SUBMISSION OF PROGRESSIVE CONTRACTORS, INC.
TO THE NATIONAL TRANSPORTATION SAFETY BOARD

COLLAPSE OF THE I-35W HIGHWAY BRIDGE
MINNEAPOLIS, MINNESOTA
AUGUST 1, 2007

October 1, 2008

Accident Synopsis

On the evening of August 1, 2007, Progressive Contractors, Inc., (PCI) was performing work on the I-35W bridge (Bridge 9340) over the Mississippi River in Minneapolis, Minnesota, when the bridge fell into the river at approximately 6:05 PM. PCI was under contract with MnDOT to make various repairs and upgrades to the bridge. At that moment, PCI's bridge crew was preparing to perform their eighth Low Slump Concrete Overlay pass on Bridge 9340. Whereas, it was common for PCI to have sixty workers on the Bridge at any given time, on this very warm evening, MnDOT directed PCI to suspend work until 7:00 PM to allow for the deck overlay concrete to be paved in the relative cool of the evening and early morning hours. As a result, PCI had sixteen workers on the bridge at the time of the collapse. Tragically, one PCI employee, Greg Jolstad, was fatally injured that evening while the others, along with one MnDOT worker suffered injuries in varying degrees. It has been well documented that several members of the traveling public, also, died and were injured on that tragic night.

The chaotic collapse scene was incredibly well managed by local, state, and federal agencies along with the generous and heroic assistance delivered by private entities and private individuals. The aftermath has had a devastating and unsettling effect upon many families and businesses. Not the least of which has been visited upon the families and the business operations of PCI. PCI had operated as a highway-heavy contractor, performing work, primarily, in the public sector since 1971. Publicly bid contracts require bid and performance bonds. Recently, our bonding company discontinued its support of PCI's work program and effectively has made it impossible for PCI to continue its business.

The investigation into the cause of the collapse began immediately. The NTSB, in its investigation has raised complex issues related to engineering, human performance, operations, material science and other system-wide safety considerations. Under unrelenting pressure to disclose their findings, areas of initial and primary focus inevitably emerged. Unfortunately, for PCI, the two, apparently equivalent, contributing factors most often mentioned were the existence of 16 under-sized gussets and PCI's construction loads. When pressed for information, the NTSB and the State of Minnesota spokespersons, time and time again, linked these two factors in the same breath, raising them to an elevated and equivalent status. The media have echoed these talking points relentlessly. The cloud created by these constant assertions was the expressed reason given by PCI's bonding company for their decision to cease their support of this 37 year-old, otherwise healthy company.

Construction Loads

PCI's construction loads in question consisted of 577,000 pounds of equipment and material (as calculated by the NTSB) that were on the bridge at the time of the collapse. According to every calculation made available to us (NTSB, MnDOT, RR&J, etc.), this weight, present at the time of the collapse, was found to have been below the design load capacity of the bridge. In fact, the combined weight as confirmed and measured by the NTSB comprised only 40% of the bridge's design live load capacity on the main trusses. These calculations take into account the fact that PCI had removed 670,000 pounds of concrete as part of their repair work under this Contract. Four of the eight lanes of traffic were shut down at the time of the collapse, reducing the live load on the bridge. Furthermore, the loads represented by PCI's net construction footprint was determined to have been less than the future design load that the bridge was intended to carry at the conclusion of the 2007 project when the bridge would have been re-opened to all lanes.

To place the impact of these loads into another perspective, PCI's activities actually reduced stress on the bridge, in most areas, by taking off two inches of concrete overlay as a part of their repair contract. Even in the areas of greatest concentration, the construction load produced only 1.2 ksi and 2 ksi shear stresses at the U10E and U10W locations, which represents only 5% to 7% of the total stress at these locations. By way of comparison, the addition of the permanent loads attributed to the major modifications to the bridge in 1977 and 1998 contributed shear stresses of 4.3 ksi at the U10E & U10W nodes, over twice the contribution of the construction loads at the points of maximum concentration.

Construction Loads Complied With All Regulations and Specifications

As to the existence of any controlling regulations or contract specifications concerning the placement of construction of loads on bridges, MnDOT produced none. The NTSB considered AASHTO Bridge Design Specification, Seventeenth Edition Division II Section 8.15.3(Construction Loads) and AASHTO Load Resistance Design Bridge Design Specification 4th Edition Section 3 as, perhaps, governing. A hypothetical scenario was given to MnDOT by the NTSB; i.e., What if the contractor (PCI in this case) had asked permission to place this load on Bridge 9340, applying the, aforementioned AASHTO test, what would your answer have been? MnDOT's analysis concluded that the operating rating factor for the main members was above 1.0, thus, by code, the steel bridge members could have safely carried the load.

The fact is no Contract Special Provision, MnDOT Standard Specification, or any AASHTO regulation specifically directs contractors' activities such as described in

the NTSB hypothetical. Past practices, on previous pours on Bridge 9340 suggest that the placement of construction materials was acceptable and the placement of materials in preparation for this specific overlay pass were, as is every aspect of projects such as this, performed under the absolute control and discretion of MnDOT personnel.

MnDOT Control of Work

The Specifications clearly confer upon the Engineer and his representatives, the Project engineer and construction inspectors, the absolute authority to control and direct every aspect of contracted work. The role of the contractor is to construct the work as described in the Plans and Specifications at the Engineer's direction. Responsibilities, authority and duties of parties signatory to MnDOT Contracts, like this one to perform repairs on Bridge 9340, can fairly be described by the following specifications excerpted from the MnDOT Standard Specifications for Construction 2005 Edition:

Scope of Work
1401
Intent of Contract

The intent of the Contract is to provide for construction of the Project and compensation for the work, according to the Contract documents. The titles and headings of the various sections and subsections of the Contract are intended for convenience of reference.

The Contractor shall construct and complete the Project in every detail as described in the Plans, Specifications, Special Provisions, and supplemental drawings.

Control of Work
1501
Authority of the Engineer

1501.1 DECIDING QUESTIONS

The Engineer will decide all questions regarding:

(a) Quality and acceptability of materials furnished and work performed.
(b) Manner of performance and rate of progress of the work.
(c) Interpretation of the Plans, Specifications, and Special Provisions.
(d) Measurement, control of quantities, and the amount of any deductions or adjustments to be made in payment.
(e) Acceptable fulfillment of all Contract provisions on the part of the Contractor.

1501.2 SUSPENDING WORK

The Engineer may suspend the work, either wholly or in part, due to failure of the Contractor to:

(a) Correct conditions unsafe for the workmen or the general public,
(b) Carry out provisions of the Contract, or
(c) Carry out orders
(d) Comply with the requirements of all permits for the project.

The Engineer may also suspend work for such periods as deems necessary due to unsuitable weather, for conditions considered unsuitable for prosecution of the work, or for any other conditions or reasons deemed to be in the public interest.

1509
Authority and Duties of the Project Engineer

The Project Engineer is the Engineer with;

(1) Immediate charge of the engineering details of the construction Project.
(2) Responsibility for the administration and satisfactory completion of the Project.
(3) Authority commensurate with the duties delegated to the Engineer,
(4) Authority to reject defective material and to suspend any work that is being improperly performed.

1510
Authority and Duties of the Inspector

Inspectors employed by the Department will be authorized to inspect all work done and materials furnished. The inspectors will not be authorized to alter or waive the provisions of the Contract, to issue instructions contrary to the Contract, or to act for the Contractor.

As a representative of the Engineer, the inspector will report progress and acceptability of the work being performed, and will call to the attention of the Contractor any failures and infringements on the part of the Contractor. Should any dispute arise as to the materials or work performance, the inspector may reject materials and suspend operations until the question at issue can be referred to and be decided by the Engineer.

MnDOT controls their Projects related to their infrastructure from design to demolition. MnDOT establishes program funding and priorities. MnDOT advertises contracts for bids, administrates the contracts and supervises and controls the construction process. MnDOT specifies construction methods, project sequencing, materials and wages.

MnDOT hired Sverdrup & Parcel to design Bridge 9340. MnDOT reviewed and approved the design. MnDOT hired contractors to build the bridge and supervised and inspected its construction. MnDOT maintenance crews performed work on-going during the life of the bridge. MnDOT hired contractors to perform small repairs and major modifications over the years. MnDOT also hired consultants to assist in the management of their responsibilities. MnDOT bridge safety inspectors performed inspections and produced inspection reports. These inspection reports became the foundation for all recommendations and repairs related to MnDOT's obligation to maintain the safe operating condition of the bridge.

Minnesota State Legislative Joint Committee Investigation – Gray, Plant, Mooty Report

In December 2007, a Minnesota Legislative Joint Committee retained the law firm of Gray Plant Mooty to serve as its Special Counsel in the investigation of the I-35W bridge collapse. After conducting interviews and gathering materials the Firm produced its Report to Joint Committee to Investigate the I-35W Bridge Collapse in May of 2008. The report addressed the history, design, inspections, assessments, repairs and collapse of Bridge 9340. The report can be found among the NTSB's docket materials and is especially insightful regarding its assessment of MnDOT's bridge inspection program and its recommendations for improvement.
The Report was especially concerned about the oral culture or the general lack of documentation and follow-up that they perceived to exist as a systemic problem in the inspection process. For example, in the 1982 bridge inspection report a bridge inspector's comment was recorded in the report regarding the roller bearing @ pier 6; "no indications the bearings are moving". The 1984 report acknowledges the failure of a downspout to redirect water from the bearings; "The West bearing is

being continuously bathed with salt water from the plugged drain above - the effects are plain to see." Yet, during the 22 year period from 1984 through 2006 observations of "moderate corrosion", "loss of section", and "clogged drains" came and went with no quantification of the problem or evidence of any implementation of repairs having been undertaken.

Similarly, in the 1993 Bridge Inspection Report it was noted that "the downstream truss @ L11 inside gusset plate has loss of section 18" long up to 3/16" deep (original thickness ½ inch)." In future inspection reports, this condition was never again quantified. Rather, the subsequent reports noted section loss without any quantification of the extent of the loss. Since the collapse, the extent of the corrosion actually measured in the debris has been variously quantified to be in a range from 5% to 25%. The Bridge Inspection Manual characterized 5-10% section loss as significant. In the case of section loss greater than 10% of a primary steel bridge element, the MnDOT Bridge Inspection Manual states that the load-carrying capacity of the member has been significantly reduced and that a structural analysis or immediate repairs may be required. Bear in mind that this structure's superstructure was rated at an NBI rating of 4 for the past 17 years, and, MnDOT's Bridge Inspection Manual describes conditions associated with a superstructure rating of 4 as follows: superstructure has advanced deterioration, members may be significantly bent or misaligned, connection failure may be imminent, and bearing may be severely restricted.

One critical significance of the corrosion inspection observation is that if the shear stresses had been reviewed for the two most common nominal sections of the ½ inch gusset plates, using the member forces shown on the original bridge design drawings, it would have been immediately obvious that the gussets were grossly under capacity and the bridge would have been closed immediately.

When repairs were suggested from inspectors' observations in the inspection reports no follow-up is documented. For example, the 2000 Inspection Report listed three Immediate Maintenance Recommendations. These three were repeated in '01, '02, '03, '04, '05, and '06. There is no evidence that any maintenance measures were undertaken, nor, does there appear to have been any policy in place to connect Immediate Maintenance Recommendations to action items. A report authored by the Minnesota State Office of the Legislative Auditor (February 2008) concluded, "*MnDOT should establish standard procedures for documenting, communicating, and following-up on bridge inspectors' maintenance recommendations".*

The Gray Plant Mooty Report (pg 50) also recounts an incident where the Metro District Bridge Safety Inspection Engineer describes an observation of bowed gusset plates on the West truss. The observation was neither documented, nor,

quantified. As a result an opportunity to have prevented the collapse of the bridge was missed because the condition was not evaluated or monitored.

These issues surrounding the Bridge Inspection program relate directly to the NTSB's discovery and subsequent assertions as to the undersized ½ inch gussets being a contributory factor in the collapse. Apparently, as reported in the NTSB's Draft Factual Report, the Safety Board investigators determined through interviews with experts in the bridge industry that the design methodology for gusset plates is normally very conservative, with the result being that a gusset plate is generally assumed to be stronger than the beams it connects. **This may be the salient point of the investigation.**

Gusset Plates – Design Error

One analysis that we believe clearly demonstrates the gross inadequacy of the gussets at the U10 and L11 nodes on the I-35W bridge is presented in the FHA Turner-Fairbanks Highway Research Center Report; *Adequacy of the U10 & L11 Gusset Plate Designs for the Minnesota Bridge No. 9340 (I-35W over the Mississippi River).*

Simply stated and clearly shown in figures 5, 6, 7 and 8, properly designed gusset plates do not have a Demand/Capacity *ratio (D/C)* that exceeds 1. Yet, the gusset plates at U10 and L11 consistently failed the D/C ratio checks that Turner-Fairbanks conducted by values indicating failure of the gussets, not just 5 or 10% which is commonly acceptable. "The capacity inadequacies were considerable for all conditions investigated with the plate providing approximately one-half of the resistance required by the design loadings." It must be emphasized that these values (for ½ inch gussets), **twice normal or allowable**, would reduce to below normal or allowable rations had a 1 inch gusset been used. Additionally, it should be noted that these checks did not take into consideration the additional stresses placed on these plates with the 4,100,000 pounds of added dead load from the two major modifications, nor, were the deleterious effects of the observed corrosion, buckling and the over length unsupported edge (an additional design error) condition present at the U10 nodes, each of which would further diminish the load carrying capacity of pristine gussets as checked.

Figures 5 through 8 (below) 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.

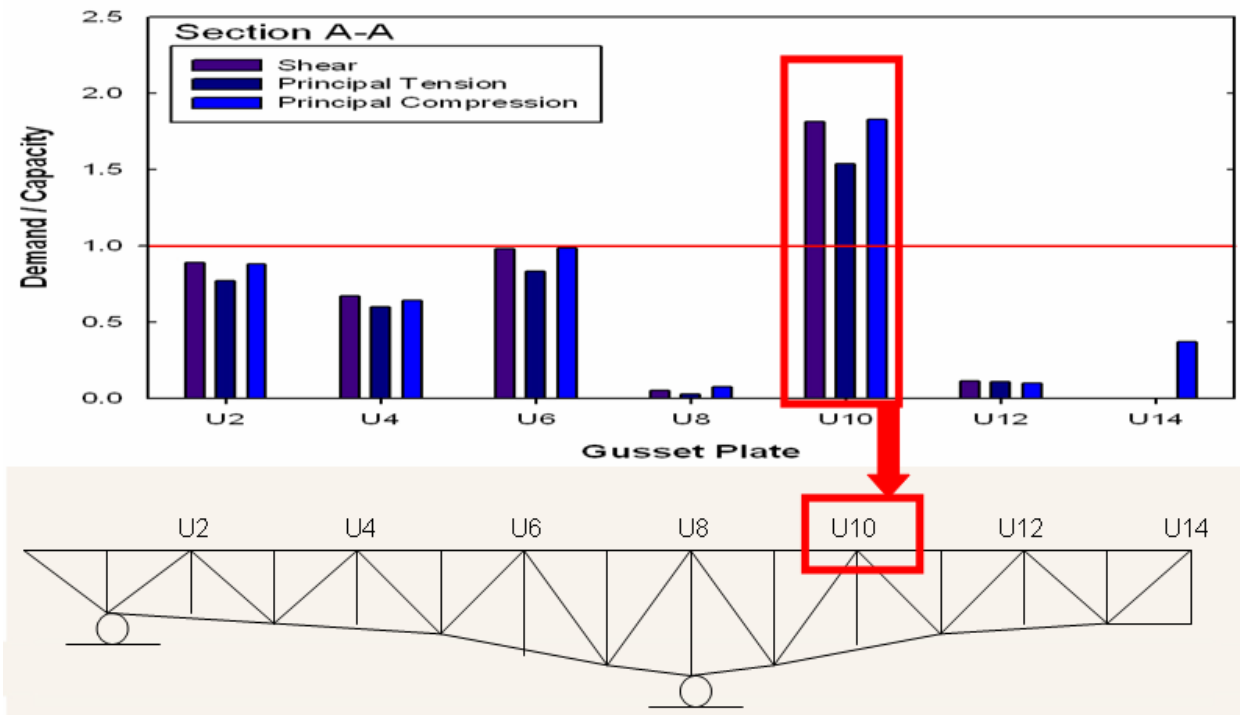

Figure 5. Demand to Capacity Ratio for Section A–A of Upper Nodes

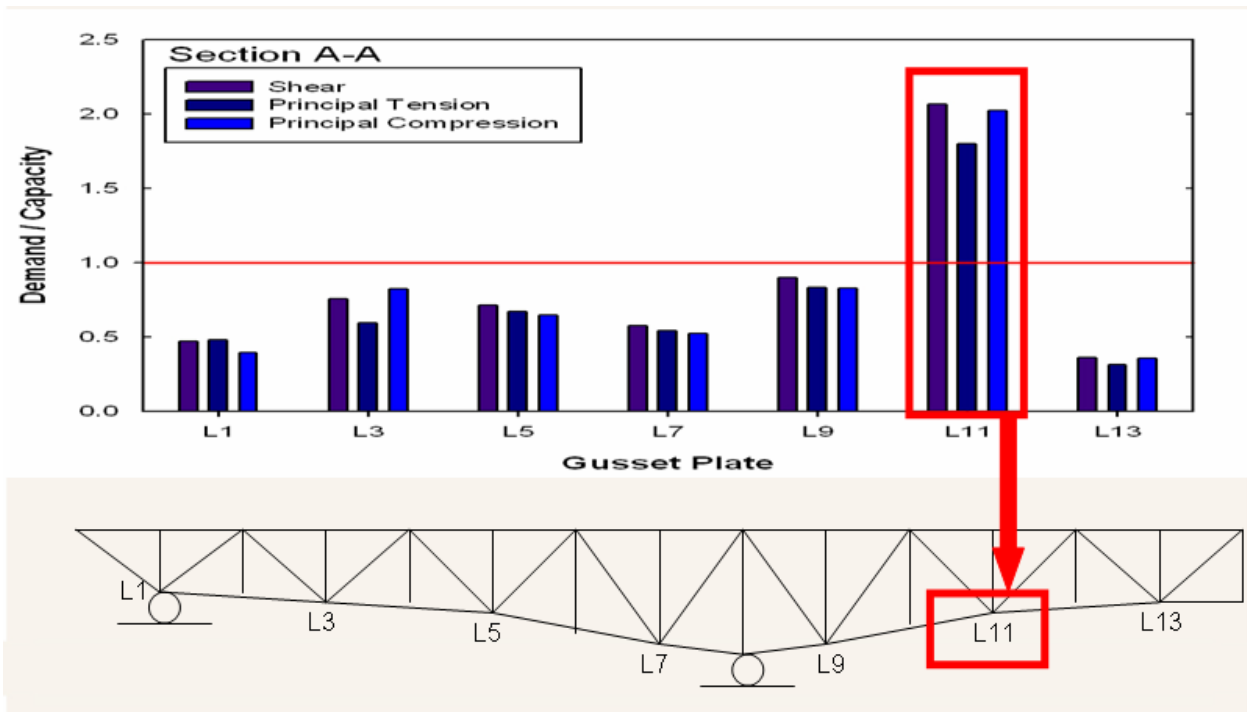

Figure 6. Demand to Capacity Ratio for Section A–A of Lower Nodes

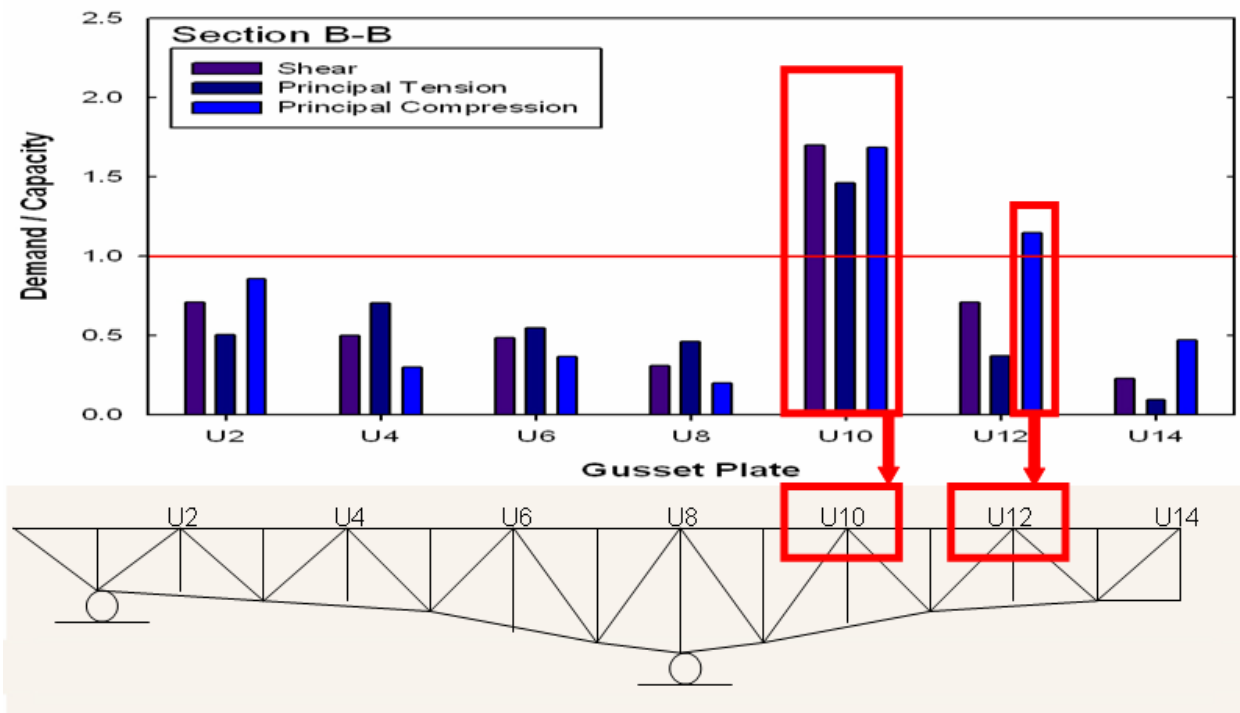

Figure 7. Demand to Capacity Ratio for Section B–B of Upper Nodes.

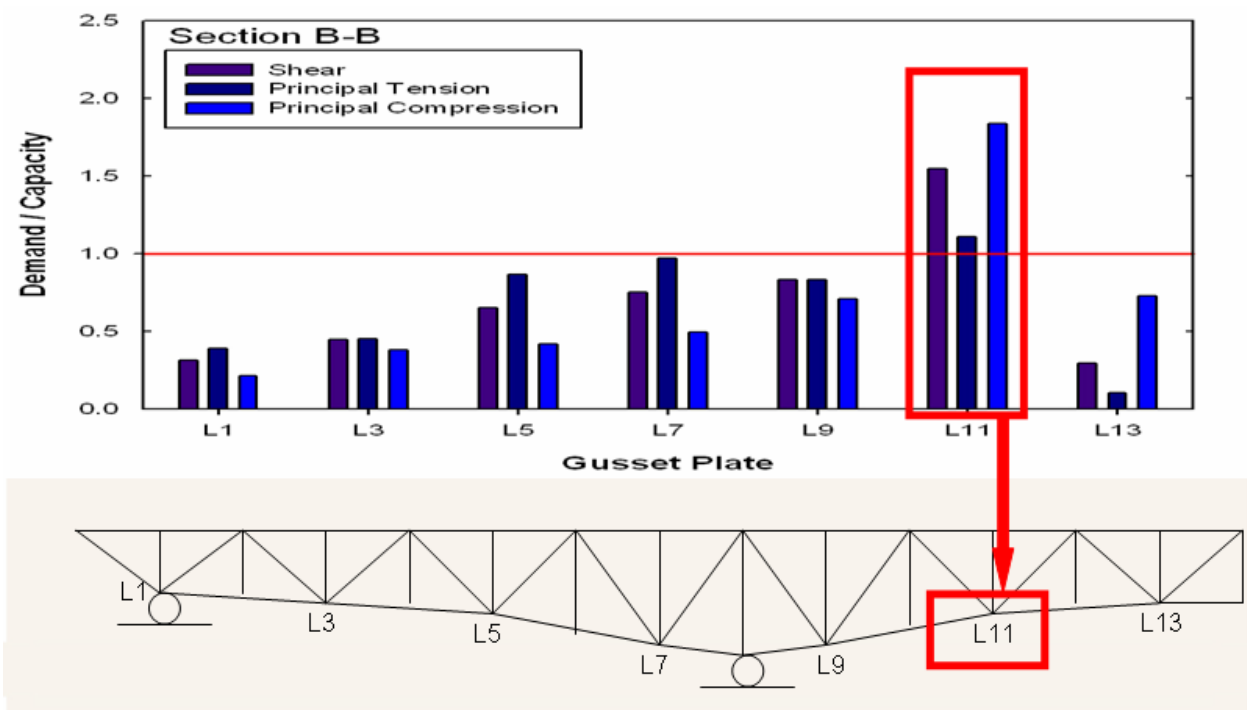

Figure 8. Demand to Capacity Ratio for Section B–B of Lower Nodes.

Another example of the gross inadequacy of the sixteen ½ inch gussets is presented in *Turner-Fairbanks Finite Element Modeling,* specifically Table 37. This table on Page 112 compares the increase in the ultimate buckling capacity of the ½ inch gussets on Bridge 9340 and properly designed 1 inch gussets. The Dead Load column reflects the fact that a properly designed 1 inch plate would have increased by ten-fold the ultimate capacity of these plates. That is, increasing the margin of safety from 7% as designed with the ½ inch gussets to greater than 10 times the margin of safety, if a properly designed gusset was called for by the designer.

These findings, as stated in the NTSB's Draft Factual Report on page 92;

The 16 gusset plates at U10(') and L11(') east and west were ½-inch thick. All these gusset plates failed the D/C ratio checks by significant amounts. Also, the length of unsupported and unstiffened-edge at U10(') was 30 inches, which exceeded the allowable maximum of 24 inches(48 x ½ inch).

triggered the Safety Board, on January 15, 2008 to issue the following safety recommendation to the FHWA:

H-08-01

For all non-load path-redundant steel truss bridges within the National Bridge Inventory, require that bridge owners conduct load capacity calculations to verify that the stress levels in all structural elements, including gusset plants, remain within applicable requirements whenever planned modifications or operational changes may significantly increase stresses.

Another examination of the inadequacy of the gussets found @ U10 and L11 is provided by the engineering firm of Raths, Raths and Johnson, Inc. (RRJ). RR&J, an engineering firm retained by PCI, analyzed the gusset plate shear stresses, comparing side by side the ½ inch gussets and the properly designed 1 inch gussets. They incorporated into this comparison, in addition to the bridge self-weight; the added components of the design live-load, the weight of the additional dead-load created by the two major modifications in 1977 and 1998, the loads present at the time of the collapse, and the weight removed by previous constructions activities. They also take into consideration the effect of the observed pre-collapse gusset buckling (bowing), as measured by the NTSB.

The attached graph (page 11), compiled by RRJ, clearly depicts the gusset plate shear stresses for the various load stages or combinations and shows the gusset plate critical buckling loads as determined by NTSB/FHWA super sophisticated non-linear finite element analysis. This graph shows that the buckling failure mode, even for a flat plate (with no bowing), is less than the steel material failure defined by overall yielding. Therefore, without the south vertical edge of the U10 gusset plates stiffened (as required by AASHTO), the factor of safety and potential failure load is less than if overall nominal or design shear yielding limit was the failure mode as represented by the 28.9 ksi line. Stated alternatively, the graph shows that

the original design provided virtually no factor of safety for the traffic loads and because of the ½ inch gusset would have been expected to fail.

Probably the most significant observation this graph facilitates is to highlight that a properly designed gusset would have sustained all of the loads present, including the 4 million pounds plus which represents the additional dead loads added by MnDOT and the construction and traffic loads at the time of the collapse, and never exceeded the ALLOWABLE stresses. When a properly designed 1 inch gusset is contrasted to the same nominal shearing stresses calculated on the underdesigned ½ inch gusset for the original design dead load, load creep and the full design Live or Traffic loads there is no factor of safety and the stresses are above the yield level of the ½ inch gusset plates.

In another exercise at a comparative analysis of ½ inch and 1 inch gussets RR&J invoked the routine design process used at the time of the I-35W bridge design. Since Sverdrup & Parcel's original gusset design calculations have not been found, RR&J utilized the method the designer used for its floor truss gussets which were available. When the gussets at the U10 nodes are checked using this design approach the magnitude of the overstress is readily apparent. When the shear stress for Design Live Load at U10 along section A is added to the Dead Load the stress equals 24.7 ksi, well above the allowable 16.5 ksi, and even approaches the yield of 28.9 ksi in shear. A comparison of the same loads applied to a 1 inch plate substituted for the gusset at U10, as reviewed by the NTSB, indicates stress for design dead load and design live load of 12.4 ksi. Well below the allowable stress. When the additional dead loads of the '77 and '98 modifications are introduced, the ½ inch gusset at U10 shear stress rise to 29.0 ksi, above the **design yield** in shear for the gusset plates. This represents a condition for which there would be no factor of safety and creates an alarming condition for the bridge.

(See graph page 11)

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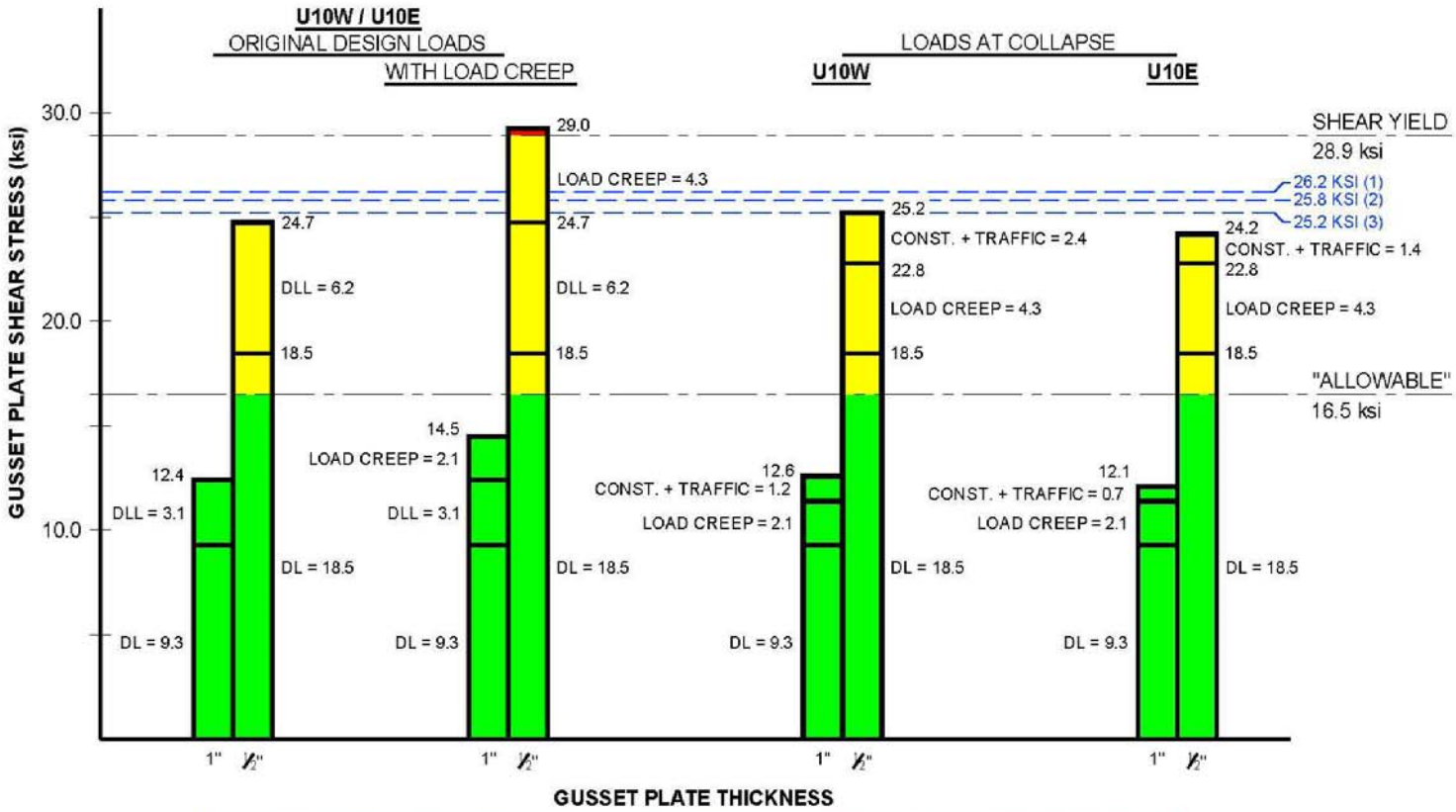
BRIDGE LOADS

DL	BRIDGE DEAD LOAD (SELF-WEIGHT)
DLL	DESIGN VEHICLE LIVE LOAD (HS20) (8 LANES OPEN)
LOAD CREEP	WEIGHT OF 2" CONCRETE TOPPING AND RAILS / MEDIAN
CONST. + TRAFFIC	WEIGHT OF CONSTRUCTION MATERIALS + EQUIPMENT - MINUS 2" TOPPING REMOVED + ESTIMATED VEHICLE TRAFFIC AT COLLAPSE



1/2" GUSSET PLATE SHEAR STRESS THAT REPRESENTS CRITICAL BUCKLING LOAD USING NTSB ANALYSIS VALUES:

(1) 26.2 KSI FOR FLAT (NOT BOWED) GUSSET PLATES.
(2) 25.8 KSI FOR 0.5" BOW AMPLITUDE IN GUSSET PLATES.
(3) 25.2 KSI FOR 0.75" BOW AMPLITUDE IN GUSSET PLATES.

Raths, Raths & Johnson, Inc.
Structural Engineers • Architects • Building Scientists
635 Midway Drive Willowbrook, Illinois 60527 630/325-6160

DATE	SCALE	CHECKED	JOB NO.
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U10 L11 GUSSET PLATE ANALYSIS

135/N BRIDGE

MINNEAPOLIS-ST.PAUL MINNESOTA

FIGURE 1

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Opportunities Missed to Discover Design Error

Sixteen under-designed gusset plates, less than one-half the size of properly sized gusset plates, were unfortunately incorporated into the design and construction of the I-35W Bridge. The design error that allowed for the incorporation of the grossly under-sized gusset plates escaped discovery in the original review process. The design error was not uncovered during the re-rating process when significant dead loads were added during the 1977 and 1998 major modifications. The design error was not uncovered or noted during forty years of State inspections. Furthermore, the design error was not uncovered after MnDOT contracted with URS in 2003 to study the bridge. This contract appears to clearly obligate URS to inspect, develop failure criteria and compute the failure forces for these connections. These tasks in their contract are outlined below.

Task 1 - Data Collection and Tabulation

1.4 Tabulate composition, attachment details and conditions for all primary and secondary members and *connections*.

Task 2 - Condition Inspection of Critical Truss Elements and Bearings

2.1 Inspect critical members, *connections* and fatigue details based on review of available data.

Task 8 - Identification of Fracture Critical Members

8.1 Develop tension and compression failure criteria for truss members and *connections* based on ultimate failure state as originally designed.

8.2 Compute tension and compression failure forces for all primary and secondary truss members and *connections* and store in database.

These tasks were apparently never completed and this opportunity to discover the design error was also missed.

Even in the wake of the eerily similar gusset failure on the Grand River Bridge in Ohio in 1996, this design flaw went undetected because of the apparent reliance on the widely-held engineering axiom that gussets are designed to be stronger than its members. According to statements made as a result of this investigation, apparently, when a bridge is rated, reviewed, inspected or evaluated, the efforts are considered to be adequate if only the truss members are carefully studied, not gussets even if they are corroded or bowed. This opinion, regarding the lack of evaluation of gusset plates by reviewing engineers, is not universally held. One insightful comment that we came upon during this ordeal concerned the observation that in the debris of a collapsed structure you can expect to find intact gussets with fractured spears of its members attached. In the wreckage of the I-35W Bridge intact members were observed with fractured undersized ½ inch gussets attached.

Grand River Bridge – Similar Gusset Failure

By way of providing background for the Grand River Bridge reference, on May 24th 1996, truss gusset plate failures occurred on the eastbound I-90 bridge over the Grand River, 30 miles east of Cleveland. The failure of these 7/16 inch truss gusset plates is strikingly similar in several respects to the failure of the underdesigned ½ inch truss gusset plates on Bridge 9430.

The FHWA analyzed the full section Grand River gusset plates using finite element analysis methods. The FHWA analysis concluded "…the design thickness of the original gusset plate was marginal, at best, and its load carrying capacity was further exacerbated by loss of thickness due to corrosion" The analysis indicated the full design load on the truss members was just about equal to the load required to buckle the gusset plates, with no safety factor. At the time of failure the gusset plates had 10% to 35% section loss due to deterioration. The section loss in the gusset plates increased the actual stresses in the remaining materials. The capacity to resist buckling in the deteriorated plates was also reduced. The estimated loads in the north truss due to dead load, construction equipment, and a legal truck was 86% of the design load. In the south truss, these loads were about 94% of the design load. Although these are estimated applied loads, it appears they were approaching the full design load.

In addition to the obvious problem of underdesigned gusset plates on the Grand River bridge, other similarities with the I-35W bridge include: corroded gusset plates, frozen expansion bearings, high temperatures, and substantial temperature gradients due to sun/shade exposure reaching peak at 6:00pm (particularly during hot August temperature recordings).

The FHWA failed to widely distribute the lessons learned at the Grand River bridge. The FHWA did not issue any Directive, Policy Memorandum, Technical Advisory, Order, or Notice with regard to the problems uncovered at the Grand River Bridge. As a result of this failure, an opportunity for the uncovering of this similar design flaw and the other exacerbating similarities may have been was missed.

Initiation of the Collapse at U10 Nodes

The summary found in the *NTSB's Materials Laboratory Sequencing Study Report No. 08-032 - Draft issued July 1, 2008* concludes, basically, that whereas, physical evidence, analytical methods, etc., are consistent with the theory that the collapse initiated with the failure of the gusset plates at U10 East and West. However, the evidence is insufficient to establish which of these nodes had the initial failure. RRJ, in the following summary of their analysis offers the following six points that lead them to believe that of these two nodes, U10E failed first:

1. The physical evidence points to the collapse initiating on the east side truss, perhaps at U10E, based on the impact and scrape marks on the bottom of the L7-L8E and L7-L8W bottom chord members that show the L8E bearing was "ahead" or north of L8W bearing during the entire sequence of the collapse, resulting in L7/L8E member failing, the east truss then falling off of Pier 6, and both trusses rolling over to the east.

2. A 3D analysis of the bridge structure under loads at collapse shows that with the end of main truss Members L9-U10E and U9-U10E disconnected at U10E, simulating failure of the gusset plates at U10E, the axial compression force in Member U9-L9E increases to greater than 1500 kips, far exceeding the design force in this member of 331 kips compression, and enough to cause the buckling and complete end connection failures described in this NTSB report. The analysis also shows the corresponding axial force in Member U9-L9W is less than 100 kips under this scenario, and this member remained relatively straight and connected to both U9W and L9W nodes of the west main truss.

3. The pre-existing bow in the U10 gusset plates as captured by the photographs taken in 1999 and 2003 and reported in the NTSB Draft Factual Highway Accident Report (August 1, 2008), Table 6, show a definite larger bow amplitude in the U10E gusset plates than the U10W gusset plates. The FHWA Report "Finite Element Modeling of I-35W Bridge Collapse" (August 2008) indicates the gusset plates analysis results show a significant reduction in critical buckling load with an increase in the bow amplitude. Greater bowing at U10E coupled with slightly lower member forces may support the initiation of the collapse at the east side regardless of the asymmetric traffic and construction loading to the west.

4. A careful examination of the gouge marks on the brass hemispherical lower bearing surfaces of the L8 East and West supports at pier 6 show much larger gouges on the West bearing indicating larger reactions on the west than the east at the time of collapse. This could be due to the large transfer of load from the east as it failed first and shifted the loading to the west increasing, momentarily the force on the bearing and causing the much larger gouging.

5. The damage on the east connections appears to be much greater and more destructive that the west connections at U10 and L11, indicating that the connections on the east literally exploded as one would expect as the failure initiates there first and then shifts to the west.

6. The debris, particularly the main truss components south of U10 was displaced to the east with the west truss laying over almost intact on top of the east truss, with the top chord of the west truss laying to the east across Pier 6. This suggests that the east truss failed first and proceeded to fall, pulling the west truss over to the east on top of it.

By way of qualification this submission is based on the information presently available to us. We based our findings primarily on information obtained from the NTSB public docket, information provided to us by the NTSB through our participation in the party process and research done by RRJ, a forensic engineering firm. We also obtained information from the MnDOT website which has thousands of documents available electronically. We have had very limited access to the actual wreckage materials, which have been under the control of NTSB and MnDOT, and thus have not had the opportunity to perform a more complete and thorough examination.

Conclusions

The stresses present at U10 nodes which can be attributed to the 2007 construction activities, are both quantifiable and minimal. On their very first day, the under-designed gusset plates likely encountered stresses at the U10 nodes that exceeded their design allowable stress. On their last day, the gusset plates at the U10 nodes were under stresses beyond which failure is initiated.

The stresses developed by the loads present at the time of the collapse acting on properly proportioned 1 inch gusset plates would fall far short of the design allowable stress and in no way would these stresses be expected to cause the gussets to fail or the bridge to collapse.

The inevitable failure of the underdesigned previously buckled gusset plates located at the U10 nodes, which went undetected for 40 years, placed PCI, unwittingly, on a path to almost certain disaster. The fact is, if the sixteen gussets present on Aug. 1, 2007, had not been grossly underdesigned, the bridge would not have collapsed under the reasonable loads present on the bridge on that day.