NATIONAL TRANSPORTATION SAFETY BOARD
Office of Research and Engineering
Materials Laboratory Division
Washington, D.C. 20594

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MATERIALS LABORATORY STUDY Report No. 08-103

A. ACCIDENT

Place	:	Minneapolis, Minnesota
Date	:	August 1, 2007
Vehicle	:	I-35W Highway Bridge
NTSB No.	:	HWY07MH024
Investigator	:	Mark Bagnard

B. TOPICS ADDRESSED

Chord-splice calculations.

C. DETAILS OF THE STUDY

Jacobs Engineering located unchecked computation sheets related to the design of connections among the main truss members of the I-35W bridge, and provided these computation sheets to the Safety Board in July 2008. The computation sheets are attached as Appendix C; page numbers indicate that some pages are missing, and several of the pages are marked VOID. The computation sheets included calculations to determine the number of rivets needed to connect the members and the rivet patterns to be used, along with calculations to evaluate the stresses in the splices between the chord members. These chord-splice calculations were used in part to size the gusset plates at the joints. The designations for the different grades of steel referred to in the computation sheets indicate that these calculations were performed for the preliminary design of the bridge; the evolution of the choices of materials used for the design of the bridge are documented in Materials Laboratory Study Report 08-074.

In this report, the methodology used to size the gusset plates in the unchecked computation sheets is described, and the calculations performed for the preliminary design of the bridge are recomputed. The same methodology is then used to evaluate the final design of the I-35W bridge. The final design differs from the preliminary design evaluated in the computation sheets in that, for specific structural elements, the member design loads changed, the gusset and splice plate materials changed, and the gusset plate and splice plate thicknesses changed. The purpose of the calculations is to determine the maximum stress (direct tension or compression stress, plus bending stress) in the top or bottom plates used to splice the chord members, for comparison against an allowable stress. Figure 1 shows an example of the geometry investigated, using the arrangement at node

U10, with an end view of the plates that connect the chord members. The chord members are typically connected by two internal vertical splice plates, two external vertical gusset plates, and external horizontal top and bottom splice plates. For some nodes, there are additional external vertical splice plates added on the outside of the gusset plates.

In the computation sheets, the force to be carried across a chord splice was determined by considering the entire main truss as a beam in bending, so that the force across a node is assumed to be equal in magnitude but opposite in sign to the force carried by the chord member above or below it. The angles of the lower chord members and splices were included in the calculations, but the effect was small. As an example, the force across node U10 is assumed to be the negative of the horizontal component of the force carried in chord member L9/L11. Similarly, the force in chord member U10/U12 is assumed to be the negative of the horizontal component of the force across node L11. If the force across the chord splice is tensile, net section properties are used to calculate the stress; if the force across the chord splice is compressive, gross section properties are used to calculate the stress (except for the horizontal top and bottom splice plates, as indicated in the summary below).

The calculation methodology is summarized below, and an example of a spreadsheet used to compute the stress is shown for node U10 in figure 2.

1. Determine the gross area and the vertical position of the center of mass of each connecting plate (splice plates and gusset plates).
2. Calculate the total gross area and the vertical position of the center of mass of all of the connecting plates together.
3. Calculate the total net area of the connecting plates by subtracting the areas of the rivet holes along the chord members and the vertical member; the rivet holes are assumed to be 1.125 inches in diameter.
4. Calculate the gross moment of inertia of each gusset plate and lateral splice plate as $(bh^3/12)+(A_g d^2)$, where b is the plate thickness and h is the plate height, A_g is the gross area of the plate and d is the distance between the center of mass of the plate and the center of mass of all the connecting plates together.
5. Calculate the net moment of inertia of the top and bottom splice plates as $A_n d^2$, where A_n is the net area of the plate and d is the distance between the center of mass of the plate and the center of mass of all the connecting plates together.
6. Calculate the total gross moment of inertia from the sum of the moments of inertia calculated in steps 4 and 5.
7. Calculate the contribution of each rivet hole to the decrease in the gross moment of inertia as $A_r d^2$, where A_r is area of the rivet hole given by 1.125 inches multiplied by the thickness of the plates through which that hole passes, and d is the distance between the center of the rivet hole and the center of mass of all the connecting plates together.
8. Calculate the total net moment of inertia by subtracting the sum of the contributions of the rivet holes in step 7 from the total gross moment of inertia in step 6.

9. The force F across the splice is assumed to act along the center of the chord members, so a bending moment M is calculated by multiplying that force times the vertical distance between the center of the chord members and the center of mass of all the connecting plates together.
10. Calculate the total stress as $(F/A)+(My/I)$, where A and I are the total area and moment of inertia (using gross section properties if F is compressive, and using net section properties if F is tensile); y is measured from the center of mass of all the connecting plates together; to get the maximum stress, y is taken at the top surface of the top splice plate for upper chord joints and at the bottom surface of the bottom splice plate for lower chord joints.
11. Compare the total stress to the allowable stress, and revise the geometry as necessary.

Appendix A summarizes the results for the preliminary design of the bridge from the computation sheets that are reproduced in Appendix C. In some cases, the computation sheets indicate that a change in thickness of the gusset plates or splice plates would be necessary based on the comparison with the allowable stress, but the stress for the revised geometry was not recalculated. The results in Appendix A were compiled by recalculating the stress using the revised geometry where necessary. The chord forces in the members opposite the node are also listed in the appendix, along with the chord splice force that was used for each calculation.

Appendix B summarizes the results of applying the chord-splice calculation methodology to the joints of the final design of the bridge. The geometry and materials were determined from the design plans and the shop drawings. The gusset plate height is taken at the centerline of the joint where the two chord members meet, along the center of the attached vertical member. Where the top or bottom edge of the gusset plate was sloped, it was assumed that the edge was 1.75 inches from the center of the outermost rivet hole on the vertical member. Top and bottom splice plates were taken to be 19 inches wide, with rows of five rivet holes in each plate. The internal welded box diaphragms at all upper chord nodes (Diaphragm A in the plans) were not considered in the calculations.

In Appendix B, for nodes U2, L3 and U4, results are summarized for four different values of the chord-splice forces. The forces shown in the appendix correspond to the different values shown in the design plans for the forces in members opposite those nodes. The geometry of the bridge is not perfectly symmetric north and south, and for these nodes, different design forces are listed for the nodes at the north end of the bridge (U2′, L3′ and U4′) than for those at the south end of the bridge. The gusset plate and splice plate geometry is the same at north and south, however, so stresses for forces from both ends of the bridge must be below the allowable stress. Also, for these nodes, the design plans indicate a reversal of the forces can occur depending on the live load, and stresses from those reversing loads must also be compared to the allowable stress. A reversing load is also shown in the plans for member U4/U6 (affecting node L5), but it is small and has been neglected.

Table 1 summarizes the results for the preliminary design of the bridge (sufficient details were not found in the computation sheets to reconstruct the calculations for nodes U2 and U4). The table lists the chord-splice force, the total stress (direct plus bending) and the allowable stress from the I-35W design plans for each material. The final column in the table compares the total stress (demand) to the allowable stress (capacity) as a calculated demand-to-capacity ratio. All of the demand-to-capacity ratios in Table 1 are less than 1, indicating that the total stress is less than the allowable stress for each node. The design of each connection was therefore appropriate for the loads that were considered in these calculations. Figure 3 plots the chord-splice demand-to-capacity ratios for the preliminary design of the bridge, with a graphic indicating the position of each node.

Table 1. Summary of chord-splice calculation results for the preliminary design of the I-35W bridge.

Node	Chord-splice force (kips)	Total stress (ksi)	Allowable stress (ksi)	Demand-to-capacity ratio
U2			27	
L3	1090	25.2	27	0.93
U4			20	
L5	-935	-18.5	27	0.69
U6	1560	35.1	45	0.78
L7	-2350	-37.1	45	0.82
U8	3200	39.1	45	0.87
L9	-2082 (est.)	-32.8	45	0.73
U10	850	21.5	27	0.80
L11	965	24.3	27	0.90
U12	-1976	-35.3	45	0.78
L13	2756	36.1	45	0.80
U14	-2958	-42.2	45	0.94

Table 2 summarizes the results of the chord-splice calculations performed for the final design of the bridge. Like Table 1, Table 2 lists the chord-splice force, the total stress (direct plus bending), the allowable stress and a demand-to-capacity ratio. For nodes with multiple chord-splice forces (resulting from considering reversing loads or different forces on the north and south sides), the force listed in the table is the one that resulted in the highest total stress. For the final design of the bridge, several of the calculated demand-to-capacity ratios exceed 1, indicating that the calculated stress exceeds the allowable stress, meaning the design was not adequate for the loads considered. Figure 3 also plots the chord-splice demand-to-capacity ratios for the final design of the bridge.

Table 2. Summary of chord-splice calculation results for the final design of the I-35W bridge.				
Node	Chord-splice force (kips)	Total stress (ksi)	Allowable stress (ksi)	Demand-to-capacity ratio
U2	722	17.1	20	0.86
L3	944	22.4	20	1.12
U4	951	24.5	20	1.22
L5	-1128	-17.5	27	0.65
U6	1674	24	27	0.89
L7	-2477	-27.8	27	1.03
U8	3335	32.4	45	0.72
L9	-2185	-23.7	27	0.88
U10	909	21.0	27	0.78
L11	929	20.9	27	0.77
U12	-2008	-25.7	27	0.95
L13	2791	28.7	27	1.06
U14	-2975	-24.9	27	0.92

As noted above, between the preliminary and final designs, the member design loads changed, the gusset and splice plate materials changed, and the gusset and splice plate dimensions changed. As a result of the changes in these variables, the demand-to-capacity ratios at several nodes increased beyond an acceptable value of 1. Some effects of these changes are addressed below.

As can be seen from the data in Tables 1 and 2, the chord-splice forces used in the calculations were generally similar for the preliminary and final designs. The force changed by 15 percent at node L3, by 17 percent at node L5 and by 7 percent at node U6. For all the other nodes, the difference was 5 percent or less.

The number of gusset plates and vertical splice plates at each node was also similar for the preliminary and final designs, except at nodes U8 and U14. The preliminary design of node U8 had two gusset plates, two internal splice plates and two external splice plates; the final design included two additional external splice plates. The preliminary design of node U14 had two gusset plates and two internal splice plates, while the final design also included two external splice plates. In both the preliminary and final designs, node L13 had two gusset plates, two internal splice plates and two external splice plates. All of the other nodes had two gusset plates and two internal splice plates in both designs.

In the computations for the preliminary design, the horizontal top and bottom splice plates were taken to be 20 inches wide and 0.375 inch thick. In the final design, the top and bottom splice plates were taken to be 19 inches wide as shown in the design plans,

and all of the top splice plates were 0.375 inch thick. The bottom splice plates in the final design were 0.5 inch thick on lower nodes and 0.375 inch thick on upper nodes. These changes alone would be expected to alter the section properties by about 5 to 10 percent.

Where details of the gusset plate and splice plate vertical dimensions and rivet spacing were found in the computation sheets for the preliminary design, they were generally similar to the final design used for the bridge. At nodes other than U8 and U14 (where the number of splice plates changed), the thicknesses of the gusset plates and splice plates would therefore be the primary factors in any changes in the section properties between the preliminary and final designs.

Between the preliminary design and the final design, the materials specified for the bridge changed; these changes reduced the allowable stress at a number of nodes, as can be seen by comparing Tables 1 and 2. The stresses for the preliminary design were all within the allowable stress for the inputs and conditions that were given or assumed. The forces between the two designs were similar, and the vertical dimensions of the gusset plates and splice plates were also similar, so one possible way to accommodate the decrease in the allowable stress resulting from the change in the materials would have been to increase the thickness of each gusset plate and splice plate by the inverse of the ratio of the final design allowable stress to the preliminary design allowable stress. For those nodes where the allowable stress decreased from 27 ksi to 20 ksi, the thickness of each plate would have to have increased by a factor of 1.35. For those nodes where the allowable stress decreased from 45 ksi to 27 ksi, the thickness of each plate would have to have increased by a factor of 1.67. Table 3 lists the thicknesses of the gusset plates and vertical splice plates for each node in the preliminary design and in the final design, along with the ratio between the final and preliminary design thicknesses, as well as the target ratio for the increase in thickness if this method had been used to adjust for the change in material. Node U6 was the only node where the thicknesses of both the gusset plates and the splice plates were increased in a manner reasonably consistent with the decrease in allowable stress. At node L7, the preliminary plate thicknesses were the same as at node U6 and the change in allowable stress was the same, but the L7 gusset and splice plates did not increase in thickness by a similar amount; the demand-to-capacity ratio for node L7 increased from 0.82 for the preliminary design of the bridge to 1.03 for the final design (as shown in Tables 1 and 2).

Table 3. Comparing gusset plate and splice plate thicknesses to the change in allowable stress for the preliminary and final designs of the I-35W bridge.

Node	Chord Splice Member	Preliminary Design Plate thicknesses (inch)	Final Design Plate thicknesses (inch)	Thickness Ratio (Final/ Prelim.)	Target Ratio Based on Material Change
U2	Gusset plate	-	0.625	-	1.35
	Internal splice plate	-	0.5	-	

L3	Gusset plate	0.625	0.5	0.80	1.35
	Internal splice plate	0.5	0.5	1.00	
U4	Gusset plate	-	0.5	-	1.00
	Internal splice plate	-	0.5	-	
L5	Gusset plate	0.5	0.5	1.00	1.35
	Internal splice plate	0.375	0.5	1.33	
U6	Gusset plate	0.625	1	1.60	1.67
	Internal splice plate	0.5	1	2.00	
L7	Gusset plate	0.625	1	1.60	1.67
	Internal splice plate	0.5	0.625	1.25	
U8	Gusset plate	0.75	0.625	0.83	1.67
	Internal splice plate	0.75	0.75	1.00	
	External splice plate	0.75	0.75	1.00	
	External splice plate	-	0.75	-	
L9	Gusset plate	0.625	1	1.60	1.67
	Internal splice plate	0.5	0.625	1.25	
U10	Gusset plate	0.5	0.5	1.00	1.00
	Internal splice plate	0.5	0.5	1.00	
L11	Gusset plate	0.5	0.5	1.00	1.00
	Internal splice plate	0.5	0.5	1.00	
U12	Gusset plate	0.5	1	2.00	1.67
	Internal splice plate	0.5	0.5	1.00	
L13	Gusset plate	0.75	1	1.33	1.67
	Internal splice plate	0.75	0.625	0.83	
	External splice plate	0.75	1.25	1.67	
U14	Gusset plate	0.75	1	1.33	1.67
	Internal splice plate	0.625	0.5	0.80	
	External splice plate	-	1	-	

An alternative possibility for the change in the demand-to-capacity ratios from the preliminary to the final design is that different forces were used in the chord-splice calculations. The force to be carried by each chord splice in Tables 1 and 2 was derived from the design load in the chord member opposite each node, which is continuous through that specific panel point. The alternative to using that force would be to sum up the components of the design loads parallel to the chord members contributed by all of the connecting members on each side of the node, and use the total force to size the elements of the chord splice. This procedure creates some additional complexity, since each

member design load corresponds to the maximum live load case calculated for that specific member, and the design loads for connecting members may have come from different arrangements of live loads. The design loads are therefore not necessarily in equilibrium, so the sums on either side of a node might not be equal. The reversing loads and differences between the north and south ends of the bridge introduce additional permutations to consider. Table 4 compares the maximum chord-splice force calculated from summing connecting member design loads in the final design with the chord-splice force in Table 2 from the chord member opposite each node. Except at nodes U4, U10 and L11, the chord-splice force from summing connecting member design loads was larger than that from the member opposite the node. Using the connecting member design loads in the chord splice calculations would in no case have reduced any of the stresses that exceeded the allowable stress in Table 2 to a level below the allowable stress, but in several cases using the connecting member loads would have increased an acceptable calculated stress to a level beyond the allowable stress.

Table 4. Comparison between chord-splice forces calculated in two different ways for the final design.

Node	Chord-splice force from summing design loads (kips)	Chord-splice force from chord member opposite node (kips)	Ratio of summation force to opposite member force
U2	1248	722	1.73
L3	1342	944	1.42
U4	871	951	0.92
L5	-1737	-1128	1.54
U6	1993	1674	1.19
L7	-2884	-2477	1.16
U8	3407	3335	1.02
L9	-2597	-2185	1.19
U10	786	909	0.86
L11	698	929	0.75
U12	-2114	-2008	1.05
L13	2900	2791	1.04
U14	-3191	-2975	1.07

Finally, a comparison was made between the chord-splice forces and the (horizontal) shear forces that would have had to be transmitted between the diagonals and the chord members. These shear forces would have been carried only by the gusset plates at each node, except at node U8, where external splice plates extend below the bottom edges of the chord members. In this case, two sets of shear forces are calculated. The first set of shear forces are calculated from the design loads in the diagonals, with the components parallel to the average direction of the chord members in each node then summed. The second set of shear forces are calculated from the design loads in the chord members, again summing components parallel to the average direction of the chord members. These summations again involve member design loads that are not necessarily in equilibrium, so different values are obtained with the two methods. The reversing loads were not considered in these calculations, but the design loads for the nodes on both the north and south ends of the bridge were considered, with the maximum magnitude used. Table 5 compares the shear forces with the chord-splice force from the member opposite the node, and the ratios of the forces are also plotted in figure 4.

Table 5. Comparison of shear forces and chord-splice forces.

Node	Shear force magnitude from design loads in diagonals (kips)	Shear force magnitude from design loads in chord members (kips)	Chord-splice force magnitude from member opposite node (kips)	Ratio of diagonal shear force to chord-splice force	Ratio of chord shear force to chord-splice force
U2	1548	2013	722	2.14	2.79
L3	132	1432	944	0.14	1.52
U4	1159	2061	951	1.22	2.17
L5	1607	850	1128	1.42	0.75
U6	1838	1317	1674	1.10	0.79
L7	2011	1660	2477	0.81	0.67
U8	271	289	3335	0.08	0.09
L9	2699	2426	2185	1.24	1.11
U10	2724	3071	909	3.00	3.38
L11	2577	2917	929	2.77	3.14
U12	2102	1866	2008	1.05	0.93
L13	1315	967	2791	0.47	0.35
U14	0	0	2975	0.00	0.00

As can be seen in Table 5, the shear forces are approximately 3 times as large as the chord-splice forces at nodes U10 and L11. Nodes U10 and L11 are positioned

approximately at the inflection point where the overall bending of the truss changes from positive in the center of the bridge to negative at node 8 (and node 8′) over the piers. The chord-splice forces resulting from overall bending are therefore small, but the shear forces are substantial. Some significant differences between shear forces and chord-splice forces are also observed in the U2, L3, U4 and L5 nodes, but these are nodes where the design loads have the most variability from reversing forces and differences north to south, so the design loads at these nodes are the farthest from equilibrium. An accurate design at each of these nodes might require a set of equilibrium loads calculated to produce the maximum shear force and chord-splice force. The design loads for the higher numbered nodes toward the center of the bridge are generally closer to being in equilibrium. Symmetry at node U14 (at the center of the bridge) and near symmetry at node U8 result in shear forces that are much smaller than the chord-splice forces for those nodes.

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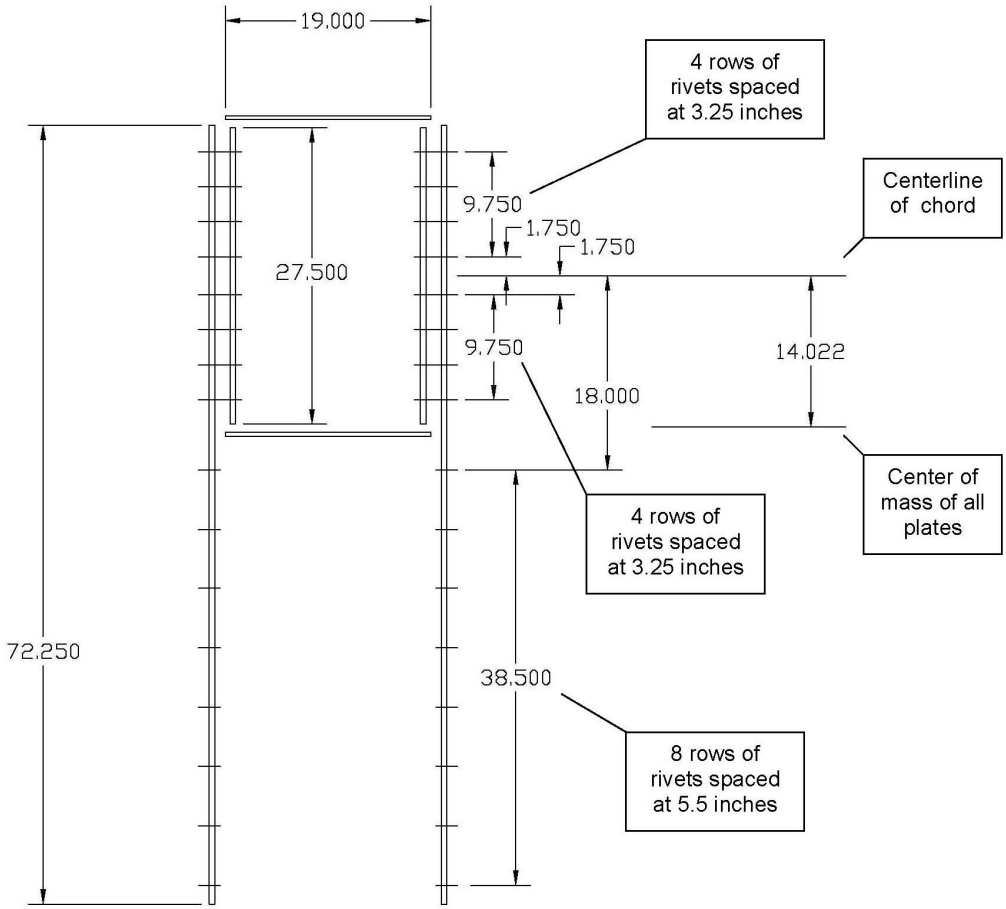


Figure 1. Cross sectional view through the chord splice components at node U10, including two gusset plates (0.5 inch thick by 72.25 inches high), two internal splice plates (0.5 inch thick by 27.5 inches high) and upper and lower splice plates (0.375 inch thick by 19 inches wide). Short lines indicate the positions of the rivets in the gusset plates and internal splice plates, and dimensions are shown. The upper and lower splice plates had five rivets each.

U10		Height	73.25	Hole diam
DWG	1516 AB	Width	104	1.125
MHD 3310		Allowable	27 ksi	

	Height	Thickness	A_gr	y	(A_gr)(y)	# rivets	A_net
Gusset	72.25	0.5	72.25	-22.125	-1598.531	16	54.25
Splices	27.5	0.5	27.5	0	0	8	18.5
	Width	Thickness					
Top	19	0.375	7.125	14.6875	104.6484	5	5.015625
Bottom	19	0.375	7.125	-14.6875	-104.6484	5	5.015625
SUMS			114	-22.125	-1598.531		82.78125

CG	-14.0222

	Height	Thickness	A_gr	d	(A_gr)(d^2)	bh^3/12	Igr
Gusset	72.25	0.5	72.25	-8.102796	4743.596	31429.13	36172.72
Splices	27.5	0.5	27.5	14.0222	5407.111	1733.073	7140.184
	Width	Thickness	A_net				
Top	19	0.375	5.015625	28.7097	4134.114		4134.114
Bottom	19	0.375	5.015625	-0.665296	2.22001		2.22001
I gross							47449.24
I net							36597.27

Hole y	Hole d	Thickness	Ad^2
11.5	25.5222	2	1465.612
8.25	22.2722	2	1116.115
5	19.0222	2	814.1495
1.75	15.7722	2	559.7154
-1.75	12.2722	2	338.8657
-5	9.022204	2	183.1504
-8.25	5.772204	2	74.96626
-11.5	2.522204	2	14.3134
-18	-3.977796	1	17.80072
-23.5	-9.477796	1	101.0572
-29	-14.9778	1	252.3762
-34.5	-20.4778	1	471.7576
-40	-25.9778	1	759.2016
-45.5	-31.4778	1	1114.708
-51	-36.9778	1	1538.277
-56.5	-42.4778	1	2029.909
		I holes	10851.97

Force	909 kips

	Tension	Bending	Total
Stress	10.98075	10.06439	21.04513

Figure 2. Sample spreadsheet calculations for the U10 chord splice.

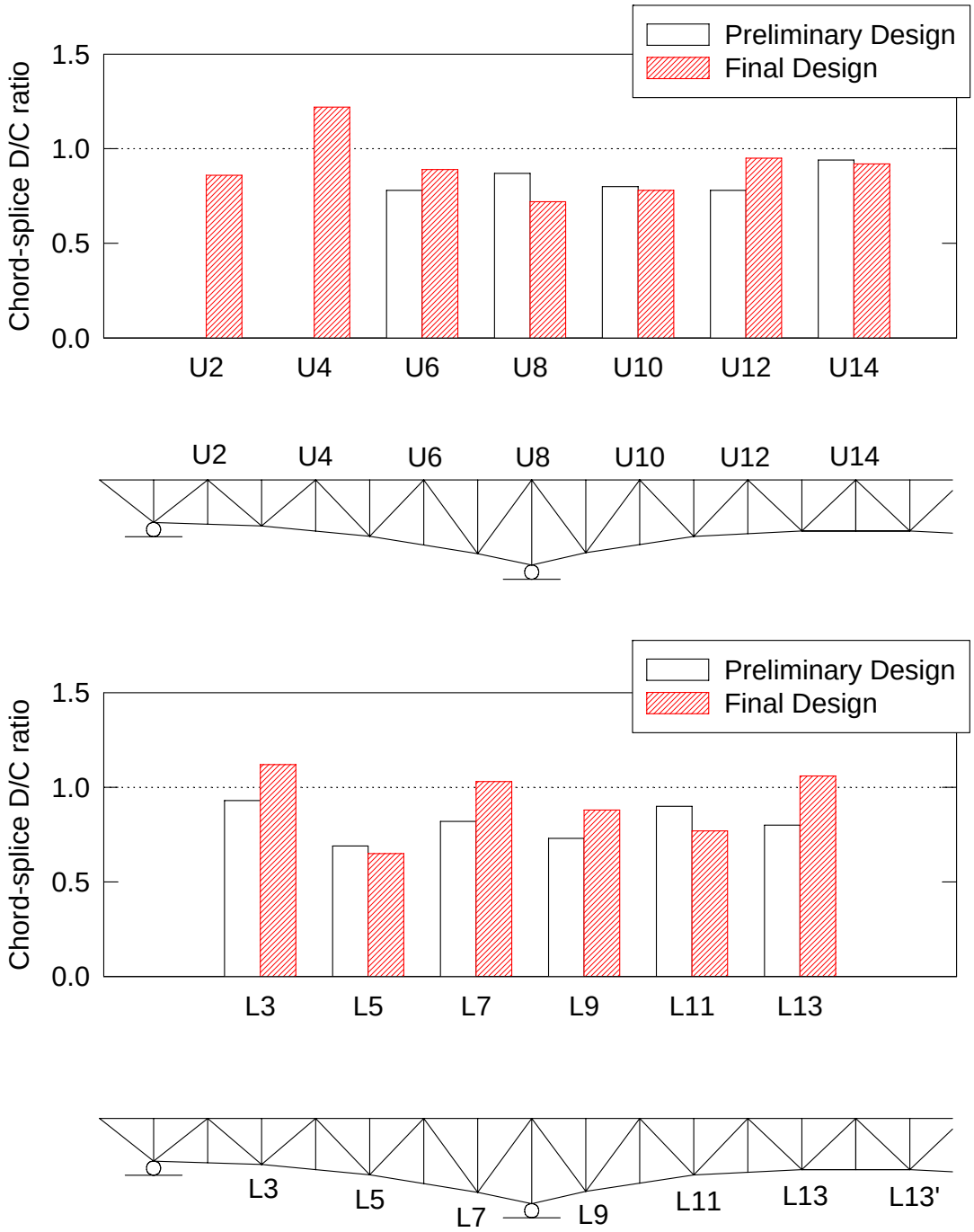


Figure 3. Demand-to-capacity ratios from the chord-splice calculations performed for the preliminary and final designs of the I-35W bridge. Demand-to-capacity ratios less than 1 (the dotted line in the plots) indicate an acceptable design for the forces considered in these calculations.

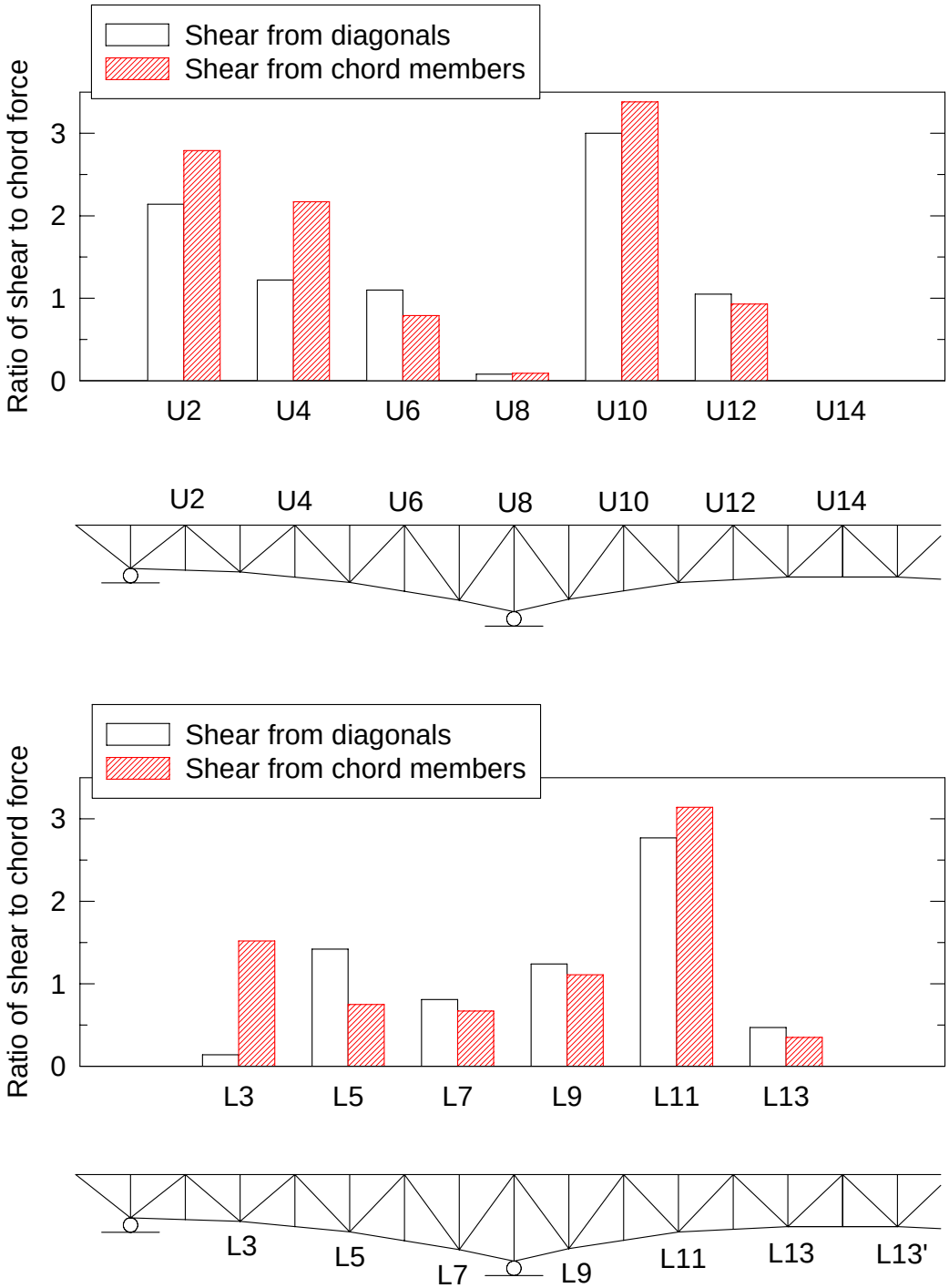


Figure 4. The ratio of the shear force to the chord-splice force for each node. Shear forces were calculated in two ways, from the diagonal member design loads and from the chord member design loads. The chord-splice force was calculated from the chord member design load opposite the node. Calculations for the final design of the I-35W bridge.

Appendix A

Summary of results from the chord-splice calculations for the preliminary design of the bridge.

This appendix summarizes the reconstructed chord-splice calculations from the unchecked computation sheets in Appendix C, which were performed for the preliminary design of the bridge. The chord forces in the member opposite each node are also listed, along with the chord-splice force used in the calculation. The computation sheets lacked sufficient detail about nodes U2 and u4 to reconstruct those calculations.

Node U2	Material – A441 Allowable stress – 27 ksi
L1/L3 chord member force (force in this member can reverse depending on the live load)	524 kips -243 kips

Node L3	Material – A441 Allowable stress – 27 ksi		
Gusset plates	53 inches high	0.625 inch thick	14 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			7.62 inches
U2/U4 chord member force (force in this member can reverse depending on the live load)			196 kips -891 kips
L3 chord splice force			1090 kips
Net area			76 in^2
Net moment of inertia			17313 in^4
Total stress			25.2 ksi

Node U4	Material – A36 Allowable stress – 20 ksi
L3/L5 chord member force (force in this member can reverse depending on the live load)	724 kips -464 kips

Node L5	Material – A441 Allowable stress – 27 ksi		
Gusset plates	60 inches high	0.5 inch thick	
Internal splice plates	27.5 inches high	0.5 inch thick	
Top splice plate	0.375 inch thick	20 inches wide	
Bottom splice plate	0.375 inch thick	20 inches wide	
Distance from center of chord to center of mass of connecting plates			10.04 inches
U4/U6 chord member force (reversing force is -184 kips compression, and neglected)			925 kips
L5 chord splice force			-935 kips
Gross area			96 in^2
Gross moment of inertia			26923 in^4
Total stress			-18.5 ksi

Node U6	Material - T-1 Allowable stress – 45 ksi		
Gusset plates	62 inches high	0.625 inch thick	17 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-10.98 inches
L5/L7 chord member force			-1588 kips
U6 chord splice force			1560 kips
Net area			83 in^2
Net moment of inertia			27276 in^4
Total stress			35.1 ksi

Node L7	Material – T-1 Allowable stress – 45 ksi		
Gusset plates	68 inches high	0.625 inch thick	
Internal splice plates	27.5 inches high	0.5 inch thick	
Top splice plate	0.375 inch thick	20 inches wide	
Bottom splice plate	0.375 inch thick	20 inches wide	
Distance from center of chord to center of mass of connecting plates			13.33 inches
U6/U8 chord member force			2312 kips
L7 chord splice force			-2350 kips
Gross area			128 in^2
Gross moment of inertia			47457 in^4
Total stress			-37.1 ksi

Node U8	Material - T-1 Allowable stress – 45 ksi		
Gusset plates	82 inches high	0.75 inch thick	23 rivets
Internal splice plates	27.5 inches high	0.75 inch thick	8 rivets
External splice plates	28 inches high	0.75 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-15.01 inches
L7/L8 chord member force			-3270 kips
U8 chord splice force			3200 kips
Net area			151 in^2
Net moment of inertia			79855 in^4
Total stress			39.1 ksi

Node L9 (similar to node L7)	Material - T-1 Allowable stress – 45 ksi		
Gusset plates	68 inches high	0.625 inch thick	
Internal splice plates	27.5 inches high	0.5 inch thick	
Top splice plate	0.375 inch thick	20 inches wide	
Bottom splice plate	0.375 inch thick	20 inches wide	
Distance from center of chord to center of mass of connecting plates			13.33 inches
U8/U10 chord member force			2044 kips
L9 chord splice force			-2082 kips
Gross area			128 in^2
Gross moment of inertia			47457 in^4
Total stress			-32.8 ksi

Node U10 (from first try at U6)	Material – A441 Allowable stress – 27 ksi		
Gusset plates	62 inches high	0.5 inch thick	17 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-10.09 inches
L9/L11 chord member force			-850 kips
U10 chord splice force			850 kips
Net area			72 in^2
Net moment of inertia			22113 in^4
Total stress			21.5 ksi

Node L11 (from first try at U6)	Material – A441 Allowable stress – 27 ksi		
Gusset plates	62 inches high	0.5 inch thick	17 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			10.09 inches
U10/U12 chord member force			-900 kips
L11 chord splice force			965 kips
Net area			72 in^2
Net moment of inertia			22113 in^4
Total stress			24.3 ksi

Node U12 (from L5)	Material - T-1 Allowable stress – 45 ksi		
Gusset plates	60 inches high	0.5 inch thick	
Internal splice plates	27.5 inches high	0.5 inch thick	
Top splice plate	0.375 inch thick	20 inches wide	
Bottom splice plate	0.375 inch thick	20 inches wide	
Distance from center of chord to center of mass of connecting plates			-9.37 inches
L11/L13 chord member force			1976 kips
U12 chord splice force			-1976 kips
Gross area			103 in^2
Gross moment of inertia			28058 in^4
Total stress			-35.3 ksi

Node L13	Material – T-1 Allowable stress – 45 ksi		
Gusset plates	54.5 inches high	0.75 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.75 inch thick	8 rivets
External splice plates	28 inches high	0.75 inch thick	8 rivets
Top splice plate	0.375 inch thick	20 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	20 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			6.02 inches
U12/U14 chord member force			-2756 kips
L13 chord splice force			2756 kips
Net area			123 in^2
Net moment of inertia			25213 in^4
Total stress			36.1 ksi

Node U14	Material - T-1 Allowable stress – 45 ksi		
Gusset plates	51.5 inches high	0.75 inch thick	
Internal splice plates	27.5 inches high	0.625 inch thick	
Top splice plate	0.375 inch thick	20 inches wide	
Bottom splice plate	0.375 inch thick	20 inches wide	
Distance from center of chord to center of mass of connecting plates			-7.17 inches
L13/L13′ chord member force			2958 kips
U14 chord splice force			-2958 kips
Gross area			127 in^2
Gross moment of inertia			24783 in^4
Total stress			-42.2 ksi

Appendix B

Summary of results from the chord-splice calculations for the final design of the bridge.

This appendix summarizes the chord splice calculations for the final design of the bridge.

Node U2 and U2′	Material - MHD 3306 Allowable stress – 20 ksi		
Gusset plates	55.125 inches high	0.625 inch thick	16 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-8.45 inches
L1/L3 chord member force			608 kips
U2 chord splice force			-608 kips
Gross area			111 in^2
Gross moment of inertia			25827 in^4
Total stress			-10.1 ksi
L1/L3 chord member force (force in this member can reverse depending on the live load)			-456 kips
U2 chord splice force			456 kips
Net area			75 in^2
Net moment of inertia			18868 in^4
Total stress			10.8 ksi
L1′/L3′ chord member force			308 kips
U2′ chord splice force			-308 kips
Gross area			111 in^2
Gross moment of inertia			25827 in^4
Total stress			-5.1 ksi
L1′/L3′ chord member force (force in this member can reverse depending on the live load)			-722 kips
U2′ chord splice force			722 kips
Net area			75 in^2
Net moment of inertia			18868 in^4
Total stress			17.2 ksi

Node L3 and L3′	Material - MHD 3306 Allowable stress – 20 ksi		
Gusset plates	53.59375 inches high	0.5 inch thick	14 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			6.66 inches
U2/U4 chord member force			408 kips
L3 chord splice force			-409 kips
Gross area			98 in^2
Gross moment of inertia			21188 in^4
Total stress			-7.0 ksi
U2/U4 chord member force (force in this member can reverse depending on the live load)			-942 kips
L3 chord splice force			944 kips
Net area			68 in^2
Net moment of inertia			15967 in^4
Total stress			22.4 ksi
U2′/U4′ chord member force			732 kips
L3′ chord splice force			-734 kips
Gross area			98 in^2
Gross moment of inertia			21188 in^4
Total stress			-12.5 ksi
U2′/U4′ chord member force (force in this member can reverse depending on the live load)			-855 kips
L3′ chord splice force			857 kips
Net area			68 in^2
Net moment of inertia			15967 in^4
Total stress			20.3 ksi

Node U4 and U4′	Material - MHD 3306 Allowable stress – 20 ksi		
Gusset plates	55.125 inches high	0.5 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-7.72 inches
L3/L5 chord member force			906 kips
U4 chord splice force			-902 kips
Gross area			97 in^2
Gross moment of inertia			21975 in^4
Total stress			-16.5 ksi
L3/L5 chord member force (force in this member can reverse depending on the live load)			-828 kips
U4 chord splice force			824 kips
Net area			67 in^2
Net moment of inertia			16157 in^4
Total stress			21.2 ksi
L3′/L5′ chord member force			785 kips
U4′ chord splice force			-781 kips
Gross area			97 in^2
Gross moment of inertia			21975 in^4
Total stress			-14.3 ksi
L3′/L5′ chord member force (force in this member can reverse depending on the live load)			-955 kips
U4′ chord splice force			951 kips
Net area			67 in^2
Net moment of inertia			16157 in^4
Total stress			24.5 ksi

Node L5	Material - MHD 3310 Allowable stress – 27 ksi		
Gusset plates	59.71875 inches high	0.5 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			9.67 inches
U4/U6 chord member force (reversing force is -56 kips compression, and neglected)			1119 kips
L5 chord splice force			-1128 kips
Gross area			119 in^2
Gross moment of inertia			33464 in^4
Total stress			-17.5 ksi

Node U6	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	62.5 inches high	1 inch thick	17 rivets
Internal splice plates	27.5 inches high	1 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-11.10 inches
L5/L7 chord member force			-1696 kips
U6 chord splice force			1674 kips
Net area			134 in^2
Net moment of inertia			41951 in^4
Total stress			24.0 ksi

Node L7	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	68.125 inches high	1 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.625 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			14.41 inches
U6/U8 chord member force			2436 kips
L7 chord splice force			-2477 kips
Gross area			187 in^2
Gross moment of inertia			72039 in^4
Total stress			-27.8 ksi

Node U8	Material - MHD 3318 Allowable stress – 45 ksi		
Gusset plates	84 inches high	0.625 inch thick	23 rivets
Internal splice plates	27.5 inches high	0.75 inch thick	8 rivets
External splice plates	28 inches high	0.75 inch thick	8 rivets
External splice plates	59.5 inches high	0.75 inch thick	16 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-14.90 inches
L7/L8 chord member force L8/L9 chord member force			-3407 kips -3420 kips
U8 chord splice force			3335 kips
Net area			201 in^2
Net moment of inertia			93424 in^4
Total stress			32.4 ksi

Node L9	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	75.125 inches high	1 inch thick	16 rivets
Internal splice plates	27.5 inches high	0.625 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			17.42 inches
U8/U10 chord member force			2147 kips
L9 chord splice force			-2185 kips
Gross area			201 in^2
Gross moment of inertia			95890 in^4
Total stress			-23.7 ksi

Node U10	Material - MHD 3310 Allowable stress – 27 ksi		
Gusset plates	72.25 inches high	0.5 inch thick	16 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-14.02 inches
L9/L11 chord member force			-919 kips
U10 chord splice force			909 kips
Net area			83 in^2
Net moment of inertia			36597 in^4
Total stress			21.0 ksi

Node L11	Material - MHD 3310 Allowable stress – 27 ksi		
Gusset plates	65.59375 inches high	0.5 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			10.91 inches
U10/U12 chord member force			-924 kips
L11 chord splice force			929 kips
Net area			79 in^2
Net moment of inertia			28786 in^4
Total stress			20.9 ksi

Node U12	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	62.125 inches high	1 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-12.77 inches
L11/L13 chord member force			2011 kips
U12 chord splice force			-2008 kips
Gross area			166 in^2
Gross moment of inertia			52269 in^4
Total stress			-25.7 ksi

Node L13	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	54.34375 inches high	1 inch thick	15 rivets
Internal splice plates	27.5 inches high	0.625 inch thick	8 rivets
External splice plates	28 inches high	1.25 inches wide	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.5 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			6.08 inches
U12/U14 chord member force			-2790 kips
L13 chord splice force			2791 kips
Net area			157 in^2
Net moment of inertia			32656 in^4
Total stress			28.7 ksi

Node U14	Material - MHD 3309 Allowable stress – 27 ksi		
Gusset plates	52 inches high	1 inch thick	14 rivets
Internal splice plates	27.5 inches high	0.5 inch thick	8 rivets
External splice plates	28 inches high	1 inch thick	8 rivets
Top splice plate	0.375 inch thick	19 inches wide	5 rivets
Bottom splice plate	0.375 inch thick	19 inches wide	5 rivets
Distance from center of chord to center of mass of connecting plates			-6.19 inches
L13/L13′ chord member force			2975 kips
U14 chord splice force			-2975 kips
Gross area			202 in^2
Gross moment of inertia			38085 in^4
Total stress			-24.9 ksi

Appendix C

Computation sheets showing chord-splice calculations for the preliminary design of the bridge.

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 1 OF

COMPUTATIONS FOR Truss Details DATE Nov, 63

BY C.J.R. CHKD

Rivet Pitch - AASHO, Art 1.6.22 & Art 1.6.23

Thickness t, in.	Stitching Compr. 12t	Stitching Tension 24t	Sealing 4"+4t
5/16			
3/8			
7/16			
1/2			
9/16			
5/8			
11/16			
3/4			
13/16			
7/8			
15/16			
1			
1 1/16			
1 1/8			
1 3/16			
1 1/4			
1 5/16			
1 3/8			
1 7/16			
1 1/2			

Member U1-L1

Stress = 587^k C

Capacity = 640^k C

1227 ÷ 2 = 614^k avg.

$^3/_4$ × 640 = 480

614 ÷ 10.6 = 58$^{(-)}$ 1" reg rivets req'd @ L1

480 ÷ 10.6 = 45 1" " " " " U1

614 ÷ 15.7 = 39$^{(+)}$ 1" H.S. " " " L1

480 ÷ 15.7 = 31$^{(-)}$ 1" H.S. " " " U1

Upper Laterals

Panel U0-U1:

Stress = 256^k C @ 125% N.U.S.

Capacity = 275^k C

531 ÷ 2 = 265^k avg.

265 ÷ 8.12 = 33$^{(-)}$ - 7/8 rivets req'd

Panels U1-U2 thru U13-U14:

Max Stress = 116^k T or C (Panel U8-U9)

Capacity = 176^k C, 310^k T

$\frac{176 + 116}{2}$ = 146^k avg

$^3/_4$ × 176 = 132^k

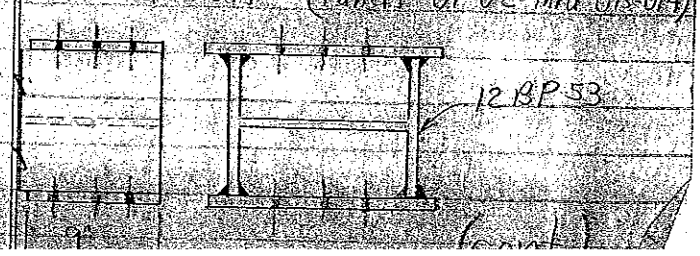
132 ÷ 8.12 = 16$^{(+)}$

18 rivets × 8.12 = 146$^{(+)k}$ > 146^k

∴ Use 18 - 7/8" rivets

18 Furn. - Pattern 3.33

Weld Req'd for Conn. Pls. @ ends of Laterals (Panels U1-U2 thru U13-U14)



SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340 — SHEET NO. 2 OF

COMPUTATIONS FOR Truss Details — DATE Nov 11, '63

BY C.J.R. CHKD

(Cont. from pg. 1)

9 rivets × 8.12 = 73^K

Size Weld Req'd:

Size × 0.707 × 9" × 4 welds × 12.4^{ksi} = 73^K

Size = 73/0.707 × 9 × 4 × 12.4

= 0.23

∴ Use 1/4" Weld

Joint U1 Gusset Pl. size

Pl. width = 21" @ Top row rivets in U1-L1

Try 1/2" A-441 Pl's

$f = P/A = \frac{587^K}{2} \div 21 \times 1/2 = 27.9^{ksi} > 27$

Try 9/16", $f = 24^{ksi} < 27^{ksi}$

∴ Use 9/16" - A441 Gusset Pl.

Joint U1 Lat. Pl size

75% × 176 = 132^K T

Net Width = say 22" - 3 rivets = 19"

Area = 3/8 × 19 = 7.13 □"

f = 132/2 ÷ 7.13 = 9.3^{ksi}

∴ Use 3/8" - A36 Lateral Pl's.

Member U0-U2 (A-36)

Capacity = ~~683^K T~~ 734^{K}T

Stress = 683^K T

708 ÷ 10.00 = 71$^{(-)}$-1" H.S. rivets req'd

Pattern 4444444466888 = 136

Bearing on 1/2 Pl = 20,000;

20,000/2 = 10.00

Pat

Member U2-U4 (A441)

Capacity = 1020^K C, 1220^K T

311 + 513 + 67 = 891^K C } Reversal

437 + 39 + 280 = 196^K T }

1087^K

1087 ÷ 10.00 = 109$^{(-)}$-1" H.S. rivets req'd

Pattern 4444444466888 = 136

Member L1-U2 (A441)

Capacity = 1218^K C

Stress = 1205^K C

1218 ÷ 15.7 = 78$^{(-)}$-1" H.S. rivets req'd

Pattern 5777753 = 82

Member U2-L2 (A441)

Capacity = 546^K T

Stress = 533^K T

546 ÷ 15.7 = 35$^{(-)}$-1" H.S. rivets req'd

Pattern 2244444 = 48 @ U2

75% × 546 ÷ 15.7 = 26 req'd @ L2 Use 22424 = 28

Member U2-L3 (A441)

Capacity = 655^K T

215 + 325 + 43 = 583^K T

217 + 24 - 193 = 48^K C

631^K

631 ÷ 15.7 = 40$^{(+)}$-1" H.S. rivets req

Pattern 2244442 = 44

631 ÷ 10.6 = 60$^{(-)}$-1" reg. rivets req

SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 4 OF

DATE Nov. '63

BY C.J.R. CHKD

Member U2-L2 (Check $s^2/4g$)

$s^2/4g = \frac{(3.75)^2}{4 \times 3.50} = \frac{14.05}{14.00} =$ say 1.00

$15 - 2 \times 9/8 = 15 - 2.25 = 12.75''$

$15 - (4 \times 9/8) + (2 \times 1.00) =$

$15 - 4.50 + 2.00 = 12.50$

Try $s^2/4g = \frac{(4.00)^2}{4 \times 3.50} = \frac{16}{14.00} = 1.14$

$15 - 4.50 + 2(1.14) = 12.78$

$> 12.75 \therefore$ OK

Use 4" pitch between Row 1 & 2

Member L1-U2 (Develop Cov. Pl.)

Area $= (20.00 - 10.00)\,1/2 = 5.00\ \square''$

Cap. $= 5.00 \times 19.02 = 95.1^K$ C

Cov. Pl Length = say 12", weld = 5/16"

Cap. $= 12'' \times 4$ welds $\times 0.707 \times 5/16 \times 14.7$ psi $=$

$156^K > 95.1^K \therefore$ OK

Member U2-L3 (Develop Web Pl)

Area $= 7.45 \div 2$

Cap. $= (7.45 \div 2)\,27 = 100^K$

Cap. $= 25'' \times 2$ welds $\times 0.707 \times 1/4 \times 14.7$ psi $=$

$130^K > 100^K \therefore$ OK

Member U2-L2 (Develop Web Pl)

Cap. $= (7.50 \div 2)\,27 = 101^K$

Cap. $= 26.5'' \times 2 \times 0.707 \times 1/4 \times 14.7$ psi $=$

$138^K > 101^K \therefore$ OK

Joint U2 - (Develop Splice Mat'l)

Assume 1/2" Splice Pl. (A441)

Area $= (27.50'' \times 1/2'') - (8 \times 9/8 \times 1/2)$

$= 13.75 - 4.50 = 9.25\ \square''$

$(9.25 \times 27) \div 10.00 = 25$ 1" H.S. rivets req'd

~~Develop 1/16" Fill Pl (A441)~~

~~Area $= (27.50 \times 1/16) - (8 \times 9/8 \times 1/16)$~~

~~$= 1.72 - 0.56 = 1.16\ \square''$~~

~~$(1.16 \times 27) \div 11.25 = 3$ rivets Req'd~~

Member U1-L1 (Develop Cov. Pl.)

Area $= (20.00 - 10.00)(1/2) = 5.00\ \square''$

Cap. $= 5.00 \times 19.71 = 99^K$

For 1/4" weld:

Length $\times$ 4 welds $\times 0.707 \times 1/4 \times 14.7 = 99^K$

$L = 99 \div 4 \times 0.707 \times 1/4 \times 14.7 = 9\,1/2''$ req'd

Joint L2 - (Gusset Pl. Size)

Try 9/16 A-441 Pls

$f = P/A = \frac{546}{2} \div (19\,1/2 - 4 \times 9/8)\,9/16$

$= 273 \div 8.43 = 32.4^{ksi} > 27^{ksi}$

Try 11/16" A-441 Pls

Area $= 15\,(11/16) = 10.3\ \square''$

$273 \div 10.3 = 26.5^K < 27^K \therefore$ OK

$\therefore$ Use 3/4" A441 Gusset Pl

SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 5 OF

DATE Nov 63

BY C.J.R. CHKD

Joint U2 - (Transferring Stress to Gusset)

For Member U0-U2

$4 \times 11.25 = 45^K$

2 add holes: $2 \times 9/8 \times 9/16 \times 20 = 25^K$

$8 \times 11.25 = 90^K$

4 add holes: $4 \times 9/8 \times 9/16 = 50^K$

For Member U2-U4

$4 \times 11.25 = 45^K$

2 add holes: $2 \times 9/8 \times 9/16 \times 27 = 34^K$

$8 \times 11.25 = 90$

4 add holes: $4 \times 9/8 \times 9/16 = 68^K$

Joint U2 - (Cover PL Splice)

Cover PL Area $= 7.69/2 = 3.85 \square''$

Try Splice PL $= 15\frac{5}{8}'' \times \frac{3}{8}'' = 5.85 \square''$

$- 4 \times 9/8 \times 3/8 = 4.16 \square''$

$4.16 \times 27 \div 10.6 = 11^{(-)}$ 1"-Regular Rivets

Lower Lateral Connections

Use Box Section - Sht 24 (A36)

Cap. $= 136^K$ C , 240^K T

Max Stress $= 48^K$ T or C

$(136 + 48) \div 2 = 92^K$

$75\% \times 136^K = 102^K$

$102 \div 8.12 = 13^{(-)}$ Req'd (7/8" Regular)

Pattern = 233 = 16 rivets

Joint

Bottom Strut Conn's.

Same section as bottom Laterals

Cap. $= 178^K$ C , 240^K T

Max Stress $= 65.4 \div 2 = 32.7^K$ T or C

$75\% \times 178^K = 134^K$

$134 \div 8.12 = 16^{(+)}$ Req'd (7/8" Regular)

Pattern 2323 - 20

Conn. U2-L2 @ L2

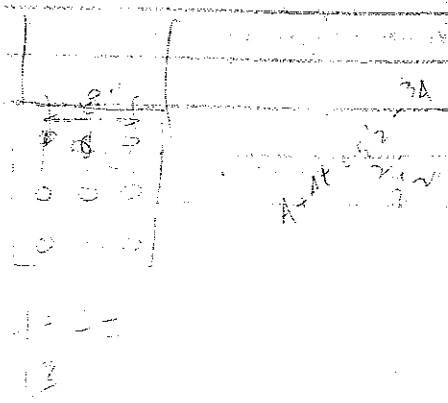
Capacity $= 555^K$ T

Max Stress @ L2 < say 10^K

$\therefore 75\% \times 555^K = 416^K$

$416 \div 10.6 = 39^{(+)}$ 1" rivets req'd

Member U2-L2 has both H.S. and regular rivets in it, but say ok since the regular rivets have very little stress.



SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 6 OF

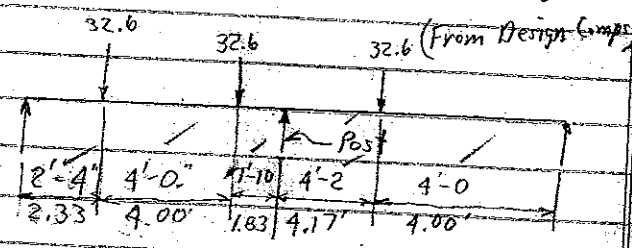
DATE Nov 21, '63

BY C.J.R. CHKD

Determine Max. Load on a Post member

Max D.L. = 32.6 Kips (From Design Comps)

Determine L.L. :



Max Ld @ post

$32.6\left(\frac{6.33}{8.16}+\frac{2.33}{8.16}+\frac{4.00}{8.16}\right) = 50.5^K$

LL + Imp = 50.5 × 1.3 = say 66^K

DL + LL + Imp = 66 + 32.6 = say 100^K

~~100 ÷ 10.6 = 9(+) 1" Reg rivets req'd~~

Member U0-L1 A-441

Stress = 871^K C

Capacity = 936^K C

18.07 = say 900^K Avg.

Furn 100 @ U0 ← 900 ÷ 10.6 = 85-1" Reg Rivets Req'd

Furn 104 @ L1 900 ÷ 15.7 = 57(+)-1" H.S. Rivets "

Develop Cover Pl

A = (20-10) ½ = 5.00 □"

Cap. = 5.00 × 18.68 = 93.3^K

93.3 = l × 4 welds × 0.707 × 5/16 × 14.7 psi

$l_{req'd}$ = 93.3 ÷ 4 × 0.707 × 5/16 × 14.7

= 7.2" say 7½"

Member L1-L3

Capacity = 702^K C, 797^K T

148 + 333 + 43 = 524^K T
345 + 31 − 133 = 243^K C } Reversal

767^K

767^K ÷ 15.7 = 49 1" H.S. rivets req'd @ L1

Pattern 4 4 6 4 6 8 = 76 furnished

Joint L1

Spl. Mat'l Furnished:

2 gussets : say 28" × 5/8 = 35 □"

2 Spl. Pls : " 28 × 5/8 = 35 □"

70 □"

− 16 holes × 9/8 × 10/8 = 22.50

Net A = 47.50 □" > 39.82 for L1-L3 ∴ say OK

Develop Spl. Pl's

5/8 × 27½ − 9/8 × 8 × 5/8 = 17.2 − 5.6 = 11.6

(11.6 × 27) ÷ 10.6 = 29(+) 1" reg rivets req'd

Develop ¼" Fill Pl

¼ × 27½ − 9/8 × 8 × ¼ = 6.88 − 2.25 = 4.63 □"

(4.63 × 27) ÷ 10.6 = 12(−) req'd.

Joint L3 U2 - (Cover Pl Splice)

Top Cov. Pl Area = 3.75 □"

Try Splice Pl 3/8" × 20" − 5 × 9/8 × 3/8

= 7.50 □" − 2.11 = 5.39 □" ∴ OK

5.39 × 20 ÷ 10.6 = 10(+) 1" reg riv.

Bottom

Cov Pl Area = 3.75 □"

Try Splice Pl 3/8 × 15½ − 4 × 9/8 × 3/8

= 5.82 − 1.69 = 4.13 □" > 3.75 ∴ OK

4.13 × 20 ÷ 10.6 = 8(+) 1" reg rivets req'd

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 9 OF

DATE Dec 4, 1963

BY CJR CHKD

Member L3-U4 (A441)

Stress = 594^K T

Capacity = 618^K T, 482^K C

1212 ÷ 2 = 606^K T avg.

606 ÷ 15.7 = 39$^{(-)}$ 1" HS rivets

606 ÷ 10.6 = 57$^{(+)}$ 1" reg. "

Pattern 43455531 = 60

Member L3-L5 (A441)

Capacity = 1180^K T, 986^K C

78 + 572 + 74 = 724^K T
490 + 44 - 70 = 464^K C } Reversal

1188^K

1188 ÷ 10.00 = 119$^{(-)}$ HS. or Reg rivets

Develop Horiz. Diaphragm

Area = 11.25$^{\square''}$

Cap = 11.25 × 20.43 = 230^K

Rivets Req'd = (11.25 × 27) ÷ 10.0 = 30$^{(+)}$

OK

Weld length req'd:

= (11.25 × 20.43) ÷ 4 × 0.707 × 14.7 × 4 welds

= 230 ÷ 10.4 = 22.1" req'd; 29.5" Furn.

say OK

SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340 SHEET NO. 10 OF

COMPUTATIONS FOR Truss Details DATE Dec 10, 1963

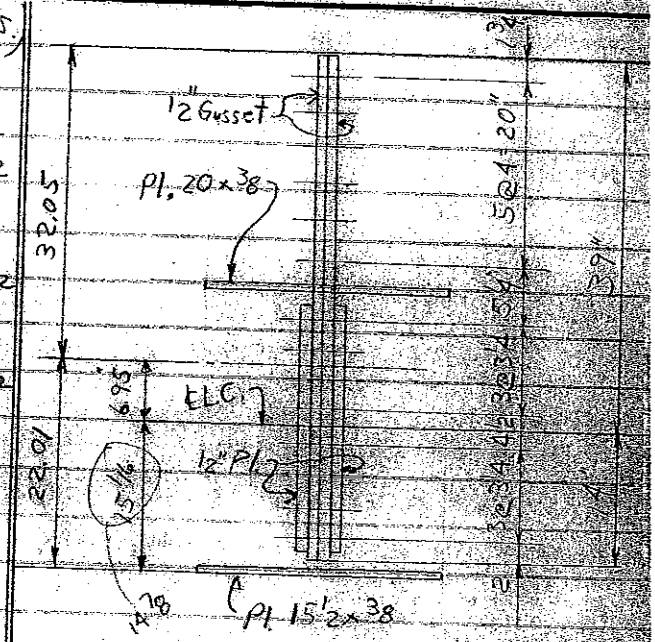
BY C.J.R. CHKD

Splice @ L3 Mtl. A441 (A.S.)

Say stress across Jt L3 =
Stress in U2-U4 = say 1090 ᵉ see sheet 2
↑ change

Holes out $= (\frac{9}{8} \times 2)(\overline{18.95}^2 + \overline{15.70}^2 + \overline{12.45}^2 + \overline{9.20}^2 + \overline{4.70}^2 + \overline{1.45}^2 + \overline{1.80}^2 + \overline{5.05}^2) + (\frac{9}{8} \times 1)(\overline{10.30}^2 + \overline{14.30}^2 + \overline{18.30}^2 + \overline{22.30}^2 + \overline{26.30}^2 + \overline{30.30}^2)$

$= 2030 + 3100 = 5130$



	Agr	y	Ay	A(net)
2 Gusset Pls 53×1/2	53.00 □"	12.5	663	37.25
2 Spl. Pls 27 1/2×1/2	27.50	0	0	18.50
1 Top Spl. Pl 20×3/8	7.50	14.88	111.5	5.39
1 Bott Spl. Pl 20*×3/8	7.50	−14.88	−111.5	5.39
	95.50 □"	12.50	663	66.53 □"

$28 \times \frac{9}{8} \times \frac{1}{2} = 15.75$ □"
$16 \times \frac{9}{8} \times \frac{1}{2} = 9.00$ □"
$5 \times \frac{9}{8} \times \frac{3}{8} = 2.11$
$5 \times \frac{9}{8} \times \frac{3}{8} = 2.11$

* Assumed Eff Pl width (Pl. is actually abt 15'2" wide)

I:	A	d	Ad^2	I_o
Gussets	53.00	5.55	1632	12,400
Spl. Pls	27.50	6.95	1330	1738
do (net)	5.39	7.93	338	~
do (net)	5.39	21.83	2580	~
			5880	14,138
				5880
				20,018
			Holes out	−5130
			I_{net} =	14,888

S_m (top) = 14888 ÷ 32.05 = 464
S_m (bott) = 14888 ÷ 22.01 = 675

Stresses

1090ᵀ across joint (see above)

At bottom

$f = \frac{1090}{66.53} + \frac{1090 \times 6.95}{675}$

$= 16.4 + 11.2 = 27.6^{ksi\,T} > 27$

∴ Use 5/8" Gussets

Conn. Spl. Mat'l		Req'd	Furn
Pl 27.50×1/2	= 9.25×27÷10 =	25	24
Pl 20×3/8	= 5.62×27÷8.1 =	19 (−)	20
Pl 15 1/8×3/8	= 4.17×27÷8.1 =	14 (−)	16

say 24 is OK since 10 ksi is conservat

SVERDRUP & PARCEL

JOB Minnesota Bridge #9340

COMPUTATIONS FOR Truss Details

SHEET NO. 11 OF

DATE Dec 11, 1963

BY C.J.R. CHKD

Member U4-U6 (A441)

Capacity = 1063^{KT}, $934^{K}C$

$352^{T} + 526^{T} + 47^{T} = 925^{KT}$ } Reversal

$443 + 58 - 317 = 184^{K}C$

1109^{K}

$1109 \div 10.00 =$ 111 H.S. or Reg rivets @ U4 req'd

$1109 \div 15.7 =$ $71^{(-)}$ H.S. rivets req'd @ U6

Member U4-L5 (A441)

Capacity = 1008^{K} C

Stress = 991^{K} C

$1008 \div 15.7 =$ $64^{(+)}$ 1" H.S. rivets req'd

$1008 \div 10.6 =$ 95 1" reg " "

Member L5-L7 (A441)

Capacity = 1588^{K} C

Stress = 1578^{K} C

$1588 \div 15.7 =$ 101 1" H.S. rivets req'd

Member L5-U6 (T-1)

Capacity = 1372^{K} T

Stress = 1259^{K} T

$2631 \div 2 = 1316^{KT}$ (avg)

$1316 \div 15.7 =$ $84^{(-)}$ 1" H.S rivets req'd.

Member U5-L5 (A441)

Capacity = 609^{K} C

Stress = 586^{K} C

$1195 \div 2 = 598^{K}$ C avg.

$75\frac{1}{2} \times 609 = 456^{K}$ C

$598 \div 15.7 =$ $38^{(+)}$ 1" H.S. rivets req'd @ Jt. L5

$456 \div 15.7 =$ $29^{(+)}$ " " Jt. U5

Joint L5

Develop 1/2" I.S. Spl. PL

$\frac{27\frac{1}{2} \times \frac{1}{2} \times 27^{ksi}}{15.7} = 24^{(-)}$ 1" H.S. rivets re

Develop 1/2" × 27 1/2" Fill PL

Same as Spl. PL = $24^{(-)}$ 1" H.S. riv

Member L5-U6 (Develop Web PL

Weld length req'd =

$\frac{8.64^{\square''} \times 45^{ksi}}{\frac{1}{4} \times 0.707 \times 14.7 \times 4 \text{ welds}} = 37.4''$ (too lon

Use 3/8" weld = $\frac{37.4 \times \frac{1}{4}}{\frac{3}{8}} = 24.9''$ (OK)

Member U6-L7 (A441)

Capacity = 1650^{K} C

Stress = 1618^{K} C

$1650 \div 15.7 =$ 105 1" H.S. rivets req'd

Pattern @ U6

Member U6-U8 (T-1)

Capacity = 2300^{K} T

Stress = 2312^{K} T

$2300 \div 15.7 =$ 153 1" H.S. Rivets req'd

Pattern

$4 \times 15.7 = 62.8^{K}$

$2 \times \frac{9}{8} \times \frac{7}{8} \times 45 = 88.6$ too high

$8 \times 15.7 = 125.6^{K}$

∴ Pattern shall begin : 4468

SVERDRUP & PARCEL

JOB Minnesota Bridge # 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 12 OF

DATE 12-13-63

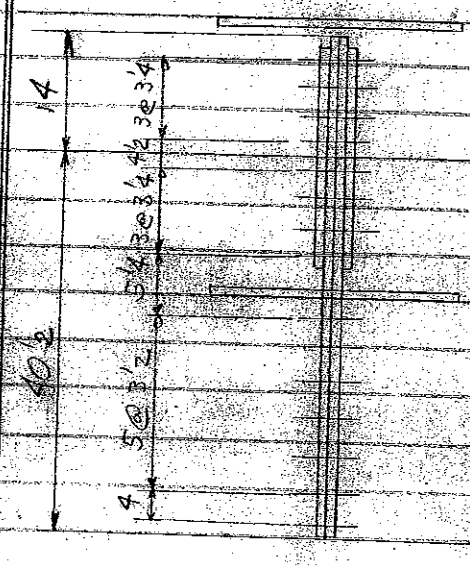
BY C.J.R. CHKD

Splice @ U4 & U24 (MH. A36)

Stress across Joint = Stress in L3-L5 or L23-L25

Design Joint For 693^k C, 630^k T

Assume "c.g." and "I" of this joint to be same as @ Jt U2



With no holes out: (Comp)

S.m. (top) = 82,023 ÷ 22.5 = 980

Stress @ top:

= f

With holes out (Tension)

S.m. (top) = 16560 ÷ 22.5 = 735

Stress @ top

$$f = \frac{464}{67.39} + \frac{464 \times 7.62}{735}$$

$$= 6.89 + 4.81 = 11.70^{ksi}\ T$$

Figure Splice Pl's with holes out beause sag stress on splice Pl's = 13^{ksi} (conservative - see comps. above)

13^{ksi} × 13.75$^{in^2}$ = 179^{kips} to develop

179 ÷ 10^{kips}/rivet = 18 req'd < 24 furnished

∴ sag OK

For lateral Pl 20 × 3/8 = 7.50$^{in^2}$ × 13^{ksi} = 98^k

98 ÷ 8.1 = 12 req'd < 20 furnished

∴ sag OK

VOID

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 13 OF

COMPUTATIONS FOR Truss Details DATE Dec. 16, 1963

BY C.J.R. CHKD

Splice @ L5 Mtl - A441 (AS)

Say stress across jt = stress in member U4-U6.

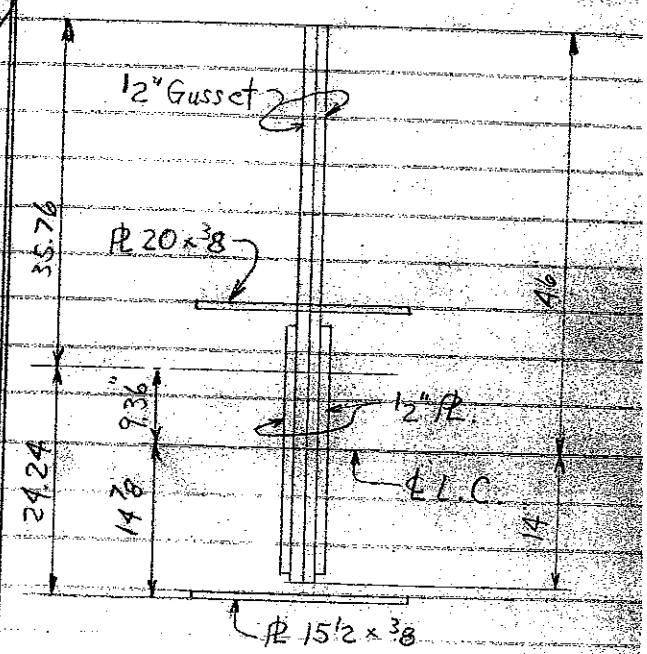
Member U4-U6 : 925^k T, 184^k C

45.92
36.12
9.80

76 — 9.80 — 76.6

(76.6 ÷ 76) × 925 = 935^k C across Jt L5

Void



	Agr	y	Ay
2 Gusset PLs 60 × 1/2	60.00	16.00	960
2 Spl. PLs 27 1/2 × 1/2	27.50	0	0
1 Top Spl PL 20 × 3/8	7.50	14.69	110
1 Bott Spl PL 20* × 3/8	7.50	-14.69	-110
	102.50	16.00	960

* Assumed effective PL width (PL is actually 15 1/2" wide)

I:	A	d	Ad²	I_o
Gussets	60.00	6.64	2640	18,000
Spl. PLs	27.50	9.36	2420	1738
do	5.39	5.33	153	~
do	5.39	24.05	3116	~
			8329	19,738
				8329
			I_{gr}	28,067

Sm (top) : 28,067 ÷ 35.76 = 786

Sm (bott) : 28,067 ÷ 24.24 = 1160

Stresses

935^k C across Joint

At bottom

$f = \frac{935}{102.50} + \frac{935 \times 9.36}{1160}$

= 9.11 + 7.53 = 16.62^{ksi} < 27^{ksi}

Use 3/8" I.S. Spl PL's Stress = 18.2^{ksi} (scratch paper)

Conn. Spl. Mat'l.	Req'd	Fun
PL 27.50 × 3/8 = 10.3 × 27 ÷ 15.7 =	18(-)	24
PL 20 × 3/8 = 7.50 × 27 ÷ 15.7 =	13(-)	15
PL 15 1/2 × 3/8 = 5.81 × 27 ÷ 15.7 =	10	12

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 14 OF

COMPUTATIONS FOR Truss Details DATE Dec 18, 1963

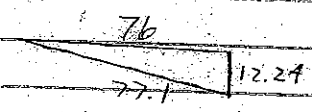
BY C.J.R. CHKD.

Splice @ U6 Mtl. (Q.T.)

Say stress across ⊥ = horiz component of stress in L5-L7

Member L5-L7 = 1588^K C

52.041
39.796
12.245



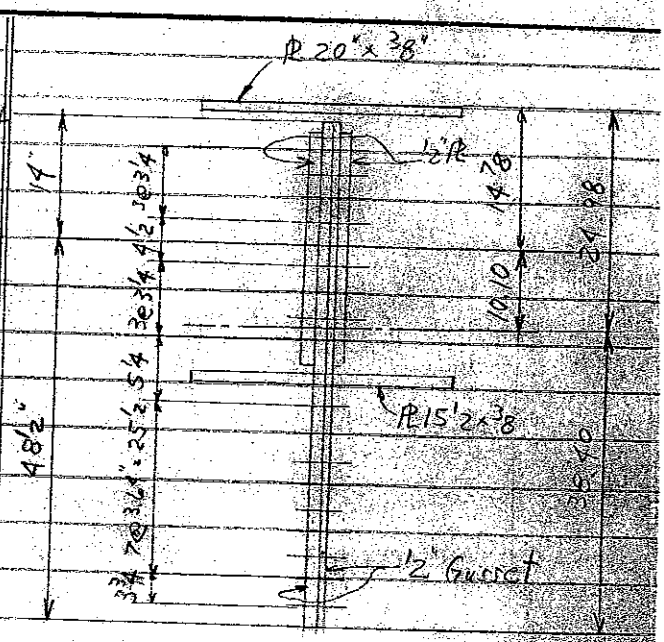
76.0

77.1 × 1588 = 1560^K T across ⊥ U6

Void

Holes out = $(\frac{9}{8}\times 2)(\overline{22.10}^2 + \overline{18.85}^2 + \overline{15.60}^2 + \overline{12.35}^2 + \overline{7.85}^2 + \overline{4.60}^2 + \overline{1.35}^2 + \overline{1.90}^2) + (\frac{9}{8}\times 1)(\overline{7.15}^2 + \overline{10.79}^2 + \overline{14.43}^2 + \overline{18.07}^2 + \overline{21.71}^2 + \overline{25.35}^2 + \overline{28.99}^2 + \overline{32.63}^2 + \overline{36.63}^2)$

= 2990 + 5700 = 8690



	Agr	y	Ay	A (net)	
2 Gusset PL's 62" × 1/2"	62.00	17.00	1054	42.90	34 × 9/8 × 1/2 = 19.10 on
2 Spl. PL's 27 1/2 × 1/2"	27.50	0	0	18.50	16 × 9/8 × 1/2 = 9.00
1 Top Spl. PL 20" × 3/8	7.50	-14.69	-110	5.39	5 × 9/8 × 3/8 = 2.11
1 Bott Spl PL 20" × 3/8	7.50	14.69	110	5.39	5 × 9/8 × 3/8 = 2.11
	104.50	17.00	1054	72.18	

*Assumed Eff. PL width (PL is 15 1/2" wide)

I	A	d	Ad²	I₀
Gussets	62.00	6.90	2950	19,850
Spl PL's	27.50	10.10	2810	1738
do (net)	5.39	24.79	3310	~
do (net)	5.39	4.59	113	~
			9183	21,588
				9183
				30,771
			Holes out	− 8690
				22,081

Sm (top) = 22,081 ÷ 24.98 = 884

Sm (bott) = 22,081 ÷ 38.40 = 575

Stresses:

At top

$f = \frac{1560}{72.18} + \frac{1560 \times 10.10}{884}$

= 21.6 + 17.8 = 39.4 < 45 ksi T too hi

∴ Use 5/8" Gusset PL's

Conn. Spl. Mat'l		Req'd	Furn
PL 27 1/2 × 1/2 =	9.25 × 45 ÷ 15.7	26(+)	32
PL 20 × 3/8 =	5.39 × 45 ÷ 15.7	15(+)	20
PL 15 1/2 × 3/8 =	4.13 × 45 ÷ 15.7	12(−)	16

Revise Drawing

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 15 OF

COMPUTATIONS FOR Truss Details DATE Dec 30, 1963

BY C.J.R. CHKD

Member U7-L7 (A441)
See U1-L1 (Sht. 1)

Member L7-U8 (T-1)
Stress = 1619^k T
Capacity = 1700^k T
3319 ÷ 2 = 1660^k T avg.

1660 ÷ 15.7 = 106 - 1" H.S. rivets req'd

Member L7-L8 (T-1)
Stress = 3266^k C
Capacity = 3300^k C

3300 ÷ 15.7 = 210 - 1" H.S. rivets req'd

Member U8-U10 (T-1)
Stress = 2044^k T
Capacity = 2036^k T

2036 ÷ 15.7 = 130$^{(-)}$ 1" H.S. rivets req'd.

Member U8-L9 (T-1)
Stress = 2028^k T
Capacity = 2129^k T
4157 ÷ 2 = 2079^k T avg.

2079 ÷ 15.7 = 133$^{(-)}$ 1" H.S. rivets req'd

Member U8-L8 (T-1)
Stress = 3252^k C
Capacity = 3380^k C
6632 ÷ 2 = 3316^k C avg.

3316 ÷ 15.7 = 211$^{(+)}$ 1" H.S. rivets req'd

Member L9-U10 (T-1)
Stress = 2228^k C
Capacity = 2255^k C
4483 ÷ 2 = 2242^k C avg.

2242 ÷ 15.7 = 143 - 1" H.S. rivets req'd

Member L9-L11 (A-441)
Stress = 850^k C
Capacity = 1020^k C
1870 ÷ 2 = 935^k C avg

935 ÷ 15.7 = 60$^{(+)}$ 1" H.S. Rivets req'd

Horiz. Web Pl
Weld Req'd - Try 3/8" weld

$\frac{20.54 \times 11.25}{3/8 \times 0.707 \times 14.7 \times 4}$ = 14.8" Req'd, 15" Furn. (say OK)

Joint L9

By inspection, use same splice mat'l as @ Jt. L7

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 16 OF

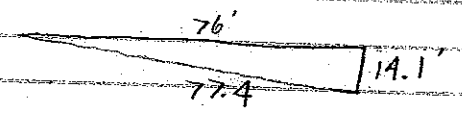
COMPUTATIONS FOR Truss Details DATE Dec 31, 1963

BY C.J.R. CHKD

Splice @ L7 Mtl. (Q.T.)

Say stress across Jt. = stress in member U6-U8

Member U6-U8 = 2312^K T



$\frac{77.4}{76} \times 2312$ = say 2350^K C across L7

	A_{gr}	y	Ay
2 Gusset Pl's 68" × 1/2"	68.00	20.00	1360
2 Spl. Pl's 27 1/2" × 1/2"	27.50	0	0
1 Top Spl. Pl 20 × 3/8	7.50	14.88	118
1 Bott Spl. Pl 20* × 3/8	7.50	−14.88	−118
	110.50	20.00	1360

*Assumed Effective Pl width (Pl is actually 15 1/2" wide)

$A_Y/A = 1360/110.5 = 12.30"$

I:	A	d	Ad^2	I_o
Gussets	68.00	7.70	4030	26,180
Spl. Pls	27.50	12.30	4160	1738
d_o	5.39	2.58	36	~
d_o	5.39	27.18	3970	~
			12,196	27,918
				12,196
				40,114

$S_{m\ top}$ = 40,114 ÷ 41.70 = 963

$S_{m\ bott}$ = 40,114 ÷ 27.36 = 1470

Stresses:

2350^K C across Joint

At bottom

$f = \frac{2350}{110.50} + \frac{2350 \times 12.30}{1470}$

= 21.2 + 19.7 = 40.9^{KSI} < 45 too high

∴ Use 5/8" Gussets

Stress = 37.9^{KSI} (scratch paper) say 6

Horizontal Web Pl in L5-L7

See Calculations for L3-L5. (Sht. 7)

Use 3/8" Weld

$\frac{20.37 \times 11.88}{3/8 \times 0.707 \times 14.7 \times 4 \text{ welds}}$ = 15.6" req'd

18" Furn. OK

For Member L7-L8:

$\frac{15.97 \times 30.90}{5/16 \times 0.707 \times 14.7 \times 4 \text{ welds}}$ = 38" req'd

35" Furn.

Use 3/8" weld — 32" req'd

35 Furn. OK

Conn. Splice Mat'l.

		Req'd	Fur.
Pl 27 1/2 × 1/2	= 13.75$^{\square''}$ × 45^{KSI} ÷ 15.7	= 40$^{\square''}$	40
Pl 20 × 3/8	= 7.50 × 45 ÷ 15.0	= 22$^{(4)}$	25
Pl 15 1/2 × 3/8	= 5.81 × 45 ÷ 15.0	= 17$^{\square''}$	20

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 17 OF

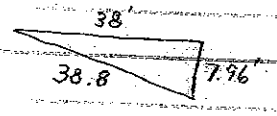
DATE Jan 3, 1964

BY C.J.R. CHKD.

Splice @ U8 (Mtl. - Q.T.)

Say stress across Jt. = horiz component of stress in L7-L8 or L8-L9

Member L7-L8 = Say 3270^K

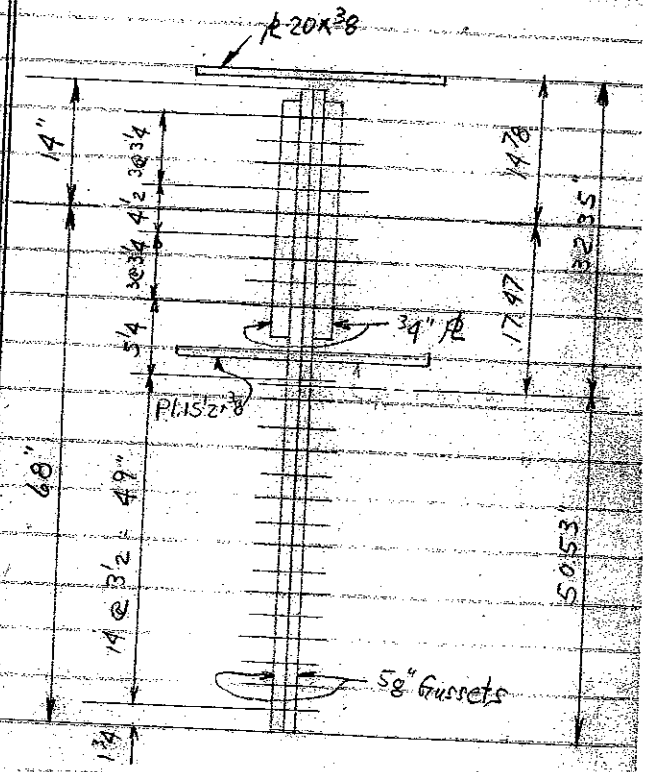


$\frac{38}{38.8} \times 3270 = 3200^K$ T across Jt U8

Holes out = $(\frac{9}{8} \times 2.75)(\overline{29.47}^2 + \overline{26.22}^2 + \overline{22.97}^2 + \overline{19.72}^2 + \overline{15.22}^2 + \overline{11.97}^2 + \overline{8.72}^2 + \overline{5.47}^2)$

$+(\frac{9}{8} \times 1.25)(\overline{0.22}^2 + \overline{3.28}^2 + \overline{6.78}^2 + \overline{10.28}^2 + \overline{13.78}^2 + \overline{17.28}^2 + \overline{20.78}^2 + \overline{24.28}^2 + \overline{27.78}^2 + \overline{31.28}^2 + \overline{34.78}^2 + \overline{38.28}^2 + \overline{41.78}^2 + \overline{45.28}^2 + \overline{48.78}^2)$

= 9130 + 18,500 = 27,630



	$A_{gr.}$	y	Ay	A(net)
2 gusset PL's 82×⅝	102.50	27.00	2770	70.20
2 Spl. PL's 27½×¾	41.25	0	0	27.75
1 Top Spl PL 20*×⅜	7.50	-14.69	-110	5.39
1 Bott Spl PL 20×⅜	7.50	14.69	110	5.39
	158.75	27.00	2770	108.73

$23 \times \frac{9}{8} \times \frac{10}{8} = 32.3^{\square''}$

$8 \times \frac{9}{8} \times \frac{3}{2} = 13.50$

$5 \times \frac{9}{8} \times \frac{3}{8} = 2.11$

$5 \times \frac{9}{8} \times \frac{3}{8} = 2.11$

* Assumed effective PL width (PL is all 15½" wide)

I:

	A	d	Ad^2	I_o
Gussets	102.50	9.53	9300	57,500
Spl PL's	41.25	17.47	12590	2600
do (net)	5.39	32.16	5550	~~~~
do (net)	5.39	2.78	42	~~~~
			27,482	60,100
				27,482
				87,582
			Holes out	− 27,630
				59,952

S_m (top) = 60,000 ÷ 32.35 = 1860

S_m (bott) = 60,000 ÷ 50.53 = 1188

Stresses:

At top

$f = \frac{3200}{108.73} + \frac{3200 \times 17.47}{1860}$

= 29.5 + 30 = 59.5^{ksi} too high

Splice @ U8 (continued)

Try:

	Agr	y	Ay	A(net)
2 Gusset Pl's 82×5/8	102.50	27.00	2770	70.20
2 Spl. Pl's 27½×¾	41.25	0	~	27.75
2 Spl Pl's 28 × ¾	42.00	0	~	28.50
1 Top Spl. Pl 20×3/8	7.50	-14.69	-110	5.39
1 Bott Spl. Pl 20×3/8	7.50	14.69	110	5.39
	200.75	27.00	2770	137.23

2770 ÷ 200.75 = 13.80"

I:

	A	d	Ad^2	I_o
Gussets	102.50	13.20	17,880	57,500
I.S. Spl. Pls	41.25	13.80	7850	2600
o.s. Spl Pls	42.00	13.80	8000	2740
do (net)	5.39	28.49	4370	~
do (net)	5.39	0.89	~	~
			38,100	62,840
				38,100
				100,940
Holes out = 9700 + 22500 (approx)				- 32,200
				68,740

Sm (top) = 68,740 ÷ 28.7 = 2390

Stress @ top

$$= \frac{3200}{137.2} + \frac{3200 \times 13.8}{2390}$$

$= 23.3 + 18.5 = 41.8^{ksi}$

SVERDRUP & PARCEL

JOB Minnesota Bridge No 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 19 OF

DATE Jan 3, 1963

BY C.J.R. CHKD

Splice @ U8 (continued)

Try:

	Agr	y	Ay	A(net)
2 Gusset Pl's 82 × 3/4	123.00	27.00	3320	84.20
5 Spl. Pl's 27 1/2 × 3/4	41.25	0	~	27.75
2 Spl. Pl's 28 × 3/4	42.00	0	~	28.50
1 Top Spl. Pl 20 × 3/8	7.50	-14.69	-110	5.39
1 Bott Spl. Pl 20 × 3/8	7.50	14.69	110	5.39
	221.25	27.00	3320	151.23

23 × 9/8 × 3/2 = 38.8

3320 ÷ 221.25 = 15.00

I:	A	d	Ad²	I₀
Gussets	123.00	12.00	17,720	73,500
1.5. Spl. Pls	41.25	15.00	9280	2600
0.5. Spl Pls.	42.00	15.00	9450	2740
do bott	7.50	29.69	6610	~
do (net)	7.50	0.31	~	~
			43,060	78,840
				43,060
				121,900
Holes out = 12,700 + 24,400 (approx.)				37,100
				84,800

S.m (top) = 84,800 ÷ 29.9 = 2840

Stress @ top

$$= \frac{3200}{151.23} + \frac{3200 \times 15.00}{2840}$$

$$= 21.2 + 16.9 = \underline{38.1^{ksi}} < 45^{ksi}$$ say OK

Connect Spl. Mat'l	Req'd	Furn
Pl 28 × 3/4 = 14.25 × 45 ÷ 15.7	41(-)	48
Pl 27 1/2 × 3/4 = 13.9 × 45 ÷ 15.7	40(-)	48
Pl 20 × 3/8 = 5.39 × 45 ÷ 15.7	15(+)	20
Pl 15 1/2 × 3/8 = 4.13 × 45 ÷ 15.7	12(+)	16

SVERDRUP & PARCEL

JOB 2083 Minnesota Bridge No 9340

COMPUTATIONS FOR Truss Details

SHEET NO. 20 OF

DATE Jan 9, 1964

BY C.J.R. CHKD

Member U10-U12 (A441)

Stress = 900^k C

Capacity = 1020^k C

1920 ÷ 2 = 960^k C avg.

960 ÷ 15.0 = 64 - 1" H.S. rivets req'd

Member U10-L11 (T-1)

Stress = 1930^k T

Capacity = 1970^k T

3900 ÷ 2 = 1950^k T avg.

1950 ÷ 15.7 = 124$^+$ - 1" H.S. rivets req'd.

Joint U10

See Jt U6 Comps.

By inspection, Use (2 - 1/2" Gussets, 2 - 1/2" Splices) (A.S.)

Connect. Splice mat'l	Req'd	Furn.
Pl 27 1/2 × 1/2 = 9.25 × 27 ÷ 15.0 =	17$^{(-)}$	22
Pl 20 × 3/8 = 5.39 × 27 ÷ 15.7 =	9$^+$	10
Pl 15 1/2 × 3/8 = 4.13 × 27 ÷ 15.7 =	7$^{(+)}$	8

1/4" Fill Pl = 4.63 × 27 ÷ 15.0 = 8$^{(+)}$

$f = \frac{850}{72.18} + \frac{850 \times 10.1}{884}$

= 11.8 + 9.71 = 21.5^{ksi} < 27 say OK

Member L11-U12 (A441)

Stress = 1683^k C

Capacity = 1745^k C

3428 ÷ 2 = 1714^k C avg.

1714 ÷ 15.7 = 109$^{(+)}$ 1" H.S. Rivets Req'd

Member L11-L13 (T-1)

Stress = 1976^k T

Capacity = 2036^k T

4012 ÷ 2 = 2006^k T avg.

2006 ÷ 15.7 = 128 1" H.S. rivets req'd

Member U11-L11 (A441)

Stress 536^k C

Capacity 584^k C

1120 ÷ 2 = 560^k C avg

75% × 584 = 438^k C

560 ÷ 15.7 = 36$^{(-)}$ 1" H.S. Riv. Req'd @ L11

438 ÷ 15.7 = 28$^{(-)}$ " " " " @ U11

Joint L11

By Inspection use same Jt mat'l as in Trial #1 @ Jt U6.

2 Gussets - 1/2"
2 Spl. Pl's - 1/2"
Top Spl Pl - 3/8"
Bot " " - 3/8"

A net = 72.18
Mat'l = A441
Stress across Jt = say 965^k

$\frac{965}{72.18} + \frac{965 \times 10.10}{884}$ = 13.4 + 11.0 = 24.4 < 27

say OK

SVERDRUP & PARCEL

JOB 2083 Minnesota Bridge No. 9340 SHEET NO. 22 OF

COMPUTATIONS FOR Truss Details DATE Jan 24, 1964

BY C.J.R. CHKD

Member U12-U14 (T-1)

Stress = 2756 k C

Capacity = 2860 k C

5616 ÷ 2 = 2808 k C avg.

2808 ÷ 15.7 = 179-1" H.S. rivets req'd

Member U12-L13 (T-1)

Stress = 1233 k T

Capacity = 1372 k T

2605 ÷ 2 = 1303 k T avg.

1303 ÷ 15.7 = 83-1" H.S. rivets req'd.

Joint U12

See Jt L5 Comps

By inspection, try (Q.T.) { 2-½" Gussets; 2-½" I.S. Spl. Pls; ⅜" Cov Pl Spl. T&B.

$f = \frac{1976}{102.50} + \frac{1976 \times 9.36}{1160}$

= 19.3 + 15.9 = 35.2 < 45 ksi say O.K.

Conn. Spl. Mat'l	Req'd	Furn.
Pl 27½ × ½ = 13.75 × 45 ÷ 15.7 =	40 (-)	40
Pl 20 × ⅜ = 7.50 × 45 ÷ 15 =	22 (+)	25
Pl 15½ × ⅜ = 5.81 × 45 ÷ 15 =	17 (-)	20
Fill Pl 27½ × ⅜ = 12.3 × 27 ÷ 15.7 =	30 (-)	32

Member L13-U14 (A-36)

Capacity = 584 k C, 700 k T

205 + 285 + 31 = 521 k C } Reversal

213 + 24 - 185 = 52 k T }

573 k

573 ÷ 15.7 = 37 (-) 1" H.S. rivets req'd.

Member L13-L15 (T-1)

Stress = 2958 k T

Capacity = 2961 k T

2961 ÷ 15.7 = 189 (-) 1" H.S. rivets req'd.

Try 4-6 pattern

4 × 15.7 = 62.8 k

2 × ⅝ × 19/16 × 45 = 121 k > 62.8 k

8 × 15.7 = 125.6 k

Try 4-4-6-8 Pattern

14 × 15.7 = 220 k

4 × ⅝ × 19/16 × 45 = 242 k > 220 k

∴ Use 4-4-6-6-8 pattern

Joint L13 (see sheet 23)

SVERDRUP & PARCEL

JOB Minnesota Bridge No. 9340 SHEET NO. 23 OF

COMPUTATIONS FOR Truss Details DATE Jan 27, 1964

BY C.J.R. CHKD

Splice @ L13 (Mtl. - T-1)

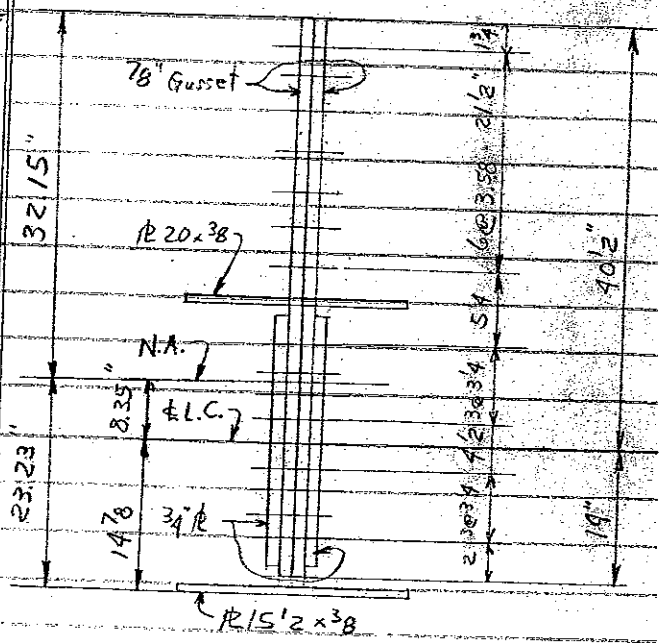
Say Stress across Jt. = stress in Member U12-U14

U12-U14 = 2756^k C

∴ Stress across L13 = 2756^k T

Holes out = $(\frac{9}{8} \times 3\frac{1}{4})(\overline{20.35}^2 + \overline{17.10}^2 + \overline{13.85}^2 + \overline{10.60}^2 + \overline{6.10}^2 + \overline{2.85}^2 + \overline{0.40}^2 + \overline{3.65}^2)$
$+ (\frac{9}{8} \times 1\frac{3}{4})(\overline{8.90}^2 + \overline{12.48}^2 + \overline{16.06}^2 + \overline{19.64}^2 + \overline{23.22}^2 + \overline{26.80}^2 + \overline{30.38}^2)$

= 3900 + 6040 = 9940



Trial #1

	Agr	y	Ay	A(net)	
2 Gusset Pl's 54 1/2 x 7/8	95.40 in²	13.25	1265	65.90 in²	30 x 9/8 x 7/8 = 29.50
2 Spl Pl's 27 1/2 x 3/4	41.25	0	0	27.75 in²	16 x 9/8 x 3/4 = 13.50
1 Top Spl Pl 20* x 3/8	7.50	14.69	110	5.39	5 x 9/8 x 3/8 = 2.11
1 Bott Spl Pl 20 x 3/8	7.50	-14.69	-110	5.39	5 x 9/8 x 3/8 = 2.11
	151.65 in²	13.25	1265	104.43	

* Assumed Effective Pl width (Pl is actually abt 15 1/2" wide)

I:	A	d	Ad²	I₀
Gussets	95.40	4.90	2290	23,600
Spl Pl's	41.25	8.35	2880	2610
do (net)	5.39	6.34	216	~
do (net)	5.39	23.04	2850	~
			8236	26,210
				8236
				34,446
Holes out:				− 9940
I (net) =				24,506

Sm (top) = 24,500 ÷ 32.15 = 764

Sm (bott) = 24,500 ÷ 23.23 = 1055

Stresses

2756^k T across Jt. (see above)

At bottom

$$f = \frac{2756}{104.4} + \frac{2756 \times 8.35}{1055}$$

= 26.4^k + 21.8^k = 48.2^{ksi} > 45^{ksi} (too high)

=

(See Page 24 for Trial #2)

Splice @ L13 (cont.)

Trial #2

	Agr	y	Ay	Anet	
2 Gusset Pl's 54½×¾	81.60 in²	13.25	1081	56.30 in²	30×⅝×¾ = 25.3
2 I.S. Spl. Pl's 27½×¾	41.25	0	0	27.75	16×⅝×¾ = 13.50
2 O.S. Spl. Pl's 28×¾	42.00	0	0	28.25	do do
1 Top Spl Pl 20*×⅜	7.50	14.69	110	5.39	5×⅝×⅜ = 2.11
1 Bott Spl Pl 20×⅜	7.50	-14.69	-110	5.39	5×⅝×⅜ = 2.11
	179.85		1081	123.08	

$\bar{y} = 6.05$

I	A	d	Ad²	I₀
Gussets	81.60	7.20	4230	20,300
I.S. Spl.	41.25	6.05	1510	2610
O.S. Spl	42.00	6.05	1540	2740
Top Spl	5.39	8.64	403	~~
Bott Spl	5.39	20.74	2320	~~
			10,003	25,650

Holes out: 10,003

Say $3900 \times \frac{4.50}{3.25} = 5400$ 35,653

$+ 6090 \times \frac{34}{38} = 5170$ − 10,570

10570 25,083

$S_m(\text{bott}) = 25{,}083 \div 20.93 = 1200$

$f_{(bott)} = \frac{2756}{123.08} + \frac{2756 \times 6.05}{1200}$

$= 22.4 + 13.9 = 36.3^{ksi} < 45^{ksi}$

Say OK

Connect Spl. mat'l

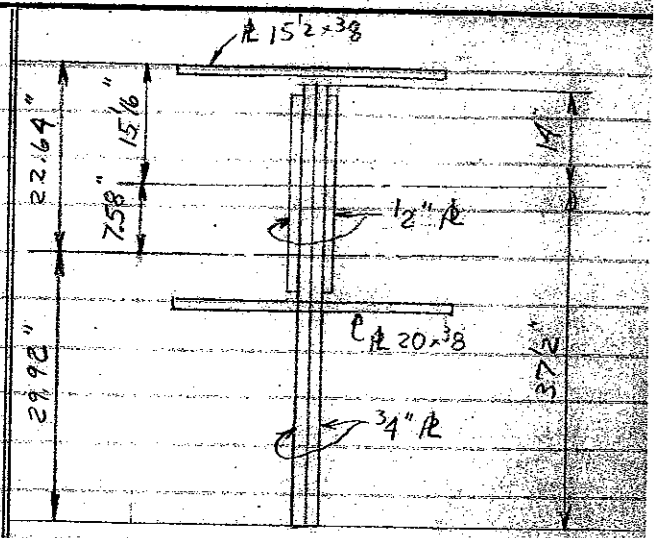
	Req'd	Furn
Pl 28×¾ = 14.12 × 45 ÷ 15.7 =	40⁺	40
Pl 27½×¾ = 13.80 × 45 ÷ 15.7 =	40⁽⁻⁾	40
Pl 15½×⅜ = 4.17 × 45 ÷ 15.7 =	12⁽⁻⁾	16
Pl 20×⅜ = 5.39 × 45 ÷ 15.7 =	15⁽⁺⁾	20
Fill Pl 27½×7/16 = 8.08 × 45 ÷ 15.7 =	23⁽⁺⁾	24

Splice @ U14 (M11-T-1)

Say stress across Jt = stress in Member L13-L15.

L13-L15 : 2958^K T

∴ Stress across Jt U14 = 2958^K C



Try:

	A gr	y	Ay
2 Gusset Pl's 51½×¾	77.20	11.75	907
2 Spl Pl's 27½×½	27.50	0	0
1 Top Spl Pl 20*×⅜	7.50	-14.88	-111.7
1 Bott Spl Pl 20×⅜	7.50	14.88	111.7
	119.70	11.75	907

* Assumed Effective Pl Width (Pl is actually abt 15½" wide)

I:

	A	d	Ad²	I₀
Gussets	77.20	4.17	1340	17,080
Spl Pl's	27.50	7.58	1580	1738
do	7.50	22.46	3800	—
do	7.50	7.30	399	—
			7119	18,818
				7119
			I_{gr} =	25,937

S_m(Top) = 25,937 ÷ 22.64 = 1147

Stress:

$$f = \frac{2958}{119.7} + \frac{2958 \times 7.58}{1147}$$

= 24.8 + 19.6 = 44.4 Too high

∴ Use ⅝" U.S. Spl. Pl's

Develop Horiz. Web Pl

Try ½" weld

$$l = \frac{15.25 \times 31.08}{12 \times 0.707 \times 14.7 \times 4} = 22.8" \text{ req'd}$$

24" Furn OK

Conn. Spl. Mat'l.

	Req'd	Furn
Pl 27½×⅝ = 17.20×45÷15.7 =	49(+)	48

Say OK since unit stress in conn. mat'l is less than 45^{ksi} and Horiz. web Pl cannot be cutback one more rivet line and still use a ½" weld. There is not enough clearance to cut the Spl. Pl around the horiz web Pl.

Pl 15½×⅜ = 5.81×45÷15.0 =	17(-)	20
Pl 20×⅜ = 7.50×45÷15 =	22(+)	25