There was
upon
A Roadway Bridge
across the
Merrimack River between
Groveland & Haverhill.

May, 1877.
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There is upon a Bowstring Girder. The bridge chosen as the subject of this thesis is a wrought iron, tubular girder built by the King Bridge & Iron Manufacturing Co. of Cleveland, Ohio.

The structure was commenced in March, 1871, under the superintendence of Col. Coffin, of Newburyport. The stone piers were designed by Mr. C. F. Putnam of Salem, and are considered the best upon the river.

It may not be out of place to give a brief history of the way the bridge came to be built. At Groveland the Merrimack makes a sharp turn and this bend has made the crossing at this...
place highly desirable. In 1834, 1835 & 1836 petitions were sent to the legislature for a charter to build a bridge at this locality, but the opposition of the proprietors of the Haverhill Bridge prevented a favorable hearing. It was not until 1870 that a charter was granted. (Acts, 1870, chap. 219.) The bridge was erected near the site of the "Chain Ferry".

"The day of inauguration may be considered one of the important events in the town's life; a good proportion of the people were out, a collation was provided, and speeches ranging from grave to gay were warmly in the approval of this object which has been cherished one for many years to obtain."

General Description of the Structure.
As in building a structure of this kind, the first thing to be considered is the foundation, as in giving a description of it I will begin with the foundation.

Cribs.
The cribs upon which the piers rest extend from the bed of the river to within one foot of the low water line. They are built of good hemlock timber 12" x 12" and securely fastened with bolts and treenails, and cross-tied at about every six feet, as shown in plan. (See next page.) The spaces between the cross ties are filled with rubble stone, well rammed and the top smoothed off with small stones. On the top of
Plan of Crib

40' 6"

Platform 5' wide

46' 6"

Platform 5' wide

Scale 6 ft = 1 inch
the crib is placed a covering of pine timber to receive the pier. Rubble stone is placed about the crib and also extending up against the pier is four feet. About 1600 tons of rubble stone were needed to fill the crib, besides, about 2000 tons placed around the pier and cribs. Also about 176,000 ft. C.M. were required for the six cribs.

Piers:
I can not describe the piers in a better manner than by copying an extract from the specifications.

"The piers (six in number) shall be of granite. The masonry shall be of first class bridge masonry and shall consist of stones cut on bed, built and joints,
and laid in regular courses of two feet each. The stretchers shall have six no cases less than two feet  
bed, and shall not be less than four feet in length. The leaders shall be not less than two feet 
in width, and shall hold the same size as the heart of the pier that they show on the face and shall occupy at least one fifth (1/5)  
of the face of the wall. Their length shall equal at least two thirds of the thickness of the pier. All joints to be thoroughly broken. The granite for the heart of the pier shall be of blocks well laid in regular courses in cement and well fitted to their place. The face of the stone generally to be left as it comes from the.
quarry except the cut water, or ice-breakers, which is to be cut to the slope shown in plan. (See next page.)

The piers shall be covered with a course of coping seven feet long, well hammered on bed and joints.

The cut water of the piers shall have five iron dowels, one and one-quarter inches in diameter and eight inches long, in each course.

The bed of the river to be properly levelled and cleared off to receive said piers."

There are about 1650 cubic yards of granite masonry in the piers.

The bed of the river was found to be in good condition to receive the piers; only a few small stones had to be removed.
Line Elevation of Pier
Plan of Pier.
The Abutments.

The abutments are of solid granite masonry laid in regular courses of from eighteen inches to two feet six, the largest stones at the bottom. The thickness varies from ten feet at the base to four feet at the top. The same rules govern the size of the headers and stretchers as govern the size of the pane stones in the pier, as already described. Where the wall is not more than five feet thick, the headers extend through the pane. All the stones are laid in cement mortar and pointed with cement.

The Wing Walls extend about thirty-five feet on either side.
of the abutment and are constructed in the same way as the abutments. On the Haverhill side of the river it was found necessary to drive about one hundred and eighty piles. These piles were driven fifteen feet in both abutments and wings about five hundred and fifty cubic yards of masonry were used.

The cost of the cubes, piers, abutments and wingwalls, also including the teaming of the rubble stone, was $48,898.35.

The cost was divided as follows:

County of Essex paid \( \frac{27}{60} \) of the entire cost.

City of Haverhill paid \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) of the entire cost.

Town of Londonderry paid \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) of the entire cost.

Town of Newbury paid \( \frac{2}{6} \) \( \frac{2}{6} \) \( \frac{2}{6} \) of the entire cost.
The superstructure.
The extreme length of the bridge is 804 feet. It is divided into six arch spans, each 126 feet, and a draw span, 48 feet. The width between the trestles is twenty-five feet. Each span (except the draw) is divided into fourteen panels or bays. The height of each arch is twelve feet from center of lower chord to the center of upper chord.

Arches: The truss arches consist of three 7½ inch channel irons and two plates of boiler iron 5⅛" x 11" at the crown and 5⅛" x 13" at the springing. The channel irons and the plates are securely riveted together with 576 rivets placed four and one half inches from center to center.
The extremities of the arches fit into places of cast iron resting upon the abutments or piers. These places have a flat bearing of about twelve or fourteen inches and they are connected with the ends of the lower chord by thread and nut connections. The ends of the arch are planed so as to give a uniform bearing. All joints in the arch lapse about thirty inches.

Lower Chords: The lower chords are composed of two flat bars of wrought iron, each 1" x 6" in sectional area. The ends are enlarged and made round for the thread and nut.

Verticals: The verticals are of flat iron. The upper ends of these
pass through the center of the arches and are connected by means of thread and nut. The lower ends pass between the lower chords and through connecting blocks of cast iron. Screw nuts, both above and below, the chords hold the posts and make them easy to be adjusted.

Diagonals: The diagonals cross each other, two in each panel, and consist of 1 1/4" tie rods in the first panel and 1" tie rods in all the rest. The ends of the diagonals pass obliquely through the arch and connecting blocks with screw nuts at the ends for adjustment.

Side Bracing. There are five side braces of three and one half inch.
plan iron connected at the top
with the arch and at the bottom
with channel iron, which run
across the bridge under the roadway.

Stay Bracing. To give lateral stead-
iness there is a pair of diag-
nonal stay braces one inch hi-
diameter in each of the panels.

Head Bracing. The bracing over
the roadway gives a clear head
room of fifteen feet and
consists of three panels. The
web of each of these panels is
made of 1/2 inch pipe iron and
the struts are formed of two
angle irons.

Fence. The fence over the draw
is made up of three rods of
pipe iron; the top rail is one inc
six diameter, and the two lower ones are 3/4" in diameter.

Wood Work. The floor beams are of Georgia Pine lumber 4" x 15" x 27' and are placed two feet from center to center. They extend three inches beyond the chord on each side. The planks are of white pine lumber, are four inches thick and laid length-wise of the bridge. They are nailed to the floor beams by five-inch spikes.

The hub-planks are pine planks 7/8" x 8", are about eighteen inches above the floor. They are secured to posts by 7/8" bolts. (These planks are not shown on the drawings that accompany this thesis.)

The whole structure was painted after being put in position.
Showing dimensions of bridge,
also the way in which the posts & diagonals
are numbered.
Limits of Stress.
The ultimate strength of the iron used in this bridge was as follows:
Mougt iron tension per sq.in. 30 tons
" compression " " 24 "
The factor of safety of four was introduced and this fixes the limit of stress:
for tension per sq. in. 7.5 tons = 15000 lbs.
for compression " 6.0 = 12000 lbs.
The loads for which the bridge was calculated are:
Dead load per linear ft. of structure 750 lbs.
Live load per sq. ft. of structure 62 lbs.
This reduced to the load per linear foot for one truss is,
Dead load per linear ft. for 1 truss 375 lbs.
Live " " " " " " " " 775 "
Total load " " " " " " " " 1150 "
Calculations

The maximum thrust would come upon the lower chord at the crown and the uniform tension in the lower chord are found by dividing the maximum bending moment by the height of the girder at that point and are as below.

\[ \text{max } M = 1150 \times 62 \times 62 - 1130 \times 62 \times 31 \]

\[ = 3210300 \text{ ft} \cdot \text{lb} = 1105.15 \text{ ft} \cdot \text{lb} \]

Horizontal thrust at crown = uniform tension in lower chord = \( 1105.15 \div 115 = 9.64 \text{ lb/ft} \)

Note: In calculating the thrust at the different points of the arch, the curve has been assumed to be a parabola; this is not strictly
true, but the error introduced will be too small to be noticed. The thrust at any section, other than the crown, will be found by simply combining the horizontal thrust with the load between the crown and the section considered.

Hence: \[ T_{10} = \sqrt{98.24^2 + 5.75^2} = 98.39 \text{ tons} \]
\[ T_{20} = \sqrt{98.24^2 + 11.50^2} = 98.90 \text{ "} \]
\[ T_{30} = \sqrt{98.24^2 + 17.25^2} = 99.71 \text{ "} \]
\[ T_{40} = \sqrt{98.24^2 + 22.42^2} = 100.71 \text{ "} \]
\[ T_{47} = \sqrt{98.24^2 + 27.02^2} = 101.85 \text{ "} \]
\[ T_{64} = \sqrt{98.24^2 + 31.06^2} = 103.00 \text{ "} \]
\[ T_{62} = \sqrt{98.24^2 + 35.63^2} = 104.50 \text{ "} \]

The subscript numbers represent the number of feet from the center to the section considered, thus: \( T_{10} \) represents the thrust at a section 10 ft. from the center.
The sectional areas required to resist the thrust at the different sections are:

At 10' from crown 16.4 Sq. in.

20' " " 16.3 " "

30' " " 16.6 " "

39' " " 16.8 " "

47' " " 16.97 " "

54' " " 17.2 " "

62' " " 17.4 " "

the crown 16.37 " "

The actual area is:

At the crown 15.505 Sq. in.

" Springing 16.75 " "

The sectional area required to resist the tension in the lower chord is 98.24 + 7.8 = 13.1 Sq. in.

The actual area is 10.68 " "

This is the area within the threads where it connects with the shoe at the extreme ends.
Calculations of the web. It has been assumed that the load upon the bridge is transferred directly to the arch. Under this supposition the diagonals the diagonals would not be needed in the case of uniform loading. In the case of unequal distribution of the load, as when the live load extended over only a part of the structure, the stress coming in any web member would be made up of two parts, (1) a part due to the dead load, this being uniformly distributed would be transmitted directly to the bow and would not call into action any of the diagonals. (2) A part due to the rolling load. This, when it extends over the longer
of the two segments into which an assumed plane, as AB Fig. 1, divides the bridge, causes an upward shear on the left of the plane of section and a downward shear on the right of section. This vertical shear is resisted by the arch itself, so that in this case the diagonals are not needed. When the rolling load extends over the shorter segment only, it causes an upward shear on the right and a downward shear on the left of the section. This shear must be resisted by the web in some way.
Tension in diagonals: From the above considerations, it appears that the only stress that can come upon any diagonal must be produced by the rolling load and that, when the shorter segment is loaded. We have just seen that this method of loading causes an upward shear on the right of the section and, since the diagonals must all act as ties, it follows that this shear must be resisted by the diagonal that slopes upward toward the center. Therefore, the stress in any diagonal is the vertical shear (caused by the shorter segment being loaded) resolved along the diagonal in question.

*Except in the panel just beyond the center, when the other diagonal must resist it.*
Vertical shear in bay No. 2 \[ \frac{780 \times 8 \times 4}{124} = 201 \text{ lb} \]

Tension in diag. 2 \[ 201 \times \frac{8.4}{4.5} = 380 \text{ lb} \]

Vert. shear in bay No. 3 \[ \frac{775 \times 15 \times 75}{124} = 702 \text{ lb} \]

Tension in diag. No. 3 \[ 702 \times \frac{0.30}{8.5} = 1120 \text{ lb} \]

Vert. shear in bay No. 4 = \[ \frac{775 \times 23 \times 11.5 + 124}{124} = 1652 \text{ lb} \]

Tension in diag. No. 4 \[ 1652 \times \frac{12.4}{8.5} = 2410 \text{ lb} \]

Vert. shear in bay No. 5 \[ \frac{775 \times 32 \times 16 + 124}{124} = 3200 \text{ lb} \]

Tension in diag. No. 5 \[ 3200 \times \frac{14.14}{10} = 4533 \text{ lb} \]

Vert. shear in bay No. 6 \[ \frac{775 \times 42 \times 21 + 124}{124} = 5513 \text{ lb} \]

Tension in diag. No. 6 \[ 5513 \times \frac{15.05}{12.5} = 7561 \text{ lb} \]

Vert. shear in bay No. 7 \[ \frac{775 \times 52 \times 26 + 124}{124} = 8447 \text{ lb} \]

Tension in diag. No. 7 \[ 8447 \times \frac{15.05}{12.5} = 11365 \text{ lb} \]

Shear in bay No. 8 = 11920 lb, Tension in diag. 8 = 16350 lb.
Tensions in verticals: From what has preceded, it is evident that the greatest tension that can come upon any vertical must be the load, both live and dead, suspended from it plus the weight of the vertical itself. The load suspended from it is as follows:

Live load per linear foot: 775 lbs.
Planking: 125 lbs.
Floor beams: 88 lbs.
Lower chord: 42 lbs.
Nuts, washers, facing: 100 lbs.

Total dead load per linear ft: 1130 lbs.

Tension in: No. 2: 1130 x 7/8 + 25 = 8500 lbs.
No. 3: 1130 x 7/8 + 50 = 8525 lbs.
No. 4: 1130 x 8/8 + 70 = 9675 lbs.
No. 5: 1130 x 9/8 + 90 = 10825 lbs.
No. 6: 1130 x 10 + 100 = 11400 lbs.
No. 7: 1130 x 10 + 110 = 11410 lbs.
No. 8: 1130 x 10 + 112 = 11412 lbs.
IV Thrust in Verticals. The only time that any thrust can come in any of the verticals is when the adjacent ties are strained. Under any condition in order that the resultant stresses in a vertical shall be compression the thrust caused by the rolling load must exceed the tension caused by the dead load. Upon following this consideration out it appears that only the middle vertical and the two on either side are ever in a state of compression.

Thrust in vert. No. 8 $16350 \times \frac{11}{15} - 10 \times 1850 = 8490\text{ lb}$

" " " No. 7 $11365 \times \frac{11.25}{13.05} - 10 \times 350 = 4947\text{ lb}$

" " " No. 6 $7561 \times \frac{11}{15} - 10 \times 350 = 2013\text{ lb}$

" " " No. 5 $4533 \times \frac{10}{14.14} - 7\frac{1}{2} \times 350 = -125\text{ lb}$

This last result shows that vertical No. 5 never acts as a strut.
The sectional area required in the diagonals and verticals.

Diagonal No. 2 requires

\[ \frac{380}{15000} = 0.02 \text{ Sq. in.} \]

Actual area = 1.22 \text{ Sq. in.}

Diag. No. 3 requires \[ \frac{1120}{15000} = 0.075 \text{ Sq. in.} \]

Actual Area \[ = 3.1416 \times \frac{1}{4} = 0.7854 \]  

Diag. No. 4 requires \[ \frac{2398}{15000} = 0.16 \]  

Diag. No. 5 \[ \frac{3200}{15000} = 0.21 \]  

Diag. No. 6 \[ \frac{7561}{15000} = 0.50 \]  

Diag. No. 7 \[ \frac{11365}{15000} = 0.76 \]  

Diag. No. 8 \[ \frac{16350}{15000} = 1.09 \]  

Vert. No. 2 \[ \frac{8500}{15000} = 0.57 \]  

" No. 3 \[ \frac{8525}{15000} = 0.57 \]  

" No. 4 \[ \frac{9675}{15000} = 0.64 \]  

" No. 5 \[ \frac{10825}{15000} = 0.72 \]  

" No. 6 \[ \frac{11400}{15000} = 0.76 \]  

" No. 7 \[ \frac{11410}{15000} = 0.76 \]  

" No. 8 \[ \frac{11412}{15000} = 0.76 \]  

No. 8 requires 2.17 p.p.m., for threeto Gordon's formula being used.
Diagram of Strain

In the vertical indicates tension.
and + signifies compression.
<table>
<thead>
<tr>
<th>Piece of Structure</th>
<th>Total Load</th>
<th>Required Sectional Area</th>
<th>Actual Sectional Area</th>
<th>Safe Intensity of Load</th>
<th>Actual maximum intensity</th>
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<tr>
<td>Bow at crown</td>
<td>98.24 tons</td>
<td>16.37 sq.in.</td>
<td>13.50 sq.in.</td>
<td>6.0 tons</td>
<td>6.34 tons</td>
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<td>Bow at Springing</td>
<td>104.50</td>
<td>17.40</td>
<td>16.75</td>
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<td>1.09</td>
<td>0.785</td>
<td>7.6</td>
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Intensities are tons on the square inch of least sectional area.
The Draw Span.

A very brief description of the draw will be sufficient since that was not made the subject of this thesis. It is situated in the middle of the bridge and it consists of two platforms of the same width as the rest of the roadway and each twenty-four feet long. When the draw is open, these platforms are lifted into a vertical position by means of chains that pass over the posts and wind about a cylinder on the adjacent span. A second chain on each side serves to hold up the outer end when the draw is closed. This second chain is of such a length that the outer end of the platform can never fall.
below the horizontal. The post is made of two plates of boiler rivet and two channel iron. It is twenty feet high and is stayed near the top from the adjoining spans by means of two rods. The beams of the draw are similar in shape but smaller in size than the cross-section of the post. The floor beams are placed just below the beams and are kept in place by a bolt running through them from the beam to a channel iron below. The draw is very heavy and hard to be lifted and does not give as much satisfaction to those who see it as the rest of the bridge does. The cost of the whole structure was $84,962.70.
In conclusion. Although the area required by the preceding calculations is a little in excess of the actual area in the chords, the bridge will probably stand under any weight that will ever come upon it. When it was tested, ox carts, four abreast and filled with stone, were driven across the bridge. The load upon each arch span was about thirty-five tons and the deflection noticed was 1/16 of an inch. The draw was tested in the same same way with about nineteen tons and there was no appreciable deflection beyond the tightening of the chains. The greatest objection to the structure is its lack of stiffness. A car-
ridge driving on to one end of it will cause the whole bridge as far as the draw to vibrate. This might have been obviated to a certain extent by making the platform of the roadway discontinuous over the piers. As the whole it gives general satisfaction to those who use it most.

Before closing I would acknowledge my indebtedness to the King Bridge Iron Manufacturing Company, of Cleveland, for information regarding the loads and limits of stress used in the calculation, and to Mr. James Kimball of Salem, one of the County Commissioners, for facts concerning the building and the testing of
the bridge and Mr. C. A. Putnam of Salem for information concerning the crib, pier &c.

Respectfully submitted,

Geo. W. Kiltridge
Dept. Civil Eng.
Class of '77