l = 144$ in. and $E = 25{,}000{,}000$. Substituting these values in formula for S on page 39 gives

$$S = \frac{(7x+231.5)\,131.5}{3.63} + \frac{(7x+160)\,216 \times 2}{5.3 + \frac{(7x+231)131.5 \times 144^2}{25{,}000{,}000}},$$

simplifying the expression gives

$$S(0.076x + 7.82) = 19.3x^2 + 5{,}648x + 135{,}600.$$

The allowed dead load unit stress is 25,000 lb. per sq. in.

and for live load it is 12,500 lb. per sq. in. This causes the allowed unit stress to change for different values of x. The value of S must be assumed and then x is to be computed from the equation. Use this value of x in finding the actual stresses in the lower chord, and if the assumed value of S does not agree with the actual, it must be recomputed. By using S as 21,000 lb. per sq. in., x. the safe live load is found to be 7 lb. per sq. ft. of floor surface, and the dead and live load unit stress are 16,700 and 4,340 lb. per sq. in. respectively. This gives an efficiency of one for the lower chord.

CONCLUSION.

A structure, in order to be economically designed, should be built so that the efficiencies of all the parts are the same. This is

not the case in this bridge. Some of the members have an efficiency of twenty times that of the lower chord. So the bridge is of poor design, and would be so if the weakest member were strong enough to take the required stress

The connections at the ends of the members are of poor construction. The lower chord connections are of cast iron which is required to take a tensile stress; and at the end pedestal the stress is transferred from the upper chord to the pedestal by an abutting joint.

By far, the weakest member is the lower chord. It is subject, under full load, to large tensile and bending stresses, either of which is greater than the safely allowable. If the bridge acts as a simple truss, there is no doubt but that the lower chord is

stressed beyond a safe limit. The computed safe floor load, which is 7 lb. per sq. ft., is about equal to that produced by thirty men walking over the bridge. The bridge probably acts as a two hinged arch, and some of the horizontal thrust is taken up at the pedestal, but the pedestal is unable to resist much of the horizontal thrust. Even if it were able to releive the lower chord of most of its tensile stress, there still would be the large secondary stress.

The efficiency of the bridge would be greatly increased, if the floor system were connected to the truss at the panel points. If this were done, the bridge would be fairly safe. The lateral system is a poor excuse and is likely to be the cause of the bridge failing. Failure of the bridge will

probably be either at the middle of the lower chord, due to excessive live load or near the end of the lower chord, caused by a strong wind.