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July 6, 2010

Mr. M. Scott Ratliff
Travelers Insurance Company
P.O. Box 662
Midlothian, VA 23113

Subject: Blacksburg High School Gymnasium Collapse

Your Claim # DMF 3428; FORCON CASE # V10155

On March 5, 2010, you asked FORCON to determine the cause of the gymnasium collapse that occurred at Blacksburg High School on February 13, 2010.

FORCON's structural engineer, Henry W. Moncure, PE, conducted site visits March 1, 2010, May 10, 2010, and May 19, 2010. He conducted a design review with available drawings and technical information. An evaluation of the remaining gymnasium structure was performed. FORCON issued a report on June 4, 2010 concluding that that the gymnasium collapse began at the south end of the west main roof truss where it was seated on the column. Please refer to that report.

On May 10 & 11, 2010, the main truss ends were removed from the scene and transported to FORCON for further evaluation and metallurgical analysis. On May 17 & 18, 2010, FORCON's mechanical and materials engineer, Walter S. Laird, PE, examined and documented the fracture surfaces and determined that extensive testing and analysis was required in order to determine the exact cause of failure at the southwest main truss seat.

On May 20, 2010, the southwest truss end was transported to Mistras in Richmond, Virginia for radiographic and ultrasonic evaluation. The radiography and ultrasonic testing was completed on May 24, 2010. Because of the complex geometry of the steel when it deformed during the collapse, the results of this testing was inconclusive and destructive analysis was required. The steel needed to be cut up for laboratory testing.

On June 3, 2010, FORCON transported the southwest and northwest main roof truss ends to Lehigh Testing Laboratories in New Castle, Delaware and consulted with principle metallurgist, Aaron B. Tanzer. The fracture surfaces were examined, scene photographs, and macro photographs were studied. A testing and analysis protocol was developed. Additional radiography and scanning electron microscopy was requested.

On June 11, 2010, FORCON traveled to Lehigh Testing Laboratories to examine sectioned fracture surfaces in the laboratory. Preliminary observations and possible causes were discussed. The status of testing protocol was reviewed for completeness and scheduling. Additional sectioning and analysis was required.

On June 25, 2010, FORCON returned to Lehigh Test Laboratories to review and analyze the results of the metallurgical testing, chemical analyses, radiography, and fractography. A root cause analysis was performed. On June 30, 2010, Lehigh Testing Laboratories issued a report with their observations and conclusions. Please refer to Lehigh's report.

This investigation determined the following:

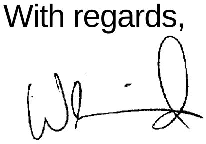
- A snow event was the initiating factor for this gymnasium collapse.
- The south support of the west main truss buckled causing it to roll and collapse.
- The steel was slightly weaker than it was supposed to be.
- The welds were weak at the truss seat. They were substandard and did not meet the American Welding Society's standards for structural welds.
- Lack of weld fusion and voids in the weld created additional stress in the metal where the failure began.
- Gusseting in the area of the truss seat plates would have been a better and stronger design and may have prevented this collapse.

FORCON concludes that the gymnasium collapse at Blacksburg High School on February 13, 2010 was caused by a snow event in combination with a poorly designed and manufactured main truss support.

Please call if you have any questions.

We appreciate the opportunity to assist you.

With regards,



Walter S. Laird, PE
Mechanical &Materials Engineer
AWS Certified Weld Inspector

June 30, 2010

Walter S. Laird
FORCON International
5424 Distributor Drive
Richmond, VA 23225

SUBJECT: Blacksburg High School Roof Truss Support – Case V10155
Lehigh Project Number: 2736-10
PO # Case V10155

BACKGROUND

This report documents the examination of a roof truss support received for evaluation from FORCON International.

Lehigh Testing Laboratories received both the southwest and northwest ends of a truss including the supports. This system had been used to support the gymnasium roof at Blacksburg High School in Virginia. The truss sections were sent after the roof had collapsed on February 13, 2010. At the time of the collapse, the roof was covered with a large amount of snow. Cracks in the wall under the truss top horizontal member on the southwest end and a masonry block on the floor were noticed during girls' basketball practice. The building was evacuated as a precautionary measure and the roof subsequently collapsed during the same day. A coach took cell phone photographs of the cracked wall prior to evacuation (Figures 1 and 2). The masonry block surrounding the support column had cracked into fissures propagating in the downward direction, and the top member of the truss had already separated from the roof adjacent to the southwest wall. In addition, the top member of the truss had already moved to the left edge of the masonry block column while the bottom member was still in the center of the column.

The type of truss construction used is illustrated in Figure 3 which shows a truss and support column in the school auditorium. While this truss is smaller than the gym roof trusses, the general design is the same. Trusses are supported on ends by a steel shape which consists of a vertically-oriented plate (or web) section and a horizontally-oriented base. Four bolt holes are located in the base which allows the truss support to be bolted to a vertical support column. The web passes between two diagonally-oriented members and is, by design, fillet welded to those members. The top of the shape is butt welded to the top horizontal member. The primary difference in truss design between the auditorium and gym roof trusses is that the length of the butt weld to the horizontal member from the gym roof was located below and within the diagonal member while most of the length of the auditorium truss weld was located above the diagonal member. Full penetration is required for this structural weld under ANSI/AWS D1.1. The plate section also was not gusseted. Gussets are perpendicularly-oriented which would have provided lateral support to the plate.

The gym roof trusses had been left in-situ and been exposed to ambient outside elements after the roof collapse. The trusses were specified to have been made from ASTM A36 structural steel. The high school building had been constructed in 1974.

The purpose of this investigation is to characterize both the observed fractures and the structure and material used in the truss support.

CONCLUSIONS

- The truss support collapsed and eventually fractured as a result of a failure mechanism of buckling.
- The support did not meet as-manufactured mechanical property requirements. Specifically, the average yield strength (yield point) measured 32.8 ksi compared with a specified minimum yield point of 36 ksi, a 9% deficiency.
- The butt weld between the support and the horizontal member was not a full penetration weld, a violation of ANSI/AWS D1.1 for structural welds.
- One area of the butt weld also exhibited apparent lack of fusion along one side of the weld.
- Contact areas between the support and the horizontal truss member were poorly prepared. This probably was a major cause for the incomplete weld penetration.
- The lack of gussets on the truss support likely contributed to failure as this enabled deflection of the top of the truss relative to the support column.
- Support composition met the compositional requirements of the specified ASTM A36 material.
- Microstructure was a combination of ferrite and fine pearlite, typical and expected for this material.
- Average support truss hardness measured 76 HRBW.

RESULTS

Both the southwest and northwest ends of the truss, including the supports, were provided by FORCON for evaluation. Focus remained on the southwest end based on observation that failure was occurring initially on that end of the truss prior to building evacuation. This report documents only the southwest truss support.

The as-received truss base is shown in Figure 4*. This section consisted of the base and part of the plate section. The top of the plate had fractured completely along its length. (We have identified this as Fracture 1). The plate was bent to one side of the base and the fracture was located below the height of the base. We assume that the base, which had been firmly bolted to the column, had not moved during the roof collapse based on relative locations in Figure 2. Figure 3 thus indicates the plate section had grossly deformed to result in a section of plate bent below the base at the time of final fracture and liberation from the rest of the plate. Evaluation of the fracture surface (Figure 5) indicated the plate had come in contact with the base. Fracture

* When appropriate, photos have deliberately been rotated to mimic original orientation of components.

appeared to have initiated at the first point of contact. River markings on the fracture revealed the fracture likely had been a rapid overload fracture and had propagated in both directions as shown in the figure. The origin location indicates this surface had been in tension at the time of fracture.

Figure 6 shows the end of truss and support on the truss side of Fracture 1. This figure is analogous to the top photo in Figure 4 but was taken from the opposite side (i.e. the southwest side). The support in end-view was J-shaped, with Fracture 1 located at the bottom of the J. The morphology suggests the support plate had deformed to an S-shape prior to fracture. Fracture had occurred in the plate base material.

Side views of the truss end are shown in Figure 7. A second fracture was observed in the plate. This fracture ran along the toes of the fillet welds to the diagonal, and then partially along the butt weld before it ended. This fracture (Fracture 2) intersected with the end of Fracture 1 that was located at the diagonal. We did not see any directionality in Fracture 2. We believe this had been rapid fracture that had propagated in the short transverse direction.

Closer views of Fracture 2 along the butt weld are shown in Figures 8 and 9. We observed that the section had twisted inward in this area such that Fracture 1 was oriented over 90º relative to Fracture 2. Fracture 2 in this area showed an area with a “ridge line” in the center of the fracture that was oriented parallel to the weld surfaces. This section of Fracture 2 was radiographed but no indications were observed. We also noticed one area in this region which did not appear to be fractured. Closer examination (Figure 10) revealed what appeared to be machining marks on half of the surface, with only a miniscule fracture area at its surface. The machining marks indicate we were seeing a section of weld where no fusion had occurred during welding (which would have obliterated the machining marks and resulted in fracture if the area had been fused).

Two metallographic cross-sections had been prepared through the butt weld. One had been in the lack of fusion (LOF) area while the other had been prepared through an intact weld area. Samples were mounted in polymeric compound, ground, polished, and etched to reveal structure. The LOF region showed almost no fusion on one side of the weld (Figure 11). Full fusion was achieved on the other side. Curvature indicates the LOF side had been in tension prior to fracture completion. A crack mid-width had initiated at the root of the weld. We think the tension in bending resulted in a crack starting at the root of the weld on the compression side, resulting in the crack that was exhibited in the form of a “ridge line”. This crack had only progressed a short distance before final fracture running along the weld fusion line was observed.

The second cross-section was prepared through an area of unfractured weld. A macroscopic view revealed incomplete penetration across the weld, with a small triangular region of unfused metal between the welds. The footprint of this region was too small to see when the weld was radiographed. The left side appeared to have had a single bevel which had run along its thickness while the right side did not appear to have been prepared at all or, at best, had been beveled only on the bottom corner. The observed lack of full penetration in both weld cross-sections is a violation of ANSI/AWS D1.1.

General microstructure was observed from the support plate from the metallurgical mounts. Etched microstructure (Figure 13) was comprised of a combination of ferrite and fine pearlite, typical and expected structure for ASTM A36 steel.

Chemical composition of the support was quantitatively evaluated. Composition is shown in the following table:

Table 1: Truss Support Chemical Composition (Weight %)

Element	**Truss Support**	**ASTM A36-81a***
Carbon	0.18	0.26 max
Phosphorus	0.006	0.04 max
Sulfur	0.022	0.05 max
Copper	0.02	0.20 min when Cu-steel is specified
Manganese	0.62	Not specified

* Oldest version of specification available at Lehigh Testing Laboratories.

The support met the compositional requirements for ASTM A36 shapes.

Evaluation was performed in the LOF region of Fracture 2 in a scanning electron microscope. While no new fractographic information was revealed, we did examine surfaces for semi-quantitative composition using energy dispersive spectroscopy. External contaminants were found including calcium, silicon, and trace amounts of sulfur, silicon, potassium, and aluminum were found on the LOF and on adjacent fracture surfaces. We believe these represent dirt and sand exposure which had occurred from exposure after failure.

Two 2”-gage length tensile specimens were machined from the base section of the support and tested per ASTM A370. Unlike the plate section, the base had not seen deformation from failure. This allowed us to produce valid test results. Tensile properties are shown in the following table:

Property	**Truss Support #1**	**Truss Support #2**	**ASTM A36-81a**
Ultimate Tensile Strength (ksi)	59,500	59,700	58 – 80
Yield Point (ksi)	32,700	32,900	36 min
% Elongation in 2”	37	37	21 min
% Reduction of Area	64	63	Not specified

The support did not meet yield point specifications. In this instance yield point and yield strength are the same.

Rockwell B hardness of the support in the plate section was measured. Average hardness measured 76 HRBW.

DISCUSSION

The mechanism of failure was buckling of the truss support. The support buckled in the plate region, pulling the top member and diagonal of the truss to one side. This meant that loading no longer was centered on the support column, which aided in plastic deformation. The support deformed into an S-shape when viewed from the end. The plate buckled to the point where a portion had bent below the support and contact load with the support coupled with tensile loading caused by the bending resulted in rapid overload fracture at Fracture 1. We consider Fracture 2 to be a secondary fracture, but both fractures were induced by the high strains and gross deformation caused by buckling of the plate.

We noted three major deficiencies which at least contributed to failure.

First was the lack of gusseting. The support design damped dynamic loading from the roof in the longitudinal direction. However, lack of gussets meant there was no lateral support for dynamic loading. An example could have been load shifting of roof snow caused by winds. This could have the effect of causing displacement of the support during the roof's 36-year lifespan. Deviation from a completely vertical alignment will have the effect of increasing stress as the amount of supported cross-sectional area is decreased.

The second major deficiency is the lower-than-specified yield strength. Of specified mechanical properties, this is most important as it defines the stress where plastic deformation begins. Yield strength was 9% below minimum requirements. This is a serious deficiency as the steel used should have been certified to meet strength requirements prior to construction.

The third deficiency had been the quality of welding. The butt weld between the horizontal member and the support plate violated requirements of ANSI/AWS D1.1 structural steel welding because full penetration was not achieved. The LOF we observed probably was not visible after welding as a miniscule area of surface weld metal was present so surface inspection techniques such as magnetic particle or penetrant inspection would not have detected it. Lack of full penetration compromises structural integrity because the load-carrying area is reduced. Since stress = load/area, reduction of loaded area will increase the local stress. This would not have been of concern after initial installation because the unsupported portion of the horizontal member would have vertically loaded onto the plate. However, deflection from dynamic lateral loading could have resulted in bending at the weld as cross-section is even more severely reduced due to the incomplete penetration. One major contributing factor to the observed lack of penetration was poor preparation of the surfaces to be welded. Several geometries result in complete fusion, including those shown in Figure 14. Most common would have been to double-bevel both surfaces so weld metal had complete access to all surfaces.

We do not know if the gymnasium would have remained intact had these deficiencies not been present. However, we do consider all three to be serious structural deficiencies for 1974 construction.

Thank you for giving us this opportunity to assist you with this investigation. Should you have any questions regarding the information provided, please contact us.

Sincerely,

Lehigh Testing Laboratories, Inc.

Aaron Tanzer

Aaron Tanzer
Principal Engineer

PS: Please note that we will retain the component(s) sent for evaluation for one year and then discard unless you provide us with an alternate disposition.

Figure 1: Cell phone picture taken during roof failure shows separation between the top member of the truss and the roof (arrow) on the southwest side.

Figure 2: Cell phone picture taken during roof collapse shows southwest side top member pulling to the left side of the column, compared to the bottom member which was located in the center of the column.

Figure 3: Truss in auditorium illustrates construction geometry. This support uses the same design as the gym roof support but is smaller.

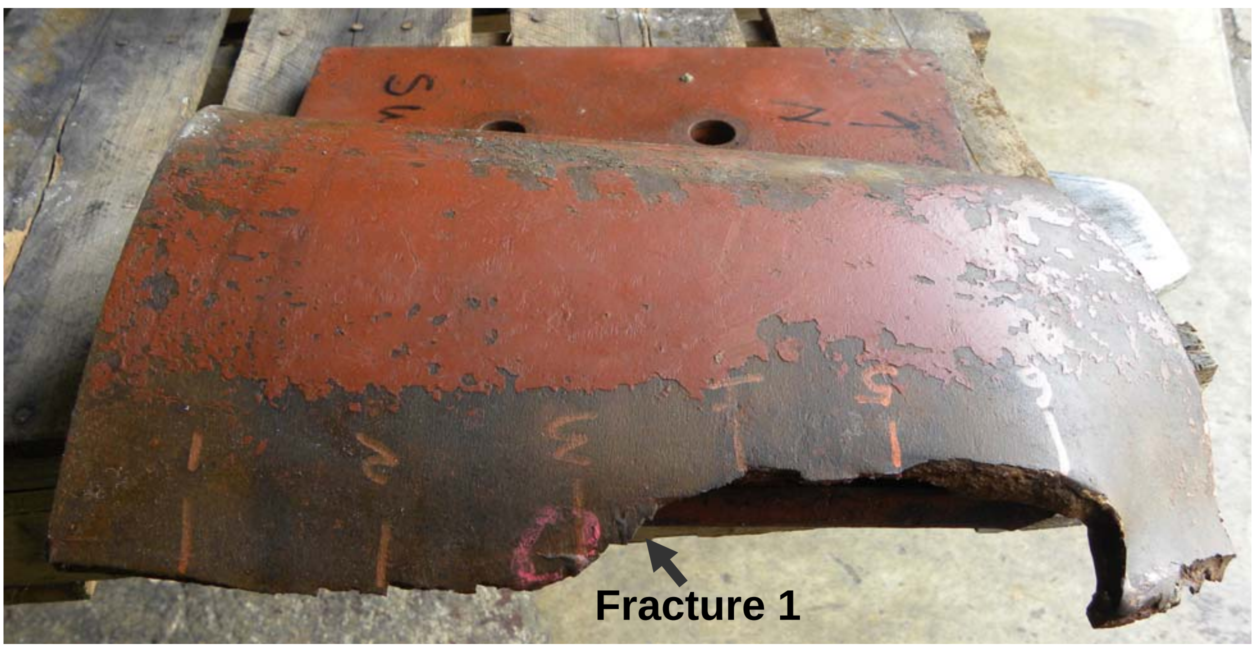


Figure 4: Base section of support after fracture. Note how web has collapsed onto base. (Note: photos, when appropriate, have been rotated to match original as-constructed orientation).

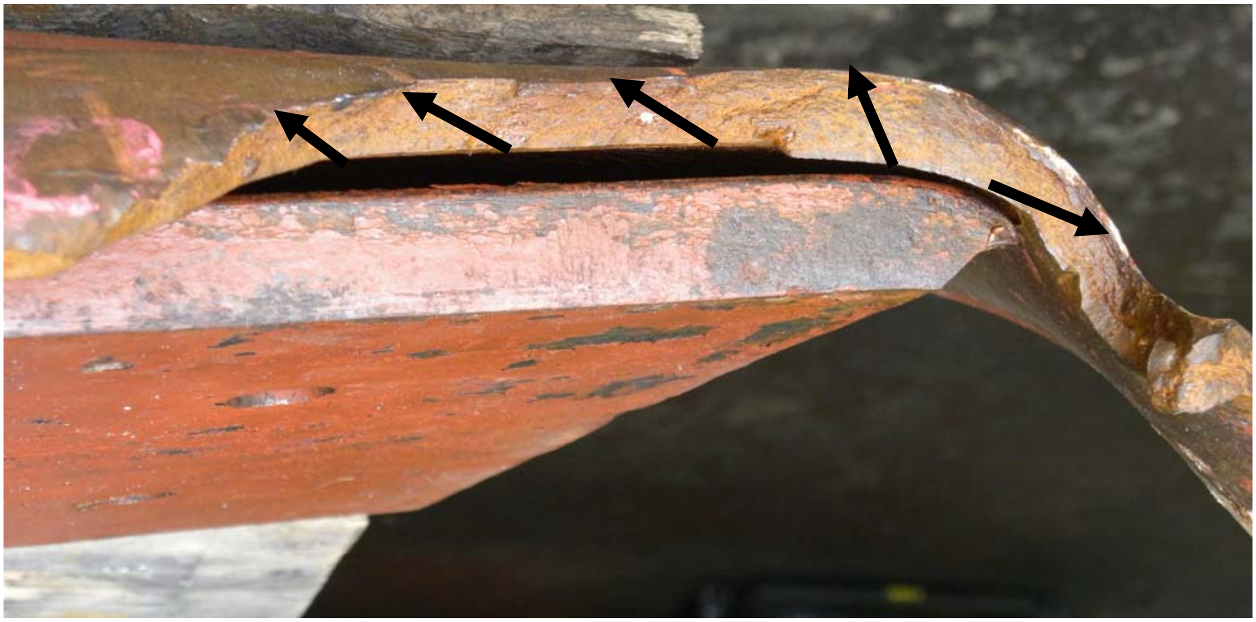


Figure 5: Close-up view of Fracture 1 on base side of fracture. Arrows show directions of fracture propagation. Morphology is consistent with fracture morphology after severe buckling has occurred.

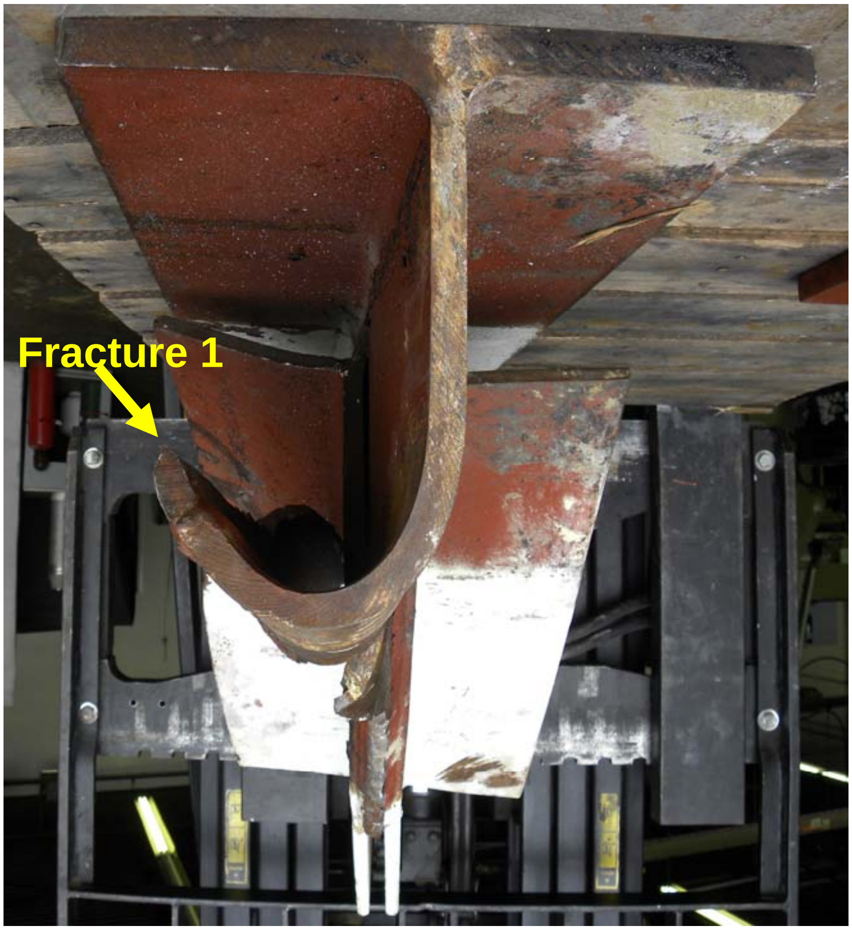


Figure 6: End view of as-received top truss member. Note almost complete bending of web on support member.

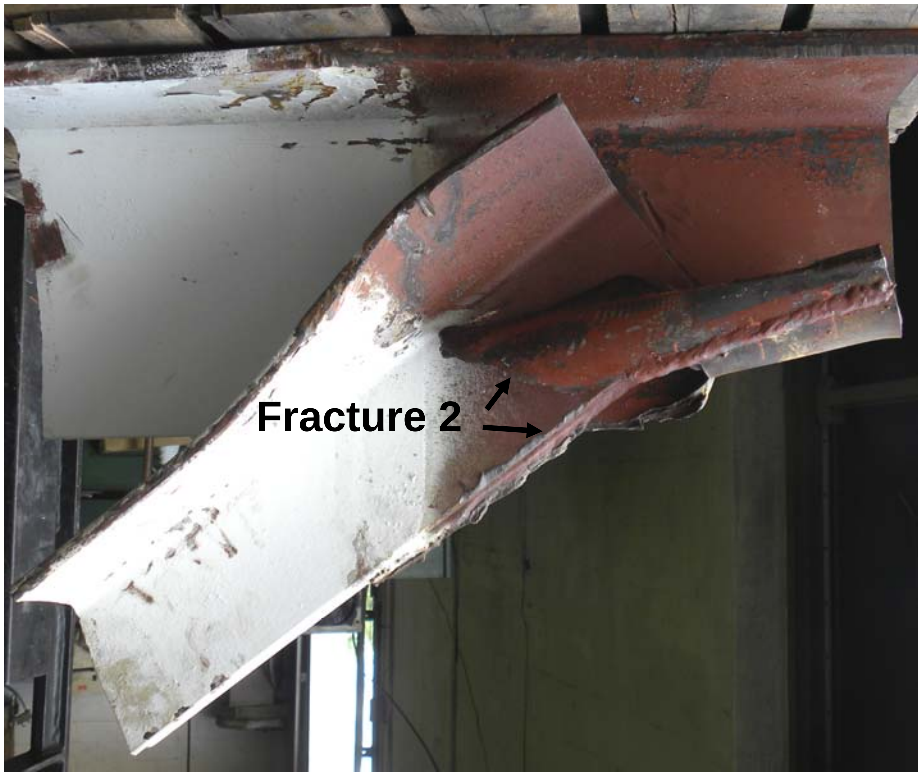
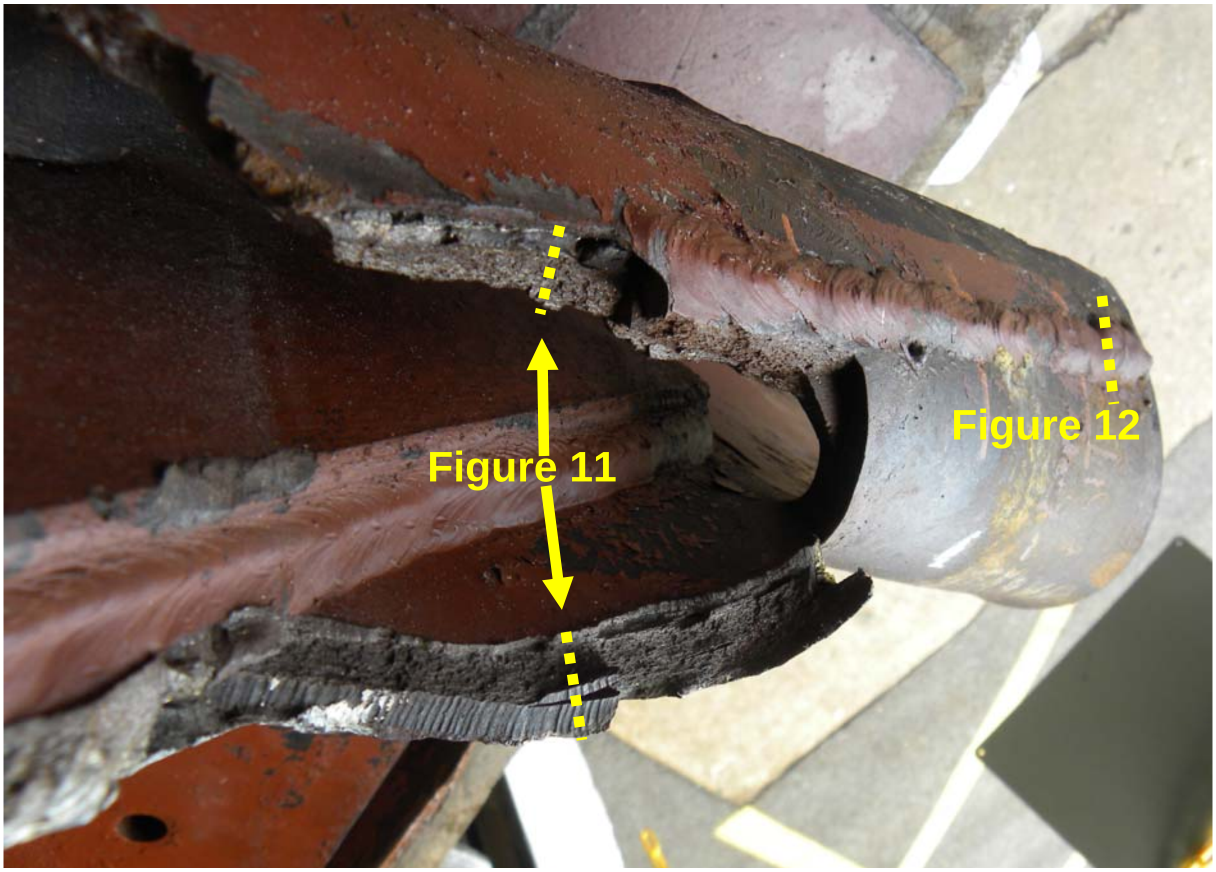


Figure 7: Top truss member as viewed from the sides. Arrows point to Fracture 2 on weld.



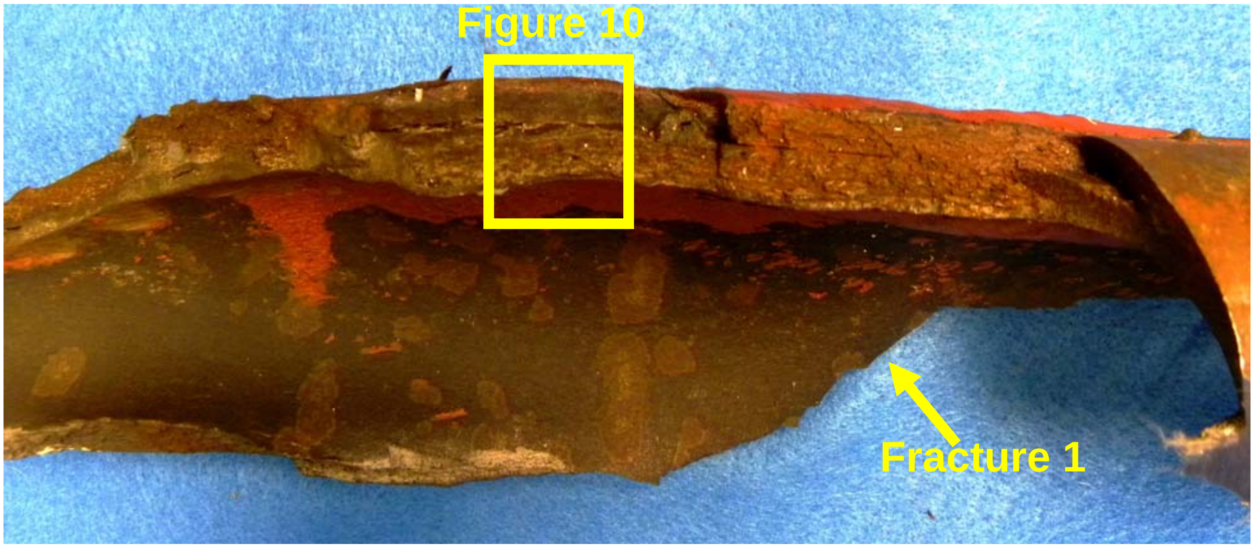


Figure 8: Closer views of Fracture 2. Note “ridge line” in center of fracture in bottom photo.

Figure 9: Fracture 2 ridge lines.

Figure 10: Macroscopic view of area outlined in Figure 9 shows a lack of fusion above the ridge line, as indicated by parallel machining marks which were on the edge of the original member being welded.

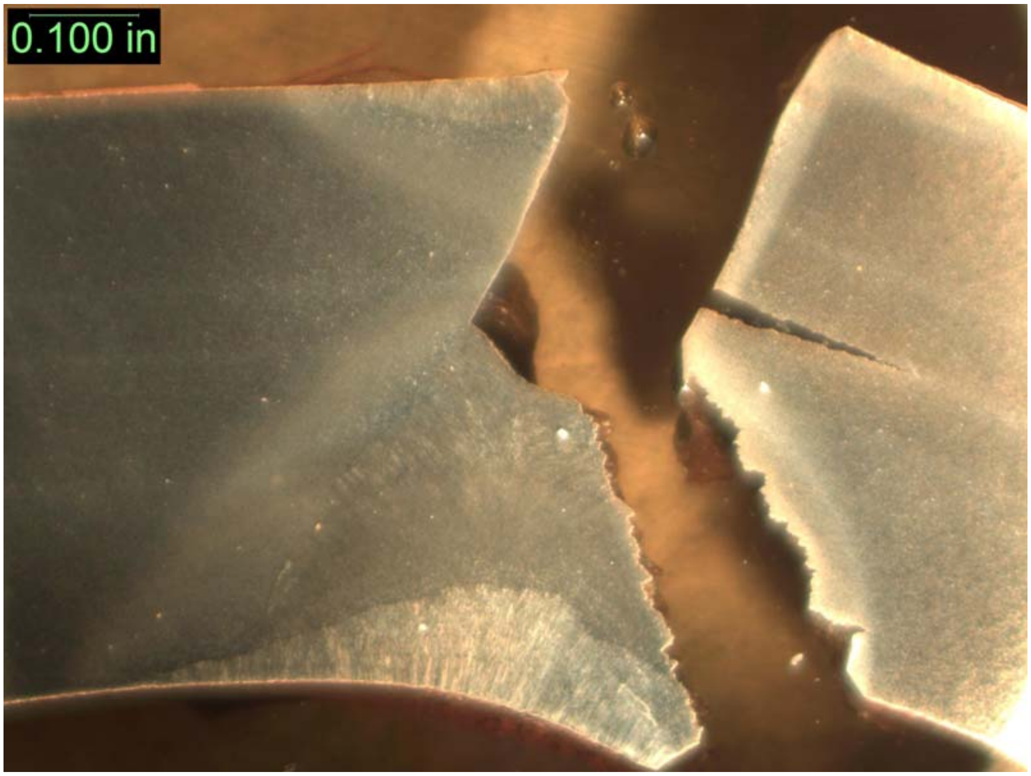


Figure 11: Lack-of-fusion area cross-section from Fracture 2. Note crack at root of weld running parallel to surfaces. Etch: 2% nital.

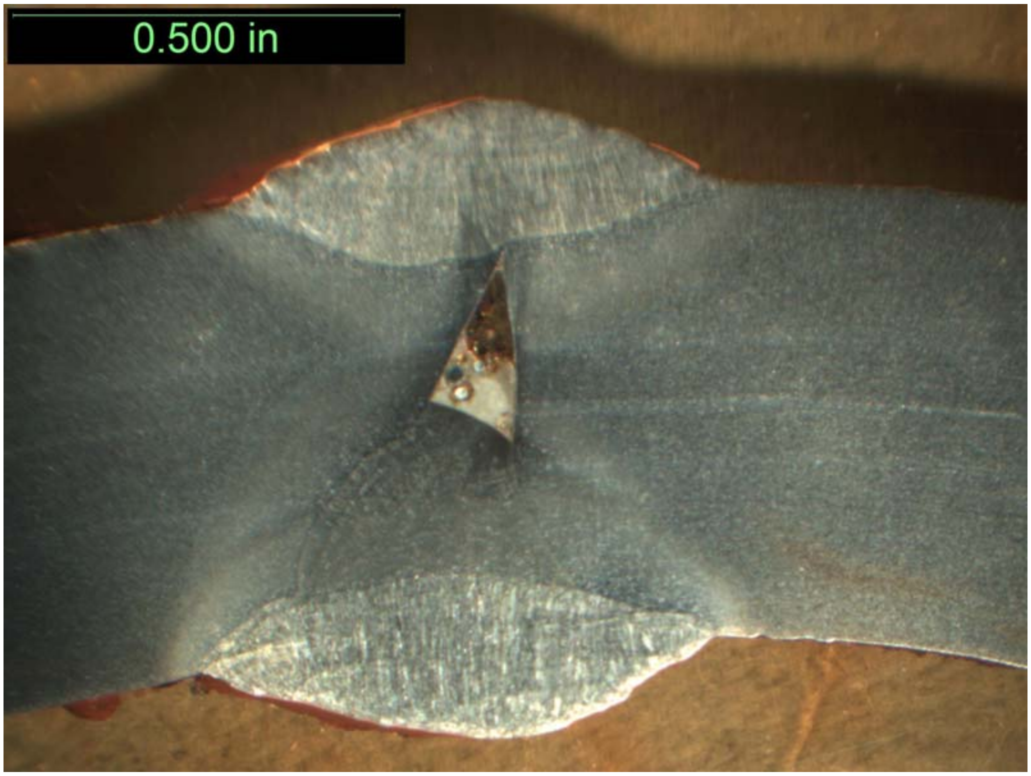


Figure 12: Cross-section prepared through intact weld area. Note lack of penetration between welds and curvature showing top weld was in tension while bottom weld was in compression during buckling.

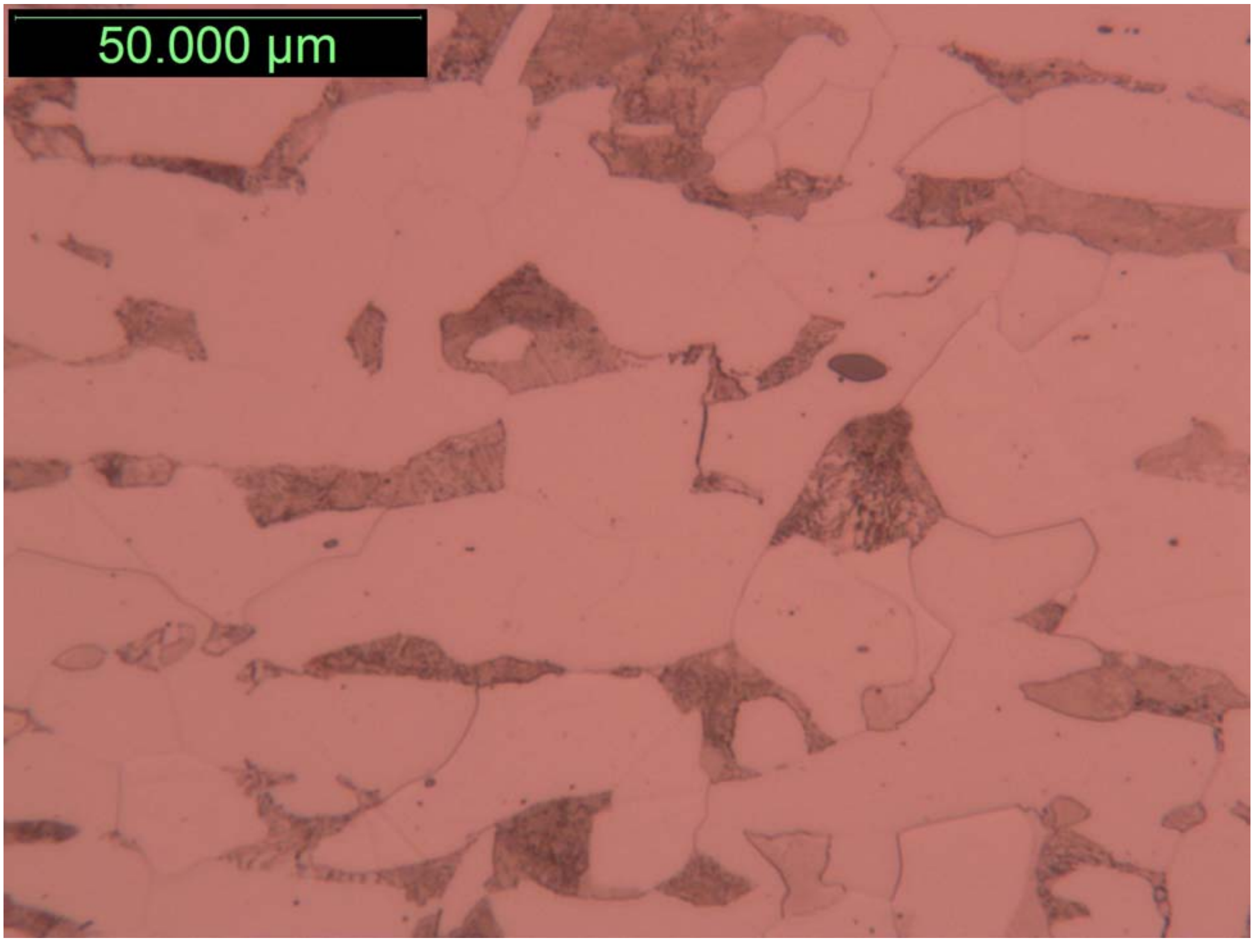
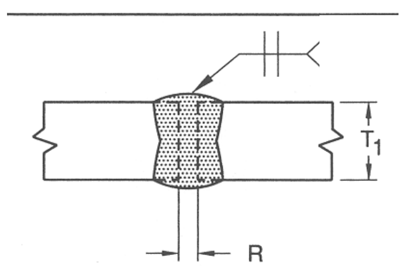
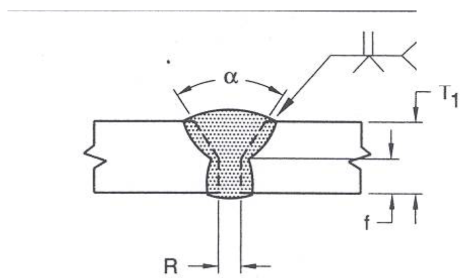


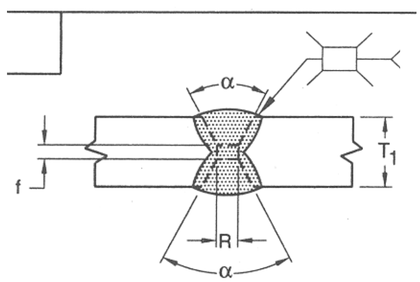
Figure 13: Support base metal microstructure. Structure was comprised of ferrite (light) and fine pearlite, typical and expected for ASTM A36 structural steel. Magnification: 1000X. Etch: 2% nital.



Square-groove weld



Single V-groove weld



Double V-groove weld

Figure 14: Acceptable butt weld geometries from ANSI/AWS D1.1.