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Fragility analysis of wind-excited traffic signal structures

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ABSTRACT

Traffic signal structures are prone to fatigue failure due to wind-induced excitation that is often difficult to characterize due to the variable response arising from a combinations of along- and across-wind effects and the uncertainties associated with the fatigue and fracture phenomena. A probabilistic framework is proposed to predict structure component fatigue life. To solve the problem of wind-induced fatigue, the model is introduced to inter-relate wind hazard to structural response and hence fatigue damage. Stress range demand variabilities are considered separately from the randomness associated with the fatigue capacity. To characterize the natural wind response of a representative structure, full-scale structural monitoring is conducted, and the data were incorporated into an example application of the framework. To successfully demonstrate the efficacy of the proposed framework, resulting wind-induced fatigue life distributions are compared against compiled inspection records for a large traffic signal structure population in Wyoming.

1. Introduction

Traffic signal structures of the type shown in Fig. 1 [1] that consist of a horizontal steel arm and vertical tubular pole are commonly used at intersections and other locations. Signal clusters are attached either horizontally or vertically at various locations along the length of the mast arm. Such a traffic signal support structure may often exhibit large-amplitude, wind-induced vibrations that over time can result in reduced fatigue life and failure of the structure. Due to the lack of redundancy and increasing span length of these structures, arm-to-pole connection fatigue is increasingly becoming of concern, especially for existing, aging structures.

In this paper, the viability of modeling wind-induced fatigue damage and hence the fatigue life of a steel structure using a direct probabilistic approach is investigated. The accumulation of fatigue damage in lightweight, wind-excited structures can be estimated by modifying the four-step direct approach of [2], which builds upon the developments of previous work in seismic risk assessment [3–5]. Specific to the work presented herein, the framework is divided into four subtasks, namely: (a) wind hazard analysis which defines the hazard intensity (wind speed) at a particular location; (b) the observed structural response that relates cyclic stress amplitudes to wind speed excitation for a structure; (c) the fatigue damage analysis which is related to cyclic stress amplitudes for as-built structural details; and (d) probable fatigue-life estimation, from which the annual rate of damage and hence fatigue life (fragility) can be assessed.

One underlying problem related in determining the fatigue life of lightweight steel traffic signal structures due to wind excitation has been the lack of consensus in determining the excitation mechanism responsible for large amplitude vibrations [1,6–9]. To this point, several advanced quantitative assessment techniques have been proposed to determine the fatigue life of wind-excited steel pole structures [10–16], but each requires a good understanding of the excitation mechanisms for modeling purposes. Such techniques can be computationally intensive. Works specifically oriented to assess the fatigue life of cantilevered signal structures have also had shortcomings. The response of these structures have been based on the full-scale response of a structure excited by a single blower [17], the brief response following natural wind gusts [18], or scaled wind tunnel results where the scale factor was not precisely known [7]. If fatigue is to be fully addressed in a probabilistic fashion, the structural response demands due to natural wind must also, where such demands are compared with the probabilistic fatigue capacity of the critical fatigue-prone connections.

Unlike past works [7,17,18], or recently recommended deterministic methods [19], to estimate the fatigue life of steel support structures, aleatoric and epistemic uncertainties present in the analysis must be considered. The research described herein modifies and expands upon the approach presented in [2] to quantify fatigue damage accumulated by lightweight steel traffic support structures under natural wind excitation. This information, along

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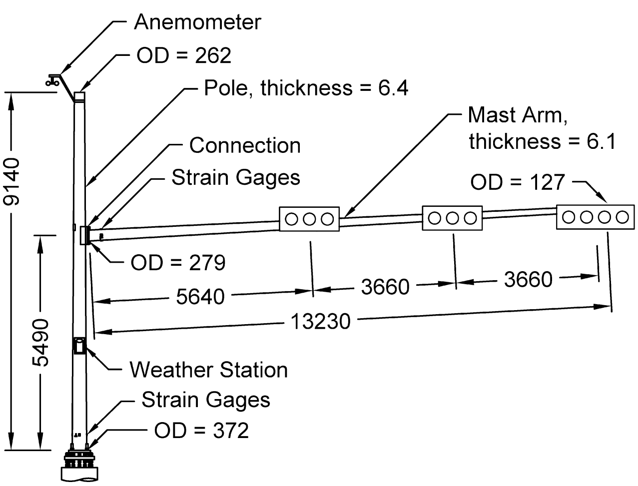
0141-0296/

(a) Full-scale traffic signal support structure



(b) Traffic signal dimensions (mm) and instrumentation schematic.

Fig. 1. Experimental traffic signal structure [1].

with quantified uncertainties, is used to form probability-based fatigue life fragility curves for these wind-excited structures. Thus, dependable service life may be specified at a given level of non-exceedance.

As a result, this paper first explains the four-step methodology specifically cast to predict the fatigue life of lightweight wind-excited steel structures in a probabilistic (fragility) manner. To illustrate the application of the proposed method, each step in the framework is detailed to enable the determination of fatigue damage accumulation in cantilevered signal structures due to natural wind excitations. Differing from previous works, structural response is based on long-term field observations of stress range versus wind speed of a prototype structure in Bryan, TX, depicted in Fig. 1(a) with dimensions given in Fig. 1(b). To successfully demonstrate the efficacy of the proposed framework, resulting wind-induced fatigue life distributions are compared to analysis performed on compiled inspection records for a large population of structures in southern Wyoming. Based on ease of implementation and the quality of results derived from the analyses performed herein, the proposed risk assessment methodology demonstrates promise for application in other areas.

2. Theoretical direct damage estimation framework

The four-step approach is introduced to estimate the fatigue damage accumulated in fatigue-prone connections of wind-excited structures by relating damage levels to recurrence rate. Each step in the framework is based on the central tendency, or median, of each involved variable. Shown in Fig. 2, the primary objective of the direct four-step approach in computing fatigue life is to relate the estimated damage to (local wind) demands and structural capacity measures. Fig. 2 visually demonstrates the interaction of the four plots, from (a) to (d), when plotted using log–log scales. These inter-relationships are best described using piecewise power functions and are described in the following paragraphs.

Power functions may be used based on the observation that most of the relationships represented in Fig. 2 can be simplified as linear segments in log–log space. The interaction of the four-step damage estimation approach may be consolidated to a single compound equation

$$\frac{D_{i+1}}{D_i} = \left|\frac{S_{r,i+1}}{S_{r,i}}\right|^{c_i} = \left|\frac{u_{i+1}}{u_i}\right|^{b_i c_i} = \left|\frac{\lambda_{i+1}}{\lambda_i}\right|^{a_i b_i c_i} \tag{1}$$

where $D =$ damage fraction; $S_r =$ stress range; $u =$ wind speed; and $\lambda =$ recurrence rate for a given wind speed. The alphabetically related exponents are equal to the slopes in log–log space of graphs $a, b, c,$ and d of Fig. 2 between points i and $i+1$, respectively. The relationships presented in Eq. (1) and the graphical representation hold true because the scales of two neighboring plots (one beside and one above/below) have similarly scaled axes.

2.1. Local hazard model

The first subtask in the proposed methodology involves analyzing the wind hazard at a particular location. Beginning with Fig. 2(a), the local wind hazard is represented by relating a wind intensity measure, the hourly mean wind speed u, to the recurrence rate λ of events $U > u$ in hours per year. When discretized, the power function for each linear segment relates wind speed to recurrence rate by

$$\frac{u}{u_i} = \left|\frac{\lambda}{\lambda_i}\right|^{a_i} \tag{2}$$

over the interval $u_i < u < u_{i+1}$ where a_i is determined between consecutive points.

For example, the local wind environment is characterized by the wind hazard model at a specific location. As exemplified in Fig. 3(a), historical, hourly wind records near the test site in Bryan, TX (Easterwood Airport, College Station, TX, 11 km from the installed test structure) are attained from the National Oceanic and Atmospheric Administration (NOAA) and analyzed independent of wind direction. An empirical probability distribution function of hour-averaged wind speeds may be generated from the historical records. Extreme wind climatology is taken into account to evaluate the stress response that may occur during the life of a traffic support structure. If wind records are over 20 years in length, it is reasonable to extrapolate the probability density for winds with a recurrence rate $\lambda < 1$ h/year (return period for hourly wind speed greater than one year). Thus, an Extreme Type I (Gumbel) distribution may be used to fit to the extreme data from the attained records [20]. Recurrence rates are then determined using the composite cumulative distribution of wind speed.

2.2. Structural response model

The second subtask in the proposed framework involves structural response analysis relating stress response to a given level of wind excitation for a particular structure. Following the arrow downward from the point in Fig. 2(a) to a correlating point on the curve shown in Fig. 2(b), it is shown that the structural