Thesis

Dover Street Draw-Bridge

May 17th 1877

Charles E. Stewart

Dover Street Draw Bridge, Boston.

I will commence by giving a brief description of the work, as described in the City Engineer's Report for the year 1877.

The plans for rebuilding this structure were prepared during the early part of the year 1876. On the 20th of July a contract was made with George H. Cavanagh, of Boston, who was the least bidder, to widen the bridge and rebuild all parts, except the draw, and to all work (except the laying of the pipes) required to put down siphons under the new draw-way, and to carry two lines of water pipes across Fort Point Channell in place of two lines removed from the old bridge.

The contract price for all work connected with the water pipes was $9000. and that for widening and rebuilding the bridge was $60000., which price I believe was very low as compared with some of the other bids.

On the 10th of August a contract was made with the Leighton Bridge and Iron Co. of Rochester N.Y. for building two lateral moving iron draws, the contract price being $14,800.

A contract was also made with the Atlantic Dredging Co. of Brooklyn N. Y. to do certain dredging in the new ship channel at a price of 17 cents per cu. yd. and after this was finished a further amount of material was removed by the same company at 20 cts. per cu. yd.

The bridge has been widened from 40 ft to 60 ft., a new opening has been made for the passage of vessels, which opening is 36 ft. wide in the clear and is at right angles to the centre line of the bridge, and the old opening has been closed; strong and convenient piers have been provided to facilitate the passage of vessels; suitable fenders have been built to protect the bridge from vessels and two sliding iron draws have been erected on heavy pile foundations. Both draws span the same channel, are side by side and each is half the width of the stationary part of the bridge. They are moved by horse power and withdraw to opposite sides of the channel.

The larger portion of the old bridge has been replaced with new work; a portion of the old piling which was in good condition has been retained.

The bridge as rebuilt is an oak pile bridge, the bents of piles on the new work are about 16 ft. apart while in the old bridge they were from 16 ft to 18 ft apart, the timbers are of Southern pine and the floor is covered with gravel and paved with small stone blocks.

Great precaution seems to have been taken to secure a water tight floor; it being laid with seasoned hard pine planks 6 inches thick, caulked and payed, covered with a layer of asphalt and a top layer of coal-tar concrete. A large number of lead scuppers were provided to drain the gravel, the lead scuppers being 1½ inches inside diameter, there being one scupper for every 100 sq. ft. of bridge surface.

The Foundations for the Draws.

There are of course two foundations, one for each sliding draw, one being on one side of the opening for vessels and the other on the other side of this opening. These two foundations are alike in construction and they are, as before mentioned pile foundations; the piles of these draw foundations were capped with 12"×12" solid hard pine caps, and great care was taken to secure a good bearing on the piles.

The lines of solid caps rest at each end upon lines of double girder caps running the other direction and the caps are bolted to them, with 1 inch screw bolt through each of the double girder caps.

The double track stringers on which the trucks for moving the draws roll by means of rails, are severally 18 in. by 18 in. and 18 in by 16 in.; the lower one is notched down 1 in. and bolted to the solid caps with 1 in screw bolts one to each cap and the two stringers are bolted to each other with 1¼ in. screw bolts, one every 4 ft. the heads of the bolts being countersunk and oak keys 4 in. square are fitted and driven between each two bolts.

The lines of track stringers are separated by hard pine blocks 5 in by 8 in. one at every 8 ft.; and the stringers are bolted together with 1 in screw bolts, there being two bolts to each stringer for each block.

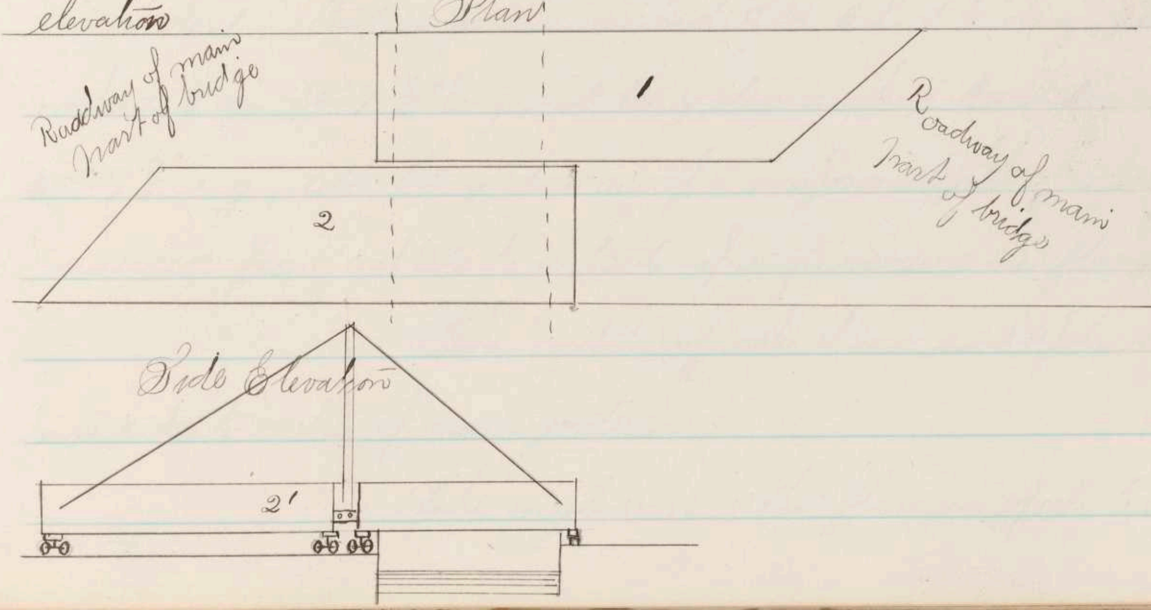
The draw foundations are floored over with 4 in spruce planking secured to the solid caps.

The Two Iron Sliding Draws.

As this was the part of the bridge which I selected for my work. I will confine myself in the rest of my space to them alone.

This style of a drawbridge, that is one consisting of two separate draws, entirely alike in construction and each working separate, one withdrawing to one side of the opening and the other to the other side, is used as far I know only in this vicinity and it has several advantages which other drawbridges do not have, for instance if one of the draws is out of order the other can still be used while the other is being repaired.

In order to more clearly show their construction I will make a brief sketch in plan and in side elevation



The figures on the page before represents the position of the draws when the opening is closed; now in order to open the draw "1" is slid upwards and to the right and "2" slides downwards and to the left.

Each draw consists of four plate girders supporting the floor and not of two each continuous over the middle support as might at first be supposed, but the two girders composing each side of the draw are not entirely independent for they are connected in the middle and the span over the channel is supported when the draw is being moved by means of rods passing over Samson posts.

Description of the parts made of Iron.

The main girders of the spans composing the draws are wrought iron plate girders of the lengths shown on Sheet No. 1; that is girder marked C is 63'–5 $\frac{1}{8}$" long, girder marked B is 26'–9 $\frac{3}{4}$" long and the girders marked A and A' are each 40'–9 $\frac{7}{8}$" long; all the girders are of a uniform depth of 4 feet 6 inches measured from outside to outside of angle irons in the flanges.

The width of each draw is 28 feet, measured from centre to centre of main girders.

The angle irons in the flanges of all the girders are 4 in. by 4 in., weighing 11 pounds per ft. and all the flange plates are 12 inches wide.

In the girders marked A and A' on Sheet No. 1. the top flange plates are $\frac{3}{8}$ inches thick and the bottom flange plates are $\frac{1}{4}$ of an inch thick.

In the girder marked B. the top flange plate is $\frac{1}{4}$ of an inch thick, and in the girder marked C the top flange plates are $\frac{5}{16}$ of an inch thick. The position of the flange splices are shown on Sheet No. 1 and the details of the same on Sheet No. 2.

The lengths and thicknesses of the web plates in the main girders are shown on Sheet No. 1 and also the cover plates and angle irons which serve to splice and stiffen them, two cover plates being used to each web splice.

The rivets used in the girders are $\frac{13}{16}$ of an inch in diameter except where otherwise stated or shown on the drawings. The girders A and A' each have at suspended end two cast iron bearing plates; one connected to the girder and the other fitted and bolted to the draw landing of the pile bridge. The other ends of these girders are connected to the cast iron bearing box at the foot of samson post by means of a pin joint.

Each end of girders B and C is fitted with cast-iron bearing boxes which have sockets to receive the truck frame bearings. All the faces of these castings to which any piece is riveted are planed, and all the rivet holes in them were drilled and not punched.

The floor beams of these draw-bridges are of an I formed section being made up of vertical web plates and flanges formed of plates and angle irons. The general floor beams marked "O" on Sheet No. 1 are of proper lengths to keep the distances from centre to centre of the main girders 28 ft. and they are 21 inches deep measured from outside to outside of the angle irons in the flanges. The web plates of these floor beams are $\frac{3}{8}$ of an inch thick and are made in one piece. The top flanges of these floor beams are made up of two $3\frac{1}{2}$ in. x $3\frac{1}{2}$ in angle irons weighing 10 lbs per ft. and one 9 in x $\frac{3}{8}$ in. plate extending 5 ft 6 in each side of the middle of the beam. The bottom flanges are made of two $3\frac{1}{2}$ in. x $3\frac{1}{2}$ in. angle irons weighing 10 lbs per ft.; the rivets in the flanges of these beams are $\frac{3}{4}$ of inch in diameter and are 4 inches apart.

Now on account of the peculiar shape of the draws the floor beams are not all alike, so I will give the dimensions and sections of the special floor beams.

The floor-beams marked Q and R are of the same section as floor beams "O" and differ from them only in regard to length. The floor beam marked S is 18 in deep, the web plate being $\frac{5}{16}$ of an inch thick; the top flange is made of two 3 in. x 3 in angle irons weighing $9\frac{3}{4}$ lbs per foot and the bottom flange is made of two 3 in x 3 in angle irons weighing $7\frac{1}{4}$ lbs per ft. The rivets in the flanges are $\frac{3}{4}$ of an inch in diameter and 3 in. pitch.

The floor beam marked T is 18 inches deep, the web plate is $\frac{1}{4}$ of an inch thick and each flange is made of two 3 in x 3 in angle irons weighing $7\frac{1}{4}$ lbs. per ft. The rivets in the flanges are $\frac{3}{4}$ of an inch in diameter and are $3\frac{1}{2}$ inches apart.

Floor beam marked U is 18 inches deep, the web plate is $\frac{1}{4}$ of an inch thick and each flange is made of one 3 in x 3 in angle iron weighing $7\frac{1}{4}$ lbs per ft. The rivets in the flanges are $\frac{3}{4}$ of an inch in diameter and 4 inches apart.

The floor beam V is formed of one 8 in I-beam, weighing 65 lbs per yd.

Floor beams marked W and W′ are 21 inches deep, the web plates being $\frac{3}{8}$ of an inch thick, and each flange is made of two 3 in. x 3 in. angle irons, ~~wt~~ weighing $7\frac{1}{4}$ lbs per foot.

The rivets in the flanges of these floor beams are $\frac{3}{4}$ of an inch in diameter and 4 inches apart. The floor-beams ττ and ττ' are connected together at the centre line of the draw, and are fitted with a cast cast-iron bearing-plate which rests on the truck frame.

In the Specifications to contractors this clause was inserted in regard to the floor beams. "All floor beams are to be straight and true to dimensions. They are to be carefully riveted to the main girders and other floor-beams. Care is to be taken not to warp or spring the main girders out of line in riveting the floor beams to them. All the floor beams are to be provided with holes in the top flanges for the floor-stringer and sidewalk timber bolts, as shown on sheet No. 1

Both the spans of each draw are braced laterally between the girders and floor beams as shown on Sheet No. 1.

The general bracing consists of tie-rods and angle-iron struts. The struts are 3 in × 3 in, angle irons weighing $7\frac{1}{4}$ lbs per foot; the tie-rods in the suspended span are $1\frac{1}{8}$ inches in diameter with $1\frac{3}{8}$ inch diameter screw ends, and in the other span they are 1 inch in diameter with $1\frac{1}{4}$ inch diameter screw-ends.

The details of the connections of the general lateral bracing to the girders and the floor beams are shown on Sheets 2 and 3. The lateral bracing in the triangle suspended by the special floor-beams consists of angle-irons and rods, which are shown on sheets 1 and 3.

The Samson posts are each made of two 10 in. channel beams, weighing 69 lbs. per yd., which are latticed together with $2\frac{1}{2}$ in $\times$ $\frac{3}{8}$ in lattice bars spaced as shown on Sheet 2. The rivets used in this lattice work are $\frac{3}{4}$ of an inch in diameter and each post has a cast iron cap fitted to its top end.

The posts are connected together at the top with a wrought iron lattice cross strut, $24\frac{1}{2}$ inches deep. The flanges of the cross strut are each made up of two $3\frac{1}{2}$ in $\times$ $3\frac{1}{2}$ in. angle irons, weighing $8\frac{1}{2}$ lbs per foot, and one 10 in $\times$ $\frac{1}{4}$ in. plate. The lattice bars of the web are 3 in $\times$ $\frac{1}{4}$ in. and the rivets in the strut are $\frac{3}{4}$ of an inch in diameter. Each post has a corner bracing connected to it and to the cross strut; the web-plate of the corner brace is $\frac{1}{4}$ of an inch thick and the angle irons are 3 in. $\times$ $3\frac{1}{2}$ in. weighing $7\frac{2}{3}$ lbs per foot. The details of the cross strut, corner braces and top end of samson post are shown on Sheet 2.

There are two suspension rods in each set from the ends of the girders over samson post. The suspension rods from ends of girders A and A' are $1\frac{5}{8}$ inches in diameter, with 2 inch screw ends in turnbuckles; the rods from the end of girder B are $1\frac{3}{4}$ inches in diameter with $2\frac{1}{8}$ in. screw ends in turnbuckles, and the rods from the end of girder C are $1\frac{5}{8}$ inches in diameter with 2 in screw-ends also in turnbuckles.

Each rod has one pipe turnbuckle, whose strength is equal to that of the rod. The turnbuckles are made of strong wrought iron welded pipes. The pins which connect these rods with the girders and the top of the samson post were turned to the proper size and the holes in the eyes of the suspension rods were first drilled and then bored.

At the end of girder B as shown on Sheet No. 1 a number of pieces of cast iron, weighing about 17000 lbs are attached to the girder, this weight acts as a counterbalance, obviating the tendency of the greater weight resting on the girder "A" to lift up the girder B.

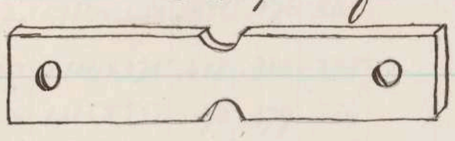
Each draw has a cast iron roadway and sidewalk curve the latter curb being provided with a number of scuppers as shown on Sheet No 1.

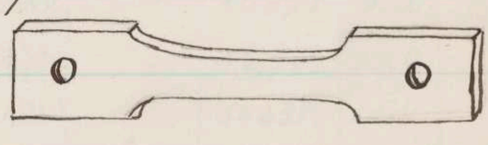
In the Specifications to contractors it was stated that all rivet holes used in the construction of the bridge should not be more than $\frac{1}{16}$ of an inch larger in diameter than the cold rivet.

A set of experiments was made in order to find out the effects of tensile stress on specimens of iron similar to those used in this bridge and others in Boston, an account of which I will give below.

Tabulated statement of the results of tests by The Colt's Patent Arms Manuf. Co. of the effects of tensile stress on specimens prepared from ten pieces of plate and two of angle iron received Dec. 21st 1876 from the City Engineer of Boston and tested for him.

Shapes of the Specimens





Reference Number	Original mark on the Sample piece	Proportions of the Specimens — Before Testing — Minimum cross section	Minimum Area of cross section	Length between Shoulders	After fracture — Minimum Area of Cross Section	Length between Shoulders	Greatest stress sustained without permanent set.	Breaking stress	Elastic limit per sq in of original cross section	Tensile strength per sq in of original cross section	Reduction of cross section by breaking	Ultimate Elongation
		Ins.	sq. Ins	Ins.	sq. Ins	Ins.	Lbs.	Lbs.	Lbs.	Lbs.	Perct.	Perct.
		"Long Specimens"										
858	•	1.117x.369	.412	4.996	.384	5.675	11,000	22,050	26,669	53,519	6.4	13.5
859	•	1.125x.372	.419	4.995	.405	5.400	11,500	20,480	27,446	48,783	3.3	8.0
861	• •	1.117x.372	.415	4.999	.354	5.500	13,000	21,740	31,325	52,385	15.0	10.0
863	• • •	1.115x.488	.544	5.000	.464	5.550	13,000	26,500	23,897	48713	15.0	11.0
864	• • •	1.112x.484	.538	4.994	.470	5.610	12,000	26200	22,305	48,699	13.0	12.0
866	• • • •	1.115x.489	.545	5.000	.527	5.665	17,000	27,200	31,193	49,908	3.3	13.0
868	• • • • •	1.113x.369	.411	4.992	.326	5.900	9,500	23,220	23,112	56496	21.0	18.0
870	• • • • • •	1.115x.363	.405	5.000	.372	5.590	12,000	25,540	29,630	63062	8.0	12.0
		"Short Specimens"										
860	•	1.119x.378	.423	.380	.389			22000		54,373	8.0	
862	• •	1.118x.380	.425	.381	.380			23800		56,000	16.0	
865	• • •	1.120x.485	.543	.383	.446			31,260		57,569	18.0	
867	• • • •	1.117x.480	.536	.376	.508			27000		50373	5.2	
869	• • • • •	1.114x.375	.418	.380	.325			23,720		56,746	22.0	
871	• • • • • •	1.115x.352	.392	.379	.334			21,340		54438	15.0	

The stresses were applied gradually in all cases. Specimen # 865 mark ... broke through the back of the eye with 29 860 lbs. Plates were then welded over the break, and the piece was broken at the neck with 31260 lbs.

The original distance between the centre of the eye and the end of the specimen was $1\frac{7}{8}$ inches, the diameter of the hole 1 inch.

The numbers obtained by dividing the breaking stresses by the areas of the least cross sections of the specimens, measured after fracture, are as follows

#858	859	860	861	862	863	864	865	866	867	868	869	870	871
57422	50568	56,555	61,440	62,632	57,112	55,744	70,090	51,613	53,150	71,165	72,985	68,656	63,893

Office of The Colt's Pat. Fire Arm Mfg. Co.
Hartford Jan 12, 1877
C. B. Richards
Engineer

Description of the Parts of this Bridge made of Wood.

The floor stringers are made of yellow pine, 4 inches thick and from 12 in. to 15 in. deep giving a crown of 3 inches to the roadway as shown on Sheet No. 2. They are placed 2 feet and 6 inches apart from centre to centre and the lengths are shown on Sheet No. 1; the stringers are bolted to the floor-beams with $\frac{3}{4}$ in. screw bolts.

There are a number of extra, short stringers placed in as shown on Sheet No. 1; these are placed where the scuppers occur because the scuppers cut through and therefore weaken the main stringers.

The spaces between the stringers over the floor beams π and π' are filled with 7 in. timbers of the same depth as the floor stringers.

The flooring of the draws is made of two courses of planks. The lower course is formed of 4 in. spruce planking, each plank extending the entire width of the course and they are spiked to the floor stringers with $\frac{1}{2}$ inch wrought spikes, there being 2 spikes at each end of the planks and one at the other bearings. The upper course of planking is made of 2 inch spruce.

The sidewalk on each girder is 8 ft wide measured from centre of girder to the edge of the curb and it has a pitch of 2 in.; it is shown in detail on Sheet No. 2.

The sidewalk stringers are made of yellow pine and are 3 in x 12 in and they rest on 8 in x 12 in spruce timbers.

The curb timber is made of white pine and is 6 in x 9 in and a cast iron curb and scuppers are fitted to it. The sidewalk planking is also formed of white pine 2 in. thick and it is tongued and grooved.

Apparatus for Moving the Draws.

Each draw is fitted with two moving gears, one for hand and the other for horse power. The Rack for the horse power moving gear is shown on Sheets 1 and 3.

I will explain briefly the manner in which the draws are moved. The horse in the horse-house passes round and round in a circle, thus turning a crank to which he is harnessed; at the bottom of the vertical rod of the crank is a cog-wheel which connects with another cog-wheel on the end of a line of horizontal shafting, at the other end of the line of shafting is another vertical cog-wheel which acts on the lower of two horizontal cog-wheels attached to the opposite ends of a vertical axle; the upper one of these two horizontal cog-wheels works directly in the vertical teeth of the rack, thus applying a sliding force to the draw, by which it is moved sidewise through the intervention of the track and trucks.

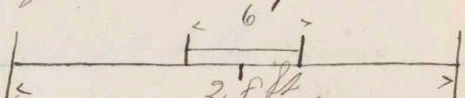
The wooden track stringers were accurately planed and leveled to the proper grades and on them were fastened steel rails on which the trucks of the draw move, these steel rails weigh 60 lbs. per yard.

At first there was great trouble in moving the draws and on investigation it was found that the tracks were laid a little too close together, thus causing the flanges of the wheels to squeeze up against them. this difficulty was overcome by laying the rails a little farther apart thus giving the wheels more play.

Calculations on the Two Sliding Draws

The Data for Calculation received from Mr. Cheney

On the General Floor-beams, 28 ft. long he made his calculations taking as a dead load, uniformly distributed over the surface of the draw, 40 lbs. per square foot. and for a live load of 100 lbs per square foot. He also made the calculations supposing that instead of the above live load, that a 20 ton wagon passes over the floor beam as is shown in the sketch



and used the stresses that produced the worst results.

For a Uniformly Distributed load he allowed as a working stress 12000 lbs per sq. in for tension and 9000 lbs per sq. in for compression.

For the calculations on the Girders he took as a Dead load 50 lbs. per square foot of the surface of the draw, this load including the weight of the trusses so it was not necessary to correct the results for the weight of the girders; for a Live load he also used here 100 lbs per sq. ft. of the surface of the bridge, but in the calculations for the shears due to the live load he substituted the 20 ton wagon.

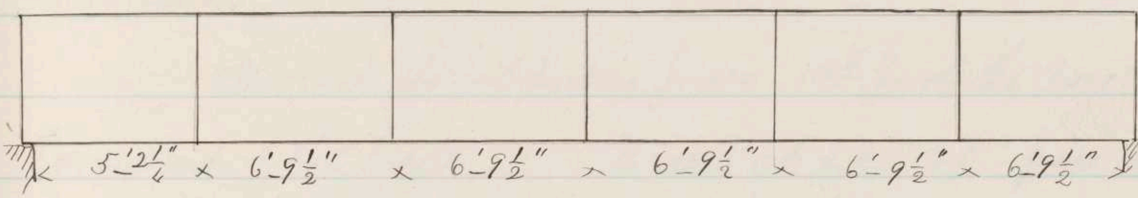
The depth of the trusses that was used

was 4'3".   For shearing of rivets 7,500 lbs per sq. in was allowed and for crippling 12000 lbs per sq in

Girders A and A'.

These two girders are just alike. Take the span for calculations from <u>centre of bearing on right to pin in centre</u>; hence span for calculations = 40'9" − 1'7$\frac{1}{4}$" = 39'1$\frac{3}{4}$"

Live load = 160 lbs per sq. ft. of the surface of the draw. and Dead Load = 50 lbs per sq. ft, which includes the wt of the trusses.



In calculating this girder the loads were supposed to act where the floor beams were connected with girders, that is at the intersections of the panels. Each girder A and A' supports a load equal to one half of that born by 5 of the floor beams marked "O"   Width of draw = 28 ft; half width = 14 ft.

Dead Load acting on each girder = 5 × 6'-9$\frac{1}{2}$" × 14' × 50 lbs = 23770 lbs

6'-9$\frac{1}{2}$"
5
30
3 11$\frac{1}{2}$
33'11$\frac{1}{2}$"

9$\frac{1}{2}$
5
47$\frac{1}{2}$" = 3' 11$\frac{1}{2}$"

34'
14
136
34
475 sq ft.
50
23800 lbs
30..
23770 lbs = Dead load

168" × $\frac{1}{2}$ = 84 sq in
124)50 (.34 lbs on sq in
84
30. lbs on $\frac{1}{2}$" in length

Total Live load = $5 \times 6'9\frac{1}{2}" \times 14' \times 100 = 2 \times 23770 = 47540$ lbs

$$\begin{array}{r} 47540 \text{ lbs} \\ 23770 \text{ "} \\ \hline 71310 \text{ "} \end{array}$$ = total load on Girder A or A'

Total load supported at each point = 14,262 lbs.

Now we can suppose the flanges as taking up the bending action of the load and the <u>web</u> as taking up the shearing action

The chord stress at any ~~point~~ section equals the bending moment at that section divided by the height of the truss

The greatest bending moments occur when the truss is fully loaded.

To find the supporting forces. 1st find the position of the resultant of the loads R acts at 225.25" from the left.

Supporting force on Right = $\frac{71310 \times 225.3"}{469\frac{3}{4}"} = \frac{71310 \times 225.3 \times 4}{1879} = 34202$ lbs

$$\begin{array}{r} 71310 \\ 901.2 \\ \hline 142620 \\ 71310 \\ 641790 \\ \hline 64264572. \end{array}$$

1879)64264572(34.202 lbs

Supporting force on left = 71310 lbs − 34202 lbs = 37108 lbs.

To find the greatest chord stress in the middle of the girder that is in the 4th panel from left.

34202 lbs = 17.1 tons 14262 lbs = 7.13 tons

$$\begin{array}{r} 234.875 \\ 17.1 \\ \hline 234875 \\ 1644125 \\ 234875 \\ \hline 4016.3625 \end{array} \qquad \begin{array}{r} 153.375 \\ 7.13 \\ \hline 460125 \\ 153375 \\ 1073625 \\ \hline 1093.56375 \end{array} \qquad \begin{array}{r} 71.875 \\ 7.13 \\ \hline 215625 \\ 71875 \\ 503125 \\ \hline 512.46875 \end{array}$$

M = 4016.36 − 1093.56 − 512.47 = 2410.33 inch tons

Chord stress in 4th panel from left = $\frac{M}{h}$ = $\frac{2410.33}{51}$ = 47.26 tons.

```
51)2410 (47.26
   204
    370
    357
     130
     102
      280
```

To find the greatest chord stress in 5th panel from left

M = 17.1 tons × 163″ − 7.13 × 81.5 = 2787.3 − 581.095 = 2206.205 inch tons

```
  163        81.5        51)2206.205 (43.26 tons
  17.1       7.13            204
  163        2445             166
 1141        815              153
 163         5705              132
 2787.3      581.095           102
 581.095                        306
 2201.205                       256
```

Greatest chord stress = $\frac{M}{h}$ = $\frac{2206.205}{51}$ = 43.26 tons.

To find the greatest chord stress in 6th panel from left.

M = 17.1 tons × 81.5 = 1393.65 inch tons

```
  81.5      Greatest chord stress = 1393.65 / 51 = 27.33 tons
  17.1
  815       51)1393.65 (27.33 tons
 5705          102
 815            373
 1393.65        357
                 166
                 153
                  135
```

To find the greatest chord stress in the 3rd panel from left

Support on left = 37108 lbs = 18.55 tons

M = 18.55 tons × 225.25″ − 7.13 × 163 − 7.13 × 81.5 = 4178.3875 − 1743.285

M = 2435.102 inch tons.

```
 225.25       163        81.5
 18.55        7.13       7.13
 4178.3875    1162.19    581.095
 1743.285     581.095
 2435.102     1743.285
```

Greatest chord stress = $\frac{2435.102}{51}$ = 47.75 tons

To find the greatest chord stress in 2nd panel from left.

$M = 18.55 \times 143.75 - 7.13 \times 81.5 = 2666.5625 - 581.095 = 2085.467$ inch tons

$$\begin{array}{r} 143.75 \\ 18.55 \\ \hline 71875 \\ 71875 \\ 115000 \\ 14375 \\ \hline 2666.5625 \end{array} \qquad \begin{array}{r} 7.13 \\ 81.5 \\ \hline 581.095 \end{array}$$

Greatest chord stress $= \dfrac{2085.467}{51} = 40.89$ tons

To find the greatest chord stress in 1st panel from left.

$M = 18.55 \times 62.25 = 1154.737$ inch tons

$$\begin{array}{r} 62.25 \\ 18.55 \\ \hline 31125 \\ 31125 \\ 49800 \\ 6225 \\ \hline 1154.7375 \end{array}$$

Greatest chord stress $= \dfrac{1154.737}{51} = 22.64$ tons

Total Chord stresses in Girder A or A'

In the 1st panel from left	=	22.64 tons
" " 2nd " " "	=	40.89 tons
" " 3rd " " "	=	47.75 tons
" " 4th " " "	=	47.26 tons
" " 5th " " "	=	43.26 tons
" " 6th " " "	=	27.33 tons.

To find the greatest shearing forces that occur at the different panel intersections

1st Find the shearing forces due to the Dead Load

(see next page)

Supporting force on Left due to dead load = 12369. lbs

" " " Right " " " " = 11401. "

Shear in 1st panel from left - 12369 lbs = 6.185 tons

" " 2nd " " " = 12369 lbs - 4754 lbs = 7615 lbs = 3.807 tons

" " 3rd " " " = 12369 - 9508 = 2861 lbs = 1.43 tons

" " 1st panel from right = 11401 lbs = 5.7 tons

" " 2nd " " " = 11401 lbs - 4754 lbs = 6647 lbs = 3.323 tons

" " 3rd " " " = 11401 lbs - 9508 lbs = 1893 lbs = .946 tons

To find the Greatest shears due to the Live load

Greatest support on left due to live load = 24738 lbs

" " " right " " " " = 22802 lbs.

The greatest shear occurs in the 1st panel on left when all the floor beams are loaded; hence greatest shear in 1st panel due to live load = 24738 lbs = 12.37 tons.

The greatest shear in the second panel from left occurs when only the 4 floor beams on the right are loaded. Amount of live load on each floor beam = 9508 lbs.

To find the supporting force on left. R acts at 203.75" from right

Support on left = $\frac{38032 \text{ lbs} \times 203.75}{469.75} = \frac{19.02 \text{ tons} \times 203.75}{469.75} = \frac{3875.32}{469.75} = 8.26$ tons

Shear in second panel from left due to live load = 8.26 tons

The greatest shear in the 3rd panel from left occurs when only 3 floor beams are loaded with live load

Racts at 163" from right support.

Support on left $= \frac{28524 \times 163}{469.75} = \frac{14.26 \text{ tons} \times 163''}{469.75''} = 4.9$ tons

$$\begin{array}{r} 14.26 \\ 163 \\ \hline 4278 \\ 8556 \\ 1426 \\ \hline 2324.38 \end{array}$$

469.75)2324.38 (4.9

Greatest shear in 3rd panel = 4.9 tons.

The greatest shear in the 1st panel from right equals the supporting force on the right with the live load all over the girder = 22802 lbs = 11.4 tons

To find the greatest shear in the second panel on right 4 floor beams loaded R acts at 184.5 from left

$\frac{19.02 \times 184.5}{469.75} = 7.5$ tons

$$\begin{array}{r} 184.5 \\ 19.02 \\ \hline 3690 \\ 16605 \\ 1845 \\ \hline 3509.19 \end{array}$$

469.75)3509.19 (7.5 tons

Greatest shear in 3rd panel on right

3 floor beams loaded R acts at 143.75 from left

Support on right $= \frac{14.26 \times 143.75}{469.75} = 4.4$ tons = Greatest shear in 3rd panel

$$\begin{array}{r} 143.75 \\ 14.26 \\ \hline 2049.875 \end{array}$$

469.75)2049.875 (4.4 tons

Total Shears

In 1st panel on left = 18.55 tons

" 2nd " " " = 12.07 tons

" 3rd " " " = 6.33 tons

" 3rd panel from right = 5.35 tons

" 2nd " " " = 10.82 tons

" 1st " " " = 17.1 tons

To find out the size to make the Flanges

We have the greatest chord stress in the 3rd panel from left = 47.75

1st take the top chord which is in compression; allow 9000 lbs or 4.5 tons per sq. in for compression.

$4.5)\underline{47.75}(10.61$ sq in to be made up in the top chord by plates and angle irons
$\underline{45}$
275

Use 2 angle irons $4'' \times 4'' \times \frac{7}{16}'' = 6.5$ sq in.

At t middle use $\frac{3}{8}''$ web plate $4 \times \frac{3}{8} = 1\frac{1}{2}$ sq in

10.61 sq in − 8 sq. in = 2.61 sq. in to be made up in section of flange plate; hence we use 1 plate on top $12'' \times \frac{3}{8}'' = 4\frac{1}{2}$ sq in.

2nd take the bottom flange, allow 12000 lbs or 6 tons per sq. in for tension $6)47.75(7.96$ sq. in to be made up in section in bottom flange; use 2 angle irons $4'' \times 4'' \times \frac{7}{16}$ to connect the flanges with web = $6\frac{1}{2}$ sq. in

Sectional area of portion of web included in flange = $1\frac{1}{2}$ sq. in; hence taking into account that we have to look out for rivet holes in bottom flange we use 1 plate $12'' \times \frac{1}{4}''$ Mr Cheney calculated this girder and also girder B with a little longer span and consequently he got his stresses a little greater; but he told me that I ought to take it as I have.

The plates in the top flanges of girders A and A' extend the whole length of the girder, but in the lower flange the plate is cut off $11'\text{-}7\frac{1}{2}''$ each side of the centre of the girder.

To find the sizes of the web plates in the different panels to resist the actions of the greatest shearing forces; in getting the sizes of these plates it is not sufficient to put in iron enough in section to merely resist the shearing action alone but we have to take into account the tendency of the shearing force to buckle the plates by using the following formula from Rankine $\dfrac{36000}{1 + \dfrac{s^2}{3000t^2}}$; in which $\underline{t}$ is the thickness of the web plate and $\underline{s}$ the distance measured along a line inclined at 45° to the horizon between two of the vertical stiffening ribs s = about 58″

Try at ends of girder $\frac{1}{2}''$ plate in web

$$\frac{36000}{1+\frac{3362}{750}} = \frac{36000 \times 750}{4112} = \frac{27000000}{4112} = 6566 \text{ lbs}$$

$$\begin{array}{r} 750 \\ 36000 \\ \hline 4500000 \\ 2250 \\ \hline 27{,}000{,}000 \end{array} \qquad \begin{array}{l} 4112 \overline{)27000000} (6566. \\ \quad 24672 \\ \quad \overline{\ 23280} \\ \quad\ 20560 \\ \quad \overline{\ \ 27200} \\ \quad\ \ 24672 \\ \quad \overline{\ \ 25280} \end{array}$$

Greatest shear ~~at~~ in 1st panel = 18.55 tons ~~4'3" = 51"~~

18.55 tons = 37100. lbs

$$\begin{array}{l} 4.25 \overline{)37100} (8730 \text{ lbs per ft of span} \\ \quad 3400 \\ \quad \overline{3100} \\ \quad 2975 \\ \quad \overline{1250} \\ \quad 1275 \end{array}$$

$12'' \times \frac{1}{2}'' = 6$ sq in

6)8730 (1455 lbs per sq in

The formula gives the ultimate resistance in lbs per sq. in and Rankine says that the intensity of the shearing action of the working load should not exceed one sixth of the resistance given by the above formula. ; the formula gives 6566 lbs per sq. in and the intensity of the shearing action in the 1st panel is 1455 lbs per sq in. hence $\frac{1}{2}$" web plate is just about right.

In the second panel the web plate is $\frac{7}{16}$" thick and in the 3rd or middle panel a $\frac{3}{8}$" web plate is used which are large enough.

To find the number of rivets required to splice the web plates at the different panel intersections.

The greatest shearing force that occurs in the girder is 18.55 tons; then the shear per ft. up and down will be

$\frac{18.55}{4.25} = 4.4$ tons allow for shearing of rivets 7500 lbs = 3.75 tons per

sq. in. $3.75)4.4($1.2 sq. in to be made up in rivets per ft up and down

the rivets are $\frac{13}{16}$" in diam. area of rivet = .52 sq. in

The rivets have to shear in two places before the splice gives way

$1.2 \div 2 \times .52 = 1.1$ rivet per ft for shearing.

For bearing or crippling t d.

cover plates are $\frac{1}{4}$" thick there being 2 of them at each splice

$\frac{13}{16} \times \frac{1}{2} = \frac{13}{32} = .41$ allow 12000 lbs = 6 tons per sq in

$6)4.4($.73 sq. in. $.41).73($1.8 rivets per ft.

$4.25 \times 1.8 = 7.6$ rivets for bearing in each row

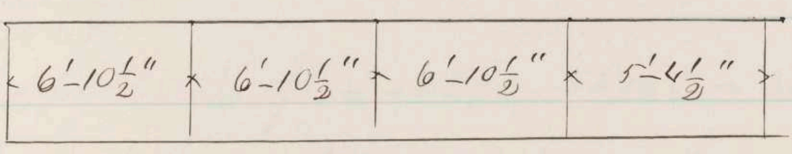
Now there were 2 rows of 11 rivets each put at each of the panel intersections all through the girders so Mr Cheney evidently simply calculated the no of rivets needed to resist the greatest shear in the longer girder and kept the same number throughout all the girders. This is not perhaps the most economical way but it is probably better to be a little too safe and save time of in calculations than to save a little money on rivets

Two angle irons each 3" x 3" weighing $7\frac{1}{4}$ lbs per ft were used as stiffening ribs midway between the panel intersections which were connected to the girders by means of 9+2 rivets in flange = 11 rivets. The same size of stiffening ribs were used throughout all the girders.

I will not complete the calculations on the number of rivets required to connect the different parts here but will wait till I have calculated the other girders for from the looks of things I should say that the number of rivets was calculated where the stresses were greatest and about the same pitch given to the rivets in other similar positions.

Calculations on Girder B.

Take the length of the Girder to calculate from centre of bearing to the centre of the pin near the foot of Samson Post or about 26'



Taking it for granted that floor beam R rests directly on the support we have the wt. of the 3 floor beams resting on the girder.

Total dead load acting on girder B

3 x 6'-10½" x 14' x 50 lbs.

6' 10½"
3
18' 2' 7½"
20' 7½"

14'
20
280 sq. ft.
50
14000 lbs.
430 "
14430 "

now the dead load for ½" in length = 28.5 lbs
7½" = 15/2"

28.5
15
1425
285
427.5 "

Live load = 100 lbs. per sq. ft. 14430 lbs

Total live load = 2 x 14430 lbs = 28860 lbs
43290 "

1st find the Reactions.

We suppose the loads concentrated where the floor beams are joined to the girders. Resultant of loads acts at 2 x 6'-10½" from left = 13'-9"

Support on Right due to both live and dead load

$$= \frac{\overset{21645}{\cancel{43290}} \times \overset{1.06}{\cancel{13.75}}}{\cancel{26}} = 22940 \text{ lbs.}$$

21645
1.06
129870
21645
22943.70

Support on Left = 43290 - 22940 = 20350 lbs.

To find the greatest chord stresses, they occur when the girder is loaded all over with both the live and dead loads.

The chord stress at any point = M at that point divided by h

Greatest bending moment in the 1st panel =

$20350 \times 6'\text{-}10\frac{1}{2}''$ $\qquad$ $6'\text{-}10\frac{1}{2}'' = 82.5''$

$h = 51''$

$$\begin{array}{r} 20350 \\ 82.5 \\ \hline 101750 \\ 40700 \\ 162800 \\ \hline 1{,}678875 = M \end{array} \qquad \begin{array}{l} 51)1678875(32917 \\ \;\;153 \\ \;\;\;148 \\ \;\;\;102 \\ \;\;\;\;468 \\ \;\;\;\;459 \\ \;\;\;\;\;\;87 \\ \;\;\;\;\;\;51 \\ \;\;\;\;\;\;325 \end{array}$$

Greatest chord stress in this panel = 32917 lbs = 16.46 tons

To find the greatest chord stress in the 2nd panel from left.

$M = 20350 \times 165 - 14430 \times 82.5 = 3357750 - 1190475 = 2167275$ inch lbs

$$\begin{array}{r} 20350 \\ 165 \\ \hline 101750 \\ 122100 \\ 20350 \\ \hline 3357750 \\ 1190475 \end{array} \qquad \begin{array}{r} 14430 \\ 82.5 \\ \hline 72150 \\ 28860 \\ 115440 \\ \hline 11904750 \end{array}$$

Greatest chord stress $= \frac{2167275}{51} = 42495$ lbs $= 21.25$ tons.

$$\begin{array}{l} 51)2167275(42495 \\ \;\;204 \\ \;\;\;127 \\ \;\;\;102 \\ \;\;\;\;252 \\ \;\;\;\;204 \\ \;\;\;\;\;\;48 \end{array}$$

Greatest chord stress in 2nd panel from Right

$M = 22940 \times 147 - 14430 \times 82.5 = 3372180 - 1190475 = 2191705$

Greatest chord stress $= \frac{2191705}{51} = 42974$ lbs $= 21.49$ tons

$$\begin{array}{r} 22940 \\ 147 \\ \hline 160580 \\ 91760 \\ 22940 \\ \hline 3372180 \\ 1180475 \\ \hline 2191705 \end{array}$$

Greatest chord stress in 1st panel from Right

$M = 22940 \times 64.5 = 1479630$ in. lbs.

Greatest chord stress $= \frac{1479630}{51} = 29012$ lbs $= 14.5$ tons

To find the Greatest Shears that occur at the different panel intersections. 1st find the shears due to the dead load

The support on the right due to the dead load is 7647 lbs.

" " " " left " " " " " " 6783 "

Dead load suspended at each point = 4810 lbs.

Greatest shear in the 1st panel due to dead load = 6783 lbs = 3.39 tons

" " " " 2nd " " to dead load = 6783 lbs − 4810 lbs = 1973 lbs = .99 tons

" " " 1st panel from right " " " = 7647 lbs = 3.82 tons

" " " 2nd " " " " " " " = 7647 − 4810 = 2837 lbs = 1.42 tons

To get the shears due to the live load.

The greatest shear occurs in the 1st panel on left when all three of the floor beams are loaded with the full load. Live load suspended at each point = 9620 lbs. The support on the left = 13566 lbs = 6.78 tons = the shear in the 1st panel on left due to live load

The greatest shear occurs in the 2nd panel on left when two floor beams are fully loaded R acts at $13'-9'' + 3'-5\frac{1}{4}'' = 17'-2\frac{1}{4}''$ from left $= 8'-9\frac{3}{4}''$ from right

The support on the Left $= \dfrac{19240 \times 105.75}{312} = 6521$ lbs $= 3.26$ tons

$$\begin{array}{r} 19240 \\ 105.75 \\ \hline 96200 \\ 134680 \\ 96200 \\ 19240 \\ \hline 2034630. \end{array}$$

312) 2034630 (6521
1872
1626
1560
663
624

Greatest shear in the 1st panel on Right occurs when the 3 floor beams are fully loaded. The support on the right due to live load = 15294 lbs = 7.65 tons = Shear in the 1st panel on right.

Greatest shear in the second panel on the right occurs when the two floor beams on the left are loaded with live load.

Racts at 6'-10½" + 3'-5¼" = 10'-3¾" = 123.75"

Support on Right = $\frac{19240 \times 123.75}{312}$ = 7631 lbs = 3.82 tons

```
   123.75
    19240
  -------
   495000
   24750
  111375
  12375
  -------
  2380950.

312)2380950(7631
    2184
    ----
     1969
     1872
     ----
       975
       936
       ---
        390
```

Total Shears

Shear in 1st panel on left = 3.39 + 6.78 = 10.17 tons

" " 2nd " " " = .99 + 3.26 = 4.25 tons

" " 1st " " right = 3.82 + 7.65 = 11.47 tons

" " 2nd " " " = 1.42 + 3.82 = 5.24 tons

To find out the sizes to make the flanges

The Greatest chord stress = 21.5 tons.

The top chord is in compression allow 9000 lbs = 4.5 tons per sq in

$4.5\overline{)21.5}($ 4.8 sq. in in section to be made up in top flange
$\frac{180}{350}$

Use 2 L's $4'' \times 4'' \times \frac{7}{16}''$ Sectional area of 2L's = 6.5 sq. in

$\frac{6.5}{1.75}$
8.25 sq. in

Try $\frac{7}{16}''$ web in centre of girder

$\cancel{4} \times \frac{7}{\cancel{16}_4} = 1\frac{3}{4}$ sq. in + 6.5 sq. in = 8.25 sq. in.

Now there was a top plate put on $12'' \times \frac{1}{4}'' = 3$ sq. in which plate was not needed to resist the chord stresses but was put on to finish off the top.

For the Bottom Flange which is in tension allow 12000 lbs = 6 tons per sq. in for tension

$6\overline{)21.5}$
3.6 sq in

2 L's $4'' \times 4'' \times \frac{7}{16}'' = 6.5$ sq in besides the portion of the web $4'' \times \frac{7}{16}'' = 1.75$ make up the bottom chord, there being no plate on this chord.

To find the sizes to make the Web plates

Taking into account the buckling of the plates we have the formula $\dfrac{36000}{1 + \dfrac{s^2}{3000\,t^2}}$

$4' = 48'$
$\frac{48}{384}$
$\frac{192}{2304}$
2304
4608

take s^2 = about 4608 by t at ends of girder $= \frac{1}{2}''$

$$\frac{36000}{1 + \dfrac{4608}{750}} = \frac{36000 \times 750}{4608} = \frac{27000000}{4608}$$

$4608\overline{)27000000}($ 5860 lbs
23040
39600
36864
27360
~~17~~

Greatest shear in the 1st panel = 11.47 tons

$51'' \times \frac{1}{2} = 25.5$ sq. in $25.5 \overline{)11.47}$ (.45 tons per sq in = 900 lbs per sq in
$\underline{1020}$
1270

The formula gives 5860 lbs per sq. in as the ultimate resistance and the intensity of the shearing action due to the working load is 900 lbs per sq. in and this is less than $\frac{1}{6}$ of the ultimate resistance hence $\frac{1}{2}$" web plate will do at the ends. In the centre panels a $\frac{7}{16}$" web plate was used which is large enough.

In the middle panels one 3" L stiffener was used in the centre of the panels.

I will not take up the calculations on the number of rivets required, at present.

Calculations on the Long Girder "C"

The first thing to do is to find out the loads that rest on the girder. We take the loads resting on half the length of the floor beams.

The Dead load resting on the 3 half floor beams marked "O" is $3 \times 6' - 10\frac{1}{2}'' \times 14' \times 50$ lbs. = 14430 lbs

Live load = 100 lbs per sq. ft. Live load on these 3 half floor beams

= 28860 lbs Dead load on each half floor beam "O" = 4810 lbs

Live " " " " " " = 9620 lbs.

Total " " " " " " = 14430 lbs = 7.22 tons

To find the load supported by floor beam R

28' x 3'-5¼" 28' = 336" 3'-5¼" = 41.25"

$$\begin{array}{r} 41.25 \\ 336 \\ \hline 24750 \\ 12375 \\ 12375 \\ \hline 13860.00 \text{ sq in} \end{array}$$

144)13860(96.25 sq ft.
1296
900
864
360
288
720

Length of Floor beam S = 32' 5" ½ of 7' = 3.5'

" " " R = 28'

2)50'5"

25-2.5" = 25.2'

$$\begin{array}{r} 25.2' \\ 3.5' \\ \hline 1260 \\ 756 \\ \hline 88.20 \text{ sq ft.} \end{array}$$

$$\begin{array}{r} 96.25 \text{ sq ft.} \\ 88.20 \text{ " "} \\ \hline 184.45 \text{ " "} \end{array}$$

Dead load supported on floor beam R = 50 x 184.45 = 9222.50 lbs

Live " " " " R = 100 x 184.45 = 18445. lbs

now ½ of the weights on this floor beam rests on Girder C.

Dead load on ½ floor beam R = 4611.25 lbs

Live " " " " " = 9222.50 " = 4.61 tons

Total " " " " R = 13833.75 lbs = 6.92 tons.

To find the load supported on floor beam S

Length of floor beam S = 22'5" = 22.42'

22.42' x 7' = 156.94 sq. ft.

Dead load supported by floor beam S = 156.94 × 50 = 7847 lbs

Live " " " " " " = 156.94 × 100 = 15694. lbs

Now $\frac{1}{2}$ of the wt on this floor beam rests on girder C

Dead load on $\frac{1}{2}$ of floor beam S = 3923.5 lbs

Live " " " " " " " = 7847. lbs. = 3.92

11770.5 lbs = 5.88 tons

To find the load supported on floor beam T

Floor beam T is 16'–10" long   16'–10" × 7' = 16.83 × 7 = 117.81 sq. ft.

Dead load supported by floor beam T = 117.81 × 50 = 5890.5 lbs

Live " " " " " " = 117.81 × 100 = 11781. lbs.

$\frac{1}{2}$ of this load is hung on girder C

Dead load on $\frac{1}{2}$ of floor beam T = 2945.25 lbs.

Live " " " " " " T = 5890.50 " = 2.94 tons

8835.75 " = 4.42 tons.

To find the load supported on floor beam U

Length of floor beam U = 11'2"   11.17' × 7' = 78.19 sq. ft.

Dead load on floor beam U = 78.19 × 50 = 3909.5 lbs

Live " " " " U = 78.19 × 100 = 7819. lbs

$\frac{1}{2}$ of this is supported on Girder C.

Dead load on $\frac{1}{2}$ of floor beam U = 1954.75 lbs.

Live " " $\frac{1}{2}$ " " " " = 3909.5 lbs = 1.95 tons

5864.25 lbs = 2.93 tons.

To find the load supported on floor-beam "V"

Length of floor beam V = 5'-7"

5'-7" × 7' = 5.58' × 7' = 39.06 sq. ft.

Dead load supported on floor beam V = 39.06 × 50 = 1953 lbs

Live " " " " " V = 39.06 × 100 = 3906 lbs.

½ of this is supported on girder C.

½ Dead load supported on floor beam V = 976.5 lbs.

½ Live " " " " " " = 1953.0 lbs = .98 tons

2929.5 lbs = 1.46 tons.

To find the supporting forces. 1st find the position of the resultant of the loads by taking moments about left support. For the figure see next page.

1.46 × 7 = 10.22
2.93 × 14 = 41.02
4.42 × 21 = 92.82
5.88 × 28 = 164.64
6.92 × 35 = 242.20
7.22 × 41.875 = 302.34
7.22 × 48.75 = 351.97
7.22 × 55.625 = 401.61

43.27) 1606.82 (37.14 ft.
1298 1
30872
30289
5830
4327
5030

Res. is at 37.14 ft from the left.

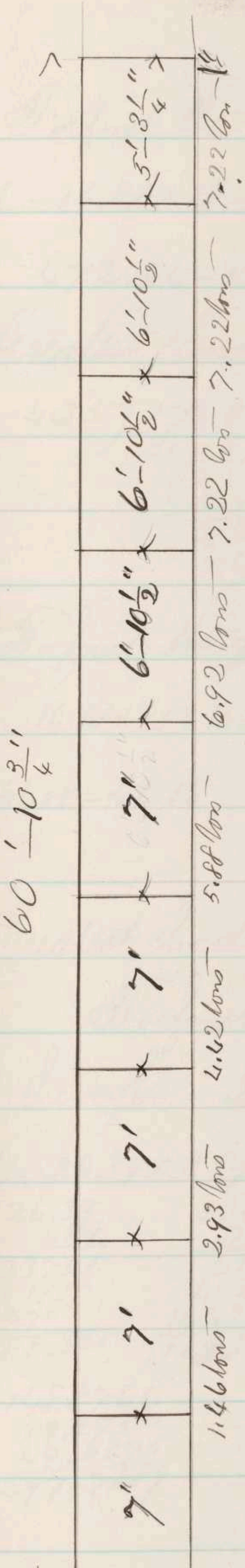
Total length of girder for calculation = 60'-10¾"
= 60.9'

The support on the right = $\frac{43.27 \times 37.14'}{60.9'}$ = 26.39 tons

43.27
37.14
17308
4327
30389
12981
1607.0478

60.9) 1607.0478 (26.39
1218
3890
3654
2364
1827
5370

The support on the left = 43.27 tons − 26.39 tons = 16.88 tons



To find the greatest chord stresses in the different panels

Greatest bending moment in the 1st panel on left $= 16.88 \times 7 = 118.16$ ft. tons the depth of the girder for calculation $= 4'-3''$

Greatest chord stress in the 1st panel on left

$= \frac{118.16}{4.25} = 27.8$ tons

$$
\begin{array}{r}
4.25 \,)\, 118.16 \,(27.8 \\
\underline{850} \\
3316 \\
\underline{2975} \\
3410
\end{array}
$$

To find the greatest chord stress in the 2nd panel on left. $M = 16.88 \times 14 - 1.46 \times 7$

$= 236.32 - 10.22 = 226.10$ ft. tons.

$$
\begin{array}{r}
16.88 \\
\underline{14} \\
6752 \\
\underline{1688} \\
236.32
\end{array}
$$

Greatest chord stress $= \frac{226.10}{4.25} = 53.2$ tons

To find the greatest chord stress in the 3rd panel on left.

$M = 16.88 \times 21 - 1.46 \times 14 - 2.93 \times 7 = 354.48 - 20.44 - 18.51 =$

315.53 ft. tons.

Greatest chord stress $= \frac{315.53}{4.25} = 74$ tons.

To find the greatest chord stress in the 4th panel on left

$M = 16.88 \times 28 - 1.46 \times 21 - 2.93 \times 14 - 4.42 \times 7 = 472.64 - 30.66 - 39.02 - 30.94$

$= 472.64 - 100.62 = 372.02$ ft. tons

$$\begin{array}{r} 30.66 \\ 39.02 \\ 30.94 \\ \hline 100.62 \end{array}$$

Greatest chord stress $= \frac{372.02}{4.25} = 87.53$ tons

$$\begin{array}{l} 425 \overline{)\,372.02\,(} 87.53 \\ \quad\;\; \underline{3400} \\ \quad\;\;\; 3202 \\ \quad\;\;\; \underline{2975} \\ \quad\;\;\;\; 2270 \\ \quad\;\;\;\; \underline{2125} \\ \quad\;\;\;\;\; 1450 \end{array}$$

To find the greatest chord stress in the 5th panel on left.

$M = 16.88 \times 35 - 1.46 \times 28 - 2.93 \times 21 - 4.42 \times 14 - 5.88 \times 7 = 590.80 - 40.88 - 55.53$

$- 61.88 - 41.16$

$$\begin{array}{r} 40.88 \\ 55.53 \\ 61.88 \\ 41.16 \\ \hline 199.45 \end{array}$$

$M = 590.80 - 199.45 = 391.35$ ft. tons

Greatest chord stress $= \frac{391.35}{4.25} = 92.08$ tons

To find the greatest chord stress in the 4th panel on the right. The support on the right $= 26.39$ tons.

$M = 26.39 \times 25.9 - 7.22 \times 20.62 - 7.22 \times 13.75 - 7.22 \times 6.875$

$$\begin{array}{r} 26.39 \\ \underline{25.9} \\ 23751 \\ 13195 \\ 5278 \\ \hline 683.501 \end{array} \qquad \begin{array}{r} 20.62 \\ \underline{7.22} \\ 4124 \\ 4124 \\ 14434 \\ \hline 148.8764 \end{array} \qquad \begin{array}{r} 13.75 \\ \underline{7.22} \\ 2750 \\ 2750 \\ 9625 \\ \hline 99.2750 \end{array} \qquad \begin{array}{r} 6.875 \\ \underline{7.22} \\ 13750 \\ 13750 \\ 48125 \\ \hline 49.63750 \end{array}$$

$M = 683.501 - 297.79$
$= 385.71$ ft. tons

$$\begin{array}{r} 148.8764 \\ 99.2750 \\ 49.6375 \\ \hline 297.7889 \end{array}$$

Greatest chord stress $= \frac{385.71}{4.25} = 90.75$ tons

To find the greatest chord stress in the 3rd panel on Right

$M = 26.39 \times 19.025 - 7.22 \times 13.75 - 7.22 \times 6.875 = 502.07 - 99.27 - 49.64$

```
   19.025
    26.39
   171225
   57075
  114150
  38050
  502.07
```

```
  99.27
  49.64
 148.91
```

$M = 502.07 - 148.91 = 353.16$ ft tons

Greatest chord stress $= \dfrac{353.16}{4.25} = 83.1$ tons.

Greatest chord stress in the second panel on the right

$M = 26.39 \times 12.14 - 7.22 \times 6.875 = 320.37 - 49.64 = 270.73$ ft tons

```
   26.39
   12.14
   10556
   2639
  5278
  2639
 320.3746
```

Greatest chord stress $= \cancel{139}\ \dfrac{270.73}{4.25} = 63.7$ tons

Greatest chord stress in the 1st panel on right

$M = 26.39 \times 5.27 = 139.0753$

```
   26.39
    5.27
   18473
   5278
  13195
  13907.53
```

Greatest chord stress $= \dfrac{139.0753}{4.25} = 32.72$ tons

```
425.)13907.53(3272
     1275
     1157
      850
      3075
      2975
       100
```

To find the greatest Shearing Stresses that occur at the different panel intersections. The greatest shear occurs at the ends when the whole truss is loaded both with the live and dead loads, hence the greatest shear at the left = 16.88 tons

Now to find the other shears I will treat the dead and live loads separately.

The support on the left due to the Dead load = 5.63 tons.

The greatest shear at the 1st intersection due to the dead load = 5.63 - .49 = 5.14 tons

The shear at the 2nd panel intersection due to the dead load = 5.63 - .49 - .98 = 4.16 tons

The shear " " 3rd " " " " " " = 5.63 - .49 - .98 - 1.47 = 2.69 tons

The shear " " 4th " " " " " " = 5.63 - .49 - .98 - 1.47 - 1.96 = .73 tons

The greatest shear on the right due to the total load = 26.39 tons

Shear at the 1st panel intersection on right due to dead load = 8.80 tons - 2.41 = 6.39 tons

Shear at the 2nd panel intersection on right due to dead load = 8.80 - 2.41 - 2.41 = 3.98 tons

Shear at the 3rd " " " " = 8.80 - 2.41 - 2.41 - 2.41 = 1.57 tons

Shear at the 4th panel intersection ....
= 8.80 - 2.41 - 2.41 - 2.41 - 2.31 = -.74 tons.

To find the Shears due to the live load.

The greatest shear occurs in the 2nd panel on the left when all but the floor beam on the left are loaded with live load.

I suppose the live load of 100 lbs per sq ft to roll off the floor beams successively

To find the greatest shear in the second panel 1st find the position of the resultant of the loads by moments about left.

1.95 × 14 = 27.30
2.94 × 21 = 61.74
3.92 × 28 = 109.76
4.61 × 35 = 161.35
4.81 × 41.875 = 201.42
4.81 × 48.75 = 234.49
4.81 × 55.625 = 267.56
27.85) 1063.62 (38.19
8355
22812
22280
5320
2785
2535

R acts at 38.19 ft. from left.

60.90
38.19
22.71

Support on left = the greatest shear in the second panel due to live load $= \frac{27.85 \times 22.71}{60.9}$

$= \frac{632.4735}{60.9} = 10.39$ tons.

27.85
22.71
2785
19495
5570
5570
632.4735

To find the greatest shear in the 3rd panel due to the live load

Suppose the live load removed from the 2 floor beams on left.

To find the ~~position~~ position of the resultant R by moments about left.

25.90) 1035.32 (39.98 R acts at 39.98' from left.

60.9
39.98
20.92 ft

Support on left = greatest shear in the 3rd panel due to the live load

$= \frac{25.9 \times 20.92}{60.9} = \frac{541.828}{60.9} = 8.90$ tons

To find the greatest shear in the 4th panel due to the live load
Suppose the load removed from the 3 floor beams on left.
To find the position of the resultant of the load.

$$22.96\overline{)973.58}(42.41 \text{ ft from left}$$

9184
5518
4582
9360
9184
176

$$\begin{array}{r} 60.90' \\ 42.41 \\ \hline 18.49 \end{array} \text{ ft from right}$$

Support on the left = the greatest shear in the 4th panel due to live load

$$= \frac{22.96 \times 18.49}{60.9} = \frac{424.530}{60.9} = 6.97 \text{ tons}$$

To find the greatest shear in the 5th panel from left due to the live load
Suppose live load removed from 4 floor beams on left.

$$19.04\overline{)863.82}(45.37 \text{ ft from left}$$

7616
10222
9520
7020
5712
13080

$$\begin{array}{r} 60.90' \\ 45.37 \\ \hline 15.53 \end{array} \text{ ft. Rast from right}$$

Greatest shear in the 5th panel from left due to live load

$$= \frac{19.04 \times 15.53}{60.9} = \frac{295.6912}{60.9} = 4.85 \text{ tons}$$

$$\begin{array}{r} 15.53 \\ 19.04 \\ \hline 6212 \\ 13977 \\ 1553 \end{array}$$

$$60.9\overline{)2956912}(4.85$$

2436
5209
4872
3370

The greatest shear in the second panel on the right occurs when all but the 1st. floor beam on right are loaded

1st find the position of the resultant load by taking moments about right support.

4.81 × 12.145 = 58.42
4.81 × 19.02 = 91.48
4.61 × 25.9 = 119.30
3.92 × 32.9 = 128.97
2.94 × 39.9 = 117.40
1.95 × 46.9 = 91.45
.98 × 53.9 = 52.82

24.02) 659.84 (27.47 ft. from right

60.9
27.47
R acts at 33.43 ft from left.

Support on right = shear = $\frac{24.02 \times 33.43}{60.9}$

= 13.17 tons.

To find the greatest shear in the 3rd. panel on the right

19.21) 601.42 (31.31 R acts at 31.31 ft from the right

60.9
31.31
29.59 ft from the left. Support on the right = $\frac{19.21 \times 29.59}{60.9}$

= 9.33 tons.

To find the greatest shear in the 4th. panel on the right due to the live load

14.40) 509.94 (35.41 ft. from right
4320
7794
7206
5940
5760
1800

60.9'
35.41
R acts at 25.49 ft. from left.

Support on the left = shear = $\frac{14.40 \times 25.49}{60.9}$ = 6.03 tons.

60.9) 367056 (6.03 tons.

Total Greatest Shears

In the 1st panel from left. = 16.88 tons

" " 2nd " " " = 5.14 + 10.39 = 15.53 tons

" " 3rd " " " = 4.16 + 8.90 = 13.06 tons

" " 4th " " " = 2.69 + 6.97 = 9.66 tons.

" " 5th " " " = .73 + 4.85 = 5.58 tons

" " 6th " " " = 1.57 + 6.03 = 7.60 tons

" " 7th " " " = 3.98 + 9.33 = 13.31 tons

" " 8th " " " = 6.39 + 13.17 = 19.56 tons

" " 9th " " " = 26.39 tons.

To find out the sizes to make the flanges.

The greatest chord stress occurs in the 5th panel from left = 92.08 tons

Take the upper chord which is in Compression allow 9000 lbs = 4.5 tons per sq. in for compression.

4.5)92.08(20.46 sq. in to be made up in section in the top flange in 5th panel from left.
90
208
180
280

The angle irons used here to connect the web with flanges were 4"×4"× $\frac{7}{16}$

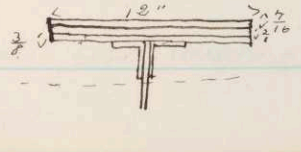
~~2L's =~~ 2 L's = 6.5 sq in in section

web plate at this point $\frac{3}{8}$" 4 × $\frac{3}{8}$" = 1.5 " " " "

Use 1 plate 12 × $\frac{7}{16}$ = 5.25 " " " "

Use 2 plates 12"× $\frac{3}{8}$" = 2× $\frac{9}{2}$ = 9.00 " " " "

22.25 " " " "



To find where to cut off the plates reduce the sections by the top plate $12'' \times \frac{7}{16}'' = 5.25$ sq. in

$$\begin{array}{r} 22.25 \text{ sq. in} \\ 5.25 \text{ " "} \\ \hline 17.00 \text{ " "} \end{array}$$

$$\begin{array}{r} 4.5 \\ 17 \\ \hline 315 \\ 45 \\ \hline 76.5 \end{array}$$

The greatest chord stress in 3rd panel on left = 74 tons hence on the left cut the top plate off in this panel as shown on the drawings and the greatest chord stress in the 3rd panel on right = 83.1 hence cut the top plate off in this panel as shown in the drawings making the top plate 26'-3" long.

Next reduce the cross section by the plate $12'' \times \frac{3}{8} = 4.5$ sq. in.

$17.00 - 4.5 = 12.5$ sq in

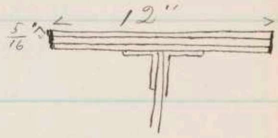
$$\begin{array}{r} 12.5 \\ 4.5 \\ \hline 625 \\ 500 \\ \hline 56.25 \end{array}$$ tons.

The greatest chord stress in the 2nd panel on left is 53.2 hence cut the plate off in this panel as shown in the drawings this plate is cut off on the other end at 2nd panel on right making the plate 38'-3" long. We can not cut off the lower plate so it extends along the whole top of the girder

In the Bottom flange

Greatest chord stress = 92.08 tons; use 12000 lbs = 6 tons for tension $6\overline{)92.08}$ 15.35 sq in to be made up in section of plates and angle irons.

Use 2 angle irons $4'' \times 4'' \times \frac{7}{16}''$ = 6.25 sq. in



portion of web included in flange $4'' \times \frac{3}{8}''$ = 1.25 sq. in

There are 3 plates used $12'' \times \frac{5}{16}''$ = 11.25 sq. in

18.75 " "

For the bottom chord we have to look out for rivet holes because it is under a pull hence this section is about right.

To find where to cut off the 1st plate reduce the section by $12'' \times \frac{5}{16}''$ = $3\frac{3}{4}$ sq. in 18.75 − 3.75 = 15 sq. in

6
99

This lower plate is cut off to correspond to the top plate in the top flange. and the next plate is also cut off to correspond to the 2nd plate on the top.

If we reduce the area by 3 plates 11.25 sq. in leaves 4.10
6
hence we can cut off the third plate & in the bottom chord 24.6 tons
in the 1st panel as shown in the drawings

To find out the sizes to make the webs to withstand the shearing action of the loads; taking into account the buckling of the plates

$$\frac{36000}{1 + \frac{s^2}{3000t^2}}$$

Take the 1st panel on the right

s = the distance measured at an angle of 45° with the horizon between two vertical stiffeners try t as $\frac{1}{2}''$ $t^2 = \frac{1}{4}$ take $s^2 = 1800$

$$\begin{array}{r} 36000 \\ 750 \\ \hline 1800000 \\ 252000 \\ \hline 27000000 \end{array} \qquad \frac{36000}{1+\frac{1800}{750}} = \frac{36000\times 750}{2550} = \frac{27000000}{2550}$$

2550) 27000000 (10590 lbs.

Greatest shear in this panel = 26.39 tons = 52780 lbs.

heighth of girder for calculation = 4' 3" = 51" $51\times\frac{1}{2} = 25.5$

25.5) 52780 (2070 lbs.

Now the ultimate resistance given by the formula 10590 lbs per sq. in, the intensity of the shearing action of the working load is 2070 now Rankine says that this last intensity should not exceed $\frac{1}{6}$th of the ultimate resistance, it will be seen in this case that it is about $\frac{1}{5}$th but it is safe enough to use $\frac{1}{2}$" web plate at the ends; the web plate in the second panel from the ends is also $\frac{1}{2}$ of an inch thick; in the 3rd panels from the ends the web plates are $\frac{7}{16}$ of an inch thick and in the other panels they are $\frac{3}{8}$ of an inch thick. The sizes and arrangement of the cover plates and stiffeners are shown in the drawings on sheet No. 1

Calculations on Rivets

To find out the number of rivets required to connect the web plates at the panel intersections in the long girder C

The greatest shear occurs in the 1st panel on the right = 26.39 tons. The shear per ft. up and down = 4.25) 26.39 (6.21 tons
25.50
890
850

allow 3.75 tons per sq. in for shearing of rivet 3.75) 6.21 (1.66 sq in
3 75
2460
2250
210

use $\frac{13}{16}$" rivets area = .52 sq in

the rivets have to shear in 2 places 1.04) 1.66 (1.6 rivets per ft for shearing

for crippling use 2 cover plates each $\frac{1}{4}$" thick

$\frac{13}{16} \times \frac{1}{2} = \frac{13}{32} = .41$

allow 6 tons per sq. in for crippling 6) 6.21
.41) 1.03 (2.5 rivets per ft.

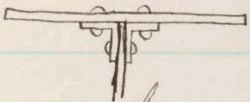
hence the rivets were placed at 4.5 inches apart; this pitch was kept throughout the girder at the web splices. At most of these intersections so many rivets are not needed but Mr. Cheney says that it is not good policy to change the pitch of the rivets to often on account of the greater liability to mistakes in the construction.

There were 11 rivets put into each of the central stiffeners counting the two in the flanges.

The number of rivets required to connect the web with flanges..

We may consider the shear to act in the web at 45° to the horizon; now since the highth of the girder is 4'-3" and the vertical shear at the right is 26.39 tons we may consider the chord stress for 4' - 3" on the right to be

26.39 tons   stress per ft. = 4.25) 26 39 (6.21 tons



hence the 1st set of rivets were put 4" apart and the same pitch was kept throughout the top and bottom flanges; this employs more rivets than are needed to withstand the chord stress, but the same pitch was used throughout for the reasons stated on the other page.

To find out how many rivets to employ to connect the plates to the angle irons in the flanges at ends.

sectional area of 2L's $= 4'' \times 4'' \times \frac{7}{16}'' = 6.5$ sq. in.

" " " portion of web $4'' \times \frac{1}{2}'' = 2.0$ " "

" " " plate $12'' \times \frac{3}{8}'' = 4.5$ " "

13.0 " "

take say $\frac{1}{2}$ of 6.21 tons per ft. as passing through 2nd set of rivets or 3 tons per ft.

use $\frac{13}{16}$" rivets   area of $\frac{13}{16}$" rivets = .52 sq in

3.75) 3.00 (.8   52) .80 (1.5 rivets per ft for shear

for bearing allow 6 tons per sq. in

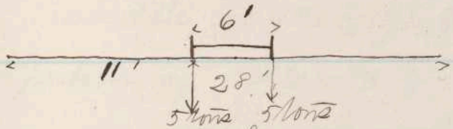
6) 3.00 (.5 $\frac{13}{16} \times \frac{3}{8} = \frac{39}{128} = .3$

3) .5 (1.6 rivets per ft.

Now here again the rivets in each of the two rows are placed 4 inches apart and the same pitch kept throughout; now this number of rivets according to my calculations is too many to simply resist the action of the chord stress and he evidently put in just twice as many rivets as he used in the 1st set of rivets. The same pitches of rivets are used in girders A and A' as in girder C.

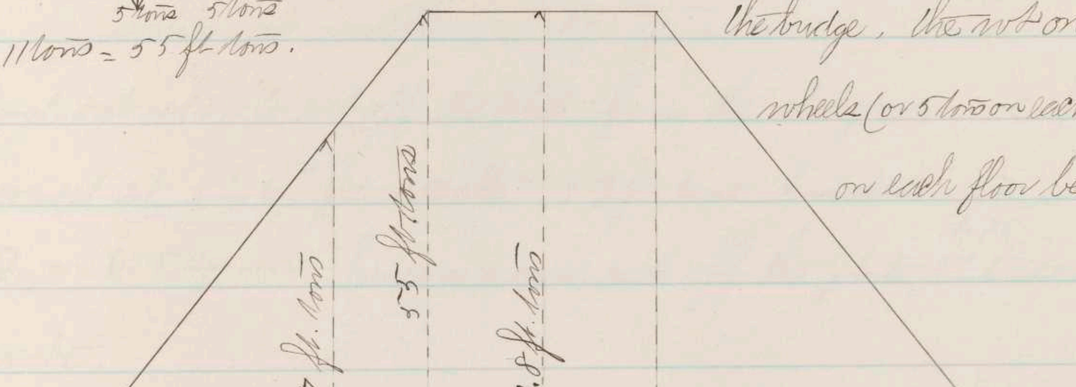
Calculations on the Floor beams

General Floor Beam "O".



5×11 tons $= 55$ ft-tons.

Suppose a 20 ton wagon to be on the bridge, the wt on two wheels (or 5 tons on each) rests on each floor beam



The moment curve for the dead load which is a uniformly distributed load of 50 lbs per sq ft. is a parabola whose middle ordinate is $\frac{1}{8} wl^2 = \frac{1}{8} \times 4.8 \times 28' = 16.8$ ft tons

Dead load on "O" = 9620 lbs = 4.8 tons

hence the total moment at the centre = 71.8 ft. tons depth of floor beam = 21" Greatest Chord stress = $\frac{71.8}{1.75}$ = 41.03 tons

For the top Flange which is in compression; for strains due to the wagon he used 1000 lbs per sq. in for compression and for tension in the lower chord 15000 lbs per sq in. as this wagon is an unusual load.

$$5)\underline{41.03}$$
$$8.2 \text{ sq. in}$$

Use 2 L's $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{7}{16}"$ = 6 sq. in

" in middle one $9" \times \frac{3}{8}"$ plate = $3\frac{3}{8}$ " "

portion of web $4" \times \frac{3}{8}"$ = $1\frac{4}{8}$ " "

section at middle = $10\frac{7}{8}$ " "

To find out where to cut off the plate from the diagram we find the moment at 5'–6" from centre = 57 ft. tons chord stress = $\frac{57}{1.75}$ = 32.7 tons

$\frac{32.7}{5}$ = 6.5 sq. in hence we can cut off the plate 5'–6" each side of the centre.

For the bottom flange allow 15000 lbs = 7.5 tons

$7.5)\ 41.03\ (5.5$ sq. in

hence in the bottom flange simply use 2 L's $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{7}{16}"$ = 6 sq in

and the portion of the web = $1\frac{1}{2}$ sq in.

To find out the sizes to make the web-plate.

Greatest shear at support of floor beam = 5 + 2.4 = 7.4 tons.

$$\frac{36000}{1+\frac{882}{421.9}} = \frac{36000 \times 42.9}{1303.9} = 11650 \text{ lbs per sq in} = \text{ultimate resistance} \quad \ell^2 = 882 \quad \text{try } t = \tfrac{3}{8}"$$

421.9
421.9

$1303.9)\ 15188400.\ (11650$ (36000)

$21 \times \frac{3}{8} = \frac{63}{8} = 8$ sq in

7.4 tons = 14800 lbs $\quad \frac{14800}{8}$ = 1850 lbs

1850 lbs per sq in = working shear now it is just about $\frac{1}{6}$ th of the ultimate resistance hence use $\frac{3}{8}$" web

How to find out how many rivets to use in the flanges

The chord stress for 21" at the ends = 7.4 tons

" " " " 1 ft. " " " = $\frac{7.4}{1.75}$ = 4.2 tons ~~per ft.~~

use $\frac{3}{4}$" rivets = .44 sq in allow 3.75 tons for shearing $\frac{4.2}{3.75}$ = 1.1 sq in

rivets shear in two places $\frac{1.1}{.88}$ = 1.3 rivets per ft for shearing

for bearing allow 6 tons per sq. in. 6)4.2 .7 sq in $\frac{3}{4} \times \frac{3}{8} = \frac{9}{32}$ = .28

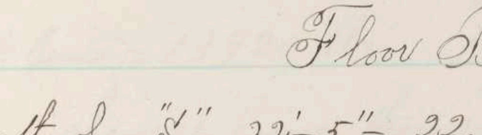
$\frac{.7}{.28}$ = 2.5 rivets per ft. The rivets were put in at 4" pitch

The details of the connections of these floor beams "O" with the main girder are shown on sheet No. 2.

The floor beams "Q" and "R" are of the same sections as the floor beams "O" and differ from them only in length.

Floor Beam S

Length of "S" = 22'-5" = 22.42'



5 × 8.21 = 41.05 ft. tons

Dead load supported by floor beam S = 7847 lbs = 3.92 tons

M at middle due to dead load = $\frac{1}{8}wl^2 = \frac{1}{8} \times \cancel{3.92}\ .49 \times 22.42$ = 10.99 tons

total M at middle = 41.05 ft + 10.99 tons = 52.04 tons

depth of floor beam = 18" = 1.5 ft. $\frac{52.04}{1.5}$ = 34.7 tons = greatest chord stress allow 5 tons for compression $\frac{34.7}{5}$ = 7. sq in to be made up in the top flange

Use 2 L's $3'' \times 3'' \times \frac{1}{2} = 5\frac{1}{2}$ sq. in
web $3 \times \frac{5}{16} = \frac{1}{6\frac{1}{2}}$ " "

Now really it is too much to count the live load on this floor beam as 2 wheels of a 20 ton wagon for it is shorter than the floor beams "O" so this section will be large enough.

For the Bottom flange allow 7.5 tons per sq in.

7.5) 34.7 (4.6 sq in. Use 2 L's $3'' \times 3'' \times \frac{3}{8}'' = 4.1$ sq in
portion of web $3 \times \frac{5}{16}'' = \frac{1}{5.1}$ sq in.

To find the size of the Web.

Greatest shear = 5 tons + 1.96 tons = 6.96 tons $\quad s^2 = 648$, try $t = \frac{5}{16}$

$$\frac{36000}{1 + \frac{648}{3000\,t^2}} = \frac{36000}{1 + \frac{648}{292.9}} = \frac{36000 \times 292.9}{940.9} = \frac{10544400}{940.9} = 11206 \text{ lbs.}$$

= the ultimate resistance per sq in. $\quad 18 \times \frac{5}{16} = \frac{45}{8} = 5.62$ sq in

6.96 tons = 13920 lbs $\quad \frac{13{,}920}{5.6} = 2500$ lbs.

as my shear is larger than what the shear really is; a $\frac{5}{16}$" web plate is right. To find the number of rivets to use in the flanges

1.5) 6.96 (4.64 tons per ft for greatest chord stress at end

3.75) 4.64 (1.2 sq in use $\frac{3}{4}$" rivets = .44 sq in

rivets have to shear twice .88) 1.2 (1.3 rivets per ft for shearing

for bearing allow 6 tons 6) 4.64 (.77 $\quad \frac{3}{4} \times \frac{5}{16} = \frac{15}{64} = .23$

$\frac{.77}{.23} = 3.4$ rivets per ft for bearing. pitch of rivets = 3 in.

Floor beam T is 16'–10" long = 16.83'

Dead load on floor beam T = 5890.5 lbs = 2.95 tons

Live " " " " " = 11781 lbs = 5.89 "

8.84 tons

The greatest M in the middle $= \frac{1}{8} w l^2 = \frac{8.84 \times 16.83}{8} = \frac{148.78}{8} = 18.6$ ft tons

depth of floor beam = 18" $\frac{18.6}{1.5} = 12.4$ tons = chord stress at the centre

allow 4.5 tons per sq in for compression. $\frac{12.4}{4.5} = 2.8$ sq in in section

Use 2 L's $3" \times 3" \times \frac{3}{8}" = 4.1$ sq in.

The Bottom flange is also made up of 2 L's $3" \times 3" \times \frac{3}{8}"$

To find out the size of the web plate

The greatest shear at the end of the beam = 4.42 tons

$$\frac{36000}{1 + \frac{648}{187.5}} = \frac{36000 \times 182.5}{835.5} = \frac{6750000}{835.5} = 8080 \text{ lbs}$$

$18 \times \frac{1}{4} = 4.5$ sq in 4.42 tons = 8860 lbs

$\frac{8860}{4.5} = 1960$ lbs. hence a $\frac{1}{4}$" plate will be safe enough but taking into account the buckling of the plate I think that I would have used a $\frac{5}{16}$" plate.

To find the number of rivets to use in the flanges; supposing the shear in the web to be equal to two forces acting at 45° to the horizon we have the stress for the 1st 18" in the flange = 4.42 tons $\frac{4.42}{1.5} = 3$ tons per ft.

For shearing of rivet allow 3.75 tons per sq. in.

$\frac{3.0}{3.75} = .8$ sq in use $\frac{3}{4}$" rivet $= .44$ $.88)\overline{.8}($ 1. rivet per ft.

for bearing allow 6 tons per sq. in. $6)\overline{3.75}$ $.62$ sq in

$\frac{3}{4} \times \frac{1}{4} = \frac{3}{16}$ $16)3(.19$ $\frac{.62}{.19} = 3.3$ rivets per ft.

put the rivets in the flanges $3\frac{1}{2}$ inches apart from centre to centre.

Floor beam "U" is $11'-2" = 11.17'$ long

The total load on the floor beam $= 5.86$ tons Greatest M $= \frac{1}{8} wl^2$

$= \frac{1}{8} \times 5.86 \times 11.17 = \frac{65.46}{8} = 8.18$ ft. tons. depth $= 18"$

Greatest chord stress $= \frac{8.18}{1.5} = 5.4$ tons.

allow 4.5 tons per sq in. for compression. $\frac{5.4}{4.5} = 1.2$ sq in

to be made up in area in the top flange.

Use one L $3" \times 3" \times \frac{3}{8}" = 2$ sq. in

In bottom flange allow 6 tons per sq. in for tension

$\frac{5.4}{6.} = .9$ sq in to be made up in the bottom flange

hence use in the flange one L $3" \times 3" \times \frac{3}{8}" = 2$ sq. in.

$\frac{1}{4}$ of an inch web plate was used and since this was large enough in the floor beam "T" it is large enough here.

The greatest shear at the ends $= 2.93$ tons

= the chord stress for 18" from the ends in the flange

$\frac{2.93}{1.5} = 2$ tons for the 1st ft. allow 3.75 tons per sq in for shearing

$\frac{2}{3.75} = .54$ sq. in use $\frac{3}{4}$" rivets $= .44$ rivets only have to shear once here $\frac{.54}{.44} = 1.2$ rivets per ft. for shearing

$\frac{2 \text{ tons}}{6 \text{ tons}} = .33$ sq in. $\frac{3}{4} \times \frac{1}{4} = \frac{3}{16} = .19$ $\frac{.33}{.19} = 1.7$ rivets per ft. for bearing now the rivets were put in 4 inches apart or 3 rivets to the ft. which I should say was closer than was required to withstand the stress.

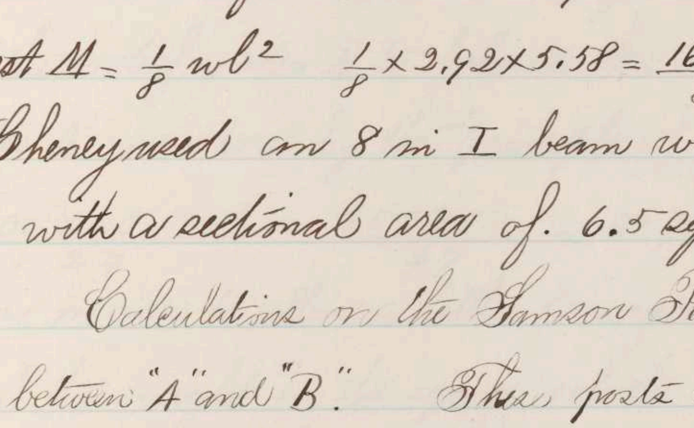
Floor beam "V" is 5'-7" = 5.58' long

Total load on the floor beam = 2.92 tons

Greatest $M = \frac{1}{8} wl^2$ $\frac{1}{8} \times 2.92 \times 5.58 = \frac{16.29}{8} = 2.04$ ft. tons

Mr. Cheney used an 8 in I beam weighing 65 lbs per. yard with a sectional area of. 6.5 sq in.

Calculations on the Samson Post and Suspension Rods between "A" and "B". These posts and rods act when the draw is off



To find out the additional weight that has to be placed at the end of girder B to counterbalance the greater wt suspended at the end of girder A $\frac{41'-5''}{2} = 20'-8.5'' = 20.7'$

$20.7' \times 14' \times 50$ lbs $=$ 14490. lbs. Besides this dead load of 50 lbs per sq. ft. he supposed a snow load of 10 lbs per sq. from the draw and also that 12 men at 120 lbs apiece were standing at the end A. $289.8 \times 10 = 2898$ lbs 12 men at 120 = 1440 lbs

14490
2898
1440
18828 lbs finally he considered the wt acting at the end of girder A as 21000 lbs. Now to find the weight that should be at the end of girder B to balance this weight supported at end of girder A, we do it by the "principle of the lever"

$\frac{21000 \times 40.75'}{27} = \frac{855750}{27} = 31,700$ lbs

Now in order to allow a little leeway ~~I~~ will only take into account the weight in the square which I have marked off. as supported at the end of B

Dead load supported at end of B $= 14' \times 14.3' \times 50 = 10000$ lbs

+ Snow load $= 14' \times 14.3' \times 10 = 2000$ lbs $= 12000$ lbs.

31700
12000
19700 lbs Mr Cheney told me that the dead load at this end of the draw is really more than 50 lbs per sq. ft. In the Specifications to Contractors there was a clause stating that this Counter-Balance should not be less than 17000 lbs.

Samson Post

Taking into account that there is a possibility of the girder "A" lifting the girder "B"; we may take as the thrust which acts through the posts and is transmitted to the central supports as 21000 lbs + 34000 lbs = 55000 lbs. Take the length of the post from the centre of the top pin to fastening at bottom = 25.2 ft = 302.4 inches

Formula from Rankine page 523 $\frac{P}{S} = \frac{36000}{1 + \frac{l^2}{36000 r^2}}$ for struts with both ends fixed. For struts with both ends hinged put 9000 instead of 36000 in the denominator; for struts with one end fixed and the other rounded

$$\begin{array}{r} 9000 \\ 36000 \\ \hline 2)\,45000 \\ \hline 22500 \end{array}$$

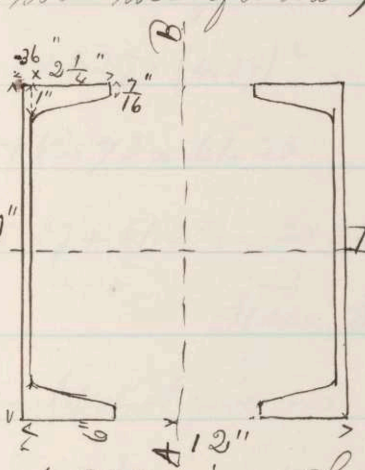
put 22500 for 36000 in the denominator and we have $\frac{P}{S} = \frac{36000}{1 + \frac{l^2}{22500 r^2}}$ where P is the breaking load; S the sectional area; l the length of the strut and r^2 is the square of the least radius of gyration, that is to say r^2 is the mean of the squares of the distances of the particles of the cross section from a neutral axis traversing its centre of gravity in that direction which makes r^2 least.

In this case we have the working load P = 55000 lbs.

The formula as given above gives the ultimate resistance per sq. in; hence the formula for the working intensity will be $\frac{P}{S} = \frac{9000}{1 + \frac{l^2}{5625 r^2}}$ We have taken the working intensity for compression as 9000 lbs but this intensity is for direct crushing hence for the post I will try say 2 tons = 4000 lbs as the working intensity of the stress per sq. inch.

$\frac{P}{S} = 4000$ lbs $S = \frac{55000}{4000} = 13.75$ sq. in to be made up in section of the strut.

We will use for the post a section of 2 Channel bars.

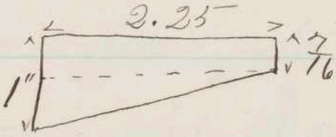


I find in looking over a list of some channel bars that a channel bar of the dimensions shown in the figure and weighing 23 lbs per ft. or 69 lbs per yd. has a section $= 10 \times .36 = 3.6 + 2\,\frac{1+\frac{7}{16}}{2} \times 2\frac{1}{4}" = 3.24$ $= 6.84$ sq. in. Section of 2 of these channel bars $= 2 \times 6.84$ sq in $= 13.68$ sq. in which is very near the section which I have supposed to be required. To find the square of the least radius of gyration for the above section $\frac{I}{A} = r^2$ 1st Find I about the neutral axis, I tried it first about axis marked A-B

I of one web about its own neutral axis $= \frac{hb^3}{12} = \frac{10\times(.36)^3}{12} = \frac{.46656}{12}$

$= .0389$ I of one web about AB $= .04 + 3.6\times(5.82)^2 = .04 + 3.6\times 33.87$

$= .04 + 121.93 = 121.97$



I of 2 Rectangles of flanges

1st about their own neutral axis $= \frac{2\times\overline{2.25}^3\times\frac{7}{16}}{12} = \frac{2\times 11.39\times\frac{7}{16}}{12}$

$= \frac{2\times 4.98}{12} = .83$ I of these about AB $= .83 + 2\times 2.25\times\frac{7}{16}\times(4.525)^2$

$2\times 2.25\times\frac{7}{16} = 1.96$ $(4.52)^2 = 20.43$ $20.43\times \cancel{\frac{7}{16}}\, 1.96 = 40.04$

$40.04 + .83 = 40.87$

The two triangles included in the flanges   I of these about AB $=$

$.5 + 2\times 1.27\times(4.89)^2$ $\overline{4.89}^2 = 23.91$ $23.91\times 2.54 = 60.73$

$.5 + 60.73 = 61.23$.   I of the entire section about AB $= 2\times(121.97$

$+ 40.87 + 61.23) = 2\times 224.07 = 448.14$

Then I took the axis in the position CD to see if it would not give me a smaller I.

I of 2 webs $= 4\times 1.62\times(4.69)^2$ $(4.69)^2 = 22.$ $4\times 1.62\times 22 =$

$= 4\times 35.64 = 142.56 \quad + 1.4 = \quad 144$

Total I about CD $= 144 + 60 = 204$. This is much smaller than the I about AB hence the least $r^2 = \frac{204}{13.7} = 14.9$

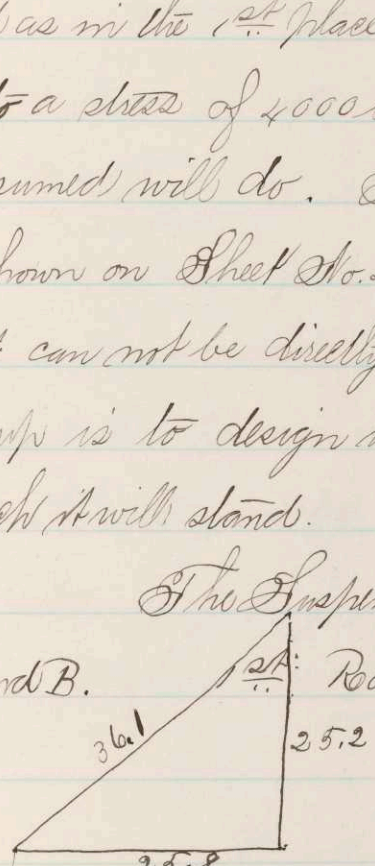
$$\frac{P}{S} = \frac{9000}{1 + \frac{(302.4)^2}{5625\times 14.9}} = \frac{9000}{1 + \frac{91445.76}{83812.5}}$$

$$= \frac{9000 \times 83812.5}{83812.5 + 91445.76} = \frac{754,312,500}{175258.5} = 4304 \text{ lbs.}$$

Now this shows that this post will bear a working stress of 4304 lbs on the square inch with a sufficient factor of safety and as in the 1st place in making my section I only subjected it to a stress of 4000 lbs per sq. in. hence the section that I assumed will do. The bracing which connects these bars is shown on Sheet No. 2. and also the cross strut; this cross strut can not be directly calculated for but the best way to make it up is to design it and then make an estimate of how much it will stand.

The Suspension Rods from the Post between girders A and B.

2d: Rods from top of post to end of B



$$\overline{25.2}^2 = 635.04$$
$$\overline{25.8}^2 = \underline{665.64}$$
$$1300.68 = \overline{36.1}^2$$

34000 lbs

The direct stress on the rods $= \frac{36.1}{25.8} \times 34000 = 48707$ lbs

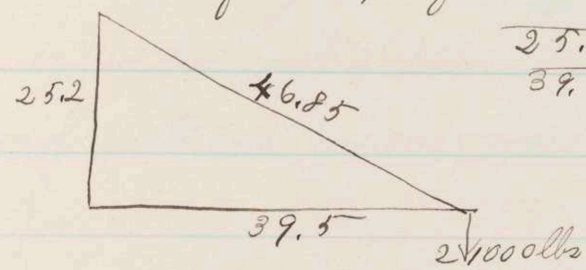
$$\begin{array}{r} 36.1 \\ 34000 \\ \hline 1227400.0 \end{array}$$

$25.8)1227400.(48707$

allow 12000 lbs per sq. inch for tension $12000)48707(4.06$ sq in

Use 2 rods $\frac{4.06}{2} = 2.03$ sq in. area of each rod 2 Rods $1\frac{5}{8}''$ in diameter will do but it is better to use 2 rods $1\frac{3}{4}''$ each in diameter.

Rods from top of Post to end of girder A



$$\overline{25.2}^2 = 635.04$$
$$\overline{39.5}^2 = \underline{1560.25}$$
$$2195.29 = \overline{46.85}^2$$

Direct stress on rods $= \frac{46.85}{25.2} \times 21000$ lbs $= 39042$ lbs

$$\begin{array}{r} 46.85 \\ 21000 \\ \hline 983850. \end{array} \qquad 25.2)983850(39042$$

allow 12000 lbs per sq. in. for tension. $\frac{39042}{12000} = 3.25$ sq in.

Two rods were used here also $\frac{3.25}{2} = 1.63$ sq. in hence use 2 rods each $1\frac{5}{8}$" in diameter. These rods are connected with the casting on top by a pin $2\frac{3}{4}$" in diameter. The greatest shearing force of pin $= 48707$ lbs. allow say 8000 per sq in for shear $\frac{48707}{8000} = 6.09$ sq in.

Conclusions.

My time will not allow me to go any further into the discussion of this work; Although I have not gone as minutely into the details as I should have done if I had been designing the structure and as the designer did, I will say that I have been through most of the work and in all my results have agreed very closely with Mr. Cheney's, in no case, I believe, finding his sections of iron too small and the only discrepancy in his work between theory and practice is in the number of rivets used in some cases which is greater than theory calls for.

I will end by saying that I am very well pleased with the treatment that I received at City Hall and I am especially indebted to Mr. Cheney for his cordial aid.

Chas. E. Stewart

May 11th 1877