Technical Note

Risk-Informed Mean Recurrence Intervals for Updated Wind Maps in ASCE 7-16

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Abstract: ASCE 7 is moving toward adopting load requirements that are consistent with risk-informed design goals characteristic of performance-based engineering (PBE). ASCE 7-10 provided wind maps that correspond to return periods of 300, 700, and 1,700 years for Risk Categories I, II, and combined III/IV, respectively. The risk targets for Risk Categories III and IV buildings and other structures (designated as essential facilities) are different in PBE. The reliability analyses reported in this paper were conducted using updated wind load data to (1) confirm that the return periods already in ASCE 7-10 were also appropriate for risk-informed PBE, and (2) to determine a new risk-based return period for Risk Category IV. The use of data for wind directionality factor, K_d, which has become available from recent wind tunnel tests, revealed that reliabilities associated with wind load combinations for Risk Category II structures are, in fact, consistent with the reliabilities associated with the ASCE 7 gravity load combinations. This paper shows that the new wind maps in ASCE 7-16, which are based on return periods of 300, 700, 1,700, and 3,000 years for Risk Categories I, II, III, and IV, respectively), achieve the reliability targets in Section 1.3.1.3 of ASCE 7-16 for nonhurricane wind loads. **DOI:** 10.1061/(ASCE)ST.1943-541X.0002011.

Introduction

ASCE 7 is moving toward load requirements that are intended to be consistent with reliability-based design goals stipulated in performance-based engineering (PBE). The enabling requirements for PBE are found in Section 1.3.1.3 of ASCE 7-10 (ASCE 2010). The numerical target reliability goals for buildings and other structures in Risk Categories I–IV (Table 1.5-1 of ASCE 7-10), which formerly appeared in Commentary C1.3.1, are now are located in Section 1.3.1.3 of ASCE 7-16 (ASCE 2017). The target reliability GGgoals for nonseismic design are based on component performance (Ellingwood et al. 1980; Galambos et al. 1982), while the target reliability goals for earthquake-resistant design are consistent with the probability of failure of a structural system (NEHRP 2012). For example, the reliability-based seismic performance goal for Risk Category II buildings is a 1% (or less) probability of collapse in 50 years (ASCE 7-10, Section 21.2.1). These seismic reliability-based performance goals provide a uniform risk of failure or collapse, rather than a uniform probability of hazard occurrence at all sites.

Similarly, the wind load provisions in ASCE 7-10 introduced, for the first time, a set of wind maps based on the notion of uniform wind *risk* (i.e., probability of failure) rather than uniform wind *hazard* (i.e., probability of hazard occurrence), stipulating return periods of 300, 700, and 1,700 years for the 3-s gust wind speeds for Risk Categories I, II, and combined III/IV, respectively. At the same time, the previous wind load factor 1.6 was reduced to 1.0, making the uniform risk approach for wind similar to that for earthquake effects. These uniform risk maps resolved the perennial difficulty of ensuring essentially uniform risk in hurricane-prone and nontropical regions of the United States, but were not entirely consistent with the target reliability goals stipulated in Commentary C1.3.1.3. The target reliabilities for PBE were moved from C1.3.1.3 of ASCE 7-10 to Section 1.3.1.3 of ASCE 7-16. Concurrently, new wind data became available that prompted a reexamination of the wind speed return periods. Furthermore, separate wind maps for Risk Categories III and IV needed to be developed for ASCE 7-16 using a structural reliability-based procedure described in this paper to support the reliability goals for Risk Categories III and IV structures in Section 1.3.1.3 of ASCE 7-16.

This paper summarizes the reliability analyses conducted to determine mean wind speed return periods to meet the reliability goals for Risk Categories I–IV. In addition to utilizing an expanded database (Lombardo et al. 2009; NIST 2012), a key part of these analyses was a reexamination of the wind directionality factor that supported the original development of the wind load combinations in Section 2.3 of ASCE 7 using recent wind tunnel data.

Wind Directionality Effects on Reliability Assessment

The wind speeds and aerodynamic coefficients stipulated in ASCE 7 [and its predecessor, ANSI A58 (ANSI 1982), which dates back to before 1972], are independent of direction and are maximum or envelope values. It has been recognized for more than three decades that the most unfavorable orientation of a structure and the most unfavorable wind direction rarely coincide, and that some reduction in the forces determined from those direction-independent quantities is warranted. This directionality effect was first introduced into the load combinations for strength design in ANSI A58, Section 2.3, in 1982, where it was embedded in the wind load factor of 1.3. In the 1998 edition of ASCE 7 (ASCE 1998), an explicit wind directionality factor, $K_d = 0.85$, was introduced for rectangular buildings. This factor has remained essentially unchanged

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(directionality factors are listed in ASCE 7-10, Table 26.6-1), while the wind load factor was increased from 1.3 to 1.6. The wind directionality factor accounts for two effects: (1) a reduced probability of maximum winds coming from any given direction, and (2) a reduced probability of the maximum pressure coefficient occurring for any given wind direction. The current value of $K_d = 0.85$ is based on a relatively simple directional analysis conducted as part of the original ANSI A58/ASCE 7 load factor development three decades ago (Ellingwood et al. 1980; Ellingwood 1981), at a time when only limited data on the effects of wind directionality were available. This directionality factor was derived independently from the reduction factor 0.8 recommended by Davenport (1983) for a similar purpose, which was adopted by the Canadian code.

One of the underlying assumptions of previous wind reliability analyses (Ellingwood et al. 1980, 1982) was that the wind directionality factor, $K_d = 0.85$, is statistically unbiased, i.e., the mean value of K_d, which is a random variable, is 0.85. While this assumption appeared to be confirmed by a later Delphi study (Ellingwood and Tekie 1999), recent data presented by Isyumov et al. (2014) and Habte et al. (2014) indicate that that assumption was conservative. Isyumov et al. (2014) presented relative frequency diagrams of wind directionality effects on building accelerations, moments, and pressures, which were determined from wind tunnel studies of three buildings. The relative frequencies for flexure (moments) presented by Isyumov et al. (2014) and Habte et al. (2014) are consistent with the notion of an equivalent uniformly distributed load (EUDL), which is customarily used in reliability analysis for strength limit states (Ellingwood et al. 1980). The mean directionality factors listed by Isyumov et al. (2014), illustrated in Fig. 1, are substantially lower than 0.85 for extratropical regions, but only slightly less than 0.85 for hurricane (tropical) regions. Data presented in Habte et al. (2014) lead to similar conclusions.

The Wind Load Subcommittee of ASCE 7 stipulated (and the ASCE-7 Main Committee) that the current values of K_d in Table 26.6-1 of ASCE 7-10 should be retained in ASCE 7-16. However, the conservative bias in K_d must be taken into account in the reliability analyses for determining the wind speed return periods required to meet the target reliability goals in Section 1.3.1.3 of ASCE 7-16, as described in the following section.

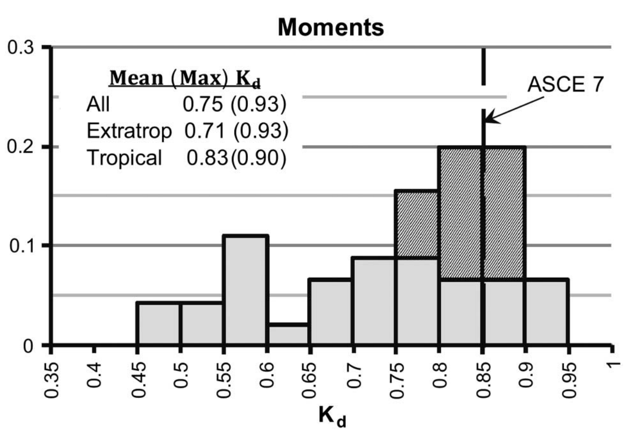

Fig. 1. Wind directionality coefficient values based on wind tunnel tests of three buildings in two wind environments (adapted from *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 133; Nicholas Isyumov, Eric Ho, and Peter Case, "Influence of Wind Directionality on Wind Loads and Responses," pp. 169–180, © 2014, with permission from Elsevier)

Wind Load Reliability Analyses

The reliability of structural members can be determined through a Monte Carlo analysis in which the random variables and their assumed probability distributions [and therefore their means and coefficients of variation (COVs)] are based on statistical data. (If required by local site conditions, nonparametric distributions of hazards based on data can also be used.) Eqs. (1)–(3) provide a brief example of this determination for the combination of dead and live loads, which provides the benchmark for all nonseismic reliabilities stipulated in ASCE 7. The familiar LRFD design equation is provided by Eq. (1). The limit state is defined by Eq. (2), and in a normalized form to facilitate reliability analysis in Eq. (3). Failure is assumed to occur when $G(\boldsymbol{X}) < 0$

$$0.9R_n = 1.2D_n + 1.6L_n \tag{1}$$

$$G(R, D, L) = R - D - L = 0 \tag{2}$$

$$G(X) = \left(\left[1.2 + 1.6\left(\frac{L_n}{D_n}\right)\right] \Big/ 0.9\right) X_1 - X_2 - \left(\frac{L_n}{D_n}\right) X_3 = 0 \tag{3}$$

In Eqs. (1)–(3), R is the random strength, R_n is the nominal strength, D is the random dead load, D_n is the nominal dead load, L is the random live load, and L_n is the nominal live load. The normalized form of Eq. (2), involving the nondimensional random variables $X_1 = R/R_n$, $X_2 = D/D_n$, and $X_3 = L/L_n$, allows the reliability analysis to be performed for design situations involving different grades of material and a range of L_n/D_n, which typically ranges from 0.5 to 4.0. Table 1 provides the statistics for the random variables in Eq. (3) used in the reliability analysis of a compact, laterally supported A992 steel flexural member (Ellingwood 2000).

If $L_n/D_n = 2.0$, the probability of failure (P_f) in a 50-year service period can be determined as 0.00298 using Monte Carlo simulation; the corresponding reliability index is $\beta = \Phi^{-1}(1 - P_f) = 2.75$. On an annual basis, the probability of failure is approximately 6×10^{-5}. These reliabilities are typical for steel flexural members (Ellingwood 2000), and can be compared with the reliability targets presented in ASCE 7. Such reliability indexes may not exactly match the target values in ASCE 7 because it is not possible to achieve the reliability target exactly in all cases using a small number of discrete load and resistance factors [Eqs. (1)–(3)]. In some cases, the reliability index for gravity load design will be slightly higher than 3.0, while in other cases it will be slightly less, but the average value will be approximately 3.0.

A similar analysis was conducted to determine the reliabilities for wind-resistant design. The design must satisfy Eq. (4), i.e.

$$0.9R_n = 1.2D_n + 1.0W_T \tag{4}$$

in which W_T = nominal wind load, corresponding to a 3-s gust wind speed with a return period of T years (in ASCE 7-10, $T = 700$ years for Risk Category II structures). The limit state is defined by

$$G(R, D, W) = R - D - W_{\max} = 0 \tag{5}$$

Table 1. Statistics of Load and Resistance Random Parameters

Variable	Mean	COV	Probability function
X_1	1.08	0.09	Lognormal
X_2	1.05	0.10	Normal
X_3	1.00	0.25	Type I

in which random variable W_{max} = maximum wind load to occur in 50 years. In the absence of structural deterioration, this implies that the reliability assessment is based on a service period of 50 years; this was the basis for the original reliability analyses, leading to the load combinations in Section 2.3 of previous editions of ASCE 7 (Ellingwood et al. 1980). As previously, the limit state is written in nondimensional form

$$[1.2D_n + 1.0W_T]X_1/0.9 - X_2D_n - X_W W_{50} = 0 \qquad (6)$$

in which $X_W W_{50} = (W_{max}/W_{50})W_{50} = W_{max}$ and W_{50} = wind load corresponding to a 50-year return period wind speed. The term W_T for any return period is related to W_{50} (see ASCE 7-10, C26.5-3) as follows:

$$W_T = W_{50}[0.36 + 0.1\ln(12T)]^2 \qquad (7)$$

The ratio W_{50}/D_n varied from 0.5 to approximately 4 in the original study (Ellingwood et al. 1980).

The key to determining the reliability index, β, or the limit state probability, P_f, for wind load combinations lies in the proper determination of the probability model of X_W (the nondimensional variables X_1 and X_2 are defined in Table 1). Wind loads in ASCE 7-10 are defined by

$$W = CGC_pK_zK_dV^2 \qquad (8)$$

in which C = constant; G = gust factor; C_p = pressure coefficient; K_z = exposure factor; and $V = 3 - \mathrm{s}$ gust wind speed. The nondimensional wind variable, X_W, is defined in Eq. (9):

$$X_W = \frac{W_{max}}{W_{50}} = \left(\frac{C}{C}\right)\left(\frac{GC_p}{GC_{pn}}\right)\left(\frac{K_z}{K_{zn}}\right)\left(\frac{K_d}{K_{dn}}\right)\left(\frac{V_{max}}{V_{50}}\right)^2 \qquad (9)$$

Under the assumption that term $K_d/K_{dn} = K_d/0.85$ is unbiased, the uncertainty in X_W is defined by a Type I distribution of largest values, fitted to the 90th percentile and above the cumulative density function of W, with mean and COV of 0.90 and 0.35, respectively.

Structural Reliability Based on Mean of $K_d = 0.85$

Fig. 2 shows the reliabilities determined if the directionality coefficient is assumed to be unbiased [i.e., the mean (μ_{Kd}) and nominal value (K_{dn}) of K_d both equal 0.85], which is comparable to the assumption made in the first generation of load combinations for strength in ANSI A58/ASCE 7 (Ellingwood et al. 1980) The solid line shows the average value, which was used to set target reliability values. The reliability index, β, is approximately 2.5 for a return period of 700 years using a wind load factor of 1.0 (the current return period stipulated for Risk Category II structures in ASCE 7-10). Approximately the same result is obtained if the wind load calculated from a 50-year return wind speed is factored by 1.6, as in ASCE 7-05 (ASCE 2005) and earlier editions of ASCE 7. For the past three decades, there has never been a clear explanation for the difference between the apparent reliability indexes of 2.5 and 3.0 for members designed for wind loads and gravity loads, respectively The new data on wind directionality coefficients provided in Isyumov et al. (2014) (Fig. 1) provided an explanation for the apparent difference, as summarized in the following section.

Structural Reliabilities Based on Updated K_d

Using the recently published directionality data (Isyumov et al. 2014), the reliability analyses can distinguish between the nominal K_d (or K_{dn}, traditionally 0.85), and the mean of K_d, or μ_{Kd}. If $\mu_{Kd} = 0.71$, as suggested by Fig. 1, but K_{dn} remains at 0.85, as stipulated in ASCE 7-16, the mean value of X_W as calculated by Eq. (9) decreases from 0.9 to approximately 0.75. With this change in the statistics for K_d, but with all other parameters the same, the mean reliability index β for a return period of 700 years using a wind load factor of 1.0 increases from approximately 2.5 to approximately 3.0 for Risk Category II structures, which is comparable to the reliability achieved with the ASCE 7-16 gravity load combinations.

Fig. 3 shows the nominal return period versus the reliability index for extratropical winds utilizing the revised directionality factor. The return periods of 300 and 1,700 years for Risk Categories I and III structures are consistent with the target reliability goals of 2.5 and 3.25, respectively, in Table 1.3.1a of ASCE 7-16. To achieve the desired reliability target $\beta = 3.5$ for Risk Category IV structures designated as essential facilities, the nominal return period must be increased to approximately 3,000 years; this nominal return period has been adopted for ASCE 7-16, where facilities in which continued function following the design-basis event, as well as life safety, are of concern.

The reliability analyses for the wind load combinations in ASCE 7-16 and the results presented in Figs. 2 and 3 are based on data

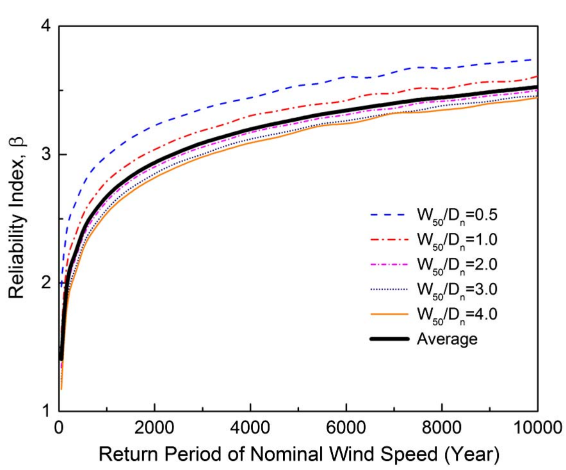

Fig. 2. Reliability versus return period of nominal wind speed ($K_{dn} = 0.85$; $\mu_{Kd} = 0.85$)

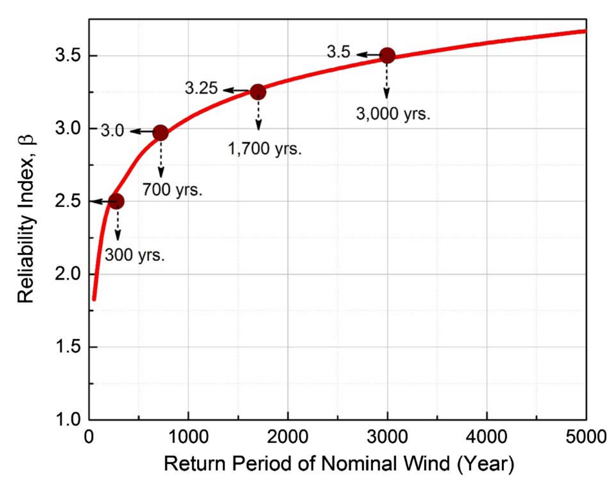

Fig. 3. Recommended mean return periods for wind maps in ASCE 7-16 ($K_{dn} = 0.85$; $\mu_{Kd} = 0.71$)

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for extratropical storm regions. The COV for hurricane wind speeds tends to be much larger than the COV for extratropical wind speeds, and the hurricane wind speed distributions have longer tails. Moreover, Fig. 1 indicates that the mean of K_d for hurricane wind speeds is close to the nominal value of 0.85. While these differences imply that the ASCE 7-16 wind load combinations are likely to result in lower reliabilities in hurricane-prone regions, additional data on hurricane wind speeds and directionality factors are required to resolve this issue conclusively.

Conclusions

ASCE 7 is moving toward adopting load requirements based on uniform risk rather than uniform hazard that are consistent with reliability-based design goals stipulated for PBE. This paper presented the reliability basis for the new wind maps adopted for ASCE 7-16, which correspond to return periods of 300, 700, 1,700, and 3,000 years for buildings and other structures in Risk Categories I, II, III, and IV, respectively. This is the first time that wind return periods and associated reliabilities for design have differentiated between Risk Categories III (primarily public assembly) and IV (essential facilities). As a side benefit, taking advantage of recent (2014) data for extratropical wind directionality effect, the reliabilities associated with the ASCE 7 wind load combinations for Category II buildings in extratropical regions of the United States are shown to be comparable to the reliabilities associated with the gravity load combinations that were first introduced in 1982 [ANSI A58 (ANSI 1982)].

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