Central Avenue Bridge

Over the Neponset River,

Dorchester Mass.

Henry H. Carter. Class of "77.
Mass. Inst. Tech.

Index.

Central Avenue Bridge.

This bridge was designed by Mr. J. E. Cheney to cross the Neponset River on the line of Central Avenue, Dorchester (Lower Mills.) The river at this point is about 155 feet wide and is divided into two branches by an island which extends from the Lower Mill's dam, to a point about 400 ft. below. This island is about 30 ft. wide and affords an opportunity to build a pier on it, so that the bridge is divided into two spans whose lengths bear the relation to each other of 3 to 2. The bridge is a deck bridge, the lower chord being 18 inches above the level of the highest known flood of the river, and the top chord 1½ ft. below the level of the road. The slight depth of the bridge is necessitated by the short distance of the surface of the river from the bed of the road. The track of a branch of the Old

Colony Railroad crosses Central Avenue about 30 feet from the southerly end of the bridge, thus making it impossible to raise the grade of the avenue so as to give a more economical depth to the bridge. The Neponset river, owing to its short length and the nature of the country through which it flows, is not subject to freshets or to any great variation in level, so that in putting the lower chord only 18 inches above the surface or the river, no danger is apprehended of the bridge being carried away by a flood. As the depth of the bridge is so slight in comparison to its length it has been necessary to brace the girders together very strongly; a glance at the plan shows the manner in which this has been done. The bridge has a small angle of skew of 3° 13', or a deflection of $5\frac{9}{16}$ inches in every 8 feet 4 inches.

Iron Work.

Each span consists of six wrought-iron single intersection lattice girders placed 8 ft. 4 in. apart, from centre to centre, and conforming to the skew shown on the plan. The girders of the long span are 94 ft. long, those of the short span 63 ft. 6 in. long; the girders of both spans are 6 ft. 5 in. deep, measured from outside to outside of the angle iron in the chords. The 94 ft. span is cambered 3/4 in. and the 63 ft. span, $\frac{1}{2}$ in.
The chords and end-posts of the girders are of T shape section and are spliced as shown on the plan. All rivets used in the bridge are $\frac{13}{16}$ of an inch in diameter except in certain parts which will be mentioned further on. The girders are connected together at the top and bottom chords with lateral

bracing and at the ends and other points with vertical cross bracing. Holes are provided in the top chords of the gir-ders to receive 3/4 inch floor timber bolts. Each girder rests at its pier end on a cast iron bearing plate and at its abutment end on two cast iron roll-er-plates furnished with a set of turned wrought-iron rollers. The rollers are covered by a hood of leath-er fastened to the upper and lower roller plates. The web-plates of the bottom chords at each end of each girder are fitted to wrought-iron shoe plates to which the cast iron bearing-plates are connected. The stone at the pier and abutments is cut and dressed to receive the bearing and roller plates which are set with cement.

Each sidewalk is provided with a cast iron curb which is bolted to the woodwork with 3/4 inch screw-bolts. Cast iron scuppers are placed in the side-walk curb in order to drain away the water which may collect on the floor of the bridge. The following are some of the directions concerning the quality of the iron and the erection of the bridge. All plate-iron to be C No. 1. or iron of equal strength, to be straight and free from warps. All bar iron to be best bridge iron, straight, true section, and to have full smooth edges. All angle-iron to be of the best quality made and to have legs of uniform thickness. All rivets are, to be of the best quality, their sizes being understood to mean the diameter of the rivet when cold. The rivet holes are not to be more than $\frac{1}{16}$ inch larger in diameter than the cold rivet. All rivet holes are to fit accurately over each other so as to allow the easy passage of a rivet through them. The parts to be riveted are to be firmly bolted or clamped together, the rivets driven at a red heat, and in all cases they are to fill the holes. All loose rivets are to be cut out and replaced with others. All rivet heads are to be full size and neatly finished.

Wood Work.

The floor timbers are of yellow pine, 15 inches deep at the centre of the road, moulded to 12 inches in depth at the sidewalk curbs. The floor timbers that extend the whole width of the bridge are 6 inches thick and rest on the chords of the girder at panel points. Their projecting portions are chamfered as shown on the plan. The floor timbers that extend from outside to outside girders are 4 inches thick and rest on the chords of the girders midway between the 6 inch timbers.

The timbers are bolted to the girders with 3/4 inch bolts. The roadway planking is in two courses; the lower course of 4 inch yellow pine is securely spiked to the floor timbers; and the upper course of 2 inch spruce is spiked to the lower course. The sidewalk curb timber of yellow pine 7 in. by 8 in. is spike bolted to every floor timber with 3/4 inch bolts and is fitted with a cast iron curb and scuppers. The sidewalk stringers of yellow pine 3 in. by 9 in. are spike bolted to the floor timbers at bearings. The sidewalk plank is 2 inch white pine, tongued and grooved, and planed on one side to an even thickness. The fences are 4 ft. high and are made of white pine. The fence post are 4 in. by 6 in. fastened to each projecting floor timber and sidewalk stringer with one 3/4 in. screw bolt through each. Each post is let into the floor timber 2 in.

The upper fence rail is 4 in. by 6 in. mortised and pinned to the post. The middle rail 8 in. by 1½ in. and the lower rail 10 in. by 1½ in. are both let into the post ¾ of an inch. White pine facia boards 1½ in. thick are nailed to the posts. All timber used in construction must be sound, straight grained, free from all knots and imperfections, and of full size.

Paint.

All the surfaces of iron that come in contact have one good coat of paint before being put together and as soon as they are completed they are again painted. The paint used is made of linseed oil and mineral paint.

The fences and exposed sides of the facia boards are painted with three coats the last two coats being sanded.

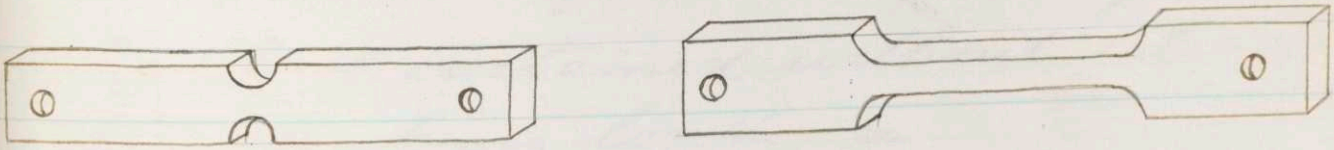
Masonry

The masonry includes two abutments, a pier, and a retaining wall which is connected with the abutment on the Dorchester side of the river. Trenches are excavated to the ledge-rock below the water-level of the river for the foundations of the southerly abutment and the pier. For the northerly abutment and the retaining-wall the trenches are excavated to solid earth. The top of the ledge-rock is roughly cut to a level surface so as to make a good bed for the foundation stones. The masonry is built of Quincy granite laid solid in cement mortar, which is composed of two parts of clean sand and one part of the best hydraulic cement. The exposed faces of the masonry are composed of large stones; one fifth part of the stones above the foundations are headers running clear through the wall from front to rear. The abutments, pier, and retaining wall have a batter on their faces of one inch in 24 inches and are levelled off to receive the bridge seat courses, the roadway and sidewalk curbs, and the coping of the pier, and retaining wall. The bridge-seat courses are rough hammered on all surfaces except the front and back vertical faces, which are straight quarry split. The bridge seat courses are two feet rise four feet six inches in width for the southerly abutment, and four feet in width for the northerly abutment. The roadway and sidewalk curbs conform to the crown of the roadway and pitch of the sidewalk. They are dowelled to the courses below, which in turn are dowelled to the next lower courses, with one and one half inch iron dowels, ten

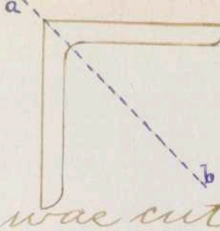
inches long; two are put in each stone.
The coping of the pier consists of stones
of two feet rise, 4 feet 4 inches in width
and are rough hammered on their
beds, and vertical joints.
The coping stone is held in place
by cramps of one half inch iron
well coated with coal tar, and
leaded in.

Determination of the strength of the iron used in the construction.

In accordance with a clause in the specifications which reads: "The contractor shall furnish suitable specimens of the iron he is using, when required by the engineer, that the same shall be tested"; experiments were made at the works of the Colts Fire Arms Company at Hartford, Conn. to determine the tensile strength of plate and angle iron used in the construction of the bridge. The iron experimented upon was divided into two classes, "Long Specimens", and "Short Specimens" and the samples were cut from actual pieces of the bridge iron.



Shapes used in Testing.



The angle iron was cut through the plane ab as shown in the figure and was then shaped to the forms drawn on page 12. The specimens used, were attached to a machine for testing iron by means of the holes drilled in their ends, and were carefully observed while the weights were gradually applied. The dimensions of the smallest cross section and the length between the shoulders were measured both before and after the testing, the results being given in the table. The weight at which a permanent set took place was noted and wts. were then slowly added until the iron gave way. The columns headed Elastic Limit per sq. in. and Tensile strength per sq. in. were found respectively from the load sustained without set and the breaking load.

	Before Testing			After Testing		Without set	Breaking load	Elastic limit per sq. in. of original cross sec.	Tensile strength per sq. in.	Reduction of Cross Sec. by breaking in Per Cent.	Elongation in per cent
	Cross section in in.	Area of C.S.	Length between shoulders	Length between shoulders	Area of Cross Sec.						
PLATES	1.117 X.369	.412	4.996	5.675	.384	71000	22050	26699	53519	6.4	13.5
	1.125 X.372	.419	4.995	5.400	.405	11500	20480	27446	48783	3.3	8.0
	1.117 X.372	.415	4.999	5.500	.354	13000	21740	31325	52385	15.0	10.0
	1.115 X.488	.544	5.000	5.550	.464	13000	26500	23897	48713	15.0	11.0
	1.112 X.484	.538	4.994	5.610	.470	12000	26200	22305	48699	13.0	12.0
	1.115 X.489	.545	5.000	5.665	.527	17000	27200	31193	49908	3.3	13.0
ANGLE IRON	1.113 X.369	.411	4.992	5.900	.326	9500	23220	23112	56496	21.0	18.0
	1.115 X.363	.405	5.000	5.590	.372	12000	25540	29630	63062	8.0	12.0

The above results were obtained from experiments on the long pieces.

1.119 X.378	.423	.380		.389		22000		54373
1.118 X.380	.425	.391		.380		23800		56000
1.120 X.480	.543	.383		.446		31260		57569
1.117 X.480	.536	.376		.508		27000		50373
1.114 X.375	.418	.380		.325		23720		56746
1.115 X.352	.392	.379		.334		21340		54438

Short Specimens

Limits of Stress.

The average ultimate strength of the iron tested was 54000 lbs per square inch of section, so that if we introduce a factor of safety of about $4\frac{1}{2}$ we obtain 12000 lbs per square inch for our limiting stress in tension. In the top chord 10000 lbs per square inch is the greatest allowable stress in compression.

Struts and other parts in compression are calculated by the formula

$P = \frac{f S}{1 + \frac{l^2}{c r^2}}$; in which P = the breaking load, S = the area of the section of the strut, l = its length, r = its least radius of gyration, and c and f being two constants which for wrougth iron equal 38000 lbs.

The greatest shearing stress for rivets is taken as 7500 lbs per square inch; for bearing, 12000 lbs per square inch is used. In the calculation of the struts, a factor of safety of 4 is used for the stresses produced by the rolling load of 100 lbs per square foot while a factor of $3\frac{1}{2}$ is used for those produced by the wagon load of 20 tons.

The calculations which follow refer to the long span of 94 feet.

Amount and Distribution of Load.

Each girder is calculated to sustain a live load of 100 lbs per sq. ft. of area supported by the girder

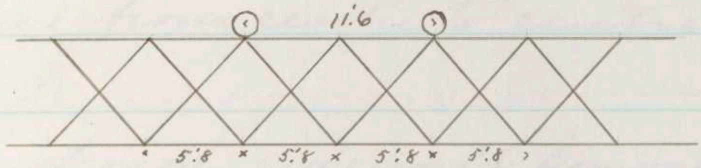
Also a dead load of 50 lbs per sq. ft. of area supported by the girder.

The stresses in both the upper and lower chords are calculated on the supposition that this live and dead load of 150 lbs extends over every sq. ft. of surface supported by the girder.

The girders are also calculated to sustain a load of 100 lbs per sq. ft extending over the bridge gradually from one end to the other, in addition to the dead load of 50 lbs per sq. ft extending over the whole bridge. As this method of loading gives comparatively small stresses in the middle diagonals a further supposition is made.

A 20 ton wagon is rolled from one end of the bridge to the other after the live load of 100 lbs per sq. ft.

before mentioned has been taken off.
The distance between the wheels of this wagon is 11.6 ft. so that when one of the wheels is over the apex of one system of diagonals, the other wheel is over the next apex of the same system



The width of the wagon is 8' 4" so that only 10 tons can come on one girder.
The stresses in the diagonals are calculated both for the wagon load and the rolling load of 100 lbs. per sq. ft. The stresses in any given diagonal are found by both of these methods of loading, and the amount of iron to be used in the diagonal is calculated from the greatest stress given by one or the other methods of loading. A dense crowd of people going over the bridge and the crossing of a heavy team, such as a road-roller, are provided for by these methods of loading.

Dimensions used in the Calculation.

Long Span; length between centres of support - 93 ft.
Width from outside to outside truss - - - 41 ft 8 in
Distance from centre to centre of each truss - 8 ft. 4 in
Width of each sidewalk from outside girder 4 ft 2 in
Total Width of bridge - - - - - - - - - - - - - - - - 50 ft.
Number of panels for long span - - - 16
Length of each panel - - - - - - - - 5.8125 ft.
Height from centre to centre of chords - - - 6 ft 3"
Ratio of Length to height - - - - - - - - - .93
Short Span: Length between centres of support - 62 ft 6 in
Number of panels - - - - - - - - - - - - - - 10
The centres of support of both spans are inside of the pier and abutment faces a distance of 15 inches

Calculation of Chord Stresses.

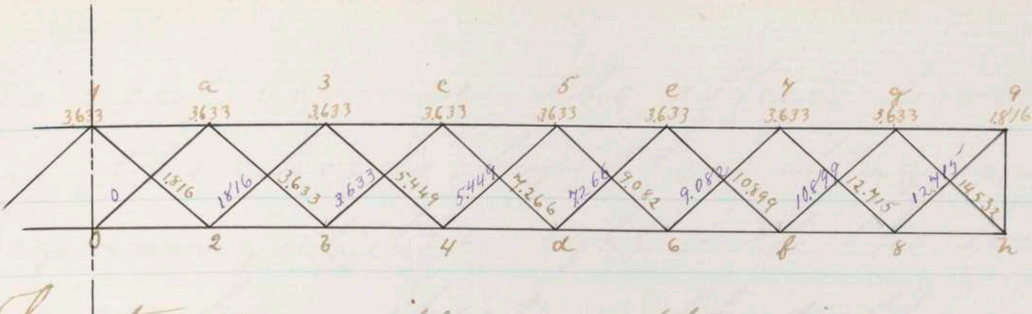
The stresses in the chords are calculated as before stated, on the supposition that every square foot of surface of the bridge is loaded with 150 pounds. The amount of weight on one girder is equal to its floor area multiplied by 150 lbs. The floor area supported by one girder is equal to the length of the girder multiplied by the distance between two adjacent girders.

93 x 8.3333 = 774.9969 sq. ft supported by one girder.

774.9969 x 150 = 116249.5 lbs supported by one girder.

116249.5 ÷ 16 = 7265.595 lbs distributed on one panel.

This panel weight collected at the apices of the diagonals, gives 3.633 tons on every apex, except the two at the ends of the girder which support only half this weight, or 1.816 tons.



The stresses written on the diagonals in the above figure are the vertical effects produced by the weights on the apices 1, a, 3, &c. Starting with the general principle that when the bridge is symmetrically loaded, the weights work from the middle towards the ends, we have one half the weight on the apex 1 producing a stress in the bar 1,2; this also acts in 2,3 and is then reinforced by the weight on 3 making a stress whose vertical effect on 3,4 is 5.449 tons. The stresses in the diagonals throughout this system are followed out in this manner, the resultants being written on the figure. The other system is similar, except that the whole of the weight on the apex "a" works towards the right.

We next consider the effects produced in the chords by these diagonal stresses. Beginning at the right with the lower chord we see by inspection that the only force tending to produce stress in 8.h, is the stress in g,h. The stress written on the diagonal is the vertical effect of the weight transmitted to the abutment through this diagonal. The ratio of the length of a panel to its height is .93, so that if we multiply 14.532 tons by .93 we obtain 13.51476 tons as the stress in 8.h

Passing now to the bar f,8 we see that the stress in it is made up of three components: 1st The stress in 8h which pulls on f,8; 2nd The stress in 7,8 resolved along f,8, and 3rd The stress in 8,9 also resolved along f,8.

13.51476 + 12.715 x .93 + 12.715 x .93 = 37.16466 tons

In the same manner the stress in 6.f is made up of the pull in f.8 and the stresses in ef and fg resolved along 6.f

37.16466 + 10.899 x .93 + 10.899 x .93 = 57.4368 tons

57.4368 + 9.082 x.93 + 9.082 x.93 = 74.32932 tons stress in d,6

74.32932 + 7.266 x.93 + 7.266 x.93 = 87.84408 tons stress in 4,d

87.84408 + 5.449 x.93 + 5.449 x.93 = 97.97922 tons stress in b,4

97.97922 + 3.633 x.93 + 3.633 x.93 = 104.73660 tons stress in 2,b

104.7366 + 1.816 x.93 + 1.816 x.93 = 108.11436 tons stress in 0,2

This brings us to the middle of the bridge so that 108 tons is the greatest stress in the lower chord.

Top Chord.

The stresses in the top chord are calculated in the same way; the stress in g,9 being produced by the pull from 8,9. and the stresses in the remainder of the bars being made up of three components as in the bottom chord.

12.715 x.93 = 11.82495 tons stress in g,9

11.82495 + 14.523 x.93 + 10.899 x.93 = 35.47578 stress in 7,g

35.47578 + 12.715 x.93 + 9.082 x.93 = 55.74699 tons " " e,7

55.74699 + 10.899 x.93 + 7.266 x.93 = 72.64044 tons " " 5,e

72.64044 + 9.082 x.93 + 5.449 x.93 = 86.15427 tons " " c,5

86.15427 + 7.266 x.93 + 3.633 x.93 = 96.29034 tons " " 3,c

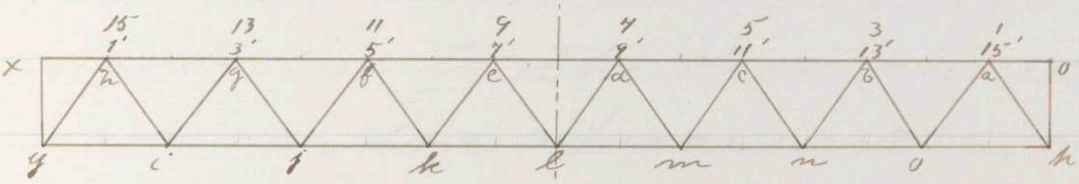
96.29034 + 5.449 × .93 + 1.816 × .93 = 103.04679 tons stress in a.3

108.04679 + 3.633 × .93 + 0 = 106.42548 " " " a 1

106 tons is therefore the greatest stress in the upper chord.

Diagonal Stresses.

We first calculate the stresses produced in the diagonals by the rolling load of 100 lbs. per sq. ft. of surface of the floor. As before, the surface supported by one panel amounts to 48.4373 sq. ft. A load of 100 lbs per sq. ft. of this surface gives a weight of 2.421 tons at each diagonal apex. A dead load of 50 lbs per sq. ft. of this surface gives a weight of 1.211 tons at each diagonal apex. As the two diagonal systems are entirely distinct from one another, each system is calculated separately. The greatest stress in any given diagonal occurs when the longer of the two segments, into which it divides the bridge, is loaded and the shorter segment is unloaded; or since the dead load is always on the bridge: when the longer segment is loaded with the dead and live load both, and the shorter segment loaded only with the dead load. For convenience in calculation we make use of "triangular numbers", which, for any series of numbers, are formed by successive additions of those numbers. We now proceed to calculate the stresses in the system drawn below



The apices on the top chord are numbered from the left towards the right and from the right towards the left with odd numbers as shown in the figure. Vertical lines drawn from all the apices onto the chords divide the girder into 16 equal lengths; the numbers above any apex show how many sixteenths of the length of the girder, the apex is from either end. By this method of numbering the apices we have a way of at once determining the maximum stress in any diagonal. For example, if we wish to determine the maximum stress in fg; we know that this greatest stress occurs when the long segment fo is loaded with both the dead and live load and the short segment fx is loaded only with the dead load, so that if we find the reaction at the left abutment due to the dead and live loads on the apices 1, 3, 5, 7, 9, 11, and the reaction at the right abutment due to the dead load on the apices 1' and 3'; the difference of these two reactions multiplied by the ratio of the length of the diagonal to its height will give the maximum stress in this bar. By means of triangular numbers we find the reactions at the abutments. Since the live and dead load extends from 0 to f, by the lever, we have 1+3+5+7+9+11 sixteen ths of the live and dead load going to the left abutment and 1+3 sixteenths of the dead load going to the right abutment.

1, 3, 5, 7, 9, 11, 13, 15 Apex numbers
4, 9, 16, 25, 36, 49, 64 Triangular numbers

That is $\frac{36}{16}$ of 3.632 tons goes towards the left and $\frac{4}{16}$ of 1.211 goes towards the right. 8.172 - .3028 = 7.8692 tons which multiplied by 1.365 the ratio of the length of the diagonal to its height gives 10.741458 tons as the maximum stress in the strut f j. By this process all the diagonal stresses are calculated.

Diagonal h, g.

$$1.365\left(\frac{1+3+5+9+11+13+15}{16}\times 3.632 - 0\times 1.211\right) = 39661.44 \text{ lbs.}$$

Diagonals h, i and g i

$$1.365\left(\frac{1+3+5+9+11+13}{16} \text{ of } 3.632 - \frac{1}{16} \text{ of } 1.211\right) = 30161.04 \text{ lbs.}$$

Diagonals g j and f j.

$$1.365\left(\frac{1+3+5+9+11}{16}\times 3.632 - \frac{1+3}{16}\times 1.211\right) = 21482.92 \text{ lbs.}$$

Diagonals fk and ke

$$1365\left(\frac{1+3+5+7+9}{16}\times 3.632-\frac{1'+3'+5'}{16}\times 1.211\right)=13632.8\text{ lbs}$$

Diagonals el and ld

$$1.365\left(\frac{1+3+5+7}{16}\times 3.632-\frac{1'+3'+5'+7'}{16}\times 1.211\right)=6609.33\text{ lbs}$$

Diagonals dm and mc

$$1.365\left(\frac{1+3+5}{16}\times 3.632-\frac{1'+3'+5'+7'+9'}{16}\times 1.211\right)=412.23\text{ lbs}$$

The diagonals ld, dm, and mc correspond with the diagonals el, ek, and kf so that the stresses found above for ld, dm, and mc are counter stresses produced when the short segment is loaded. A calculation for cn gives a minus result, showing that cn and the diagonals beyond never act as counters. This minus result in cn occurs because the dead load working towards the right abut-ment from the apices 1', 3', 5', 7', 9', 11' exceeds the dead and live load working towards the left abut-ment from the apices 1 and 3.

Summing up our previous results we have the following maximum stresses in the diagonals

Struts.

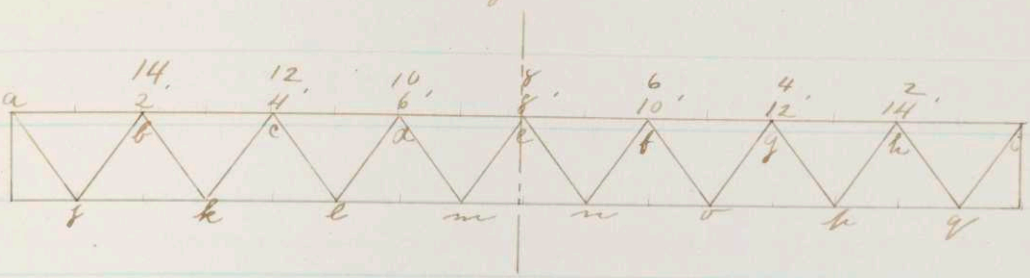
hg	39661.44 lbs.
gi	30161.04 lbs.
fj	21482.92 lbs
ek	13632.80 lbs
dl	6609.33 lbs
cm	412.23 lbs

Ties.

hi	30161.04 lbs
gj	21482.92 lbs
fk	13632.80 lbs
el	6609.33 lbs
dm	412.23 lbs

	Strut	Tie.
el	6609.33 lbs	6609.33 lbs.
ek	13632.80 "	412.23 "
fk	412.23 "	13632.80 "

Second System



The calculations for this system are made like those for the first system, the only difference being that the apices are numbered with the even instead of the odd numbers

Diagonal aj and bj. lbs.

$$1.365\left(\frac{2+4+6+8+10+12+14}{16}\times 3.632-0\times 1.211\right)=34703.76$$

Diagonals bk and kc

$$1.365\left(\frac{2+4+6+8+10+12}{16}\times 3.632-\frac{2}{16}\times 1.211\right)=25614.49$$

Diagonals cl and ld.

$$1.365\left(\frac{2+4+6+8+10}{16}\times 3.632-\frac{2+4}{16}\times 1.211\right)=17351.33$$

Diagonals dm and me

$$1.365\left(\frac{2+4+6+8}{16}\times 3.632-\frac{2+4+6}{16}\times 1.211\right)=9914.26$$

Diagonals en and nf.

$$1365\left(\frac{2+4+6}{16}\times 3.632-\frac{2+4+6+8}{16}\times 1.211\right)=3303.30\text{ lbs.}$$

The remaining diagonals give minus results showing that they do not act as counters.

Struts.

bf	34703.76 lbs
ck	25614.49 "
dl	17351.33 "
em	9914.26 "
fn	3303.30 "

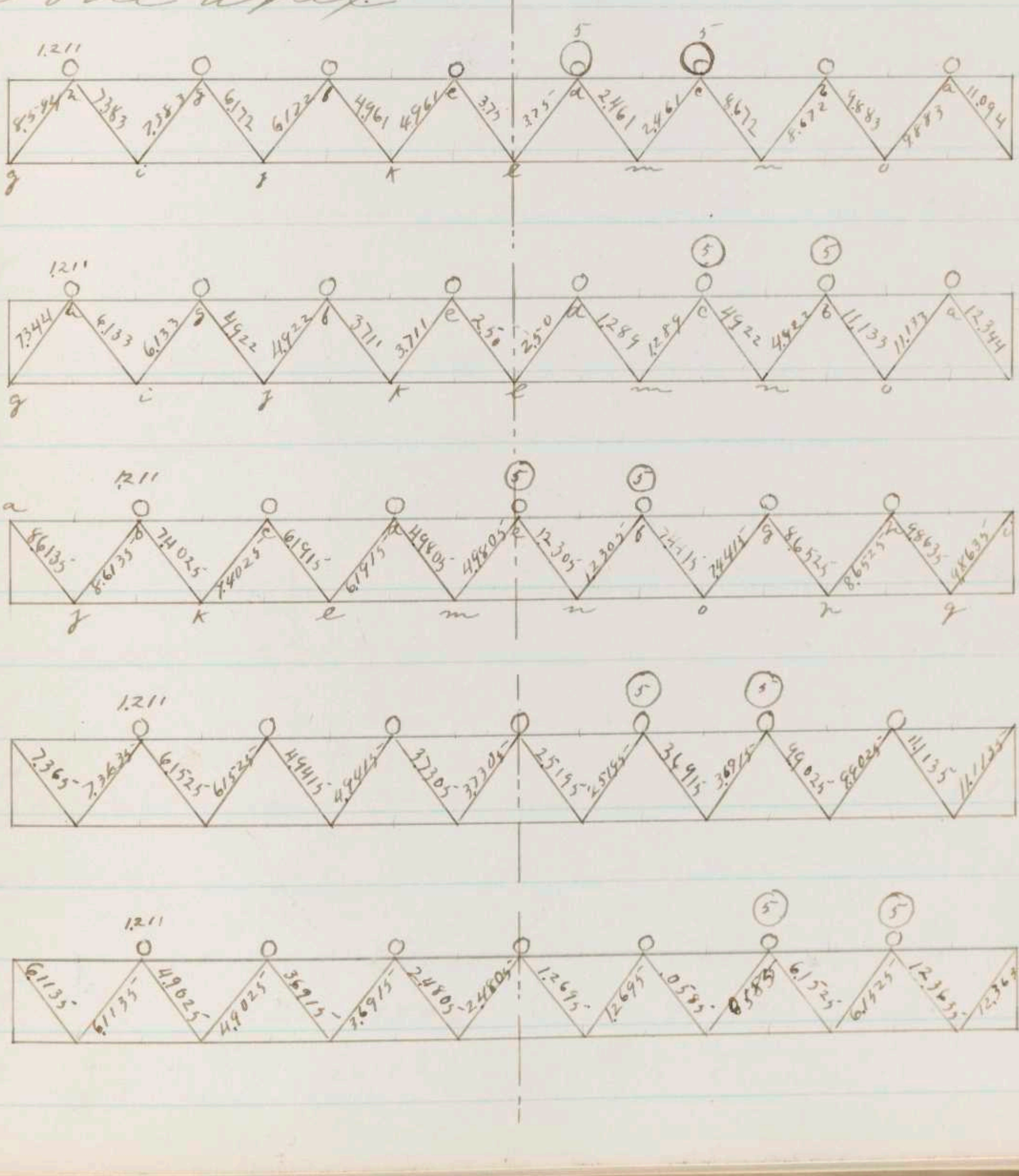
Ties.

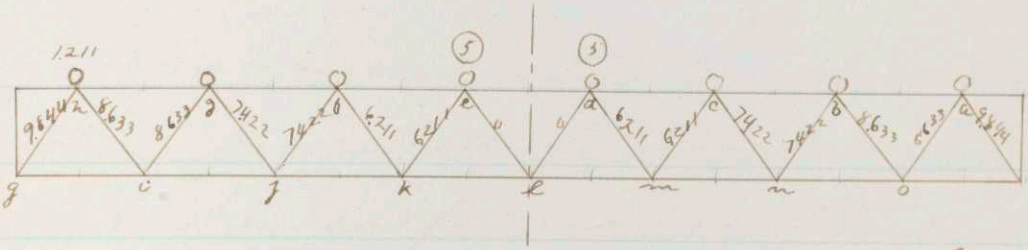
af	34703.76 "
bk	25614.49 "
cl	17351.33 "
dm	9914.26 "
en	3303.30 "

	Strut.	Tie
em	9914.26	3303.30
dm	3303.30	9914.26

Wagon Load Stresses.

As stated under "Distribution of Load" the wagon is supposed to weigh 20 tons, 10 tons being the maximum load ever coming on one girder, and 5 tons being the maximum load on one apex.





Summary of Maximum Stresses.
First System
Struts.

hg - - - - - - - - - - - - - - - - - - - 24688 lbs

gi - - - - - - - - - - - - - - - - - - - 22266 "

fj - - - - - - - - - - - - - - - - - - - 17344 "

ek - - - - - - - - - - - - - - - - - - - 12422 "

dl - - - - - - - - - - - - - - - - - - - 7500 "

cm - - - - - - - - - - - - - - - - - - - 2578 "

Ties.

hi - - - - - - - - - - - - - - - - - - - 22266.6 "

gj - - - - - - - - - - - - - - - - - - - 16744. "

fk - - - - - - - - - - - - - - - - - - - 12422 "

el - - - - - - - - - - - - - - - - - - - 7500 "

dm - - - - - - - - - - - - - - - - - - - 2578 "

	Strut	Tie.
el	7500	7500
ek	12422	2578
fk	2578	12422

By a comparison with the maximum strains produced by the live load of 100 lbs per sq ft. we see that the wagon load gives greater stresses in the bars ed, ek, and fk.

Second System.

Struts.

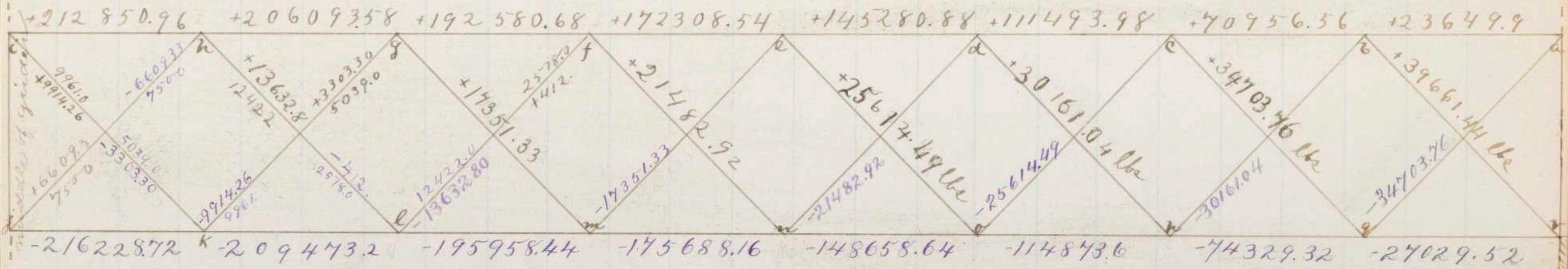
bg	24727 lbs
ck	19805 lbs
dl	14883 lbs
em	9961 lbs
fn	5039 lbs

Ties.

ag	24727 lbs
bk	19805 lbs
cl	14883 lbs
dm	9961 lbs
en	5039 lbs

	Strut	Tie.
em	9961 lbs	5039 lbs
dm	5039 "	9961 "

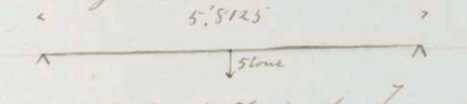
Stress Sheet



Black Ink indicates compression
Violet Ink indicates tension
Stresses without + or - signs are produced by the wagon load; those with signs are produced by the load of 100 lbs per sq. ft.

Bending Moment in the Top Chord.

By reference to the sheet containing the general elevation of the bridge, it will be seen that only alternate floor beams rest directly on the apices of the diag-onals, and that the floor beams be-tween rest on the chord midway be tween the apices. These beams resting between the apices produce a bending moment in the chord so that in determining the amount of iron necessary this bending moment must be considered as well as the direct thrust. The great-est B.M in the chord occurs when one of the wheels of the 20 ton wagon rests directly on the floor beam.



10000 x 17.437 = 174375 inch pounds = B.M.

This B.M. requires a large amount of iron to resist its action, but at the same time the total amount required in the chord is comparatively small owing to the small stresses due to direct compression. [It must be remembered that when the wagon load is on the bridge. the load of 100 lbs per sq. ft. is taken off.]

The total amount of iron necessary in the chord is greatest when we suppose the bridge loaded with 150 lbs. per sq. ft.; the bending moment being produced by that proportion of the live load of 100 lbs per sq. ft. which rests on the floor beams between the apices.

8.3333 × 1.4531 = 12.109 sq. ft. of floor area resting on the beam

12.109 × 100 = 1210.9 lbs producing bending

1210.9 × 17.437 = 21114.46 inch pounds. B.M

A small T shaped section containing .82 sq. in. in the vertical web, and .27 sq. in. in the horizontal flange would resist this B.M., so that as our chord is T shaped the addition of 1 sq. in. of iron to the amount necessary to resist the direct compression gives area enough to sustain the combined bending and compressive stresses.

Determination of Dimensions.

Top Chord.

As the horizontal distance between the apices of the diagonals does not exceed six times the width or depth of the chord, Gordon's formula for cross-breaking has not been used; the amount of iron necessary in any given bay being determined by dividing the stress in that bay by our limiting stress, and adding to this quotient one square inch to resist bending. In the following determination of dimensions, the letters naming any bar refer to the "Stress Sheet", on page 36 from which the stresses in the bars are taken. No allowance is made for rivet holes in the top chord as the rivets are supposed to fit the holes tightly and to transmit the stress as well as the solid plate.

Top Chord.

Table of Stresses and Dimensions. 40.

Bay	Stress	Iron necessary to resist the stress	Iron necessary to resist the bending action	Total amount of iron required	Form and dimensions of iron used in building the bridge	Total amount of iron used
	Pounds	Square Inches	Square Inches	Square Inches		Square Inches
ab	23649.9	$\frac{23649}{10000} = 2.36$	1.1	3.46	1 plate 12" x 5/16" 2 angle irons 3½" x 3½" x 10 lbs 1 web plate 11" x ½"	15.25
bc	70956.6	$\frac{70956}{10000} = 7.09$	1.1	8.19	1 plate 12" x 5/16" 2 angle irons 3½" x 3½" x 10 lbs 1 web plate 11" x ½"	15.25
cd	111493.9	$\frac{111494}{10000} = 11.15$	1.1	12.26	1 plate 12" x 5/16" 2 angles 3½" x 3½" x 10 lbs 1 web plate 11" x ½	15.25
de	145280.8	$\frac{145281}{10000} = 14.53$	1.1	15.63	1 plate 12" x 5/16" 2 angles 3½ x 3½" x 10 lbs 1 web plate 11" x ½" 1 plate 12" x 3/8 (halfway)	15.25 19.75
ef	172308.5	$\frac{172309}{10000} = 17.23$	1.1	18.33	1 plate 12" x 5/16" 2 angles 3½" x 3½" x 10 lbs 1 web plate 11" x ½ 1 plate 12" x 3/8	19.75
fg	192580.7	$\frac{192581}{10000} = 19.26$	1.1	20.36	1 plate 12" x 5/16" 2 angles 3½ x 3½" x 10 lbs 1 web plate 11" x ½" 1 plate 12" x 3/8" 1 plate 12" x 1/4" (halfway)	19.75 22.75
gh	206093.6	$\frac{206094}{10000} = 20.61$	1.1	21.71	1 plate 12" x 5/16 2 angles 3½ x 3½ x 11 lbs 1 web plate 11" x ½" 1 plate 12" x 3/8 1 plate 12" x 1/4"	22.75
hi	212850.9	$\frac{212851}{10000} = 21.28$	1.1	22.38	1 plate 12" x 5/16 2 angles 3½ x 3½ x 10 lbs 1 web plate 11" x ½" 1 plate 12" x 3/8 1 plate 12" x 1/4	22.75

In the bay de the 12"×3/8" plate only extends from the middle of the bay towards the centre of the bridge, so that the first half of the bay is smaller than the required amount of iron by .38 inches, while the other half is larger by 4.12 inches. In the same way the 12"×1/4" plate only extends over 1/5 of the bay fg so that 4/5 of the bay is too small by .61 inches and the other fifth too large by 2.39 inches. Throughout the whole chord the iron used exceeds the necessary amount in all cases, with the exception of the two bays just mentioned. In regard to de the difference between the necessary amount and the amount taken is too small to be noticed, but I should have thought it better to extend the 12"×1/4" plate over the whole of fg.

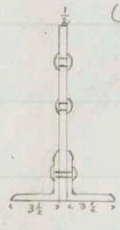
Bottom Chord.

Table of Stresses and Dimensions

Bay	Stress	Iron necessary to resist the stress	Form and Dimensions of iron taken.	Gross Area	Rivet Reduction	Net Area
rq	-27029.5	$\frac{27029}{12000} = 2.25$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs	11.5	2.0	9.5
qp	-74329.3	$\frac{74329}{12000} = 6.19$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs	11.5	2.0	9.5
po	-114873.6	$\frac{114874}{12000} = 9.57$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs	11.5	2.0	9.5
on	-148658.6	$\frac{148659}{12000} = 12.39$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs 1 plate 12" x ¼"	14.5	2.68	11.82
nm	-175688.16	$\frac{175688}{12000} = 14.64$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs 1 plate 12" x ¼ 1 plate 12" x 3/8	19.0	2.68	16.32
ml	-195958.4	$\frac{195958}{12000} = 16.33$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs 1 plate 12" x 1/4" 1 plate 12" x 3/8	19.0	2.68	16.32
lk	-209473.2	$\frac{209473}{12000} = 17.46$	1 web plate 11" x ½" 2 angles 3½ x 3½ x 10 lbs 1 plate 12" x ¼" 1 plate 12" x 3/8 1 plate 12" x 1/4"	22.	2.68	19.32
kj	-216228.7	$\frac{216229}{12000} = 18.02$	1 web plate 11" x ½" 2 angles 3½" x 3½" x 10 lbs 1 plate 12" x 1/4" 1 plate 12" x 3/8" 1 plate 12" x 1/4"	22.	2.68	19.32

In finding the area to be deducted on account of rivets in the first three bays, we see by an inspection of the lower chord that the number of rivets in any one plane perpendicular to the tensile stress in no case exceeds three

Diameter of rivet $= \frac{7}{8}''$ Thickness of plate $= \frac{1}{2}''$



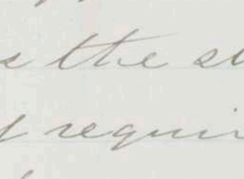
$\frac{7}{8} \times \frac{1}{2} + \frac{7}{8} \times \frac{1}{2} = \frac{7}{8}$ sq. inch " of angle iron $= \frac{13}{32}''$

$\left(\frac{13}{32} + \frac{1}{2} + \frac{13}{32}\right) \times \frac{7}{8} = \frac{147}{128}$ sq. inch $\frac{7}{8} + \frac{147}{128} = 2.02$ sq. inches

For the remaining bays the chord increases in area as we approach the centre of the bridge but as the struts and ties are smaller and require a less number of rivets, I have considered 4 rivets passing through the chord at the middle bay as giving the greatest reduction to be made for these bays.

$2\left(\frac{13}{32} + \frac{8}{32} + \frac{12}{32}\right) \frac{7}{8} = \frac{462}{256}$ sq. in.

$\frac{7}{8} \times \frac{1}{2} + \frac{7}{8} \times \frac{1}{2} = \frac{224}{256}$ sq in



$\frac{462}{256} + \frac{224}{256} = 2.68$ sq. inches

Ties.

Tie	Stress	Iron necessary to resist the stress	Form and Dimensions of iron taken	Gross Area	Rivet Reduction	Net Area
aq	-34703.7	$\frac{34703}{12000} = 2.89$	2 angle irons 3"×3" × 8⅓ lbs per ft.	5.0	.77	4.23
bp	-30161.1	$\frac{30161.1}{12000} = 2.51$	1 angle 5"×3"×10 lbs	3.0	.383	2.62
co	-25614.5	$\frac{25614}{12000} = 2.13$	1 angle 4"×3"×9½ lbs	2.85	.383	2.47
dn	-21482.9	$\frac{21482.9}{12000} = 1.79$	1 angle 3"×3"×8½ lbs	2.55	.38	2.17
em	-17351.3	$\frac{17351.3}{12000} = 1.45$	1 angle 3"×3"×8⅓ lbs	2.5	.392	2.11
fl	-13632.8	$\frac{13633}{12000} = 1.14$	1 angle 3"×3"×8⅓ lbs	2.5	.39	2.11
gk	-9961.0	$\frac{9961}{12000} = .83$	1 angle 2¾"×2¾"×6¾ lbs	2.025	.326	1.699
hj	-7500.0	$\frac{7500}{12000} = .63$	1 angle 2¾"×2¾"×6 lbs	1.8	.301	1.499
ik	-5039.0	$\frac{5039}{12000} = .42$	1 angle 2¾"×2¾"×6 lbs	1.8	.301	1.499
hl	-2578.0	$\frac{2578}{12000} = .22$	1 angle 2¾"×2¾"×6¾	2.025	.326	1.699

Also act as struts

Dimensions of Angle Iron	Thickness	Area cut out by 7/8" rivet
3"x3"x9 3/4 lbs. per ft.	.535 inches	.535 x 7/8 = .468 sq. in
3"x3"x8 1/2 " " "	.4600 "	.460 x 7/8 = .402 " "
3"x3"x8 1/3 " " "	.4480 "	.448 x 7/8 = .392 " "
3"x3"x7 1/2 " " "	.4000 "	.400 x 7/8 = .350 " "
5"x3"x10 " " "	.4375 "	.437 x 7/8 = .383 " "
5"x3"x13 1/2 " " "	.5000 "	.500 x 7/8 = .437 " "
4"x3"x9 1/2 " " "	.4375 "	.437 x 7/8 = .383 " "
2 3/4"x2 3/4"x6 3/4 " "	.3750 "	.375 x 7/8 = .326 " "
2 3/4"x2 3/4x6 " "	.3430 "	.343 x 7/8 = .301 " "

Column No. 3 is also the bearing area of a 7/8" rivet on angle irons of the dimensions given in the table; it also gives the numbers under "Rivet Reduction" on page 44.

Rivets.

Diagonal	Stress.	Rivet area for shear in sq. in	No. of 7/8" rivets	Sq. inches required for bearing	Bearing area of a single rivet.	No. of 7/8" rivets	No. of rivets taken
aq	34703.7	$\frac{34703}{7500} = 4.63$	8	$\frac{34703}{12000} = 2.89$	$.448 \times 7/8 = .392$	7	8
bp	30161.1	$\frac{30161}{7500} = 4.02$	7	$\frac{30161}{12000} = 2.51$	$.4375 \times 7/8 = .383$	7	7
co	25614.5	$\frac{25614}{7500} = 3.42$	6	$\frac{25614}{12000} = 2.13$	$.4375 \times 7/8 = .383$	6	6
dn	21482.9	$\frac{21482}{7500} = 2.86$	5	$\frac{21482}{12000} = 1.79$	$.460 \times 7/8 = .402$	5	5
em	17351.3	$\frac{17351}{7500} = 2.31$	4	$\frac{17351}{12000} = 1.45$	$.448 \times 7/8 = .392$	4	4
fl	13632.8	$\frac{13632}{7500} = 1.82$	3	$\frac{13632}{12000} = 1.14$	$.448 \times 7/8 = .392$	3	3
gk	9961.0	$\frac{9961}{7500} = 1.33$	3	$\frac{9961}{12000} = .83$	$.375 \times 7/8 = .326$	3	3
hj	7500.0	$\frac{7500}{7500} = 1.00$	2	$\frac{7500}{12000} = .63$	$.343 \times 7/8 = .301$	2	3
br	39661.4	$\frac{39661}{7500} = 5.28$	9	$\frac{39661}{12000} = 3.31$	$.535 \times 7/8 = .468$	7	9
cq	34703.7	$\frac{34703}{7500} = 4.63$	8	$\frac{34703}{12000} = 2.89$	$.460 \times 7/8 = .402$	7	8
dp	30161.1	$\frac{30161}{7500} = 4.02$	7	$\frac{30161}{12000} = 2.51$	$.400 \times 1/8 = .350$	7	8
eo	25614.5	$\frac{25614}{7500} = 3.42$	6	$\frac{25614}{12000} = 2.13$	$.500 \times 1/8 = .437$	5	6
fn	21482.9	$\frac{21483}{7500} = 2.86$	5	$\frac{21483}{12000} = 1.79$	$.4375 \times 7/8 = .383$	5	5
gm	17351.3	$\frac{17351}{7500} = 2.31$	4	$\frac{17351}{12000} = 1.45$	$.4375 \times 7/8 = .383$	4	4
hl	13632.8	$\frac{13632}{7500} = 1.82$	3	$\frac{13632}{12000} = 1.14$	$.448 \times 7/8 = .392$	3	3
ik	9961.0	= 1.33	3	= .83	$.343 \times 7/8 = .301$	3	3

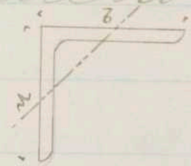
Column No. 4 is obtained by dividing the no. of sq. in. in No. 3 by .6" (area of a 7/8" rivet.)

Column No 5 divided by No. 6 gives No. 7.

Struts

In calculating the struts the formula $P = \frac{fS}{1 + \frac{l^2}{cr^2}}$ has been used, in which P equals the crushing load, S equals the sectional area in square inches, l equals the length in inches, and c & f equal two constants whose values for wrought iron are 38000 lbs. The above formula is obtained from the well known one of Gordon's by substituting for $\frac{l^2}{r^2}$ the quantity $\frac{l^2 S}{12 I}$; I, the least moment of inertia, being equal to $r^2 S$, in which r^2 is the square of the radius of gyration. The struts being fastened to the top chord through one flange only and at the same time being riveted to the intersecting ties make it a very doubtful matter as to the manner of their giving way. If a strut should bend out perpendicularly to the plane of the girder it would have to spring out the tie to which it is riveted, while on the other hand if it bent in the plane of the girder it would have to be considered as two struts each hinged at one end and and fastened at the other. This uncertainty as to the manner of the giving way of the struts has made it difficult to determine the proper radius of gyration in each case. In order to be on the safe side, however, that radius which gives the greatest amount of iron for the given stress has been taken without regard as to the probability of the strut giving way in any particular plane. The struts br, cq, dp, being formed of two angle irons riveted together have their moments of inertia taken about an axis which traverses the centre of gravity parallel to their longest sides.

The moments of inertia about this axis being divided by the respective areas of the struts gives .81 as the square of the radius of gyration. The remaining struts being formed of a single angle iron the neutral axis is taken so as to give the least value to r^2, which is calculated by the



formula $\frac{b^2h^2}{12(b^2+h^2)}$

In the table on page 50 the fourth column is obtained by multiplying the stress by the factor of safety. The breaking weight thus found is substituted in the formula in the fifth column and the areas of the different struts then found. The seventh column gives the areas actually taken to resist the stress.

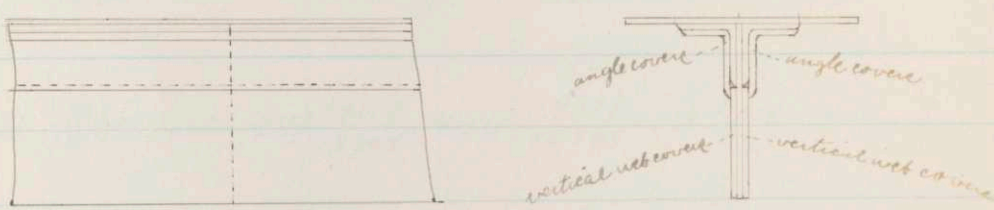
Struts.

Diagonal.	Stress.	Factor of Safety	Breaking Weight = Stress × Fac. of S.	area required $= \frac{P(cr^2+l^2)}{cr^2 \times f}$	Form and Dimensions taken	Area
br	39661.4	4	158645.6	5.6	2 angle irons 3"×3"×9¼ lbs.	5.85
cq	34703.7	4	138815.1	4.9	2 angle irons 3"×3"×8½ lbs	5.10
dp	30161.1	4	120644.2	4.2	2 angle irons 3"×3"×7½ lbs	4.50
eo	25614.5	4	102458.0	3.7	1 angle iron 5"×3½×13 lbs	3.90
fn	21482.9	4	879317	3.4	1 angle iron 5"×3"×10 lbs.	3.00
gm	17351.3	4	69405.3	2.7	1 angle iron 3"×4"×9½ lbs	2.85
hl	13632.8	4	545312	2.5	1 angle iron 3"×3"×8⅓ lbs	2.50
ik	9961.0 9914.1	3.5 4	34864.0 39656.4	1.8	1 angle iron 2¼×2¼×6 lbs	1.80
jh	7500.0 6609.3	3.5 4	30000.0 26437.2	1.4	1 angle iron 2¼×2¼×6 lbs	1.80
hk	5039.0 3303.3	"	20156.0	These bars require more iron when they act as ties.	1 angle iron 2¼×2¼×6¾ lbs	2.025
jl	2578.0		10312.0		1 angle iron 3"×3"×8⅓ lbs	2.50

Cover Plates.

The vertical web in each chord consists of 4 lengths of about 23 feet each. These lengths are spliced together by two vertical cover plates one on each side of the chord.

The angle iron in each chord consists also of 4 lengths; they are spliced together by two angle cover plates one on each side of the chord



From this figure it will be seen that whenever the vertical web is cut (and therefore spliced) the angle irons are cut also. The horizontal plates consist of various lengths breaking joint with those on each side so that they require no particular splicing.

Adjoining lengths of the vertical web, the angle irons, and the 12"×3/8" plate meet near the centre of the girder so that five cover plates are required. The figure on the preceding page shows four of these covers, the fifth cover being placed on top of the horizontal plates. The number of rivets in each cover plate is calculated in the following table

Cover Plate at Centre of Girder.

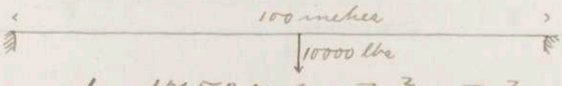
Part of Chord	Area	Rivet Reduction	Proportion of Stress	Area required for bearing	Area required for shearing	No of 7/8" rivets required	No. of rivets taken
Vertical Web	5.5	.875	$\frac{4.625}{19.32} \times 216228 = 51706$	$\frac{51706}{12000} x = 4.31$	$\frac{51706}{15000} = 3.45$	$4.3 \times \frac{16}{7} = 10$	11
Angle Irons	6.0	.71	$\frac{5.29}{19.32} \times 216228 = 59317$	$\frac{59317}{12000} = 4.86$	$\frac{59317}{15000} = 3.9$	$\frac{3.9}{.6} = 7$	8
Plate 12"×3/8"	4.5	.66	$\frac{3.84}{19.32} \times 216228 = 48154$	$\frac{48154}{12000} = 4.01$	$\frac{48154}{15000} = 3.2$	$\frac{3.2}{.6} = 6$	7

The web cover plates are 2'8" × 7½" × 7/16"
The angle iron cover plates are 3½" × 3" × 10 lbs per ft.
The horizontal cover plate is 5'4" × 12" × 3/8"
The cover plates calculated in the above table evidently require the greatest amount of iron and the greatest number of rivets so that in making the remain-

covers like the one at the middle we have greater strength than absolutely required.

Floor Timbers.

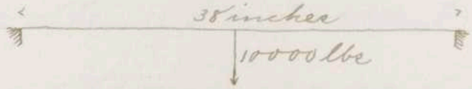
The floor timbers are of yellow pine. 15 inches deep at the centre of the roadway and moulded to 12 inches at the sidewalk curbs. On account of this difference in depth the cross section of the beams is variable so that their calculation as continuous over the girders would be very complex. Larger dimensions are also obtained by calculating the timbers as resting on the girders and not continuous over them. The passage of the 20 ton wagon evidently causes the maximum stresses in the timbers so that if they are proportioned to resist this stress they have a large surplus of strength when the only load on the bridge is the distributed load. Assuming the width of the timbers as six inches we obtain the necessary height by substituting in the formula $M_0 = \frac{fI}{y}$ or $mWl = nfbh^2$



$$\frac{1}{4} \times 10000 \times 100 = \frac{1}{6} \times 1750 \times 6 \times h^2; \quad h^2 = \frac{250000}{1750.0} = 143''$$

h = 12 inches.

The ultimate resistance of yellow pine to cross breaking is about 7500 lbs per square inch so that introducing a factor of safety of 4.5 we obtain 1750 for our value of "f". This factor of safety at first sight appears small but it must be remembered that not only is a load of this sort a very unusual one, but the timber is continuous over the girders also. It now remains to calculate the roadway planking. This may be done in two ways: either assume the dimensions of the planking and calculate the required spacing of the floor timbers, or assume the spacing and calculate the dimensions required for the planking; I have chosen the latter way as more convenient.



$mWl = nfbh^2$; $h^2 = \frac{mWl}{nfb}$; assuming $b = 12$ in.

$h^2 = \frac{6 \times 10000 \times 38}{4 \times 1750 \times 12} = 27. \therefore h = 5$ inches.

The upper course of spruce plank distributes the stress throughout the lower course so that in taking the depth of our planking as 4" we are still on the side of safety

Cross Bracing.

As the bridge is of slight depth in comparison with its length, the cross bracing has been made very strong so as to counteract the limberness which the bridge would otherwise have. Any attempt to calculate this bracing on the supposition of the wind's acting against the bridge would be useless, the bracing required to keep the bridge stiff being greatly in excess of that required to resist wind pressure. Experience has been the only guide in the matter of arranging the bracing both as regards position and dimensions so that no attempt is made here to discuss the subject.

Conclusion.

The strength of the Central Avenue Bridge compares favourably with that of any bridge in this city and at the same time greatly exceeds the strength of the majority of road bridges in this country whose dimensions are frequently calculated on as low a basis as a load of 40 lbs. per square foot. In conclusion I wish to express my indebtedness to Mr. J. E. Cheney of Boston, whose kindness and courtesy in answering all questions relating to this bridge, and the general subject of bridge building, have been of the greatest assistance to me.

H. H. Carter

May 11th 1877.