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Aerodynamic Testing of Variable Message Signs

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FLORIDA INTERNATIONAL UNIVERSITY

Miami, Florida

AERODYNAMIC TESTING OF VARIABLE MESSAGE SIGNS

A dissertation submitted in partial fulfillment of

the requirements for the degree of

DOCTOR OF PHILOSOPHY

in

CIVIL ENGINEERING

by

Debbie Meyer

2014

To: Dean Amir Mirmiran
College of Engineering and Computing

This dissertation, written by Debbie Meyer, and entitled Aerodynamic Testing of Variable Message Signs, having been approved in respect to style and intellectual content, is referred to you for judgment.

We have read this dissertation and recommend that it be approved.

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ABSTRACT OF THE DISSERTATION
AERODYNAMIC TESTING OF VARIABLE MESSAGE SIGNS

by

Debbie Meyer

Florida International University, 2014

Miami, Florida

Professor Arindam Gan Chowdhury, Major Professor

The increasing nationwide interest in intelligent transportation systems (ITS) and the need for more efficient transportation have led to the expanding use of variable message sign (VMS) technology. VMS panels are substantially heavier than flat panel aluminum signs and have a larger depth (dimension parallel to the direction of traffic). The additional weight and depth can have a significant effect on the aerodynamic forces and inertial loads transmitted to the support structure. The wind induced drag forces and the response of VMS structures is not well understood. Minimum design requirements for VMS structures are contained in the American Association of State Highway Transportation Officials Standard Specification for Structural Support for Highway Signs, Luminares, and Traffic Signals (AASHTO Specification). However the Specification does not take into account the prismatic geometry of VMS and the complex interaction of the applied aerodynamic forces to the support structure. In view of the lack of code guidance and the limited number research performed so far, targeted experimentation and large scale testing was conducted at the Florida International University (FIU) Wall of Wind (WOW) to provide reliable drag coefficients and investigate the aerodynamic instability of VMS. A comprehensive range of VMS

geometries was tested in turbulence representative of the high frequency end of the spectrum in a simulated suburban atmospheric boundary layer. The mean normal, lateral and vertical lift force coefficients, in addition to the twisting moment coefficient and eccentricity ratio, were determined using the measured data for each model. Wind tunnel testing confirmed that drag on a prismatic VMS is smaller than the 1.7 suggested value in the current AASHTO Specification (2013). An alternative to the AASHTO Specification code value is presented in the form of a design matrix. Testing and analysis also indicated that vortex shedding oscillations and galloping instability could be significant for VMS signs with a large depth ratio attached to a structure with a low natural frequency. The effect of corner modification was investigated by testing models with chamfered and rounded corners. Results demonstrated an additional decrease in the drag coefficient but a possible Reynolds number dependency for the rounded corner configuration.

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1 INTRODUCTION

Intelligent transportation systems (ITS) improve safety and mobility by integrating advanced communication technology into the transportation infrastructure. Variable message signs (VMS) are a cornerstone of intelligent transportation systems and provide a high tech compliment to static flat panel signs. These remotely programmable traffic control devices relay real-time advisories to motorists about changing highway conditions and hazards such as inclement weather, traffic accidents, construction activity, congestion, and public service alerts.

VMS are significantly heavier than flat panel aluminum signs and have a larger prismatic profile parallel to the direction of traffic. Dimensions and weight vary depending on application and maintenance access requirements. For example, small rear access and lift-face VMS can be used where only a short message is required. Maintenance access to internal components is provided by rear doors or lifting the front face of the VMS. These VMS may be deployed on low volume roadways where maintenance can be accomplished by closing a lane and utilizing a lift truck. However, the largest walk-in style signs deployed on high volume highways can be as large as 9.1 m wide x 3 m high x 1.2 m deep and weigh more than 1814 kg (Figure 1.1). The larger profile and additional weight amplifies the wind load and inertial effects that are transmitted to the support structure.

VMS are typically installed on cantilever or bridge-type support structures. Figure 1.2 shows typical cantilever and bridge (or non-cantilever) VMS support structures. Cantilever support structures are characterized by a single vertical upright which supports

a horizontal mast arm or truss. These structures have low natural frequencies in the range of 1 to 3 Hz and damping typically less than one percent of critical, rendering the structures to be highly flexible. These conditions make cantilever support structures particularly susceptible to large amplitude vibrations and fatigue problems (Kaczinski et al., 1998). Bridge type or non-cantilever structures are supported at each end by single or double vertical uprights and can span longer distances. They are typically more rigid than cantilever support structures and have a higher natural frequency. Table 1.1 shows the natural frequency of typical cantilever and non-cantilever structures provided by various past research studies.

Structure types can be categorized into four primary groups depending on the geometry of the horizontal span; 1) box truss, 2) tri-chord truss, 3) two-chord truss, and 4) monotubes. A typical example of each structure type is shown in Figure 1.3. Box trusses consist of four main chords positioned to form a three dimensional box. The main horizontal chords are typically large diameter tubes in single or multiple sections bolted together for longer spans. The truss elements consist of round welded pipe members or angles joined with gusset plates. Tri-chord trusses are similar to box trusses but consist of three main members positioned to form a three dimensional triangle. Truss members, like the box truss, are typically round welded pipe members or angles joined with gusset plates. Two-chord trusses are planar structures consisting of two primary horizontal members with secondary members spanning vertically in-between. The truss may be composed of rounded or square tubular members arranged in triangular or rectangular patterns. Monotubes are large tubular structures of constant diameter or linearly tapered

elements. The horizontal beam may be a single element or multiple linear sections joined together with bolted connections.

There is no standard method for attaching VMS to the support structure. The number and size of the hanger and connectors will depend on the sign size, weight, and shape. Typically, aluminum extrusions (z shapes, channels, W shapes, etc.) are arranged vertically and horizontally in a grid pattern depending on the access configuration (walk-in, rear access, front access) and support structure type. Generally, the hangers evenly distribute the weight of the sign to the attachment points on the support structure.

Several State Department of Transportations have reported wind induced vibrations and fatigue related problems in VMS support structures (Dexter and Ricker, 2002). These problems can lead to costly repairs, premature removal, and occasional structural failure. To address this issue, the American Association of State Highway and Transportation Officials Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals (AASHTO Specification, 2013) has identified four wind loading mechanisms with potential for inducing large amplitude vibrations in VMS support structures: 1) galloping, 2) vortex shedding, 3) natural wind gusts, and 4) truck induced gusts. As indicated in Table 1.2, each VMS structure support type is susceptible to different wind loading phenomena.

1.1 Statement of problem and research objectives

Minimum design requirements for VMS support structures are contained in the AASHTO Specification (2013). Table 3.8.6-1 of the AASHTO Specification suggests a

drag coefficient (C_d) of 1.7 for VMS to be used in the wind pressure equation for strength and fatigue design.

AASHTO does not indicate how this current estimation of C_d was determined but it appears to be based on the results of wind tunnel testing of flat panel signs. However, the drag coefficient for two dimensional panels can be significantly different than the drag coefficient for a three dimensional VMS. Research has shown that the C_d of a prismatic shape is dependent on wind turbulence, aspect ratio, and the depth ratio (Bearman and Trueman, 1972; Laneville et al, 1975; Letchford, 2001). It is hypothesized that partial reattachment of flow over and around a prismatic VMS will reduce drag forces resulting in a smaller drag coefficient for use in design.

Fouad et al. (2003) pointed out that the current AASHTO Specification was developed based on wind acting on flat plates, not prismatic VMS and suggest that wind pressures, drag coefficients, and uplift forces must be revised to account for the VMS geometry.

The effect of wind-induced drag forces and the response of VMS structures to those forces are not well understood. Due to a lack of code guidance and limited research, targeted experimentation is needed to provide more reliable wind loading data for VMS.

The primary objective of this research is to investigate wind loading, especially wind-induced drag, on VMS by testing large-scale models in a simulated suburban atmospheric boundary layer flow. The work includes an extensive literature review, investigation of industry standards, large-scale experimental testing in the 12-fan Wall of Wind (WOW) facility at Florida International University (FIU), an in-depth analysis of obtained data and formulation of drag coefficient recommendations for incorporation into the AASHTO Specification. Experimental objectives are as follows:

- Determine drag coefficients for VMS with various aspect (b/c) and depth (d/c) ratios
- Investigate Reynolds number effects on VMS drag coefficients
- Characterize the mitigating effects of rounded and chamfered corners on VMS
- Evaluate potential design cost saving benefits of using the new drag force evaluation methodology for VMS (including reduction in drag with modified aerodynamic design)

Drag coefficient results from this study can be used by other researchers for finite element modeling/analysis (FEM) of full-scale VMS structures. Such analyses may focus on stresses generated at connections, design and selection of member sizes, base shear and moment, and foundation forces.

Since galloping and vortex shedding are also potentially important phenomena, the opportunity was taken to gather additional data. Since the drag calculation also involves the gust factor, a theoretical evaluation of the potential range of gust factors was carried out. These objects were as follows:

- Evaluate the galloping potential of VMS structures
- Evaluate vortex shedding for VMS structures
- Estimate gust factors and recommend drag force evaluation methodology for incorporation into the AASHTO Specification

1.2 Terminology

Figure 1.4 is a schematic representation of an elevated VMS illustrating the model's primary dimensions: lateral length b , height c , depth (along wind length) d , and the distance from the ground to the top of the model h . Geometric ratios used throughout this study were:

$$\text{Aspect ratio} = b/c \quad (1-1)$$

$$\text{Depth ratio} = d/c \quad (1-2)$$

$$\text{Clearance ratio} = c/h \quad (1-3)$$

Two model support configurations were used for testing. The models were mounted on cantilever support masts which incorporated load cells. Two support masts were used for the longer models and a single support mast was used for the shorter models. These will be called the double and single support configurations respectively. Figure 1.5 shows the axes and force orientation for the single and double support configuration used throughout the study. Body-axes have been used in this study as shown. The x-axis is positive in the along wind direction and the y-axis is positive to the right. The z-axis is positive in the upward direction and counterclockwise is positive for the torsional moment, M_z . The loads and moments measured by individual load cells for this study are also shown. Angle θ defines the horizontal wind approach direction and 0° is defined as wind normal to the front face of the model.

1.3 Force and moment coefficients

The time averaged aerodynamic forces on VMS subjected to wind loading can be expressed non-dimensionally as force coefficients defined as

$$C_{Fx} = \frac{F_x}{\frac{1}{2}\rho\bar{U}^2(bc)} \quad (1-4)$$

$$C_{Fy} = \frac{F_y}{\frac{1}{2}\rho\bar{U}^2(bc)} \quad (1-5)$$

$$C_{Fz} = \frac{F_z}{\frac{1}{2}\rho\bar{U}^2(bc)} \quad (1-6)$$

where F_x , F_y and F_z are the resultant forces along the x , y and z axis respectively, ρ is the density of air (assumed as 1.225 kg/m^3), $\bar{U}$ (m/s) is the mean wind speed, b (m) is the lateral length of the VMS, and c (m) is the height of the VMS. Refer to Figure 1.5 for axis and force orientation. The resultant forces used in this study for the single and double support configurations were determined as follows

Single Support Setup

$$F_x = F_{xl} \quad (1-7)$$

$$F_y = F_{yl} \quad (1-8)$$

$$F_z = F_{zl} \quad (1-9)$$

Double Support Setup

$$F_x = F_{xl} + F_{x2} \quad (1-10)$$

$$F_y = F_{yl} + F_{y2} \quad (1-11)$$

$$F_z = F_{zl} + F_{z2} \quad (1-12)$$

The force coefficients in this study were normalized using the area of the front face of the model ($b \times c$), unless otherwise noted, and the forces were resolved into orthogonal components parallel and perpendicular to the body-axes of the structure (Figure 1.5). The resultant forces (F_x , F_y and F_z) and corresponding force coefficients (C_{Fx} , C_{Fy} , and C_{Fz}) along the x, y, and z axis are conventionally termed normal force, lateral force and vertical lift respectively. Numerical subscripts for F_x , F_y , and F_z refer to the component reactions at each load cell. See Figure 1.5 for the location and orientation of component forces. The moment about the z axis is termed the twisting moment and defined as

$$C_{Mz} = \frac{M_z}{\frac{1}{2} \rho \bar{U}^2 (b^2 c)} \quad (1-13)$$

Where M_z is the moment around the centroidal z axis and calculated as

Single Support Setup

$$M_z = M_{z1} - F_{y1} \cdot a_2 \quad (1-14)$$

Double Support Setup

$$M_z = M_{z1} + (F_{x1} \cdot a_1) - (F_{y1} \cdot a_2) + M_{z2} - (F_{x2} \cdot a_1) - (F_{y2} \cdot a_2) \quad (1-15)$$

where the centroidal moment arm for the x axis is a_1 and the centroidal moment arm for the y axis is a_2 . Numerical subscripts for M_z refer to the moment at each load cell.

The eccentricity ratio about the central vertical (z) axis is defined as

$$\frac{r}{b} = \frac{C_{Mz}}{C_{Fx}} \quad (1-16)$$

where C_{Mz} is the mean twisting moment coefficient about the centroidal z axis, C_{Fx} is the mean normal force coefficient, r is the distance from the centroid of the model to the

point of application of the force, and b is the lateral length of the model. See Figure 1.5 for the location and orientation of component forces.

Table 1.1 Typical natural frequencies for cantilever and non-cantilever structures

Support Type	Truss Type	Material	Truss Span (m)	VMS	Natural Freq.		Source
					Out of plane (Hz)	In plane (Hz)	
Cantilever	New Jersey; 2-chord	Steel	13.3	Yes	0.8	1.2	NCHRP 469 (2002)
Cantilever	California; 4-chord box	Steel	8.7	Yes	1.8	1.9	NCHRP 469 (2002)
Cantilever	Illinois; Vierendeel	Steel	10.3	Yes	1.3	1.9	NCHRP 469 (2002)
Cantilever	Illinois; 4-chord box	Alum.	9.4	Yes	1.0	1.4	NCHRP 469 (2002)
Cantilever	IDOT; 4 chord box	Alum.	9.1	No	2.3	2.5	Foutch et al (2006)
Cantilever	Monotube mast arm	Steel	10.6	No	1.2	1.3	Fouad et al (2002)
Cantilever	2 Chord	Steel	13.9	No	1.0	1.6	Fouad et al (2002)
Non-cantilever	IDOT; 4 chord box	Alum.	28.7	Yes	3.3	4.2	Foutch et al (2006)
Non-cantilever	IDOT; 4 chord box	Alum.	28.7	Yes	2.9	3.4	Foutch et al (2006)
Non-cantilever	WisDOT; 4 chord box	Steel	21.7	Yes	-	6.3	Ginal (2003)
Non-cantilever	WisDOT; 4 chord box	Steel	32.4	Yes	-	2.4	Ginal (2003)
Non-cantilever	WisDOT; 3 chord	Steel	20.5	No	-	8.4	Ginal (2003)
Non-cantilever	PennDOT; 4 chord box	Steel	59.1	No	2.4	1.9	Kacin (2007)
Non-cantilever	Monotube	Steel	30.5	No	0.9	1.2	Fouad et al (2002)
Non-cantilever	Monotube	Steel	18.3	No	1.1	2.1	Fouad et al (2002)
Non-cantilever	3 chord	Steel	25.0	No	4.1	3.1	Fouad et al (2002)
Non-cantilever	3 chord	Alum.	45.7	No	2.1	2.1	Fouad et al (2002)

Table 1.2 AASHTO fatigue design requirements for VMS structures (AASHTO, 2013)

Type of Structure	Galloping	Vortex Shedding	Natural Wind	Truck Gusts
Cantilever Sign (1-, 2- or 3-chord)	X	*	X	X
Cantilever Sign (four-chord)	**	*	X	X
Non-cantilever Sign		*	X	X

* Vortex shedding has occurred in a monopole bridge support and can occur in cantilevered structures if the sign attachment is not attached

** Owner may choose to exclude galloping loads for 4-chord trusses

Figure 1.1 VMS structure on Florida's Turnpike (a) side elevation (b) front elevation
(photo courtesy Meyer)

Figure 1.2 VMS structure types (a) cantilever (b) bridge or non-cantilever (photo courtesy Meyer)

Figure 1.3 VMS support types (a) four-chord or box truss (b) three-chord or tri-chord truss (c) monotube (d) two-chord truss (photo courtesy Meyer)

Figure 1.4 VMS primary dimensions

(a)

(b)

Figure 1.5 Force orientation (a) Single support configuration (b) Double support configuration

2 BACKGROUND OF AERODYNAMICS RELATED TO SIGN STRUCTURES

2.1 General aerodynamics

VMS are generally rectangular prismatic shapes that are considered bluff bodies aerodynamically. The wind flow around a VMS is therefore characterized by separation from the upstream leading edges of the structure. This forms an outer flow region and an inner turbulent wake region separated by a thin layer of high shear and vorticity. This unstable free shear layer generates discrete vortices which are shed into the downstream wake region. The base pressure in the wake region determines the amount of vorticity that is shed from each side of the body which in turn affects the distance to vortex formation and the strength of the fully formed vortices (see Figure 2.1). This complex equilibrium between the vorticity shedding, the distance to vortex formation, and the base pressure significantly influences the pressure distribution and total forces experienced by the structure (Bearman and Trueman, 1972).

2.1.1 Flow around flat plates

Aluminum flat panel signs and VMS with small depth ratios are examples of flat panels. The primary force on a flat panel mounted normal to the direction of the wind is the drag force. Positive pressure from the oncoming wind on the front face provides approximately 60% of the drag and 40% results from the negative pressure on the rear face (Holmes, 2001).

The drag effect on a flat panel can be influenced by several factors such as 1) aspect ratio, b/c , of the sign (see Fig. 1.4 for geometric notation), 2) vertical clearance above ground (clearance ratio; c/h), and 3) turbulence in the oncoming flow (Letchford, 2001; Laneville et al., 1975).

The drag coefficient for a square shape ($b/c = 1$) will be smaller than the drag coefficient for an elongated rectangular shape. This is because wind flows equally around all four sides of the square shape. However, the longer dimension of a very tall and narrow or short and wide shape provides a higher resistance flow path and greater opportunity for interaction of the shear layers separating from around the elongated sides (Holmes, 2001). Wind is forced to travel faster around each of the tall vertical sides or over the top and bottom of the wide sign panel. The faster winds speeds cause more entrainment from the wake region into the shear layers resulting in a lower base pressure and an increase in drag (Letchford, 2001).

Drag decreases as the vertical clearance decreases (c/h increases). In the case of a suspended two dimensional strip (infinitely wide), strong vortices are shed into the wake region which causes increased entrainment and higher drag. However, shedding of these vortices can be suppressed and the increased drag effect can be eliminated by the introduction of a splitter plate into the wake region. In much the same way, shear layer interaction is stifled as the vertical clearance beneath the sign is reduced (c/h increases) and ultimately eliminated when the sign is attached to the ground ($c/h = 1$) which serves the same purpose as a splitter plate in reducing the drag (Holmes, 2001). Figure 2.2 illustrates how the suppression of vortices affects drag for the case of a splitter plate and a wall on the ground ($c/h = 1$).

Wind tunnel tests and flow visualization experiments by Laneville et al. (1975) indicated that an increase in free stream turbulence in the oncoming flow can influence the force coefficient of a sharp edged bluff body. Experimental results showed that reattachment does not occur after separation from the leading edge of thin bodies. However, depending on the intensity, turbulence can increase mixing and entrainment of air into the separated shear layers. Increased entrainment reduces the base pressure which in turn decreases the radius of curvature of the main streamlines. This results in an increase in drag (Laneville et al., 1975). Figure 2.3 graphically shows a comparison of turbulent and less-turbulent flows around a thin plate and the resulting effect on the drag.

2.1.2 Flow around prismatic shapes

Flow around square and rectangular prismatic shapes has been investigated extensively because these shapes have many practical engineering applications. In general, flow around square and rectangular prismatic shapes can be divided into two categories: bluff bodies with separated flow and bluff bodies with separated flow followed by reattachment along the sides.

Yen et al. (2011) provided an in-depth description of the flow structure around square bluff bodies ($d/c = 1$) by conducting wind tunnel experiments and applying visualization techniques in the wake region. Three characteristic modes were identified which describe the flow around prismatic shapes; 1) leading edge, 2) separation bubble and 3) attached flow (refer to Figure 2.4). The leading edge mode occurred for small vertical angles of incidence and was characterized by flow separation at the leading edge. In the separation bubble mode, flow separated near the front edge but reattached to the side face where a

counter clockwise separation bubble was formed. D'Auteuil (2006) describes the separation bubble as the flow region between the leading edge corners and the separation bubble. He also mentioned that reattachment will depend on the shape of the body and the depth ratio. In addition, the length and intensity of the separation bubble could be influenced by the state of the boundary layer before separation at the leading edge. Yen et al. (2011) found that the separation bubble flow mode occurred at a vertical incidence angle of approximately $\theta = 12^\circ$ (refer to Figure 2.4(b)) which also corresponded to the lowest measured drag coefficient. In the attached flow mode which occurred at vertical incidence angles greater than 30° , no leading edge separation occurred and the flow moved along the square cylinder.

Many researcher have studied rectangular prisms with different depth ratios to evaluate the potential for reattachment along the sides and the resulting effect on drag (Nakaguchi et al., 1968; Bearman and Truman, 1972; Laneville et al., 1975; Larose and D'Auteuil, 2008; Sohankar, 2008). Initially it was thought that the drag coefficient for flat plates was the same as for prismatic bluff bodies (i.e. for 2D and 3D shapes) with a value around 2. However, research conducted by Nakaguchi et al. (1968) reported a peak in the drag coefficient for a depth ratio around $d/c = 0.6$. Bearman and Trueman (1972) investigated the high drag results obtained by Nakaguchi et al. (1968) for rectangular cylinders at the critical depth ratio of $d/c = 0.62$. They conducted 2D wind tunnel experiments on rectangular cylinders for Re between 2×10^4 and 7×10^4 with depth ratios in the range of $0.2 < d/c < 1.2$. Measurements of the surface pressure for depth ratios $d/c = 0.2$ and 1.0 showed a uniform base pressure distribution. However, measurements for

$d/c = 0.6$ showed that the base pressure was significantly lower towards the center of the base indicating that vortices were forming along the base centerline.

A primary result of Bearman and Trueman's (1972) work was the explanation of the difference between flow around a thin plate (small values of d/c) and flow around a prismatic rectangular cylinder (large values of d/c). They demonstrated that the distance from the leading edge separation point to the onset of vortex formation is longer (further downstream) for the elongated shape. Their results indicated that drag is affected by the downstream distance to vortex formation and the narrowness of the wake. Since vortices represent regions of low pressure the further downstream that they form, the higher the base pressure and the lower the drag. The Experiments conducted by Bearman and Trueman (1972) showed that as d increased from that of a flat plate, the size of the separated wake region is reduced because the increased volume of the afterbody occupies part of the wake. However, the same entrainment process continues to act on the smaller wake volume causing the base pressure to decrease and the drag to increase. If d remains small enough the afterbody will not interfere with the inward curvature of the streamlines (see Figure 2.3) and full strength vortices will form closer to the rear of the body resulting in a lower base pressure. The base pressure will continue to drop with increasing d until the minimum is reached at about $d/c = 0.62$. When d is sufficiently large, the downstream corners of the afterbody will begin to interfere with the inward curvature of the streamlines. The shear layers may reattach along the sides of the afterbody delaying the interaction between shear layers. Although the shear layers will eventually separate again from the trailing edge corners, vortex formation will be further

downstream, the wake will be smaller, and the entrainment will be weaker. This will result in lower drag (Bearman and Trueman, 1972).

Laneville and Yong (1983) also investigated the high drag result for $d/c = 0.62$. They used visualization techniques to obtain time averaged patterns and the geometric configuration of vortex shedding in the wake region of rectangular cylinders. They introduced two new parameters, V_p which is the distance from the trailing edge of the body to the shed vortex center and V_c which is the lateral distance between the vortex centers (see Figure 2.1). Smaller values of V_p indicate that the vortices are forming close to the trailing edge of the structure. Smaller values of V_c indicate that the vortices are forming very close together which results in a non-linear pressure distribution on the rear surface of the structure. They found that V_c is large for high and low d/c ratios but very small for intermediate d/c ratios. A significant variation in drag was noted for d/c in the range of $0.3 < d/c < 0.8$ with the highest drag occurring at approximately $d/c = 0.6$. This corresponded with smaller values of V_c and V_p and confirmed the results obtained by Bearman and Trueman (1972).

For elongated shapes (those with higher d/c) the presence of free stream turbulence can effect drag by promoting reattachment of separated shear layers (Laneville et al., 1975). As discussed previously, Laneville et al. (1975) showed that free stream turbulence increases mixing and entrainment in the separated shear layers. Their experiments showed that in the case of thin shapes, where no reattachment occurs, this causes a decrease in base pressure (i.e., high magnitude of suction) and a reduced radius of curvature resulting in an increase in drag as shown in Figure 2.3(a). However, in the case of elongated shapes, their results indicated that reattachment along the sides of the

shape may occur if the shear layer entrainment is large enough. Although the streamlines eventually separate from the trailing edges, the wake is narrower and the distance to vortex formation is further downstream resulting in reduced drag (Laneville et al, 1975). The turbulent and non-turbulent dividing streamlines for flow around an elongated shape is shown in Figure 2.3(b). VMS structures with large depth ratios represent such elongated shapes and are expected to have lower drag in turbulent flows.

Nakamura and Ohya (1984) expanded the results of Laneville et al. (1975) and investigated the effects of turbulence intensity and turbulence scale on the base pressure of 2D rectangular cylinders with various depth ratios. Their results confirmed a peak in drag at the critical depth ratio of $d/c = 0.62$ in smooth flow but noted that the value of the base pressure coefficient was reduced and the peak occurred at a smaller depth ratio ($d/c \approx 0.5$) when subjected to a flow with small scale turbulence. Their results for the effect of free stream turbulence can be summarized as follows: small scale turbulence was shown to decrease the base pressure of short rectangles but increase the base pressure for longer rectangles. Large scale turbulence appears to increase the base pressure for all rectangles (Nakamura and Ohya, 1984).

Sitheeq et al. (1997) extended the previous research to 3D cylinders. They performed wind tunnel tests to determine the effect of turbulence scale on mean and fluctuating pressures. Tests were conducted on 3D square prisms subjected to three boundary layer flows. The three simulated flows had significantly different integral length scales of 0.69 m, 0.48 m, and 0.22 m. However, the turbulence intensity at the prism height of 100mm was similar for two of the boundary layers with turbulence intensities of 18% and 15% and only slightly lower for the third at 10%. The power law exponent for all three

boundary layers was around $\alpha = 0.27$ but the velocity at the model height was different for each boundary layer. The value of the calculated pressure coefficients for all three boundary layer flows were very similar. Sitheeq et al. (1997) concluded that there was “no indication of a significant effect of integral length scale on the mean and rms pressure distribution.” It is suggested by the researchers that the mean and fluctuating pressures on the prism are primarily controlled by the small scale turbulence content of the flow and to a much lesser extent by the integral turbulence length scale. The results of this study are consistent with studies conducted by Tieleman and Atkins (1990) and Tieleman (1992).

Li and Melbourne (1999) investigated the effect of large scale turbulence on pressure fluctuations in separated and reattaching flows by conducting 2D wind tunnel tests on rectangular cylinders over a wide range of turbulence intensities and integral length scales. Results of their testing indicated that mean pressure distributions are strongly dependent on turbulence intensity but not significantly affected by turbulence scale over range of turbulence scale ratios from 0.4 to 3.6. However, at a turbulence scale ratio of 6, mean pressure data was shown to be dependent on turbulence scale. In addition, the mean pressure coefficients measured at the leading edge for very large scale turbulence flows (turbulence scale ratio > 12) were similar to the values obtained in smooth flow. Li and Melbourne (1999) explained that for relatively small scale turbulence, an increase in turbulence intensity reduces the size of the separation bubble. On the other hand, large scale turbulence (turbulence scale ratio > 5) reduces the magnitude of the mean pressure coefficient in the region near the leading edge. A further increase of turbulence scale causes the mean reattachment length to increase which corresponds to an increase in the

mean pressure coefficient near the separation location and a decrease of the coefficient value in the reattachment zone. Li and Melbourne (1999) noted that this was in agreement with results obtained by Nakamura and Ozono (1987) and suggested that turbulence of very large scale is equivalent to a flow with slowly fluctuating velocity which can no longer effectively influence the mean flow of a bluff body.

2.1.3 Corner modification

The aerodynamics of bluff bodies like VMS depends largely on the behavior of the separated shear layers. As a result, forces due to wind loading on these structures can be reduced by controlling the separated flow around the structure. Several research studies have investigated the effects of modified corners on drag. Following is a summary and discussion of three key research studies that focused on the aerodynamic effects of rounded and chamfered corners.

Tamura and Miyagi (1999) conducted wind tunnel tests on 2D and 3D square cylinders each with sharp, chamfered or rounded corners in smooth and turbulent flows. Two dimensional flow characteristics were maintained by using end plates above and below the model. The same model was used in three dimensional tests with only the top plate removed. The Reynolds number for the test setup was 3×10^4 based on a length dimension of 50 mm and the longitudinal turbulence intensities were 6.5% and 14%. Their results for cylinders with sharp edges indicated that the drag coefficient decreases as turbulence intensity increases. This is consistent with previous research results. The research also showed that the value of the drag coefficient was affected by corner modification and found to decrease for both uniform and turbulent flows. The largest

decrease in the drag coefficient was obtained for cylinders with rounded corners but a reduction was also noted for cylinders with chamfered corners. Tests on 2D and 3D models with all corner configurations quantitatively indicated that value of the drag coefficient for 3D cylinders is smaller and varies less with vertical angle of attack than 2D cylinders. A plot of Tamura and Miyagi (1999) lift coefficient (C_L) data in turbulent flow indicates a negative slope for square cylinders with sharp and chamfered corners at a vertical attack angle of 0° . The researchers suggests this is an indication of complete separation of flow in the wake region. However, the slope is positive for cylinders with rounded corners which they suggest indicates a separation of flow followed by reattachment. Tamura and Miyagi (1999) concluded that corner modification decreases the wake width and promotes reattachment resulting in reduced drag forces.

Yamagishi et al. (2010) also examined the flow characteristics around cylinders with modified corners. They conducted wind tunnel experiments, numerical analysis, and applied visualization techniques on square cylinders with various chamfer dimensions. The cylinder cross section dimensions ($d \times d$) used in the wind tunnel experiments were 30 x 30 mm and 420 mm in length. Chamfers were cut along each edge of the cylinder. Three models were configured for the tests, C1, C3, and C5 with chamfer sizes of $C = 1$ mm, 3 mm, and 5 mm which corresponded to a $C/d = 0.033$, 0.100 and 0.167 respectively. Turbulence intensity for the wind tunnel flow was 0.65% with a maximum wind speed of approximately 35 m/s. This corresponded to test Reynolds numbers ranging from 1×10^3 to 6×10^4 . Numerical analysis was performed using Fluent 6.3 utilizing the finite volume method. Wind tunnel tests indicated that the drag coefficient remained constant in the tested Reynolds number range for all cylinder configurations.

Computational results were in good agreement with the experimental results. Wind tunnel tests for a Reynolds number of 6×10^4 and computational analysis showed that the drag coefficient for the C1 cylinder and the cylinder without chamfers ($C/d = 0$) was approximately equal with a value around 2. However, the drag coefficient dropped abruptly (40%) to approximately 1.2 for C3 ($C/d = 0.1$). It then increased slightly (20%) to approximately 1.5 for C5 ($C/d = 0.167$) Yamagishi et al. (2010). Visible flow patterns obtained by Yamagishi et al. (2010) using a propylene glycol mist tracer showed that the separation area of C3 was smaller than the areas for the sharp edge square and C5. They also noted that the tangential velocity of the corner end side of C5 was large compared to C3 which they thought contributed to the enlarged separation area of C5 and increased drag (the drag coefficient for C5 was still smaller than the cylinder with square corners). Visualization techniques also confirmed that the drag coefficient increased as the width of the wake behind the cylinder became larger Yamagishi et al. (2010).

Larose and D'Auteuil (2008) carried out experiments to study the effect of Reynolds number on chamfered edge corners for 2D rectangular prisms with depth ratios of 2, 3, and 4. Models were tested in a pressurized wind tunnel to determine the static force coefficients of drag, lift, and pitching moment. The test Reynolds number ranged from 0.15×10^6 to 4×10^6 based on a constant model depth of 76.2 mm. Pitch angles from -2° to 10° were tested for each run of the three depth ratios (2, 3, and 4), three edge configurations (sharp, small chamfer and large chamfer) and two flow conditions (smooth and turbulent). The researchers chose the shapes because they were suspected to have separated flow regions and possible flow reattachment. Results for the sharp edge configuration with a depth ratio of 2 showed a gradual increase in the drag coefficient as

the Reynolds number increased from 0.25×10^6 to 1.0×10^6 . The lift coefficient remained constant over the same Reynolds number test range. However, the configuration with large chamfers showed a significantly different trend. The lift coefficient varied with Reynolds number, even having a sign reversal when the Reynolds number was increased from 0.4×10^6 to 1.0×10^6 , while the drag coefficient remained invariant over the same Reynolds number test range. Larose and D'Auteuil (2008) noted that this difference in results for sharp edges and large chamfers demonstrated the sensitivity of edge configuration to Reynolds number for the specified test conditions.

It should be noted that in tall building design, corner geometry changes have played an important role in reducing wind loads. For example, corner refinements on the 509 m tall Taipei 101 building led to a 25% reduction in the wind-induced base moments (Irwin, 2008).

2.2 Aeroelastic instability

The primary load on VMS structures is generally due to wind-induced drag. However, aeroelastic phenomena, caused by the interaction between the natural wind and the structure's vibrational characteristics, can lead to significant vertical forces. These are due to the phenomena of vortex shedding and galloping instability. The resulting vibrations can lead to fatigue damage and ultimately failure of the structure (Kaczinski et al., 1998). Note that vortex shedding and galloping responses are characterized by aeroelastic feedback whereby motions of the structure normal to the direction of wind flow affect the flow itself and amplify the aerodynamic force fluctuations.

2.2.1 Vortex shedding

Vortex shedding occurs during steady uniform flow when vortices, from each face of the structure, with alternating rotational direction are periodically shed into the wake of a structure. The alternating pattern of vortices is commonly referred to as von Karman vortex street. The frequency at which the vortices are shed (f_s) is proportional to the approaching mean wind speed ($\bar{U}$) and inversely proportional to the lateral height (c) of the structure. This is expressed non-dimensionally as the Strouhal number

$$S_t = \frac{f_s c}{\bar{U}} \quad (2-1)$$

The Strouhal number for a circular cross section depends on the Reynolds number of the oncoming flow. However for sharp edged cross sections like VMS where flow separation occurs at the leading edge, the effects of Reynolds number are expected to be much less. However the Strouhal number has been shown to be a function of the depth ratio. A relationship between the Strouhal Number and depth ratio for rectangular cross sections was developed by Dyrbye and Hansen (1997) and is reproduced in Figure 2.5. Since VMS are principally rectangular prismatic shapes, their Strouhal number should be in the range of 0.06 to 0.15 (Ginal, 2003).

Parkinson (1989) compiled results from Brooks (1960), Hoerner (1965), Nakamura and Tomonari (1977), and Washizu et al. (1978) and showed how the Strouhal number is affected by the depth ratio. For smooth flow and a depth ratio in the range of $0 < d/c < 1.0$, the base pressure changes abruptly with increasing d while the Strouhal number only changes slightly. A plausible explanation for this is provided by Bearman and Trueman (1972). If d is small enough, the downstream edges of the afterbody will

not interfere with the inward curvature of the streamlines and full strength vortices will form closer to the rear of the cylinder. This lack of interference with the shear layers is why the Strouhal number is not significantly affected. When d is sufficiently large, the downstream edges of the afterbody will begin to interfere with the inward curvature of the streamlines forcing the vortices to form further downstream and the shear layers to be more diffused. This results in a smaller vortex shedding frequency and a drop in the Strouhal number (Gerrard, 1966). Reattachment of flow along the sides of the afterbody results in an increase in base pressure and a sharp increase in the Strouhal number. The abrupt change arises due to the decrease in lateral spacing of the shear layers after separating from the downstream corners. Smaller lateral spacing of the shear layers corresponds to a smaller stream wise spacing of vortices and therefore a larger vortex shedding frequency (Parkinson, 1989).

The alternating shedding of vortices produce periodic forces that result in oscillations in a plane normal to the direction of wind flow. Significant oscillation can occur when the frequency of vortex shedding is close to the natural frequency of a flexible structure. These lateral vibrations have a strong organizing effect on the vortex shedding pattern which can increase the strength of the vortices and couple the vortex shedding frequency to the natural frequency of the structure. This phenomenon is known as lock-in. The critical wind speed (U_{cr}) at which lock-in occurs can be estimated using the Strouhal number relation

$$U_{cr} = \frac{f_n c}{S_t} \quad (2-2)$$

Where in this case f_n is the natural frequency of the structure. The amplitude of the vibrations resulting from lock-in are limited by the balance between the energy input into the motion by the vortices and the dissipation of that energy by structural damping. Eventually large oscillations of the structure interfere with the uniform shedding of vortices and therefore the maximum amplitude of vortex induced vibrations may be self-limiting (Blevins, 1977). The drag force on a structure vibrating at or near the vortex shedding frequency is also a function of vibration amplitude. Bearman and Obasaju (1982) showed that drag on a square structure increases at resonance.

The range of wind speeds at which vortex shedding can occur is bounded by the ability of the vortices to stay locked onto the structural motion. According to the commentary in the AASHTO Specification (2009), vortices shed at wind velocities below approximately 5 m/s do not possess sufficient energy to excite most sign support structures and at wind speeds greater than about 20 m/s the natural turbulence in the flow disturbs the formation of vortices. This would imply VMS structures are susceptible to vortex induced vibrations only for the range of wind speeds between 5 m/ and 20 m/s. In fact there is little evidence that the turbulence intensity increases with wind speed. Therefore vortex excitation at speeds above 20 m/s cannot be discounted.

2.2.2 Galloping

Galloping of a prismatic structure is the self-excited response to natural wind due to instability of the structure. Galloping is typically characterized by predominately horizontal wind flow and vertical motion of the structure. To initiate galloping there must be an initial displacement of the structure that changes the angle of attack (α) of the wind

flow relative to the structure. The initial displacement may be due to fluctuations in the wind or vortex shedding. The vertical velocity ($\dot{z}$) of the structure results in the angle of attack of the wind being

$$\alpha = \arctan\left(\frac{\dot{z}}{\bar{U}}\right) \quad (2-3)$$

Where $\bar{U}$ is the horizontal velocity of the oncoming wind. For $\dot{z} \ll \bar{U}$ this becomes

$$\alpha = -\left(\frac{\dot{z}}{\bar{U}}\right) \quad (2-4)$$

If evaluation of the Den Hartog criterion yields

$$\frac{dC_L}{d\alpha} + C_D < 0 \quad (2-5)$$

where, in the present notation, C_L is the vertical lift coefficient, α is the angle of attack, and C_D is the normal force coefficient, the motion of the structure results in a force given by

$$F_z = -\frac{1}{2}\rho U \dot{z} A \left(\frac{dC_L}{d\alpha} + C_D\right) \quad (2-6)$$

The force is in the same direction as the structural motion $\dot{z}$ if $\frac{dC_L}{d\alpha} + C_D < 0$ and proportional to the structural velocity. Therefore it is effectively like a negative aerodynamic damping force and if it exceeds the positive damping force due to the structural damping it results in unstable oscillations, called galloping, that grow in amplitude. Galloping from equilibrium can only occur if the magnitude of the negative aerodynamic damping is greater than the positive mechanical damping of the structure.

Therefore the minimum wind speed required to initiate galloping is proportional to the mechanical damping of the structure (Novak, 1972).

Experimental and theoretical research conducted by Parkinson (1963) demonstrated that the magnitude of the angle of attack (α) is important to the galloping characteristics of prismatic shapes. Parkinson (1963) explained that flow reattachment will occur along the sides of a prismatic shape for sufficiently large values of α . The galloping characteristics of the structure will be affected by the resulting asymmetry in the pressure distribution which produces a net force in the vertical direction.

Parkinson (1989), using data from Brooks (1960), Smith (1962), Laneville (1973), Novak (1974), and Nakamura and Tomonari (1977), showed that rectangular cylinders with $d/c < 0.75$ in smooth flow did not gallop from rest (hard galloping) and required an initial vibration. Sections in the range $0.75 < d/c < 3.0$ did gallop spontaneously from rest (soft galloping). The lower boundary for the beginning of soft galloping corresponds to the abrupt change in base pressure noted previously when the trailing edges of the afterbody begins to interfere with the inward curvature of the streamlines. For $d/c > 3.0$ no galloping could be induced. This upper boundary corresponds to an afterbody length that is sufficiently long enough for reattachment of the shear layer to occur. Parkinson (1989) also demonstrated that as turbulence in the oncoming flow is increased, soft galloping sections experience weaker galloping and eventually become stable and hard galloping sections become soft galloping.

A structure that meets the Den Hartog criterion for galloping may not gallop under service conditions. Kaczinski et al. (1998) had difficulty reproducing galloping results in wind tunnel experiments. A model observed to gallop in one test did not gallop in another

test under identical test conditions. The same unpredictability has been observed in the field. Only one structure experienced galloping in a series of structures subjected to the same wind conditions. The researchers attributed the difficulty in reproducing galloping results in the laboratory and the unpredictability observed in the field to the sensitivity of these structures to very specific conditions such as the dynamic properties of the structure, aerodynamic properties of the attachments, and flow characteristics (Kaczinski et al., 1998).

In-service VMS attached to cantilever support structures with fundamental frequency of 1.04 – 1.10 Hz and structural damping ratio of 0.4 – 0.7% have been observed galloping. Structural failure due to galloping was documented for a structure in California in 1995. Instrumentation and monitoring of a cantilever bent monopole VMS support structure showed that it was galloping in steady winds (Dexter and Ricker, 2002).

The susceptibility of non-cantilever (i.e. bridge-type) VMS support structures to galloping is still unclear. Dexter and Ricker (2002) did not expect VMS mounted on non-cantilever structures to be susceptible to galloping “due to the rigidity of the sign bridge.” Fouad et al. (2003) points out that the question of susceptibility of non-cantilever support structures to galloping (and vortex shedding) is unresolved and additional laboratory testing and field evaluation is necessary. However, Fouad et al. (2003) did not specifically address VMS.

Ginal (2003) evaluated the galloping instability of two non-cantilever VMS support structures (S-40-404, S-67-402) in Wisconsin. Structure S-40-404 was a steel four chord pipe truss with a span of 32.3 m. Structure S-67-402 was also a steel four chord pipe truss but with a span of 21.6 m. Each structure supported a 2.3 x 7.3 m VMS. The dimensions

of the VMS correspond to an aspect ratio of $b/c \approx 3$ and a depth ration $d/c \approx 0.5$. A modal analysis using FEM was performed and the onset wind velocities were determined using the universal galloping response curves for rectangular prisms developed by Novak and Tanaka (1974). Based on calculations and assumptions by Ginal (2003), the range of onset wind velocities for the two structures were determined to be

$$98 \text{ m/s} \leq U_{\text{crit S-40-404}} \leq 133 \text{ m/s} \quad (2-7)$$

$$212 \text{ m/s} \leq U_{\text{crit S-67-402}} \leq 289 \text{ m/s} \quad (2-8)$$

Due to these high velocities, Ginal (2003) concluded that galloping vibrations do not need to be considered as a loading scenario for the non-cantilevered VMS support structure as configured in his test group. He adds however, that galloping of sign support structures are highly dependent on the characteristics of the structure including cross section, total damping ratio, and natural frequencies as well as flow characteristics and suggests that a wind tunnel study is conducted to validate the results of his research.

2.2.3 Summary of NCHRP reports on fatigue induced vibration due to vortex shedding and galloping

An important source of information on the susceptibility of sign structures to galloping and vortex shedding are the National Cooperative Highway Research Program (NCHRP) reports. The following is a discussion of two in-depth initiatives sponsored by AASHTO that focused on fatigue induced vibrations resulting from galloping and vortex shedding in sign support structures. Although these studies did not specifically address VMS support structures, current AASHTO procedures and VMS design requirements are based on the research performed for these reports. The first study, Kaczinski et al. (1998)

investigated the susceptibility of cantilever structures to galloping and vortex shedding and the second study, Fouad et al. (2003) investigated susceptibility of non-cantilever structures.

Research conducted by Kaczinski et al. (1998) included aerodynamic and aeroelastic wind tunnel tests on scaled models of cantilever support structures. Results of the aerodynamic tests demonstrated that cantilever sign and signal support structures are potentially susceptible to galloping due to the aerodynamic characteristics of attachments such as signs and signals. The aerodynamic test program also indicated that the susceptibility of signs to galloping was independent of aspect ratio and whether or not the approach wind was from the front or rear. However, these tests did not include oblique wind directions. The aeroelastic test program included wind tunnel tests and finite element modeling of two cantilever sign support structures (single arm sign support and a two-chord truss structure). The scaled models were mounted to a dynamic balance and positioned so that the vertical plane of the model was normal to the wind tunnel flow. The balance was used to measure the base moment and strain gages measured the drag and lift moments. Several conclusion regarding the susceptibility of cantilever sign support structures to galloping and vortex shedding were determined from the testing program. The single arm sign support exhibited significant vertical plane oscillations consistent with galloping but the cantilever truss structure did not exhibit significant vertical plane oscillations. The researchers did not conclude that cantilever truss structures were less susceptible to galloping since there have been reported failures of in-service cantilever trusses caused by vertical plane vibrations which are consistent with galloping. Instead they noted that galloping is definitely possible for most types of

cantilever structures but that galloping is very sensitive to specific conditions, does not occur frequently, and is difficult to reproduce in wind tunnel experiments. They concluded by recommending that cantilever structures installed on high volume high speed facilities should be designed for galloping.

Regarding vortex shedding, they concluded that the Strouhal number provides a reasonable estimate of the critical wind velocity at which vortex shedding can be expected. None of the models with attachments (signs) exhibited vortex induced vibrations even though the predicted critical wind velocity was above the 4.5 m/s threshold. The sign attachments were thought to interfere with the formation of vortices. The single arm cantilever support did exhibit a significant vortex induced response without attachments. The researchers concluded that attachments on horizontal supports of cantilever structures disrupts the *spanwise correlation* of vortex shedding. As a result they suggested that vortex shedding induced vibrations should only be considered during construction prior to the installation of the sign attachments. Finite element analysis was then used to develop equivalent static limit state load ranges to be used in the fatigue design of cantilever support structures.

Fouad et al. (2003) conducted a survey of state departments of transportation to determine fatigue and vibration concerns for non-cantilevered support structures. Although most responders indicated satisfactory performance of non-cantilevered support structures, some concerns for fatigue and vibration were reported. Furthermore, it was revealed that steel non-cantilevered support structures rarely have vibration problems but more problems have been noted in aluminum structures. It was also noted that when vibration did exist there was typically substantially less mass attached to the structure

than what was intended in the design. The researchers conceded that the susceptibility of non-cantilevered structures to galloping and vortex shedding is unresolved and that additional experimental laboratory and field work is needed. Since the conditions for wind induced vibrations exist for non-cantilevered structures, the researchers proceeded by conservatively assuming non-cantilevered support structures were susceptible to the same fatigue and vibration inducing mechanisms as cantilevered structures including galloping and vortex shedding. Finite element analysis was conducted on six representative structures and *equivalent static fatigue loads* (AASHTO Specification, 2009; equations 11-1 through 11-6) were established for non-cantilevered structures. Based on their study, the researchers recommended that the same galloping and vortex shedding equivalent static fatigue loads used for cantilever structures should also be used for non-cantilever structures.

2.3 Literature review for drag on elevated signs

Experimental research on elevated panels and VMS support structures is very limited. The following is a summary of the major research relevant to drag coefficient development for elevated flat panel signs.

A wind tunnel study by Ginger et al. (1998) was conducted at the Cyclone Structural Testing Station (CSTS), James Cook University to determine net pressures on a variety of fence (wall) and hoarding (elevated panel) configurations. Aspect ratio (b/c) and wind direction (θ) was investigated for walls and clearance ratio (c/h) and wind direction (θ) were investigated for the elevated panels with a fixed aspect ratio of $b/c = 2$. Results for the elevated panel tests are discussed here. A 80 mm x 160 mm x 8 mm (height x length x

thickness) pressure tapped model mounted on two legs was configured to determine pressures on elevated panels. At a 1:50 length scale, this represented a 4 m x 8 m x 0.4 m full-scale panel. The clearance below the panel was adjusted by changing the height of the legs and measurements were made for clearance ratios of 0.57, 0.67 and 0.80. The pressures were measured for approach wind directions of -90° through 0° to 90° , in 15° increments. The mean velocity and turbulence profile simulated in the CSTS wind tunnel fell between the Australian code (AS1170.2) terrain categories of 2 and 3. Ginger et al. (1998) noted that previous studies indicated that the loading on walls and elevated panels behave quasi-statically. Therefore only mean pressures were obtained in this study. Table 2.4 shows a comparison of the Ginger et al (1998) CSTS results for mean net pressure coefficients and results obtained by using the equation proposed by Letchford (2001) (see Equation 2-9). The Ginger et al (1998) results are in good agreement with the mean force coefficient values predicted by Letchford's formula. Also shown in Table 2.4 are the Ginger et al (1998) CSTS results for the $\theta = 45^\circ$ wind direction. The Ginger et al. (1998) results indicate an apparent reduction in the drag coefficient for the $\theta = 45^\circ$ direction which is not in agreement with the wind direction independence noted by Letchford (2001). Ginger et al. (1998) also compared their CSTS results to code values in the Australian (AS1170.2), the American (ASCE 7-95) and the European (ENV 1991-2-4) standards for $\theta = 0^\circ$ and $\theta = 45^\circ$. The results obtained by Ginger et al. (1998) are lower than the code values contained in AS1170.2 and ENV 1991-2-4. The CSTS results were considerably larger than the values contained in the ASCE 7-95 standard which was the applicable standard at the time of the study. However, the coefficients contained in the current ASCE 7-10 standard are based on Letchford's results and consequently are in

good agreement with the CSTS results. It should be noted that the values contained in ASCE 7-10 are obtained by multiplying Letchford's results by 1.18 to make them compatible with the design procedure used in the code.

Letchford (2001) conducted the first comprehensive investigation of the effect of drag on a variety of elevated flat panel sign geometries. A series of wind tunnel tests were performed in a simulated turbulent boundary layer on 1:50 scale flat rectangular sign models to investigate the effect of aspect ratio (b/c), clearance ratio (c/h), porosity, and wind direction on drag coefficients. Letchford's (2001) results for porosity are not discussed here since they were not included in the scope of this research study. The terminology and geometry used in Letchford's (2001) study is consistent with the current study (refer to Figure 1.4). Model sizes used to obtain the normal force coefficients ranged in size from 50 x 50 mm to 100 x 400 mm with gaps beneath the panels extending from 5 mm to 150 mm. The model thickness for the first force balance model was not given but the panel thickness for the second model was 5 mm. The mean velocity and turbulence intensity profiles for Letchford's test wind tunnel were between rural and suburban terrains. He noted however, that the wind tunnel spectrum of longitudinal turbulence had significantly higher energy content in the high frequency range (small-scale turbulence) than the corresponding full-scale spectrum (Figure 2.6). A summary of mean coefficient results for several model configurations of aspect and depth ratio is included in Table 2.1. Letchford's results demonstrated that for aspect ratios greater than one, as the sign panel becomes short and wide, the mean drag force increases for bigger gaps and decreases for smaller gaps. For aspect ratios less than one, as the panel becomes tall and skinny, the drag force increased irrespective of the clearance ratio. The study also

investigated how drag is affected by clearance ratio. The tests showed that drag decreases as the gap below the panel is reduced (c/h increases) and is smallest for the condition $c/h = 1$ when the panel is on the ground. As expected, results for small gaps ($c/h > 0.8$) are consistent with those obtained for walls while results for large gaps ($c/h < 0.2$) are consistent with results for an isolated panel. Letchford also noted that gaps in the intermediate region are dependent on both aspect and clearance ratio.

Figure 2.7 is a graph provided by Letchford (2001) that compares Flachsbart's (as reported by Letchford, 2001) results ($c/h = 0.2$) and Letchford's ($c/h = 0.16$) results for isolated panels along with the Australian (AS1170.2, 1989) and the U.S. (ASCE 7, 1995) code values. It is clear that Letchford's (2001) drag coefficients results were higher than Flachsbart's results which were obtained in smooth uniform flow. Letchford (2001) attributed this to the increased turbulence present in his boundary layer flow compared to Flachsbart's smooth flow results. However, he also noted that turbulence scale likely contributed to the differences and adds that excessive high frequency, small-scale turbulence present in the wind tunnel flow will lead to larger drag forces. Letchford provides an explanation based on Gartshore's results that increased turbulence in the flow can lead to a reduced radius of curvature of the streamlines resulting in a reduced base pressure (higher suction in magnitude on the back face) and an increase in drag. (Letchford, 2001).

A review of Letchford's results for mean drag as a function of wind direction for a panel with aspect ratio $b/c=2$ shows that the mean normal force coefficient remains approximately constant for wind approach directions up to 45° then decreases linearly.

The study proposed a formula to calculate the normal force coefficient (C_f) for aspect and clearance ratios in the range of $0.2 < b/c < 5$ and $0.2 < c/h < 1.0$ as

$$C_f = 1.45 + 0.5(0.7 + \log_{10}(b/c))(0.5 - c/h) \quad (2-9)$$

For aspect ratios outside of this range it is suggested that the free ends need to be considered. However, Letchford (2001) noted that using the results from earlier works, such as Flachsbart's (see Figure 2.7) may be unconservative due to the smooth flow conditions of those studies.

In the same year as Letchford (2001), Quinn et al. (2001) instrumented and monitored full scale roadside signs in the grass field at the Silsoe Research Institute. The parameters investigated were: sign shape, sign size, and inclination angle. Wind force coefficient data was collected on a representative range of 750 mm and 1500 mm temporary roadside signs at wind speeds of 4-12 m/s. Forces were measured directly using three load cells mounted on a triangular steel framework. The 1 minute mean values of wind speed and force were used to calculate the force coefficient. The mounting could be attached at different heights and inclination angles of 0° and 22.5° . A picture of the test set up is shown in (Figure 2.8). The results of Quinn et al. (2001) confirm that wind induced forces obey a quasi-steady mean dynamic pressure model for mean force. The sign shape did not appear to contribute significantly to the magnitude of the wind force coefficient (C_f). However, the addition of a frame around the square 750 mm sign increased the force coefficient. The researchers thought this was a result of the disruption of the flow around the sharp edge of the sign. It was suggested that the addition of a frame increases the aerodynamic area of the sign leading to an increase in C_f . A summary of selected results is presented in Table 2.2. Quinn's et al. (2001) results for mean force coefficients

do not agree well with Letchford's (2001) data. Although the turbulence intensity for both studies was approximately 25%, Quinn's et al. (2001) results were considerably lower than the study conducted by Letchford (2001). Quinn et al. (2001) noted a cosine reduction in the mean force coefficient for signs mounted at 1 m with a horizontal wind direction angle less than $\pm 30^\circ$. The dependence on wind direction was not noted for the signs mounted at 2 m. This led the authors to suggest that mounting height or the ratio of sign size to mounting height maybe an important parameter for future research. The study also investigated how large velocity gradients close to the ground affect the force coefficient. The authors suggested that parts of the structure closer to the ground experience wind speeds less than the nominal wind speed. This causes asymmetrical curvature of the streamlines around the structure. Spectral analysis showed that the estimated admittance function is generally in agreement with wind tunnel results for flat plates. However, the authors noted an increased response in the reduced frequency range of 0.1 – 1 indicating that the signs are responding more to atmospheric eddies of 1 – 10 times the characteristic length of the signs. This was thought to be a result of asymmetric full-scale eddies near the ground and the potentially large difference in longitudinal and vertical turbulence length scales.

The wind tunnel tests by Letchford (2001) and the full scale tests by Quinn et al. (2001) were followed by a limited wind tunnel study by Paulotto et al. (2006). Experimental tests were performed to investigate the effects of wind action on a frame-type signboard embedded in a regular array of identical buildings at the Inter-university Research Center for Building Aerodynamics and Wind Engineering boundary layer wind tunnel in Prato, Italy. For the first series of tests, the model was mounted in the center of

the turntable with no other obstacles on the surface. This was denoted as the “free standing condition”. The sign was tested in a boundary layer characterized by full scale turbulence intensity at the model height of 19% and turbulence integral length scale of 20 m. Measurements were obtained for wind directions in 45° increments for the full range of 360°. An average force coefficient of 1.65 for the 0° wind approach angle was determined from the free standing tests. This was in good agreement with Letchford’s (2001) comparable result of approximately $C_f = 1.6$. The force coefficient obtained for the 45° horizontal wind approach angle was significantly lower ($C_f = 1.15$). This finding is not consistent with Letchford (2001) whose values for C_f were approximately constant for angles between 0° and 45°. The authors attributed this discrepancy in part to the difference in the thickness of the model signboards used in each study. In Letchford’s (2001) study the model sign thickness was 0.5 mm and the sign thickness for Paulotto et al. (2006) was 9 mm. However the authors thought it was more significantly a consequence of the aerodynamic interference between the signboard and the posts in Paulotto et al. (2006) model. Figure 2.9 shows graphic representations of the different model schemes used by Letchford (2001) and Paulotto et al. (2006). The velocity field obtained by PIV technique showed clearly that the wake of the windward post in Paulotto et al. (2006) model reattached to the signboard and as a consequence, the value of C_f was significantly reduced. The authors suggest that the relative position of the posts and signboard strongly influences the wind load and should be accounted for in codes. The second series of tests by Paulotto et al. (2006) included a proximity model with its surface covered by a regular array of identical buildings which simulated the “embedded” condition. The mean values of the normal force coefficient for the “embedded” signboard

were smaller than those corresponding to the “free-standing” condition. The authors also noted that wind effects are less dependent on the wind direction in the “embedded” condition than in the “free-standing” condition.

Giannoulis et al. (2012) conducted a literature review of wind loads on vertical panels. The authors evaluated and compared experimental results for force coefficients obtained in wind tunnels and by full-scale testing with the corresponding provisional values in the European (EN 1991-1-4, 2005) and the American (ASCE 7-10, 2010) codes. Their review includes both permeable and non-permeable panels for walls and elevated panels. However, permeable panels are not relevant to this research project and consequently not included in the following summary. Force coefficient results for solid vertical panels on the ground (walls) and elevated (signs) are discussed. The authors note that several studies were conducted in the 1990s to clarify how wind loads on walls are affected by geometry, wind direction, and return corners. The results of these studies provided the basis for many of the international codes including EN 1991-1-4 and ASCE 7-10. Initial wall research focused on area averaged results for mid-width force coefficients and the effects of geometry and impinging wind direction. This research demonstrated that C_f ranges from 1.05 to 1.2 in most cases for 0° impinging wind and decreased for horizontal wind directions other than 0° . It was determined however, that the maximum load occurs at the free end of the wall where C_f may be as much as three times larger than C_f at mid-width. This was determined to be a result of high suctions forming leeward of the flow separation at the free end. Additional wind tunnel and full-scale studies were initiated to investigate this result. Research indicated that free end

force coefficients are dependent on aspect ratio (l/h), return corner length, and impinging wind direction. The following conclusions were made:

- For 0° impinging wind; wind loads at free ends are independent of aspect ratio.
- C_f at the free end for a specific aspect ratio increases as the wind direction increases from 0° to 45° .
- For the 45° horizontal wind direction; as the aspect ratio increases, the value of C_f increases to a maximum ranging from 3.0 to 3.5.
- Return corners reduce loads at the free end depending on the aspect ratio and return corner length.

The authors concluded that the research results for panels on the ground (walls) are in good agreement. The code provisions of ASCE 7-10 are also in agreement with the literature but the provisions adopted in EN 1991-1-4 were determined to be comparatively more conservative.

The two primary studies by Letchford (2001) and Quinn et al. (2001) are compared and discussed in detail by Giannoulis et al. (2012). Specific results for these studies were discussed previously and only a comparison of the results is discussed here. In general, Quinn's et al. (2001) results are consistently lower than Letchford's (2001). Giannoulis et al. (2012) attributes this to a difference in the terrain category and the thickness of the model used in each study. The authors point out that Quinn's et al. (2001) field measurements at Silsoe corresponded to a rural terrain category and Letchford's (2001) wind tunnel simulation was between rural and suburban terrain categories (higher roughness length). They also noted that the addition of a frame in Quinn's et al. (2001)

study increased the force coefficient for a sign with aspect ratio 1 and clearance ratio 0.55 from $C_f = 0.92$ to 1.1. Quinn et al. (2001) attributed the difference to an increase in aerodynamic area due to the addition of the frame. Quinn et al. (2001) did not provide the thickness of the frame so a comprehensive evaluation was not made by Giannoulis et al. (2012). However, they concluded that this was probably the dominant reason for the discrepancy. Force coefficients for different aspect and clearance ratios as reported by Quinn et al. (2001) and Letchford (2001) was provided by Giannoulis et al. (2012) and reproduced here in Table 2.3. Table 2.3 also includes the code values from the EN 1991-1-4 and ASCE 7-10 code. Giannoulis et al. (2012) points out that Letchford's (2001) results form the basis of the ASCE 7-10 code but the published code values multiply Letchford's (2001) reported results by a factor of 1.18 to make them compatible with the way the gust factor is incorporated into the design process. The published ASCE 7-10 values are shown in parentheses. The EN 1991-1-4 code provides a single value $C_f = 1.8$ for all aspect ratios with a reference height to the mid height of the panel. The reference height used by ASCE 7-10 is the top of panel but the EN 1991-1-4 values shown in the Table 2.3 were adjusted to be compatible with the ASCE 7-10 reference height at the top of the panel. Giannoulis et al. (2012) highlights the variability in the results of Quinn et al. (2001), Letchford (2001), and the two codes.

Giannoulis et al. (2012) compared the three main studies that investigated the effects of impinging wind direction (Letchford 2001, Quinn et al. 2001, and Paulotto et al. 2006). Letchford (2001) and Quinn et al. (2001) concluded that the force coefficient for elevated panels is independent of wind direction for the range of $0^\circ - 45^\circ$. Although Paulotto et al. (2006) result for 0° impinging wind ($C_f=1.65$) was in good agreement with

Letchford's (2001) value of $C_f = 1.6$, Paulotto et al. (2006) noted a reduction in C_f for the 45° impinging wind ($C_f = 1.15$). This is not in agreement with wind direction independence demonstrated by Letchford (2001) and Quinn et al. (2001).

Giannoulis et al. (2012) also compared the provisions of the European EN 1991-1-4 code and the American ASCE 7-10 code. The following major differences in base assumptions were noted:

- EN 1991-1-4 uses a logarithmic velocity profile; ASCE 7-10 uses a power law profile.
- EN 1991-1-4 had 5 roughness categories; ASCE 7-10 has 3 roughness categories.
- EN 1991-1-4 basic velocity is an average of 10 minutes at 10 m; ASCE basic velocity is a 3 second gust at 10 m.
- EN1991-1-4 reference height is at the mid panel; ASCE 7-10 reference height is at the top of the elevated panel.

As a result of these differences, the calculated design forces vary depending on the selected code. Giannoulis et al. (2012) demonstrated the effects of different code methodologies by calculating the design force for ground mounted panels with aspect ratios equal to 1, 2, 5, and 10 using the recommended provisions of each code. A comparison of the results revealed that the force coefficients used in EN 1991-1-4 are only about 10 - 17% larger than those used in ASCE 7-10. However, the characteristic design forces calculated using EN 1991-1-4 were in the range of 61 - 80% higher than comparable forces calculated using ASCE 7-10.

The previously discussed research studies focused on flat panel signs but recent studies conducted by Smith et al. (2014) and Zuo et al. (2014) investigated wind loading on rectangular box signs. The first study by Smith et al. (2014) consisted of measuring the wind loads on a full scale box sign (VMS) constructed at the Wind Science and Engineering field site located at the Reese Technology Center in Lubbock, Texas. The dimensions of the sign were 7.5 x 3.75 x 1.75 m and the top of the sign was 7.5 m above ground level. This geometry corresponded to $b/c = 2$, $d/c = 0.47$, and $c/h = 0.5$. The mean force coefficient was calculated as a weighted average of the net pressure coefficients for 12 transducers and was determined to be $C_{Fx} = 1.13$. The eccentricity ratio (r/b) results for angles of attack from 180° to 270° were generally less than 0.15. The full scale data was compared to the flat panel wind tunnel studies by Letchford (2001), Warnitchai et al. (2009), and Zuo et al. (2014). Smith et al. (2014) makes the point that the results of the small scale wind tunnel studies on flat panel signs provides an upper bound to the full scale results.

The second study conducted by Zuo et al. (2014), consisted of testing five configurations of small scale models (1:50) in the simulated open terrain boundary layer at the Texas Tech University Boundary Layer Wind Tunnel. Testing included a scaled model of the full size sign tested in Smith's et al. (2014) study and was used to validate Zou's et al. (2014) wind tunnel results. The width and height of the test model face was the same for all five models which were configured to represent a rectangular box sign, a single plate sign, an equal spaced double, a 30° V-shaped signs and a 10° V-shaped sign. The drag coefficient result of the full scale rectangular box obtained by Smith et al. (2014) was noted to be in good agreement with the small scale wind tunnel result by Zuo

et al. (2014). It was also noted that the mean force coefficient is largest when the horizontal wind direction was normal to the sign face and the mean torque about the vertical axis was largest when the sign was at a 45° horizontal angle to the mean wind direction. The results of the single and double plate signs are not relevant to this study and the results are not discussed further. Zuo et al. (2014) also tested thirty nine 1:50 scale rectangular box shapes with various configurations in the wind tunnel. Dimensions of the models are included in Table 2.5. The effect of depth ratio was not investigated by Zuo et al. (2014) even though the researchers acknowledged that the depth of the model was expected to affect the wind loading. The chosen depth for all models was 3.68 cm which corresponds to 1.84 m at full scale. Wind load data was obtained using both pressure and force transducers which produced similar results. Non-dimensional force and eccentricity coefficients were calculated and compared to previous flat panel results. Selected mean force coefficients and eccentricity coefficients are included in Table 2.6. It was evident from the data that the mean force coefficient for the rectangular box models were consistently lower than the results obtained for single plate signs with the same aspect and clearance ratio. This finding was attributed to partial reattachment of flow as reported by Laneville et al. (1975) and discussed in detail previously. Based on these results, the researchers concluded that rectangular box sign structures (i.e. VMS) are over designed since the current standards are based on the experimental results of single plate signs. The dependency of the force coefficient on aspect ratio was also evaluated. It was shown that the force coefficient increases with increasing aspect ratio when the clearance ratio is smaller than 0.7 which is consistent with results obtained by Letchford (2001) for single plate signs.

Results for the synthesized mean eccentricity coefficient and the synthesized mean extreme eccentricity coefficient for the box sign models were also significantly smaller than comparable results for the single plate sign model. Once again the authors point out that the current design procedure may over-design wind induced torque for box sign models. They also noted that the critical value of the force coefficients and the eccentricity coefficient did not occur at the same horizontal wind direction angle.

Table 2.1 Selected mean drag force coefficient results from Letchford (2001)

Clearance Ratio	Aspect Ratio							
	0.1	0.2	0.25	0.5	1	2	4	5
1.00		1.42	1.41	1.17	1.15	1.14	1.08	1.04
0.95		1.43	1.43	1.33	1.27	1.24	1.14	
0.90	1.55	1.44	1.45	1.41	1.34	1.33	1.20	1.15
0.80		1.46	1.49	1.44	1.43	1.39	1.32	
0.67				1.46	1.42	1.38	1.35	1.32
0.50				1.47	1.38	1.42	1.45	1.44
0.30					1.42	1.45	1.53	1.57
0.16						1.48	1.51	

Table 2.2 Selected coefficient results from Quinn et al. (2001)

Sign	Size (b x c)	Mounting Height	Clearance Ratio	Aspect Ratio	CW(0°)
Rectangle	750 x 1050 mm	1 m	0.69	0.71	1.03
Square	750 mm	1 m	0.55	1	1.05
Square + frame	750 mm	1 m	?	1	1.25
Square	750 mm	2 m	0.32	1	1.08
Rectangle	1500 x 2000 mm	2 m	0.67	0.75	1.22

Table 2.3 Force coefficient results for different aspect and clearance ratios as reported by Giannoulis et al. (2012)

Research Study	Aspect Ratio	Clearance Ratio					
		0.1	0.14	0.27	0.32	0.55	0.66
Letchford (2001)	0.71						1.44
ASCE 7-10 (2010)							(1.70)
Quinn (2001)							0.86
EN 1991 -1 4 (2005)							1.5
Letchford (2001)	0.75						1.43
ASCE 7-10 (2010)							(1.70)
Quinn (2001)							1.05
EN 1991 -1 4 (2005)							1.55
Letchford (2001)	1				1.42	1.38	
ASCE 7-10 (2010)					(1.68)	(1.63)	
Quinn (2001)					1.01	0.92	
EN 1991 -1 4 (2005)					1.69	1.57	
Letchford (2001)	8	1.63	1.63	1.55			
ASCE 7-10 (2010)		(1.85)	(1.85)	(1.80)			
Quinn (2001)		1.52	1.17	0.74			
EN 1991 -1 4 (2005)		1.76	1.74	1.67			

Note: ASCE 7-10 (2010) values in parentheses are Letchford (2001) values multiplied by 1.18 to make them compatible with the design procedure in the code

Table 2.4 Comparison of Ginger et al. (2001) mean net pressure coefficient results and results obtained using Letchford's (2001) equation

c/h	CSTS Study		Letchford (2001)
			Equation
	$\theta = 0^\circ$	$\theta = 45^\circ$	$\theta = 0^\circ$
0.57	1.39	1.12	1.41
0.67	1.39	1.27	1.37
0.80	1.43	1.08	1.30

Table 2.5 Model geometry for Zuo et al. (2014)

Aspect Ratio (b/c)	Depth Ratio (d/c)
0.2	0.145
0.3	0.145
0.5	0.242
1	0.290
2	0.483
4	0.580
5	0.527
7.5	0.725

Table 2.6 Selected results from Zuo et al. (2014)

Coefficient	Aspect Ratio (b/c)			
	1	2	4	5
Mean Force ($c/h = 0.5$)	1.25	1.22	1.21	1.27
Critical Mean Force ($c/h = 0.5$)	1.25	1.23	1.21	1.3
Critical Mean Force ($c/h = 0.25$)	-	1.39	1.33	1.41

Figure 2.1 Distance to vortex shedding (a) Thin shape (b) Elongated shape

Figure 2.2 Vortex suppression (Holmes, 2001) (a) 2D plate (b) 2D plate with splitter plate (c) 2D wall on the ground

Figure 2.3 Sketches of dividing streamlines (Laneville et al. 1975) (a) Thin shape (b) Elongated shape

Figure 2.4 Streamline patterns (Yen et al. 2011) (a) Leading edge separation (b) $\theta = 12^\circ$ separation bubble (c) $\theta = 30^\circ$ attached flow (d) $\theta = 45^\circ$ attached flow

Figure 2.5 Strouhal Number versus depth ratio (Dyrbye and Hansen, 1997)

Figure.2.6 Spectrum of longitudinal turbulence (Letchford, 2001)

Figure 2.7 Mean normal force coefficient for solid panels $c/h < 0.2$ (Letchford, 2001)

Figure 2.8 Picture of Quinn's experimental rig (Quinn et al., 2001)

Figure 2.9 Wind tunnel model schemes (a) Letchford (2001) (b) Paulotto (2006)

3 CODE SPECIFICATIONS FOR DRAG DEPENDENT LOADS ON VMS STRUCTURES

Minimum design requirements for VMS support structures are contained in the AASHTO Specification (2013). The methodology adopted by the current AASHTO Specification (2013) for the design of steel and aluminum VMS support structures is allowable stress design (ASD) and the wind load provisions are based on research and formulations presented in the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-95, 1995). The AASHTO Specification (2013) includes design provisions for extreme wind induced forces (strength design) and low force wind induced vibrations (fatigue design).

3.1 Wind induced forces – strength design

AASHTO design loads for VMS structures include dead loads, live loads, and ice loads but the primary load that governs most designs is the natural wind load. The wind load that is calculated and applied horizontally to the projected frontal area of all surfaces of the structure including vertical supports, luminaires, signs, catwalks, and attachments. The strength design wind pressure is calculated using AASHTO Specification (2013) Equation 3.8.3-1:

$$P_z = 0.613K_zGV^2I_rC_d \quad (\text{Pa}) \quad (3-1)$$

where P_z is the wind pressure at height z , K_z is the height and exposure factor, G is the gust effect factor, V is the basic wind speed in m/s, I_r is the wind importance factor, and C_d is the non-dimensional drag coefficient.

Wind speeds vary with height depending on the ground surface roughness. This variation is caused by the frictional drag associated with different terrain exposures. ASCE 7-10 (2010) defines three exposure categories B, C and D (Exposure A was dropped in the 2005 edition). The calculated design wind pressure at a particular height increases as the exposure category progresses from Exposure B to Exposure D. Since sign structures are typically installed along roadways that can generally be characterized as open corridors, the AASHTO Specification (2013) has conservatively adopted Exposure C as the standard. This represents open terrain with scattered obstructions (Fouad and Calvert, 2001).

The AASHTO Specification (2013) suggests a single value for the gust effect coefficient for both wind sensitive and non-wind sensitive structures. However, it is noted that virtually all structures addressed by the AASHTO Specification should be classified as wind sensitive. According to the AASHTO Specification (2013) the gust effect factor (G), “adjusts the effective velocity pressure for the dynamic interaction of the structure with the gustiness of the wind.” The current suggested minimum value of $G = 1.14$ is based on the historical gust coefficient ($G_{\text{historical}} = 1.3$) that was incorporated into previous versions of the AASHTO Specification since 1961. The gust effect factor (G) referred to in the AASHTO Specification (2013) should not be confused with the gust effect factor (GEF or G_f) used in the ASCE 7 Specification. The historical gust coefficient ($G_{\text{historical}} = 1.3$) used in previous editions of the AASHTO Specification is the ratio of peak gust to the design mean wind speed. It does not take into account the structures vibrational characteristics or dynamic response (Fouad et al., 2002). The current gust effect factor ($G = 1.14$) is essentially the same historic gust coefficient

adjusted for 3-second gust wind speeds. The ratio of fastest mile wind speed to 3-second gust wind speed is approximately equal to 0.82. Therefore, the historical gust coefficient of 1.3 is converted to a 3-second gust coefficient by

$$(1.3) \times (0.82) = 1.07 \quad (3-2)$$

The gust coefficient in previous AASHTO Specifications multiplied the wind speed but the current gust effect factor multiplies the pressure. Therefore the gust effect factor is calculated as

$$(1.07)^2 = 1.14 \quad (3-3)$$

Commentary in the AASHTO Specification (2013) notes that the “use of the gust effect factor of 1.14 with an assumed 3-s gust speed results in essentially the same wind pressure as was determined by the 1994 edition of [the] Specifications using the 1.3 gust coefficient and the equivalent fastest-mile wind speed.” The use of the gust coefficient concept is continued in the current AASHTO Specification since, according to the commentary, it has performed well in the past (AASHTO Specification, 2013).

The basic wind speed used in the AASHTO Specification wind pressure equation (Equation 3-1) is the maximum extreme event 3-second gust wind speed that occurs over a 50 year period. This corresponds to a 2% annual probability that the wind speed will be met or exceeded. The values provided in the AASHTO Specification (2013) basic wind speed maps are for 10 m above ground and Exposure C (open terrain).

An importance factor (I_r) is used to convert the calculated wind pressure associated with a 50 year mean recurrence interval to wind pressures associated with other mean recurrence intervals like 10, 20, and 100 year. The I_r for VMS support structures is equal to 1.0 based on a typical specified minimum design life of 50 years.

The applicable drag coefficient for each wind loaded member is given in AASHTO Specification (2013) Table 3.8.6-1. The drag coefficient provided for flat panel signs is based on aspect ratio only and no consideration is given to the mounting height. Table 3.1 lists the AASHTO Specification (2013) suggested drag coefficients for flat panel signs.

Table 3.1 AASHTO drag coefficients for flat panel signs

Length/Width	C_D
1.0	1.12
2.0	1.19
5.0	1.20
10.0	1.23
15.0	1.30

The flat panel drag coefficients in the Specification have remained unchanged since at least the 1985 Specification. AASHTO Specification (2013) Table 3.8.6-1 suggests a drag coefficient of 1.7 for use in the design of VMS structures. However, a footnote to the Table states “A value of 1.7 is suggested for Variable Message Signs (VMS) until research efforts can provide precise drag coefficients” (AASHTO Specification, 2013). No information or reference is provided about how this value was determined.

Commentary in the AASHTO Specification (2013) states that the validity of the drag coefficients included in the standard was evaluated in the study conducted by McDonald et al. (1995). Based on the results of that study and independent examinations of the coefficient provided in the AASHTO Specification Table 3.8.6-1, the AASHTO Specification (2010) drag coefficients were only changed to account for metric units and 3-second gust speeds (AASHTO Specification, 2010). This did not result in any changes to the drag coefficients for flat panels or VMS. These values remain unchanged in the current AASHTO Specification (2013).

3.2 Wind induced vibration – fatigue design

Fatigue is the accumulation of damage caused by repeated high cycle stress fluctuations due to oscillations in the wind (Ocel et al., 2006). The AASHTO Specification (2013) identifies three wind loading mechanisms that have the potential to create large amplitude vibrations and excessive movement which can lead to fatigue damage in VMS support structures. They are 1) galloping; 2) natural wind gusts; and 3) truck induced wind gusts (vortex shedding was included in previous versions of the Specification but omitted for sign structures in the current edition). The structures' response and resulting stress ranges to these wind loading mechanisms is dependent on the flexibility and damping of the support structure. Cantilever support structures are highly flexible and have a low natural frequency (approximately 1 Hz). This coupled with an extremely low critical damping ratio ($< 1\%$ of critical damping) makes them particularly susceptible to resonant vibration of the wind. However, bridge type supports are less susceptible due to the torsional rigidity of the horizontal truss system (Dexter and Ricker, 2002). The AASHTO Specification (2013) fatigue design is based on the infinite life approach and the fatigue stress limits are based on constant-amplitude fatigue limits (CAFL). VMS support structures are designed to ensure that stresses due to calculated equivalent static design pressures are within the CAFL identified in the code. Fatigue failure may occur if the stress resulting from wind induced vibration exceeds the fatigue thresholds identified for each critical detail. According to Fischer (1993), fatigue cracking can occur if as few as 1 cycle in a 1000 exceed the CAFL. Infinite life is

typically considered to be longer than the service life of the structure which for a VMS support structure is typically 50 years (AASHTO Specification, 2013).

Table 1.2 indicates the susceptibility and subsequent design requirement for each fatigue wind loading mechanisms based on structure type. The application of complex dynamic wind loads due to the fatigue wind loading mechanisms shown in Table 1.1 are simplified in the AASHTO Specification (2013) design process through the use of equivalent static loads that reasonably represent the dynamic response of the structure.

The AASHTO Specification (2013) fatigue design equivalent static shear pressure range for galloping is

$$P_g = 1000I_F \quad (\text{Pa}) \quad (3-4)$$

where I_F is the importance factor that adjusts the level of structural reliability and accounts for the risk of hazard in the event of failure. For example, a structure located on a high volume roadway will have a higher importance factor since the probability of damage is greater if the structure fails. Importance factors for sign support structures are given in AASHTO Specification (2013) Table 11.6-1. Cantilevered VMS support structures are designed for galloping induced fatigue by applying the calculated pressure vertically to the surface area of attachments, signs and VMS. According to Dexter and Ricker (2002), cantilever support structures with four-chord trusses do not appear to be susceptible to galloping and are not required by the AASHTO Specification (2013) to be designed for galloping induced fatigue. The AASHTO Specification (2013) does not require non-cantilever support structures to be designed for galloping. However, Fouad et

al. (2003) recommends that non-cantilevered monotubes should be designed for galloping.

The primary fatigue design load for cantilever and non-cantilever VMS structures is natural wind. Natural wind gusts are caused by variability in the direction and velocity of the wind. These changes in direction and velocity cause fluctuating pressures which may lead to vibration of the structure and fatigue damage. The AASHTO Specification (2013) equivalent static natural wind gust pressure range used in the fatigue design of VMS structures is

$$P_{NW} = 250C_dI_F \quad (\text{Pa}) \quad (3-5)$$

The calculated pressure range is applied in the horizontal direction on the projected area of all members of the support structure and the VMS. The highest potential for fatigue related problems on VMS support structures are locations with constant winds. These locations are typically not the same locations that have the highest design wind speeds for strength. Coastal locations usually have the highest strength design wind speeds because the maximum wind speeds are usually associated with gales or extreme events like hurricanes (Dexter and Ricker, 2002). The AASHTO Specification (2013) fatigue pressure range for natural wind gusts is based on a 0.01 percent chance of exceeding a yearly mean wind speed of 5 m/s. This was supported by data from 59 weather stations that showed 87% of U.S. cities had yearly mean wind speed less than 5 m/s. As indicated previously, strength design wind speeds occur on average for 3 seconds every 50 years. However, the fatigue wind speeds are expected to occur for an hour or two each year.

The AASHTO Specification (2013) also requires cantilevered and non-cantilevered VMS support structures to be designed for an equivalent truck induced pressure range of

$$P_{TG} = 900C_dI_F \quad (\text{Pa}) \quad (3-6)$$

The pressure is applied vertically to any 3.7 m length that creates the maximum stress and is based on a truck speed of 30 m/s. The effects of truck induced gusts are beyond the scope of this research project and not discussed further.

The drag coefficients provided in AASHTO Specification (2013) Table 3.8.6-1 are used in both the fatigue design equations (Equations 3-4 to 3-6) and the strength design equation (Equation 3-1). Therefore, the current AASHTO Specification (2013) recommended fatigue drag coefficient used in the design of VMS support structures is 1.7.

Analytical and experimental research was conducted by Dexter and Ricker (2002) to specifically address VMS fatigue issues and improve provisions in the AASHTO Specification. It was initially thought that fatigue problems caused by retrofitting structures with a VMS were results of overloading the structure due to the additional weight of the VMS. Although the additional weight will tend to decrease the natural frequency of the structure, Dexter and Ricker (2002) noted that this effect is typically countered by an increase in the stiffness to control the dead load deflection. Instead these problems are likely caused by an increase in the sign area that results in an increase in stress from wind and trucks gusts resulting in vibration and fatigue related problems.

Field testing and modeling of a bent monotube cantilevered VMS support structure was performed for the California Department of Transportation (Caltrans) and reported by Dexter and Ricker (2002). Field measurements, free vibration tests, and FEM analysis of the structure found that the natural frequency was in the range of 1.04 – 1.10 Hz and the percent of critical damping ratios were in the range of (0.4 – 0.7%). Galloping was

observed during the field monitoring period and the measured stress ranges during the monitoring period indicated that the static equivalent loads were as high as 2000 Pa, which is twice the recommended value suggested in the AASHTO Specification (2013). However, it was concluded that the equivalent static pressure range of 1000 Pa recommended in AASHTO Specification (2001) (which remained the same in the AASHTO Specification (2013)) for galloping was sufficiently conservative since this location in California was thought to have the worst steady-state wind conditions in the country. In addition it was thought that bent monopole cantilever support structures may be particularly susceptible to galloping (Dexter and Ricker, 2002).

Dexter and Ricker (2002) reported the dynamic analysis results of four different cantilever VMS structures. Finite element models were utilized to determine the first six mode shapes and corresponding modal frequencies. The four cantilever structures modeled were the: (1) New Jersey 2-chord truss, (2) California 4-chord truss, (3) Illinois Vierendeel truss, and (4) Illinois trussed flat panel structure. A description of the structures and a summary of the results are provided in Table 1.1. The natural frequencies provided in the Table are an average of the experimental results and several FEM modal analyses. Only the natural frequencies for the first mode (out-of-plane horizontal truss movement) and the second mode (in-plane vertical truss movement) are provided since, according to Dexter and Ricker (2002), fatigue problems in cantilever support structures most likely occur as a result of vibration of these modes. A review of field data by Dexter and Ricker (2002) indicated that most cantilever sign support structures tested have natural frequencies in the range of 0.5Hz to 2 Hz. This is consistent with the results of the FEM analysis. The report explains that this is probably due to the design for dead load

deflections since the natural frequency is proportional to the square root of the stiffness divided by the mass. An incremental increase in mass is accompanied by an approximately proportional increase in stiffness which maintains an almost constant ratio with the natural frequency (Dexter and Ricker, 2002).

A spectral analysis was also performed by Dexter and Ricker (2002) using the finite element models of the New Jersey 2-chord truss and the California 4-chord truss to determine the applicability of the AASHTO Specification (2001) equivalent static loads for natural wind gusts for VMS structures. The Davenport wind velocity spectrum was used to model the dynamic response of the structure. Equivalent static pressure ranges were back calculated from the dynamic analysis. For a yearly mean wind speed of 5 m/s, the analysis results for equivalent static pressures were in good agreement with the natural wind gust pressures provided in the AASHTO Specification (2001) which have not changed in the current AASHTO Specification (2013).

In summary, the AASHTO procedure for strength and fatigue design of VMS structures is dependent on the drag coefficient. In addition, the strength design formula for wind loads and the fatigue equations for wind and truck induced gusts all use the same value of the drag coefficient. It is unclear how the current AASHTO suggested drag coefficient value of 1.7 was determined. However, it is acknowledged that this value is conservative (AASHTO Specification, 2013). This study will determine reliable drag coefficients for use in design by testing a comprehensive range of large scale VMS models in the WOW.

The current gust effect factor ($G = 1.14$) used in the AASHTO Specification (2013) has produced adequate designs in the past but does not accurately take into account the

structure's vibrational characteristics or dynamic response. This research will assess the degree to which the 1.14 gust factor covers the real expected range of gust effects.

Although a few studies (Ginal, 2001; Dexter and Ricker, 2002) have looked into the susceptibility of VMS structures to galloping and vortex shedding, it still is not clear whether these structure are vulnerable to excessive vibration and subsequently fatigue damage. This research will be the first to evaluate the current AASHTO design assumptions for galloping by testing VMS models in the wind tunnel. Aerodynamic data will be obtained for multiple vertical wind approach angles to determine the critical onset wind velocity for galloping.

4 EXPERIMENTAL DESIGN, PROCEDURE, AND INSTRUMENTATION

4.1 Industry specifications and model size selection

The experimental design was initiated by contacting major manufacturers of VMS signs. Three manufacturers Daktronic, Ledstar, and Skyline provided design specifications and typical sign dimensions. The aspect ratio (b/c) and depth ratio (d/c) for each sign configuration was calculated and plotted in Figure 4.1. Prototype aspect ratios ranged from 0.62 to 9.81 and depth ratios range from 0.11 to 0.91. Three aspect ratios ($b/c = 1, 3$ and 5) and three depth ratios ($d/c = 0.1, 0.4$ and 0.7) were selected to encompass a wide range of typical VMS sizes and shapes. The ranges of selected ratios covered in this study are delineated by the square in Figure 4.1. The selected geometries represent approximately 89% of the aspect ratios and 98% of the depth ratios provided. A fixed clearance ratio of $c/h = 0.24$ was used for all tests. This value is based on a full scale sign height of 1.8 m and a minimum vertical clearance of 6 m which corresponds to a full scale minimum mounting height of 6.9 m. A length scale of 1:3 was utilized to minimize blockage effects and ensure that the largest model was completely immersed in the FIU WOW wind field.

4.2 Model design

A total of 13 models were constructed for testing. Models 1 - 9 have sharp edges and vary in length from 0.6 m to 3 m with depths varying from 61 mm to 427 mm. Model 10 was constructed with 32 mm round edges and Model 11 was constructed with 32 mm chamfered edges. Models 12 and 13 are flat panels consisting of a single sheet of

plywood with a depth dimension of approximately 13 mm. Model 12 is 3.0 x 0.6 m and Model 13 is 1.5 m x 0.3 m. Scaled dimensions of the models and applicable geometric ratios are included in Table 4.1.

The models were fabricated from 13 mm shop grade maple plywood and reinforced with 51 x 102 mm southern pine. Models 4-11 were fitted with removable top panels to facilitate connection to the support structure and Models 1-3 and 12-13 were through bolted to the support structure with threaded 13 mm countersunk bolts. All wood to wood connections were joined with 25 mm screws and structural wood glue. Model dimensions, sections, and details are shown in Figures 4.2, 4.3, 4.4 and 4.5.

The model support structure consisted of either a single or double rigid steel cantilever system. Figures 4.6 and 4.7 shows elevation drawings for both support systems and a side elevation drawing showing the major components and dimensions of the support system is illustrated in Figure 4.8. The single support system was used for the $b = 0.6$ m models (Models 1, 4, and 7) and the double support system was used for the $b = 1.8$ m and $b = 3.0$ m models (Models 2-3, 5-6, and 8-13). The horizontal cantilever arm for both the single and double support systems was 762 mm long and fabricated from 152 x 152 x 6 mm steel tubes. A vertical 305 x 305 x 6 mm steel plate was welded to the front end of the cantilever arm to provide a mounting point for the models. A second horizontal 305 x 305 x 6 mm steel plate was welded to the rear end of the cantilever arm which was then attached to the top plate of the load cell with four 13 mm lag bolts. The vertical supports were also fabricated from 152 x 152 x 6 mm steel tubes. A 305 x 305 x 6 mm steel plate was welded to the top of each vertical support which was then connected to the bottom plate of the load cell with four lag bolts. The vertical posts of the double support

system were spaced 1.2 m apart. Each vertical support was bolted to the base with four 13 mm bolts and then rigidly connected to the WOW floor with 19 mm bolts. A total of three complete support systems were required and assembled for testing.

All models were mounted at mid height 2.3 m above the WOW test facility floor and attached to the cantilever arm steel connection plate with four 13 mm bolts centered horizontally and vertically on the model. The bolt holes (14 mm) in the back face of the models were drilled slightly larger than the 13 mm bolt. This was done to accommodate any minor misalignments in the support structure that might induce initial stresses in the setup. Additional stability was provided by three wire cables attached to eye bolts at the top of each vertical support (below the load cells) and fixed to the WOW floor. The model mounting details are illustrated in Figure 4.9.

4.3 Instrumentation

A multi-axis load cell supplied by JR3, Inc. (model 75E20S4 – M125D – AF 1350L) was mounted on top of each vertical support to simultaneously measure forces, moments and torques. A picture of the load cell is shown in Figure 4.10. The load cells are equipped with six degree of freedom force torque sensors which simultaneously measure forces along three orthogonal axes (x, y, and z) and moments and torques about those axes. The load cells are oriented with the x- and y- axes at the horizontal mid-plane of the load cell and the z axis along the central vertical axis. The load cells used in this study were oriented with respect to the VMS model axes and positioned with the positive x axis in the along wind direction, the positive y axis to the right, and the positive z axis up. Refer to Figure 1.5 for axis and force orientation. Measured forces and moments were

multiplied by the manufacturer's calibration matrix to obtain actual forces and moments (the calibration matrices are included in Appendix A). A total of three load cells were assembled and wired in accordance with the manufacturer's specifications.

Cobra probes supplied by Turbulent Flow Instruments (Figure 4.11) were used to measure the wind velocity and turbulence in three orthogonal directions. Probes were mounted vertically 0.9 m above the model at an elevation of 3.5 m above the WOW floor. This distance was considered sufficient to ensure that the cobra probe wind speed readings were not affected by the flow around the model (this is discussed in more detail in Section 4.6). Each probe was mounted on a vertical uni-strut support suspended from the fixed gantry. Three probes spaced at 1.2 m on center were utilized with the middle probe centered horizontally on the model. Figure 4.12 shows the configuration of the cobra probes with a model in place. In addition, wind speed measurements were also conducted for the free flow case where the speeds were measured without the model in place at the mid-height of the model (refer to Section 4.5.4).

4.4 Wall of Wind (WOW)

Experiments were conducted in the Wall of Wind (WOW) facility at Florida International University (FIU). The test section of this open jet system is 6.1 wide x 4.3 m high and wind is generated by two rows of six electric fans arranged in a convex arc (Figure 4.13). Each fan has a maximum flow rate of 113.3 cubic meters/second and the rotational speed is controlled by variable frequency drives (VFD). The wind flow from each fan merges into a contraction zone which is designed to generate a high velocity uniform flow field. Vertical vanes at the contraction exit guide the flow in the

longitudinal direction. A 9.8 m flow simulation box consisting of triangular spires and floor roughness elements help develop the desired atmospheric boundary layer (ABL) profile.

4.4.1 Velocity profile

Figure 4.14 shows a graph of the target suburban terrain mean wind speed profile (power law exponent $\alpha = 0.25$) and the non-dimensional profile obtained at the WOW. As indicated, the simulated wind speed profile generated in the WOW for these tests was close to the target suburban terrain profile. Measurements were conducted for several throttle percentages which represent the percentage of fan motor speed and correspond to a test wind speed. A summary of test throttle percentages and corresponding measured mean wind speeds for the probes at a height of 2.3 m and 3.5 m (without a model in place) is given in Table 4.2. The measured mean wind velocities used in this study at the mounting height of the models (2.3 m) were approximately 15 m/s (25% throttle) and 40 m/s (65% throttle).

4.4.2 Partial turbulence simulation

Figure 4.15 is a plot of the turbulence intensity generated in the WOW. The turbulence intensity of the WOW laboratory flow is lower than the full-scale target suburban terrain ABL. This is due to the partial turbulence simulation method that is utilized in the WOW which reproduces only the high frequency portion of the ABL spectrum (see Yeo and Gan Chowdhury, 2013 and Fu, 2013). The full spectrum for suburban terrain simulated in the wind tunnel by Fu (2013), the WOW partial spectrum,

and the dimensionalized Kaimal spectrum are shown in Figure 4.16. As indicated, the high frequency portion of the WOW partial spectrum matches well with the wind tunnel full spectra and is in good agreement with the inertial subrange of the dimensionalized Kaimal spectrum. The partial simulation flow in the WOW and the full-scale ABL flow have similar high frequency spectral content but the partial spectrum flow in the WOW is characterized by significantly weaker low frequency fluctuations. The missing low frequency content of the WOW spectrum represents the large scale turbulence associated with slowly moving gusts. This large scale turbulence does not significantly affect local vortex formation, flow separation, or flow reattachment but does augment the sustained wind speed (Fu et al., 2012). However, the proper simulation of small-scale or high frequency turbulence is critical in the investigation of aerodynamic parameters and flow separation from sharp edged objects like VMS. It was shown by Fu et al. (2012) that two flows with approximately similar high-frequency spectral content will produce comparable aerodynamic effects even though one flow is characterized by weak low frequency fluctuations as long as the mean wind speed of the flow with weak low frequency fluctuations is increased to compensate for the missing low frequency fluctuations. The incremental increase in the mean wind speed may be viewed as a flow fluctuation with zero frequency and perfect spatial coherence (Fu et al, 2012). More information about the partial turbulence simulation used in the WOW and the validation of aerodynamic results can be found in Fu et al. (2012), Yeo and Gan Chowdhury (2013), and Fu (2013).

4.5 Tare tests

Tare tests were conducted to isolate and correct for the secondary aerodynamic forces applied to the support system. Two configurations, Setup A and B were utilized and shown schematically in Figures 4.17 and 4.18 respectively. Setup A was used to determine the tare correction for the single support configuration. This setup consisted of independently supporting Model 7 25 mm in front of Vertical Support 1. The model was held in place by mounting Support 2 to the front face of the model. Wire cables were attached to each vertical support to provide additional stability. Load cells were mounted and wired on both Support 1 and 2 but only data from Load Cell 1 was utilized. The positive x-axis for Load Cell 1 was oriented to correspond with the longitudinal along-wind direction. Setup B as illustrated schematically in Figure 4.18, was used to determine the tare correction for the double support configuration. This setup consisted of detaching Model 8 from the horizontal cantilever arm and independently supporting it 25mm in front of Vertical Support 1. The position of Support 1 corresponded to the same location of the support during the force coefficient testing and the leading windward support for the 45° tests. The model was held in place by rotating Supports 2 and 3 ninety degrees and attaching them to each end of the model. This preserved the flow pattern around the model and isolated the aerodynamic forces applied to the support structure. Wire cables were attached to each vertical support to provide additional stability. Load cells were mounted on all three supports but only data from Load Cell 1 was utilized. The positive x-axis for Load Cell 1 was oriented to correspond with the longitudinal along-wind direction. Figure 4.19a shows a picture of tare test Setup A configured for the 0° horizontal wind approach direction and Figure 4.19b is a picture of tare test Setup A

configured for the 45° horizontal wind approach direction. Similarly, Figure 4.20a shows a picture of tare test Setup B configured for the 0° horizontal wind approach and Figure 4.20b is a picture of tare test Setup B configured for the 45° horizontal wind approach.

Tare test data was sampled at 100 Hz for 1 minute. Both setups were tested for horizontal wind approach directions of 0° and 45°. Setups A and B were rotated clockwise for the 45° horizontal wind approach test. For Setup B this ensured that data was measured on the windward load cell (Load Cell 1). Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. Tare tests were conducted for wind speeds of 12 m/s, 15 m/s, 30 m/s, and 40 m/s.

The measured tare correction for Setup A was applied to Models 1, 4, and 7 and the tare correction obtained using Setup B was applied to Models 2-3, 5-6, and 8-13. A summary of tare test results for the single support (Setup A) configuration is included in Table 4.3 and a summary of results for the double support (Setup B) configuration is included in Table 4.4.

4.6 Wind field adjustments

The presence of the model has two effects on the wind field: (1) local flow field distortion and (2) blockage. Local flow field distortion is caused by accelerations and decelerations in the flow field very close to the model as the air negotiates its way around the VMS. These effects attenuate as you move upwards or downwards away from the model so ideally speed measurements should be taken far enough away to be out of this zone. The second effect due to the presence of the model is blockage of the whole flow

field. This is the difference between the model being in an infinitely large wind field (as in real wind) and being in a 6.1 x 4.3 m jet near the mouth of the flow simulation box. The general flow at the model could either experience an overall acceleration or deceleration depending on whether the constraint of the simulation box dominated over the lack of constraint of the jet or vice versa. A blockage test was performed to get an overall measure of whether the wind speed measurements were sampled sufficiently far enough away and to correct for local flow effects.

4.6.1 Blockage correction test

The blockage test consisted of testing Model 12 with dimensions of 3.0 x 0.6 m at a wind speed of 19 m/s and Model 13 with dimensions of 1.5 x 0.3 m (1/4 blockage) at twice the wind speed or 38 m/s. Both models were flat panels ($d = 13$ mm) with $b/c = 5$. Pictures of the blockage test setup for Models 12 and 13 in the WOW are shown in Figure 4.21.

The blockage test was conducted for 0° and 45° horizontal wind approach directions in the simulated suburban boundary layer flow. Data for each test run was sampled at 100 Hz for 1 minute. Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. The double cantilever support (refer to Figure 4.6) was used for the blockage test and the applicable tare correction was applied to the measured blockage test data for the 0° horizontal wind approach. Tare corrections were derived from testing large prismatic models (Models 7 and 8) which were not representative of the models used in the

blockage tests. Therefore, no tare correction was applied to the measured blockage test data for the 45° horizontal wind approach direction.

The measured wind speed at the reference height of 3.5 m was intended to be far enough away from the model to avoid the local flows. However the test revealed that the measurements may not have been completely free of local effects. Tests using Model 12 (1:3 scale) showed that the model was probably still in a slightly accelerated local flow. This caused the drag coefficient results to be slightly low. For Model 13 at half the scale (1:6), the flows were less affected which resulted in larger drag coefficients. To account for this effect, the zero blockage correction was extrapolated from the plot of the normal force coefficient (C_{Fx}) versus the ratio of the model area to the wind field area (A/A_s) as shown in Figure 4.22 for the 0° horizontal wind direction. Where A is the area of the front face of the model ($b \times c$) and A_s is the area of the WOW wind field equal to 26 m². As shown in the graph, a negative blockage effect was observed (the net effect was to reduce the drag compared to the zero blockage case). The zero blockage drag coefficient extrapolated from the graph for Model 12 (0° horizontal wind direction) was 1.04. A summary of the 0° blockage correction percentage used for each model is shown in Table 4.5. The graph for the 45° horizontal wind direction is shown in Figure 4.23. The zero blockage drag coefficient extrapolated from the graph for Model 12 for the 45° direction was 0.86. The percent blockage correction applied to each model was then calculated as a proportion of A/A_s . A summary of the 45° blockage correction percentage used for each model is shown in Table 4.6.

4.6.2 Wind speed spatial relationship

The mean wind speed ($\bar{U}$) used in the calculation of the drag coefficient was determined by averaging the mean temporal wind speeds measured at different locations during model testing. Wind speed measurements were recorded during each test run using three cobra probes mounted at the reference height of 3.5 m which is 0.9 m above the model. The probes were spaced horizontally 1.2 m apart with the middle probe centered with respect to the model (see Figure 4.10). The mean temporal wind speed for the one minute test length was determined for each probe. The measured mean wind speeds at 3.5 m (with the model in place) are provided in Table 4.7. Analysis of the mean temporal wind speeds for each probe showed that the wind profile along the horizontal length of the model was not quite uniform. The measured wind speed at the center probe was approximately 4% higher than the wind speeds measured at each of the two outer probes for the 0° horizontal wind direction and 5% higher for the 45° wind direction. The percent difference between the mean measured wind speed at 3.5 m and the mean adjusted spatial wind speed for each model test is shown in Table 4.8. Consequently, a linear interpolation was applied to calculate the wind speed at a horizontal location corresponding to the outer edge of each model. These three spatial wind speeds (left edge, center, and right edge) were averaged to determine the mean spatial wind speed at the reference height of 3.5 m. This is the wind speed used in the force coefficient calculation. A summary of the calculated spatial wind speeds at the reference height of 3.5 m for each model tested is provided in Table 4.9.

4.6.3 Free flow wind

Free flow wind speed measurements were conducted to determine the relationship between the wind speeds at the reference height of 3.5 m and the desired wind speeds at the model mounting height of 2.3 m.

The Free Flow Wind Testing used an array of cobra probes mounted on a grid of vertical and horizontal uni-strut supports. Three vertical uni-struts at 1.2 m on center were attached to the fixed gantry at the top and bolted to the WOW floor at the bottom. The gantry was bolted to tracks on the WOW floor at the desired location. Wire cables attached at the top of the gantry and bolted to the WOW floor provided supplemental support and stability to the gantry. Additional uni-strut bracing was provided at the 2/3 point of the uni-strut vertical span. An array of six cobra probes were mounted in two horizontal rows. The top row of three probes was mounted at 3.5 m and spaced horizontally 1.2 m on center with the middle probe centered on the model (however no model was in place). The bottom row of three probes was mounted at 2.3 m. and also spaced horizontal at 1.2 m on center. Looking upstream, the probes on the top row were numbered from right to left 1-3 and the bottom row from right to left were numbered 4-6. Figure 4.24 shows the free flow wind test setup in the WOW and the configuration of the cobra probes. Testing was conducted in the simulated suburban boundary layer flow (power law exponent $\alpha = 0.25$) and data was sampled at 100 Hz for 1 minute. Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. The Free Flow Wind Test was only conducted for the 0° horizontal wind direction. A summary of the measured free flow wind speeds is provided in Table 4.2.

The ratio of the measured free flow wind speed at 3.5 m to the mean wind speed at 2.3 m was established for each set of corresponding probes. The mean vertical height adjustment (*VHA*) ratio for the 15 m/s and 40 m/s tests was calculated using the mean ratio of Probe 1: Probe 4, Probe 2: Probe 5, and Probe 3: Probe 6. The calculated *VHA* ratios are tabulated in Table 4.10. The measured wind speed used in the drag coefficient calculation was adjusted to the mounting height wind speed by multiplying the calculated drag coefficient at 3.5 m by the mean *VHA* squared for each tested wind speed as follows

$$C_{Fx_{15}} \times (VHA_{15})^2 \quad (4-1)$$

$$C_{Fx_{40}} \times (VHA_{40})^2 \quad (4-2)$$

where $C_{Fx_{15}}$ and $C_{Fx_{40}}$ are the drag coefficients calculated at 3.5 m for the 15 and 40 m/s wind speeds respectively. VHA_{15} and VHA_{40} are the mean vertical height adjustment ratios for the 15 and 40 m/s test wind speeds. The mean *VHA* ratio for the 15 m/s wind speed was determined to be $VHA_{15} = 1.10$ and the mean *VHA* ratio for the 40 m/s wind speed was also determined to be $VHA_{40} = 1.10$.

4.7 Test setups and procedures

4.7.1 Force coefficient tests

Models 1-9 were utilized in the force coefficient testing. Testing consisted of 18 model configurations and 36 individual aerodynamic tests. The test assembly began with bolting the steel base to the WOW floor then bolting either the single or double cantilever support system to the base depending on the model size. The single cantilever support system was utilized for models 1, 4 and 7 and the double cantilever support system was

utilized for models 2-3, 5-6, and 8-9. The single or double cantilever support system and the base remained in place while the models were interchanged for each test. Baseline data indicated that erection of the model and bolt tightening induced minor stresses in the system. Loosening and then retightening the load cell bolts after mounting the model reduced this effect. Three wire cables bolted to the WOW floor were attached to the top of each vertical support (below the load cell) to provide additional stability and reduce vibration in the system. The wire cables were tightened and adjusted after each model was erected. All models were mounted to the cantilever support at mid height which was 2.3 m measured from the WOW floor. This provided 2 m vertical clearance, except for Model 13 which had a vertical clearance of 2.1 m ($c/h = 0.13$). Based on the model height of 0.6 m, this corresponded to a clearance ratio of $c/h = 0.24$. Typical model setups in the WOW for the 0° and 45° tests are shown in Figures 4.25 and 4.26. As shown in the figures, three cobra probes were mounted at the reference height of 3.5 m which provided 0.9 m of clearance above the top of the model. Each probe was mounted on a uni-strut suspended vertically from the fixed gantry. The probes were spaced horizontally 1.2 m on center with the middle probe centered on the model. A horizontal cable attached to the fixed gantry and the bottom of the uni-struts helped to reduce vibration of the cobra probes.

All models were tested for 0° and 45° horizontal wind directions. Zero degree was defined as the direction of the approaching wind normal to the front face of the model. The base was rotated clockwise about the vertical axis of the model to obtain the 45° approach direction. Wind tunnel tests for each model were conducted at fan motor speed percentages of 25% and 65% which corresponds to test wind speeds of 15 m/s and 40

m/s. The ambient temperature and air pressure readings were monitored and updated before each test. Data was sampled at 100 Hz for 1 minute for each test run. Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. The appropriate tare correction (Table 4.3 and 4.4), blockage correction (Table 4.5 and 4.6), and wind speed adjustments (Section 4.5.3 and 4.5.4) were applied to all test data.

4.7.2 Corner modification tests

This series of tests investigated the effects of modifying the model edges with round (Model 10) and chamfered (Model 11) corners. It was hypothesized that modified corners would reduce the drag coefficient by delaying the separation of flow around the VMS. Model 10 and 11 have the same overall dimensions as Model 9 (3.0 m. x 0.6 m. x 427 mm) but were constructed with 32 mm (5% of the vertical height) round and chamfered corners. Figures 4.27 and 4.28 shows the model setups in the WOW and Figure 4.29 shows close-up photos of the modified corners.

Testing was performed using the same parameters and conditions as the sharp edge tests. Corner modification tests were conducted for 0° and 45° horizontal wind approach directions in the simulated suburban boundary layer flow. Tests were conducted for 15 m/s and 40 m/s wind speeds. Data for each test run was sampled at 100 Hz for 1 minute. Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. The appropriate tare correction (Table 4.3 and 4.4), blockage correction (Table 4.5 and 4.6) and wind speed adjustments (Section 4.5.3 and 4.5.4) were applied to all test data.

4.7.3 Galloping tests

Tests were conducted to investigate the galloping potential of VMS. Model 9 ($b/c = 5$, $d/c = 0.7$) was attached to the double support configuration with a series of 2.5° and 4.5° wedges to vary the angle of attack of the approaching wind. The wedges were positioned to simulate both negative and positive angles of attack. The tests were conducted for wind attack angles of $\alpha = -4.5, -2.5, 0, 2.5$, and 4.5 at wind speeds of 15 and 40 m/s. Figure 4.30 shows a schematic of the galloping test setup and Figure 4.31 shows pictures of the test setup in the WOW.

The tests conditions were identical to the test conditions for the force coefficient tests for Models 1-9. Tests were conducted in the simulated suburban boundary layer for only the 0° horizontal wind approach direction. Data was sampled at 100 Hz for one minute. Baseline data sampled one minute before and one minute after each test run was subtracted from the measured test data to account for zero offset effects. The appropriate tare correction (Table 4.3 and 4.4), blockage correction (Table 4.5), and wind speed adjustments (Section 4.5.3 and 4.5.4) were applied to all test data.

4.7.4 Wind driven rain test

Model 8 (3.0 m x 0.6 m x 427 mm) was subjected to a simulated high velocity wind driven rain environment by utilizing spray nozzles mounted on the exhaust side of the WOW fans. Figure 4.32 shows the configuration of four vertical lines of equally spaced spray nozzles utilized for the test setup. Since cobra probes could not be utilized during the wind driven rain test, the wind speed was assumed to be equal to the average wind speeds for the Model 8 force coefficient test (wind only). Testing was performed using

the same parameters and conditions as the force coefficient tests for Model 8. This test was only conducted for the 45° horizontal wind direction. Tests were conducted for 15 m/s mph and 40 m/s wind speeds in the simulated suburban boundary layer flow. Data for each test run was sampled at 100 Hz for 1 minute. To account for zero offset effects, baseline data was subtracted from the measured test data. The appropriate tare correction (Table 4.3 and 4.4), blockage correction (Table 4.6), and wind speed adjustments (Section 4.5.3 and 4.5.4) were applied to all test data.

Table 4.1 Model dimensions and geometric ratios

Model No.	Width b (m)	Height c (m)	Depth d (m)	Aspect Ratio b/c	Depth Ratio d/c
1	0.6	0.6	0.06	1	0.1
2	1.8	0.6	0.06	3	0.1
3	3.0	0.6	0.06	5	0.1
4	0.6	0.6	0.2	1	0.4
5	1.8	0.6	0.2	3	0.4
6	3.0	0.6	0.2	5	0.4
7	0.6	0.6	0.4	1	0.7
8	1.8	0.6	0.4	3	0.7
9	3.0	0.6	0.4	5	0.7
10	3.0	0.6	0.4	5	0.7
11	3.0	0.6	0.4	5	0.7
12	3.0	0.6	0.013	5	0.021
13	1.5	0.3	0.013	5	0.042

Table 4.2 Free flow wind speeds at 2.3 m and 3.5 m

Throttle	Mean Free Flow Wind Speed	
	at 2.3 m	at 3.5 m
	(m/s)	(m/s)
20%	12	13
25%	15	17
30%	18	20
40%	24	27
50%	30	33
60%	37	40
65%	40	44
70%	43	47
80%	49	54
90%	55	60
100%	61	67

Table 4.3 Summary of tare test results – single support (Model 7)

0°						
Wind Speed	Forces (N)			Moments (N-m)		
	F_{x 1}	F_{y 1}	F_{z 1}	M_{x 1}	M_{y 1}	M_{z 1}
12 m/s	-0.24	0.14	1.46	-0.02	-0.06	-0.01
15 m/s	-0.59	0.12	1.06	-0.01	-0.10	0.00
30 m/s	-3.69	0.95	1.98	0.08	-0.40	0.00
40 m/s	-6.86	1.77	4.68	0.16	-0.65	-0.09

45°						
Wind Speed	Forces (N)			Moments (N-m)		
	F_{x 1}	F_{y 1}	F_{z 1}	M_{x 1}	M_{y 1}	M_{z 1}
12 m/s	1.62	-8.00	15.07	-1.05	-0.29	-0.56
15 m/s	1.25	-12.95	10.40	-1.66	-0.48	-0.79
30 m/s	0.80	-51.46	24.76	-6.68	-2.24	-3.73
40 m/s	1.67	-85.64	38.43	-11.05	-3.92	-6.52

Table 4.4 Summary of tare test results – double support (Model 8)

0°						
Wind Speed	Forces (N)			Moments (N-m)		
	F_{x 1}	F_{y 1}	F_{z 1}	M_{x 1}	M_{y 1}	M_{z 1}
12 m/s	0.25	-0.02	10.99	-0.02	0.16	0.30
15 m/s	-0.16	0.22	6.64	0.03	0.21	0.56
30 m/s	-4.84	2.03	2.25	0.37	0.67	2.31
40 m/s	-9.86	3.97	-7.14	0.79	0.96	4.05

45°						
Wind Speed	Forces (N)			Moments (N-m)		
	F_{x 1}	F_{y 1}	F_{z 1}	M_{x 1}	M_{y 1}	M_{z 1}
12 m/s	-2.21	-8.65	2.45	-0.97	0.17	-1.07
15 m/s	-4.10	-13.22	2.16	-1.48	0.33	-1.61
30 m/s	-20.13	-49.23	11.01	-5.45	2.19	-5.71
40 m/s	-36.43	-79.37	21.35	-8.62	4.79	-9.33

Table 4.5 Blockage correction results - 0° wind direction

Model	Blockage Correction
1, 4, and 7	1.3%
2, 5, and 8	3.9%
3, 6, and 9	6.4%
10 and 11	6.4%
12	6.4%
13	1.6%

Table 4.6 Blockage correction results - 45° wind direction

Model	Blockage Correction
1	1.50%
2	4.22%
3	6.94%
4	1.91%
5	4.63%
6	7.35%
7	2.31%
8	5.04%
9, 10, 11	7.76%
12	6.84%
13	1.72%

Table 4.7 Mean wind speeds at 3.5 m

Model No.	Measured Mean Wind Speed (m/s)			
	0° Wind Direction		45° Wind Direction	
	25% Throttle	65% Throttle	25% Throttle	65% Throttle
1	18	48	18	48
2	18	48	15	48
3	18	48	15	48
4	18	48	18	48
5	18	48	15	48
6	18	48	18	48
7	18	48	18	48
8	18	49	18	48
9	19	49	18	48
10	18	49	18	48
11	19	49	18	48

Table 4.8 Percent difference between center and edge probes at 3.5 m

Model No.	% Difference Between Center and Edge Probes			
	0° Wind Direction		45° Wind Direction	
	25% Throttle	65% Throttle	25% Throttle	65% Throttle
1	-5%	-6%	-5%	-5%
2	-2%	-3%		-4%
3	-2%	-2%		-4%
4	-5%	-6%	-5%	-5%
5	-3%	-4%		-4%
6	-2%	-2%	-4%	-5%
7	-5%	-6%	-5%	-6%
8	-4%	-5%	-5%	-7%
9	-2%	-3%	-6%	-6%
10	-4%	-3%	-4%	-5%
11	-4%	-4%	-4%	-5%
Mean	-3%	-4%	-5%	-5%

Table 4.9 Spatial wind speeds at 3.5 m

Model No.	Mean Spatial Wind Speed (m/s)			
	0° Wind Direction		45° Wind Direction	
	25% Throttle	65% Throttle	25% Throttle	65% Throttle
1	19	49	19	49
2	18	48	15	48
3	18	47	14	47
4	19	49	19	49
5	18	49	15	48
6	18	48	18	48
7	19	49	19	49
8	19	49	19	48
9	18	49	18	48
10	18	49	18	47
11	18	49	18	47

Table 4.10 Vertical height adjustment (VHA) ratios

Throttle	20%	25%	30%	40%	50%	60%	65%	70%	80%	90%	100%
Wind Speed (m/s)	12	15	18	24	30	37	40	43	49	55	61
Probe 1:Probe 4	1.17	1.13	1.15	1.13	1.13	1.14	1.14	1.14	1.13	1.12	1.12
Probe 2:Probe 5	1.11	1.11	1.11	1.11	1.11	1.11	1.12	1.12	1.11	1.12	1.12
Probe 3:Probe 6	1.07	1.07	1.06	1.06	1.06	1.05	1.05	1.05	1.05	1.04	1.04
Mean Correction	1.12	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.09	1.09

Figure 4.1 Prototype geometric ratios

Figure 4.2 Model design schematic – Models 1-3

Figure 4.5 Model design schematic - Sections

MODELS 7-9 (depth ratio = 0.7) - PLAN VIEWS

Figure 4.6 Cantilever support system -0.6 and 1.8m models

Figure 4.4 Model design schematic – Models 7-9

Figure 4.7 Cantilever support system – 3m models

Figure 4.9 Model mounting configuration

Figure 4.8 Side elevation of cantilever support system

DETAIL

Figure 4.10 JR3 load cell (photo courtesy Meyer)

Figure 4.11 Turbulent Flow Instruments cobra probe (photo courtesy Meyer)

Figure 4.12 Cobra probe configuration with model in place a) looking upstream b) side elevation (photo courtesy Meyer)

Figure 4.13 Wall of Wind (WOW) (photo courtesy Meyer)

Figure 4.14 Suburban terrain wind speed profile

Figure 4.15 Turbulence intensity plot

Figure 4.16 Spectrum comparison (Mooneghi, 2014)

Figure 4.17 Schematic of tare test Setup A – single support (Models 1, 4, and 7)

Figure 4.18 Schematic of tare test Setup B – double support (M

Figure 4.19 Tare test Setup A in the WOW (a) 0° configuration (b) 45° configuration
(photo courtesy Meyer)

Figure 4.20 Tare test Setup B in the WOW (a) 0° configuration (b) 45° configuration
(photo courtesy Meyer)

Figure 4.21 Blockage test setup in the WOW (a) 0° configuration (b) 45° configuration (photo courtesy Meyer)

Figure 4.22 Blockage correction graph - 0° wind direction

Figure 4.23 Blockage correction graph - 45° wind direction

Figure 4.24 Free flow wind test setup in the WOW a) looking upstream b) side view (photo courtesy Meyer)

Figure 4.25 Force coefficient test setup in the WOW - 0° wind direction (a) Looking upstream (b) Side view (photo courtesy Meyer)

Figure 4.26 Force coefficient test setup in the WOW - 45° wind direction (a) Looking upstream (b) Side view (photo courtesy Meyer)

Figure 4.27 Round corner test setup in the WOW (a) Side view (b) Close-up (photo courtesy Meyer)

Figure 4.28 Chamfer corner test setup in the WOW (a) Side view (b) Close-up (photo courtesy Meyer)

Figure 4.29 Modified corner close-up photos (a) Round corner (b) Chamfer corner (photo courtesy Meyer)

Figure 4.30 Galloping test schematic (a) Side elevation (b) Wedge details

Figure 4.31 Galloping test setup in the WOW (a) Positive 4.5° angle (b) Negative 4.5° angle (photo courtesy Meyer)

Figure 4.32 Wind driven rain test setup in the WOW (a) looking upstream (b) nozzle configuration (photo courtesy Meyer)

5. TEST RESULTS AND DISCUSSIONS

Wind tunnel tests in the WOW were performed with the models and test configurations discussed in Chapter 4. A comprehensive research plan and test procedure was developed to attain the objectives of this project. The plan provided a logical sequence of work and ensured efficient use of the WOW facility. Table 5.1 shows the testing matrix for this study. Laboratory testing was conducted at the FIU WOW from May 28, 2013 to June 7, 2013.

Table 5.1 Testing matrix

Model No.	Test Speed						Wind Approach	
	20% 12 m/s	25% 15 m/s	30% 18 m/s	50% 30 m/s	60% 37 m/s	65% 40 m/s	0°	45°
1	✓	✓				✓	✓	✓
2	✓					✓	✓	✓
3	✓					✓	✓	✓
4	✓	✓				✓	✓	✓
5	✓					✓	✓	✓
6	✓	✓				✓	✓	✓
7	✓	✓				✓	✓	✓
8	✓	✓	✓	✓	✓	✓	✓	✓
9	✓	✓				✓	✓	✓
10	✓	✓		✓		✓	✓	✓
11	✓	✓		✓		✓	✓	✓
12			✓				✓	✓
13					✓		✓	✓

5.1 Force coefficients

Mean normal (C_{Fx}), lateral (C_{Fy}), and vertical lift (C_{Fz}) force coefficients were calculated from the measured data for each model using Equations 1-4, 1-5, and 1-6 respectively. Raw force measurements were adjusted using the load cell calibration

matrix (see Appendix A) and initial baseline loads (no wind) were subtracted from the data. Applicable tare corrections (refer to Section 4.5) and wind field adjustments (see Section 4.6) were applied based on the model setup and test configuration. All force coefficients, except noted otherwise, were normalized using the area of the front face of the model ($A_x = b \times c$). A summary of mean force coefficient results for the 0° horizontal wind direction is included in Table 5.2 and the results for the 45° horizontal wind direction is included in Table 5.3.

5.1.1 Effect of aspect ratio

Variation of the normal force coefficient (C_{Fx}) as a function of aspect ratio (b/c) for the 0° horizontal wind direction is presented in Figure 5.1 for the selected depth ratios $d/c = 0.1, 0.4$, and 0.7 . The plotted results for both the 15 m/s and 40 m/s wind speeds clearly show that the drag coefficient increased with aspect ratio. This is consistent with the normal force coefficient trends noted in the studies by Letchford (2001) and Zuo et al. (2014). The effect of increasing drag with increasing aspect ratio is attributed to the model becoming more two-dimensional as the aspect ratio increases. For models with a $b/c = 1$, flow is directed around the model equally, producing a narrow three-dimensional wake region. This results in less drag and a lower drag coefficient. As the width increases ($b/c = 3$ and $b/c = 5$), the end effects become less dominant and the flow interaction with the model is more two-dimensional. The flow is forced to travel faster over the top and bottom of the model which produces a wider, more two-dimensional wake region resulting in a higher drag coefficient. The lowest drag coefficient is obtained for model (Model 7) which has the smallest aspect ratio ($b/c = 1$) and the largest depth ratio ($d/c =$

0.7). This is because the model with lowest aspect ratio and highest depth ratio has the potential of forming the narrowest 3-D conical downwind wake due to dominant end effects and partial flow reattachment on the side faces as well as the top and bottom faces.

Variation of the normal force coefficient (C_{Fx}) as a function of aspect ratio (b/c) for the 45° horizontal wind direction is presented in Figure 5.2 for the depth ratios $d/c = 0.1$, 0.4, and 0.7. The results show that for a given depth ratio, the drag coefficient generally decreases as the aspect ratio increases. For $d/c = 0.4$ and 0.7, C_{Fx} increases slightly from $b/c = 1$ to $b/c = 3$ then decreases to $b/c = 5$. The sharpest decrease in the normal force coefficient was measured for the models with the smallest depth ratio ($d/c = 0.1$) corresponding to Models 1, 2 and 3. This is a reversal of the trend observed for the 0° horizontal wind direction. In general, the largest drag coefficient for each tested depth ratio was obtained for the model with the smallest aspect ratio.

The lateral force coefficient (C_{Fy}) was calculated using the following values for area in the drag coefficient equation

$$A_x = b \times c \quad (5-1)$$

$$A_y = d \times c \quad (5-2)$$

A summary and comparison of the result are included in Tables 5.4 and 5.5. As expected, the results for C_{Fy} using A_x are small compared to the results for C_{Fy} using A_y since the area of the front face for all the models is significantly larger than the area of the side face. Consequently, the results for the 0° wind direction are approximately zero and only the 45° wind direction results are discussed further. Figure 5.3 is a plot of C_{Fy} using A_x as a function of aspect ratio for the tested depth ratios of 0.1, 0.4, and 0.7 in the 45°

horizontal wind direction. All results indicate a decrease in C_{Fy} with increasing aspect ratio. The largest decrease was measured for models with $d/c = 0.7$ (Model 7) which decreased from $C_{Fy} = -0.54$ for $b/c = 1$ to $C_{Fy} = -0.11$ for $b/c = 5$ (Model 9) resulting in 80% reduction in C_{Fy} . A negative value for C_{Fy} represents a result in the direction normal to the front face of the model (refer to Figure 1.5 for force orientation). The value of C_{Fy} using A_x for $b/c = 5$ appears to converge at approximately -0.05 for the 15 m/s wind speed and -0.07 for the 40 m/s wind speed. C_{Fy} lateral using A_x was obtained for Model 7 ($b/c = 1$ and $d/c = 0.7$) which was approximately $C_{Fy} = -0.55$. The smallest value of C_{Fy} using A_x was approximately $C_{Fy} = 0$ obtained for Model 3. The results for C_{Fy} using A_x are significantly affected by dividing the measured force by the comparatively large area of the front face. Consequently, it was decided that C_{Fy} using A_y would be a better representation of the findings.

As shown in Table 5.4, the results for C_{Fy} using A_y in the 0° wind direction for the 15 m/s test speed varies from approximately 0 to -0.84 and for the 40 m/s test speed varies from 0 to -0.39. For the 15 m/s wind speed the largest value ($C_{Fy} = 0.84$) was obtained for Model 2 ($b/c = 3$ and $d/c = 0.1$) and the smallest value ($C_{Fy} \approx 0$) was obtained for Models 1 and 6-9. For the 40 m/s wind speed the largest value ($C_{Fy} = 0.39$) was obtained for Model 3 ($b/c = 5$ and $d/c = 0.1$) and the smallest value ($C_{Fy} \approx 0$) was obtained for Models 1, 4, and 6. Figure 5.4 is a plot of C_{Fy} using A_y as a function of aspect ratio for the tested depth ratios of 0.1, 0.4, and 0.7 in the 0° horizontal wind direction. For the 15 m/s wind speed the value of C_{Fy} using A_y was approximately $C_{Fy} = 0$ for $d/c = 0.7$ (Models 7, 8, and 9). This was thought to occur because the flow partially reattached along the sides of the models and no lateral force develops. However, for $d/c = 0.1$ and 0.4, C_{Fy} using A_y

increases dramatically from $b/c = 1$ to $b/c = 3$ then decreases for $b/c = 5$. This can be explained by noting that the flow around models with $b/c = 3$ and $b/c = 5$ is predominately over the top and bottom and an unequal force is produced in the lateral direction due to the alternating formation of vortices around the sides close to the rear of the model. This effect is more pronounced for the thinnest models ($d/c = 0.1$). The largest C_{Fy} using A_y for the 0° wind direction was $C_{Fy} = 0.84$ obtained for Model 2 ($b/c = 3$ and $d/c = 0.1$) and the largest increase in C_{Fy} using A_y was between $C_{Fy} \approx 0$ for Model 1 ($b/c = 1$ and $d/c = 0.1$) and $C_{Fy} = 0.84$ for Model 2. Similar trends in C_{Fy} using A_y were noted for the 40 m/s wind speed test but the values were generally smaller. An exception was also noted for models with $d/c = 0.1$. There was a slight increase in C_{Fy} using A_y from $b/c = 3$ to $b/c = 5$ instead of the sharp decrease noted in the 15 m/s wind speed tests. In summary, mean lateral forces are expected to be small, 1 or 2% of the drag force. The values obtained in this study are an indication of the resolution of the experiment.

Figure 5.5 shows the trend of C_{Fy} using A_y as a function of aspect ratio for the 45° wind direction. In contrast to the results for the 0° wind direction, all results for the 45° wind direction are negative (refer to Figure 1.5 for the force orientation). This result is expected due to the positioning of the model with respect to the oncoming flow and the positive orientation of F_y which is 90° to the front face of the model. The values for C_{Fy} using A_y for the 45° wind direction are generally larger than the results obtained for the 0° wind direction. This is also due to the 45° orientation of the model with respect to the oncoming flow since the side face closest to the WOW outlet (see Figure 4.25) is directly impacted by the oncoming flow. For the 15 m/s wind speed test with $d/c = 0.1$, C_{Fy} using A_y decreases from $C_{Fy} = -1.16$ to $C_{Fy} = -0.31$ (73% decrease) with increasing aspect ratio.

The values of C_{Fy} using A_y for models with $d/c = 0.4$ and 0.7 increase slightly from $b/c = 1$ to $b/c = 3$ then decrease to $b/c = 5$. The largest C_{Fy} using A_y was $C_{Fy} = -1.16$ obtained for Model 1 and the smallest was obtained for Model 3 which was $C_{Fy} = -0.31$. The 40 m/s wind speed test results for $d/c = 0.4$ and $d/c = 0.7$ are similar to the trends noted for the 15 m/s wind speed tests. However, C_{Fy} using A_y for $d/c = 0.1$ increases sharply with increasing aspect ratio from $C_{Fy} = -1.05$ to $C_{Fy} = -2.21$ for a 110% increase. The largest $45^\circ C_{Fy}$ using A_y measured was $C_{Fy} = -2.21$ for Model 3 and the smallest was $C_{Fy} = -0.75$ obtained for Model 9.

5.1.2 Effect of depth ratio

Figure 5.6 shows the variation of the normal force coefficient (C_{Fx}) for the 0° horizontal wind approach as a function of depth ratio for the selected aspect ratios $b/c = 1, 3, \text{ and } 5$. The plotted results for the 15 m/s and 40 m/s wind speeds showed a decreasing trend in the drag coefficient with increasing depth ratio for all aspect ratios tested. These results suggest that partial reattachment of flow over a prismatic VMS reduces the drag force resulting in a smaller drag coefficient for use in design. The sharpest decrease in normal force was noted for the square models, $b/c = 1$ (Models 1, 4 and 7). As the model becomes deeper, flow reattachment is promoted over the sides and the top and bottom faces. Model 7 (largest depth ratio and lowest aspect ratio), produced the narrowest 3-D conical downwind wake which resulted in the smallest normal force coefficient among all models tested.

Figure 5.7 shows the variation of the C_{Fx} for the 45° horizontal wind approach as a function of depth ratio (d/c) for the aspect ratios $b/c = 1, 3, \text{ and } 5$. The graphs indicated

that C_{Fx} for the 45° wind direction also decreased with increasing depth ratio. Similar to the 45° horizontal wind approach trends for aspect ratio, the normal force coefficient decreased as the depth of the model increased (increased depth ratio). For each aspect ratio, C_{Fx} decreases as the length of the side face increases (depth ratio increases). The smallest C_{Fx} for the 15 m/s wind speed test ($C_{Fx} = 0.89$) was obtained for Model 9 ($d/c = 0.7$, $b/c = 5$), the deepest and longest model. Conversely, the largest ($C_{Fx} = 1.23$) was obtained for Model 1 which had the smallest aspect ratio ($b/c = 1$) and the smallest depth ratio ($d/c = 1$).

The results for C_{Fy} using A_x in the 0° horizontal wind direction are approximately zero and consequently not discussed. Figure 5.8 is a plot of C_{Fy} using A_x as a function of depth ratio for the tested aspect ratios of 1, 3, and 5 in the 45° horizontal wind direction. C_{Fy} using A_x is shown to increase with increasing depth ratio for all model tested. Figure 5.9 shows the plot of C_{Fy} using A_y as a function of the depth ratio for the 0° wind direction for the tested aspect ratios of 1, 3, and 5. For $b/c = 1$ the value of C_{Fy} using A_y is approximately equal to zero. As already discussed in conjunction with the results for C_{Fx} as a function of aspect ratio in the 0° wind direction (Section 5.1.1), the flow around models with $b/c = 1$ produced a narrow 3D wake region and no significant lateral forces develop. However, for the 15 m/s wind speed and $b/c = 3$ and 5, C_{Fy} decreases with increasing depth ratio and converges to approximately zero at $d/c = 0.7$. Again this could be due to partial reattachment of flow along the sides of models with $d/c = 0.7$ resulting in minimal force development in the lateral direction. The plot for the 40 m/s wind speed test was similar except for a slight increase in C_{Fy} for $b/c = 5$ from $d/c = 0.4$ to $d/c = 0.7$.

Figure 5.10 is a plot of C_{Fy} using A_y as a function of depth ratio for the tested aspect ratios of 0.1, 0.4, and 0.7 in the 45° horizontal wind direction. For the 15 m/s wind speed, C_{Fy} using A_y for $b/c = 1$ is shown to decrease with increasing depth ratio (34% decrease), C_{Fy} using A_y for $b/c = 5$ is shown to increase with increasing depth ratio (145% increase) and C_{Fy} using A_y for $b/c = 3$ remains approximately constant at $C_{Fy} = -0.91$. For the 40 m/s wind speed, C_{Fy} using A_y decreases with increasing depth ratio for all models tested.

5.1.3 Effect of wind direction

A comparison of C_{Fx} results in the 0° and 45° wind direction is shown in Figure 5.11. As illustrated, the C_{Fx} results for the 45° horizontal wind approach direction for all models tested were smaller than the 0° horizontal wind approach direction except for Model 1. C_{Fx} for the 45° direction for Model 1 ($C_{Fx} = 1.24$) was 7% larger than the 0° result ($C_{Fx} = 1.16$). This finding was noted in preliminary testing and again during final testing. The square shape and small depth ratio may have contributed to this unique result. One possible explanation is that the negative pressure created on the rear leading edge is larger for structures with a short width (b) dimension. In addition, as the depth (d) dimension increases, the streamlines are deflected around the shape instead of curling around the rear edge. This reduces the negative pressure on the rear leading edge resulting in a reduced mean normal force coefficient. The largest variation for the 15 m/s wind speed was generally noted for models with $b/c = 5$ (Models 3, 6 and 9) with an average difference of 0.25 and the smallest variation was noted for models with $b/c = 1$ (Models 1, 4, and 7) with an average difference of 0.06. The results for the 40 m/s wind speed tests were similar. The average difference for models with $b/c = 1$ was 0.08 and the

average difference for models with $b/c = 5$ was 0.23. The largest difference between C_{Fx} results for the 0° and 45° wind directions was approximately 0.30 for Model 9.

Figure 5.12 shows a comparison of results for 0° and 45° wind directions for C_{Fy} using A_y . As noted previously, the results for the 0° wind direction are approximately equal to zero (except for the small forces measured for Models 2, 3 and 5) while the results for the 45° wind direction are larger and negative due to the wind impacting the end faces directly. The largest difference for C_{Fy} using A_y was 1.7 (Model 2) for the 15 m/s wind speed and 2.6 (Model 3) for the 40 m/s wind speed.

5.1.4 Reynolds number effect

All models were tested for wind speeds of 15 m/s and 40 m/s which corresponds to a Reynolds Number (Re) test range of 5.96×10^5 to 1.59×10^6 . Re is defined as:

$$Re = \frac{c\bar{U}}{\nu} \quad (5-15)$$

Where c is the height of model (0.6 m), $\bar{U}$ is the mean free flow wind speed (m/s) at 2.3 m, and ν is the viscosity of air ($1.51 \times 10^{-5} \text{ m}^2/\text{sec}$).

The normal and cross wind force coefficients as a function of the Reynolds number are plotted in Figure 5.13. Test results were evaluated to determine the sensitivity of sharp edge models to Re effects. Letchford (2001) noted a decreasing trend in C_{Fx} with higher wind speeds but concluded that there is little evidence to support Re dependency. In contrast, the results of this study indicate a small increase in C_{Fx} and C_{Fy} using A_y with wind speed for sharp edge models (Models 1-9) but this was thought to be caused by the introduction of additional vibration in the test setup at higher speeds. Consequently Re effects were not considered in this study. However, a possible Re dependency was noted

for the increase in C_{Fx} for the round edge configuration (Model 10). This illustrates the importance of edge configuration in determining Re effects.

5.2 Twisting coefficient and eccentricity estimates for fatigue and strength design

The twisting moment coefficient was calculated using Equation 1-13 with the sign conventions shown in Figure 1.5. The non-dimensional eccentricity ratio was calculated using Equation 1-16. Table 5.6 is a summary of the results for the twisting moment (C_{Mz}) and eccentricity ratio (r/b) for the 0° horizontal wind direction and Table 5.7 is a summary of the results for the 45° wind direction. The eccentricity for the 0° horizontal wind approach direction is approximately zero for all the models tested. Consequently, only the results for the 45° wind direction are discussed in detail. The largest calculated eccentricity is $r/b = 0.13$ for Model 2 which is still lower than the ASCE 7-10 (2010) suggested code value of 0.2. It should be noted that the current AASHTO specification (2013) does not include a design requirement for eccentricity.

5.2.1 Effect of aspect ratio

Variation of the twisting moment (C_{Mz}) as a function of aspect ratio (b/c) for the 45° horizontal wind direction is graphed in Figure 5.14 for the selected depth ratios $d/c = 0.1$, 0.4, and 0.7. The results show that for a given depth ratio, the twisting moment coefficient increases from $b/c = 1$ to $b/c = 3$ then decreases at $b/c = 5$. The largest twisting moment coefficient ($C_{Mz} = 0.15$) was obtained for Model 2 and the smallest ($C_{Mz} = 0.03$) for Model 7. The average C_{Mz} for models with $b/c = 1$, 3 and 5 was 0.07, 0.12 and 0.10 respectively. The results of the eccentricity ratio as function of aspect ratio are plotted in

Figure 5.15. As expected, the plotted trend for the eccentricity ratio follows the plotted results of the twisting moment coefficient. The largest eccentricity ratio was calculated to be 0.13 (Model 2) and the smallest was 0.03 (Model 7).

5.2.2 Effect of depth ratio

Variation of the twisting moment (C_{Mz}) as a function of depth ratio (b/c) for the 45° horizontal wind direction is plotted in Figure 5.16 for the selected aspect ratios $b/c = 1, 3$, and 5. The results show that for a given aspect ratio, the twisting moment coefficient decreases with increasing depth ratio. The results of the eccentricity ratio as function of depth ratio is plotted in Figure 5.17 and follows the same trend as the twisting moment results.

5.3 Corner Modification

This series of tests investigated the effects of modifying the model changing the sharp corners to round (Model 10) or chamfered (Model 11) corners. Refer to Table 4.1 for model dimensions and Figures 4.26, 4.27 and 4.28 for pictures of the test configuration and model setup in the WOW. Testing was performed using the same parameters and conditions as the sharp edge tests. The drag coefficient test results for the modified models are tabulated in Tables 5.8 and 5.9.

Figure 5.18a is a plot of C_{Fx} in the 0° wind direction for the models with modified corners as a function of wind speed. The results for the sharp corner model (Model 9) are included in the graph for comparison. Results show that drag coefficients for models with modified corners are lower than the drag coefficients for the sharp corner models. As

outlined by Tamura and Miyagi (1999), this suggests that chamfered and round corners decrease the wake width resulting in reduced drag. At a wind speed of 15 m/s, the drag coefficient decreases in the order of sharp edge, chamfered and round corners. These results agree qualitatively with Tamura and Miyagi (1999) and Yamagishi et al. (2010). The drag coefficient for the chamfered model is 22% smaller than the sharp edge model and the drag coefficient for the round edge model is 40% smaller than the sharp edge model. However, at a wind speed of 40 m/s, Figure 5.18a shows that the drag coefficient decreases in the order of sharp edge, round and chamfered corners. Here the drag coefficient for the round edge model is larger than the coefficient for the chamfered model. The round model is still 16% smaller than the sharp edge model and the drag coefficient for the chamfered is 21% smaller than the sharp edge model. A slight increase in C_{Fx} with wind speed was noted for the sharp and chamfered models. However, C_{Fx} for the round corner model increased significantly which may indicate a possible Re dependency for the round corner model. The chamfered modification is recommended for wind load mitigation purposes as it has less Re dependency and is easier to fabricate.

Figure 5.18b shows the C_{Fx} in the 45° wind direction. As illustrated, the coefficient for the modified corner models are significantly lower than the sharp edge results for Model 9. Similar to the 0° wind direction, the 45° wind direction coefficient decreases in the order of sharp edge, chamfered, and round corners for the 15 m/s wind speed test and in the order of sharp edge, round, and chamfered corners for the 40 m/s wind speed tests.

5.4 Wind Driven Rain

Model 8 (1.8 x 0.6 x 0.4 m) was subjected to a simulated high velocity wind driven rain environment by utilizing spray nozzles mounted on the exhaust side of the WOW fans. The configuration of the wind driven rain test is discussed in Section 4.6.4 and the test setup in the WOW is shown Figure 4.31. This test was only conducted for the 45° wind direction.

The measured C_{Fx} for Model 8 with and without wind driven rain was 0.96 and 0.93 respectively for the 15 m/s wind speed. The results for the 40 m/s wind speed were 0.94 and 0.95. It does not appear from this limited testing that wind driven rain has any effect on C_{Fx} .

5.5 Discussion and comparison with previous results

Table 5.10 lists the normal force coefficient results for the 0° horizontal wind approach direction obtained in this study along with results from Letchford (2001), Quinn et al. (2001), Smith et al. (2014), and Zuo et al. (2014). The AASHTO Specification (2013) flat panel and VMS code recommended values are also included. An examination of the results shows good agreement with the wind tunnel results obtained by Zuo et al. (2014) and Smith, et al. (2014). Figure 5.19 graphically compares these results as a function of aspect ratio. As shown in the graph, the flat panel results by Letchford (2001) are larger than the prismatic results obtained in this study and wind tunnel experiments by Zuo et al. (2014). The lower drag coefficient values obtained for prismatic rectangular shapes may be in part due to model thickness and partial reattachment of flow after initial separation from the leading edge of the model. As discussed in Sections 2.1.2 and 5.1.2,

reattachment of flow along the sides of the prismatic shape leads to vortex formation further downstream, a smaller downstream wake region, and a weaker entrainment process resulting in a lower drag coefficient. In addition, the excessive energy content in the high frequency range of Letchford's (2001) wind tunnel ABL simulation may have contributed to the larger results. The partial simulation method used in the WOW ensures that the high frequency spectral content of the WOW flow matches the high frequency portion of the ABL spectrum (see Section 4.4.2). Proper simulation of the high frequency range of the spectrum is critical in the investigation of sharp edge shapes like VMS since small scale high energy fluctuations significantly affect aerodynamic behavior such as local vortex formation, flow separation, and flow reattachment (Fu et al., 2012). Since the current AASHTO Specification (2013) recommended drag coefficient is 1.7, it is suggested that VMS structures designed using these values may be overdesigned and uneconomical.

The drag coefficient result of $C_{Fx} = 1.08$ from Quinn's et al. (2001) full scale study at Silsoe for a sign with $b/c = 1$ and $c/h = 0.32$ is in good agreement with the WOW result for Model 1, 4, and 7 ($C_{Fx} = 1.15, 1.12$, and 0.98) which have comparable geometry ($b/c = 1$ and $d/c = 0.1, 0.4$ and 0.7). The slightly higher results of the WOW study may be due in part to the smaller clearance ratio (larger gap beneath the sign) of $c/h = 0.24$ used in this study as opposed to $c/h = 0.32$ by Quinn et al. (2001). In addition, the Smith et al. (2014) full scale result of $C_{Fx} = 1.13$ for $b/c = 2$ and $d/c = 0.47$ is in excellent agreement with the interpolated WOW results for Models 4 and 5 which was $C_{Fx} = 1.17$.

The measured WOW normal force coefficients for the 45° horizontal wind approach direction were compared to the results obtained by Letchford (2001), Quinn et al. (2001),

Ginger et al. (1998), Paulotto et al. (2006) and Zuo et al. (2014). In the WOW study, all normal force coefficient results for the 45° horizontal wind approach direction (except the results for Model 1 which is discussed separately in Section 5.1.3) were lower than the results for the 0° horizontal wind approach direction. This finding is consistent with Ginger et al. (1998), Zuo et al. (2014) and the limited results obtained by Paulotto et al. (2006) but is not consistent with Letchford's (2001) and Quinn's et al. (2001) suggestion that C_{Fx} is independent of the wind approach direction for approach angles between 0° and 45°. However, graphs of Letchford's (2001) drag coefficient results versus wind direction show a significant reduction in the drag coefficient for wind direction angles larger than 45°. Letchford's (2001) result for the wind direction of 45° with $b/c = 2$ and $c/h = 0.5$ is approximately $C_{Fx} = 1.5$, but decreases to approximately $C_{Fx} = 1.1$ at the wind direction of 60°. In Zuo et al. (2014), a graph of drag coefficient versus wind direction shows a significant drop in C_{Fx} for the 45° yaw angle (horizontal wind direction). The reduction increases with increasing wind direction angles up to 90°.

The eccentricity ratios computed in the FIU WOW study were compared to the wind tunnel values from Letchford (2001) and Zuo et al. (2014) and the full scale results from Smith et al. (2014). The eccentricity ratios in this study were mostly below 0.13 and in good agreement with the full scale results obtained by Smith et al. (2014) and Zuo et al. (2014) but considerably lower than the results obtained by Letchford (2001) and the suggested ASCE 7-10 value of 0.2. However, Smith et al. (2014) tested a range of attack angles and the maximum eccentricity ratio occurred around an angle of attack of 70° (200° using the model orientation of Smith et al. 2014). Consequently the FIU WOW results may not be conservative since only the 45° angle of attack was tested.

In summary, the FIU WOW results are in very good agreement with the full scale tests results by Quinn et al. (2001) and Smith et al. (2014). It is also evident from the results of this study and the wind tunnel results by Zuo et al. (2014) that the drag coefficient values for prismatic signs are lower than the values for flat panels with corresponding aspect and clearance ratios.

Table 5.2 Summary of mean force coefficient results for the 0° wind direction

Model No.	b (m)	d (m)	c (m)	Aspect Ratio b/c	Depth Ratio d/c	0° Wind Direction					
						15 m/s			40 m/s		
						C _{Fx}	C _{Fy}	C _{Fz}	C _{Fx}	C _{Fy}	C _{Fz}
1	0.6	0.06	0.61	1	0.1	1.16	0.00	-0.06	1.15	0.00	-0.05
2	1.8	0.06	0.61	3	0.1	1.20	0.03	-0.08	1.22	0.01	-0.01
3	3.0	0.06	0.61	5	0.1	1.24	0.01	-0.07	1.28	0.01	0.00
4	0.6	0.24	0.61	1	0.4	1.05	0.03	-0.12	1.12	0.00	-0.03
5	1.8	0.24	0.61	3	0.4	1.17	0.03	-0.07	1.21	0.02	0.00
6	3.0	0.24	0.61	5	0.4	1.23	0.00	-0.06	1.25	0.00	0.00
7	0.6	0.43	0.61	1	0.7	0.97	0.00	0.00	0.98	0.01	0.01
8	1.8	0.43	0.61	3	0.7	1.12	-0.01	-0.05	1.16	0.02	0.04
9	3.0	0.43	0.61	5	0.7	1.19	0.00	-0.05	1.22	0.01	0.03
10	3.0	0.43	0.61	5	0.7	0.85	0.03	0.24	1.06	0.04	0.12
11	3.0	0.43	0.61	5	0.7	0.97	0.03	0.04	1.01	0.04	0.12
12	3.0	0.01	0.61	5	0.02	1.27	0.01	-0.04	-	-	-
13	1.5	0.01	0.30	5	0.04	-	-	-	1.27	0.01	0.06

Table 5.3 Summary of mean force coefficient results for the 45° wind direction

Model No.	b (m)	d (m)	c (m)	Aspect Ratio b/c	Depth Ratio d/c	45° Wind Direction					
						15 m/s			40 m/s		
						C _{Fx}	C _{Fy}	C _{Fz}	C _{Fx}	C _{Fy}	C _{Fz}
1	0.6	0.06	0.61	1	0.1	1.23	-0.12	-0.02	1.24	-0.10	0.06
2	1.8	0.06	0.61	3	0.1	1.13	-0.03	0.00	1.18	-0.06	0.04
3	3.0	0.06	0.61	5	0.1	1.02	-0.01	-0.04	1.11	-0.04	0.02
4	0.6	0.24	0.61	1	0.4	0.99	-0.35	-0.08	1.02	-0.36	0.01
5	1.8	0.24	0.61	3	0.4	1.03	-0.13	-0.06	1.06	-0.16	-0.01
6	3.0	0.24	0.61	5	0.4	0.99	-0.04	-0.05	1.03	-0.07	-0.04
7	0.6	0.43	0.61	1	0.7	0.93	-0.54	-0.07	0.94	-0.56	0.01
8	1.8	0.43	0.61	3	0.7	0.93	-0.21	0.05	0.95	-0.22	0.08
9	3.0	0.43	0.61	5	0.7	0.89	-0.11	0.02	0.91	-0.11	0.04
10	3.0	0.43	0.61	5	0.7	0.69	-0.07	0.00	0.74	-0.11	0.03
11	3.0	0.43	0.61	5	0.7	0.74	-0.06	0.10	0.72	-0.10	-0.06
12	3.0	0.01	0.61	5	0.02	1.04	-0.11	0.04	-	-	-
13	1.5	0.01	0.30	5	0.04	-	-	-	1.04	-0.47	0.22

Table 5.4 Summary of results for C_{Fy} using A_x and A_y for the 0° wind direction

Model No.	b (m)	d (m)	c (m)	Aspect Ratio	Depth Ratio	A_x (m^2)	A_y (m^2)	C_{Fy} for 0° Wind Direction			
								15 m/s		40 m/s	
								Using A_x	Using A_y	Using A_x	Using A_y
1	0.61	0.06	0.61	1	0.1	0.37	0.04	0.00	0.01	0.00	0.02
2	1.83	0.06	0.61	3	0.1	1.11	0.04	0.03	0.84	0.01	0.35
3	3.05	0.06	0.61	5	0.1	1.86	0.04	0.01	0.37	0.01	0.39
4	0.61	0.24	0.61	1	0.4	0.37	0.15	0.03	0.08	0.00	0.00
5	1.83	0.24	0.61	3	0.4	1.11	0.15	0.03	0.24	0.02	0.13
6	3.05	0.24	0.61	5	0.4	1.86	0.15	0.00	0.03	0.00	0.04
7	0.61	0.43	0.61	1	0.7	0.37	0.26	0.00	0.01	0.01	0.01
8	1.83	0.43	0.61	3	0.7	1.11	0.26	-0.01	-0.03	0.02	0.07
9	3.05	0.43	0.61	5	0.7	1.86	0.26	0.00	0.01	0.01	0.08

Table 5.5 Summary of results for C_{Fy} using A_x and A_y for the 45° wind direction

Model No.	b (m)	d (m)	c (m)	Aspect Ratio	Depth Ratio	A_x (m^2)	A_y (m^2)	C_{Fy} for 45° Wind Direction			
								15 m/s		40 m/s	
								Using A_x	Using A_y	Using A_x	Using A_y
1	0.61	0.06	0.61	1	0.1	0.37	0.04	-0.12	-1.16	-0.10	-1.05
2	1.83	0.06	0.61	3	0.1	1.11	0.04	-0.03	-0.86	-0.06	-1.89
3	3.05	0.06	0.61	5	0.1	1.86	0.04	-0.01	-0.31	-0.04	-2.21
4	0.61	0.24	0.61	1	0.4	0.37	0.15	-0.35	-0.87	-0.36	-0.89
5	1.83	0.24	0.61	3	0.4	1.11	0.15	-0.13	-0.95	-0.16	-1.22
6	3.05	0.24	0.61	5	0.4	1.86	0.15	-0.04	-0.48	-0.07	-0.87
7	0.61	0.43	0.61	1	0.7	0.37	0.26	-0.54	-0.77	-0.56	-0.79
8	1.83	0.43	0.61	3	0.7	1.11	0.26	-0.21	-0.91	-0.22	-0.93
9	3.05	0.43	0.61	5	0.7	1.86	0.26	-0.11	-0.76	-0.11	-0.75

Table 5.6 Summary of twisting moment and eccentricity ratio results for the 0° wind direction

Model No.	15 m/s			40 m/s		
	C_{Mz}	r/b	r	C_{Mz}	r/b	r
1	0.00	0.00	0.00	0.00	0.00	0.00
2	-0.01	-0.01	-0.02	-0.01	-0.01	-0.01
3	-0.01	-0.01	-0.02	0.00	0.00	0.00
4	-0.04	-0.04	-0.03	0.00	0.00	0.00
5	-0.01	-0.01	-0.02	-0.01	-0.01	-0.02
6	0.00	0.00	-0.01	0.00	0.00	-0.01
7	0.01	0.01	0.00	0.00	0.00	0.00
8	-0.01	-0.01	-0.02	0.00	0.00	0.00
9	-0.01	-0.01	-0.03	0.00	0.00	-0.01

Table 5.7 Summary of twisting moment and eccentricity ratio results for the 45° wind direction

Model No.	15 m/s			40 m/s		
	C_{Mz}	r/b	r	C_{Mz}	r/b	r
1	0.13	0.11	0.06	0.13	0.10	0.06
2	0.14	0.12	0.22	0.15	0.13	0.23
3	0.11	0.11	0.33	0.13	0.11	0.34
4	0.05	0.05	0.03	0.06	0.06	0.04
5	0.11	0.11	0.19	0.13	0.12	0.22
6	0.09	0.09	0.27	0.10	0.10	0.30
7	0.02	0.02	0.01	0.03	0.03	0.02
8	0.08	0.09	0.16	0.08	0.08	0.15
9	0.06	0.07	0.20	0.06	0.07	0.21

Table 5.8 Summary of modified corner results for the 0° wind direction

Model No.	0° Wind Direction					
	15 m/s			40 m/s		
	C _{Fx}	C _{Fy}	C _{Fz}	C _{Fx}	C _{Fy}	C _{Fz}
9	1.19	0.00	-0.05	1.22	0.01	0.03
10	0.85	0.03	0.24	1.06	0.04	0.12
11	0.97	0.03	0.04	1.01	0.04	0.12

Table 5.9 Summary of modified corner for the 45° wind direction

Model No.	45° Wind Direction					
	15 m/s			40 m/s		
	C _{Fx}	C _{Fy}	C _{Fz}	C _{Fx}	C _{Fy}	C _{Fz}
9	0.89	-0.11	0.02	0.91	-0.11	0.04
10	0.69	-0.07	0.00	0.74	-0.11	0.03
11	0.74	-0.06	0.10	0.72	-0.10	-0.06

Table 5.10 Comparison with previous research results for mean normal force coefficients

Study	Type	Depth Ratio (d/c)	Clearance Ratio (c/h)	Aspect Ratio (b/c)				
				1	2	3	4	5
FIU - Models 1, 2, 3	Large Scale Wind Tunnel	0.10	0.24	1.15	1.19*	1.22	1.25*	1.28
FIU - Models 4, 5, 6	Large Scale Wind Tunnel	0.40	0.24	1.12	1.17*	1.21	1.23*	1.25
FIU - Models 7, 8, 9	Large Scale Wind Tunnel	0.70	0.24	0.98	1.07*	1.16	1.19*	1.22
Zuo (2014) WT	Small Scale Wind Tunnel	0.29	0.50	1.25	-	-	-	-
Zuo (2014) WT	Small Scale Wind Tunnel	0.48	0.50	-	1.22	-	-	-
Zuo (2014) WT	Small Scale Wind Tunnel	0.58	0.50	-	-	-	1.21	-
Zuo (2014) WT	Small Scale Wind Tunnel	0.53	0.50	-	-	-	-	1.27
Smith (2014)	Full Scale	0.47	0.50	-	1.13	-	-	-
Quinn (2001)	Full Scale	Flat Panel	0.32	1.08	-	-	-	-
Letchford (2001)	Small Scale Wind Tunnel	Flat Panel	0.30	1.42	1.45	-	1.53	1.57
Letchford (2001)	Small Scale Wind Tunnel	Flat Panel	0.50	1.38	1.42	-	1.45	1.44
AASHTO Flat Panel	N/A	All	All	1.12	1.19	-	-	1.2
AASHTO VMS	N/A	All	All	1.70	1.70	1.70	1.70	1.70

* Interpolated results

Figure 5.1 Effect of aspect ratio (b/c) on the normal force coefficient (C_{Fx}) for the 0° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.2 Effect of aspect ratio (b/c) on the normal force coefficient (C_{Fx}) for the 45° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.3 Effect of aspect ratio (b/c) on the lateral force coefficient (C_{Fy}) using A_x for the 45° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.4 Effect of aspect ratio (b/c) on the lateral force coefficient (C_{Fy}) using A_y for the 0° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.5 Effect of aspect ratio (b/c) on the lateral force coefficient (C_{Fy}) using A_y for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.6 Effect of depth ratio (d/c) on the normal force coefficient (C_{Fx}) for the 0° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.7 Effect of depth ratio (d/c) on the normal force coefficient (C_{Fx}) for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.8 Effect of depth ratio (d/c) on the lateral force coefficient (C_{Fy}) using A_x for the 45° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.9 Effect of depth ratio (d/c) on the lateral force coefficient (C_{Fy}) using A_y for the 0° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.10 Effect of depth ratio (d/c) on the lateral force coefficient (C_{Fy}) using A_y for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.11 Comparison of normal force coefficients (C_{Fx}) for the 0° and 45° wind directions (a) 15 m/s (b) 40 m/s

Figure 5.12 Comparison of lateral coefficients (C_{Fy}) using A_y for the 0° and 45° wind directions (a) 15 m/s; (b) 40 m/s

Figure 5.13 C_{Fx} and C_{Fy} using A_y as a function of wind speed

Figure 5.14 Effect of aspect ratio (b/c) on the twisting moment coefficient (C_{Mz}) for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.15 Effect of aspect ratio (b/c) on the eccentricity ratio (r/b) for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.16 Effect of depth ratio (d/c) on the twisting moment coefficient (C_{Mz}) for the 45° wind direction (a) 15 m/s; (b) 40 m/s

Figure 5.17 Effect of depth ratio (d/c) on the eccentricity ratio (r/b) for the 45° wind direction (a) 15 m/s; (b) 40 m/s)

Figure 5.18 Comparison of sharp (Model 9) and modified corner (Model 10 and 11) results (a) 0° wind direction (b) 45° wind direction

Figure 5.19 Comparison of FIU WOW and past research results for normal force coefficient

6 ASSESSMENT OF AERODYNAMIC INSTABILITY

6.1 Predictions of vortex shedding amplitudes from spectra of lift

Vortices shed from the upper and lower edges of the VMS cause fluctuating forces primarily in the crosswind direction. The simplest theory for predicting vortex excitation assumes the excitation force oscillates in time t in a pure sinusoidal manner, with the excitation force per unit length being given by

$$f(t) = \frac{1}{2} \rho U^2 c C'_L \sin \omega_v t \quad (6-1)$$

in which C'_L is a fluctuating “lift coefficient”, c = cross-section width normal to wind, and

$$\omega_v = 2\pi N_v .$$

where N_v is the shedding frequency of the vortices.

Considering crosswind motions in the direction, z , the deflection $z(y,t)$ of the structure in each of its natural modes of vibration may be expressed as

$$z(y,t) = \phi(y)q(t) \quad (6-2)$$

where $\phi(y)$ = deflection shape of the mode of vibration, and $q(t)$ = generalized coordinate for the mode of vibration. It can then be shown that the equation of motion in a particular mode of vibration is

$$M_G (\ddot{q} + 2\varpi_0 \zeta \dot{q} + \varpi_0^2 q) = F_G(t) \quad (6-3)$$

where

$$M_G = \text{generalized mass} = \int_0^L m(y) \phi(y)^2 dy \quad (6-4)$$

and

$$F_G = \text{generalized aeodynamic force} = \int_0^L f(y, t) \phi(y) dy \quad (6-5)$$

where m = mass per unit length, and L = length of structure. As we are assuming, for simplicity, that the force fluctuation is uniform along the length of the structure, Equation 6-5 becomes

$$F_G = \frac{1}{2} \rho U^2 C'_L c \times \int_0^L \phi dy \times \sin(\omega_v t) \quad (6-6)$$

The solution to Equation 6-3 can be obtained by assuming

$$q = q_0 \sin(\omega_v t + \varphi)$$

where φ = phase angle, which leads to a predicted maximum response when $\omega_v = \omega_0$, with the amplitude being

$$|q_0| = \frac{\frac{1}{2} \rho U^2 C'_L c \int_0^L \phi dy}{\omega_v^2 M_G \cdot 2\zeta} \quad (6-7)$$

If the mass distribution is uniform then this becomes

$$|q_0| = \frac{\rho U^2 C'_L c}{4\omega_v^2 m \zeta} \times \frac{\int_0^L \phi dy}{\int_0^L \phi^2 dy} \quad (6-8a)$$

Equation 6-8a implies that if the mode shape ϕ were anti-symmetric, as for a complete sine wave for example, then the response would be zero. In fact, it is found that the rotation of the vortices often is able to switch sign when the mode shape changes sign and a response still occurs. Therefore, recognizing this we replace ϕ in the numerator of

Figure 6.1 VMS lift power spectrum for Models 7, 8, and 9

Equation 6-8a by its absolute value. Also, we can group the variables into non-dimensional parameters that help in assessing when vortex excitation might be a problem.

$$\frac{|q_0|}{c} = \frac{C'_L}{16\pi^2} \left(\frac{U}{N_v c} \right)^2 \left(\frac{\rho c^2}{m \zeta} \right) \times \frac{\int_0^L |\phi| dy}{\int_0^L \phi^2 dy} \quad (6-8b)$$

The parameter $\frac{4\pi m \zeta}{\rho c^2}$ has been named the Scruton number S_c after Scruton, who first identified it as a key parameter by which to assess the susceptibility of a structure to vortex excitation. It has also often been expressed as $\frac{2m\delta}{\rho c^2}$ where δ = the logarithmic decrement, which equals $2\pi\zeta$ for the low values of ζ of interest in wind engineering. The higher the Scruton number the less will be the vortex excitation amplitude. Also, from Equation 6-8b, the parameter $\frac{N_v c}{U}$ is the Strouhal number, S_t . Therefore Equation 6-8b can be expressed as

$$\frac{|q_0|}{c} = \frac{C'_L}{4\pi S_t^2 S_c} \times \frac{\int_0^L |\phi| dy}{\int_0^L \phi^2 dy} \quad (6-8c)$$

From the power spectra of the fluctuating lift obtained on the VMS signs with $d/c = 0.7$ a peak in the excitation spectrum was noted at a non-dimensional frequency $fc/U = 0.10$. This may be identified as the Strouhal number for the VMS signs. From the spectra included in Figure 6.1, the area under the peak was measured and the following lift coefficients determined:

Table 6.1 Lift coefficients determined from spectra

b/c	S_t	S_c	C'_L
1	0.10	4.9	0.035
3	0.10	4.9	0.046
5	0.10	4.9	0.052

In estimating the Scruton Number S_c the damping ratio was taken as 0.005, the vertical dimension as 1.8 m and the weight per unit length of the VMS as 201 kg/m. The weight of the truss was assumed to be approximately 217 kg/m. As suggested by Ginal (2003), the entire mass of the VMS and 50% of the mass of the truss was used in the calculation. With knowledge of the natural frequency and the inputs in the above table one may then estimate the amplitude of oscillation of the VMS due to vortex excitation using Equation 6-8c. The ratio of integrals in Equation 6-8c was taken to be that for a sinusoid, i.e. $4/\pi \approx 1.27$.

Based on the above calculations, a sign with $c = 1.8$ m and $b = 9.1$ m is predicted to experience vertical sinusoidal oscillations due to vortex shedding with amplitude of about 203 mm. The predicted 3 second gust speeds where oscillations would start to build to the 203 mm amplitude are dependent on the natural frequency of the sign structure in the vertical direction and are plotted in Figure 6.2. Vortex oscillations take some time to build up and the gust speeds plotted here are used only as an indicator of when the accompanying sustained speeds are sufficient to generate oscillations.

These results are for $d/c = 0.7$ which is expected to be the worst case, since it has most horizontal area for the vortices to act on. While lift spectra were not obtained for the other d/c values, the vertical excitation forces are expected to be reduced roughly in

proportion to the plan area of the sign, i.e. for $d/c = 0.4$ an amplitude of approximately 116 mm would be expected. At $d/c=0.1$ the amplitude is expected to reduce further but the plane area of the support structure would then be comparable to that of the sign so a straight proportional reduction is probably no longer valid.

Figure 6.2 Critical gust speeds in winds that will cause vortex shedding oscillations for $c = 1.8$ m

From the above discussion it can be concluded that significant vortex induced oscillations are possible of VMS with 1.8 m vertical dimension within wind speed ranges of concern if the natural frequency is low.

6.2 Galloping tests

Galloping in the lateral direction for VMS with $d/c = 0.7$ was investigated. Figure 6.3 is a graph showing the variation of C_L (vertical lift coefficient) and C_D (normal

Figure 6.3 Graph of C_D and C_L versus angle of approach

force coefficient) as a function of the wind angle of attack (α). The graph shows that the slope of C_L is negative in the range $-4.5^\circ < \alpha < 4.5^\circ$ for both wind speeds. For the 15 m/s wind speed, the slope of C_L ($dC_L/d\alpha$) is -2.31 and $C_D = 1.19$ at $\alpha = 0^\circ$. Evaluation of the Den Hartog criterion for the 15 m/s wind speed yields

$$\left[\frac{dC_L}{d\alpha} + C_D < 0 \right]_{\alpha=0} = -1.12 \quad (6-9)$$

This suggests the model, as configured, is susceptible to galloping. A similar calculation for the 40 m/s wind speed shows that ($dC_L/d\alpha$) is -2.32 and $C_D = 1.22$ at $\alpha = 0^\circ$. Evaluation of the Den Hartog criterion for the 40 m/s wind speed yields

$$\left[\frac{dC_L}{d\alpha} + C_D < 0 \right]_{\alpha=0} = -1.10 \quad (6-10)$$

This results also suggests that the model, as configured, is susceptible to galloping.

The critical galloping onset wind velocity (U_{crit}) for a typical span truss VMS support structure was calculated. Starting with the vertical force per unit length being given by

$$Z = \frac{1}{2} \rho U^2 c \frac{dC_Z}{dC_\alpha} \alpha \quad (6-11)$$

and

$$\alpha = -\frac{\dot{q}}{U} \quad (6-12)$$

Therefore

$$Z = -\frac{1}{2} \rho U^2 c \frac{dC_Z}{dC_\alpha} \frac{\dot{q}}{U} \quad (6-13)$$

Assuming the vertical mode of vibration for the structure can be modeled as a SDOF oscillator, the equation of motion including the aerodynamic force in the vertical direction can be written

$$M_G(\ddot{q} + 2\omega_0\zeta\dot{q} + \omega_0^2q) = - \int_{sign} \frac{1}{2}\rho U^2 b \frac{dC_Z}{d\alpha} \cdot \frac{\dot{q}}{U} \phi^2 dy \quad (6-14)$$

Rearranging the equation and combining the right hand term with the $\dot{q}$ term on the left yields

$$M_G(\ddot{q} + 2\omega_0\dot{q}\left(\zeta + \frac{\rho U \int_{sign} b \frac{dC_Z}{d\alpha} \phi^2 dy}{4\omega_0 M_G}\right) + \omega_0^2 q) = 0 \quad (6-15)$$

The $dC_Z/d\alpha$ term is assumed to be zero along parts of the structure where the VMS is not present. Therefore the integral is only evaluated over the part of the structure where the VMS is present. From Equation 6-15, the total damping of the system becomes

$$\zeta_{total} = \zeta + \frac{\rho U \int_0^b c \frac{dC_Z}{d\alpha} \phi^2 dy}{4\omega_0 M_G} \quad (6-16)$$

Where ζ is the structural damping ratio; ρ the density of air; b is the length of the VMS; c is the height of the VMS; $dC_Z/d\alpha$ is the slope of C_Z at 0; ϕ^2 is the modal deflection shape; ω_0 is the natural circular frequency and M_G is the generalized mass.

At critical velocity the total damping of the system is zero,

$$\zeta + \frac{\rho U_{crit} \int_0^b c \frac{dC_Z}{d\alpha} \phi^2 dy}{4\omega_0 M_G} = 0 \quad (6-17)$$

Where U_{crit} is the onset wind velocity for galloping. Solving Equation 6-17 for the critical velocity yields

$$U_{crit} = - \frac{4\zeta\omega_0 M_G}{\rho \int_0^b c \frac{dC_Z}{d\alpha} \phi^2 dy} \quad (6-18)$$

Since the mass is assumed constant along the length of the VMS, U_{crit} becomes

$$U_{crit} = - \frac{4\zeta\omega_0 M_G}{\rho b c \frac{dC_Z}{d\alpha} \phi^2} \text{ for } \frac{dC_Z}{d\alpha} < 0 \quad (6-19)$$

Equation 6-19 was used to estimate the critical galloping onset wind velocity for a typical VMS support structure. The structure consisted of a 3-chord steel truss 1.8 m high x 1.5 m deep with a horizontal truss span of 30.5 m. The weight of the truss was increased 5% to account for the additional weight of connections. A 9.1 m long x 1.8 m high x 1.2 m deep walk-in style VMS was assumed for this calculation. The mass of the VMS was assumed to be 201 kg/m and the mass of the truss (including a catwalk and 5% additional for connections) was assumed to be 217 kg/m. As discussed in Section 6.1, the entire mass of the VMS and 50% of the mass of the truss was used in the critical onset velocity calculation. Damping ratios (ζ) from 0.0025 to 0.0075 and natural frequencies (f) from 0.8 to 3.0 were used to cover a wide range of conditions.

Using Equation 6-19 and the parameters in Table 6.2 with $dC_z/d\alpha = -1.12$, the critical galloping onset wind velocity (U_{crit}) was calculated and graphed in Figure 6.4.

Table 6.2 Parameters for critical galloping onset wind velocity calculation

ρ (kg/m ³)	c (m)	b (m)	Span (m)	m_{VMS} (kg/m)	m_{Truss} (kg/m)	ϕ	ζ (%)	f (Hz)
1.227	1.8	9.1	30.5	201	217	0.9	0.25 - 0.75	0.8 - 3.0

Figure 6.4 Critical galloping onset wind velocity

The U_{crit} calculated for this research are lower than the velocities determined for two span sign structures by Ginal (2003). The details and geometry of Ginal (2003) structures are discussed in Section 2.2.2 but the results are repeated below for reference:

$$98 \text{ m/s} \leq U_{crit} \leq 133 \text{ m/s} \quad (f = 2.4 \text{ Hz}; \zeta = 0.004) \quad (6-20a)$$

$$212 \text{ m/s} \leq U_{crit} \leq 289 \text{ m/s} \quad (f = 6.3 \text{ Hz}; \zeta = 0.004) \quad (6-20b)$$

The lower critical wind speed result for the WOW study was anticipated since according to Parkinson (1963) there is an increased potential for shapes with a longer afterbody length to gallop. This occurs because the afterbody length interferes with the vortex formation in the downstream wake region. Flow that separates from the windward edges can reattach along one side of the afterbody length which leads to an asymmetric surface pressure distribution and a net force in the z -direction. Since the resulting galloping instability is a function of the afterbody length, the results of this study based on $d/c = 0.7$ is expected to be lower than Foley et al. (2004) which was based on $d/c =$

0.5. Since the critical galloping onset wind velocity for structures with low natural frequency and low critical damping is in the range of likely wind speeds experienced in many areas of the country, it is suggested that galloping potential be investigated further and considered in the design of VMS structures.

6.3 Buffeting and Aerodynamic Damping

Figure 6.5 shows a horizontally aligned long, line-like, flexible structure of length L , width b_l , vertical depth d_l , and mass per unit length $m(y)$, with wind of uniform mean velocity normal to its length. To begin with we assume that b_l and d_l are very small relative to L ; hence term “line-like”. Later we will deal with the fact that they may not be small relative to L . Although the instantaneous turbulence velocities at different points are different, statistically the turbulence is homogeneous along the span. We will make the quasi-steady assumption that the fluctuating wind loads can be determined from the aerodynamic force coefficients measured in steady flow.

Figure 6.5 Flexible line-like structure

We consider forces and motions in the x direction, which is aligned with the mean wind direction.

The deflection $x(y,t)$ of the structure in each of its natural modes of vibration may be expressed as

$$x(y,t) = \phi(y)q(t) \quad (6-21)$$

where $\phi(y)$ = deflection shape of the mode of vibration, and $q(t)$ = generalized coordinate for the mode of vibration. It can then be shown that the equation of motion in a particular mode of vibration, that has frequency ϖ_0 and damping ratio ζ , is

$$M_G(\ddot{q} + 2\varpi_0\zeta\dot{q} + \varpi_0^2q) = F_G(t) \quad (6-22)$$

where

$$M_G = \text{generalized mass} = \int_0^L m(y)\phi(y)^2 dy \quad (6-23)$$

and

$$F_G = \text{generalized aerodynamic force} = \int_0^L f(y,t)\phi(y)dy \quad (6-24)$$

where m = mass per unit length and f = force per unit length. Using the quasi-steady assumption, and ignoring second order effects, the equation for the fluctuating aerodynamic force in the along wind direction may be expressed as

$$f(y,t) = \rho U C_{F_x} d_1 (u(y,t) - \dot{x}(y,t)) \quad (6-25)$$

where C_{F_x} = aerodynamic drag coefficient. Therefore the generalized aerodynamic force may be written

$$F_G = \int_0^L \rho U C_{Fx} d_1 (u(y, t) - \phi(y) \dot{q}(t)) \phi(y) dy \quad (6-26)$$

The term involving $\dot{q}$ may be taken over to the left hand side of Equation 6-22 and combined with the damping term already there. The Equation of motion may then be written as

$$M_G \left(\ddot{q} + 2\varpi_0 \left[\zeta + \frac{\rho U \int_0^L C_{Fx} d_1 \phi^2 dy}{2\omega_0 M_G} \right] \dot{q} + \varpi_0^2 q \right) = \rho U \int_0^L C_{Fx} d_1 \phi u dy \quad (6-27)$$

In this equation it can be seen that the left hand side is the same as that of Equation 6-22 except that the total damping ratio is now given by the structural damping ratio, ζ , plus the aerodynamic damping ratio, ζ_a : i.e.

$$\zeta_{tot} = \zeta + \zeta_a \quad (6-28)$$

where

$$\zeta_a = \frac{\rho U \int_0^L C_{Fx} d_1 \phi^2 dy}{2\omega_0 M_G} \quad (6-29a)$$

Note that for the case where C_{Fx} and d are constant along the span, and where the mass per unit length m is also constant, this relationship reduces to

$$\zeta_a = \frac{\rho U C_{Fx} d_1}{2\omega_0 m} \quad (6-29b)$$

Using random vibration theory Equation 6-27 may be used to derive the power spectrum of generalized deflection of the structure.

$$S_q(n) = \frac{(\rho U)^2}{M_G^2 \omega_0^4} \left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right|^2 S_u(n) \int_0^L \int_0^L C_{Fx} C'_{Fx} d_1 d'_1 \phi \phi' \bar{S}_{uu}(n, y, y') dy dy' \quad (6-30)$$

in which $\left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right|$ is the mechanical admittance given by

$$\left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right| \equiv \sqrt{\frac{1}{\left[1 - \left(\frac{n}{n_0}\right)^2\right]^2 + 4\zeta_{tot}^2 \left(\frac{n}{n_0}\right)^2}} \quad (6-31)$$

$\bar{S}_{uu}(n, y, y')$ is the cross-spectrum of the velocity fluctuations at locations y and y' along the span, divided by the power spectrum, S_u (which, like the mean velocity and all turbulence statistical properties, is assumed to be homogeneous along the span).

Equation 6-30 was derived on the basis that the dimensions b_1 and d_1 were so small that the turbulence velocity u remains perfectly correlated over these dimensions. In many situations this assumption is somewhat approximate and introduces too much conservatism. To overcome this, a correction can be made by introducing a two dimensional aerodynamic admittance function $|\chi_{2D}(n, b_1, d_1)|$ that models the lack of correlation over the dimensions b_1 and d_1 . With this correction term the power spectrum of deflection becomes

$$S_q(n) = \frac{(\rho U)^2}{M_G^2 \omega_0^4} \left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right|^2 S_u(n) \int_0^L \int_0^L C_{Fx} C'_{Fx} d_1 d'_1 \phi \phi' \bar{S}_{uu}(n, y, y') |\chi_{2D}(n, b_1, d_1)|^2 dy dy' \quad (6-32)$$

Allowing for the fact that C_x , b_1 and d_1 may vary along the span, Equation 6-32 may be written as

$$S_q(n) = \frac{(\rho U C_{Fx,ref} d_{1ref} L)^2}{M_G^2 \omega_0^4} \left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right|^2 \left| \chi_y(n, L) \right|^2 \left| \chi_{2D}(n, b_{1ref}, d_{1ref}) \right|^2 S_u(n) \quad (6-33)$$

where $C_{Fx,ref}$, b_{1ref} and d_{1ref} are reference values at a selected location along the span, and

$$\left| \chi_y(n, L) \right|^2 \left| \chi_{2D}(n, b_{1ref}, d_{1ref}) \right|^2 = \frac{1}{L^2} \int_0^L \int_0^L \left(\frac{C_{Fxo} C'_{Fxo}}{C_{Fx,ref}^2} \right) \left(\frac{d_1 d'_1}{d_{1ref}^2} \right) \frac{\left| \chi_{2D}(n, b_1, d_1) \right|^2}{\left| \chi_{2D}(n, b_{1ref}, d_{1ref}) \right|^2} \phi \phi' \bar{S}_{uu}(n, y, y') dy dy' \quad (6-34)$$

Also the aerodynamic damping ratio may be written

$$\zeta_a = \frac{\rho U C_{Fx,ref} d_{1ref} \int_0^L \frac{C_{Fx}}{C_{Fx,ref}} \frac{d_1}{d_{1ref}} \phi^2 dy}{2 \omega_0 M_G} \quad (6-35)$$

The variance of generalized deflection may be evaluated by integrating Equation 6-34 over all frequencies. For a lightly damped structure this integration can be simplified considerably by noting that nearly all the contribution to the integral comes from a very narrow range of frequencies close to the natural frequency. This a consequence of the form of the mechanical admittance term $\left| H\left(\frac{n}{n_0}, \zeta_{tot}\right) \right|^2$. As a result we may express the integration of Equation 6-33 as

$$\sigma_q^2(n) = \frac{(\rho U^2 C_{Fx,ref} d_{1ref} L I_u)^2}{M_G^2 \omega_0^4} \left(\int_0^\infty \left| \chi_y(n, L) \right|^2 \left| \chi_{2D}(n, b_{1ref}, d_{1ref}) \right|^2 S_u(n) dn + \left| \chi_y(n_0, L) \right|^2 \left| \chi_{2D}(n_0, b_{1ref}, d_{1ref}) \right|^2 \frac{n_0 S_u(n_0)}{\sigma_u^2} \frac{\pi}{4 \zeta_{tot}} \right) \quad (6-36)$$

6.3.1 Gust Factor Calculation of ASCE 7-10

The general form of Equation 6-36 is reflected in the detailed gust factor calculation for along wind response in ASCE 7-10, with the integral term being the background factor and the last term being the resonant contribution. Empirical expressions have been used in ASCE 7 to account for the aerodynamic admittance terms and simplify the calculation in a convenient fashion suitable for a building code or standard. However, as part of the simplification process, and as a conservative approximation, the aerodynamic contribution to total damping ratio has been omitted in ASCE 7. Although this is a reasonable approach for most large structures, for flexible sign structures this can lead to significant over-estimates of the gust factor.

Therefore, in order to estimate the gust factor for the VMS signs the ASCE 7-10 procedure can be adopted with one modification, which is to add the aerodynamic damping to the structural damping. As can be seen from Equation 6-29a the aerodynamic damping depends on the deflection shape ϕ and generalized mass of the mode of vibration. In general these will be specific to each sign structure, including the wind drag and mass not only of the sign itself but also of the truss or other structure supporting the sign. However, to provide rough estimates of the contribution of aerodynamic damping to total damping in the present study the following simplifying assumptions have been made.

1. The generalised mass of the entire sign structure, including the sign, is assumed to be the mass of the VMS and 50% of the mass of the truss.
2. The sign is assumed to be located at the point of maximum deflection.

3. The aerodynamic drag forces on the support structure are ignored for the calculation of aerodynamic damping.

The effects of these three assumptions are expected to result in a conservative estimate of the aerodynamic damping.

6.3.2 Estimates of gust factor based on ASCE 7-10 with aerodynamic damping included

Using the ASCE 7-10 formulae for the gust factor, but including the aerodynamic damping term as described above (Equation 6-29b), gust factors were estimated for a selection of sign geometries and weights. These estimates are provided here to give an indication of the sensitivity of the gust factor to size, natural frequency, weight, and wind velocity. The signs selected were as follows.

1. Width $b = 9.1$ m, height $c = 2.4$ m, depth $d = 1.2$ m. Two weights were examined, 201 kg/m and 104 kg/m.
2. A large sign with $b = 15.2$ m, $c = 3.7$ m and $d = 1.8$ m. Two weights were examined, 335 kg/m and 179 kg/m.
3. A small sign with $b = 3.7$ m, $c = 1.8$ m and $d = 0.9$ m. Two weights were examined, 112 kg/m and 60 kg/m.

Figures 6-6 through 6-11 show the behavior of the gust factor plotted versus 3 second reference gust speed for various natural frequencies. The exposure was taken as open terrain (Exposure C of ASCE 7) and the reference wind speed was the standard speed at 10 m height in open terrain. Appropriate adjustments were made to the speed to determine the speed at the mid height of the sign. A representative drag coefficient of

1.25 was used in the calculations. The structural damping ratio was taken to be 0.005 but tended to be far outweighed by the aerodynamic damping at high wind speeds. As a result the gust factor calculations were not overly sensitive to the assumed structural damping.

It can be seen in Figures 6-6 to 6-11 that these rough estimates show a wide range of possible gust factors, some below the existing 1.14 value in AASHTO and some above. For the mid-sized sign at heavier weight, Figure 6-6, and for wind speeds above 45 m/s, the gust factor G ranges from about 1.07 for a fairly stiff sign with frequency 3 Hz to about 1.34 for a very flexible sign with a low frequency of 1 Hz. For the light-weight mid-sized sign, Figure 6-7, the aerodynamic damping is relatively higher. This reduces the gust factor so that at speeds above 45 m/s it ranges from about 1.00 for the stiffest case (frequency = 3 Hz) to 1.18 for the most flexible (frequency = 1 Hz). The larger sign sees generally lower gust factors, ranging from about 0.95 to 1.24 for speeds above 45 m/s, Figures 6-8 and 6-9. The smallest sign sees the highest gust factors ranging from about 1.11 up to 1.43 at speeds above 45 m/s (Figures 6-10 and 6-11).

Although the scope of the current studies was focused on aerodynamic drag coefficient of VMS signs this preliminary assessment of the gust factor allows some conclusions to be drawn on this other important factor that forms part of the aerodynamic drag calculation. It indicates that gust factors in excess of the standard 1.14 value assumed by AASHTO are possible for the more flexible sign structures. A future, more detailed study of gust factor would be warranted in view of these results.

Figure 6.6 Estimated gust factor for mid-sized sign (heavier weight)

Figure 6.7 Estimated gust factor for mid-sized sign (lighter weight)

Figure 6.8 Estimated gust factor for large sign (heavier weight)

Figure 6.9 Estimated gust factor for large sign (lighter weight)

Figure 6.10 Estimated gust factor for small sign (heavier weight)

Figure 6.11 Estimated gust factor for small sign (lighter weight)

7 DISCUSSIONS AND RECOMMENDATIONS

7.1 AASHTO recommendations

One objective of this study was to determine accurate drag coefficients for VMS that could be incorporated into the AASHTO Specification. The matrix shown in Table 7.1 is the synthesis of the drag coefficient results from this research. Since no significant difference was noted in the results for the tested fatigue level wind speed of 15 m/s and the extreme event wind speed of 40 m/s, a single design matrix is suggested for use in both fatigue and strength design. The C_{Fx} values are based on the results obtained for the 40 m/s wind speed in the 0° horizontal wind direction since these values were slightly larger than the values obtained for the 15 m/s wind speed and the 45° wind direction. The one noted exception is the value for $b/c = 1$ with $d/c = 0.1$. The value of $C_{Fx} = 1.21$ is a rounded value that takes into account the large result obtained for the geometric configuration of Model 1 in the 45° horizontal wind direction. Intermediate values for $b/c = 2$ and 4 and $d/c = 0.2, 0.3, 0.5$, and 0.6 were interpolated from actual tested geometric configurations.

Table 7.1 Drag Coefficient design matrix for C_{Fx}

Aspect Ratio (b/c)	Depth Ratio (d/c)						
	0.1	0.2	0.3	0.4	0.5	0.6	0.7
1.0	1.21	1.18	1.15	1.12	1.08	1.04	0.98
2.0	1.21	1.20	1.18	1.17	1.14	1.11	1.09
3.0	1.22	1.22	1.22	1.21	1.20	1.18	1.16
4.0	1.25	1.25	1.24	1.24	1.23	1.22	1.21
5.0	1.28	1.27	1.26	1.25	1.24	1.23	1.22

The maximum value of the eccentricity ratio (r/b) obtained in the WOW study was 0.13. This value is in good agreement with the r/b ratios obtained by Smith et al. (2014) which were less than 0.15. Based on these results, an r/b value of 0.15 is recommended for use in design. However, it is important to note that the current AASHTO Specification (2013) does not include a recommendation for r/b .

Another objective of this study was to estimate gust effect factors and recommend drag force evaluation methodology. Although the findings of the study are limited, preliminary results indicate that gust factors in excess of the AASHTO Specification (2013) suggested value of 1.14 are possible for flexible sign structures. Additional research is suggested to explore this finding in more detail to see if a change to the current 1.14 value is warranted.

The dynamic analysis conducted as part of this research indicates that VMS signs with a large depth ratio attached to structures with a low natural frequency may be susceptible to vortex shedding and galloping when subjected to a range of likely expected wind speeds. Based on these findings, it is suggested that the AASHTO design procedure be expanded to evaluate VMS structures for vortex shedding and galloping potential.

7.2 Impact of Recommendations – Case Studies

Three case studies are presented to illustrate the impact of the newly proposed drag coefficients obtained in the FIU WOW research. A typical tri-chord overhead span sign structure was analyzed using the current AASHTO drag coefficient ($C_{Fx} = 1.7$), the FIU WOW representative drag coefficient ($C_{Fx} = 1.25$), and the drag coefficient obtained in

the modified corner test for chamfered corners ($C_{Fx} = 1.00$). The VMS and basic structure dimensions are shown in Table 7.2. The sign structure configuration and design is based on the FDOT Design Standard Index Drawing 11320 which is included in Appendix B along with the corresponding detailed dimensions and geometry.

The FDOT Span Overhead Sign Program V6.0 was used to analyze member stresses and fatigue limits in accordance with the AASHTO Specification (2013). The program uses PC-SAP IV developed by Klaus-Jurgen Bathe, Edward L. Wilson, and Fred E. Petersen of the University of California at Berkley to analyze the structure as a 3D space frame. The horizontal span was analyzed as a tri-chord truss composed of three members forming an equilateral triangle (Figure 1.3b shows a picture of a typical tri-chord structure). A mounting height or wind height of 7 m was used based on the VMS vertical height of 2.4 m and a minimum vertical clearance of 6 m. This corresponds to a clearance ratio of $c/h = 0.24$. The dimensions of the walk-in style VMS were 9.1 x 2.4 x 1.2 m which corresponds to geometric ratios of $b/c = 4$ and $d/c = 0.5$. The VMS was offset 13.7 m from the right upright. A 2.4 x 1.2 m steel walkway was also included since this is typically utilized with VMS structures of this size. The VMS was mounted on the two chord face of the truss and attached with seven vertical hangers spaced at 1.3 m. The weight of the hangers and associated connectors was included in the weight of the VMS (81.5 kg/m^2). All forces for the VMS and the walkway were applied at the location where the vertical hangers attach to the truss. Web members of the truss were considered to be pinned and only capable of resisting axial loads. The truss chords were considered to be continuous with a constant cross section.

The structure was assumed to be located in Miami Dade County and the applicable AASHTO Specification (2013) and Florida Department of Transportation Structure Design Guidelines (2014) design wind speed of 67 m/s (150 mph) was applied for estimation of ultimate wind loading on all members. Equation 3-1 was used to calculate the strength design wind pressure. The structure was also analyzed for fatigue loads due to natural wind gusts and truck induced gusts only. The equivalent static natural wind gust pressure was calculated using Equation 3-5 and the equivalent static pressure range for truck gusts was calculated using Equation 3-6. Galloping and vortex shedding were not included in this analysis since they are not required to be considered in the design of tri-chord span trusses in the current AASHTO Specification (2013). Consistent with the AASHTO Specification (2013) for fatigue, the program generates up to eight static load cases to account for wind directions and placement of the VMS. The load cases are evaluated using the SAP 3D frame analysis component. Individual member and component stresses were compared to the established AASHTO Constant Amplitude Fatigue (CAF) thresholds. A printout of the input data and program output for all three case studies is included in Appendix B.

A summary of the combined stress ratio (CSR) results and the maximum reactions at the base of the uprights is provided in Table 7.3. A summary of the generated fatigue stress is provided in Table 7.4. Structure members were first sized using $C_{Fx} = 1.7$ so that comparative stress reductions could be shown for $C_{Fx} = 1.25$ and $C_{Fx} = 1.00$. As indicated in the Tables, the CSR's and fatigue stresses are below the acceptable limits for all values of C_{Fx} . Tables 7.3 and 7.4 also show the percentage decrease in CSR, maximum reactions, and fatigue stress from $C_{Fx} = 1.70$ to 1.25, $C_{Fx} = 1.25$ to 1.00, and $C_{Fx} = 1.70$ to

1.00. The data is shown graphically in Figures 7.1, 7.2, and 7.3. As indicated, the average decrease in CSR was 20% from $C_{Fx} = 1.70$ to 1.25 and 30% for $C_{Fx} = 1.70$ to 1.00. The average decrease in the reactions at the base of the upright was 12% for $C_{Fx} = 1.70$ to 1.25 and 18% for $C_{Fx} = 1.70$ to 1.00. The largest reduction was for the normal moment which was 18% and 30% for $C_{Fx} = 1.70$ to 1.25 and $C_{Fx} = 1.70$ to 1.00 respectively.

The results demonstrate a significant reduction in the fatigue stresses as well. In particular, the average decrease in the chord member connection was 22% and 33% for $C_{Fx} = 1.70$ to 1.25 and $C_{Fx} = 1.70$ to 1.00 respectively. The largest decrease was noted for the gusset plates. The average fatigue stress decrease for $C_{Fx} = 1.70$ to 1.25 was 25% and the average decrease for $C_{Fx} = 1.70$ to 1.00 was 38%.

Based on these results, a cost savings maybe realized if a reduction in C_{Fx} permits the use of a smaller member size. A substantial cost savings may also be realized if the reduction in stress due to the use of a smaller drag coefficient permits an existing structure supporting a flat panel sign to be retrofitted with a VMS instead of requiring a new structure to be designed and constructed. A significant reduction in the drag coefficient and corresponding reduction in stress was noted for the chamfered edge case. This minor modification in manufacturing may also result in substantial cost savings and design advantages.

Table 7.2 VMS geometry

Case Study VMS Structure Geometry	
Truss Length	30.5 m
Left Truss Height	7.0 m
Right Truss Height	7.0 m
Number of Truss Panels	13 @ 2.34 m Total length 30.5 m
Vertical Truss Depth	1.8 m
Horizontal Truss Depth	1.6 m
Chord Member	168 mm O.D. Pipe 11 mm Wall
Web Member	Angle 102 x 102 x 9.5 mm
Left Upright Member	610 mm O.D. Pipe 19 mm Wall
Right Upright Member	610 mm O.D. Pipe 19 mm Wall
Camber	25 mm

Table 7.3 CSR and maximum reactions

Member Description	Unit	C _{Fx}			% Decrease in C _{Fx}		
		1.70	1.25	1.00	1.70 to 1.25	1.25 to 1.00	1.70 to 1.00
Combined Stress Ratio Check							
CSR_Chord	n/a	0.595	0.457	0.384	23%	16%	35%
CSR_Web	n/a	0.382	0.308	0.268	19%	13%	30%
CSR_Left_Upright	n/a	0.420	0.347	0.307	17%	12%	27%
CSR_Right_Upright	n/a	0.480	0.393	0.345	18%	12%	28%
Average % Decrease					20%	13%	30%
Max. Reaction at Base of Left Upright							
Left_Long_Moment	kN-m	491.15	401.89	352.40	18%	12%	28%
Left_Long_Shear	kN	70.33	57.65	50.62	18%	12%	28%
Left_Trans_Moment	kN-m	169.59	146.77	134.11	13%	9%	21%
Left_Trans_Shear	kN	53.91	48.53	45.55	10%	6%	16%
Left_Axial_Load	kN	69.79	69.21	68.90	1%	0%	1%
Left_Axial_Torque	kN-m	34.66	31.21	29.29	10%	6%	16%
Average % Decrease					12%	8%	18%
Max. Reaction at Base of Right Upright							
Right_Long_Moment	kN-m	572.16	463.58	403.36	19%	13%	30%
Right_Long_Shear	kN	81.85	66.32	57.74	19%	13%	29%
Right_Trans_Moment	kN-m	165.86	143.27	130.84	14%	9%	21%
Right_Trans_Shear	kN	53.87	48.53	45.55	10%	6%	15%
Right_Axial_Load	kN	73.35	72.77	72.46	1%	0%	1%
Right_Axial_Torque	kN-m	34.65	31.19	29.29	10%	6%	15%
Average % Decrease					12%	8%	19%

Table 7.4 Fatigue stress check

Fatigue Stress Check		C _{Fx}			% Decrease in C _{Fx}		
		1.70	1.25	1.00	1.70 to 1.25	1.25 to 1.00	1.70 to 1.00
Left Base Connection							
Left_Upright_Fatigue_Check	MPa	1.61	1.38	1.26	14%	9%	22%
Left_Base_Fatigue_Check	MPa	1.03	0.89	0.81	14%	9%	21%
Left_Anchor_Bolts_Fatigue_Check	MPa	3.39	2.92	2.66	14%	9%	22%
Average % Decrease					14%	9%	22%
Right Base Connection							
Right_Upright_Fatigue_Check	MPa	1.67	1.4	1.25	16%	11%	25%
Right_Base_Fatigue_Check	MPa	0.99	0.9	0.8	9%	11%	19%
Right_Anchor_Bolts_Fatigue_Check	MPa	2.83	2.92	2.66	3%	9%	6%
Average % Decrease					9%	10%	17%
Chord Member Connection							
Chord_Splice_Fatigue_Check	MPa	1.82	1.47	1.28	19%	13%	30%
Chord_Slot_Fatigue_Check	MPa	0.37	0.28	0.24	24%	14%	35%
Half_Chord_Fatigue_Check	MPa	0.29	0.23	0.19	21%	17%	34%
Left_Truss_Bolts_Fatigue_Check	MPa	0.14	0.11	0.1	21%	9%	29%
Right_Truss_Bolts_Fatigue_Check	MPa	0.14	0.11	0.09	21%	18%	36%
Average % Decrease					21%	14%	33%
Gusset Plates							
Back_Plates_Fatigue_Check	MPa	0.06	0.04	0.03	33%	25%	50%
Back_End_Plates_Fatigue_Check	MPa	0.27	0.212	0.19	21%	10%	30%
Front_Plates_Fatigue_Check	MPa	0.09	0.07	0.06	22%	14%	33%
Front_End_Plates_Fatigue_Check	MPa	0.28	0.21	0.18	25%	14%	36%
Top_Center_Plate_Fatigue_Check	MPa	0.2	0.15	0.12	25%	20%	40%
Bot_Center_Plate_Fatigue_Check	MPa	0.19	0.15	0.12	21%	20%	37%
Average % Decrease					25%	17%	38%
Web Members							
Web_Member_Fatigue_Check	MPa	0.83	0.63	0.52	24%	17%	37%

Figure 7.1 Graph of fatigue stress results

Figure 7.2 Graph of CSR results

Figure 7.3 Graph of reaction results

8 CONCLUSIONS AND FUTURE RESEARCH

8.1 Summary of findings

The increasing national interest in ITS and the need for more efficient transportation systems have led to an expanding use of VMS technology. This necessitated an evaluation of the current design criteria and code guidance for VMS. The FIU WOW study investigated wind effects on VMS, focusing specifically on wind induced drag and the response of VMS. The research included a comprehensive literature review, investigation of industry specifications, testing of large scale VMS models, extensive data analysis, and discussion of results.

Drag coefficients were obtained for thirteen models at wind speeds of 15 m/s and 40 m/s in horizontal wind directions of 0° and 45°. Based on the extensive large scale testing in the WOW, several conclusions were drawn regarding wind induced drag on VMS. The drag coefficient results in the FIU WOW study are generally in good agreement with the results obtained in full scale studies such as the ones conducted by Quinn et al. (2001) at Silsoe and Smith et al. (2014) at the Reese Technology Center in Texas. A comparison of results for full scale and small scale drag coefficients shows that the values obtained in small scale wind tunnel studies were generally larger. It was suggested that this may be in part due to excessive energy content in the high frequency range of the wind tunnel ABL simulation. The WOW testing confirmed that the prismatic geometry of a VMS can significantly affect wind loading. A comparison of the FIU WOW results with earlier flat panel results suggests that the normal force coefficient for prismatic signs such as VMS is lower than the drag coefficients obtained for flat panel signs. Consequently, the FIU

WOW results are lower than the AASHTO Specification (2013) recommended value of 1.7.

Testing also revealed trends in the drag coefficients based on aspect ratio, depth ratio, and wind direction. It was demonstrated that the drag coefficient increases with increasing aspect ratio which is consistent with the results from Letchford (2001) and Zuo et al. (2014). It was also shown that the drag coefficient decreases with increasing depth ratio resulting in a reduced drag force and a smaller drag coefficient for VMS. Wind direction results for the 45° horizontal wind direction were considerable less than the results obtained in the 0° direction (except for Model 1 which is discussed in Section 5.1.3). This finding was consistent with the studies conducted by Zuo et al. (2014), Ginger (1998), and Paulotto (2006) but inconsistent with trends noted in the studies by Letchford (2001) and Quinn (2001).

The effect of corner modification was investigated by testing models with chamfered and round (radial) corners. Chamfered corner tests demonstrated an additional decrease in the drag coefficient resulting in drag coefficient value around $C_{Fx} = 1.00$. This minor modification in manufacturing could have a significant impact on VMS wind loading and lead to more efficient structure designs. Results from the round corner tests indicated a possible Reynolds number dependency.

The maximum value obtained for the non-dimensional eccentricity ratios for the 45° wind direction ($r/b = 0.13$) was lower than the ASCE 7-10 suggested value of 0.2. The FIU WOW results are in good agreement with the study conducted by Zuo et al. (2014) which also reported eccentricity ratios below 0.15. However, eccentricity is not currently considered in the AASHTO Specification (2013) design procedure.

Wind driven rain results were compared to the wind only results for the same model. Based on the limited testing conducted as part of this study, it does not appear that drag on VMS is affected by wind driven rain in the Reynolds number test range of 5.96×10^5 to 1.59×10^6 .

Vortex shedding of a typical VMS with $d/c = 0.7$ was investigated by analyzing the power spectra of the fluctuating lift measured in the WOW. Vertical oscillations with an amplitude of about 203 mm was predicted when 3 second gust speeds are as low as 26 m/s for structures with a natural frequency of 1 Hz. Since this is in the range of likely wind speeds, further experimental confirmation is suggested. Galloping instability of a model with $b/c = 5$ and $d/c = 0.7$ was investigated experimentally and analyzed using basic galloping theory. Evaluation of the Den Hartog criterion suggested that the model as configured is susceptible to galloping. Based on this finding, galloping is possible for VMS with a large depth ratio attached to a flexible structure with a low natural frequency. It is suggested that galloping potential for VMS structures should be investigated further. Gust factors were estimated for a selection of sign geometries and weights using the ASCE 7-10 formula including the aerodynamic damping term. This was done to provide an indication of the sensitivity of the gust factor to size, natural frequency, weight, and wind velocity. Results showed a wide range of possible gust factors, both above and below the current AASHTO Specification (2013) value of 1.14. Based on this preliminary assessment, gust factors in excess of 1.14 are possible for the more flexible sign structures and warrant a more detailed study.

An alternative to the AASHTO drag coefficient value of 1.7 was presented in the form of a design matrix (see Table 7.1). However a single representative value of $C_{Fx} = 1.25$ may be used. Three case studies were also presented to illustrate the impact of reducing the drag coefficient. A prototype tri-chord non-cantilever structure was analyzed using the current AASHTO recommended drag coefficient of 1.7. The generated stresses were compared to two other cases using drag coefficients of 1.25 and 1.00. A comparison of results showed that the stress at key locations was reduced significantly. Combined stress ratios were reduced as much as 20% for a drag coefficient reduction from $C_{Fx} = 1.7$ to 1.25 and almost 30% for a reduction from $C_{Fx} = 1.7$ to 1.0. The case studies further emphasize that the current AASHTO drag coefficient value of 1.7 may result in more costly and inefficient designs in some cases. However, the current AASHTO Specification (2013) does not provide guidelines for aeroelastic behavior which can lead to unsafe designs for very flexible VMS.

8.2 Recommendations for future research

Based on the findings of this study, future research and testing may include:

- Comprehensive large scale testing of additional VMS geometries and configurations. This should include a more inclusive range of horizontal wind directions (0° to 180°) and a representative range of clearance ratios.
- Finite element modeling of typical VMS structures using the new FIU WOW drag coefficients. This should include dynamic analysis to evaluate the susceptibility of VMS to aeroelastic instabilities.

- Additional large scale wind tunnel testing to evaluate the sensitivity of VMS structures to galloping and vortex shedding. Testing should include a larger range of attack angles and VMS geometries.
- Comprehensive wind tunnel testing of various corner modifications. Testing should evaluate various sizes and additional shape options. The use of rounded corners should be investigated further.
- The results for the square VMS in the 45° horizontal wind direction needs to be investigated in more detail. Although the large results for Model 1 in the 45° wind direction were obtained in preliminary testing and again during final testing, the magnitude of the difference with the 0° wind direction was not anticipated.
- Full scale sectional models of VMS structures should be tested in the wind tunnel to investigate fluctuating lift and shed more light on the potential for aeroelastic instabilities. Use of innovative dampers can be investigated at full scale to reduce potential aeroelastic instabilities.
- Further investigation of the gust effect factor should be carried out.
- Field measurements should be performed to facilitate comparison of wind induced responses to the wind tunnel results for both fatigue and ultimate conditions.

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APPENDICES

Appendix A – Load Cell Calibration Matrices

Multi-Axis Load Cell Technologies

FORCE-MOMENT SENSOR SPECIFICATION SHEET

JR3, Inc.
22 Harter Avenue
Woodland, CA 95776

(530) 661-3677
Fax (530) 661-3701
e-mail: jr3@jr3.com

75E20S4-M125D-AF 1350L

Serial Number 4541

Analog calibration.

	Electrical Load Settings	Sensor Load Ratings	Calibration Loads used
Fx	1,350 lbs	1,350 lbs	400 lbs
Fy	1,350 lbs	1,350 lbs	400 lbs
Fz	1,500 lbs	2,700 lbs	400 lbs
Mx	6,000 in-lbs	10,125 in-lbs	2,600 in-lbs
My	6,000 in-lbs	10,125 in-lbs	2,600 in-lbs
Mz	3,000 in-lbs	10,125 in-lbs	2,600 in-lbs

Calibration Matrix: Multiply the calibration matrix and the sensor
Voltage vector to determine the loads. (lb and in-lbs)

267.26	-1.67	-0.62	-4.88	4.00	-1.68	Fx Volts
-0.78	269.98	1.41	-1.46	-14.34	-0.22	Fy Volts
2.68	-8.55	295.98	1.84	-0.02	-8.23	Fz Volts
23.09	-42.46	2.29	1202.55	3.23	2.24	Mx Volts
-2.98	26.82	-0.94	-0.73	1207.56	4.08	My Volts
-9.90	12.21	18.09	-2.13	-9.35	603.50	Mz Volts

Color	
Brn	Fx
R	Fy
O	Fz
Y	Mx
Grn	My
Blu	Mz
V(pink)	+12V (12 to 15)
Gry	-12V (12 to 15)
W	Power COMMON
Tan	Gnd, Signal Reference
Bare	Shield drain, not connected to sensor body

Final Inspection:

Calibration Date	3/8/09
Calibration Matrix	✓
Axis Orientation	✓
Units (lb and in-lbs)	✓
Hardware Correct	✓
Label Correct	✓
Functional Test	✓
Inspection Date	4/2/09
Inspector Initial

Multi-Axis Load Cell Technologies

FORCE-MOMENT SENSOR SPECIFICATION SHEET

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e-mail: jr3@jr3.com

75E20S4-M125D-AF 1350L

Serial Number **4542**

Analog calibration.

	Electrical Load Settings	Sensor Load Ratings	Calibration Loads used
Fx	1,350 lbs	1,350 lbs	400 lbs
Fy	1,350 lbs	1,350 lbs	400 lbs
Fz	1,500 lbs	2,700 lbs	400 lbs
Mx	6,000 in-lbs	10,125 in-lbs	2,600 in-lbs
My	6,000 in-lbs	10,125 in-lbs	2,600 in-lbs
Mz	3,000 in-lbs	10,125 in-lbs	2,600 in-lbs

Calibration Matrix: Multiply the calibration matrix and the sensor
Voltage vector to determine the loads. (lb and in-lbs)

261.17	0.23	-0.15	-6.46	2.69	-0.86	Fx Volts
-0.91	262.77	1.35	-0.93	-8.94	0.79	Fy Volts
8.73	-1.49	297.54	2.06	1.86	1.56	Fz Volts
-11.39	-29.62	0.43	1190.83	-6.66	-1.37	Mx Volts
44.62	-0.01	5.45	-8.27	1218.80	-1.01	My Volts
-8.19	7.77	13.29	-5.46	-7.61	592.36	Mz Volts

Color	
Brn	Fx
R	Fy
O	Fz
Y	Mx
Grn	My
Blu	Mz
V(pink)	+12V (12 to 15)
Gry	-12V (12 to 15)
W	Power COMMON
Tan	Gnd, Signal Reference
Bare	Shield drain, not connected to sensor body

Final Inspection:	Calibration Date	3/27/09
	Calibration Matrix	✓
	Axis Orientation	✓
	Units (lb and in-lbs)	✓
	Hardware Correct	✓
	Label Correct	✓
	Functional Test	✓
	Inspection Date	4/2/09
	Inspector Initial	JK

Appendix B – Case Studies Data

Span Sign Structure Dimensions – FDOT Standard Index 11320

Location	Dimension	Location	Dimension
A	100'	LH	3/16"
B	23'	RA	7/8"
C	23'	RB	4
D	13	RC	1/2"
E	72"	RD	1/2"
F	6.63" x 0.432"	RE	1/4"
G	4" X 4" X 3/8"	RF	1/4"
H	24" X 0.75"	RG	3/16"
J	24" X 0.75"	RH	3/16"
K	1"	BA	1 3/4"
SA	6" X 6" X 1/2"	BB	10
SB	6	BC	1 1/2"
SC	1"	BD	1/2"
PA	1 1/2"	BE	1' 10 1/2"
PB	2"	BF	5/16"
PC	3/8"	BG	5/16"
PD	7/16"	BH	5/16"
PE	1"	BJ	3/16"
PF	12	CA	1 3/4"
GA	1/2"	CB	10
GB	3/4"	CC	1 1/2"
GC	1' - 3 1/2"	CD	1/2"
GD	6 1/4"	CE	1' - 10 1/2"
GE	1' - 3 1/4"	CF	5/16"
GF	5 1/2"	CG	5/16"
GG	1' - 8 3/4"	CH	5/16"
GH	7 1/2"	CJ	3/16"
GJ	8"	DA	19'
GK	9 1/4"	DB	4' - 6"
GL	3/16"	DC	15
LA	7/8"	DD	5
LB	4	DE	12"
LC	1/2"	FA	20'
LD	1/2"	FB	4' - 6"
LE	1/4"	FC	15
LF	1/4"	FD	5
LG	3/16"	FE	12"

REVISION		DESCRIPTION	
LAST	07/20/10		
FDOT 2014		DESIGN STANDARDS	
SPAN SIGN STRUCTURE			
INDEX	NO. 11320		
SHEET	NO. 1 of 5

VITA

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- 1984 - 1988 University of Miami
 B.S., Architectural Engineering
- 1988 – 1993 Winningham & Fradley, Inc.
 Project Engineer
- 1993 – 1996 FDOT District 4 – Environmental Management
 Project Engineer
- 1996 – 1997 FDOT District 4 – Office of Production
 Production Staff Engineer
- 1997 – 1999 FDOT District 4 – Structure Design
 Structural Design Engineer
- 1998 - 2000 Florida International University
 M.S, Civil Engineering
- 1999 – 2007 FDOT District 4 – Structures and Facilities
 Bridge Repair Engineer
- 2007 – 2014 Florida's Turnpike Enterprise
 Maintenance Operations Engineer
- 2009 - 2014 Florida International University
 Doctoral candidate, Civil Engineering
- Professional Engineer: Florida PE 51439
Certified General Contractor: Florida CGC 062929
Certified Public Manager: Florida State University