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TURNER-FAIRBANK HIGHWAY RESEARCH CENTER REPORT

Adequacy of the U10 Gusset Plate Design for the Minnesota Bridge No. 9340 (I-35W over the Mississippi River)

FINAL REPORT

Reggie Holt, PE
Federal Highway Administration

Joseph Hartmann, PhD, PE
Federal Highway Administration

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INTRODUCTION

The I-35W Bridge over the Mississippi River in Minneapolis, MN had 14 spans and a total length of 1,907 feet. The primary structure of this bridge was a variable depth steel deck truss of 1,064 feet in length that carried I-35W over the river and gorge. On August 1, 2007 a failure in the river span of the deck truss caused a complete collapse of the entire truss structure and some of the approach spans resulting in the tragic loss of 13 public motorist lives. The National Transportation Safety Board (NTSB) is the primary agency investigating this failure to determine a probable cause. The Federal Highway Administration (FHWA) is assisting and collaborating with both the onsite and broader activities of the NTSB investigators. The FHWA team consists of personnel from the Turner-Fairbank Highway Research Center (TFHRC), the Office of Bridge Technology (HIBT), the Resource Center (RC) and several Division offices. One of the tasks performed by the FHWA team was a review and assessment of the original bridge design calculations by Sverdrup & Parcel. This report will focus on the findings of this assessment unique to the gusset plate design methodology used for the primary truss and more specifically the design of the gusset plates at location U10. The initial onsite investigation of the collapsed structure identified the failure of the U10 gusset plates as occurring early in the event.

DESIGN SPECIFICATIONS

Minnesota Bridge No. 9340 (herein referred to as the I-35W Bridge) was designed by structural engineering consultant Sverdrup & Parcel for the Minnesota Department of Transportation (Mn/DOT) in the early 1960s. The General Notes on the construction drawings indicate that Mn/DOT commissioned the design to be in accordance with Division I of the AASHO "Standard Specification for Highway Bridges," 1961 Edition and 1961 and 1962 Interim Specifications modified by Minnesota Highway Department standards on allowable stresses. The following sections summarize the appropriate provisions from those specifications relevant to the design of the gusset plates on the I-35W Bridge.

Section 1.4.1 – GENERAL

The provisions of this section outline the various combinations of loads and forces for which structures are designed to withstand and what percentage of material allowable stress will be used for the design resistance calculations associated with each combination. The gusset plate design assessment conducted herein considered the Group I load combination which typically governs the design for this component. This combination includes the effect of dead load, live load, impact, earth pressure, buoyancy, and stream flow. Designing for Group I loads is intended to produce a structure with adequate strength to resist a credible extreme force effect. As this is an ultimate strength condition for the component, use of 100% of the appropriate allowable stresses are permitted in the design.

Section 1.4.7 – HIGH STRENGTH LOW-ALLOY STRUCTURAL STEEL

The provisions of this section specify the allowable stresses that are to be used for design with high-strength low-alloy (HSLA) structural steels. These stresses are dependent on material thickness and direction of loading. For HSLA structural steel less than ¾-inch in thickness such as was used in the U10 gusset plates on the I-35W Bridge, the following allowable stresses were specified in AASHO:

- 27,000 psi for axial tension
- 15,000 psi for shear
- 22,000 psi – 0.56 $(L/r)^2$ for compression

where *L* (in.) is the unsupported length and *r* (in.) is the radius of gyration of the member being designed. These allowable stresses were not modified by the Minnesota Highway Department design standards. This assessment uses these AASHO specified allowable stresses with one exception; no reduction was taken for component instability in compression.

Section 1.6.34 – GUSSET PLATES

These provisions are intended to inform the designer about elements and details associated with the design of gusset plates. They include the statement "gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress." In addition, this section of the specifications indicates that if the length of an unsupported edge of a low-alloy steel gusset plate exceeds 48 times its thickness, the edge shall be stiffened.

LOADS

As summarized above, the Group I load combination was used to determine the strength demands for this gusset plate design assessment. Dead load, live load and live load impact force effects are needed for this load group. These loadings were generated based on the information contained on the original design plans and verified by independent sources within and outside[1] of the assessment team. The generated loads were consistent with the values used in the Sverdrup & Parcel calculations[2] and the truss member forces shown on the design plans. As such, the forces indicated on the design plans were used for the assessment of the gusset plate design. It is important to note that, consistent with accepted design practice, the member forces shown on the design plans are for the maximum force carried by that member due to the varied application (envelope) of live load applied to the bridge. However, these forces are not necessarily produced concurrently for all the members connected by a gusset plate and, therefore, often result in combinations of design loads that do not satisfy static equilibrium at the node.

The following sections provide a brief description of how each component of loading was used in the reconstructed design calculation.

[1] BSDI, Ltd., Coopersburg, Pennsylvania.
[2] Obtained from Mn/DOT.

Dead Load
The permanent loads on the bridge were separated into two dead load components. The first component included the weight of the superstructure below the deck stringers. That is, the weight of the primary truss, floor trusses, and all bracing members. The second component included the weight of the elements that transfer load to the primary truss at panel points through the deck stringers. The weight of the bridge deck, curb, barrier and deck stringers were included in this second component. The dead load forces were assumed to be distributed equally to each truss line. That is, for the purposes of design, the east and west lines of the primary truss were considered to resist one-half of the total dead load of the bridge.

Live Load
The General Notes on the construction drawings also specified the live load model that was used for design: H20–S16–44. This model consists of a placing either a single three axle 72,000 pound truck (truck load) or a uniform 640 pound/foot load in combination with one or more concentrated loads (lane load) in each lane of the bridge to produce the maximum force effect for the component being designed. As expected for bridges with spans of this length, lane load governed the live load design. The governing lane load case was generated by using seven traffic lanes placed transversely across the deck in order to maximize the loading that occurs in one line of the primary truss. The application of live load indicated in Figure 1 placed a maximum demand of 4.23 lanes of lane load on one line of the primary truss.

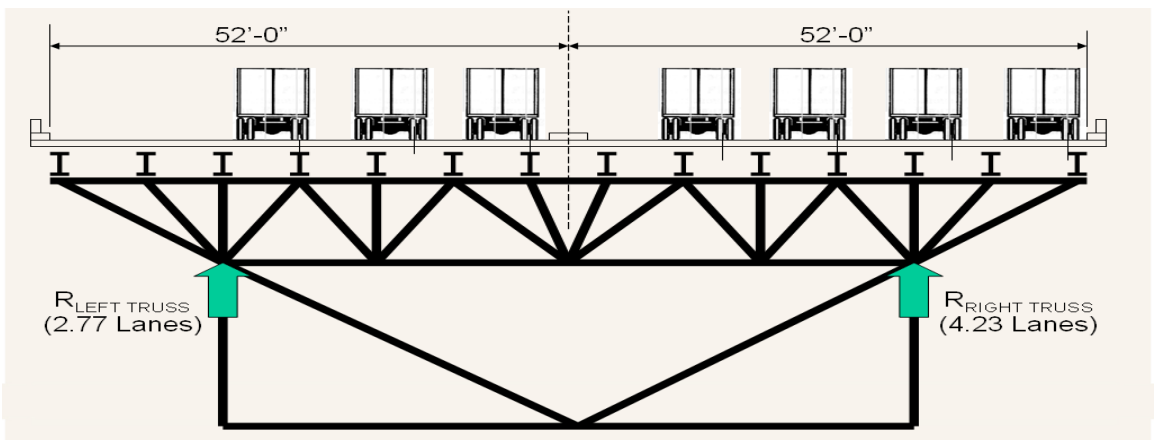


Figure 1. Live Load Lane Placement Used for Design.

Impact
Impact loadings were generated as per the Section 1.2.12 of the AASHO Specifications. These loadings account for the dynamic, vibratory and impact effects on the bridge caused by the moving live load. In application these effects scale up the governing live load by a small fraction called an impact factor. For the I-35W Bridge, the impact factors computed in the original design were 9% for the main span and 13% for the back spans.

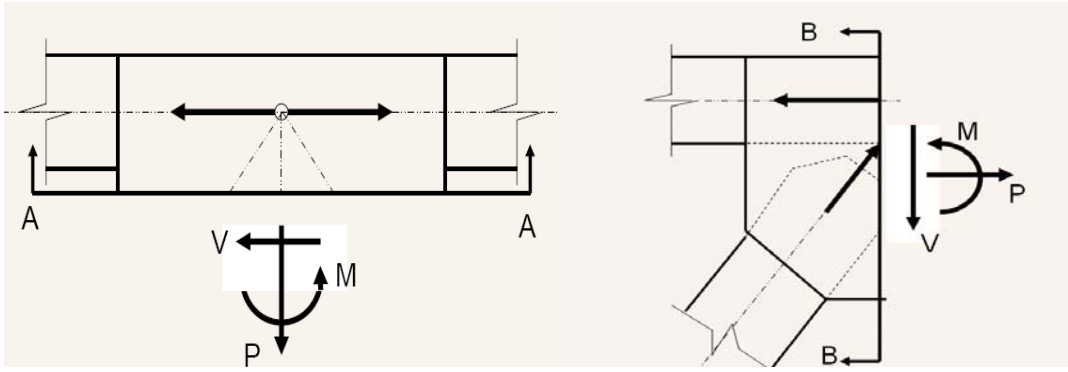
DESIGN METHODOLOGY

The original design calculations by Sverdrup & Parcel for the I-35W Bridge which were supplied by Mn/DOT did not include any information for the primary truss gusset plate designs. It was therefore impossible to check or comment on the original design calculations for the gusset plates in the main trusses. However, the Sverdrup & Parcel design calculations for the gusset plates on the floor trusses of the I-35W Bridge were included in the documents supplied by Mn/DOT. For this report, the basic design methodology used by Sverdrup & Parcel for the floor truss gusset plates was employed to recreate design calculations for the main truss gusset plates. This design methodology is significantly consistent with the methodology used in several other truss bridge designs reviewed by the assessment team:

- Sewickley Bridge over the Ohio River, Pennsylvania, 1979
- Chelyan Bridge over the Kanawha River, West Virginia, 1993
- I-90 over the Grand River (Condition evaluation calculations), Ohio, 1996

All of the design calculations studied used the same general procedures to design gusset plates. The methodology employs general beam theory structural mechanics to analyze the gusset plates and estimate design stresses along a critical section. One of the assumptions of beam theory is that the individual components of stress caused by shear, axial force and bending can be decoupled from the aggregate complex stress state and analyzed independently without a significant loss in accuracy. This approach is consistent with the language used in the governing AASHO Specifications of the era which state that "gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress."

For the purposes of this investigation, this design methodology was used to establish the capacity of the primary truss gusset plates for the I-35W Bridge. In doing so, two gusset plate critical sections were considered. These two sections, herein referred to as Section A–A and Section B–B, are shown in Figure 2. Section A–A, is the plane located between the chords and diagonals of a node and is oriented parallel to the chord (essentially horizontally throughout most of the structure). Section B–B is the typically vertical plane located between the chord and diagonal on one side of the node and the remainder of the node.



SECTION A–A **SECTION B–B**

Figure 2. Gusset Plate Design Sections.

Sections A–A and B–B are typical sections for gusset plate design. A complete design would not be limited to only capacity calculations at these two sections but would include capacity checks on several other planes of significance. However, investigating the capacity of the gusset plate along these two sections will result in an adequate determination of overall performance. Using the truss member forces at each node, the equilibrating shear (V), moment (M) and axial (P) forces shown in Figure 2 are easily determined. The demands of these equilibrating forces on the gusset plate can then be compared to the flexural stress, direct stress and shear stress limitations specified in order to evaluate performance.

RECONSTRUCTED DESIGN CALCULATION RESULTS

This section contains the reconstructed design calculations for the primary critical sections of the U10 gusset plate. The demonstrated methodology was subsequently used to evaluate all of the I-35W Bridge primary truss gusset plates.

GUSSET PLATE U10 GEOMETRY

The geometry of U10 gusset plates and the location of Sections A–A and B–B are shown on Figure 3. The plate dimensions (other than thickness) shown were scaled from the design plan set. The differences between the as-built plate sizes and those shown (scaled) from the plans were minimal. As the dimensions shown on the plan set are the best representative of the original design, the plate dimensions scaled from the plans have been used to reconstruct the design calculations.

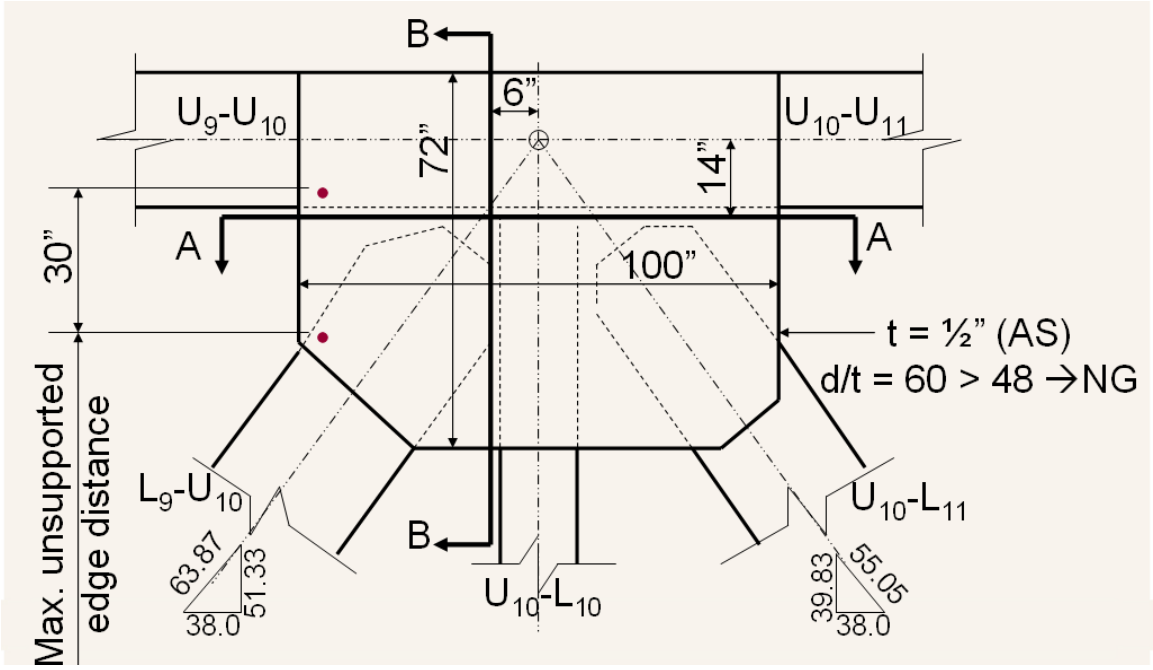
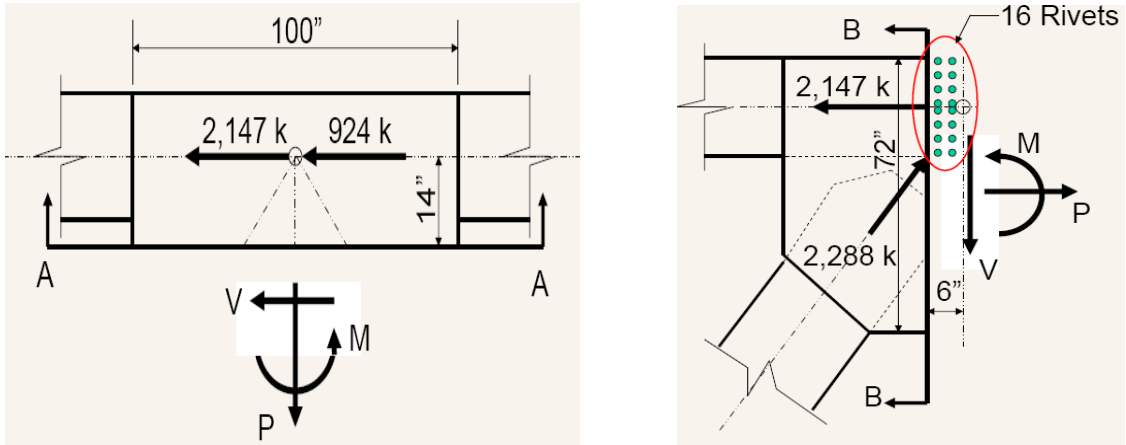


Figure 3. Gusset Plate U10 Geometry.

It should be noted that the offset dimensions that locate Sections A–A (14 inches from the centerline of the top chord) and B–B (6 inches from the centerline of the vertical) in Figure 3 are for the geometry of the U10 gusset plate. The offsets for these sections at other nodes can vary. This is particularly the case for Section B–B as it is located such that there will be no contribution of the connection plates to the resistance supplied by the gusset plates.

GUSSET PLATE U10 RECONSTRUCTED DESIGN CALCULATIONS

Figure 4 shows the free body diagrams used to evaluate the demands of the equilibrated loadings on the critical sections of the U10 gusset plate.



SECTION A–A **SECTION B–B**

Figure 4. Gusset Plate U10 Free Body Diagrams.

The reconstructed design calculations for each of the critical sections indicated on Figure 4 are shown below.

Section A–A

- Section Properties (1 Plate)

$$A = h(t) = 100\left(\tfrac{1}{2}\right) = 50\ in^2$$

$$I = \frac{bh^3}{12} = \frac{\left(\tfrac{1}{2}\right)(100)^3}{12} = 41{,}667\ in^4$$

$$S = \frac{I}{\left(\frac{h}{2}\right)} = \frac{41{,}667}{\left(\frac{100}{2}\right)} = 833\ in^3$$

- Equilibrating Forces

$$V = 2{,}147 + 924 = 3{,}071\ k$$

$$M = 3{,}071(14) = 42{,}994\ k - in$$

- Stresses along section (1 Plate)

$$f_a = P/A = 0$$

$$f_b = M/S = \tfrac{1}{2}\left(\frac{42{,}994}{833}\right) = 25.8\ ksi$$

$$f_{v-avg} = V/A = \tfrac{1}{2}\left(\frac{3{,}071}{50}\right) = 30.7\ ksi$$

- Principal Stresses (at neutral axis)

$$R = \sqrt{\left(\frac{f_a}{2}\right)^2 + \left(f_{v-avg.}\right)^2} = f_{v-avg.}$$

$$f_{ten} = -f_{v-avg.} = -30.7\ ksi$$

$$f_{comp} = +f_{v-avg.} = 30.7\ ksi$$

Section B–B

- Section Properties (1 Plate)

$$A = h(t) = 72\left(\tfrac{1}{2}\right) = 36\ in^2$$

$$I = \frac{bh^3}{12} = \frac{\left(\tfrac{1}{2}\right)(72)^3}{12} = 15{,}552\ in^4$$

$$S = \frac{I}{\left(\frac{h}{2}\right)} = \frac{15{,}552}{\left(\frac{72}{2}\right)} = 432\ in^3$$

- Equilibrating Forces

$$P = 2{,}288\left(\frac{38.0}{63.9}\right) - 2{,}147\left(\frac{\overset{rivets}{56-16}}{56}\right) = -173\ k\ (tension)$$

$$V = 2{,}288\left(\frac{51.3}{63.9}\right) = 1{,}839\ k$$

$$M = 2{,}147\left(\frac{\overset{rivets}{56-16}}{56}\right)(36-14) - 2{,}288\left(\frac{38.0}{63.9}\right)\left[36 - \left(14 + 6\left(\frac{51.3}{38.0}\right)\right)\right]$$

$$M = 14{,}826\ k-in$$

- Stresses along section (1 Plate)

$$f_a = P/A = \tfrac{1}{2}\left(\frac{-173}{36}\right) = -2.4\ ksi$$

$$f_b = M/S = \tfrac{1}{2}\left(\frac{14{,}826}{432}\right) = 17.2\ ksi$$

$$f_{v-avg} = V/A = \tfrac{1}{2}\left(\frac{1{,}839}{36}\right) = 25.5\ ksi$$

- Principal Stresses (at section neutral axis)

$$R = \sqrt{\left(\frac{f_a}{2}\right)^2 + (f_v)^2} = \sqrt{\left(\frac{-2.4}{2}\right)^2 + (25.5)^2} = 25.5\ ksi$$

$$f_{ten} = \frac{f_a}{2} - R = \frac{-2.4}{2} - 25.5 = -26.7\ ksi$$

$$f_{comp} = \frac{f_a}{2} + R = \frac{-2.4}{2} + 25.5 = 24.3\ ksi$$

GUSSET PLATE U10 DEMAND TO CAPACITY RATIOS

Comparing the reconstructed design stresses computed for the U10 gusset plate shown above to the allowable stresses specified in the AASHO Specification, Section 1.4.7 results in demand to capacity (D/C) ratios that illustrate the expected performance of the gusset plate. The D/C ratio is a comparative measure of the efficiency of the design. A D/C value less than 1 indicates a conservative design; a D/C ratio of 1 indicates an efficient design, and a D/C ratio greater that 1 indicates a liberal design with a reduction in the intended factor of safety. Liberal designs are not common but are sometimes acceptable based on the professional judgment of an engineer. The D/C ratios for the U10 gusset plate along Sections A–A and B–B are summarized in Tables 1 and 2.

Table 1. Gusset Plate U10 Section A–A Demand to Capacity Ratios.

Force Type	AASHO Allowable Stress	Reconstructed Design Stress	Demand to Capacity Ratio
Shear ($f_{v\text{-}avg}$)	15.0 ksi	30.7 ksi	2.05
Principal (f_{ten})	27.0 ksi	30.7 ksi	1.14
Principal (f_{comp})	22.0 ksi	30.7 ksi	1.40

Table 2. Gusset Plate U10 Section B–B Demand to Capacity Ratios.

Force Type	AASHO Allowable Stress	Reconstructed Design Stress	Demand to Capacity Ratio
Shear ($f_{v\text{-}avg}$)	15.0 ksi	25.5 ksi	1.70
Principal (f_{ten})	27.0 ksi	26.7 ksi	0.99
Principal (f_{comp})	22.0 ksi	24.3 ksi	1.10

I-35W GUSSET PLATE DEMAND TO CAPACITY RATIOS

Using the method and procedures demonstrated for the design of the U10 gusset plate above, demands and capacities were determined for all of the unique primary truss gusset plates on the I-35W Bridge except those at node U0. This two member node at the end of the bridge was not considered because its geometry is very different than the typical five member nodes elsewhere in the structure. These demands and capacities are reported herein graphically on one half of the symmetric primary truss. When the node is labeled with a prime (‘) after the panel point number (i.e. U2’ as opposed to U2), it indicates that the governing demand for that node came from the north back span. Although the I-35W Bridge primary truss is symmetric about its centerline, loading in the back spans varied slightly due to geometric differences in the approach spans. The results for the Section A–A design calculations are shown graphically on Figures 5 and 6. Similarly, the results for the Section B–B calculations are shown graphically on Figure 7 and 8. The results of all of these calculations are summarized numerically in Table 3.

For the entire set of primary truss gusset plates, the principal stress demands and capacities were determined at the neutral axis of the critical section. Therefore, the data shown in the figures and tables of this section may not represent the actual maximum principal stress along a critical section at locations where combined bending and axial stresses are a significant portion of the aggregate stress state. However, when shear stress dominates (as is the case in these gusset plate analyses), the principal stress calculated at the neutral axis of the critical section is also the most likely maximum principal stress.

Figures 5 through 8 contain bar charts indicating the demand and capacity for a given gusset plate which is located on an accompanying figure of the primary truss. There are individual figures to address shear stress and principal stress comparisons. Demands that are greater than capacities are highlighted with a red box.

Demand to capacity ratios greater than 1.00 in Table 3 are indicated with a gray field.

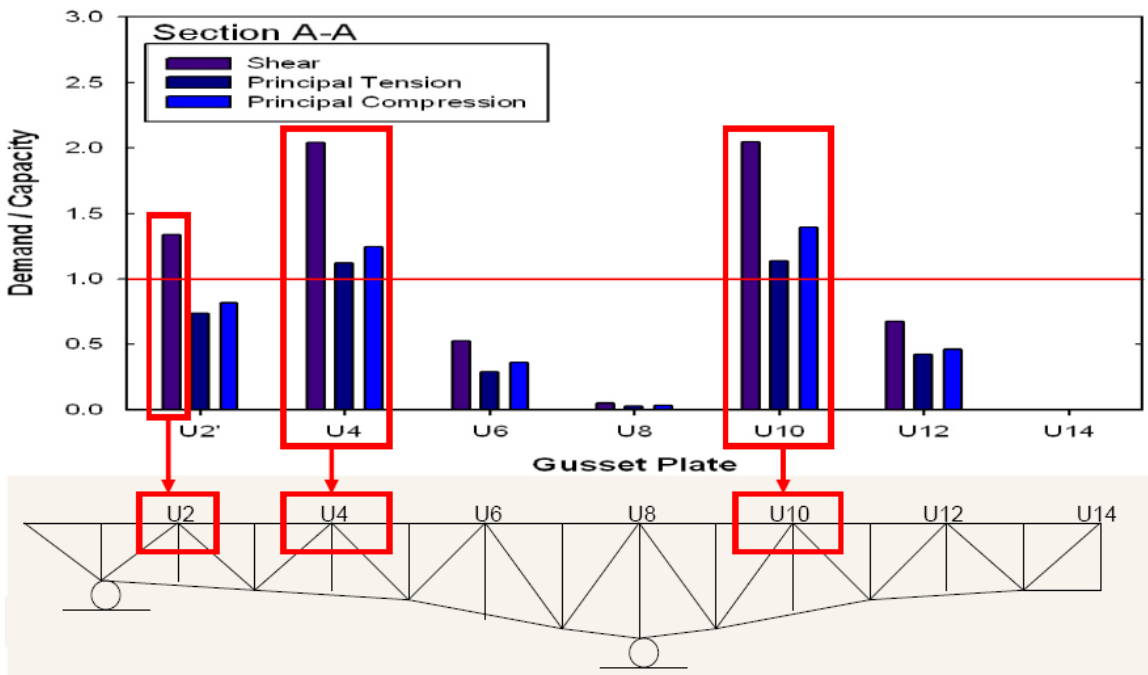


Figure 5. Demand to Capacity Ratio for Section A–A of Upper Nodes.

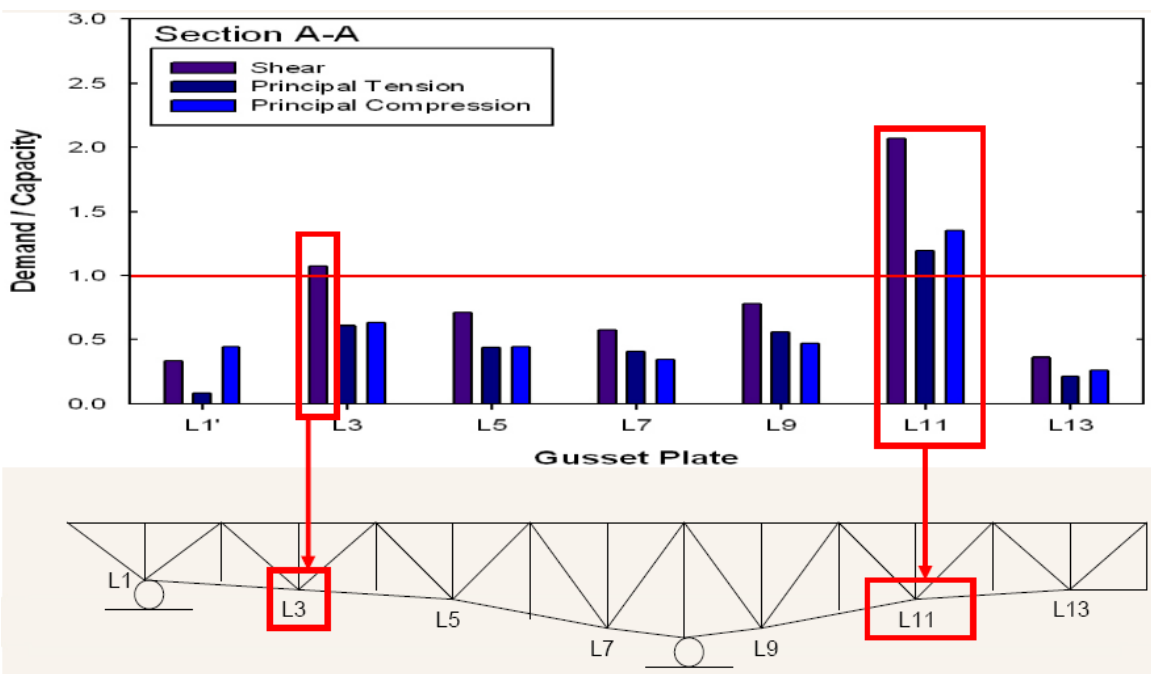


Figure 6. Demand to Capacity Ratio for Section A–A of Lower Nodes.

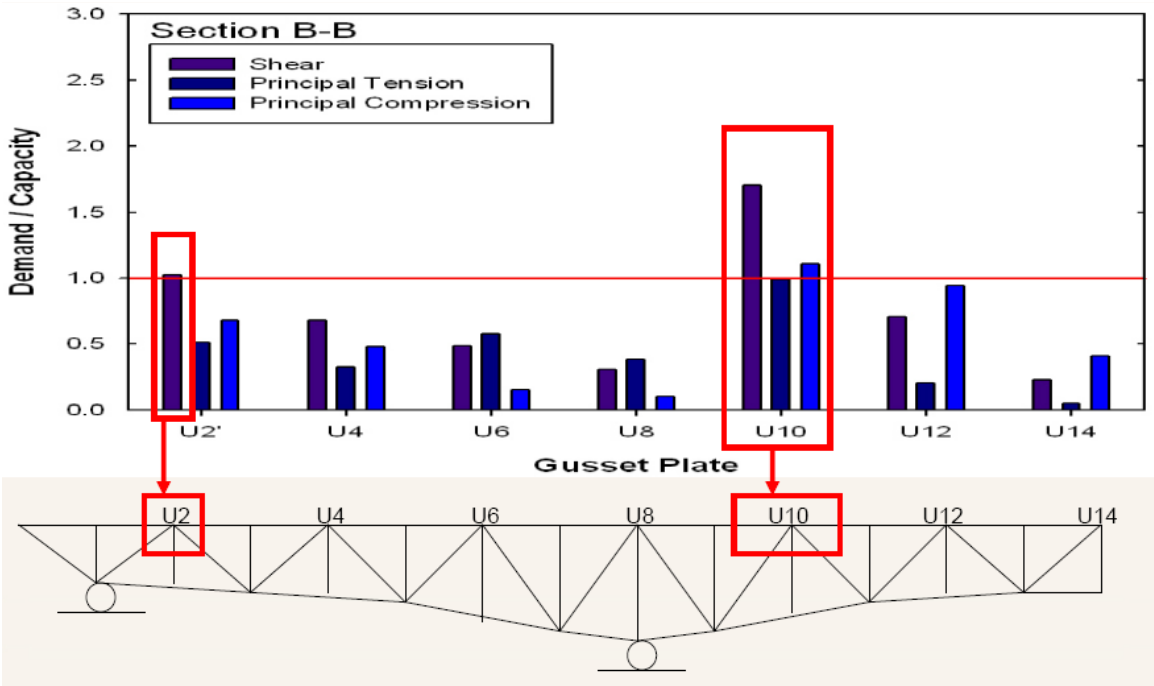


Figure 7. Demand to Capacity Ratio for Section B–B of Upper Nodes.

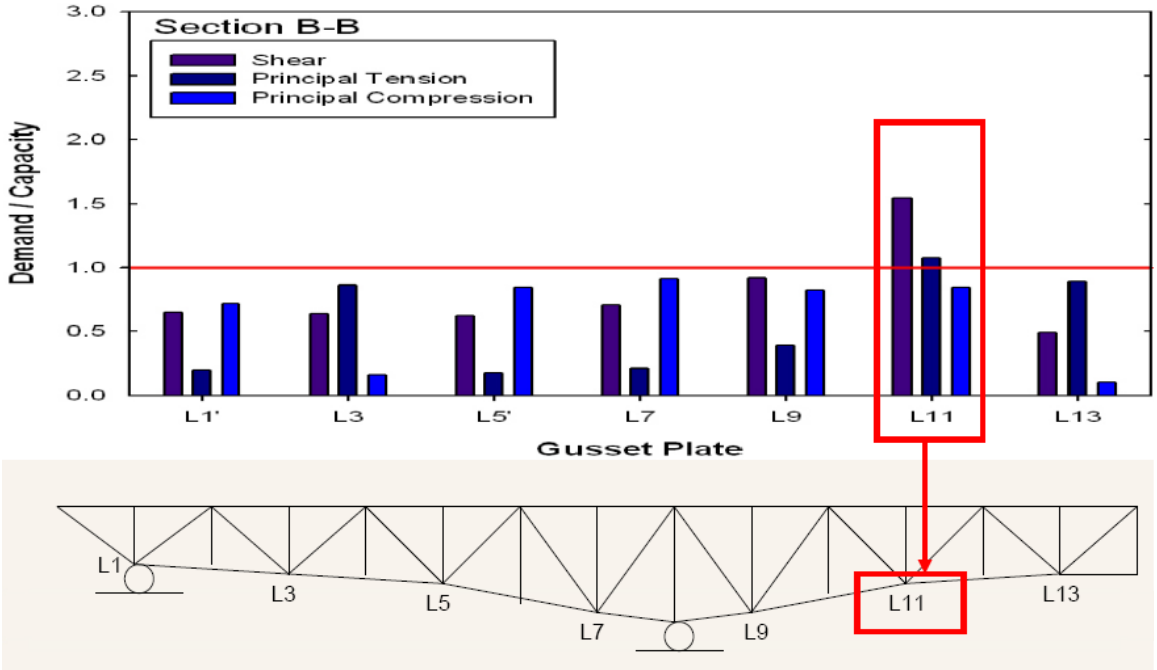


Figure 8. Demand to Capacity Ratio for Section B–B of Lower Nodes.

Table 3. Demand to Capacity Ratios for the Primary Truss Gusset Plates.

Gusset Plate	Thickness in.	Steel Grade ksi	Demand/Capacity					
			Section A–A			Section B–B		
			Shear	Principal Tension	Principal Compression	Shear	Principal Tension	Principal Compression
Upper Nodes								
U2’	5/8	36	1.34	0.74	0.82	1.02	0.52	0.68
U4	1/2	36	2.04	1.12	1.25	0.68	0.32	0.48
U6	1	50	0.52	0.29	0.36	0.48	0.58	0.15
U8	1 3/8*	100	0.05	0.03	0.04	0.31	0.38	0.10
U10	1/2	50	2.05	1.14	1.40	1.70	0.99	1.11
U12	1	50	0.68	0.42	0.46	0.71	0.20	0.94
U14	1	50	0.00	0.00	0.00	0.23	0.05	0.41
Lower Nodes								
L1’	1*	36	0.34	0.09	0.44	0.65	0.20	0.72
L3	1/2	36	1.08	0.61	0.64	0.64	0.86	0.16
L5	5/8	50	0.71	0.44	0.44	0.62	0.18	0.80
L7	1	50	0.58	0.41	0.35	0.71	0.21	0.91
L9	1	50	0.79	0.56	0.47	0.92	0.39	0.82
L11	1/2	50	2.07	1.20	1.35	1.55	1.07	0.84
L13	1	50	0.36	0.21	0.26	0.49	0.89	0.10

*Thickness of built-up (multi-ply) gusset plate.

I-35W GUSSET PLATE DETAILING

Figure 9 presents a visual survey of the primary truss gusset plate thicknesses and steel grades from the as-built I-35W Bridge. In the back-span of the primary truss, a gradual decrease in gusset plate thickness is observed as the distance to the node from the points of support increases. In the main-span, a less gradual transition in plate size is evident. For this comparison, the thickness of the built-up gusset plates at nodes L1 and U8 are represented in the figure.

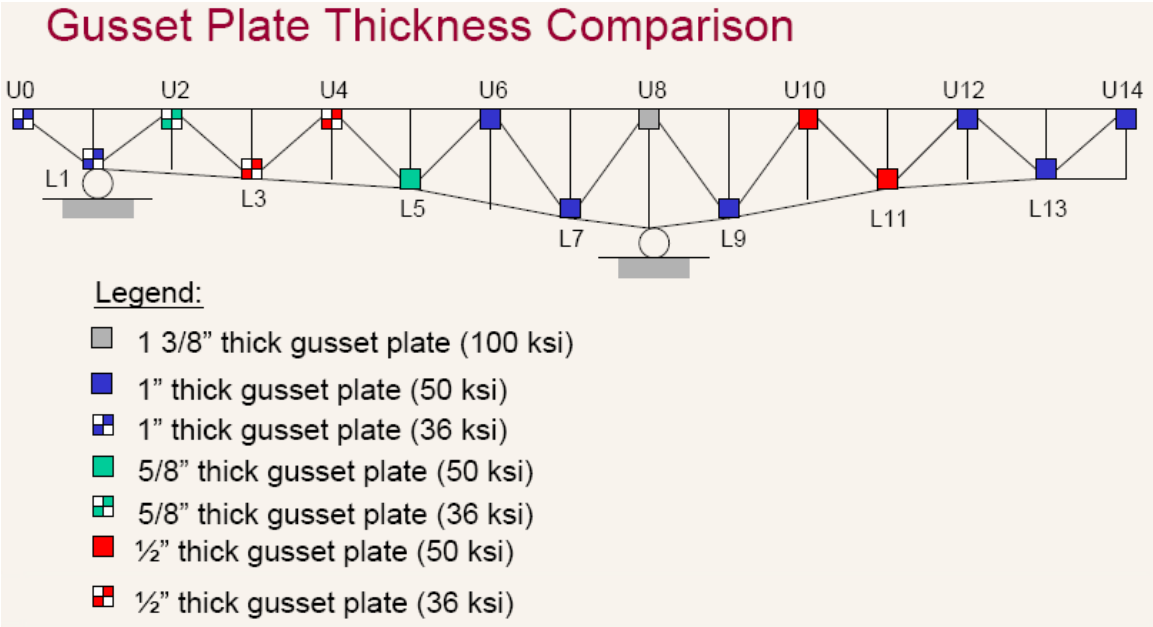


Figure 9. Gusset Plate Thickness Comparison.

The AASHO Specifications of the era also included a detailing requirement for gusset plates in Section 1.6.34. For low alloy steels of the type used in the construction of the gusset plates of this bridge, the specification limited the slenderness of the unsupported edge length to 48 times the plate thickness (for carbon steels this limit increases to 60). If the unsupported edge length exceeded this limit, stiffening was required. Stiffening of the edge in these cases was needed in order to avoid prematurely compromising the capacity of the gusset plate due to buckling potentially caused by compression from the primary truss diagonals.

Table 4 assesses the adequacy of the unsupported edge lengths of the gusset plates provided on the I-35W Bridge. An initial comparison of the unsupported edge lengths and limits presented indicates that the gusset plates at U8, L8 and U10 did not meet the specified slenderness limit. However, the diagonals at U8 carry a net tension load due to all the load groups considered mitigating the need for edge stiffening, and edge stiffening was provided at L8 to bring that gusset plate into compliance with the specification. Therefore, based on the reconstructed design assessment, only the gusset plates provided at U10 were not in compliance with the specified limits (indicated by the gray shading in the table).

Table 4. Gusset Plate Unsupported Edge Adequacy.

Gusset Plate	Thickness of Unsupported Edge (in.)	Unstiffened Unsupported Edge Limit (in.)	Unsupported Edge Length (in.)	Assessment/Compliance
Upper Nodes				
U2	5/8	37.5	30	OK
U4	1/2	30	16	OK
U6	1	48	22	OK
U8	5/8	30	36	Tension Diagonals Only – OK
U10	1/2	24	30	**Inadequate – No Good**
U12	1	48	22	OK
U14	1	48	16	OK
Lower Nodes				
L1	1	60	36	OK
L3	1/2	30	26	OK
L5	5/8	30	20	OK
L7	1	48	16	OK
L8	1	48	54	Edge Stiffening Provided – OK
L9	1	48	22	OK
L11	1/2	24	22	OK
L13	1	48	18	OK

INTERPRETATION OF RESULTS

Contrasting a review of the D/C ratios in Table 3 with the assessment of unsupported edge lengths shown in Table 4 indicates that the thickness of some of the primary truss gusset plates were seemingly dictated by the demands of the applied loading while others were determined by the geometric needs of the connection or the resulting slenderness requirements of the unsupported edge. However, there are several gusset plates with D/C ratios so in excess of an efficient design for which the only explanation can be an error in either the analysis or calculation of the designer.

From the reconstructed design and detailing information, it appears, as usually is the case, that the width and length of many gusset plates was dictated by the number of connectors (rivets) needed to fasten the truss members to the gusset plate. Given the equal and opposite direction of the chord forces at supports and mid-spans it is not uncommon to see extremely low D/C values for gusset plates at these nodes which likely explains the thickness and steel grade combinations used at L1, U6, U8 and U14. In addition, it seems likely that the thickness of the gusset plates supplied at L5, L7, L9, U12 and L13 were the result of the demands of the applied loading. All of these gusset plates have one or more D/C ratio equal to or greater than 0.80 for the thickness and steel grade detailed.

The gusset plates at U2, L3, U4, U10 and L11 all failed the D/C ratio check for shear along Section A-A. These gusset plates all have another common feature; they all are at locations in the truss where the force in the primary chord they connect changes from either tension to compression or vice versa. While it could be argued that the calculated overstress at L3 could be the result of a liberal design, the gussets at U4, U10 and L11 fail this check considerably. As shear significantly dominates these 3 connections they also fail the principle stress D/C ratio checks on Section A-A.

The gusset plates at U10 and L11 also fail the shear D/C ratio check on Section B-B by a significant margin. The 2% overstress for shear indicated for the U2 gusset on Section B-B in Table 3 is negligible. The U10 and L11 plates also have principle tension and compression overstresses, respectively, that are again a result of the dominate shear on the section considered.

Only the U10 gusset plate violated the unsupported edge limitations.

Reviewing the entire set of D/C ratios in total reveals a consistent mis-handling of shear by the designer. The gusset plates at U10 where the most under-designed as a result of this error. When the resulting lack of capacity of these gusset plates was coupled with a violation of the detailing requirements for the unsupported edge, they became the most vulnerable plates in the truss to the affects of any applied loads. In summary, the capacity inadequacies for the U10 gussets were considerable for all conditions investigated with the plate providing approximately one-half of the resistance required by the design loadings without consideration for instability.