PARAMETRIC STUDY OF FATIGUE IN LIGHT POLE STRUCTURES

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Master of Science

Maryam Sadat Hosseini
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ABSTRACT

The fatigue behavior of light pole structures under wind loadings (specifically, wind gust) were studied. Fatigue failures have occurred in luminaire structures as a result of wind-induced vibration under severe environmental conditions. The increasing number of failures is due to vibration near the fundamental mode and lack of appropriate damping and motion control. In some sense the problem has been exacerbated because the majority of fatigue design specifications were established prior to 2001. As a result, the 4th edition of the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals now provide clear design criteria that incorporate loading conditions and connection detail categories.

Initiative for the study was the occurrence of failures due to rapid fatigue resulting from wind induced vibration. Several failures have been associated with welded hand-hole details that have not been the subject of comprehensive experimental studies. As a result there is a lack of available data for some aluminum structural details like the hand hole. As such, Finite element models for several parametric studies were constructed and analyzed in order to gain insight into the behavior of the light pole structures during wind loads (specifically, wind gust). Parametric studies were conducted to increase understanding of the important geometric features and their influence on local stresses.
that derive the fatigue response specifically; the hand-hole reinforcement width, hand-hole geometry and shoe base connection were examined.

ANSYS Workbench was used to complete all of the analysis (ANSYS, 2012). The fatigue behavior of light pole structures subjected to 3-sec wind gusts will be studied with particular emphasis on two critical locations. In particular, emphasis will be placed on hand-hole and shoe-base connections. The results indicate that, light pole structures with pole thicknesses larger than 0.375 in are not susceptible to fatigue failure around shoe base connection for the loading and sizes considered. In addition, by increasing pole thicknesses, the maximum stress around shoe base decreases. For the case of the hand-hole with different widths, hand-hole with width of 0.5 in shows better performance as compare to 0.1 in and 1 in width. Finally, the results indicate that increasing hand-hole thicknesses will reduce fatigue failures around the hand-hole.
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CHAPTER I
INTRODUCTION

1.1 Background

Sign, signal and luminaire structures are used widely on highways for traffic control and roadway lighting. The unanticipated failure of a sign, signal, luminaire structures could cause severe injuries, property damage, and accidents. Several incidents of failures have developed in light pole structures in past decades. Failures have occurred around the pole to base and hand-hole connections, as a result of the fatigue crack growth. Most of these localized failures are caused by wind induced vibration, resulting in stress cycles at the weld toes (Azzam2006).

Light poles are designed to provide particular environmental and loads resistance. In addition, the American Association of state Highway and Transportation Officials (AASHTO), American National Standards Institute (ANSI), the Aluminum Design Manual (ADM) and many building codes present somewhat different standard guidelines and design provisions. These design provisions are based on theoretical analysis, research and industry practice and consider direct wind pressures on the pole, bending, shear, axial and torsional stresses. In general, poles are designed based on a three second gust for the 50-year mean wind map found in AASHTO, ANSI, or local building codes.

There are two types of sign structures used along the side of highways in the United States. 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 the top. The second type is a Traffic signal structure. A traffic signal structure with only one pole is a cantilever structure and a traffic structure with two or more columns are sometimes referred to as a Sign Bridge or Overhead Structure.

Fatigue failures take place due to the repeated application of variable stresses that are much lower than the stress required to cause failure during a single application. Fatigue failures result from repeated loadings and the resulting stress cycles. Each effective stress cycles above the active threshold causes a certain amount of damage. The most distinguishing locations of fatigue failures in components are at areas of stress concentrations such as welds, notches, holes and changes in section. Typical areas that may exhibit fatigue cracking in sign, signal and luminarie structures are at welds, anchor bolts, hand-hole and base connections.

Motivated by several fatigue failures, AASHTO provided support for several research projects. One such project was funded through The National Highway Research Program (NCHRP) and its focus was to develop a reliable fatigue-resistant design procedure for cantilevered signal, sign, and light luminaries (Ocel et al. 2006). According to NCHRP Report 469, most states have experienced some failure of support structures. Therefore, understanding both wind loads and fatigue life behavior and analysis of the light poles is critical for the eventual mitigation of the problem. Fatigue cracking typically occurs along the top outside weld toe. Wind forces cause cyclic stress ranges that contribute to
fatigue damage in pole. Damage usually appears in the form of cracks that grow over time and lead to final failure through ductile rupture or brittle fracture.

1.1.2 Loads Affecting Luminaire Structure

(AASHTO) has published specification for structural supports for highway signs, luminaries and traffic Signals that identify four types of wind induced loadings to be considered in design, including: Galloping, Vortex shedding, Natural wind gusts, and Truck-induced wind gusts.

Galloping and vortex shedding are aeroelastic phenomena caused by the action of wind and structural vibration. Galloping-induced fluctuation primarily occurs in flexible, lightly damped structures with non-symmetrical cross sections. Vortex shedding typically occurs during steady uniform flows and creates resonant fluctuation in a plane normal to direction of flow. Galloping results from unbalanced periodic forces, while vortex shedding is caused by the shedding of vortices in the wake behind of structural element.

Natural wind gust arise from changing in velocity and direction of air flow. Truck-induced wind gust is defined as the passage of trucks below cantilevered support structures resulting in induced gust load on the front area and underneath the members. The effect of these loads is most considerable in cantilevered sign-support structures which have large front area. Unlike galloping and vortex shedding arise in constant-amplitude, the amplitude of the response to natural wind gusts and truck-induced wind gusts are variable.
It is generally believed that luminaire structures are not subjected to *Galloping* and *Truck-induced wind gusts* but, *Vortex shedding* and *Natural wind gusts* are the major cause of fatigue damage in luminarie structures (NCHRP Report 469)

1.1.3 Light Pole Components

A cantilevered light pole structure consists of the following elements: Pole shaft, base, anchor bolts, hand-hole and luminaire as shown in Figure 1-1. A common structural detail that joins the base and hand-hole to the pole shaft are fillet-welded connections. The base of the round tubular pole is inserted into base that contains a circular hole. Fillet welds are placed between the pole and base as shown in Figure 1-2. The outside fillet weld is classified as a category E’ fatigue detail and is an area of design concern in light pole structures.

![Figure 1-1, Cantilevered Light Pole framework](image)
1.2 Objective of the Study

Shoe Base and Hand-hole connections have experienced fatigue problems in the United States (Kaczinski et.al 1998). The AASHTO specification categorizes fillet weld base and hand-hole connections as E’ details. Aluminum light poles are typically fabricated from alloy 6063-T6 or in some cases 6005-T5 extruded tubes. Aluminum pole structures are primary focus of this research. All models prepared have geometric features similar to those produced commercially. The behavior of aluminum light poles was studied for comparison purpose.

A research program has been undertaken to evaluate fatigue performance of luminaire structures under wind loading. An extensive parametric study conducted using Finite
Element Code (ANSYS Workbench 14.5). Both the pole and hand-hole thicknesses were studied to see how stress concentration, Fatigue Lives changed.

1.3 Research Motivation

The focus of this study is to analyze the effects of wind forces and wind gusts on cantilevered light pole and observe the critical stresses, as well as the fatigue life of light pole structures by using finite element code.

The results of a literature review of highway luminaire structures is presented in Chapter II. According to ASSHTO specification, fatigue applications were studied. Moreover, the history of fatigue design, fatigue behavior, fatigue life characterization rules (e.g. Miner’s rule, Paris law and Rain flow counting method) and types of loadings affecting luminaire structures are discussed. At the end of this chapter, significant research conducted on cantilevered sign, signal and luminaire structures is discussed.

Chapter III describes the development and implementation of a light pole finite element model using the structural analysis program ANSYS Workbench 14.5 that was used to obtain the stresses at the weld toe connections both in hand-hole and shoe base details. Moreover, a static wind load applied to the finite element model and, the equivalent wind pressures due to basic wind speed were calculated. At the end of this chapter, a method to model one cycle of wind gusts is illustrated.

Chapter IV discusses fatigue due to wind loadings. Shoe base and hand-hole fatigue results are presented. A parametric study was conducted to determine the variation in
stresses, strain, and fatigue lives for different geometric cases. Conclusions and further recommendations were described in Chapter V.
CHAPTER II
LITREATURE REVIEW

2.1 Highway Luminaire Structures

Luminaire, traffic signals and sign structures are widely used for traffic control all over the world. There are different types of luminaire structures but most of them consist of a vertical pole with or without a cantilevered arm for signals or signs.

Light pole structures are flexible due to their long span thickness and relatively small-cross sectional area. This may cause or result in low natural frequencies for the first several modes. Moreover, provided damping has not been augmented, the damping in luminaire structures is very low, approximately around one percent of critical damping (Kaczinski et al., 1998). As a result, luminaire structures may be susceptible to fatigue failure under wind loading. During the past three decades, fatigue damage has been occurred in luminaire structures. Fatigue is localized structural damage that occurs when a material or component is subjected to cyclic loading. Damage may occur even if the nominal maximum stresses values are less than the ultimate tensile stress limit or below the yield stress limit of the material. Finally, a crack will reach the critical size, become unstable and cause the structure to fail suddenly. Obviously, damage costs are significant, even on per occurrence. It is essential to perform periodic inspections of sign structures to
detect potential problems and avoid serious accidents. A literature study was conducted to examine.

1) History of fatigue and applications

2) Various types of wind loadings which affected luminaire structures

3) Researches were conducted to investigate fatigue in luminaire structures

2.2 History of Fatigue

Early studies of fatigue begin with W.A.J. Albert, a German mining administrator, who reported the failure of iron mine-hoist chins resulting from repeated loading in 1829 (Schutz, 1996). Rankine, recognize the fatigue failure of railway axles caused by initiation and growth of cracks from the shoulder and other stress concentrations on the shaft. In 1842, he presented his work as a paper to the Institution of Civil Engineers. Unfortunately, many engineers ignored his work because they believed in “re-crystallisation” of metal which was wrong (Schutz, 1996). In 1860 S-N curves were first expressed by Wohler. He illustrated fatigue test results on railway axles in the form of tables. But, his Colleague, Spangenberg re-draw the Wohler’s conclusion as curves and referred them as Wholer 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). In 1910 Basquin represented the finite life region of the “Wohler curve” in the form of $\log \sigma$ on the ordinate and $\log N$ on the abscissa and called it, Basquin Law $(\Delta \sigma (N_f)^b = C)$ . The numerical values of C and b were presented in a table presented by Basquin (Basquin, 1910).
Bauschinger defined the cyclic stress strain curve. The *Bauschinger* effect defines as a characteristic of materials where the material's stress/strain properties change as a result of the microscopic stress distribution of the material (Bauschinger, 1886). In 1903 Ewing’s and Humphries developed the idea of fatigue crack initiation using cyclic deformation. In 1924, Palmgern proposed linear damage rule for variable amplitude loading and this idea was further developed by Miner in 1945 (Schutz, 1996). 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.

2.2.1 AASHTO and Fatigue Categorization of Connection Details

The cantilever luminaire structures should be designed for infinite fatigue life based on AASHTO sign structure design specification (AASHTO, 2001). Although the luminaire structure fatigue limits are based on the AASHTO bridge specifications, the geometrical configuration of typical cantilever aluminum luminaire structures connection details vary significantly from details found in the AASHTO bridge specifications.

Fatigue connection details such as, hand hole-to-column and column-to-base were acknowledged from state department of transportation standard drawings and manufacturer literature as low fatigue strength. According to the AASHTO Bridge specification, the categorization of sign, signal and luminaire structure connection details was included in NCHRP Report 412. This categorization of luminaire structure connection details to the existing AASHTO specification is based upon a general understanding of fatigue behavior, development of fatigue design curves with respect to previous research and experience with structural failures. In addition, by evaluating the
stress concentrations and locations where cracks form, the relationships between the fatigue resistance of luminaire support structure connection details and the existing knowledge of bridge details had been investigated (Kaczinski, et al., 1998; Fisher, et al., 1981). The provisions of the AASHTO bridge specifications for the design of structures for fatigue are based on a nominal stress approach in which details are grouped into categories according to their relative fatigue resistance.

The AASHTO specification for sign structures (2001) includes nine design categories labeled A through K2. Each of these nine categories decreases in fatigue strength. Examples of connection details applicable to sign structures include anchor bolts (Category D detail), mechanical clamps and U-bolts Category D details), column-to-base-plate or mast-arm-to-flange-plate socket connections (Category E’ details) and hand-hole openings (Category E detail). Most cantilever sign structure connection details are classified as having low levels of fatigue resistance.

2.2.2 AASHTO Fatigue Life Rules and S-N Curves

In 2004, AASHTO published the fatigue design requirements. AASHTO is among the most-widely used standards to design highway bridges for truck induced cyclic loadings. In addition, the fatigue design requirements in AASHTO (2001) for sign structures draw heavily from the detailed information and procedures provided for bridge structures.

In current study, fatigue life is mostly derived from the AASHTO specification requirements for the fatigue strength (resistance) of structural details. The fatigue design strength given in the AASHTO bridge design specification (2004) corresponds to S-N
curves derived two standard deviations below the mean fatigue resistance. In Figure 2.1, S-N design curves are shown and plotted with stress range on the vertical axis and the number of cycles to failure on the horizontal axis for eight different detail categories. Axes are logarithmic representations. Beyond a certain point, which depends on the detail category, the fatigue life line is horizontal (shown as dashed line in Fig. 2.1) or is infinite. The stress range that identifies the horizontal portion of each curve is called constant-amplitude fatigue limit (CAFL), below which the fatigue life is infinite. The sloping portions of the S-N curves can be represented as the following:

\[ N = M \Delta\sigma^m \]  

(2-1)

Where

\( N \) = number of cycles to failure

\( \Delta\sigma \) = constant-amplitude stress range

\( m = 3 \)

\( M \) = a constant that depends primarily on geometry of the structural detail

Table 2.1 lists eight detail categories and corresponding \( M \) and CAFL values. The fatigue strength curves are based upon a large database of constant-amplitude-loading tests of specimens with the structural detail of interest. Design curves are obtained by shifting the fitted curve from the test data horizontally to the left by roughly two standard deviations of the scattered data to provide fatigue strength with the desired level of safety (Fisher et al., 1998). The details of sign structures are classified based on associations with existing...
details categories in the AASHTO bridge specification, the AWS D1.1 structural welding code, and testing in other research (Kaczinski et al., 1998).

Table 2-1, Constants for different steel fatigue detail categories (AASHTO, 2004)

<table>
<thead>
<tr>
<th>Detail Category</th>
<th>$M$ (MPa)$^3$</th>
<th>CAFL (MPa)</th>
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<tr>
<td>A</td>
<td>$82.0 \times 10^{11}$</td>
<td>165</td>
</tr>
<tr>
<td>B</td>
<td>$39.3 \times 10^{11}$</td>
<td>110</td>
</tr>
<tr>
<td>B’</td>
<td>$20.0 \times 10^{11}$</td>
<td>82.7</td>
</tr>
<tr>
<td>C</td>
<td>$14.4 \times 10^{11}$</td>
<td>69.0</td>
</tr>
<tr>
<td>C’</td>
<td>$14.4 \times 10^{11}$</td>
<td>82.7</td>
</tr>
<tr>
<td>D</td>
<td>$7.21 \times 10^{11}$</td>
<td>48.3</td>
</tr>
<tr>
<td>E</td>
<td>$3.61 \times 10^{11}$</td>
<td>31.0</td>
</tr>
<tr>
<td>E’</td>
<td>$1.28 \times 10^{11}$</td>
<td>17.9</td>
</tr>
</tbody>
</table>
2.2.3 Miner’s Rule

This method accounts for the damage that result when fatigue loading is not applied at a constant harmonic amplitude. Miner’s rule is a linear damage rule and it assumes that the damage fraction that results from a particular stress range level is a linear function of the number of cycles which takes place at that stress range. Damage done by repeated application stress range $S_i$ is given by:

$$D_i = \frac{n_i}{N_i}$$

(2-2)

Where

$n_i =$ number of cycles occur at stress range level $i$

$N_i =$ number of cycles to failure at stress level $i$
It is also significant to realize that the term “failure” should be interpreted as reaching the allowable fatigue life rather than the actual structure failure. To calculate $N_i$ the following AASHTO equation is used (AASHTO 2009):

$$N_i = \frac{A}{S_{i}^{\beta}}$$  \hspace{1cm} (2-3)

Where $A$ is the category constant specified by AASHTO and $S_i$ is the stress range (ksi). As a result, the damage done by all stress range is less than one ($\sum D_i \leq 1$). Structural failures occur when the sum reaches unity. Miner’s rule states that the finite fatigue life, in years, can be determined by;

$$\frac{1}{\sum D_i}$$  \hspace{1cm} (2-4)

If the number of occurrences for each stress range is identified, the finite fatigue life can be determined. This method has some restrictions (Bannantine et al., 1990): First, it does not consider chain effects and it is independent of the average stress in the cycle. In addition, when residual stresses are high and plasticity is restricted, both factors are not consistent with observed behavior of metal fatigue (similar to sign structure under wind loading). It is known that these factors only have a small influence (Bannantine et al., 1990). Last but not least, the approach provides a sufficient correlation with test data and it has the significant benefit that it is easy to use (Fisher et al., 1998). The AASHTO LRFD Bridge Design Specification (AASHTO, 2004) also advises that the Miner’s rule can be used to account for cumulative damage. Hence, Miner’s rule is applicable to this research.
2.2.4 Paris Law

In 1961 Paris and Erdogan recognized that crack growth was directly related to the stress intensity factor range, \( \Delta K \) and is now referred to as the Paris Law, \( \frac{da}{dN} = C(\Delta K)^n \). According to the American Standard of Testing Materials (ASTM), C and n are material constants and can be determined by implementing fatigue crack growth tests. \( \Delta K \) is the stress intensity factor range \( (K_{max} - K_{min}) = Y\sigma\sqrt{a} \) (ksi - MPa), a is the crack thickness, Y is a geometry factor and N is the Number of cycles.

\[
\frac{da}{dN} = C(\Delta K)^n
\] (2-5)

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 described as a single parameter, K, may be viewed as the intensity of 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. Region I is the fatigue threshold where the \( \Delta K \) is too low to propagate a crack. Region II encompasses data where the rate of crack growth changes roughly linearly with a change in stress intensity fluctuation. In region III, small increases in the stress intensity amplitude produce relatively large increases in crack growth rate since the material is nearing the point of unstable fracture. Figure 2-2 shows the Paris law regions.
The Paris law was not readily acknowledged as the $\Delta K$ parameter is based on an elastic stress field in the crack front, 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”
2.2.5 Rainflow Counting

The Rainflow method is one of the most popular and probably the best method of cycle counting. The rainflow method is used for counting fatigue cycles from a time history. The fatigue cycles are stress-reversals. The rain flow method allows the application of Miner’s rule in order to assess the fatigue life of a structure subjected to complex loading. Luminaire structures are subjected to variable wind loads with time. As a result, the stress histories of the luminaire structures have variable amplitudes. Variable amplitude fatigue analysis is similar to a constant amplitude fatigue analysis with the addition of cycle counting and damage summation. In a variable amplitude loading history, equivalent constant amplitude cycles must be identified. This process referred to rainflow counting. A few of the methods that are used for counting stress ranges include the reservoir method, the rainflow method, the peak count, and the mean-crossing peak count method (Bannantine et. al., 1990). Rainflow algorithm counts a history of peaks and valleys of stress time history in sequence which has been rearranged to begin and end with the maximum peak or minimum valley (Downing and Socie (1982)). The AASHTO LRFD Bridge Design Specification (AASHTO, 2004) stipulates that the Variable amplitude, Fatigue limit (VAFL) is to be taken as one-half of the CAFL. Determining fatigue life under variable stress cycles continues the sloping straight line in the S-N curve down to 50% of the CAFL. The VAFL is defined as 50% of the CAFL. All the stress cycles below the VAFL provide no fatigue damage, while all the stress cycles above the VAFL cause fatigue damage, which includes the impact of variable amplitude on the fatigue life. Use of a simple straight-line S-N curve without a fatigue limit assumes that the fatigue strength curve continues below the CAFL with the same constant slope of 3. It means that
there is no fatigue limit existing and all stress cycles contribute to fatigue damage. The straight-line method will produce more conservative fatigue lives than the VAFL method.

2.3 Types of wind load for Fatigue on sign structures

As stated in NCHRP Report 412, four types of loading are considered in the assessment of fatigue damage on signal, sign, and luminaire structures: a) Galloping, b) Vortex Shedding, c) Natural Wind Gusts and d) Truck Induced Gusts (Kaczinski et al., 1998). Galloping and Vortex Shedding can induce nearly constant-amplitude vibrations at a natural frequency and may lead to fatigue failures in a short period of time. Natural wind gusts and truck induced gusts produce along-wind vibrations that may produce accumulated fatigue damage at critical connection details over the life of the structure. In some extreme events, wind gusts may also result in excitation near the natural frequency of these structures.

*Galloping* is an unstable aerodynamic damping forces which are created due to structural vibration-induced variations in the angle of attack of the wind flow (Simiu and Scanlan, 1986). Moreover, galloping-induced fluctuations primarily occur in flexible, lightly damped structures with non-symmetrical cross-sections. Most of cantilevered luminaire structures have circular cross sections. The symmetry of a circular cylinder causes development of a pure drag force when subjected to a periodically varying angle of attack of the wind flow. Therefore, the aerodynamic force is always oriented opposite to the direction of wind motion. As a result, circular cylinders are not susceptible to the galloping failure.
**Vortex shedding** is defined as a steady uniform flows which produce resonant fluctuations in a plane normal to the direction of flow. Vortex shedding is caused by the shedding of vortices in the wake of the structural element. Vortices which are shed in the wake behind the element in an alternating pattern referred to as a von Karman vortex street (Figure 2.3). The frequency, at which vortices are shed from element, $f_s$ is given by the Strouhal relation:

$$f_s = \frac{SV}{D} \quad (2-6)$$

Where,

$S =$ Strouhal number

$D =$ the across wind dimension of the element

$V =$ Free stream wind velocity

If the frequency of vortex shedding does not match on the natural frequencies of the luminaire structures, the effect of vortex shedding maybe ignored. However, when the frequency of vortex shedding reaches one of the natural frequencies of the structure, significant stress ranges and displacement ranges can result. The result is a tendency for the vortex shedding frequency to become coupled to the natural frequency of the structure. This phenomenon referred to lock-in and the critical wind velocity is given by Strouhal relation as follow:

$$V_{cr} = \frac{f_s D}{S} \quad (2-7)$$

Where,
\( f_n = \) Natural frequency of the structure

\( V_{cr} = \) Critical wind velocity

D = the across wind dimension of the element

Natural wind gusts arise from natural variability in the velocity and direction of air flow. Natural wind gusts are characterized by a spectrum of fluctuating velocity components which oscillate over a board range of frequencies as a result of turbulence inherently present in any natural air flow. The variable stress ranges in the structural elements of the luminaire structure are caused by random vibrations, will produce fatigue damage in the long term. For ultimate strength design, one of the best approaches for predicting the maximum pressure (load) imposed on a structure by a gust is through use of the gust factor. A gust factor is defined as the ratio of the expected peak displacement

Figure 2-3, Von Karman Vortex shedding Phenomenon
(load) during a specified period to the mean displacement (load). Research related to wind gust indicates that the intensity of turbulence of wind is typically, between 5% to 25% (Davenport, A.G., et al.). The AASHTO specifications conservatively use 30% intensity of turbulence in the design of luminaire structures. Since the pressure imposed is proportional to the square of velocity, the actual gust factor is equal to the square of 1.3, or 1.69. This factor has been used by specifications since 1959.

**Truck-Induced Wind Gusts** are produced by the passage of trucks beneath sign structures and act on the front area facing the oncoming traffic and underside of the members of the sign structures. It is more likely to be critical for sign structures with large frontal areas parallel to the ground, e.g. the considerable width associated with variable message sign structures. Cantilever sign structures, on the other hand, have little dimension parallel to the ground and should not sustain large truck-induced wind loads.

### 2.4 Research conducted on fatigue failures in luminarie structure

In this section, most of the significant studies of fatigue failures in the area of aluminum and steel luminarie structures are reviewed. Numerous studies and tests have been carried out on sign, signal and luminaire structures but the objective of the studies differed depending on the individual scope of the projects.

**2.4.1 Creamer (1979), Fatigue Loading of Cantilever Sign Structure (1979)**

This research was carried out to develop a method for checking the fatigue of cantilever highway sign structures for truck-induced gust loading. It was concluded that truck-induced gusts can produce significant sign response and a large number of stress
fluctuations. Moreover, a peak pressure of 1.23 psf with period of 0.375 sec is
noteworthy characteristics of truck-induced pressure plusses (Creamer, 1979).

2.4.2 Fisher and colleagues (1981), Steel Through Plate Socket Connection Tests

Fisher, et al was the first group which studied fatigue behavior of standard light poles. Fatigue tests were conducted 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 steel built having unequal fillet weld legs was evaluated as well. Two additional specimens from Ameron were fabricated separately for comparison purposes. Poles built by Valmont were designated as V1-V6, poles build by Ameron were designated as A1-A6 and the two additional specimens were designated as A7 and A8. Fatigue tests confirmed that poles with equal 45° fillet weld legs had a fatigue strength less than Category E’. However, poles with unequal fillet weld legs had fatigue strength equal to Category E. 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. It was observed that when poles were galvanized, small fatigue cracks were hard to identify. The galvanized coating allows large cracks to form before the coating breaks. As a result, most of the fatigue life was exhausted before cracks were identified. In this project the issue of base flexibility was also related to the fatigue results (Fisher et.al 1981).

2.4.3 Gilani (1997), Fatigue Life Evaluation of Steel Post Structure

This paper summarizes a research effort conducted for the California Department of Transportation (CALTRANS) after the collapse of a Changeable Message Sign (CMS)
structure. The research included both field and analytical studies. Field measurements were undertaken to quantify the natural frequencies of vibration and structural characteristics. The analytical study was carried out using FE analysis to estimate 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 CMS structures. This report also briefly evaluated the wind loading experienced by CMS 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).

2.4.4 Kaczinski (1998), NCHRP Report 412

Many structural engineers believed that the design guidance in AASHTO should be improved with respect to fatigue. NCHRP project 412 was proposed to develop guideline for fatigue design of sign, signal and luminarie structures. The magnitude of stress ranges in many cases is critical and causes fatigue cracks. Among the 36 states which responded to a survey, approximately 50% had problems with wind-induced vibration of cantilevered support structures. The report indicates that truck and natural wind-induced vibrations were 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 on full scale cantilever sign structures. These studies indicate that cantilever structures with signal attachments are susceptible to galloping and become worse when back plates are added to the signal attachments. Moreover, vortex shedding does not need to be taken in to
account for wind velocities less than 5 mph. Finite Element Analyses were conducted to calculate wind pressures on cantilever support structures during vortex shedding, galloping, natural wind and truck induced vibrations for both static and dynamic approaches.

It was recommended 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)

2.4.5 Azzam (2006), Fatigue Behavior of Aluminum Light Pole Structures

This research was conducted to develop fatigue study on full-scale welded aluminum light pole structures. 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 lower bound resistance suitable for design in terms of S-N curves. Fatigue tests revealed relatively low strengths for the through plate socket connection as compared to the shoe base details. Parametric studies revealed that residual stresses play an important role in the behavior of light pole structures. Moreover, attempts to stiffen through plate socket connections using triangular plate stiffeners resulted in longitudinal stresses at the tip of the stiffeners for short stiffeners that contradict AASHTO specification fatigue categories for light pole structures. (Azzam, 2006)
CHAPTER III

FINITE ELEMENT MODELING

3.1 Finite Element (FE) Study-Introduction

Finite element models for several parametric studies were constructed and analyzed in order to gain insight into the behavior of the light pole structures during wind loads (specifically, wind gust). Parametric studies were conducted to increase understanding of the important geometric features and their influence on local stresses that derive the fatigue response specifically; the hand-hole reinforcement width, hand-hole geometry and shoe base connection were examined. ANSYS Workbench was used to complete all of the analysis (ANSYS, 2012).

3.2 Geometry

The light pole geometry was created in Design Modeler (DM) in three dimensions (3-D) has all details from measurements and engineering drawings. A box has been added around the model to include the environment around the pole. Upstream of the pole has a dimension of 15R (R is the radius of the pole). Downstream has been expanded to 60R. On sides and top the distance of 15R was considered (Anderson, J.D, 2006).

The pole body has been subtracted from the fluid (air) region in this part. The geometry has one inlet and outlet and three symmetric walls on the sides and one wall at the bottom. Symmetric walls generate no friction on the air (fluid) as it has zero gradients for
velocity and other parameters normal to its plane. As such, there is no need to model an infinite surrounding. Figure 3-1 shows the geometry and Figures 3-2 and 3-3 show it in top and side view. The geometry is sliced with different cut planes to have additional control on mesh size. A finer mesh is applied close to the pole to capture velocity and pressure gradients more exactly.

Figure 3-1, Fluid geometry used for Finite Volume Method
Figure 3-2, Top view of the geometry for CFD (Computational Fluid Dynamics)

Figure 3-3, Side view of the pole for CFD (Computational Fluid Dynamics)
3.3 Meshing

The flow domain shown in Figures 3-1 and 3-2 is meshed for finite volume calculations conducted via the ANSYS Fluent code. There are different methods to generate a mesh in ANSYS meshing. Here, the mesh is generated by using assembly meshing (cutcell method). Assembly meshing or cutcell generates a cartesian mesh so the domain is meshed by hexahedral elements. Inflation has been added to cover the walls of the pole. Inflation helps to capture the velocity gradients and pressure distribution close to the pole with more resolution. Mesh orthogonal quality is always above 0.08 to avoid convergence issues in solution. Mesh is shown in Figure 3-4 and Figure 3-5 (in 2-D and 3-D) with a cut plane which passes through the pole.

Figure 3-4, CFD Mesh on a cut plane (slice)
Figure 3-5, CFD Mesh on a cut plane (slice)

Figure 3-6, a sample applied mesh sizing
3.4 Flow Field Calculation

In this section, we first describe the Navier-Stokes flow equations and the numerical scheme for solving these equations using the fluid dynamics code from ANSYS Fluent. We also present our mesh-independence study.

3.4.1 Governing Equations

A steady state, turbulent, incompressible model has been adopted for the flow regime inside the domain. The finite volume method (Patankar 1980) implemented in The Fluent code is used to solve continuity and conservation momentum. Since it would be computationally prohibitive to model the often very small scale and high frequency fluctuations in fluid velocity seen in turbulent flow in production environments, time averaged methods of simulating turbulence effects have been derived. In these models, terms are introduced to simulate the average turbulent flow field, so that the small scale turbulent behavior does not have to be solved explicitly by the Navier-Stokes equations. Instead, transport equations are solved in order to bring the model to closure and model the full range of turbulent flow scales. In the case of the $k - \varepsilon$ turbulence model, the values of $k$, the turbulent kinetic energy, and $\varepsilon$ epsilon, the turbulence dissipation rate must be determined. These so-called Reynolds-average Navier-Stokes (RANS) turbulence models significantly reduce the processing power required and make turbulence modeling practical for a wide range of turbulent flow problems. In turbulence models that involve Reynolds averaging, the exact Navier-Stokes flow solution is broken down into its varying and time averaged components.
The general form of any scalar flow property is shown in Equation 3-1.

\[
\phi = \bar{\phi} + \phi'
\]  

(3-1)

where \( \bar{\phi} \) is the mean property value, and \( \phi \) is the varying value. If the time averaged values of the flow variables are substituted into the standard Navier-Stokes equations, the Reynolds-averaged Navier-Stokes equations can be obtained:

\[
\frac{\partial \rho}{\partial t} + \frac{\partial}{\partial x_i} (\rho u_i) = 0
\]  

(3-2)

\[
\frac{\partial}{\partial t} (\rho u_i) + \frac{\partial}{\partial x_j} (\rho u_i u_j) = - \frac{\partial p}{\partial x_i} + \frac{\partial}{\partial x_j} \left[ \mu \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} - \frac{2}{3} \frac{\partial}{\partial x_i} \delta_{ij} \right) \right] + \frac{\partial}{\partial x_j} (- \rho u'_i u'_j)
\]  

(3-3)

The two-equation RANS turbulence models utilize the Boussinesq hypothesis which says that the transfer of momentum caused by turbulent eddies can be modeled by an eddy viscosity, \( \mu_t \). The theory states that the Reynolds stress tensor, \( \tau_{ij} \), is proportional to the rate of strain tensor, \( \bar{S}_{ij} \), defined as

\[
\bar{S}_{ij} = \frac{1}{2} \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right)
\]  

(3-4)

They can be combined as

\[
\tau_{ij} = 2 \mu_t 2 \bar{S}_{ij} = - \frac{2}{3} k \delta_{ij}
\]  

(3-5)

This can also be written as

\[
- \rho u'_i u'_j = \mu \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) - \frac{2}{3} \left( \rho k + \mu \frac{\partial u_k}{\partial x_k} \right) \delta_{ij}
\]  

(3-6)

The Boussinesq assumption provides a method of calculating the turbulent viscosity which comes at a low computational cost. However, its main disadvantage is assuming that the
Reynolds stress tensor is proportional to the strain rate tensor. This is not strictly true and is in fact invalid for a range of flows including those with strong accelerations or high curvature (Fluent theory guide).

The standard k-epsilon model is a simple yet robust two-equation turbulence model that has been utilized widely in industry for practical flow analyses over the years.

\[
\frac{\partial}{\partial t} (\rho k) + \frac{\partial}{\partial x_i} (\rho k u_i) = \frac{\partial}{\partial x_j} \left[ \mu + \frac{\mu_t}{\sigma_k} \right] \frac{\partial k}{\partial x_j} + G_k + G_b - \rho \varepsilon - Y_{st} + S_k
\]

\[
\frac{\partial}{\partial t} (\rho \varepsilon) + \frac{\partial}{\partial x_i} (\rho \varepsilon u_i) = \frac{\partial}{\partial x_j} \left[ \mu + \frac{\mu_t}{\sigma_{\varepsilon}} \right] \frac{\partial \varepsilon}{\partial x_j} + C_{\varepsilon} \frac{\varepsilon}{k} (G_k + C_{\varepsilon} G_b) - C_{\varepsilon} \rho \frac{\varepsilon^2}{k} + S_{\varepsilon} \tag{3-7}
\]

The turbulent viscosity is calculated as

\[
\mu_t = \rho C_{\mu} \frac{k^2}{\varepsilon} \tag{3-8}
\]

In the case of the standard k-epsilon model, \( \mu_t \) is a constant. The term \( G_k \), which is the production of turbulent kinetic energy can be defined as

\[
G_k = -\rho u'_{i} u'_{j} \frac{\partial u_j}{\partial x_i} \tag{3-9}
\]

or,

\[
G_k = \mu_t S^2, \tag{3-10}
\]

where \( S \) is the modulus of the mean strain rate tensor

\[
S = \sqrt{2S_{ij}S_{ij}} \tag{3-11}
\]
Re (Reynolds number) is calculated to predict the flow regime \( \rho = 1.225 \frac{kg}{m^3}, V = \frac{50 m}{s} \), \( D = 0.1016 m, \mu = 1.7894e - 5 \ Pa.s \)

\[
Re = \frac{\rho V D}{\mu} = 3.47e5
\]

The flow around the streamlined airfoil remains attached, producing no boundary layer separation and comparatively small pressure drag. However, the flow around the less aerodynamic circular cylinder separates, resulting in a region of high surface pressure on the front side and low surface pressure on the back side and thus significant pressure drag.

Consider an element of the cylinder surface of thickness \( ds = rd \). The force per unit span on the element due to a pressure normal to the element is

\[
df = prd
\]  \hspace{1cm} (3-12)

The drag component of this force is the component acting in the direction of the free-stream velocity

\[
df = pr \cos \theta \ d
\]  \hspace{1cm} (3-13)

The integral of this around the cylinder circumference gives the total drag on the cylinder per unit span \( d \).

\[
F_{d, \text{pressure}} = \text{Drag Force due to pressure} = \int_0^{2\pi} p r \cos \theta \ d\theta
\]  \hspace{1cm} (3-14)
Pressure drag is drag due to the integrated surface pressure distribution over the body. Therefore, in general, the total drag coefficient of a body can be expressed as

$$C_D = C_{D,\text{pressure}} + C_{D,\text{friction}}$$ \hspace{1cm} (3-15)

$$C_D = \frac{F_{D,\text{total}}/A}{\frac{1}{2} \rho U_x^2} = \frac{F_{D,\text{pressure}}/A}{\frac{1}{2} \rho U_x^2} + \frac{F_{D,\text{friction}}/A}{\frac{1}{2} \rho U_x^2}$$ \hspace{1cm} (3-16)

Which factor, pressure or friction drag, dominates depends largely on the aerodynamics (streamlining) of the shape and to a lesser extent on the flow conditions (Fluent theory guide).

Typically, the most important factor in the magnitude and significance of pressure or form drag is the boundary layer separation and resulting low pressure wake region associated with flow around non-aerodynamic shapes.
3.5 Workflow in Workbench

Workbench is relatively new and may be used to connect different solvers. The Computational Fluid Dynamics (CFD) solver in ANSYS is based on a Finite Volume Method. The used workflow is shown here.

Block A in Figure 3-8 is the geometry block in Design Modeler. This geometry was discussed in Figure 3-1. It contains a flow domain around the pole. Block B contains mesh information which was discussed in Figures 3-3 and 3-4. By using another wire, information of Block B was passed to Block C which is the CFD solver. After solving Fluid Flow the pressure distribution on walls of pole is transferred as an input data to Block D which is the FEA solver and called ANSYS Structural. Please notice that the mesh in CFD and Mechanical can be totally different. Behind the scene ANSYS data mapping takes care of only differences. It does an interpolation between the data from the CFD solver to the FEA solver. The geometry of the pole for FEA analysis is considered in Block E. This geometry will be discussed in the next paragraph. The information associated with this geometry is passed to Static Structural for solution. Meshing and other settings for FEA solver are in D4 which is called Model.
3.6 Geometry and Mesh for FEA Solver

The geometry is modeled based on measurements of parts and engineering drawings. The geometry is shown in Figure 3-9 shows slices which used for mesh generation. By sliding the geometry, it is divided into sweepable bodies which can have a hexahedral mesh and reduce number of mesh elements. This is important as ANSYS Mechanical does not scale well (ANSYS-FLUENT is totally scalable and you can use as many CPUs as you needed to improve the speed and reduce the turnaround of simulation). The geometry is shown in Figure 3-9.
The mesh generated for FEA analysis is shown in Figure 3-10. As is obvious, it has two parts, including a tetrahedral mesh as it is difficult to use a hexahedral mesh for these regions.
Figure 3-10, FEA mesh generated in ANSYS Mechanical
3.7 CFD Results

The imported load from Fluent will be used in FEA analysis. The CFD results are shown in this section. In Figure 3-11, velocity contours on y-plane are shown at a 1m distance from the pole bottom.

![Figure 3-11, Velocity Contours in 3-D](image)

After reviewing this 3-D view, specific results will be focused on 2-D planar views. First, above velocity contour is shown in 2-D.
Figure 3-12, Velocity Contours in 2-D

By showing velocity vectors behind the cylinder, the wake is obvious. This is shown in Figure 3-13.

Figure 3-13, Velocity Vectors on y-plane slice at 1m distance from bottom
Figure 3-14, Velocity contours on a constant z-plane which passes through pole (z=0.15 m)

Figure 3-15, Gauge Pressure Contours on a top view in Pascal
Figure 3-16, Gauge Pressure contours on z=0.15 m

In order to have a better image of these contours on a plane z=0.15 m. The following figures show contours by zooming into region around bottom part of the pole. Figure 3-17 shows velocity contours on z=0.15 m and Figure 3-18 shows the velocity vectors for this slice plane.

Figure 3-17, Velocity contours for region around the bottom of the pole
Figure 3-18, Velocity vectors for region around the bottom of the pole
3.8 Parametric Schemes

In this study, three different parametric schemes are utilized Case (1) involves the applications of different tube widths and investigating fatigue results for the location of shoe-base-connection; Case (2) involves the application of different hand-hole widths; and Case (3) examine different hand-hole widths. In Case (2) and (3) fatigue results are studied around hand-hole-connections. Finally, details of the geometry parameters for each of the cases are shown in Table 3-1).

<table>
<thead>
<tr>
<th>Pole Thickness (in) Case (1)</th>
<th>Hand-hole width (in) Case (2)</th>
<th>Hand-hole Thickness (in) Case (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.188</td>
<td>0.1</td>
<td>1</td>
</tr>
<tr>
<td>0.25</td>
<td>0.5</td>
<td>1.5</td>
</tr>
<tr>
<td>0.3125</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>0.375</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.8.1 Total Deformation

Total Deformation of light pole structures under wind loads were calculated through ANSYS Workbench. The results indicate that by increasing tube thickness, the total deformation will be reduced in case (1). Total deformation for each of the models is shown in Figure 3-19. Maximum total deformation is equal to 68.9 mm and occurs in pole with thickness of 0.188 in. The minimum total deformation is equal to 37.4 mm and occurs in pole with a thickness of 1 in, as expected.
3.8.2 Equivalent Elastic Strain

Equivalent elastic strain was determined for all of the three cases studied through ANSYS Workbench. The focus will be in critical locations such as the hand-hole and shoe base. Both the hand-hole and base locations are considered for case (1). However, for two other cases only the hand-hole location is considered. Figure 3-20 shows the maximum elastic strain was reduced around the hand-hole and base when the tube width increased.
Figure 3-20, Equivalent Elastic Strain for different pole thicknesses, case (1)
3.8.3 Equivalent Stress

A significant spatial variation in the effective stress distribution for different cases can be clearly observed in Figures 3-21, 3-22, 3-23. For different pole thicknesses (Case1), the equivalent stresses are reduced by increasing pole thickness. In Case (2), by reducing hand-hole width to 0.1 in, the amount of stresses increases and reaches to 107.7 Mpa around the hand-hole. Moreover, for hand-hole widths of 0.5 in and 1 in, the maximum equivalent stress is equal to 46.8 Mpa and 48.5 Mpa respectively. It is also important to note that the area in which maximum stresses occur around hand-hole with width of 1 in is larger than hand-hole with width of 0.5 in. As a result a hand-hole width of 0.5 in is recommended in this study.

Case (3) indicates that, the equivalent stress is not dependent on hand-hole thicknesses because in all three cases, the maximum equivalent stresses have minor different compare to each other.

Finally, the maximum stresses with respect to pole thicknesses, hand-hole widths and hand-hole lengths are shown in Figures 3-24, 3-25 and 3-26 respectively.
Figure 3-21, Equivalent Stress for different pole thicknesses, case (1)
Figure 3-22, Equivalent Stress for different pole widths, case (2)

Figure 3-23, Equivalent Stress for different pole thicknesses, case (3)
Figure 3-24, Maximum stresses for different pole thicknesses

Figure 3-25, Maximum stresses for different hand-hole widths
Figure 3-26, Maximum stresses for different hand-hole Thicknesses
CHAPTER IV
FATIGUE

4.1 Fatigue Overview

Fatigue is a cumulative damage process caused by the repeated loading. Fatigue may result from the repeated application of stress levels much lower than the tensile strength of a given material. Fatigue is typically divided into two categories;

1) Low-cycle fatigue
2) High-cycle fatigue

*High-cycle fatigue* occurs when the number of cycles (repetition) of the load is relatively large (e.g., $1 \times 10^4$ – $1 \times 10^9$). As a result, the stresses are usually low compared with the material’s strength. Stress-Life approaches are used for high cycle fatigue and cyclic strains occur mostly in the elastic range. These are global stresses. Locally, stresses to cause damage are plastic.

*Low-Cycle fatigue* occurs when the number of cycles of the load is relatively low. Plastic deformation often accompanies low-cycle fatigue, which explains the relatively short fatigue life ($N < 10000$). Strain-Life approaches are best suited for low-cycle fatigue evaluation. The focus of this study is high-cycle fatigue or Stress-Life. In the vast majority of cases, light poles and similar structures are designed for infinite life. In this Chapter, Fatigue behavior of light poles under three-second wind gusts will be studied.
4.2 Types of Cyclic Loadings

Static stresses are analyzed using calculations for a single stress state. However, fatigue damage occurs when stress at a point changes over time. There are essentially four classes of fatigue loading. These classes are described as follow;

1) Constant Amplitude, Proportional Loading
2) Constant Amplitude, Non-Proportional Loading
3) Variable Amplitude, Proportional Loading
4) Variable Amplitude, Non-Proportional Loading

When minimum and maximum stress levels are constant, constant amplitude loading results otherwise, the loading is known as a variable amplitude or non-constant amplitude. The second identifier, proportionality, describes whether the changing load causes principal stress axes to change. If the principal stress axes do not change, then the loading is proportional. If the principal stress axes change, then the cycle cannot be continued simply and it is non-proportional loading. Each Stress-Life and Strain-Life approach includes variety of loadings. Stress-Life includes; a) Constant Amplitude, Proportional Loading, b) Variable Amplitude, Proportional Loading and c) Constant Amplitude, Non Proportional Loading. However, The Strain-Life approach only contains Constant Amplitude, Proportional Loading. Each type of loading describes shortly as follow:

*Constant Amplitude, Proportional Loading* is constant amplitude because only one set of finite element stress results along with the loading ratio is required to calculate alternating stress and mean values. The loading ratio is defined as the ratio of minimum
to maximum load. Common types of constant amplitude loading are fully reversed (apply a load, then apply an equal and opposite load for a load ratio of -1) and zero based (apply a load and then remove it; the load ratio is zero). Therefore, at a single set of finite element results can determine critical fatigue locations. For a better understanding, constant amplitude load fully reversed and zero based types are shown in Figure 4-1 and Figure 4-2 respectively.

![Figure 4-1, Constant Amplitude Load Fully Reversed](image1)

Constant Amplitude Load
Fully Reversed

![Figure 4-2, Constant Amplitude Zero-Based](image2)

Constant Amplitude Load
Zero-Based

*Constant Amplitude, Non-Proportional Loading*, implies that the loading is of constant amplitude but the principal stress and strain axes can change between two load sets. The critical location may occur in a location that is not easily identifiable. Common types of non-proportional loadings can describe as follow;
a) Alternating between two different load cases such as bending and torsional loads

b) An alternating load superimposed on a static load

c) Nonlinear boundary conditions

Variable Amplitude, Proportional Loading; In this case, the load ratio changes over time and it requires one set of finite element results. Fatigue critical locations may be determined by looking at a single set of finite element result. However, the damage accumulation caused by fatigue loading may not easily be determined. Total damage may be calculated through cumulative damage rules such as Miner’s rule and Rainflow counting. Cycle counting is used to simplify complicated load histories to a number of constant amplitude test data. Results from cycle counting are commonly summarized in a histogram format. The ANSYS Fatigue Module uses a “quick counting” technique to reduce runtime and memory.

Variable Amplitude, Non-Proportional Loading, in this type of loading the variety of stress cases included has no relationship to one another. Fatigue critical locations are easily determined life is not determined. In addition, the combination of loads which caused damage is not obvious. As a result, a multiaxial critical plane method and advanced cycle counting techniques are required.

4.3 Types of Results

Both Stress-Life and Strain-Life are considered for all cases. In all cases, the Stress-Life approach is better suited for evaluation of luminaire structures. Therefore, only Stress-life
results are discussed. Types of Fatigue results conducted in this research are listed as below:

a) Fatigue Life
b) Fatigue Damage
c) Fatigue Safety Factor
d) Stress biaxiality
e) Fatigue Sensitivity

*Fatigue Life* is defined as the number of stress cycles which an element under cyclic loadings can resist before failure. Comparative results are shown using ANSYS Workbench for different cases. The resulting contour plots shows the available life for a given fatigue analysis. If loading is constant amplitude, the results provide the number of cycles. However, if loading is variable amplitude, results describes the number of loading blocks until failure.

*Fatigue Damage* is defined as the design life divided by the available life. For fatigue damage, values greater than one indicate failure before the design life is obtained.

*Fatigue Safety* is a factor of safety with respect to a fatigue failure at a given design life. The maximum factor of safety is equal to 15 and values less than one describes failure before the design life.

*Biaxiality* is defined as the smaller principal (magnitude) divided by the larger principal stress with the principal stress nearest to zero ignored. A biaxiality of zero
refers to uniaxial stress, a value of 1 refers to a pure biaxial state and a value of -1 refers to pure shear.

*Fatigue Sensitivity* describes how the fatigue results vary as a function of loading at the critical location in the model. Sensitivity may be calculated for life, damage or the safety factor. The life sensitivity for all of the cases is investigated by changing the load percentage. A value of 100% corresponds to the life at the current loading.

4.4 Parametric Study of Fatigue

A parametric study was conducted on luminaire structures by using the commercially available finite element code (ANSYS Workbench 14.5). This study provides insight into the nature of fatigue critical areas, around the weld toe adjacent to both the hand-hole and base. In addition, the influence of geometry changes on the local stress fields, fatigue life, damage, factor of safety and fatigue sensitivity is studied. These Models dimensions take from HAPCO, a well known firm specializing in manufacture of light poles. A light pole structure shown in Figure 4-3 is used as the basis for finite element analyses.
4.5 Fatigue Behavior of Light Pole Structures

In the following subsections, the fatigue behavior of light pole structures subjected to 3-sec wind gusts will be studied with particular emphasis on two critical locations. In particular, emphasis will be placed on hand-hole and shoe-base connections.

4.5.1 Fatigue Life

Fatigue life analyses are conducted for luminaire structures with different pole widths, case (1). Case (2) & (3) examine the influence of reinforcement width and thickness of the hand-hole on fatigue life.

Significant variations in fatigue life can be clearly observed. In case (1), by changing the pole thickness from 0.188 in to 1 in, the improvement in fatigue life is
nearly a factor of 12300 \( \left( \frac{1 \times 10^9}{80833} \right) \). It is observed that, the minimum fatigue life occurs around the shoe base to pole connection. Therefore, increasing pole thickness has a positive influence in increasing fatigue life of light pole structures. Fatigue life around the base is shown in Figure 4-4.

The next step is changing the hand-hole reinforcement widths. In case (2), three different hand-hole widths are modeled and fatigue life. The hand-hole widths modeled include 0.1 in, 0.5 in and 1 in. Figure 4-5 shows the fatigue life around the hand-hole. The results indicate the value of fatigue life becomes more critical as the hand-hole reinforcement width becomes thinner. Moreover, a butterfly stress pattern is observed in a hand-hole with a width equal to 0.1 in. Therefore, the thinnest hand-hole is not recommended.

Last but not least, fatigue life was due to changing hand-hole thicknesses was investigated (case (3)). The three considered different thicknesses (1 in, 1.5 in and 2 in). Fatigue life for each of these three models is shown in Figure 4-6. It is observed that the hand-hole with the largest thickness has improved fatigue life as compared with other models. As a result, hand-holes with the largest thicknesses are suggested.
Figure 4-4, Fatigue life response for different pole thicknesses, case (1)

Figure 4-5, Fatigue life response for different Hand-hole Widths, Case (2)
4.5.2 Fatigue Damage

Fatigue damage analyses are conducted in all three cases. In case (1), damage will decrease by increasing pole thickness. The contours beside each figure indicate that damage occurs in locations in which the damage contour number is greater than one. Critical locations are taken place around the pole-to-shoe base connection. For additional details please see Figure 4-7. A damage analysis for case (2) is shown in Figure 4-8. The maximum critical damage occurs in a hand-hole with a width equal to 0.1 in. However, the damage which occurs in a hand-hole with a width equal to 1 in includes a larger area around hand-hole. As a result, the hand-hole with a width equal to 0.5 in is the most beneficial model in case (2).

Case (3) describes fatigue damage with respect to the hand-hole thickness. Three different hand-hole thicknesses were considered for this case. The results indicate that increasing the hand-hole thickness from 1 in to 2 in, decreases the amount of damage
around hand-hole. In addition, fatigue damage will affect a smaller area around hand-hole. Figure 4-9 is shows the fatigue damage for case (3).

Figure 4-7, Fatigue damage for different pole thicknesses, case (1)

Figure 4-8, Fatigue damage for different Hand-hole widths, Case (2)
4.5.3 Safety Factor

Factor of Safety analyses were carried out in the finite element models. Factor of Safety contours differ from 0 to 15. Contour numbers less than one indicate failures before the design life. A safety factor as low as 1.2 may be used when using the more conservative AWS fatigue category data. In either case, a calculated factor of safety less than 1.0 indicates potential fatigue issues.

The parametric study results for the factor of safety analysis in case (1) is shown in Figure 4-10. A factor of safety equal to 1.4 or 1.6 is recommended for design. As Figure 4-10 shows, only poles with thicknesses equal or bigger than 0.375 in, provide an acceptable safety factor for 3-sec wind-gust loading.

Although the factor of safety is not large enough in any of the models in case (2), the hand-hole with a width equal to 0.5 in is recommended. A safety factor for each model in case (2) is shown in Figure 4-11.
Finally, the effect of changing the hand-hole thicknesses on safety factor is studied in case (3). Safety factor for all of the models are shown in Figure 4-12. It is observed that, by increasing hand-hole thickness, the factor of safety will increase.
Figure 4-10, Safety factor for different pole thickness, Case (1)
Figure 4-11, Safety factor for different Hand-hole widths, Case (2)

Figure 4-12, Safety factor for different Hand-hole thicknesses, Case (3)
4.5.4 Biaxiality Indication

The biaxiality indication is defined as the ratio of the smaller to larger principal stress (with principal stress nearest to 0 ignored). Hence, locations of uniaxial stresses report a value of 0; pure shear report a value of -1, and biaxial stresses reports a value of 1. Usually fatigue test data is reflective of a test specimen under uniaxial stress. The biaxiality indication helps to determine if a location of interest is in a stress state similar to testing condition.

It is observed that, almost the entire model is under uniaxial stress except the details: hand-hole and shoe base. In addition, by increasing the pole thickness, the effect of pure shear is reduced in the shoe-base. The biaxiality indication for case (1) is shown in Figure 4-13. The biaxiality for the hand-hole in case (1) is similar to that of all the models, because we do not change hand-hole geometry.

In case (2), the biaxiality indication study is examined around the hand-hole. The results indicate that, a hand-hole with width equal to 0.5 in is more suitable for design. The biaxiality indication for case (2) is shown in Figure 4-14.

Lastly, a biaxiality study of hand-hole thicknesses in case (3) shows that; the hand-hole with the largest thickness is under influence of pure shear as compared to hand-holes with smaller thicknesses. Figure 4-15 is shown the result from case (3).
Figure 4-13, Biaxiality indication for different pole thicknesses, Case (1)
4.5.5 Fatigue Sensitivity

A fatigue sensitivity chart displays how life, damage or the safety factor at critical locations varies with respect to load. In case (1) the critical location is around the shoe-base and the other two cases critical locations are around the hand-hole. Figure 4-16, 4-17 and 4-18 presents the fatigue sensitivity charts for cases (1), (2) and (3) respectively.
Figure 4-16, Fatigue Life sensitivity with respect to load for different pole thicknesses, Case (1)
Figure 4-17, Fatigue Life sensitivity with respect to load for different hand-hole widths, Case (2)
Figure 4-18, Fatigue Life sensitivity with respect to load for different hand-hole thicknesses, Case (3)
CHAPTER V
CONCLUSIONS

Failures caused by fatigue cracking often occur around welded structural details, some welded details include the light pole support base and hand-hole. Many of these failures are caused by wind-induced vibration, resulting in various applied stress cycles at the weld toe. This report analyzed the application of wind forces and specifically wind-gusts at the weld toes of light pole structures. Predicting fatigue life, damage, stress and strain were the goals of this study. Based on this parametric study, the following results were obtained:

1) Based on parametric studies, the fatigue strength of light pole structures may be improved by increasing pole thicknesses. An increase in pole thickness from 0.188 in to 1 in decreased Total deformation and the maximum equivalent stress adjacent to the weld toe by at least 42% in all cases evaluated.

2) An increase in hand-hole reinforcement width did not significantly change the maximum equivalent stress in the hand-hole adjacent to the weld toe.

3) An investigation into the role of hand-hole reinforcement width (i.e., 0.1, 0.5 and 1 in) indicates that the maximum equivalent stress in the hand-hole with a width of 0.5 in has the smallest equivalent value. Total fatigue life around the shoe base connection may be enhanced by increasing pole thickness.
4) Total fatigue life around the hand-hole may be enhanced by increasing hand-hole reinforcement thickness. However, increasing hand-hole reinforcement width does not increase fatigue life.

5) Fatigue damage occurs around the shoe base, with pole thicknesses less than 0.375 in.

6) A hand-hole with reinforcement width of 0.5 in, exhibits higher predict fatigue life and lower fatigue damage as compared to 0.1 in and 1 in hand-hole width.

7) Poles with thicknesses less than 0.375 in are not recommended for this loading and geometry because safety factor is less than one.

8) This safety factor shows value less than one around the hand-hole, specifically for small widths.

9) An increase in hand-hole thickness may improve the safety factor around hand-hole. Hand-holes with width of 2 in exhibit better performance than 1 and 1.5 in hand-hole’s thicknesses.

10) Shear increases when the hand-hole width is increased.
RECOMMENDATIONS FOR FUTURE WORK

1) Full scale fatigue tests on Aluminum light pole structures should be done to validate FEM results.

2) Long term monitoring sign/signal structures to validate pressure and load recommendations for vortex, galloping, truck induced and wind load effects.

3) Unsteady analysis in ANSYS Fluent to get more accurate results in modeling wind-gusts.
REFERENCES


ANSYS Workbench (14.5)


