I-35W Bridge Collapse

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Abstract: The I-35W bridge over the Mississippi River in Minneapolis, Minnesota, collapsed suddenly on August 1, 2007. This note briefly summarizes an analysis based on original design drawings, an investigation of material evidence provided by the National Transportation Safety Board (NTSB), and a full-scale load rating of the bridge superstructure. The results of the investigation and conclusions of the analysis include. (i) The thickness of gusset and the thickness of the side wall of the upper chords were designed proportional to the bending moment solution of a one-dimensional influence line analysis. This fact reveals that the NTSB-disclosed undersized gusset plates are the consequence of a bias toward a "one-dimensional model" in the original design that did not give sufficient consideration to the effects of the forces from diagonal truss members. (ii) Although the bridge's truss-cell structure was appropriately designed, the design of the node that connected the floor members to the main truss-frame was inadequate to effectively distribute live and dead loads. Consequently, the local redundancy provided by the truss-cells was significantly reduced. (iii) A three-dimensional, nonlinear, finite-element, computation-based load rating indicates that some of the gusset plates had almost reached their yield limit when the bridge experienced the design load condition. The bridge was sustained by the additional safety margin provided by the ultimate strength of the ductile steel that comprised the gusset plates.

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Introduction

The interstate highway bridge I-35W over the Mississippi River in Minneapolis, Minnesota, collapsed suddenly at 6:05 p.m. on August 1, 2007 (see Fig. 1). Approximately 1,000 ft of the 1,907-ft-long bridge fell to the water and ground, resulting in 13 fatalities and 145 injuries, with 111 vehicles involved in the collapse.

Designed in 1964 and opened to traffic in 1967, this bridge was a three-span continuous deck-truss structure flanked by steel-girder and concrete-slab approaches. The deck-truss spans contained a 458-ft-long main span, two 265-ft-long side spans, six through-traffic lanes, and two auxiliary lanes. The deck was 108 feet wide from curb to curb and 113.25 ft wide overall.

According to the National Transportation Safety Board's [National Transportation Safety Board (NTSB) 2008a,d] investigation, roadway construction was underway on the deck-truss portion of the bridge, while four of its eight lanes were closed because of parked machineries and stock-piled paving materials on the bridge at the time of the collapse (see Fig. 1). There were also at least two recorded major rehabilitations done to the bridge previously, one in 1977 and another in 1998. As part of these rehabs, the average thickness of the concrete deck was increased from 6.5 to 8.5 in. and the original six traffic lanes were widened to 8.

The NTSB's investigation found that the bridge's gusset plates U10 and L11 were undersized and concluded that the bridge would have stood if the gussets were twice as thick; for instance, 1 in. in thickness as was the nearby U12 gusset plate. The following questions, therefore, remain: Was there any reason for such an undersized design for this crucial bridge over the Mississippi River? Why did neither the original designer nor the managers of the subsequent rehabilitations notice the abnormality of these gussets and the underlying risks?

Based on the information released to the public [Minnesota Department of Transportation (MDOT) 2007; Minnesota's State Government Libraries (MSGL) 2007; National Transportation Safety Board (NTSB) 2007a,b,c, 2008a,b,c,d] and the evidence disclosed by the NTSB, this report briefly summarizes the writer's analysis (Hao 2007, 2008, 2009) of these concerns. It is the writer's wish that this analysis will trigger more prominent discussions and shed light on the potential safety issues that may exist in the truss bridges that are currently in service.

Procedure for Analysis

The purpose of this study is to evaluate the amplitude of the stresses in each structural component within the deck-truss section of the bridge according to design drawings and the *American Association of State Highway and Transportation Officials Bridge Design Specification* (AASHTO 2007). A two-level computational model has been developed for this purpose. Plotted in Fig. 2 is a global-level, three-dimensional (3D) finite-element model

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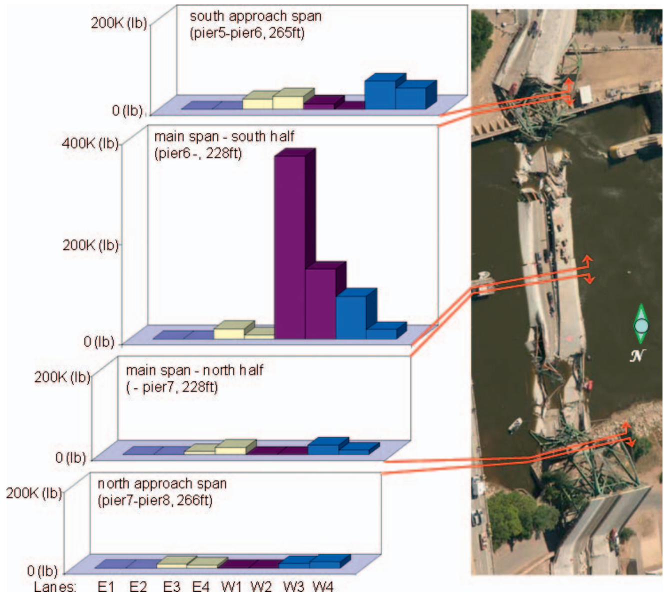

Fig. 1. (Color) Live load on each lane from east to west (left to right in the figure) over the four parts of the I-35W deck-truss spans, respectively, at the moment of collapse. Data were collected from National Transportation Safety Board (NTSB) (2008b).

that includes the two main truss frames on the west and east sides of the bridge, respectively, connected by 27 floor truss frames, lateral bracings, steel girders, and the concrete deck. The structural components of the bridge were categorized into three groups: long structural members (trusses and girders), stiffening components (gusset plates, stiffeners, and struts), and the concrete deck, represented in the model by the beam, thick-shell, and thick-plate finite elements, respectively. The reinforcement of a truss to the attached gusset plate was modeled by laying the beam elements partially over the shell elements, while the sizes of the overlapping were assigned according to design drawings. The deck's live load was modeled by the similarly sized 3D brick elements with the density consistent to their weight. Based on the geological information for the local area, the bridge's substructure was assumed to be rigid, represented by the fixed boundary conditions imposed on the nodes at the corresponding piers. The computation of this model gave accurate (in terms of numerical analysis) forces and bending moments for the long structural members and concrete deck. It also demonstrated the stress-strain states of these stiffeners, such as gussets, with moderate accuracy because some geometric details were included. The global-level solution provided boundary conditions for the submodel of each gusset plate with detailed geometry, which will be introduced later.

According to official documents [National Transportation Safety Board (NTSB) 2007b,c] the bridge's steel members were made of grade 50 mild steel with Young's modulus of 200 GPa (29,000 ksi), a Poisson's ratio of 0.3, a yield strength of 348 MPa (50.5 ksi), and an engineering ultimate strength of 593 MPa (86 ksi) at 10% normal strain. The J_2-incremental plasticity-based constitutive law (Hill 1951) had, therefore, been employed on these members. The concrete deck slab was modeled as linear elastic with Young's modulus of 21 GPa (3,000 ksi) with the T_{cut} at 69 MPa (10 ksi). The density of the steel used was $7{,}800\ \text{kg/M}^3$ (490 pcf) and the density of the concrete was $2{,}290\ \text{kg/M}^3$ (143 pcf).

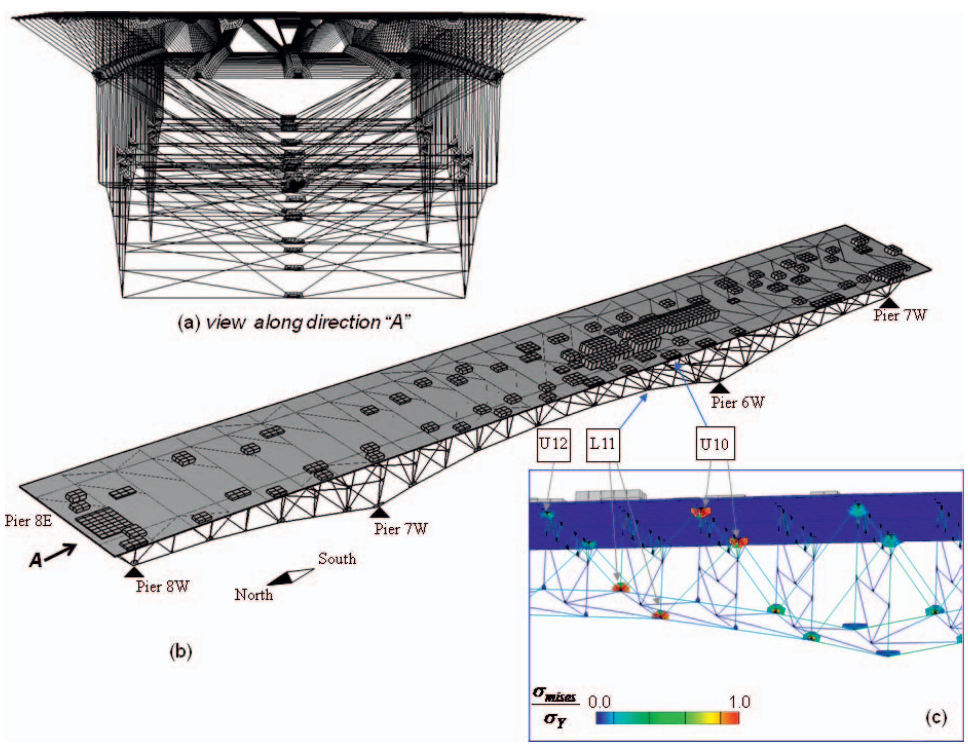

Fig. 2. (Color) Global 3D finite-element model of the I-35W bridge: (a) view of the model from north; (b) overview of the model, where the deck live load at the moment of collapse is modeled by brick finite elements; and (c) the computed stress contours, showing that gusset plates U10 and L11 had the highest stress under the deck load

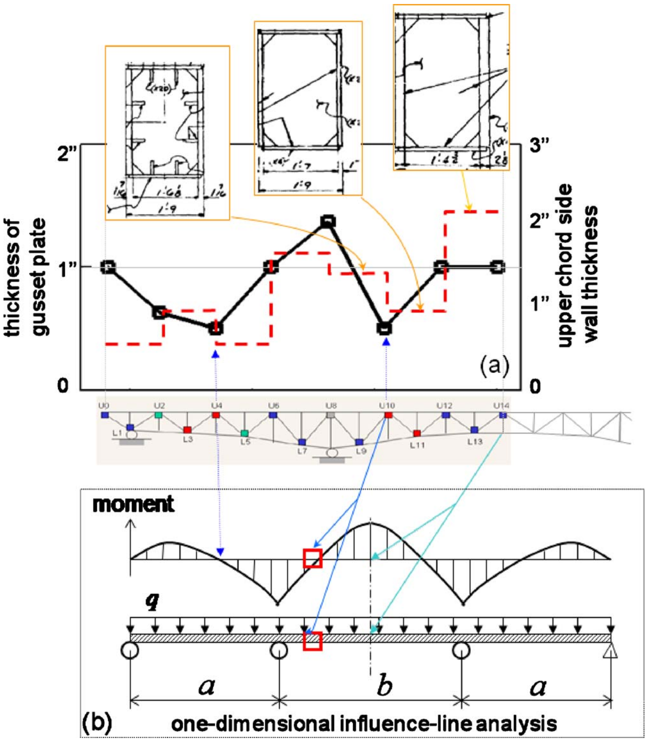

Fig. 3. (a) Thicknesses and locations of I-35W gusset plates (the empty squares connected by solid line) and the side-wall thicknesses of the upper chords (dashed line); (b) one-dimensional influence-line solution for a three-span bridge model that was used to explain I-35W design (Hao 2007). A consistency can be found between the distributions of these thicknesses and the bending moment solution.

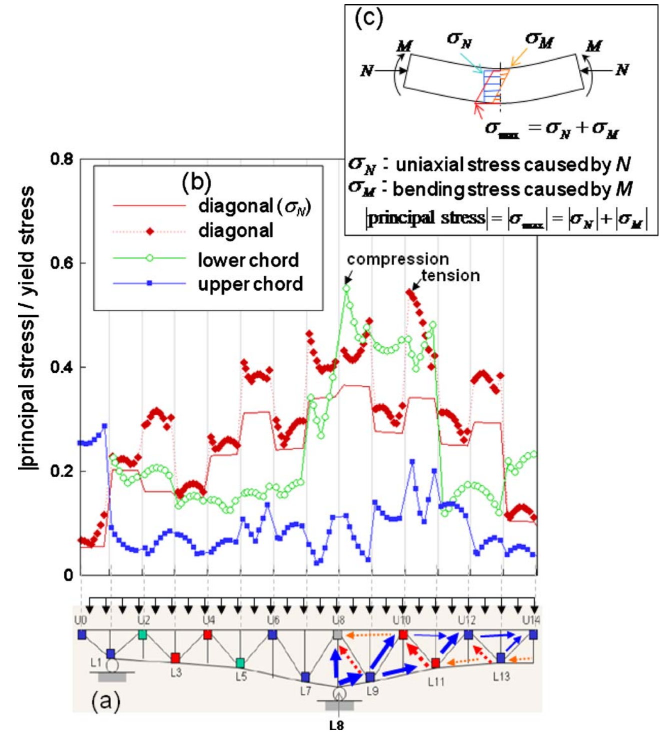

Fig. 4. (a) One-half of the main truss frame with an illustration of the load path in the center span; (b) the corresponding amplitudes of the stresses in western main frame under the collapse load. Here, the |principal stress| can be either the first principal stress (tension) or the third principal stress (compression) but with the maximum amplitude.

Why the Gusset Plates Were Undersized

NTSB's investigation revealed that the failure of the undersized U10 gusset plate initiated the bridge's collapse. Gusset plates U10, L11, and U4 (see Fig. 3) were only 0.5 in. thick, whereas other gusset plates were 1 in. thick or greater. To explore the reason why the gusset plates were undersized, Plot (a) of Fig. 3 shows the locations of the gusset plates (represented by the squares connected by solid lines) and the thicknesses of the upper chords' side wall (the dashed lines). Plot (b) of Fig. 3 demonstrates the one-dimensional influence-line solution for the three-span bridge model that was used to explain the original design (Hao 2007). By comparing the thickness distributions of these structural components with the bending moment solution, one can find an obvious consistency. This fact reveals that the original design, at least for the main frame gusset plates and upper chords, was based on similar one-dimensional models. By contrast, Plot (b) of Fig. 4 displays the stresses in the western upper, lower, and diagonal main-frame truss members under the load at the time of the collapse, which demonstrates that the principal stress in the diagonal members was about four times higher than in the upper chords near the U10 gusset plate. Consequently, the forces from the diagonal members dominated the stress state in the gusset plate.

The computational results in Plot (b) of Fig. 4 can be summarized as following: (i) the stress levels in the diagonal and lower chords were generally higher than in the upper chords; (ii) two stress peaks occurred: a tension peak in diagonal member U10-L11 near gusset plate U10 and a compression peak in the lower chord L8-L9 near gusset plate L8. According to design drawings, the node L8 was reinforced by a 1-in.-thick gusset plate overlapped by another small 0.625-in.-thick cover plate. This analysis focuses on gusset plate U10 and the diagonal members. (iii) By comparing the uniaxial stress, σ_n, in diagonal member (represented by the solid line) and its principal stress (the solid diamonds connected by a dotted line), the difference between the two stresses was the contribution of the bending moment. In the area around gusset plate U10, the contribution was about one-third of the total stress. Plot (a) of Fig. 4 represents one-half of the main frame of the bridge with an illustration of the load path; the solid arrows demonstrate the resistance flow from the supporting pier to the upper chords and the dashed arrows represent the tension flow that has one peak on the lower chords at the middle of the center span and another peak on the upper chord just above the supporting pier. The gusset plates with a distance to the supporting pier between one-sixth and one-third of the total center span length, like U10, are the pivots that transfer deck-load and center-span weight to the lower chords and piers through diagonal members. One-dimensional influence-line models only give estimates of the stresses in the upper and lower chords and are generally unable to provide information of the force flows in the diagonal members. The amplitudes of the forces in each truss member under different load conditions were analyzed (Hao 2007). As indicated in Gee (2008), more detailed analysis is necessary for design and rating of this kind of steel bridges.

Cause of the Collapse

Plotted in Fig. 5 are the stresses in the diagonal members under three loading conditions: dead load of the original bridge with a

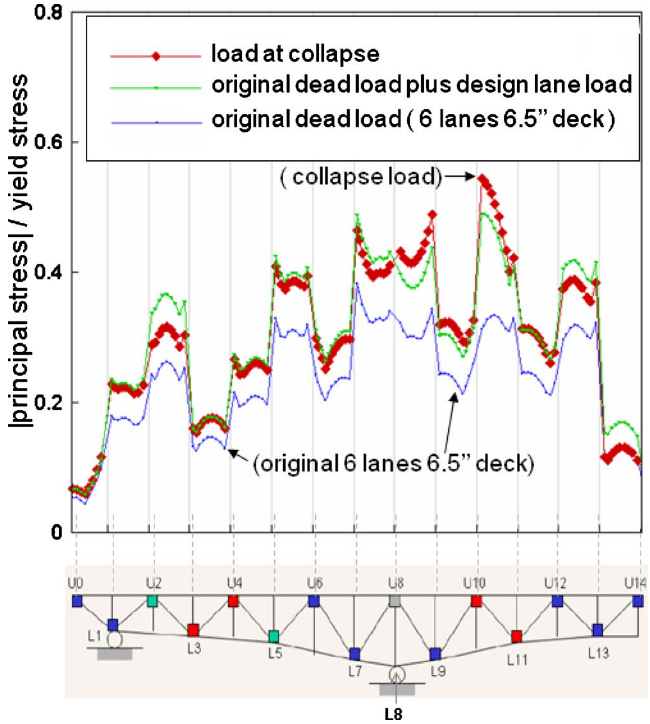

Fig. 5. Principal stress in truss diagonals under three load conditions which show that the peak stress appears at the location near gusset plate U10 when the deck load is present

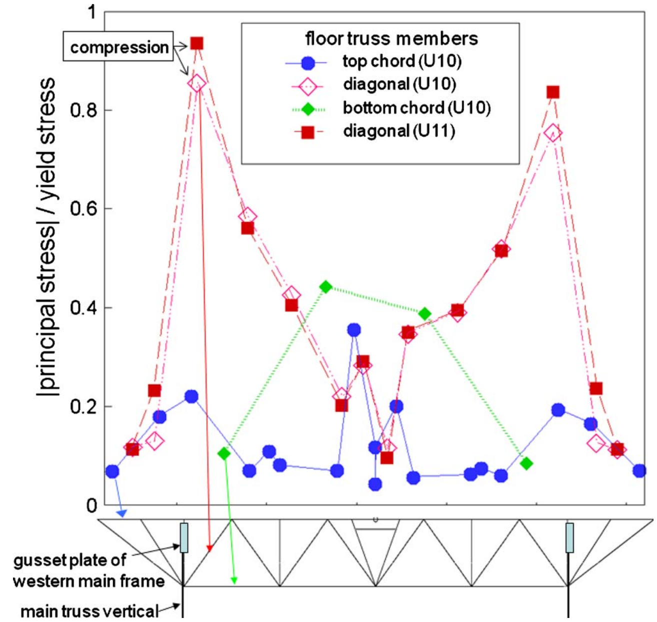

Fig. 7. Stresses on the floor truss frames above nodes U10 and U11 under collapse load. The peak stress occurs on the diagonal truss member between the top floor truss and the main truss vertical member, which caused the elevated stress in main frame truss vertical member (see Fig. 6).

6.5-in.-thick deck, the dead load plus the design lane load, without the design truck load, and the load at the time of the collapse. This plot demonstrates that the peak stress under the collapse load is about 10% higher than the peak stress in the area near gusset plate U10 under the second load condition. Fig. 6 represents the stress distribution in the vertical truss just beneath the western

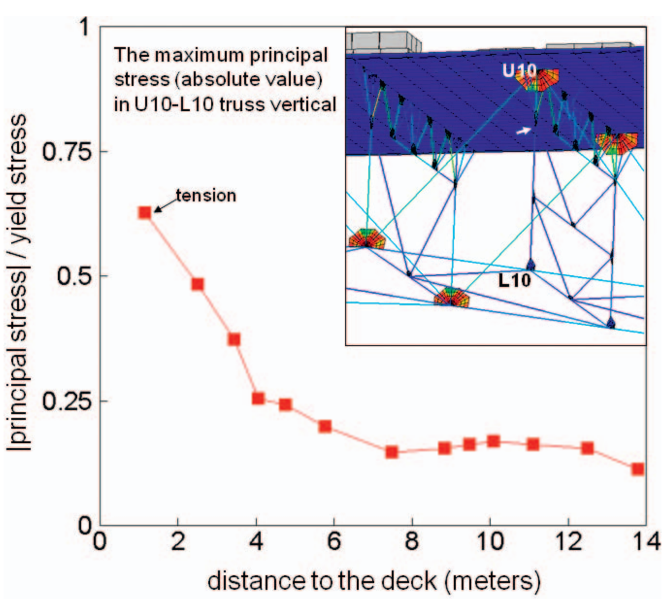

Fig. 6. (Color) Principal stress in the vertical truss between gusset plates U10 and L10. The amplitude rises about three to four times within the member between the floor truss joint (indicated by the white arrow) and gusset plate U10.

gusset plate U10 under the collapse load, demonstrating the rising amplitude at its end attached to the gusset. Displayed in Fig. 7 are the stresses in the members of the floor truss frames above nodes U10 and U11 under the collapse load. These frames were just beneath the stock-piled construction materials when the bridge collapsed. In both locations, the maximum compressive stress occurred at the diagonal members that connected the top floor trusses and the western main frame vertical truss.

According to Figs. 4–7, the principal stresses in the truss members were generally below the steel yield strength under the collapse load. On the other hand, remarkably high stresses can be found on the ends of the trusses attached to the western gusset plate U10. Fig. 8(a) demonstrates the connection between the floor truss, cantilever, and main frame trusses within the triangular area centered at gusset plate U10. Fig. 8(b) shows the corresponding finite-element model using 3D tetrahedral elements. This design enabled the bridge to support a wider deck but required stronger gusset plates to sustain the two associated load paths: compression from the deck onto the plate's top and down pulling from the vertical truss to its bottom. The particular design of the floor truss frame in I-35W also induced lateral forces through the vertical truss to the gusset plate.

As indicated in Fig. 4, the diagonal truss member U10-L11 carried the tension stress with the highest amplitude among all of the main truss members. This stress was corresponding to the weight of the middle part of the central span and the deck load above. In that plot, the low-stress amplitude in the upper chords attached to gusset plate U10 implies that these horizontal members contributed limited load capacity to the gusset; therefore, the majority of the vertical load on this node was balanced by the compression force and bending moment carried by the diagonal member U10-L9. It seems as if the designer noticed this high compression because in the original drawings of the U10-L9 member, there was a box beam with a side-wall thickness of 2 in. and a section area larger than any of the other truss members

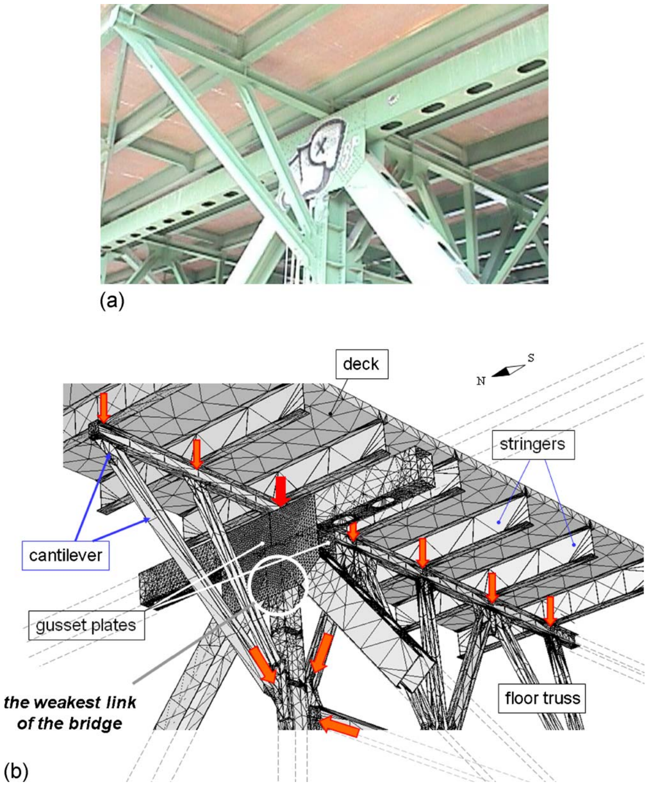

Fig. 8. Design of the nodes that connect floor trusses and main frames: (a) the node U10; (b) finite-element model

attached to gusset plate U10, which is why the stress on this member was lower than others. However, the total force input from this member to the gusset plate was the highest (Hao 2007). Thus, although all the truss members remained elastic, the stress level in this 0.5-in.-thick gusset plate was much higher. It is well known that a rivet hole introduces stress concentrations with a factor of 2 under uniaxial tension (Timoshenko and Goodier 1970). The finite element solution in Fig. 9 and the computations

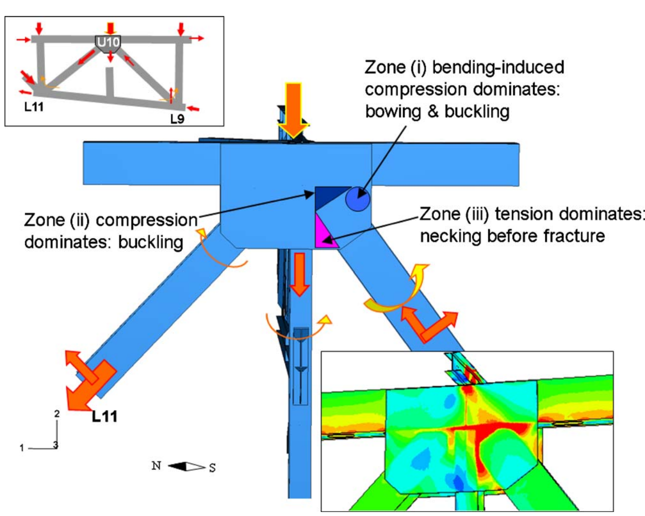

Fig. 9. Illustration of the forces and moments around gusset plate U10, which produced three stress concentration zones on the side with the compressive diagonal member

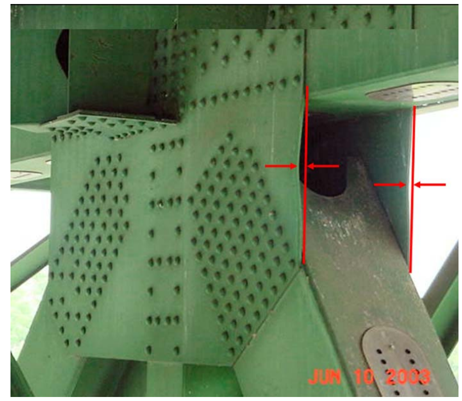

Fig. 10. Bowing at gusset plate U10 before the collapse [National Transportation Safety Board (NTSB) 2008d]

by Hao (2008) indicate that the elastic concentration factor could be double this value.

Fig. 9 illustrates the stress state of gusset plate U10 computed by the submodel in Fig. 8(b). When a diagonal member introduces compressive force, stress distribution on the gusset can be characterized by three zones: Zone (i), the area between horizontal and diagonal members where the compressive stress induced by the bending moment dominated and it is this compression that may have caused bowing and buckling; Zone (ii), the triangular area between the ends of the horizontal, vertical, and diagonal members where buckling could be a dominant failure mode; and Zone (iii), the area between the diagonal and vertical truss members where a tension stress state might dominate. The bending-moment-induced tension along the horizontal direction and the pulling from the vertical truss caused the downward tension.

Under this condition, the scenario of the collapse could be as follows: the biaxial tension in Zone (iii) induced plastic deformation that resulted in thickness reduction and necking. Such a deformation pattern is compatible with a simultaneous bowing in Zone (i). The latter might trigger a buckling in Zone (ii). These mechanisms, in conjunction with the lateral force induced by the diagonal floor truss, promoted movement of the diagonal member U10-L9, which tore the gusset plate off and transferred the vertical compression to the vertical truss member U10-L10. The latter was a redundant component in the design and, therefore, had a much smaller section area and moment of inertia. It buckled immediately due to the high down-pulling load imposed on the gusset by the diagonal member U10-L11. This led to a complete failure of node U10, followed by the fractures of the gusset plates L11 and U11, and, finally, the subsequent collapse of the bridge.

This scenario is evident by the findings of National Transportation Safety Board (NTSB) (2007b,c, 2008a,b,c,d). As demonstrated in Fig. 10, bowing in the aforementioned zone (i) caused by bending moment was noticeable by this 2003 photograph. In Fig. 11, the destroyed pieces of gusset plate U10 reveal the necking along the horizontal direction near the bottom of Zone (iii) and the tension-induced fracture in the upper-right portion of the image, or the ligaments between the rivets. Fig. 12 shows the wreckage of the entire bridge that was reassembled by the National Transportation Safety Board (NTSB) (2008d). Almost all of the vertical trusses attached to nodes U9, U10, and U11 were severely damaged within the triangular area demonstrated in Figs. 8 and 9.

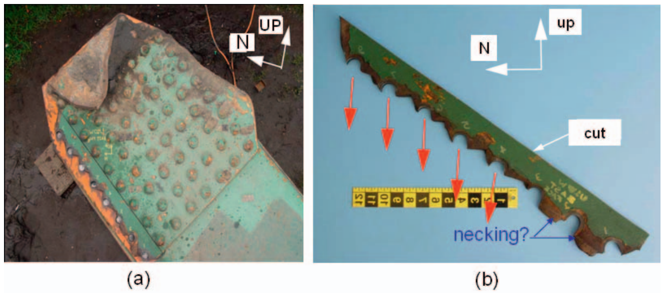

Fig. 11. (Color) Fractography of gusset plate U10 [National Transportation Safety Board (NTSB) 2007c]: (a) the diagonal truss member U10-L9 after it tore off the gusset; (b) a cut from the gusset, where the red arrows indicate the direction of tension, which shows the downward-pulling and horizontal tension-induced necking; where "N" refers to north

Why the Bridge Sustained for 40 Years

The results plotted in Fig. 5 suggest that under the original dead- and design-lane loads, the maximum principal stress in the diagonal member near gusset plate U10 was about half of the yield strength of the steel. On the other hand, elastic stress concentrations in the attached gusset plate could be more than double that of the stresses in truss members. These facts imply that the undersized gussets could have been partially yielded, locally buckled, or both, when the bridge was at the designed service load long before the collapse. The bowing of the gusset in Fig. 10 is evidence of this possibility.

However, the steel used to build the bridge allowed 10% plastic strain at the ultimate strength of 86 ksi, which was about 70% higher than its yield limit (50.5 ksi) in the design. This additional safety margin provided by the ultimate strength, in conjunction with the steel's ductility, held the bridge until the time that the accumulated damages caused by environmental factors, additional weight, and increased traffics induced material fatigue and nibbled the margin away.

Lessons Learned

1. The load path of the I-35W's shallow-arch deck-truss structure determines that gusset plates within a distance to the nearby pier, between one-sixth and one-third of the total center span length (like U10 in this bridge), are the pivots that transfer deck load and weight to supporting piers. This load path results in the force flow with high amplitude in the diagonal members attached to these gussets. On the other hand, according to the conventional one-dimensional influence line model, the amplitude of bending moment in this area is zero or very small due to its transition from positive to negative sign. This coincidence may have led to an undersized design of the structural components within this area. This was also the likely cause for the undersized gusset plates in the I-35W bridge.
2. The inadequate gusset plates' thickness, in conjunction with the inadequacy in the particular design of the upper node that connected the floor truss and main frame, essentially removed the local redundancy of each truss cell (see Fig. 8) in this bridge and caused high localized stresses in these gussets.
3. In design specifications of current bridges, the fatigue limit is determined by the amplitude of cycling stress corresponding to live loads. For long-span bridges or deck-truss bridges, such as I-35W, the dead load causes a high mean stress. When the mean stress is close to the material's yield point, the fatigue life could be significantly reduced although live load-induced cycling stresses may be very low.
4. When the situations listed above meet concurrently, while multiple heavy trucks or heavy construction loads are present simultaneously on each lane of a deck-truss bridge, the probability of another incident such as the failure of I-35W could be higher than expected.
5. Thus, for major and complex bridges, an independent quality assurance inspection is highly recommended to assure the design and load rating adequately address the above items.
6. The undersized design in I-35W seems to be part of the efforts to reduce weight and costs, as there was a lack of the means for accurate calculation at the time. Although several concurrent factors triggered the bridge's fall, this tragedy confirms again the intrinsic and close correlations between technological developments, economic engineering applications, safety, and security. It also draws attention to the potential risks in our infrastructure systems (Reid 2008).

The writer emphasizes that these analyses and conclusions solely reflect his professional opinion.

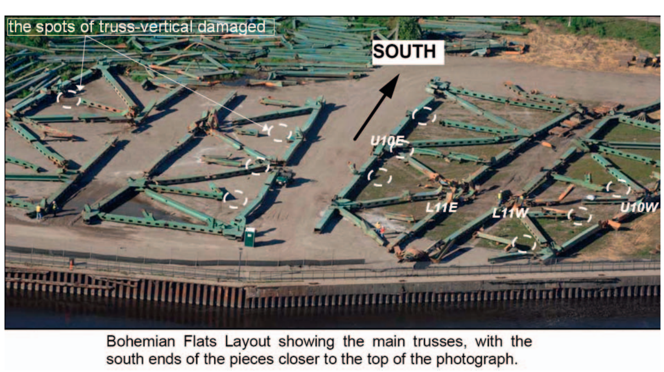

Fig. 12. (Color) This NTSB photograph [National Transportation Safety Board (NTSB) 2008a,b,c,d] shows that many vertical trusses were severely damaged within the triangle area defined by Fig. 8

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