Disclaimer Statement

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA 1998 – 009		

4. Title and Subtitle	5. Report Date
Fatigue Performance Of Variable Message Sign & Luminaire Support Structures Volume I – Fatigue Related Wind Loads on Highway Support Structures	April 1998
	6. Performing Organization Code CAIT/Rutgers/ATLSS/Lehigh
7. Author(s) Gary R. Consolazio, Kevin W. Johns, and Robert J. Dexter	8. Performing Organization Report No. FHWA 1998 - 009
9. Performing Organization Name and Address New Jersey Department of Transportation CN 600 Trenton, NJ 08625	10. Work Unit No.
	11. Contract or Grant No.
12. Sponsoring Agency Name and Address Federal Highway Administration U.S. Department of Transportation Washington, D.C.	13. Type of Report and Period Covered Final Report 09/13/1996 - 6/30/1998
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

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17. Key Words	18. Distribution Statement
fatigue, wind, loads, highway, support structures, cantilever, VMS	

19. Security Classif (of this report)	20. Security Classif. (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	104	

Form DOT F 1700.7 (8-69)

ADVANCED TECHNOLOGY FOR
LARGE
STRUCTURAL SYSTEMS

Lehigh University

Fatigue Related Wind Loads on Highway Support Structures

Final Report

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ATLSS Report No. 98-03

April 1998

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Acknowledgments

The research reported herein was performed at the Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University. The project was sponsored by the New Jersey Department of Transportation. The authors would like to acknowledge Professor John W. Fisher for providing invaluable guidance over the course of the research program. Robert Connor assisted with the instrumentation, data acquisition and field testing throughout the duration of the project. Ed Tomlinson assisted in the instrumentation and data acquisition process. John Hoffner, Larry Heffner, Steve Leonard, Roger Moyer, Todd Anthony and other ATLSS technicians provided assistance with all laboratory work.

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Abstract

In order to determine equivalent static pressures for fatigue loads on cantilevered highway sign support structures a cantilevered Variable Message Sign (VMS) located along Interstate 80 westbound at mile marker 48.5 in northern New Jersey was continuously monitored for three months. The structure was instrumented with strain gages, pressure transducers, and a wind sentry. All the data was collected with a Campbell Scientific CR9000 digital data acquisition system. A cellular phone transceiver enabled remote communication with the data logger. The system and instrumentation was powered with solar powers and marine batteries. Short-term testing was performed on the structure to determine the dynamic characteristics such as stiffness, natural frequency, and percent of critical damping. Results of the short-term test indicated that the stiffness was 0.24 kN/mm, the first and second mode natural frequencies were 0.87 cycles/s and 1.22 cycles/s respectively, and the percent of critical damping for the first and second modes were 0.57 percent and 0.25 percent respectively. Long-term monitoring was performed to capture the structure's response to natural wind gusts, galloping, and truck-induced wind gusts. This data would then be used to determine appropriate fatigue design wind loads for future sign support structures. During the three months of monitoring the structure did not experience galloping, which is a phenomena highly dependent on location. A galloping design pressure of 1000 Pa was recommended based on previous research. The summer months, which is when the structure was monitored, were not conducive to the strongest natural wind patterns in northern New Jersey. The highest natural wind speed that was recorded was 7.5 m/s. It is believed that much stronger winds are present in winter and spring, therefore a natural wind gust design pressure of 250 Pa was recommended. Truck-induced gusts were measured and a linear gradient for the truck-induced gust design pressure was determined. The truck-induced gust design pressure ranged linearly from 1760 Pa at 0 to 6 m above the surface of the road to 0 Pa at 10.1 m and over.

Chapter One: Introduction

1.1 Problem

Cantilevered sign and signal support structures are used extensively on major interstate highways and at local intersections for the purposes of traffic control. The cantilevered support structures are attached to a single vertical support as opposed to two supports for traditional overhead structures. The single support increases motorist safety by minimizing the probability of vehicle collision and is more economical than overhead support structures.

The 1994 AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals offers little guidance in the design for fatigue. The specification is currently being revised and will have better guidance for fatigue design. Until the new specification is available, designers do not have adequate guidance for fatigue related design issues in signs, signals and luminaire support structures. Consequently, many cantilevered sign and signal support structures across the country have exhibited excessive displacement due to wind-induced vibration and several have even failed due to fatigue cracking. In Michigan, fatigue cracks developed in the anchor bolts of a cantilevered sign support structure, resulting in the death of a motorist. Cracking has also occurred in many of the welded details of cantilevered sign and signal support structures, such as the connection of the mast arm or truss to the column, or the connection of the column to the base plate.

There are some obvious reasons for the sensitivity to vibration of cantilevered support structures. The single support significantly increases the flexibility of the cantilevered structures relative to overhead structures. The flexibility of these structures has increased over the years due to longer span lengths to accommodate more traffic lanes and a desire to set the column farther away from the road to increase motorist safety. Today, it is not unusual for the cantilever to span more than 12 meters (40 ft). The ratio of stiffness to mass consistently gives these structures a low natural frequency of about 1.0 Hz. These cantilevered support structures also have extremely low critical damping ratios, typically less than one percent of critical damping. These conditions make cantilevered support structures particularly susceptible to large-amplitude vibration and/or fatigue cracking due to wind loading.

The wind-induced vibration of cantilevered support structures was recently studied at the Center for Advanced Technology for Large Structural Systems (ATLSS) for the National Cooperative Highway Research Program (NCHRP). The project was NCHRP 10-38, "Fatigue-Resistance Design of Cantilevered Signal, Sign, and Light Supports". It was found that excessive vibration of cantilevered sign and signal support structures may be due to three different phenomena, possibly acting together at times. These three phenomena are 1) buffeting by natural-wind gusts; 2) buffeting by gusts caused by trucks passing under the structure; and 3) galloping. Any of these may cause large-amplitude displacement ranges and associated stress ranges in sign and signal support structures. Therefore, the details of these structures must be designed for fatigue resistance by considering typical stress ranges resulting from these phenomena.

Equivalent static load ranges were recommended in the NCHRP Project 10-38 final report which can be used to estimate stress ranges at details for the three wind-loading phenomena. These fatigue design loads are less than the ultimate design loads used for strength design, and therefore should not be considered in the strength design checks.

Depending on their geometry, sign and signal support structures may be more or less affected by the three wind-loading phenomena. For example, truck-induced wind gusts act primarily vertically upward on the projected area on a horizontal plane. Flat signs, signals, and their support structures are not susceptible to truck-induced wind gusts, due to their relatively small area projected on a horizontal plane. Variable message signs (VMS) are potentially susceptible to truck-induced wind gusts, because of their width in the direction of traffic and the large area projected on a horizontal plane.

The VMS is a relatively new type of sign that is capable of displaying any message on an electronic LED face. This feature enables motorists to be provided with the most recent information regarding road conditions and traffic flow. The sign is controlled from an office that is given information such as traffic speed and congestion from cameras and radar guns that are mounted beside the VMS. The research described in this report was motivated by reported large-amplitude vibration of a VMS support structure in New Jersey.

Many other VMS have had problems with excessive vibration and fatigue. Failures of cantilevered support structures for VMS have occurred in Virginia[16] and California[17]. A second California cantilevered VMS support structure was instrumented and monitored. Stress ranges of 140 MPa (20 ksi) were recorded during one wind event, which is well above the fatigue threshold for typical details. Truck-induced wind gusts are believed to have been the problem in New Jersey and in Virginia, whereas galloping was identified as the cause of the excessive vibration in the California VMS. The California VMS are mounted on a curved monotube support structure, which is much different and more susceptible to galloping due to the low torsional rigidity of the mast arm. A typical truss-type structure, such as is used in New Jersey, is torsionally stiff and is therefore not as susceptible to galloping.

The 10-38 research was focussed primarily on galloping. The recommended fatigue design load range for truck-induced wind gusts was based on some uncertain assumptions. Therefore, this additional research was sponsored by New Jersey DOT (NJDOT) to gather additional data on the magnitude of the truck-induced wind gust loads.

This research also addresses anchor bolt tightening and tightening of the truss to stub connections at the column. The anchor bolts used in these structures are as large as three inches in diameter. There was not much guidance available on the proper tightening procedure for this size of bolt. Improper tightening of the anchor bolts can cause changes in the dynamic characteristics of the sign support structure.

There was also concern about the amount of contact on the faying surfaces for the truss to stub connection. The concern was over the fact that it is near impossible to get 100 percent mating between the two surfaces, therefore different alternatives were looked at to solve this problem.

1.2 Purpose

The primary objective of the research described in this report was to gather data on the magnitude of truck-induced gust loads and, if necessary, refine the equivalent static load range for truck-induced wind gusts recommended in NCHRP 10-38. To accomplish this objective, a cantilevered VMS was instrumented and monitored. The VMS that was chosen by NJDOT spans Interstate 80 West at mile marker 48.5 in northern New Jersey (Figure 1-1). Short-term and long-term field tests were conducted. The short-term test had two main goals: 1) to obtain the static and dynamic characteristics of the support structure, such as stiffness, natural frequency, and damping ratio; and, 2) to drive trucks under the sign in an attempt to quantify the magnitude of the truck-induced gusts. The long-term test lasted three months and would attempt to measure any significant dynamic response from random truck traffic and other wind-loading phenomena.

1.3 Scope

This report summarizes previous research relevant to the design of VMS support structures to resist truck-induced gusts and other wind-loading phenomena. The measurements included pressures near the VMS and strains at all critical locations. Pressure transducers were intended to measure the pressure resulting from the upward gust of air that large trucks produce when going under the signs at high speeds. The stress ranges deduced from the strain histories would be compared to stress ranges calculated using the fatigue design load ranges recommended in NCHRP 10-38. The design guidelines recommended herein can be used to design future structures to be resistant to fatigue and excessive dynamic displacement despite the worst-case truck-induced wind loads. This research was not concerned with changing the geometry or adding damping devices to mitigate the vibration problems. Mitigation is to be addressed in the phase two studies of project 10-38, which are ongoing at the University of Minnesota under the direction of Robert Dexter.

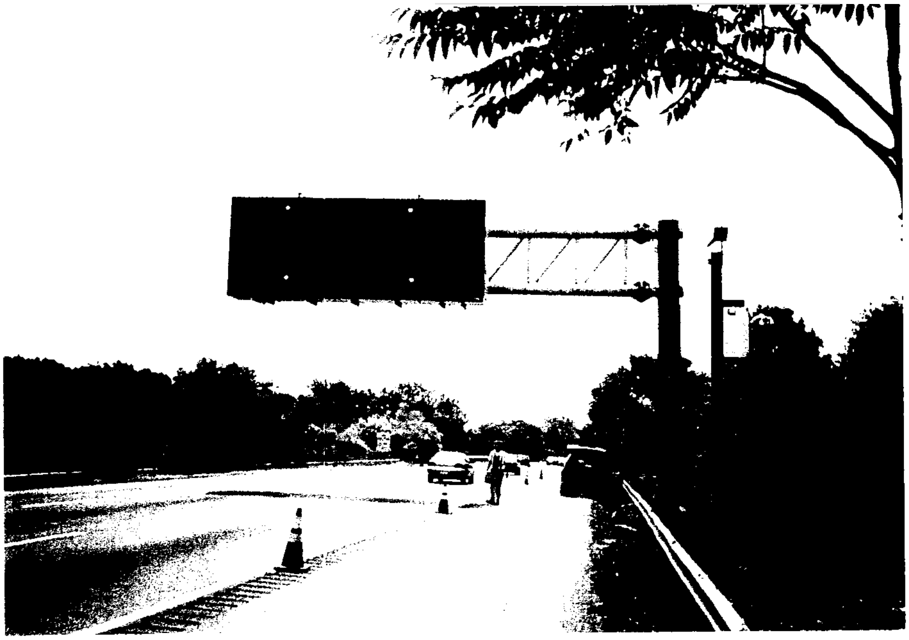

Figure 1-1: Cantilevered VMS on Interstate 80 West Bound Mile Marker 48.5.

Chapter Two: Background

2.1 Wind Loading Phenomena Relevant to VMS Support Structures

All types of support structures are potentially susceptible to natural wind gusts and truck-induced wind gusts. In cantilevered structures it is easier for the mast arm to twist (i.e. for the sign panel to change angle of attack relative to the wind) than it is in overhead structures. This change in angle of attack facilitates galloping, as is explained below. Therefore, in cantilevered structures, galloping is also a primary concern, in addition to natural and truck-induced gust loading. Vortex shedding is a different phenomenon than galloping but also results in vertical displacements; therefore, these phenomena are often confused. Previous research in Project 10-38 showed that vortex shedding did not occur in cantilevered sign and signal support structures as long as the sign or signal was attached, presumably because the galloping began to occur first.

Because overhead support structures are not susceptible to galloping, they are potentially susceptible to vortex shedding. Irwin and Peeters[1] discovered a problem with vortex shedding in an overhead sign support structure that failed in Calgary, Canada. The vortex shedding occurred from two 8 m (25 ft) long sign panels mounted on the structure.

To our knowledge vortex shedding has never been reported to be a problem for cantilevered support structures and therefore, vortex shedding will not be discussed further.

2.1.1 Natural Wind Gusts

The response of typical cantilevered support structures to natural wind gusts was modeled using spectral finite-element analysis. The structure is broken up into several continuous areas such as signs or exposed portions of the structure. The fluctuating wind force on each area and the resulting response variables (such as column base moment) during a short interval are characterized as stationary random processes. The response spectrum can be related back to the expected variable-amplitude history of the response as a function of time. Specifically, the root-mean-square (RMS) of the random response time history is found from the integration of the response spectrum over all significant frequencies. In the case of cantilevered support structures, there are only a few significant frequencies; therefore, the integration is performed by simply summing the response at these frequencies.

The wind force spectrum is derived from the velocity spectrum. A standard wind velocity spectrum (which depends on the mean hourly wind velocity) was selected from the literature[7]:

$$S_V(f) = \frac{4KV_{10}{}^2 x^2}{f(1+x^2)^{4/3}} \tag{2-1}$$

where $S_V(f)$ is the spectral density of the velocity (which has units of velocity squared multiplied by time), f is the cyclic frequency (cycles/s), K is a terrain coefficient $(m/s)^2$, V_{10} is the mean wind velocity (m/s) at a reference height of 10 meters, and x is the quantity (1200 meters * f)/V_{10} (dimensionless for V_{10} in m/s). The terrain coefficient K was taken as 0.005 which is typical for open grassy terrain[7,8].

The drag force is proportional to the square of the velocity and both the force and the velocity can be represented as the sum of their mean and fluctuating components. Through algebraic manipulation of these relationships, the following relations can be obtained[8]:

$$\left(\frac{d}{v}\right)^2 = \frac{4\ D^2}{V^2} = 4\ C^2\ A^2\ V^2 \tag{2-2}$$

where d and D are the fluctuating and mean value of the drag force respectively, v and V are the fluctuating and mean value of the wind velocity respectively, A is the total frontal area of the surface which is causing the drag, and C is a constant equal to $0.5\rho C_d$ with ρ equal to the density of air and C_d equal to the drag force coefficient. The density was taken as 1.22 kg/m^3 which is the value for "standard air" (one atmosphere pressure at 14°C).

The force and velocity spectra are proportional to the square of the fluctuating components of force or velocity, therefore the ratio of these spectra is equal to the ratio in Equation 2-3, i.e.:

$$S_F(f) = 4\ C^2\ A^2\ V^2\ S_v(f) \tag{2-3}$$

The force spectrum must be calculated for the total frontal area of a surface and cannot be broken down into sub-areas. One spectrum is calculated for each sign and signal attachment. Additional spectra are calculated for each continuous exposed portion of the mast arm or column. These spectra must be completely correlated to each other in the analysis.

Important assumptions must be made regarding 1) the mean wind velocity at which the support structures should be analyzed; and, 2) estimating the effective stress range from the RMS of the variable amplitude response. It is impractical to forecast the future wind history at each location for cantilevered support structures therefore, some very simple assumptions were made. The design procedure is based on a spectral analysis using the mean hourly wind velocity, which was exceeded in only 0.01 percent of all hours. It is accepted that the probability of exceedence of mean hourly wind velocity at a location is a Rayleigh distribution, which depends only on the yearly mean wind velocity V_m[9], i.e.:

$$P_E(v) = e^{\frac{-\pi v^2}{4 V_m^2}} \tag{2-4}$$

where $P_E(v)$ is the probability that a randomly-occurring mean hourly velocity is greater than the velocity magnitude v and e is the base of the natural logarithms. The limit-state mean hourly velocity is found by setting P_E equal to 0.01 percent and solving for v.

The yearly mean wind velocity also varies from place to place. A collection of yearly mean wind speed data from weather stations at 59 cities across the U.S. was examined. Most weather stations are located at airports, therefore the data should be representative of most open terrain. The data showed that 81 percent of the cities had a mean wind velocity of less than 5 m/s (11 mph) at 10 m (33 ft) above the ground. It was decided to use 5 m/s (11 mph), which was exceeded in only 19 percent of U.S. cities, as the baseline case for a static design pressure in the specifications. The mean hourly wind velocity for this yearly mean wind speed is 17 m/s (37 mph).

The result of the analysis, the spectral density of the response, has units of the response (such as moment or stress) squared multiplied by time. When the spectral density of the response is integrated across a range of frequencies, the result (the area under the spectrum) is equivalent to the variance of the response about the mean. The square root of this area is the root-mean-square (RMS) of the response. The time history of the response is narrow-banded (concentrated about one frequency), since the response is still dominated by the resonant frequency. For random, narrow-band time histories, the average or effective stress range S_r^{eff} can be estimated from the relationship which gives the stress range for a constant-amplitude response in terms of the rms of the stress response σ_{rms}[10], i.e.:

$$S_r^{eff} = 2.8\,\sigma_{rms} \tag{2-5}$$

A variety of sign, signal, and luminaire support structures were analyzed at a mean wind velocity of 17 m/s and values of normalized equivalent static pressures for these structures ranged from 170 to 300 Pa (3.6 to 6.3 psf). Considering the numerous uncertainties in this analysis, not enough is known to assign greater or lesser loads to different types of structures. Also, separate loading for different types of structures would unnecessarily complicate the design process. Therefore, these values were averaged and rounded to 250 Pa (5.2 psf), which is recommended for design. This natural wind gust pressure must be applied to a variety of surfaces with widely varying drag coefficients. Therefore the recommended static design pressure must be multiplied by the appropriate drag coefficient and then may be applied to the surface. The structures should be designed so that the stress ranges resulting from the application of this load range are below the CAFL[6].

These calculations indicate that most structures will eventually be susceptible to cracking from natural wind gusts, but the recommended loads are not so large as to predict rapid failure. These results are consistent with observed service fatigue failures that can be attributed to natural wind gusts. Because of the uncertainty in these assumptions, the recommended equivalent static load range can be easily adjusted for other mean wind speeds.

2.1.2 Galloping

Galloping, also known as Den Hartog instability, is an areoelastic phenomenon caused by a coupling between the aerodynamic forces which act on a structure (caused by the action of wind) and the structural vibrations[2]. Galloping is characterized by large amplitude, vibrations normal to the direction of wind flow. Galloping-induced oscillations primarily occur in flexible, lightly damped structures with non-symmetrical cross-sections (e.g. circular cylinders are not susceptible to galloping-induced vibrations because they are symmetric).

Galloping-induced oscillations are caused by forces, which act on a structural element as it is subjected to periodic variations in the angle of attack of the wind flow. The periodically varying angle of attack is generated by across-wind oscillation of the structure. When the forces are aligned with the direction of across-wind motion, the result is successively larger amplitudes of oscillation, i.e. galloping.

The potential susceptibility of a structure to galloping from the equilibrium position is evaluated using the Den Hartog stability criterion[4].

$$\left(\frac{dC_{Fy}}{d\alpha}\right)\Big|_{\alpha=0} = -\left(\frac{dC_L}{d\alpha} + C_D\right)\Big|_{\alpha=0} > 0 \qquad (2\text{-}6)$$

where C_{Fy} is the aerodynamic lift force coefficient acting normal to the free stream velocity, a is the angle of attack, C_L is the lift force coefficient, and C_D is the drag force coefficient which acts in respect to the relative wind velocity. The free stream velocity is defined as the relative wind velocity times the cosine of the angle of attack. The Den Hartog stability criterion states that "a section is dynamically unstable if the negative slope of the lift curve is greater than the ordinate of the drag curve." As is evident from Equation 2-6, this condition is satisfied when the slope of the lift force coefficient normal to the free-stream velocity, dC_{Fy}/da , is positive (in other words, when the term $dC_L/da + C_D$ is negative). This condition is referred to as "negative aerodynamic damping".

Galloping from the equilibrium position can only occur if the magnitude of the negative aerodynamic damping is greater than the magnitude of the positive mechanical damping possessed by the structure (i.e. galloping can only occur if the effective damping is less than zero). Therefore, the minimum wind velocity required to initiate galloping is directly proportional to the mechanical damping possessed by the structure[3]. This onset wind velocity is also proportional to the mass and stiffness of the structure and the inverse of the slope of the lift force coefficient curve, C_{Fy}.

Thus, a highly flexible structure with low damping (such as a typical cantilevered support structure) will be susceptible to galloping-induced oscillations at relatively low wind velocities provided, of course, that the Den Hartog stability criterion is satisfied.

Although the structure on Interstate 80 never experienced galloping while it was monitored, there has been other very recent research that has been investigated to formulate a recommended fatigue design galloping pressure range.

A majority of cantilevered support structures are composed of structural members with circular cross-sections. Circular cylinders are not susceptible to the galloping instability. This fact is important because it indicates that the across-wind vibrations observed in the cantilevered support structures in the field are the result of the aerodynamic characteristics possessed by the attachments to these structures (i.e. signs/signals).

This fact was confirmed by McDonald et al.[5] at Texas Tech University. McDonald's tests indicated that the configuration of the signal attachments and the direction of flow significantly influence the susceptibility for galloping. Signal attachments configured with backplates and subjected to flow from the rear were found to be most susceptible to galloping (i.e. the slope of the lift force coefficient curve, C_{Fy}, was greatest for this configuration and flow direction).

A full-scale 12.2 m (40 ft) structure configured with signal attachments was observed to experience galloping oscillations with displacement amplitudes at the tip of the horizontal support estimated at between 300 to 400 mm (12 to 16 in). The results of tests on a 14.6 m (48 ft) structure were similar. Galloping was observed in this structure at a wind velocity equal to 4.5 m/s (10 mph) with a maximum measured stress range in the vertical support (at a location 330 mm (13 in) from the base) equal to approximately 34 MPa (4.9 ksi).

These observations regarding signals with backplates were confirmed in wind-tunnel tests conducted at the Wright Brothers facility at M.I.T. as part of the NCHRP project 10-38. These wind-tunnel experiments were such that the scaled up cantilevered sign and signal support structures would be subjected to equivalent static lift-pressure ranges between 1150 and 1770 Pa (24 and 37 psf) during occurrences of galloping-induced vibrations. These pressures were derived from the maximum loads obtained at the highest wind velocities (about 13 m/s) applied in the tests. It was observed that the magnitude of the loads increases with wind velocity, such that much larger loads are theoretically possible at higher wind velocities. However, larger velocities were not used in the wind-tunnel tests in order to minimize potential damage to the test specimens.

Finite-element models were prepared for several structures that were observed to gallop in the field. In the Texas Tech tests, strain measurements at the column base were available. In other cases mast arm displacement amplitudes were estimated from videotapes. The finite-element analyses showed that these structures were subjected to equivalent static pressure ranges from 775 to 1290 Pa (16.2 to 27.0 psf) during the observed galloping. Thus, the wind-tunnel data are conservative and reasonably consistent with respect to the field observations.

The most recent research that has obtained data on galloping was done in California. Caltrans has instrumented a cantilevered VMS that is supported by a monotube structure. Figure 2-1 shows a stress range of 145 MPa (21 ksi) in the column of the Caltrans VMS support structure. VMS on monotubes will be more susceptible to galloping due to their lack of stiffness in the cantilevered section. This lack of stiffness allows the sign to easily change angle relative to the oncoming wind. As described previously, this circumstance is required for galloping to occur. The equivalent static galloping pressure that would cause this stress in the column is about 2000 Pa (42 psf).

Considering the inherent variability in the response of a structure to galloping, the variation in equivalent static pressure ranges observed in NCHRP 10-38 where remarkably consistent. Based upon these results, it is recommended that an equivalent static lift-pressure range equal to 1000 Pa (21 psf) be used in the design of cantilevered sign and signal support structures for galloping-induced fatigue. The stress ranges resulting from the application of this load range should be less than the constant amplitude fatigue limit (CAFL), to obtain essentially infinite life[6].

The value of 1000 Pa (21 psf) is the median (rounded to two significant figures) of the loads from the field observations (775 to 1290 Pa). This equivalent static lift-pressure range should be applied vertically as a shear stress on the surface area of all sign and signal attachments mounted to the horizontal mast arm as seen in the normal elevation.

Of course it would be preferred to mitigate or prevent the galloping. It is theoretically possible, that much larger loads could be experienced if the wind velocity increases significantly. Several research efforts are currently underway to investigate ways to mitigate galloping. However at this time none of these have been shown to be effective. Without a reliable and cost effective means of mitigation, it is advised that structures be designed to resist these recommended fatigue loads.

Most present structures are not designed to withstand this large a load and will require more fatigue resistant details and possibly increased sections in order to meet these criteria. Therefore, many agencies are concerned that these loads are too conservative. On the other hand, the Caltrans measurements show the galloping loads could be as high as twice the recommended value of 1000 Pa in some cases. Larger design loads may be appropriate for monotube cantilevered support structures such as those used in California. However, this is believed to be a special case that is not applicable to the much stiffer truss-type structures. Therefore, the recommended load of 1000 Pa represents a compromise between safety and practicality.

2.1.3 Truck-Induced Wind Gusts

The passage of trucks beneath cantilevered support structures tends to induce gust loads on the frontal area and the underside of the members and the attachments mounted on the mast arms of these structures. The magnitude of a natural wind gust pressure is much larger than the pressures from truck-induced loading (in the horizontal direction). Therefore, for the purposes of fatigue design, truck-induced wind loads normal to the sign are not critical.

The VMS are particularly susceptible to truck-induced wind gusts. These signs have a relatively large width (up to approximately 1.2 m or 4 ft) in the direction parallel to traffic flow. Figure 2-2 is a VMS structure that failed due to anchor bolt fatigue. Figure 2-3 shows a closer view of the fractured anchor bolts. Prior to failure, VMS structures in Virginia and California were observed to be vibrating in the vertical plane. Therefore, it is believed to be the vertical pressure acting on the horizontal area that caused these vibrations.

A study by Creamer, Frank, and Klingner[13] has been the most extensive program performed to date on the subject of truck-induced gust loads. This study's suggested loading function is represented by a horizontal triangular pressure distribution applied to the face of the sign panel with a peak pressure of 60 Pa (1.25 psf) and a vertical uniform pressure distribution of 60 Pa (1.25 psf) applied to any walkways or lighting fixtures. The development of this forcing function was based upon the maximum loading event observed in the field and corresponds to a gust velocity of approximately 8.5 m/s (19 mph).

In addition to the above research program, several other studies have been conducted which support the conclusions drawn by Creamer. For example, field testing of one support structure conducted by the University of North Carolina determined that the maximum pressure induced on the face of the sign was equal to 67.5 Pa (1.41 psf)[14]. Another field study conducted by the Michigan Department of Transportation determined that the maximum axial stress range, induced in the anchor bolts of a cantilevered sign support structure, is equal to just over 21 MPa (3 ksi)[15]. This axial stress range was observed during the simultaneous passage of two trucks beneath the structure.

Wind tunnel data from a test on a two dimensional one-eighth model cantilevered sign and signal support structure performed at M.I.T's Wright Brothers Facility by Philip Mark Cali and Eugene Covert[12] indicates that the shape of the truck had a large influence on the magnitude and shape of the graph of the pressure ranges created by the truck-induced wind gusts. M.I.T. used two different model test trucks-one box-shaped and one source-shaped. The box truck produced the maximum pressure. It is confirmed by this data that the maximum pressure on the vertical area of the signs acts toward the trucks. The interesting observation of M.I.T. was that this negative truck gust came when the leading edge of the truck was in the vicinity of the edge of the sign.

A range of cantilevered sign and signal support structures were analyzed including a fatigue damaged VMS. The resulting stress ranges caused by the loads suggested by Creamer et al[10] in both the horizontal and vertical direction were very small, which is not consistent with the failures that have occurred.

Ronald Cook, et al[11], (University of Florida) measured pressures from truck-gusts by mounting pressure transducers on the side of an overpass. By measuring the pressures at varying elevations, Cook observed a vertical gradient in the pressure from truck-induced gusts. The maximum observed pressure by Cook was about 50 Pa, which is much higher than observed on Interstate 80. However, Cook's measurements were taken about 600 mm closer to the trucks.

Desantis[16] modeled the sign structure that failed in Virginia. He used a simple model for the truck-gust load that assumed the velocity of the wind in the upward direction is equal to the truck velocity. The equivalent static truck-gust pressure is determined by using the static wind pressure formula where V = 105 kph (65 mph). To account for an increase in the relative truck speed due to head winds, the gust factor of 1.3 is also included. Since the applied truck-gust pressure will lift the mast arm vertically, the pressure obtained above is doubled to represent the entire truck-gust pressure range. This doubling of the pressure is based on the assumption that on the first cycle the downward and upward forces are equal. Based on these assumptions, an equivalent static vertical pressure range of 1760 Pa (36.6 psf) can be obtained. This pressure range must be multiplied by the appropriate drag coefficient and horizontally projected area in order to determine the proposed vertical truck-gust load. Using this equivalent static pressure Desantis was able to match the displacement ranges observed for the structure in Virginia that failed. Table 2-1 shows a comparison of the different pressures measured by different researchers.

Table 2-1 Summary of Pressures from Various Researchers

Researcher	Model	Pressure
Creamer et al	Uniform vertical loading based on maximum loading event observed in the field	60 Pa
Univ. of North Carolina	Max pressure on the face of a sign	67.5 Pa
Cook et al	Pressure transducers at various heights and angles	*50 Pa
Desantis	Upward wind gust is equal to truck velocity	1760 Pa
ATLSS	Back calculated from strain gage data	525 Pa

* Measured dynamic pressure, not an equivalent static pressure.

In the NCHRP project Desantis' model was recommended for truck-gust loading. The structures should be designed so that their stress ranges resulting from the application of this load range are below the CAFL. In order to check the Desantis model it was applied to the two structures that failed. One structure was observed vibrating immediately after installation and developed fatigue cracks after approximately six months of service. The other structure failed after approximately 18 months of service. From analysis, the ratio of the stress ranges to the fatigue thresholds was 5.2 and 2.7. The predicted fatigue lives are consistent with the relative service lives prior to failure. Therefore, the simple Desantis model seems reasonable for design purposes.

Assuming a drag coefficient of 1.45 the Desantis model would imply an equivalent static pressure of 2550 Pa. Note that this is almost five times bigger than the maximum equivalent static pressure from strain gage data in the New Jersey research. However, during the monitoring in New Jersey there were no significant headwinds and it is not known what synergetic effect this may have.

2.2 Background Relevant to Fatigue Resistance of Details

Experience with multiaxial loading experiments on large-scale welded structural details indicates the loading perpendicular to the local notch or the weld toe dominates the fatigue life. The cyclic stress in the other direction has no effect if the stress range is below 83 MPa (12 ksi) and only a small influence above 83 MPa (12 ksi). Since the combination of multiaxial loading does not have to be considered. The recommended approach for multiaxial loads is:

1) decide which loading (primary or secondary) dominates the fatigue cracking problem (typically the loading perpendicular to the weld axis or perpendicular to where cracks have previously occurred in similar details); and,

resistance expected for a particular detail. The welding process also does not typically have an effect on the fatigue resistance. The independence of the fatigue resistance from