



Thesis.

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May 18, 1876

The Iron Bridge over the Merrimac River at Tyngsboro, Mass.

This bridge was built in the year 1871 by the National Bridge Co. of East Boston. The total span is 596 feet made up of four river spans, two of 136 feet $2\frac{3}{4}$ in. and two of 137 feet $11\frac{3}{8}$ in., and one span of 48 feet over the Boston, Lowell and Nashua R.R.

The design of the river spans is a "Simville" or a "Whipple of double intersection" and that of the span over the railroad is a simple "Pratt".

The roadway is level over the short railroad span but over the four river spans it has a regular fall of one foot in 55.5 feet, being 16 ft. 9 in. above the abutments at the commencement of the first river span and only 4 ft. 9 in. above the abutments at the end of the last river span.

This roadway consists of three inch planking nailed to four inch by twelve inch stringers which rest directly on the floor beams. One end of the railroad span rests on a granite wall with a vertical face and with wing-walls on each side, the other rests on the end post of the first river span.

The abutments for the first end of the first river span and the last end of the last river span are of granite. The abutments for the intermediate spans were made as follows: first, a number of square piles were driven together as closely as possible in bunches of about six feet diameter, at the points where the abutments were wanted; then cast iron cylinders six feet in diameter and one inch thick were placed over these piles and driven into the sand at the bottom of the river to a distance of five or six feet; the space between the top of the piles and the top of the cylinder was filled with rubble and concrete.

A large block of granite was imbedded in the concrete for the ends of the trusses to rest upon. There are three pairs of these cylinders, and those in each pair are eighteen feet apart from center to center. The cylinders on the up river side are armed with ice breakers, made of sheet iron, supported by piles and concrete, so that but little of the thrust they receive is transmitted to the cylinders. Each pair of these cylinders are braced by two cast iron beams, about ten feet apart, and also by iron rods running from each end of these beams and bolted to a ring placed midway between the beams and the cylinders.

Around each of these cylinders large quantities of ballast was dumped amounting in all to about 100 tons

The loads for which this bridge was calculated are given in the following statement -

" Static Load made up as follows
About 12,000 ft B.M. = 1000 cu ft. weighing 32 lbs per cu. ft. 32,000 lbs
Total weight of Iron work 54,000 "
Possible accumulation of Snow 27,000 "
Total Static Load 113,000 lbs

Static load, factor five (5) 565,000 lbs
= Ultimate strength of static load

Moving load 162,000 lbs factor six (6) 972,000 lbs

Total Ultimate strength = 1,537,000 lbs

Working load found by dividing by factor six (6)
= 256,166 lbs

Number of panels 13

Panel weight on two trusses = 19,705 lbs

" " " one truss = 9,852 lbs = 4.9 tons

Moving load = 3.1 tons

Static load = 1.8. " "

We see from the above statements that after the dead load was decided upon it was summed up and multiplied by a factor five; then the live load was multiplied by a factor six, both

results added together and their sum divided by a factor six. The effect of this multiplication and division has been simply to reduce the dead load by one sixth and has not changed the live load. There may be a reason for this but I have failed see it. It seems to me, that, after having estimated the dead as nearly as possible, it is very unwise to use only five-sixths of it in the calculations, unless it is intended to use the same factor of safety for the dead as for the live load.

The proportion of the live load to the dead, was found by taking the ratio, which the live load multiplied by a factor six, bore to the dead load multiplied by a factor five.

Calculation of Stresses

To produce the greatest chord stresses the live load must be all over the bridge, therefore at each apex there would be a weight of 4.9 tons.

There are two systems of diagonals and struts in this truss which are entirely independent of each other and which are not symmetrical about the centre.

Fig 1, page 7 shows the necessary diagonals for this condition alone,

The vertical components of the stresses of the diagonals are marked on them; the stresses in the chords are also marked. The stresses in the diag-

onals were found as follows. In the diagonal from O to a there is a stress from $\frac{12}{13}$ of the load at (a) + $\frac{10}{13}$ of the load at (c) + $\frac{8}{13}$ of the load at (e) + $\frac{6}{13}$ of the load at (g) + $\frac{4}{13}$ of the load at (i) + $\frac{2}{13}$ of the load at (k) = $(12 + 10 + 8 + 6 + 4 + 2) \times \frac{4.9}{13} = 15.83$ tons

In the diagonal from O to b we have $(11 + 9 + 7 + 5 + 3 + 1) \times \frac{4.9}{13} = \frac{36 \times 4.9}{13} = 13.57$ tons.

The vertical components of the stresses in these two diagonals should

Fig. 1.

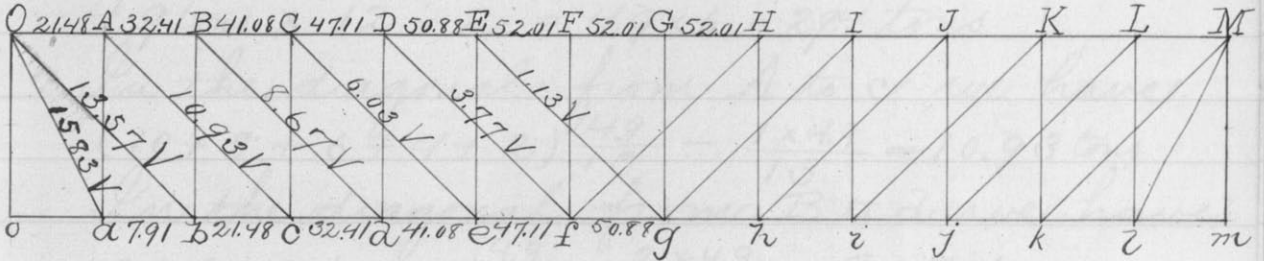


Fig. 2.

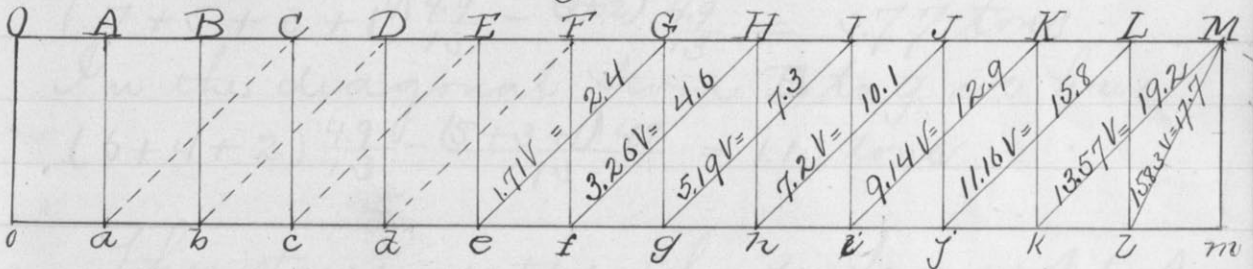
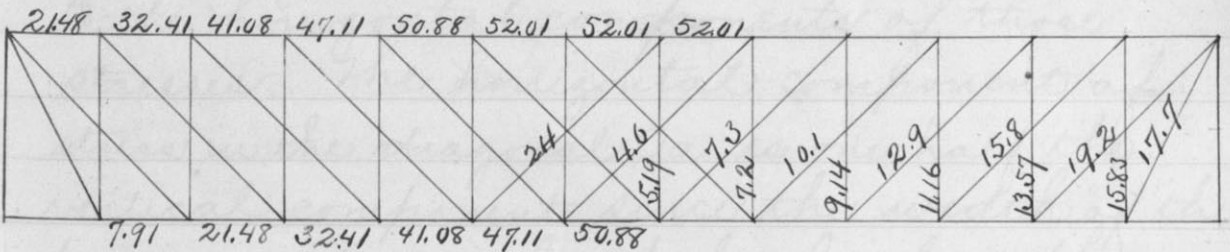


Fig. 3



equal the supporting forces.

$$15.83 + 13.57 = 29.4 \text{ tons}$$

$$4.9 \text{ tons} \times 12 \div 2 = 4.9 \times 6 = 29.4 \text{ tons}$$

In the diagonal from A to c we have

$$(10 + 8 + 6 + 4 + 2) \frac{4.9}{13} - \frac{1 \times 4.9}{13} = 10.93 \text{ tons}$$

In the diagonal from B to d we have

$$(9 + 7 + 5 + 3 + 1) \frac{4.9}{13} - \frac{2 \times 4.9}{13} = 8.67 \text{ tons}$$

In the diagonal from C to e we have

$$(8 + 6 + 4 + 2) \frac{4.9}{13} - \frac{(3 + 1) \times 4.9}{13} = 6.03 \text{ tons}$$

In the diagonal from D to f we have

$$(7 + 5 + 3 + 1) \frac{4.9}{13} - \frac{(4 + 2) \times 4.9}{13} = 3.77 \text{ tons}$$

In the diagonal from E to g we have

$$(6 + 4 + 2) \frac{4.9}{13} - \frac{(5 + 3 + 1) \times 4.9}{13} = 1.13 \text{ tons}$$

The stress in the Chord from O to A is that produced by the stresses in the two diagonals Oa and Ob and is equal to the horizontal components of those stresses. The horizontal component of stress in the diagonal Oa is one half the vertical component since the width of the bay equals one half the height. The horizontal component of the stress in the diagonal Ob is equal to the vertical component since the diagonal is at an angle of 45°

Therefore the stress in the chord from O to A equals $\frac{1}{2}$ of $15.83 + 13.57 = 21.48$
 The stress in the chord from A to B is the stress in the chord from O to A with the addition of that due to the stress in the diagonal AC Therefore stress in the chord from A to C = $21.48 + 10.93 = 32.41$ tons

$$\text{Stress from B to C} = 32.41 + 8.67 = 41.08 \text{ tons}$$

$$\text{" " C " D} = 41.08 + 6.03 = 47.11 \text{ "}$$

$$\text{" " D " E} = 47.11 + 3.77 = 50.88 \text{ "}$$

$$\text{" " E " F} = 50.88 + 1.13 = 52.01 \text{ "}$$

These stresses may be proved by the method of moments, as follows.

We must keep each system separate.

Therefore to find the stress from O to A we take 15.83 multiply it by $\frac{1}{2}$, since the height of the truss = 1 and each of the distances $oa, ab \text{ \& } c = \frac{1}{2}$, and take 13.57 and multiply it by 1 , since $ob = 1$ and height of truss = 1 . Therefore stress in OA = $15.83 \times \frac{1}{2} + 13.57 \times 1 = 7.91 + 13.57 = 21.48$ tons

$$\text{Stress in AB} = 15.83 \times \frac{3}{2} + 13.57 \times 1 - 4.9 \times 1 = 32.41$$

$$\text{" " BC} = 15.83 \times \frac{3}{2} + 13.57 \times 2 - 4.9 \times 2 = 41.08$$

$$\text{" " CD} = 15.83 \times \frac{5}{2} + 13.57 \times 2 - 4.9 \times 4 = 47.11$$

$$\text{" " DE} = 15.83 \times \frac{5}{2} + 13.57 \times 3 - 4.9 \times 6 = 50.88$$

Stress in $EF = 15.83 \times \frac{1}{2} + 13.57 \times 3 - 4.9 \times 9 = 52.01$ tons.

The stress in the lower chord from o to a is nothing. The stress in the chord from a to b is that due to the diagonal Oa and is equal to the horizontal component of the stress in that diagonal and since the vertical component = 15.83 the horizontal component = $15.83 \div 2 = 7.91$

The stress in bc is the stress in ba with the addition of the stress produced by the diagonal O_b . Therefore the stress in $bc = 7.91 + 13.57 = 21.48$ tons.

The stresses in the other parts of the lower chord may be found in the same manner, but we see that the stresses in the opposite ends of the parallelograms, formed by the ties and segments of the chords, are equal.

Therefore, having found the stresses in the upper chord, if we run these stresses down through these parallelograms we have the stresses in the lower chord

To find the greatest stresses in the diagonals and to find the necessary counter diagonals we suppose the dead load only to be over the whole bridge and the live load to be over a part of it. The greatest stress in any diagonal occurs when the live load extends from the further end of the bridge to the foot of the diagonal. Fig 2, page 7, shows the greatest stresses on the diagonals also the necessary counter diagonals.

First we suppose the live load to be at (a) that is a load of 4.9 tons at this point and a load of 1.8 tons at all the other points

$\frac{1}{3}$ of this load 4.9 will go to the right abutment but at the same time there will be $\frac{3.0}{13}$ of 1.8 going through the same panel to left abutment. Therefore no counter diagonal will be needed in this panel. Now suppose the live load has reached the point (b), then at each of the points a and b there will be a load of 4.9 tons and at all the other points a load of 1.8 tons

$\frac{2}{3}$ of the load at b , 4.9 tons, goes to the right abutment, but at the same time there is $\frac{2.5}{13}$ of 1.8 tons going to the left abutment, therefore no counter diagonal is needed in this panel. In the same manner it could be shown that no counter diagonals are needed until we get to the point (e).

When the live load has reached this point there is $(5+3+1)\frac{4.9}{13} = 3.39$ tons going to the right abutment and $(6+4+2)\frac{1.8}{13} = 1.68$ tons going to the left abutment, therefore the difference between 3.39 tons and 1.68 tons is the vertical component of the stress in the diagonal eG

$3.39 - 1.68 = 1.71$ tons. Since the ratio of length of the tie to the depth of the truss is the $\sqrt{2}$ to 1, 1.71 tons multiplied by the $\sqrt{2} = 1.414$ will give the actual stress in the diagonal. $1.71 \times 1.414 = 2.4$ tons

the actual stress in the diagonal eG. To get the greatest stress in the diagonal fH suppose the live load to have reached that point, and to get the greatest stress in g*i*, suppose the live load

to have reached that point, and so on for any diagonal.

$$\text{Stress in fH} = \left\{ (6+4+2) \frac{4.9}{13} - (5+3+1) \frac{1.8}{13} \right\} \times 1.414 = 4.6 \text{ tons}$$

$$\text{" " gI} = \left\{ (7+5+3+1) \frac{4.9}{13} - (4+2) \frac{1.8}{13} \right\} \times 1.414 = 7.3 \text{ "}$$

$$\text{" " hJ} = \left\{ (8+6+4+2) \frac{4.9}{13} - (3+1) \frac{1.8}{13} \right\} \times 1.414 = 10.1 \text{ "}$$

$$\text{" " iK} = \left\{ (9+7+5+3+1) \frac{4.9}{13} - \frac{2 \times 1.8}{13} \right\} \times 1.414 = 12.9 \text{ "}$$

$$\text{" " jL} = \left\{ (10+8+6+4+2) \frac{4.9}{13} - \frac{1 \times 1.8}{13} \right\} \times 1.414 = 15.8 \text{ "}$$

$$\text{" " kM} = \left\{ (11+9+7+5+3+1) \frac{4.9}{13} - 0 \right\} \times 1.414 = 19.2 \text{ "}$$

$$\text{" " lN} = \left\{ (12+10+8+6+4+2) \frac{4.9}{13} - 0 \right\} \times 1.118 = 17.7 \text{ "}$$

It will be noticed that in the case of the diagonal lN the ratio of the length of the tie to the depth of the truss is the $\sqrt{\frac{5}{4}}$ to 1. $\sqrt{\frac{5}{4}} = \frac{1}{2}\sqrt{5} = 1.118$.

The greatest stresses in the posts occur under the same conditions of loading as the diagonals. Therefore the greatest stress in the post lN is 15.83 tons as is shown in Fig 3 page 7 which shows all the necessary members with the stresses marked on them.

The form of the upper chord is that of a built channel bar, therefore the formula for r^2 , the radius of gyration, in Rankine page 523 may be applied to this case.

Instead of that formula I have used the formula from which that was derived and which I think is easier to work with if you have a table of squares at hand. The formula I have used is $r^2 = h^2 \left\{ \frac{A^2 + 4AB}{12(A+B)^2} \right\}$ where

h = depth of flanges + $\frac{1}{2}$ thickness of web;

B = area of web, A = area of flanges

For the first five panels, that is up to the point E Fig. 1 page 7 the cross section does not change; for this section, $B = 14\frac{1}{2} \times \frac{1}{4} + 6 \times \frac{1}{4} = 5.12$ sq in.

$$A = (12 + 12) \times \frac{1}{4} + 6 \times \frac{1}{4} = 7.5 \quad "$$

$$h = 12.12 \text{ in. } h^2 = 146.89$$

$$r^2 = h^2 \left\{ \frac{A^2 + 4AB}{12(A+B)^2} \right\} = 146.89 \left(\frac{56.25 + 153.6}{1911.12} \right) = 16.13$$

substituting this value in Gordon's formula $\frac{P}{S} = 36000 \div \left(1 + \frac{r^2}{36000} \right)$ we have

$$\frac{P}{S} = 36000 \div \left(1 + \frac{(126)^2}{36000 \times 16.13} \right) = 35041.9 \text{ lbs}$$

Therefore the breaking strength of this cross section is 35041.9 lbs per square inch.

For the three middle panels that is from E to H the cross section is heavier. In the formula for $R^2 = h^2 \left\{ \frac{A^2 + 4AB}{12(A+B)^2} \right\}$, $B = 14\frac{1}{2} \times \frac{3}{8} \times 6 + 6 \times \frac{5}{16} = 7.3 \text{ sq in}$

$$A = (12'' + 12'') \frac{5}{16} + 6 \times \frac{5}{16} = 9.4 \text{ sq in } h = 12.2 \text{ in } h^2 = 148.84$$

$$R^2 = h^2 \left\{ \frac{A^2 + 4AB}{12(A+B)^2} \right\} = 148.84 \left\{ \frac{88.36 + 274.48}{3346.68} \right\} = 16.14$$

Substituting this value of R^2 in Gordon's formula and we have

$$\frac{P}{S} = 36000 \div \left(1 + \frac{(126)^2}{36000 \times 16.14} \right) = 35042.5 \text{ lbs}$$

Therefore the breaking stress is 35042.5 lbs per square inch.

At the bottom of page 16 is a table the first column contains the number of the panel. Panel No 1 refers to the same part of the truss as OA in Fig 1 page 7. The second column contains the stress in lbs.

The third column contains the stress per square inch found by the division of the stress by the area in square inches. The fourth column contains the breaking stress per square inch as given by Gordon's formula. The fifth column gives the factor found by dividing the breaking stress per sq. in. by the actual stress per sq. in.

It will be noticed the factor of safety in the first and second panels is very large. The plates and angle irons in these panels are one fourth of an inch thick and this is the least thickness, it is claimed, that single plates should have in any structure

Upper Chord

| No. of Panel | $P =$ Stress | $\frac{P}{S}$ | $\frac{P}{S}$ (Gordon's form) | Factor of Safety |
|-----------------|-----------------|---------------|----------------------------------|------------------------|
| 1 | 42960 lbs | 3409.5 lbs | 35041.9 lbs | 10.3 |
| 2 | 64820 " | 5144.4 " | 35041.9 " | 6.8 |
| 3 | 82160 " | 6520.6 " | 35041.9 " | 5.4 |
| 4 | 94220 " | 7477.8 " | 35041.9 " | 4.7 |
| 5 | 101760 " | 8076.2 " | 35041.9 " | 4.3 |
| 6 | 104020 " | 6288.6 " | 35042.5 " | 5.6 |

The struts have the form of an I beam. The formula for r^2 , the radius of gyration, for this section as given by Rankine (page 523) is $r^2 = \frac{b^2}{12} \cdot \frac{A}{A+B}$ where b = breadth of flanges, A = the joint area of flanges, B = the area of the web. There are only two different sizes of I beams in the bridge, one, a heavy 8 in. I beam with a flange $5\frac{1}{8}$ " wide, and a web $\frac{1}{2}$ " thick, and the other a light 8 in I beam with a flange only 4" wide and a web $\frac{5}{16}$ " thick. For the heavy I beam $r^2 = \frac{b^2}{12} \cdot \frac{A}{A+B} = \frac{(5\frac{1}{8})^2}{12} \cdot \frac{8.19}{10.94} = 1.76$

Substituting this in Gordon's formula $\frac{P}{S} = 36000 \div (1 + \frac{63504}{90000 r^2})$ we have —

$$\frac{P}{S} = 36000 \div (1 + \frac{63504}{90000 \times 1.76}) = 7186.8 \text{ lbs}$$

For the light I beam $r^2 = \frac{b^2}{12} \cdot \frac{A}{A+B} = \frac{(4)^2}{12} \cdot \frac{5.24}{6.62} = 1.004$

Substituting this value of r^2 in Gordon's formula and we have —

$$\frac{P}{S} = 36000 \div (1 + \frac{63504}{90000 \times 1.004}) = 4484.4 \text{ lbs}$$

Struts

| No. | Stress | $\frac{P}{S}$ | $\frac{P}{S}$ Gordon's Form. | Factor of Safety |
|-----|------------|---------------|---------------------------------|------------------------|
| 1. | 31660 lbs. | 2893.9 lbs. | 7186.8 lbs. | 2.5 |
| 2 | 27140 " | 2480.8 " | 7186.8 " | 2.9 |
| 3 | 22320 " | 3134.8 " | 4484.4 " | 1.4 |
| 4 | 18280 " | 2567.4 " | 4484.4 " | 1.7 |
| 5 | 14400 " | 2022.5 " | 4484.4 " | 2.2 |
| 6 | 10380 " | 1457.9 " | 4484.4 " | 3.1 |

In the above table the first column contains the number of the strut, No. 1 referring to the strut aA in fig 1 page 7. No. 2 refers to the strut bB in fig 1 page 7. &c. The second column gives the total stress in pounds. The third column gives the stress in pounds per square inch found by dividing the total stress by the area in square inches. The fourth column gives the breaking stress in pounds on the square inch as found by Gordon's formula. The fifth column gives the factor of safety.

found by dividing the breaking stress in pounds on the square inch as given by Gordon's formula by the actual stress in pounds on the square inch. It will be noticed that the factors of safety are very small indeed. In no case is the factor of safety large enough to make the struts as strong in proportion as the other members of the bridge. The factor of safety for strut No. 3 is only 1.4 the actual stress being almost as much as the breaking stress. At this point the heavy I beam of struts Nos. 1 & 2 is changed to the light I beam. This bridge was calculated for a live load of about 1200 lbs per running foot while the actual travel over it has been light pleasure carriages and light wagons, there having been scarcely any heavy travel over it since it was built. This probably accounts for the I beams having stood thus long without showing signs of giving way.

Diagonal Ties

| Tie | Stress | Area | Stress per Sq. in. |
|-----|------------|--------------|-----------------------|
| MU | 35,400 lbs | 4.148 sq.in. | 8534.2 lbs |
| Mk | 38,400 " | 4.148 " " | 9257.4 " |
| Lj | 31,600 " | 2.97 " " | 10637.7 " |
| Ki | 25,800 " | 2.454 " " | 10513.4 " |
| Jh | 20,200 " | 2.454 " " | 8231.4 " |
| Ig | 14,600 " | 2.454 " " | 5949.4 " |
| Hf | 9,200 " | 1.988 " " | 4627.7 " |
| Ge | 4,800 " | .994 " " | 4828.9 " |

In the above table the first column gives the tie referred to, the letters being found on fig 2 page 7. Taking the tenacity of wrought iron rods, as given by Rankine, between 60,000 lbs and 70,000 lbs per square inch we see that in the case of every tie there is a factor of safety of at least six and in one case a factor of safety of fourteen.

Lower Chord

| Panel | Stress | Effective Area | Stress per sq. in. |
|-------|-----------|----------------|--------------------|
| 2 | 15820 lbs | 4 sq. in. | 3955 lbs |
| 3 | 42960 " | 6.2 " " | 6929 " |
| 4 | 64820 " | 6.28 " " | 10322 " |
| 5 | 82160 " | 8.5 " " | 9666 " |
| 6 | 94220 " | 8.4 " " | 11217 " |
| 7 | 101760 " | 9.6 " " | 10600 " |

In the above table the first column gives the number of the panel referred to. The second column gives the total stress in pounds. The third column gives the effective area in square inches found by subtracting the area cut out for pins, from the total area. The fourth column gives the stress in pounds per square inch found by dividing the total stress in pounds by the effective area in square inches.

Taking the tenacity of wrought iron plates as between 50,000 lbs and 60,000 lbs we see from the above table that the least factor of safety is about five and the greatest about fourteen.

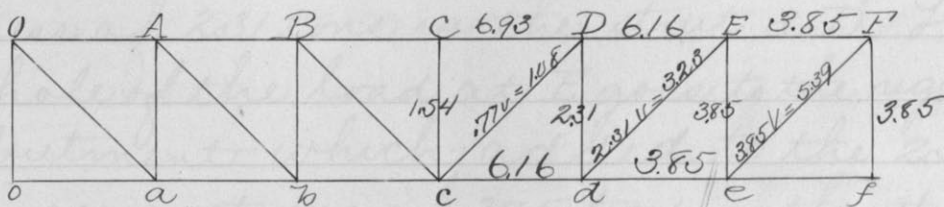
Pins

| Pin | Diameter | Maximum Stress | Planes of Shear | Stress per sq. inch |
|-----|----------|----------------|-----------------|---------------------|
| a | 2 in. | 35400 lbs | 4 | 2818 lbs |
| b | 1 7/8 " | 42960 " | 2 | 7783 " |
| c | 1 3/4 " | 64820 " | 2 | 13504 " |
| d | 1 3/4 " | 82160 " | 6 | 5705 " |
| e | 1 3/4 " | 94220 " | 6 | 6543 " |
| f | 1 1/8 " | 101760 " | 6 | 17131 " |
| A | 2 " | 23660 | 2 | 3767 " |
| B | 1 3/4 " | 19740 | 2 | 3576 " |
| C | 1 3/4 " | 14320 | 2 | 2983 " |
| D | 1 3/4 " | 10280 | 2 | 2142 " |
| E | 1 3/4 " | 6400 | 2 | 1333 " |
| F | 1 1/8 " | 2380 | 2 | 1293 " |

In the above table the first column gives the pin referred to, the letters being found on fig 1 page 7. The second column gives the diameter of the pin. The third column gives the greatest stress that comes on the pin. The fourth column gives the planes of shear, the large number of planes, six being caused by the distribution

of the stress into four plates. The fifth column gives the stress in pounds per square inch found by dividing the maximum stress by the planes of shear, and dividing this quotient by the area of the pin in square inches. It will be seen from the table that for a good quality of steel pin the stress per square inch is very light in all cases except the pins (c) and (f). The stress on these two pins is rather heavy unless the pin is of an extra fine quality of steel. It seems unfortunate that we have such light pins at these points when at other points where the stress is much less we have a heavier pin, for instance at the point (f) we have a stress on the pin of $101760 \text{ lbs} \div 6 = 16960 \text{ lbs}$ and a pin $1\frac{1}{8}$ in. diameter, while at the point (a) we have a stress on the pin of $35400 \text{ lbs} \div 4 = 8850 \text{ lbs}$, only about half as much as in the other case, and a pin 2 in. in diameter, the areas being respectively .99 in and 3.14 in.

Floor Beam



The floor beam has for a dead load its own weight and a part of the planking of the bridge.

This gives a dead load of .3 ton per panel. It has the same live load as the large truss $3.1 \text{ tons} \times 2 = 6.2 \text{ tons}$. This added to the dead load gives a panel weight of 1.54 tons.

The greatest stress comes on this floor beam when when the whole truss is loaded with both live and dead load. The panels are 3 ft square, the ties running at an angle of 45° .

Calculation of the stresses.

One half of load at (C), 1.54 tons, goes to the right abutment and one half to the left abutment producing ^{a stress of} a vertical component .77 tons in the tie c D

The whole of the load at D goes to the right abutment, which added to the .77V gives a stress of 2.31 tons in the strut d D. The whole of the load at E goes to the right abutment which added to the 2.31 tons gives a stress of 3.85 tons in the strut e E. The load on the strut Dd goes to the right abutment through the tie a E producing a stress in that tie whose vertical component is 2.31 tons. The load on the strut e E goes to the right abutment through the tie e F producing a stress in that tie whose vertical component is 3.85 tons. The stress in the Chord EF is the vertical component of the stress in the tie e F. = 3.85 tons. The stress in DE is 3.85 tons + 2.31 tons = 6.16 tons. The stress in CD = 6.16 tons + .77 tons = 6.93 tons

We can also obtain these stresses by the method of moments —

Stress in EF = $3.85 \times 1 - 0 = 3.85$ tons

" " ED = $3.85 \times 2 - 1.54 \times 1 = 6.16$ "

" " DC = $3.85 \times 3 - 1.54 \times 3 = 6.93$ "

As the ratio of the length of tie to the depth of the truss is the $\sqrt{2}$ to 1. The vertical components of stress multiplied

by the $\sqrt{2} = 1.4$ will give the actual stresses in the tie. —

$$\text{Stress in } cD = .77 \text{ tons} \times 1.4 = 1.08 \text{ tons}$$

$$\text{" " } dE = 2.31 \text{ " } \times 1.4 = 3.23 \text{ "}$$

$$\text{" " } eF = 3.85 \text{ " } \times 1.4 = 5.39 \text{ "}$$

For the upper chord the form of section is two angle irons $2\frac{1}{4} \times 2\frac{1}{4} \times \frac{5}{16}$. These angle irons act independently as they are $3\frac{1}{8}$ " in. apart and are not rivetted except at the panel joints. The formula for r^2 , the radius of gyration, given by Rankine page 523 is $r^2 = \frac{b^2}{24}$ where b = breadth of the arms.

$$r^2 = \frac{b^2}{24} = \frac{(2\frac{1}{4})^2}{24} = .21 \quad \text{Substituting this value of } r^2 \text{ in Gordon's formula}$$

$$\frac{P}{S} = 36000 \div \left(1 + \frac{b^2}{36000 r^2}\right) \text{ we have}$$

$$\frac{P}{S} = 36000 \div \left(1 + \frac{1296}{7560}\right) = 30732 \text{ lbs}$$

In the upper chord from F to E, taking half the stress in each angle iron we have

$$\frac{P}{S} = 2629 \text{ lbs showing factor of safety of } 11.7$$

In the upper chord from E to D

$$\frac{P}{S} = 4488 \text{ lbs factor of safety } 6.8$$

In the upper chord from D to C

$$\frac{P}{S} = 5134 \text{ lbs. factor of safety } 5.9$$

The struts are angle irons $2" \times 2" \times \frac{1}{4}"$
 $r^2 = \frac{b^2}{24} = \frac{(2)^2}{24} = .17$ substituting this in
 Gordon's formula we have
 $\frac{P}{S} = 36000 \div \left(1 + \frac{1296}{36000 \times .17}\right) = 29709 \text{ lbs}$

In the strut c C $\frac{P}{S} = 2133 \text{ lbs}$
 showing a factor of safety of 13.9
 Strut d D, $\frac{P}{S} = 4738 \text{ lbs}$ factor of safety 6.2
 " e E, $\frac{P}{S} = 7897$ " " " 3.8

Tie c D effective area = .6 sq in
 $\frac{P}{S} = 3600 \text{ lbs}$ factor of safety about 10
 Tie d E effective area = .6 sq in
 $\frac{P}{S} = 10766 \text{ lbs}$ factor of safety nearly 6
 Tie e F effective area = .8 square inches
 $\frac{P}{S} = 13475 \text{ lbs}$ factor of safety about 4.4

For tie e F shearing area of two $\frac{7}{8}$ in
 rivets, one shearing in two places and
 one shearing in one place, is 1.8 sq in
 Stress in e F = 10780 lbs stress per
 square inch = 5933 lbs, a very light
 stress, 7500 lbs per square inch being
 the general rule for rivets. The least
 bearing area of these rivets is .65 sq in
 giving a stress of 16585 lbs per sq in

for bearing. This stress is too large for safety, 10,000 lbs per sq. in being the stress generally taken for bearing. This large bearing stress shows that the rivets would tear out the plate quicker than they would shear off. For the tie $2E$ we have the same shearing area and the same bearing area but the stress is only 6460 lbs. The shearing stress per square inch is 3588 lbs. The bearing area is 9785 lbs per sq. in. These rivets would also tear out the plate before they would shear. For the tie CD the stress is 2160 lbs., the rivet area for shear is 1.2 giving a shearing stress per square inch of 1800 lbs. The bearing area is .33 sq. in. giving a bearing stress of 6576 lbs per square inch thus showing that the rivet would tear out the plate before it would shear. In the lower chord from e to d the stress is 7700 lbs the area is $2.7 \text{ sq. in} - .5 \text{ sq. in} = 2.2 \text{ sq. in}$ giving a stress of only 3500 lbs per square inch. In lower chord (dc) the stress is 12320 lbs

The area is 2.2 sq. in. giving a stress of 5600 lbs per square inch.

The end posts of the large truss have the same form and dimensions of cross section as the upper chord in the first panel. The radius of gyration, r^2 , is as before = 16.13. Substituting this in Goddons formula and we have —

$$\frac{P}{S} = 36000 \div \left(1 + \frac{(252)^2}{9000 \times 16.13} \right) = 25044 \text{ lbs}$$

The stress on these posts is 58,800 lbs. The area is 12.6 sq. in. The stress per sq. in. is 4666 lbs giving a factor of safety of 5.4

The wooden stringer which lies between the floor beams is in the condition of a sloping rafter simply supported at the ends. The angle which it makes with the horizontal is $1^{\circ} 7'$. Formula (5) Rankine

page 293 gives the intensity of the longitudinal thrust in pounds on the square inch for any section x'

$$p' = \left(\frac{W \cos^2 i}{2 \sin i} + w x' \cos i \sin i \right) \div A$$

In this formula, W = the total load = 3192 lbs, being one fifth of the live load per panel for two trusses + its own weight + one fifth of the weight of the planking in one panel.

$w = W \div l \cos i$ = the intensity of the load per lineal inch measured horizontally. A = the area of the cross section = 4 in. x 12 in = 48 sq in.

Substituting these values in the above formula taking $x' = \frac{1}{2} l$ we have $p' = 171$ lbs. In addition to this thrust there is a stress due to the bending moment, a thrust in the upper, and a tension in the lower side of the beam

The intensity of the stress due to the bending moment is $p'' = \frac{M \cdot m \cdot h}{I}$

$$M = 50264 \text{ in. lbs.}, m \cdot h = 6 \text{ in.}, I = 576$$

Substituting these values in the formula we have $p'' = 523.6 \text{ lbs}$

$p = p' + p'' = 171 \text{ lbs} + 523.6 \text{ lbs} = 747.6 \text{ lbs} =$
the thrust in the upper half of the beam.

$$p''' = p'' - p' = 523.6 \text{ lbs} - 171 \text{ lbs} = 352.6 \text{ lbs}$$

We see from the above stresses that the stringer is able to stand with safety the loads for which the bridge was calculated.

The Weight of the Bridge

| | |
|------------------------------------|---------------|
| The upper chord one truss weighs | 6282.2 lbs |
| The struts " " " | 6347. " |
| The ties " " " | 4311.6 " |
| The lower chord " " " | 4679.9 |
| The rivet heads " " " | <u>1074.4</u> |
| | 22695.1 lbs |
| Weight of two trusses — | 45390.2 " |
| " " Cross bracing | 1652. " |
| " " Lateral bracing | 1558.2 " |
| " " Floor beams | <u>7346.4</u> |
| | 55946.8 |
| Weight of planking and stringers = | <u>27568.</u> |
| | 83514.8 lbs |

We see from the above that the dead load estimated was a trifle too large