Safety of Structures in Strong Winds and Earthquakes: Multihazard Considerations

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Abstract: In accordance with the ASCE Standard 7-05, in regions subjected to wind and earthquakes, structures are designed for loads induced by wind and, separately, by earthquakes, and the final design is based on the more demanding of these two loading conditions. Implicit in this approach is the belief that the standard assures risks of exceedance of the specified limit states that are essentially identical to the risks inherent in the provisions for regions where only wind or earthquakes occur. We draw the attention of designers, code writers, and insurers to the fact that this belief is, in general, unwarranted, and that ASCE 7 provisions are not risk consistent, i.e., in regions with significant wind and seismic hazards, risks of exceedance of limit states can be up to twice as high as those for regions where one hazard dominates. This conclusion is valid even if the limit states due to wind and earthquake are defined differently, as is the case in ASCE 7. We propose an approach to modifying ASCE 7 provisions which guarantees that risks implicit in minimum ASCE 7 requirements for regions where one hazard dominates are not exceeded for structures in regions with strong wind and seismic hazards.

DOI: 10.1061/(ASCE)ST.1943-541X.0000108

CE Database subject headings: Earthquakes; Limit states; Hazards; Structural safety; Wind loads.

Author keywords: Building technology; Earthquakes; Limit states; Multihazards; Safety; Wind loads.

Introduction

The design of structures in the United States is governed by load combinations specified in the ASCE Standard 7 (ASCE 2005). In regions prone to both earthquakes and strong winds, structures are designed for loads induced by wind and, separately, by earthquakes. The final design is based on the more demanding of these two loading conditions. Implicit in this approach is the belief, which has so far prevailed in the code-writing community, that the standard assures risks of exceedance of the limit states being considered that are essentially identical to the risks inherent in standard provisions for regions where only wind or earthquakes occur.

The purpose of this paper is to draw the attention of the design, code-writing, and insurance communities to the fact that this belief is, in general, unwarranted. We show that ASCE 7 Standard provisions are not risk consistent, in the sense that structures in regions with significant wind *and* seismic hazards can have risks of exceedance of limit states that can be up to twice as high as corresponding risks implicit in the provisions for regions where only one of these hazards dominates. This is true in spite of the fact that such risks are notional; that the failure modes for wind and earthquake can differ from each other; and that the ASCE 7 Standard uses different design approaches for wind and earthquakes.

The paper is organized as follows. First we explain, by using probabilistic tools, the nature of the misconception that has led to the development of the current—inadequate—ASCE design criteria for multihazard regions. Next, we show that a simple modification of the ASCE 7 Standard criteria can assure that risks for structures in multihazard regions are not higher than risks for structures in regions where one hazard dominates. Finally, we consider a case study—a water tower—to illustrate the main points presented in this paper.

Risk of Exceedance of Limit States Induced by Two Hazards

We now show that implicit in ASCE 7 provisions are risks of exceedance of limit states due to two distinct hazards that can be greater by a factor of up to 2 than risks for structures exposed to only one hazard. An intuitive illustration of this statement follows. Assume that a motorcycle racer applies for insurance against personal injuries. The insurance company will calculate an insurance premium commensurate with the risk that the racer will be hurt in a motorcycle accident. Assume now that the motorcycle racer is also a high-wire artist. In this case, the insurance rate would increase as the risk of injury, within a specified period of time, in either a motorcycle or a high-wire accident will be larger than the risk due to only one of those two types of accident. This is true even though the nature of the injuries sustained in a

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Note. This manuscript was submitted on September 8, 2008; approved on August 14, 2009; published online on September 18, 2009. Discussion period open until August 1, 2010; separate discussions must be submitted for individual papers. This technical note is part of the ***Journal of Structural Engineering***, Vol. 136, No. 3, March 1, 2010. ©ASCE, ISSN 0733-9445/2010/3-330–333/$25.00.

motorcycle accident and in a high-wire accident may differ. The argument is expressed formally as

$$P(s_1 \cup s_2) = P(s_1) + P(s_2) \tag{1}$$

where $P(s_1)$=annual probability of event s_1 (injury in motorcycle accident); $P(s_2)$=annual probability of event s_2 (injury in high-wire accident); and $P(s_1 \cup s_2)$=annual probability of injury due to a motorcycle or a high-wire accident (note that the probability of s_1 and s_2 occurring simultaneously is negligible).

Eq. (1) also holds for a structure for which $P(s_1)$=probability of the event s_1 that the wind loads are larger than those required to attain the limit state associated with design for wind, and $P(s_2)$ =probability of the event s_2 that the earthquake loads are larger than those required to attain the limit state associated with design for earthquakes (note that, as in the earlier example, it is assumed that s_1 and s_2 have zero probability of simultaneous occurrence). $P(s_1 \cup s_2)$=probability of the event that, in any one year, s_1 or s_2 occurs. It follows from Eq. (1) that $P(s_1 \cup s_2) > P(s_1)$ and $P(s_1 \cup s_2) > P(s_2)$, i.e., the risk that a limit state will be exceeded is increased in a multihazard situation with respect to the case of only one significant hazard. If $P(s_1) = P(s_2)$, the increase is twofold. Note that s_1 and s_2 *can differ*, as they typically do under ASCE 7 design provisions. In spite of such differences, it is the case that, both for earthquakes and wind, inelastic behavior is allowed to occur during the structure's lifetime. For seismic loading, only the mean recurrence interval (MRI) (i.e., the inverse of the risk of exceedance) of the maximum considered earthquake is specified; the MRI of the onset of postelastic behavior is unknown. For wind loading, the MRI of the onset of nonlinear behavior is specified; however, nonlinear behavior is also possible and allowed to occur during the structure's life.

We now consider a model developed by Pearce and Wen (1984) that has been invoked in support of the current ASCE 7 provisions for design in regions with both strong earthquakes and hurricanes. For illustration purposes we consider, in this model, the case in which two time-dependent loads $X_1(t)$ and $X_2(t)$ occur. If

$$Z = \max[X_1(t) + X_2(t)] \text{ in } (0,t) \tag{2}$$

the probability that Z exceeds a level s during the interval (0, t) is

$$G_Z(s) \approx 1 - \exp(-\{\nu_1[1 - P_{X1}(s)] + \nu_2[1 - P_{X2}(s)] + \nu_{12}[1 - P_{X1+X2}(s)]\}t) \tag{3}$$

in which the terms ν_1, ν_2, and ν_{12}=annual mean s-upcrossing rates of X_1, X_2, and X_1+X_2; and P_{X1}, P_{X2}, and P_{X1+X2}=marginal cumulative distribution functions of X_1, X_2, and X_1+X_2, respectively. The rate ν_{12} can be approximated by $\nu_1\ \nu_2\ (\tau_1+\tau_2)$, in which τ_1 and τ_2 are the durations of X_1 and X_2; it represents (approximately) the annual probability of a coincidence in time of the loads X_1 and X_2 (Ellingwood, personal communication, 2008); ν_{12} for coincident wind and earthquake is negligibly small, so wind and earthquake may be treated as mutually exclusive. However, an inspection of Eq. (3) shows that, in spite of the fact that wind and earthquake may be considered mutually exclusive, $P_Z(s)$ is increased if both wind and earthquakes can occur at a site, contrary to the assumption implicit in the ASCE 7 provisions.

The assumption is that, because wind and earthquake hazards have negligible probability of occurring at the same time, structures may be analyzed first as if they were subjected to only one of the hazards and second as if they were subjected only to the other hazard; the design selected for each member then corresponds to the higher of the respective demands. In this approach the increase in the probability of exceeding a limit state in the presence of two hazards is not taken into account. This would imply that the insurance rates for structures subjected to two significant hazards should be the same as for their counterparts subjected to one hazard. This implication would be correct only if the current ASCE 7 Standard design criteria applied to regions with both wind and earthquake hazards were modified so that risks in such regions may be brought in line with risks in single-hazard regions.

Proposed Approach to Modification of Current Design Criteria

The current ASCE 7 standard design criteria for wind and earthquake include load factors that imply intended nominal MRIs of limit states. Eq. (1) shows that, if the intended MRI of the limit state induced by wind is $N_1 = N$, and the intended MRI of the limit state induced by seismic loads is $N_2 = N$, the MRI of a limit state induced by wind or seismic loads is $N/2$. To guarantee MRIs of limit states induced by wind or seismic loads equal to N, as intended by the Standard, rather than $N/2$, the design criteria must be such that the MRI of the limit state induced by seismic loads is approximately $2N$, and the MRI of the limit state induced by wind loads is also $2N$. This is also seen from Eq. (3), which can be approximated as follows by noticing that the argument of the exponential function is small compared to 1, and by neglecting the ν_{12} term

$$G_Z(s) \approx \{\nu_1[1 - P_{X1}(s)] + \nu_2[1 - P_{X2}(s)]\}t \tag{4}$$

$$G_Z(s) \approx \left(\frac{1}{N_1} + \frac{1}{N_2}\right)t \tag{5}$$

$$\text{If } N_1 = N_2 = 2N, \quad G_Z(s) \approx \left(\frac{1}{N}\right)t \tag{6}$$

Inherent in the ASCE 7 wind load factor is an MRI of the factored wind load of length N. Doubling of that MRI is achieved approximately through multiplication of the wind load factor by an importance factor of 1.15 [see e.g., ASCE 2005, p. 104, Table 5, and Eq. (3)]. Hence, for structures for which the MRIs of effects induced by wind or earthquakes are halved in relation to MRIs of effects induced, separately, by wind or by earthquakes, the requisite increase can be achieved by multiplying the load factors specified in the standard by approximately 1.15 (the multiplication factor would be larger for the seismic load factor, in keeping with the larger importance factors used for earthquake design). The use of the multiplication factors would then ensure larger MRIs for the events s_1 and s_2, and an adequate MRI for the event s_1 or s_2.

We do not aim to propose "exact" values of the multiplication factors just discussed. Given the numerous approximations inherent in ASCE 7, the search for such exact values would not be warranted. However, the proposed factors would ensure that safety levels implicit in ASCE 7 provisions are not lower in multihazard than in one-hazard regions. The use of the proposed factors in cases in which the MRIs of the two limit states would not be equal would be conservative. On the other hand, failure to use such factors would result in unconservative designs.

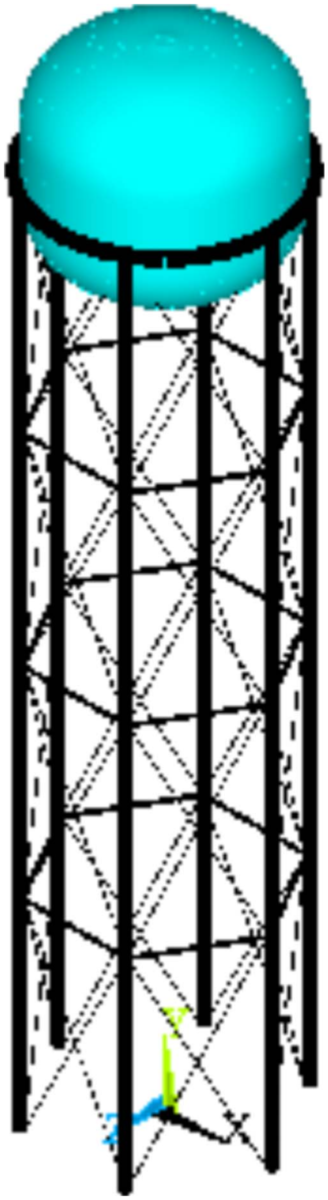

Fig. 1. Water tower

Case Study

To illustrate the concepts discussed above, we present a case study for locations in South Carolina where the effects of wind and earthquake are important. Fig. 1 shows a $1{,}500\text{-m}^3$ water tank consisting of a cylindrical middle part with diameter of 13.68 m and height of 4.50 m, and semiellipsoidal roof and bottom with major axis of 13.68 m and minor axis of 8.90 m. The tank is supported at 1.12 m above the juncture between the bottom and the middle parts by a balcony ring girder of square box cross section, supported in turn by six vertical 53.64-m legs. The legs are braced by three sets of hexagonal horizontal braces placed at equal vertical distances between the ground and the balcony girder. In addition, the water tower is stiffened with diagonal braces consisting of steel tubes prestressed with a tensile stress of 125 MPa. The material is steel with 200-GPa modulus of elasticity, 2.0-GPa tangent modulus, and 400-MPa yield strength. Table 1 lists the outside dimensions (diameter D_o or side a) and thickness t of the structural members. The tank model also has a hollow 1.09-m-diameter and 6.3-mm-wall-thickness vertical core used for pumping water (not visible in Fig. 1).

Table 1. Structural Member Dimensions

Structural member	Shape	D_o or a mm	t mm
Tank roof	Ellipsoidal shell		6.3
Tank middle	Cylindrical shell		9.5
Tank bottom	Ellipsoidal shell		9.5
Balcony girder	Square box	609.6	12.7
Legs	Circular tube	711.2	12.7
Horizontal braces	Circular tube	304.8	6.35
Diagonal braces	Circular tube	50.8	6.35

Since P-Δ (load-deflection) effects are important, a nonlinear large deflection finite-element model (FEM) of the water tower is created to accurately capture the deformations of the balcony girder and the top end conditions of the legs. The FEM uses 4,200 thin shell elements (with four nodes each) to model the tank and core, and 1,800 beam elements (with two nodes each) to model the balcony ring girder, legs, and braces. Static pushover analysis is used, whereby load is applied gradually, i.e., first gravity, followed by wind or earthquake. The most unfavorable direction of lateral loading is parallel to a pair of diametrically opposite legs denoted as $+x$ and $-x$.

Wind Load

The basic wind speed is 45 m/s (100 mi/h) (ASCE 2005, Fig. 6.1) and the required importance factor is 1.15. The strength design load combination is $0.9\ D+1.6\ W$, where D is the dead load and W is the wind load. The weight of water, with the tank filled to maximum operating capacity (94% of tank volume), is included in D. For this load combination, the total foundation reaction is 13.9 MN vertical and 572 kN horizontal. The $+x$ and $-x$ legs' vertical reactions are 3.54 and 1.10 MN, respectively, so the wind effects account for a relative difference of $\pm 53\%$ from the reactions without wind. The maximum deflection is 0.140 m (0.23% of total height). For this loading, the structure is close to the limit of small deflection, linear elastic range.

Seismic Load

According to ASCE (2005), Eq. 11.4.6 and Fig. 22.2 (map for 1 s spectral response acceleration $S_1=0.20$ g at location selected, where g is the acceleration of gravity), the design spectral response horizontal acceleration S_a for this structure is

$$S_a=\frac{S_{D1}}{T}=\frac{(2/3)F_v S_1}{T}=\frac{(2/3)\cdot 1\cdot 0.20\ \text{g}}{4.08}=0.033\ \text{g}. \qquad (7)$$

where S_{D1}=design spectral response acceleration parameter at 1-s period; F_v=long-period site coefficient; and $T=4.08$ s=first natural period of the structure. ASCE 7-05 provides maps for parameters S_s and S_1 that correspond to the ground motion of 0.2- and 1.0-s spectral response accelerations with 5% of critical damping of the maximum considered earthquake (with 2% probability of exceedance within a 50-year period) for site Class B. Load effects include vertical seismic design acceleration A_v

$$A_v = 0.14 \quad S_{DS} = 0.14 \cdot (2/3) F_a \quad S_S = 0.14(2/3) \cdot 1.0 \cdot 0.75 \;\; \mathrm{g} = 0.07 \;\; \mathrm{g}, \tag{8}$$

where S_{DS}=design spectral response acceleration parameter at 0.2-s period and F_a=short-period site coefficient.

For the strength design load combination $0.9D+1.0E$, the total foundation reaction is 15.0 MN vertical and 500 kN horizontal. The $+x$ and $-x$ legs' vertical reactions are 3.80 and 1.20 MN, so the seismic effects account for a relative difference of $\pm 52\%$ from what the reactions would be without earthquake loads. The maximum deflection of the structure is 0.155 m (0.25% of total height). The structural behavior is seen to be very similar under seismic load and wind load. This strongly suggests that the risk of exceedance of an undesirable state is greater for the structure subjected to wind or earthquakes than for the structure subjected just to wind or just to earthquakes.

It is not possible in the present state of the art to calculate the risk of exceedance of the seismic limit state. Nevertheless, it is clear that in the case of two hazards the risk of exceedance of an undesirable state—a limit state associated with the one or both hazards—can be significantly greater than the risk associated with only one of the hazards. Increasing the design loads as proposed in this paper will guarantee that the risk is not larger under the two hazards than is the case for regions where only one hazard is present.

Conclusions

The notional risk of exceedance of limit states implicit in the ASCE 7 Standard can be greater by a factor of up to 2 for regions where both wind and earthquake loads are significant than for regions with only one significant hazard. This is true even if, as in the ASCE 7 Standard, the limit states differ for wind and earthquake. We propose an approach to modifying ASCE 7 provisions which guarantees, in most cases conservatively, that designs for regions in which earthquake and wind hazards are significant satisfy minimum requirements with respect to safety implicit in provisions for regions where only one hazard matters. In addition to design implications, our argument has implications for insurance assessments, which depend on the type of structure. This dependence needs to be better understood, and further research is required for this purpose (Crosti et al. 2009).

Acknowledgments

We thank M. Grigoriu of Cornell University, P. Gould of the Washington University in St. Louis, H. S. Lew, J. Main, and F. Sadek of the National Institute of Standards and Technology, and D. Veneziano of the Massachusetts Institute of Technology, for helpful comments. Eq. (3) and related material were kindly communicated by B. R. Ellingwood of the Georgia Institute of Technology.

References

ASCE. (2005). "Minimum design loads for buildings and other structures." *ASCE 7-05*, Reston, Va.

Crosti, C., Duthinh, D., and Simiu, E. (2009). "Wind and seismic multihazard design of buildings." *CTA XXII Congress, Steel for a Sustainable Future*.

Pearce, T. H., and Wen, Y. K. (1984). "Stochastic combinations of load effects." *J. Struct. Eng.*, 110(7), 1613–1629.