The Second Research Report

Submitted To:
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Why the Gusset Plates of I-35W Bridge Are "Undersized"?

The Potential Risk in Today and A Proposed "Bridge Safety Monitoring System" *

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* This report is based on the NTSB's finding released at Jan. 15, 2008 for the ongoing investigation of bridge 9340 Collapse (Aug. 1st, 2007), the FHWA internal report dated at Jan. 12, 2008, and the analysis performed on the author's research report of 9340 collapse submitted to NTSB at Sept. 2007.

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INTRODUCTION

The NTSB safety recommendation H-08-1[1] of the I-35W accident indicates that the "gusset plates U10, U10' of the bridge are undersized", "However, because the detailed calculations (of original design) … could not be located,…, the Safety Board has not yet determined whether the error was due to a calculation mistake, a drafting error, or some other error in the design process".

In order to assist the ongoing NTSB investigation, this report provides

(1) a possible reason to cause undersized gusset plate's size in original design
(2) a simple analysis of the subsequent risk in fatigue failure
(3) introduction of a proposed "hybrid monitoring system" for the safety operation of the bridges with undersized components or undetected flaws.

1. WHY THE GUSSET PLATES ARE UNDERSIZED?

This report concludes that the undersized gusset plate U10 is a part of original design based on the "consistency" explored between the plate thickness and the force distribution in horizontal truss members; by which the effects of diagonal truss force on the gusset plate were underestimated. Also, it seems that the gusset plates at "T" joints U7,U9 are also undersized.

Fig.1 is the gusset's plate thickness of the I-35W bridge. The FHWA's analysis [2] confirms the U10 plate undersized; **whereby a question is raised: why is the plates' thickness varying from 0.5 to 1.375 inches**?

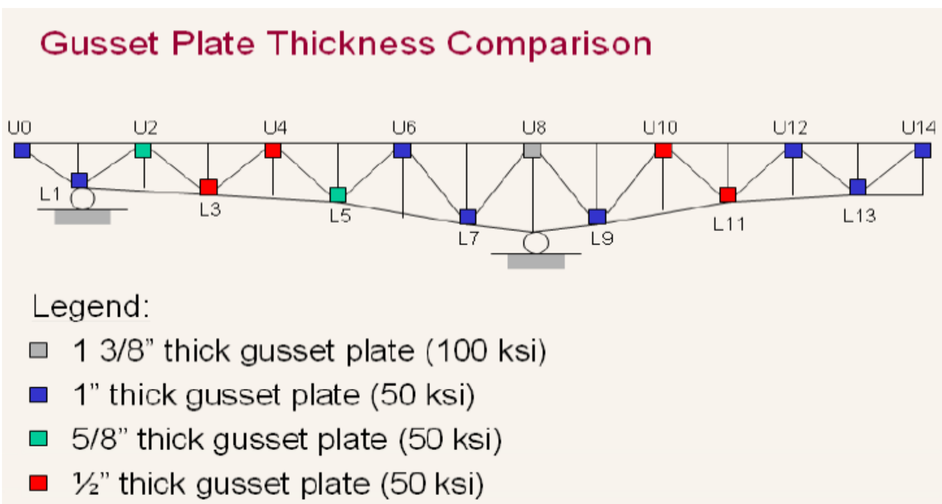


Fig. 1 Distribution of the gusset plates' thicknesses (figure 9 of FHWA internal report [2])

To answer the question, plotted in Figs.2 and 3 are the comparisons between the actual gusset thicknesses and the distributions of computed truss forces in upper- and lower-chards[4]; by which one sees the same trend of two curves except the end nodes (U0,L1). The half inch gusset plates (U4, U10, L3, L11) are at or close to the locations with the lowest truss force.

To explain these coincidence, a statement has been given in the third paragraph, section 5 of the report [4] of last September: "*Due to the limited computation capability in past, it seems that the original design and subsequent early investigations could only treat the 9340 bridge as a truss-assembled structure, which led to the focus onto the forces and damage conditions in truss members….*". Obviously, the truss force in upper and lower level chords had been used as the governing parameters for the gusset plate design, which led to the "consistency" demonstrated in Figs. 2 and 3. However, the corresponding stress concentration in the gusset plate did not obtain sufficient consideration.

On other hand, it seems that the effect of diagonal truss force on gusset plate was ignored in the design. Fig. 4 is the same plot as Fig. 2 but with the force distribution in diagonal truss members: while the truss in upper horizontal chords approaches to its low valley near U10, the diagonal truss force reaches its highest peak. This peak did being taken into account in original design with focusing on the truss member; this is because, according to the original blue print listed in Appendix I, the diagonal truss L9-U10 has relatively larger section area.

It should also be noticed that in the scanned blueprint (Appendix I) all gusset plates on the "T" joint, e.g. U7 and U9, have the same half inched thickness which is about one third of the thickness of the nearby node U8. These two nodes are also the spots with highest risk in fatigue failure and fracture.

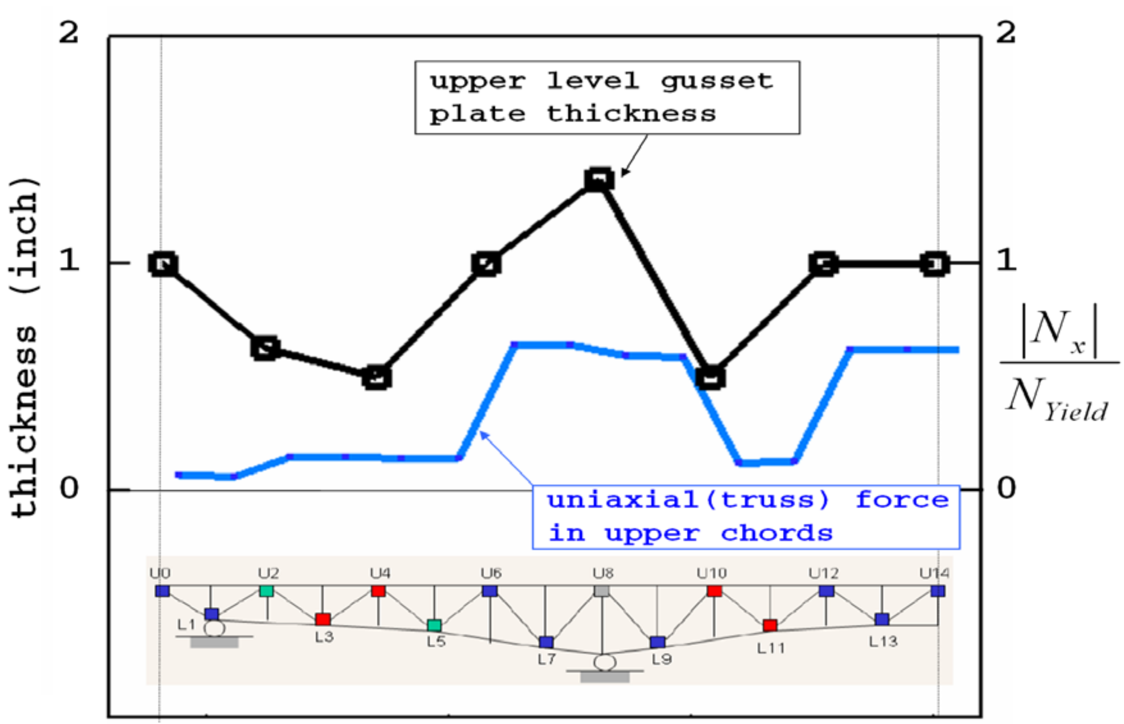


Fig. 2 Comparison between the upper level gusset plate thickness and the truss force distribution in upper chords; the latter is picked up from the Fig. 8a of [4]

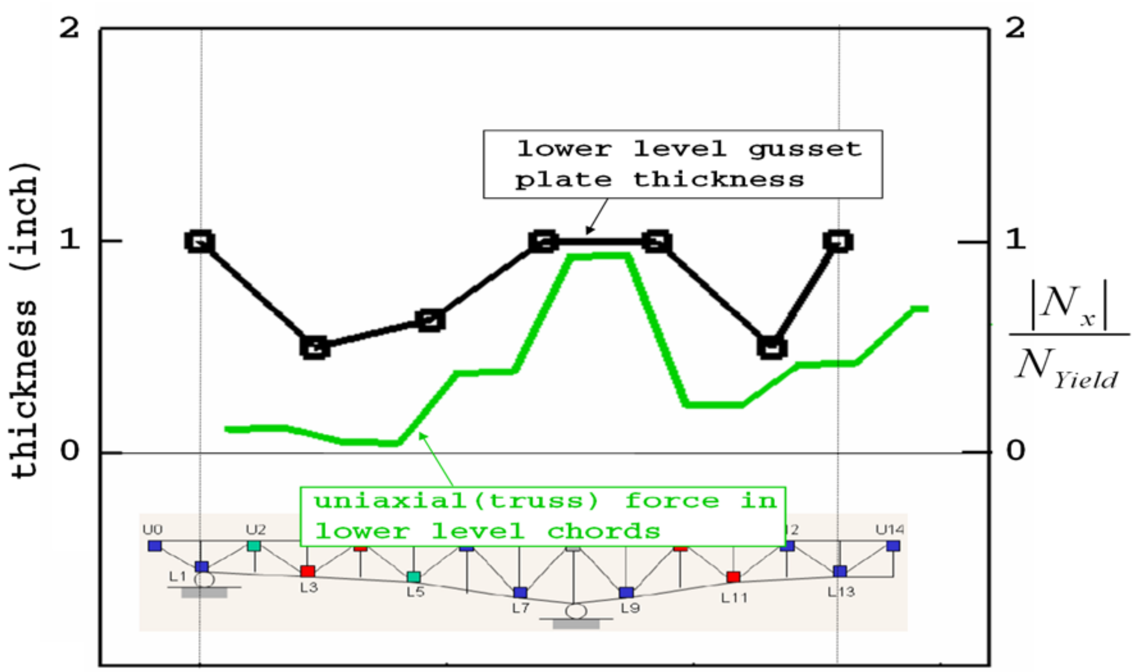


Fig. 3 Comparison between the lower level gusset plate thickness and the truss force distribution in lower chords; the latter is picked up from the Fig. 8a of [4]

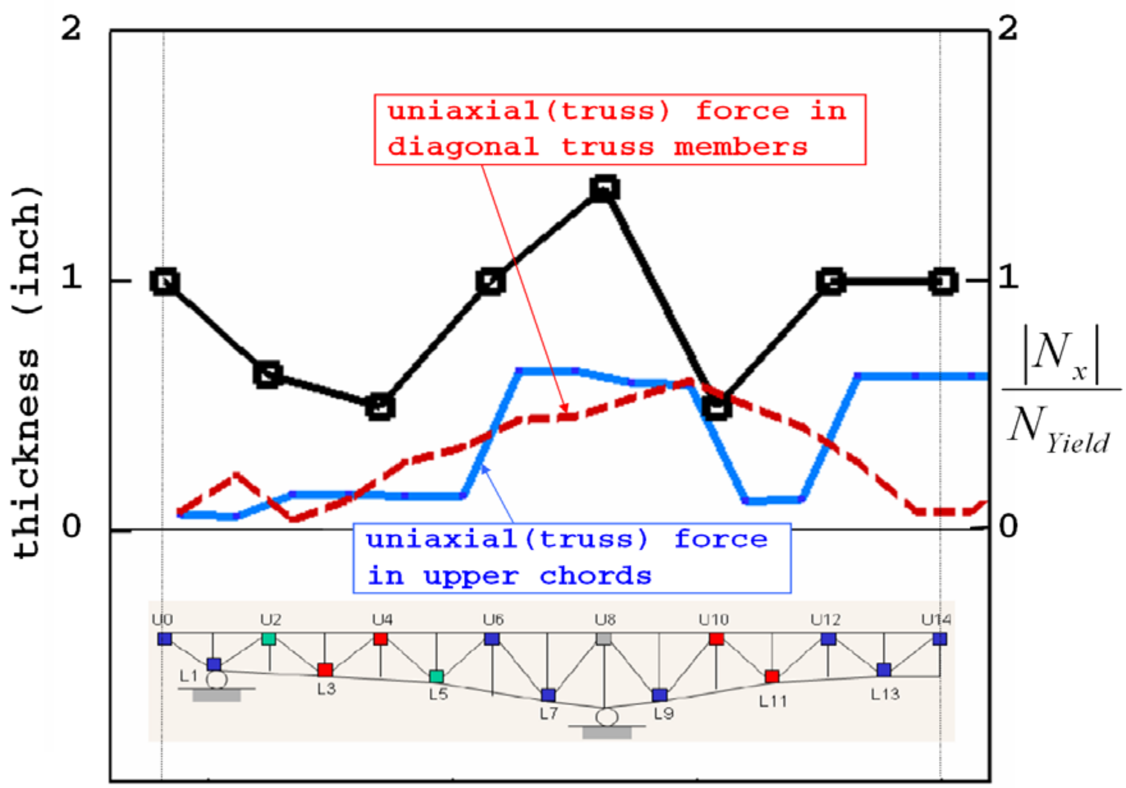


Fig. 4 Comparison between the upper level gusset plate thickness and the diagonal truss force distribution.

2. WHAT IS THE SUBSEQUENT RISK?

An undersized plate thickness, in conjunction with the bending moment that is usually ignored in truss-based design, will elevate the amplitude of stress concentration remarkably with subsequent localized material's yielding, resulted in shorter fatigue life than expected. This effect becomes more significant particularly when thermal-induced stress presents while, e.g., roller bearing of a bridge is locked.

In order to investigate the effects of stress concentration, Fig. 5a is a 3D computer (finite element) model of the node U10 and Fig. 5b is that of U9. They are created according blueprints [3] listed in Appendix. However, since the distribution of the living-load on the bridge deck when it collapsed and the material's constants are not available, in finite element computation the boundary conditions applied to the horizontal, diagonal and vertical truss members are picked up from [2] whereas that to the lateral bracing and floor truss members are referring to the results obtain in [4]. All materials obey linear elastic law.

2.1 Bending Moment

There are at least two kinds of bending moments effectively acting upon a gusset plate: the first one is that caused by the layout of adjoined trusses members in the plate and the differences in directions and amplitudes of the associated truss forces; the second is that in a beam with tension in one side and compression in another side.

The FHWA internal report [2] has confirmed that the stress relevant to the first kind of moment is still within safety margin in U10 gusset plate but the shear stress and principle stresses surpass the plate's capacity.

For a bridge like I-35W, in a steel beams the stress caused by the above-mentioned secondary bending moment is generally very small as compared with (uniaxial) truss stress; so it usually is omitted through simplifying a beam structure into a truss structure. This is widely applied successful approximation which has been proven accurate enough for load-capacity calculation.

However, from the viewpoints of fatigue life assessment and structural integrity analysis, the consideration will be somewhat different. This is because, at a joint like gusset plate, such a bending moment may cause a stress concentration with the same or higher amplitude as that caused by truss force. It becomes more significant especially when the thermal-induced stress presents. Plotted in Fig. 6 are the distributions of the above-mentioned secondary bending moment in the upper chords under different conditions including the cases that the bridge's all roller bearings are locked while temperature changes. Accordingly, Fig. 7 is a comparison of the stress distributions at the U10 gusset plate between the case that the node is only under (uniaxial) truss force and that with the above-mentioned secondary bending moment. In the latter, the stress caused by bending moment in beams is 30% higher than the case without bending whereas that is about 70% higher in the gusset plate.

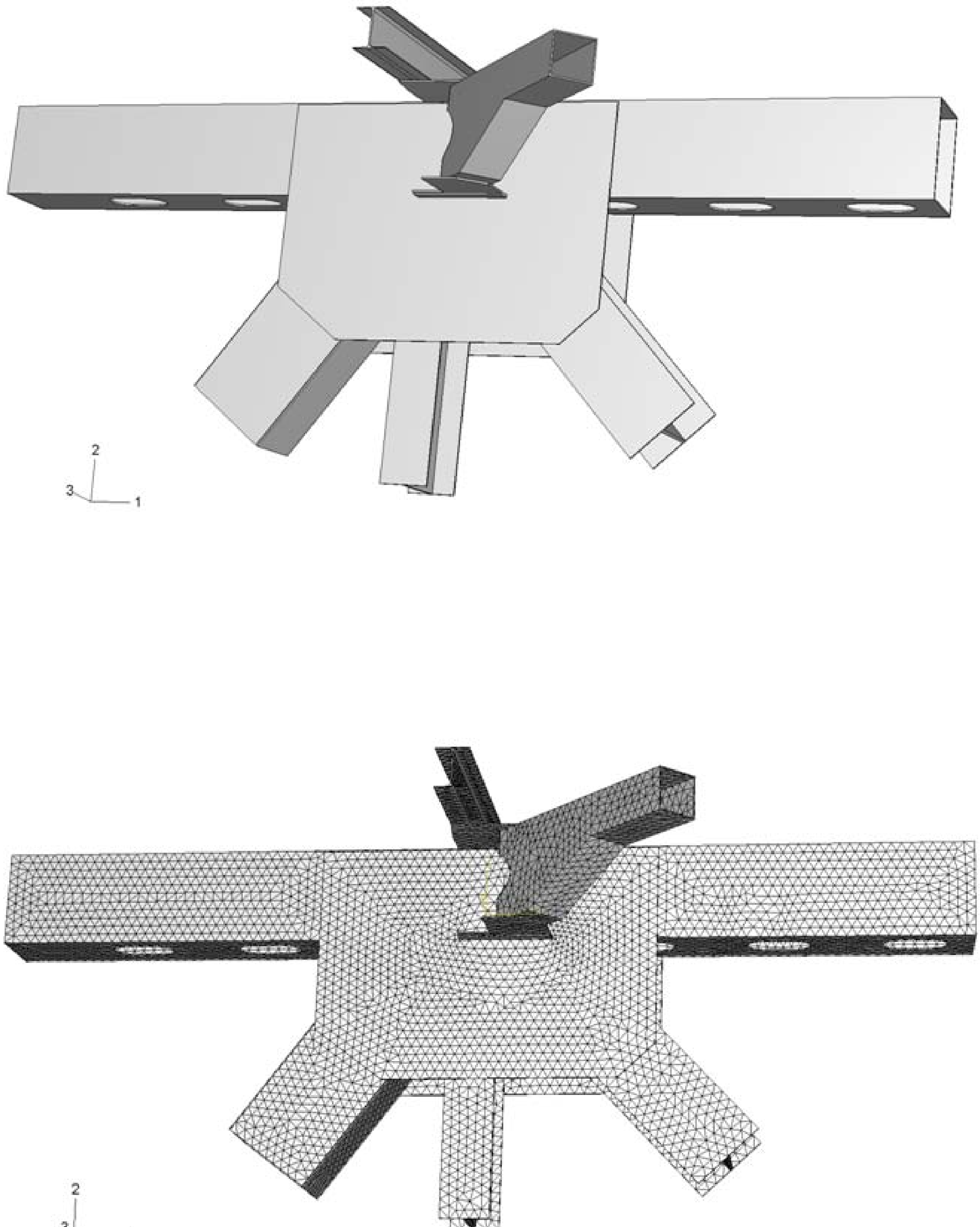


Fig. 5a Computer model of the node U10(west) according the blueprint (see Appendix)

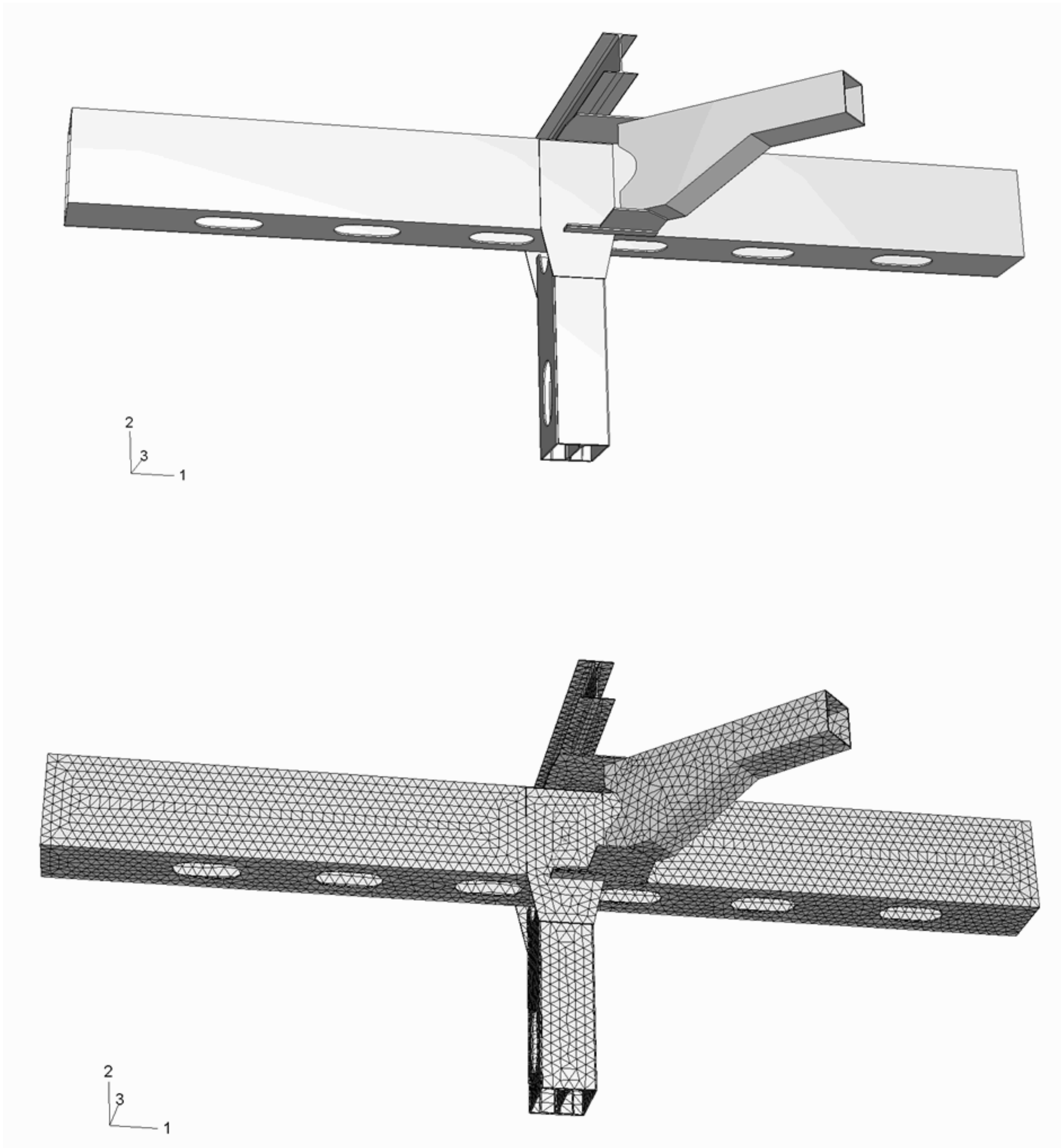


Fig. 5b Computer model of the node U9 according the blueprint (see Appendix)

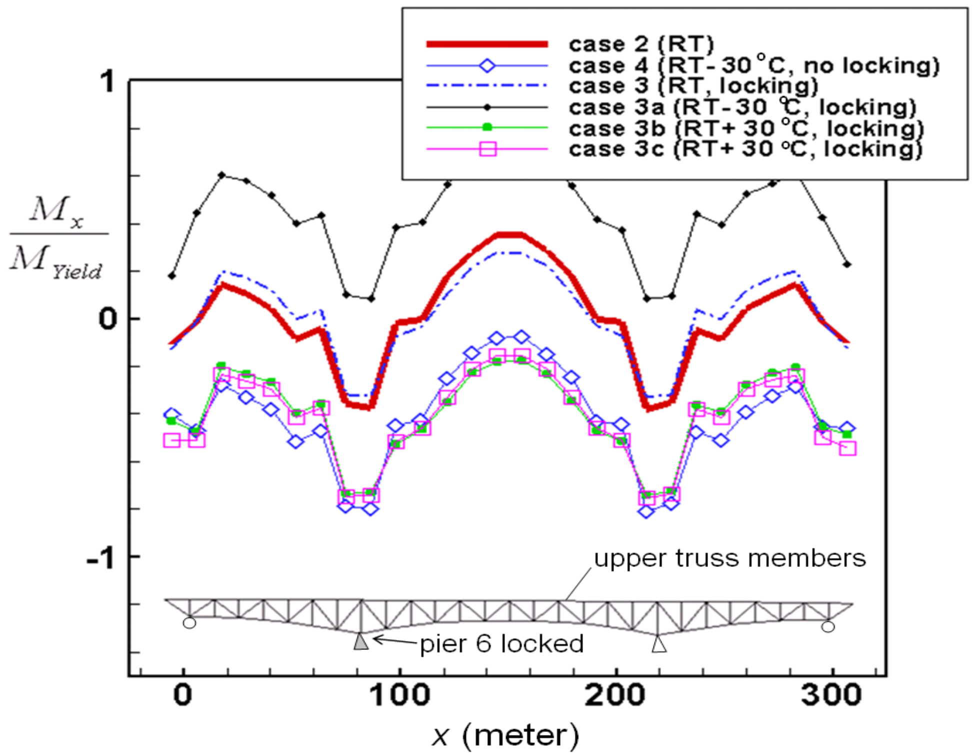


Fig. 6: The distributions of bending moment in upper truss members for the cases with locked-roller bearing or temperature changes or both; where “RT” stands for room temperature and “locking” means the lock of roller bearing at pier 6; picked up from the Fig.9a in [4]

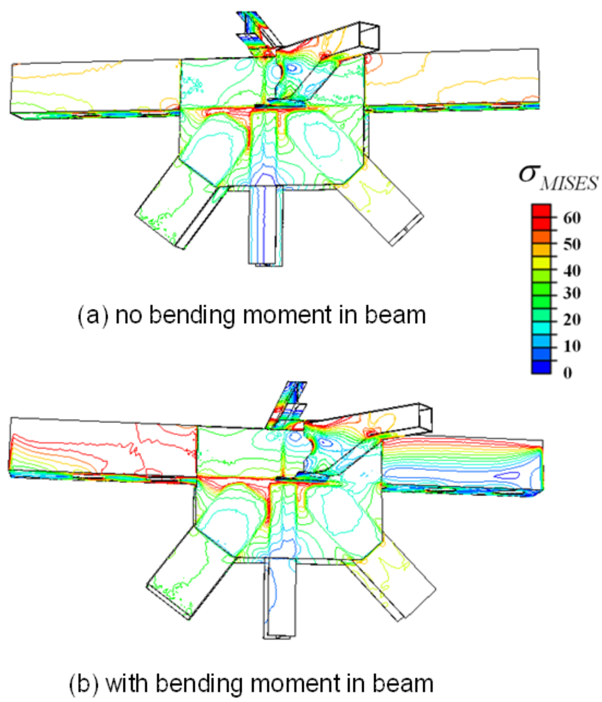
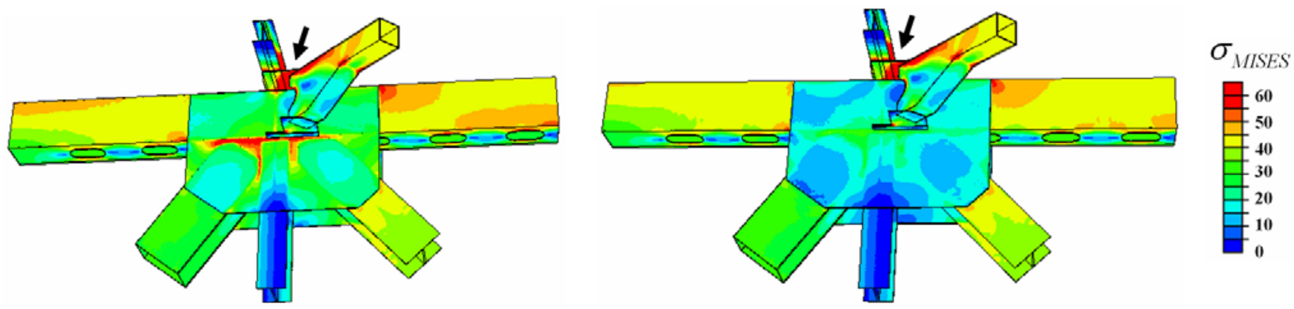
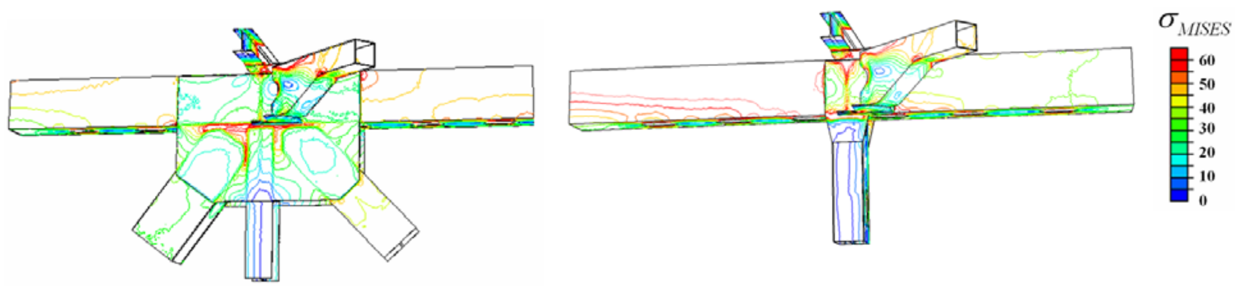


Fig. 7 A comparison of the computed effective stress (Von Mises stress) with and without bend moment in horizontal truss members



(a) U10(west) with 0.5' gusset plate   (a) U10(west) with 1' gusset plate

Fig.8: A comparison of the effective stress for the node U10 with 0.5' and 1' gusset plates, respectively; the 1atter significantly reduced the level of peak stress. However, as pointed by the black arrow, remarkable stress concentration can be seen in the plate connecting lateral bracing



(a) U10 (U10')   (b) U9 (U9)

Fig. 9 A comparison of the effective stress (Von Mises stress) on U10 and U9 joints (no bending moment); both have the same half inch plate

2.2 Stress Concentration and Fatigue Life

Fig. 8 is a comparison of the computed effective stress in the U10 node with 0.5' and 1' gusset plates. Unfortunately, the effect of the stress concentration-induce fatigue life reduction was not taken into account in the previous fatigue analysis of the bridge. As mentioned previously, the half inch gusset plates were also used for the upper level main truss members' T joints. Fig. 9 is a comparison of the stress distributions on U10 and U9 joints; the stress concentration on the later seems higher than the former. To obtain more accurate results of stress distribution during the collapse, load distribution on the bridge deck and computer model including floor truss structure will be needed.

Now we discuss the effect of stress concentration on structural safety. It is well-known that the capacity of a steel structure against applied load is characterized by its "limit load", by which the average stress in one or more structural section(s) reaches the material's yielding strength so large scale plastic deformation occurs. A stress concentration, regardless the sources, usually has relatively less effect on a structural limit load. For example, the beams and gusset plates used in bridges are usually made of middle-low strength steels; by which the enhanced ductility can effectively smear out a stress concentration through localized plastic deformation. This might be a reason that the I-35W bridge stood for more than 4 decades with half-sized gusset plates and additional deck weight.

On other hand, stress concentration and localized plastic strain have strong effects on material's fatigue failure. The difference between structural failure and fatigue induced fracture can be explained by the example of thin-wall cylinder vessel test illustrated in Fig. 10. It is well-known that in the cylinder wall the amplitude of hoop stress σ_h is twice higher than the longitude stress σ_l. However, around the circumferential notch with one third depth of the wall thickness, the average longitude stress is about $0.75\sigma_h$ but near the notch tip a stress concentration exists with higher peak stress. A structural failure of the cylinder takes place when a monotonic loading applies until its maximum average stress, i.e. the hoop stress, reaches yielding strength regardless the stress concentration elsewhere. On contrast, a fatigue crack initiation and growth present at the notch tip due to the stress concentration when the vessel is under repeated load at low amplitude.

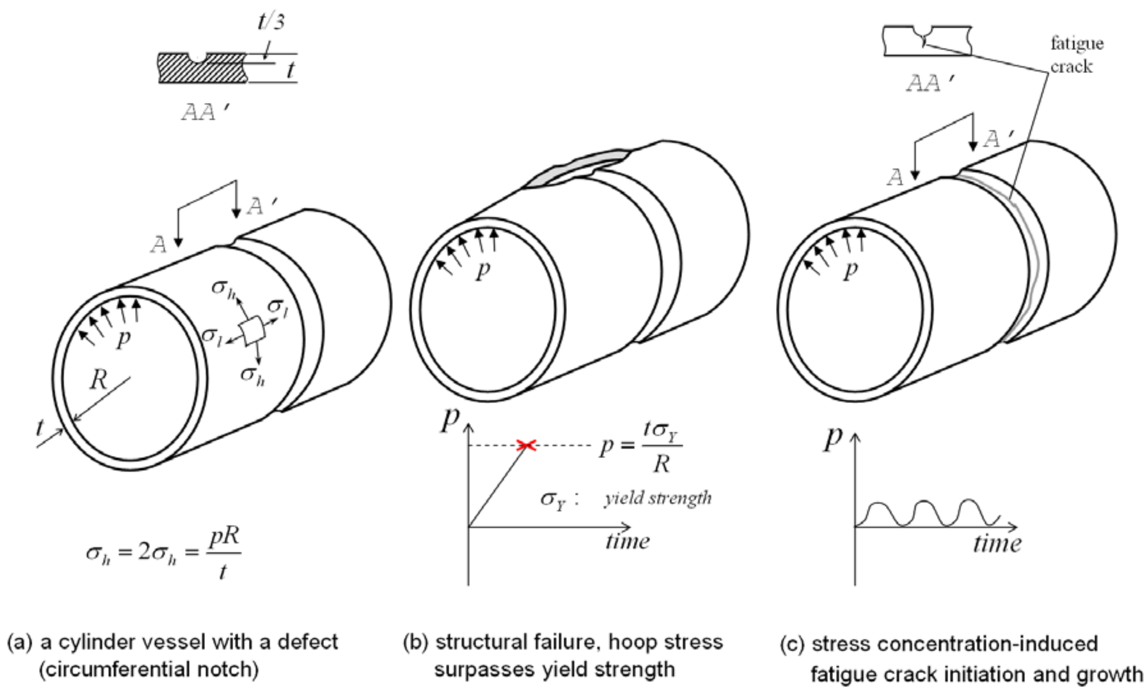


Fig. 10: (a) an example to explain the two failure models in a defected cylinder vessel under internal pressure: (b) a structural failure governed by the maximum average stress regardless stress concentration; (c) a fatigue failure governed by stress concentration-induced damage accumulation that results in crack initiation and growth.

The both failure modes illustrated in the cylinder vessel test of Fig. 10 occurred during the 9340 bridge's collapse. By examining the photographs of the collapse site released to publics

[3], among many structural-failure characterized wrecked debris one can find the traces of fatigue crack propagations, which are these old and rusted surface parts surrounded by the shining fresh fractured surfaces, as pointed by the arrows in Figs. 11a and 11b.

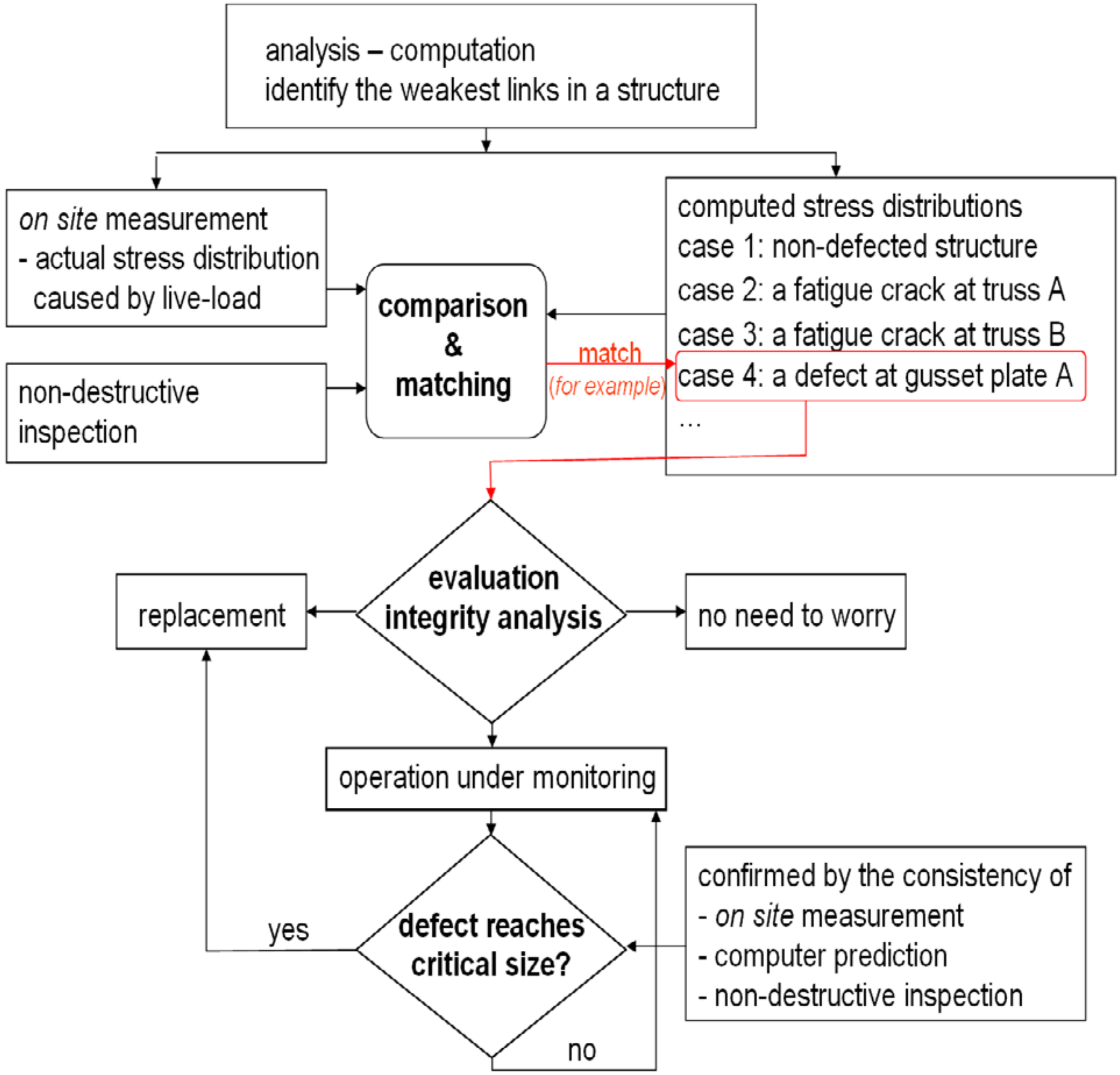
Fig. 11 Fractographies of wrecked gusset plates of I-35W bridge; these photos were made at the day after the collapse (downloaded from www.dot.state.mn.us/i35wbridge/photos/ground/aug2/ at 3/5/08). The rusted old surface parts surrounded by fresh fractured surfaces, as pointed by the white arrows, look like fatigue cracks. Some of them are in welded joints.

3. A Hybrid Living-Monitor System for Safe Operation of Bridges with Defects

Since there are more than hundred of highway bridges national wide with the similar design as I-35W, a "hybrid bridge living-monitor system" has been proposed in [4]; which, essentially, is a procedure that combines computer simulation, *in situ* (on site) measurement, nondestructive inspection, and structural integrity analysis. This procedure enables to provide a clear picture of the "health" condition of a bridge, so as to ensure continuing safe operation. The introduced methodology can also be considered as a new technology to improve the accuracy of bridge inspection and nondestructive detection.

It is no doubt that current applied inspection systems and safety monitor procedures are efficient and successful. However, in some cases, due to limitation of available equipments, risks exist in missing or unable to obtain certain crucial information of some developed defects in a structure. To minimize the probabilistic for these kinds of odds, the central of the proposed procedure is to use computation as an extension of measurement; the difference and consistency between thorough computation and careful measurement at key-locations provide the key to understand structural "health condition" and to identify possible defect.

This concept is briefed by the flow chart I, which can be divided into two phases: the phase one is to identify the weakest link of a structure and to find possible defects; the second phase is the safety operation of a defected structure under monitoring. Through "matched and consistent" information obtained from different methods, it enables to perform fatigue and fracture mechanics-based quantitative structural integrity analysis and to make further decision for operation.



Flow Chart I: The concept of the proposed “hybrid monitoring system”

This “matching” procedure can be explained by a sample example - the three-truss structure illustrated in Fig. 12, where the trusses *A* and *B* are neither visible nor detectable but the dead load P_{dead} and live load P_{live} are known. Theoretical analysis or accurate computation can predict the stress level on the truss *C* when either *A* or *B* is broken or both of them are normal. By comparing the computation and measured results at *C*, the situations of *A* and *B* are under control.

Plotted in Fig. 13 are the computations of the stress distribution for gusset plate U10 between the case of no defect and the case that fatigue cracks cut 12 rivals between the plate and the horizontal chord U9U10. The significant difference in stress distributions on the outside surface implies the feasibility to detect and monitor inside fatigue crack growth through the proposed methodology. Also, by measuring to the deviation of strain increment from linear monotonic loading of live-load in measured strain, it is possible to estimate the stress caused by dead load and the level of plastic strain.

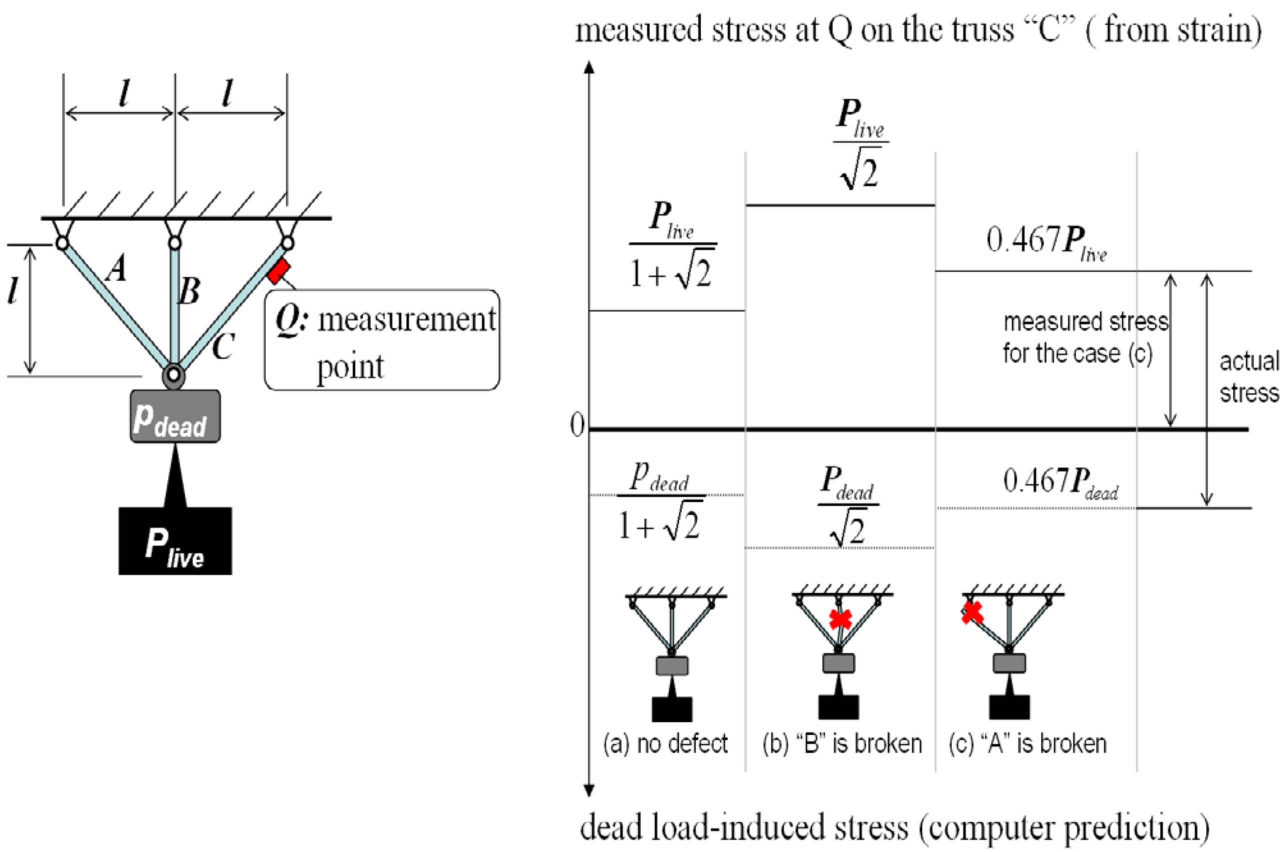


Fig. 12 An example of the proposed “hybrid monitor system” – the three-truss structure on the left where the trusses A and B are neither visible nor detectable. However, as plotted on right, the information about the damage in A or B can be obtained by comparing the measured result at truss *C* and the computed stress

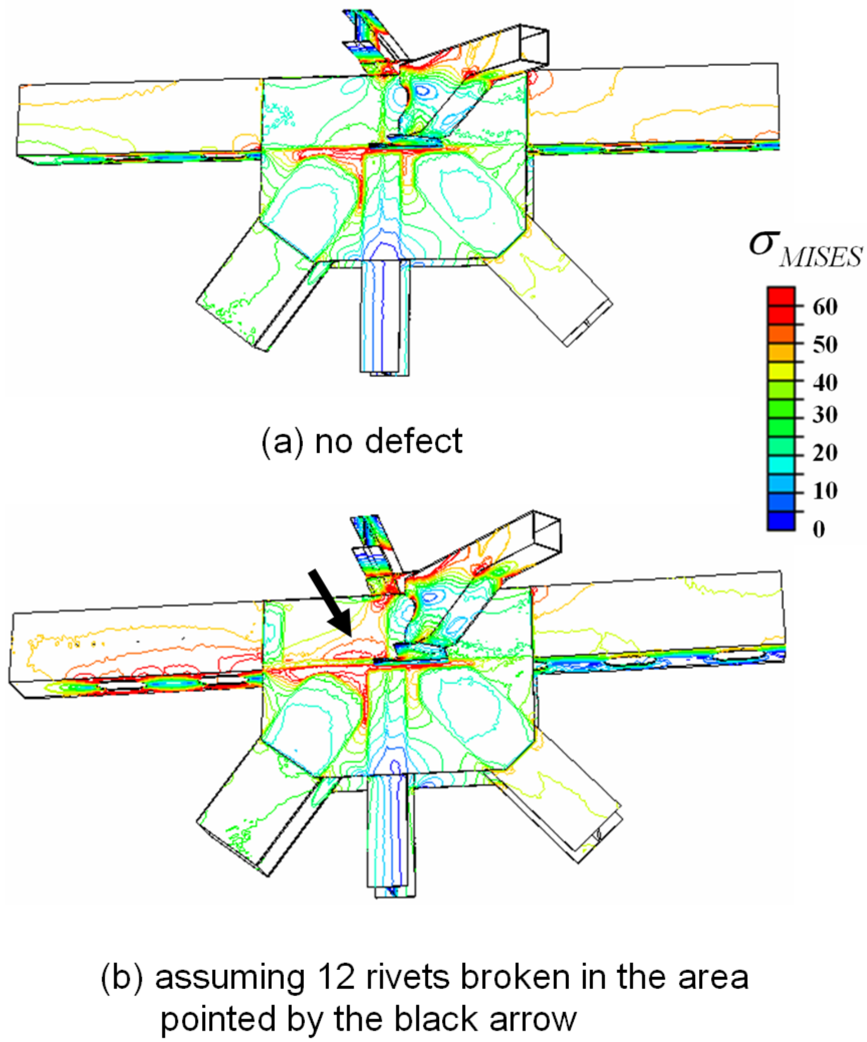


Figure 13: Another example of the proposed "hybrid monitor system" – a comparison of the stress distribution on the outer surface of gusset plate between defect-free and the case with 12 rivets broken

4. Summary

(1)The consistency between the gusset plate thicknesses, according to NTSB's finding, and the horizontal truss force distributions implies that the latter had been used as a governing parameter in the original design; this is because all half-inched plates were at or closed to the locations with the lowest horizontal force. However, the design obviously did not give sufficient attention to the effect of diagonal truss force – which reached its peak value near U10 gusset plate.

(2) The blueprints of the bridge show that the gusset plates at all "T" joints of upper chords are also with half inch thickness. The computer simulation indicates that the stress concentration on the U9(U9') plate is higher than that on U10(U10') plate.

(3) Beside truss force, the computer simulation indicates that bending moment in truss members can cause higher stress concentration around gusset plates; particularly when a bridge supporting bearing locked and thermal induced stress becomes significant.

(4) Although these stress concentrations may not have immediate effect on the load-capacity of the bridge; the resulted localized plastic strain can significantly accelerate damages evolution that leads to reduced fatigue life.

(5) A safety-monitoring system is proposed for a defected structure, whereby the key issues are (i) knowing first the weakest link of the structure through thorough computation; (ii) utilizing advance numerical simulation as an extension of on-site measurement and nondestructive inspection; (iii) using the “consistency” and “matching” between measurement and computation to identify the evolution of defect.

References

[1] National Transportation Safety Board, Safety Recommendation (H08-1), Jan. 15, 2008

[2] Reggie Holt & Joseph Hartmann, “Adequacy of the U10 & L11 Gusset Plate Designs for the Minnesota Bridge No. 9340” FHWA, Turner-Fairbank highway research center report, Jan. 12, 2008

[3] www.dot.state.mn.us/35wbridge/index.html

[4] Su Hao, “A Preliminary Analysis of the Bridge 9340 Collapse at August 1st, 2007 and Some Suggestions”, ACII, A research report submitted to NTSB, Sept. 22, 2007