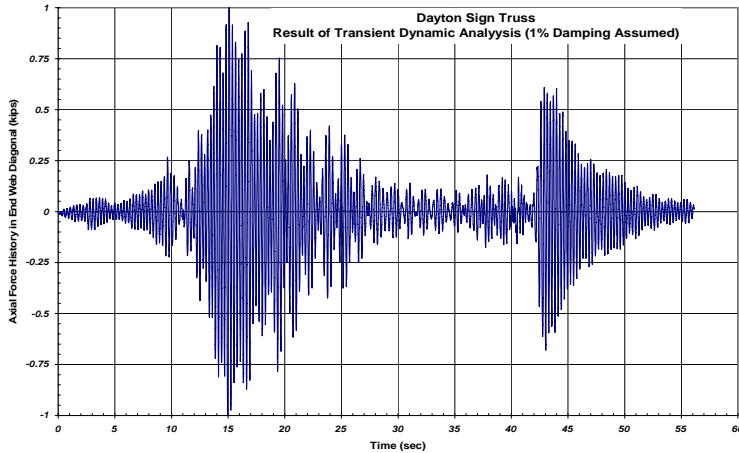


# Investigation of the Dayton IR 75 Sign Truss Failure of 9/11/06



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Report for the Ohio Department of Transportation  
Office of Traffic Engineering  
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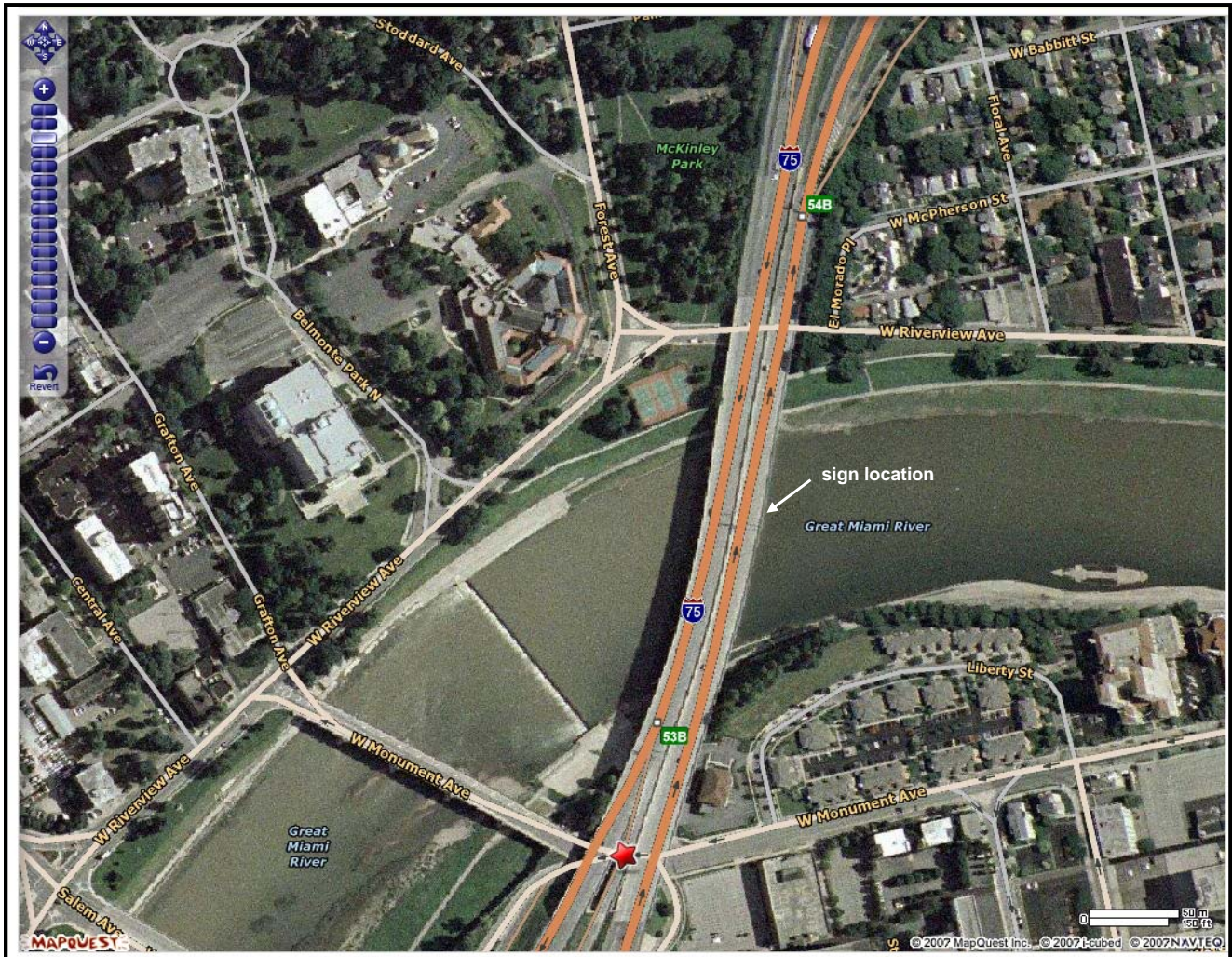
## Executive Summary

Based upon a combination of in-situ field monitoring of traffic-induced bridge vibrations at the location of the failed sign support truss, finite element simulation of the expected dynamic response of the original truss in such an environment, the length of service of the truss at the time of its failure, the volume of truck traffic on the bridge in question during that time of service, and metallurgical examination of the failed components of the sign truss, the conclusion of the investigation is that extremely high-cycle fatigue of the chord/web diagonal welded connection was the cause of the truss failure. The in-service effective stress range of the AASHTO Category ET connection in question was most likely below the currently specified AASHTO constant amplitude fatigue limit (CAFL) for the detail, but the enormous quantity of response cycles (approaching or even exceeding 1 billion cycles) such a bridge mounted sign on a heavily traveled route accumulates over a service lifetime of 30 or 40 years, exceeds anything currently considered in the design codes for such structures. The implications of this fact on the current inventory of such structures in similar installations and of the same age range, i.e., installed with the original interstate routes, largely in the 1960's, are obvious.

The one somewhat puzzling aspect of the lower chord fracture of this structure, namely that the fracture passed near the weld toe but not actually contacting the weld toe, does not appear to be particularly significant. The fracture did indeed pass close to the weld and certainly within the heat affected zone of the weld. The fracture surfaces were too abraded to positively identify the initiation point; small cracks, however, were also identified within the weld under microscopic examination. The original flaw which eventually propagated and produced the rupture, apparently in this case, just happened to be near the weld rather than in the weld.

## Background

On the afternoon of September 11, 2006, a structural failure was discovered by ODOT District 7 personnel on an aluminum sign support truss, located on the Great Miami River IR75 Bridge in Dayton, OH (see Figure 1). The support truss was of a 4

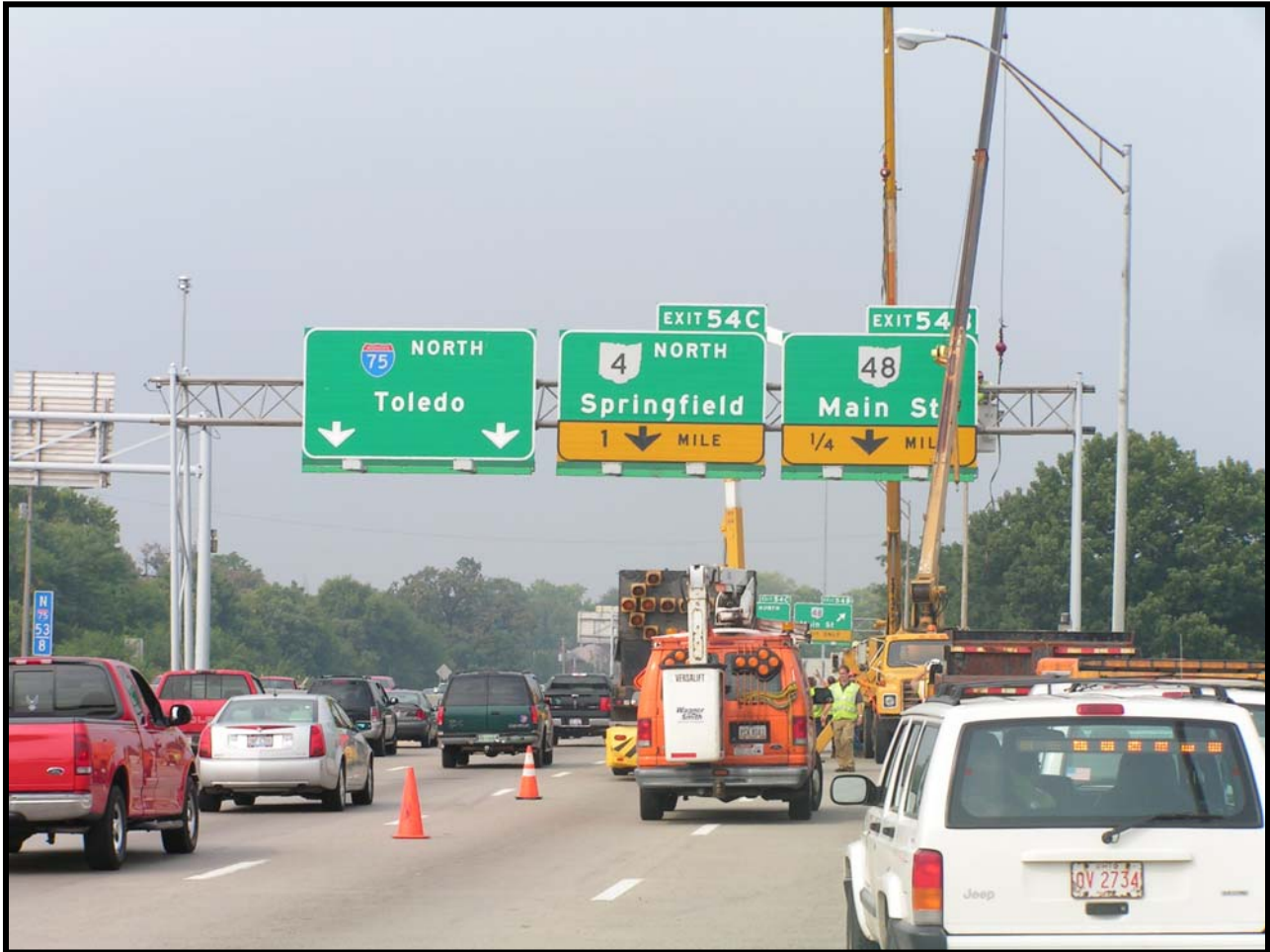


**Figure 1: Location of Sign Support Structure**

chord, 3 ft x 3 ft welded box truss configuration; the chord members were 4.75 in O.D. x 0.188 in wall pipe sections, the web diagonals were 1.66 in O.D. x 0.14 in wall pipe sections and the internal and chord diagonals were 2.00 in O.D. x .188 in wall pipe sections. The truss span was 64.93 ft, running from parapet to parapet of the northbound 4 lane expressway. The sign truss was mounted near the midspan point of the 9<sup>th</sup> span (spanning from pier 8 to pier 9) of the 12 span bridge carrying northbound IR 75 over the Great Miami River in Dayton, OH. The bridge piers have a right forward



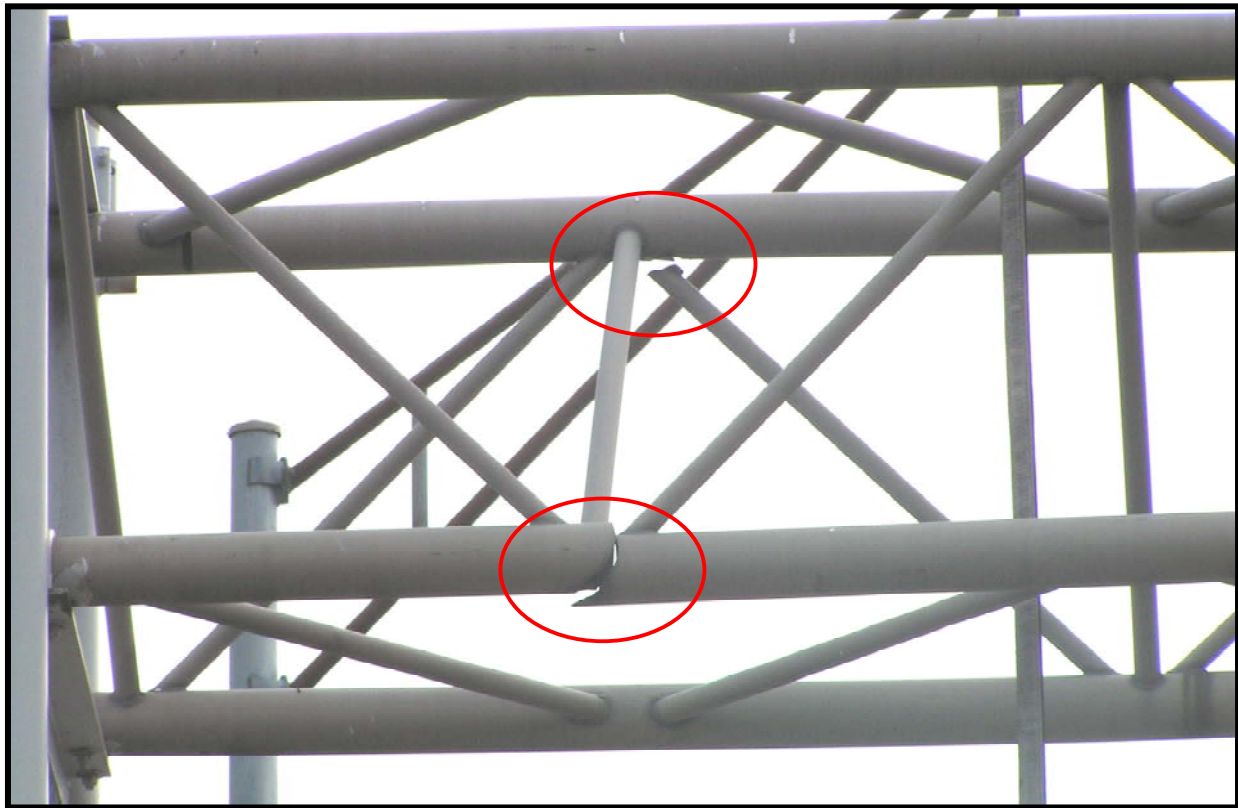
The sign truss carried a payload consisting of 3 signs (see Figure 3), totaling approximately 468 ft<sup>2</sup> of signage, weighing approximately 1170 lbs. Allowing for lighting hardware, wiring, etc., it was presumed, for the dynamic vibration analysis of the sign truss, that it carried a total payload of 1500 lbs. The design of this stretch of IR75 was completed in 1963; for the fatigue analysis of the sign support truss, the time in service, at the time of failure, was assumed to be approximately 40 years. The observed failure consisted of the complete fracture of a bottom chord member near the eastern end of the truss and the pullout of an adjacent web diagonal member from the diagonally opposite top chord member (see Figure 4).



**Figure 3: Sign Truss at the Time of Its Removal**

### **Inspection of Truss Remnants on 11/16/06**

On Thursday, November 16, 2006 the authors viewed the remnants of the failed sign structure at the District 7 Dixie Outpost, where it had been stored since its removal from the Dayton IR75 Bridge. The truss at this time had been separated into two separate segments at the flanged midspan chord splices, and the signage had been removed (see Figure 5).



**Figure 4: Damage Locations of the Failed Sign Support Truss**



**Figure 5: Remnants of Sign Truss at the District 7 Dixie Outpost**

Close-up views of the fractured chord are shown in Figures 6 and 7. The crack obviously propagated completely around the circumference of the chord cross section, passing closely to, but not actually contacting, the toe of the fillet weld of the diagonal/chord connection. This type of tubular chord to tubular diagonal welded connection is categorized as an AASHTO category ET fatigue detail, in the *Standard Specification for Structural Supports for Highway Sign, Luminaire and Traffic Signals* (AASHTO, 2001).

Category ET is lowest of the AASHTO fatigue categories in the *Sign and Luminaire* specification, assigned a constant amplitude fatigue limit (CAFL) of only 0.44 ksi (axial stress measured in the diagonal) for aluminum members. The CAFL criteria published in the AASHTO *Sign and Luminaire* specification is intended to provide for adequate fatigue lifetimes in the normal wind load environment to which sign and luminaire support structures are subjected. The traffic-induced seismic fatigue environment, to which bridge-mounted sign and luminaire support structures are subjected, however, is not specifically addressed in the specification.



**Figure 6: Close-Up View of Fractured Lower Chord Section**



**Figure 7: Additional Close-Up View of the Fractured Lower-Chord Connection**

The fracture surfaces shown in Figure 7, in particular, appear to be rather badly abraded, presumably from impacts and rubs suffered during the load cycles after the crack initiated and while it propagated to the point of failure. Nevertheless sections of both the upper and lower chord members and connecting diagonals at the failure locations were removed for later microscopic metallurgical examination and analysis at CWRU. The upper chord failure surface, shown more closely in Figure 8, did obviously pass through the chord/web diagonal welded connection.





**Figure 8: Close-Up View of the Failed Upper-Chord Connection**

### **Field Test of 12/10/06**

On Sunday, December 10, 2006, traffic-induced vertical accelerations were measured at the bases of both of the towers (see Figure 9), where the sign support structure had previously been mounted, on the IR75 bridge over the Great Miami River in Dayton, OH. Accelerations were measured at a 100 hz sampling rate, and digitally recorded during passages of an ODOT tandem axle dump truck, weighing approximately 50 kips, as well as a number of other trucks of unknown weights, in the usual Sunday morning traffic stream. Accelerometers were mounted at both sign support towers, with an instrumentation cable temporarily suspended overhead from the west tower accelerometer to the data logger, located on the shoulder at the east tower (see Figure 10).



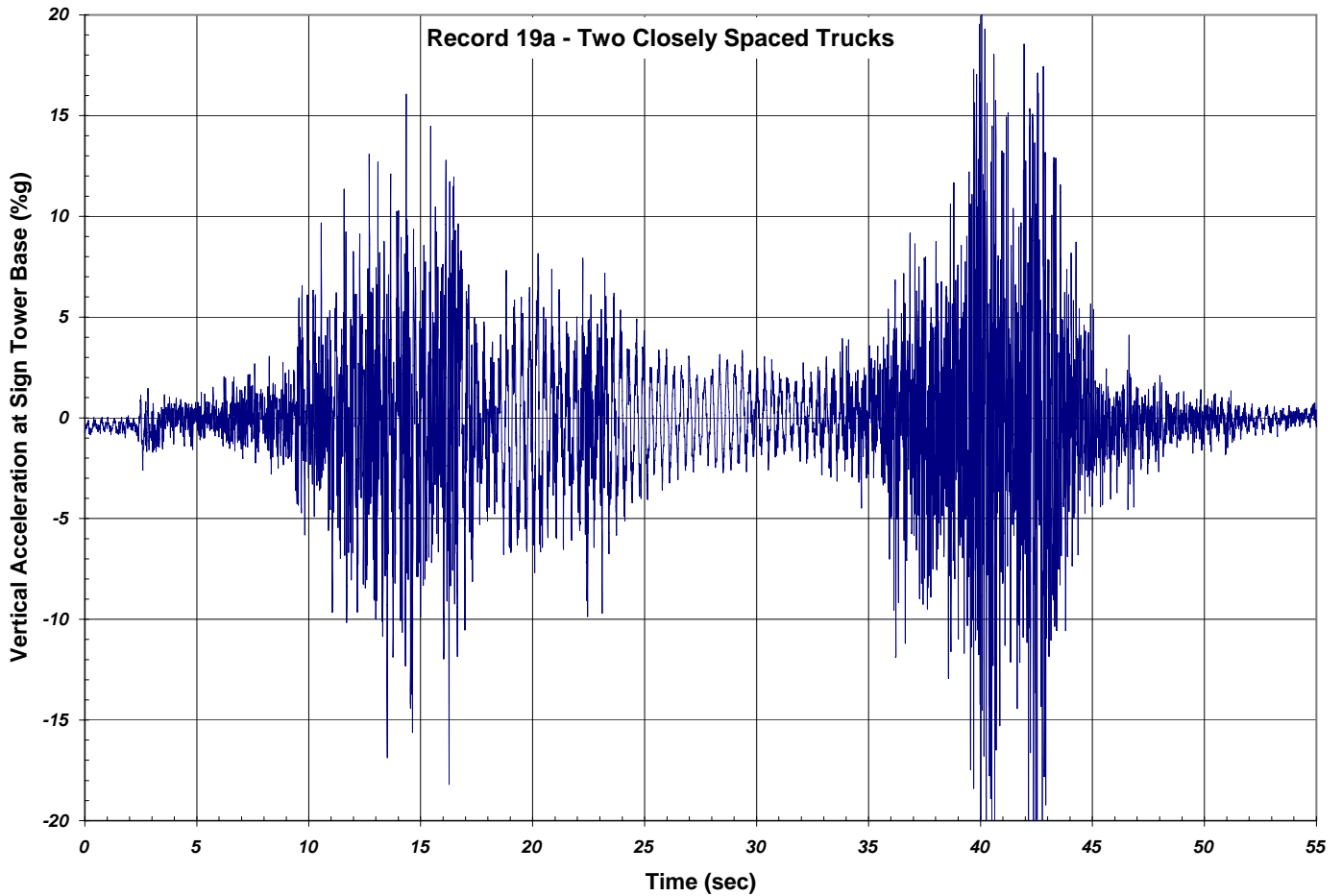
accelerometer mounted on  
sign tower baseplate

**Figure 9: Accelerometer Mounted on Tower Base**



**Figure 10: Instrumentation Cable Suspended Between Sign Towers**

Traffic-induced vertical accelerations of the bridge, at the location of the sign tower bases, were monitored for approximately a 2 hour time interval. During that time, passages in each lane were monitored for the loaded ODOT tandem axle dump truck made available for the test, as well as for the usual flow of traffic for the bridge on a Sunday morning. (The bridge was opened to general traffic once the installation of the instrumentation was completed.) One such acceleration record, made during the passage of two closely spaced (non-ODO) trucks is shown in Figure 11.



**Figure 11: Representative Traffic-Induced Bridge Acceleration Record**

As can be seen in Figure 11, measurable vertical vibrations produced by a single heavy truck passage can easily extend over a time interval on the order of 15 seconds or more in duration, and exhibit peak acceleration amplitudes from 10% to as high as 20% of gravity. The frequency content of this traffic-induced vibration will tend to be concentrated around the natural frequencies of the bridge in vibration modes excited by the traffic. Typically, the fundamental flexural and torsional modes will tend to dominate the traffic-induced response; vehicles in the center lanes will excite the flexural mode primarily, while vehicles in the outer lanes will have a greater tendency to excite the torsional mode. For a typical slab on girder bridge, these two response modes exhibit

very closely spaced natural frequencies. The frequency content of the vertical vibrations measured on this bridge were observed to be concentrated around a frequency of 3 hz. An ensemble of 19 separate truck passage records were preserved and utilized for the subsequent dynamic FE analysis of the failed sign structure.

### Results of Dynamic Vibration of Analysis of Sign Truss

A finite element model of the failed sign structure was created utilizing the general purpose structural analysis software package SAP2000. The sign support structure, including the towers, was modeled with elastic 3D frame elements. Continuity was assumed for all structural connections; added mass totaling 1.5 kips of weight, representing signage, luminaires, wiring, etc., was distributed along the top and bottom chords of one vertical face of the truss. The aluminum truss members were assumed to have an elastic modulus of 10,000 ksi and a weight density of 0.10 lb/in<sup>3</sup>. The steel tower members were assumed to have an elastic modulus of 29,000 ksi and a weight density of 0.283 lb/in<sup>3</sup>. Nominal cross sections and dimensions were taken from the ODOT standard drawing (see Figure 2) as well as field measurements taken from the actual truss during the Dixie Outpost inspection of 11/16/06. The resulting FE model of the sign support structure is shown in Figure 12, and the first three natural modes of vibration are shown in Figure 13..

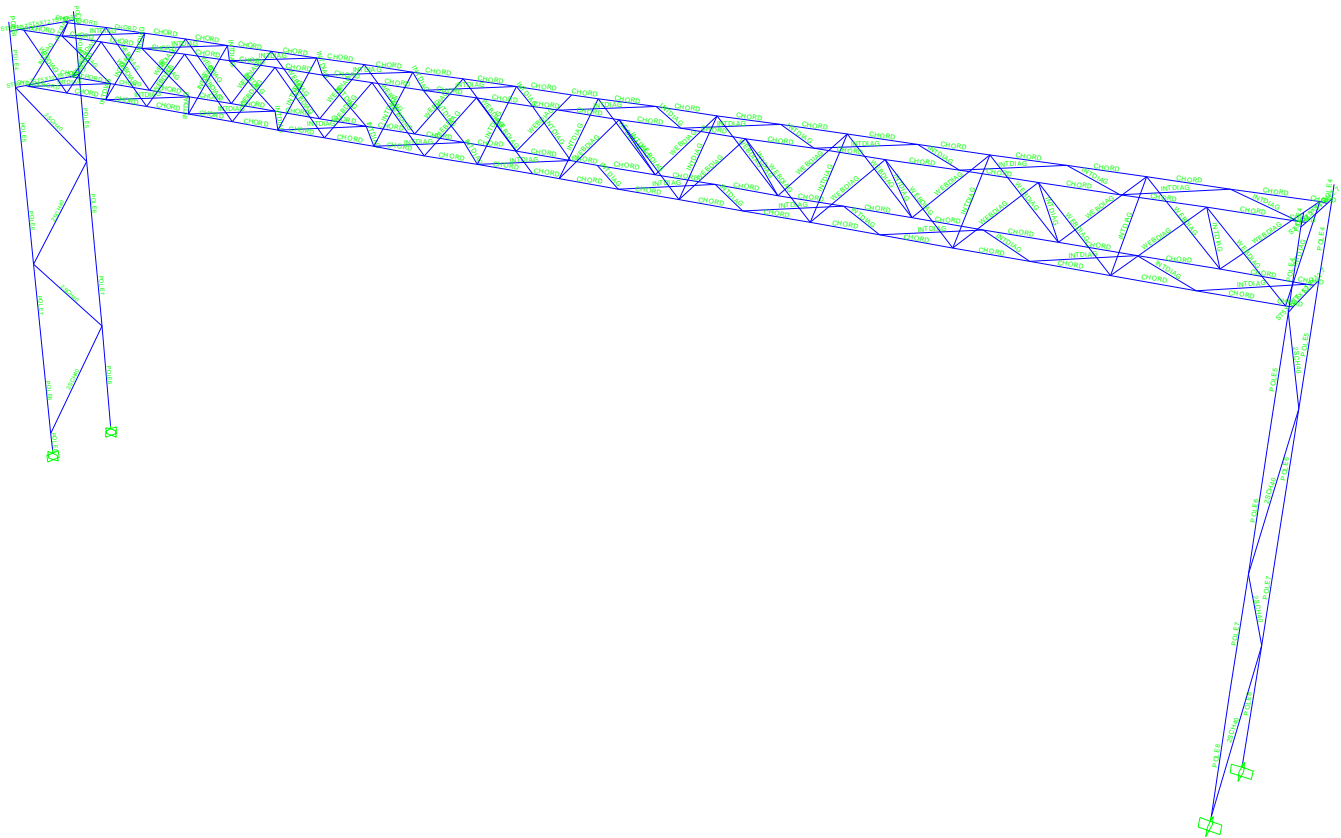
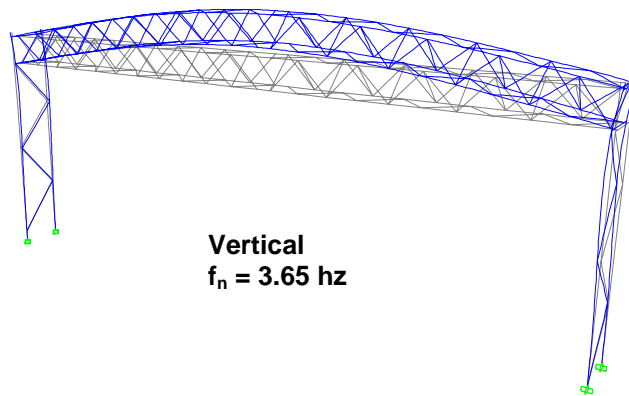
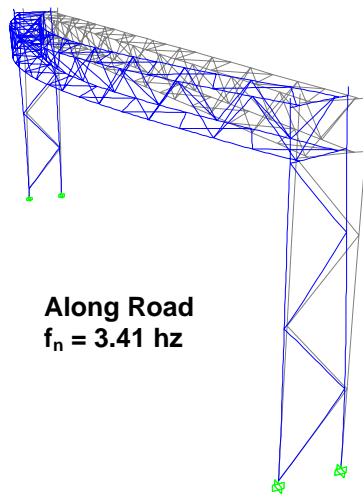
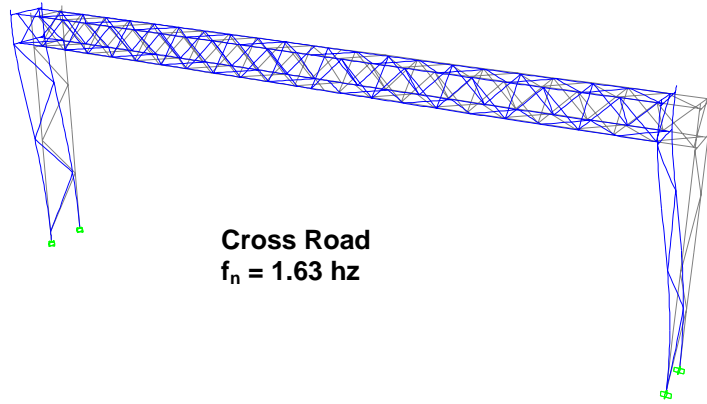
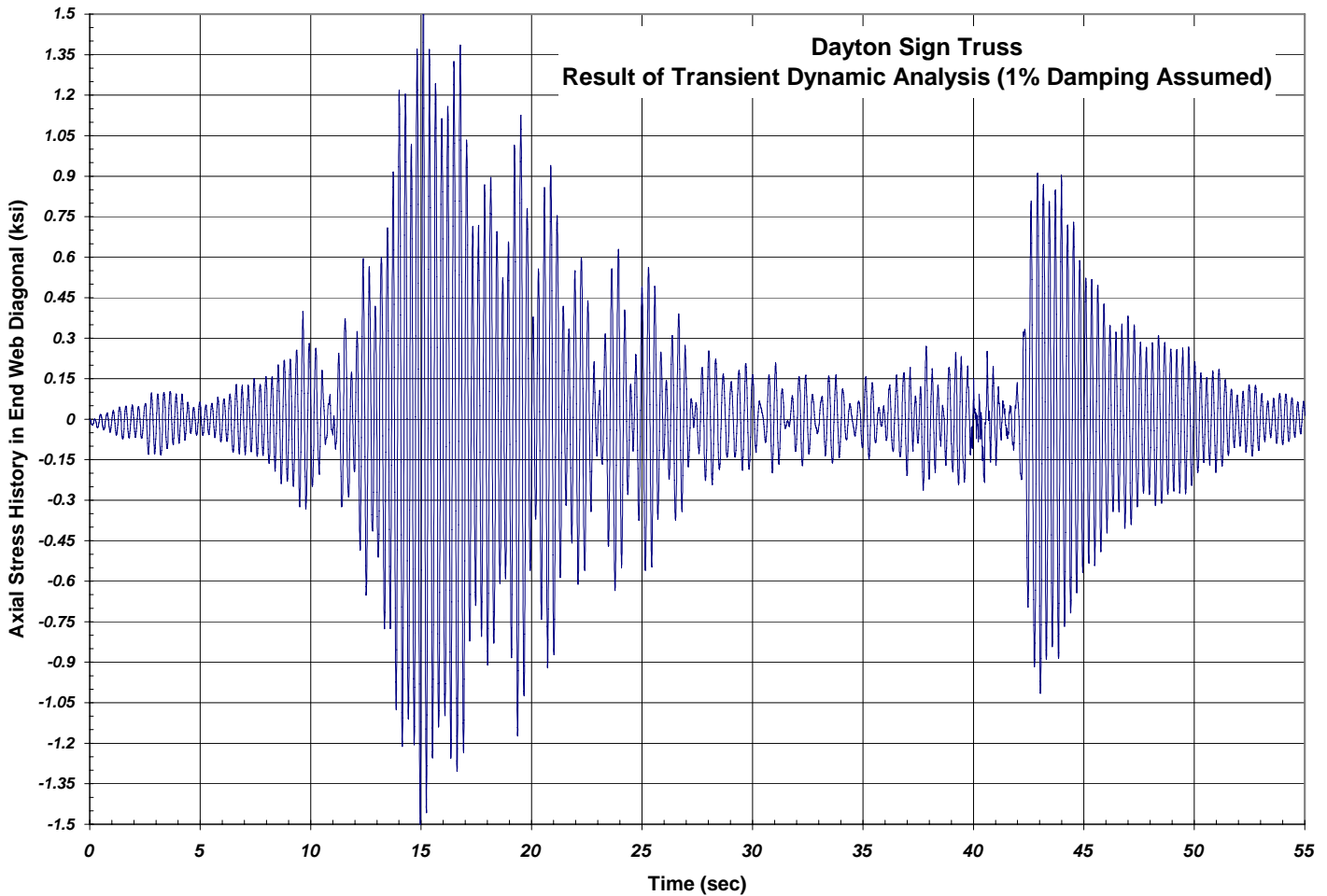


Figure 12: FE Model of Sign Truss



**Figure 13: First Three Natural Modes of Vibration of Sign Truss**

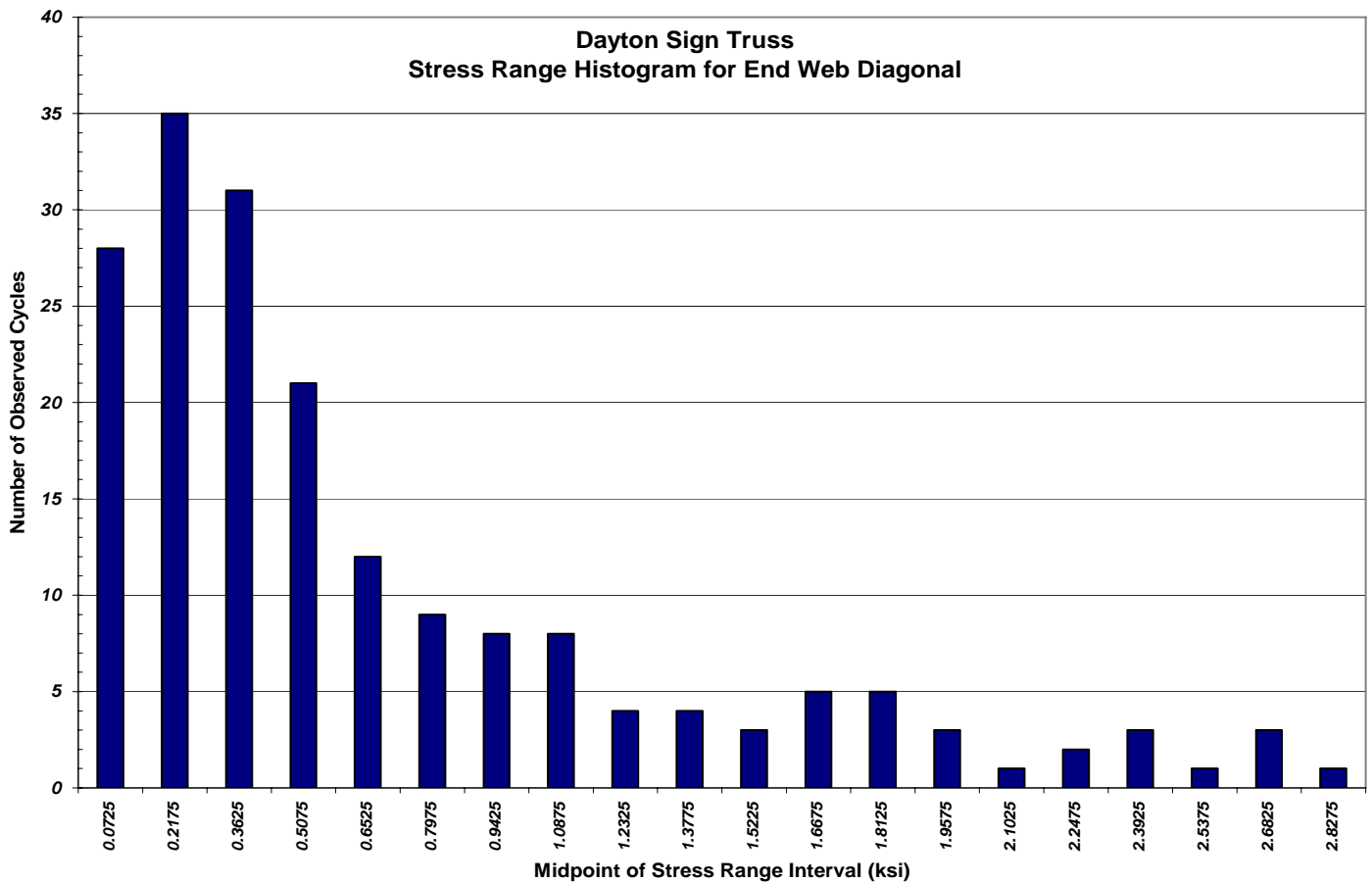
The FE model depicted in Figure 12 was subjected to seismic excitation, consisting of vertical acceleration time histories applied to the support points at the tower bases. The acceleration time history records utilized were those recorded during the field tests of 12/10/06. A damping level of 1% of critical was assumed for all the dynamic analyses. The primary response quantities of interest were the axial stress levels in the web diagonals nearest the support towers, as they represent the most critical fatigue detail (AASHTO Category ET), subjected to the highest stress amplitudes. A sample record of such a response time history, computed for the tower acceleration record shown in Figure 11, is shown in Figure 14.



**Figure 14: Axial Stress Time History for End Web Diagonal**

For a fatigue lifetime analysis in such a variable amplitude cyclic stress environment, a useful analytical tool is the stress range histogram. To construct such a histogram, the entire stress range (peak to peak) covered by the response is subdivided into “bins”, defined by the sub-interval of the total stress range included within that particular bin, ie.g., if the total stress range is 3 ksi (+1.5 to -1.5 ksi) and it is divided into

20 such “bins”, the sub-intervals would logically be  $< 0 \text{ to } .15 >$ ,  $< .15 \text{ to } .30 >$ ,  $< .30 \text{ to } .45 >$ , ..... ,  $< 2.85 \text{ to } 3.0 >$ . Each bin is then typically identified by its sub-interval midpoint value, i.e., the  $< 0 \text{ to } .15 >$  sub- interval is the .075 ksi bin, the  $< .15 \text{ to } .30 >$  sub-interval is the .225 ksi bin, etc. In processing a response time history into a stress range histogram, each time a particular cycle of response is “completed”, the current count in the appropriate stress sub-interval or “bin” for its peak-to-peak amplitude is incremented, until the entire response time history is processed. This type of data processing is known as “rainflow” cycle counting, and is a built-in “real-time” feature of many commercial dynamic data acquisition/processing software packages. A stress range histogram constructed for the response time history shown in Figure 14 is shown in Figure 15.



**Figure 15: Stress Range Histogram for the Response in Figure 13**

To estimate an expected fatigue lifetime for a particular structural detail, a constant amplitude stress range/number of cycles to failure relationship is typically

assumed, which is an analytical approximation of a traditional empirical (S-N) curve. A typical relationship commonly used is that given in Eq. 1:

$$N_f = C_f S_R^{-1/m} \quad (1)$$

where  $N_f$  is the number of cycles to failure,  $C_f$  and  $m$  are constants dependent on the material and weld detail, and  $S_R$  is a constant amplitude stress range.

The *Standard Specification for Structural Supports for Highway Sign, Luminaire and Traffic Signals* (AASHTO, 2001) does not specify an assumed S-N curve for aluminum weld details, but merely specifies a constant amplitude fatigue limit (CAFL) for each weld detail, which presumably corresponds to an “adequate” fatigue lifetime.

The *LRFD Bridge code* (AASHTO, 2004) does specify parameters for assumed aluminum S-N curves, but does not include a Category ET weld detail. It was decided for this analysis to utilize an assumed category ET fatigue lifetime S-N curve, based upon an extrapolation made from a combination of provisions in the *Sign, Luminaire and Traffic Signal* (AASHTO, 2001) and *LRFD Bridge codes* (AASHTO, 2004).

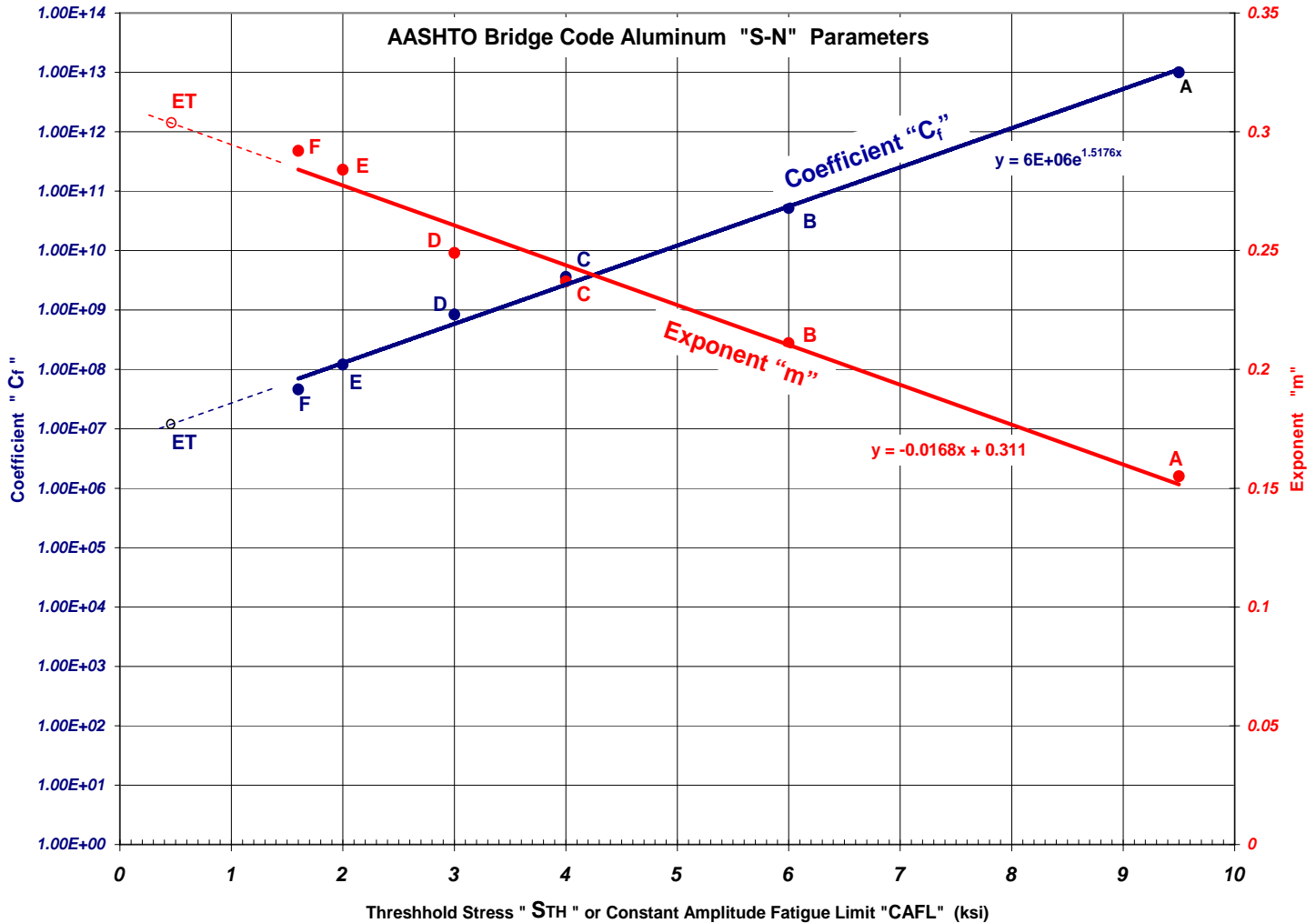
The parameters ( $C_f$  and  $m$ ) of the specified S-N curves for fatigue categories A through F in the *LRFD Bridge Code* (AASHTO, 2004) are listed in Table 1, along with the corresponding values of the specified fatigue threshold ( $S_{TH}$ ), which is taken as equivalent to the CAFL parameter in the *Sign and Luminaire Code*. Also included in the Table 1 is the CAFL of a category ET fatigue detail, from the *Standard Specification for Structural Supports for Highway Sign, Luminaire and Traffic Signals* (AASHTO, 2001).

Fatigue Category	$C_f$ (cycles / ksi <sup>m</sup> )	$m$	$S_{TH}$ or CAFL (ksi)
Bridge Code “A”	1.00 E+13	.155	9.5
Bridge Code “B”	5.20 E+10	.211	6.0
Bridge Code “C”	3.60 E+9	.237	4.0
Bridge Code “D”	8.40 E+8	.249	3.0
Bridge Code “E”	1.20 E+8	.284	2.0
Bridge Code “F”	4.60 E+7	.292	1.6
Sign Code “ET”			.44

**Table 1: AASHTO Fatigue Parameters**

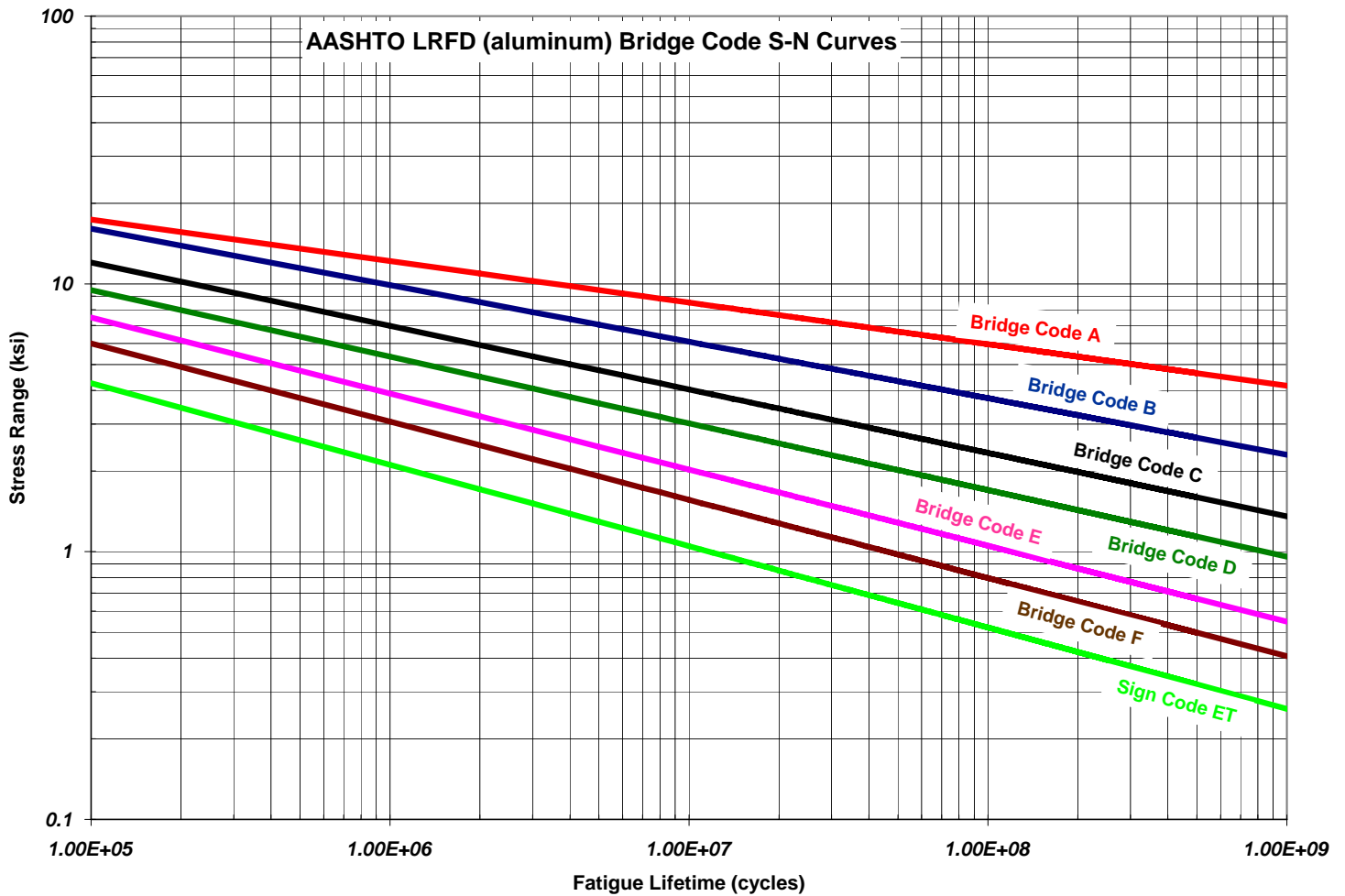


These data were graphed and fit to analytical curves, utilizing the  $S_{TH}$  and CAFL as an independent variable, as shown in Figure 16. From these analytical curves the parameters of the ET fatigue lifetime curve were estimated as  $C_f = 1.17 \text{ E}+07$  (cycles /  $\text{ksi}^m$ ) and  $m = .304$ .



**Figure 16: Extrapolation of Category ET S-N Parameters**

The S-N curves for aluminum fatigue detail categories from the AASHTO LRFD Bridge code are plotted in Figure 17, along with the extrapolated curve assumed for the *Standard Specification for Structural Supports for Highway Sign, Luminaire and Traffic Signals* category ET fatigue detail. The cutoffs for threshold and CAFL stress values were intentionally not included in Figure 17 in order to illustrate the potential implications of very high cycle counts on a material lacking any apparent endurance limit.



**Figure 17: Extrapolated S-N Curve for Fatigue Category ET**

In variable amplitude fatigue environments it is common practice to make use of the Palmgren-Miner rule to develop an equivalent constant amplitude stress from a stress histogram representation of that observed variable amplitude environment. The Palmgren-Miner Rule is given as:

$$\sum_{i=1}^k D_i = \sum_{i=1}^k \frac{n_i}{N_{fi}} \quad (2)$$

where  $D_i$  is the damage fraction,  $k$  is the number of discrete stress range amplitudes considered in the histogram,  $n_i$  is the number of cycles at stress  $S_i$  and  $N_{fi}$  is the number of cycles to failure at stress  $S_i$ . Failure is assumed when:

$$\sum_{i=1}^k D_i = 1 \quad (3)$$

Incorporating the Palmgren-Miner Rule (Eq 2) in to the analytical representation of an S-N curve (Eq 1) results in the expression given in Equation 4 for the cumulative damage fraction of a given stress range histogram:

$$\sum_{i=1}^k \frac{n_i}{N_{fi}} = \sum_{i=1}^k \frac{S_i^{1/m} n_i}{C_f} = \frac{S_{eq}^{1/m}}{C_f} \cdot \sum_{i=1}^k n_i \quad (4)$$

From Equation 4 an equivalent stress range,  $S_{eq}$ , which produces an identical degree of cumulative damage, when applied at a constant amplitude for the same number of total cycles in the histogram, can be defined as:

$$S_{eq} = \left( \frac{\sum_{i=1}^k S_i^{1/m} n_i}{\sum_{i=1}^k n_i} \right)^m \quad (5)$$

Examining the computed responses of the end web diagonals of the failed truss for the ensemble of truck passage events monitored during the field tests of 12/10/06 produced the following statistics:

- 1. average number of response cycles per truck passage  $\cong 90$**
- 2. average duration of vibration per truck passage  $\cong 24.66$  seconds**
- 3. average equivalent stress range for the resulting histogram  $\cong .30$  ksi**

The B&C commercial vehicles (trucks) per day for this section of IR75 reported for 2006 were approximately 19,000, out of an AADT (average annual daily traffic) count

of approximately 136,000, from the most recently available vehicle count data. (ODOT, 2006) : available at [www.dot.state.oh.us/techservsite/offceorg/traffmonit/default.htm](http://www.dot.state.oh.us/techservsite/offceorg/traffmonit/default.htm)) Interstate route IR75 is the main artery along the corridor connecting Cincinnati, Dayton and the Toledo area. Just north of the bridge in question is the intersection, as well, with another busy interstate highway, IR70, so this bridge is very heavily traveled.

Assuming half of the total trucks, or 9,500 per day, to be northbound, and half of those trucks, or 4750 per day, to be “fully loaded”, i.e., weights comparable to the ODOT tandem axle dump truck utilized for the field test, one could easily speculate that the sign truss would essentially be in an almost constant state of vibration, as 4750 trucks per day is equivalent to a truck roughly every 18 seconds, and it takes the vibration from a “typical” truck more than 20 seconds to “damp out”. One could therefore potentially anticipate as many as 300,000 vibrations cycles per day for the sign truss, or more than 100,000,000 cycles per year. At that rate of response cycle accumulation, at an equivalent stress range of roughly 0.30 ksi, the expected fatigue lifetime of  $7 \times 10^8$  cycles for the extrapolated AASHTO S-N curve could be reached in as little as seven years. The fact that the sign truss endured as long as it did is most likely attributable to the following factors:

1. The AASHTO S-N curves represent fairly conservative, lower bound estimates of expected fatigue lifetime, which itself exhibits considerable “spread”.
2. Truck traffic is not spread uniformly over the course of a 24 hour day, 7 days per week; i.e., there are intervals of lighter traffic when the sign truss may not be in a state of continuous vibration.
3. It is also possible that the weight spectrum of the traffic on the bridge the Sunday morning of the field monitoring (12/10/06) was not completely representative of the typical in-service weight spectrum.
4. There has been considerable growth in traffic volume over the lifetime of the structure, i.e., the 2006 traffic volume is also not necessarily representative of the lifetime of service.

Even given these mitigating facts, however, a fatigue failure of such a bridge-mounted sign support structure is quite explicable, and points out concerns for other such bridge-mounted structures of similar construction and vintage.

## **Results of Metallurgical Examination**

Figure 18 shows the sample examined from the upper chord, designated sample A. A higher magnification optical view of failed member A (boxed region in Figure 18) is shown in Figure 19. Higher magnification optical views shown in Figure 19 clearly reveal the presence of fatigue beach marks in the weld region. Also highlighted in Figure 19 is the presence of various regions of porosity in the weld of sample A.



Figure 18: Metallurgical Sample Taken From Upper Chord of Failed Truss

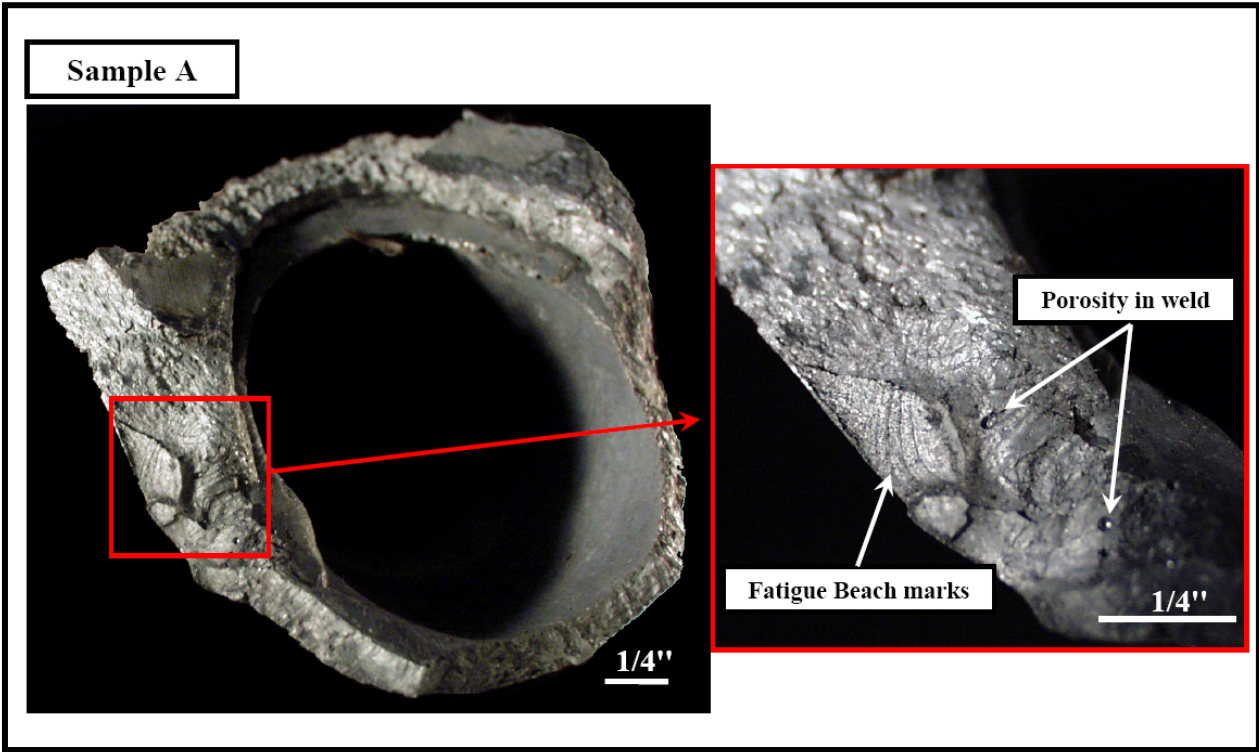


Figure 19: Higher Magnification Views of Boxed Region of Figure 18

Figure 20 (boxed region) shows the regions investigated for sample B, taken from the lower chord of the truss. Failure adjacent to the weld, in the base metal, was exhibited by failed member B as shown in both Figure 20 and 21. Higher magnification views of the failed tube in Figure 21 could not reveal any characteristic features of the fatigue progression, as the fracture surfaces had been destroyed by their continued contact and rubbing together during the failure (propagation) process.



**Figure 20: Sample B Taken From Failed Truss Lower Chord**

Metallographic sectioning was conducted to examine the weld metal, base metal, and heat affected zone (HAZ) of sample A as shown in Figure 22. Representative microstructures are shown in the various regions. The base metal exhibited a somewhat large grain size in all areas examined. Figure 23 shows a similar analysis for Sample A' for an intact weld/strut combination. Clear evidence of porosity in the weld is shown both in the as-sectioned (top right) and polished (bottom left and right) photos.

Metallographic sections taken from failed sample B are shown in Figure 24. A crack in the weld region is shown (top right) while the weld and base metal are shown in the bottom two photos. Figure 25 provides a similar analysis for Sample B'. Porosity is again evident in the weld region (top right photo), while metallography of the weld and base metal is shown in bottom left photo. The photo on the bottom right compares the microstructure present in the smaller tube (left) to that of the larger tube (right), showing that the grain sizes are similar.

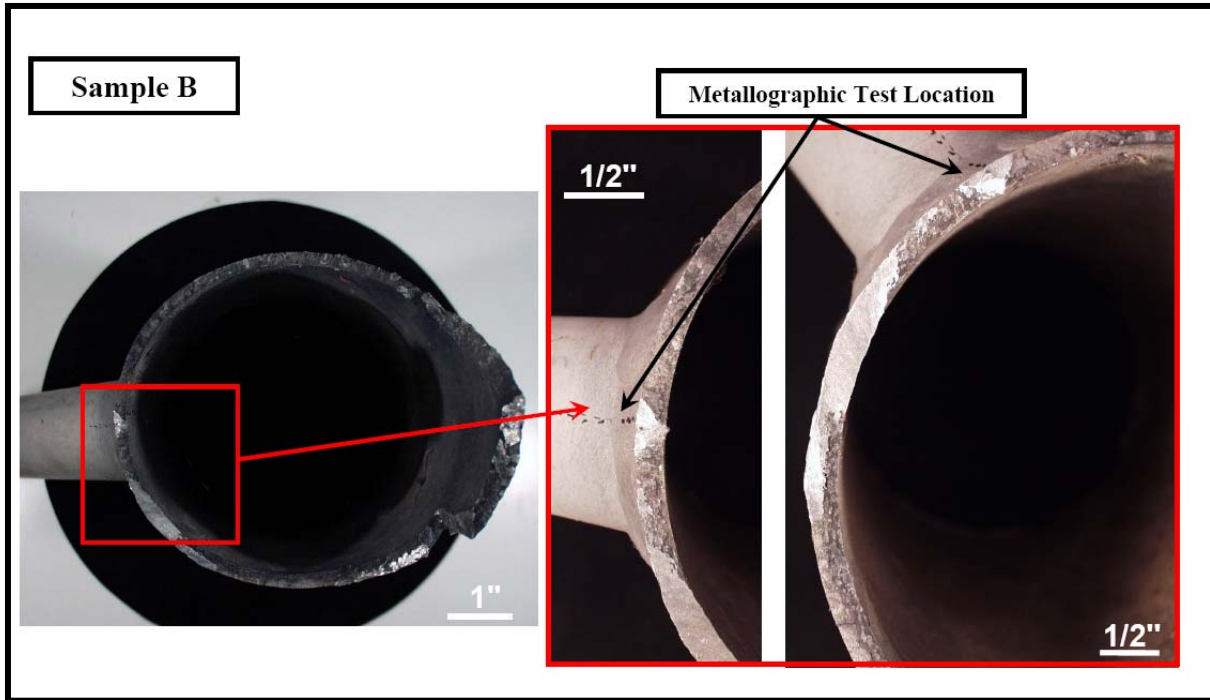


Figure 21: Higher Magnification Views of Boxed Region From Figure 20

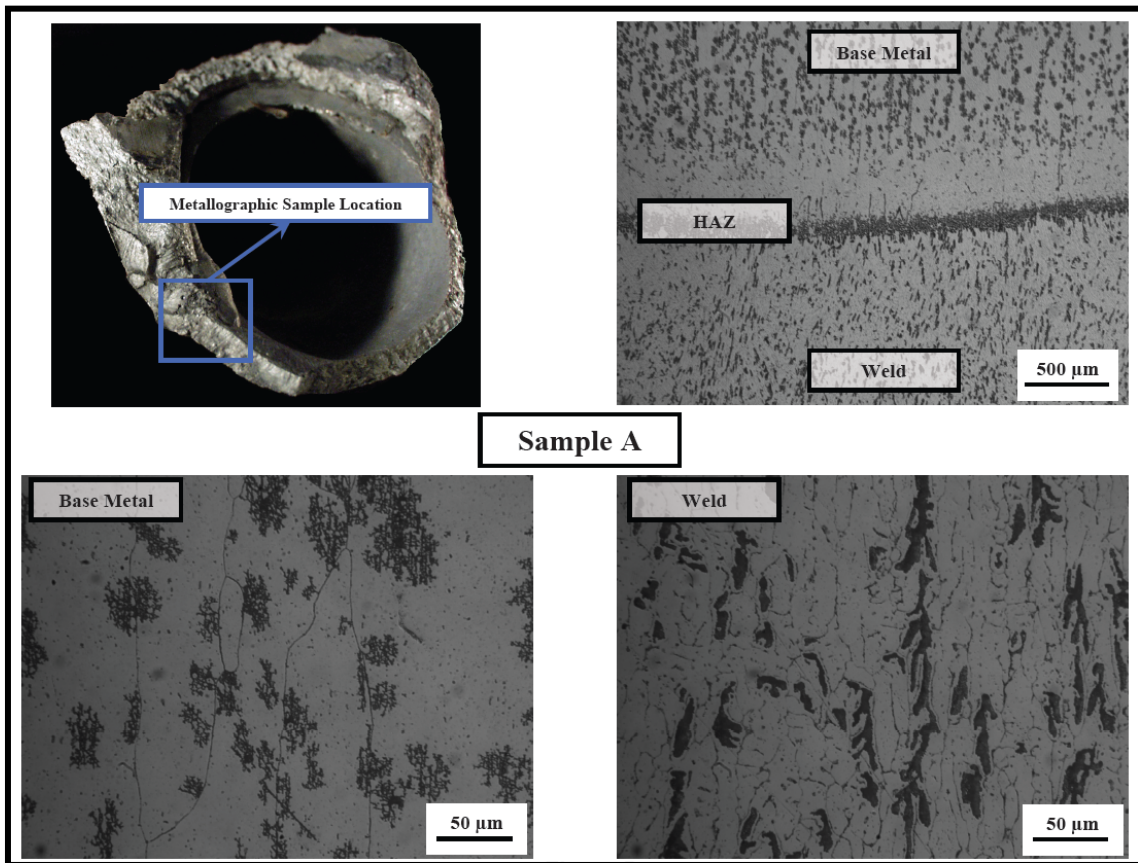
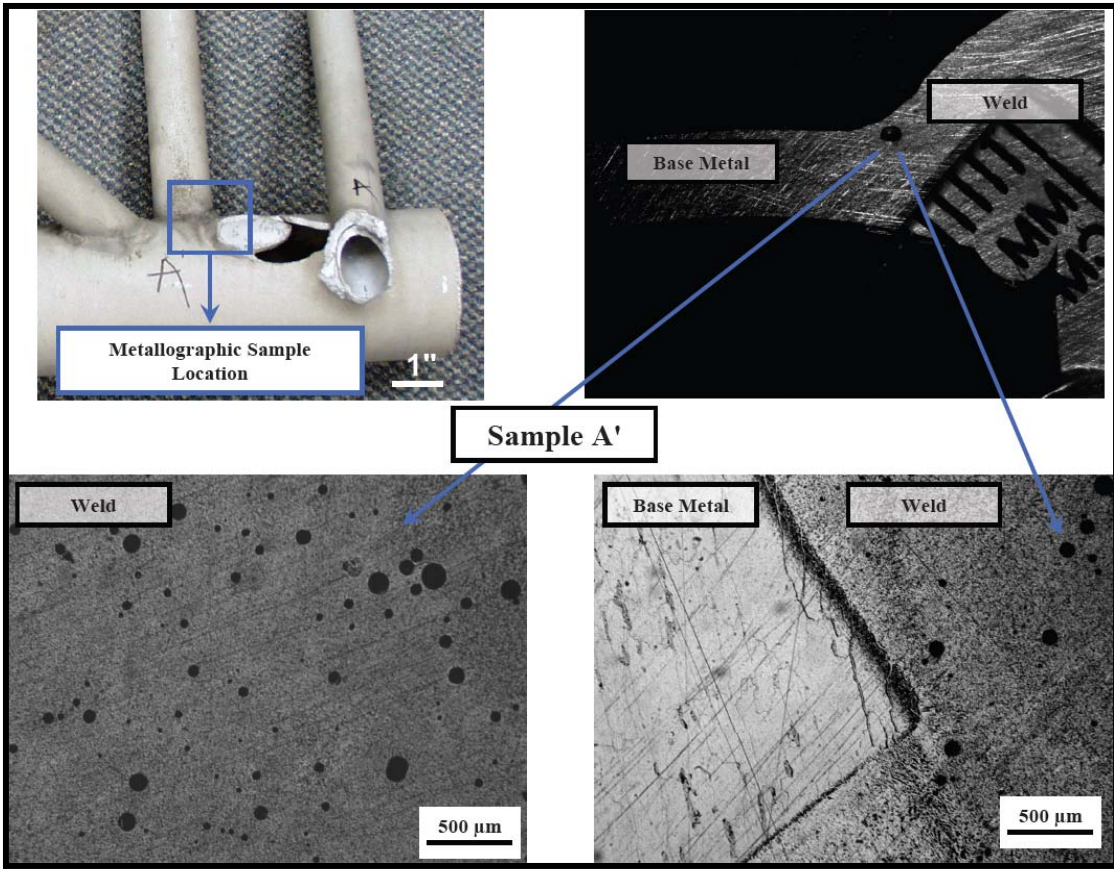
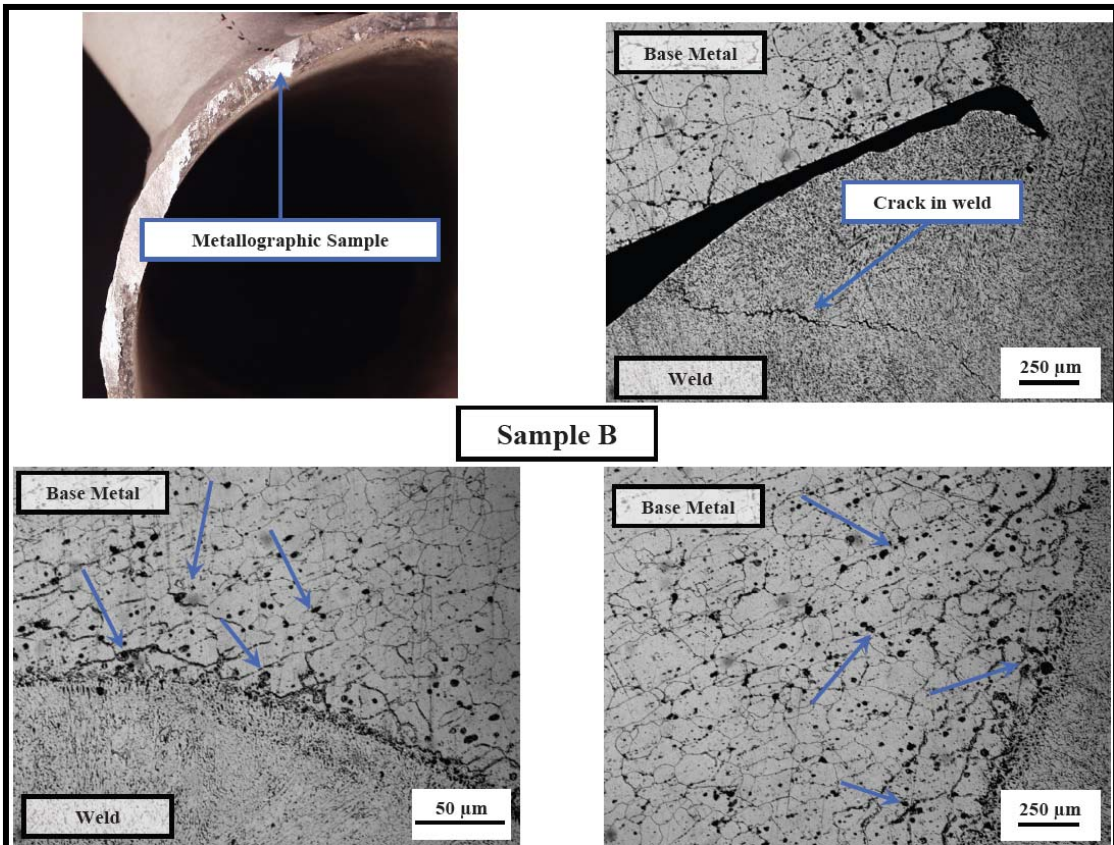


Figure 22: Metallographic Sections From Sample A (Upper Chord)



**Figure 23: Metallographic Sections  
From Sample A' (Upper Chord)**



**Figure 24: Metallographic Sections  
From Sample B (Lower Chord)**



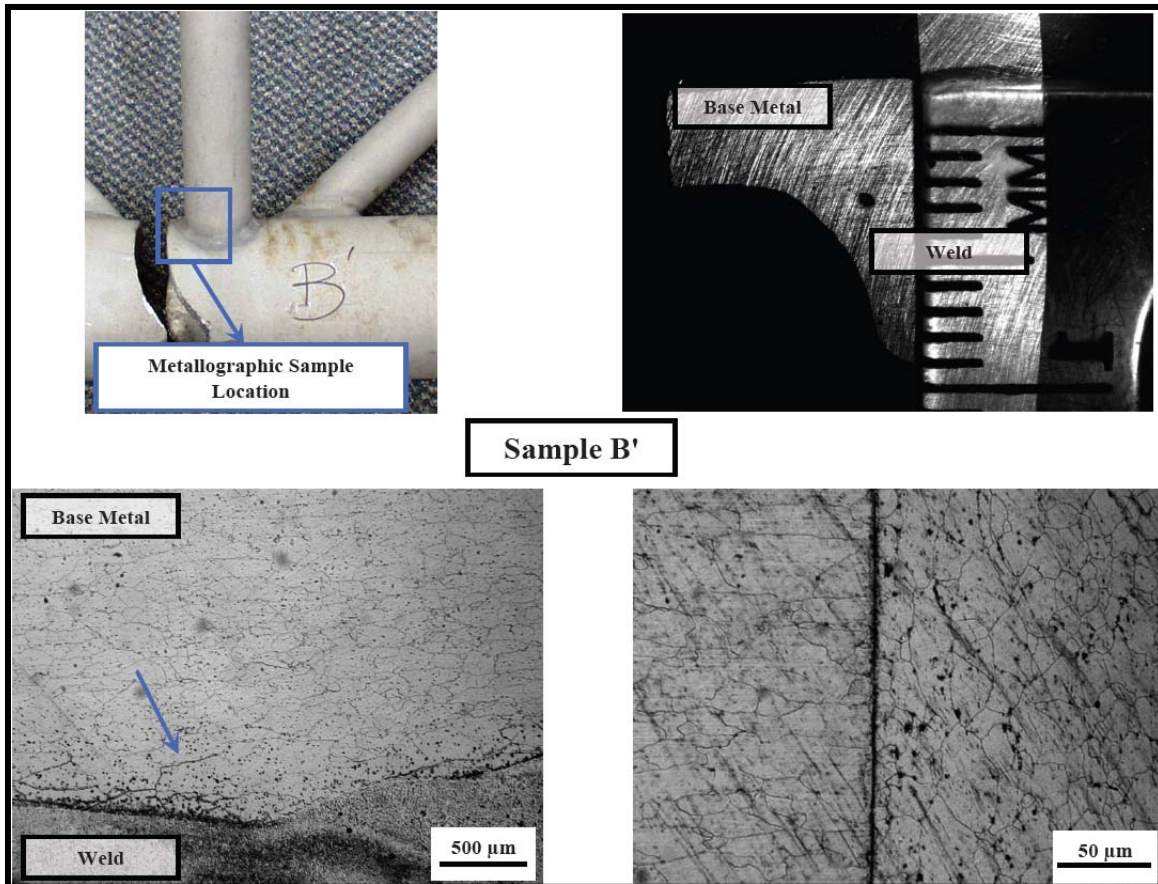


Figure 25: Metallographic Sections From Sample B' (Lower Chord)

## Hardness Values of Struts (O.D.)

	Sample ID #			
	Sample A	Sample A'	Sample B	Sample B'
<b>HRB (100 Kgf)</b>	<b>53</b>	<b>51</b>	<b>51</b>	<b>56</b>
	<b>55</b>	<b>51</b>	<b>53</b>	<b>56</b>
	<b>54</b>	<b>47*</b>	<b>54</b>	<b>51</b>
Average	<b>54.0</b>	<b>51.0</b>	<b>52.7</b>	<b>54.3</b>
StDev.	<b>1.00</b>	<b>0.00</b>	<b>1.53</b>	<b>2.89</b>

\* Near the weld

Figure 26: Rockwell B Hardness Numbers From Surfaces

Figure 26 provides the Rockwell B hardness numbers taken on the OD surfaces of the tubes shown. The somewhat lower hardness measured near the weld in sample A' may be due to the difficulty in taking the hardness value due to the geometry of the sample/weld.

Fatigue is clearly responsible for the failure. The presence of porosity in the weld of Sample A could also affect the mechanical behavior of the weld in Sample A. The location of the failure in Sample B occurred somewhat removed from the weld region. Preliminary exam of these regions showed a sub-critical crack in the weld and nothing remarkable in the region of failure in the base metal of tube B.

## **Summary / Conclusion**

Based upon a combination of in-situ field monitoring of traffic-induced bridge vibrations at the location of the failed sign support truss, finite element simulation of the expected dynamic response of the original truss in such an environment, the length of service of the truss at the time of its failure, the volume of truck traffic on the bridge in question during that time of service, and metallurgical examination of the failed components of the sign truss, the conclusion of the investigation is that extremely high-cycle fatigue of the chord/web diagonal welded connection was the cause of the truss failure. The effective stress range of the AASHTO Category ET connection in question was most likely below the currently specified AASHTO constant amplitude fatigue limit (CAFL) for the detail, but the enormous quantity of response cycles (approaching or even exceeding 1 billion cycles) such a bridge mounted sign on a heavily traveled route accumulates over a service lifetime of 30 or 40 years, exceeds anything currently considered in the design codes for such structures. The implications of this fact on the current inventory of such structures in similar installations and of the same age range, i.e., installed with the original interstate routes, largely in the 1960's, are obvious.

The one somewhat puzzling aspect of the lower chord fracture, namely that the fracture passed near the weld toe but not actually contacting the weld toe, does not appear to be particularly significant. The fracture did indeed pass close to the weld and certainly within the heat affected zone of the weld. The fracture surfaces were too abraded to positively identify the initiation point; small cracks were identified within the weld, however. The original flaw which eventually propagated and produced the rupture apparently, in this case, just happened to be near the weld rather than in the weld.

## **Acknowledgements**

Ralph Van Kirk and his crew at District 7 very ably made the field monitoring on a very busy bridge possible and provided us, as well, with numerous design documents for the bridge and sign structure. Jim Roth of the ODOT Division of Operations, Office of Traffic Engineering provided valuable background information utilized for the modeling and analysis of the sign structure. Professor John Lewandowski and his students in the Department of Material Science & Engineering at the Case School of Engineering expertly performed the metallurgical examination and analysis.

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