FINAL CONTRACT REPORT

**DETERMINATION OF REMAINING FATIGUE LIFE OF WELDED STUD DETAILS
ON OVERHEAD ALUMINUM SIGN PANELS IN VIRGINIA**

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ABSTRACT

Some overhead highways signs in Virginia using a specific welded threaded stud and clip connection have failed while in service. From inspection of the signs it was determined that the failure was caused by fatigue of the weld connecting the threaded stud to the back of the sign panel. It was also observed that lower edge connections failed first and the failures progressed upwards in an unzipping pattern. A combination of natural and truck-induced wind gusts is the cause for the fatigue failure. Although signs with these connections are no longer produced by VDOT and all production was halted in early 2004, there are still approximately 4,000 signs in Virginia with this connection detail.

The objective of the research project described here in was to determine priorities for an inspection and retrofitting plan for the remainder of the signs in Virginia. Specifically an S-N curve, which is a plot of stress range versus the number of cycles to failure, was to be developed to aid in predicting the remaining service life of sign panels using this connection detail.

The authors opted to test single connections instead of multiple connection systems (i.e., an entire sign or portion thereof) because of material availability, the timeliness of testing, and the readily available equipment for testing. Connections simulating interior and exterior connections were tested under a pseudo-static load as well as for fatigue. Fatigue tests of interior and exterior sign connections developed failures of the aluminum panel instead of the expected weld fracture. Because the failure and fatigue threshold were not representative of failures found in the field, a proper S-N curve to help develop retrofitting priorities could not be developed. Recommendations from this program include increasing retrofit and inspection efforts, gauging and monitoring full-scale signs in service to understand loading conditions, and testing full-scale signs.

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INTRODUCTION

From the time interstates were constructed, highway signs were used to give drivers locations and directions. These signs, as seen in Figure 1, can be supported by either bridges or cantilevers over the road, as seen in Figures 2 and 3, or on roadside supports, as seen in Figure 4.

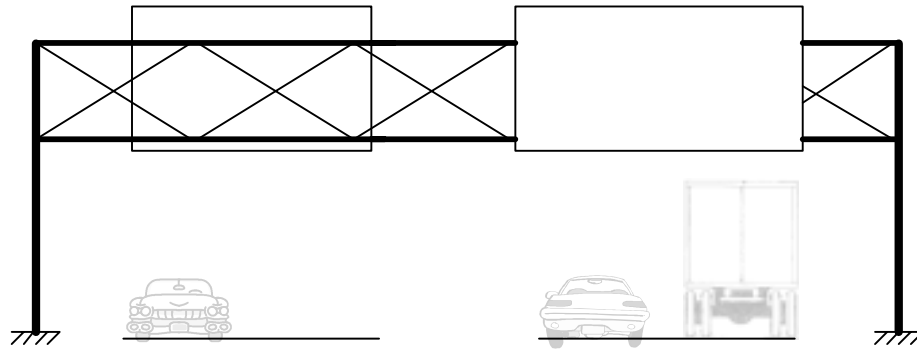

Figure 1. Typical Highway Sign.
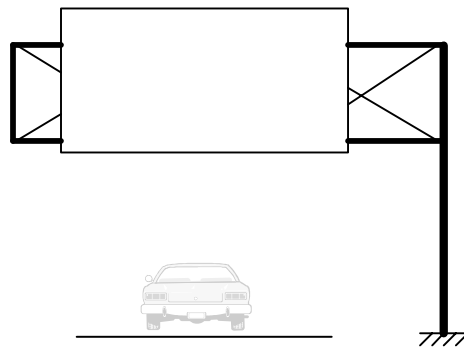
Figure 2. Bridge Support Detail.

Figure 3. Cantilever Support Detail.
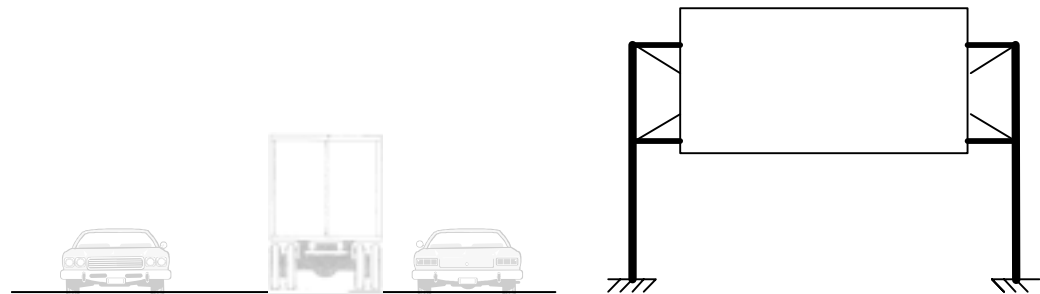
Figure 4. Roadside Sign.

From the early 1980s until recently, the Virginia Department of Transportation (VDOT) has used a sign detail, as seen in Figure 5, connecting the sign panel to the support structure. The signs consist of multiple aluminum panels, varying in width from 2 to 5 feet with heights up to 18 feet, which are joined together using the connection clip detail seen in Figure 6. Threaded studs that are welded to the back of the panel connect the sign panel to the supporting structure. Figures 7 and 8 show the clip and z-bar connection detail. These welded connections are spaced 14 to 16 inches horizontally on center and 20 to 51 inches vertically on center over the entire sign as seen in Figure 9. There are currently about 4,000 signs that contain this clip and z-bar detail in Virginia.

W

14" c-c

Zee Bar 6061-T6
Aluminum Alloy

6" c-c

2'-0" Minimum to 5'-0" maximum width sign panels.

Figure 5. Sign Detail.

Sign face

Studs to be electrically
welded to back of sign panel
by Capacitor Discharge
Method

6061-T6 Aluminum Extrusion
1/8" thick backing strip

7075-T6 Aluminum Alloy
0.255 I.D. – 0.493 O.D. x
0.062" thick spring
lockwasher

5356-H32 Aluminum Alloy
¼" – 20 x ½" stud

6061-T6 Aluminum Alloy
¼"-20 hexagon head nut to
be installed with torque not
to exceed 25 in-lbs

Figure 6. Connection Clip Detail.

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6061-T6 Aluminum Alloy
¼"-20 hexagon head nut to
be installed with torque not
to exceed 25 in-lbs

5356-H32 Aluminum Alloy
¼"-20 x ½" long stud

Type 304 stainless steel
cup point set screw 3/8"-
16 x ½" long NC-2 socket
head to be provided in
each panel clip

Zee Bar 6061-T6
Aluminum Alloy

7075-T6 Aluminum Alloy
0.255 I.D. 0.493 O.D. x 0.062"
thick spring lockwasher

6061-T6 Aluminum
panel clip

Studs to be
electrically welded
to back of sign
panel by
Capacitor
Discharge Method

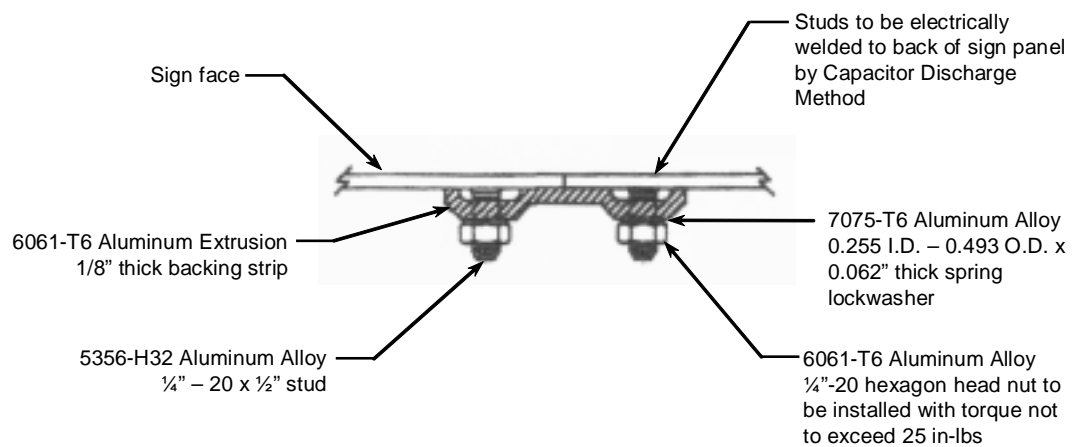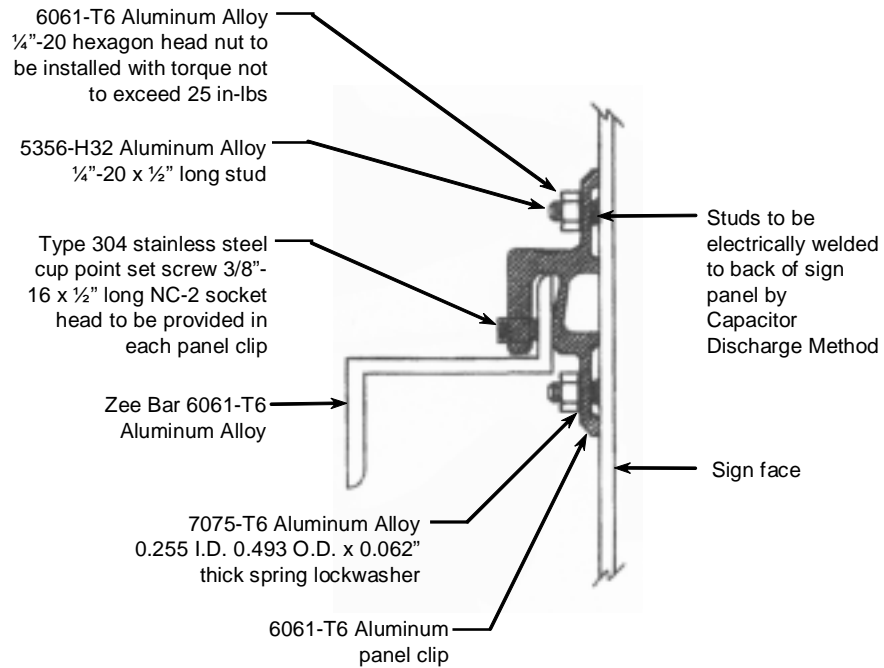Sign face
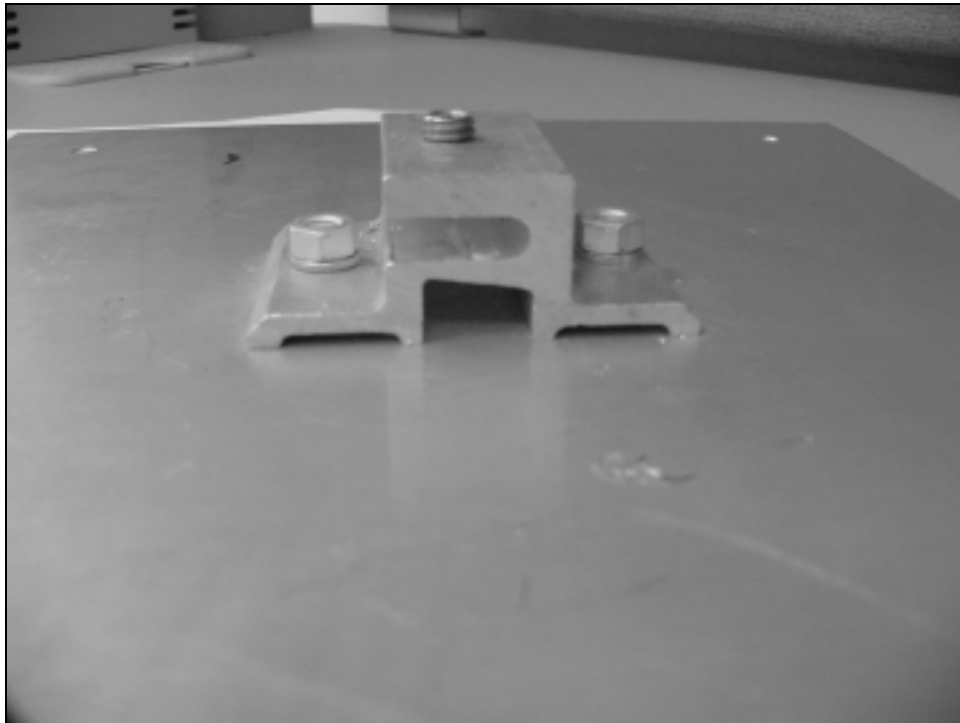
Figure 7. Connection Detail.

Figure 8. Sign Connection.

Figure 9. Sign Connected to Support.

VDOT halted its production of the signs with this clip-angle connection detail many years ago, however, some contractors continued to use this detail until early in 2004. One of the reasons some VDOT Districts stopped using the clip and z-bar connection was because the studs can shear if the nuts were more than "hand-tightened" during installation. Figure 10 shows studs that failed during tightening in the laboratory. Other districts, however, did not stop allowing contractors to install this connection detail until early in 2004, after a few known sign failures in the field.

In April 2003, a sign failed and fell in the Hampton Roads District causing a traffic accident.   Another sign failed in the Fredericksburg District in March 2004, luckily not causing injury. It was observed that these failures occurred due to the threaded studs "popping off" the sign panels. The failure mechanism was observed to be fatigue of the weld connecting the panel to the stud, caused by natural and/or truck-induced wind gusts. From inspection of other signs that had failed connections, but had not fallen, it was concluded that the bottom edge and corner connections are more susceptible to failing. After the edge connections fail, the interior connections start to fail. This can be described as an unzipping of the sign connections. A failed sign and stud can be seen in Figures 11 and 12.
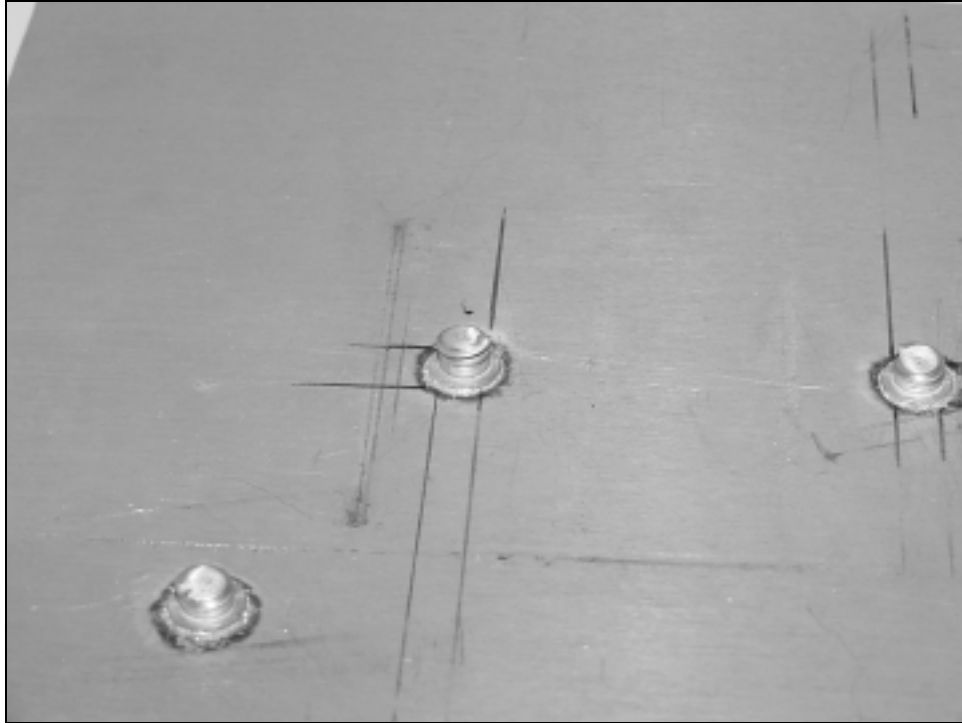
Figure 10. Stud Failure Due to Over Tightening.

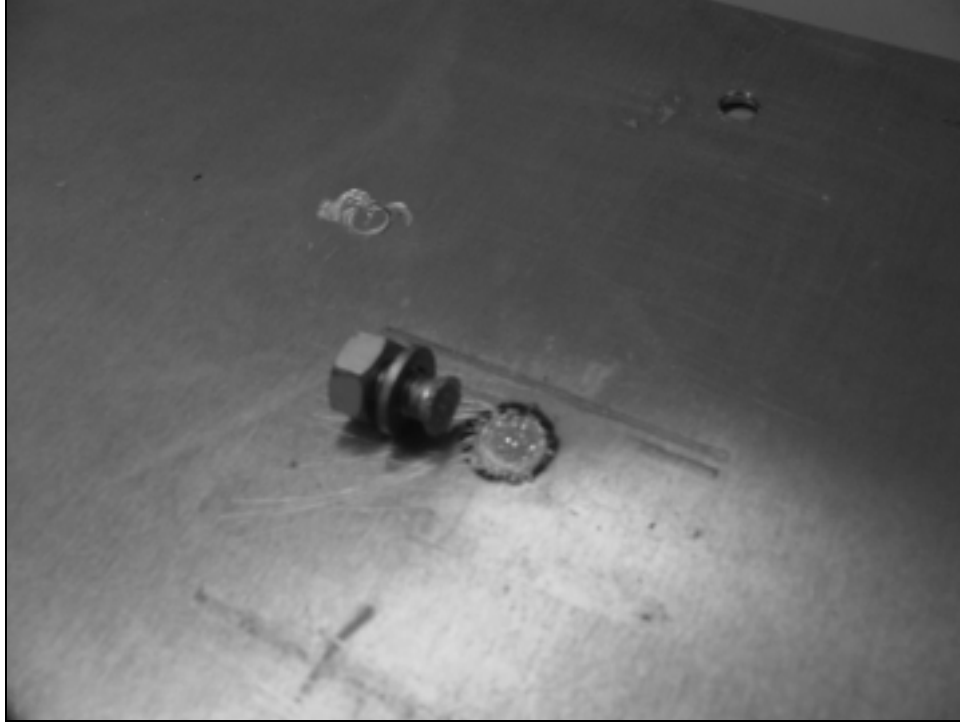

Figure 11. Failed Stud on Sign.

Figure 12. Failed Stud Due to Fatigue.

PURPOSE AND SCOPE

This project was designed to address the sign failures by determining priorities for a retrofitting plan of the existing signs. S-N curves representing the relationship between fatigue stress range to number of cycles to failure for the panel connection detail were developed from sample tests. Specifically a testing procedure was developed to test connections using pseudo-static and fatigue loads. The test fixture was designed to allow for the testing of interior and exterior connections. The test connections were obtained from a new sign panel from the Hampton Roads District of VDOT. The sign panel contained 48 individual clip and z-bar connections.

CURRENT CODE AND SPECIFICATIONS

The current design specifications for highway signs is the *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*, 4th edition, published by the American Association of State Highway and Transportation Officials (AASHTO, 2004). The specifications use Equation 1 to convert wind speeds, V (mph), into wind pressure, P_z (psf).

$$P_z = 0.00256 K_z G V^2 I_r C_d \qquad \text{[Equation 1]}$$

where

K_z = height and exposure factor.
G = gust effect factor.
I_r = gust importance factor
C_d = drag coefficient.

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The wind speed is determined from wind maps found in the AASHTO specifications. Most of Virginia lies in the 90 mph wind category; the eastern coast is the exception with wind speeds up to 120 mph. The pressure for a sign in western Virginia was found to be 28.4 psf and 50.4 psf for coastal hurricane winds. For a theoretical sign with connections spaced 14 inches on the horizontal and 30" on the vertical, force per connection approximately 83 lb for 90 mph wind speeds and 147 lb for 120 mph hurricane winds. These calculations can be found in the Appendix.

The AASHTO specifications account for fatigue design by developing equivalent pseudo-static forces for natural wind gusts as well as truck-induced gusts. These pressures, which are intended for design of cantilever superstructures, are calculated using empirical formulas. An equivalent pseudo-static pressure for natural wind gusts are found using Equation 2 applied to the horizontal face. Equation 3 is used to determine equivalent pseudo-static pressure for truck-induced gusts applied vertically to the sign support structure.

$$P_{NW} = 5.2 C_d I_F \ \textit{(psf)} \quad\quad\quad\quad \text{[Equation 2]}$$
$$P_{TG} = 36.6 C_d I_F \ \textit{(psf)} \quad\quad\quad\quad \text{[Equation 3]}$$

where
C_d = drag coefficient
I_F = fatigue importance factor

Since these calculations are for cantilever support structures, and not for panel connections, they may not be reliable or accurate for this application.

TEST PROCEDURES

Quantifying Pretension of Threaded Studs

Because there is no standard limit on the pre-torque in the studs, a torque sensor and bolt gages were used to determine the pretension of the stud due to the tightening of the nut. A torque sensor, as seen in Figure 13, was developed in-house and used to quantify the "as-built" level of pre-torque in connections of our sample sign (from VDOT's Hampton Roads District sign shop). The torque sensor, machined out of aluminum, was accurate to one inch-pound of torque. The studs were instrumented using bolt gages, as seen in Figure 14, to measure bolt tension during tightening. Torque was applied to the instrumented studs to determine the relationship between pre-torque and bolt tension.
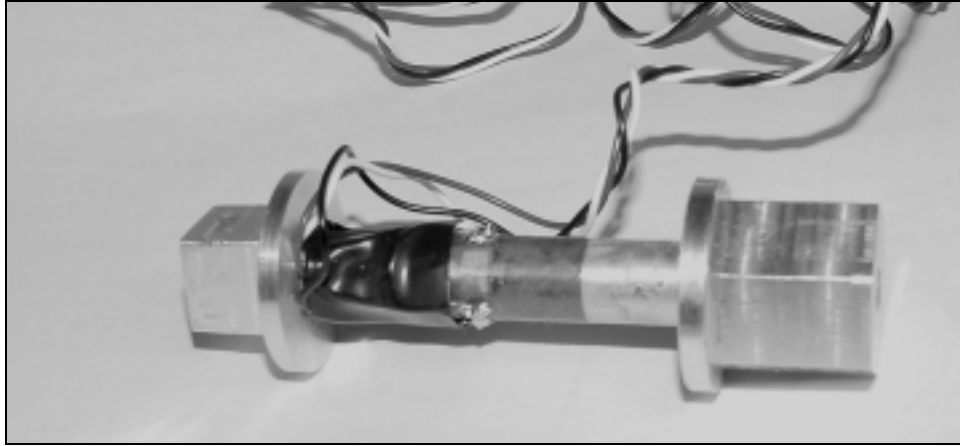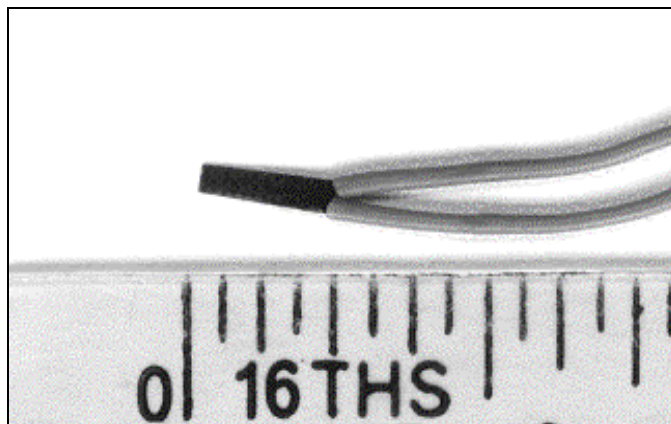
Figure 13. Torque Sensor.

Figure 14. Bolt Gage.

Pseudo-Static and Fatigue Test Setup

Both full-scale tests and smaller single connection tests were considered when developing the testing program.   The single tests were determined to be a better choice for many reasons. The availability of signs with this detail is limited, and since they are not manufactured anymore it was very difficult to obtain enough signs to get sufficient data if testing full signs. The donated sign panel from the Hampton Roads District was the only new test specimen available for this testing. It was also very difficult and impractical to test complete signs because of the space and size of the equipment that was required. Application of a uniform wind load to a large sign panel is a very difficult if not impossible task. By testing a single connection at a time, it was thought that more tests can be run and thus give statistically significant results. It was also a more readily available way to test the signs because a single connection can be more easily tested in a servo-controlled, hydraulic test frame, thus limiting time and resources needed to construct a test setup.

The single connection test setup, as seen in Figure 15, was developed to allow for testing of various boundary conditions. Boundary conditions to simulate interior and exterior connections were used. A half-inch aluminum base plate with ¼-inch bolts was used to pin

edges of the sign. Two types of connections were used to pull on the studs. For the interior supports, a pinned connection as seen in Figure 16 was used to add a degree of freedom, removing induced bending moments due to possible testing frame misalignment, thus allowing a more uniform distribution. For exterior connections, that degree of freedom was taken away to simulate a stiffer connection. The sign panel itself was trimmed to 12" by 10" rectangle with holes along the side so the panel could be pinned to the base plate. A schematic of the test setup can be seen in Figure 17.

Figure 15. Test Setup.

Interior Connection Testing

The sign specimen was first pinned on all four sides, as seen in Figure 18, to simulate an interior connection. The nuts were hand tightened on the studs. Repetitive pseudo-static tests were first performed on the interior test specimen to determine the stress distribution between the forces in the two studs. This was done by loading and unloading the specimen to 200 lb force. Bolt gages were installed in the threaded studs for these tests to determine the force distribution between the two studs during loading. Once this relationship was established, multiple pseudo-static tests at loading rates of 2-3 lb/sec were run to failure to determine the ultimate pseudo-static strength of a connection. Figure 19 shows a specimen during a pseudo-static test. Fatigue tests were then run at stress ranges starting at half of ultimate pseudo-static strength based on the pseudo-static test results. Table 1 shows the type of tests run and how many specimens were run at each load.
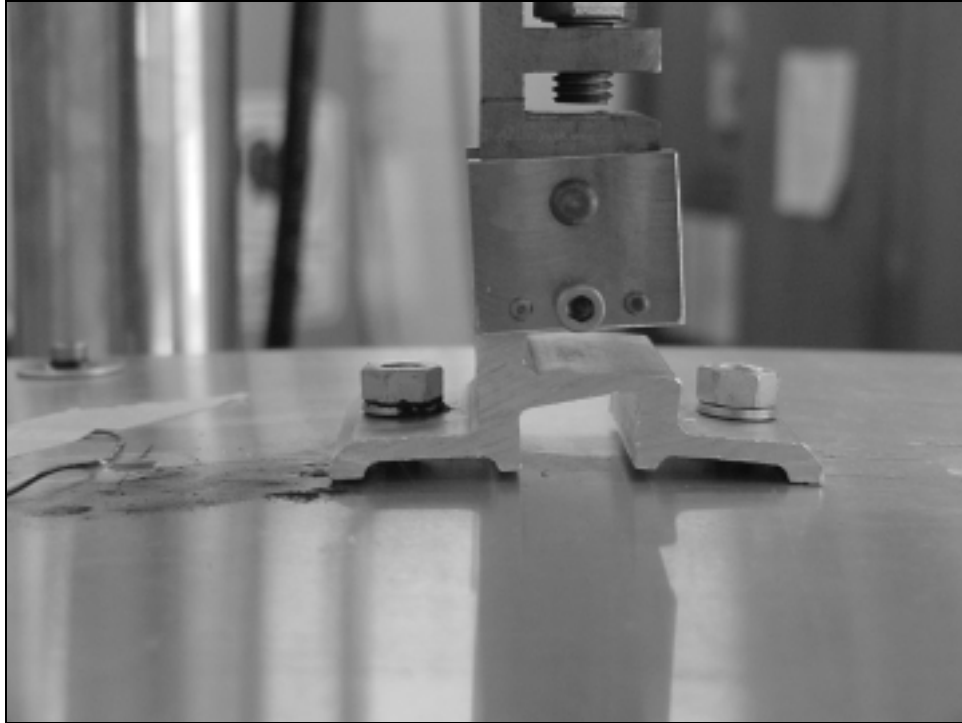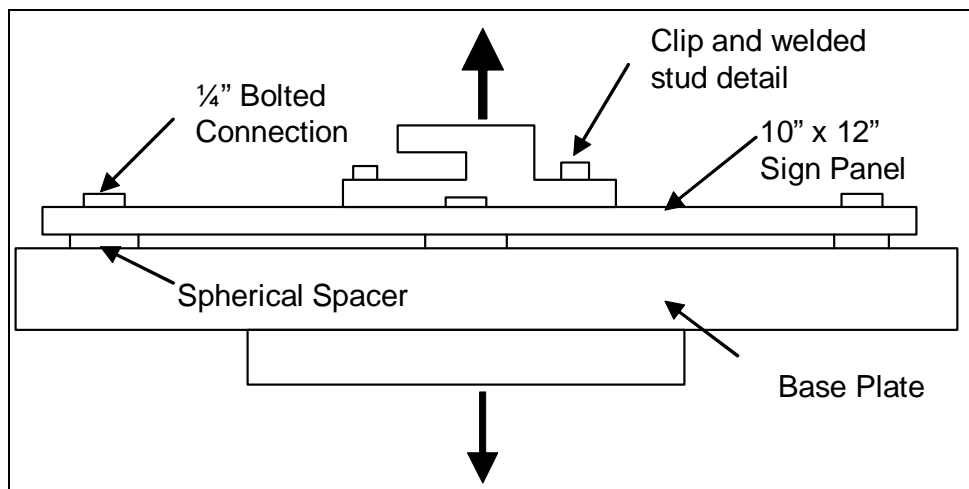
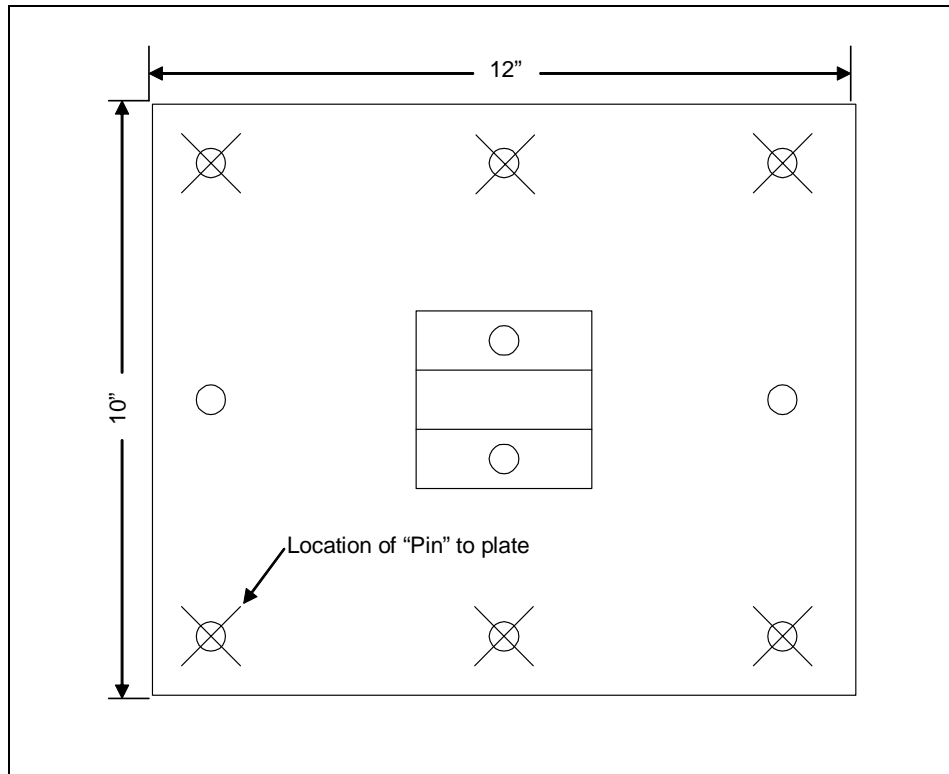**Figure 16. Pinned Connection.**

Figure 17. Test Panel.

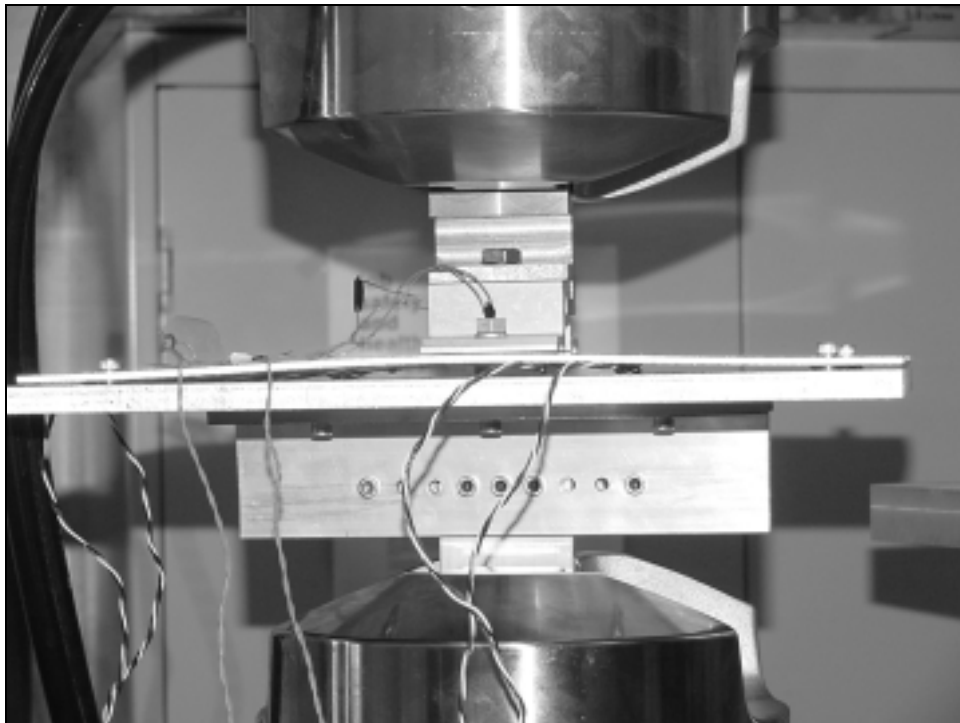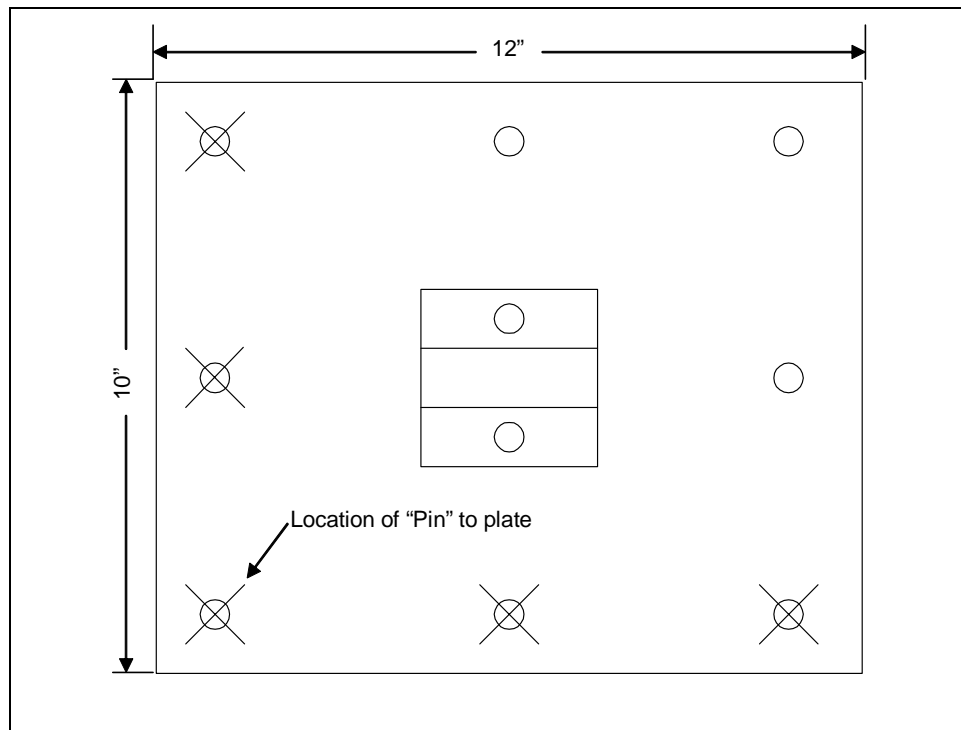**Figure 18. "Interior" Connection.**

Figure 19. Specimen During Pseudo-static Testing.

Table 1. Interior Connection Tests

Test Type:	Pseudo-static	50-500 lb	50-275 lb
Number of Samples:	7	2	2

Corner Connection Testing

To simulate an exterior connection, the specimen was pinned on two sides, as seen in Figure 20, with the nuts hand tightened on the studs. Pseudo-static tests to failure were first run to determine the maximum static strength. The first fatigue tests were then performed at the same levels as the interior connections. Fatigue tests were also run at other stress levels as seen in Table 2.

Figure 20. "Exterior" Connection.

Table 2. Corner connection Tests

Test Type:	Pseudo-static	60-600 lb	50-500 lb	40-400 lb	27-275 lb	22-220 lb	17-170 lb	10-100 lb
Number of Samples:	3	3	3	3	5	4	3	1

ANALYSIS AND RESULTS

Pretension of Stud

The hand-tightened nuts on the studs of the Hampton Roads sign panel were tensioned to a torque between 5 and 10 in-lb. This translated into approximately 3-5 lb (95-158 psi) of pretension on the studs. This is a negligible amount of stress in comparison to the tensile strength of the connection (less than 1%). A significant tension force can be developed in the studs when they are more than hand tightened with a wrench. Normally this would lead to a shear failure.

Interior Connection

The interior connection was first tested to determine the stress distribution between the two studs. Figure 21 shows the orientation of stud 1 and stud 2 with respect to the clip orientation. From low load pseudo-static tests, it was approximated that stud 2 carries about 90% of the tension force that which stud 1 will carry. Figure 22 shows a plot of the stresses in stud 2 with comparison to what stud 1 carries. The difference between the forces in the two studs can be attributed to the stiffness of the clip.
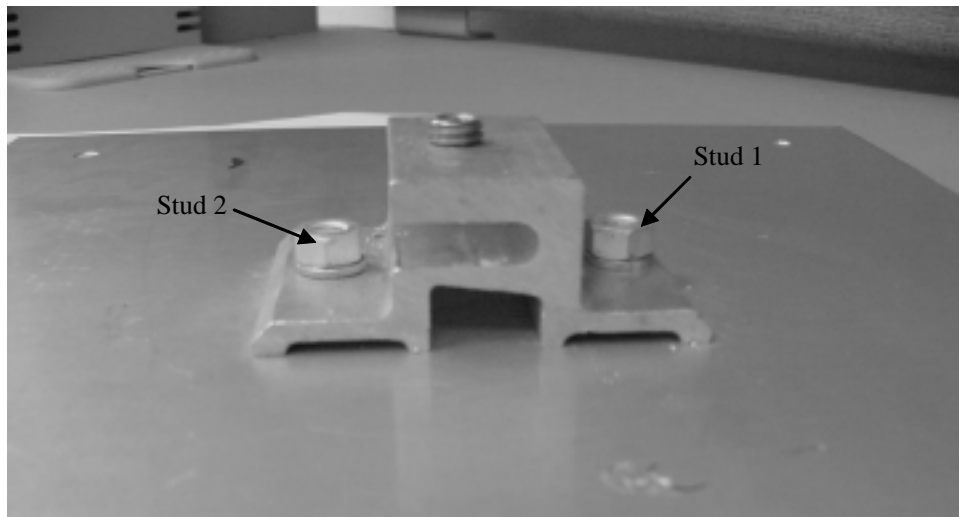

Figure 21. Stud Orientation.

The results of the seven pseudo-static tests to failure are shown in Table 3. The average pseudo-static strength of the connection is 967 lb. The weld failed for all specimens, as seen in Figure 23. This failure mechanism was representative of most failures found in the field.
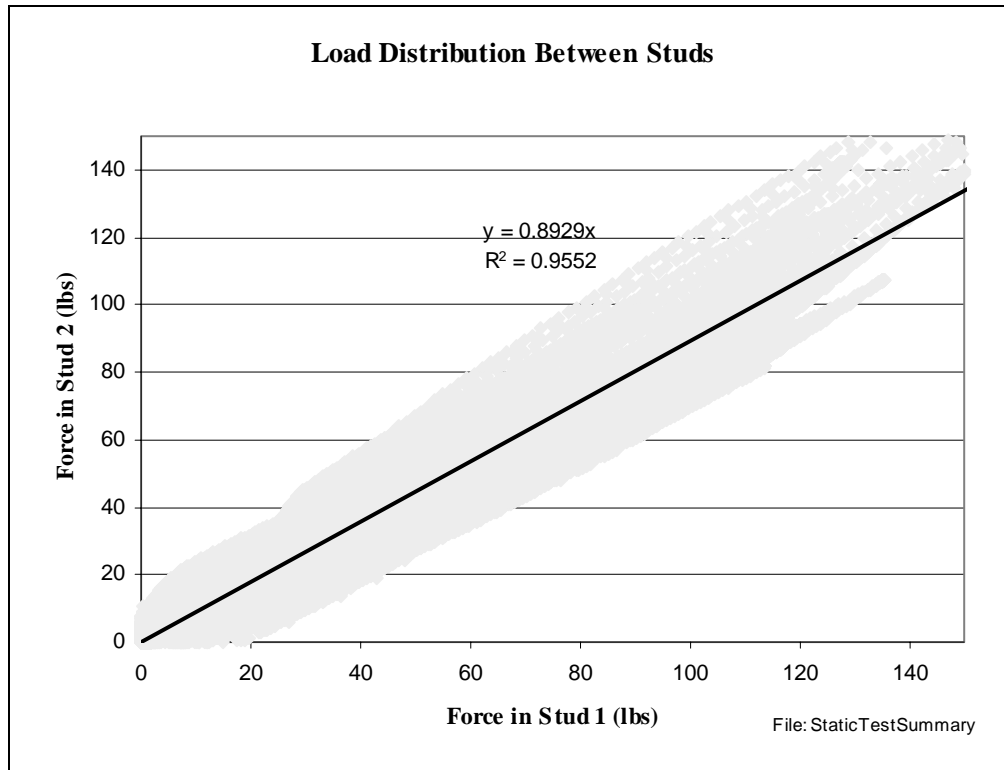
Figure 22. Load Distribution Among Studs.

Table 3. Pseudo-static Strength Results

Test :	1	2	3	4	5	6	7	AVG
Pseudo-static Strength:	903 lb	920 lb	1025 lb	1020 lb	840 lb	940 lb	1118 lb	967 lb

The connections tested for fatigue at stress range of 50-500 lb failed in the same manner as the pseudo-static tests. The connections tested cyclically at lower stress ranges developed cracks in the aluminum panel prior to the stud pulling out, as seen in Figure 24. In these cases, the welds did not fail. Crack initiation for these tests was estimated by changes in stiffness plots. Because these failures were not representative to any found in the field, the testing of interior connections was stopped. It was assumed that this was not the correct boundary condition and loading configuration that was in the field. The fatigue results can be seen in Table 4. There was no S-N curve developed for this loading condition because of the small number of samples tested.
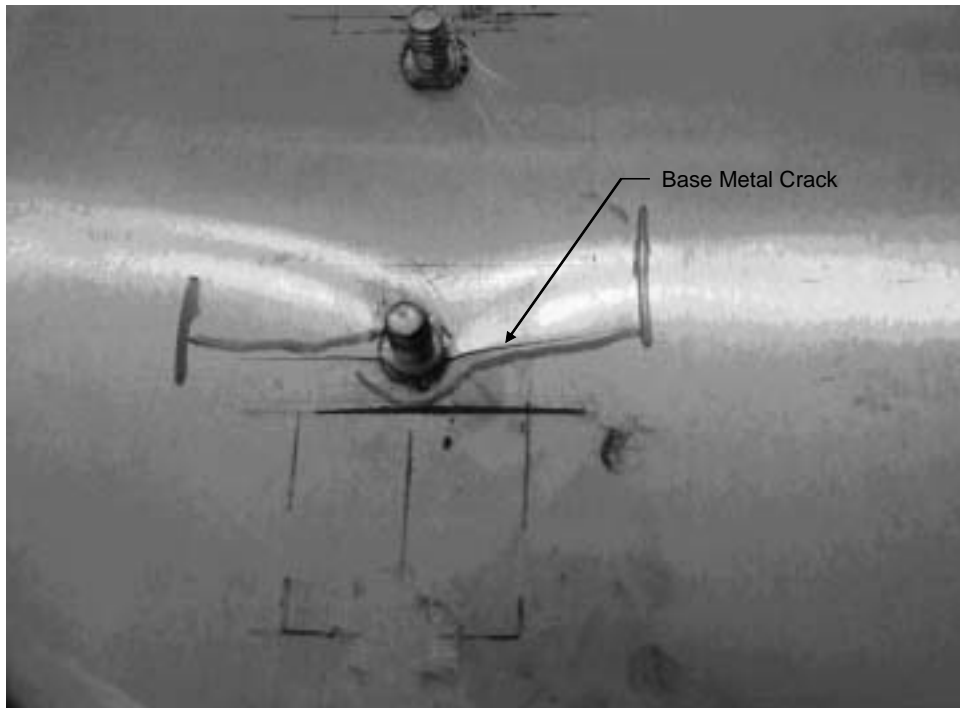
Figure 23. Failure of Weld.

Base Metal Crack

Figure 24. Cracked Panel Leading to Stud Pullout.

Table 4. Fatigue Results

Stress Range:	50-500 lb	50-500 lb	50-275 lb	50-275 lb
Number of Cycles:	130	51	~150,000	~150,000
Failure Occurred in:	weld	weld	panel	panel

Exterior Connection

The average pseudo-static strength was found to be 780 lb from the three pseudo-static tests. The individual strengths were 790, 720, and 830 lb. The failures were similar to that of the interior connections tested under pseudo-static loading, showing that the welds popped off.

The connections tested at higher stress range exhibited failures of the weld between the stud and panel. Lower stress ranges exhibited cracking of the panel. Table 5 shows the results of the tests run, as well as the failure mechanisms. An S-N curve was developed from the exterior connection tests as seen in Figure 25.

Table 5. Exterior Connection Fatigue Results

Stress Range	Number of Cycles			
60-600 lb	37	103	20	
50-500 lb	55	47	49	
40-400 lb	63	268	157	
27-275 lb	1,160	1,417	2,800	2,050
22-220 lb	52,800	19,640	35,000*	25,000*
17-170 lb	200,000*	125,000*	23,000*	
10-100 lb				

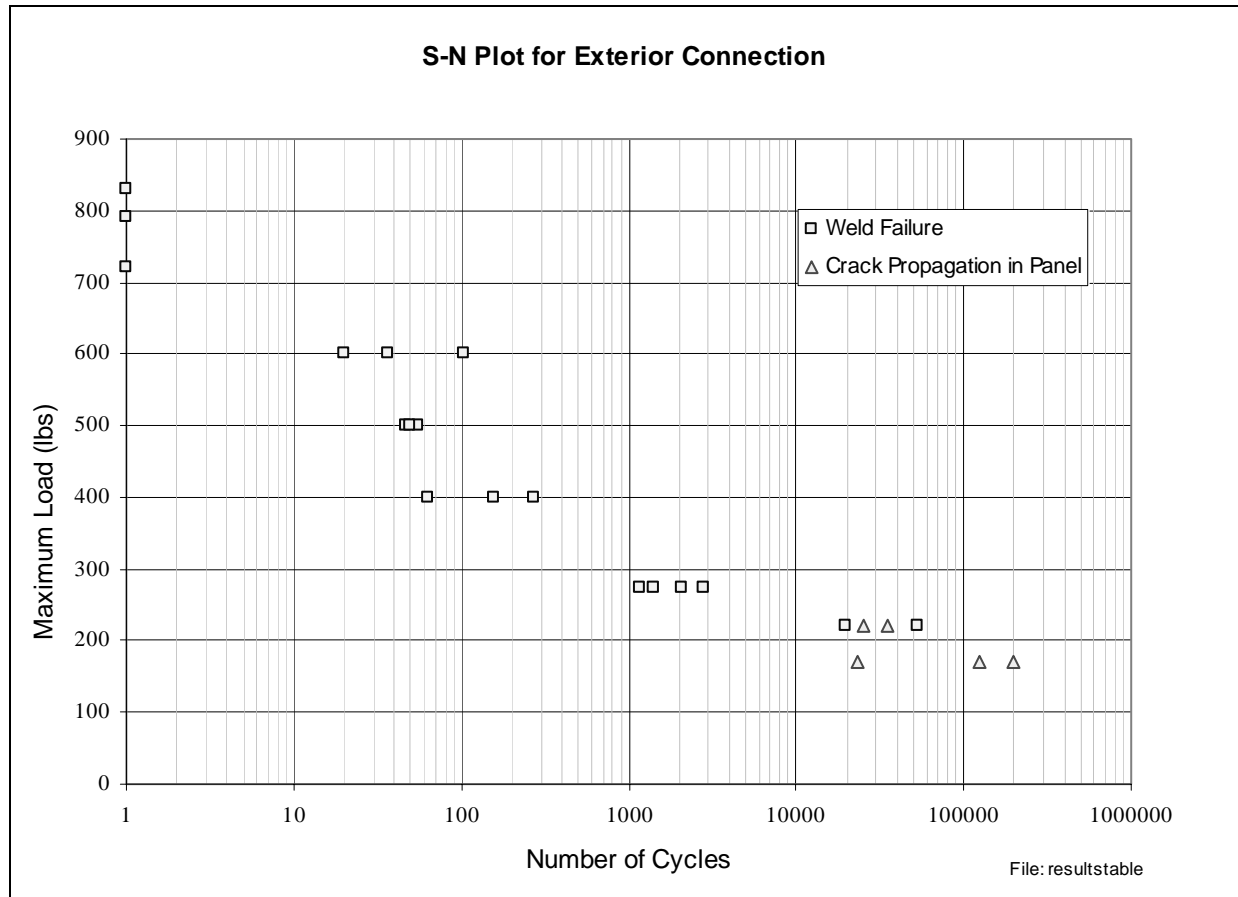* crack in panel, not weld failure

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Figure 25. S-N Plot of Exterior Connection.

DISCUSSION

Interior Connection

The objective behind the testing procedure was to first test an interior, uniform loading condition and work towards the non-uniform edge connections. It was thought that the interior connection would be better understood and simpler to model. It was also the objective of the testing procedure to start with a high range and work down to "infinite fatigue" life. The loads at which the base metal failure was found was about twice that of the gust pressure on the sign, which was calculated above. This failure mechanism, as previously mentioned, is not consistent with the failures found in the field. It was assumed at this point in time that the loading conditions and boundary conditions used in the lab did not properly model the loading conditions of a full sign in the field. Because of this, the interior connection fatigue testing was stopped. Since the field failures were observed to start at the edge, a simulation of an exterior connection was then attempted.

Corner Connection

The exterior connection was tested with the same objective as the interior fatigue connection, to start at a high stress range and progress to lower stress ranges until the infinite life stress range was determined. The exterior connections were observed to have lower pseudo-static strengths than the interior connections. This was expected due to the larger deformation and bending stresses induced on the stud connection. The weld fracture was the controlling failure mechanism at approximately 1.5 times that of hurricane design forces. The test panels exhibited base metal failure and cracking instead of a weld fracture at load ranges lower then 220 lb. The test panel tested at a load range of 100 lb exhibited an extended fatigue life. This load range is still above hurricane gusts of 90 mph and just below gusts of 120 mph for the sample sign. Under this loading type, the signs should not fail under common wind gusts. Because the tests did not model the failures found in the field, it is presumed that the loading conditions used for these tests did not properly simulate those found on full size signs in service.

CONCLUSION

The highway sign failure is a dangerous and urgent issue to address. These tests were designed with the intention of addressing this problem and determining a solution as quickly as possible with the limited amount of material available for testing. It must be noted that these results are based on testing sample connections from one new sign panel. There are many variables (including weld quality and pre-torque) involved in manufacturing and erection that add to uncertainties in the quality of the signs in service and the test sign. Unfortunately, the testing procedures described did not adequately model the loading conditions found in the field because the load ranges that found "infinite fatigue" life cycles are still higher than the fatigue loads due to wind gusts in the field. Because the failures found in the test specimens did not match the failures found in in-service signs, a proper S-N curve could not be produced. Without the proper S-N curve, a priorities list for inspection and retrofitting cannot be formulated.

RECOMMENDATIONS

1. *Inspections and retrofitting scheduling should be increased because of the statistical uncertainty.* Because there are so many uncertainties observed about how the signs are manufactured and erected it is suggested that the inspection process be accelerated.

2. *Full size highway signs that are still in use should be gauged and monitored to determine what type of loading to which they are subjected.* There are some issues concerning which signs should be tested because wind loads can be site specific. It is suggested that signs in the same area or setting as known failures be gauged.

3. *Full scale testing with wind loads may be the proper way to test the highway signs.* This requires multiple signs, which can be a problem because they are not readily produced anymore.

REFERENCE

American Association of State Highway & Transportation Officials. *Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals,* 4th Ed. Washington, DC, 2001.

APPENDIX

LOAD CALCULATIONS ACCORDING TO AASHTO

The following calculations are following Section 3.8 in the AASHTO Specifications (AASHTO 2001).

$$P_z = 0.00256 K_z G V^2 I_r C_d \quad (psf)$$

V = 90 mph for most of Virginia

   = 110-120 mph for the coastal areas of Virginia

C_d = 1.2 (assuming a L/W ratio of 5.0)

K_z = 1.0 (assuming sign is 32 ft above ground at center point, can be conservative)

G = 1.14

I_r = 1.0

For 90 mph wind velocities:

$$P_z = 0.00256(1)(1.14)(90)^2(1.0)(1.2) = 28.4 \text{ psf}$$

Assuming connection spaces are 30" along the vertical and 14" along the horizontal:

Force per connection = 28.4 psf (1.17 ft)(2.5 ft) = 83 lb

For 120 mph wind velocities:

$$P_z = 0.00256(1)(1.14)(120)^2(1.0)(1.2) = 50.4 \text{ psf}$$

Assuming connection spaces are 30" along the vertical and 14" along the horizontal:

Force per connection = 50.4 psf (1.17 ft)(2.5 ft) = 147 lb