Load Testing and Evaluation of Aluminum Sign Trusses in the State of Iowa

FINAL REPORT

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Abstract

The Iowa Department of Transportation uses four chord aluminum tubular truss structures to support Dynamic Message Signs (DMS). Inspections of these aluminum trusses have revealed that about half of these structures have cracks. Nearly all of these cracks are located at the internal diagonal-to-main chord fillet welded connections. The internal diagonal-to-main chord members are those which connect opposite upper and lower chords and maintain the geometry (i.e., they keep the truss square) of the truss. For highly indeterminate structures it is often possible for a structure to carry the design load even when one or more members fail in the structure. At the time of inception of this research program, the ability of the cracked aluminum trusses to redistribute load around a fractured member was unknown. Large scale destructive tests and frame analyses were conducted to establish the ability of the trusses to carry the design load in various faulted states. In the summer of 2007 the Iowa DOT shipped two aluminum trusses, one 70 ft long and the other 55 ft long, to Purdue University for destructive testing. Both the 70 ft truss and the 55 ft truss were subjected to a simulated design wind load and a simulated DMS dead load.

Under the full simulated loading, each respective truss has two main chords that resist nominal axial tension and two main chords that resist nominal axial compression. This research program focused on the tension chords. The 70 ft truss did support the simulated design loads when one of the two main tension chords (5 inch O.D.) and all of the internal diagonals (2 inch O.D.) were completely severed. The 70 ft truss failed after both tension chords and all of the internal diagonals were severed. The 55 ft truss did support the simulated design loads when one of the two tension chords was severed. The 55 ft truss did support the simulated DMS and wind load when both tension chords were severed and retrofitted. The retrofits were removed and then the truss was tested again. The 55 ft truss with both tension chords severed (and no retrofit) failed to support the design load. In light of the 55 ft truss experimental test results, the authors concluded that the retrofit concept is appropriate and effective for use in practice. Analytic frame analysis was conducted for both trusses. The experimental test results and analysis appear to indicate that the loss of all internal diagonals has a negligible effect on the structures' ability to carry the design load. It also appears that the loss of two of the four main chords, whether they are in nominal tension or compression, will most likely inhibit the truss capacity to support the full design wind load.

Several crack surfaces were exposed and examined. In addition, several connections where no cracks were observed were also sectioned and polished. No evidence of fatigue cracking was observed and all fracture surfaces appeared to be brittle fracture. It is believed that cracks were due to significant constraint that developed during fabrication.

1.0 Summary

Destructive testing of a 70 ft and 55 ft aluminum four chord truss was conducted at Purdue University. The dead weight of the DMS sign was simulated by bolting scrap steel beams to one exterior side of the truss near mid-span. The scrap steel had nearly the same weight as the reported weight of the DMS sign. The wind load was simulated by hydraulic jacks and a wiffle-tree loading configuration. The force transmitted by the hydraulic jacks was monitored with load cells. Simple steel frames were fabricated for the purpose of mimicking the sign truss end supports. The U-bolt connections and saddles were arranged to provide boundary conditions that were similar to the truss-to-end frame connections that are found in the field. A fracture in a member was simulated by cutting the cross section of the tube with a reciprocating saw. Testing of a main chord retrofit was conducted on the 55 ft truss. Frame analysis using SAP 2000 Version 11 was conducted and the analysis predictions were compared to the experimental data. Samples of crack faces were examined for the purpose of ascertaining the cause of cracking. Recommendations for the practicing engineer's response to cracked aluminum trusses are given at end of this document.

2.0 Testing of the 55 ft Long Truss

2.1 Test Setup

In the summer of 2007, the 55 ft truss was received by Purdue University. According to the Iowa DOT this truss was part of structure number 57002. The structure was inspected in the "as-delivered" state at Purdue University. No major dents or bent members were detected. However, many cracks were found. The types and locations of the cracks were consistent with the inspection reports and photos provided by Iowa DOT. No new cracked connections were detected. Some of the previously documented cracks were larger than what was documented in the inspection reports and pictures, but no member was completely severed. It is not known whether the discrepancy in crack length was the result of crack growth after the last field inspection, due to handling and transporting, or whether the crack sizes were underestimated during the last field inspection.

The truss was assembled and placed on the end supports in the laboratory in November 2007. Twenty strain gages were attached to the truss at selected locations. A Campbell Scientific CR9000 data logger recorded the strain gage output and string pot output. For this report the strain gage output was converted to uni-aixal stress using E =10,100 ksi. A simulated DMS sign (i.e. scrap steel beams) was then bolted to one side of the truss; strains were recorded during the attachment of the simulated sign. Note that the strain gage data does not include the self weight of the aluminum truss or the W4 X 13 steel members used to attach to the scrap beams used as dead load. It is noted that the offset of the beams used as dead load were approximately the same as the center of gravity of the actual sign. However, the additional items, such as the grated walkway and hand rail were not included. Hence, the torsional effects of the dead load were somewhat underestimated.

To be consistent, the analysis did not include the self weight or the weight of the W4 X 13 shapes. Figure 1 shows a 3-D illustration of the test set-up. The four green rings in the figure indicate the location of the strain gages that were placed on the main chord. Two jacks, each with their own respective wiffle-tree system were used to apply the simulated design wind load. Figure 2 is a photograph of the simulated dead load and the wiffle tree system. Figure 3 is a 3-D drawing of the simulated sign-to-truss cross section. Figure 4 shows the placement of the strain gages that will be discussed in this report, the approximate location of these gages are shown as green rings in Figure 1. The lateral displacement of the truss was measured at one collar connection and the end supports. Figure 5 shows a picture of the string pot attached to the collar connection that was used to measure the deflection of the midspan. The deflection at the end supports was measured in order to determine the amount of rigid body movement of the supports. The deflection of the supports could then be subtracted from the midspan deflections so that the 'true' midspan deflection of the truss could be determined.

The total combined weight of the two scrap beams was estimated to be between 3.80 kips and 4.0 kips. This is very close to the estimated weight for of the DMS reported by the Iowa Department of Transportation. The calculated weight of the scrap steel is based on the product of the standard density of steel (490 lb/ft^3) and the calculated volume of the beam. The weight distribution of the simulated sign on

the truss is of course somewhat different than the weight distribution of the DMS on the truss structures that are found in the field; however the difference is either negligible or somewhat conservative. The placement of the DMS on real structures in field is offset from the center and as a result the axial forces in the chords are most likely less than what would be present for a DMS placed in the center. Though the offset is somewhat less than would be found in the field, the researchers believe it was still reasonable for the purpose of this test. The simulated sign was loosely bolted to the W4 X 13 shape. This was done so that the simulated sign did not provide a substantial beneficial force path or additional bending stiffness to the truss as horizontal loading was applied. The W4 X 13 was attached to the truss using U-bolts. The Iowa Department of Transportation provided the total design wind load of 3.746 kips, applied uniformly on the truss members and a total of 9.207 kips spread uniformly on the DMS. Thus the total design lateral wind load is equal to 12.953 kips. It was determined that the laboratory test loads would meet or exceed a total lateral load of 13.2 kips. The total lateral load was applied at eight discrete points through the wiffle tree system. This results in 1.65 kips per point at a total load of 13.2 kips. This approximation of the actual wind loading was used because spreading the load uniformly would have been cost prohibitive. Furthermore, the application of the wind load at discrete points provides a conservative yet reasonable simulation of the wind load as applied in service.

Results – 55 Foot Truss

The 55 ft truss in the "as-delivered" condition was capable of supporting the simulated sign dead load and full simulated wind load during several load trials. The structure was inspected after the simulated wind load was removed (i.e. the pressure in the hydraulic jacks was released). There were no visual signs of plasticity or buckling in the members or in the strain data. However, some of the existing cracks appeared to have opened or propagated a small amount. The stress before application of the wind load and after release of the wind loaded differed by less than 1.5 % of the maximum stress at each respective strain gage shown in Figure 4. The relative displacement before application of the wind load and after release of the wind loaded differed by less than 0.5 % of the of the maximum displacement. Note that the relative displacement is defined as the lateral displacement of the collar connection minus the average lateral displacement of the truss end supports. Figure 6 presents graph of the lateral load vs. relative displacement for several tests configurations and several load cycles per configuration. As shown in Figure 6, the "as-delivered" curve is slightly non-linear, but the deviation from the ideal straight line is small. This small difference in behavior is attributed to be due to non-linearity in the U-bolted saddle connections (i.e., the supports).

Following the testing in the 'as-delivered' condition, a lower tension chord was then cut as shown in Figure 7, to simulate a fracture in one of the main chords. The chord was cut while the truss supported the sign dead load but no applied lateral load. The truss was connected to the overhead crane as a precaution in case the cutting of the chord resulted in instability. Once it was determined the truss was stable in the faulted condition, the truss was subjected to the sign load and the full lateral load. No new noticeable deformation, other than the cut chord, was observed after the lateral load returned to zero kips. The stress before application of the lateral load and after release of the lateral load differed by less than 8 % of the maximum at all but two locations (i.e., it was 8% at two strain gages). Note that 14 of the 16 gages placed on the main chord were less than 3 %. The displacement before application of the wind load and after release of the wind loaded differed by less than 1.5 % of the of the maximum displacement. This suggests that the global truss behavior was nearly linear elastic, though a small amount of localized nonlinear behavior likely occurred. (*It is noted that the two strain gages which indicated the 8% residual strain may have simply 'electrically drifted' during the test.*)

An illustration of the cross section view of the cut and retrofitted chords is shown in Figure 8. The retrofit consisted of inserting a square 3 inch x 3 inch x 0.25 inch 6061-T6 aluminum section inside of the 5 inch O.D. main load carrying chord member. During installation of the square tube in the lower chord, a 'birds nest' of what appeared to be welding wire and some solid material was found inside the chords (see Figure 9). This was removed so that the square tube could be inserted inside the round tube. The square tubes are sold in 20 ft lengths, so simple tension member splices were designed to splice the square tube sections together. The splice is shown in Figure 10. Three 20 ft long members were spliced together

and inserted inside the 55 ft long main chord. Holes were then drilled through the main chord and the internal square tube. Stainless steel bolts were inserted at defined intervals to provide shear connections and continuity between the chord and the square tube. The holes were drilled using a magnetic base drill. Since the truss is made from aluminum, a simple steel stand was attached to the aluminum chord so that the magnetic drill could be secured to the truss (see Figure 11). A set of three stainless steel bolts were then installed at 2.5 ft from the end of the truss and then the same bolt pattern was installed at about 10 ft intervals along the main chord. Figure 12 shows a picture of the bolted chord. The bolts were tightened by hand with a spud wrench until they were snug tight. It is not possible to create a slip critical connection because it would not be possible to pretension the joint due to the large gap between the square tube and the circular main chord.

The truss with one retrofit chord was capable of supporting the full DMS load and the full lateral load. The stress before application of the lateral load and after release of the lateral load differed by less than 27 % of the maximum stress at each respective strain gage shown in Figure 4. The displacement before application of the lateral load and after the release differed by less than 2.0 % of the of the maximum displacement. In Figure 6 the data labeled “lower chord retrofit” appear to be more non-linear than the “as delivered” truss as expected. However, the deviation from the ideal straight line is still small. This nonlinearity in the strain gage data and displacement data is most likely due to some combination of the bolted square tube-to-main chord connection sliding into bearing, and friction in the saddle connections.

A second tension chord was then cut and retrofitted using the same methods as previously described. The cut location of the second member was near midspan and selected because the authors estimated that this would be the worst case failure scenario for the case of two severed tension chords. In this condition (i.e., with two chords cut and retrofitted), the truss was capable of supporting the sign load and the lateral load. The stress before application of the lateral load and after release of the lateral load differed by less than 25.0 % of the maximum stress at each respective strain gage shown in Figure 4. The displacement before application of the wind load and after release of the wind loaded differed by less than 20.0 % of the of the maximum displacement. In Figure 6, the “lower and upper chord retrofit” curve is highly non-linear. The nonlinearity in the strain gage data and displacement data is most likely due to local yielding at highly stresses locations as well as other factors previously described.

Next, the two chord retrofits were severed and the truss was loaded laterally. As lateral load was applied, a plastic hinge formed in the lower compression chord. As additional load was applied, a portion of the cross section at this location fractured at a lateral load of 5.6 kips. This is approximately 40 % of the lateral design load. The test was then stopped due to avoid damage to the fixtures. Figure 13 shows the third fractured chord. (This was not a simulated fracture.) Hence, the 55 ft truss, with two chords cut, was not able to support the full lateral design load.

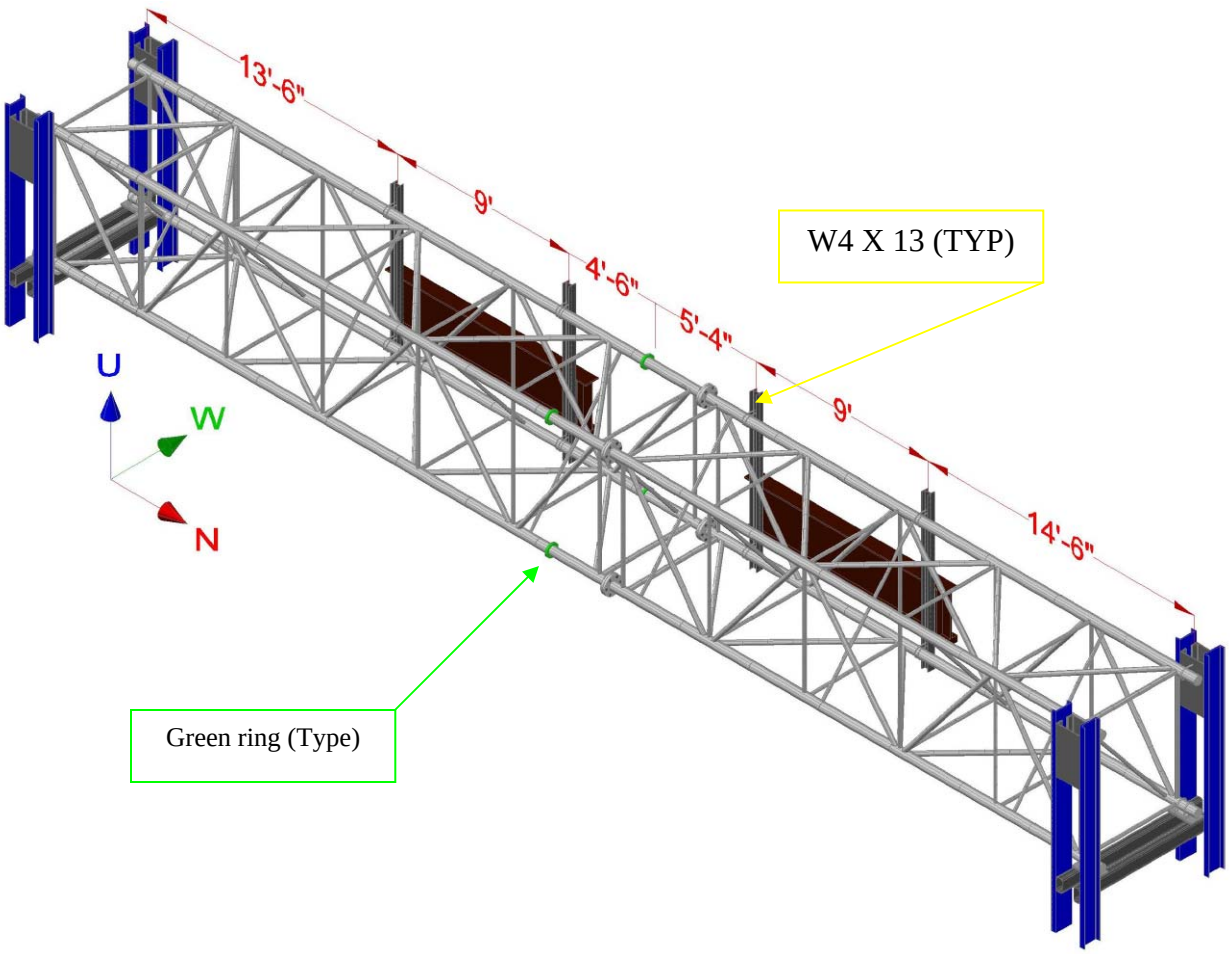

Figure 1 – Illustration of truss 55 ft test configurations, green rings (near mid-span) indicate the location of strain gages that were placed on the main chords

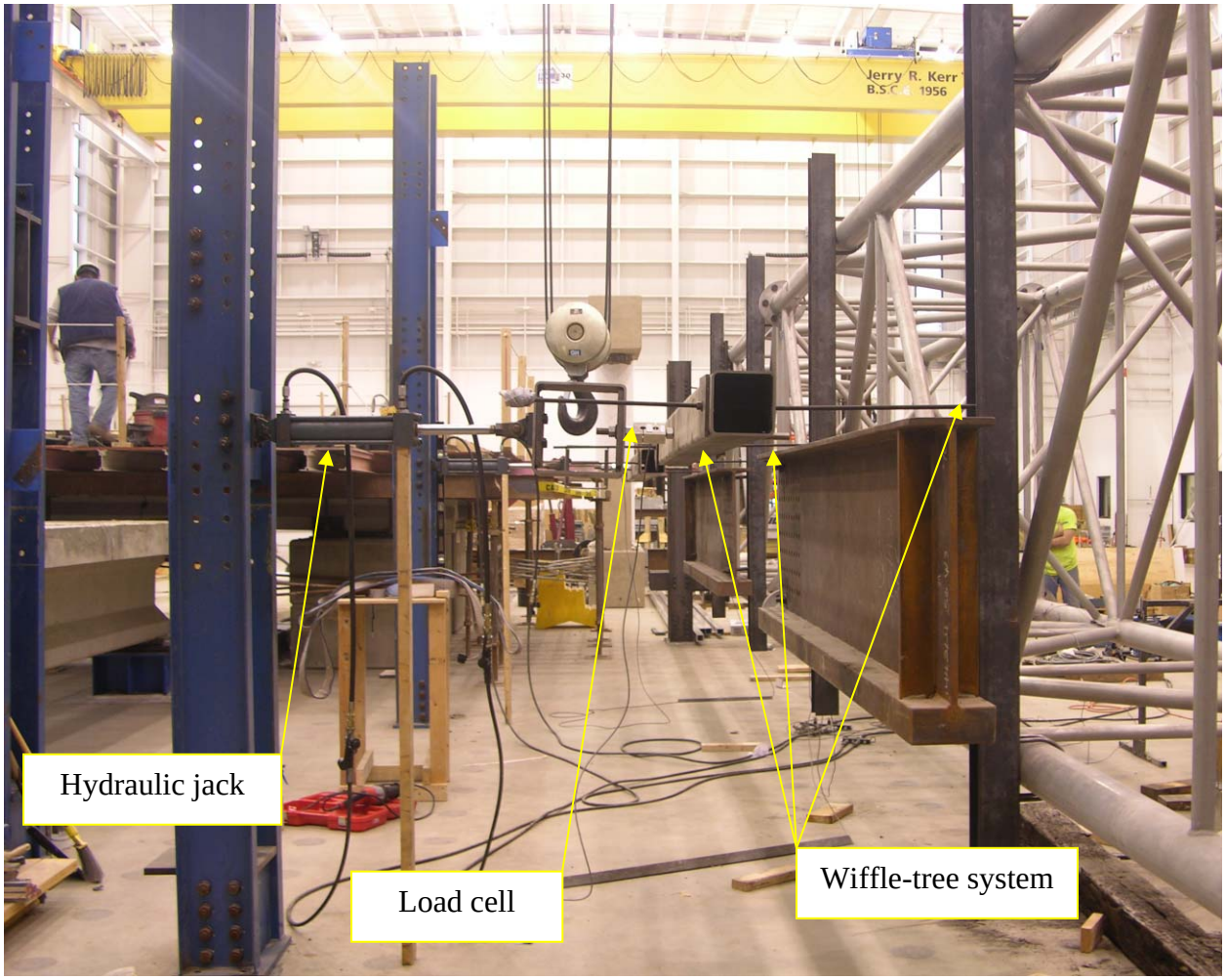

Figure 2 – Lateral load setup, 55 ft truss is shown

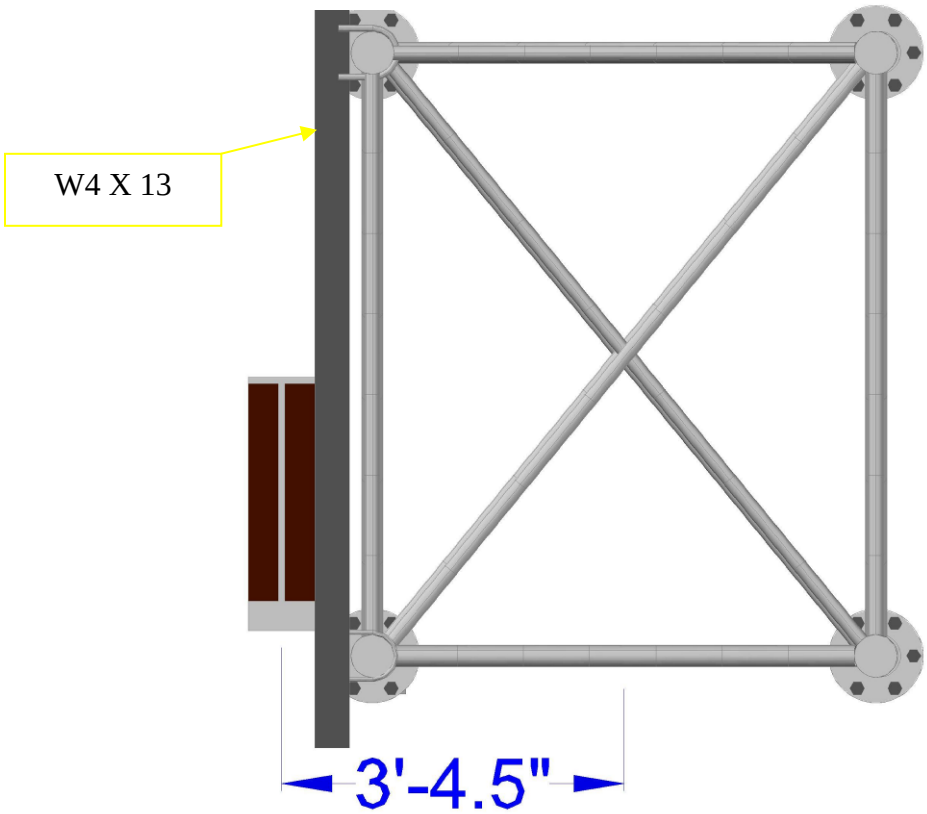

Figure 3 - Cross section view of simulated DMS sign; the scrap beam's center of gravity was placed 3' 4.5" away from the truss center line

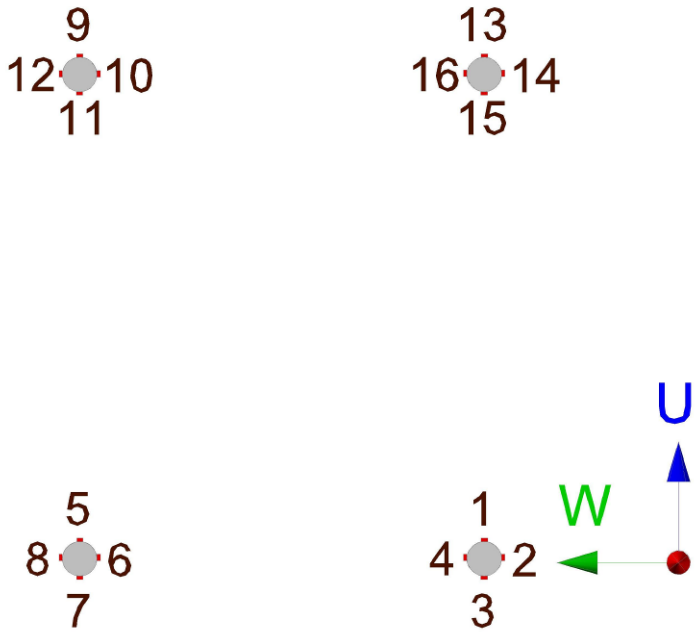

Figure 4 – Cross section view of strain gages that were placed on the main chords, the location of the strain gages are shown as green rings in Figure 1

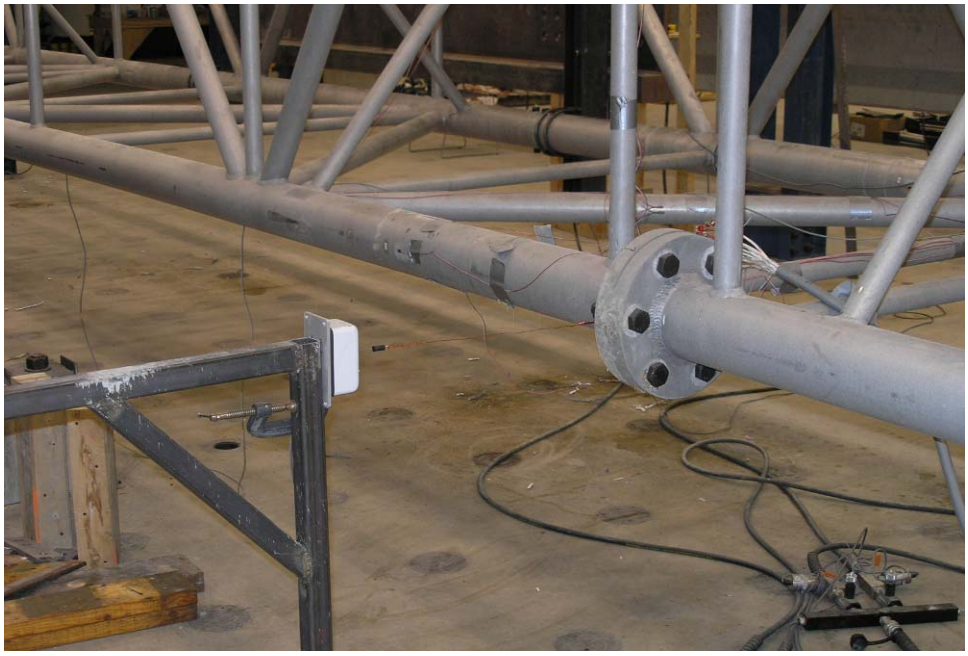

Figure 5 – String pot attached to the 55 ft truss at north bolted collar connection

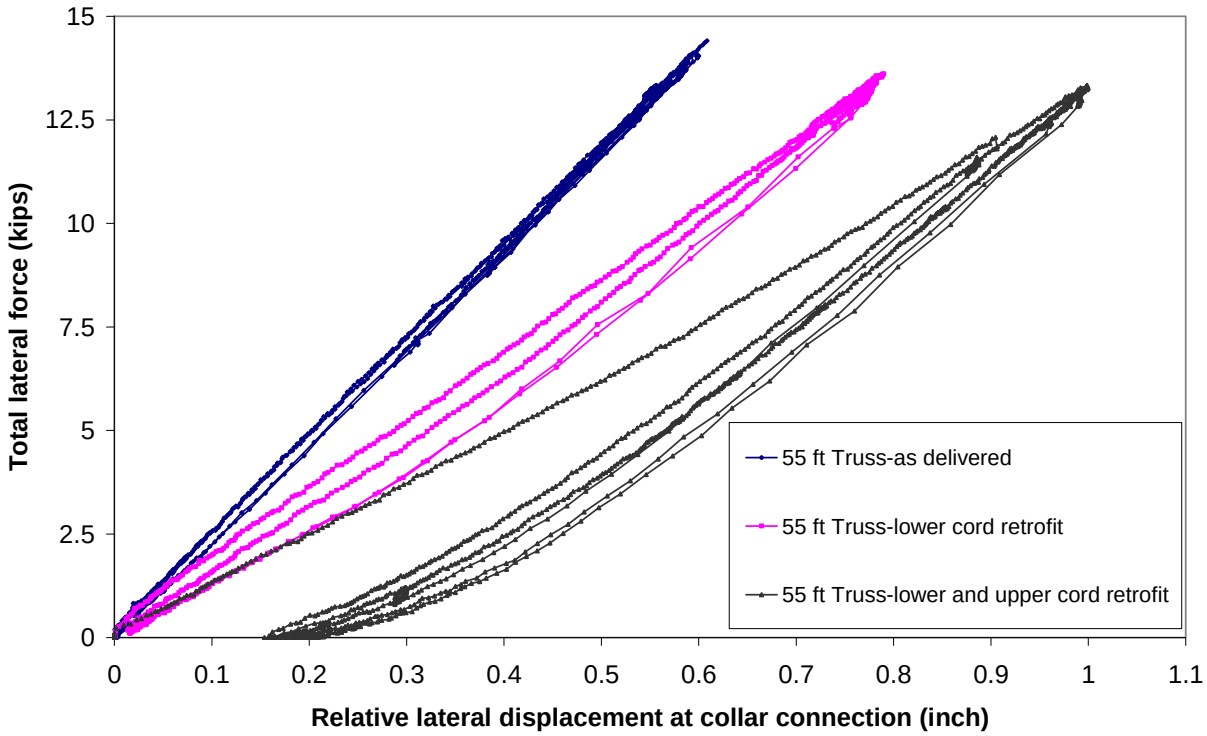

Figure 6 – Lateral load vs. relative displacement for several 55 ft truss configurations; note that the displacement was zeroed before each test

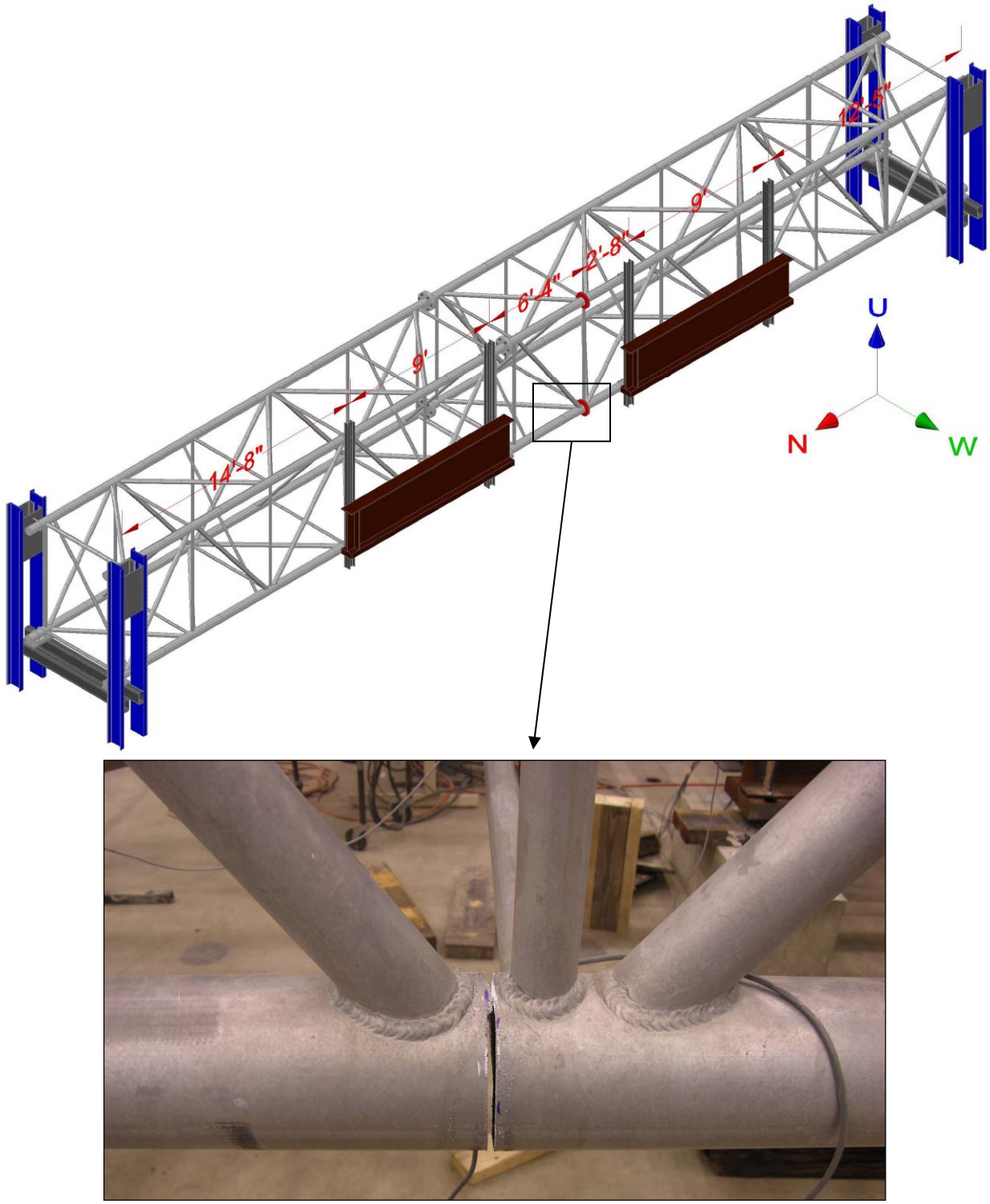

Figure 7 – Red rings indicate the location of simulated fractures on the 55 ft truss; note that the lower chord (shown in the picture) was cut first, and the top chord was cut later in the testing sequence

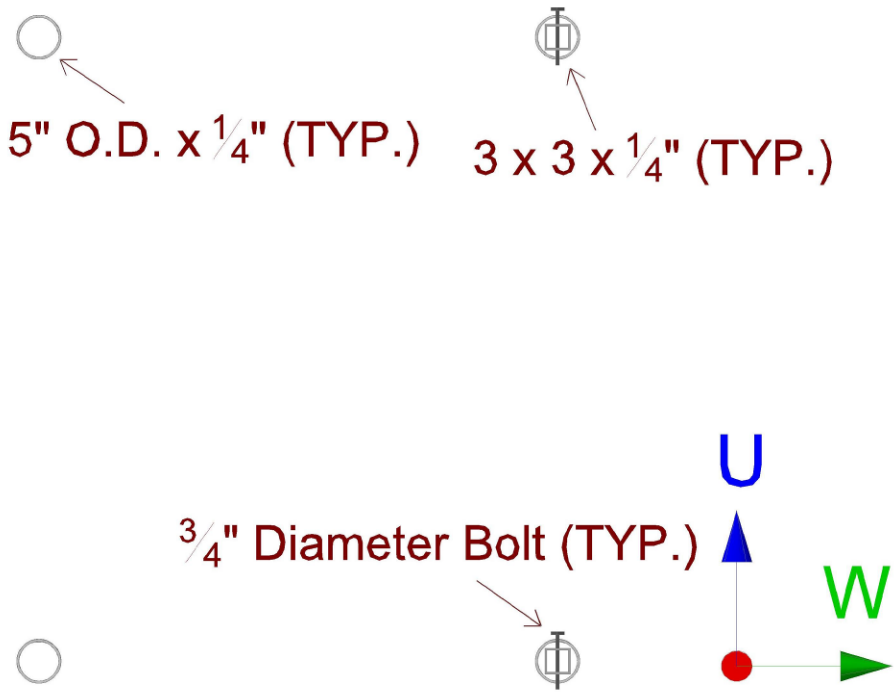

Figure 8 – Cross section view showing typical retrofit concept

Figure 9 – Material found inside the retrofitted lower chord of 55 ft truss

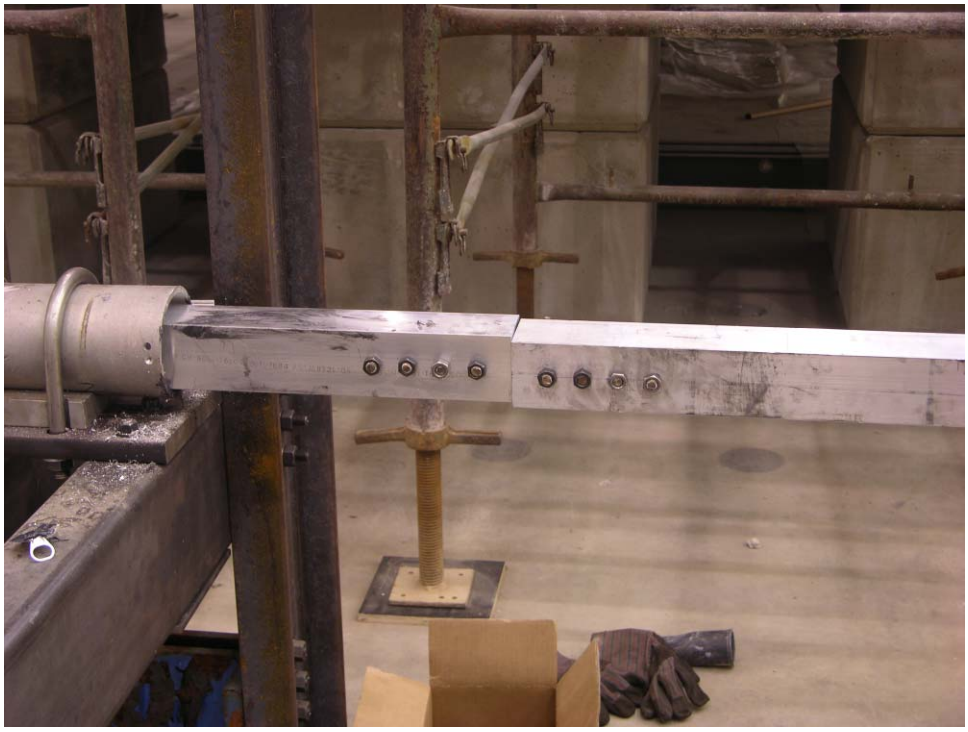

Figure 10 - Picture of splice detail for square tube-to-square tube connection (the splice plates are inside the square tubes); note that this picture was taken while the square tube was being inserted inside the main chord

Figure 11 - Picture of drilling apparatus

Figure 12 - Three bolt pattern for main chord-to-square tube connection (the square tube is inside the main chord, the bolts were stainless steel, but the nuts in this picture were not)

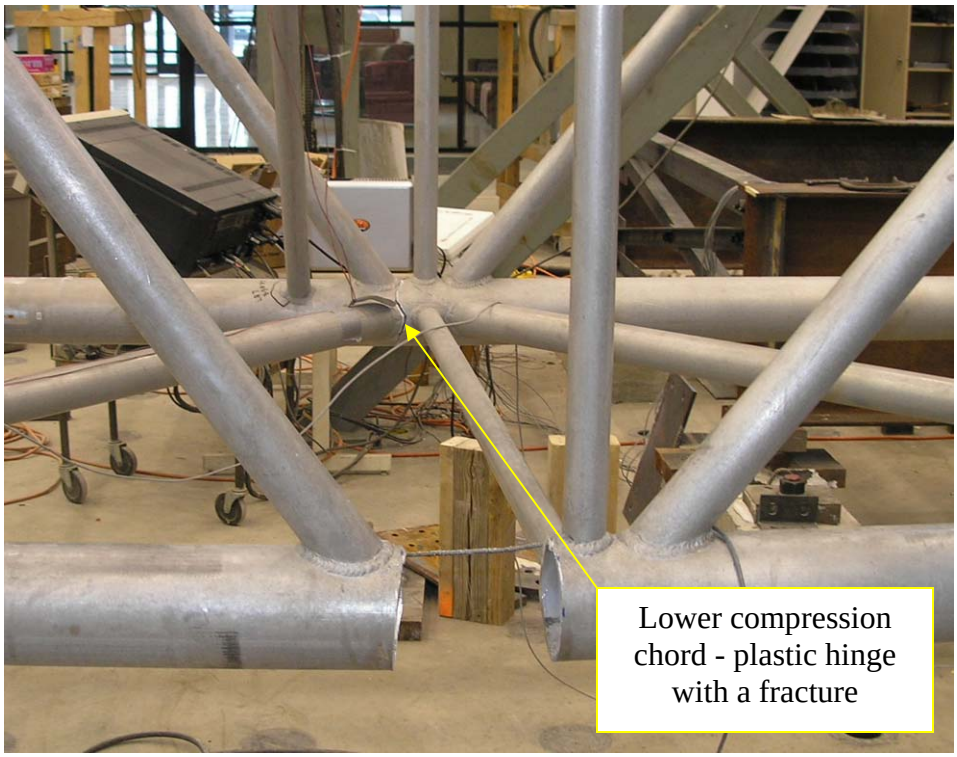

Figure 13 - Picture of 55 ft truss after the completion of all testing, far lower chord failed by plastic hinge with a fracture, near lower chord was cut with a saw (see Figure 7)

2.1 Analysis of the 55 ft Long Truss Model

An analytic model was created for the purpose of gaining further information about the behavior of the truss. A frame model was created in SAP 2000 version 11. The member geometry properties (e.g. cross section area, and moment of inertia) were based on the cross section dimensions shown in the construction drawings. The aluminum 6061-T6 material constants in SAP 2000 are E = 10100 ksi, v = 0.33, G = 3796 ksi, and F_y = 35 ksi. The member types and geometry of the truss were taken from the drawings, with the exception of the truss camber. The camber of the truss was not included in the model. The member-to-member connections were defined as perfectly rigid moment connections.

The boundary conditions presented a minor challenge. The truss-to-support frame connections in the lab set-up and in the real structure cannot be easily defined. The U-bolt connection cannot be define as "free" or "fixed" since the contact behavior and the friction in the connection is not accurately quantified. Furthermore the end support itself is an elastic body and thus the elastic response of the end support affects the force path within the truss.

In light of the complexity of the boundary conditions, simplified boundary conditions were used. Figure 14 illustrates the boundary conditions. All the main chord supports (8 total) were fixed in the Z-direction (vertical) as defined in Figure 14. One of the ends of one of the main chords was fixed in the X-direction so that the analysis would be stable. The X direction reaction was 0.0 kips at this joint, as expected. Note that the X-direction is along the length of the truss. Three of the four main chords were fixed in the Y direction. The boundary condition for the fourth chord, shown as the double roller in Figure 14, was chosen because the loading caused this chord and the U-bolt to loose contact in the Y direction. This phenomenon was observed during testing.

As stated, the simplified boundary conditions do not capture the true rotational resistance and axial resistance that are present in a real connection. As a result the model will tend to over predict the nominal axial stresses at the center of the truss and under-predicted the nominal axial stresses (generate by any potential constraint) at the ends of the truss. (A qualitative way to visualize the difference to is to compare the moment diagram for a loaded beam with pin and roller end supports to a loaded beam with fixed end supports.)

Table 1 reports the predicted stresses and the experimental stresses for the case of the truss in the as delivered state. The predicted stresses were calculated by using elementary mechanics of material relationships (i.e. axial stress = axial load/cross section area, and bending stress = Moment*radius/moment of inertia). The analysis was linear elastic. The ratio of the analytic results to the experimental results is bounded by - 26% and + 16 % error, which is acceptable for the purpose of this study. The experimentally derived stresses and predicted stresses were well below the level where yielding of a member would be a concern. The majority of the error is most likely due to the inability of the model to capture the boundary conditions at the truss ends and the true load versus deformation behavior of the welded connections. Table 2 reports the measured deflection and the predicted deflection for several truss configurations.

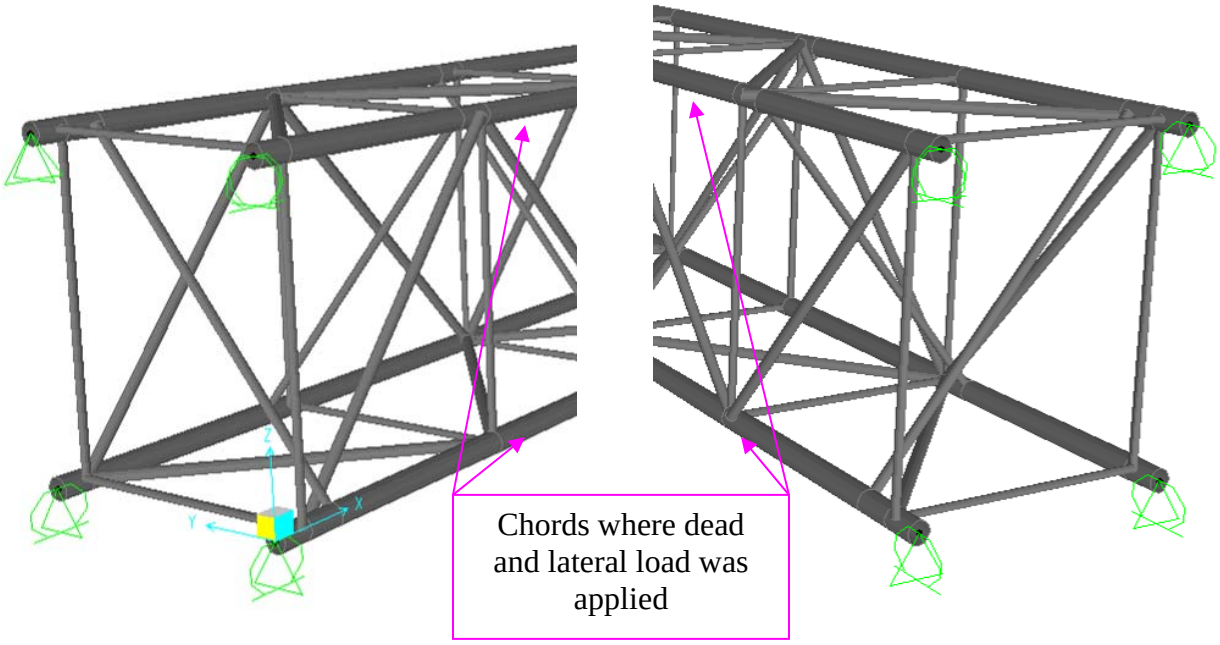

Figure 14 – Boundary conditions for each end of the 55 ft truss; double triangles represent fixed displacement in X, Y, and Z and free rotation degrees of freedom; double rollers represent fixed displacement in Z and all other degrees of freedom are free; one triangle and one roller represent fixed displacement in the Y and Z and all other degrees of freedom are free

The analysis was then repeated for the case where the bottom chord was severed. The analytic model showed that no plastic hinge would form in the frame. The analysis was also repeated for the case of the severed and retrofitted bottom chord. The analytic model showed that the results were very sensitive to how the bolted square tube-to-main chord retrofit connection was modeled. Table 2 reports on the extreme case of perfectly rigid retrofit connections (displacement of 0.55 inch) and very compliant connections (displacement of 1.02). The real measured deflection was 0.77 inch at 13.2 kips of lateral load. Thus it is reasonable to infer that the connections had a stiffness that was somewhere between the two extremes (perfectly stiff or perfectly compliant). More experimental work would need to be done on calibrating the connection before a frame analysis model could be used to quantitatively predict the behavior of the frame with retrofitted severed chords. No analytical models were built for the case of two retrofitted severed chords as the analysis would most likely also be very sensitive to how the connection was modeled. Hence, the value of the analysis would be questionable unless a full analytical calibration study was undertaken. The authors do not believe such an effort is warranted as the objective of the retrofit was met (i.e., carry the design loads in the retrofit connection). Quantification of subtle variations in response due to boundary conditions, connection slip, etc. would require considerable effort with little payback in terms of actual value added. Furthermore, the resulting analysis would only be as good as the assumptions made regarding boundary conditions and connection slip.

The linear elastic analysis was repeated for the case of two cut chords. At the total applied DMS and lateral load the SAP analysis reported moments that far exceeded the plastic moment capacity of the two intact chords. Thus the linear elastic analysis suggests that the frame would not be able to support the full design load when two chords are severed. More refined analysis with a geometrically nonlinear and material nonlinear formulation was not attempted since this type of analysis would require material and connection load verses deformation behavior data that was not available. Again, the authors do not believe such effort is warranted.

Table 1 – Stress results for the 55 ft truss subject to the simulated design sign load and simulated lateral wind load (for the as delivered configuration)

Strain Gage Name (see Figure 4)	Predicted Stress (ksi)	Experimental Stress (ksi)	Experimental/Predicted
1	-2.17	-2.34	1.09
2	-2.31	-2.58	1.12
3	-2.08	-2.43	1.16
4	-1.94	-2.16	1.11
5	4.60	3.44	0.75
6	4.96	4.08	0.82
7	4.26	3.65	0.86
8	3.90	3.02	0.77
9	2.21	2.58	1.17
10	2.80	2.57	0.92
11	2.03	1.57	0.78
12	1.44	1.59	1.11
13	-4.54	-3.36	0.74
14	-4.62	-4.02	0.87
15	-4.28	-3.87	0.90
16	-4.20	Error-no reading	NA

Table 2 – Lateral deflection results for the 55 ft truss subject to the simulated lateral wind load (response due to dead load is not included)

Truss Configuration	Experimental Relative* Lateral Displacement of Collar Connection (inch)	Analytic Relative Lateral Displacement of Collar Connection (inch)
As delivered	0.55	0.59
Cut bottom chord	0.94	1.02
Cut bottom chord with retrofit	0.77	0.55-to-1.02
Cut top and bottom chord with retrofit	0.99	No attempt

*****The measured displacement of at the collar connection minus the average displacement of the saddle connection at the truss-to-end support location

In conclusion, the results of the experimental and analytical studies are in reasonable agreement for the truss in the as delivered state. In the opinion of the authors, the experimental and analytic results support the use of a linear elastic analytic model for the purpose of investigating any scenario of a single fractured member. Linear elastic analysis is of course limited, in that it cannot calculate plastic redistribution of load or frame buckling; however, this is not seen as a limitation for the level of analysis that is needed for investigating the effect of member loss. At this time, the retrofitted truss behavior is not easily described by analysis. Further research would be required to calibrate the retrofit behavior. However, because the retrofit was able to carry the full design load, thereby satisfying the objectives of the research, such effort is not warranted.

2.2 Conclusions for the 55 ft Long Truss Model

Several conclusions, in regards to the structural integrity of the 55 ft truss subjected to the design load in the laboratory, are apparent.

1. The failure of one tension chord near the center of the truss does not affect the ability of the truss to support the design DMS load and the full design lateral wind load. The behavior should essentially remain linear elastic under this load.
2. The loss of two tension chords near the center of the truss does not affect the ability of the truss to support only the design DMS load and a nominal percentage of the design wind load.
3. The failure of two tension chords near the center of the truss does inhibit the ability of the truss to support the design DMS load and the full design lateral wind load.
4. Main chords that contain cracks, or chords that are severed, can be retrofitted. The retrofit concept shown in this report will restore the ability of the chord to carry the chord's portion of the full design load. The square tube and the bolted connections should be conservatively designed for the full axial force that the damaged chord must carry.
5. A cracked and retrofitted structure will be less stiff than the as designed structure. The decrease in stiffness for the case of two retrofitted chords was less than 40 % of the as delivered structure. Results in the field may vary based on the connection fit up, square tube wall thickness, and other factors.
6. Quantitative predictions for the retrofitted truss are very sensitive to how the retrofit bolted connection is modeled. If more precise analytical predictions are desired for the case of the retrofitted chords, then further research would be required. As long as the retrofits are designed with conservatism this research is most likely not necessary.
7. In the field, the wind will change direction. A severed tension chord may become a non-existent compression chord when the wind changes to the opposite direction if the fracture surfaces were to slide with respect to one another. For example, this could occur if the fracture surface was at 45 degree angle. It should be noted, that for this to occur, the geometry of the given chord would have to change and the truss would have to be out of square" due to the loading.

   This research program did not attempt to examine the buckling capacity of the truss when compression chords are removed. A simple elastic frame analysis in combination with the relevant buckling design equations can be used to examine the buckling capacity for the case of one intact compression chord and one removed compression chord. The loss of any two chords would most likely have the same conclusions described in "2." and "3.".

3.0 Testing of the 70 ft Long Truss

3.1 Test Setup

The 70 ft span truss was delivered to the Bowen Laboratory in the summer of 2007 and was identified as structure number 77263, according to Iowa DOT design plans. The structure was inspected in the "as-delivered" condition at Purdue University. No major dents or bent members were detected. However, many cracks were detected. The types and locations of the cracks were consistent with the inspection reports and photos provided by the Iowa DOT. No additional cracked connections were detected. Some of the previously documented cracks were larger than what was documented in inspection reports and pictures provided by Iowa DOT, but no member was completely severed. It is not known whether the discrepancy in crack length was the result of crack growth after the last field inspection or whether the crack sizes were underestimated during the previous field inspection.

The truss was assembled and placed on the end supports in August 2007. Fifty strain gages were then attached to the truss. Sixteen of the strain gages were placed on the main chords near mid span at the 12, 3, 6, and 9 o'clock position. The remaining gages were placed on internal diagonals and on various diagonal members near the end of the truss. Only the measurements on the 16 gages installed on the chords will be discussed in this report as very low stresses were measured in all of the internal diagonals. The dead load of the DMS sign was simulated using the same scrap beams as previously discussed. Figure 16 shows a 3-D illustration of the entire truss as erected in the setup. Figure 17 is a photograph of the set-up. The simulated lateral wind load was applied using the same wiffle tree set-up that was used for the 55 ft truss tests. Two jacks, each with their own respective wiffle-tree system were used to apply the simulated design wind load. Iowa DOT provided the worst case wind load of 4.874 kips spread uniformly on the truss members and 9.207 kips spread uniformly on the DMS. Thus the total lateral wind load is equal to 14.081 kips. It was determined that the laboratory test would meet or exceed a total load of 14.08 kips applied at 8 discrete points (1.76 kips per point).

Results – 70 ft Span Truss

The 70 ft truss in the as-delivered condition was able to support the dead load and the full simulated design lateral wind load. The structure was inspected after the simulated wind load was removed (i.e. the pressure in the hydraulic jacks was released). There were no visual signs of plasticity or buckling. Some of the cracks had appeared to have opened or propagated a small amount after multiple cycles of loading. However, there were no sudden cracks or noises were observed and none of the members or connections had severed. The stress before application of the lateral load and after release of the lateral loaded differed by less than 3.0 % of the maximum stress at each respective strain gage placed on one of the four main chords near the center of the truss. (See Figure 18 for gage locations). Thus the data suggest that response was nearly linear elastic.

One of the bolted collar connections was unbolted while the truss supported the sign dead load (see Figure 19). This was done to investigate the behavior of the truss when one of the load paths in a tension chord was removed. The benefit of un-bolting the connection is that the connection can be re-bolted together after the test with little or no residual damage. The truss was connected to the overhead crane as a precaution in case this might cause instability. The truss with one unbolted collar connection was subjected to the full simulated lateral wind load. The lateral load was removed and then the collar connection was re-bolted together. No new noticeable permanent deformation was observed. The stress before application of the lateral load and after release of the lateral loaded differed by less than 9.0 % of the maximum stress at each respective strain gage placed on one of the four main chords near the center of the truss. Thus the response was approximately linear elastic. The nonlinearly most likely due to friction in the truss-to-end support connections. The joint was easily re-bolted together after the lateral load was released. One worker with two wrenches was able to re-bolt the connection. There was no visible evidence that suggested that large scale plastic deformation had occurred around the region of the truss were the collar connection was un-bolted.

A lower chord was then partially cut as (Figure 20 illustrates the locations where the chords were severed). In order simulate the sudden release of energy that would take place in the event of a chord

fracture, the chord was only partially cut. Then, lateral load was applied until the chord fractured. The truss was connected to the overhead crane as a precaution in case the cutting of the chord cause instability. After the chord fracture, additional lateral load was applied up to the full design lateral wind load. The truss with one fractured chord was capable of supporting the full dead load and lateral load. No additional permanent deformation, other than at the fractured chord, was observed after the lateral load returned to zero.

Failure of individual internal diagonals was evaluated by sequentially cutting selected diagonals, applying the lateral load, unloading, cutting more internal diagonals, and then repeating the process until all diagonals were cut. Hence, at the completion of the internal diagonal tests, all internal diagonals were cut. In retrospect such sequencing was probably not necessary; one test with all diagonals severed could have been a more efficient means of verifying the minimal contribution of the diagonals to the overall strength of the truss.

At this stage, the truss was in a severely damage condition as all internal diagonals and one tension chord were completely severed. In this condition, the simulated lateral load was then applied. Even in this condition, the truss was able to support the full lateral design load.

An interesting observation was made when cutting the internal diagonals. At several locations, water drained out of the diagonal as it was being cut. The most likely explanation is that water entered the member through a crack while the truss was in service or during the fabrication process. The water could potentially cause damage to the internal diagonal because the water in the member changes to ice in the winter and expands. Thus, there is a question of whether ice expansion caused the member to crack. This does not appear to be the case because no bulging of the internal diagonals was found. Furthermore it would be expected that the cracking patterns would favor the lower diagonal-to-chord connections, since that is where the water would accumulate because of gravity. However no such pattern exists. At this time the authors' best estimation is that the water in the diagonals did not contribute to the formation of cracks in the internal diagonal-to-chord connections. It is worth noting that the water was very dirty and clearly contained various contaminants. Thus, it is unlikely the water would actually freeze at 32F (as pure water does) and the freezing point was potentially much lower due to the presence of the pollutants.

Finally, the upper tension chord was cut. The truss with two tension chords and all internal diagonals cut was capable of carry the full dead load (i.e., self weight and weight of the simulated DMS). Next, the simulated lateral wind load was applied. At a total lateral load of 7.2 kips, or approximately half of the full lateral load, the truss had approximately 10.5 inches of lateral deflection at the mid-span. The test was stopped because the authors were concerned that the truss would permanently damage the end supports.

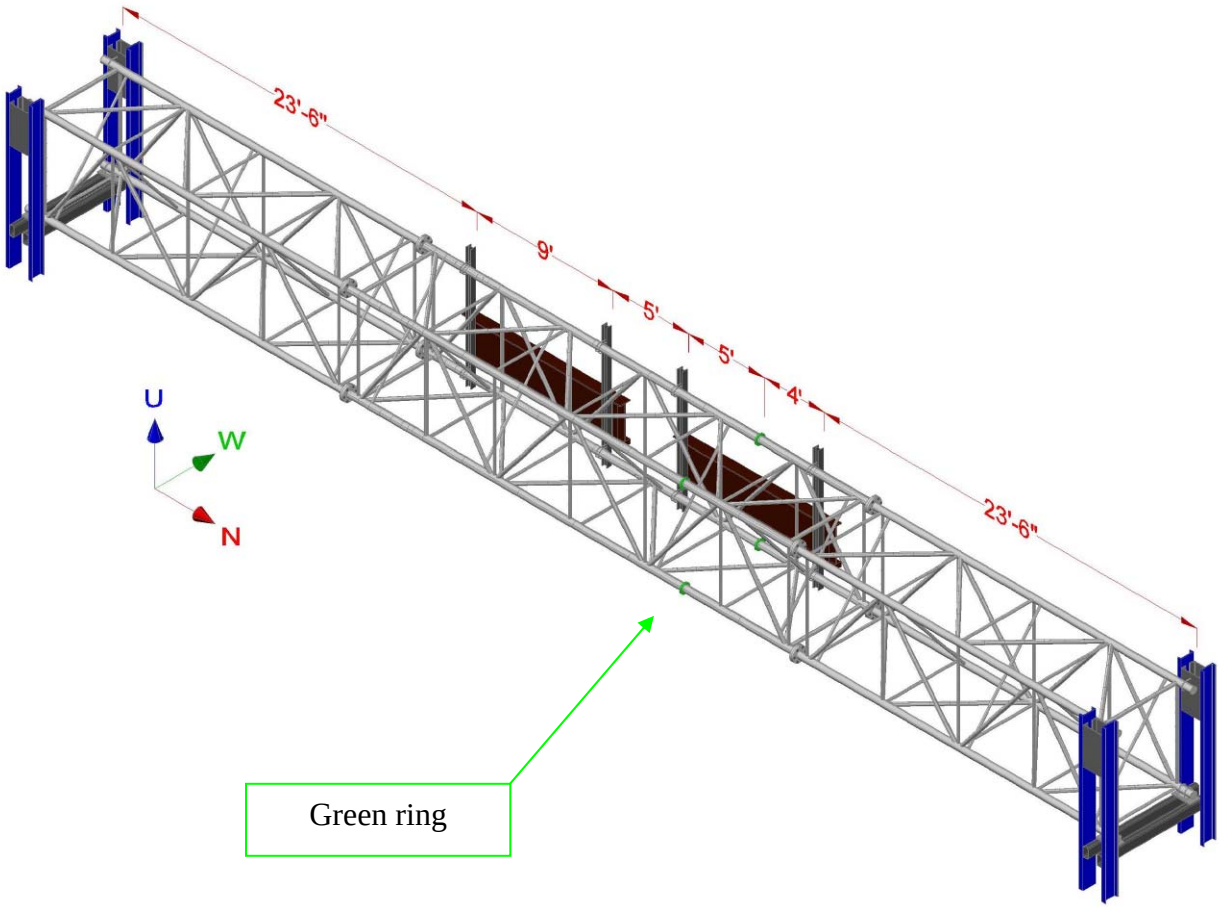

Figure 16 - Illustration of truss 70 ft test configurations, green rings (near mid-span) indicate the location of strain gages that are shown in Figure 18

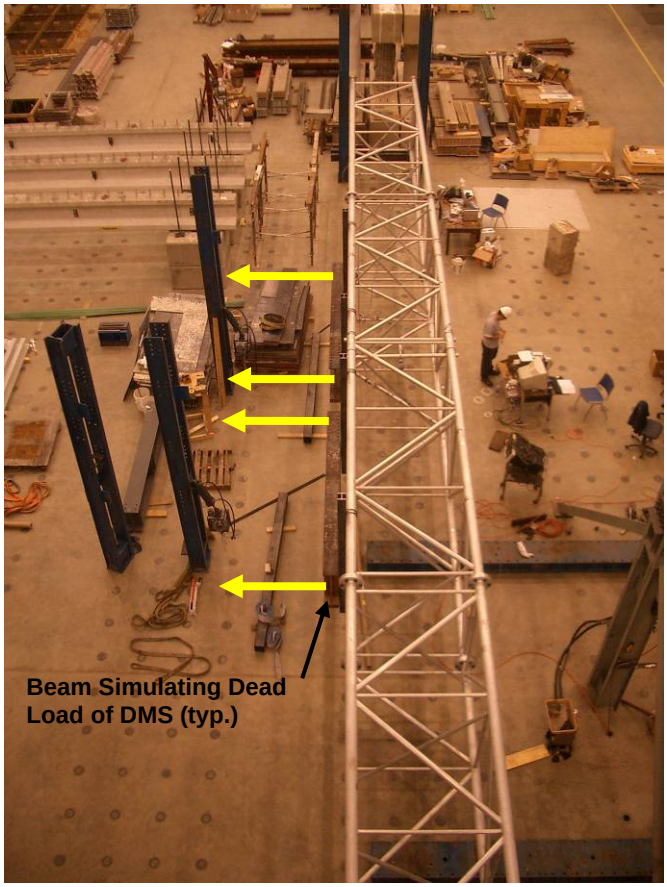

Figure 17 – Picture of set-up of 70 ft truss, yellow arrows indicate the direction of applied lateral loads

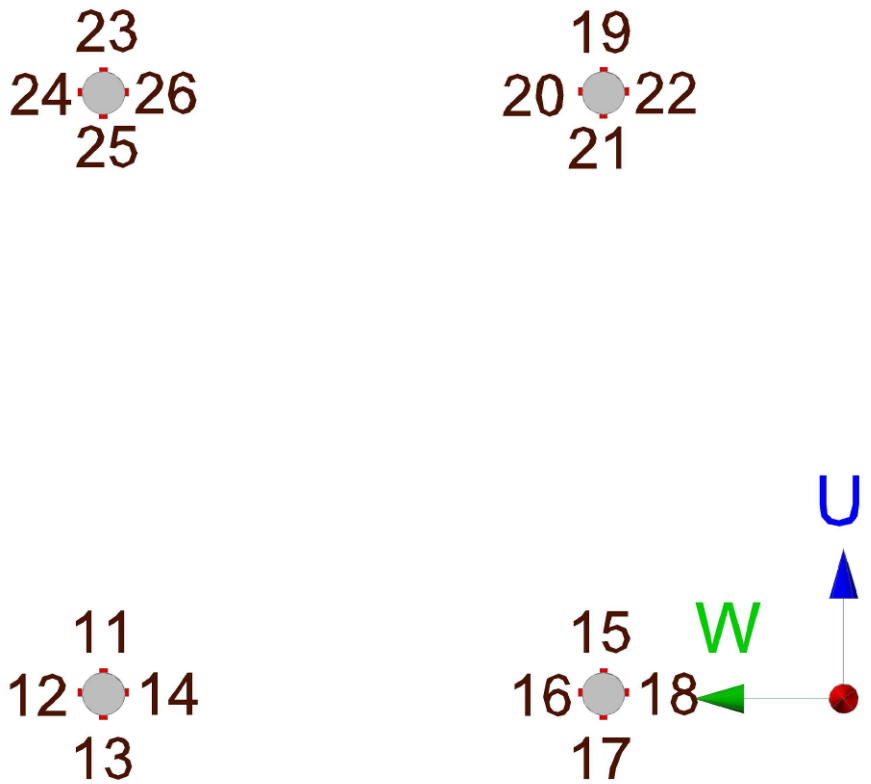

Figure 18 – Cross section view of strain gages that were placed on the main chords, the location of the strain gages are shown as green rings in Figure 16

Figure 19 – Picture of unbolted collar connection, 70 ft truss

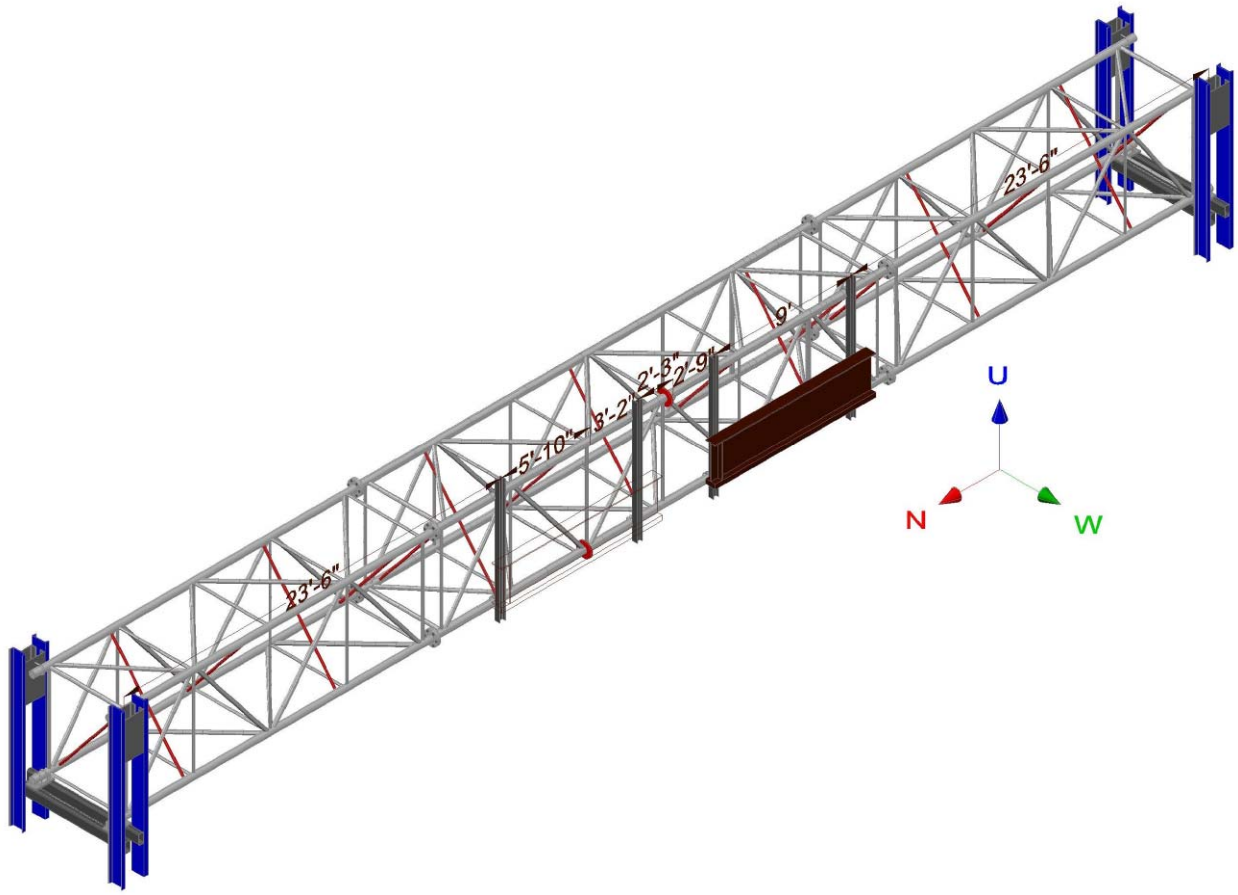

Figure 20 – Red rings on the main chords and red internal diagonals indicate the location of simulated fractures; note that the sequence of testing was 1.) cut the lower chord and then test, 2.) cut internal diagonals and then test, repeat cutting more internal diagonals and testing until all diagonals are cut, 3.) cut the top chord and then conduct final test

3.1 Analysis of the 70 ft Long Truss Model

The 70 ft truss was analyzed using the same analysis methods and boundary conditions reported previously for the 55 ft span truss. The geometry was slightly simplified by setting internal diagonals at 5 ft intervals instead of the exact position. Table 3 compares the predicted stresses from the linear elastic model to the stresses measured on the main chords for the truss in the as delivered state. The ratio of the analytic results to the experimental results is bounded by - 22% and +12 % error, which is acceptable for the purpose of this study. The differences are attributed to the fact that the boundary conditions were simplified. As stated previously, the actual boundary conditions are not ideal 'pins' or 'fixed' supports. Hence, differences between the analytical results and the measured data are expected. It is important to note that the measured stresses and predicted stresses were well below the level where yielding of a member would be a concern.

The analysis was repeated with the tension chord and all internal diagonals being severed. The analytic model showed that no plastic hinge would be expected form in the truss and the behavior should be linear elastic. The analytical results were consistent with the observed experimental behavior.

The linear elastic analysis was also performed for the case with two chords and all internal diagonals severed. As expected, under the applied dead and total lateral wind load, the analysis resulted in moments that far exceeded the plastic moment capacity of the two remaining chords. Thus the linear elastic analysis suggests that the frame would not have been able to support the full design load when two chords are severed near mid-span. This was also consistent with the experimental results.

Table 3 – Stress results for the 70 ft truss subject to the simulated design sign load and simulated lateral wind load (for the as delivered configuration)

Strain Gage Name (see Figure 18)	Predicted Stress (ksi)	Experimental Stress (ksi)	Experimental/Predicted
11	5.76	4.91	0.85
12	4.81	4.36	0.91
13	5.29	5.05	0.96
14	6.24	5.75	0.92
15	-3.95	-4.10	1.04
16	-3.45	-3.50	1.01
17	-3.76	-3.94	1.05
18	-4.26	-4.76	1.12
19	-5.80	-4.87	0.84
20	-5.26	-4.10	0.78
21	-5.57	-5.84	1.05
22	-6.12	-5.36	0.88
23	4.06	4.43	1.09
24	3.11	2.70	0.87
25	3.55	3.40	0.96
26	4.51	4.45	0.99

In conclusion the truss experimental data and analytic truss data are in reasonable agreement for the truss in the as delivered state. Thus, the linear elastic analytic model can be used to investigate the any scenario of fractured members.

3.2 Conclusion for the 70 ft Span Truss Model

Several conclusions regarding the structural integrity of the 70 ft truss subjected to the design load in the laboratory are apparent.

1. The loss of one tension chord near the center of the truss does not affect the ability of the truss to support the dead load and design lateral wind load.
2. The loss of both tension chords near the center of the truss will not affect the ability of the truss to support only the design sign dead load.
3. The loss of both tension chords near the center of the truss will inhibit the ability of the truss to support the full dead load and the full design lateral wind load.
4. The loss of all internal diagonals does not affect the ability of the truss to support the full design load.
5. In field, the wind will change direction. A severed tension chord may become a non-existent compression chord when the wind changes to the opposite direction. This research program did not attempt to examine the buckling capacity of the truss when compression chords are removed. A simple elastic frame analysis in combination with the relevant buckling design equations can be used to examine the buckling capacity for the case of one intact compression chord and one removed compression chord. The loss of any two chords would most likely have the same conclusions described in “2.” and “3.”.

4.0 Results of Fractographic Examinations

4.1 Properties of Base Metal

Although no material tests were conducted to determine the exact material properties, some general information is presented based on published data regarding this aluminum alloy. According to the design drawings, the material is an Aluminum alloy designated as 6061-T6. This is aluminum alloy, containing magnesium and silicon as the major alloying elements. It is one of the most commonly used general purpose alloys, possesses good mechanical properties and is rather weldable. It gains its strength through a heat treatment process (precipitation hardening). T6 temper 6061 has an ultimate tensile strength of at least 42 ksi and yield strength of at least 35 ksi.

As stated, Alloy 6061 is highly weldable using the tungsten inert gas welding (TIG) or metal inert gas welding (MIG). Since 6061-T6 is a heat-treatable alloy, the strength of the T6 version of this alloy can be reduced in the region of welds. Typically, after welding, the properties near the weld are reduced due to the local heat treatment at the weld which can result in a loss of strength of 20% to 40%. The material can be re-heat-treated to restore -T4 or -T6 temper for the whole piece. For design purposes, the heat affect zone is typically taken as one inch either side of the center of the weld. Yield strength in welded zone goes from nominally 35 ksi to about 20 ksi. The tensile strength reduces from nominally 38 ksi - 42 ksi to about 24 ksi.

As with all aluminum alloys, 6061-T6 is susceptible to hydrogen-induced weld metal porosity and special care must be taken during fabrication to ensure a high-quality weld. As may be expected, porosity forms during solidification due to the abrupt drop in hydrogen solubility when going from liquid to solid. Porosity can best be avoided by minimizing the available hydrogen during welding. Nevertheless, 6061-T6 is generally considered a weldable alloy as long as the appropriate filler metal is selected. In contrast to most steel weldments, the weld metal of heat-treatable aluminum alloys is typically the weakest part of the joint and is typically the location of failure when the joint is loaded in a strength test.

4.2 Examination of the Fracture Surfaces

Based on the results of the visual inspection, it was clear that the cracks observed at the weld toes likely entered into the base metal as well as traveled through the weld throat. In order to examine these potential defects, several of the cracked connections were saw cut and the crack surface exposed. Connections at internal and shear diagonals were examined as well as cracked and uncracked connections. No cracks were observed at connections that did not exhibit obvious visual cracking. Several cross-section were also saw cut and polished for examination (Pieces 1 and 4).

Figure 21 contains photographs of two cracks that were examined. As is clearly seen, the crack in the left photograph in Figure 21 is entirely in the weld throat while the other (right photograph) is located at the toe and enters the base metal (*as indicated by the red circles*). Also apparent is the complete lack of fusion between the diagonal member and the chord. However, fillet welds were specified on the design drawings. Nevertheless, it would seem some level of penetration/fusion would be expected.

Exposure of the crack surfaces revealed no evidence of fatigue crack initiation or propagation. Several specimens were examined using a light microscope at magnifications of up to 20X. Two such specimens are shown in Figure 22 and are typical of all crack surfaces examined. Considering the size of the cracks observed and the lack of any evidence of fatigue crack growth, it would be unreasonable to suggest that fatigue was the initiator or cause for crack growth.

Macro examination of the fracture surface reveals some insight into the cause the fracture. The weld metal did not show evidence of any porosity or defects that would be considered rejectable per the Structural Welding Code for Aluminum, (AWS D1.2 2003). AWS D1.2 contains criteria for fillet weld soundness in Section 3.10. The specific criteria are presented in Section 3.10.3.1, where it is stated that:

> *"If the specimen folds flat on itself, the specimen shall have satisfactorily passed the test. If the specimen fractures, the broken surface shall be examined visually. To pass, it shall show complete fusion to the root of the joint, and shall exhibits no inclusion of porosity larger than 3/32 in. (2 mm) in the greatest dimension. The sum of the greatest dimensions of all inclusion and porosity shall not exceed ¾ in (20 mm) in each 4 in (100 mm)."*

No rejectable defects, per AWS D1.2 described above were found in any of the crack surfaces examined. Since no rejectable macro defects were found, several images were taken using a scanning electron microscope (SEM) of several of the crack surfaces. Two typical examples are presented in Figure 23.

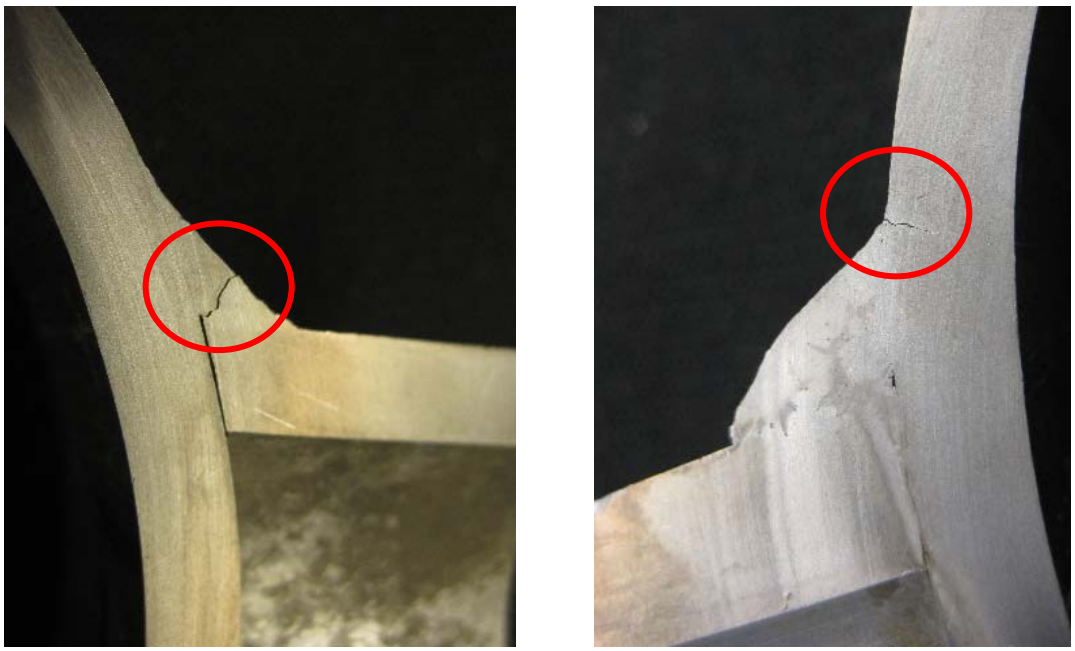

Figure 21 - Photograph of cracks observed through weld throat and weld toe.

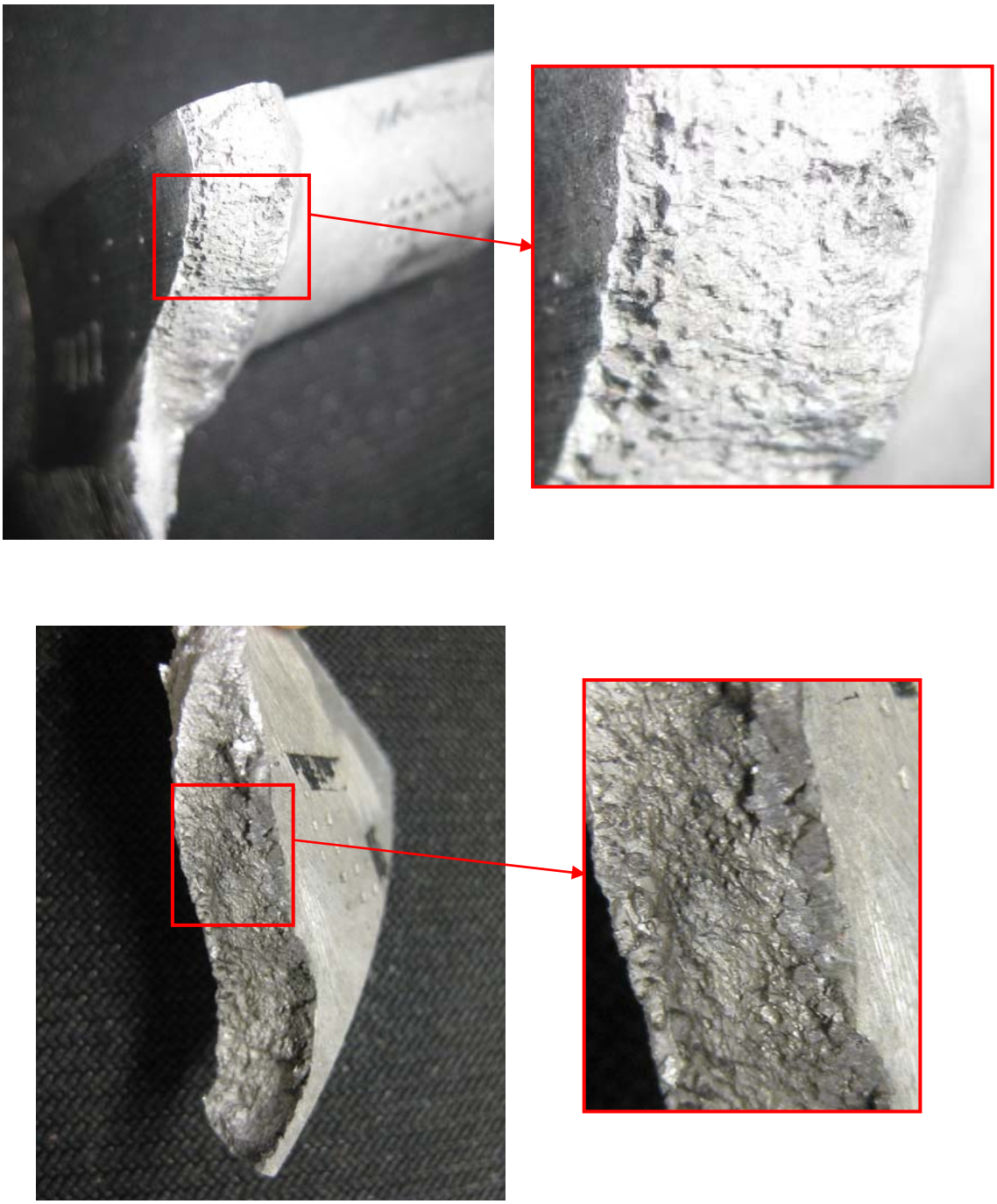

Figure 22 - macro photographs of fracture surface from two joints

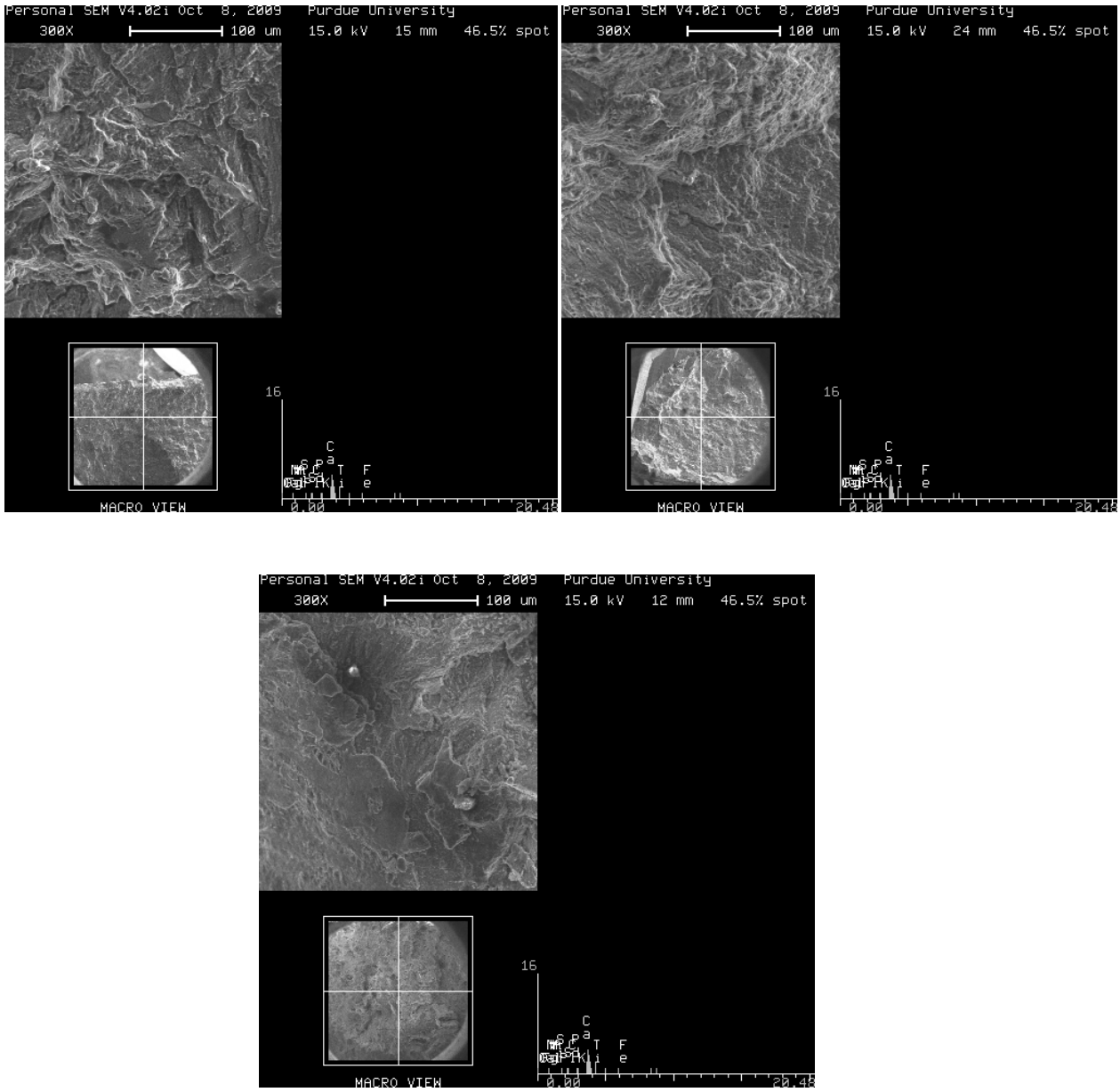

Figure 23 - SEM images from selected crack surfaces

The fractographic examination suggests the cracking was primarily brittle in as evidenced by the relatively ‘smooth’ fracture surfaces as shown in the photographs in Figure 23. However, near the free edges of the cracks, there was some evidence of ductile fracture, likely due to the fact that there was little constraint at the free edges as evidenced by the presence of small shear lips. Such an observation is common in brittle fractures.

5.0 Conclusions and Recommendations

5.1 Causes for the Observed Cracking

The causes for the cracking is believed to be due to significant load applied to the joints. Obviously, the question becomes what is the source of the significant loading. Although a definitive answer is not possible to develop, it is the opinion of the authors that significant residual stresses (local welding residual stresses and global member restraint) led to very high forces being locked in the members. In the presence of normal and acceptable (per AWS D1.2 standards) fractures initiated at the welds leading to the observed cracks. No evidence of fatigue cracking was found which would suggest that fatigue cracks propagated to an unstable size, resulting in the observed fractures.

Discussions with fabricators suggest that the trusses are fabricated such that the internal diagonals are the last members to be installed and often are used to 'force' the four chords into their final and true geometry. As stated, the experimental measurements confirmed that there is little stress produced in the internal diagonals during any of the load cases examined in the laboratory testing of the 55 ft and 70 ft truss.

It is suspected that some of the cracks developed shortly after fabrication or shortly after the truss was erected in its final position. As stated, the weld itself is likely a lower strength than the base metal. Hence, it is suspected that the cracks initiated in the weld. In some cases the crack remained in the weld metal while in others the fracture propagated into the base metal.

5.2 Recommendations for the Aluminum Truss Inventory with Similar Design and Construction

The 55 ft truss and the 70 ft truss are both from the same truss design concept. According to the drawings provided by the Iowa DOT, the state of Iowa has five standard space trusses for this concept. The five standard truss types differ primarily in the total length (50 ft, 55 ft, 60 ft, 65 ft, and 70 ft) and the location of the collar splice. Otherwise the member type, construction practices, and connections are the same. In light of the similarities for the five truss types, it is acceptable to extrapolate the results of the 55 ft and 70 ft truss test to the 50 ft, 60 ft and 65 ft truss test. Thus all five truss types should be able to support the design dead load (3.95 kips or less) and the respective design wind load even when one tension chord and up to all of the internal diagonals are severed. All five truss types should be able to support the design dead load when two chords are severed. All five truss types most likely will <u>not</u> be able to support the design dead load and the design lateral wind load when any two chords near the center are severed. The case of two main chords severed near the end supports was not investigated. In lieu of further analysis and testing, the authors recommend that the conclusions for the mid-span severed chords can also be used for severed chords near the supports.

A severed chord can be successfully repaired with the retrofit concept detailed in this report. The retrofit tube and connections should be designed for the load that the chord would need to carry in the as built condition. A tube with a cross section <u>larger</u> than 3 inch x 3 inch is not recommended for placing inside the 4.5 inch I.D. (5.0 inch O.D.) main chord because it may not be able to fit. The only significant constructability concern for the retrofit detail is that some type of blockage (e.g. debris from construction) inside the main chord may inhibit the installation of the retrofit. The inside of the chords should be viewed to look for blockage prior to the inserting of a tube inside of the main chord. The chord retrofit is not designed to be a permanent fix. The retrofit's long-term corrosion resistance and fatigue resistance is not known. (*It is noted that corrosion is not anticipated to be a problem as it is aluminum and stainless steel fasteners should be used. Fatigue is also not believed to have contributed to the observed cracking, however in service live load stress ranges are unknown.*) In the absence of further test data or analysis, it may be reasonable to place a limit on the effective design life of the retrofit. For example, without any formal inspection program that examines the integrity of the retrofit, Iowa DOT may wish to limit the life of the retrofit to say two or three years. With some form of inspection program, the life may be extended.

The conclusions for the experimental data and analysis can be used to form recommendations for the action that should be taken in regards to similar aluminum truss structures. If cracks are found in internal diagonals, then there is no immediate safety concern. The maintenance engineer should see that these cracks are removed in an expedited manor so that member will not fall out onto the roadway.

If cracks are found in only one main chord, then there does not appear to be immediate safety concern. The truss should be retrofitted or replaced as scheduling permits. It may be prudent to increase inspection frequency to ensure additional cracks do not develop undetected in other main chords. If cracks are found in more than one main chord or if at least one chord is completely severed, then there may or may not be a safety concern. The amount of wind will be the controlling factor for whether there is a safety concern. Obviously since high winds can occur at any time, a conservative response strategy should be implemented should cracks be found in more than one chord or if a chord is fractured. Trusses with more than one cracked chord or at least one completely severed chord should be a top priority for replacement or chord retrofit. Of course, each situation in the field is unique and these recommendations should not be taken as absolute or definitive.

5.3 Acknowledgments

The primary investigator for this research project was Robert J. Connor. Bruce Brakke (Iowa DOT) was the project manager. Steven Altstadt served as the research engineer, managed the day-to-day operations for this project, and conducted the final analysis. Sangdo Hong conducted some of the preliminary analysis at the start of this project. Special thanks are in order for the undergraduate assistants Dan Kurdziel, Trevor Lighty, and Joe Mueller. They often came into the laboratory on nights and weekends to help construct the test set-up and instrument the trusses. Dan Kurdziel also created the 3-D illustrations. Lastly, thanks to Professor Jason Weiss of Purdue University for assisting with obtaining the SEM photographs.