**Structural Monitoring and Maintenance**, *Vol. 1, No. 4 (2014) 393-409*
DOI: http://dx.doi.org/10.12989/smm.2014.1.4.393


# Effectiveness of strake installation for traffic signal structure fatigue mitigation

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*(Received August 2, 2014, Revised November 21, 2014, Accepted December 3, 2014)*

**Abstract.** Across-wind response is often the cause of significant structural vibrations that in turn cause fatigue damage to welded and other connections. The efficacy of low-cost helical strakes to mitigate such adverse response is presented for a traffic signal structure. Field observations are made on a prototype structure in a natural wind environment without and with helical strakes installed on the cantilevered arm. Through continuous monitoring, the strakes were found to be effective in reducing across-wind response at wind speeds less than 10 m/s. Estimates of fatigue life are made for four different geographical locations and wind environments. Results for the class of traffic signal structure show that helical arm strakes are most effective in locations with benign wind environments where the average annual wind speed is not more than the vortex shedding wind speed, which for this investigation is 5 m/s. It is concluded that while strakes may be effective, it is not the panacea to mitigating connection fatigue at all locations.

**Keywords:** traffic signal structure; fatigue; helical strakes; vortex shedding; full-scale monitoring

## 1. Introduction

Traffic signal structures that consist of a horizontal, cantilevered steel arm attached to a vertical tubular pole are commonly used to support traffic signals over roadways. Depending on the geographic location, traffic signal clusters are attached either horizontally or vertically along the length of the mast arm. Because they are most commonly long and slender, traffic signal support structures consistently exhibit wind-induced vibrations. These vibrations are commonly broken into two components related to the plane of the structure for a horizontal mast arm: in-plane motion relates to vertical vibrations due to across-wind effects, while out-of-plane motion arises from the unsteady nature of along-wind (or gusting) effects. The resulting stress reversals that persist over the years can lead to fatigue, particularly in welded connections.

Multiple mechanisms have been identified to cause the large-amplitude vibrations undergone by cantilevered traffic signal structures, but the reason for certain common characteristics, particularly the in-plane motion induced by across-wind effects, remains under dispute. The mechanism behind observed large-amplitude responses had been attributed to wind-induced

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http://www.techno-press.org/?journal=smm&subpage=7 ISSN: 2288-6605 (Print), 2288-6613 (Online)

galloping of the signal clusters (McDonald *et al.* 1995, Kaczinski *et al.* 1998). Galloping is a form of aerodynamic instability caused by negative aerodynamic damping that results from periodic changes in lift forces due to the oscillation of a body in a constant flow regime. As a result, fatigue design loads were specified to account for natural wind gust, truck-induced gusts, and galloping in the 2001 AASHTO specifications (AASHTO 2001).

To better understand the excitation mechanism, recent studies have found that vibrational response with signal clusters attached closely resembled the response due to vortex shedding (Letchford *et al.* 2008, Cruzado *et al.* 2013). Vortex shedding, like galloping, leads to vibrations perpendicular to the approaching wind. The vortex shedding mechanism occurs due to variations in pressure on the leeward side of a bluff body caused by the periodic detachment of low-pressure vortices from alternating sides of the body subjected to a flow regime. These studies related the vibrations to the height of the attached signal clusters. As a result of identifying vortex shedding as an excitation mechanism, both galloping and vortex shedding were accounted for in the fatigue provisions of the 2009 Standard Specifications (AASHTO 2009). Recently, large-amplitude vibrations have been attributed to the aerodynamic instability of the cantilevered mast arm (Zuo and Letchford 2010). Regardless of the excitation mechanism, both galloping or vortex shedding produce “across-wind” effects.

Various types of mitigation devices and measures, such as aerodynamic damper plates (Pulipaka *et al.* 1998), were developed based on the understanding that galloping was the primary means of excitation. However, the effectiveness of such mitigation devices has been limited, indicating either improper installation or a potential inadequacy in the current understanding of the problem. To mitigate response due to signal cluster vortex shedding, visual back plates were removed from the signal cluster housing (Letchford *et al.* 2008, Cruzado *et al.* 2013), which is also a potential hindrance to traffic light visibility. Excitation-independent solutions, such as impact dampers (Cook *et al.* 2001), viscous dampers (Hamilton *et al.* 2000, McManus *et al.* 2003), or integrated tuned-mass dampers (Christenson and Hoque 2011) have been developed to increase damping. Unfortunately, their effectiveness cannot be optimal for each structure due to the large variability in the configuration of these structures. A single device cannot be equally effective across multiple arm lengths or signal cluster configurations due to the sensitivity of the fundamental frequency.

The motivation behind reducing response is clear. These wind-induced vibrations lead to stress ranges at critical connections which can significantly reduce the fatigue life of cantilevered traffic signal support structures. Unfortunately, recent changes to the 2013 AASHTO specifications (AASHTO 2013) have eliminated vortex shedding as a design excitation mechanism for fatigue, only considering galloping, natural wind gust, and truck-induced loading. If an effective vibration mitigation device is instituted to reduce response, then galloping forces are allowed to be neglected during design—leading to longer, more slender structures. With vortex shedding-induced vibrations not accounted for during design, this leaves the potential for a reduced fatigue life for current AASHTO code-based designs.

With questions pertaining to whether or not vortex shedding is a significant excitation mechanism, there is need to study and evaluate the effectiveness of traditional aerodynamic damping methods to disrupt vortex formation and reduce excitation. Several strategies and devices have been used to alleviate vortex-induced vibrations (Ahearn and Puckett 2010). The most common add-on aerodynamic damping device for cylindrical structures is helical strakes (Warpinski 2006). Helical strakes have been known to significantly disrupt shear layer formation, along with vortex formation and shedding along tall, slender structures (Scruton and Walshe 1973).

Optimal strake sizes and installation geometry have been recommended by Kumar *et al.* (2008). Such geometries include the height and pitch of the helical strakes, where the pitch is the longitudinal length of one strake revolution. Considerations must be made to take into account structure specificity. Ideally, structural monitoring is necessary to assess the behavior of a traffic signal structure before and after the installation of aerodynamic damping devices.

This paper seeks to evaluate a low-cost, aerodynamic vibration suppression device for cantilevered traffic signal support structures. The approach involves the use of helical strakes in the form of a braided rope wound around the arm in a triple helix formation. Ropes have been successfully used by other researchers for mitigating vortex shedding induced vibration in high mast structures (Blevins 1977, Kaczinski *et al.* 1998, Ahearn and Puckett 2010, Ahearn 2010, Connor *et al.* 2012). The simple solution may be used as a retrofit and is intended to improve the fatigue life of certain existing fatigue prone systems. However, strakes could potentially be used in new construction for structures installed in suitable locations. As a means to determine the effectiveness of helical strake installation, the stress response at the pole-to-arm connection is observed in a natural wind environment before and after strake installation. To quantify the potential benefit of arm strake installation, the fatigue life of the structure is then estimated considering any changes in natural wind response for several distinct wind environments.

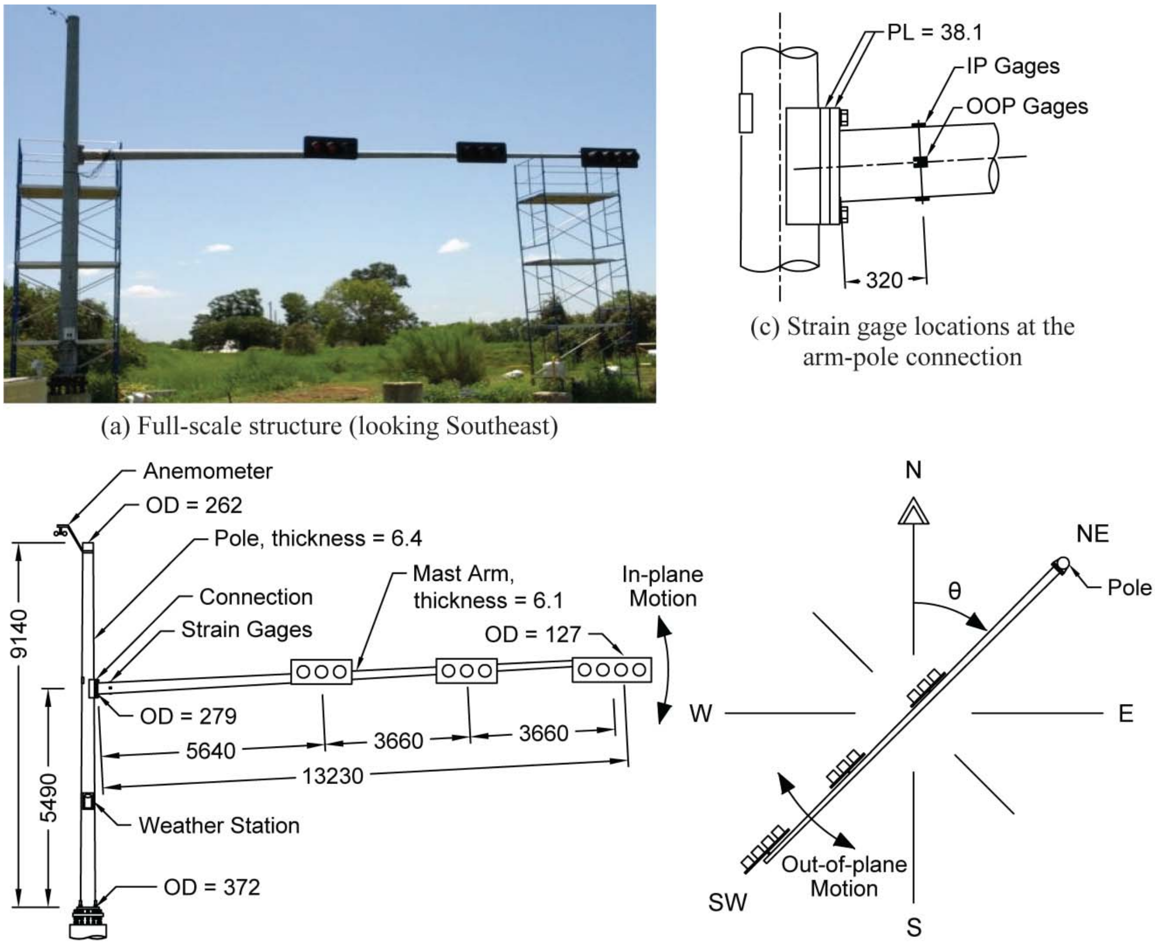
## 2. Experiments on a prototype traffic signal structure

To investigate the response of a traffic signal structure under realistic ambient conditions, field experiments were conducted in a natural wind environment. A decommissioned cantilevered traffic signal structure, with slender dimensions and a signal cluster orientation susceptible to large displacement vibrations, was installed atop a drilled shaft foundation at the Texas A&M University Riverside Campus in Bryan, Texas. Fig. 1 depicts the installed structure and the location of the installed instrumentation. The photograph in Fig. 1(a) shows the traffic signal structure that consists of two tapered cylindrical members. The dimensions of the tested structure are denoted in Fig. 1(b). A structure with an arm of this length is typically used to support three signal clusters.

Fig. 1(c) depicts the placement of several strain gages in regions where significant dynamic response was expected to occur. Strain gages were installed near the arm-pole connection, but far enough away from the welded transverse base plate to eliminate local stress effects. This distance is greater than the diameter of each section to ensure elimination of local stress effects. As a result, measured stresses are appropriately scaled to be representative of the nominal stresses at the section nearest the toe weld near the transverse plate. Weldable strain gages were used within a full-Wheatstone bridge configuration to measure in- and out-of-plane bending strains while allowing for temperature compensation and the exclusion of strain effects strain effects not specific to the intended measurement. All channels were continuously sampled at 100 Hz and the resulting voltage time histories were logged to a mobile data acquisition system.

To measure wind speed and direction, a weather station, including a windmill-style anemometer, was installed on the traffic signal structure pole as indicated in Fig. 1(b). The included sensor suite was also connected to the data acquisition system. To ensure that the response of the structure was captured over a wide variety of wind conditions, data was continuously logged and consisted of meteorological (wind speed and direction) and strain gage channels. The dynamic response of the standard structure was observed for a period of four and a half months, from April 24 to September 9, 2012. Following this time, strakes were affixed to the mast arm to investigate their efficacy in mitigating in-plane response for one month, from September 13 to October 13, 2012.

The orientation of the structure was such to achieve maximum response to the local wind environment, where the mast arm projected from the northeast to the southwest away from the pole, as shown in Fig. 1(d). Analysis of the available 40-year wind history for the region has shown the greatest proportion of the wind is derived from the Gulf Coast from the South (180°). As a result, about one-half of winds approach the rear face of the signal clusters. Such approaching winds lead to an amplified response (McDonald *et al.* 1995, Kaczinski *et al.* 1998, Letchford *et al.* 2008, Zuo and Letchford 2010, Cruzado *et al.* 2013). The recorded wind data was considered sufficient to establish similarity with trends found in the historical record.




(a) Full-scale structure (looking Southeast)

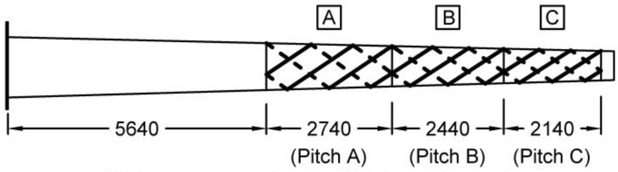
(c) Strain gage locations at the arm-pole connection

(b) Northeast to Southwest Elevation

(d) Plan view

Fig. 1 Experimental (a) traffic signal structure, along with its (b) dimensions and included instrumentation, (c) strain gage locations, and (d) orientation at Texas A&M University's Riverside Campus, Bryan, Texas (dimensions in mm)

(a) Triple-helix strake installation on the mast arm (looking Northeast)




(b) Mast arm strake application schematic

Fig. 2 Full-scale traffic signal structure with (a) helical strakes installed on the mast arm corresponding to (b) predetermined layout (dimensions in mm)

Typically, three equispaced strakes are provided on slender members, such that the projected height of the strake is approximately 10% of the diameter of the section with an installation pitch of 15 diameters (Kumar *et al.* 2008). Although such triple helix strakes have been shown to be an effective and economical method to control vortex-induced vibrations (in-plane response), they also give rise to a higher drag coefficient which may in turn affect the out-of-plane response of the mast arm. Regardless, braided polypropylene rope with a constant diameter of 16 mm was installed on the existing structure between the installed signal clusters by wrapping it around the exterior of the mast arm in the triple helix pattern as shown in Fig. 2(a). The helical strakes were attached to the arm using flexible bands (at minimum 3 per strake revolution) adhered to the strake and arm section using a heavy adhesive. Each segment (pitch length) denoted in Fig. 2(b) represents one complete revolution of the installed strakes using parameters presented in Table 1, where $d_{avg}$ refers to the average diameter of each tapered arm segment, as denoted in Fig. 2. The selected configuration was intended to achieve recommended strake size and pitch using a constant diameter strake.

Following the field acquisition of the data, post-processing of the recorded voltage time histories was performed in MATLAB. Similar to previous research by others (Zuo and Letchford 2010, Wieghaus 2015, Bartilson *et al.*, Wieghaus *et al.*), one-minute statistics were used to describe the ambient behavior. The weather station was set to record one-minute weather statistics (mean wind speed and direction). Aligning with this interval, equivalent stress ranges were described. Intrinsic functions were used (Mander *et al.* 1992, Shah 1993, Letchford *et al.* 2008, Wieghaus 2015, Bartilson *et al.*, Wieghaus *et al.*) to calculate one-minute statistics describing the ambient response with a constant amplitude sinusoid. Each one-minute statistic could then be paired with a corresponding wind condition. Since all points in the time history were used in this analysis, the equivalent one-minute constant amplitude stress range $S_r$ was inferred using

$$S_r = 2\sqrt{2}\sigma_{std} \tag{1}$$

where $\sigma_{std}$ = the standard deviation is taken over a one-minute moving window. Results from the analysis include one-minute statistics for wind (speed and direction) and structural response in terms of stress range. Fatigue equivalent stress ranges are later calculated and discussed.

Only wind-induced stress responses were recorded as the structure's location is not near an active roadway. Although response have been attributed to natural wind and traffic gusts, the response due to traffic effects are significantly lower than those due to natural winds (Chen *et al.* 2001, Albert *et al.* 2007). Consequently, it has been assumed that the monitored response is primarily responsible for the fatigue damage to these structures.

## 3. Deterministic fatigue analysis methods

Fatigue life using a constant amplitude stress range is typically defined in the form of

$$N = A\left(S_r\right)^{-3} \tag{2}$$

where $S_r$ = double amplitude (peak-to-trough) stress range amplitude and $A$ = AASHTO fatigue category coefficient as recommended from physical experiments (Frank 1980, Keating and Fisher 1986). The arm-to-base plate welded connection at the arm-pole connection has traditionally been compared against AASHTO Category E' (Fisher *et al.* 1983). Fig. 3 presents a comparison of relatively recent traffic signal connection fatigue tests of common traffic signal connections (Koenigs *et al.* 2003, Roy *et al.* 2011) against their AASHTO fatigue classification, where $A = 1.28\times10^{11}$ MPa. In the case that the 1-in-10,000 observed stress range exceeds the constant amplitude fatigue limit for "infinite life" analyses, a straight line extension of the S-N curve is to be used in fatigue analyses (Roy *et al.* 2011).

Following calculation of the one-minute responses over the observation period, a measure of fatigue equivalent stress range $S_{re}$ may be used to account for the variability in response at each incremental wind speed. The root mean cube stress range is an equivalent stress range that can be used for fatigue life estimation under variable amplitude stress ranges when using the governing AASHTO fatigue strength (S-N) curves above and below the constant amplitude fatigue limit (Yen *et al.* 2013). The fatigue equivalent response may be appropriately extrapolated for wind speeds higher than those observed.

Table 1 Design parameters for arm strake installation

| Segment | Pitch Length, $L_p$ (mm) | Average Diameter, $d_{avg}$ (mm) | $d_{rope}/d_{avg}$ | $L_p/d_{avg}$ |
|---|---|---|---|---|
| A | 2740 | 195 | 0.08 | 14 |
| B | 2445 | 165 | 0.10 | 15 |
| C | 2130 | 140 | 0.11 | 15 |

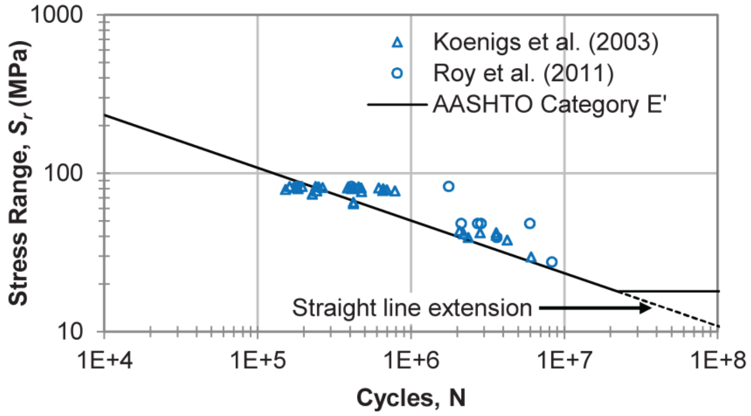



Fig. 3 Fatigue performance of mast arm-to-base plate connection (cycled about elevated mean stress)

The number of stress cycles undergone by the structure per year can be determined as a function of wind speed. Using historical wind records attained from the National Oceanographic and Atmospheric Administration (NOAA) at select locations of interest, probability distribution functions for wind speed may be empirically generated. These functions, along with the first in- and out-of-plane natural frequencies of the structure (1.09 Hz and 0.99 Hz, respectively), may be used to determine the number of cycles $n_U$ for a given wind speed $U$ as

$$n_U = t\, p_U\, f_n \tag{3}$$

where $t$ = the period of time under consideration in seconds; $p_U$ = the probability associated with wind speed $U$; and $f_n$ = the response frequency of the structure in Hertz.

Incorporating the fatigue equivalent stress range response as a function of wind speed, as well as the number of stress cycles expected to occur annually, the distribution of fatigue damage annually accumulated at the arm-pole connection may be found applying Miner's Rule (Miner 1945). For the case of traffic signal structure connections,this damage relationship is evaluated using the AASHTO Category E' fatigue classification. For which case, the total annual damage accumulated may be shown

$$D = \sum_U \frac{n_U}{N} = p_U\, f_n \left(\frac{S_r}{16}\right)^3 \tag{4}$$

where $N$ = the fatigue life (cycles) corresponding to the fatigue equivalent constant amplitude stress range amplitude at each wind speed. The inverse of annual fatigue damage is the fatigue or service life as determined using the deterministic means set fourth using AASHTO analysis concepts. Utilizing this procedure allows the effect of natural wind response to be visually related to fatigue damage and thus dependable service life.

## 4. Experimental observations

### 4.1 Response of standard (as-built) prototype structure

Fig. 4 presents the inferred stress range results as a function of wind speed for the standard structure based on experimental observations. As seen in Fig. 4(a), roughly one-half of all winds approached from a southerly direction. As previously mentioned, a measure of fatigue equivalent stress range was used to account for the variability in response at each incremental wind speed.

When comparing the total in- and out-of-plane response at the arm-pole connection as depicted in Figs. 4(b) and 4(c), respectively, it is clear that although the equivalent stress ranges begin similarly, the large dispersion in the across-wind response causes the equivalent response to vary significantly, between 3 m/s and 7 m/s. Despite a lack of observed wind speeds greater than 13.5 m/s, fitted trends indicate that out-of-plane response is greater than the in-plane response. This is typically the case with a response that arises due to the unsteady nature of natural winds (buffeting). Because the largest in-plane responses occur when winds approach from the back side of the structure (90°<θ<180°), the stress response is greater than when winds approach from the other directions. This variation in low-speed response is depicted in Fig. 4(d), where the equivalent stress range is shown to vary with wind direction. In contrast, the out-of-plane response appears wind direction invariant in Fig. 4(e). When studying the fatigue equivalent in- and out-of-plane response, two observations were clear: (1) the response is typically small; and (2) the in-plane response is more dependent on wind direction at lower wind speeds.

Based on these findings and the observations depicted in Fig. 4, the contribution of vortex shedding can be confirmed using the following rationale. Over 75% of all inferred one-minute in-plane stress ranges above 10 MPa occurred between 4 m/s and 6 m/s. Because the largest in-plane responses occurred when winds approached from the back side of the structure, the perpendicular wind speeds most affecting in-plane response center about 5 m/s. The vortex shedding frequency is often described by the non-dimensional Strouhal relationship

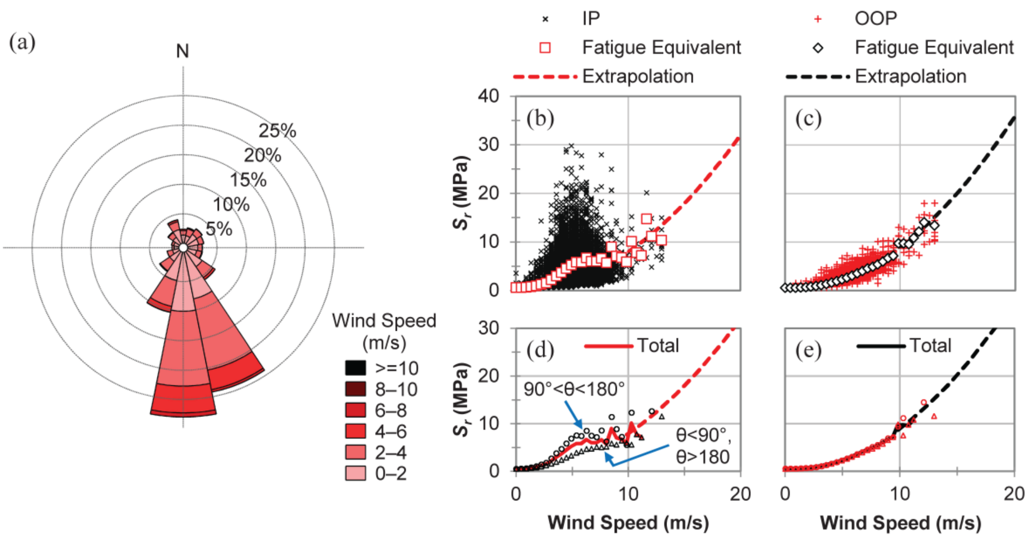



Fig. 4 Standard (unmodified) structure: (a) Measured wind conditions, total (b) in-plane and (c) out-of-plane stress responses at the arm-pole connection, (d) in-plane and (e) out-of-plane fatigue equivalent response as a function of wind speed and direction

$$St = \frac{f_{vs} D}{\overline{U}_{perp}} \tag{5}$$

where $St$ = Strouhal number, $f_{vs}$ = the vortex-shedding frequency, $D$ = cross-wind (characteristic) body dimension, and $\overline{U}_{perp}$ = mean speed of the oncoming flow. Using the experimentally determined 1.09 Hz in-plane natural frequency and a signal cluster backplate height of 600 mm, the Strouhal number when wind approached from the back side was 0.13. This corresponds with literature since the Strouhal number for a rectangular object with a small depth-to-height ratio is approximately 0.15 (Blevins 1977, Yu *et al.* 2013). As a result, it is concluded that large-amplitude in-plane observations are influenced by the propensity of signal cluster vortex shedding at lower wind speeds. The overall in-plane response is attributed to a combination of vortex shedding, galloping, and natural wind gusts.

As depicted in Fig. 4, the fatigue equivalent in- and out-of-plane responses were observed to increase with the square of wind speed when not influenced by across-wind effects, thus the extrapolations follow this relation. This response arises due to the unsteady nature of the along-wind response (buffeting). In a similar fashion, the observations clearly demonstrate that the out-of-plane response is direction independent and also attributed to buffeting.

### *4.2 Response of structure with arm modified with helical strakes*

The preceding results demonstrate that an ordinary traffic signal structure is prone to increased dynamic response due to across-wind effects. Observations made are similar to those previously reported (Zuo and Letchford 2010). However, their observations showed significant in-plane responses with winds approaching the front side of their large-diameter mast arm structure. Hence, mast arm vortex shedding was the excitation mechanism, not signal cluster vortex shedding (Letchford *et al.* 2008, Cruzado *et al.* 2013).

Regardless of this finding, helical strakes, traditionally a means to reduce vortex shedding from long circular sections, were installed on the mast arm of the cantilevered structure in an effort to appraise the viability of strakes as a response and connection fatigue mitigation measure. Helical strakes effectively disrupt the formation of a boundary layer, adversely affecting shear layer roll up, to prevent vortex formation and shedding (Kumar *et al.* 2008). As depicted in Fig. 2(a), braided rope was used to form helical strakes on the mast arm at an optimal arrangement utilizing three separate pitch lengths. Winds of primary interest were captured, allowing ample opportunity to complete the desired studies.

Fig. 5 depicts the response of the structure with mast arm strakes installed. As seen in Fig. 5(a), it was seen that almost one-half of the observed winds approached the structure from the backside. Therefore, a similar proportion of winds found to cause an increased low-speed in-plane response (direction dependent) were measured over the two observation periods. It was thus deemed appropriate to compare results obtained without and with arm strakes installed.

Fig. 5(b) indicates that the structure's arm-pole connection experienced no inferred one-minute in-plane stress ranges over 12 MPa. With the vast majority of observations under 10 MPa, the structure displayed no tendency to exhibit large across-wind lock-in vibrations. Arm strake installation decreased the in-plane fatigue equivalent stress response, nearly eliminating the influence of vortex-induced vibrations. When comparing the out-of-plane response at the arm-pole connection without and with strakes, the response appears unchanged. The fatigue equivalent in-and out-of-plane responses was extrapolated using the same relationship as used for extrapolating the response of the standard structure as the buffeting response remained similar. The additional drag assumed with strake installation showed no appreciable effect during observation.

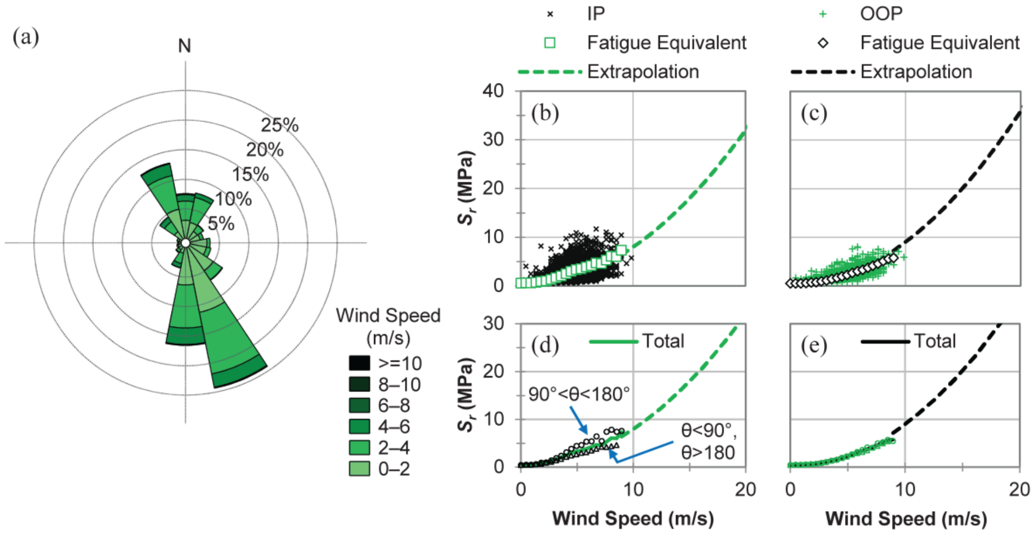



Fig. 5 Structure with arm strakes: (a) Measured wind conditions, total (b) in-plane and (c) out-of-plane stress responses at the arm-pole connection, (d) in-plane and (e) out-of-plane fatigue equivalent response as a function of wind speed and direction

## 5. Fatigue analysis and implications

The fatigue life of the structure is estimated considering changes in natural wind response as a result of arm strake installation for several distinct wind environments to assess (1) the role of vortex shedding on fatigue damage accumulation; and (2) the effectiveness of helical strake installation to extend the fatigue life. Based on the observed stress response at the pole-to-arm connection before and after strake installation, the fatigue life was estimated based on changes in natural wind response for four distinct wind environments: College Station, TX; Chicago, IL; Corpus Christi, TX; and Cheyenne, WY.

Fig. 6 not only depicts the process used to characterize the fatigue performance and fatigue life under several distinct wind environments, but more importantly depicts this analysis to determine in-plane fatigue for the standard and straked structure (Figs. 6(a) and 6(b), respectively). The first row depicts the in-plane stress range as a function of wind speed for the responses calculated and extrapolated using the individual one-minute observations at each incremental wind speed. As seen in Fig. 6, the difference in in-plane response reflects the reduction of low-speed across-wind effects. Correlating with the information presented in Figs. 4 and 5, it is evident that the in-plane across-wind fatigue equivalent response below 10 m/s is quite small in comparison to the response above 10 m/s, although it was observed that large, low-speed across-wind responses occur. It should be noted that each response above 10 m/s is based on extrapolation of stress range made previously based on experimental observations, where response extrapolation before and after strake installation is similar.

The second row of Fig. 6 represents the annual number of stress cycles undergone as a function of wind speed, whose shapes resemble the empirical probability distribution at each of the selected locations. The third row depicts the fatigue damage annually incurred as a function of wind speed, where the following row depicts the cumulative fatigue damage incurred over ranging wind speeds, whose sum is the inverse of service life.

As seen in the first column of the presented damage plots in Fig. 6, a fair amount of fatigue damage occurs atop the arm-pole connection at speeds related to lock-in across-wind effects for the standard traffic signal structure observed. This is apparent in all assessed wind environments. Through visual inspection of the damage plots without and with strakes installed, it is evident that the reduction in in-plane low-speed fatigue equivalent response nearly eliminates all amplified fatigue damage occurring at speeds below 10 m/s. This, however, has little impact on fatigue life unless the structure is located in a rather benign wind environment such as College Station, TX. As evidence, in Cheyenne, WY, where the probability of wind speeds above 10 m/s is quite high, significant fatigue damage is accumulated annually. As a result, for both the standard and straked arm structure in non-benign wind environments, a significant amount of accumulated fatigue damage is attributed to wind speeds over 10 m/s and is thus sensitive to the prevalence of winds above this threshold, as reflected in the fatigue life estimates depicted in the final row of Fig. 6.

Fig. 7 presents fatigue analyses to characterize the out-of-plane (along-wind) performance and dependable fatigue life of the traffic signal structure without and with arm strakes installed. Since the out-of-plane behavior of the structure showed little change with the addition of helical arm strakes, the out-of-plane fatigue performance showed little to no change. Further, extrapolated out-of-plane buffeting response is greater than the in-plane buffeting response, thus out-of-plane fatigue may control in certain wind environments. As was previously the case for in-plane fatigue, most out-of-plane fatigue damage accumulated is attributed to wind speeds over 10 m/s, and is thus sensitive to the proportion of winds that occur over this threshold.

## 6. Discussion

Following months of ambient response observations, change in the natural wind response of the structure was studied and quantified using fatigue equivalent parameters. Arm strakes nearly eliminated the influence of vortex-induced vibrations on the stress response. Using inferred one-minute statistics, the fatigue equivalent response was then extrapolated for higher wind speeds, where strake installation was not observed to change the expected response of the structure at high wind speeds. Using historical wind records from four distinct locations in the United States, the fatigue life of the structure was estimated considering any changes in natural wind response for several distinct wind environments to assess: (1) the role of vortex shedding on fatigue damage accumulation; and (2) the effectiveness of helical strake installation to extend the fatigue life of cantilevered traffic signal structures. Table 2 presents service life estimates related to in- and out-of-plane excitation, as each separately results in damaging stress reversals at different locations along the welded connection detail.

To relate these results against inspection records, Cheyenne, WY, was used for comparison. Based on the analyses performed, the predicted fatigue life in that wind environment is approximately 7 years (using Category E' extended). Previously compiled inspection records (Price *et al.* 2008) for 172 similar traffic signal structures in Cheyenne, WY, were statistically analyzed assuming random, right-sided censoring. Based on this analysis, the 2.5% non-exceedance probability corresponding to fatigue crack formation and/or detection relates to a life of approximately 8 years (Wieghaus 2015, Wieghaus *et al.*). Given the uncertainty in the fatigue resistance of these connections and that the fatigue category E' is assigned somewhat conservatively (Keating et al. 1986), it is not unreasonable that this AASHTO-type analysis would yield a dependable service life of some 7 years in that wind environment.

Based on the findings of the performed research as presented in Table 2, it is clear that the effectiveness of helical arm strakes as a fatigue life mitigation strategy depends greatly on wind environment. Fig. 8 depicts the impact of wind environment on the effectiveness of helical strake installation. Fig. 8(a) graphically depicts the results of the preceding fatigue analysis, fit with respect to wind speed. Additionally, regions are identified depicting the benefit of arm strake installation. As seen, the effectiveness of helical strake installation increases with decreasing mean annual wind speed. To further categorize wind environment, Fig. 8(b) relates the portion of commonly occurring winds below 10 m/s to damage accumulation. It can be seen that the corresponding fatigue life estimates directly coincide with this wind speed portion, where Cheyenne, WY, has the least expected life and College Station, TX, has the greatest. The effectiveness of mast arm strakes is related to the portion of winds that occur below 10 m/s, a measure to quantify the severity of the wind environment, whereas Figs. 6 and 7 demonstrate the influence of winds above 10 m/s on the fatigue service life of traffic signal structures.

During fatigue assessments, empirical distributions based on wind records attained from NOAA were used to describe wind speed distribution; extreme wind climatology was not incorporated into the analysis. Taking into account the probability of large windstorm events lessens the effectiveness of arm strakes in extending dependable life, however not significantly as it has been shown that more prominent winds are responsible for most fatigue damage accumulation. During an extreme wind event, however, the effectiveness of helical arm strakes in vibration and fatigue mitigation diminishes. Since arm strake installation only reduces the influence of across-wind effects at low wind speeds, it does not reduce the fatigue damage accumulated during high wind speed events where significant damage occurs. For this reason, the structure remains susceptible to large decreases in service life due to extreme events.

Table 2 Location-based fatigue life estimates based on historical wind records and inferred structural response

| Location | Mean Annual Wind Speed (m/s) | Configuration | In-plane Service Life (years) | Out-of-plane Service Life (years) |
|---|---|---|---|---|
| College Station, TX | 3.6 | Standard | **33.9** | 65.1 |
| | | Arm Strakes | **64.8** | 65.3 |
| Chicago, IL | 4.6 | Standard | **20.2** | 28.1 |
| | | Arm Strakes | **31.4** | 28.2 |
| Corpus Christi, TX | 5.3 | Standard | **14.6** | 18.5 |
| | | Arm Strakes | 21.3 | **18.5** |
| Cheyenne, WY | 5.7 | Standard | 7.3 | **6.8** |
| | | Arm Strakes | 8.5 | **6.8** |

*Critical case is in bold typeface

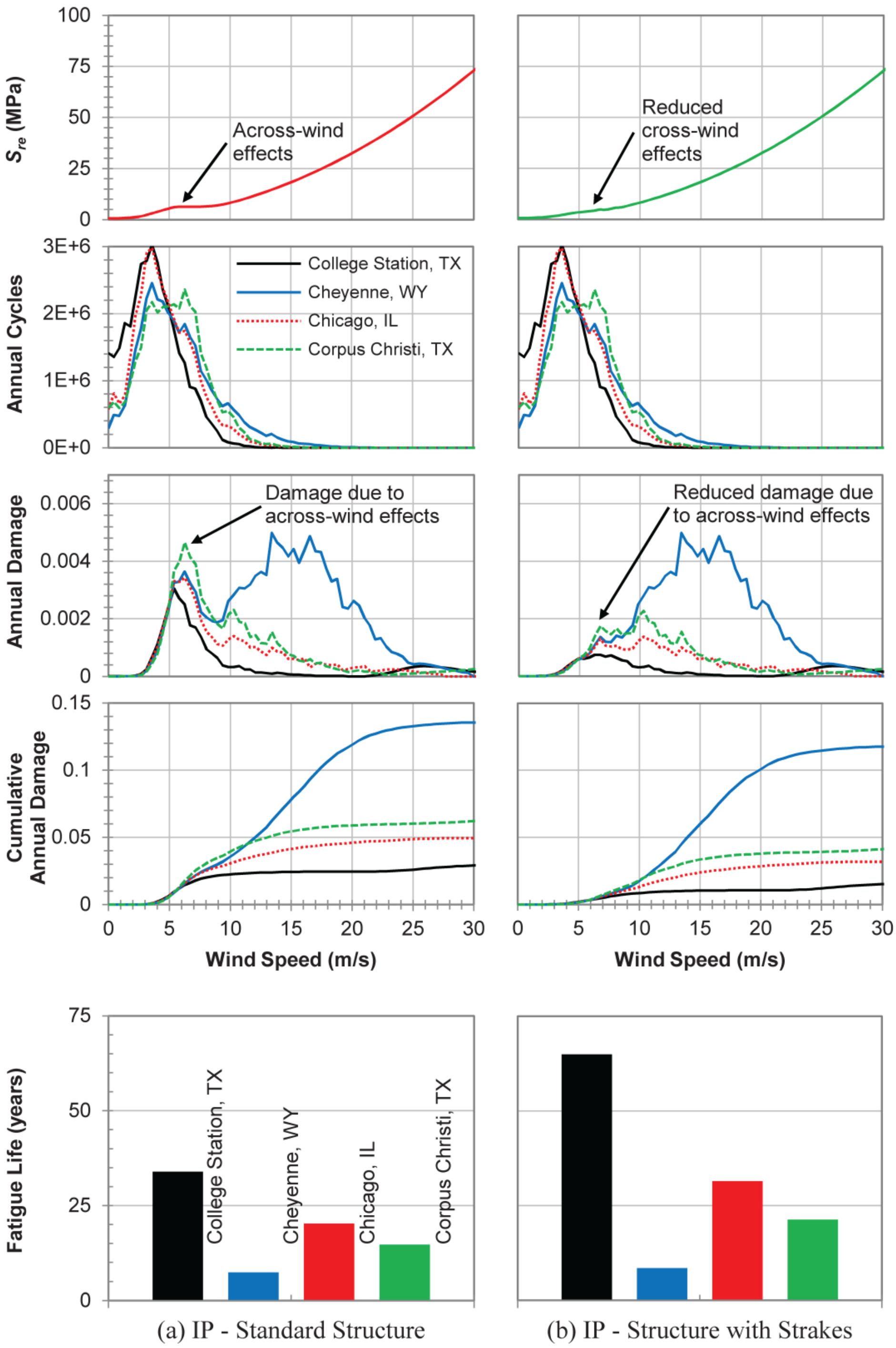



Fig. 6 The in-plane (IP) fatigue assessment for the (a) standard structure and (b) structure with helical arm strakes installed

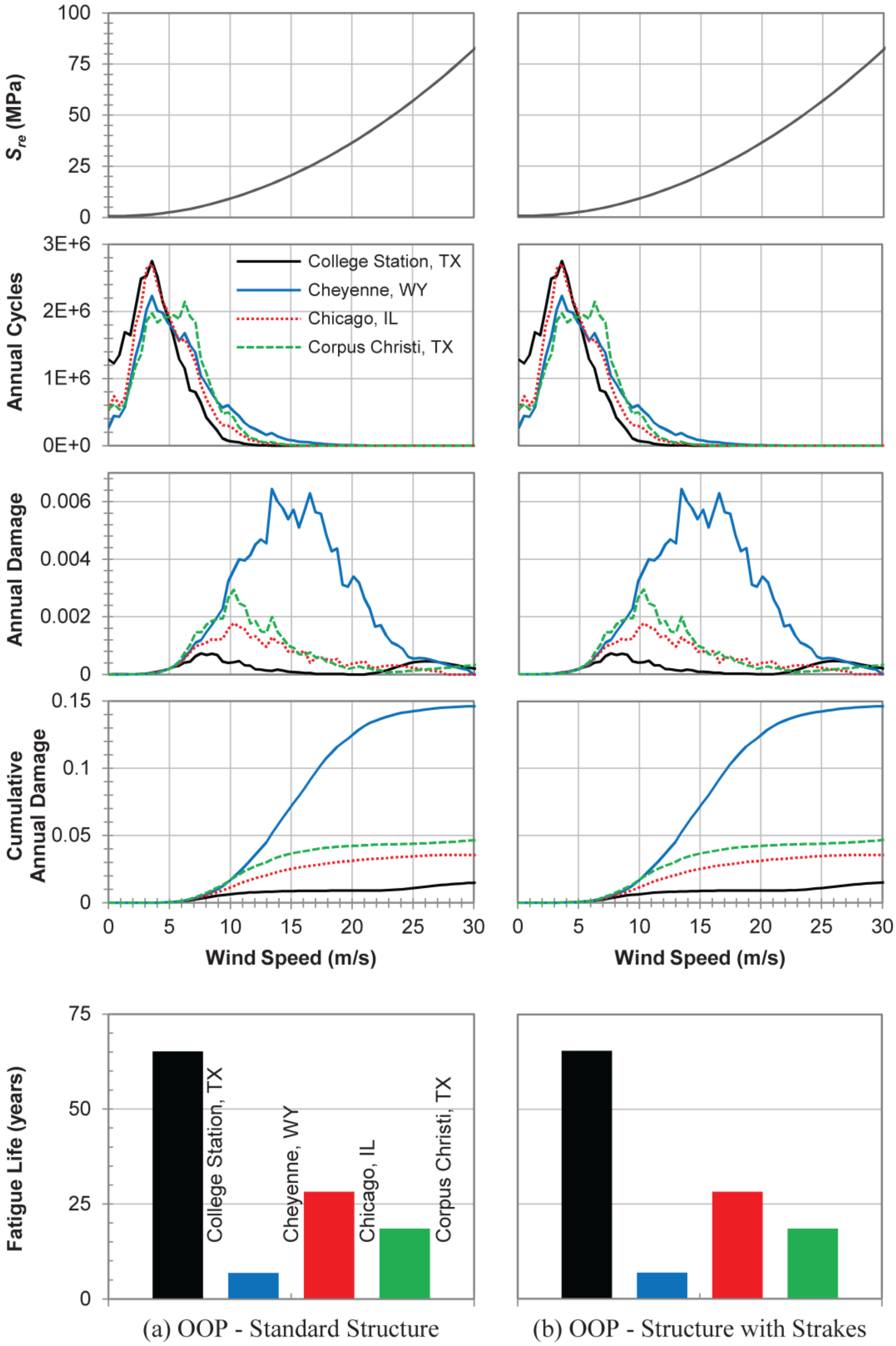
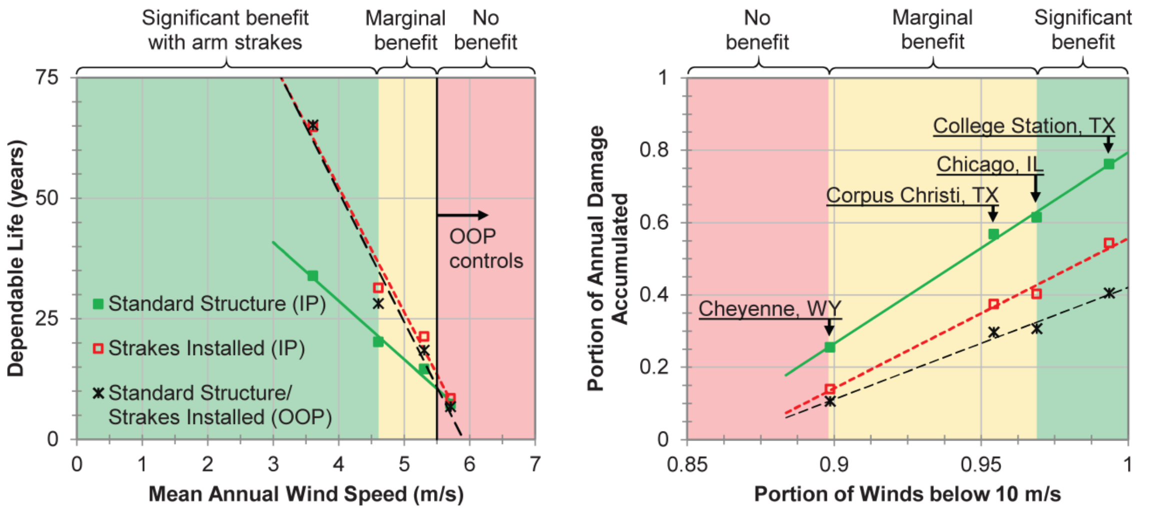



Fig. 7 The out-of-plane (OOP) fatigue assessment for the (a) standard structure and (b) structure with helical arm strakes installed




(a) Dependable life with mean wind speed (b) Common wind influence on damage accumulation

Fig. 8 Influence of local wind environment on arm strake mitigation viability

In addition to the work presented herein, further natural wind testing in conjunction with computational modeling should be conducted to further investigate the efficacy of helical mast arm strakes for a broader variety of traffic signal structure dimensions and attachment configurations. It may be possible for future developments to further improve strake effectiveness with the potential for in-plane dependable life to surpass the life related to out-of-plane excitation. Therefore, it is also important to strive for improved performance for out-of-plane motion. One possibility may be to use louvered signal cluster (visual) backplates or other damping devices (tuned mass or viscous) to further reduce out-of-plane response. While these methods reduce the fatigue demand, it is considered more important to improve the fatigue resistance capacity. To this end, the authors are presently investigating the reduction of connection mean stress to improve dependable fatigue life.

## 7. Conclusions

Based on recent findings that identify vortex shedding as a significant excitation mechanism responsible for amplified across-wind traffic signal structure responses at low wind speeds, helical strakes were installed to reduce wind-induced response. To investigate the response of a traffic signal structure without and with helical arm strakes, field experiments were conducted in a natural wind environment. The orientation of the structure was such to achieve maximum response to the local wind environment to best study the effect of helical strake installation.

The installation of traditionally-effective helical arm strakes to the cantilevered traffic signal structure mast arm significantly reduced the large amplitude response associated with vortex shedding of the signal clusters. This demonstrates that helical strakes can effectively disrupt the formation of a boundary layer and vortex formation on the back side of the signal clusters. Consequently, helical arm strakes are effective in reducing across-wind response at low wind speeds.

The effectiveness of helical arm strake installation in improving the fatigue life of cantilevered traffic signal structures greatly depends on the role of across-wind effects, and in particular vortex shedding, on fatigue damage accumulation. Arm strake installation has been shown to be very effective at reducing in-plane across-wind effects at low wind speeds, thus greatly reducing fatigue damage accumulated at those speeds. As a result, the practical benefit of helical arm strake installation on service life depends on local wind environment, where over 90% of all winds should be below 10 m/s. Because helical arm strakes did little to change the out-of-plane response of the cantilevered traffic signal structure, their installation is of little benefit, if any, in increasing the out-of-plane service life as assessed in this work. Unfortunately, the installation of helical arm strakes is not a panacea for mitigating connection fatigue.

Although strakes were shown to be effective at reducing in-plane fatigue damage accumulation for existing structures in mild wind environments, alternative fatigue mitigation measures should be sought in an effort to find a single mitigation solution effective across all wind environments. Instead of attempting to mitigate fatigue by reducing the demand, new methods are emerging whereby the improved fatigue resistance (capacity) is achieved through post-tensioning the connections (Hurlebaus and Mander 2014).

## Acknowledgements

This work was partially funded by the Texas A&M Transportation Institute (TTI) and is gratefully acknowledged. The authors would also like to thank the Texas Department of Transportation (TxDOT) for donating the traffic signal support structure and signal clusters, and the City of College Station for donating a controller cabinet.

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