Vulnerability assessment of sign-support structures during transportation and in service

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Vulnerability assessment of sign-support structures during transportation and in service

by

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The student author, whose presentation of the scholarship herein was approved by the program of study committee, responsible for the content of this thesis. The Graduate College will ensure this thesis is globally accessible and will not permit alterations after a degree is conferred.

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DEDICATION

To my beloved parents, Hadi and Zahra Arabi, for making me who I am.
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ABSTRACT

Dynamic Message Signs (DMSs) are increasingly used in highways as an effective means to communicate time-sensitive information with motorists. To ensure their long-term performance, it is critical to ensure that the truss structures that hold them can resist not only extreme loading events, but also fatigue induced by service loads. For this purpose, a comprehensive study has been conducted on the fatigue performance of this important category of structures. The current study evaluates the fatigue performance of DMS-support structures during transportation under road-induced excitations as well as throughout their service life under loads induced by environmental stressors. In the first section, fatigue analysis of these structures during transportation under road-induced excitations is conducted. To investigate this, a comprehensive field test and numerical study are conducted. For short-term monitoring, one span of a sign-support structure is instrumented. Additionally, detailed finite element simulations are conducted to obtain an in-depth understanding of the potential modes of damage under the road-induced excitations. The outcome of this study is expected to determine the extent of fatigue of DMS-support structures during transportation. In the second part, vulnerability assessment of these structures is investigated under thermal loads. For that purpose, two DMS-support structures located in Iowa are selected for long-term monitoring. Moreover, detailed finite element simulations are conducted to obtain an in-depth understanding of the expected extent of damage. Based on the results obtained from this study, the DMS-support structures are found to demonstrate a great performance under thermal loads, which strongly supports the recent transition from aluminum to steel truss structures. Finally, fatigue performance of DMS-support structures under the combined effects of diurnal temperature changes and natural wind excitations is investigated. Field monitoring has been
paired with detailed FE simulations to understand the fatigue performance of DMS-support structures under multiple stressors. The current study is concluded with the investigation of wind directionality effects. The outcome of this study is expected to not only contribute to the long-term performance and safety of DMS-support structures, but also pave the way to implement similar multi-stressor perspectives for other transportation infrastructures in service.
First of all, I would like to thank my thesis adviser Dr. Behrouz Shafei. His office door was always open whenever I had any trouble or question about my research. His support, guidance and ideas significantly helped to accomplish this work. I would also like to acknowledge Dr. Brent M. Phares and Dr. Ming-Chen Hsu as my committee members and I am gratefully indebted to their valuable comments on this thesis. I would also thank the Iowa Department of Transportation (DOT), which have funded this study as well as its several engineers who have helped in various parts of the project.

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CHAPTER 1. INTRODUCTION

Overhead highway signs, luminary poles, and traffic signals play an important role in guiding the traffic and ensuring the public safety. Among them, the use of Dynamic Message Sign (DMS) has been rapidly increased, primarily because of advancements in the DMS technology and its unique capability to provide the motorists with live traffic information. DMSs are mounted on either a cantilever or bridge type truss structure, which is prefabricated elsewhere and shipped to the site. Considering that the truss structures offer an economic and dependable means of holding the DMS cabinet, their structural performance has become a subject of interest for those working on the design and maintenance of transportation infrastructures. Since hollow pipes and angle sections are commonly used in such structures, the DMS-support structures have a relatively small mass, high flexibility, and low damping ratio (in the range of 1%). The listed properties make this category of traffic structures particularly vulnerable to high-cycle, low-amplitude excitations [1]. Failure of DMS-support structures has been reported in several cases throughout the United States and beyond. The most common cause of failure is believed to be metal fatigue. Mast arm to column connection, column to base plate connection, and anchor bolts are among the critical parts of the support structures susceptible to failure [2–4]. In addition, joints and weldments are often the areas of concern, as they experience fatigue-induced damage. Cracks have been observed, especially in the joints within the toe and leg of fillet welds with the possibility of propagating into the main chords [5].

The current study evaluates the fatigue performance of DMS-support structures during transportation from the fabrication site to the location of installation under road-induced excitation, as well as throughout their service life under load induced by environmental stressors. In the first
section of this study, fatigue analysis of these structures during transportation under road-induced excitations is presented. To investigate this aspect, a comprehensive field test and numerical study are conducted. For the field test, one span of a four-chord, overhead sign-support structure is instrumented with strain gauges. Additionally, detailed finite-element (FE) simulations are conducted to obtain an in-depth understanding of the extent and potential modes of damage. The stresses that the truss structure experience during transportation directly depend on the vibrational excitations, which require an integrated model for simulating the road profile and the suspension system of the truck used to transport the prefabricated structure. To achieve this goal, a set of road profiles are generated based on ISO Specifications [6]. In the next step, a passive suspension system is modeled. This model transfers the excitations induced by the road roughness to the structure considering the mass, stiffness, and damping parameters. Using a Simulink model, the suspension-induced movements, which excite the truss structure at its supports, are calculated during transportation. The excitation time-history of the supports is employed as an input to the detailed FE model of the truss structure. This model, which has been developed using the ABAQUS package, is utilized to obtain the time-history of strains and stresses at various structural members and their joints. The FE model is validated first with the field data collected from the strain gauges during transportation. Stress time histories of the most critical members of the truss are then extracted from the FE model. Using the rainflow cycle counting method [7], the stress ranges and their corresponding cycles are derived for fatigue analysis. Based on the Miner’s linear fatigue damage accumulation rule, the fatigue-induced damage is calculated using field data and numerical simulation results. The outcome of this study contributes to determine the intensity of damage that the structure may experience during transportation and if this has a critical effect on the expected service life.
In the second section of the current study, the contribution of thermal stressors on the fatigue performance of DMS-support structures is presented. This section is particularly in response to the recent transition from aluminum to steel overhead sign-support structures. Through a systematic fatigue evaluation of steel DMS-support truss structures (and comparison with their aluminum counterparts) under thermal loads, this is the first-known study that sheds light on the promise of using steel materials to avoid fatigue-induced damage in this important category of structures. For this purpose, two DMS-support structures are instrumented with vibrating-wire gauges, which are capable of recording both strain and temperature. Using an array of gauges in critical locations, strain and temperature time histories are recorded for close to one year. The collected time histories are then employed for the fatigue assessment of the DMS-support truss structures. In addition to the study of the data collected from the field, detailed FE models are generated to obtain an in-depth understanding of the extent and potential of damage (after validation with the field data). For calculating the fatigue life, the rainflow cycle counting method [7] and the Miner’s linear fatigue damage accumulation rule [8] are utilized. This part of the study, is concluded by a direct comparison between the fatigue performance of steel and aluminum DMS-support structures under thermal loads.

Despite the contribution of the existing studies to evaluate fatigue in DMS-support structures due to temperature- and wind-induced loads, there is a gap in the literature concerning the combined effects of temperature and wind on this important category of structures, which often experience both during their service life. This has been the primary motivation of the third section of the current study, which aims at establishing the very first multi-stressor perspective for the evaluation of DMS-support structures. For this purpose, a field study paired with detailed numerical simulations are performed. With the implementation of a dense sensor layout, strain,
temperature, and wind speed and direction are recorded on a regular basis. While the field data provide a direct assessment of the strains and stresses that the individual structural elements experience, they will be further utilized to validate a FE model, which is subjected to a number of loading scenarios. This model is employed to identify the most critical structural elements under fatigue induced by individual and combined stressors. The fatigue calculations are based on the rainflow cycle counting method [7] and the Miner’s fatigue damage accumulation rule [8]. The current study is then extended to evaluate wind directionality effects based on historical data available for a 50-year time window. This provides detailed information about the contribution of the involved stressors and illustrates how a site-specific assessment can affect the estimated fatigue life.
CHAPTER 2. FATIGUE ANALYSIS OF SIGN-SUPPORT STRUCTURES DURING TRANSPORTATION UNDER ROAD-INDUCED EXCITATIONS

Abstract

Dynamic Message Signs (DMSs) are increasingly used in highways as an effective means to communicate time-sensitive information with motorists. To ensure the long-term performance of DMSs, it is critical to ensure that the truss structures that hold them can resist not only extreme loading events, but also fatigue induced by service loads. The existing studies, however, are primarily focused on the loads that DMS-support structures experience during their service life and neglect the potential contribution of stresses induced during the transportation of them from the fabrication site to the location of installation. As a result, the potential damage that this important category of structures may sustain during transportation is largely unknown. To investigate this aspect, a comprehensive field test and numerical study were conducted. For field investigation, one span of a four-chord, overhead sign-support structure was instrumented to perform a short-term structural health monitoring. In addition, detailed finite element simulations were conducted to obtain an in-depth understanding of the potential modes of damage under the excitations induced by the road profile. The outcome of this study is expected to determine the extent of fatigue and structural vulnerability of DMS-support structures during transportation.

Keywords: Sign-support structures; Transportation; Fatigue; Finite-element analysis; Road profile; Suspension system; Stress concentration factor.
Introduction

Overhead highway signs, luminary poles, and traffic signals are critical transportation infrastructure components, which play an important role in guiding the traffic and ensuring the public safety. Among them, the use of Dynamic Message Sign (DMS) has been rapidly increased, primarily because of advancements in the DMS technology and its unique capability to provide the motorists with live traffic information. DMSs are mounted on either a cantilever or bridge type truss structure, which is normally prefabricated elsewhere and shipped to the site. Considering that the truss structures offer an economic and dependable means of holding the DMS cabinet, their structural performance has become a subject of interest for those working on the design and maintenance of transportation infrastructures. Since hollow pipes and angle sections are commonly used in such structures, the DMS-support structures have a relatively small mass, high flexibility, and low damping ratio (in the range of 1%). The listed properties make this category of traffic structures particularly vulnerable to high-cycle, low-amplitude excitations [1]. Failure of DMS-support structures has been reported in several locations in the United States. The most common cause of failure is believed to be metal fatigue. Mast arm to column connection, column to base plate connection, and anchor bolts are among the critical parts of the support structures susceptible to failure [2–4]. In addition, joints and weldments are often the areas of concern, as they experience fatigue-induced damage. Cracks have been observed, especially in the joints within the toe and leg of fillet welds propagating into the main chords in some cases [5].

A review of the existing literature indicates that Barle et al. (2011) investigated the truss joints by modeling the stress fields under static and dynamic loads. This study was able to estimate the fatigue life of the structure and make suggestions for design improvements [6]. Roy et al. (2010) examined the weld geometry at the joints and evaluated crack modes through experimental
fatigue tests [7]. In a study not limited to the joints, Rice et al. (2012) combined numerical simulations with field measurements to better understand the fatigue characteristics of sign structures [8]. Sanz-Andrés et al. (2003) provided a qualitative explanation of the main characteristics that capture the evolution of vehicle force causing fatigue damage [9]. Huckelbridge and Metzger (2007) conducted a fatigue analysis of a sign structure under the vibration of truck passage [10]. Different types of overhead truss structures were studied by Fouad et al. (2003) to understand the influence of natural wind gusts [11]. In a separate effort, Kacin et al. (2012) conducted a fatigue life analysis for an overhead sign-support structure [12]. They utilized the ANSYS package to model the structure and identify the critical members in both pristine and damaged conditions. Wind-induced fatigue analysis of high-mast light poles was conducted by Chang et al. (2009 and 2010) using long-term field monitoring data [13,14]. In a later study, Chang et al. (2014) investigated the effects of thermal loads on the aluminum DMS-support structures [15].

Despite the contribution of the former studies, they are found to be primarily focused on the loads that the structure experiences during the service life, disregarding the potential contribution of stresses induced during transportation from the fabrication site to the location of installation. As a result, the potential damage that this important category of structures may sustain during transportation is largely unknown. To investigate this aspect, a comprehensive field test and numerical study is conducted in the current study. For this purpose, one span of a four-chord, overhead sign-support structure is instrumented with strain gauges. The main objective of this short-term monitoring is to capture the strains induced in the truss members because of the excitations caused by the road profile.
In addition to the investigation of data collected from instrumentation, detailed finite-element (FE) simulations are conducted to obtain an in-depth understanding of the extent and potential modes of damage. The stresses that the truss structure experience during transportation directly depend on the vibrational excitations, which require an integrated model for simulating the road profile and the suspension system of the truck used to transport the prefabricated structure. To achieve this goal, a set of road profiles are generated based on ISO Specifications [16]. This covers a range of road surface conditions from good to poor. In the next step, a passive suspension system is modeled. This model transfers the excitations induced by the road roughness to the structure considering the mass, stiffness, and damping parameters. The Simulink package is used for solving the differential equations associated with the suspension system. Using the developed Simulink model, the suspension-induced excitations, which excite the truss structure at its supports, are calculated during transportation.

The excitation time-history of the supports is employed as an input to the detailed FE model of the truss structure. This model, which has been developed using the ABAQUS package, is utilized to obtain the time-history of strains and stresses at various structural members and their joints. The FE model is validated first with the field data collected from the strain gauges during transportation. Stress time histories of the most critical members of the truss are then extracted from the FE model. Using the rainflow cycle counting method [17], the stress ranges and their corresponding cycles are derived for fatigue analysis. Based on the Miner’s linear fatigue damage accumulation rule, the fatigue-induced damage is calculated using both experimental data and numerical simulation results. The outcome of this study contributes to determine the intensity of damage that the structure may experience during transportation and if this has a critical effect on the expected service life.
Field Study

To evaluate the effects of road-induced excitations on DMS-support truss structures during transportation, a field study has been conducted on one of the three blocks of a truss structure, which was shipped from the State of Kansas to Iowa for installation. The truss under consideration was transported for a distance of approximately 110 miles (1 mile = 1.609 km). The average speed of truck was in the range of 40-50 miles per hour. During monitoring, twelve one-axial strain gauges were mounted on the truss and the strain data were recorded in 33 data sets for a period of three hours with the frequency of 100 Hz. Figure 2-1 shows the transported truss and the position of some of the mounted strain gauges. Figure 2-2 provides the identification number of all the strain gauges used for instrumentation and Figure 2-3 illustrates the strain time histories recorded by the Sensor S1 and S12 over 120 seconds for the 3rd and the 30th data sets as two examples. A review of the data collected from all the 12 strain gauges indicates that the strain time histories follow a similar pattern overall and there is no abnormal strain in the recorded time histories. Figure 2-3(a) clearly shows that there are peaks in the strain time histories, which occur at the same time in all the data sets (see for example the strain at the 10th and 115th second). This trend can be related to the road profile. The similarity observed in the time histories, as well as the absence of any major outliers, highlight that the collected data have an acceptable quality. To obtain a more in-depth understanding of the range of strains, the collected data have been processed and the maximum and minimum strain values for each sensor in each of the 33 recorded data sets are summarized in Figure 2-4. As it can be seen in this figure, the strains are mostly in the range of -20 to +20 micro-strain, although there are some data points which exceed this range. Table 2-1 summarizes the maximum and minimum strain values for each sensor. Since the data have been recorded in separate time intervals during transportation, the average of maximum and minimum
values obtained for each time interval has also been included in Table 2-1. This table provides an envelope with a peak and average value for each sensor, which reflect the range of strains that each truss member may experience. This range is also used to evaluate the accuracy of predictions obtained from the FE model generated for this study.

As failure due to fatigue is the most critical mode of failure in sign-support structures, a detailed fatigue analysis is conducted using the data collected from the field. The goal of this analysis is to evaluate the potential contribution of road-induced excitations to the fatigue-induced damage. For this purpose, the rainflow cycle counting method is utilized to count the number of cycles for various stress ranges that the structure has experienced during transportation. The Miner’s rule is used in the current study to determine if any fatigue-induced damage occurs during transportation [18]. To conduct the fatigue analysis, the strain time histories are first converted to the stress time histories using the well-known Hook’s Law. Although this direct conversion may involve some approximation (due to shear and/or bending moment), the percentage of error is deemed negligible because the truss consists of long and hollow members, mainly under a uniaxial loading condition.

Based on the stress time-histories obtained along the length of truss members, the stress time histories at the joints can be predicted. Despite the similarities between the patterns of stress time histories at the middle and end of a truss member, it is known that the stresses are magnified at the joints due to strain concentration effects. The strain concentration factor (SCF) is reported in the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (hereafter referred to as AASHTO Specifications) based on the type of connection [19]. Using the magnified stress time histories, the number of cycles for various stress ranges are obtained for the fatigue analysis. Figure 2-5 presents the histograms of stress ranges
and their respective number of cycles for 6 sensor/joint locations. The histograms are plotted in a logarithmic scale to depict the number of cycles clearly. A review of the developed histograms shows that all of the cycles are below the constant amplitude fatigue threshold (CAFT), which is 4500 psi (1 psi = 6895 Pa) for the slotted tube to gusset plate connections [19]. This eliminates the concern about experiencing a high level of fatigue-induced damage during transportation.

**Numerical Study**

In addition to the field investigation, a detailed FE model of the instrumented truss structure has been generated in the ABAQUS package to obtain a comprehensive assessment of the structural response of individual truss members further to their potential modes of damage and failure. To model the joints with necessary details, shell elements are used for the truss members and connecting gusset plates. This approach allows for the determination of stress distribution within the thickness of each truss member. Figure 2-6 shows an overview of the FE model together with a close view of connection details. Mesh patterns reflect a fine mesh at critical joint locations, while a coarse mesh has been used for the rest of the elements. The FE simulations conducted in the current study pursue three key objectives: (1) To determine the effect of road profile on the range and number of cycles of stress, which can lead to fatigue-induced damage; (2) To identify the most vulnerable members of the truss and their connecting joints during transportation; and (3) To evaluate the extent of stress magnification from the middle to the end of individual truss members. To achieve the listed objectives, the road-induced excitations are applied to the truss model. This requires modeling of the road profile and the truck’s suspension system following the procedure discussed in the next sections.
Road Profiles

There are several studies devoted to the generation of road profiles and related subjects. Among all the procedures available for road roughness generation, Power Spectral Density (PDS)-based procedures have received a great attention [20–23]. In this study, artificial road profiles are generated based on the PSD-based procedure introduced by ISO 8608 [16]. This involves using the following formula to generate the road profile, $h(x)$, for different road surface classes:

$$h(x) = \sum_i \sqrt{\Delta n} 2^k \left( \frac{n_0}{n_i} \right) \cos(2\pi n_i x + \varphi_i)$$  \hspace{1cm} (1)

where $\Delta n$ is the frequency band, $k$ is the constant value determined based on the road profile class, $n_i$ is the spatial frequency, $n_0 = 0.1$ cycle/meter, and $\varphi_i$ is the random phase angle distributed uniformly from 0 to $2\pi$. According to ISO 8608, the road surface will degrade by increasing the value of $k$. The $k$ values for different road classes from good to poor have been tabulated in Table 2-2. A detailed explanation on the generation of reliable artificial road profiles has been provided by [24]. It must be noted that Equation 1 does not capture the unexpected road bumps and potholes. This, however, is expected not to influence the predictions in any significant way, as the number of cycles associated with bumps and potholes in the recorded strain time histories was found to be negligible. Road profiles generated for a sample of 250 meters are shown in Figure 2-7. After generating the necessary road profiles, the next step is to model a suspension system capable of transferring the excitation induced by the road roughness to the truss structure that is being shipped by the truck.
Suspension System

There are different types of car models available in the literature for different purposes. While the quarter car model is utilized for spectral analysis, the full car model is used for advanced control and comfort studies [25]. Among the studies conducted on modeling of suspension systems, Shpetim Lajqi (2012) presented a mathematical model of a quarter-car suspension system to study the performance of active, semi-active, and passive systems. For this purpose, Simulink was used to solve the differential equation of the suspension system. Chavan et al. [26] took the nonlinearity of the suspension system parameters into account to introduce a precise suspension model. Sharma et al. [27] conducted a study aimed at preventing the transfer of vibrations to the driver. A quarter-car model with two degrees of freedom was used for simulation purposes. Alexandru et al. [28] compared the ride and handling of passive and semi-active suspension systems. The study found that the acceleration that the body’s mass experiences can be reduced if a semi-active suspension system is used. Similar studies of suspension systems with the main objective of evaluating the contributing parameters can be found in [29,30].

In the current study, the quarter-car model is used to capture the response of a passive suspension system and the consequent excitation transferred to the truss structure during transportation. The suspension diagram shown in Figure 2-8(a) is a quarter-car system, in which $m_s$ is the sprung mass, $m_{us}$ is the unsprung mass, $k_s$ is the stiffness coefficient of the suspension system, $k_{us}$ is the stiffness of the tire, $c_s$ is the damping coefficient of the suspension system, $Z_s$ is the displacement of the sprung mass, $Z_{us}$ is the displacement of the unsprung mass, and $Z_r$ is the excitation from the road roughness. Since the primary focus of this study is on the vertical excitation that the truss structure experiences, only vertical mass movements are modeled and the movements in the other directions of the vehicle are disregarded. It must be noted that the accuracy
of the generated road-induced excitations could be further improved if further details regarding the 
tire-road contact, the truck dynamic, and its suspension system, were available to be included in 
the developed model.

Equations of motion can be introduced using the Newton’s second law for each mass and the 
Newton’s third law for their interactions.

\[
m_s \frac{d^2 Z_s}{dt^2} + c_s \left( \frac{dZ_s}{dt} - \frac{dZ_{us}}{dt} \right) + k_s (Z_s - Z_{us}) = 0
\]
\[
m_{us} \frac{d^2 Z_{us}}{dt^2} + c_s \left( \frac{dZ_{us}}{dt} - \frac{dZ_s}{dt} \right) + k_s (Z_{us} - Z_s) + k_{us} (Z_{us} - Z_r) = 0
\]

Since solving Equations (2) and (3) is nontrivial, the Simulink package is used. Figure 2-8(b) 
depicts the Simulink model for solving the coupled differential equations of the suspension system. 
There are some suggested values for the suspension system parameter in the literature for a car, 
bus, and heavy truck. However, there are no parameters available for the pickup truck used for the 
transportation of the truss. Hence, the suspension system parameters of the most similar car are 
utilized to solve the differential equation of the suspension system, because those values are the 
closest ones to the pickup truck \[24\]. It must be mentioned that the parameters have been adjusted 
to include the weight of the truss. The selected parameters are listed in Table 2-3. It is assumed 
that the tires do not lose contact with the pavement during transportation. The input file of the 
Simulink model includes road roughness values, which are an array of equally spaced data points. 
Three road classes with \( k = 3, 4, \) and 5 are investigated in the current study. Figure 2-8(c) shows 
the Simulink results for the sprung mass, unsprung mass, and road profile for various \( k \) values. 
The body displacement obtained at this stage is used as an input to the FE model of the truss 
structure for the purpose of capturing the road roughness effects.
**Validation of FE Model**

To validate the multi-step procedure developed to investigate the structural response of truss structures through numerical simulations, the strain time histories obtained from the FE model for the individual truss members are compared with the envelope of strain time histories obtained from the field study (i.e., Figure 2-4). Two strain time histories extracted from the FE model for $k = 3$ and $5$ are shown in Figure 2-9. The strain time histories are recorded at the exact location of the strain gauges mounted on the actual truss. All the strains obtained from the field study fall in the envelope provided by the FE simulation results with $k$ in the range of 3 to 5. This highlights that the developed FE model can successfully capture the strains that the truss members experience during transportation.

**Results and Discussions**

**Stress Concentration Factor**

Stress Concentrations Factor (SCF) is an essential factor for both fatigue design and analysis of sign-support structures. As it is proven that a higher SCF leads to a lower fatigue life, an accurate estimate of the SCF is expected to be critical to obtain an efficient and economic structural design and configuration. In addition, from an analysis point of view, the fatigue life of a structure can be predicted with the least underestimation or overestimation if an accurate SCF is utilized. Following the procedure provided by the AASHTO Specifications [19] for fatigue analysis, the stresses must be calculated based on the net cross-sectional area of the truss members and then magnified by a prescribed constant value of 4.0. The SCF, however, is known to be influenced by the direction of applied loads and the position of individual truss members, in addition to their boundary conditions. In this study, the direction of loads applied to the truss
structure during transportation is completely different from the direction of loads that the structure is expected to experience after installation in the field. As a result, for the fatigue analysis of the structure during transportation, a revised set of SCFs must be obtained. To achieve this goal, the developed FE model is utilized and the SCFs are derived with the following steps: (1) The most critical element at the end of each truss member is identified by comparing the suspected critical elements at each joint; (2) The stress time history is derived at the middle of the truss members, where the sensors were mounted during transportation; and (3) The SCFs are estimated by comparing the stress time histories obtained for the middle and end of each truss member.

Based on the AASHTO Specifications [19], the stress time history intended to be used for fatigue analysis must be derived from the areas adjacent to the welded parts. Therefore, a comprehensive procedure is implemented to select the most critical element for each truss member. Following is the procedure used for this purpose: All the elements located on one weld line are selected. The stress time histories are then obtained for the selected elements and comparisons are made to identify the elements that constantly experience the highest stress values. Figure 2-10 shows the elements selected for deriving the stress time histories, as well as the stress time histories derived from the numerical simulations.

The outcome of this investigation highlights that the elements with the highest stress are those closest to the hole of the truss members. This criterion leads to four elements at each end of the truss member. There are eight elements in each tubular member adjacent to the hole, which directly experience the welding effects. Figure 2-10 depicts four of those eight elements at one end of a member. The stress time histories for the eight elements are illustrated in Figure 2-10. The element with the highest value of stress in this figure is the most critical element of the member. For example, Element 322 is the most critical element in the member that carries Sensor S1. The
next step is to examine the stress time history at the middle of the member. In reality, a sensor can be mounted anywhere on the perimeter of the truss member. Thus, the effect of the sensor’s mounting location must be considered as well. To address this aspect, all the elements at the middle of the truss member are selected in one cross section for deriving the averaged stress time histories. Since the truss members normally experience pure compression or tension, averaging the stress in the cross section is expected not to introduce a major approximation. Figure 2-11 shows how the stress time histories obtained for the middle of the member are compared with the one recorded at the most critical element at the end of the same member.

For deriving the ratio of stress in the critical element to the stress at the middle of the member, a MATLAB code has been generated to pick the highest value in each peak and valley. This is an important step to avoid the outliers that may appear if the stress values are close to zero. This ratio has been shown in Figure 2-12 for Sensor S1 and S2. Table 2-4 summarizes the SCFs obtained from the numerical simulations for various members of the truss. Although the fillet welds have not been explicitly included in the FE model, the calculated SCFs can be below or above 4.0, which is the SCF recommended by the AASHTO Specifications [19].

**Fatigue-Induced Damage Analysis**

The stress life method is utilized in the current study for the fatigue analysis of the truss structure. This method is based on the S-N curves, which represent the stress ranges versus the number of cycles. Following is the equation of each S-N curve:

$$N_i = \frac{A}{S_i^3}$$  \hspace{1cm} (4)
where $A$ is a constant value based on the connection category [19], $N_i$ is the number of cycles needed to cause failure at the $i$-th stress range, and $S_i$ is the corresponding stress range. The AASHTOSpecifications [19] provides a table, including various types of connection for sign-support structures, including groove welded and fillet welded connections. The connection categories are labeled as A, B, B’, C, D, E, E’, ET and K2. In this study, the type of connection is a slotted tube to gusset plate connection with the coped hole, which falls in the E Category. Figure 2-13 shows the connection of interest in the current study as illustrated by the AASHTO Specifications [19]. The parts highlighted in red are the most critical areas, prone to fatigue-induced cracks.

To investigate probabilistic considerations in the fatigue life analysis, a set of S-N curves are developed for different confidence percentiles. This reflects the fact that the samples that were subjected to fatigue tests provided a distribution of stress ranges under a given number of cycles. Using the statistical properties reported by Moses et al. [32] for the S-N curves of connections in the E Category, Figure 2-14 shows the S-N curves for the mean ($\mu$) and mean plus various standard deviations, i.e., $\mu + \sigma$, $\mu + 2\sigma$, and $\mu + 3\sigma$, representing 50, 84.1, 97.7, and 99.9 percentiles, respectively. For comparison purposes, the S-N curve provided by the AASHTO Specifications [19] (equivalent to 94 percentile) is included in this figure as well. The AASHTO recommended curve has been used for the fatigue analysis presented in this section.

Based on the Miner’s rule, damage resulted from a given stress range is a linear function of the number of cycles of that stress range. The Miner’s rule formula can be expressed as:

$$D_i = \frac{n_i}{N_i}$$ (5)
where \( N_i \) is the number of cycles needed to cause failure at the \( i \)-th stress range \( (S_i) \), and \( n_i \) is the number of cycles of the stress range \( S_i \). The value of \( N_i \) can be calculated by using Equation (4) and the value of \( n_i \) can be found by applying the rainflow cycle counting method on the stress time histories. Once the damage for each specific stress range \( S_i \) is determined, the total damage \( D \) can be calculated using the following equation:

\[
D = \sum_i D_i
\]

According to the Miner’s rule, the failure occurs if the total damage, \( D \), becomes equal to 1.0. It must be noted that the Miner’s rule disregards the mean stress correction. This, however, has been shown to have negligible effects on the fatigue analysis of overhead sign-support structures [5,8,12].

In the current study, fatigue analysis is conducted on (1) data collected from the field and (2) results obtained from the FE simulations. Based on the stress ranges and cycles obtained from the field data (as illustrated in Figure 2-5), the percentage of fatigue-induced damage is calculated for all the truss members that were instrumented with a strain gauge. The outcome of the fatigue analysis on the field data is shown in Figure 2-15. The most critical member of the truss has experienced 0.01% of failure damage during transportation.

The fatigue analysis has been conducted on the numerical simulation results as well. This includes three road classes A, B, and C \( (k = 3, 4, \) and \( 5, \) respectively). The three most critical members have been identified for fatigue analysis. The first member is the lower chord of the truss adjacent to the point where the truss is sitting on the truck. The second member is one of the interior diagonals and the third member is one of the vertical diagonals. Figure 2-16 shows the results of the rainflow cycle counting analysis of the identified critical members for different road classes. The outcome of this fatigue analysis has been summarized in Figure 2-17, which depicts the
percentage of fatigue-induced damage for each of the critical members under the three road classes. According to this figure, the percentage of damage for the most critical member of the structure is approximately 0.002% of failure damage for the road class A \((k = 3)\) and 0.03% of failure damage for the road class B \((k = 4)\). The obtained values are in a close agreement with the damage percentage determined from the field data. The location of the highest fatigue-induced damage in the FE model is consistent with the findings from the field investigation, which identified the lower chord of the truss as the most vulnerable member. It must be noted that the fatigue-induced damage to the structure can increase to approximately 0.1% of failure damage if the truss is transported in the road class C \((k = 5)\).

Conclusions

A comprehensive field study paired with detailed numerical simulations were conducted on a four-chord sign-support structure during transportation from the fabricator to the installation site. For the field study, 12 strain gauges were mounted on various truss members and a short-term structural health monitoring was completed. Further to the field study, a detailed FE model of the same truss was generated using the ABAQUS package to evaluate the distribution of strains and stresses within the entire truss members. To investigate the effects of road roughness on the potential of experiencing fatigue-induced damage, three artificial road profiles were generated following the ISO guidelines. To capture the vibrations transferred to the truss structure when it was being shipped by the truck, a quarter car suspension system was designed and the Simulink package was used to solve the corresponding partial differential equations of motion. The time-history of vertical vibrations at the truss supports were used as input to the detailed FE model of the truss generated in the ABAQUS package. Following a set of numerical simulations for various
road surface conditions, the stress time histories were obtained and the SCFs were estimated. This was one of the unique contributions of the current study, which highlights how the actual SCFs are compared with the constant value reported in the AASHTO Specifications. A fatigue analysis was then conducted using both the field data and simulation results. While the potential of experiencing fatigue-induced damage changes from one truss member to the other, it was shown that the predictions obtained from the developed computational framework is in a close agreement with the observations from the field in terms of identifying the most vulnerable elements as well as the percentage of fatigue damage during transportation.

References


Tables and Figures

Table 2-1. The range of strain values recorded by the strain gauges during transportation.

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Max. Value (Micro-strain)</th>
<th>Min. Value (Micro-strain)</th>
<th>Avg. of Max. Values (Micro-strain)</th>
<th>Avg. of Min. Values (Micro-strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>28.1</td>
<td>-16.2</td>
<td>11.2</td>
<td>-8.5</td>
</tr>
<tr>
<td>S2</td>
<td>16.8</td>
<td>-21.7</td>
<td>8.5</td>
<td>-8.4</td>
</tr>
<tr>
<td>S3</td>
<td>24.4</td>
<td>-14.9</td>
<td>8.6</td>
<td>-6.5</td>
</tr>
<tr>
<td>S4</td>
<td>15.1</td>
<td>-11.4</td>
<td>6.1</td>
<td>-4.4</td>
</tr>
<tr>
<td>S5</td>
<td>17.1</td>
<td>-14.2</td>
<td>8.5</td>
<td>-6.3</td>
</tr>
<tr>
<td>S6</td>
<td>14.7</td>
<td>-17.1</td>
<td>8.1</td>
<td>-8.5</td>
</tr>
<tr>
<td>S7</td>
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<td>18.4</td>
<td>-10.5</td>
<td>7.7</td>
<td>-5</td>
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<td>3.5</td>
<td>-2.5</td>
</tr>
<tr>
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<tr>
<td>S11</td>
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<td>-13.1</td>
<td>6.2</td>
<td>-5.5</td>
</tr>
<tr>
<td>S12</td>
<td>11.9</td>
<td>-8</td>
<td>3.8</td>
<td>-2.7</td>
</tr>
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Table 2-2. Values of k for ISO road profiles.

<table>
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<tr>
<th>Road Class</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
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<tbody>
<tr>
<td>k</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>9</td>
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Table 2-3. Selected values of parameters for the quarter car suspension system.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sprung mass $m_s$ [Kg]</td>
<td>800</td>
</tr>
<tr>
<td>Unsprung mass $m_{us}$ [Kg]</td>
<td>40</td>
</tr>
<tr>
<td>Suspension stiffness $k_s$ [N/m]</td>
<td>21000</td>
</tr>
<tr>
<td>Suspension dumping $c_s$ [N-s/m]</td>
<td>1500</td>
</tr>
<tr>
<td>Tire stiffness $k_{us}$ [N/m]</td>
<td>150000</td>
</tr>
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</table>
Table 2-4. A comparison between the SCFs obtained from the numerical simulations and the constant value recommended by the AASHTO Specifications.

<table>
<thead>
<tr>
<th>Sensor ID</th>
<th>Location of the member in the truss</th>
<th>SCF derived from numerical simulations</th>
<th>Difference with the AASHTO recommendation</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Vertical diagonal</td>
<td>1.30</td>
<td>-68%</td>
</tr>
<tr>
<td>S2</td>
<td>Interior diagonal</td>
<td>3.39</td>
<td>-15%</td>
</tr>
<tr>
<td>S6</td>
<td>Horizontal diagonal</td>
<td>3.03</td>
<td>-24%</td>
</tr>
<tr>
<td>S7</td>
<td>Vertical diagonal</td>
<td>1.91</td>
<td>-52%</td>
</tr>
<tr>
<td>S3, S8</td>
<td>Lower chord</td>
<td>3.72</td>
<td>-7%</td>
</tr>
<tr>
<td>S4, S5</td>
<td>Lower chord</td>
<td>5.70</td>
<td>+43%</td>
</tr>
</tbody>
</table>

Figure 2-1. Preparation of transported truss for short-term monitoring.
Figure 2-2. Geometry and details of the truss further to the instrumentation layout. This truss was transported in three separate parts. The middle part is the subject of the current study.

Figure 2-3. Strain time histories derived from Sensors S1 and S12 for two data sets: (a) 3rd data set, and (b) 30th data set.
Figure 2-4. Maximum and minimum strain values recorded by strain gauges in each data sets.

Figure 2-5. Stress ranges and their respective number of cycles for the truss members instrumented during transportation. (1 psi = 0.006895 MPa)
Figure 2-6. Finite element model: (a) big picture of finite element model and close view of connection, and (b) mesh pattern of members.

Figure 2-7. Artificial road profiles under three different road classes for a sample of 250.
**Figure 2-8.** Suspension system simulation procedure: (a) schematic representation of truck suspension system, (b) Simulink model used for solving the suspension system equations, (c) response of suspension system for three road classes.

**Figure 2-9.** Strain time history derived from the FE model road profile A (left) and C (right).
Figure 2-10. Procedure of finding the most critical element of a member: (a) stress time histories of all the elements located on one weld line, (b) elements location on one weld line, (c) stress time histories of eight elements adjacent to the hole, and (d) elements location adjacent to the hole. (1 ksi = 6.895 MPa)

Figure 2-11. Stress time histories at the middle and the connection of two truss members obtained from FE simulations. (1 ksi = 6.895 MPa)
Figure 2-12. SCF changes and the corresponding mean value for Sensors S1 (left) and S2 (right).

Figure 2-13. Connection detail investigated in the current study.
Figure 2-14. Developed $S-N$ curves considering various confidence levels. (1 psi = 0.006895 MPa)

Figure 2-15. Fatigue damage estimated for the truss members instrumented during transportation.
Figure 2-16. Stress ranges and their respective cycles for the most vulnerable truss members obtained from the numerical simulations under three different road classes. (1 psi = 0.006895 MPa)

Figure 2-17. Fatigue damage in the most critical truss members based on numerical simulations.
CHAPTER 3: INVESTIGATION OF FATIGUE IN STEEL SIGN-SUPPORT STRUCTURES SUBJECT TO DIURNAL TEMPERATURE CHANGES

Abstract

Use of Dynamic Message Signs (DMSs) in highways has been significantly increased due to their unique capability to guide the traffic in real time. As a result, the structural performance assessment of the overhead truss structures that hold DMSs has become a subject of interest, particularly for those involved in the design and maintenance of transportation infrastructures. Despite the importance of this category of structures, there are only a few studies available in the literature, primarily focused on wind load effects. The potential contribution of thermal stresses to damage, particularly at the joints of overhead truss structures, however, is still largely unknown. This was the motivation of the current study as the damage induced by temperature fluctuations can become significant, especially in the regions that experience large diurnal temperature changes. To investigate this critical aspect, a systematic field study supported by numerical simulations is conducted. Two overhead DMS-support structures located in Iowa are selected for instrumentation and long-term monitoring. An array of vibrating-wire gauges are installed on these two DMS-support structures to understand the effects of diurnal temperature changes. Further to the analysis of collected field data, detailed finite element simulations are conducted to obtain an in-depth understanding of the expected extent of fatigue-induced damage. Based on the results obtained from this study, the DMS-support structures under consideration are found to demonstrate a great performance under thermal loads. This strongly supports the recent transition from aluminum to steel overhead truss structures to avoid vulnerability to diurnal temperature changes.

Keywords: Steel sign-support structures; Finite-element simulation; Fatigue; Structural health monitoring; Thermal loads.
**Introduction**

There are various transportation infrastructure assets, such as overhead highway signs, luminary poles, and traffic signals, which are critical to guide the traffic and ensure the public safety. Among them, cantilever and bridge-type truss structures are widely used in highways to support Dynamic Message Signs (DMSs), which are capable of communicating real-time information with the motorists. As this category of sign-support structures are shipped to the site in a prefabricated form, they offer an economic and dependable choice with the advantage of fast installation. Thus, as the use of overhead truss structures has increased, the structural performance assessment of them has become a subject of interest, particularly for those involved in the design and maintenance of transportation infrastructures. Since hollow pipes and angel sections are commonly used in such structures, they have a relatively small mass, high flexibility, and low damping ratio (in the range of 1%). These characteristics make the overhead truss structures vulnerable to high-cycle, low-amplitude excitations [1].

Failure of overhead truss structures has been observed in a number of cases in the United States and beyond. The most common cause of failure has been reported to be metal fatigue. In-service loads are commonly due to wind load effects, such as vortex shedding, galloping, and natural wind gusts. Vortex shedding occurs when a bluff body separates the flow from its surface. If the frequency of shedding vortices is closed to that of the truss structure, large displacements are anticipated. Galloping is a self-induced phenomenon due to aerodynamic forces, which create oscillations in the cross-wind direction. Finally, wind gusts can cause vibrations in the truss structure, depending on the height and location of the structure. Further to natural wind loads, the wind generated by vehicles passing underneath the structure can cause some vibrations. The vibration-induced fatigue of sign-support structures has been the subject of a number of former
studies, e.g., Hong et al. (2015), Hosch et al. (2014), Sanz-Andres et al. (2003), Ding et al. (2016), 
Holmes (2002), Barrero-Gil and Sanz-Andres (2009), Somchai1 et al. (2013), Lichtneger and Ruck 
(2015), Rice et al. (2017), and Cali and Covert (2000) [2–13].

In a DMS-support structure, truss to column and column to base plate connections are 
among the critical parts, vulnerable to structural damage and failure [14–16]. Moreover, the 
individual joints and weldments of these structures are often of particular concern, as they are 
susceptible to fatigue-induced damage. In several cases, formation and propagation of cracks have 
been reported within the toe and leg of fillet welds, with an extension to the main chords [17]. 
Among the existing studies, Barle et al. (2011) investigated the truss joints by modeling the stress 
fields under static and dynamic loads. Based on the estimated fatigue life, suggestions were made 
to improve the design guidelines [18]. In a separate study, Roy et al. (2010) examined the weld 
geometry and evaluated the crack modes through experimental fatigue tests [19]. In a study not 
specific to the joints, Rice et al. (2012) used the finite-element (FE) analysis results in conjunction 
with the data obtained from the existing sign structures to improve the understanding of their 
fatigue characteristics [20]. Kacin et al. (2012) conducted a fatigue life analysis for an overhead 
sign-support structure. For this purpose, a FE model was developed to simulate the structure and 
identify its critical members in both pristine and damaged conditions [21]. Fouad et al. (2003) 
investigated the effect of natural wind gusts on fatigue in different types of overhead truss 
structures [22]. Huckelbridge and Metzger (2007) performed a fatigue analysis on a sign structure 
under the vibrations induced by truck passage [23]. Wind-induced fatigue analysis of high-mast 
light poles was conducted by Chang et al. (2009 and 2010) using long-term field monitoring data 
[24,25]. This effort was extended by Chang et al. (2014) to evaluate the thermal damage in pole 
structures [26].
Further to the studies on performance assessment, damage detection in overhead sign-support structures has been a subject of interest due to the catastrophic consequences of their failure. The number of studies on this subject, however, is limited. Sun et al. (2006) developed a multi-level strategy for damage detection in sign-support structures [27]. In a separate study, vibration-based health monitoring was used by Kopsaftopulos and Fassois (2010) for damage identification in sign-support structures [28]. The listed studies are in addition to the ones devoted to damage detection in the truss structures not specifically used for traffic signs and signals [29–36]. It is important to note that the existing studies are primarily focused on wind-induced excitations, neglecting the contribution of thermal stressors that can potentially cause damage to overhead truss structures during their service life. This research gap has been addressed through the current study. This study is particularly in response to the recent transition from aluminum to steel overhead sign-support structures. Through a systematic fatigue evaluation of steel DMS-support truss structures (and comparison with their aluminum counterparts) under thermal loads, this is the first-known study that explores the promise of using steel materials to avoid fatigue-induced damage in this important category of structures.

In this study, a field investigation on DMS-support structures, as well as their numerical simulations, has been performed. For this purpose, two DMS-support structures are instrumented with vibrating-wire gauges, which are capable of recording both strain and temperature data. The main objective of the long-term monitoring program is to capture the strains induced in the truss members due to diurnal temperature changes. Using an array of gauges in critical locations, strain and temperature time histories are recorded for close to one year. The collected time histories are then employed for fatigue assessment purposes. In addition to the evaluation of data collected from the field, detailed FE models are generated to obtain an in-depth understanding of the extent and
potential of damage (after validation with the field data). For calculating the fatigue life, the rainflow cycle counting method [37] and the Miner’s linear fatigue damage accumulation rule [38] are utilized. This study is concluded by an analytical investigation to make a direct comparison between the fatigue life of steel and aluminum DMS-support structures.

Field Investigation and Monitoring

DMS-Support Truss Structures

A long-term field monitoring program has been established for two representative steel DMS-support structures in Des Moines, Iowa to obtain an in-depth understanding of their vulnerability to the fatigue induced by diurnal temperature changes. Figure 3-1 shows the location of the two DMS-support structures (labeled as Truss A and B) on the map. Since the selected structures are relatively close to each other, they are expected to experience comparable weather conditions. This helps the current study evaluate the patterns of structural response and extend the findings to other similar structures. The long-term monitoring of the structures includes the main truss panels and two support posts. Each truss panel consists of four main chords and several internal members. All the internal members, i.e., vertical, horizontal, and interior diagonal, are made of standard steel pipes with an outer diameter of 3 in. (1 in. = 0.0254 m). The main chords consist of extra-strong steel pipes with an outer diameter of 5 in. The post supports are 31 ft. (1 ft. = 0.3048 m) tall and made of HSS pipes with an outer diameter of 14 in. and a thickness of 0.5 in. The side panel trusses are made of extra strong steel pipes with an outer diameter of 3 in. The main horizontal truss consists of three prefabricated trusses, which are 25, 30, and 25 ft. (total 80 ft.) long for Truss A and 30, 30, and 30 ft. (total 90 ft.) long for Truss B, respectively. The entire
horizontal truss is supported with a saddle assembly placed on W10×45 beams at two ends. The top truss ends are connected to the support posts by using stainless U-bolts. The support posts are welded to the base plates. A bolted connection is used to attach the base plates to the foundation.

**Monitoring System Setup**

Campbell Scientific CR1000 has been used to record data from vibrating-wire gauges. The sampling rate of the recording system is one every hour. A total of 18 vibrating-wire gauges are used for the instrumentation of each truss. Among them, seven sensors have been installed on the middle truss panel, seven sensors have been installed on the side truss panel close to the highway shoulder, and four sensors have been installed at the bottom of the post supports. Figure 3-2 shows the instrumentation layout for both Truss A and B. Similar members have been selected from both trusses for instrumentation to make a direct comparison possible. In each panel, four out of seven sensors have been attached to the horizontal chord members (one for each chord) and the remaining three sensors have been attached to the diagonal members.

**Findings from Long-Term Data**

One of the primary objectives of this study is to investigate the effect of diurnal temperature changes on the fatigue life of steel DMS-support structures. In order to evaluate such effects, the two structures have been monitored for close to one year to capture both hot and cold seasons. Figures 3-3 and 3-4 provide the strain and temperature time histories recorded by Sensor ID 3, 10, 5, and 12 for both structures in a hot and a cold month. A review of the data collected from all the installed sensors indicates that the strain time histories follow a pattern similar to the temperature time histories and there is no abnormal value in the recorded values. For a detailed fatigue analysis
of the two DMS-support structures under consideration, the temperature-induced strain time histories are first converted to the stress time histories using the Hook’s law. Even though this direct conversion can carry some approximation, the percentage of error is negligible since the truss consists of long and hollow members with a small thickness. The rainflow cycle counting method is then utilized to count the number of stress range cycles. Finally, the Miner’s rule [38] is used for the calculation of the fatigue life of the two structures. With a special focus on the truss joints, the stress time histories obtained along the length of the truss members are used to derive the stress time histories at the joints. This is achieved by using a magnification factor to take into consideration the stress concentration effects. Based on the type of connection, the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals [39] reports the stress concentration factor (SCF). This factor is found to be 4.0 for the joints of the two case-study trusses [39]. Using the magnified stress time histories, the number of cycles for various stress ranges are obtained for the fatigue analysis of the truss joints. Figure 3-5 presents the histograms of the stress ranges and their respective number of cycles for four sensor/joint locations selected from each structure. As it is illustrated in the developed histograms, some of the cycles are higher than constant amplitude fatigue threshold (CAFT), which is 4500 psi for the slotted tube to gusset plate connection [39]. This indicates that the concern of experiencing a high level of fatigue-induced damage from diurnal temperature changes exists for this category of structures.

In addition to the strain time histories, the temperature time histories must be investigated to further correlate the effects of diurnal temperature changes to the strains and stresses induced in the two structures. Figure 3-6 provides an overview of the maximum, minimum, and averaged temperature time histories for one hot and one cold month. Based on the recorded data, a
temperature range between 20 °F and 110 °F is experienced by each of the two trusses. Based on the data collected from the field, Figure 3-7 provides detailed information about the correlation of strain, temperature, and time of the year for Truss A and B. This figure confirms the correlation between the strain change and temperature.

**Numerical Simulations**

In addition to the field study, detailed FE models of the DMS-support structures have been generated in the Abaqus software package to extend the scope of investigations beyond what can be captured in the field. The numerical simulations are conducted, particularly to obtain an in-depth understanding of the response of this category of structures under temperature-induced stresses and predict their potential vulnerability to fatigue-induced damage. In the developed FE models, shell elements are used for the truss members and gusset plates. This provides the details necessary to capture stress concentrations at the joints. Figure 3-8 demonstrates the orthogonal projection of the FE model of Truss A together with a close view of a typical truss member. After an extended sensitivity analysis, the mesh size and pattern have been decided in such a way that the connection details are captured, while the overall efficiency of the FE simulations is maintained. Since the main objective of this numerical study is to identify the truss members (and their joints) most vulnerable to temperature-induced stresses, a temperature change is applied to the FE model in 10 °F increments. The strains and stresses at the most critical locations are then obtained for a fatigue analysis.

To validate the FE models developed to investigate the structural response of the DMS-support trusses, the temperature time histories averaged from the temperature time histories obtained from the individual sensors are applied to the FE models. A coupled thermal-stress
analysis is performed to calculate the strains and stresses in the truss members under the temperature profiles similar to what they experience in the field. Figure 3-9 compares the strain time histories obtained during the field study (from March 15th to March 31st) with the ones extracted from the FE model of Truss A. The members in the same location are selected from both middle and side truss panels to evaluate the consistency of results. Based on Figure 3-9, the strain time histories derived from the FE model follow a trend very close to the temperature time histories recorded at the corresponding sensor locations in the field. Considering that temperature time histories are input to the FE models, it is observed that the generated models can simulate the overall structural response well. This, however, does not necessarily include all the details of the strain time histories recorded by the sensors. This can be attributed to the data collection strategy used in the field monitoring program to avoid unnecessary large data files. The sample rate of data collection has been set to one every hour, mainly because no drastic change in the temperature is normally anticipated within one hour. In addition, it must be noted that there can be other sources of excitation, which cause low-cycle noises. Such noises are, however, believed not to have a major influence on the fatigue life of the structures, as they produce low-cycle stress ranges. Considering that the number of cycles for each stress range is needed for a fatigue analysis, it is essential that the FE simulations deliver the cycles of strain and stress similar to the actual cycles that the DMS-support structures experience in the field. Figure 3-10 illustrates the results of rainflow cycle counting for the simulation and actual strain time histories. A review of the results shows that the number of cycles associated with the first strain range is not fully consistent between the simulation and field data. This was expected because the high-cycle, low-range noises are not captured in the numerical simulations. For the other strain ranges, however, there is a close agreement between the numbers of cycles. This indicates that the generated FE model can
successfully simulate both range and number of cycles of strain and stress, which are critical for fatigue analysis.

**Fatigue Analysis**

For the fatigue analysis of the DMS-support structures under diurnal temperature changes, the stress life method has been utilized in the current study. This method is based on the $S$-$N$ curves, which represent the relation between the stress range and the number of cycles. The general equation of $S$-$N$ curves can be expressed as:

$$\log N_i = \log A - 3 \log S_i$$

where $N_i$ is the number of cycles needed to cause failure at the $i$-th stress range, $S_i$ is the corresponding stress range, and $A$ is a constant value determined based on the connection details. The AASHTO Specifications [39] provides a table that includes the $A$ value for various types of connections used in sign-support structures. In the current study, the type of connection used in the two trusses is a slotted tube to gusset plate connection with the coped hole, which falls in the $E$ category. Based on the AASHTO Specifications [39], the connection zone, particularly the areas close to the weld between the pipe and the gusset plate, are the most critical parts prone to fatigue-induced cracks.

Based on the Miner’s rule, the damage resulted from a specific stress range can be expressed as:

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

where $D_i$ is the fatigue damage caused by the stress range $S_i$, $n_i$ is the number of cycles of the stress range $S_i$, and $N_i$ is the number of cycles needed to cause failure at the $i$-th stress range. The
$N_i$ is obtained from Equation 1. On the other hand, $n_i$ is determined from the rainflow cycle counting of the stress time histories. Once the damage for each specific stress range, i.e., $S_i$, is found, the total damage, $D$, can be estimated using $D = \sum D_i$. Based on the Miner’s rule, if the total damage, $D$, becomes equal to 1.0, failure is expected to occur.

The fatigue analysis has been conducted on the data collected from the field, as well as the ones obtained from the numerical simulations. Using the stress ranges and cycles obtained from the field data (as illustrated in Figure 3-5), the fatigue life of all the instrumented truss members is found to be infinite. A similar procedure is used for the fatigue life calculation using the FE models, which provide the flexibility of going beyond the instrumented truss members. There are two alternatives for calculating the fatigue life based on the result of the numerical simulations: The first alternative is to apply the actual temperature time history to the FE model and find the stress time histories at critical locations by conducting a coupled thermal-stress analysis. This procedure, however, is computationally expensive because of the duration of the temperature time history that needs to be applied to the structure step-by-step. The second alternative, which has been employed in the current study, includes the following steps for calculating the fatigue life: (1) The rainflow cycle counting is performed on the averaged temperature time histories and the number of cycles for each temperature range is derived, (2) The derived temperature ranges are applied to the FE model and a coupled thermal-stress analysis is conducted, and (3) The stresses generated in the truss elements due to each temperature range are obtained and then further employed for the fatigue life calculations. Further to computational efficiency, the second alternative is believed to provide acceptable results considering that the stress ranges have the same number of cycles as the temperature ranges.
Figure 3-11 shows the total number of cycles for the entire duration of monitoring for both Truss A and B. Based on this figure, the number of cycles in each temperature range is consistent for both trusses, which confirms that both of them experience similar weather conditions. A review of the recorded cycles indicates that more than 97% of them are associated with the temperature ranges less than 50 °F, which is known to be representative of the expected diurnal changes in most of the United States. The outcome of the fatigue analysis using the results obtained from the numerical simulations for Truss A has been summarized in Table 3-1. This table shows that the fatigue life of Truss A is $1 \times \frac{1}{7.27 \times 10^{-4}} = 1375$ years, which can be deemed equivalent to infinite.

**Comparison of Steel and Aluminum DMS-Support Structures**

Considering the recent transition from aluminum to steel DMS-support structures, the current study provides a unique opportunity to investigate the role of materials in the fatigue life of this category of structures under temperature-induced stresses. For this purpose, the two steel DMS-support structures under consideration are compared with an aluminum DMS-support structure, which has been in service in the same geographic area. The fatigue life assessment of the aluminum DMS-support structure has been performed by Chang et al [26]. For comparison purposes, an analytical approach is employed to find the fatigue life of a representative truss element with steel and aluminum materials. To complete the calculations, the following assumptions have been made: (1) Both structures experience the same diurnal temperature changes; and (2) The fatigue is induced by only diurnal temperature changes and other sources of excitation are neglected.

Using the Hook’s law, the thermal stress can be found from $\Delta \sigma = E\alpha \Delta T$, where $\Delta \sigma$ is the stress change, $E$ is the modulus of elasticity, $\alpha$ is the linear coefficient of thermal expansion, and
$\Delta T$ is the temperature change. Using this equation, the ratio of the stress change in an aluminum with respect to a steel truss element can be stated as:

$$\frac{\Delta \sigma_{AL}}{\Delta \sigma_{ST}} = \frac{E_{AL}\alpha_{AL}}{E_{ST}\alpha_{ST}}$$

(3)

where $\Delta \sigma_{AL}$ and $\Delta \sigma_{ST}$ are the stress range of the aluminum and steel structure, $E_{AL}$ and $E_{ST}$ are the modulus of elasticity of aluminum and steel, and $\alpha_{AL}$ and $\alpha_{ST}$ are the coefficient of thermal expansion of the aluminum and steel structure, respectively. Since the ratios of $\frac{E_{AL}}{E_{ST}}$ and $\frac{\alpha_{AL}}{\alpha_{ST}}$ can be assumed equal to $\frac{10^7\text{psi}}{2.9\times10^7\text{psi}} = 0.34$ and $\frac{24\times10^{-6}(1/\text{K})}{12\times10^{-6}(1/\text{K})} = 2$, respectively, the ratio of the stress change in Equation 3 is found to be 0.69. It must be noted that this ratio is in fact equivalent to $\frac{S_{AL}}{S_{ST}}$, which is needed for fatigue life assessment in Equation 1.

Based on the AASHTO Specifications [39], the ratio of the constant value of $A$ for the connections of an aluminum with respect to a steel truss element is equal to 0.2. This is because of the tube to tube connections used in the aluminum and the slotted tube to gusset plate connection in the steel DMS-support structures. Using the $S$-$N$ curves, the ratio of the cycles needed to cause a fatigue-induced failure for an aluminum with respect to a steel structure can be obtained from:

$$\frac{N_{AL}}{N_{ST}} = \left(\frac{S_{ST}}{S_{AL}}\right)^3 \left(\frac{A_{AL}}{A_{ST}}\right) = \left(\frac{1}{0.69}\right)^3 (0.2) = 0.61$$

(4)

The ratio of the fatigue-induced damage in an aluminum with respect to a steel structure can be derived using the following equation:

$$\frac{D_{AL}}{D_{ST}} = \left(\frac{N_{ST}}{N_{AL}}\right)^{\frac{n_{AL}}{n_{ST}}}$$

(5)
where $\frac{n_{AL}}{n_{ST}}$ is the ratio of the number of the stress range cycles of an aluminum with respect to a steel structure. In this case, the number of stress range cycles is equal to the number of temperature range cycles. As it is assumed that both structures are in similar weather conditions, they experience the same number of temperature (and as a result stress) range cycles. Thus, $\frac{n_{AL}}{n_{ST}}$ is equal to 1.0, and the ratio of the fatigue-induced damage is found to be 1.64 in Equation 5. This demonstrates that the expected damage to the new steel DMS-support structures with a slotted tube to gusset plate connection detail is 64% less than that of existing aluminum DMS-support structures with a tube to tube connection detail.

Further to the fatigue performance assessment of individual aluminum and steel truss elements, the fatigue life comparison can be extended to the entire structure. The correlation between the temperature and stress ranges derived from the FE models of the aluminum and steel structures is shown in Figure 3-12. Using this figure, the stress ranges for the most critical truss joints are obtained for fatigue analysis. The outcome of fatigue life analysis on the most critical joint of the aluminum DMS-support structure has been provided in Table 3-2. Consistent with Chang et al. [26], the fatigue life of $\frac{1}{2.30 \times 10^{-2}} = 44$ years is obtained from this analysis. A direct comparison between the two materials shows that the fatigue life of the aluminum structure is significantly less than that of the steel structure, despite the fact that the stresses that the steel structure experiences are consistently higher than those experienced by the aluminum structure. This is completely in line with the findings from the analytical study performed on both materials.
Conclusions

A comprehensive field study supported with FE numerical simulations was conducted on two steel overhead DMS-support structures located in Iowa. For the field study, the DMS-support structures were instrumented using vibrating-wire gauges for a long-term structural health monitoring program. Through this program, temperature and strain data were recorded from multiple truss elements in service. In addition to the field study, the FE models of this category of structures were developed and validated to further investigate the potential thermal damage and identify the most vulnerable members. The outcome of this systematic effort was employed for a detailed fatigue life analysis, which showed that both steel structures under consideration have an infinite fatigue life with a satisfactory structural performance. This study was then extended to compare the fatigue life of the new steel DMS-support structures with the existing aluminum ones. For this purpose, a set of analytical calculations and numerical simulations were performed at both element and structure levels. It was found that while the steel trusses commonly experience stresses higher than those generated in the aluminum trusses (under similar weather conditions), their fatigue life still remains superior. This strongly supports the recent transition initiated by transportation agencies to use steel DMS-support structures, instead of aluminum ones.

References


[2] Hong HP, Zu GG, King JPC. Reliability consideration for fatigue design of sign,


## Tables and Figures

**Table 3-1.** Fatigue analysis results of Truss A.

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<th>No. of cycles</th>
<th>Stress range (ksi)</th>
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**Table 3-2.** Fatigue analysis results of the aluminum DMS-support structure.

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Figure 3-1. Location of two steel DMS-support structures under consideration through a long-term field monitoring program.
Figure 3-2. Instrumentation layout used for Truss A and B.
Figure 3-3. Strain and temperature time histories obtained from Truss A and B during a cold month.
Figure 3-4. Strain and temperature time histories obtained from Truss A and B during a hot month.
**Figure 3-5.** Stress ranges and their respective cycles for a set of sensor locations from Truss A and B.
Figure 3-6. Averaged, maximum, and minimum value of temperature time histories recorded in each truss structure during a one-year period.
Figure 3-7. Correlation of strain change, temperature, and time of data collection for selected sensors from Truss A and B.
Figure 3-8. Details of the FE models of the sign-support structures generated for numerical simulations.
Figure 3-9. Comparison of strain time histories obtained from the FE model and during the field study.
Figure 3-10. Rainflow cycle counting of actual and finite element model simulated strain time histories.

Figure 3-11. Summary of rainflow cycle counting results.
Figure 3-12. Correlation between temperature range and the stress range derived from the FE models of steel and aluminum DMS-support structures.
CHAPTER 4: A MULTI-STRESSOR PERSPECTIVE ON FATIGUE PERFORMANCE OF SIGN-SUPPORT STRUCTURES

Abstract

A multi-stressor perspective for the analysis and design of civil infrastructure components is essential to ensure their long-term performance and safety under various environmental and mechanical stressors. This has been implemented to a good extent for buildings and bridges. However, there is a gap in the body of knowledge concerning how such a perspective can be extended to the structures that support highway signs, luminaries, and traffic signals. This has been the motivation of the current study to investigate the fatigue performance of Dynamic Message Sign (DMS)-support structures under the combined effects of diurnal temperature changes and natural wind excitations. For this purpose, a steel DMS-support structure in service has been instrumented using an array of vibrating-wire gauges and an anemometer. Through recording the sensory data for close to one year, the time-histories of a variety of parameters contributing to the fatigue life of this important category of structures are collected. The parameters of interest include strain, temperature, and wind speed and direction. This field monitoring program has been paired with detailed finite-element (FE) simulations. Upon validation of the FE model, the individual and combined effects of the contributing stressors are quantified to understand the fatigue performance of DMS-support structures under multiple stressors of different nature. This leads to the identification of the most fatigue-critical elements, which are found to be different under temperature and wind loads. The current study is then extended to use historical temperature and wind data with a focus on the investigation of wind directionality effects. The outcome of this systematic effort not only contributes to the long-term performance and safety of DMS-support
structures, but also paves the way to implement similar multi-stressor perspectives for other critical transportation infrastructures in service.

Keywords: Fatigue; Multi-Stressor Analysis; Field Monitoring; Finite-Element Simulations; Temperature; Wind Excitation

Introduction

The concepts of multi-hazard and multi-stressor analysis and design for civil infrastructure components have gained a significant attention since the past decade [1–11]. This is primarily because of the success of the developed concepts in addressing a number of issues originating from conventional analysis and design approaches. Among them is taking into consideration the interactions and interdependencies of the hazards and stressors that a structure is exposed to during its service life. A review of the existing literature shows that the analysis and design of buildings under multiple hazards and stressors have been the subject of several studies. For example, a combinatorial optimization framework was introduced by Chulahwat and Mahmoud (2017) for multi-hazard design of building systems [12]; Aly and Abburu (2015) examined the impact of wind and earthquake-induced loads on the structural behavior of high-rise buildings [13]; and Gerasimidis et al. (2017) studied the resilience of steel buildings using the sequential method of multi-hazard analysis for post-event fire [14]. Similar efforts exist for the analysis and design of bridges under multiple hazards and stressors. For example, Alipour et al. (2011) investigated the performance of deteriorating bridges under high seismic risks [4]; Ghosh and Padgett (2012) explored the impact of multiple component deterioration on the seismic vulnerability of bridge structures [15]; and Alipour et al. (2013) introduced a reliability-based framework for bridges under both scour and earthquake hazards [5]. In separate studies, the effects of aging mechanisms and environmental stressors on the capacity and performance of various building and bridge
structures have been investigated during their expected service life [16–21]. Despite the great progress made in the multi-hazard and multi-stressor analysis and design of buildings and bridges, however, there is a gap in extending the developed concepts to the structures that support highway signs, luminaries, and traffic signals.

A variety of transportation infrastructures are relied on a daily basis to guide the traffic and communicate the necessary information with pedestrians and motorists. Among them, bridge-type overhead truss structures, which are capable of supporting Dynamic Message Signs (DMS), have gained a significant attention, primarily because DMSs can provide drivers with alerts regarding traffic issues and safety concerns in close to real time. In addition, this category of overhead highway signs offers the transportation agencies an economic option, as they are shipped to the site in a prefabricated form and can be installed in a short period of time with minimum traffic disruptions. The common structural characteristics of DMS-support structures include a relatively low damping ratio, high flexibility, and small mass. Such characteristics make them particularly vulnerable to fatigue-induced damage and failure [22]. Diurnal temperature changes and natural wind excitations have been recognized as two main causes of fatigue [23–28].

In DMS-support structures, welded parts in the main truss, truss-to-column, and column-to-base plate connections are the most critical parts, vulnerable to fatigue-induced damage due to high stress concentration [29–31]. Crack initiation has been frequently reported within the toe and leg of fillet welds, with the possibility of propagation to the other structural members [22]. Among the conducted studies, stress fields were modeled by Barle et al. (2011) under static and dynamic loads to investigate how the design of truss joints can be improved [32]. Experimental tests were conducted by Roy et al. (2010) to evaluate the crack modes, taking into consideration the weld geometry [24]. In a separate study, Rice et al. (2012) used a simplified numerical framework, as
well as field investigation, to understand the structural behavior of aluminum sign-support structures under wind-induced loads [33]. Kacin et al. (2012) conducted a fatigue life analysis for an overhead sign-support structure. Historical data along with FE simulations were employed to estimate the fatigue life in both pristine and damaged conditions [34]. Fouad et al. (2003) examined the effect of natural wind gusts on various types of overhead truss structures and proposed a strategic plan to further enhance the performance of sign-support structure [35]. The impact of vibrations induced by truck passage on the fatigue of a sign structure was studied by Huckelbridge and Metzger (2009) [36]. As confirmed in a later study performed by Constantinescu et al. [37], the truck passage was found to have a negligible contribution to fatigue [22,36,38]. The fatigue performance of high-mast, light poles was evaluated by Chang et al. (2009 and 2010) through a long-term field monitoring program [39,40]. This study was then extended to evaluate aluminum DMS-support structures under thermal loads [41].

Despite the contribution of the existing studies to evaluate fatigue in DMS-support structures due to individual temperature- and wind-induced loads, there is a gap in the literature concerning the combined effects of temperature and wind on the fatigue life of this important category of structures, which often experience both during their service life. This has been the primary motivation of the current study, which aims at establishing the very first multi-stressor perspective for the evaluation of DMS-support structures. For this purpose, a field study paired with detailed numerical simulations are conducted. The field study collects the necessary data representing the involved stressors (i.e., temperature and wind) and monitors the performance measures of a steel DMS-support structure in response to them. With the implementation of a dense sensor layout, strain, temperature, and wind speed and direction are recorded on a regular basis. While the field data provide a direct assessment of the strains and stresses that the individual
structural elements experience, they will be further utilized to validate a FE model, which is subjected to a number of loading scenarios. This model is employed to identify the most critical structural elements under fatigue induced by individual and combined stressors. The fatigue calculations are based on the rainflow cycle counting method [42] and the Miner’s fatigue damage accumulation rule [43]. The current study is then extended to evaluate wind directionality effects based on historical data available for a 50-year time window. This provides detailed information about the contribution of the involved stressors and illustrates how a site-specific assessment can affect the estimated fatigue life.

**Field Monitoring and Data Analysis**

To understand the fatigue performance of steel DMS-support structures recently added to the highways in Iowa (and beyond), a representative DMS-support structure close to Des Moines, Iowa has been selected for a long-term structural health monitoring program. Figure 4-1 shows the location of the structure on the map. The horizontal truss consists of three prefabricated parts connected together on site using bolts. The entire truss is supported by two prefabricated posts in the ends. The total length of the truss is 80 ft. (1 ft. = 0.3048 m) and the support posts are 31 ft. tall. Table 4-1 summarizes the geometry and material properties of the truss members in detail. A number of sensors, including vibrating-wire gauges and an anemometer, have been mounted on the DMS-support structure. The sensor layout consists of 18 gauges, which capture the temperature and strain changes at various structural members with a sampling rate of one per hour. Among the mounted sensors, seven of them are on the middle span of the truss, seven of them are on the side panel of the truss, and four of them are on the support posts. In addition to the vibrating-wire gauges, an anemometer has been installed at the top of the post support to obtain the site-specific
information of wind speed and direction with the frequency of one recording per hour. Figure 4-2 illustrates the location and label of the mounted sensors. Figure 4-3 provides more detailed information from the site regarding the overall configuration of the DMS-support structure and the attached gauges. With the sensor layout shown in Figure 4-2, field data are recorded for close to one year to capture both hot and cold seasons. A review of the temperature and strain time histories obtained from the individual sensors confirms that all of them work with no problem and reflect a similar pattern overall for the recorded data.

**Numerical Simulations under Temperature and Wind Loads**

To extend the scope of investigations to multi-stressor scenarios and identify the most critical parts of DMS-support structures, a detailed FE model has been generated using the ABAQUS software package. The truss elements are modeled with shell elements, which are capable of capturing stress concentrations at the connections. Figure 4-4 illustrates the FE model of the DMS-support structure under consideration along with a close view of connection details. The size and pattern of mesh have been decided after conducting an extensive sensitivity analysis, in such a way that the computational efficiency is achieved, while maintaining the accuracy and convergence. Upon validation of the FE model with the recorded field data, it is further employed for the prediction of temperature and wind-induced fatigue damage.

For the structural performance assessment of the DMS-support structure under diurnal temperature changes, a thermal analysis is performed. For this purpose, a rainflow cycle counting is performed on the averaged temperature time histories obtained from the site. This provides the number of cycles for various temperature ranges. The temperature ranges are then applied to the
FE model and a coupled thermal-stress analysis is conducted. Finally, the stresses generated in the truss elements due to each temperature range are collected for fatigue life calculations. Further to computational efficiency, this approach provides accurate results considering the fact that the number of cycles for each stress range is identical to that for the associated temperature range.

Further to thermal analysis, the DMS-support structure is subjected to wind simulations. For this purpose, wind speed time histories are generated with different mean values using a detailed procedure. The wind speed, $u$, is a function of time, $t$, and height from the ground, $z$. According to the empirical power law profile, the mean wind speed, $u_z$, at the height, $z$, can be expressed as:

$$u_z = u_{z_r} \left( \frac{z}{z_r} \right)^P$$  \hspace{1cm} (1)

where $z_r$ is the reference height, and $P$ is the Hellmann exponent determined based on the terrain. Considering that wind speed has a random nature in both time and height, the wind load simulations take into consideration eddies of different sizes conveyed by the mean flow. The turbulence energy distribution is commonly expressed by power spectral density (PSD) functions. There are several mathematical representations for the micro-meteorological pick of wind fluctuation in the form of power spectrum. Among them, the Fichtl and McVehill model provides a general mathematical expression as:

$$\frac{f S_f}{\sigma^2} = \frac{4f^*}{(1+\alpha f^* \beta)^{3\beta}}$$  \hspace{1cm} (2)

where $f$, $S_f$, and $\sigma^2$ are the frequency, PSD, and variance of wind speed, respectively. In Equation (2), the following parameters are included:
\[ \alpha = 1.5 \frac{4^\beta}{b^\beta} \]  
(3)

\[ b = \frac{1.5^\beta \beta \Gamma \left( \frac{5}{3\beta} \right)}{\Gamma \left( \frac{1}{\beta} \right) \Gamma \left( \frac{2}{3\beta} \right)} \]  
(4)

\[ f^* = \frac{f L_{ux}}{u_z} \]  
(5)

where \( \Gamma(.) \) is Gamma function, and \( L_{ux} \) is the wind’s turbulence scale. Equation (2) can be simplified with common PSDs, which assume, for example, \( \beta = 1 \) (Kaimal’s), \( \beta = \frac{5}{3} \) (Panofsky’s), and \( \beta = 2 \) (Kalman’s) [45]. The wind spectra, in general, can be divided into two main categories of height independent (e.g., Davenport’s spectrum) and height dependent (e.g., Kaimal’s spectrum) [46]. To take into account the height of the DMS-support structure, the Kaimal’s spectrum is employed using the following equation:

\[ \frac{u_z S_{fz}}{u_z^2} = \frac{200z}{\left(1 + 50 \frac{f z}{u_z}\right)^{5/3}} \]  
(6)

where \( S_{fz} \) is the PSD function at the height of \( z \) above the ground, and \( u_* \) is the friction (or shear) velocity.

In the current study, the wind speed time history, \( u_t \), at the height of interest, i.e., \( z \), is estimated using the summation of two components: (i) mean wind speed, \( u_z \), and (ii) fluctuating wind, \( \tilde{u}_{z,t} \), which is a function of both time and height:

\[ u_t = u_z + \tilde{u}_{z,t} \]  
(7)
\[ \ddot{u}_{z,t} = \sum_{k=1}^{n} \sqrt{2(S_{fz})_k \Delta f} \cos(2\pi f_k t + \varphi_k), \quad f_k = k\Delta f \] (8)

where \( n \) is frequency number, \((S_{fz})_k\) is the PSD function calculated from Equation 6 for the \( k \)-th frequency \((f_k)\), \( \Delta f \) is the frequency increment, and \( \varphi_k \) is the Gaussian random phase angle, which is uniformly distributed between 0 and \( 2\pi \). It has been shown in the literature that the wind speed time histories generated by Equation (7) compare well with the measured wind records [47]. Figure 4-5 shows the wind time histories generated for the mean wind speed of \( u_z = 2.5 \text{ mph to } 42.5 \text{ mph with 5.0 mph increments. This will be further employed to calculate the pressure time history acting on the structure, } p_t, \) using the Bernoulli equation:

\[ p_t = 0.5 \rho C_d u_t^2 \] (9)

where \( \rho \) is the air density, and \( C_d \) is the drag coefficient. Drag is defined as the resistance caused by an object in a flow, which increases with the flow turbulence. The drag coefficient is dependent on the shape and size of the object in the flow [45]. For this study, \( C_d \) is assumed equal to 1.2 for the truss members and 1.7 for the DMS cabinet following AASHTO (2015) [44]. The simulated pressure time histories are then applied to the FE model one by one to identify the most critical elements and their associated fatigue life. It must be noted that the current study primarily focuses on the effects of temperature and natural wind gusts. This is because other wind-induced loads, such as vertex shedding and galloping, as well as loads induced by the passage of trucks underneath the structure, are reported to have minimal effects on the overall fatigue life (Chang et al. 2014).
Multi-Stressor Fatigue Life Investigation

The stress life method has been employed for the fatigue analysis of the DMS-support structure under consideration in the current study. This method is based on the relation between the stress ranges and number of cycles, commonly referred to as S-N curves:

\[ N_{\Delta S} = \frac{A}{\Delta S^3} \]  

(10)

where \( \Delta S \) is a given stress range, \( N_{\Delta S} \) is the number of cycles required for the given stress range to cause failure, and \( A \) is a constant determined based on the connection details. This constant has been provided by AASHTO (2015) [44] for various connections used in sign-support structures. In this study, the DMS-support structure has a slotted tube to gusset plate connection with a coped hole, which is categorized as class \( E \) by AASHTO (2015) [44]. The \( A \) constant for this category is \( 11 \times 10^8 \text{ psi}^3 \). Once \( N_{\Delta S} \) is calculated using Equation 10, the damage induced by that given stress range is predicted using the Miner’s rule [43] as \( D_{\Delta S} = \frac{n_{\Delta S}}{N_{\Delta S}} \), where \( n_{\Delta S} \) is the number of cycles of that given stress range experienced by the structure. \( n_{\Delta S} \) is determined by performing a rainflow cycle counting analysis on the stress time histories. The total fatigue damage is then calculated by \( D_T = \sum D_{\Delta S} \). The Miner’s rule indicates that the failure occurs once the total damage, \( D_T \), becomes equal to one. To conduct a multi-stressor fatigue life analysis, data recorded from the field, site-specific historical data, and data obtained from the numerical simulations are utilized.

Fatigue Analysis under Individual and Multiple Stressors

Considering that the structural response of the DMS-support structures remains in the linear range under diurnal temperature changes and natural wind gust excitations, the fatigue-
induced damage can be predicted through the accumulation of damage caused by individual stressors. Using this concept, the first stressor in this study is diurnal temperature changes, which have been captured by the vibrating-wire gauges mounted for the long-terms monitoring of the DMS-support structure under consideration. Using the recorded temperature time histories, the number of cycles for a range of $\Delta T$ is extracted using the rainflow cycle counting method for a duration of one year (see the first two rows of Table 4-2). It must be noted that the obtained number of cycles also represents the number of cycles of the corresponding stress range, $\Delta S$, predicted for each individual truss element. This prediction is based on a set of thermal analyses of the developed FE model under the temperature range of 5 °F to 85 °F with 10 °F increments. To express the correlation between the stress change and temperature change, a matrix notation is utilized by considering $n$ critical elements and $m$ temperature increments:

$$
\begin{bmatrix}
\Delta S_{11} & \cdots & \Delta S_{1m} \\
\vdots & \ddots & \vdots \\
\Delta S_{n1} & \cdots & \Delta S_{nm}
\end{bmatrix} = E_{n\times1}(\Delta T_{m\times1})^T + B_{n\times1}
$$

(11)

where $\Delta S_{nm}$ is the stress range of the $n$-th element under the $m$-th temperature increment, $E$ is the matrix of coefficients for the temperature increments, $\Delta T$ is the temperature increment matrix, and $B$ is the matrix of constant value of equations. Once the stress ranges are calculated using Equation (11), $N_{\Delta S}$ can be determined. Having $N_{\Delta S}$ and the number of cycles for each stress range, i.e., $n_{\Delta S}$, during one year, the fatigue-induced damage is identified. Table 4-2 summarizes the fatigue damage calculations for three most critical elements of the structure under thermal loads. As can be seen in Table 4-2, the implementation of the validated FE model greatly helps expand the investigations beyond the instrumented truss elements, leading to the identification of those elements that are suspected to be most vulnerable to fatigue.
The second cause of fatigue-induced damage considered in the current study is wind loads. To have a realistic estimate of wind loads, an anemometer has been attached to the top of the DMS-support structure. This anemometer records the intensity and direction of natural wind gusts every one hour. Figure 4-6 presents a wind rose diagram generated using the field data. A total of 20 bins are considered for this diagram. Wind speeds higher than 35 mph are found to occur only rarely, thus, have not been included in this diagram. For fatigue analysis, it is assumed that wind can blow in 8 main directions, i.e., N, NW, W, SW, S, SE, E, and NE. The wind rose diagram regenerated based on the 8 main wind directions has been included in Figure 4-6. The percentage of occurrence of various wind directions and speeds has been summarized in Figures 4-7 and 4-8, respectively. Considering the skewness to the left in Figure 4-8, wind speeds below 10 mph have the highest probability of occurrence, while this probability drastically decreases as the wind speed elevates.

For structural analysis purposes, the 8 main wind directions are combined to 4 primary directions perpendicular to the plane of the DMS-support structure. The primary four directions are expressed as N-S, NW-SE, W-E, and SW-NE. Considering that the probability of occurrence of a given wind speed and direction is essential to perform an accurate fatigue life calculation, a statistical analysis is performed using independent and joint probabilities. For this purpose, two probabilistic events are defined: \( P(W_D) \) is the probability of wind blowing in a given primary direction, and \( P(W_B) \) is the probability of wind blowing in a given speed range. For example, \( P(W_D) \) is equal to 22.20% for the W-E primary direction (from Figure 4-7), and \( P(W_B) \) is equal to 38.47% for wind speeds between 0 mph to 5 mph (from Figure 4-8). In the current study, however, the probability of interest is the joint probability of experiencing a given wind speed range in a given primary direction. This joint probability can be expressed as \( P(W_D \cap W_B) = P(W_D)P(W_B|W_D) \).
Knowing the joint occurrence probabilities of wind-induced loads, the fatigue life is estimated using the range and number of stress cycles obtained from the FE simulations performed on the DMS-support structure subjected to wind excitations. Table 4-3 summarizes the procedure for three most critical elements of the structure under natural wind loads. Considering that the DMS-support structure is oriented perpendicular to the W-E primary direction, the results provided in Table 4-3 do not include the other three primary directions, which are found to have a minimal contribution to the overall fatigue life. Upon the calculation of the fatigue-induced damage due to diurnal temperature changes and natural wind excitations, the accumulated effects of them are predicted using the superposition rule. Table 4-4 shows the results of the accumulated fatigue-induced damage. Since the type of each individual stressor, i.e., due to wind and temperature, is unique, the critical elements of the structure are different under each of them. Based on the final results, all of the critical elements show infinite fatigue life, which is commonly defined as a life greater than the 50-year design life.

**Fatigue Analysis Based on Historical Data Considering Wind Directionality Effects**

The fatigue life investigation performed based on the recorded temperature and wind data for a one year time window is extended in this section to (i) benefit from site-specific historical data, which are available for a long period of time, and (ii) examine the wind directionality effect, which reflects how the same wind-induced load can differently influence the fatigue life of the structure, depending on its orientation with respect to the region’s most frequent wind direction. For this purpose, the temperature and wind data recorded between January 1965 and December 2015 are obtained from the Automated Surface Observing Systems (ASOS) program for Des Moines International Airport. As shown in Figure 4-1, the air distance between the selected
weather station and the DMS-support structure under consideration is less than 10 miles. This ensures that both sites experience similar temperature, as well as wind speed and direction. The duration of the recorded historical data used for this study is chosen to be 50 years, which is consistent with the design life of DMS-support structures. Considering the year-to-year changes in the temperature and wind patterns, this is expected to provide a detailed insight regarding the fatigue life of this category of structures.

According to the ASOS, both temperature and wind data are recorded every hour. The wind data are captured at the height of 33 ft. from the ground, which is consistent with the elevation of the horizontal truss of the DMS-support structure. Figure 4-9 depicts the temperature time history extracted from the Des Moines International Airport’s weather station. The lowest and highest temperature reported during this 50-year time period is approximately -30 ºF and 110 ºF, respectively. Compared to the temperature data recorded at the field during the one-year monitoring program, this time history provides the most extreme temperature values. Using the rainflow cycle counting method, the number of cycles for various temperature ranges are extracted from the temperature time history and reported for one year in Figure 4-9. In a similar effort, the historical wind data are evaluated to generate the wind rose diagram of the wind speed and direction for the 50-year time window. A review of the diagram with 20 bins shows that the dominant wind direction is consistent with what recorded at the DMS-support structure (as shown in Figure 4-8). Based on this diagram, the highest recorded wind speed is close to 55 mph. Figure 4-11 summarizes the percentage of occurrence of each wind direction, which can be employed to predict $P(W_D)$. This figure shows that the dominant primary direction is N-S with the probability of occurrence of 35.26%. Figure 4-12 illustrates the joint probability of occurrence of each wind speed for each primary wind direction. Based on this figure, the W-E direction has the highest
probability of occurrence in low wind speeds (under 10 mph), while the NW-SE direction becomes
dominant as the wind speed increases, particularly to the range above 15 mph.

To include the wind directionality effect in the fatigue life prediction of the DMS-support
structures, the fatigue life analysis is performed under the four primary wind directions using the
full historical data. Table 4-5 presents the details of the fatigue life calculation for one of the most
critical elements (i.e., Element No. 412). This follows the same procedure implemented for fatigue
life analysis under diurnal temperature changes and natural wind excitations using the field
monitoring data. Figure 4-13 summarizes the multi-stressor fatigue life of three most critical
elements under various primary wind directions. Based on the calculated fatigue life, the NW-SE
is the most critical primary direction, while the NE-SW is the least critical one. Although the
orientation of DMS-support structures is commonly dictated by the highway path, it is found that
the risk of wind-induced fatigue can significantly change, depending on the dominant wind
direction. As the temperature and wind data are available for most of the existing highways, the
introduced multi-stressor perspective is expected to help adjust the estimated fatigue life based on
real exposure conditions.

Conclusions

Development of multi-hazard and multi-stressor approaches for the analysis and design of
structures has been a focus of several studies in the literature. While a significant body of
knowledge has been created in this regard for buildings and bridges, there is a gap to implement
such approaches for the structures that support highway signs, luminaries, and traffic signals. To
address this gap, the current study investigated the fatigue life of DMS-support structures with a
multi-stressor perspective, which captured two primary stressors contributing to fatigue-induced damage, i.e., diurnal temperature changes and natural wind excitations. For this purpose, a comprehensive field study paired with detailed numerical simulations was employed. Through the field study, a DMS-support structure located close to Des Moines, Iowa was instrumented with an array of sensors to record strain, temperature, and wind speed and direction. A FE model of the same DMS-support structure was then generated and validated with the field data. Through a detailed fatigue life analysis under both individual and combined stressors, it was found that the most fatigue-critical elements can change drastically depending on the stressor under consideration. This highlighted the importance of benefiting from a multi-stressor perspective to obtain an accurate estimate of the vulnerability of individual elements with accumulated fatigue-induced damage due to various environmental and/or mechanical stressors. The developed approach was then extended to utilize the historical temperature and wind data, which not only reflect the long-term patterns of fluctuation, but also capture the dominant wind directions. This was an important addition, as it demonstrated how a site-specific assessment can influence the predicted fatigue life. The outcome of this study is expected to go beyond the long-term performance and safety assessment of DMS-support structures, as similar multi-stressor perspectives can be implemented for other transportation infrastructures in service.

References


[38] Albert MN. Field Testing of Cantilevered Traffic Signal Structures under Truck-Induced Gust Loads. The University of Texas at Austin, 2006.


**Tables and Figures**

**Table 4-1.** Details of the main structural elements of the DMS-support structure under consideration.

<table>
<thead>
<tr>
<th>3D view</th>
<th>Members</th>
<th>Material</th>
<th>Outer diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="3D view" /></td>
<td>All the internal truss members, i.e., vertical, horizontal, and interior diagonal members</td>
<td>Standard steel pipes</td>
<td>3 in.</td>
</tr>
<tr>
<td><img src="image" alt="3D view" /></td>
<td>Main chords</td>
<td>Extra-strong steel pipes</td>
<td>5 in.</td>
</tr>
<tr>
<td><img src="image" alt="3D view" /></td>
<td>Support posts</td>
<td>HSS pipes</td>
<td>14 in.</td>
</tr>
</tbody>
</table>
Table 4-2. Fatigue damage calculation for three most critical elements of the DMS-support structure under thermal loads.

<table>
<thead>
<tr>
<th>$\Delta T (^{\circ}F)$</th>
<th>5</th>
<th>15</th>
<th>25</th>
<th>35</th>
<th>45</th>
<th>55</th>
<th>65</th>
<th>75</th>
<th>85</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of cycles</td>
<td>662</td>
<td>114</td>
<td>153</td>
<td>41</td>
<td>10</td>
<td>3</td>
<td>0</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Element No. 19152</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress range (ksi)</td>
<td>2.6</td>
<td>7.4</td>
<td>12.3</td>
<td>17.1</td>
<td>21.5</td>
<td>26.7</td>
<td>31.5</td>
<td>36.3</td>
<td>41.0</td>
</tr>
<tr>
<td>$N_{\Delta S}$</td>
<td>6.26E+7</td>
<td>2.71E+6</td>
<td>5.91E+5</td>
<td>2.20E+5</td>
<td>1.11E+5</td>
<td>5.78E+4</td>
<td>3.52E+4</td>
<td>2.30E+4</td>
<td>1.60E+4</td>
</tr>
<tr>
<td>Damage</td>
<td>1.06E-5</td>
<td>4.20E-5</td>
<td>2.59E-4</td>
<td>1.86E-4</td>
<td>9.03E-5</td>
<td>5.19E-5</td>
<td>0</td>
<td>8.70E-5</td>
<td>0.00E+0</td>
</tr>
<tr>
<td>Element No. 16433</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress range (ksi)</td>
<td>2.3</td>
<td>7.1</td>
<td>11.8</td>
<td>16.6</td>
<td>21.3</td>
<td>26.1</td>
<td>30.8</td>
<td>35.5</td>
<td>40.3</td>
</tr>
<tr>
<td>$N_{\Delta S}$</td>
<td>8.51E+7</td>
<td>3.09E+6</td>
<td>6.64E+5</td>
<td>2.42E+5</td>
<td>1.14E+5</td>
<td>6.22E+4</td>
<td>3.77E+4</td>
<td>2.45E+4</td>
<td>1.68E+4</td>
</tr>
<tr>
<td>Damage</td>
<td>7.78E-6</td>
<td>3.69E-5</td>
<td>2.30E-4</td>
<td>1.70E-4</td>
<td>8.80E-5</td>
<td>4.82E-5</td>
<td>0</td>
<td>8.16E-5</td>
<td>0.00E+0</td>
</tr>
<tr>
<td>Element No. 19458</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress range (ksi)</td>
<td>1.9</td>
<td>6.4</td>
<td>10.8</td>
<td>15.2</td>
<td>19.6</td>
<td>24.0</td>
<td>28.5</td>
<td>32.9</td>
<td>37.3</td>
</tr>
<tr>
<td>$N_{\Delta S}$</td>
<td>1.53E+8</td>
<td>4.29E+6</td>
<td>8.79E+5</td>
<td>3.13E+5</td>
<td>1.46E+5</td>
<td>7.91E+4</td>
<td>4.77E+4</td>
<td>3.09E+4</td>
<td>2.12E+4</td>
</tr>
<tr>
<td>Damage</td>
<td>4.33E-6</td>
<td>2.66E-5</td>
<td>1.74E-4</td>
<td>1.31E-4</td>
<td>6.87E-5</td>
<td>3.79E-05</td>
<td>0</td>
<td>6.47E-5</td>
<td>0.00E+0</td>
</tr>
</tbody>
</table>

$sum = 7.27E-4$ for Element No. 19152

$sum = 6.62E-4$ for Element No. 16433

$sum = 5.07E-4$ for Element No. 19458
Table 4-3. Summary of fatigue life calculation for three most critical elements of the DMS-support structure under natural wind loads.

<table>
<thead>
<tr>
<th>Wind Speed (mph)</th>
<th>2.5</th>
<th>7.5</th>
<th>12.5</th>
<th>17.5</th>
<th>22.5</th>
<th>27.5</th>
<th>32.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability of wind in the W-E primary direction</td>
<td>0.222</td>
<td>0.222</td>
<td>0.222</td>
<td>0.222</td>
<td>0.222</td>
<td>0.222</td>
<td>0.222</td>
</tr>
<tr>
<td>Probability of occurrence of the wind speed range in the W-E primary direction</td>
<td>0.3847</td>
<td>0.3946</td>
<td>0.145</td>
<td>0.0568</td>
<td>0.0162</td>
<td>0.0018</td>
<td>0.0009</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Damage due to each wind speed during 1 year</th>
<th>Element No. 6971</th>
<th>Element No. 412</th>
<th>Element No. 2552</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.75E-10</td>
<td>1.08E-5</td>
<td>1.26E-3</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>2.01E-5</td>
<td>1.33E-3</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>3.05E-6</td>
<td>4.14E-4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weighted damage due to each wind speed during 1 year</th>
<th>Element No. 6971</th>
<th>Element No. 412</th>
<th>Element No. 2552</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.35E-11</td>
<td>9.46E-7</td>
<td>4.06E-5</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>1.76E-6</td>
<td>4.29E-5</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>2.67E-7</td>
<td>1.33E-5</td>
</tr>
</tbody>
</table>
Table 4-4. Accumulated fatigue-induced damage, taking into consideration the stresses due to both diurnal temperature changes and natural wind excitations.

<table>
<thead>
<tr>
<th>Wind induced damage</th>
<th>Critical elements under wind load</th>
<th>Critical elements under temperature load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Element No. 6971</td>
<td>Element No. 412</td>
</tr>
<tr>
<td>Temperature induced damage</td>
<td>1.21E-3</td>
<td>1.45E-3</td>
</tr>
<tr>
<td>Total accumulated damage</td>
<td>1.55E-7</td>
<td>1.06E-7</td>
</tr>
<tr>
<td>Fatigue life (year)</td>
<td>826</td>
<td>691</td>
</tr>
</tbody>
</table>
Table 4-5. Fatigue damage calculation for a select element.

<table>
<thead>
<tr>
<th>Temp. Induced Fatigue Damage</th>
<th>$\Delta T$ (°F)</th>
<th>5</th>
<th>15</th>
<th>25</th>
<th>35</th>
<th>45</th>
<th>55</th>
<th>65</th>
<th>75</th>
<th>85</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of cycles</td>
<td>774</td>
<td>165</td>
<td>142</td>
<td>37</td>
<td>15</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Stress range (ksi)</td>
<td>0.412</td>
<td>0.505</td>
<td>0.598</td>
<td>0.691</td>
<td>0.784</td>
<td>0.877</td>
<td>0.970</td>
<td>1.063</td>
<td>1.156</td>
<td></td>
</tr>
<tr>
<td>$N_{\Delta S}$</td>
<td>1.57E+10</td>
<td>8.54E+9</td>
<td>5.14E+9</td>
<td>3.33E+9</td>
<td>2.28E+9</td>
<td>1.63E+9</td>
<td>1.21E+9</td>
<td>9.16E+8</td>
<td>7.12E+8</td>
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</tr>
<tr>
<td>$D$</td>
<td>4.92E-8</td>
<td>1.93E-8</td>
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<td>1.11E-8</td>
<td>6.57E-9</td>
<td>1.23E-9</td>
<td>8.30E-10</td>
<td>1.09E-9</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wind Induced Fatigue Damage</th>
<th>Wind Speed (mph)</th>
<th>Damage during one year</th>
<th>N-S</th>
<th>SW-NE</th>
<th>W-E</th>
<th>NW-SE</th>
<th>P(W_d)</th>
<th>W_d</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.5</td>
<td>0.00</td>
<td>0.35</td>
<td>0.15</td>
<td>0.22</td>
<td>0.28</td>
<td>0.10</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>7.5</td>
<td>2.01E-5</td>
<td>0.35</td>
<td>0.15</td>
<td>0.22</td>
<td>0.28</td>
<td>0.38</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>1.33E-3</td>
<td>0.35</td>
<td>0.15</td>
<td>0.22</td>
<td>0.28</td>
<td>0.38</td>
<td>0.40</td>
</tr>
<tr>
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Figure 4-1. Location of the DMS-support structure under consideration.

Figure 4-2. Instrumentation layout form the side view.
Figure 4-3. Overall configuration and instrumentation details of the DMS-support structure used for the field study.
Figure 4-4. FE model of the DMS-support structure, including a close view of connections and the mesh pattern of truss elements.

Figure 4-5. Simulated wind speed time histories for a range of mean wind speeds.
Figure 4-6. Analysis of wind data recorded in the field: (a) wind rose diagram with 20 bins, (b) wind rose diagram with 8 main directions.

Figure 4-7. Percentage of occurrence of each wind direction based on the field recorded data.
Figure 4-8. Percentage of occurrence of each wind speed based on the field recorded data.

Figure 4-9. Analysis of historical temperature data obtained from the ASOS for Des Moines International Airport: (a) temperature time history, and (b) number of cycles for various temperature ranges.
Figure 4-10. Analysis of historical wind data obtained from the ASOS for Des Moines International Airport: (a) wind rose diagram with 20 bins, and (b) wind rose diagram with 8 main directions.

Figure 4-11. Percentage of occurrence of each wind direction.
**Figure 4-12.** Probability of occurrence of various wind speeds in the four primary directions.

**Figure 4-13.** Estimated multi-stressor fatigue life of three most critical elements of the DMS-support structure under consideration in different primary wind directions.
CHAPTER 5. SUMMARY AND CONCLUSION

In the first section of the current study, a comprehensive field study paired with detailed numerical simulations were conducted on a four-chord sign-support structure during transportation from the fabrication site to the installation location. For the field study, 12 strain gauges were mounted on various truss members and a short-term structural health monitoring was completed. Further to the field study, a detailed FE model of the same truss was generated using the ABAQUS software package to evaluate the distribution of strains and stresses within the entire truss members. To investigate the effects of road roughness on the potential of experiencing fatigue-induced damage, three artificial road profiles were generated following the ISO Specifications. To capture the excitations transferred to the truss structure during transportation by the truck, a quarter car suspension system was designed and the Simulink package was used to solve the partial differential equations of motion. The time-history of vertical vibrations at the truss supports were used as input to the detailed FE model of the truss. Following a set of numerical simulations for various road surface conditions, the stress time histories were obtained and the SCFs were estimated. This was one of the unique contributions of the current study, which highlighted how the actual SCFs are compared with the constant value reported in the AASHTO Specifications. A fatigue analysis was then conducted using both field data and simulation results. While the potential of experiencing fatigue-induced damage changes from one truss member to another, it was shown that the predictions obtained from the developed computational framework is in a close agreement with the observations from the field in terms of identifying the most vulnerable elements as well as the percentage of fatigue-induced damage during transportation.
In the second part of the study, a comprehensive field study paired with detailed numerical simulations was conducted on two overhead DMS support structures located in Iowa. For the field study, two DMS support structures were instrumented using vibrating wire gauges for a long-term structural health monitoring program. In addition to the field study, the FE models of this category of structures were developed to further investigate the potential thermal damage based on the most vulnerable members. This section provided two main contributions: (1) Fatigue life analysis of steel overhead DMS support structures using the field data and FE simulations, and (2) Comparison of the fatigue performance of a new steel overhead DMS support structure (with a slotted tube to gusset plate connection detail) with an aluminum overhead DMS support structure (with a tube to tube connections detail). Evaluation of the field data and numerical simulation results showed that both steel structures have an infinite fatigue life.

The fatigue performance of the steel structure was compared with that of the aluminum structure using an analytical approach. It was demonstrated that while a steel truss experiences the stresses that are 45% higher than those of the aluminum one, the steel structure has a fatigue life 39% longer than that of the aluminum structure. In addition to the analytical approach, a direct comparison was made between the aluminum and steel DMS-support structures. For this comparison, the environmental condition of the aluminum structure was applied to the steel one and the fatigue performance of the aluminum structure was compared with the steel structure. The outcome of this comparison confirmed that the steel structure has a superior fatigue performance. This strongly supports the recent transition of the transportation agencies from the aluminum DMS-support structures to the steel ones.

The last part of the current study investigated the fatigue performance of DMS-support structures with a multi-stressors perspective, which captures two primary stressors contributing to
fatigue-induced damage, i.e., diurnal temperature changes and natural wind excitations. For this purpose, a comprehensive field study paired with detailed numerical simulations was performed. Through the field study, a DMS-support structure located close to Des Moines, Iowa was instrumented with an array of sensors to record strain, temperature, and wind speed and direction. A FE model of the same DMS-support structure was then generated and validated with the field data. Through a detailed fatigue life analysis under both individual and combined stressors, it was found that the most fatigue-critical elements can change drastically depending on the stressor under consideration. This highlighted the importance of benefiting from a multi-stressor perspective to obtain an accurate estimate of the vulnerability of individual elements with accumulated fatigue-induced damage due to various exposure conditions. The developed approach was then extended to utilize the historical temperature and wind data, which not only reflect the long-term patterns of fluctuation, but also capture the dominant wind directions. This was an important addition, as it helped obtain a site-specific assessment of the fatigue life. The outcome of this study is expected to not only contribute to the long-term performance and safety of DMS-support structures, but also pave the way to implement similar multi-stressor perspectives for other transportation infrastructures in service.
Future Study

Further efforts can be considered to extend the current study to other sign-support structures, especially those exposed to extreme events. Such studies can utilize CFD simulations and wind tunnel tests to investigate the extent of vulnerability during the expected service life. Moreover, the transportation-induced fatigue analysis of other important structures, such as wind turbine blades, which are prone to fatigue damage, is an important area of research that can be explored based on the findings of this study.
REFERENCES


