Thesis
on the
Pawtucket Bridge
at
Lowell Mass.

May 1877. R. A. Hale.

Thesis on the Pawtucket Bridge at Lowell. Mass.

Before proceeding to the description of the highway bridge which is the subject of this Thesis, a brief sketch will be given of the old highway bridge and the reasons for erecting the present one in its stead. The location of both the old bridge and the new one is the same: situated across the Merrimack River and connecting the city of Lowell and town of Dracut: Lowell at present containing 50.000 inhabitants and Dracut 2500. The old bridge was erected in 1790, and consisted of a wooden Howe Truss of 3 spans of 107 ft each and 2 stone piers. The piers were laid in dry stone and presented an obstruction of about 70 ft. which in freshets prevented the free discharge of water, as the pier at the Lowell side was only 50 ft below the dam.

The bridge also was endangered by the floating logs and ice which lodged against the piers. In addition, as frequent repairs were necessary on the bridge, it was decided to erect a new one. and in April 1871 a committee of four from Lowell and three from Dracut conferred and accepted the bid of the National Iron Works to erect a bridge, pier, and build an addition to one abutment, with the agreement, that when the pier and the addition to the abutment were completed. Lowell and Dracut should each pay $4000. And when the bridge was completed $14000. should be paid by each; making $36000 total.

The old abutment on the Lowell side was to be widened by the Locks & Canal Co (the controllers of the water-power) who were to be benefited, by having but one pier, as an obstruction in the river, instead of two, as formerly. The Bridge Co. were to widen the Dracut abutment and build the pier, in the middle of the river.

The new bridge was completed in the fall of 1871 to the satisfaction of all concerned, except the Bridge Co. who were losers to a heavy amount on their contract.

The old abutment on the Lowell side was enlarged, by building an additional wall, four feet thick, faced with granite and fastened by iron clamps to the old wall. The foundation of the abutment was rock and a trench was cut to receive the stones of the lower course, which were bolted to the solid rock. The width of the abutment is 50 ft. and it forms a part of the canal and wing wall of the dam.

The abutment on the Dracut side was widened in the same manner about 12 ft.

The pier in the middle of the river was built as follows: the foundation was solid rock. The lower course was set in a trench cut in the rock and was 40 ft in length by 12 ft wide.

The height of the pier is about 25 ft and its upper surface is 30 ft long by 9 ft wide.

The down stream end and the two sides have a batter of 1 inch to the foot. The Upstream side is pointed, with a batter of 6 inches to the foot to within 6 ft of the top thence perpendicular to the top. A small triangular space is left on the pier as shown. The rock is good sound granite and the courses are laid alternately headers and stretchers, and break joints.

The filling is good rubble work which was laid in courses level with the face work.

The up stream point of the pier is made of sound whole stone and each course doweled with the succeeding with 1½" iron pins, and the face covered with boiler iron strips ½" thick and 3" wide.

The Bridge consists of 2 spans, of the Parker Bridge Co's design, of about 160 ft span each: as both spans are exactly alike the Lowell Span is taken for discussion. As the Bridge Co. was not in existance, no plans (except an outline tracing of the principal dimensions) could be obtained, and the author was obliged to make complete measurements of all the details, which thus gave a better knowledge of it, than otherwise. The Lowell span consists of 2 trusses, the length of the up stream truss is 156'11" and the down stream truss is 151'8". The trusses are parallel and are connected by cross girders running at right angles to the line of the trusses. The line joining the ends of the two opposite trusses is obleque to the line of the trusses. thus forming a "skew". The interior angle formed by the down stream truss, at the Lowell side is 102°: and at the pier the angle is 78°. This is partly due to the difference of length in the trusses.

The width of the roadway is 24 ft and the sidewalk is 6 ft. The truss is divided into 12 panels, the length of which is 10 ft 6" except at the ends where the lengths vary as shown in the drawing.

The top chord is curved, box shaped and composed of a wrought iron horizontal plate 1.75'×½" in section; attached to 2 vertical plates 1.5'× ¼", by 2 angle irons

$3'' \times 3'' \times 1/4''$, and 2 angle irons of the same size attached to the lower ends of the vertical plates. The rivets for attaching the angle irons are $7/8''$ diameter and $5''$ apart, centre to centre, The chord is formed in lengths of 15 ft and connected by butt joints. The strengthening plates on the sides of the chord are $2ft \times 11'' \times 5/16''$ and on top are $2ft \times 1'9'' \times 1/2''$. Strengthening plates are also used in the strut pin connection $11'' \times 11 1/2'' \times 3/8''$ ✓

The chord is carried down to the abutment and connected with a cast iron ribbed box by means of bolts as shown in Fig 3. Additional plates and angle irons are attached to the chord as shown in Fig 4. EE. The lower part of the top chord has lattice cross bracing, extending throughout its whole length, to give stiffness to the chord. Its size is $2'' \times 3/8''$

The bottom chord is composed of 6 flat wrought iron bars $6'' \times 1/2''$, except at the ends of the trusses, when 4 bars are substituted, and at the centre $6'' \times \frac{5}{8}''$

The chord bars are arranged in 2 sets of 3 each the bars being $3''$ apart in each set and the distance between the sets 7".

The chord bar joints are plain butt joints with a strengthening plate 16"×½" on each side of the joint and connected with 2 pins 1" indiameter, with washers 3" dia. between the bars.

Under the upright is a pin 1¼" dia. connecting the sets of chord bars: the pin passes through 2 flanges cast on the sides of the lowerangle block: and a washer 3" dia keeps the sets of bars apart, and also forms a bearing for the upright or strut box, which has a cast iron tongue 2"×1" attached.

Strengthening plates are attached to preserve the section. 18"×6"×⅜".

The struts are of I section composed of a wrought iron plate 10"×¼" and 4 angle irons 3"×3"×⅜" rivetted to the plate by ⅞" dia. rivets 5" apart.

The lengths of the struts vary and are given farther on. The connection of the strut to the chord at the top is made by encircling a wrought iron strap ¼" thick ×8" wide about the pin which runs through the chord, and then connecting it to the strut with rivets. In addition to the strap the pin passes through 2 cast iron eye blocks, which butt against the strut. The lower end of the strut is fitted in a cast iron strut box, which also supports the cross girder.

The diagonal members are made of round bar iron of varying diameters and lengths, as shown farther on, and at the upper and lower chord pass through angle blocks, and are connected at the ends, which are enlarged, by screw threads and nuts.

The main diagonals pass outside of the struts. as also the counter diagonals, except in 2 cases in which the counters pass through the struts. holes being drilled for the purpose in the struts.

At the ends of the truss 2 vertical rods 1¼" dia. connect the upper chord with the lower. The upper attachment being a screw thread and nut, and the lower attachment a pin passing through the lower chord bar, as shown.

Each cross girder is a simple lattice girder containing 9 panels between the main trusses and 2 panels outside of each main truss as supports for the sidewalks. The drawing is shown in Fig 1.

The total length at the top of the truss is 40 ft 6" and at the bottom 25 ft 9". The height is 3 ft. 2" and length of each panel 2 ft 9". The upper and lower chords are each formed of 2 angle irons 3"×3"×3/8": the uprights are of angle irons 3"×3"×3/8" and the diagonals of flat bar iron varying from 2½" to 4" in width and 3/8" thickness. The end of the girder supporting the sidewalk has an upright midway between the main truss and the end of the girder, and also a diagonal brace from the foot of the upright to the main truss. The extreme end of the Girder has a plate 7"×6"×½" connecting the upper and lower chord. Sections of the uprights of the cross girders at the ends of the main truss, where there is no strut is shown in Fig 2: the girder is braced by plates as shown in Fig 1. In all cases, except at the ends the cross girder rests directly on the strut box and is connected with the strut angle irons and thus doing away with the bracing. The method of connecting the girder with the strut is shown in Fig 1. D.D. The girders at the ends rest on hollow cast iron boxes which fit on the lower chord.

The lateral bracing consists of wrought iron rods about 30 ft long and 1" to 1⅛" dia. and running as shown on the plan, a distance of 2 panels.

The connection with the strut box is shown in Fig 5. the cross girders being used for compression members.

The cast iron chord boxes at the Lowell end rest on 4 rollers, 6" dia. by 18" long. which rest on an iron plate 2" deep, the whole surrounded by iron plates: the lower chord is connected with the box by a wrought iron strap 6" deep by 1" thick, which encircles it and attached by bolts as shown in Fig. 3. The dia. of the bolt is 1⅜" and each bolt connects 2 chord bars with the strap.

✓H

The roadway is supported by 10 lines of stringers, of Georgia Pine 4"×12": the stringers are supported by being laid on wrought iron brackets 5"×4"×½", which are attached to the uprights of the cross girder by bolts ¾" dia. On the stringers is laid a layer of chestnut 3" thick, and to allow dust and water to pass off, and prevent rot the planks are laid 1" apart. On this layer is placed a layer of white oak 3" thick well fitted, and spiked.

A guard timber of oak 8"×8" is laid by the side of each truss, on the roadway, to prevent injury to the truss by passing drivers.

The sidewalk has 3 rows of stringers 4"×9" of Georgia Pine supported by the upper chord of the cross girders. On the stringers is laid a 2" pine planking. The sidewalk fence is a plain iron railing 3' high with posts bolted to the cross girders, a flange being cast on the bottom of the post to connect with the angle iron of the girder. The lower rail is a flat iron bar 2¼"×¼"; the upper rail is a flat bar 3"×½". and an additional strip of iron is attached as shown in the section.

The vertical rods are round iron ¾" dia and attached to the upper and lower rail, by enlarging the ends of the rods. The fence is made in sections of 10 ft 6", and connected to the posts at the lower rail by bolting to a flange cast on the side of the post, and at the upper rail by a cap which is fitted to the post and connected by bolts.

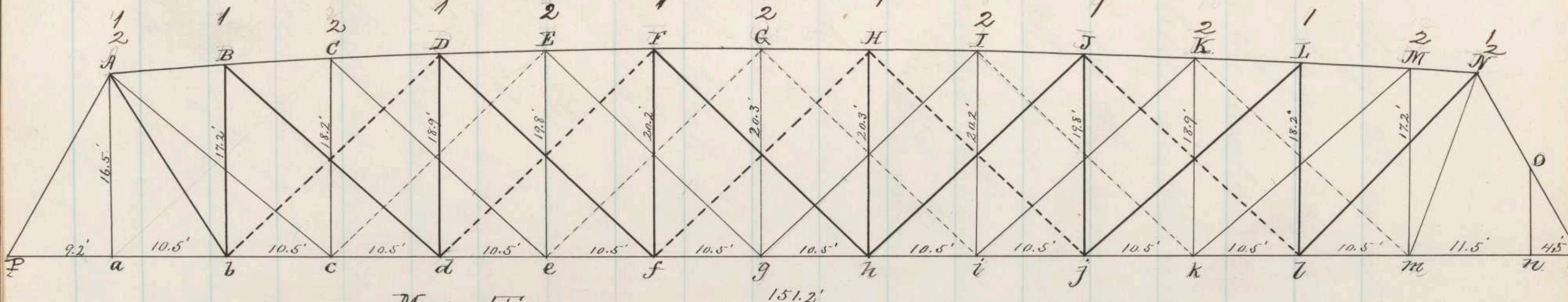
As no information could be obtained in regard to data used for the dead weight of the bridge, the weight was calculated from the dimensions, and the summary is given on the following page. The calculations of the weight were made from the measured dimensions of the separate parts, and may be considered as very nearly the truth.

Summary of the Weight of the Lowell Span.

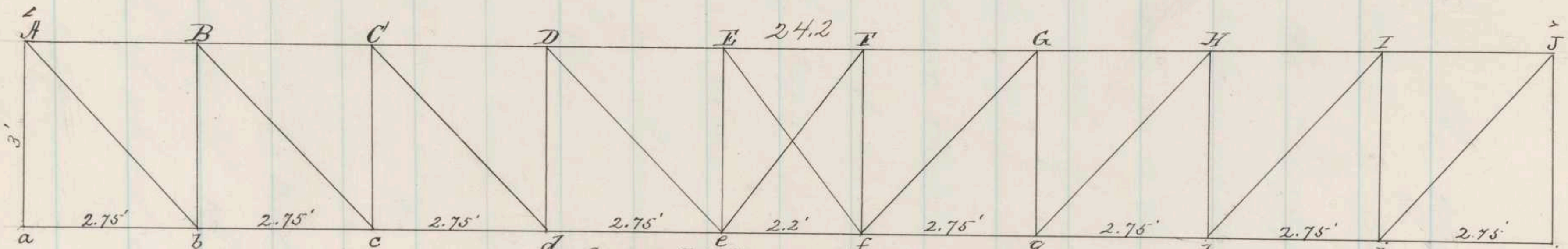
Weight of One Truss. Upper chord	lbs 13195.	
Strength Plates - sides & top	1641	
Pins and Rivets	794.	
Chord Bracing	1419	
Struts and Castings	9221	
Angle Blocks, top & bottom,	7070	
Main Diagonals	6600	
Counter "	1047	
Chord Bars	8959	
Strength Plates, Nuts & washers,	2507	lbs
Total weight of one Truss		53.833
Weight of the 2nd Truss, 202 lbs, additional		54.035
Planking and floor timbers & spikes.		72,270
Cross Girders		25.095
Fence		4950
Sway Bracing		1517
Total Weight of Bridge (Lowell Span)		211.700 lbs or 105.85 Tons.

The total weight of the bridge (Lowell Span) = 105.85 Tons or 52.92 Tons sustained by each truss, the difference in the two trusses being only 200 lbs as regards dead weight. $\frac{52.925}{151.67 \times 40.5}$ = 17 lbs per square foot. A comparison was made with the weight per running foot at the centre of the truss and the weight per foot as deduced from the total weight. The results were 670 lbs and 685 lbs per foot respectively, showing a difference of but 15 lbs. For snow, and convenience in calculation the weight of the dead load was taken as 20 lbs, and the live load as 60 lbs per square foot making a total of 80 lbs per square foot.

The diagram of the down stream truss is shown on the following page with the dimensions of the panels and heights of the truss. The end panels vary in length from those in the centre, as shown. As the truss is a "double intersection" truss, for convenience of calculation it was divided in two simple trusses as shown by the heavy and light lines, the dotted lines



Main Truss.



Cross Girder.

representing the counter braces. The system shown on the diagram by 1 at the vertices, is designated as No 1 in the Calculations. and the other system by No 2. The stresses in the upper and lower chords are produced by the combined action of both systems. while the stress in the diagonals and posts are due only to the effect of the system to which they belong. The maximum chord stresses are produced when the bridge is fully loaded and the stresses are given in the table for each system and the total effect on each piece. The method of calculation used was by Moments, 1st determining the reactions at the abutments. then finding the horizontal stress in each panel and then the stress in the chords by dividing by the depth of the truss at each panel. The horizontal stress is resolved along the upper chord by multiplying by the secant of the angle made with the horizontal and upper chord. The table of chord stresses is given on the following page.

Table of Chord Stresses.

Upper Chord compression						Lower Chord Tension					
System 1	Tons.	System 2	Tons	Total Stress	Tons	System 1	Tons	System 2	Tons	Total Stress.	Tons.
A.B.	32.4	A C	40.3	A.B.	72.7	P b	16.9	P b	15.3	P. b.	32.2
B.D	50.7	C.E.	52.6	B.C.	91.0	b d	32.3	c e	40.2	b c	47.6
D F	55.6	E.G.	57.2	C.D	103.3	d f	49.2	e g	52.4	c d	72.5
F H.	56.2	G. I.	55.8	D E	108.2	f h	55.6	g i	53.4	d e	89.4
H J	57.7	I. K	53.5	E.F	112.8	h j	51.2	i k	44.9	e f	101.6
J L	51.2	K. M.	45.0	F. G	113.4	j l	36.8	k m	25.5	f g.	108.0
L. N.	36.9	M. N.	25.5	G. H	112.0	l o	17.0	m n	15.9	g h	109.0
A.P.	36.4	A.P.	32.9	H. I.	113.5					h i	104.6
O. N.	33.6	O. N.	38.0	I. J	111.2					i j	96.1
				J. K	104.7					j k	81.7
				K L	96.2					k l	62.3
				L. M.	81.9					l m	42.5
				M. N.	62.4					o m	32.9
				A. P	69.3						
				O. N	71.6.						

For all except the end panels, the load per panel was $\frac{80 \times 20\frac{1}{4} \times 10\frac{1}{2}}{2000} = 8.5$ tons. The 2 suspending rods at A sustained 8 tons, and those at O held 4 tons.

Each system was considered as sustaining ½ the whole weight on the rods A & O. The example of finding the stress in F.H. System 1 is given: the reaction at the left is 30.3 Tons. Moments are taken about H.

H.orizontal stress $= \frac{30.3 \times 82 - 8.5(21+42+63) - 4 \times 73.5}{20.3} = 56.2$ Tons.

which resolved along the chord, does not change. as the chord is almost flat at this point.

The stress in the end post A.P is the reaction 30.3 Tons Vertical or resolved $= 30.3 \times \frac{3.606}{3} = 36.4$ Tons in the first system. the other stresses being found similarly, and given in the table. The two suspending rods at A each sustain $\frac{8}{2} = 4$ tons, which requires $\frac{4.}{5} = .8$ sq inches

The actual area of the rod 1¼" dia = 1.2 inches, which is sufficient. The greatest stress in the upper chord is 113.5 Tons; allowing 4 tons for compression this requires 28.4 sq inches. The actual section is $21 \times \frac{1}{2} + 2 \times 18 \times \frac{1}{4}$ + 4 angle irons 3" × 3" × 3/8" = 28.3 sq inches which is about enough.

If the upper chord joints were perfectly fitted the additional plates would not be needed. but as this is impossible, the whole strain may come on the plates. There are 3 plates with $36+30+30 = 96$ rivets: there are needed $\frac{113}{4 \times .6} = 46$ rivets for shear.

In the lower chord the greatest stress is 109 tons using 5 tons as the safe limit for tension per $\square''$ $\frac{109}{5} = 21.8$ sq inches required. The actual section is $5/8'' \times 6'' \times 6'' = 22.5$ sq inches.

At J K the stress is 81.7 Tons. This requires $\frac{81.7}{5} = 16.3$ sq inches The actual section is $1/2'' \times 6'' \times 6 = 18''$ square inches.

In O M the stress is 32.9 Tons which requires $\frac{32.9}{5} = 6.6$ sq inches the chords vary in this panel but the smallest are 2 bars $6'' \times 1'' = 12$ sq inches which is sufficient.

At the lower chord connections of greatest stress, the steel pins are $1\frac{5}{8}''$ dia. Allowing 6 tons per square inch shear for steel, there are required for each set of 3 bars, $\frac{109}{2 \times 6} = 9$ sq inches for shearing area: as the pin would have to be sheared at 6 sections this reduces the area to $\frac{9}{6} = 1.5''$. The actual area of the pin is 2.08 sq inches.

For bearing area, allowing 6 tons per sq. inch as the safe

limit for wrought iron, we have $\frac{109}{2\times6} = 9''$ area.
the thickness of the bars and plates is $(\frac{3}{8}+\frac{3}{8}+\frac{5}{8})3 = \frac{33}{8}''$.
then the diameter $= \frac{9}{\frac{33}{8}} = 2.1''$ which shows the pin too small. At JK the pin is $1\frac{3}{8}''$ dia: the stress is 81.7 Tons. For shear $\frac{81.7}{2\times6} = 6.8$ sq inches: for 6 sections of shear $\frac{6.8}{6} = 1.1''$ are required. The actual area is $1.5''$.

For bearing we must have $\frac{81.7}{2\times6} = 6.8$ sq. inches: the thickness is $(\frac{4}{8}+\frac{3}{8}+\frac{3}{8})\times3 = \frac{30}{8}''$: the diameter $= \frac{6.8}{\frac{30}{8}} = 1.8''$ required.

At Om the stress is 32.9 Tons, and 5 sections to be sheared: this requires $\frac{32.9}{2\times5\times6} = .55$ sq. inches. The area of the pin, $1\frac{3}{8}''$ dia, is 1.48 sq inches which is ample.

For bearing we have $\frac{32.9}{2\times6} = 2.74$ sq inches required: the thickness is $\frac{5}{2}''$ and the dia. $= \frac{2.74}{\frac{5}{2}} = 1.1''$ required.

Near the end of the abutment at Q the chords are connected by an $1\frac{1}{4}''$ steel pin. For shear is required $\frac{32.9}{2\times6} = 2.6''$ and being double shear $= 1.3$ sq inches

The actual area is 1.2 square inches. For bearing is required $\frac{32.9}{2\times6} = 2.7''$ $t = 1''+1'' = 2''$, and the dia. $= \frac{2.7}{2} = 1.35''$

The actual dia. is $1.25''$. The strength of the pins at the foot of the uprights are discussed farther on.

Diagonal stresses

The maximum stress on any main diagonal occurs when the live load covers the longer segment of the bridge up to the foot of that diagonal, and the need of counter braces is determined by moving the live load so that the shorter segment is covered with the live load and the longer segment unloaded. The stresses are shown in the following table, with the required areas, number of rods, and actual areas: and the mode of calculating them is as follows. The horizontal chord stress in any panel is occasioned by the action of the diagonals, and the increased chord stresses towards the centre show the effect of each diagonal, therefore the greatest difference in the horizontal stresses in any two panels resolved along the diagonal in that panel in which the stress is the greatest will give the maximum stress in that diagonal, thus for the diagonal E g. System 2, the live load extends from a to g.

Diagonal Stresses.

System 1						System 2.					
Diagonal	Tensile Stress Tons	Area Required. sq. inches	No of Rods.	Dia. of Rod inches	Actual Area in Sq. inches	Diagonal	Tensile Stress Tons	Area Required sq. inches	No. of Rods.	Dia of Rods inches	Actual Area in sq. inches
Main						Main					
Ab	26.3	5.3	2	2.	6.3	Ac	30.5	6.1	2	2¼	8.0
Bd	24.3	4.9	2	1⅞	5.5	Ce	21.3	4.3	2	1⅞	5.5
Df	13.9	2.8	2	1⅜	3.0	Eg	12.5	2.5	2	1⅜	3.0
Fh	8.6	1.7	2	1⅜	3.0	Ig	10.3	2.1	2	1⅜	3.0
Jh	14.7	2.9.	2	1⅜	3.0	Ki	15.2	3.0	2	1⅜	3.0
Lj	21.4	4.3	2	1⅞	5.5	Mk	26.7	5.3	2	1⅞	5.5
Nl	30.9	6.2	2	2	6.3	Nm	16.0	3.2	2	1⅞	5.5
Counters						Counters					
Hf	8.0	1.6	2	1⅜	3.0	Ge	.7	.2	1	1⅜	1.5
Fd	3.7	.7	1	1⅛	1.0	Gi	5.7	1.2	2	1⅜	3.0
Hj	2.8	.6	1	1⅜	1.5	Ik	1.6	.3	1	1⅛	1.0
Db	not required.		1	1	.8	Ec	not required.		1.	1	.8
Jl.			1	1	.8	Km			1	1	.8

In the above table 5 tons per sq. inch is allowed for safe limit of tensile strains.

The Reaction at the left is 14.8. Tons, then for moments about g. we have $\frac{14.8 \times 72.2 - 2.1(21+42) - 1 \times 63}{20.3} = 43$ Tons horizontal stress in e.g: for moments about e we have $\frac{14.8 \times 51.2 - 21 \times 2.1 - 1 \times 42}{19.8} = 33.9$ Tons horizontal stress in ce: then $43.0 - 33.9 = 9.1$ the horizontal stress in Eg; resolved along its length $= 9.1 \times \frac{28.9}{21} = 12.5$ Tons.

The stress in the diagonal Ab is the horizontal stress in the 1st bay minus the horizontal component of the stress in the end post: as the end post being inclined assists in the compression of the chord. The ends of the rods are upset for the screw threads and the diameters in the table are the effective diameters.

The depth of the nuts are equal to the effective diameter of the bolts, which show them to be safe against stripping the threads.

The maximum stresses on the struts are determined similarly to the diagonals by resolving the difference between the horizontal stresses in 2 panels, vertically and then deducting the weight, directly borne by the strut. or really which passes into the diagonal, leaves the vertical stress on the strut. Thus for the strut Ee in system 2. we have the Reaction 14.8 Tons: taking moments about c. $\frac{14.8 \times 30.2 - 21}{18.2} = 23.4$ Tons horizontal stress in 1st Bay. taking moments about. e. $\frac{14.8 \times 51.2 - 2.1 \times 21 - 42}{19.5} = 33.9$ Tons: the difference is 10.5 Tons horizontal stress, resolved vertically is $10.5 \times \frac{18.5}{21} = 9.1$ Tons vertical stress in Bay ec. deducting 2.1 Tons the dead load leaves 7. tons.. the vertical stress on Ee.

The only stresses on the centre posts of each system are those due the vertical components. of the inclined chord stresses on either side, and which produce tension in the posts which however is very slight as the upper chord is nearly horizontal at these points.

The tables of the maximum strut stresses are given in the following table.

Stresses in Uprights

1st System			2nd System		
Upright	Comp. Tons	Tension Tons	Upright	Comp Tons	Tension Tons.
Bb	20.0		Cc	21.9	
Dd	10.7		Ee	7.0	
Ff	6.7		Gg		1.08
Hh		1.6	Ii	4.6	
Jj	12.2		Kk	12.7	
Ll	17.6		Mm	20.8	

The greatest compression is 21.9 Tons which needs $\frac{21.9}{4} = 5.4$ sq inches. the sectional area of the strut is ($10 \times \frac{1}{4}$ + 4 angle irons 2.1 area each) = 10.9 sq inches which is sufficient. All of the struts are of the same sectional area, and therefore sufficiently strong.

Applying Gordons formula $\frac{P}{S} = \frac{8000}{1 + \frac{l^2}{r^2 \times 9000}}$ in which $r^2 = \frac{I}{A} = 14.1$ and $l = 218''$ and finding the value of P we have P = 29.5 Tons which the strut is capable of resisting. At B. in the strut connection to the upper chord the steel pin is 2¼" diameter. For shear we must have $\frac{21.9}{6\times 2} = 1.8$ sq. in. as this is double shear; the area of the pin is 4" which is ample. For bearing we have $\frac{21.9}{6} = 3.6$ sq. in. required. $t = 14" + \frac{6}{8}" + \frac{1}{2} = 15\frac{1}{4}"$: $d = \frac{3.6}{15.14} = .2"$ required.

The strap to connect the strut to the pin is fastened by six rivets and although the strut also bears against the eye block, if the joint were not perfect the whole strain might come on the rivets. For shear $\frac{21.9}{4} = 5.5$ sq. inches required. The area of 7/8" rivet is .6 inches. $\frac{5.5}{.6} = 9$ rivets required. For bearing $\frac{21.9}{6} = 3.6$ sq. inches required $t = \frac{3}{8} + \frac{3}{8} + \frac{2}{8} = 1$, $1 \times 7/8$ = bearing area. $\frac{3.6}{7/8} = 4.$ rivets needed.

At D, the pin is 2" diameter and the stress is 10.7 Tons. For shear is required $\frac{10.7}{6\times 2} = .9$ sq. inches. The area of the pin is 3.1 sq. inches. For bearing $\frac{10.7}{6} = 1.8$ sq. inches required $t = 15\frac{1}{8}$. $d = \frac{1.8}{15\frac{1}{8}} = .1"$ diameter required. At the bottom of the uprights are steel pins 1¼" diameter which pass through all of the chords. For shear we need $\frac{21.9}{6\times 10} = .36"$ The area of the pin is 1.2 sq. inches.

For bearing we need $\frac{21.9}{6} = 3.6$ sq inches

$t = (\frac{1}{2} \times 6) + 3 + \frac{18}{8} = 8\frac{1}{4}''$. $d = \frac{3.6}{8\frac{1}{4}} = .4''$ diameter needed

In all cases the strengthening plates added where pins or joints occur, far exceed the loss of metal from drilling the holes for pins.

Cross Girder.

The diagram of the centre lines of the cross girder is shown on page 15 with the dimensions used for calculation. The part A.J. is first considered. The total length of A.J. is 24.2 ft. and each panel is 2.75 ft in length except the centre one, which is 2.2 ft. The effective depth is 3 ft. Each girder supports 1 panel load and the maximum chord stresses occur when the panel is fully covered by the live and dead load.

The dead load is the weight of the planking and floor beams which equals 7860 lbs. or $\frac{7860}{24.2} = 325$ lbs per lineal foot: or $325 \times 2.75 = 894$ lbs per panel of the girder. The live load per panel length is

$10\frac{1}{2} \times 60 \times 2\frac{3}{4} = 1522$ lbs.. The total load = 2416 lbs or 1.21 Tons. per panel. The middle panel sustains $\frac{1.21}{2} + \frac{1.05}{2} = 1.13$ Tons. The chord stresses are found by resolving horizontally, the vertical stresses in each panel, with the full load on the girder. and then combining these stresses which increase from the ends to the centre, being greatest in the centre: as the girder is symmetrical about the centre, the stresses on each side are the same.

The greatest stress on any diagonal and strut. is, when the live and dead loads cover the longer segment of the girder, up to that panel in which the stresses are required; and the remaining panels are covered with the dead load only. Thus the greatest stress on Cd and Dd is when the girder is loaded from J to D with live and dead loads, and the remainder with dead load. The vertical stress is then resolved along the diagonal by multiplying by the secant

of the angle made by it and the Vertical
The tables of maximum chord and diagonal stresses are given below, also for struts.

Strut, Chord, and Diagonal Stresses.

1 Chord Panel	2 Upper Chord Comp. in Tons	3 Lower Chord Tension in Tons.	4 Diagonals	5 Tensile Stress in Diagonals Tons	6 Required Area Sq inches	7 Size of Diagonal inches	8 Actual Area Square inches	9 Strut	10 Comp. Stress in Struts Tons	Required. Area sq inches	Actual Area in Sq inches
AB&IJ	4.38		Ab-Ji	5.47	1.09	4"×3/8	1.5	Aa&Jj	5.36	1.34	4.0
BC&HI	7.64		Bc&Ih	4.91	.98	3"×5/16"	.94	Bb&Ii	4.76	1.19	"
CD&GH	9.79		Cd&Hg	3.49	.70	2½"×7/16"	1.1	Cc&Hh.	3.63	.91	"
DE-EG	10.8		De&Gf	2.19	.44	2½×7/16"	1.1	Dd&Gg	2.59	.65	"
ab&ij		0	Ef&Fe	1.10	.22	2"×3/8"	.75	Ee&Ff	1.63	.41	"
bc&hi		4.38	Ig&Ed	.15	.03	none					
cd&gh		7.64									
de&fg		9.79									
ef		10.8									

In the above table 4 tons is used for compression and 5 tons for tension per square inch. The struts are all the same size composed of 2 angle iron 3"×3/8: the area = 4 inches which is sufficient. The greatest upper chord stress is 10.8 Tons. This needs $\frac{10.8}{4}$ = 2.7 square inches. The

actual area of the 2 angle irons is 4"; which is ample.
The maximum stress in the lower chord is 10.8 Tons
This requires $\frac{10.8}{5} = 2.2$ inches; the actual area is 4".
The rivets are 3/4" diameter. For shear in the diagonal Ab is needed $\frac{5.47}{4} = 1.37$ sq. inches
The area of a 3/4" rivet is .44": for double shear the number required is $\frac{1.37}{.88} = 2$ rivets which is the number used. For bearing we must have $\frac{5.47}{5} = 1.09"$
$t = 3/8 + 3/8 + 3/8 = 9/8$." dia. = 3/4" then $\frac{1.09}{9/8 \times 3/4} = 1.3$ rivets required.
In Bc the stress is 4.91 Tons. For shear we need $\frac{4.91}{4} = 1.23"$: for double shear $\frac{1.23}{.88} = 1.4$ rivets are needed. One is used. For bearing we need $\frac{4.91}{5} = .98$ sq inches
$t = 9/8"$ then $\frac{.98}{9/8 \times 3/4} = 1.2$ rivets required.
In the panel gh. the joint in the lower chord is connected by 2 angle irons 3"×3"×1/4" and 12 rivets. The total stress is 7.64 Tons, of which 1/4 or 1.91 Tons are considered as acting on the rivets in each horizontal arm of the angle iron and $\frac{1}{4} + \frac{1}{4} = \frac{1}{2}$ or 3.82 Tons on the rivets in the vertical arm. For shear in the vertical arm we need, double shear, $\frac{3.82}{4 \times 2} = .48"$ or

$\frac{.48}{.44}$ = 1 rivet required, 4 rivets are used.

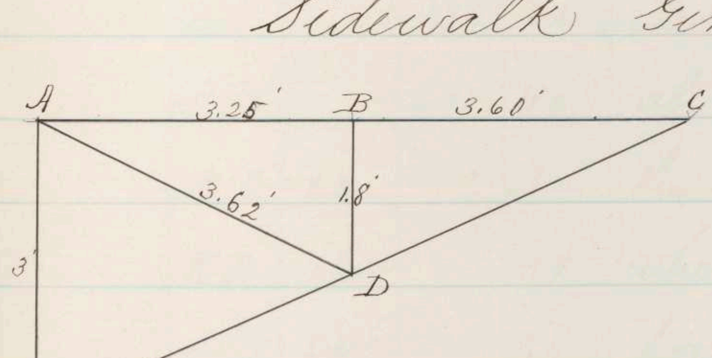
For bearing we need $\frac{3.82}{5}$ = .76 inches $t = \frac{1}{4} + \frac{3}{8} + \frac{3}{8} = 1''$ then $d = \frac{.76}{3/4}$ = 1 rivet needed.

For the horizontal arm, for shear we need $\frac{1.91}{4}$ = .48 sq. in. single shear: this requires 1 rivet

For bearing we need $\frac{1.91}{5} = .38''$ $t = \frac{1}{4} + \frac{3}{8} = 5/8''$

$5/8 \times \frac{3}{4} = \frac{15}{32}''$: then $\frac{.38}{\frac{15}{32}}$ = .75 rivets needed. In all these last shearing and bearing cases, 2 rivets at least must be used to make the connection.

Sidewalk Girder.



The sidewalk projection of the truss supports 3 floor stringers at A.B.C. at which the loads are concentrated. The dead load is the weight of planking and fence = 692 lbs or 700 lbs per foot. for the live and dead. the live being 6.86 × 10½ × 60 = 4116 lbs and the totals = 4808 lbs. For the panel at B the load is 3.42 × 700 = 1.2 Tons. At C the load is .63 Tons

The stresses as given in the table below were obtained by the graphical method for the braced arch given by Du Bois; the required and actual areas are also shown

Stresses in sidewalk girder.

Name of Piece	Comp Stress. Tons.	Tensile Stress Tons	Required Area
A.B		1.75	.35
B.C		1.27	.25
A.D		2.00	.40
B D	1.2		.3
E.D	1.96		.49
C.D	1.43		.36

2 Angle irons are used for all of these pieces. 2½" × ¼": the area is 1.5 × 2 = 3.0 sq inches which is ample; except for A D which is a flat bar 2¼ × ¼ = .6 sq inches.

The strain at A where the girder is attached to the main cross girder is 1.2 Tons.

For ~~tension~~ this requires $\frac{1.2}{5}$ = .24 inches area.

The area of 2 bolts and two rivets, each ¾" dia is 1.62 sq inches, which is sufficient to hold it. By comparing the stresses in the last table, with those of the cross girder. the former are so much smaller, that no calculation is necessary for the bearing and shearing areas of the rivets, as the rivets are placed as in the greatest stresses in the cross girder.

Roadway Stringers.

For the strength of the Roadway stringers 12 Tons were taken, as the maximum load concentrated on one floor beam. The bending moment is greatest midway between the cross girders; then

$M = 1/4\, Wl = \frac{1}{4} \times 12 \times 10.5 = 31.5$ Tons.

The moment of resistance is $\frac{fI}{y}$, taking f as 1000 lbs per square inch for wood. $= \dfrac{1000 \times \frac{4 \times 12 \times 12 \times 12}{12}}{6} = 48$ Tons. which show the stringers (4" x 12") sufficiently strong.

In the calculation for cross bracing, the effect of wind was taken at 20 lbs per square foot and the surface acted on $= (160' \times 3') + 20 = 500$ sq. ft.

The total force is $500 \times 20 = 1000$ ft-lbs. Considering the force as uniformly distributed we have $M = \frac{1}{8} Wl$

$= 1000 \times 20 = 10$ Tons at the centre of the bridge or 5 tons on each cross brace, at the centre which resolved along the cross brace $= 5 \times \frac{35}{27} = 6.5$ Tons

This requires $\frac{6.5}{5} = 1.3$ sq inches. The rods are 1¼" diameter and the area = 1.2 sq inches which are sufficeant nearly. The skew of the bridge was not considered as the difference in strain in the two trusses, occasioned by a moving load is very slight, being a highway bridge.

As a test for the bridge 4 large stone teams fully loaded were driven abreast, across the bridge at the same time but the weights were not known. The weight of a stone team very heavily loaded is 10 tons: this would make 40 tons live load on the bridge. No deflection could be noticed although careful observations were made with a level by the city engineer. The bridge was thoroughly inspected a few months ago and with the exception of a few loose nuts. (which were tightened up) everything was in good condition

From the investigations it may be concluded that the bridge and its connections are perfectly safe for the purpose for which they are intended.

May. 1877. Richard A. Hale