Reliability Analysis of Concrete Structures Considering Different Hazard Curves for Western and Eastern United States

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Summary

In the deterministic design of structures, according to the current standards, the uncertainties inherently present in the evaluation of actions and resistances are accounted for through partial safety factors. This paper discusses the reliability levels obtained in the design of reinforced concrete structures under pure bending, using the safety criteria defined in the ACI 318-05 and in the ASCE/SEI 7-05. The reliability analyses consider the different shapes of the seismic hazard curves, corresponding to the different rates of change of maximum ground accelerations versus corresponding annual probability of occurrence, between locations in Central and Eastern and in the Western United States. Combinations of dead, live and seismic loads are analyzed. It is shown that the design considering the present code provisions leads to a seismic risk that is not uniform throughout the country; very different levels of reliability are obtained for the different locations.

Keywords: safety factors; earthquake loads; concrete structures; reliability analysis.

1. Deterministic design

The deterministic flexural design of a typical reinforced concrete beam is performed. The concrete design criteria, including the reduction factors ϕ , as defined in the ACI 318-05 (Ref. [1]) are followed. Several combinations of dead (*D*), live (*L*) and seismic loads (*E)* are analyzed.

2. Definition of the probabilistic seismic input

The seismic hazard maps for the United States were defined by the USGS, (Ref. [2]). The obtained hazard curves indicate that the rate of change of ground motion versus their annual probability is not constant in the United States. The Figure 1 presents two of these typical curves, showing the 0.2 sec Spectral Acceleration, for a system with 5% of damping, against the Return Period T_M, for a typical location in the less active areas in the Central and Eastern U.S.(Charleston), and for the Western U.S. (San Francisco). The Gumbel functions used in the reliability analyses are also shown.

3. Reliability analyses and conclusions

The probabilistic model for the reliability analyses considers data from previous studies of Szerszen et al. (Ref. [3]) for dead and live loads. For seismic loads, a Gumbel distribution for the accumulated probability of non-exceedance of maximum accelerations has been considered.

Main results of the analyses are shown in Figure 2, the reliability indexes β, regarding a reference period of 50 years. The parameter χ is the relationship between nominal values of dead and total applied loads. A reference value for the minimum required reliability index is also plotted (β = 3.8).

Adequate values for β are only attained in the cases where dead loads are predominant. In the range of low seismic forces ($E \leq 0.6\ L$), reliability indexes β between 3.0 and 4.4 are obtained. In the

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range of high seismic forces, ($E \geq 0.6\ L$), the reliability indexes β obtained for San Francisco are much smaller than the ones of Charleston. This difference is explained by the different return periods for the design accelerations that result from the considered deterministic criteria.

The obtained results show some inconsistency in the criteria defined in the standards, since very different levels of reliability are obtained for similar loading conditions in different locations.

4. References

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[3] SZERSZEN M.M., SZWED A., and NOWAK, A.S., "Reliability Analysis for Eccentrically Loaded Columns", *ACI Structural Journal*, *102* (8), 2005, pp. 676-688.

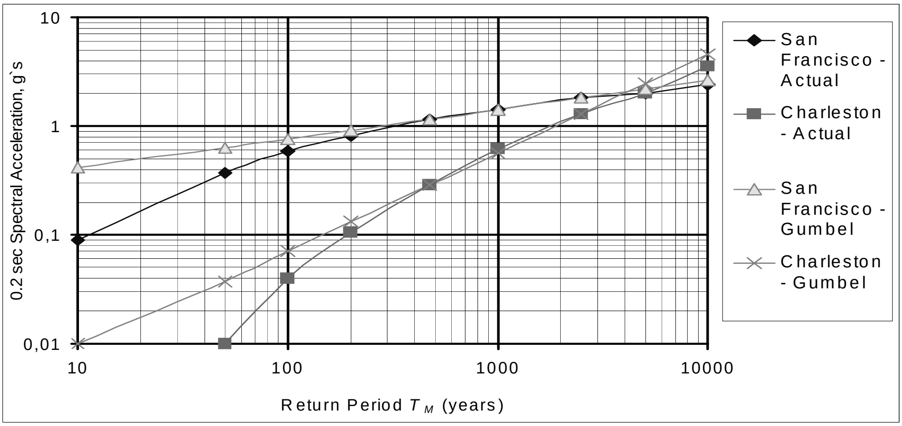

Fig. 1: Hazard curves for San Francisco and Charleston: actual values and Gumbel functions

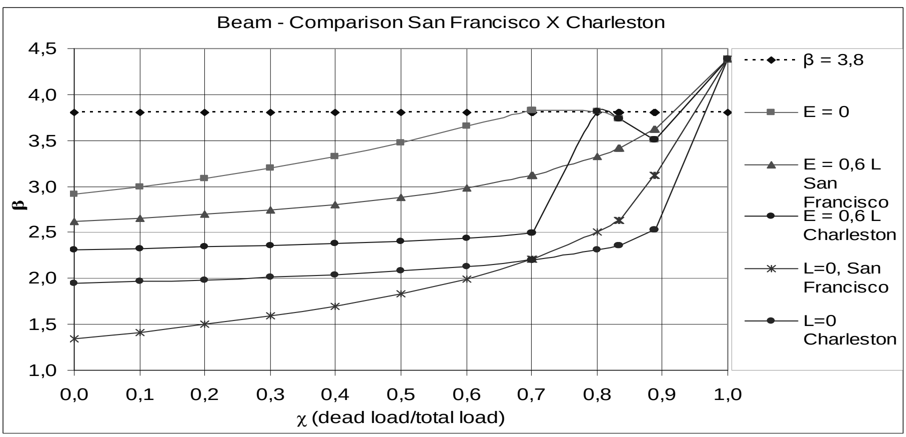

Fig. 2: Results of the reliability analyses for San Francisco and Charleston