SEISMIC RELIABILITY FUNCTIONS OF MULTISTORY BUILDINGS THEIR SENSITIVITY TO SEVERAL ANALYSIS AND DESIGN VARIABLES

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ABSTRACT :

A study is made about the sensitivity of seismic reliability functions for multistory buildings, expressed in terms of Cornell's reliability index β, to the criterion used to evaluate it (deformation capacity, secant-stiffness reduction index) and the type of structural arrangement. Results show that the reliability index β varies in a much faster way with the normalized intensity for a dual wall-frame system than for a frame system; this may be due to the fact that, in the former case, the lateral strength and stiffness are concentrated in the wall, while they are distributed among different structural members in the other case. Base-shear *vs* roof displacement curves obtained by pushover analysis may show large differences in the lateral strength remaining after the deformation capacity (that corresponding to a 20 percent reduction in the base-shear, with respect to its maximum throughout the curve) is exceeded. These differences partly explain the large differences in the values of β corresponding to given values of the ground motion intensity normalized with respect to the deformation capacity.

KEYWORDS: Seismic reliability function, deformation capacity, multistory systems

1. INTRODUCTION

Conventional approaches to the evaluation of the seismic reliability of multistory systems are based on the concept of comparing the amplitude of the peak lateral distortion with the deformation capacity (both expressed in probabilistic terms). However, the resulting criteria are affected by severe limitations, associated with the difficulties implicit in the determination of the deformation capacity of a complex system responding to a ground motion excitation at its base: such capacity depends, among other variables, on the lateral configuration of the system at the instant of impending collapse, and this configuration is unknown. The need for an explicit estimation of the ultimate capacity can be circumvented through the use of the incremental dynamic analysis. IDA (Vamvatsikos and Cornell, 2002), or of a secant-stiffness reduction index (I_{SSR}), as an indicator of the proximity to system collapse. Use of the latter is presented as an alternative to IDA, due to the possibility it offers of obtaining consistent estimators of Cornell's reliability index (ratio of mean value to standard deviation of safety margin) through substantially lower computational efforts. The objective of this paper is to make a brief presentation of the secant-stiffness reduction index, as well as to show the results of some exploratory studies about a) the sensitivity of the seismic reliability functions based on the concept of deformation capacity to both the type of structural arrangement and the criteria used to estimate it, and b) the relations between the reliability functions obtained using different definitions of deformation capacity and those developed with the aid of the index described above.

2. SECANT-STIFFNESS REDUCTION INDEX

Esteva and Díaz-López (2006) present several alternative criteria for the estimation of the seismic reliability function β(*y*) for a complex nonlinear system, where β is Cornell's reliability index and *y* is the ground motion intensity. In all cases, the collapse condition is expressed in terms of a secant-stiffness reduction index defined as follows:

$$I_{SSR} = \frac{(K_0 - K)}{K_0} \quad (2.1)$$

Here, $K=V_b/\psi H$ is the value of the secant stiffness of a nonlinear system at the instant when its global distortion ψ reaches its peak absolute value during its response to a seismic excitation; V_b is the base shear at the instant where the peak value of ψ is reached and H is the height of the system with respect to its base. K_0 is the value adopted by K under conditions of linear response; it is determined from the results of a pushover analysis of the system of interest subjected to a system of lateral forces obtained by a modal superposition criterion for an expected response spectrum proportional to that specified for design. Collapse of the system corresponds to the condition I_{SSR}. Failure probability can thus be readily estimated, provided the probability distribution of this index can be determined; otherwise, the reliability function is expressed in terms of Cornell's index β, as mentioned above.

3. RELIABILITY FUNCTIONS

The index proposed in Eqn. 2.1 can be applied to estimate the seismic reliability function of a structural system. For this purpose, the seismic capacity of the latter is expressed in terms of Z_F = ln Y_F, where Y_F is the minimum value of the seismic intensity required to produce its collapse, determined by the condition I_{SSR} = 1.0. For an earthquake ground motion with intensity equal to y, the safety margin Z_M would be equal to the natural logarithm of Y_F/y. The reliability index would then be equal to

$$\beta(y) = \frac{E(Z_F) - \ln y}{\sigma(Z_F)} \quad (3.1)$$

Here, $E(\cdot)$ and $\sigma(\cdot)$ denote the expected value and the standard deviation, respectively. An expression for Z as a function of I_{SSR} can be obtained from a sample of pairs of values of Z and u = I_{SSR}. If all the values in the sample are smaller than 1.0, both $E(Z(u\ ))$ and $\sigma(Z(u\ ))$ can be estimated by means of a conventional regressions analysis. However, this kind of analysis does not apply to those cases when the sample includes points with values of I_{SSR} equal to 1.0: it is then necessary to resort to a maximum likelihood analysis, as proposed by Díaz-López *et al* (2008).

4. CASES STUDIED

The criterion described above was applied to determine the seismic reliability functions of a number of structural systems representative of those normally built in Mexico City; they were designed in accordance with Mexico City Building Code and the corresponding Complementary Technical Norms (NTCS-RCDF, 2004). The structures selected were assumed to be built on soft soil; soil-structure interaction was taken into account, both during the design process and for the evaluation of their seismic reliability functions.

A study was made about the sensitivity of the reliability of the systems considered to different analysis and design variables: a) Criterion adopted to evaluate β, b) type of structural arrangement (rigid frame and dual wall-frame system), c) slenderness ratio, d) constitutive functions describing the behavior of structural members and e) stiffness and strength variation along the height of the system.

Two alternative criteria were adopted to evaluate the reliability functions: a) that described in Sections 2 and 3, using index I_{SSR}, and b) on the basis of a deformation capacity estimated by means of conventional pushover analysis, as described by Esteva *et al* (2002). The results of these studies are summarized in the following.

4.1. Rigid frame and dual wall-frame systems: structural arrangement and slenderness ratio

A family of four twelve-story buildings was analyzed: a rigid frame and three dual wall-frame systems. The latter were similar to that sketched in Figure 1. The three systems studied differed in the value of the shear-wall width, L_w, which was equal to 4m, 3m and 6m for systems 12NB, 12NC and 12ND, respectively. For the rigid frame system (12NSM), the shear wall was replaced by an intermediate bay, 6m wide. The different values of the shear-wall widths gave place to different slenderness ratios of the corresponding systems.

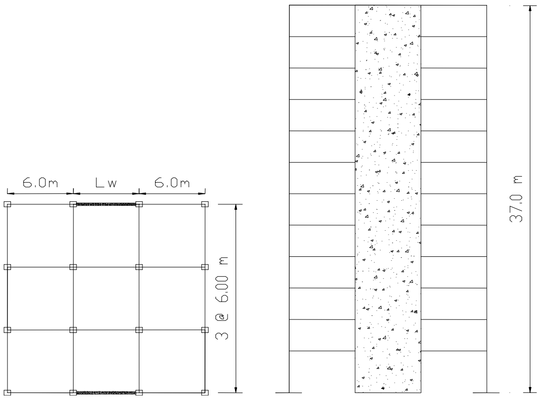


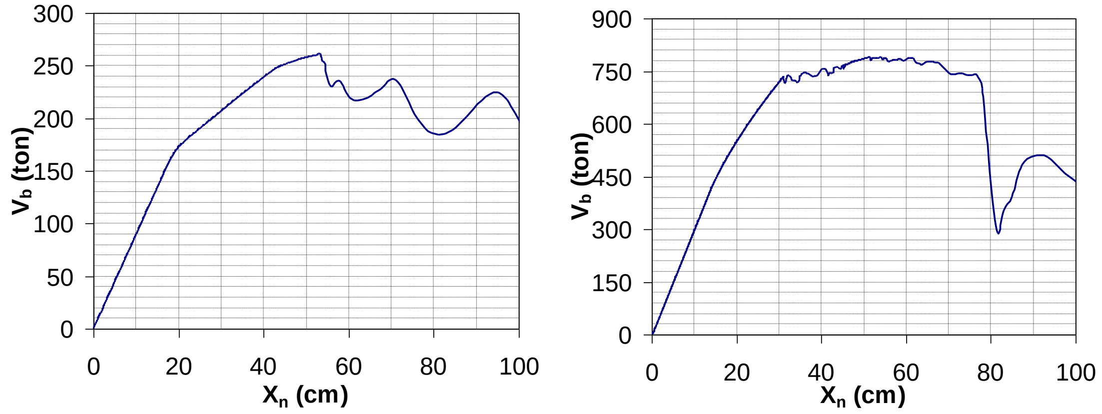
Figure 1. 12-story dual wall-frame system

In order to apply the criteria described in Section 2 and 3, a sample of pairs of values (y, I_{SSR}) was generated for each case studied, proceeding as follows:

i. A pushover analysis is perform to obtain the curve relating base-shear force with roof displacement, taking the (uncertainly known) gravitational loads and mechanical properties of the structural members equal to their expected values.
ii. This curve is used to estimate δ_F, the deformation capacity of the system, taken equal to the roof displacement necessary to produce a reduction of 20 percent in the maximum value of the base shear force.
iii. A sample of structural systems is simulated, taking no account the probabilistic descriptions of their corresponding gravitational loads and mechanical properties.
iv. For each simulated structural system, a ground motion time history is simulated for a pre-established intensity (Ismael & Esteva, 2006). The latter is measured by its normalized value, $\eta = S_{dl} / \delta_F$, where S_{dl} is the ordinate of the linear displacement response spectrum for 0.05 damping, and δ_F was defined above.
v. A step-by-step dynamic response analysis is performed for each pair of system and ground motion time-history. This will lead to a sample of pairs of values (Z, I_{SSR}), where $Z = \ln \eta$.
vi. These pairs of values are plotted and used to estimate $E(Z)$ and $\sigma(Z)$ as functions of I_{SSR}.
vii. The values of $E(Z_F)$ and $\sigma(Z_F)$ are those corresponding to corresponding to I_{SSR} = 1.0. They are used to determine the reliability function of the system, according to Eqn. 3.1.

Figures 2a and 2b show the base-shear *vs* roof displacement curves obtained for cases 12NSM and 12ND. It can be observed that the dual system (Figure 2b) shows a much higher lateral strength, but a smaller deformation capacity than the rigid frame system (Figure 2a).

Figure 3 shows the reliability functions $\beta(\eta)$, expressed in terms of the normalized intensity defined above. It can be observed that the rigid frame system (12NSM) is characterized by higher reliability values (for the same value of η) than the dual wall-frame system (12ND). This can be explained by the fact that the rigid frame system preserves a higher portion of its base-shear capacity than the dual wall-frame system, after the corresponding deformation capacities are reached. The sudden decay in the base-shear capacity of stem 12ND is due to the fact that the wall provides a very large portion of that capacity.



a) 12NSM b) 12ND

Figure 2. Pushover curves for systems 12NSM and 12ND

Figure 4 shows the reliability functions obtained in accordance with the criterion of deformation capacity determined from a pushover analysis (Esteva *et al*, 2002). In this case, the seismic intensity is measured by S_a, the ordinate of the linear pseudo-acceleration response spectrum for 0.05 damping, for the fundamental natural period of the system. In spite of the fact that Figures 3 and 4 are expressed in terms of different intensity measures, they serve to show significant differences in the relative variations of the reliability functions obtained for the different systems studied. According to Figure 4, wall-frame systems present higher β values than rigid frames: this is a consequence of the higher base-shear capacities of the former, which dominate in these cases with respect to the higher deformation capacity of the rigid-frame system. It is also observed that the values of β based on the concept of deformation capacity are higher than those obtained on the basis of the collapse intensity, Y_F.

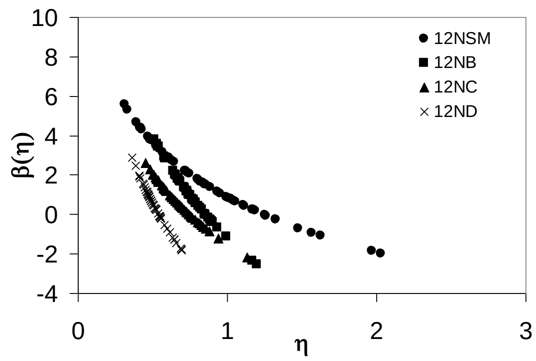


Figure 3. Reliability functions of 12-story systems, using the criterion based on ISSR

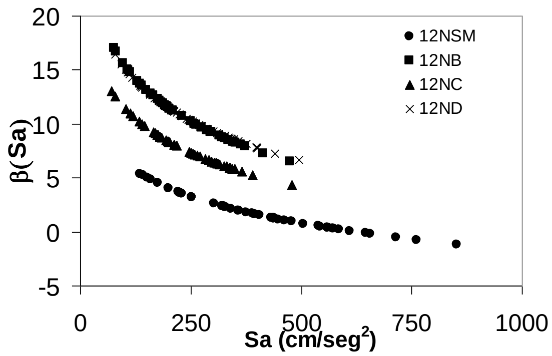


Figure 4. Reliability functions of the 12-story systems, using the criterion based on deformation capacity

Similar studies were conducted on a set of three 20-story dual wall-frame systems, with different slenderness ratios. A sketch of the systems studies is shown in Figure 5. Systems denoted as SD1, SD2 and SD3 have the same height (H = 61m), but different base widths: system SD1 has a base (L_1 x L_2) of 18 x 18m; system SD2 is 21m x 21m and system SD3 is 26 x 24m. Similarly to what was done with the systems presented in Figure 1, the reliability functions for these new systems were obtained using both criteria: one based on the use of index I_{SSR} and another based on the concept of deformation capacity.

Figure 6 shows the base-shear *vs* roof displacement curve for case SD1. The reliability functions obtained in terms of I_{SSR} are shown in Figure 7. In this case, the ground motion intensity was measured by the ordinate of the linear pseudo-acceleration response spectrum for the fundamental period of the system. The results show increasing values of β with increasing values of the slenderness ratio.

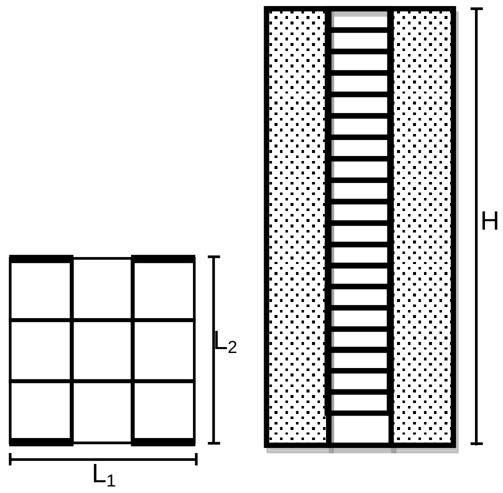


Figure 5. 20-story dual wall-frame system

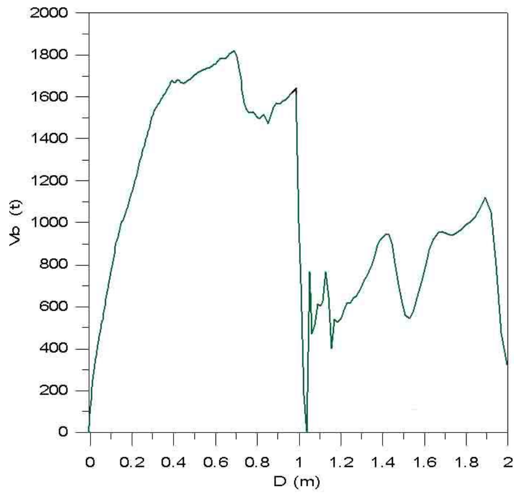


Figure 6. Pushover curves for 20-story SD1 system

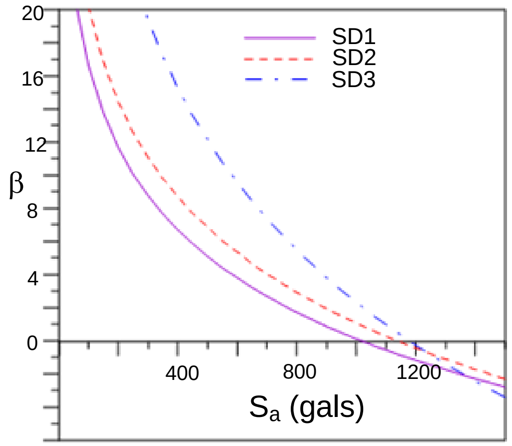


Figure 7. Reliability functions for the 20-story systems, using the criterion based on ISSR

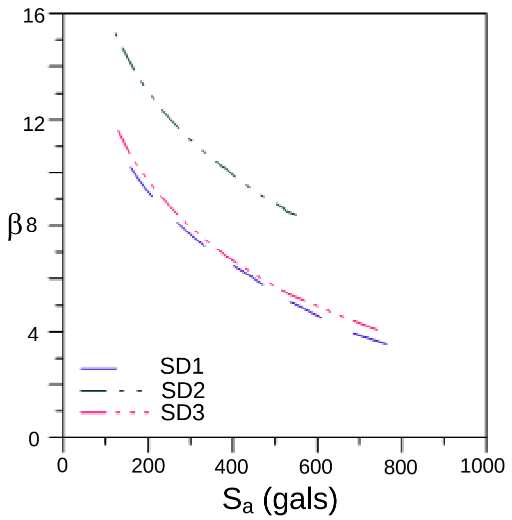


Figure 8. Reliability functions for the 20-story systems, using the criterion based on deformation capacities

Significant differences between the reliability values obtained by both criteria are also observed when the results shown in Figures 7 and 8 are compared. In the interval of low intensities, the reliability values estimated by means of index I_{SSR} are higher than those obtained with the criterion based on deformation capacities. However, this behavior is reversed for higher intensity values.

4.2. Along-height variation of stiffness and strength

Three 12-story rigid frame reinforced concrete buildings were studied. Detailed information about them has been presented by Díaz-López et al (2008). Building B1 was designed in accordance with the 2004 issue of Mexico City Building Code and its corresponding Complementary Technical Norms for Earthquake Resistant Design (NTCS-RCDF, 2004). The strength and stiffness properties of structural members in building B2 were derived from those obtained for building B1, in such a manner that the resulting values of the story strength and stiffness were equal to those of building B1 multiplied by a linear function varying from 0.8 at the bottom to 1.0 at the top. For building B3, a function equal to .15 at the bottom and 1.0 at the top was applied. In this manner, it was intended to obtain some information about the influence of irregular distributions of lateral story strength and stiffness on the seismic reliability functions of the systems.

The reliability functions were obtained using the criterion based on the concept of deformation capacity. The value of the latter was alternatively estimated by means of two different criteria: a) as in previous cases, by means of a pushover analysis, and b) taking the peak lateral distortion at failure equal to 0.03, which is the acceptable value of the peak lateral distortion for reinforced concrete frame buildings, according to Appendix A of the Complementary Technical Norms for Earthquake Resistant Design of Mexico City Building Code.

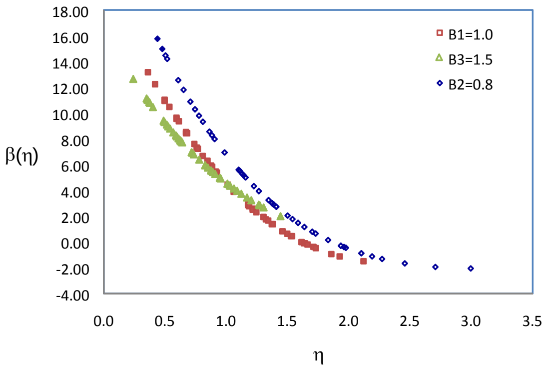


Figure 9. Reliability functions for systems with different forms of along-height variation of strength and stiffness, determining the deformation capacity by means of a pushover analysis

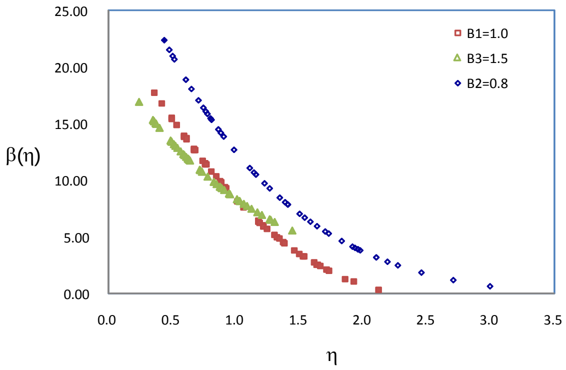


Figure 10. Reliability functions for systems with different forms of along-height variation of strength and stiffness, taking the deformation capacity equal to that established by Mexico City Building Code (0.03)

Figures 9 and 10 show the results obtained for β as a function of the normalized intensity η. Figure 9 corresponds to the case when the deformation capacity is estimated by means of a pushover analysis, taking it equal to the lateral deformation corresponding to 20 percent reduction of the maximum value attained by the base-shear capacity. Figure 10 corresponds to the case when the deformation capacity is taken equal to a lateral distortion of 0.03. It can be observed that, in general, system B2 shows the maximum values of β for a given value of η. Also, system B3 shows values of β smaller than those corresponding to system B1, for values of η smaller than 1.0. The base-shear capacity of system B2 is lower than those of the other two systems; however, its natural fundamental period is longer than those of the other two cases, which seems to lead the system to a region of the spectrum associated with lower displacement responses than those corresponding to the other two cases when the effective natural period is elongated as a consequence of nonlinear behavior.

The influence of the criterion used to define the deformation capacity is evident when the results of Figures 9 and 10 are compared: the reliability values are higher when the deformation capacity is taken equal to 0.03.

4.3. Constitutive functions of the behavior of structural members

Different models have been presented in the literature to represent the constitutive functions describing the nonlinear behavior of structural members and critical sections under the action of cyclic loads. Here, use is made of Campos and Esteva (1997) bilinear model, based on that formerly proposed by Wang and Shah (1987). The model considers stiffness and strength degradation as a function of a damage index ε, which depends in turn on the sum of the amplitudes of the curvature or rotation response cycles experienced at the end of each structural member. These amplitudes are taken into account by means of a cumulative damage index, *D*. These damage indicators are determined in accordance with the following equations:

$$D = \sum_{i}^{n} \frac{\theta_i}{\theta_F} \tag{4.1}$$

$$\varepsilon = 1 - e^{-\alpha D} \tag{4.2}$$

In these equations, ε may vary in the range (0, 1.0), θ_i is the amplitude of the distortion in the *i*-th cycle, at the member or critical section considered (for instance, the local curvature or the rotation of a plastic hinge at the end of a flexural member), and θ_F the corresponding value at failure. The value of α is established using information obtained from laboratory tests. On this basis, Campos and Esteva propose to take α = 0.0671 for reinforced concrete flexural members with tensile and transverse reinforcement ratios complying with the requirements specified in Mexico City Building Code for a global ductility factor equal to 4.

The influence of damage on the cyclic behavior of a structural member is introduced as a reduction in the ordinates of the moment-rotation functions at the plastic hinges of flexural members. The reduced moment is taken as $M_D = M(\theta)(1\text{-}\varepsilon)$, where θ is the maximum rotation previously reached at the plastic hinge considered, $M(\theta)$ is the moment corresponding to a rotation equal to θ for the initial, undamaged, moment-rotation bilinear function, and M_D is the reduced moment associated with an accumulated damage equal to *D*, as given by Eqn. 4.1.

In this study, values of α equal to 0.0671, 0.120 and 0.290 were adopted, for comparative purposes. They resulted from several fittings to the laboratory test initially used by Campos and Esteva; the quadratic errors associated with the differences between the fitted functions and the experimental results do not show variations larger than 10 percent from their average value.

Figure 11 shows the reliability functions for system B1, considering different values of α. No significant influence is observed for the range of values of η covered.

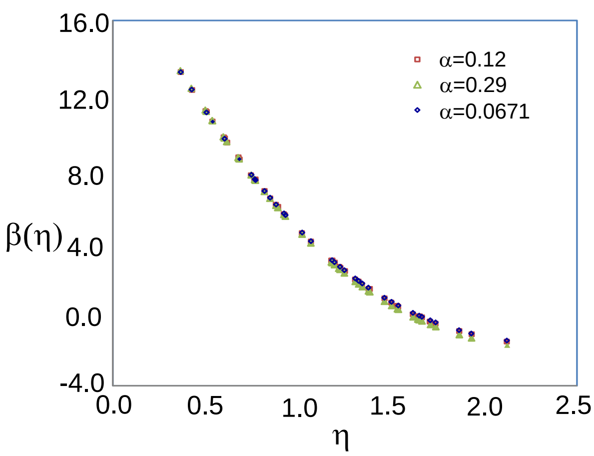


Figure 11. Reliability functions for system B1 with different values of α

5. CONCLUDING REMARKS

Some exploratory studies have been presented about the sensitivity of seismic reliability functions of nonlinear multistory systems to the criteria used to define system collapse, to the type of structural arrangement, and to the definition of lateral deformation capacity, for those cases when this concept is used. The following conclusions have been reached:

a) Base-shear *vs* roof displacement curves obtained by pushover analysis may show large differences in the lateral strength remaining after the deformation capacity (that corresponding to a 20 percent reduction in the base-shear, with respect to its maximum throughout the curve) is exceeded. These differences partly explain the large differences in the values of β corresponding to given values of the ground motion intensity normalized with respect to the deformation capacity.
b) When the concept of deformation capacity is not used, the seismic capacity of the system is expressed in terms of the (random) value of the ground motion intensity required to initiate collapse. Uncertainties about this value are associated with both the detailed characteristics of a ground motion with a given intensity and the uncertainties on the mechanical properties of the system.
c) The derivation of practically applicable design criteria with given target reliability levels is tied to the availability of easy-to-apply tools to make reasonably accurate estimates of the seismic reliability functions of the systems of interest. These tools must also be generally applicable to wide families of structural arrangements. Their development requires identifying indicators of system capacity that may be used to define adequate measures of normalized intensity.

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