THE PLATE GIRDER BRIDGE
OF
THE COLORADO AND SOUTHERN RAILROAD
AT
BOULDER, COLORADO

GRADUATION THESIS

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1905
PREFACE.

The work of writing up the Boulder Plate Girder Bridge was suggested by Mr. Howard C. Ford, assistant in Civil Engineering at The University of Colorado. It was afterwards suggested to investigate the strength of the bridge and make a thesis of the whole. Whatever success this thesis may win should therefore be credited to Mr. Ford who originated the idea.

The work of watching the progress of the erection was very interesting; it being a great satisfaction to the author to know just the steps followed in replacing a railroad bridge by a more modern structure without interfering with traffic. Thanks is due Mr. W. E. Hague, foreman in charge of the work, for his kindness and willingness to answer questions or to aid in looking up points which he might not know.

The investigation of strength was aided very decidedly by the complete set of detail drawings of the bridge which The American Bridge Company kindly furnished. Tracings of these sheets have been placed at the end of the work. The interest of the thesis has been greatly increased by the figures on the costs of the various parts of the work.
These figures were given by Mr. O. J. Travis, Superintendent of Bridges and Buildings of The Colorado and Southern Railroad at Denver, Colorado.

No expression of thanks is adequate for the assistance given by Professor M. S. Ketchum. His willingness to clear up matters not understood was only exceeded by the skill with which he satisfied his questioner.

Archibald C. Barrett.

June 15, 1905.
THE COLORADO AND SOUTHERN PLATE GIRDER BRIDGE

Across Boulder Creek, Boulder, Colorado.

Introduction:— The object of this thesis is to give a detailed description of the method of replacing the old wooden bridge across Boulder Creek by a steel plate girder without interfering with the traffic; also to investigate the strength of the girder in all its parts.

The first part of the thesis includes a description of the bridge, and its abutments; the erection of the superstructure, the manner of riveting, and a summary of the costs.

The second part is an investigation of the strength of the various parts of the steel work according to Cooper's Specifications for Railroad Bridges (1904 Edition.)

PART I.

Description of Bridge:— The Colorado and Southern plate girder span over Boulder Creek at Boulder was erected in December 1904 to take the place of the two wooden, queen post truss spans which formerly carried the track across the creek.

The bridge now in place was designed and fabricated by the American Bridge Co. at Chicago. It is a through plate-girder, single-track, 80 ft. span; 16 ft., 4 ins. deep and made of soft Open Hearth steel. The bridge was designed
for Coopers Class E-50 loading. The total weight of the bridge and track is 186,000 lbs. or 93 tons.

Abutments:— The girders are supported on two concrete abutments with wing walls on the up-stream side. They reach 5 ft. above the creek level and extend 15 feet below. The north abutment was placed 10 ft. outside and the south one 20 ft. inside the ends of the old bridge.

Work on the concrete abutments began about the middle of October under the Colorado and Southern foreman in charge of eight men. Sheet piling made of two-inch tongue and groove plank was driven, inclosing the area of the abutments; and as the material was excavated the piling was driven on down. The water was kept out by two horizontal displacement pumps, for which steam was applied by a portable vertical boiler. The material excavated was gravel and boulders, and after a depth of 15 feet was reached without striking bed-rock, the concrete was started. The concrete was put in through a chute and spread and tamped by two men. The composition was:—one part Iola Portland cement, three parts screened creek sand, and five parts broken sandstone, all sizes below 2 1/2 inches. The cement, sand and rock were measured in wheelbarrows, the water was measured in barrels and added during the mixing. The concrete was mixed by turning twice with a shovel.

When the coffer-dams were filled up to the ground level, forms of 2-inch planks were built and the concrete
continued up to fill the forms. The top was leveled up by a layer of cement mortar one inch thick.

The concrete was allowed about a month to set before the forms were removed. The total cost was $996.01 for material and $1233.44 for labor.

Erection of Superstructure:— The fabricated steel for the plate girder arrived in Boulder about the time work was begun on the abutments. The girders were packed side by side on two flat cars and were firmly braced and bolted together. The remainder of the steel work was loaded promiscuously.

The erection of the bridge began November 28th, 1904. The erection gang consisted of the Colorado and Southern foreman, W.E. Hague and ten laborers. The workmen were inexperienced in bridge work and received $2.50 per day.

The first work was to arrange the old bridge so that the girders could be put in place without disturbing the track and interfering with traffic. In the first place the floorbeams of the old bridge, which were bolted to the underside of the lower chords, were supported on bents made of 12x12 inch posts braced with 2x18 in. planks as shown in the accompanying photograph, Plate I.

When the floorbeams were thus supported, the old wooden trusses no longer carried any load and were unbolted from the floorbeams and were shoved outward about a foot.
Showing method of supporting floorbeams of old bridge.

At the same time the ends of the ties and their supporting stringers were sawed off and removed, leaving only two feet of tie on each side of the rails, thus making a space so that the girders could be set on the floorbeams. The photograph in Plate II shows men sawing off the ends of the ties on the left hand side, and shows the space left for the girder on the right after the ends have been sawed off.

The girders were then unpacked and set in notches on 12"x12" sills, 6'-0" long, at each end, and braced by two 4"x4" batter posts from the top flange to the ends of the sills. A rope was also thrown over the girder at each end and fastened to the ends of the sills. (See Fig. I)
Plate II.

Showing men sawing off ends of ties.

Fig. I

End view of girder packed on trucks.

Each sill was set on a pair of car trucks and separated from them by 3/4" rods laid transversely so the girders would slide easily from the trucks. The first girder was towed into place by a switch-engine and jacked from the trucks on to the blocking support-
ing each end. A jack was used at each end jacking against the old trusses and the truck-wheels. The blocking was made of ties built up on the floorbeams.

While the girder was being shoved on the block-hung two sets of falls from the middle of the upper flange were fastened, one on each side, to trees in the creek bottom, and were slackened on one side and tightened on the other as the girder was moved over. The outside set of falls was used to hold the girder against the old trusses during the night. The clearance between the girder and the track was so small that while the girder rested over night on the blocking, the plates on the ends of the bottom flange for connecting with the diagonal bracing had to be cut off to allow trains to pass. The next morning a screw jack was placed under each end of the girders on the floorbeams. The girder was jacked up slightly to loosen the blocking and then lowered as the blocking was removed. The bracing which held the girder on the trucks was first removed. When the girder rested on the floorbeams it was allowed to remain temporarily and the girder on the other side put in the same way.

It took the erection gang three days to set both girders on the floorbeams, two days being req-
uired to place the first girder.

After placing the girders, the remainder of the steel was set on the sides of the track in the order in which it would be needed; the Colorado & Southern pile-driving outfit with a switch-engine being used to handle the steel.

The girders were next supported from the abutments on jacks, the ends of the floorbeams on which they had rested were sawed off and the girders slowly lowered on the abutments. The photograph in Plate III shows men at work sawing off the ends of the floorbeams preparatory to lowering the girders onto the abutments.

PLATE III

Showing men sawing off the floor-beam ends preparatory to lowering the girders to the abutments.
The gang was now ready to put in the floor-system; which consisted of: four intermediate and two end floorbeams, ten main-track stringers, ten safety stringers, with diagonal and sway bracing. The plan of the floor system is shown in Plate IV.

The floor system was erected a panel at a time beginning at the North end. The old wooden stringers were cut about a foot beyond the second panel point and all the track, ties, and supporting timbers between the cut and the north end were removed, thus leaving an open space to the ground. This work is shown in Fig. 2.

![Diagram of floor system](image)

**Fig. 2**

Showing old track removed from one panel.

The pile diver then brought up the end floorbeam on the derrick and and let it down into the vacant panel, short lengths of rail being laid so that the track came to the edge of the opening. The
end floorbeam was swung around and put in place, the free connecting angles, as will be explained later, were put on and field bolts run through the rivet holes. The second floorbeam was brought up and put in in the same way. However there was considerable difficulty encountered with this one for the following reason:— Each end floorbeam is connected to the girder web by two angles, one on each side of the web of the floorbeam (Plate IV). One of these connecting angles is shop-riveted to the girder and the other is shipped loose, to be field-riveted on after the floorbeam is in place. The shop-riveted angles are on the side of the floorbeam toward the ends of the bridge, changing sides at the middle as shown by Fig. 2. Now the position of the second floorbeam was on the other side of the angles from the erecting machine and as the floorbeam could not be let down squarely into position on account of the top flanges of the girders and could not be entered diagonally and turned behind the angles on account of the small space between the angles and the track ahead; one end had to be put in place and the girders spread apart with jacks until the other end could clear its angle and pass behind. When the floorbeam was in place and before it was bolted, it was shoved back slightly and the stringers put in and the whole bolted up. The main stringers
could not be put in after the floorbeams were bolted as in swinging around into place they were blocked by the shelf angles of the safety stringers. As soon as the stringers were bolted up, the ties were laid over them, the track put down and trains allowed to pass. In this way all the panels were put in; making two panels a day after the first panel. Work was started after the morning trains had passed, the tracks were relaid for the noon trains; and another panel was put in during the afternoon in time for the evening trains.

After three floorbeams were put in in making two panels and the fourth floorbeam was reached it seemed as though the erection could go on much faster since the connecting angles were turned the other way and the floorbeams could be set on the same side of the angles as the derrick, but it was soon evident that if the floorbeams were put in their proper places in front of the angles they could not be shoved back as before to allow the main stringers to be put in, so one end of the floorbeam had to be left on the wrong side of the angles until the stringers were in place then jacked by the angles into place as had been done before with the others. Each time when the girders were spread to allow the floorbeam to pass its angle, all the floorbeams in place, save the end one, had to be unbolted at one end to let the girder
It seems as if all this labor might have been obviated had the connecting angles been both shop-riveted to the floorbeams, or if the angle which was riveted to one girder had been for the opposite side of the floorbeam to the one on the other girder. The number of field rivets in each case would have been the same. A great saving would also have been made if the rivets in the upper flanges of the girders on the inside and adjacent to the connecting angles had been countersunk; as in every case the floorbeam web plates had to be pried past these heads.

The bridge was erected and bolted together ready for riveting with the exception of the cross frames and diagonals, in ten days after the first girder was brought over or twelve days from the beginning of the work.

Riveting:— On Saturday December 10th, riveting was started. The riveting gang consisted of five men; four riveters and one rivet-heater. The men were entirely new to the work and consequently did very slow work and wasted many rivets. However each day showed an advance in the number of rivets driven and in the quality of the work done. Likewise the men soon learned the tricks of the trade and could make very good heads on very poorly driven rivets.

The first rivets driven were in the east end of the north end-floorbeam as can be easily seen by comparing them with the others. Two men were put in each side of
the joint and every hole was drifted before receiving the rivets. The field bolts which were in every other hole were removed as the riveting progressed.

The method of riveting was as follows:— The rivet heater ran over with a hot rivet in his tongs, man No.2 took it in his tongs and set it in the hole, No.3 drove it up, No.2 set a dolly, or heavy iron maul with a die in the head, over the rivet head and Nos. 2 and 3 pulled back on the dolly handle against a hook, bolted in the hole above, which held the handle near the head and acted as a fulcrum. This held the rivet head tight while No.4 and No.5 headed it upon the other side. No.4 first upset the rivet thus spreading it and filling the hole. No.5 then placed his "snap" over the end and No.4 hammered the "snap" till the head was formed.

The first day the end of the floorbeam was riveted up, 44 rivets being driven and 8 rivets wasted. The second day 118 rivets were driven and six wasted.

While the riveters were at work the other men were bolting up the cross frames and diagonal bracing, clearing away the old timbers and putting in the cross ties. All but the riveters stopped work on December 18th, the latter working on till January 10th. In all 1549 rivets were driven. The time occupied in driving the rivets was 27 days or an average of 57 1/2 rivets per day. With five men at $2.50 per day or $12.50 per day for the gang, the
rivets cost \( \frac{12.50 \times 27}{1549} = 22 \) cents a piece, which is a very high cost, 10 to 15 cents being an average cost.

A criticism on the rivet driving is that the lengths of the rivets were not determined from the drawings but were obtained by trial. For instance in riveting the connecting angles of the intermediate floorbeams to the girders, 3 inch rivets were used while the plans required 3 5/8 inch rivets. This showed that the rivets were not "upset" sufficiently to fill the holes before heading them up. This was apparent from the method of heading up. When a hot rivet came through it was first bent over against one side of the hole, then was hit a few raps on the head supposedly to fill the hole, but in reality only mashing the end against the outside of the hole, after which the snap was put on and the head formed.

The cost of erecting the bridge was as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>$129.70</td>
</tr>
<tr>
<td>Labor</td>
<td>869.55</td>
</tr>
<tr>
<td>Total</td>
<td>$999.25</td>
</tr>
</tbody>
</table>

The total weight of the bridge without track was 155185 lbs. or 77.6 tons. Then the cost of erection per ton was \( \frac{999.25}{77.6} = 12.87 \) dollars. An average cost for such work is from $6 to $10 per ton.
<table>
<thead>
<tr>
<th>Description</th>
<th>Material</th>
<th>Labor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavating and putting in two abutments, 996.01</td>
<td>$996.01</td>
<td>$1233.40</td>
</tr>
<tr>
<td>Erecting and riveting steel girders, 129.70</td>
<td>129.70</td>
<td>869.55</td>
</tr>
<tr>
<td>Cost of girder with inspection at bridge works and freight from Chicago,</td>
<td>4178.93</td>
<td>54.95</td>
</tr>
<tr>
<td>Labor cleaning up and filling approaches,</td>
<td></td>
<td>1450.00</td>
</tr>
<tr>
<td>Totals,</td>
<td>$5304.64</td>
<td>$3607.90</td>
</tr>
<tr>
<td>Total cost, $8912.54</td>
<td></td>
<td></td>
</tr>
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</table>
PART II.

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specifications:- This investigation is to be carried on according to Cooper's Specifications for Railroad Bridges, 1901 Edition, and the following loadings will be used as they are the ones for which the bridge was designed.

Live Load: Cooper's Class E-50.

Dead Load:- Weight of metal, 2000 lbs. per ft.

" track, 400 " " ".

Wind Load:- Fixed wind load, 300 " " ".

Moving " , 450 " " ",

Depth and Spacing:- The depth of girders varies in common practice from 1/10 of the span to 1/12, an average ratio being 1/10.5. In this bridge the ratio is 8 1/3 : 80 or 1/9.6. The distance C to C of girders is 16'-4". Cooper specifies a clearance of 14 ft. and an additional clearance in inches of .8 the degree of the curve, on each side, for curvature. The track over this bridge has a (8.24) degree curvature so an additional clearance of .8 x .84 = 6.7 or a total specified clearance of 14 + .3 = 14.3".
The weight of an intermediate floorbeam, as marked on the piece, is 4100 lbs. An intermediate floorbeam differs from an end floorbeam in the following parts:

<table>
<thead>
<tr>
<th>Piece</th>
<th>No. of Pieces</th>
<th>Cross Section</th>
<th>Length in Ft.</th>
<th>Weight per Ft.</th>
<th>Total Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>I s</td>
<td>4</td>
<td>6X6X3/4</td>
<td>15.30</td>
<td>26.35</td>
<td>512</td>
</tr>
<tr>
<td>Gus. pl.</td>
<td>2</td>
<td>15/2X1X7</td>
<td>6.56</td>
<td>26.35</td>
<td>33</td>
</tr>
<tr>
<td>L s</td>
<td>4</td>
<td>3&quot;X3&quot;X3/3</td>
<td>7.2</td>
<td>6.56</td>
<td>19</td>
</tr>
<tr>
<td>Spl. pls</td>
<td>4</td>
<td>12 1/2&quot;X1/2</td>
<td>333</td>
<td>21.25</td>
<td>23</td>
</tr>
<tr>
<td>Fills.</td>
<td>4</td>
<td>14&quot;X3/4</td>
<td>167</td>
<td>35.7</td>
<td>24</td>
</tr>
<tr>
<td>Fills.</td>
<td>4</td>
<td>12 1/2&quot;X3/4</td>
<td>167</td>
<td>31.88</td>
<td>21</td>
</tr>
<tr>
<td>L s</td>
<td>2</td>
<td>3/2&quot;X5&quot;X5/8</td>
<td>1.5</td>
<td>10.4</td>
<td>31</td>
</tr>
<tr>
<td>L s</td>
<td>2</td>
<td>3/2&quot;X5&quot;X5/8</td>
<td>1.5</td>
<td>10.4</td>
<td>31</td>
</tr>
</tbody>
</table>

**Difference in weight** = 489 lbs

**Weight of intermediate floorbeam** = 4100 lbs

**Weight of end floorbeam** = 3611 lbs

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**Safety Stringers.**

<table>
<thead>
<tr>
<th>I beam</th>
<th>1</th>
<th>20&quot;</th>
<th>15.292</th>
<th>65.</th>
<th>994</th>
</tr>
</thead>
<tbody>
<tr>
<td>Con L s</td>
<td>4</td>
<td>6&quot;X6&quot;X1/2</td>
<td>1.458</td>
<td>19.6</td>
<td>114</td>
</tr>
<tr>
<td>Fill pl</td>
<td>1</td>
<td>6 1/4X1/2</td>
<td>0.333</td>
<td>10.63</td>
<td>4</td>
</tr>
</tbody>
</table>
| Rvt. Hds | 23  | 7/8" | 24.29/100 | 7 | Total weight = 1119 lbs
<table>
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<tr>
<th>Piece</th>
<th>No of pieces</th>
<th>Cross Section</th>
<th>Length (in ft.)</th>
<th>Weight (per ft.)</th>
<th>Total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main stringers</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web pl.</td>
<td>1</td>
<td>31 3/4&quot;X1/2&quot;</td>
<td>15.29</td>
<td>53.93</td>
<td>325</td>
</tr>
<tr>
<td>Fl. Ls</td>
<td>4</td>
<td>33/2&quot;X5&quot;X 5/8&quot;</td>
<td>15.32</td>
<td>16.8</td>
<td>1030</td>
</tr>
<tr>
<td>Con. Ls</td>
<td>4</td>
<td>6&quot;X6&quot;X 1/2&quot;</td>
<td>2.56</td>
<td>19.6</td>
<td>206</td>
</tr>
<tr>
<td>Fill pls</td>
<td>4</td>
<td>9&quot;X 5/8&quot;</td>
<td>2.08</td>
<td>19.13</td>
<td>159</td>
</tr>
<tr>
<td>Stif. Ls</td>
<td>4</td>
<td>33/2X3X5/8&quot;</td>
<td>2.56</td>
<td>8.5</td>
<td>87</td>
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<tr>
<td>Fill pl.</td>
<td>4</td>
<td>1/2&quot;X 5/8&quot;</td>
<td>2.0</td>
<td>7.44</td>
<td>59</td>
</tr>
<tr>
<td>Con. pl.</td>
<td>1</td>
<td>12&quot;X 3/8&quot;</td>
<td>2.00</td>
<td>15.3</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9&quot;X 3/8&quot;</td>
<td>1.5</td>
<td>11.43</td>
<td>17</td>
</tr>
<tr>
<td>Rvt. Hds</td>
<td>353</td>
<td>7/8&quot;</td>
<td>24.29</td>
<td>87</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>Total weight</strong></td>
<td>2501 lbs</td>
</tr>
<tr>
<td>Stringer Brackets</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fl. Ls</td>
<td>2</td>
<td>33/2&quot;X5X 5/8&quot;</td>
<td>1.166</td>
<td>16.8</td>
<td>39</td>
</tr>
<tr>
<td>Web pl.</td>
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<td>2.646</td>
<td>23.38</td>
<td>62</td>
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<tr>
<td>Fill pl.</td>
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<td>4&quot;X 5/8&quot;</td>
<td>2.354</td>
<td>8.5</td>
<td>40</td>
</tr>
<tr>
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<tr>
<td>Rvt. Hds</td>
<td>26</td>
<td>7/8&quot;</td>
<td>6.3</td>
<td><strong>Total weight</strong></td>
<td>238 lbs</td>
</tr>
<tr>
<td>Safety Stringer Brackets</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I beam</td>
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<td>20&quot;</td>
<td>1.166</td>
<td>65</td>
<td>76</td>
</tr>
<tr>
<td>Con Ls</td>
<td>2</td>
<td>6&quot;X6&quot;X 1/2&quot;</td>
<td>1.458</td>
<td>19.6</td>
<td>57</td>
</tr>
<tr>
<td>Fill</td>
<td>1</td>
<td>6 1/4&quot;X 1/2&quot;</td>
<td>0.333</td>
<td>10.63</td>
<td>4</td>
</tr>
<tr>
<td>Rvt. Hds</td>
<td>14</td>
<td>7/8&quot;</td>
<td>24.29</td>
<td><strong>Total weight</strong></td>
<td><strong>3</strong></td>
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</tbody>
</table>

Total weight = 2501 lbs

**Total weight** = 238 lbs
<table>
<thead>
<tr>
<th>Piece</th>
<th>No of</th>
<th>Cross</th>
<th>Length</th>
<th>Weight</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>in ft.</td>
<td>per ft.</td>
<td></td>
</tr>
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<td></td>
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</tr>
</tbody>
</table>

Total weight of safety stringer brackets: 140 lbs

**Cross Frames.**

<table>
<thead>
<tr>
<th>L's</th>
<th>2</th>
<th>3&quot;X3&quot;X3/8&quot;</th>
<th>4.771</th>
<th>7.2</th>
<th>68.6</th>
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<tbody>
<tr>
<td>L's</td>
<td>2</td>
<td>3&quot;X3&quot;X3/8&quot;</td>
<td>4.193</td>
<td>7.2</td>
<td>60.4</td>
</tr>
<tr>
<td>Pls.</td>
<td>4</td>
<td>9 1/2&quot;X3/8&quot;</td>
<td>1.167</td>
<td>12.12</td>
<td>56.6</td>
</tr>
<tr>
<td>Rvt. Hds: 50</td>
<td>7/8&quot;</td>
<td>24.29</td>
<td>100</td>
<td>12.1</td>
<td></td>
</tr>
</tbody>
</table>

Total weight = 197.7 lbs

**Stringer Laterals.**

| L   | 1   | 3"X3"X3/8" | 5.599 | 7.2 | 40.0 |

Total weight = 40.1 lbs

**Bottom Laterals, BL & BL2.**

<table>
<thead>
<tr>
<th>L</th>
<th>2</th>
<th>5X3 1/2&quot;X 1/2&quot;</th>
<th>1.060</th>
<th>11.1</th>
<th>383.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>8</td>
<td>5X3 1/2&quot;X 1/2&quot;</td>
<td>20.135</td>
<td>13.6</td>
<td>547.4</td>
</tr>
<tr>
<td>L</td>
<td>8</td>
<td>3 1/2&quot;X3/2&quot;X 3/8</td>
<td>1.125</td>
<td>3.5</td>
<td>76.5</td>
</tr>
<tr>
<td>Pl.</td>
<td>8</td>
<td>9&quot;X3/8&quot;</td>
<td>1.125</td>
<td>11.43</td>
<td>103.3</td>
</tr>
<tr>
<td>L</td>
<td>2</td>
<td>5&quot;X3/2&quot;X 1/2&quot;</td>
<td>9.776</td>
<td>13.6</td>
<td>266.0</td>
</tr>
<tr>
<td>L</td>
<td>2</td>
<td>5&quot;X3/2&quot;X 1/2&quot;</td>
<td>9.333</td>
<td>13.6</td>
<td>263.3</td>
</tr>
<tr>
<td>L</td>
<td>4</td>
<td>3 1/2&quot;X3/2&quot;X 1/2&quot;</td>
<td>1.125</td>
<td>11.1</td>
<td>50.0</td>
</tr>
</tbody>
</table>

Total weight = 1459.1 lbs
### Bottom Laterals, BL3&BL4.

<table>
<thead>
<tr>
<th>Piece</th>
<th>No. of</th>
<th>Cross</th>
<th>Length:Weight</th>
<th>Total:</th>
</tr>
</thead>
<tbody>
<tr>
<td>: L</td>
<td>: 2</td>
<td>: 3(\frac{1}{2})&quot;X 3(\frac{1}{2})&quot;X 1(\frac{1}{2})&quot;</td>
<td>: 20.219: 11.1</td>
<td>: 449.3 :</td>
</tr>
<tr>
<td>: L</td>
<td>: 8</td>
<td>: &quot; X &quot; X (\frac{3}{2})&quot;</td>
<td>: 1.125: 8.5</td>
<td>: 76.5 :</td>
</tr>
<tr>
<td>: L</td>
<td>: 2</td>
<td>: &quot; X &quot; X (\frac{1}{2})&quot;</td>
<td>: 9.943: 11.1</td>
<td>: 220.6 :</td>
</tr>
<tr>
<td>: L</td>
<td>: 2</td>
<td>: &quot; X &quot; X (\frac{1}{2})&quot;</td>
<td>: 9.901: 11.1</td>
<td>: 220.0 :</td>
</tr>
<tr>
<td>Plate</td>
<td>: 8</td>
<td>: 9&quot;X 3/8&quot;</td>
<td>: 1.125: 11.48</td>
<td>: 103.3 :</td>
</tr>
<tr>
<td>: L</td>
<td>: 4</td>
<td>: 3(\frac{1}{2})&quot;X 3(\frac{1}{2})&quot;X 1(\frac{1}{2})&quot;</td>
<td>: 0.625: 11.1</td>
<td>: 27.3 :</td>
</tr>
<tr>
<td>Plate</td>
<td>: 2</td>
<td>: 7 1/2&quot;X 3/8&quot;</td>
<td>: 1.375: 9.57</td>
<td>: 57.9 :</td>
</tr>
</tbody>
</table>

**Total weight**

= 1170.3 :

### Bottom Laterals, BL5&BL6.

<table>
<thead>
<tr>
<th>Piece</th>
<th>Cross</th>
<th>Length:Weight</th>
<th>Total:</th>
</tr>
</thead>
<tbody>
<tr>
<td>: L</td>
<td>: 1</td>
<td>: 3(\frac{1}{2})&quot;X 3(\frac{1}{2})&quot;X (\frac{3}{8})&quot;</td>
<td>: 20.219: 3.5</td>
</tr>
<tr>
<td>: L</td>
<td>: 1</td>
<td>: &quot; &quot; &quot;</td>
<td>: 9.943: 8.5</td>
</tr>
<tr>
<td>: L</td>
<td>: 1</td>
<td>: &quot; &quot; &quot;</td>
<td>: 9.901: 3.5</td>
</tr>
<tr>
<td>: L</td>
<td>: 4</td>
<td>: &quot; &quot; &quot;</td>
<td>: 1.125: 3.5</td>
</tr>
<tr>
<td>: L</td>
<td>: 2</td>
<td>: &quot; &quot; &quot;</td>
<td>: 0.625: 8.5</td>
</tr>
<tr>
<td>Pl.</td>
<td>: 4</td>
<td>: 9&quot;X 3/8&quot;</td>
<td>: 1.125: 11.48</td>
</tr>
<tr>
<td>Pl.</td>
<td>: 1</td>
<td>: 7 1/2&quot;X 3/8&quot;</td>
<td>: 1.625: 9.57</td>
</tr>
</tbody>
</table>

**Total weight**

= 473.9 :
Table. 1
Summary of Weights of Floor System.

<table>
<thead>
<tr>
<th>No. of pieces</th>
<th>Name of piece</th>
<th>Weight each</th>
<th>Total weight:</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>End floorbeam</td>
<td>3611</td>
<td>7222</td>
</tr>
<tr>
<td>4</td>
<td>Int. floorbeam</td>
<td>4100</td>
<td>16400</td>
</tr>
<tr>
<td>10</td>
<td>Main Stringer</td>
<td>2501</td>
<td>25010</td>
</tr>
<tr>
<td>10</td>
<td>Safety Str.</td>
<td>1119</td>
<td>11190</td>
</tr>
<tr>
<td>4</td>
<td>Str. bracket</td>
<td>238</td>
<td>952</td>
</tr>
<tr>
<td>4</td>
<td>S. Str. bracket</td>
<td>140</td>
<td>560</td>
</tr>
<tr>
<td>10</td>
<td>Cross frame</td>
<td>197.7</td>
<td>1977</td>
</tr>
<tr>
<td>15</td>
<td>Str. lateral</td>
<td>40</td>
<td>600</td>
</tr>
<tr>
<td>2</td>
<td>B.L. 1--B.L. 2</td>
<td>1459.1</td>
<td>2918</td>
</tr>
<tr>
<td>2</td>
<td>B.L. 3--B.L. 4</td>
<td>1170.3</td>
<td>2342</td>
</tr>
<tr>
<td>1</td>
<td>B.L. 5</td>
<td>473.9</td>
<td>474</td>
</tr>
</tbody>
</table>

Total weight of floor-system: 69645 lbs.

*******

Stringers.

The main stringers are spaced 5'-0" C to C and the centers of the safety stringers are 2'-0" outside of the main stringers.

The main stringers are designed to carry all the load and the safety stringers are made half as strong as the main stringers.

Dead Load: Weight of main stringer = 2500 lbs.
Cross-frames & bracing = 500 ".
Track @ 400 lbs. per ft. 3100 ".
Total stringer dead load = 6100 ".
Dead load moment = \( \frac{6100 \times 15.5 \times 12}{8} = 141825 \text{ in.} \text{lbs.} \)

Live load moment (Cooper pg. 30) = 1937500 " 

Total moment = 2129325 " 

Economic Depth:— Johnson's formula for economic depth is,

\[ X = 1.41 \sqrt{\frac{m}{f t}} \]

\( m = \text{total moment} = 2129325 \text{ in.} \text{lbs.} \)

\( f = \text{allowable stress on gross area} = 85\% \text{of 9000 lbs} \)

or approximately 7650 lbs.

\( t = \text{thickness of plate} = 1/2 \text{ inch} \)

Then \( X = 1.41 \sqrt{\frac{2129325}{7650 \times 0.5}} = 33.3 \text{ inches} \)

The actual depth of the stringer web plate is 31.75 inches which is not economic according to this formula. Practice however uses a less depth than this formula.

The effective depth is 32 - 2 \( \times \) 0.95 = 30.1 inches

NOTE:—The allowable stress for soft steel in tension is 9000 lbs./sq.in.

Shear in the web:— Live load end shear = 51560 lbs.

Dead load end shear = \( \frac{3050}{1} \)

Total end shear = 54610 "

The gross section of the web is 31.75 \( \times \) 0.5 = 15.87 sq.in.

Sectional area of 9 rivet holes = 4.5 "

Net area of web plate------ = 11.37 "

Unit end shear in web plate = \( \frac{54610}{11.37} = 4800 \text{ lbs./sq.ft.} \)

Allowable unit shear (Lewis' Specifications) for soft
Steel is 5000 lbs. Therefore the section is ample.

Angles:- The flange stress is \( \frac{2129325}{70300} = 30.1 \text{ lbs.} \)

Area required = \( \frac{70300}{9000} = 7.67 \text{ sq.in.} \)

The flanges are made of 2-angles 3-1/2" X 5" X 5/8", the area of which = 9.86 sq.in.

Stiffeners:- The specifications call for stiffeners if the shearing stress exceeds 10000 ft-lb/in this case, 6190.

Since the end shearing stress is 4800 lbs., stiffeners are not required. There is however a pair of stiffeners five ft. from the end and connected by 8 rivets.

Connecting Angles:- The connecting angles are:
2-angles 6"X6"X 1/2" and are therefore of ample section to resist bearing.

Rivet Spacing in Flanges:-
Rivet spacing \( = P = \frac{r h}{s} \) (Mill Buildings pg.183)

\( r \) = allowable bearing on one rivet.
\( h \) = distance between rivet lines.
\( s \) = maximum shear. The maximum end shear = 54610 lbs.

This value is taken as produced by a uniform load, erring on the safe side, and the shear at different sections computed.

Req'd spacing at end = \( \frac{4630 \times 23}{26400} = 2.4 \text{ inches.} \)

Actual spacing = 2.63 inches up to a point from the end 4'-1-7/16".

Req'd spacing at 4' point = \( \frac{4630 \times 23}{26400} = 4.96 \text{ inches.} \)
The maximum spacing in the stringer is 5" at the middle so the rest of the spacing is within the limit.

Rivet Spacing in Connecting Angles between Angles and Web Plate:
The end shear per stringer = 54610 lbs
The number of rivets required in the stringer in bearing = $\frac{54610}{4680} = 11.7$ and their are 13 rivets spaced in two rows.

Floorbeams
The floorbeams are spaced 15-5"C to C.

Live load: The live load maximum floor-beam reaction is 69687 lbs. per stringer (Cooper pg. 30)
The live load moment = 69687 X 12 X 5-2/3 = 4738716 in. lbs
(The centers of main stringers are 5'-8" from the centers of the girders.)

Dead load: Weight of floorbeam = 4100 lbs.
" " stringer = 2501 "
" " Crossframes etc. = 500 "
" " Track = 3100 "
" " Safety Stringer = 1119 "

Moment of main stringer and track =
$(2501+500+3100) X 12 X 5-2/3 = \frac{414800}{in. lbs.}$

Moment of safety Str. = 1119 X 12 X 3-2/3 = 47000 " "
" " Floorbeam = $\frac{4100 X 12 X 16-1/3}{8} = \frac{100450}{in. lbs.}$

Total dead load moment  
" live " = \frac{4738716}{in. lbs.}
" combined " = \frac{5300966}{in. lbs.}
Economic Depth: \( X = 1.41 \sqrt[2]{\frac{5300966}{7650 \times 1/2}} = 52.2 \text{ inches.} \)

The actual depth is 42.25 inches.

The effective depth is \( 3' - 6\frac{1}{2}'' \text{ minus } 3\frac{1}{2}'' = 39'' \)

The flange stress is \( \frac{5300966}{39} = 136000 \text{ lbs.} \)

The rivets in the flanges are spaced alternately in two rows (Fig. 3) and are so close that the diagonal distance between adjacent rivets does not exceed by 30% the transverse distance between rivet lines, therefore the rupture must be considered along the diagonal line and the area of two rivet holes must be deducted from the gross area of each angle.

![Fig. 3](image)

Showing section of spacing in floorbeam flange.

The flange angles are 2 angles 6"X6"X3/4"

whose area together is 16.83 sq.in.

deduct 4 rivet holes or 3.00 " "

leaves a net section of 13.88 sq.in.

The unit stress is \( \frac{136000}{13.88} = 9730 \text{ lbs. per sq. in.} \)

The allowable stress is 9000 lbs. per sq. in.

Rivets for connecting Angles, between Stringer & Floorbeam:

The live load floorbeam reaction = 69637 lbs.

" Dead " shear = 6100 "

Total end " = 75737 "
The number of rivets required is \( \frac{75787}{3150} = 24.1 \)

The actual number is 22.

The same criterion determines the number of rivets in the connecting angles between the angles and the web of the floorbeam.

**Rivets between Connecting Angles and Web of Girder:**

One angle is shop riveted and the rivets are in single shear, so the allowable stress will be taken as 3600 lbs. The number required is \( \frac{77837}{3600} = 22 \)

The actual number is 23.

**Rivet Spacing in Flanges:**

\[ p = \frac{r h}{s} = \frac{4680 \times 36}{77837} = 2.17 \text{ inches at the ends.} \]

The rivets are in two rows 1-1/4 inches apart and the pitch is 2-1/2 inches outside of the stringers and 4 inches between stringers.

**Shear in Web:**

The gross section of the web is

\[ 42.5 \times 1/2" = 21.25 \text{ sq.in.} \]

```
<table>
<thead>
<tr>
<th>Deduct 10 rivets</th>
<th>Net area</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>16.25</td>
</tr>
</tbody>
</table>
```

Maximum end shear \( = 77837 \text{ lbs.} \)

Unit stress in web \( = \frac{77837}{16.25} = 4780 \text{ lbs.per sq.in.} \)

The allowable stress is 5400 lbs.per sq.in.

The section is therefore ample.
End Floorbeam

The web section is the same as for the intermediate floorbeam so it is more than sufficient.

Live Load: The live load shear is 103124 lbs.

(Cooper pg. 30)

The live load moment = \( \frac{103124}{2} \times 12 \times \frac{5-2/3}{2} \)

= 3507000 inch lbs.

Dead Load: Moment of stringer and track =

3050x12x5-2/3 = 207400 in. lbs.

Moment of safety str. = \( \frac{1119}{2} \times 12 \times 3-1/3 \) = 23500 " "

" "Floorbeam = \( \frac{3611 \times 12 \times 16-1/3}{8} \) = 33400 " "

Total dead load moment = 319300 " "

" live " " = 3507000 " "

" combined " = 3826300 " "

The effective depth is 42.5 - 2.16 = 40.3 inches.

The flange stress is \( \frac{3326300}{40.3} \) = 94940 lbs.

The flange angles are 2 angles 6"X 4"X 3/4"

whose area = 13.88 sq.in

Deduct 2 rivet holes 5/16 " "

Net area = 12.38 " "

The unit stress is \( \frac{95200}{12.38} \) = 7680 lbs per sq.in.

The allowable unit stress is 9000 lbs. per sq.in.

Therefore the flanges are of sufficient strength.
Web Section:-

Live Load:— Maximum live load end shear in panel \( L_0 L_1 = 151341 \text{ lbs.} \) (Cooper p.30)

Dead Load:— Assumed weight of metal = \( 2000 \text{ lbs. per ft.} \)

\[ \text{track} \quad = \quad 400 \text{ lbs. per ft.} \]

(per ft. of bridge) Dead Load \( = \quad 2400 \text{ lbs. per ft.} \)

or \( 1200 \text{ lbs. per linear ft. of girder.} \)

Total dead load = \( 2400 \times 77.5 = 186000 \text{ lbs.} \)

minus weight of track = \( 400 \times 77.5 = 31000 \text{ lbs.} \)

leaves the assumed weight of steel = \( 155000 \text{ lbs.} \)

This does not include the ends of the girders which extend beyond the supports.

The actual weight of the bridge steel is:-

Weight of two girders \( = 85550 \text{ lbs.} \)

Total weight of floor-system \( = 69645 \text{ lbs.} \)

Total weight of bridge steel \( = 155195 \text{ lbs.} \)
This value differs from the estimated weight by only 195 lbs. which certainly is a remarkably close estimate. The assumed weight or 155000 lbs. will be used in this investigation.

Total dead load on one girder = 136000 lbs.
Panel load on one girder = \(\frac{186000}{5 \times 2} = 18600\) lbs.

Dead load shear = 37200 lbs.
Live " " = 151341 "
Total end shear = 188541 "

The unit shearing stress is taken as 7200 lbs. then the required net section of the web is 99.5" X 3/8" = 37.3 sq.in.

Area of 23 rivet holes = 8.6 " "
Net area = 23.7 " "

Sectional Area of Flanges:– The maximum live load moment (Cooper pg. 30) is \(-2551406\) ft.lbs.

Maximum dead load moment = \(\frac{93000 \times 77.5}{8} = 901700\) " "
Total moment = 3453106 " "
or 41437272 inch lbs.

Composition of Flanges:– The flanges are made up as follows:–
2 angles 8"X8"X3/4" area = 22.33 sq.in.
1 cover plate 18"X5/8" = 11.25 " "
1 " " " = 11.25 " "
1 " 18"X1/2" = 9.0 " "
Total gross area = 54.38 " "
Areas of Rivet Holes Deducted:

Considering rupture along a diagonal line of holes, the path between center of rivets is \( \sqrt{5^2 + 3^2} = 5.83 \) in

- \( 1.0 \) in

edges of holes ------- = 4.83 in

The distance between rivet lines is = 3.0 in.

half of a rivet hole = 0.5 in

The transverse line across rivet holes = 2.5 in

Since 4.83 exceeds 2.5 by more than 30%, the transverse line will be considered and two rivet holes will be deducted from each angle and each plate.

Areas of holes deducted:

4 holes in angles at .75 sq.in. each = 3.00 sq.in.

2 plates at .63 " " " = 2.52 " "

2 " " 1 " " .5 " " " = 1.0 " "

Total rivet hole area = 6.52 " "

Gross area = 54.38 " "

Total net area of flanges = 47.36 " "

Fig. 5. Smallest rivet spacing in lower flange near center of girder.
Effective Depth of Web:-

Center of gravity of angles from backs = 2.19 inches
" " " plates " same = 0.875 "

Call area of angles ----- $A_1$
" " " plates ----- $A_2$

Center of gravity of gross flange from angle backs is
$$\frac{A_1 \times 2.19 - A_2 \times 0.875}{A_1 + A_2} = \frac{50 - 27.5}{54.38} = 0.414 \text{ inches.}$$

Twice this value or 0.828 inches added to the depth of web plate = 100.0 inches gives an effective depth of 100.83 inches. The unit stress for soft steel in tension is 9000 lbs. Therefore the area required is
$$\frac{M}{SH} = \frac{41437273}{9000 \times 100.83} = 45 \text{ sq.in.}$$

The section is therefore of sufficient area.

Web Splices:- the web of the girder is spliced at each panel point and the splices are all similar. The splice requiring the greatest strength is that at $L_1$ and it will therefore be investigated. Since the loads are carried to the girders at the panel points by the floorsystem in a through girder, the shear must be considered as constant between panels.

Consider the shear at $L_1$ as that in the panel $L_1L_2$.

Live Load Shear in Panel $L_1L_2$:–

Try wheel 2 of Cooper's E-50 loading, at $L_2$, then the total load on the girder is 142000 lbs. The average panel load is $\frac{142}{5} = 28400$ lbs. The load in the panel $L_1L_2$ is 10 to 30 thousand pounds.
This position therefore produces a maximum shear. The rule is: The maximum live load shear occurs when the load in the panel is equal to the total load on the bridge divided by the number of panels. With the loads in this position, wheel number 9 is 6.5 ft. from the right end of the bridge and the shear in

\[ \frac{3496 + 142 \times 6.5}{77.5} \times \frac{80}{15.5} = 51600 \text{ lbs. for E-40} \]

or for E-50 loading

\[ = 64500 \text{ lbs.} \]

The dead load shear in \( L_1 L_2 = 13600 \) "

Total shear in \( L_1 L_2 = 83100 \) "

The splice consists of two plates, one on each side of the web, 14-1/2x3/8" and consists of two rows of 21 rivets on each side of the joint. The plates are therefore of ample section to transmit the shear, so that the riveting is the only thing needing investigation. One of these rows of rivets is of field rivets holding one connecting angle from the floor-beam. The direct shear taken by each rivet is \( S = \frac{83100}{42} = 1978 \text{ lbs.} \) Besides the direct shear, each rivet must take shear due to moment, caused by the eccentricity of the connection. The center of the rivet lines is 4 inches from the line of action of the direct shear, therefore the total moment of the direct shear is \( M = 83100 \times 4 = 332400 \text{ inch lbs.} \)

Each rivet will take an additional shear due to its share of this moment which varies as its distance
GRAPHIC SOLUTION OF STRESS IN RIVETS OF WEB SPlice DUE TO ECCENTRICITY OF CONNECTION.
from the center of gravity of all the rivets. If 
d = the distance of each rivet from the center of 
gravity and 
a = the resulting shear due to bending moment in a 
rivet and 
m = the shear in any rivet due to bending moment and 
M = the total moment, Then m = aXd , and a = \frac{M}{\sum d^2} 
and the resultant shear, R, is the resultant of m and S. 
Table 2 gives values of d, m, S and R for each rivet 
on one side of the neutral axis.

Table 2.

<table>
<thead>
<tr>
<th>Rivet</th>
<th>d</th>
<th>d^2</th>
<th>m</th>
<th>S</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.37</td>
<td>19.06</td>
<td>234</td>
<td>1978</td>
<td>1900</td>
</tr>
<tr>
<td>3</td>
<td>8.19</td>
<td>67.06</td>
<td>438</td>
<td>1960</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>12.12</td>
<td>147.06</td>
<td>648</td>
<td>1980</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>16.09</td>
<td>259.06</td>
<td>862</td>
<td>2050</td>
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</tr>
<tr>
<td>6</td>
<td>20.07</td>
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<td>2180</td>
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<tr>
<td>7</td>
<td>24.06</td>
<td>579.06</td>
<td>1238</td>
<td>2280</td>
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</tr>
<tr>
<td>8</td>
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<td>787.06</td>
<td>1500</td>
<td>2400</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>32.04</td>
<td>1027.06</td>
<td>1715</td>
<td>2520</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>36.04</td>
<td>1299.06</td>
<td>1930</td>
<td>2700</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>40.03</td>
<td>1603.06</td>
<td>2143</td>
<td>2850</td>
<td></td>
</tr>
</tbody>
</table>

\[ \sum d^2 = 6190.6 \]
\[ a = \frac{332400}{6190.6} = 53.6 \]
The rivets are in double shear so bearing in the web
will govern. The bearing values of 7/8" rivets are:—
for shop rivets 4410 lbs., for field rivets 2970 lbs.
The bearing of each rivet is thus less than the
allowable, and since this splice is efficient the
others must be alright.

Stiffeners:— Cooper specifies the use of stiffeners
where the unit shearing stress exceeds that allowed
by the formula:—

\[ S = 1000 - 75 \times \frac{H}{H} \]

where \( H = \frac{d}{t} \)

then

\[ S = 1000 - 75 \times \frac{99.5}{\frac{3}{8}} = 10000 - 19375 = -9875 \]

The unit shear exceeds this amount so stiffeners are
required. All stiffeners must be capable of carrying
the maximum vertical shear without exceeding the
allowed unit stress:—

\[ P = 10000 - 45 \times \frac{l}{r} \]

The stiffener angles in the first panel are

\( \frac{3}{2} " \times \frac{5 \times 1"}{2} \times 100 " \). The area of the two is 8.0 sq.in.

and \( r = 2.43 \). Then

\[ P = 10000 - 45 \times \frac{100}{2.43} = 8155 \text{ lbs.} \]

The total shear in the first panel is 145700 lbs.
then the stiffening area required is

\[ \frac{1457000}{8155} = 17.87 \text{ sq.in.} \]

This is more than twice as large as the actual
area, but in practice the stiffeners seldom come up to
Cooper's Specifications. Johnson pronounces stiffeners
sufficient if the outside legs are 1/30 of the depth
of the web. According to this, these stiffeners are
ample.

In panel 2 the stiffeners are 3/8" thick and the
area is 6.1 sq.in. The total shear is 33100 lbs.
making the required area = \(\frac{33100}{3155} = 10.2\) sq.in.
This also is insufficient \(\frac{31750}{3155}\) according to Cooper
In panel 3 the stiffeners are 3/8" thick and must take only the live load shear which is 31,750 lbs., giving a required area of \(\frac{31750}{3155} = 3.89\) sq.in. These stiffeners are therefore ample.

The stiffeners are spaced at distances varying from 5'-0\(\frac{1}{2}\)" to 5'-3\(\frac{1}{4}\)" which just exceeds Cooper's rule by the odd inches.

At the ends of the girders at panel points L\(_0\) & L\(_5\) the web is stiffened by:

4 angles 3\(\frac{1}{2}\)" X 5" X \(\frac{1}{2}\)" area = 16. sq.in.
2 fills 7 \(\frac{3}{4}\)" X \(\frac{3}{4}\)" \(\frac{3}{4}\)" = 11.62 ""
1 " 4 \(\frac{3}{8}\)" X \(\frac{1}{2}\)" \(\frac{1}{2}\)" = 2.38 ""
Total area = 30.0 ""
Besides this the upper flange angles are bent down over the ends of the girder and take some shear, so the end shear at L\(_0\) is amply provided for.

Flange Plates: "Flange plates must extend beyond their theoretical length, two rows of rivets at each end."
"Where two or more plates are used in the flanges, they shall be either of equal thickness or shall decrease in thickness outward from the angles."

Johnson gives the following formula for the theoretical lengths of cover plates:-(pg.302)

\[ X_n = L \sqrt{\frac{a_1 + a_2 + \cdots + a_n}{A}} \]
where \( X \) = length of plate; \( l \) = the span; \( a_1, a_2, \ldots, a_n \) = areas of 1st, 2nd, \ldots, nth, etc. plates from outside; 
\( A \) = total area. The theoretical and actual lengths of the plates are given in Table 3.

<table>
<thead>
<tr>
<th>Theoretical length:</th>
<th>+4 rows of rivets: Actual length</th>
</tr>
</thead>
<tbody>
<tr>
<td>( X_1 = 77.5 \sqrt{\frac{8}{47.88}} ) = 31.6</td>
<td>32.6</td>
</tr>
<tr>
<td>( X_2 = 77.5 \sqrt{\frac{18}{47.88}} ) = 47.5</td>
<td>48.5</td>
</tr>
<tr>
<td>( X_3 = 77.5 \sqrt{\frac{28}{47.88}} ) = 59.4</td>
<td>60.7</td>
</tr>
</tbody>
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Table 3.

The plates are therefore of the right lengths.

Rivet Spacing in Girder Flanges:

<table>
<thead>
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<th>Shears in the panels.</th>
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<td>Dead load shear</td>
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<td>Live &quot; &quot;</td>
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<tr>
<td>Total &quot; &quot;</td>
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</tbody>
</table>

Table 4.

Rivet spacing = \( p = \frac{rh}{s} \)  \( r = 4410, h = 91, s = \) values given in table 4.

Theoretical and actual spacing in panels 1, 2, 3.

<table>
<thead>
<tr>
<th>Theoretical spacing</th>
<th>Actual spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>( p_1 = \frac{4410 \times 91}{145700} = 2.75 ) inches</td>
<td>3 inches</td>
</tr>
<tr>
<td>( p_2 = \frac{4410 \times 91}{83100} = 4.825 ) &quot;</td>
<td>4 &quot;</td>
</tr>
<tr>
<td>( p_3 = \frac{4410 \times 91}{31750} = 12.63 ) &quot;</td>
<td>5 &quot;</td>
</tr>
</tbody>
</table>

Table 5.
Thus the actual spacing is in efficient in the first panel.

Spacing in Flange Plates:— The proportions of flange stress taken by one, two and three plates respectively are approximately, 26, 41, and 51 per cent.

\[ p = \frac{rh}{s} \]

\[ r = 4860 \text{ (rivets are in single shear.)} \]

\[ h = 100, 101.25, \text{and } 102.5. \]

\[ s = .26 \times 145700, .41 \times 83100, \text{and } .5 \times 31750. \]

Spacing in first panel for first plate.

\[ p_1 = \frac{4860 \times 100}{145700 \times .26} = 12.8 \text{ inches.} \]

Spacing in first panel for second plate.

\[ p_2 = \frac{4860 \times 101.25}{145700 \times .41} = 8.25 \text{ inches.} \]

Spacing in second panel for third plate.

\[ p_3 = \frac{4860 \times 102.5}{83100 \times .51} = 11.8 \text{ inches.} \]

The largest actual spacing in the flange plates is 6 inches and this is less than the minimum which is necessary, so all the spacing is within the limit.

The actual spacing however is very irregular due to the necessity of passing of the plate joints, stiffeners and floorbeams in an easy, symmetrical manner; also
Laterals.

Wind Load:— The fixed wind load is 300 lbs. per foot making the dead panel load = 300 \times 15.5 = 4650 lbs.
The live load is 450 lbs. per ft. or 6975 lbs. panel load.

\[ W = \text{dead panel load} = 4650 \text{ lbs.} \]
\[ P = \text{live " } = 6975 \text{ lbs.} \]
\[ \theta = 43^\circ 30', \ W \sec \theta = 6420, \ P \sec \theta = 9625. \]

Combined stresses.

![Diagram showing dead, live, and combined stresses.]

- **Dead load stresses.**
  - **North Panel:** 4220, 6420, 0
  - **South Panel:** 4220, 0, 6420

- **Live load stresses.**
  - **North Panel:** 1920, 11520, 3770
  - **South Panel:** 1920, 3770, 11520

- **Combined stresses.**
  - **North Panel:** 32070, 7960, 5770
  - **South Panel:** 32070, 5770, 7960
Allowable tensile stress on field rivets = 16200 lbs per sq in.

Area required = \( \frac{32070}{16200} = 1.98 \) sq in.

Gross section = 1 angle 5"X 3\( \frac{1}{2} \)"X 3\( \frac{1}{2} \" = 4 sq in.

Deduct two rivet holes = 1 " "

Net section = 3 " "

The angles are fastened by both legs so the strength of the full section is developed.

Allowable shearing stress on field rivets = 5400 lbs

Number of rivets required to fasten each end = \( \frac{32070}{5400} = 5.95 \)

The actual number is 8 which is sufficient to allow for the eccentricity of the connection.

BL\( \text{2} \) is cut to pass BL\( \text{1} \) and the two parts are joined by a plate 9"X \( \frac{1}{2} \" in section which is riveted by two rivets to BL\( \text{1} \). Both parts of BL\( \text{2} \) are connected to the plate by 8 rivets and an extra angle connecting the vertical leg, thus developing the full strength of the section.

BL\( \text{3} \) & BL\( \text{4} \): Area required = \( \frac{17960}{16200} = 1.11 \) sq in.

Gross section = 1 angle 3 \( \frac{1}{2} \"X3 \( \frac{1}{2} \"X1\( \frac{1}{2} \" = 3.25 sq in.

Only one leg of this angle is connected so the area considered is only \( \frac{1}{2} \) of the above = 1.62 sq in.

Deduct one rivet = .5 " "

Net area = 1.12 " "

Number of rivets required = \( \frac{17960}{5400} = 3.3 \)

There are 5 rivets and they are in direct shear.

L & BL :- Area required = \( \frac{5770}{16200} = .356 \) sq.in.

Gross area = 1 angle 3 1/2"X3 1/2"X3/8" = 2.49 sq.in.

Deduct one rivet = \( .37 \) " "

Net area = 2.12 " "

The angles are connected by only one leg so only one half of the section is counted, or 1.06 sq.in.

Number of rivets required at the ends = \( \frac{5770}{5400} = 1.1 \)

There are 4 rivets in direct shear.

In the case of the lateral, in each panel, which is cut; the ends are joined by a plate of sufficient section, by means of an extra angle to connect the vertical leg, and as many rivets are used as are at the ends of the laterals.

Conclusion:- Each part of the bridge has now been investigated. Most of the parts have stood the tests, but the following have not come up to the specifications:

1. The rivet spacing in the ends of the stringer flanges exceeds by .22 inches the theoretic pitch. The pitch however is the minimum allowed. This shows, as did the investigation of the depth of the web, on page , that the depth of the stringer is insufficient.

2. The depth of the floorbeams is 10 inches less than the economic depth.
3. The unit tensile stress in the floorbeam flanges is slightly greater than the allowable stress.

4. There is an insufficient number of rivets, by 3, between the stringer connecting angles and the floorbeams, and also between the floorbeam connecting angles and the floorbeam webs, which require the same number of rivets.

5. The rivet pitch at the ends of the floorbeam flanges is too great by .33 inches.

6. The intermediate girder stiffeners fall far below Cooper's Specification.

7. The rivet spacing in the girder flanges of the first panel is .25 inches too large.

All but numbers 6 & 7 could be corrected by deepening the stringers and floorbeams, Nos. 6 & 7 could be easily corrected without interfering with anything else.

If the bridge had been fabricated of medium steel instead of soft steel there is little doubt but that all the specifications would have been met with, except possibly in the case of the stiffeners. Stiffeners seldom come up to Cooper's Specifications in common practice.

The End.
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