Division of Operations Research On-Call Task #2 - Evaluation of the Effectiveness of Flexible Retroreflective Backplates

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16. Abstract		
Retroreflective backplates for traffic signals have a proven record of reducing traffic accidents and, currently, all new traffic signals are designed with backplates. While including backplates in the design of new signals and support structures is straightforward, adding backplates to existing signals can be problematic as doing so increases wind loads on the signals and the supporting structures, possibly to the extent that the existing structure is not adequate. This report describes a research project centering on an examination of pertinent design standards, specifications, and typical practice, to determine if broad recommendations could be made regarding whether the addition of backplates to existing signals would be acceptable or would render existing support structures inadequate for the increased wind loads. The University of Cincinnati research team, in concert with Kittelson and Associates, Inc., performed analyses of representative mast-arm and span-wire signal support structures and was able to identify areas where standard design procedures sometimes result in conservative designs. Additionally, the research team evaluated a commercially available flexible backplate design that can potentially reduce wind loads associated with the addition of retroreflective backplates to new or existing signals. Based on these analyses, a procedure for evaluating existing signs support structures for the increased wind loads due to the addition of rigid or flexible backplates is proposed.		
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Problem Statement:

Retroreflective backplates (RBPs) for traffic signals have a proven record of reducing accidents (FHWA 2021). Currently, all new ODOT traffic signals and existing traffic signals are in the process of being upgraded with ridged louvered retroreflective backplates with goal of alerting nighttime drivers to the presence of traffic signals in dark conditions. The problem to be studied is to evaluate the effect of flexible retroreflective backplates (FRBPs) on the stresses, strains, and base bending moments and torsion moments of traffic signal supports, both considering mast arm configurations and strain poles with span wires and tethers.

Retroreflective backplates increase the wind cross-section of signal heads considerably, consequently increasing the stress/strain demands on mast arms and the moment demands at the bases of the poles. While this might not be an issue for new installations, other than the larger sizes required for the supports to resist the increased demands, this renders retrofits problematic to the point that they are seldom considered.

This project will focus on the evaluation of the response of signal support systems with FRBPs installed, to assess whether a retrofit with flexible plates will sufficiently limit the increase in demand to allow the use of the current supports, as opposed to forcing a complete rebuild. The cost difference is estimated to be more than tenfold, and thus a positive outcome of this project would contribute to substantial savings while improving safety.

This project will provide design aids in the form of modified area moment design factors (K factors) and design tables for mast arms, as well as modification coefficients for use to account for FRBPs in the SWISS program input.

Research Approach:

The goal of the proposed research is to assess the impact of reflective backplates and flexible reflective backplates on the demands associated with signal support structures.

The objective of the proposed research is to develop a set of recommendations for ODOT regarding the addition of reflective backplates and flexible reflective backplates to design processes and design aids currently used by ODOT engineers and contractors, including flowcharts and block diagrams, spreadsheets, and recommendations for modifications to input/output for the program SWISS.

The proposed work is subdivided into a series of tasks, some of which overlap, and the completion of which will lead to a positive outcome for the project. The tasks are further detailed in the following.

Task 1: Preparatory Work:

As part of Task 1, the researchers held a kick-off meeting with engineers from Kittelson, the ODOT technical advisory committee (TAC), and representatives from the ODOT research office to discuss expectations and scheduling of further progress meetings. The initial activities focused on the scope of signal support structure types, including mast arm and span-wire systems. The research team requested ODOT for sample plan sets of representative signal support structures to be used as case studies during the remaining tasks. As part of this task, the researchers familiarized themselves with ODOT standard practices including applicable standard construction drawings, design tables, and the program SWISS. Additionally, the research team expanded on their literature review searching for additional results associated with reflective backplates and flexible reflective backplates, investigating both published scientific literature and manufacturers' technical specifications.

The deliverables of this task were a well-defined scope of work and a set of sample signal designs to consider as case studies for the project.

Task 2: Wind Analysis of Signal Support Structures

A primary task of this study was to assess drag coefficients associated with reflective backplates and flexible reflective backplates. To that end, recent works addressing flexible backplates (Bridge et al, 2018; DeMallo, 2018) were reviewed and included in the analysis where appropriate. The impact of adding RBPs and FRBPs to existing signals on the signal support structures was studied, as well as the impact of including RBPs and FRBPs in the design of new signals and signal support structures. This task was focused primarily on wind events included in the Extreme I limit state, though limited recommendations can be made with respect to the impact of RBPs and FRPBs on the fatigue limits of signals and signal support structures. The deliverables associated with this task are a set of drag coefficients and the modification factors to use when FRBPs are used in place of RBPs. These data were used in the subsequent tasks.

Task 3: Recommendations for Updated Standard Practices:

Work associated with Tasks 1 and 2 culminated in a process for computing the demands on signal support structures including RBPs and FRBPs that is illustrated as block diagrams. Design aids are provided in the form of modified area moment design factors (K factors) and modified exposure areas (A_i) that can be used with design tables for mast arms, and modified design pressures and exposure areas that can be used with SWISS. Additionally, other modifications and updates are recommended as part of this task."

Task 4: Summarizing and Reporting Activities:

This task consisted of the preparation of a set of recommendations as the deliverable for this project, and the writing of a final report draft to be submitted to Kittelson and ODOT for feedback. After feedback is received, a final report will be authored and submitted to Kittelson and ODOT to conclude the project.

Research Background:

The governing specification for traffic signals is the AASHTO LRFD Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, herein referred to as the AASHTO LRFD LTS-1 Specification (AASHTO, 2018). Although different in many aspects from the AASHTO Bridge Design specifications, the AASHTO LRFD LTS-1 Specification does include the language, "Where appropriate, the language and intent of [these] Specifications is kept the same as in the AASHTO LRFD Bridge Design Specifications and the AASHTO LRFD Bridge Construction Specifications."

The design of signal support structures in the State of Ohio is based primarily on gravity loads and wind loads. The AASHTO LRFD LTS-1 requires consideration of four limit states including Strength, Extreme, Service, and Fatigue. Load combinations that are applicable to the design of signal support structures for each of these limits are shown in Table 1. In these load combinations, DC is the self-weight of the structure and its components, LL is the live load, W is the sustained wind load, and W_{Fat} is the fatigue wind load, which consists of truck gusts, natural wind gusts, vortex-induced vibrations, and galloping wind, applied individually.

Table 1: AASHTO LRFD LTS-1 Load Combinations Applicable to Signal Support Structures

Strength I	$1.25DC + 1.6LL$	Gravity Loads
Extreme IA	$1.10DC + 1.0W$	Wind Loads
Extreme IB	$0.90DC + 1.0W$	Wind Loads
Service I	$1.00DC + 1.0W$	Deflections
Fatigue I	$1.00DC + 1.0W_{Fat}$	Fatigue

The Strength I load combination generally applies when gravity loads are considered, though since live loads are minimal on signal support structures (a single 500^{lb} load representing the weight of a person), this will likely control only in rare circumstances.

The Extreme I load combination applies when a sustained static wind load is considered. In addition to lateral forces, wind can also cause forces that act vertically upwards or downwards; because of this, two versions of the Extreme I load combination are presented. Extreme IA considers the case where the vertical force acts downwards and is additive to the gravity loads and Extreme IB considers the case where the vertical force acts upwards and is mitigated by the gravity loads. For the cases considered in this report, only lateral wind loads will be considered with the Extreme I load combination, Extreme IA will control in all cases, and thus Extreme IB will not be considered. Throughout this report, the terms "wind" and "static wind" will be used to refer to the Extreme I limit state.

The Service I load combination is used for the consideration of deflection criteria. The AASHTO LRFD LTS-1 also includes additional service load combinations used to address crack control in prestressed concrete structures, but since the scope of this study is limited to steel structures those combinations are not considered herein.

The AASHTO LRFD LTS-1 includes both Fatigue I and Fatigue II load combinations that are identical mathematically. However, the Fatigue I limit applies to the design of new structures using a basis of infinite life design while the Fatigue II limit applies to the evaluation of existing structures for fatigue. The scope of the study described herein was limited to the consideration of fatigue within the context of infinite life and not finite life, as is consistent with the AASHTO LRFD LTS-1, and thus only Fatigue I is considered herein. Throughout this report, the terms "fatigue" and "fatigue wind" will be used to refer to the Fatigue I limit state.

Strength and Extreme Limit States:

The available strength of the sign supports and required demands with respect to gravity loads are affected only slightly when backplates are added to the signals, as the weight of the backplates is small compared with the weight of the signs, signals, and self-weight of the mast arms and poles. However, the increase in exposure area associated with the addition of backplates can be substantial, ranging from 82% to almost 200% for signals with 12" lamps, as shown in Table 2, and can lead to increases in wind loads that may potentially render the signal support structures inadequate.

Table 2: Exposure Area for Signals with 12” Lamps

Signal Configuration	Width (in)	Height (in)	Area w RBP (ft^2)	Area w/o RBP (ft^2)	Increase (%)
1 Section	14	14	4.0	1.4	194
2 Sections	14	28	6.3	2.7	133
3 Sections	14	42	8.7	4.1	112
4 Sections	14	56	11.0	5.4	102
5 Sections - Vertical	14	70	13.3	6.8	95.9
5 Sections - Cluster			12.4	6.8	81.6

Wind pressures for which the support structures are designed are stipulated in Section 3.8 of the AASHTO LRFD LTS-1 Specification. Specifically, the design pressure mirrors the approach taken in the ASCE 7-10 standard and is given as,

$$P_z = 0.00256 K_z K_d G V^2 C_d \tag{1}$$

where P_z is the design wind pressure, V is the base wind velocity, K_z is a factor addressing height above ground and exposure, K_d is a wind directionality factor, G is a gust factor, and C_d is a drag coefficient associated with the element subjected to wind. The exposure factor, K_z, is determined based on the elevation of the signals and elements of the support structure above ground, and the directionality factor, K_D, is based primarily on wind direction and on the cross section of the support structure elements. Drag coefficients, C_D, are determined based on the type of support structure (mast arm vs. span wire, signs vs. signals, etc., within the context of this proposed study).

The most important component of Equation 1 is the base wind velocity, V, since it is squared. The base wind velocity is determined based on mapped values after assessing the risk category of the support structure and the average daily traffic (ADT) of the roadway that the traffic signal serves. Maps for mean recurrence intervals of 1,700 years, 700 years, 300 years, and 10 years are included in the AASHTO LRFD LTS-1 and in the ASCE 7 standard. Standard ODOT drawings for signal supports (TC-81.22) indicate that standard designs in Ohio are based on a mean recurrence interval of 700 years with $V = 115^{MPH}$ but are based on an ADT greater than 10,000, which is inconsistent with AASHTO LRFD LTS-1 Table 3.8-1, reproduced in Table 3 herein, which indicates that wind velocity should be based on a 1,700 year mean recurrence interval for most signal supports. Mapped wind velocities for Ohio in the AASHTO LRFD LTS-1 are shown in Table 4.

Table 3: Risk Determination for Sign and Signal Support Designs (AASHTO LRFD LTS-1 Table 3.8-1)

Traffic Volume	Risk Category Typical	High	Low
ADT $\leq$ 100	300	1,700	300
100 < ADT $\leq$ 1,000	700	1,700	300
1,000 < ADT $\leq$ 10,000	700	1,700	300
10,000 < ADT	1,700	1,700	300

Typical Risk: Failure could cross a travelway
High Risk: Failure could stop a lifeline travelway
Low Risk: Failure could not cross a travelway
Roadside Sign Supports: Base wind velocity on a 10 year MRI

Table 4: AASHTO LRFD LTS-1 Mapped Values for Base Wind Velocity in Ohio

MRI	Wind Velocity, V
1,700 years	120^{MPH}
700 years	115^{MPH}
300 years	105^{MPH}
10 years	76^{MPH}

The wind pressures that are used in the design of signal support structures are a function of the exposure conditions associated with the location of the signal. Exposure conditions are categorized as B, C, or D (ASCE 7-10). Exposure B corresponds to "urban and suburban areas, wooded areas, or other terrain with numerous, closely spaced obstructions that have the size of single-family dwellings or larger." Exposure C corresponds to "open terrain with scattered obstructions that have heights generally less than 30 ft…including flat, open country and grasslands." Exposure D corresponds to "flat, unobstructed areas and water surfaces… including smooth mud flats, salt flats, and unbroken ice." Older versions of the ASCE 7 standard included an Exposure A that corresponded to urban environments with tall buildings similar to downtown Columbus, Cleveland, or Cincinnati, but that exposure condition was removed prior to the adoption of the ASCE 7-10 standard.

The exposure category for the site is reflected in the exposure factor, K_z, used in calculating the design pressure, as shown in Equation 1. The AASHTO LRFD LTS-1 Specification assumes an Exposure C by default, and for an elevation of $z = 20$ ft above the ground, $K_z = 0.897$. Typical values for the directionality factor, K_d, are taken from AASHTO LRFD LTS-1 Table 3.8.5-1, which shows $K_d = 0.85$ for signals and signs and $K_d = 0.95$ for round or nearly-round pole supports. The gust factor, G, is generally taken as 1.14 for the design of signal support structures. Drag coefficients are provided in AASHTO LRFD LTS-1 Table 3.8.7-1 where it is shown that for a signal, $C_d = 1.20$, for members $C_d = 0.45$ to 1.10, and for signs $C_d = 1.12$ to 1.30. Using these values, the design pressure for a signal can be computed as,

$$P_{z,Signal} = (0.00256)(0.897)(0.85)(1.14)(115^{\text{mph}})^2(1.20) = 35.3^{\text{psf}} \tag{2}$$

Similarly, the design pressure for a mast arm can be computed as,

$$P_{z,Arm\ Min} = (0.00256)(0.897)(0.95)(1.14)(115^{\text{mph}})^2(0.45) = 14.8^{\text{psf}} \tag{3}$$

$$P_{z,Arm\ Max} = (0.00256)(0.897)(0.95)(1.14)(115^{\text{mph}})^2(1.10) = 36.2^{\text{psf}} \tag{4}$$

For mast arm systems, the AASHTO LRFD LTS-1 Specification identifies common failure modes for steel structures in Chapter 5, including the static moment strength of the mast arm(s), static shear strength of the mast arm(s), static strength of the connection(s) between the mast arm(s) and pole, static axial strength of the pole, static moment strength of the pole, and static shear strength of the mast arm. Additionally, the connections between the mast arm(s) and pole and between the pole and the base plate must be evaluated for fatigue.

The AASHTO LRFD LTS-1 addresses the analysis of span-wire systems in Appendix A where both simplified and detailed methods of analysis are presented. In either case, the analysis procedures consider the Strength I and Extreme I load combinations and result in maximum cable tensions and reaction forces at locations of cable supports. After the analysis is complete, the cable tension is evaluated based on manufacturer data, though the resistance factor to be applied is not clearly identified in the AASHTO LRFD LTS-1 Specification. Additionally, the cable reactions are applied to the supporting strain poles and the poles are then evaluated for the appropriate limit states in Chapter 5.

Service Limit State:

Serviceability criteria in the form of deflection checks are imposed. The AASHTO LRFD LTS-1 states in Chapter 10 that for structures "supporting signs and traffic signals, the maximum vertical deflection of the horizontal support resulting from Service I load combination shall be limited to $L/150$, where L is the span length." Note that sag in span-wire systems is considered explicitly during the design process when strength and static wind limit states are considered. Thus, the Service I limit and load combination are not considered for span-wire systems.

Fatigue Limit State:

Fatigue effects of wind gusts acting on signal support structures must be considered during their design. The AASHTO LRFD LTS-1 specifically identifies several sources of gusting wind loads, including, galloping, vortex-induced vibrations, natural wind gusts, and truck-induced gusts. Each of these sources of cyclic loading are to be considered individually. Traditionally, fatigue of structures has been evaluated either within the context of finite life, where a number of load cycles that can be sustained safely at a given stress range is determined, or in the context of infinite life, where an unlimited number of load cycles can be sustained as long as the stress range is below a certain limit. Signal support structures are traditionally designed considering infinite life because of the highly variable nature of the cyclic loading.

Galloping is an oscillation of the structure at its natural frequency that occurs in the plane perpendicular to the direction of wind flow. Signal support structures are particularly sensitive to galloping because they tend to be relatively flexible and have little inherent damping. The AASHTO LRFD LTS-1 states, "in lieu of designing for galloping or vortex-shedding limit state fatigue wind load effects, mitigation devices may be used as approved by the Owner." Since ODOT requires mechanical damping for larger support structures, galloping will not be considered herein. Vortex shedding can result in oscillations as a fluid passes around an object. In the current context, the fluid is air and it is passing around a pole or mast arm. However, the presence of attachments such as signals and signs on mast arms tends to reduce or eliminate issues associated with vortex shedding.

Natural wind gusts represent the variable nature of the horizontal wind acting on the exposure area of the sign, signals, and supporting structure. Wind gusts cause a cyclic loading in the structure that needs to be considered within the Fatigue limit state, whereas a sustained wind is considered to be a static load evaluated with the Extreme limit state. The pressure due to natural wind gusts acts horizontally on an area that is the horizontal projection of the signs, signals, and structure on a vertical plane located at the axis of the mast arm. The range of wind pressures resulting from natural wind gusts is represented as shown in Equation 5, where I_F the fatigue importance factor.

$$P_{NW} = 5.2 C_d I_F \tag{5}$$

Truck-induced gusts are idealized as a pressure pulse that acts on signs, signals, and their supporting structure when a truck passes beneath them. The pressure pulse acts vertically upwards on an area that is the vertical projection of the signs, signals, and structure on a horizontal plane located at the nominal height of the signs or signals. The range of wind pressures resulting from the passage of a truck is represented as shown in Equation 6.

$$P_{TG} = 18.8 C_d I_F \tag{6}$$

After the ranges of wind pressure from the appropriate sources have been determined and have been applied to the appropriate areas, the resulting stress ranges at critical structural details are computed and compared to limiting stress values for common details that are tabulated in Chapter 11 of the AASHTO LRFD LTS-1.

Note that span-wire systems need not be evaluated for fatigue. AASHTO LRFD LTS-1 Appendix A states that "only wind loads acting perpendicular to the span usually need to be considered" for span-wire systems. Further, AASHTO LRFD LTS-1 Chapter 11 states, "the provisions of this Section are not applicable for the design of span-wire (strain) poles." Thus, fatigue for span-wire systems is not considered herein.

<u>ODOT Design Approach for Mast-Arm Support Systems:</u>

The Ohio Department of Transportation employs standard drawings and spreadsheets to aid in the design of mast-arm support systems. The procedure centers around the calculation of the Area Moment Design Factor, K, shown in Equation 7 and illustrated in Figure 1, where b_i is the distance of the i^{th} sign or signal supported by a mast arm from the near face of the supporting pole, and A_i is the exposure area of the i^{th} sign or signal. This design factor mathematically reflects the fact that exposure areas farther away from the support pole lead to higher bending moments on the mast arm and higher torsional moments on the support pole. After the design factor is computed based on the required locations of signs and signals to be supported, the factor is used in tables to select the appropriate standard design.

$$K = \Sigma b_i A_i \tag{7}$$

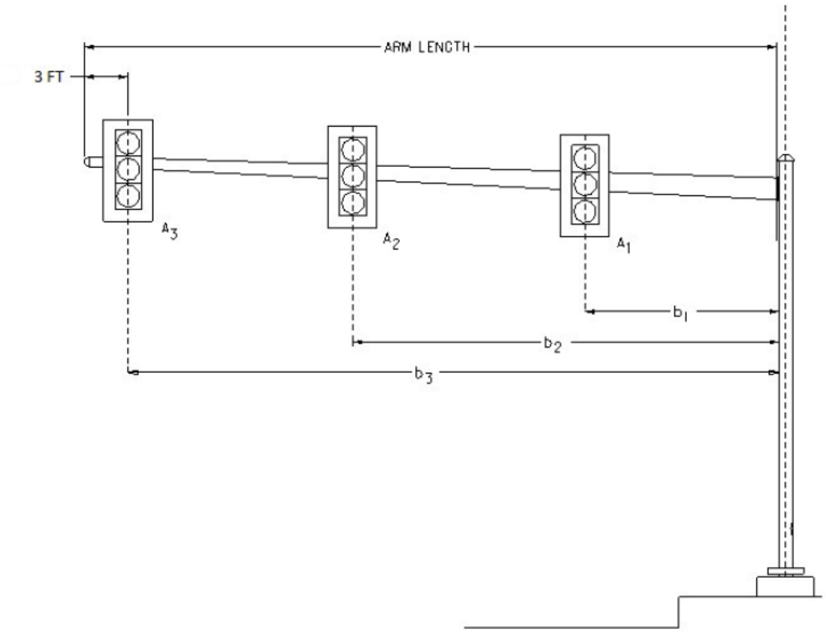

Figure 1: Distances, b_i, and Areas, A_i, Used in the Area Moment Design Factor (ODOT TEM 2021)

ODOT Design Approach for Span-Wire Support Systems:

The Ohio DOT employs the Span Wire Signal Support software SWISS for the design of span wire systems. The designer enters pertinent information describing the geometry of the system including the overall layout, exposure areas, area factor multiplier, and design data including design wind pressure.

The SWISS documentation (SWISS 2010) states that a design pressure of 42^{psf} is typically used, which corresponds to "AASHTO Criteria for a wind pressure on traffic signals with 90^{MPH} winds." It is posited that this is referring to the 2009 AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO 2009) wherein the design wind pressure is computed as (McDonald et al. 1995),

$$P_z = 0.00256(1.3V)^2 C_h C_d \tag{8}$$

where C_h is a height factor similar to K_z in the AASHTO LRFD LTS-1. With V taken as 90^{MPH} for all of Ohio in that version of the specification, C_h taken as 1.00 for $h = 20$ ft, and C_d taken as 1.2 for signals, the resulting design pressure is $P_Z = 42.05^{psf}$.

It is further noted, however, that the 2009 AASHTO ASD LTS employed a fundamentally different design approach including different load combinations, a ⅓ stress increase in some cases, and different strength models for the strain poles.

Other Literature:

A review of the pertinent literature was performed, and several works were reviewed (in addition to the specifications and standards that were summarized in the previous section), including Albert et al. (2007), McDonald et al. (1995), Connor et al. (2012), and others. While some of these reviewed works are cited throughout this report, the information provided by Pelco (2021) and the research performed by Bridge et al. (2018) are described separately in the following two subsections.

Pelco AeroFlex Retroreflective Backplate:

The AeroFlex flexible retroreflective backplate made by Pelco features 5" wide side panels that are attached with an ABS hinge that enables the side panels to "flex" out of plane during high wind events (Pelco 2021). When this occurs, the exposure area of the signal and flexible backplate are reduced relative to a rigid backplate. Pelco test results indicate that a reduction in force of 47% was achieved during testing when 5¼"×12" flexible side panels were tested at Oklahoma Christian University in a wind tunnel at wind speeds of 75^{MPH} (Pelco 2018). Additional product testing was completed at the Florida International University's "Wall of Wind" in Nov. 2018 (Zisis et al. 2018).

Data in the Pelco report summary shows that at a wind speed of 75^{MPH}, the AeroFlex flexible panel displaced by an angle of 48 deg. and carried a force of 7.62^{lbs}. A rigid louvered aluminum panel that was also subjected to a wind speed of 75^{MPH} the carried a force of 14.4^{lbs}. Thus, the AeroFlex panel carries a load equal to $7.62^{lbs}/14.4^{lbs}$ = 52.9% of the load carried by the rigid panel, which corresponds to a 47.1% reduction in force.

University of Florida Testing:

A 2018 report published by the University of Florida and funded by the Florida Department of Transportation (Bridge et al. 2018, Demallo 2018) detailed results of reduced-scale wind-tunnel testing associated with mast arms and traffic signals. As part of that study, the investigators examined several modifications to backplates that were intended to reduce the wind loads on the supporting structure. The investigators concluded that a reduction in loads was achieved using folding, rotating, and mesh side panels, but they suggested additional full-scale testing to quantify the level of load reduction achieved.

Research Findings and Conclusions:

Findings of the research indicate that FRBPs cannot simply be substituted for RBPs without consideration of the strength and serviceability of the supporting structure. The addition of FRBPs may in many cases lead to the calculated available strength of the support structure to be less than the demand when both strength and demand are computed based on the assumptions and design criteria traditionally adopted by ODOT. However, many of the assumptions and criteria can be justifiably modified to either increase the strength of the structure or reduce the demand on it. Possible modifications that can be taken into consideration are described in the following subsections.

<u>Modified Exposure Area and Drag Coefficients of FRBPs for Strength Design:</u>

The exposure area of a signal with an AeroFlex backplate can be divided into an area that is flexible and an area that is rigid. The flexible area consists of two vertical strips, each 5" wide along each side of the signal; the remaining area can be considered rigid. As was mentioned earlier, wind tunnel testing conducted at Oklahoma Christian University and described by Pelco (2018) demonstrated that a reduction in stress of 47% was measured when a louvered aluminum panel was replaced by an AeroFlex side panel was subjected to a wind of 75^{MPH}. By multiplying the flexible area of the AeroFlex FRBP by a reduction factor of 0.53 that reflects the experimental results, and then adding the rigid area, an effective area can be computed for each of the signal configurations, as shown in Table 5.

By taking the ratio of the effective area to the total area, a reduction factor can be computed that can be applied to the total area. Thus, when a support structure is designed or evaluated for a signal with an AeroFlex FRBP, the wind force acting on a signal can be determined either (a) using the effective area, (b) using the total area multiplied by the associated reduction factor, or (c) using the total area along with a modified drag coefficient that is equal to the product of 1.20 and the associated reduction factor. With the exception of the 5 Section Cluster configuration, the modified drag coefficient would have a value of (0.804)(1.20) = 0.965. The modified drag coefficient for the 5 Section Cluster configuration would have a value of (0.857)(1.20) = 1.03.

Table 5: Effect Area and Reduction Factors for AeroFlex FRBPs

Signal Configuration	Area w RBP (in²)	Flexible Area (in²)	Rigid Area (in²)	Total Area (ft²)	Effective Area (ft²)	Reduction Factor (in²)
1 Section	576	240	336	4.0	3.2	0.804
2 Sections	912	380	532	6.3	5.1	0.804
3 Sections	1,248	520	728	8.7	7.0	0.804
4 Sections	1,584	660	924	11.0	8.8	0.804
5 Sections - Vertical	1,920	800	1,120	13.3	10.7	0.804
5 Sections - Cluster	1,780	520	1,190	11.9	10.2	0.857

Note that the total area of the AeroFlex 5 Section Cluster RBP is slightly smaller than the area of a rigid RBP. This is because the geometry of the 5 Section Cluster RBP found in the ODOT Signal Design Reference Packet (ODOT SDRP 2021) is slightly different than the geometry of the AeroFlex 5 Section Cluster RBP, as shown in Figure 2.

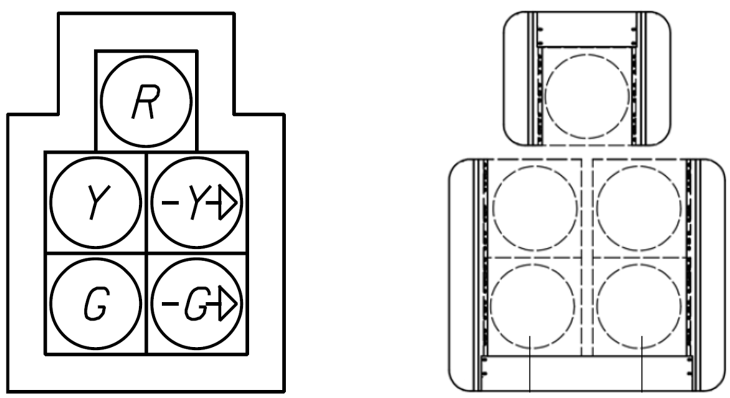

Figure 2: 5 Section Cluster Configurations: ODOT (left) and Pelco AeroFlex (right)

Design for Lower Base Wind Velocities:

The wind load provisions in the AASHTO LRFD LTS-1 Specification are based on ASCE 7-10, and the maps in both the AASHTO LRFD LTS-1 Specification and in the ASCE 7-10 standard show a base wind velocity of 115^{MPH} for all of the State of Ohio, based on a mean recurrence interval of 700 years. However, a newer version of the ASCE standard, ASCE 7-16, is widely available, has been adopted by many jurisdictions, and contains revised wind load maps that show a base wind velocity of 107^{MPH} for all of the State of Ohio based on a mean recurrence interval of 700 years. It would be justifiable for an engineer to use a base wind velocity of 107^{MPH} instead of 115^{MPH} for the design of signal supports in the State of Ohio. While this reduction might at first seem modest, since the design pressure is a function of the square of the wind velocity, as shown in Equation 1, this reduction results in a 13.4% decrease in design wind pressure. This reduction could be reflected in existing design procedures by multiplying the Area Moment Design Factor by a reduction factor of 0.866, as shown in Equation 9

$$K_{mod} = 0.866\Sigma b_i A_i \tag{9}$$

Design for Alternate Risk Categories:

Standard ODOT drawings for signal supports indicate that standard designs in Ohio are based on a mean recurrence interval of 700 years with a base wind velocity of 115^{MPH}. However, no mention is made in the drawings of alternate wind velocities being used for locations with lower risk. Wind speeds in Ohio rarely exceed 70^{MPH}, and in fact wind speeds associated with an F1 tornado range from 73 to 112^{MPH} (Weather 2021). Given this, designing for a wind velocity less than 115^{MPH} (AASHTO 2018; ASCE 7-10) or 107^{MPH} (ASCE 7-16) could be justified. In that case, the Area Moment Design Factor could be reduced by a factor equal to the square of the ratio of the design wind velocity to 115^{MPH}, as shown in Equation 10.

$$K_{mod} = \left(\frac{V}{115^{\mathrm{MPH}}}\right)^2 \Sigma b_i A_i \tag{10}$$

For example, if a wind velocity of 79^{MPH} was used for design, then the Area Moment Design Factor could be reduced by a factor of $(79^{MPH} / 115^{MPH})^2 = 0.472$, which would essentially offset the increase in exposure area associated with the addition of even a rigid backplate to a 3-Section Signal.

A rational approach to implementing this alternative might be to base the design wind velocity on a 300 year wind instead of a 700 year wind, which would result in a 105^{MPH} velocity if the maps in the AASHTO LRFD LTS-1 and ASCE 7-10 standards were referenced and would result in a velocity ranging from 99^{MPH} to 102^{MPH} for various locations in Ohio if the maps in the ASCE 7-16 standard were referenced.

Design for Site-Specific Exposure Categories:

The AASHTO LRFD LTS-1 Specification defaults to an Exposure C but affords the engineer the option of selecting other exposure conditions by including the language, "exposure coefficients for other terrain conditions may be used per ASCE/SEI 7-10 with permission of the Owner." In addition to being dependent on the exposure category, K_z is also a function of the elevation above ground. Though K_z is constant at elevations less than 15 ft above ground, its value increases at elevations above 15 ft. K_z can be computed at any arbitrary elevation but is tabulated at elevations of 0-15 ft, 20 ft, 25 ft, 30 ft, etc. Exposure factors for different exposure conditions are shown in Table 6 for an elevation of 20 ft, considered to be typical of most mast arms or span wires.

Table 6: Exposure Factors for Elevation of 20 ft

Exposure Condition	Exposure Factor for $z = 20$ ft
A	0.356
B	0.621
C	0.897
D	1.078

Note: Data for Exposure Category A taken from Taly (2003).

If a signal support structure is located in an area that can be defined as Exposure B, then a lower value of the exposure factor, K_z, could be used in the design of the structure. The design pressure, and thus the Area Moment Design Factor, are proportional to the exposure factor, thus selecting Exposure B instead of the default Exposure C could be reflected in the design of support structures with an elevation of 20 ft by multiplying the Area Moment Design Factor by a factor of 0.621/0.897 = 0.692.

If a signal support structure is located in an area that can be defined as Exposure A, it may be possible to further reduce the exposure factor and the Area Moment Design Factor, but since the Exposure A condition is no longer included in the ASCE 7 standard, this is not recommended without a site-specific study to address issues like wind channeling and vortex shedding. This is likely cost prohibitive.

Intentional Aerodynamic Shielding:

In cases where multiple signs and signals are supported by the same mast arm, a strategy of locating signs and signals at the same location on the arm could result in aerodynamic shielding that would reduce the exposure area and thus the wind force acting on the support structure. The mast arm shown in Figure 3 shows a case of partial shielding where the two left-turn-only signs partially shield or are partially shielded by the two signals on the opposite side of the arm. In cases where an existing signal support structure is being evaluated, a modest reduction in wind load could be achieved by considering the projected area of both sign and signal at each location on a vertical plane. This reduction could be increased if the sign or signals were shifted slightly so that the amount of overlapping area was maximized. This could be implemented in the evaluation of a mast arm by using modified areas in the determination of the Area Moment Design Factor and could be implemented in the evaluation of span-wire systems by modifying the area factor multiplier in SWISS.

Figure 3: Example of Partial Aerodynamic Shielding on a Mast Arm

Aerodynamic shielding of the mast arm by signs and signals is substantiated by Bridge et al. (2018) and could also be considered. However, this would be challenging to implement given the current ODOT design procedures where the Area Moment Design Factor is a function only of the exposure areas of the signs and signals and the exposure of the mast arm itself is not considered explicitly. If this option is implemented, it is worth considering that an increase in shielding can be achieved if the signals were oriented horizontally instead of vertically and mounted at the same elevation as the mast arm, as shown in Figure 4. This was the configuration considered by Bridge et al., but it is acknowledged that this signal orientation is less common in Ohio than in Florida.

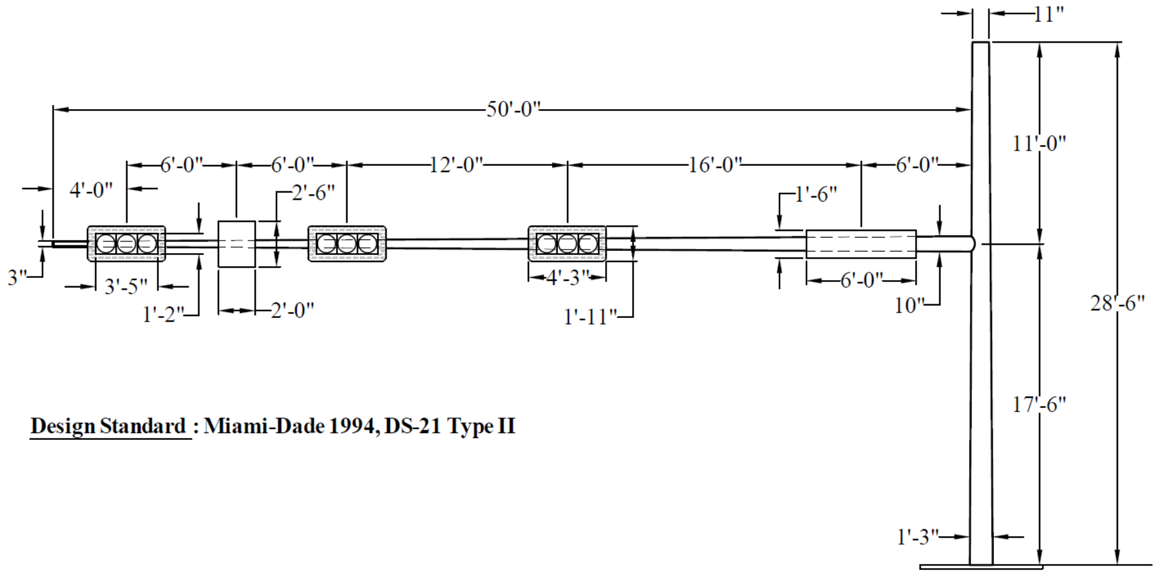

Figure 4: Horizonal Orientation of Signals to Maximize Shielding of the Mast Arm (Bridge et al. 2018)

Fatigue Loading of Mast Arm Systems:

As was mentioned earlier, fatigue need not be considered at all in the design of span-wire systems and fatigue loading for mast arm systems is based on four sources. After eliminating galloping and vortex-induced vibrations by employing mechanical dampers, all that remains are natural wind gusts (P_{NW}) and truck gusts (P_{TG}). Natural wind gusts create cyclic flexure in the horizontal plane of the mast arm (out of the plane of the structure) whereas truck gusts or truck induced pressure pulses induce cyclic flexure in the vertical plane of the mast arm (in the plane of the structure).

Natural Wind Gusts: Natural wind gusts were not found to control in the mast-arm calculations that were performed by the UC Research Team and do not appear to be considered explicitly in the ODOT design procedure for mast arms. In cases where natural wind is found to control a design, consideration could be given to designing for a reduced pressure, P_{NW}, or to allowing less conservative design assumptions associated with the fatigue category assigned to the details being evaluated.

The design pressure for natural wind gusts in the AASHTO LRFD LTS-1, shown in Equation 5, is based on a yearly mean wind velocity of 11.2^{MPH}. However, the specification allows for a reduced pressure to be considered in areas where the yearly mean wind velocity is smaller, as shown in Equation 11.

$$P_{NW} = 5.2C_d \left(\frac{V_{mean}}{11.2^{\text{MPH}}} \right)^2 I_F \tag{11}$$

For example, the yearly mean wind velocity in Columbus, OH, is approximately 9.0^{MPH}, which would correspond to a fatigue pressure, P_{NW}, that is $(9.0/11.2)^2 = 64.6\%$ of the nominal pressure.

Finally, details that are typically evaluated for fatigue stresses include (i) the welded joint between the mast arm and pole, (ii) bolts connecting the mast arm to the pole, (iii) the welded joint between the pole and baseplate, and (iv) anchor rods connecting the baseplate to the foundation. Increases in fatigue strength could possibly be realized for the bolted joint between the mast arm and pole if the bolts were considered to be pretensioned instead of snug tight. While the AASHTO LRFD LTS-1 does not indicate a distinction in Table 11.9.3.1-1 where the fatigue details and associated design constants are presented, the AASHTO LRFD Bridge Specification (AASHTO 2017) provides a higher fatigue strength for pretensioned bolts than for nonpretensioned bolts. In that specification, pretensioned bolts are considered to be a Category A detail with a threshold stress of $\Delta F_{TH} = 24.0^{\text{ksi}}$, while nonpretensioned bolts are considered to be a Category D detail for infinite life with a threshold stress of $\Delta F_{TH} = 7.0^{\text{ksi}}$. While direct adoption of the approach used in the bridge specification is not recommended for signal support structures without studying underlying assumptions and differences in the application, it does provide a possible mechanism for increased strength from the same connection detail if adequate pretension can be provided in the bolts.

<u>Truck Gusts:</u> An examination of Equations 5 and 6 will show that the pressure associated with a truck gust is approximately 3.6 times more intense than that associated with a natural wind gust (depending on the fatigue importance factor, which is often different for natural wind than for truck induced gusts). The truck induced gust, while significantly larger than the natural wind gust, acts on an exposure area that is generally much smaller than the exposure area acted upon by the natural wind gusts. Furthermore, the pressure pulse associated with a truck gust is highly dependent on the type and speed of the truck passing beneath the signal support structure. It is noted in the commentary to the AASHTO LRFD LTS-1 that during a field study of the effects of trucks on signal support structures, it was found that out of "over 400 truck events […] recorded covering a variety of truck types and vehicle speeds; only 18 trucks produced even a detectable effect on the cantilevered traffic signal structures (AASHTO 2018; Albert et al. 2007)." The commentary to the AASHTO LRFD LTS-1 also states that "the given

truck-induced gust loading should be excluded unless required by the Owner for the fatigue design of overhead traffic signal structures (AASHTO 2018)."

Based on the arguments in the preceding paragraph, the UC design team recommends that ODOT should not require the consideration of fatigue due to truck gusts. If consideration of truck gusts is to remain a design criterion, then it should be noted that the addition of flexible RBPs – or rigid RBPs, for that matter – will not appreciably increase the exposure area on which the truck induced pressure pulse, P_{TG}, acts. Thus, no change is expected in the fatigue design associated with the addition of FRBPs to signals supported by mast-arm systems. Additionally, if truck gusts are to be taken into account during design, consideration could be given to using a pressure, P_{TG}, smaller than given Equation 6. The AASHTO LRFD LTS-1 allows for a reduced pressure to be considered (Equation 12) in areas where the speed of the trucks passing beneath a signal is expected to be lower than 65^{MPH}, and a case can be made that, in most cases, if there is a signal on the road, that the posted speed limit for the road is likely at most 55^{MPH} and may be substantially lower. Using a truck speed of 55^{MPH} results in a reduction of 28% in the associated pressure, using a truck speed of 45^{MPH} results in a reduction of 52% in the associated pressure, and using a truck speed of 35^{MPH} results in a reduction of 71% in the associated pressure. This, coupled with the idea that the area on which the truck induced pressure acts is relatively small and with the idea that the pressure magnitude itself is conservative because of its unpredictability, reinforces the notion that ODOT may want to not require consideration of truck gusts for fatigue design of mast arms.

$$P_{TG} = 18.8 C_d \left(\frac{V_T}{65^{\text{MPH}}} \right)^2 I_F \qquad (12)$$

Finally, if truck gusts are taken into account during design, the same strategy to increase in fatigue strength for the bolts that was described in the context of natural wind gusts, i.e. accounting for bolt pretension, could be implemented within the context of truck gusts.

Recommendations for Implementation:

The University of Cincinnati research team conducted a literature review of pertinent research results, design specifications, standards, and drawings, and has identified several areas where the design procedures associated with signal support structures can be modified to accommodate the addition of flexible retroreflective backplates without the need for replacing the supporting structures. Those modifications are summarized in the following list and in the flowcharts found in the appendix.

- Effective exposure areas modified drag coefficients are presented for the Pelco AeroFlex flexible retroreflective backplates in Table 5.
- If necessary, it is justifiable to design for reduced wind velocities found in ASCE 7-16 (107^{MPH}) instead of those found in ASCE 7-10/AASHTO LRFD LTS-1 (115^{MPH}). This represents a reduction in wind load of approximately 13%.
- If necessary, it is justifiable to design for site-specific exposure categories. If the site of the signal is in an urban or suburban area, then the support structure can be designed for Exposure B conditions, resulting in a reduction in wind load of approximately 31%.
- If necessary, it is justifiable to account for aerodynamic shielding in the design or evaluation of the support structure. The resulting reduction in wind load would be a function of the specific signal configuration.

There is a need for an update to the ODOT design procedures for mast arm support systems and an urgent and compelling need for an update to the SWISS software used for the design of span wire support systems. Upon examination of the documentation and the source code of the SWISS software, it appears in fact that the software, which is still in use for the design of span-wire systems, is based on design specifications that are almost 30 years old. The AASHTO ASD LTS, which refers to ASCE 7-95 is based on a design philosophy (ASD) that despite still being permitted for signal support structures, is generally considered to be outdated and is no longer permitted by the Federal Highway Administration for the design of bridges.

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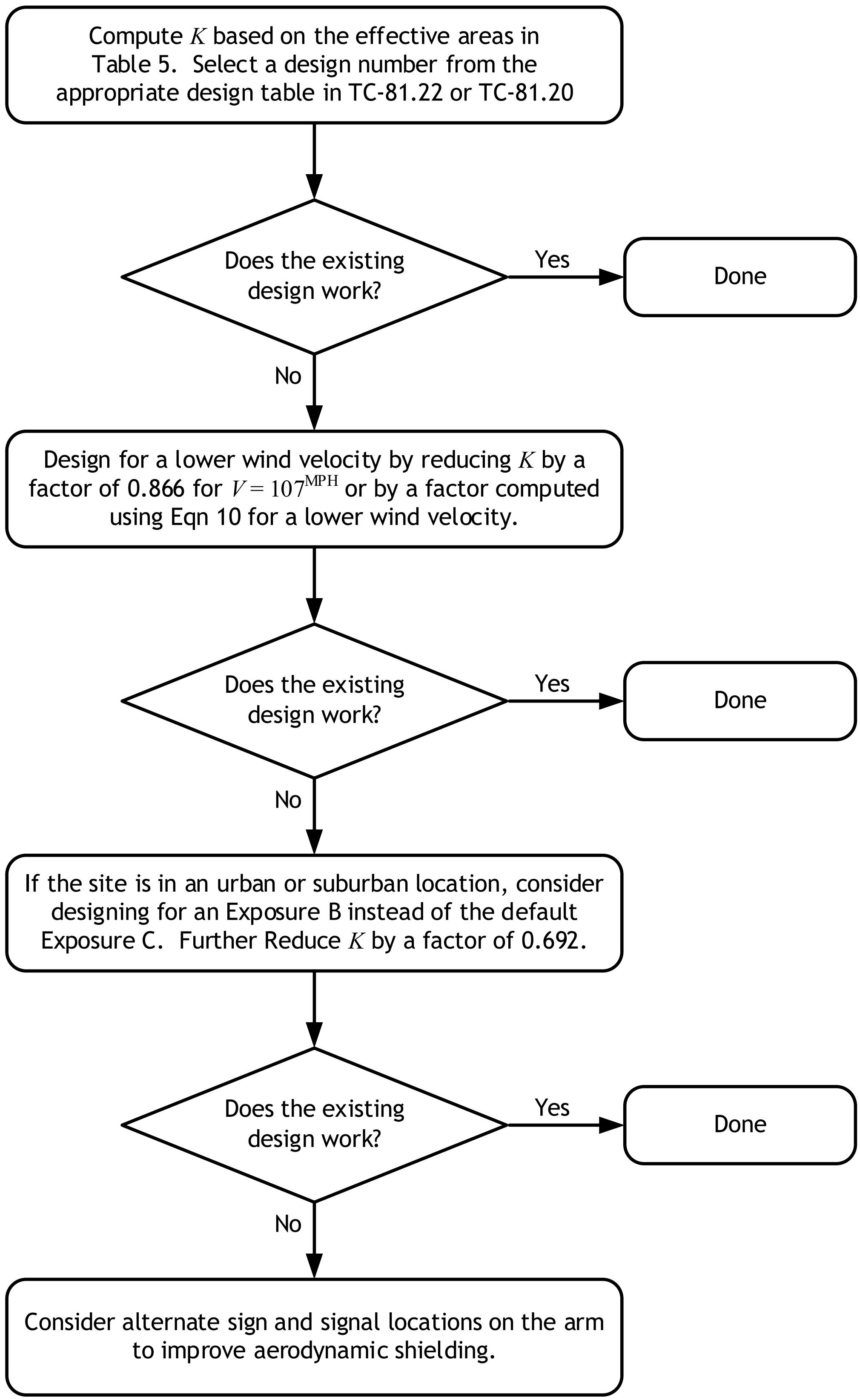
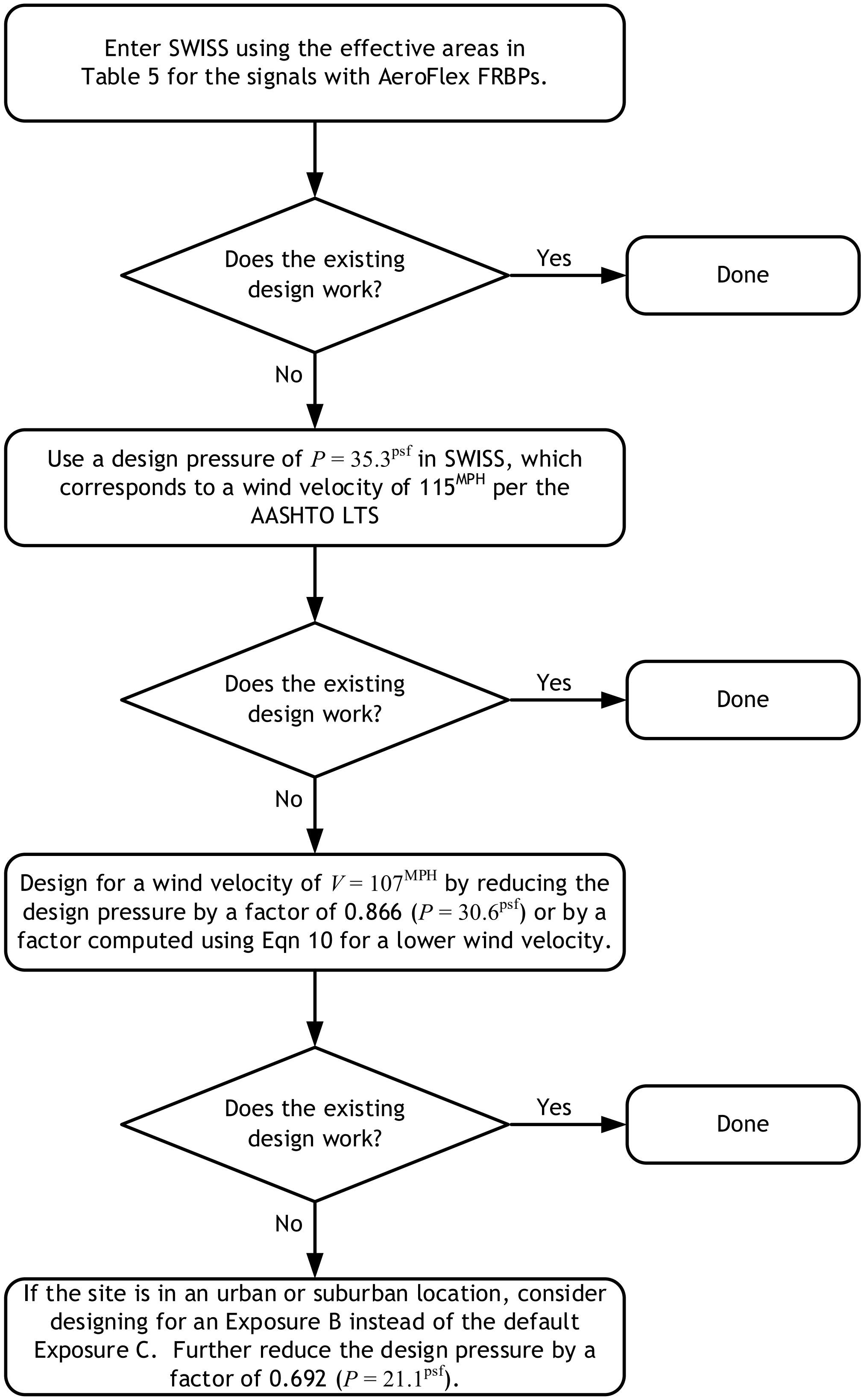
Appendix:

Flowcharts to be used for the evaluation of mast arm and span-wire support systems are shown in this section.

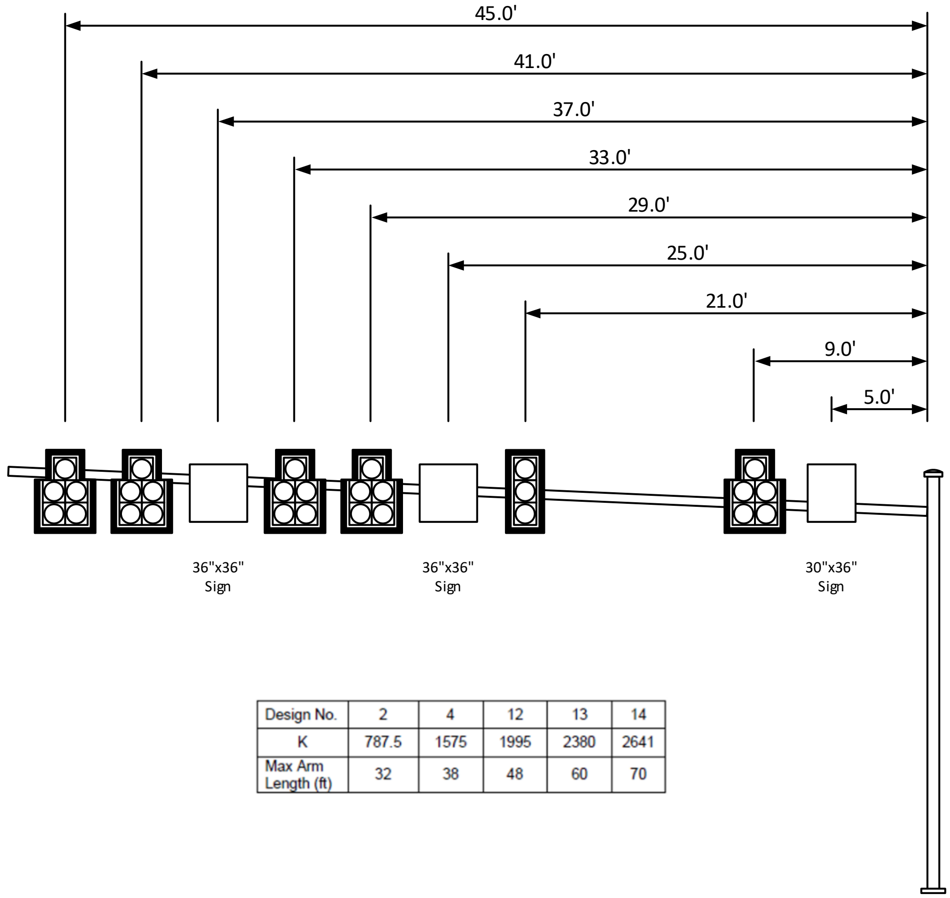
Flowchart for Evaluation of Existing Mast-Arm System for AeroFlex Flexible Backplates



Flowchart for Evaluation of Existing Span-Wire System for AeroFlex Flexible Backplates



Example Problem: Consider the hypothetical sign design shown below.



Design No.	2	4	12	13	14
K	787.5	1575	1995	2380	2641
Max Arm Length (ft)	32	38	48	60	70

Original Design without Backplates:

Compute the Area Moment Design Factor, K, for the system without backplates.

$$K = \Sigma b_i A_i$$

$$K = [(5')(7.5\text{ ft}^2) + (9')(6.8\text{ ft}^2) + (21')(4.1\text{ ft}^2) + (25')(9.0\text{ ft}^2) + (29')(6.8\text{ ft}^2)+\ldots$$

$$\ldots+ (33')(6.8\text{ ft}^2) + (37')(9.0\text{ ft}^2) + (41')(6.8\text{ ft}^2) + (45')(6.8\text{ ft}^2)] = 1{,}749\text{ ft}^3$$

From ODOT TC-81.22 Design Table, select a Design Number 12. $(1{,}749\text{ ft}^3 < 1{,}995\text{ ft}^3)$

Evaluate Design with Rigid Backplates:

$$K = [(5')(7.5\ \text{ft}^2) + (9')(12.4\ \text{ft}^2) + (21')(8.7\ \text{ft}^2) + (25')(9.0\ \text{ft}^2) + (29')(12.4\ \text{ft}^2) + \ldots$$

$$\ldots + (33')(12.4\ \text{ft}^2) + (37')(9.0\ \text{ft}^2) + (41')(12.4\ \text{ft}^2) + (45')(12.4\ \text{ft}^2)] = 2{,}725\ \text{ft}^3$$

Per ODOT TC-81.22, a Design Number 12 would no longer work. ($2{,}725\ \text{ft}^3 > 1{,}995\ \text{ft}^3$)

Evaluate Design with AeroFlex Flexible Backplates:

$$K = [(5')(7.5\ \text{ft}^2) + (9')(10.2\ \text{ft}^2) + (21')(7.0\ \text{ft}^2) + (25')(9.0\ \text{ft}^2) + (29')(10.2\ \text{ft}^2) + \ldots$$

$$\ldots + (33')(10.2\ \text{ft}^2) + (37')(9.0\ \text{ft}^2) + (41')(10.2\ \text{ft}^2) + (45')(10.2\ \text{ft}^2)] = 2{,}344\ \text{ft}^3$$

Per ODOT TC-81.22, a Design Number 12 would no longer work. ($2{,}344\ \text{ft}^3 > 1{,}995\ \text{ft}^3$)

Evaluate Design with AeroFlex Flexible Backplates using $V = 107^{\text{MPH}}$:

$$K = [(5')(7.5\ \text{ft}^2) + (9')(10.2\ \text{ft}^2) + (21')(7.0\ \text{ft}^2) + (25')(9.0\ \text{ft}^2) + (29')(10.2\ \text{ft}^2) + \ldots$$

$$\ldots + (33')(10.2\ \text{ft}^2) + (37')(9.0\ \text{ft}^2) + (41')(10.2\ \text{ft}^2) + (45')(10.2\ \text{ft}^2)] = 2{,}344\ \text{ft}^3$$

$$K_{mod} = 0.866K = (0.866)(2{,}344\ \text{ft}^3) = 2{,}030\ \text{ft}^3$$

Per ODOT TC-81.22, a Design Number 12 would no longer work. ($2{,}030\ \text{ft}^3 > 1{,}995\ \text{ft}^3$)

Evaluate Design with AeroFlex Flexible Backplates using $V = 107^{\text{MPH}}$ and Exposure B:

$$K = [(5')(7.5\ \text{ft}^2) + (9')(10.2\ \text{ft}^2) + (21')(7.0\ \text{ft}^2) + (25')(9.0\ \text{ft}^2) + (29')(10.2\ \text{ft}^2) + \ldots$$

$$\ldots + (33')(10.2\ \text{ft}^2) + (37')(9.0\ \text{ft}^2) + (41')(10.2\ \text{ft}^2) + (45')(10.2\ \text{ft}^2)] = 2{,}344\ \text{ft}^3$$

$$K_{mod} = (0.692)(0.866)K = (0.692)(0.866)(2{,}344\ \text{ft}^3) = 1{,}405\ \text{ft}^3$$

Per ODOT TC-81.22, a Design Number 12 would be acceptable. ($1{,}405\ \text{ft}^3 < 1{,}995\ \text{ft}^3$)