FATIGUE BEHAVIOR OF HIGHWAY WELDED ALUMINUM LIGHT POLE SUPPORT STRUCTURES

A Dissertation

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ABSTRACT

A number of localized failures have developed in cantilevered supports of highway signs, luminaries and traffic signals over the past ten years. Failures due to fatigue crack growth around welded structural details have occurred in socket connections within New Jersey, Iowa, Florida, Wisconsin, California, Massachusetts and Wyoming. Many of these failures have resulted from the interaction of the wind and the structures, resulting in numerous applied stress cycles.

Accordingly, fatigue tests were conducted on full-scale welded aluminum light poles containing through plate and shoe base socket connections. Through plate and shoe base socket connections are used to anchor aluminum light poles to a break away foundation.

Initiative for the study was a lack of available data for aluminum structural details, and the result of detrimental changes to specifications governing the design and proportioning of welded aluminum luminaire supports subjected to fatigue loading. As such, fatigue tests were conducted on light poles containing both shoe base and through plate socket connection details order to study the fatigue behavior and determine a lower bound resistance suitable for design in terms of S-N curves. Parametric studies using the finite element method were conducted on both detail types in order to understand the
nature of the local stress fields governing fatigue behavior and how changes in geometry affect the local stresses.

As stress range is the primary parameter used to describe fatigue strength, hole drilling strain measurements were utilized to examine whether the light pole details contained significant tensile residual stresses normally assumed to exist with welded construction.

Fatigue tests revealed relatively low strengths for the through plate socket connections as compared to the shoe base details. Near the constant amplitude fatigue limit, the difference in strength was nearly a factor of 3.5. Residual stress measurements revealed the existence of compressive residuals stresses on the surface of the tubes for both types of details measured to be close to -18 ksi. Results of the parametric study of the through plate socket connection showed a 30% reduction in the longitudinal stress on the surface of the tube by increasing the base plate thickness from 1 to 2 inches. For a 1 inch base plate thickness, additional bending through the tube wall and elevated longitudinal stresses were observed opposite to the bolt location on the tension side of the pole. Attempts to stiffen the through plate socket connections using triangular plate stiffeners resulted in elevated longitudinal stresses at the tip of the stiffeners for short stiffeners that contradict current AASHTO specification fatigue categories for such type of structures. SEM examination of typical fracture surfaces showed the existence of fine striations, secondary cracking and a region of ridges and grooves.

Future recommendations from this study includes (1) fatigue tests on steel and aluminum through plate socket connections with the proposed 3 inch base plate thickness, (2) Long term monitoring of Signal Sign structures in the State of Ohio to
validate pressure and load recommendations for vortex, galloping, truck induced and wind load effect and (3) Vibration tests on luminaire support structures to measure the first modal natural frequency for such types of details.
DEDICATION

To the seven closest people in my life:

Dad: Your support throughout my studies will never be forgotten.

Mum: Your prayer and patience was the foundation of my achievements.

Sisters and Brother

Dr.Duua: Hope your soon to come baby boy/girl will appreciate his uncle continuing education.

Walaa and Ghaidaa and Brother/Tharaa: Proud to be part of this family
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CHAPTER I
INTRODUCTION

1.1 Statement of the Problem

1.1.1 Fatigue and Luminaire Structures

A lack of fatigue test data on highway aluminum light pole structures is in direct contrast to the fact that fatigue plays an important role in the design and life of such structures. Typically, a round tubular member is inserted into a hole cut into a plate or a specially designed integrally stiffened casting (shoe base). Fillet welds are placed between the pole and plate or casting in two locations. The presence of a fillet weld at top of the cast base to tube connection or flat base plate to tube joint is an area of some design concern on such luminarie structures. Until now, there has been a lack of fundamental knowledge on the behavior of both detail types, and historically have been treated in an identical manner as low fatigue strength details according to AASHTO (AASHTO, 2001). This, in spite of a long history of good field performance for the cast shoe base socket connection detail.

It’s common throughout the United States to observe two types of pole supports along sides of a highway. The first type, which will be the focus of this research, is a luminaire structure. The two most common configurations are a single support with a cantilevered arm and a single, straight support with the light directly on top. Single or truss arm types are used for traffic signal and sign structures and are usually described
based on the vertical and horizontal members. The vertical members are typically referred to as columns, poles, posts, or masts. Traffic signal structures with only one pole are referred to as cantilever structures, whereas a structure with two or more columns may be referred to as sign bridge or overhead structure. The horizontal members of the structure consist of either one member or a truss. The single member is called a monotube or mast-arm. The truss structure may have two or more chords and is referred to as a two, tri or quad chord truss.

1.1.2 Loads Affecting Luminaire Structures

In general, cantilever sign, signal, and luminaire support structures are susceptible to at least one of four types of wind induced-loading as described in the Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals published by the American Association of State Highway and Transportation Officials (AASHTO). Loading is given as a function of the dynamic behavior of the structure, and includes:

1) Galloping;
2) Vortex shedding;
3) Natural wind gusts; and,
4) Truck-induced wind gusts.

It is generally believed that, luminaire structures are not susceptible to galloping and truck-induced wind gusts. However, vortex shedding and natural wind gusts are the primary cause of fatigue damage in luminaire structures.
Vortex shedding is defined as a steady uniform airflow that travels over the face of the pole, and results in thin sheets of tiny vortices on the back side of the pole. Structural poles should be designed against vortex shedding to avoid approaching one of the first several natural frequency modes of the structure. Historically, excitation in the second mode has been attributed to vortex shedding. However, if the structure is flexible, natural wind gusts may excite the first mode natural frequency. Light pole structures are generally susceptible to vortex-induced vibrations in the range of wind velocities between 5 and 15 m/s (11 to 33.5 mph). If pole experiences a loading close to one of the natural frequencies, it will result in significant displacements and stress ranges, eventually leading to failure.

Natural wind gusts arise from the changes in velocity and angles of attack of the air stream. A common approach for estimating the maximum pressure imposed on a structure by a gust is through the use of a gust factor $G_f$. The gust factor is used as a multiplier to increase the wind speed in a certain location. In simplest form, a single wind gust results in a single stress cycle. However, the actual situation is more complex. A single gust may result in more than a single stress cycle on the structure when a dynamic response is triggered. The gust factor accounted for in AASHTO, is an attempt to account for the dynamic interaction of the wind and the structure.

1.1.3 Failures and the Need of Research

Both types of socket connections experienced failures in either aluminum or steel during the last decade. The aluminum shoe base detail failures that occurred in 1996 along route 147 along in New Jersey, were for 45ft high poles. It is believed that during
the night of the failure, poles experienced repeated stress ranges exceeding 12 ksi. Modal Finite element analysis of the failed poles revealed that vortex shedding in the fifth mode controlled the shoe base-to-pole connection. Further, the structure was believed to be inadequately designed for fatigue. It should be noted that there are two approaches that may be taken to decrease the likelihood of such failures. First, the detail may be redesigned, or vibrations may be mitigated through use of dampers. Pole fabricators manufacture dampers to mitigate second mode vibration and in many cases, are supposed to be standard for 45 ft tall poles.

Through plate socket connections for steel have experienced fatigue issues in different states from west to east coast. In one study, it was reported that 33% of poles in service already have fatigue cracks (Gray, 1999). In another study, response from 36 states revealed that approximately one-half had fatigue problems associated with wind induced vibrations on cantilevered structures (Kaczinski et.al 1998). Subsequent to the New Jersey failures of aluminum light poles, limited fatigue tests were conducted. Conclusions of the study stated that “fatigue strength of the shoe base socket connection detail was equal to Category E’ ” (Johns and Dexter 1998). However, the data clearly shows that all failures were significantly above an E-Category, indicating a shallower trend than other welded details.

In order to better understand the fatigue behavior of both types of details, fatigue tests, residual stress measurements and finite element models of welded aluminum light poles were conducted.
1.2 Objective of the Study

1.2.1 Fatigue Tests and Proposed S-N Curves

Both through plate and shoe base connections have experienced fatigue problems in both the US and Canada. Both are classified as fillet-weld socket connection details and categorized as E’ according to AASHTO. Aluminum luminaire structures are the primary aspect of the research described here, and it is clear that the shoe base detail has not received as much attention as the through plate socket connection. To establish accurate results from fatigue tests, full scale aluminum light poles were tested. Twenty shoe base connection details made of aluminum alloy 6063-T6 extruded tubes and A356 castings were fatigue tested. In addition, ten through plate details fabricated from aluminum 6063-T6 extruded tubes and 6061 plate were evaluated for comparison purposes. Tests were conducted using small positive R ratios, and applied stress ranges that varied from 3.5 to 9 ksi. All samples were prepared according to the current processing path employed by HAPCO, the project sponsor. Strain gages were installed to monitor the stress range for each test article.

Data from the fatigue tests was gathered and analyzed using statistical techniques. Data was gathered, plotted and compared to current design S-N curves from the Aluminum Design Manual (ADM) (ADM, 2005). Resulting fatigue data proved to be sufficient enough to allow for determination of an S-N curve for the shoe base detail.

1.2.2 Residual Stress Measurements

Residual stresses play a significant role in the fatigue resistance of welded structures. Presence of residual stresses on the shoe base connection has raised many
questions on the failure of those types of poles. It was important that the samples for the residual stress measurements be put through the same processing path as the fatigue test samples. Four gages were installed on each side of the pole 90° apart as close as possible to weld toe, to obtain strain readings using the hole drilling method at the expected critical area.

1.2.3 Finite Element Models (FEM)

The issue of plate flexibility is an interesting one, first documented by Sharp and Nordmark in 1976 on tests of aluminum truss connections. Results showed that fatigue strength was primarily the result of bending through the tube wall and local stresses were not accurately predicted by a simple mechanics approach (Sharp et al. 1997). Since then, research to study the behavior of the through plate detail, especially in steel cantilever mast arm signals and signs has been conducted. In support of this work, the behavior of aluminum through plate details was studied for comparison purposes and specifically, to check if the behavior trends were the same as experienced in steel structures. The effect of base flexibility is not currently included in AASHTO specifications. For that reason, extensive parametric studies using the Finite Element Method (FEM) software were conducted. In addition, both tube diameter and plate thickness for commercially used light poles was studied to see how the stress concentration changed. Also, a weld leg parametric study was conducted to validate the behavior of unequal weld legs as observed in previous research by Fisher (Fisher et al. 1981). Further, the use of collar stiffened socket connections, effect of holes and triangular gusset stiffeners were modeled and examined.
1.2.4 Fracture Mechanics and Life Predictions Based on FEM Results

Life predictions of the welded aluminum poles were based on fracture mechanics models. Stress gradient solutions from the finite element models were used to estimate the life of the poles. Therefore, a close and accurate estimate of $K_t$, the stress concentration near the toe (top of weld) was necessary to calculate $F_g$, the stress gradient correction. With the aid of AFGROW, fatigue lives of both structures were computed and compared to test results.
2.1 Highway Traffic Control Structures

Signal, sign and luminaire structures are widely utilized for traffic control all over the states. Signal and sign structures consist mostly of a vertical pole, with an attached cantilever arm for the signals and signs. Luminaire structures generally consist of a single support with a cantilevered arm or a single, straight support, with the light placed directly on top. Signs are often seen to help define interstates, exits and street numbers. Signals aid in displaying different colored lights or colored lighted arrows in red, yellow or green.

Along a major highway, luminaire structures may be seen every $\frac{1}{10}$ of a mile. Signal structures often occur nearly every half a mile. From documented cases, it appears that these structures started to experience fatigue problems in the last three decades. The general public might not be aware of the problem, because if such a failure occurs, the structure is replaced. Those working in the fatigue area realize that this issue is a serious matter. Clearly, the damage is costly, costing up to thousands of dollars per occurrence. Traffic control devices may be fabricated using steel or aluminum and often depends on the agency responsible for maintenance.
In the AASHTO specifications, an unequal leg fillet-welded socket connection is classified as a category E’ detail. The term “socket” refers to the way the baseplate is cut-out to allow the pole to fit inside. Two fillet welds connect the base plate or casting to the pole. The first and most structurally significant is applied at the top of the base plate or casting. The second fillet weld is applied inside the cut-out in the base plate, between the bottom surface of the pole and the sides of the base plate. This weld is much smaller, and in general, much more irregular. The lower weld serves a smaller role, and primarily to helps to prevent corrosive materials from entering the gap between the tube wall and the base plate or casting as shown in Figure 2-1. To fulfill the infinite life design requirement of the fatigue provisions, the anticipated stresses at the location of the socket weld must be lower than the 1 ksi which is the Constant Amplitude Fatigue Limit (CAFL) for E’ details. The CAFL is the stress range below which no crack is expected to initiate and grow (AASHTO, 2001). Both details studied in our research are categorized as fillet-welded socket connections.

Figure 2-1. Fillet-welded through plate socket connection detail
One important reason to investigate both detail types is to examine the influence of base plate flexibility on fatigue performance of socket connections. In short, tests and analyses were conducted on the through plate socket and shoe base connection details to examine base flexibility as originally observed by (Sharp et al. 1996).

In order to determine the state of current practice for these particular fatigue issues encountered in aluminum luminaire details, a literature study was conducted and divided into four parts:

1) Fatigue concepts, history and applications.
2) Types of highway poles and discussion of vortex shedding and wind induced loads.
3) Failures and lab testing that occurred in the last three decades on through plate and shoe base socket connections.
4) Finite element analysis, residual stress and fracture mechanics for fatigue life prediction.

2.2 History of Fatigue

Studies go back to the early 1800’s when the people started examining fatigue failures and mechanisms. Definitions of fatigue are of the form of “the weakening of a metal when subjected to repeated vibrations or strains or the tendency of a material to break under repeated cyclic loading at a stress less than the tensile strength” (Schultz, 1986). The word fatigue is used because failures occur after a period of time and generally, it’s catastrophic, because the results is soften fracture of the part. Fatigue
failures initiate in a wide variety of structures including aircraft, cars, bridges, buildings and in sign, signal and luminaire structures.

Albert, a civil servant for mines, first noted this concept in 1829 with the failure of conveyer chains which were constructed and subsequently failed in service in coastal coal mines. In 1839, Poncelet utilized the word “fatigue” to describe a metal component or structure as being tired or worn out. Five years later in 1844, Rankine studied the stress concentration effect on railroad axles. During this time, the Versailles accident occurred, where 60 human lives were lost (Schultz, 1986). In 1860 S-N curves were first described by Wohler. He represented fatigue test results on railway axles in the form of tables. But, his successor, Spangenberg, plotted them as curves, but still referred to them as Wohler curves. He presented a final report with the following conclusions “Material can be induced to fail by many repetitions of stresses, all of which are lower than the static strength….” (Brooks and Choudhury, 2002). Basquin represented the finite life region of the “Wohler curve” in the form of $\log S$ on the ordinate and $\log N$ on the abscissa. He observed an endurance limit, called the Basquin Law (Basquin, 1910). The S-N curve is the mean curve drawn or calculated to represent survival of 50 percent of the data points.

As the twentieth century approached; Baushinger defined what is called the cyclic behavior of materials ($\sigma - \varepsilon$) or the cyclic stress strain curve (Bauschinger, 1886). In 1903 Ewing’s and Humphries developed the idea of fatigue crack initiation using cyclic deformation. The linear damage rule for variable amplitude loading was first proposed by Palmgren in 1924 and was further developed by Miner in 1945. Today the method is commonly referred to known as Miner's Rule (Nishida, 1990). It states that the damage done by each cycle is assumed to be proportional from the first to the last cycle.
Therefore, when N cycles have been applied; the amount of damage is the fraction \( \frac{N}{N_f} < 1 \) if safe. In 1954, Coffin and Manson observed in the low cycle regime, there is a nominal plastic strain in each cycle which causes the accumulation of “fatigue damage” (Bannantine, et.al 1990).

2.3 Fatigue Applications and AASHTO Specifications

Ten publications are prepared daily around the world dealing with fatigue (Sharp et.al, 1996). In spite of this, fatigue failures are costly, and represent approximately 4% of the gross national product (GDP). Failures of aircraft components are also worth mentioning, as this is a national safety issue. In 1958, two Boeing B-47 nuclear bombers crashed due to fatigue failure of a wing. Further, consider that the B-47 was the only aircraft capable of reaching the USSR at that period of time.

Loading conditions were and often are characterized by stress amplitude alone. With the rapid development of fracture mechanics, there was a need for research on fatigue crack propagation. The main issue was concerned with an appropriate description of the material response \((da/dn \text{ i.e. the crack growth rate})\). A new age and generation of fatigue was founded in 1961 when Paris and Erdogan realized that crack growth was directly related to the stress intensity factor range, \(\Delta K\). What is now referred to as the Paris Law, \[ \frac{da}{dN} = C(\Delta K)^n \] where C and n are material constants and can be determined by conducting fatigue crack growth tests according to the American Standard of Testing Materials (ASTM), \(\Delta K\) is the stress intensity factor range \((K_{\text{max}} - K_{\text{min}}) = Y\sigma\sqrt{\pi a} \text{ (ksi}\ldots\text{in})\).
in $\frac{1}{2}$-MPa m$^{\frac{1}{2}}$), $a$ is the crack length, $Y$ is a geometry factor and $N$ is the Number of cycles.

$$\frac{da}{dN} = C\Delta K^n$$

Where, $\frac{da}{dN}$ = increment in crack growth

$C$ = constant for material

$\Delta K$ = stress intensity factor range

and $n$ is a material constant

The stress intensity factor, characterized as a single parameter, $K$ may be defined as the stress field ahead of a sharp crack in a test specimen or a structural member (Fisher and Viest, 1964). In general, three main regions of crack growth are recognized. The first region is near the threshold, a range where crack growth is slow or non existent and the number of cycles tends to infinity. The second is the steady state region where the crack is propagating in a stable manner and the third region, is the fast fracture region where the structure or component is nearing complete failure as shown in Figure 2-2.
The Paris law was not readily accepted as the $\Delta K$ parameter is based on an elastic stress field in the crack front vicinity, and for that reason the paper was rejected by three leading journals until it first appeared in the Trend in Engineering, a periodical published by the University of Washington in St.Louis (Schultz, 1986). Nearly 20 years later, in 1982, Paris said the following:

“Ironically, the paper was promptly rejected by three leading journals, whose reviewers uniformly felt that it is not possible that an elastic parameter such as $K$ can account for the self-evident plasticity effects in correlating fatigue crack growth rates”

The Welding Research Council published the first specifications for the fatigue of steel welded joints in 1940. In 1961, tests were conducted on steel bridges to examine the
effect of cracks that formed along welded toes of cover-plated beams in steel structures (Kosteas, 1981). Aluminum structures did not have any formal universally accepted rules until 1986. Before 1986, data belonged to government laboratories, experiments and individual companies.

In the late 1970’s, the European Community started to update the fatigue design provisions for aluminum structures (Kosteas et.al, 1985). The Institute for Steel Structures at the Technical University of Munich was entrusted by the European Committee for Constructional Steelworks (ECCS) to produce the first draft of the code. It should be noted that all data on welded aluminum structures up to this period of time were the result of non full-scale structures. In 1974, the first Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signal by American Association of State Highway and Transportation Officials (AASHTO) was published, but fatigue was not included as a design specification.

The first comprehensive set of full-scale tests not performed for a private entity on welded aluminum beams was conducted in the 1980’s at the Technical University of Munich[15]. Beam samples were tested using Stress ratio’s (R) from 0.1 and –1 respectively. All experiments were conducted on alloys 5083 and 7020. The results showed that the mean S-N curves for the beams were at a lower level than similar details on small-scale specimens.

Although research started to increase in the area of fatigue analysis of signal, sign and luminaire structures in that period of time a second AASHTO specification was published in 1985 and again, didn’t include specific fatigue design specifications and guide chapters. Research under National Cooperative Highway Research Project 17-10
was conducted, with the objective to develop an up-to-date comprehensive specification, and an accompanying commentary, for structural supports for highway signs, luminaries, and traffic signals. Tests were performed at The University of Alabama (Fouad et.al, 1998). The new specification reflected the state-of-the-art design philosophies and manufacturing processes at the time, and included guidance for poles fabricated from steel, aluminum and other materials. However, it still did not include a separate fatigue guidance chapter. Project, NCHRP10-38, conducted at Lehigh University beginning in 1997, dealt with the study of the fatigue resistance of cantilevered signal, sign, and light support structures (Kaczinski et.al, 1998). A third draft of AASHTO was proposed in 1999 (AASHTO, 1999). Not until 2001, when the fourth edition for structural supports for Highway Signs, Luminaires, and Traffic Signal was published, did, for the first time, include new fatigue design provisions in Chapter 11 of the manual.

The 4th edition of the specification provides fatigue importance factors ($I_f$’s) in tabular format. Theses importance factors are similar to those found in seismic design and relate the severity of the consequences of a fatigue category I structure failure. Category I corresponds to the percentage of wind loading ($I_f = 1.0$) applied to high level lighting poles adjacent to major highways in which the vehicle speed is great enough to cause damage in the event of failure. Category I is also for those structures with high level lighting poles exceeding 90 ft. Fatigue importance factors for Category II and III structures are smaller and the structures are designed to resist lesser factored wind loads (ASCE 97-5). Importance factor II corresponds to the previous AASHTO edition’s most severe fatigue loading. The increase in fatigue design loading was consistent with the recommendation of NCHRP report 412 (Kaczinski et.al 1998).
2.4 Factors Influencing Fatigue Failures in Luminaire Structures

By in large, the number of cantilevered and overhead structures found along major highways across the country are well-engineered structures, void of problems (Ginal, 2003). Despite a relatively low failure rate, estimated to be on the order of one pole in 10,000, welded aluminum light supports have been scrutinized as the result of several well-publicized failures that have occurred over the past ten years (Fisher (1977), Dexter and Johns, (1998), and Kolousek, (1984)). In a majority of cases, the failures were attributed to sudden, severe weather events that have resulted in the poles being subjected to a number of large stress cycles or “sudden fatigue”. Several of these incidents coincided with an effort to update and revise specifications for the design of cantilevered supports for luminaries, signs and traffic signals. However, as will be highlighted in this section, these structures have experienced performance issues which have concerned departments of transportation throughout the US and Canada. The performance of these structures may be related to complex loading schemes, inappropriate to design choices, the use of fatigue sensitive details, the quality of fabrication, and combinations thereof (Kosteas, 1981). These structures are subjected to both static and dynamic loads; it is the dynamic component that directly affects fatigue performance.

Luminaires are used at various locations on major highways and in towns for the purpose of roadway illumination. Luminaire supports come in a variety of configurations and materials. The two most common configurations are a single support with a cantilevered arm and a single, straight support with the light directly on top. They’re
made from aluminum or steel. It’s believed that four loading conditions affect the fatigue behavior of signal, signs and luminaire structures and those are:

(1) vortex-shedding; (2) galloping-induced vibration (3); natural wind pressures (4) truck-induced wind pressures. Luminaire structures are typically not susceptible to galloping and truck-induced loads.

Vortex shedding occurs on a light pole with a discrete range of wind speeds when air flows past a pole structure causing vortices to shed on alternating sides. A sinusoidal pattern of vortices forms on the leeward side of the pole and is known as a Von Karman sheet. The pressure distribution created around the pole results in forces across the sides of the pole. The Strouhal relation gives the frequency, $f_s$, of the shedding vortices in the equation:

$$f_s = \frac{SV}{D}$$  \hspace{1cm} (2-1)

where $S$ is the Strouhal number, taken as 0.18 for round tubes, $D$ is the across-wind dimension of the element, and $V$ is the free-stream wind velocity (Kaczinski et.al 1998). When the frequency of vortex shedding does not match one of the natural frequencies of the structure, the structure will show a minimal response. However, when the frequency of vortex shedding approaches a natural frequency of the pole, the result is an increase in vortex strength, an increase in the correlation of the shedding forces, and a tendency for the shedding frequency to become locked-in to one of the natural frequencies of the structure. The phenomenon known as” lock-in” occurs at a speed given by the Strouhal relation:

$$V_{CR} = \frac{f_s D}{S}$$  \hspace{1cm} (2-2)
where \( f_n \) the natural frequency of the structure.

Previous research indicates that the level of turbulence associated with wind
to velocities above about 40 mph limits the formation vortices (Cook and Bloomquist
induced vibration for wind speeds between 10 to 40 mph (Kaczinski, 1998). In general, it
vortex shedding does not play a significant role for fatigue loading of
cantilever mast-arms but, probably influences luminaire structures. However, it is
possible that vortex shedding could play a role in initiating galloping.

Natural wind gusts arise from changes in speed and direction of airflow. Natural
are characterized by a spectrum of velocities that occur with a broad range of
movement. Estimation of the maximum loading pressure on a structure by a gust is
through use of a gust factor. The gust factor, \( G_f \), corrects the effective wind speed, \( V \), for
the interaction of the pole and wind. The gust factor, \( G_f \), should not be confused with
the gust coefficient. Although both factors accomplish essentially the same purpose, the
gust factor, \( G_f \), is multiplied by the pressure, while the gust coefficient is multiplied by
the wind speed. A gust factor is a ratio of the expected peak load during a time period to
the average displacement load as given by \( \frac{Y_{\text{max}}}{Y} \). Several parameters combine to
produce a gust factor. These include roughness of the surrounding terrain, height of the
structure, and the pole geometry. A design wind pressure may be found from the gust
factor. A design wind represents the largest expected static wind pressure to produce the same response a structure subjected to maximum dynamic wind loading (Gray 1999). Previous versions of the specifications incorporated wind speed maps for 10, 25, and 50-year mean occurrence intervals. A new map presenting the variation of 3-second gust wind speeds was adopted in the 2001 AASHTO Specifications.

The change from fastest-mile wind speed \((V_{fm})\) to 3-second gust wind speed \((V_{3-sec})\) represents a major change to the specifications. Experience suggests that use of traditional gust coefficient of 1.3 has resulted in successful designs (ASCE 7-95, 1995). AASHTO, however, has adopted a uniform gust factor of 1.14, which is calculated using Equation 2-3 below,

\[
G_{3-sec} = \left( G_{fm} \frac{V_{fm}}{V_{3-sec}} \right)^2
\]  

(2-3)

Where,  

\(G_{fm}\) is the gust effect factor = 1.3,

\(V_{fm}\) is fastest-mile wind speed,

\(V_{3-sec}\) is the gust duration for fastest-mile wind speed (t, sec),

\[
\frac{V_{fm}}{V_{3-sec}} = 0.82 \text{ (Fouad, et.al 1997),}
\]

\[
\therefore \left( G_{fm} \frac{V_{fm}}{V_{3-sec}} \right)^2 = (1.3 \times 0.82)^2 = (1.066)^2 = 1.14
\]
2.5 Failures and Research Conducted on Cantilevered Sign, Signal and Luminaire Structures

Several studies and tests have been performed on cantilevered sign, signal and light pole structures. The objective of the studies differed, depending on the individual case and task of the project. In this section, most of the relevant studies and failures in the area of cantilever sign, signals and light pole structures for both aluminum and steel are reviewed.

Texas Loading of Structural Steel Sign Supports (1979)

This research was conducted to determine the fatigue loading on cantilever highway signs from gusts produced by passing trucks. A peak pressure of 1.23 psf with a duration of 0.375 sec are notable characteristics of truck-induced pressure pulses. It was concluded that truck-induced gusts produce significant sign support response in the form of a large number of stress ranges for each truck passage (Creamer, 1979).

Canada (1980) Failure of Overhead Sign Bridge

As a result of an overhead Sign Bridge failure in Calgary, Canada, this research focused on evaluating the vibrational and aerodynamic characteristics of overhead sign support structures through both field measurements and wind tunnel studies. One recommendation was to reduce the solid area of the sign, by 30-40%. Further, the box mounted on the back of traffic signals has a negative effect on aerodynamic behavior of structure. Subsequent wind tunnel tests showed that 1.8% of critical damping (0.005) is sufficient to eliminate vortex shedding (Irwin and Peeters 1980).
Fisher and colleagues (1981)  

Steel Through Plate Socket Connection Tests

This was the first research to study the fatigue behavior of standard light poles. Fatigue tests were carried out on a series of poles fabricated from A283 Grade D steel built by Ameron Pole Products Division having standard 45° equal weld legs and a series fabricated from A595 Grade A steel built by Valmont having unequal fillet weld legs. Two additional specimens from Ameron were fabricated separately for comparison purposes. Poles built by Valmont were designated as V1-V6, poles built by Ameron were designated as A1-A6 and the two additional specimens were designated as A7 and A8. Fatigue tests showed that poles with equal 45° fillet weld legs were below Category E’ while poles with unequal fillet weld legs had fatigue strength equal to Category E (Figure 2-3). Table 2-1 summarizes the CAFL for steel structures by AASHTO. Fatigue cracks appeared at the toe of the welds at the base of the signal arm and at the base of the pole at approximately same number of cycles. One conclusion was that small fatigue cracks were difficult to detect when poles were galvanized. The galvanized coating permits large cracks to form before it breaks and exposes the crack. Hence, most of the fatigue life was exhausted before cracks were detected. In this project the issue of base flexibility was also related to the fatigue results (Fisher et.al 1981).
Figure 2-3. Summary of stress range versus cycle life for A and V series

Table 2-1. Constant amplitude fatigue limit for welded steel structures

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>Steel Threshold</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
</tr>
<tr>
<td>A</td>
<td>165</td>
</tr>
<tr>
<td>B</td>
<td>110</td>
</tr>
<tr>
<td>B'</td>
<td>83</td>
</tr>
<tr>
<td>C</td>
<td>69</td>
</tr>
<tr>
<td>D</td>
<td>48</td>
</tr>
<tr>
<td>E</td>
<td>31</td>
</tr>
<tr>
<td>E'</td>
<td>18</td>
</tr>
<tr>
<td>ET</td>
<td>8</td>
</tr>
<tr>
<td>$K_2$</td>
<td>7</td>
</tr>
</tbody>
</table>

North Carolina State University conducted this project, with effort focused on wind loading and dynamic behavior of cantilevered truss sign supports. Wind tunnel tests along with analytical efforts resulted in an understanding of the response of these structures. Models were used to evaluate vortex shedding of a two-chord cantilever truss configuration. Wind tunnel tests showed Strouhal numbers equal to 0.196, to about 0.201. The study also attempted to measure the wind pressures that resulted from vehicular-induced wind gusts. The maximum pressure recorded on a sign due to vehicular-induced gusts was 1.41 psf. Creamer reported results of 1.25 psf, and appears that the two separate tests are consistent (Edwards and Bingham 1984).


Research conducted at The University of Illinois was intended to;

“Combine pertinent existing wind loading and vibration theory, fatigue damage theory and experimental data into a usable fatigue analysis method for overhead sign and signal structures”

The research included instrumentation of a traffic structure in Springfield, Illinois for the purpose of collecting wind speed data. An anemometer was mounted to a pole approximately 4 feet above the traffic signal mast-arm (25 feet above the ground). Wind speed data was collected from August 7, 1991 to January 25, 1993. Results from field monitoring were as follows:
1-Experimental investigations indicated that both welds and anchor bolts are subjected to a large number of stress cycles at relatively low stress levels as compared to static analysis procedures using wind speeds in excess of 70 mph.

2-Experimental investigation of mast arm signal structures indicated that vibration of the structure tends to occur in the first and second modes.

3-Example computations with strain-gauge data indicated that 30% of the fatigue damage is caused by stress ranges of about 4.5 ksi for steel through plate details.

4-Analytical procedures used in the report indicated that all significant damage was due to wind speeds ranging from 16-27 mph.

One significant final conclusion was that static analysis methods overestimated the fatigue life of structural details (South 1994).

McDonald (1994) Failure of Cantilevered signal poles

This research was initiated after the failure of a cantilever signal pole in Dallas, Texas in 1991. The purpose of this study was to “prepare revisions to the wind load section of the Texas DOT standard for highway signs, luminaries and traffic signal structures and to develop strategies for mitigating certain large amplitude vibrations in single arm traffic signal structures”. The report points out some of the limitations present in what was the current AASHTO standard (1985). Most notably, the lack of a gust response factor and omission of terrain roughness adjustments were cited as deficiencies. It is interesting to note that a new specification was published (AASHTO 1994) soon after this report was issued. In addition, water table experiments were carried out to study the effect of vortex shedding.
Results and conclusions from this report were as follows:

1-Traffic signal and arm configurations gave vortex shedding frequencies in the range of 1.20 <fvs<2.5 (Hz) for 10 mph wind speeds. For 20 mph, frequencies increased to 2.40<fvs<5.1 (Hz). Based on FE analysis, vortex shedding could be expected for lock-in wind speeds of less than 10 mph.

2-Water table experiments concluded that relatively low lock-in velocities at which vortex shedding occurs are not large enough to cause large amplitude vibrations of cantilever traffic signal support structures (McDonald et.al 1995).

Cook (1996) Florida Tests on variable message signs

This study was an effort to measure truck-induced gust loading on Variable Message Signs (VMS element). Pressures induced by 23 random trucks passing under the truss sign bridge were measured at a height of 17 feet above the roadway. The speed of each truck was determined using a radar gun and a truck–induced pressures were recorded for each truck event. Conclusions from experimental research indicated that pressure magnitudes from the study were low, with an average of approximately 1psf and a peak of 2.0 psf. Dominant frequencies of the truck-induced pressure pulses were in the range of 0.5-3.0 Hz. As a result, the FDOT currently makes attempts to ensure that natural frequencies of the supporting structure do not approach 0.5-2.0 Hz (Cook et.al 1996).
DeSantis and Haig (1996) Failure of Steel Sign Truss in Virginia

This paper outlines an investigation of the factors leading to the failure of a truss-type overhead cantilevered highway sign support structure in Virginia. The structure was less than one year old and supported a variable message sign. The failure mode described in this paper was due to fatigue. Cracking occurred around the circumference of a steel pole, along the toe of the base plate weld in the heat-affected zone (HAZ). An analytical investigation into the cause of the VMS collapse and truck-induced gust loading was conducted.

The report concluded that the pressure exerted on the underside of the sign was nearly 26.5 psf. Upward lifting of the cantilever truss was followed by a “rebound effect”. It was shown that a stress of nearly 10.8 ksi could exist at the base of the vertical support each time a truck passed. Truck-induced gusts were verified by using field observations, and it was reported that signs moved “about one foot” up and down each time a truck passed beneath. Therefore, the pressure needed to cause the pre-mentioned observation corresponds to a truck speed of approximately 60 mph (DeSantis and Haig, 1996).

Gilani (1997) Failure of Variable Message Sign in California

This report summarizes a research effort conducted for the California Department of Transportation (CALTRANS) after the collapse of a Variable Message Sign (VMS) support structure. The research involved both field and analytical studies. Field measurements were undertaken to quantify the natural frequencies of vibration and structural characteristics. The analytical study was conducted using FE analysis to
evaluate the stress distributions around access holes and within base plates. The experimental and analytical investigations revealed that a conduit hole was a significant source of fatigue concern in the VMS support. This report also briefly evaluated the wind loading experienced by VMS structures. Vortex shedding was considered possible although it would occur at a low velocity (5 mph). At low wind speeds stress ranges would not be of any practical significance (Gilani et.al 1997).

Sparks (1997)  
Failure of welded Aluminum Light Poles in Massachusetts

This study was carried out after failure of 4 welded aluminum light poles with shoe base connection details in Boston, Massachusetts. This report represents one of the first documenting fatigue failures of shoe base connection details. Poles were 50’ high with 0.219 inch thick walls and 10 inch diameter tubes. Poles were more than 20 years old and would not meet current design standards. Scanning electron microscope (SEM), optical microscope examination, tensile properties and chemical analysis were carried out on four samples to study the causes that led to failure of these details. Failure of the poles was attributed to fatigue initiating in corrosion pits. Both outside diameter (OD) and inside diameter (ID) surfaces of the tubes, and weld deposits showed pitting attack. The pitting is attributed to exposure of the poles to chemical road salts containing chlorine used in snow melting operations. Sulphur was present in the corrosion deposits and is indicative of automotive exhaust products. (Sparks, 1997)
P&K Pole Products (1998)  Failure of Aluminum light poles in Massachusetts

This was a report developed by P&K Pole Products for the four poles that failed in Massachusetts in 1997. The results were outlined as follows:

1-The poles were over-welded at the direction of the Mass Highway Department at the time. Using a half inch weld subjected the material to excessive temperatures for extended periods.
2-The 50 ft. high poles were mounted on barrier only inches from high speed traffic, and as a result; subjected the poles to more than normal wind loads and vibration stresses.
3-Aluminum poles with a 50 ft. mounting height on median barriers should be protected with a two-part epoxy coating for the first six feet as a precaution (Riggs, 1998).


This report documented the second case of poles failures supported by shoe base details. In this report, the fracture of two aluminum pole components in New Jersey were examined to characterize the mechanism of fracture and determine possible causes for failure. It was reported that the Luminaires had been in service for less than one year prior to the failure. Scanning Electron Microscope (SEM) examinations were conducted on both samples. Results for the SEM analysis stated that: (a) visual and microscopic examination demonstrated that failure resulted from fatigue loading induced by wind. Fatigue cracks initiated along the weld toe of the fillet weld joining the pole to the shoe base and propagated through the wall thickness and around nearly 80% of its circumference. (b) Characteristics of the crack surface showed the crack to have propagated at a high rate. (c) Transformer base fracture surfaces indicated that the cracks
did not form by fatigue, but rather resulted from an overload condition (Kaufmann et.al 1997).

Johns and Dexter (1998) Research and Failure of Aluminum light poles in New Jersey

A total of fourteen luminaire supports failed along Route 147 in New Jersey. These cantilevered supports experienced cracking around the shoe base-to-pole weld and at the welds around the hand access holes.

Twelve luminaire support standards were sent to Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University to determine the fatigue resistance of the socket joint at the pole to shoe base connection. Six cantilevered supports (C) and six standard poles (S) were included. Pull tests were also performed to determine the structural characteristics, including stiffness, natural frequency and percent of critical damping. Finite element analyses were completed to evaluate the natural frequencies of shoe base socket connection luminaire supports for straight poles with top mounted lights as well as the geometry of the poles that failed on Route 147 as shown in Tables 2-2 and 2-3.

For the third mode natural frequency, the critical wind velocity was estimated to be,

$$V_{cr} = \frac{f_n \cdot D}{S} = \frac{(5.24 \text{cycles/s})(0.1778 \text{m})}{0.18} = 5.2 \text{m/s}$$,

where, D is the average tapered diameter of the New Jersey poles. Recall that $V_{cr}$ to cause vortex shedding is between 5 and 15 m/s. Therefore, this vibration mode shape is consistent with vortex shedding. As such, it is believed that vortex shedding played a role in the failure of the aluminum support standards along Route 147.
Table 2-2. Straight support standard modal analysis (Johns and Dexter, 1998)

<table>
<thead>
<tr>
<th>Mode Shape</th>
<th>Natural Frequency</th>
<th>Critical Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.74*</td>
<td>0.7</td>
</tr>
<tr>
<td>3</td>
<td>5.24</td>
<td>5.2</td>
</tr>
<tr>
<td>5</td>
<td>15.2</td>
<td>15.0</td>
</tr>
<tr>
<td>7</td>
<td>29.9</td>
<td>29.5</td>
</tr>
</tbody>
</table>

*Based on experimental data from pull tests, all others are based on FEA

Table 2-3. Cantilevered support standard modal analysis

<table>
<thead>
<tr>
<th>Mode Shape</th>
<th>Natural Frequency</th>
<th>Critical Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.79</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>1.02*</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>1.7</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>3.9</td>
<td>3.5</td>
</tr>
<tr>
<td>5</td>
<td>7.2</td>
<td>7.1</td>
</tr>
<tr>
<td>6</td>
<td>7.5</td>
<td>7.4</td>
</tr>
<tr>
<td>7</td>
<td>10.1</td>
<td>10.0</td>
</tr>
<tr>
<td>8</td>
<td>16.1</td>
<td>15.9</td>
</tr>
</tbody>
</table>

*Based on experimental data from pull tests, all others are based on FEA

Fatigue tests were conducted on 12 specimens, six cantilevered and six straight support standards as shown in Table 1-4. Tests were conducted according to the following test procedures; if a specimen ran for over 2 million cycles without cracking, then the next specimen would be run at a higher stress range. If this sample cracked, the next specimen would be cycled at the mean stress range of the two previous tests.

Conclusions of this report stated that fatigue tests results were consistent with Category E details. However, Figure 2-5 shows that results are significantly above category E, and closer to the behavior of Category D details. Further, the fatigue test data shows a shallower trend than the Category D, E or E’ details.

Fatigue tests at ATLSS were not sufficient to propose an S-N curve for this type of detail. For that reason, it was believed that more fatigue tests and research were
essential to validate and propose a general S-N curve for shoe base socket connection detail.

Table 2-4. Support standard fatigue test summaries (Johns and Dexter 1998)

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Stress Range (ksi)</th>
<th>Cycles to failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>5</td>
<td>2000000</td>
</tr>
<tr>
<td>S-2</td>
<td>10</td>
<td>55000</td>
</tr>
<tr>
<td>S-3</td>
<td>7.5</td>
<td>861000</td>
</tr>
<tr>
<td>S-4</td>
<td>5.6</td>
<td>2000000</td>
</tr>
<tr>
<td>S-5</td>
<td>7.0</td>
<td>197000</td>
</tr>
<tr>
<td>S-6</td>
<td>10.0</td>
<td>553000</td>
</tr>
<tr>
<td>C-1</td>
<td>9.3</td>
<td>109000</td>
</tr>
<tr>
<td>C-2</td>
<td>7.5</td>
<td>515000</td>
</tr>
<tr>
<td>C-3</td>
<td>8.0</td>
<td>2660000</td>
</tr>
<tr>
<td>C-4</td>
<td>12.2</td>
<td>36000</td>
</tr>
<tr>
<td>C-5</td>
<td>5.7</td>
<td>5710000</td>
</tr>
<tr>
<td>*C-6</td>
<td>5.0</td>
<td>5710000</td>
</tr>
</tbody>
</table>

*Only cantilevered support standard tested that was mounted to a transformer base

Figure 2-4 Fatigue test results in ATLSS lab facility
Another issue relevant to the investigations was damper installation. Dampers are only standard on the 45 ft straight support standards; the other support standards are fitted with dampers only if they exhibit excessive vibrations. These dampers have only been proven effective on the second mode of vibration of a straight support standard and may have not had any effect if a higher mode of vibration were achieved.

Washer installation was another issue in the investigation. There were reports that many of the heavy washers required for the transformer base to achieve its full strength were missing on the Route 147 installation. Missing washers were estimated to decrease the static strength by 50%.

Calculation of the fatigue strength for the failed poles was completed to check. From dynamic FEA runs for various mode shapes, moments from wind loads and vortex shedding were calculated, and it was believed that the 5th vibration mode was dominant. The stress range was found to be 16.7 ksi. Classification of the top fillet weld of the shoe base-to-pole socket connection is an category E detail. The corresponding CAFL is 1.9 ksi as shown previously in Table 2-1 Since the calculated stress range was greater than the CAFL, the shoe base-to-pole socket connection was determined to be inadequately designed (Johns and Dexter 1998).

Johns and Dexter (1998c) New Jersey Aluminum Tests

This journal article contains a summary of the previously described research studies conducted by Dexter et al. A very valuable piece of the document, not included in the final report is shown in Table 2-5. Table 2-5 is essentially a susceptibility matrix for sign, signal, and luminaire support structures (Johns and Dexter 1998)
<table>
<thead>
<tr>
<th>Table 2-5. Wind load susceptible loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Galloping</td>
</tr>
<tr>
<td>Sign</td>
</tr>
<tr>
<td>Signal</td>
</tr>
<tr>
<td>Luminaire</td>
</tr>
</tbody>
</table>

Note: X indicates structure is susceptible to this type of loading

* Vortex shedding occurred in an overhead sign bridge

Fouad (1998) NCHRP 411

Governing design specifications, used by many practicing engineers were considered outdated in 1997 and as such, the National Cooperative Highway Research Program (NCHRP) sponsored a project to update the 1985 AASHTO specifications. This research report recommended moving from fastest-mile wind speed maps to 3-second gust maps for 50-year mean recurrence interval winds. This change of map definition directly affects the drag coefficient $C_d$ used to compute effective velocity pressures. The recommendations also devoted an entire chapter to fatigue resistance and loading of signs, signals and luminaire support structures based on the work of Kaczinski (Fouad 1998)

Kaczinski (1998) NCHRP 412

NCHRP project number 412 was intended to develop guidelines for the fatigue design of cantilever sign, signal, and luminaire support structures. A survey of state DOT’s, revealed that the occurrence of problems was increasing, and among the 36 states which responded, approximately 50% had problems with wind-induced vibration of cantilevered support structures. The report suggests that truck and natural wind-induced
vibrations are responsible for the accumulation of fatigue damage in structures that have been in service for a number of years.

The report discusses aerodynamic studies and aero elastic studies performed at MIT in the Wright Brothers Memorial Wind Tunnel on full scale cantilever sign structures. Results of the aerodynamic studies suggest that cantilever structures with signal attachments are susceptible to galloping. Further, galloping become worse when back plates are added to the signal attachments. Further, it was concluded that vortex shedding does not need to be considered for wind velocities less than 5 mph. On the other hand, galloping was found to excite most types of sign and signal structures.

Static and dynamic FE analyses were carried out to estimate wind pressures on cantilever support structures during vortex shedding, galloping, natural wind and truck-induced vibrations. An equivalent 21 psf was recommended for the design of cantilevered sign support structures applied vertically. It was suggested that only cantilever support structures not having attachments and having lock-in velocities greater than 10 mph be considered susceptible to vortex shedding.

Final recommendations suggest that dynamic and static analyses should be conducted to determine and estimate equivalent static load ranges for the four load types (Kaczinski et.al 1998)

Cook (1998) Tests of Steel Mast Arms and light poles in Florida

The purpose of this study was to develop a damping device to mitigate vibrations in mast arms and light poles due to any type of wind-induced vibrations. Many devices were developed and tested in the laboratory on 37 ft mast arms. The effective devices
were then tested in the field on a variety of different length mast arms. The results of this study indicate that:

1-The dominant frequencies present in cantilevered mast arms are in the range of 0.7 Hz – 1.4 Hz.

2-Fatigue failures occurring in mast arms may be reduced with the implementation of the damping device.

3-The developed damping mechanism was tapered and consisted of a spring and mass that dissipates energy through impact and damps the arm quickly. This device yielded 3% critical damping on a range of mast arms (Cook 1998).

Gray (1999) Failure of Two Signal Structures in Wyoming

This research initiated as a result of the collapse of two signal structures in Wyoming. Fatigue cracking at the base of the fillet weld between the box connection and vertical pole resulted in the failures shown in Figure 2-5. The Wyoming Department of Transportation (WYDOT) inspected all of the poles in the inventory and found that one-third had fatigue cracks. Research focused on the determination of wind loadings and methods for evaluating the fatigue resistance of traffic signal structures. Tests were performed in both the field and the lab.
Field monitoring was performed to determine in-service loading conditions on the mast arm. In the lab, fatigue tests were conducted on signal structures removed from service. The focus of the testing was to develop and compare various methods for detecting fatigue damage. Two non destructive evaluations (NDE) included dye penetrant and ultrasonic testing. Results showed that dye penetrant testing was a suitable method for determining the crack lengths on the exterior surface of the pole. Ultrasonic testing showed highly variable results from inspection to inspection. Estimating the fatigue life of the connection was attempted, by applying Miners rule. Miners rule was used to find an equivalent Constant Amplitude Stress Range (CASR). This method worked well with data from field monitoring. The resulting CASR was greater then the CAFL of the detail provided by AASHTO. As a Category K₂ detail, the connection is susceptible to fatigue (Gray, 1999).

The dynamic characteristics of a cantilevered traffic structures were determined experimentally and theoretically. Experimental procedures involved forced vibration of a 50ft cantilevered structure by a variable speed mass oscillator. Theoretical characteristics were determined using computer modeling (Figure 2-6). Several devices were applied to the structure to improve the inherent damping.

![Figure 2-6. Cantilevered mast arm tested in WYDOT district yard Courtesy of U of WyomingDOT](image)

The response of the structure with the damping devices were compared to the response of the as-built structure. The following conclusions were drawn from the analyses:

1-Forced vibration by an eccentric mass is an effective method of determining the dynamic characteristics of cantilevered traffic signal structures.

2-Retrofit devices increased the overall damping of the structure (McManus, 2000).

The objective of this study was to evaluate a tuned mass impact damper under in-service conditions. A traffic signal structure was instrumented and the wind load effects with and without the dampers were evaluated. The cantilevered traffic signal structure was monitored for nominal stress readings at the box that connects the mast arm to the support pole. Also, wind speed and direction were monitored and recorded. Rain flow counting was used to determine stress range histograms and real-time stress/strains were used to evaluate the response for single wind events. Important conclusions were as follows:

1-Fatigue life expended per year based on a K2 classification was reduced from 3.26% to 0.96% for in-plane bending and from 6.54% to 2.74% for out-of-plane action using the tuned-mass damper.

2-The percentage of fatigue life based on an ET category was reduced from 0.35% to 0.11% for in-plane bending and from 0.662% to 0.25% for out-of-plane action by using the tuned-mass damper.

3-The number of cycles for an extrapolated 15-year service life was reduced by nearly 21 million for in-plane bending and 3.4 million for out-of-plane action by the use of the damper (Brisko 2002).

Chen (2003)         Failure of cantilevered Mast Arms in Missouri

Research was initiated after the Missouri Department of Transportation (MoDOT) had discovered and documented failures in several cantilever mast arms. The failures were primarily by fatigue at the weld of the arm to the base plate attached to the mast.
With over 6000 mast arms in service, the failures raised concerns with the existing mast arm inventory and future design. Two mast arms were instrumented in the field to determine the stress history and to develop applicable wind loads for Missouri. Five arms were fatigue tested, some with a proposed “fatigue-resistance” detail as shown in Figure 2-7.

![Fatigue test data conducted at U of Missouri-Columbia lab facility](image)

Figure 2-7. Fatigue test data conducted at U of Missouri-Columbia lab facility  
Courtesy of U of Missouri-Columbia

Results determined that many of the premature fatigue failures of mast arms in Missouri may be attributed to poor weld quality. Load cycles experienced by the mast arms are not necessarily critical if the weld is of high quality. Recommendation included levels of weld quality and the installation dampers on the mast arms (Chen, et.al 2003).


Research began as the rate of reported fatigue related problems increased and resulted in raised awareness of fatigue concerns in traffic signal mast arms. Prior research indicated that most commonly used connections details in these structures exhibit poor
fatigue performance. Further, a larger variety of connection details and a weld treatment method were examined in addition to the standard socket type connections. Fifty-five full-size mast arm connection detail specimens were fatigue tested as shown in Figure 2-8.

Research included the examination of base flexibility on fatigue performance. The results of two specimens with 2” thick endplates showed considerable improvements in fatigue resistance as compared to the 1.5” endplate thickness specimens. Table 2-6 shows fatigue test results for the 2” endplate thickness specimens compared to the average data for the 1.5” thick endplate welded socket connection. It was stated that the current AASHTO specification does not include the thickness of the base plate as one of the factors that determines the fatigue categorization of the detail. Further, the factors that lead to an improved fatigue life due to the thickness of the base plate are not fully understood. In addition, the present specifications were found to overestimate the fatigue
life of connections with stiffeners by a significant amount. Results indicate that the Ultrasonic Impact Treatment weld treatment can significantly improve the fatigue life of fillet-welded socket connection details (Koenigs et.al 2003).

Table 2-6 U of Texas fatigue test results (Courtsey of Mark)

<table>
<thead>
<tr>
<th>Plate Thickness (inch)</th>
<th>Specimen</th>
<th>Cycles to failure, N</th>
<th>Stress Range(ksi)</th>
<th>A*</th>
<th>A average</th>
<th>Detail Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>VALNu</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>VALNU 2 A</td>
<td>5,144,528</td>
<td>11.9</td>
<td>86.69</td>
<td></td>
<td>E'</td>
</tr>
<tr>
<td>2</td>
<td>VALNU 2 B</td>
<td>1,683,127</td>
<td>11.8</td>
<td>27.65</td>
<td></td>
<td>C</td>
</tr>
</tbody>
</table>

\[ A = \frac{N * S^3_r}{10^8} \]


This research consisted of a long term field monitoring of the new light pole installation on the Bronx-Whitestone Bridge, in New York City. Dynamic analyses were conducted on existing light poles to determine natural frequencies. Results showed that:

1-Maximum measured stresses in the prototype pole was 2.2 ksi.

2-Vibrations of the poles appear to be dominated by traffic induced excitation (Hodgson and Connor 2004).


Field testing and monitoring of selected cantilevered mast-arm signal structures in Philadelphia was conducted. In addition, dynamic tests were conducted in order to
determine the natural frequency, vibration characteristics, and the amount of damping in the structures.

Results showed that:

1-Wind speeds at both poles regularly exceed 20 mph.
2-Wind speeds at both rarely exceed 40 mph.
3-The maximum wind speed recorded at either pole during the nine month monitoring period was 54 mph.
4-Maximum wind speeds are at a direction perpendicular to the mast arm.
5-Stress ranges less than 1 ksi were measured at both poles. Effective stresses were less than the CAFL of 2.6 ksi (for category E’) (Connor et.al 2004).


After the failure of two cantilevered sign structures in Hazelton, lab tests were carried out at Lehigh University in the Advanced Technology of Large Structural System (ATLSS) laboratory. In addition, a painted cantilevered sign structure, previously removed from service due to excessive vibration, was also examined for any signs of cracking. The collapse was a result of cracking along the welded baseplate connection. It was concluded and recommended that:

1-Cracks were located at the upper weld toe connecting the baseplate to the pole.
2- The cracks covered 28% of the pole’s circumference.
3-Inspection may be aided by dye-penetrate or magnetic-particle tests.
4-Fatigue assessment should be conducted of structures of the same design (Connor and Mahmoud 2004).

In 1998, a 40-foot tall luminaire assembly support mast collapsed. This collapse initiated data collection and inspection of 474 of these luminaire support masts in the metro-Milwaukee area. Another in-service performance failure that Wisconsin DOT has experienced is the excessive vibration of a full-span sign support bridge. The structure was put in service in August 1996 and removed from service in February 1997. The purpose of the research conducted was to provide fatigue life estimates for these structures.

Results were as follows:

1-Dynamic analysis results showed a great impact of base plate thickness in the modal frequency of the structure.

2-Design of full-span sign support structures of the configuration examined need not consider galloping vibrations.

3-Truck-induced pressure pulses were found to cause very little damage. The stress ranges due to trucks passing were very small and were on the order of 180 psi.

4-Vortex shedding induced vibrations need not be considered in the design of full-span support structures.

5-The fatigue design loading of 5.2 psf for galloping design found in (AASHTO 2001) is satisfactory for the Milwaukee area.

6-There is only a 1% chance that the wind speed will exceed a 50-mph mean wind in Wisconsin (Foley 2004).
Two types of failures were examined in Illinois. The first occurred during a winter storm in February 2003, where approximately 140 tapered aluminum light poles in western Illinois collapsed as shown in Figure 2-9. The mechanism of collapse appears to have been due to fatigue failure in the connections or the pole wall.

The second was the repeated failure of light bulb filaments due to excessive vibration of light poles installed on the I-80 Bridge. The mechanism for this oscillation was evaluated (likely causes are traffic- or wind-induced vibration) and mechanisms for mitigation were proposed (University of Illinois at Urbana-Champaign official website).

A study conducted at Lehigh University investigated the effect of base flexibility on the behavior and fatigue performance of welded socket connections in cantilevered sign structures. The result of a three specimen static load testing program and extensive
FE analysis, calibration, and parametric study were discussed. The study found that base flexibility, primarily base plate thickness has a large influence on the fatigue behavior. Increasing baseplate thickness is shown to decrease stresses at weld toe where most cracks initiate (Hall III, 2005).

(Azzam) 2005  Failure of steel pole in Akron

A light pole failure was observed recently in the Akron area on June, 12, 2005 as shown in Figure 2-10. It was not clear what caused the failure but a closer look at the pictures shows that failure might be fatigue in the anchor bolts, which connects to the concrete base!!.
CHAPTER III
RESIDUAL STRESS MEASUREMENTS

3.1 Introduction

Fatigue strength of welded metal structures is primarily a function of the detail type and applied stress range (Fisher 1977, Keating and Fisher 1986). Detail classification schemes employed in many different specifications represent a ranking of the stress concentration condition of the joints (AASHTO, 2001). Details exhibiting failures that initiate from internal flaws or defects typically have the highest allowable fatigue strengths as there is no stress concentration worse than the discontinuity itself. Those welded connections with fatigue failures initiating from the toe have lower fatigue strength as cracks develop under locally elevated stress fields. The use of stress range as the primary parameter describing the fatigue strength of a welded structural detail has been justified by the existence of relatively large tensile residual stresses which develop as a result of the non-uniform heating and cooling associated with the joining process (Lehmus et.al 2005).

Residual stresses are usually defined as stress that remains in mechanical parts not subjected to any outside loading. Those stresses develop as a result of welding or fabrication processing. In welding, heat is introduced locally, and softens the metal that is located adjacent to the weld and causes thermal expansion. Cooler metal away from the weld attempts to restrain the expansion and introduces compressive stresses, that in some
instances, are large enough to cause plastic deformation. Upon cooling, the heated area will try to contract. Tensile stresses are introduced as the surrounding metal resists the contraction. Tensile stresses must reach a level close to yield strength of the heat affected zone before relief from plastic deformation is possible. Tensile residual stresses are balanced by compression in regions away from the weld (Kandil et.al 2001).

Previous fatigue tests conducted on shoe base socket connections showed fatigue strength in excess of category E. Further, the fatigue data appeared to fall along a trend line with a significantly shallower slope than current S-N curves, As such, residual stress measurements were determined to be critical toward the development of an understanding of the fatigue behavior of the connections.

3.1.1 Heat Process for Both Types of Details

Figures 3-1 and 3-2 depict the welded shoe base and through plate socket connection details examined in the current study.

Figure 3-1. Shoe base socket connection (Detail A)
Both are used to join a round extruded aluminum tube to a foundation through a break-away transformer base. Each connection type is fabricated by inserting an extruded pole into a shoe base casting or through an opening cut in the plate. Fillet weld(s) are placed around the top joint first, followed by a fillet weld between the tube and bottom of the casting or plate. In the majority of cases, the extruded tube is a heat treatable, readily extruded aluminum alloy. In this instance, the extruded tube is 6063, while the casting is aluminum alloy 356. The 6XXX series of alloys are most often encountered in structural work. 6XXX series alloys are considered crack-sensitive if welded autogenously or with insufficient filler metal conditions. Fillet welds were deposited using the Metal Inert Gas (MIG) process with 4043 as the filler material. Different gases are used to shield the arc for welding aluminum and may consist of pure argon or an argon-helium mix is used. Welds were completed with the tube and casting/plate in the T4 condition, and subsequently, the entire assembly was aged to a near T6 temper by heat treating to about 350°F for a set period of time. Alloys are often heat –treated by initially heating the material to approximately 1000°F, holding the
temperature for a short time, and then quenching in water, or suitable solution. This particular heat treatment for the tube assemblies consisted of holding the material at approximately 350ºF for a few hours. During this time, the alloying additions that were dissolved in the prior heat treatment precipitate in a controlled manner, which strengthens the alloy. Material in this condition is designated as a T6 temper (www.advantagefabricatedmetals.com (2005).

3.1.2 Residual Stress on Welded Details

Residual stresses are dependent upon the order in which pieces are joined as well as other process parameters. Figure 3-3 shows fatigue data from welded steel cover plate details (Fisher et.al 1993). In one instance, cover plates were attached to flanges prior to welding the section together. The other data set was obtained by testing beams that were assembled prior to welding the cover plates. Fatigue behavior is quite different, and the improved performance of those with cover plates attached to the flanges prior to completing the section was attributed to a more favorable distribution of residual stresses.

Figure 3-3. Fatigue data from welded steel cover plate detail
Figure 1-5 depicts fatigue data obtained during an earlier study of welded aluminum shoe base connections. The data provided are for those specimens that failed around the toe of the upper fillet weld. Also shown are the fatigue design S-N curves for details classified as Category D, E & E’ by the Aluminum Design Manual (ADM, 2005). The data appear to follow a trend that may be described as “flat” or of a slope that is shallower than the typical design S-N curves for socket type welded aluminum details, or class E in this case. Such behavior is indicative of details with low levels of residual stress. In fact, the European Convention for Constructional Steelworks (ECCS) recognized this during early development of recommendations for fatigue design of aluminum components and structures (Kosteas, 1981). For those details with favorable stress states, the S-N curves were rotated around a pivot point, resulting in a design line with a shallower slope.

3.1.3 Experimental Program

Residual stresses in both types of welded details were evaluated during this study by the hole drilling method (Measurements Group 1995). Data was reduced using both the power series and integral methods, and the resulting stress profiles compared. Measurement error is discussed and general data trends examined. Implications for the fatigue behavior of light poles using the cast shoe base and through plate connection details are discussed.

A total of five full-scale light poles samples were used to measure residual stresses adjacent to the upper weld toe using standard hole drilling techniques. Three of the samples were fabricated with the cast shoe base, designated for the purposes of this
study as A1 to A3, and the through plate connection details were designated as B1 and B2. Figure 3-4 depicts the 0/+45/-45 rosette strain gages that were placed around the perimeter of the tube. Such rosette geometry is more sensitive to hole placement eccentricity, and may produce additional measuring errors as compared to other gage types. However, this gage type was chosen because it was necessary to measure stresses as close as possible to the weld toe area.

Figure 3-4. Schematic of the rosette gages (062-UM-120) used in the light pole tests
A total of 21 gages were installed following the Measurement Group recommendations for strain gage installation using a room cure adhesive (Measurements Group INC, 1995). Four rosettes were placed along the perimeter of the tube shown in Figure 3-5. Use of several gages allowed for the averaging of results at each depth. Such an averaging scheme was used to wash out errors and to help ensure the validity of the results. One additional test was conducted on the bottom base of pole A2, adjacent to the casting to tube weld. A Micro-Measurements RS-200, as shown in Figure 3-6, milling machine used to drill holes in the appropriate location on each strain gage (Measurements Group, 1996).

Figure 3-5. Rosette strain gage installed close to weld (Detail A)
Figure 3-6. RS-200 milling machine installed on a shoe-base connection detail

Strains were recorded at depth increments of 0.01 inch using a Micro-Measurements 5000 data acquisition system (Measurements Group, 1995). At each depth increment, material is removed and the relieved strains are measured using the bonded gages. Drilling was stopped at a depth of 0.09 inch. As the strain gradient was found to be non-uniform, the data was reduced using both the integral and power series methods (Schajer, 1988a).

3.1.4 Analysis Techniques

Schajer first introduced the power series method as an approximate technique to analyze residual stresses for non-uniform stress fields (Schajer, 1988b). This technique provides a limited amount of resolution by assuming a linear variation residual stress with depth. FEM calculations were used to compute series of coefficients that were used for calibration. The coefficients resulted from strain relaxation simulation, as load increments
were applied over a small portion of a mesh that employed a fine gradation of elements near the points of hole drilling. Further, it was assumed that the residual stresses act in planes parallel to the specimen surface, and that the out of plane components are small and negligible.

The coefficients are used as the basis of a least squares method of analysis of the measured strains. The mathematical formulation is discussed by Schajer and a tables provide the expected calibration coefficients in each test as a function of hole geometry. (Schajer, 1988a).

In short, the model utilizes a best fit curve using least squares, which is considered the primary advantage in using the power series method when more than five or six hole depth or increments are used.

Bijak-Zochowski, Niku-Lari, Flaman and Manning developed the integral method for the evaluation of residual stresses (Bijak-Zochowski, 1978, Niku-Lari 1985 and Flaman, 1985). Not unlike the power series technique, the method was calibrated using finite element calculations. Residual stresses are evaluated within each depth increment, and the method is particularly suited when a rapid change in stresses is expected and few hole depth increments are used. However, it is the most sensitive of the techniques to measurement errors.

3.1.5 Errors

Errors are an issue in hole drilling measurements, as errors in strain are often magnified in the calculation of residual stress (Schajer, 1986). There are a number of possible error sources. Strain gage installation is critical as an inadequate bond may lead
to gage drift and poor soldering may lead to inaccurate strain readings. Human errors in milling the holes should be considered. Excess speed may lead to a build up of heat and failure of the milling tool. Further, excessive speed may lead to the introduction of an eccentricity of the hole from the center of strain gage rosette. Hole depth measurement requires accurate reading of a graduated vernier. Strain measurements may be missed if the operator becomes impatient during drilling. Errors in measurement of the hole diameter and depth can result in significant errors in the calculated stresses at those depths. Small misalignment of the milling cutter, say 0.001 inch, may lead to a 3% error in calculated stress (Redner and Perry, 1981). Further, errors in material constants, including uncertainties in the input of elastic constants of the specimen material will lead to errors in data reduction.

In a research study conducted by Daurelio and co-workers (Daurelio et al. 2001), it was concluded that a maximum of 4 or 5 hole increments be utilized if the integral method was employed. However a maximum limit of 20 steps for the power series technique was recommended. In their case, the test article was a stainless steel sheet, 0.08 inch thick as compared to 0.25 inch for the light poles. For the light poles, to test through the entire thickness would require a different technique, like layer removal. The hole depth increments were used to evaluate the sign and magnitude of the residual stresses adjacent to the weld using the hole drilling method.

3.1.6 Data Reduction Approach

Residual stresses were determined by the power series and integral methods for both the aluminum shoe base connection as well as the through plate details. Multiple
steps for the reduction of the residual strain data were followed throughout the course of
the testing program. Further, strain data was processed in three ways.

First, strain data for each gage location was reduced to residual stresses as a single
result. Second, strain data from individual poles was averaged at each depth increment
prior to the calculation of residual stresses. This represented an attempt to wash out any
errors encountered during the tests. Third, strain data was averaged across all poles
containing the cast shoe base details prior to development of residual stresses. Also,
strain data was averaged for all poles containing the through plate detail before stresses
were calculated. Both the magnitude and trend in residual stresses for each detail, using
both types of averaging, are reported on here.

All strain data was evaluated using H-drill software provided by Measurements
Group (Measurements Group, 1996). An initial check is made on the uniformity of the
strain data. If found to be “uniform”, the strains are reduced using a simplified uniform
stress, blind hole analysis. For data found to be non-uniform, either the power series or
integral method is employed. Residual strain data developed through use of the software
are given within an upper and lower bound range. The given range has a 90% probability
of containing the actual residual stresses. The 90% range is considered valid as long as
the residual stresses vary linearly with depth. Results were then converted using Mohr’s
circle to evaluate stresses normal to the weld axis. Engineering judgment was used to
make a final decision on the validity of results.

Figure 3-7 shows a portion of the light pole strain data in a normalized form as
Figure 3-7. Percent strain vs. normalized depth to check uniformity of strain data

The lines show the upper and lower limits of the expected variation with normalized hole depth of several strain quantities. The strain quantities are given as

\[
p = \frac{(\varepsilon_1 + \varepsilon_2)}{2} \quad (3-1)
\]

\[
q = \frac{(\varepsilon_3 - \varepsilon_1)}{2} \quad (3-2)
\]

\[
t = \frac{(\varepsilon_3 - 2\varepsilon_2 + \varepsilon_1)}{2} \quad (3-3)
\]

where

\[P, q, t\] = strain quantities
\[\varepsilon_1\] = strain response for gage number 1
\[\varepsilon_2\] = strain response for gage number 2
\[\varepsilon_3\] = strain response for gage number 3
Each strain quantity is calculated and expressed as a percentage of its value at a depth equal to 40% of the hole diameter. Individual data points for the “p” strain combination, and the numerically larger of the “q” or “t” quantities is plotted against the normalized depth (Measurements Group, 1996). If the data varies from the expected limits by more than +/- 3%, then the data is considered non-uniform. In the ideal case, both data points and the solid lines should coincide if the residual stresses are uniform. The plot shown in Figure 3-7 is typical in that the residual stresses for the light poles are non-uniform.

Strains were measured using CEA-13-062UM-120 gage rosettes. This particular configuration allowed placement of the gages on the light pole samples adjacent to the fillet weld toe. Output resulting from the reduction of the measured strains included two principal stresses and an orientation angle. The angle, \( \beta \), provides the orientation of the maximum principal stress. Once determined, the residual stress component acting along an axis normal to the weld toe was found, as it is expected to have the greatest influence on the development of a typical fatigue crack. Figure 3-8 illustrates a step by step procedure on how the orientation angle, minimum and maximum stresses were used to measure maximum residual stress at the tube surface. Note that fatigue cracks for this particular detail form along the weld toe and grow through the thickness of the pole and around the circumference.
A -42° angle was computed using H-drill software

### Step by step procedure on measurement of Residual Stresses using Mohr circle Principle

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Step 2</th>
<th>Step 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle measured from H-drill = -42°</td>
<td>Angle measured is negative indicating counterclockwise direction</td>
<td>Angle normal to weld Hence, 90-3=87 x 2 = 174</td>
</tr>
</tbody>
</table>

![Diagram showing strain directions E1, E2, E3 and Smin, Smax.]  
**Hint:** Angle is measured Clockwise from E1 to Smax

![Diagram showing maximum residual stresses measured Clockwise "from left to right".](image)

Figure 3-8. Measurement of residual stresses using mohr circle Principle

3.1.7 Experimental Results

Figure 3-9 shows the variation of residual stress with hole depth along an axis normal to the weld toe for shoe base detail A2. The curves shown were obtained by the power series and the integral methods. In both cases, strains were measured in four locations around the perimeter of the light pole and the points shown are the average values of residual stresses for each depth. In both instances, the near surface residual stresses are compressive and become more positive as the hole depth increases. The magnitude of the residual stress on the surface of the pole adjacent to the weld toe is on the order of -15 to -18 ksi, or about 50 to 60% of the 6063-T6 typical yield strength of 31 ksi.
Figure 3-9. Residual stress normal to weld vs. hole depth for A2 detail

Figure 3-10 shows the variation of residual stress with depth for the through plate detail designated as B1. Figure 3-11 is a similar plot for the through plate detail labeled B2. Again, the residual stresses are normal to the weld toe, and shown are average curves for both the power series and integral methods. As with the shoe base details, the surface residual stresses are compressive. As the hole depth increases, the magnitude of the residual stress decreases, or becomes more positive. While the trends are the same, the magnitude of the calculated residual stresses is larger for those obtained by the integral method. Both through plate details show surface residual stresses adjacent to the weld toe that are on the order of -3.78 to -10 ksi, or about 13% to 33% of the 6063-T6 typical yield strength.
Figure 3-10. Residual stress normal to weld for B1 detail

Figure 3-11. Residual Stress Normal to Weld for Detail B2
Residual stress data developed for the three light poles consistently show the surface adjacent to the top fillet weld to be in compression. As the hole depth increases, the magnitude of the compressive stress decreases, or becomes more positive. Calculations have been based on strain data averaged for each individual light pole. Any particular strain employed in the analysis represents an average of readings from four gages. For instance, the response for gage 1 at a particular hole depth represents the average of four gage elements at that depth. Taking the analysis one step further, a single set of average strains was obtained, one for each detail type as presented in Table 3-1.

Table 3-1 Average strain data obtained for both details

<table>
<thead>
<tr>
<th>Depth (inch)</th>
<th>E1 (ue)</th>
<th>E2 (ue)</th>
<th>E3 (ue)</th>
<th>Depth (mm)</th>
<th>E1 (ue)</th>
<th>E2 (ue)</th>
<th>E3 (ue)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.01</td>
<td>40</td>
<td>48</td>
<td>39</td>
<td>0.254</td>
<td>28</td>
<td>50</td>
<td>90</td>
</tr>
<tr>
<td>0.02</td>
<td>94</td>
<td>108</td>
<td>107</td>
<td>0.508</td>
<td>51</td>
<td>100</td>
<td>128</td>
</tr>
<tr>
<td>0.03</td>
<td>121</td>
<td>152</td>
<td>135</td>
<td>0.762</td>
<td>70</td>
<td>138.5</td>
<td>158</td>
</tr>
<tr>
<td>0.04</td>
<td>136</td>
<td>180</td>
<td>155</td>
<td>1.016</td>
<td>86</td>
<td>169</td>
<td>174</td>
</tr>
<tr>
<td>0.05</td>
<td>146</td>
<td>193</td>
<td>162</td>
<td>1.27</td>
<td>91</td>
<td>186.5</td>
<td>183</td>
</tr>
<tr>
<td>0.06</td>
<td>147</td>
<td>196</td>
<td>163</td>
<td>1.524</td>
<td>92</td>
<td>198.5</td>
<td>188</td>
</tr>
<tr>
<td>0.07</td>
<td>147</td>
<td>196</td>
<td>163</td>
<td>1.778</td>
<td>93</td>
<td>205.5</td>
<td>194</td>
</tr>
<tr>
<td>0.08</td>
<td>147</td>
<td>197</td>
<td>163</td>
<td>2.032</td>
<td>94</td>
<td>210</td>
<td>195</td>
</tr>
<tr>
<td>0.09</td>
<td>147</td>
<td>197</td>
<td>163</td>
<td>2.286</td>
<td>94</td>
<td>212.5</td>
<td>196</td>
</tr>
</tbody>
</table>

Figure 3-12 shows the variation in average strain versus hole depth for detail A. As shown, the strain increases until a depth of 0.04 inch is reached, after which the strain appears to level out or become constant.
Figure 3-12. Strain as a function of hole depth for detail A

Figure 3-13 shows the residual stresses obtained from using the average strains for both details with the power series method. Figure 3-14 presents results obtained by the integral method for the same data set. Both show the surface of the poles to be on the order of -12.5 to -18 ksi in compression. However, the residual stresses obtained using the power series method become more positive, while the data reduced by the integral technique eventually flattens out and appears to stay compressive. It is interesting to note that the compressive surface residual stresses are consistent with the fabrication sequence used to fabricate the light poles and the fatigue tests obtained to date. The reason behind this compressive stress is that the surface compression is introduced as the bottom or second weld begins to cool and shrink. As the bottom weld shrinks, it tries to pull the extruded tube down into the socket, but restraint is provided by the fillet weld around the top of the tube.
Figure 3-13. Residual stress normal to weld vs. hole depth by the power series method

Figure 3-14. Residual stress normal to weld vs. depth by the integral method

Figure 3-15 presents residual stresses resulting from a single test for a gage placed on the shoe base casting adjacent to the bottom weld of light pole A2. The maximum
principal stress is shown as a function of hole depth, resulting from the application of the power series technique. Results were interesting, and show large tensile residual stresses reaching nearly 31 ksi. This stress is close to the yield point of the parent material in the casting, and is consistent with the idea that the surface residual stresses develop primarily as a result of the shrinkage of the bottom fillet weld.

![Graph showing maximum principal residual stress vs. hole depth for shoe-base detail](image)

Figure 3-15. Maximum principal residual stress vs. hole depth for shoe-base detail

3.1.8 Results and Recommendations

Some interesting points were revealed from the residual stress measurements for both details. Surface residual stresses of -15 ksi are present in the shoe base connection detail (Detail A), which might explain the shallower fatigue data trend for the shoe base connection details. The through plate detail (detail B) however, showed tensile residual
stresses of up to 8 ksi beyond a hole depth of 0.025 inches. Power series method results were deemed to be more reliable than those obtained by the integral method results, primarily due to its sensitivity for human and instrumentation errors. It can be seen that high tensile residual stresses are present in the bottom casting of shoe base connection detail adjacent to the fillet weld; the stresses reached nearly 31 ksi which is close to the yield point of the cast material. Residual stresses obtained for the details are consistent with the fabrication sequence difference in residual stress results as we experienced throughout the test stages.

Finally, it’s apparent that the tensile residual stresses results for the through plate detail will play role in the fatigue resistance of this type of detail, because the presence of such stresses will have a negative influence on the crack growth behavior, and hence, shorten the fatigue life for the through plate detail. On the other hand, the presence of compressive stress on surface of shoe base detail gave the one possible reason why the shallower trend observed in earlier studies on this type of detail.
CHAPTER IV

FATIGUE

4.1 Introduction

A total of 29 fatigue tests on welded aluminum light poles were conducted, including 19 shoe base socket connection details and 10 through plate socket connection joints. Parametric studies of the two details using the finite element method provided insight into the nature of the critical areas, mainly top of the weld or toe, as well as the influence of changes in joint geometry on the local stresses. Other details, including triangular and collar stiffeners were examined as well. Figure 4-1 and 4-2 depicts the designs for both the shoe base socket connection and through plate socket connection details used in the experimental program. Test samples, fabricated by HAPCO, a well known firm specializing in the manufacture of highway light poles, used a processing path identical to that employed during product manufacture. Light pole specimens were 5 inch radius extruded tubes with 0.25 inch thick walls, fabricated from aluminum alloy 6063 in the T4 temper.
Figure 4-1. Through plate socket connection dimensions for specimens used in fatigue and finite element studies.

Figure 4-2. Shoe base socket connection dimensions for specimens used in fatigue and finite element studies.
4.2 Fatigue

4.2.1 Experimental Program

Each light pole sample was tested in a horizontal position as depicted in Figure 4-3. Specimens were mounted to a heavy steel framework using high strength, 25.4mm (1 in.) diameter bolts. Constant amplitude displacements were applied at the far end of the poles through a mechanical test machine capable of applying up to 63.5mm (2.5 in.) at a frequency of 1 Hz (1 Cycle/Sec) as shown in Figure 4-4.

![Fatigue test set up at the University of Akron Lab Facility](image1)

**Figure 4-3. Fatigue test set up at the University of Akron Lab Facility**

![Fatigue Test machine and data acquisition used in fatigue tests](image2)

**Figure 4-4. Fatigue Test machine and data acquisition used in fatigue tests**
Strains were monitored throughout the duration of each test using single element gages and a Micro Measurements System 6000 for reading and recording the strain data as shown in Figure 4-4 (Measurements Group 1995).

To monitor maximum stress ranges \( \sigma_{\text{max}} - \sigma_{\text{min}} \) during the test period, gages were mounted some distance away from the weld toe in order to isolate strain readings from the stress concentration associated with the details as shown in Figure 4-5. Crack initiation was detected by a simple microscopic instrument to validate crack location sights. Failure, was determined when the crack depth reached 6.35mm (0.25 in.) or through the pole thickness. Fatigue data for that specific test was then reported.

![Figure 4-5. Strain gage used in fatigue tests](image)

4.2.2 Loading Frequency

Luminaire structures can be found to be in excess of 150 ft throughout the United States. During elevated wind speeds and gusts, such structures may vibrate and sometimes approach the first natural frequency and cause “resonance” which could lead to a catastrophic failure. In an effort to prevent excessive vibrations, dampers were
introduced and are installed for such types of structure. A majority of current damping devices used in signal sign and luminaire support structures are to prevent second mode vibrations. However, for long mast arms or tall structures wind gusts may cause the structure to exhibit the first mode vibration which will lead to a rapid catastrophic failure. Although dynamic analysis is beyond the scope of our study. An effort to simulate natural frequency of the structure was made. Finite element models (FEM) were built and analyses conducted to measure first natural frequency of high light pole structures. A first mode natural frequency for highmast luminaire support structures is expected to be close to 1 Hz in the field.

ANSYS was used to determine the natural frequency of a highmast arm structure. Pipe elements were used with a 0.12 inch/ft taper from the bottom to the top of the structure. First mode natural frequency was estimated to be 0.962Hz for a 50ft tall aluminum pole as shown in Figure 4-6 and 4-7. Therefore, a 1 Hz test frequency was used for all tests conducted. As such, the loading frequency selected is expected to simulate real life applications.
Figure 4-6. Dynamic model to measure natural frequency of a structure

Figure 4-7. 3-D Dynamic model of a pipe element structure
4.2.3 Shoe Base Fatigue Results

A total of 19 specimens were tested for the shoe base socket connection detail. Specimens were tested at various displacements to cover a wide range of stress ranges as shown in Table 4-1.

Table 4-1. Shoe base displacement and stress range summary

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Displacement (inch)</th>
<th>Stress range (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A24</td>
<td>0.875</td>
<td>5.8</td>
</tr>
<tr>
<td>A23</td>
<td>0.875</td>
<td>4.8</td>
</tr>
<tr>
<td>A22</td>
<td>0.875</td>
<td>4.1</td>
</tr>
<tr>
<td>A21</td>
<td>0.875</td>
<td>5.3</td>
</tr>
<tr>
<td>A20</td>
<td>1.75</td>
<td>8.6</td>
</tr>
<tr>
<td>A19</td>
<td>1.75</td>
<td>8.1</td>
</tr>
<tr>
<td>A18</td>
<td>1.75</td>
<td>7.3</td>
</tr>
<tr>
<td>A17</td>
<td>1.75</td>
<td>8.1</td>
</tr>
<tr>
<td>A16</td>
<td>1.25</td>
<td>6.5</td>
</tr>
<tr>
<td>A15</td>
<td>1.25</td>
<td>5.6</td>
</tr>
<tr>
<td>A14</td>
<td>1.25</td>
<td>6.1</td>
</tr>
<tr>
<td>A13</td>
<td>1.25</td>
<td>5.4</td>
</tr>
<tr>
<td>A12</td>
<td>1.0</td>
<td>4.7</td>
</tr>
<tr>
<td>A11</td>
<td>1.0</td>
<td>5.1</td>
</tr>
<tr>
<td>A10</td>
<td>0.75</td>
<td>3.5</td>
</tr>
<tr>
<td>A9</td>
<td>0.75</td>
<td>4.0</td>
</tr>
<tr>
<td>A8</td>
<td>1.5</td>
<td>7.3</td>
</tr>
<tr>
<td>A7</td>
<td>1.5</td>
<td>6.6</td>
</tr>
<tr>
<td>A6</td>
<td>0.875</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Figure 4-8 plots the fatigue test data associated with the shoe base detail. Also included are several design S-N curves taken from the Aluminum Design Manual (ADM, 2000). It is believed that the presence of compressive residual stresses at the surface of shoe base detail, is the primary reason behind shifting the data above Category E’ as defined by AASHTO, 2001 especially at lower stress ranges. It is interesting to note that a majority of the data points fall above the Category D curve.
All data points plotted had failures that developed from the weld toe area as shown in Figure 4-9. In almost 90% of cases, cracks initiate at the farthest distance from the neutral axis and propagated through the thickness and along the weld toe area. In two samples, fatigue cracking developed in the root area of the fillet and progressed through the weld throat, likely due to inadequate penetration.

Figure 4-8. University of Akron Fatigue test results for shoe base socket connection detail
Figure 4-9. Cracks at the upper weld toe of a shoe base detail

Figure 4-10 presents the proposed lower bound design curve for the shoe base socket connection detail compared to category D, E and E’ from both the ADM and AASHTO specifications (ADM, 2005 and AASHTO, 2001). A lower bound curve is shown, located two standard deviations below the best fit line and provides an approximate 97.5% probability of survival (Menzemer and Fisher, 1993). The lower bound curve was developed in a manner consistent with current fatigue design philosophies, which represents a cut-off point or Constant Amplitude Fatigue Limit (CAFL), details with stresses below the CAFL are expected to have an infinite life.
Figure 4-10. Lower bound design curve proposed for shoe base socket connection detail

The lower bound S-N curves possess a shallow slope as compared to current aluminum fatigue design provisions. Based solely on geometry of the shoe base socket connection detail, a Category E classification would seem appropriate. A mathematical expression which represents our lower bound curve is given as follows:

\[ S = 29.698N^{-0.1368} \]  

(4-1)

Where

\[ N = \text{Fatigue life} \quad \text{and} \quad S = \text{Stress range} \]
However, it is clear from the data that the high cycle fatigue resistance is significantly greater than what is provided by a Category E classification. Luminaire supports are typically designed for long or infinite fatigue life. Based on the above analysis and results, one may estimate the constant amplitude fatigue limit for this type of detail to be on the order of 3.0 to 3.5 ksi.

4.2.4 Through Plate Socket Fatigue Test Results

All tests were carried out using the same procedure as described earlier for shoe base detail. A total of ten specimens were fatigue tested with displacements ranging from 0.25 to 0.875 inches as shown in Table 4-3. Figure 4-11 plots fatigue data associated with the through plate socket connection base detail, and include both a best fit and a lower bound resistance curve. The lower bound is significantly below what would be provided by either a Category E or E’ classification. All of the through plate socket connection detail fatigue data shows a resistance below Category E’.

Table 4-2. Through plate displacement and stress range summary

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Displacement (inch)</th>
<th>Stress range (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B12</td>
<td>0.875</td>
<td>2.9</td>
</tr>
<tr>
<td>B11</td>
<td>0.875</td>
<td>1.4</td>
</tr>
<tr>
<td>B10</td>
<td>0.875</td>
<td>2.0</td>
</tr>
<tr>
<td>B9</td>
<td>0.875</td>
<td>1.0</td>
</tr>
<tr>
<td>B8</td>
<td>0.25</td>
<td>0.9</td>
</tr>
<tr>
<td>B7</td>
<td>0.25</td>
<td>0.9</td>
</tr>
<tr>
<td>B6</td>
<td>0.875</td>
<td>4.5</td>
</tr>
<tr>
<td>B5</td>
<td>0.875</td>
<td>3.5</td>
</tr>
<tr>
<td>B4</td>
<td>0.5</td>
<td>2.2</td>
</tr>
<tr>
<td>B3</td>
<td>0.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>
Figure 4-11. University of Akron Fatigue test results for through plate socket connection detail

Figure 4-12 presents the lower bound design curve predicted for the through plate detail. The same statistical technique used for shoe base detail was applied to approximate the lower bound curve for through plate detail analysis. The lower bound curve equation was estimated to be equal to

\[ S = 6.2N^{-0.1262} \]  

(4-2)

Where, \( N \) is the number of cycles and \( S \) is the stress range
Figure 4-12. Lower bound design curve for through plate detail

It is interesting to note that fatigue cracking developed along the weld toe in all of the through plate socket connection samples as shown in Figure 4-13. However, 80% of the time, cracks formed along the toe opposite the bolt(s). This has been termed the butterfly trend, where stresses are highest opposite to the bolts on the tension side of the pole and not in the location of maximum bending stress as would be calculated using a simple mechanics approach (Hall III, 2005). A plot of the longitudinal stress in the pole adjacent to the weld toe is shown in Figure 4-14. As may be seen, the shape is similar to that of a butterfly. As a large percentage of cracks initiated along the weld toe opposite to the bolts on the tension side of the pole, six strain gages were placed on two of the samples as depicted in Figure 4-15. Such behavior results from the use of a flexible base that induces additional bending through the tube wall.
Figure 4-13. Crack propagation at weld toe and inside of tube for two through plate socket connection detail

Figure 4-14. Butterfly trend observed on longitudinal stress in the pole at top of weld (Toe)

Figure 4-16 shows a strain trace for each of the strain gages obtained during cyclic loading for a through plate socket connection. Note that gage 3 is located opposite
to one of the bolts on the tension side of the pole with a stress range measured to be equal to 4.5 ksi, while gage 1 is located at the greatest distance from the neutral axis with a stress range of 3.1 ksi. Strains from gage 3 are nearly 30% greater than those experienced by gage 1.

Figure 4-15. Gage Locations used to measure butterfly trend for through plate socket connection detail

Figure 4-16. Strain gage response for a through plate socket connection detail
5.1.1 Finite Element (FEM) Study-Introduction

Finite element models of both types of samples were constructed and analyzed in order to gain insight into the behavior observed during testing. Parametric studies of both details were conducted to increase understanding of the important geometric features and their influence on local stresses that drive the fatigue response. ANSYS was used to complete all of the analyses (ANSYS, 2004). Through plate socket connection models examined the variation of plate thickness, hole size, number of holes and hole configuration. Also several models were constructed with unequal fillet weld legs and still others examined the use of several types of stiffeners. For the cast shoe base detail, the base thickness on the bottom of the casting was varied and the stress near the hot spot area examined.

A majority of models utilized solid elements with 20 nodes. SOLID95 is an element that is relatively tolerant of irregular shapes. Given the geometry of the through plate and cast shoe base details, some irregularly shaped elements were anticipated. Each of the 20 nodes of the solid element possesses three translational degrees of freedom. Models of the shoe base detail utilized 40,000 to 50,000 elements with 82,000 to 90,000
nodes. In the case of the through plate detail, the base plate thickness, tube diameter and wall thickness were varied as shown in Table 5-1. It is important to note that the r/t (tube radius/wall thickness) ratios were taken to cover the range of values typical of commercial products.

<table>
<thead>
<tr>
<th>Plate thickness (inch)</th>
<th>Tube thickness (t) (inch)</th>
<th>5 inch tube radius (r)</th>
<th>4 inch tube radius (r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.188</td>
<td>27</td>
<td>0.188</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>20</td>
<td>0.25</td>
</tr>
<tr>
<td>3</td>
<td>0.3125</td>
<td>16</td>
<td>0.3125</td>
</tr>
<tr>
<td>4</td>
<td>0.375</td>
<td>13</td>
<td>0.375</td>
</tr>
</tbody>
</table>

Analytical models of the through plate details employed 33,000 to 41,000 elements with 62,000 to 74,000 nodes. All models were constrained around the top and bottom perimeter of the bolt holes in the x, y and z directions in an attempt to match field conditions. Models utilized a 10 ft long tube with a concentrated load applied at the tip. In a number of instances, geometry was created using Pro-Engineer and was subsequently transported to ANSYS for mesh development, solution and post-processing (Pro-E, Wild Fire 2004).

Both, shoe base and through plate socket connections were modeled with a 0.2 in. element size at the weld toe area which represents 0.17% of total model length. A sensitivity study was conducted prior to analysis to determine a suitable element size at critical areas, hence, weld toe area as shown in Figure 5-1. Recent research has indicated that elements with 0.25 inch lengths along the weld toe area, representing 0.3% of the model height, adequately captured the behavior of the connection details (Hall III, 2005).
5.1.2 Shoe Base Socket Connection

Figures 5-2 depict the standard geometry for the shoe base socket connection detail. Maximum normal stress is plotted as a function of the thickness of the bottom portion of the cast base as shown in Equation (5-1).

\[
\sigma_{\text{nom}} = \frac{Mc}{I}
\]  

where

- \( M \) is the moment at weld toe,
- \( c \) is the distance to top fiber,

and \( I \) is the moment of inertia of the structure.
Figure 5-2. Finite element model and simulation for shoe base detail

Figure 5-3 presents the results of the parametric study of the cast shoe base thickness. There was virtually no difference in the maximum stress for any of the models. Apparently, the original design has sufficient stiffness such that further increases in base thickness had little influence on the maximum normal stress in the tube. As a result weld leg studies were terminated to allow more studies on the through plate socket connection
detail and to better understand the poor fatigue behavior of such type of detail.

![Graph of σ_max/σ_nom vs Plate thickness (inch)](image)

Figure 5-3. Shoe base plate thickness parametric Study

5.1.3 Through Plate Socket Connection

In Figure 5-4, the mesh and normal stress contours are shown for the 1 inch thick base plate model of the through plate socket connection detail. The butterfly trend examined earlier in fatigue experimental studies is clearly shown with a 1 inch plate thickness and is presented in Figure 5-4. In order to better understand the local stresses and resulting butterfly pattern, parametric studies with various plate thicknesses and different tube thicknesses were studied as detailed in Table 5-2. Figure 5-5 shows the same specimen but with a thicker base plate.
Stresses were normalized to the nominal stress of the structure using Equation 5-1. A step by step example procedure was followed, and is consistent across all parametric study results. Normalized stress along the weld toe is plotted as a function of base plate thickness for different r/t ratios. Figure 5-6 presents the results of the through plate parametric study as the base plate thickness increases for a 5 inch tube radius. Results showed that increasing the plate thickness from 1 to 2.5 inches will decrease the normalized stresses by 25% as shown in Figure 5-6. Figure 5-7 shows a similar plot for a 4 inch radius tube of varying wall thicknesses. Trends are similar, with a reduction in normalized stress accompanying an increase in base plate stiffness. Again maximum reduction occurs as the thickness of the base is increased from 1 to 2.5 inches. No specific trend with the r/t ratio was identifiable.
For illustration purposes a sample calculation, for the 5 inch tube radius and 0.25 inch tube thickness case, is given below. Figure 20 and 21 are plotted with the normalized stress as shown below.
5.1.4 Design Example

Step 1- Dimensions

Distance to top fiber $c = 5.0\,\text{inch}$, tube thickness, $t = 0.25\,\text{inch}$, distance to center of wall thickness ($r$) was calculated using Equation (5-2) below

$$r = c - \frac{1}{2}(t)$$

$\therefore r = 5 - \frac{1}{2}(0.25) = 4.875\,\text{inch}$

Length from weld toe to at tip of model, $L = 118.625\,\text{inch}$, Unit load at tip of structure = 1 kip, Moment ($M$) = 1*118.625 = 118.625 k-in

Step 2-Moment of Inertia

Calculate Moment of Inertia ($I$) using Equation (5-3);

$$I = \pi r^3 t$$

$$I = 3.14 \times 4.875^3 \times 0.25 = 90.95\,\text{in}^4$$

Step 3: Nominal stress

Calculate Nominal stress $\sigma_{\text{nom.}} = \frac{Mc}{I}$

$$\sigma_{\text{nom.}} = \frac{118.625 \times 5}{90.95} = 6.52\,\text{ksi}$$

Step 4: From FE model, stress at weld toe

Step 5: Normalized stress $= \frac{\sigma_{\text{max.}}}{\sigma_{\text{nom.}}} = \frac{17.4}{6.52} = 2.67$ as shown in Figure 4-20
Figure 5-6. Parametric Study results for through plate socket connection detail with 5 inch radius tube

Figure 5-7. Parametric Study results for through plate socket connection detail with 4 inch radius tube
5.2 Weld Leg Finite Element Study

In 1981 Fisher et.al conducted fatigue tests on welded steel through plate socket connection details. As part of the investigation, several tests were conducted on assemblies utilizing unequal leg fillet welds. One of the conclusions from the study was that improved fatigue resistance was realized when unequal leg fillet welds were used and specifically, when the long leg was placed on the tube.

As such, several models were constructed with unequal and equal fillet weld legs with a 3 inch base plate thickness to negate the flexibility effect revealed from the previous parametric study. Table 5-2 describes the parameters included in the weld leg study. For the cast shoe base socket connection detail, the base thickness on the bottom of the casting was varied and the stress near the weld toe area examined. It should be noted here that after a review of the first set of FE results for the shoe base socket connection detail, subsequent work on the parametric study for the shoe base socket connection detail was stopped and only through plate socket connection was examined for weld leg properties.

<table>
<thead>
<tr>
<th>Weld leg design parameters</th>
<th>Vertical Length (inch)</th>
<th>Horizontal Length (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case study I(Unequal weld leg)</td>
<td>0.250</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>0.312</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>0.375</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>0.437</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>0.563</td>
<td>0.250</td>
</tr>
<tr>
<td>Case study II(Equal weld leg)</td>
<td>0.125</td>
<td>0.125</td>
</tr>
<tr>
<td></td>
<td>0.200</td>
<td>0.200</td>
</tr>
<tr>
<td></td>
<td>0.250</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>0.300</td>
<td>0.300</td>
</tr>
</tbody>
</table>
Results showed that the stress concentration around the weld toe is higher with equal weld legs as compared to unequal weld legs. Figure 5-8, shows that as vertical weld leg increases, normal stresses decreases for both 4 inch and 5 inch tube radius. The same trend is observed for equal weld legs as shown in Figure 5-9. Normalized stress decreases as equal weld leg length increases. However, stresses are still higher when compared to unequal weld leg.

Figure 5-8. Parametric study results for unequal weld leg study
5.3 Hole Studies

5.3.1 Round Hole Size Parametric Study

The butterfly trend, resulting from base flexibility, led to the study the influence of hole size and number of holes on local weld toe stresses. The first thought is that increasing the hole size or increasing the number of holes will result in more constraint to the base, and will lead to a more “fixed” base structure reducing stresses adjacent to the fillet weld toe.

For the hole size study, the hole diameter was increased from 1 inch to 2.5 inches. In a second case, the number of holes was increased from 4 to 8. Results from both cases will be compared with the original structure.

Figure 5-9. Parametric study results for equal weld leg study
Figure 5-10 shows the resulting stress distribution when the hole diameter was increased from 1 inch to 2.5 inches. A decrease in local stress was observed and is critical, as 80% samples of our test samples had cracks initiate in this particular area. Results showed that increasing the hole size from 1 inch to 2.5 inches will cause a 25% reduction in maximum longitudinal stresses opposite to the bolt area.

5.3.2 Number of Holes Study-Round Holes

A total of 4 holes were added to the base plate, resulting in a total of 8 holes. Figure 5-11 shows the stress distribution for 8 hole bolt pattern. The butterfly trend vanishes due to the increased restraint. The maximum stresses are concentrated and located in the expected region, at the furthest distance from neutral axis.
5.3.3 Slot Hole Parametric Study

A new type of detail was included in this research study in an effort to examine the behavior of various through plate socket connection details currently used in the light pole industry.

A 22’ through plate socket connection detail with bolt slots was modeled as shown in Figure 5-12. A parametric study was conducted to on 1, 1.75, 2, 3, 4 and 5 inch base plates to determine the stresses on top of weld toe as shown in Figure 5-13.
The butterfly trend is clearly shown for this detail in Figure 5-14. Higher stresses are present opposite to the bolt constraint area with a thin base plate. Results of the base plate parametric study as shown in Figure 5-15 revealed a 28% decrease in stress when base plate thickness is increased from 1 to 2 inch and correlates with the circular hole study of the through plate socket connection detail.
5.4 Collar Socket Connection Detail

A new detail has started to enter the light pole market, and is known as the collar socket connection detail. It consists of a through plate socket connection welded from top and bottom. In addition, a 5 inch collar tube is used to stiffen the base and pole as shown in Figure 5-16. This specific detail is currently not included in the AASHTO, 2001
specifications, but is expected to be included as an alternative structure in future specification releases.

A parametric study was conducted to examine the level of stress around the top of the collar (both with slot holes) and compare with the previous socket connection models with slotted holes (Figure 5-16). Results shown in Figure 5-17, show that the as plate thickness increases from 1 to 3 inches, the stresses on bottom area decreased by 21% as compared to 28% on the same area without the 4” collar tube. The effect of base plate thickness is negligible at the top of the collar with no significant change in stresses.
Therefore, the role of the collar will be to shift the highest stress concentration from the bottom of collar to the top portion for thicker bases.

![Stress vs Base Plate Thickness Graph]

Figure 5-17 Parametric study results and comparison between the 4 inch collar tube and the original socket connection with slot holes

5.5 Gusset Stiffener Detail

Pole to base plate stiffeners are classified as longitudinal attachments for design purposes within the specification. Stiffeners are mainly used to alleviate the elements they are attached to. In the case of stiffeners utilized with cantilevered pole type structures, gussets assist in stiffening and stabilizing the tube wall. In these applications, the tube to base plate would consist of a socket connection with stiffeners attached to both elements. The main objective was to investigate the stresses on the tip of gusset stiffener and investigate the stress Category adopted in current AASHTO specifications.
Stiffeners will be referred to as short, medium and long. Table 5-3 describe the gusset stiffener details and the stress category at tip of the stiffener by the current AASHTO specifications with long stiffeners described as the worst with a Category E.

<table>
<thead>
<tr>
<th>Construction</th>
<th>Detail</th>
<th>Stress Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Attachments</td>
<td>Non-load bearing longitudinal attachments with partial-or full-penetration groove welds, or fillet welds, in which the main member is subjected to longitudinal loading</td>
<td></td>
</tr>
<tr>
<td>SHORT</td>
<td>L &lt; 2 inch</td>
<td>C</td>
</tr>
<tr>
<td>MEDIUM</td>
<td>2 inch &lt;L&lt;12t and 4inch</td>
<td>D</td>
</tr>
<tr>
<td>LONG</td>
<td>L&gt; 12t or 4 inch when t&lt;1 inch</td>
<td>E</td>
</tr>
</tbody>
</table>

5.5.1 Recent Gusset Stiffener Fatigue Test Results

Recent research studies by Macchietto, 2001 at Valmont industries has revealed a better fatigue Category for socket connections without stiffeners as compared to gusset stiffener details as shown in Figure 5-17. The results in Figure 5-17 show a higher fatigue resistance for socket connection detail compared to gusset stiffener details. Current AASHTO, 2001 specifications describe the socket connection detail as Category E’ while medium and long stiffeners as Category D and E respectively.
Another extensive study was carried out by Koenigs (Koenigs et.al, 2004) at The University of Texas at Austin which included 25 full scale fatigue tests for medium and long gusset stiffeners. Current AASHTO specifications as shown in Table 5-3 specify medium stiffeners as Category D details and long stiffeners as E details. Results shown in Figure 5-19 reveal that at the same stress ranges, at least 3 long stiffener specimens are close to or above Category D detail.
5.6 Gusset Stiffener FEM Study

In order to better understand the performance of the gusset stiffener socket connection detail, parametric studies were conducted on base plate and tube thickness for short, medium and long gusset stiffeners. Parameters included in the study are detailed as shown in Table 5-4.

Figure 5-19 Fatigue tests results conducted at The University of Texas at Austin on medium and long gusset stiffener socket connection detail
Table 5-4 FEA design parameters for various stiffener lengths

<table>
<thead>
<tr>
<th>Stiffener length (inch)</th>
<th>Base plate thickness (inch)</th>
<th>Tube thickness (inch)</th>
<th>Total FEA Runs</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1,2,2.5,3&amp;4</td>
<td>0.188,0.25,0.3125 and 0.375</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td>20</td>
</tr>
</tbody>
</table>

Analyses were conducted using ANSYS and specifically, the parametric design language option (Swanson Analysis Systems, 2004). SHELL63 was employed for all models for simplicity, reduced running time and cost reduction. Figure 5-20 shows models for a short (2 inch), medium (3 inch) and long (16 inch) long stiffener details.

Figure 5-20. Gusset stiffener details for the 2, 3 and 16 inch long stiffeners
5.6.1 FEM Results for Short Stiffeners

Figure 5-21 presents results for a 0.188 inch tube thickness for a 2 inch (short stiffener) with a 1 and 3 inch base plate thickness. Results show higher stresses on the order of 27 ksi at the tip of the stiffener for the 1 inch plate thickness, as compared to 17 ksi for the 3 inch base plate thickness. A reduction of 37% in stress with increasing base plate thickness is revealed. However, higher stress concentrations still exist at the tip of the stiffener with a 3 inch plate thickness.

![1 inch Plate thickness](image1.png)  ![3 inch Plate thickness](image2.png)

Figure 5-21 Short stiffener FEM contour stresses for a 1 and 3 inch base plate thickness.

5.6.2 FEM Results for Medium Stiffeners

Finite element results for a 3 inch (medium stiffener) are shown in Figure 5-22, a local stress reduction of 35% is realized by increasing the base plate from 1 inch to 3 inches.
5.6.3 Parametric Study Summary

Stress concentrations for 1 inch and 3 inch base plate thicknesses are presented in Figures 5-23 and 5-24 below. Results showed that the role of base plate thickness for stresses at the tip of the stiffener. A reduction in stress is realized as the tube thickness increases from 0.188 to 0.375 inches. Another interesting behavior in Figure 5-23 is that stresses becomes constant after the stiffener reaches a 4 inch vertical lengths, and shows that stresses are highest for short stiffeners that are currently categorized as a Category C detail according to AASHTO,2001. Fatigue tests are essential to validate such behavior.

Figure 5-24 shows a constant and negligible reduction in stress concentration with a 3 inch base plate thickness. Again, same trend with tube thickness is apparent.
Figure 5-23. Gusset stiffener parametric study results for a 1 inch plate thickness at tip of stiffener for short, medium and long stiffeners with various tube thicknesses.

Figure 5-24. Gusset stiffener parametric study results for a 3 inch plate thickness at tip of stiffener for short, medium and long stiffeners with various tube thicknesses.
5.7 Dynamic Analysis Results

ANSYS was used to conduct a study of the first mode natural frequencies of 8 and 10 inch diameter tubes with tube thicknesses of 0.188, 0.25, 0.3125 and 0.375 inches. Pole heights studied herein ranged from 10 to 50 ft, to represent real life applications.

Pipe elements were used to model the structure as described earlier. PIPE16 is a uniaxial element with tension-compression, torsion, and bending capabilities. The element has six degrees of freedom at two nodes: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. This element is based on the 3-D beam element, and includes simplifications due to its symmetry and standard pipe geometry. The primary objective was to determine the first mode natural frequency of the poles as the tube thickness and diameter increases; hence, a stiff structure is expected, with larger natural frequencies.

First mode circular frequency of a structure $\omega_n$, depends on stiffness, $k$ and mass, $m$ frequencies as shown in Equation 5-4.

$$\omega_n = \sqrt{\frac{k}{m}} \quad (5-4)$$

Therefore, the behavior is dominated by either mass or stiffness.

Figures 5-25 represents results for the 8 inch diameter tube. As the tube thickness increases, the first mode natural frequency increases for the same height. It’s clearly shown that the structure is stiff at a height of 10ft and becomes more flexible as the height increases as shown in Figure 5-26. The natural frequency approaches to 2 Hz (cycles/sec) at a height of 30ft. As the structural dimension increases, the mass increases at a lower rate than the accompanying increase in stiffness.
A comparison between the 8 and 10 inch tubes shows a higher natural frequency for the 10 inch diameter tube compared to 8 inch at the same heights. Figure 5-27 represents a close up view for the 8 and 10 inch diameters with a 0.25 inch tube walls. A 30% difference in natural frequency for heights 30, 40 and 50 ft poles is observed.

As a summary, results revealed that as the tube thickness increases, the natural frequency of the structure increases. Higher natural frequencies are exist for 10 inch diameter tube poles as compared to 8 inch diameter tubes, primarily because the section modulus is larger for the 10 inch diameter. First mode natural frequency for a 10 inch diameter tube is 30% higher as compared to 8 inch pole.

Figure 5-25 First modal natural frequency for an 8 inch diameter tube with varied tube thicknesses at heights between 10 and 50 ft
Figure 5-26 First modal natural frequency for an 10 inch diameter tube with varied tube thicknesses at heights between 10 and 50 ft

Figure 5-27 First modal natural frequency for an 10 inch diameter tube with varied tube thicknesses at heights between 10 and 50 ft
5.8 Gust Effect Factor

Gust effect factor corrects the effective wind speed, $V$, for the interaction of the pole and wind. Currently, AASHTO uses a 1.14 gust factor which has resulted in successful designs. An effort was made to study the effect of the gust factor varying the first mode natural frequency and the damping ratios of 50 ft high light pole structure. ASCE, 1995, currently uses equation (5-5) to estimate the gust effect factor on a Category III-Flexible or dynamically sensitive structure, as the case in a high light pole structure

$$G = \frac{1 + 2gI_z \sqrt{Q^2 + R^2}}{1 + 7I_z}$$  \hspace{1cm} (5-5)

Where, $R$ is the resonant response factor,

$I_z$ is the intensity of turbulence at a specific height

$z$, is the background response

and $g$ the peak factor taken as 3.5.

Figure 5-28 represent results for the natural frequency effect on the gust factor.

Results show that for a first mode natural frequency of 1 Hz, the gust factor approaches 2. Therefore, the current 1.14 used by AASHTO is conservative for a high light pole structures. Further, these structures are susceptible to first mode vibration in windy areas.
Figure 5-28 Gust factor vs. natural frequency of a high light pole structure
CHAPTER VI

FRACTURE MECHANICS

6.1 Introduction

Fracture mechanics fatigue crack growth analysis is widely used in the assessment of welded structures. Cracks and flaws occur in many structures and components which have the potential to lead to catastrophic failures (Zhang et.al 2002). The aircraft industry is concerned with the structural behavior of each plane currently in service and hence, use fracture mechanics principles in determining the appropriate scheduling of inspection intervals (Basant et.al 1998). They have developed safe-life or fail-safe design approaches where a component is designed in a way that if a crack forms, it will not grow to a critical size between specified inspection intervals. Fracture mechanics deals with the answers to three questions:

A) How much load will it support?

B) How long will it support the load?

C) Are you sure?

Both crack initiation and propagation are essential to fatigue. The manner through which the crack propagates through the material gives great insight into the mode of fracture.
In fatigue, crack growth curves are presented with the crack length, $a$ as a function of the number of cycles depicted in Figure 6-1. A majority of the structures life is spent while the crack length is very small.

![Crack growth curve](image)

Figure 6-1 $a$ vs. $N$ for fatigue life estimates

Crack growth rate is defined as crack extension per cycle, $da/dN$, shown previously in Figure 2-2. Fatigue damage may primarily be attributed to propagation of a crack from an initial flaw or discontinuity at a location such as the weld toe as in the case of aluminum light pole structures.

The driving force behind crack propagation is the fluctuation of the stress intensity factor, $K$. The stress intensity factor characterizes the stress field around a crack. Normally a cyclic plastic zone forms at the crack tip, and the growing crack leaves behind a plastic wake. If the plastic zone is sufficiently small that it is embedded within an elastic zone, the conditions at the crack tip are uniquely defined by $K$. Therefore, values of log $da/dN$ can then be plotted versus log $K$ for a given crack length as shown in Figure 6-2 for 6061-T6 aluminum alloy. (AFGROW, 2005)
A crack growth law has been suggested by Paris and Erdogan which is now well established and is of the form

$$\frac{da}{dN} = C(K)^m$$  \hspace{1cm} (6-1)

where \(C\) and \(m\) are material constants, usually obtained from curve fits of experimental data. For the light pole alloy, \(C\) and \(m\) were taken as \(1.32 \times 10^{-08}\) and 2.95 respectively in the AFGROW Material data base. While expressions for \(K\) values are available for many two dimensional cases, solutions for three dimensional crack fronts are not so straightforward.
6.1.1 Crack Closure Effect on R-ratios

The idea of crack closure or plastic wake shielding was first introduced by Elber and was used to resolve the effects of R ratios (Herman et.al, 1998). In addition to the residual stress ahead of the crack, these deformations produce effects behind the crack tip that have an effect on the crack propagation behavior. Crack closure processes operate on the wake of the crack. The increasing use of damage tolerant concepts in component design has necessitated fatigue crack propagation (FCP) data be generated to assess the cyclic life of engineering structures. FCP data generated under conditions of decreasing stress intensity at low R-ratios \( \left( \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}} \right) \) can lead to overly optimistic estimates of component life as a result of high levels of crack closure. By contrast, typical service loading conditions involving the presence of tensile stresses, involve crack propagation under conditions of greatly diminished closure. Closure reduces the effective stress intensity \( (\Delta K_{\text{eff}}) \) by shielding the crack tip from a portion of the applied cyclic load since the opening stress intensity level \( (K_{\text{op}}) \) may be above the minimum applied stress intensity \( (K_{\text{min}}) \). The effective driving force for crack growth can be defined as

\[
\Delta K_{\text{eff}} = K_{\text{max}} - K_{\text{op}}
\]  

(6-2)

Where \( K_{\text{max}} \) is the minimum applied stress intensity level. Therefore, under low R-ratio test conditions, corrections for closure are extremely important, especially in the low growth rate region where closure levels may be high (Kumar and Garg 1989).
6.2 Fracture Mechanics Study plan

6.2.1 Finite Element Modeling (FEM)

Stresses obtained from FEM studies for through plate socket connections with 1 inch base plate thickness, the recommended 3 inch base plate thickness and shoe base socket connection details were used to estimate correction factors. The correction factors will be used as input to AFGROW, to estimate fatigue lives and compare with lab test results.

6.2.2 Residual Stresses

Residual stresses obtained and discussed in Chapter III were also used as input parameters to AFGROW to account for the presence of residual stresses in the tube. Kumar and Raju, 1999 conducted extensive studies on the effect of crack closure on 6063-T6 aluminum alloys and showed that no crack closure is observed for higher R values. To better estimate the fatigue lives of the structure in the presence of compressive stresses, a study of the original test results at a stress ratio of R=0 and at higher stress ratios taken as 0.5 and 0.7 were included for comparison to actual fatigue test results.

6.2.3 Correction Factors

Numerical methods have been used to estimate the stress intensity factor ($K$) for un-cracked bodies depending on results from FEM packages (Zettlemoyer, 1976). Welded details contain flaws of various shapes and sizes, but are often approximated by an elliptical or semi-circular shape described in Equation (6-3) below.

$$K = c * S \sqrt{\pi a}$$  \hspace{1cm} (6-3)
Where $c$ is the correction factor given by Equation (6-4)

\[ c = F_g \times F_w \times F_s \times F_e \quad (6-4) \]

$F_g$ is the stress gradient correction factor which accounts for the nonuniform stresses acting on the crack resulting from the detail geometry. Hence, gradient corrections for cracks growth through the tube thickness in the light pole structures were found by using Equation (6-5) below.

\[ F_g = \frac{2}{\pi} \sum_{j=1}^{m} \frac{K_j}{\sqrt{a^2 - L_j^2}} \times \Delta L_j \quad (6-5) \]

Where $a$ is the crack length, $L_j$ is the element length, $K_j$ is the stress concentration factor at each node through the tube thickness and $\Delta L_j$ is the length difference between two elements. In our case, $L_j$ was case taken as 0.005 inch.

$F_w$ accounts for the effect of a finite width plate and the correction will be taken to be

\[ F_w = \sqrt{\sec\left(\frac{\pi t}{2t_f}\right)} \quad (6-6) \]

Where $t_f$ is the tube thickness (0.25 in.) and $a$ is the crack length assumed as 0.005 inch.

Welded details contain flaws that may be modeled as semielliptical in shape. In such case, the crack shape correction factor is given by $F_e$ and evaluated in the present study by using Equation (6-7) below

\[ F_e = \frac{1}{E(K)} \quad (6-7) \]
Where $E(K)$ is the complete elliptical integral of the second kind, i.e. given by Equation (6-8)

$$E(K) = \frac{\pi}{\int_0^\theta [1 - K^2 \sin^2 \theta] d\theta}$$

(6-8)

Where $K^2 = \frac{c^2 - a^2}{c^2}$

The solution depends only upon the ratio, $\frac{a}{c}$, and was assumed in a ratio of 1:2.

$F_s$ is the correction factor associated with a free surface or edge crack and is given by Equation (6-9) below.

$$F_s = 1 + 0.12 \times (1 - \frac{a}{c})^2$$

(6-9)

Where $a$ represents the crack in the direction of the tube thickness and $c$ (2a) is the crack in the circumferential direction as shown in Figure 6-3 below.

Figure 6-3 Crack length (a) through the tube and (c) through the Circumferential direction

Equation (6-1) may be rearranged as shown in Equation (6-10) by integrating the crack growth expression from the initial crack size, $a_i$, to the critical size, $a_{cr}$ and an estimated fatigue life using the correction factors may be found using AFGROW.
\[ N = \int_{a_i}^{a_f} \frac{da}{1.32 \times 10^{-8} (F_g F_w F_s F_v \sqrt{\pi a})^{2.95}} \]  

(6-10)

6.2.4 AFGROW Software

AFGROW fracture mechanics software (Figure 6-4) was used to predict fatigue lives. An initial crack length of 0.005 inch was assumed and is consistent with previous studies of flaws in welded aluminum structures (AFGROW, 2004). Material properties are implemented in AFGROW for various types of materials from previous lab research studies. AFGROW permits the input of stresses determined using finite element models to help determine the stress intensity factor or allows input of independently determined correction factors.

![AFGROW user interface for a crack through pipe](image)

Figure 6-4 AFGROW user interface for a crack through pipe

6.3 Analysis and Fatigue Life Prediction

Correction factors were calculated for the through plate socket connection with a 1 inch base plate thickness the 3 inch plate thickness and the shoe base socket connection details as shown in Figure 6-5. A 3 in. thickness represents a previously proposed plate
thickness for light pole structures, which showed a 30% increase in local stresses compared to 1 in. plate (Azzam and Menzemer Accepted 2006) and is close to the behavior of shoe base socket connection details.

It’s clear that the 1 inch flexible base plate resulted in higher correction factors, and is a maximum of approximately 4.1 as compared to 1.89 for the through plate socket connection and 2.0 for shoe base socket connection detail.

Figure 6-5 Correction factor results for 1 and 3 inch through plate and shoe base socket connection.

To input the stress range for each case, AFGROW allows the user to input a Stress Multiplication Factor (SMF) that takes into account for both the stress range and correction factors obtained at each crack length as shown in Equation (6-11)

\[
SMF = c \star \sigma_r
\]  

(6-11)

Where \( c \) is the correction factor along the crack path and \( \sigma_r \) is the stress range in ksi.
Analysis was carried out using the same stress ranges conducted in the lab tests as presented earlier in Table 4-2 and 4-3.

The primary objective of the fracture mechanics study will be three fold:

1) Compare fatigue lives obtained in the lab with AFGROW analysis for three types of details with different AASHTO Categories.

2) Run analysis using fatigue crack propagation data found using higher stress ratios \( \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}} \) \( R=0.5 \) and 0.7 to examine the role of closure. Chart 1 represents a step by step procedure that was carried out in AFGROW to estimate S-N curves and compare with lab test results for both types of details.

3) Compare crack length vs. number of cycles (N) for the three details at different fatigue crack growth stress ratios.
Chart Flow for Fracture Mechanics Analysis using AFGROW

Material Data base: NASGRO

Model Configuration: Part Through Crack in a pipe

Assumed crack length, \(a_i\) (Through tube) = 0.005 in.
C (crack in circumferential direction) = 2 \(a_i\)

Tube Dimension:
\(D_i = 9.5\) in.
\(D_o = 10\) in.

Stress Multiplication Factor (SMF)
\[ SMF = \frac{\sigma_r}{\sigma_{r,c}} \times C \]

Correction Factors obtained from Finite Element and numerical solutions to modify stress intensity factor at each crack depth

Enter Residual stress results obtained from test experiments

Repeat for
R=0
R=0.5
&
R=0.7
6.4 Fatigue Life Estimates

6.4.1 Stress Ratio (R=0)

The first set of analyses were conducted for the through plate socket connection detail with a 1 inch plate thickness, using a low fatigue crack growth rate stress ratio, (R=0) with and without residual stresses. Figure 6-6 shows significant scatter in fatigue data for the through plate socket connection with t=1, and is believed to be the result of the limited number of samples tested. AFGROW predictions resulted in higher fatigue lives as compared to the lab tests when no residual stress was included in the analysis. When residual stresses were included, the fatigue life at the same stress ranges was nearly the same. The Constant Amplitude Fatigue Limit (CAFL) is estimated to be close to 1.0 ksi. The previously estimated CAFL based on lab test results is expected to be between 0.5 – 1.0 ksi. Figure 6-6 shows the results of a 3 inch base plate thickness with R=0 as compared to test data and exhibited higher fatigue lives as compared to a 1 inch plate thickness at the same stress ranges.

Figure 6-7 shows the results for the shoe base socket connection detail at low stress ratios. At higher stress ranges, AFGROW fatigue lives are greater for the same stress range. However, the fatigue live are closer for lower stress ranges, specifically between 3.5-5.5 ksi.
Figure 6-6 Fatigue life comparison at R=0 between lab and AFGROW for 1 inch through plate socket connection detail

Figure 6-7 Fatigue life comparison at R=0 between lab and AFGROW for 3 inch through plate socket connection detail
6.4.2 Stress ratio (R=0.5 & R=0.7) and the Effect of Crack Closure

The presence of surface compressive residual stresses complicated fracture mechanics modeling. In order to account for the residual stresses, fatigue crack propagation data obtained at higher stress ratios, R=0.5 and 0.7 was used for the predictions and compared to lab test results. In addition to the high stress ratio fatigue crack growth data, the average residual stress distributions will be superimposed. By using crack growth data with minimal levels of closure combined with the residual stress data, for crack closure effects may be accommodated through the stress field and not in addition to the intrinsic material crack growth kinetics. Figure 6-9 shows the analysis results for R=0.5 and R=0.7 fatigue crack growth stress ratios with the presence of residual stresses as compared to lab test results. The results show that by using crack
growth data with minimal closure, combined with the actual measured residual stress, fatigue lives become closer to the experimental data obtained in the lab.

Figure 6-9 Fatigue life comparison at R=0.5&0.7 between lab and AFGROW for t=1 inch through plate socket connection detail

Results for details with 3 inch base plate thicknesses are shown in Figure 6-10, and represents a longer fatigue lives as compared to 1 inch through plate thickness lab test results. Analyses were also compared to the shoe base socket connection lab test results, which showed the fatigue lives at nearly the same stress ranges to be comparable. This is believed to be the result of the lower stress concentration present due to the rigidity of the integrally stiffened shoebase or 3 inch base plate as compared to the in a 1 inch through plate detail.

Figure 6-11 presents results for shoe base socket connection details at higher fatigue crack growth stress ratios. It’s evident that at higher stress ratios, fatigue lives
shift more to the left and follows the trend of shoe base socket connection detail test results obtained in the lab.

Figure 6-10 Fatigue life comparison at R=0.7 between lab and AFGROW for $t=3$ through plate socket connection detail

Figure 6-11 Fatigue life comparison at R=0.5 and 0.7 between lab and AFGROW for shoe base socket connection detail
6.5 Crack length (a) vs. Number of cycles (N) for 3 Types of Details

6.5.1 3.5, 5.4 and 8.6 ksi Stress Ranges for Shoe Base Socket Connection Details

Fatigue predictions obtained so far revealed that crack growth data obtained at an R=0.7 combined with residual stress distributions obtained from the experimental program predicts the fatigue test data reasonably well. Crack length (a) vs. Number of cycles (N) curves were obtained for the shoe base socket connection at R=0.7 for stress ranges equaling 3.5, 5.4 and 8.6 ksi. This covers both the lower and higher stress ranges used in lab tests. Assuming an initial crack length of 0.005 inches and lower stress ranges, considerable numbers of cycles are required for the structure to reach failure. Higher stress ranges translate into fewer cycles for the structure to reach failure as shown in Figure 6-12.
6.5.2 5.5 ksi Stress Range for t=1 and t=3 inch Plate Thickness

Figure 6-13 is a comparison between fatigue lives of a through plate socket connection detail with both t=1 and 3 inch base plate thicknesses. Results show that at a stress range of 5.5 ksi, 1,000,000 cycles are required to reach failure as compared to 400,000 cycles for the 1 inch base plate thickness case. A 50% difference in cycles between the two cases clearly shows the effect of base plate thickness for such details.

![Through plate graph](image)

Figure 6-13 a vs. N for through plate socket connection at t=1 & 3 base plate thicknesses at a 5.5 ksi stress range

6.5.3 Comparison of Crack Length vs. Fatigue Life for 3 Types of Details

Figure 6-14 presents a comparison between 1 and 3 inch base through plate and the shoe base connection detail as the crack length increases. It shows that at the identical stress range (5.5 ksi), a 1 inch base plate thickness exhibits a lower fatigue resistance as compared to both the t=3 inch and shoe base socket connection details.
6.6 Effect of Crack Growth Stress Ratio, R

The effect of stress ratio, R, on the fatigue life of 1 and 3 inch base plate thickness and a shoe base socket connection are presented in Figures 6-15, 6-16 and 6-17. Results demonstrate that as R increases, fatigue life decreases. However, the presence of compressive residual stresses on the surface combined with crack growth data obtained under conditions of minimal closure (R=0.7) are closest to lab test results.
Figure 6-15 a vs. N for shoe base socket connection detail at different stress ratios.

Figure 6-16 a vs. N for through 1 inch plate socket connection at different stress ratios.
6.7 Lower Bound Curves

Lower bound curves for different AASHTO Category details are compared with results obtained from lab tests and AFGROW analysis for the through plate with both 1 and 3 inch base plate thickness and shoe base socket connection details as given in Figure 6-18 and 6-19 respectively. Results for the shoe base socket connection details correlate well with lab test results at lower stress ranges close to the CAFL. Taking infinite life to be 10 million cycles, the CAFL is about 3.3 ksi as compared to 3.8 ksi from AFGROW analysis. However, AFGROW over estimated fatigue lives at higher stress ranges as compared to lab test results.

Figure 6-20 presents fatigue life predictions for 1 and 3 inch base plate thickness and its clear that greater fatigue life is experienced by using a 3 inch plate thickness. A
CAFL of 3.6 ksi for the 3 inch base plate as compared to 1 ksi for t=1 inch base plate shows that the 3 inch base plate thickness is close to the behavior of shoe base socket connection detail. There appears to be a need for fatigue tests on a 3 inch plate thickness to prove such behavior. Further, more data samples for the 1 inch plate thickness should be tested to validate the assumptions stated herein.

Figure 6-18 Lower bound curves for AFGROW and LAB test results for shoe base socket connection detail
6.8 Scanning Electron Microscopy (SEM) of Aluminum-6063 for Shoe Base Socket Connection Detail

SEM was conducted for samples taken from a shoe base socket connection detail, tested at a stress range of 5.3 ksi and, exhibiting a fatigue life of \( (N) = 1.6 \times 10^6 \). The two samples are shown in Figure 6-20 and 21 respectively. SEM examinations conducted at different magnification levels that range between 5 and 65 micrometers in depth (\( \mu m \)). Description of the crack propagation sequence is illustrated underneath each micrograph and shown in Figures 6-20 through 6-27.
Figure 6-20 Sample 1A for a section of a shoe base socket connection detail

Figure 6-21 Sample 1C for a shoe base socket connection detail
Figure 6-22 Scanning electron micrographs of aluminum alloy light pole (Sample 1A) at 50µm. Cyclically deformed at stress range of 5.3 Kσi at 1 Hz resulting in $1.6 \times 10^6$ cycles to failure, showing:

(a) Overall morphology showing fine striation-like features in the region of early crack growth.

(b) Coarse striations reminiscent of rapid crack growth prior to the onset of unstable fracture
Figure 6-23 Scanning electron micrographs of aluminum alloy light pole (Sample 1A) at 20 µm.
Cyclically deformed at stress range of 5.3 Ksi at 1 Hz resulting in $1.6 \times 10^6$ cycles to failure, showing:
(a) Cracking along the recrystallized grain boundaries observed in the region of early crack growth.
(b) Fine irregularly spaced striations-like features or ripples reminiscent of localized micro-plastic deformation in the region of stable crack growth.
Figure 6-24 Scanning electron micrographs of aluminum alloy light pole (Sample 1-A) at 5 µm

Cyclically deformed at stress range of 5.3 Ksi at 1 Hz resulting in 1.6 x 10⁶ cycles to failure, showing

(a) A non-linear crack separating the region of slow and stable crack growth (early crack growth) from the stable crack growth region.
(b) Fine microscopic cracking along the recrystallized grain boundaries in the region of slow and stable crack growth.
Figure 6-25 Scanning electron micrographs of aluminum alloy light pole (Sample 1-C) at 25 µm

Cyclically deformed at stress range of 5.3 Ksi at 1 Hz resulting in 1.6 x 10^6 cycles to failure, showing:

(a) Overall morphology showing the regions of early crack initiation and early crack growth.
(b) High magnification of (a) showing fine striation like features in the region of stable crack growth.
(c) High magnification of the region of early crack initiation showing the striations forming a ripple-like pattern.
(d) The region of slow and stable crack growth indicative of shallow striations and reminiscent of localized micro-plastic deformation.
Figure 6-26 Scanning electron micrographs of aluminum alloy light pole (Sample 1-C) at 10 µm.

Cyclically deformed at stress range of 5.3 Ksi at 1 Hz resulting in 1.6 x 10^6 cycles to failure, showing:

(a) A non-linear crack traversing the recrystallized grain boundary in the region of early crack growth
(b) Cracking along the grain boundary separating the regions of early crack growth and stable crack growth.
Cyclically deformed at stress range of 5.3 Ksi at 1 Hz resulting in $1.6 \times 10^6$ cycles to failure, showing:

(a) An array of fine microscopic cracks in the region of stable crack growth

(b) A waveform of ridges and grooves in the region of unstable crack growth.
1) Based on the analysis of the shoe base fatigue data, the constant amplitude fatigue limits is expected to be on the order of 3 to 3.5 ksi.

2) In the high cycle regime, all of the fatigue data for the shoe base detail falls between Category C and D design curves.

3) 90% of the shoe based details tested failed by developing cracks along the weld toe.

4) Based on the limited fatigue tests for the through plate detail, the constant amplitude fatigue limit falls between 1.0 – 1.3 ksi.

5) All of the through plate details tested had fatigue strengths below Category E.

6) Fatigue strength of the shoe base details is significantly larger than that of the through plate joint.

7) Based upon parametric studies, the fatigue strength of the through plate detail may be improved by increasing the base thickness. An increase in thickness from 1 to 2 inches decreased the maximum normal stress adjacent to the weld toe by at least 24% in all cases evaluated.

8) An increase in base thickness for the shoe base detail did not appreciably change the maximum normal stress in the tube adjacent to the weld toe.
9) A parametric study on unequal fillet welds with a 3 inch plate thickness in the through plate detail revealed a 12% decrease in the hot spot stress as the weld leg increased as compared to 17% when both vertical and horizontal weld legs were increased equally.

10) Parametric studies with 1 inch base plate thickness and unequal weld legs resulted in an increase in fatigue resistance as the vertical weld increases. A 53% increase in area of opposite to bolt constraint compared to 34% on farthest distance from neutral axis. In the same study it was shown that more decrease in stress at hot spot area is seen in farthest distance from neutral axis when compared to opposite to bolt constrain area which is a result of base flexibility issue.

11) The r/t parametric study with 1 inch base plate thickness showed that as tube thickness increases, local stresses decreased and fatigue resistance would be expected to increase.

12) Increasing the bolt hole size to 2.5 inches resulted in a 25% reduction in stresses when compared to adding 4 more holes and the original 4 hole geometry. Local stresses were reduced 8 holes were modeled, but stress concentrations were moved to the furthest distance from neutral axis.

13) Fatigue test results appear to be consistent with residual stress measurements performed on both types of details.

14) Parametric studies on collar socket connection details showed that the presence of the collar decreases the stresses along the lower portion with increasing base plate thickness.

15) The effect of base plate thickness is negligible at the top of the 4 inch collar detail
16) Butterfly trend is clearly observed at the socket connection with slotted holes.

17) The gusset stiffener study revealed a higher stress concentration at short stiffeners (L<2 inches) that contradicts current AASHTO,2001 specifications.

18) The effect of base plate thickness on short, medium and long stiffener is pronounced with a 35% reduction in stresses when increasing the base plate thickness from 1 to 3 inches.

19) At same stress ranges, the 1 inch plate thickness exhibited lower fatigue life compared to 3 inch and shoe base socket connection details.

20) A 40% difference in fatigue life was found between 1 and 3 inch base through plate socket connections details. This shows that base plate thickness plays a role in the fatigue life of such type of detail.

21) Fatigue tests correlate well with AFGROW predictions using fatigue crack growth data determined at higher stress ratios(R=0.7) combined with the presence of compressive residual stresses.

22) At identical stress ranges, the 1 inch plate thickness exhibits lower predicted fatigue life as compared to 3 inch and shoe base socket connection details.

23) As the fatigue crack growth stress ratio increases, fatigue life decreases, for all types of details.

24) The fatigue life of a 3 inch base plate thickness is close enough to shoe base socket connection detail.

25) Fatigue strength of 1 inch base plate thickness falls close to Category E’ detail.

26) As the tube thickness increases, natural frequency increases.
27) Higher natural frequencies are presented for 10 inch diameter tube compared to 8 inch tube.

28) Coarse striations reminiscent of rapid crack growth prior to the onset of unstable fracture.

29) The region of slow and stable crack growth indicative of shallow striations and reminiscent of localized micro-plastic deformation.

30) A waveform of ridges and grooves in the region of unstable crack growth.

RECOMMENDATIONS FOR FUTURE WORK

1) Full scale fatigue tests on steel and aluminum through plate socket connection details with 3 inch base plate thickness to validate FEM results.

2) Full scale fatigue tests on gusset stiffener socket connection details to validate AASHTO, 2001 fatigue category details for short, medium and long stiffeners.

3) Full scale fatigue tests on the collar socket connection detail with slotted hole for future AASHTO adoption.

4) Residual stress measurements during fatigue tests for through plate socket connection detail to measure stress distribution “if any”.

5) Long term monitoring of Signal Sign structures in the State of Ohio to validate pressure and load recommendations for vortex, galloping, truck induced and wind load effects.

6) Vibration tests on luminaire support structures to measure the first modal natural frequency of such type of details.
REFERENCES

AFGROW (2005) version 4.0010.13  AFRL, WPAFB.


