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Reliability consideration for fatigue design of sign, luminaire, and traffic signal support structures under wind load

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ABSTRACT

Fatigue failure of sign, luminaire, and traffic signal support structures has been observed. One of the causes is attributed to the along-wind loading (i.e., the natural wind gusts). The Canadian Highway Bridge Design Code (CHBDC) is not specific on the fatigue design wind pressure for these structures. The fatigue design wind load for the support structures recommended in American Association of State Highway and Transportation Officials (AASHTO) is developed based on the "infinite-life" approach, and by assuming that the stress range can be estimated considering that the response due to natural wind gusts can be represented as a constant amplitude sinusoid. The adequacy of this assumption and the implied structural reliability by using the recommended fatigue design wind load are unknown. This study assesses the statistics of the stress range caused by natural wind gusts for the support structures that are amenable for simplified structural representation. The statistics are used together with a probabilistic model of hourly-mean wind speed and fatigue capacity to estimate the fatigue reliability. The results indicate that the recommended fatigue design requirement for the support structures in AASHTO results in a widely varying reliability index. This is because the recommendation does not take into account the damping ratio or stress cycles or spatially varying statistics of wind climate. The results are also used as the basis to suggest an alternative requirement for fatigue design. The parameters involved in the requirement are calibrated based on selected target reliability, and a ready to use Canadian wind map for the fatigue design is provided.

1. Introduction

Sign, luminaire, and traffic signal support structures play a significant role in the traffic management system. Their safe and intended performance is necessary to maintain adequate traffic flow and safety. However, there are documented failures of these systems due to fatigue in many locations, including those in the States of California, Missouri, Texas and Wyoming (Hartnagel and Barker, 1999; Chen et al., 2001; Dexter and Ricker, 2002). The causes of fatigue failure are attributed to galloping, vortex shedding, natural wind gusts and/or truck induced wind gusts (Kaczinski et al., 1998; Chen et al., 2001; Li et al., 2006; Letchford and Cruzado, 2008).

There is no consensus on whether the traffic signal structures are susceptible to vortex shedding (Kaczinski et al., 1998; Letchford and Cruzado, 2008). The passing traffic can induce back-and-forth (or out-of-plane) bending as well as up-and-down (or in-plane) bending of the mast arm of cantilever sign support structures. Whether the design should consider the truck induced gust pressures in the vertical or horizontal direction was discussed in Kaczinski et al. (1998), Hartnagel and Barker (1999), Chen et al. (2001), and Letchford and Cruzado (2008). The studies for the natural wind gusts for the support structures (i.e., sign, luminaire, and traffic signal support structures) given by Kaczinski et al. (1998) and Johns and Dexter (1998) have lead to the recommended fatigue design wind load implemented in AASHTO (2001, 2009). In deriving the recommendation for the fatigue design wind load, it was assumed that the use of "infinite-life" approach (i.e., 0.01% or fewer cycles exceeding the constant-amplitude fatigue limit (CAFL)) for fatigue design is adequate, and stress range can be estimated based on the assumption that the response due to natural wind can be considered as a constant amplitude sinusoid. The consideration of 0.01% or fewer cycles exceeding the CAFL has lead Kaczinski et al. (1998) and Johns and Dexter (1998) to adopt the (1–0.01%)-quantile of the hourly-mean wind speed, $U_{0.01\%}$; the assumption of constant sinusoid response has lead these studies to consider that the stress range equals 2.8 times the standard deviation of the response due to fluctuating wind. This assumption implicitly ignores the potential effect of damping. By combining these, the basic wind pressure is estimated to be 250 Pa (for $U_{0.01\%}$ equal to 17 m/s). However, to our

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knowledge, no verification has been carried out to investigate if the estimated stress range in such a manner is adequate; the implied reliability by using the mentioned approach is unavailable in the literature.

Besides the loading, the parameters and the CAFL for the fatigue design in AASHTO (2001) are taken from AASHTO Standard specifications for highway bridges (AASHTO, 1996), which are used for redundant load path structures. Letchford and Cruzado (2008) indicated that such an attitude may be due to the fact that the adopted values are for more than 2 million stress cycles, and that perhaps Kaczinski et al. (1998) felt that "the 'nonredundant' values were too conservative for cantilever supporting structures of signs, signals, and lighting". Moreover, the potential effect of the uncertainty in the structural fatigue capacity is not explicitly discussed in recommending the fatigue design wind load for the support structures.

The above mentioned studies are focused on the development of the fatigue design practice for the support structures in AASHTO. Although the design of the support structures in Canada is governed by the Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-06, 2006), the code is not specific on the fatigue design wind pressure for these structures. Therefore, there is a need to carry out a reliability-based design code calibration focused on the support structures for the CHBDC and to suggest a fatigue design requirement for their design. The main objectives of this study that addresses this need are to: investigate the stress range distribution for the support structures under natural wind gusts; assess the statistics of hourly-mean wind speed applicable to Canada; estimate the implied fatigue reliability if the infinite-life approach used to develop fatigue design wind load in AASHTO (2001) is considered; and recommend a reliability-based fatigue design wind pressure for CHBDC. The analysis procedure and results leading to these objectives are described in the following sections. The fatigue reliability estimation for galloing, vortex shedding, and/or truck induced wind gusts, that is beyond the scope of the present study, deserves future investigation.

2. Stress range distribution of simplified structural system under stochastic wind load

2.1. Wind statistics and wind spectrum

Wind is characterized by mean and fluctuating components for the purpose of estimating the structural responses. Studies related to the Canadian structural design codes under wind load for the ultimate limit state are focused on the statistics of annual maximum hourly-mean wind speed rather than the statistics of the hourly-mean wind speed, although for the fatigue limit state the latter is required. To obtain an overview of the statistics of the hourly-mean wind speed for Canadian sites, historical wind speeds recorded at 14 meteorological stations across Canada are considered. The stations, which are shown in Table 1 and Fig. 1, are located in the national capital and the capital cities of each province and territory.

Wind speed records for the considered stations are obtained from the Environment Canada (EC) HLY01 digital archive. The archive has been maintained by EC since January 1953. The reported wind speed in the archive consists of 1- or 2-min average wind speed recorded just before the top of the hour, or 10-min average wind speed recorded just before the top of the hour. To obtain the wind speed for the standard condition that is referred to in most design codes (i.e., open terrain at 10 m height), the wind speed measurements at each station were adjusted for anemometer height and for exposure or roughness corrections. For the height adjustment, the power law with an exponent of 1/7 (NRCC, 2010) is employed. Exposure or roughness corrections considering the surrounding terrain conditions are based on a simplified version of the method recommended in ESDU (2002) (Mara et al., 2013). This simplification uses a single correction factor for all directions, rather than wind direction-dependent correction factors. The assessment of the uncertainty due to anemometer type and instrumentation are not considered because of the lack of detailed information. It is considered that the adjusted wind speed is representative of hourly-mean wind speed; it could be conservative but by less than 5% (Hong et al., 2014). The sample mean and standard deviation of the (adjusted) hourly-mean wind speed U for the considered 14 stations are shown in Table 1 as well. The results show the mean varying from 3 to 6.9 m/s. The calculated values of the coefficient of variation (cov) of U, ν_U, are within 0.55–0.84.

Table 1
Selected Canadian meteorological stations and estimated return period values of hourly mean wind speed.

Location	Prov.	Climate ID	# of years	Statistics of the hourly-mean wind speed				
				Mean (m/s)	St. dev. (m/s)	cov	α	β (m/s)
Victoria Int'l A	BC	1018620	46	3.01	2.27	0.75	1.339	3.28
Whitehorse A	YT	2101300	47	3.74	2.85	0.76	1.327	4.07
Yellowknife A	NT	2204100	46	3.79	2.29	0.60	1.702	4.25
Iqaluit A	NU	2402590	46	4.13	3.46	0.84	1.199	4.45
Edmonton Int'l A	AB	3012205	50	3.47	2.39	0.69	1.476	3.84
Regina Int'l A	SK	4016560	46	5.32	3.06	0.58	1.796	5.98
Winnipeg Int'l A	MB	5023222	46	4.93	2.85	0.58	1.788	5.54
Ottawa Int'l A	ON	6106000	50	4.32	2.70	0.63	1.639	4.82
Toronto Int'l A	ON	6158733	47	4.76	3.10	0.65	1.570	5.30
Quebec Int'l A	QC	7016294	42	4.14	2.90	0.70	1.450	4.57
Fredericton A	NB	8101500	42	3.70	2.66	0.72	1.412	4.07
Halifax Int'l A	NS	8202250	50	5.14	2.90	0.56	1.837	5.79
Charlottetown A	PE	8300300	40	5.01	2.74	0.55	1.898	5.64
St. John's A	NL	8403506	34	6.87	3.86	0.56	1.844	7.73

Also, the data are used to fit the Weibull distribution, $F_U(U)$,

$$F_U(U) = 1 - \exp(-(U/\beta)^{\alpha}), \tag{1}$$

where α and β are the distribution parameters. The fitted distributions using the method of moments are illustrated in Fig. 2 together with the empirical cumulative distribution; the obtained distribution parameters are depicted in Table 1. Since the use of the (1–0.01%)-quantile of U, $U_{0.01\%}$, for the fatigue design is suggested by Kaczinski et al. (1998), the estimated $U_{0.01\%}$ using the fitted distribution is tabulated in Table 2 and compared with that obtained from the empirical cumulative distribution. The comparison shows good agreement except for Whitehorse and Iqaluit, where the fittings are inadequate in the upper region of the empirical distribution (see Fig. 2). If only the upper tail of the distributions is of interest, the use of least-squares method to fit the upper tail region can be considered, in such a case, the cov values estimated from the fitted distribution are smaller than those calculated directly from the samples.

As the hourly-mean wind speed follows the Weibull distribution, the annual maximum hourly-mean wind speed, U_{AH}, follows the Gumbel, distribution, $F_{GU}(x)$ (Jordaan, 2005),

$$F_{GU}(u_{AH}) = \exp(-\exp(-(u_{AH} - u_n)/a_n)), \tag{2a}$$

where,

$$u_n = \beta(\ln n)^{1/\alpha}, \text{ and } a_n = \beta(\ln n)^{1/\alpha - 1}/\alpha \tag{2b}$$

u_{AH} is the value of U_{AH}, and n equal to 8766 is the number of hours in a year. The mean and standard deviation of the Gumbel variate defined in Eqs. (2a) and (2b) are $u_n + 0.5772a_n$ and $1.2826a_n$. The estimated mean and standard deviation of U_{AH} in this manner