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Reliability and optimization in the formulation of seismic design codes

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1 INTRODUCTION

Writing of seismic design codes (of any building code, as well) is a specially challenging activity. Doing it means being able to formulate rules oriented to lead to an optimum balance between safety and economy. This balance should be attained by both the population of all the structures to be covered by those rules and each individual member of that population. The degree to which this double condition may be attained in practice is determined by the variability of the systems to be covered, by their complexities, and by the simplifications that need to be introduced in order to cope with those complexities. The wider is the diversity of cases to be covered by a set of simplified rules, the more difficult it is to reach optimum safety requirements for each individual system. Thus, consistency is sacrificed in sake for simplicity, on the grounds that lack of simplicity in a building code leads to inefficiencies or errors in its application, if not to its conscious overlooking or violation.

Ideally a structural design code should consist of a) a set of basic principles, applicable to any system, b) a number of sets of operational rules, each set applicable to a given family or type of structure, and c) a collection of guidelines and specific recommendations applicable to detailed safety analysis and optimization of individual structures belonging to given families.

The set of basic principles should apply to any type of system. It should contain information about the loads and disturbances to be considered, as well as about the diverse uncertainties associated to them and to their effects on structures, their probabilistic models and the rules to combine them. It should also describe the uncertainties associated to the mechanical properties of structural members and systems and to their relation with response and behavior. Finally, it should deal with general criteria for formulating probabilistic models of behavior, and hence of damage and safety, and for making decisions aimed at attaining the optimum balance between safety and economy.

The sets of operational rules constitute the core of present building codes. Within the framework advocated in this paper, they are intended to achieve the objectives set forth by the basic principles. In order to fulfill their purpose, they must be simple and general, and they must be efficient for any system to which they may be applied. Because efficiency for individual cases may be in conflict with simplicity and generality, operational rules should not be taken, as they usually are, as the desirable design requirements but only as the best options under conditions of limited or imperfect information.

The value of better information and of its impact on decisions, and hence on expected risk and benefits, is less often than desirable studied for the purpose of arriving at designs better than those complying with conventional codes. A collection of guidelines and recommendations for the detailed safety analysis of individual structures would permit optimum decisions under improved information conditions, and hence its implementation is a desirable goal, in spite of the fact that, initially, the application of those guidelines would take place only for very important systems.

The basic concepts of statistical decision theory (1) have been used for the establishment of a formal framework for safety related decisions for the design of structures under seismic risk conditions (2-4). However, the role of that formal framework in the development of seismic design codes has been only that of a basic philosophy to guide somewhat subjective decisions, rather than that of a set of procedures and algorithms for the analysis and processing of quantitative measures of uncertainties, as well as of expected behavior and consequences. In the face of so complex and uncertainty-plagued a problem as that of recommending seismic design rules leading to optimum safety levels, code writers have followed the path of relying on intuition, experience and engineering judgment for the purpose of defining the values of the basic design parameters and functions (base-shear coefficients design response spectra, safety factors), and of using formal theory only for the purpose of adjusting the relative values recommended for those parameters for different cases, in order to reach nearly consistent, however poorly defined, safety levels (5). This approach can be largely explained, if not justified, by the lack of sufficiently developed models for describing and handling in probabilistic terms some of the most significant variables involved. Strong debates are held about the best ways to describe seismic hazard, as well as about the large non-statistical uncertainties hindering the formulation of probabilistic hazard models. Very little is known about the variation of structural reliability of different structures in terms of design requirements, earthquake intensities and alternate assumptions about the conditions leading to collapse or other failure modes, and equally meager is our knowledge about the validity of any of these assumptions. The same can be stated about the sensitivity of safety to structural configuration and peculiarities, and practically no efforts have been devoted to calibrating calculated values of reliability with the observations of seismic behavior of actual constructions (6). Other concepts requiring substantial research efforts are those related to the decision process: the relations between safety and cost, as well as between structural behavior and economic and social consequences; and the philosophy for the establishment of optimum safety levels and/or acceptable risk values.

This paper presents an overall-even if brief-description of the steps in the formulation of reliability based seismic design codes, including a discussion about present development and required research, as well as illustrations inspired on cases arising in the development of some Mexican seismic design codes. These concepts are organized as follows:

a) Formal decision criteria.
b) Main variables and uncertainties: seismic hazard; structural response and behavior; damage and failure.
c) Expected consequences and utilities.
d) The future of seismic design codes and practice.

2 UNCERTAINTIES AND DECISIONS IN SEISMIC DESIGN

A model for making safety related decisions in seismic design is presented in refs. 2-4, for various assumptions regarding a) the stochastic process representing seismicity, and b) the uncertainties associated to structural properties and response. The model is based on the principle that an optimum decision related to the performance of a system to be subjected to future disturbances of unknown magnitudes, occurring at unknown times, is that which leads to a maximum of **U**, the expected value of utility, as given by the following equation, where C stands for expected value of initial construction costs, and **B** and **D** stand respectively for present values of a) expected benefits, and b) costs of damage, failure and maintenance.

$$U = B - C - D \qquad (1)$$

When the problem is to decide about optimum seismic design criteria, all the variables appearing in eq. 1 are functions of the design parameters (base shear coefficients, load and strength factors), to be represented in the following by a single variable y, which may be loosely taken to be a measure of the earthquake intensity that the structure considered is capable of resisting. For the general case, B and D are given by eqs. 2 and 3 respectively.

$$B = \int_0^\infty b(\tau)\ e^{-\gamma\tau}\ d\tau \qquad (2)$$

$$D = \int_0^\infty d(\tau)\ e^{-\gamma\tau}\ d\tau \qquad (3)$$

In these equations, $b(\tau)$ and $d(\tau)$ are respectively expected benefits and expected costs (damage, failure and maintenance), per unit time, associated to the performance of the system, and γ is a discount (or interest) rate used to transform values of future benefits and expenditures into the values they would have, were they made at time t = 0, that is, at the time when the system is built. For a system which fails at a random instant T, described by its cumulative probability distribution function $F_T(\cdot)$, $b(\tau)$ is equal to the product of b_o by $\left(1-F_T(\tau)\right)$, where the first term is the value of the benefit when the system remains in operation and the second term represents the probability that the system survives at least up to instant τ. Both $F_T(\tau)$ and $d(\tau)$ may be expressed as functions of the seismic design parameters and the probabilistic description of seismic hazard at the site of interest.

Explicit expressions for **B** and **D** are obtained in ref. 4 for a number of assumptions regarding the stochastic model of seismicity, as well as about the uncertainties associated to the model itself. The expressions presented account also for uncertainties about structural properties and response. For this purpose, the following groups of stochastic variables are distinguished in the mentioned reference:

A. Random fluctuations in structural response relative to computed effects of disturbances of given intensities. These fluctuations reflect uncertainty in disturbance characteristics, vary from one disturbance to another and are completely uncorrelated.
B. Random discrepancies between actual and computed structural properties. Variables in this group remain constant for a given system as long as the latter is not rebuilt, even if it is repaired following damage, but they can change upon rebuilding and are completely uncorrelated from one structure to the next.
C. Systematic uncertain deviations, independent of time. This group includes concepts such as error in computed seismicity; systematic effects of seismic microzonation; and engineering ignorance factors, such as inaccuracies in failure criteria and in formulas for analyzing structural response.

For the particular case when seismic hazard at the site of interest is represented by a Poisson process with independent random selection of intensities, and under the assumption that in the case of collapse the structure is rebuilt in accordance with the same specifications, B is independent of the design parameters and D is equal to D_c and D_d, which correspond respectively to the costs of collapse and other failure models, and may be computed as follows:

$$D_\tau = \bar{A}\, E_c\, (\delta) \tag{4}$$

$$D_d = E_c\, (\beta\, \delta) \tag{5}$$

In these equations, $\bar{A}$ is the expected cost in case of collapse, E_c means expectation with respect to uncertainties associated to variables in the group C defined above, and

$$\delta = \frac{E_B(\mu)}{1 - E_B(\mu)} \tag{6}$$

$$\mu = \frac{E_A(\Lambda)}{1 + E_A(\Lambda)} \tag{7}$$

$$\beta = E_2 \int_0^X \left(\frac{\Lambda'_u}{\Lambda} \right) w\left(\frac{u}{X} \right) du \tag{8}$$

in which $\Lambda = \lambda/\gamma$, λ is the mean rate of exceedance of the intensity which produces collapse, X is structural strength, u stands for structural response and w is the cost of damage for u<X (that is, before collapse). Also, $\Lambda'_u = d\Lambda_u/d_u$, $\Lambda_u = \lambda_u/\gamma$, and λ_u = rate of exceedance of response u. E_A and E_B mean expectation with respect to variables in groups A and B, respectively. In some cases, it may be of practical advantage to approximate D by the expression

$$D = E\left[\lambda_1 \left(p\, \bar{A} + w_o \right) \right] \tag{9}$$

where λ_1 is the rate of occurrence of earthquakes of engineering significance; and p and w_o are respectively the conditional probability of collapse and the conditional expected cost of other failure modes given the occurrence of once such events.

The corresponding equations for the cases when seismic hazard is represented by a renewal process model are presented in ref. 4 and will not be discussed here, except to say that D is an increasing function of the time elapsed since the last large earthquake.

Maximization of U in eq. 1 is only one of various alternate approaches to determining optimum seismic design parameters. Other approaches advocate the definition of upper bounds to risk levels to be adopted by society or the study of marginal investments that a given society can make to save one human life per unit time. In one case study presented in the following a combination of two of these approaches was applied.

Decisions related to seismic safety of structures imply the investment of present resources for the benefit of both, present and future generations; therefore, they must consider the very significant but formally unresolved problem of transfer of resources and risks between generations. A very simple example illustrates the problem: the value of D for a structure to be built near a seismic gap (potentially hazardous seismic source with a long history of quiescence after having generated large shocks) will be higher than that which would be obtained on the basis of the Poisson assumption with the same rate of occurrence of large events. Therefore, the seismic design requirements would be more demanding than for the Poisson case. In the opposite situation, that is a short time after the occurrence of an episode including a large event and its aftershocks, the hazard per unit time would be lower than in the Poisson case with equal rate of event-occurrence, and so would be D. Following the same reasoning as above, the design requirements would now be less strict than for the Poisson case, and, with time, the hazard would grow, as well as the risk of failure, which might thus make future generations face excessively high risk values. The writer is not aware of any answers given to the questions arising from this problem.

3 UNCERTAINTY ANALYSIS

3.1 Seismicity and seismic hazard·

For highly active seismic sources it is relatively simple to obtain magnitude-recurrence functions (rates of exceedance of different magnitudes, per unit time), on the exclusive basis of statistical information in each source region. Accordingly, the estimation of intensity-recurrence curves at sites in the vicinity of those sources is within easy reach of code writing committees. Somewhat more difficult is to obtain estimates of probability density functions of the maximum magnitude or the maximum intensity to be expected during long time intervals. In some of these cases uncertainties associated to variables in group C of section 2 may be important.

A feature of highly active sources is that dispersion in the times of occurrence of large earthquakes are smaller than those implied by the Poisson process model. Seismic hazard in the vicinity of those sources is essentially determined by the so called "characteristic earthquakes", that is events with magnitudes contained in a relatively narrow range of high values, occurring at relatively uniform time intervals. For those cases when the average of these intervals is of the order of a few tens of years, it is reasonable to base seismic design regulations on a specified fractile of the probability distribution of the intensity produced at the site of interest by one of those characteristic earthquakes. A significant uncertainty for the calculation of this probability distribution is that associated to the coordinates of the focus or the rupture area of each characteristic earthquake generated at a given source area. In some other cases it is preferred to base the determination of seismic design requirements on the probability distribution of the maximum intensity to be expected during a given time interval (a few tens of years). In those cases it is necessary to count with reasonably good estimates of the probability distributions of the waiting times between characteristic earthquakes on each of the neighboring seismic sources, as well as the distributions of the magnitudes of those earthquakes. Because of the usual shortness of statistical samples at the potential seismic sources which determine seismic hazard at given sites, the determination of the required probability functions has to be based on relevant information gathered not only at the sources of interest, but also at other, similar or related sources. Uncertainties associated to variables in group C may be large in these cases too. Ref. 7 describes an approach for the definition of the form of the probability density function of the times between characteristic earthquakes, based on the criterion of minimizing the expected negative utilities arising when making seismic design decisions on the basis of an erroneous choice of the mentioned form.

In areas of moderate or low seismic activity the difficulties to determine the probability functions defined above may be larger. Their estimates will have to be based on statistical information corresponding to the sources of interest and to others (2, 9-10), and will be affected by large uncertainties of group C. The significance of these uncertainties on decisions related to selection of seismic design coefficients may not be too large when the criterion adopted is that of minimizing the sum of initial cost and the present value of future losses and maintenance costs; however, their significance for those cases based on the criterion of maximum tolerable risk deserves careful study.

The influence of local soil or topography conditions on the expected characteristics of future earthquakes may significantly deviate from that predicted by conventional one-dimensional wave propagation models, even for cases formerly thought to possess conditions favoring the applicability of those models, such as Mexico City (7, 8). A large fraction of those deviations may be systematic and therefore corresponding to variables in group C, while other may be related to the path and source characteristics of each earthquake, and therefore corresponding to variables in group A.

3.2 Structural reliability

A key step in the formulation of reliability-based seismic design codes is the calculation of failure and damage probability functions, expressed in terms of ground motion intensity, for various families of structural systems. For linear single-degree-of-freedom systems, the problem offers no difficulties, once the stochastic model of the ground motion for a given intensity is defined. The study of more realistic representations of actual structural systems is far more complicated, as a consequence of the influence of a large number of variables, which for many of them is far from understood. Among those variables deserve special attention the following:

a) Structural configuration and peculiarities, such as number of degrees of freedom, height-to-width ratio, along- height variation of strength and stiffness, and in-plan asymmetries;
b) Variation of safety factors with respect to location throughout the structures, and with respect to the different types of members and/or failure modes;
c) Models of mechanical behavior of materials and structural members; and

d) Assumptions concerning failure conditions and mechanisms for structural systems.

The following paragraphs describe some recent studies about the influence of the variables just mentioned. It will become evident that, although in some cases those studies succeed at showing the relevance of the variables under consideration, the quantitative relations between those variables and the failure and damage probability functions are far from understood. Available results can be (and have been) used in modern seismic design codes to grossly account for the most significant variables, but much is still to be learned along this line, which will serve to improve the consistency between safety levels of different structures designed in accordance with a given code.

3.2.1 Reliability functions and failure rates

The distribution of the maximum response of nonlinear systems, both hysteric and strength- and stiffness-degrading, to stochastic ground motion has been studied by means of analytical models based on approximations such as the equivalent linearization technique (11, 12). The original criteria, applicable only to the case of stationary excitation, have been refined to cover also some non-stationary models, but the computational difficulties and the restrictive assumptions implied by those criteria prevent their widespread application to large and complex structural systems, similar to those which must be studied in order to understand the influence of the variables most importantly affecting the reliability of usual real-world systems. These difficulties may be circumvented by means of Monte Carlo simulation, at the expense of long computation times. This approach was followed in ref. 13, which aims at assessing the influence of a number of structural parameters on computed failure probabilities of systems designed with the same safety factors for the same nominal intensities. For this purpose, it is assumed that building frames fail in a ductile manner by the formation of plastic hinges at those member sections where the acting bending moment reaches the local bending capacity, and that a brittle failure limit state is reached when the ductility demand at any given story, expressed in terms of lateral deformations of that story, reaches the available capacity of ductile deformation.

The studies in ref. 13 start from the assumption that, under the action of an earthquake of a given intensity, the structure may fail in any of n different modes, each corresponding to the exceedance of the capacity for ductile deformation at a given story. For the purpose of computing probabilities, failure is assumed to take place in the mode for which the ratio between ductility demand and capacity is largest, regardless of the order in which this condition may be reached at the different potential failure modes. This assumption permits to represent the results of simulations in terms of means and variances of the maximum values of the ratio between demand and capacity as functions of intensity, and hence to obtain estimates of failure probabilities, also as functions of intensity. These functions are later used in combination with assumed forms of the intensity-recurrence function at the site of interest, to obtain values of the quotient of the rate of failure of a structural system divided by the rate of exceedance of the intensity for which the system was nominally designed. Wide variations (from 0.003 to 0.213) were observed in the mentioned quotient for single- and multi-story frames with different numbers of stories and natural periods, all designed in accordance with the same code requirements. Among several conclusions, it is found that the number of degrees of freedom has a pronounced influence on the failure probabilities, and the structural failure rate decreases when the available ductility (and correspondingly, its design value) increases. This can be explained in terms of the contribution of the available lateral load capacity that any continuous frame has even if it has not been specifically designed for that type of load. The higher the capacity of the structure to take ductile deformations, the lower the additional lateral strength required to resist a specified set of lateral forces; therefore, the higher the design ductility, the higher, in proportion, is the contribution of the member resistances needed for vertical loads to the lateral strength required to take an earthquake of given intensity, and the higher are the earthquake intensities that may be resisted by the strength reserves due to the differences between expected and nominal values of member resistances. Most systems considered in the study under discussion are assumed to develop significant load yielding at several critical sections before a failure limit state is reached, and, as a consequence, neither the results reported nor the conclusions reached are valid if the safety factors with respect to local brittle failure modes are not sufficiently high with respect to those associated to ductile (bending) modes as to prevent the occurrence of the former. Finally, the variability of the failure probabilities obtained for the few cases studied is significant enough as to justify the development of new studies designed to gain greater understanding of it. Future investigations should not only widen the ranges of cases studied, but they should also explore better representations of the mechanical behavior of structural members and systems.

3.2.2 Influence of number of degrees of freedom

An introductory study of the variation of the seismic reliability of complex multistory building frames with the number of stories is undertaken in ref. 14, which presents criteria for the calibration of seismic reliability models characterized by different degrees of refinement. The study aims at providing tools for estimating the reliabilities that would result from detailed representations of the mentioned structures on the basis of those computed for simplified models. The final aim is to facilitate the development of systematic, parametric studies about the reliabilities of complex systems. Failure probabilities obtained from models with various degrees of refinement are calculated and compared. One family of simplified models studied is that of the single-degree-of-freedom systems with mass, stiffness and strength equivalent to that of the original, complex system. Another family includes multistory frames where the stiffnesses and strengths of beams and columns are concentrated on a smaller number of members (arranged in a single-bay frame). Uncertainties on masses, stiffnesses and strengths of components of the simplified "equivalent" systems were chosen so as to lead to the same mean values and variances of the global properties (mass, lateral stiffness and strength) as the original system, in accordance with the criteria stated in the mentioned paper. The reliability analysis for each system was formulated in terms of the first two moments of the probability density function of the behavior index, defined as the maximum value, for any story, of the ratio of the response to the capacity, both expressed as values of lateral story deformation.

The results show that for high excitation values the failure probabilities for the original systems may be significantly higher than for their simplified counterparts. Also, it was shown that it is possible to obtain transformation rules relating seismic failure probabilities computed for simplified and detailed models of structural systems belonging to a given family.

3.2.3 Structural configuration and peculiarities

The performance of a structure subjected to earthquakes is closely related to the values of ductility demand and cumulative damage, relative to both total or story displacements and local deformations, but the code requirements are expressed in terms of member forces and resistances, calculated on the basis of simplified models of the structure and its response: thus, the restrictions imposed on the maximum allowable stresses and deformations resulting from a static or a linear dynamic seismic response analysis for a specified base shear coefficient or design spectrum serve to put a bound on the probability that the critical ductility ratios or damage indexes will be exceeded under the action of all the earthquakes to which the structure may be subjected. The higher the correlation between the values of the control variables derived from the simplified models and the responses to be controlled in the actual systems, the greater the dependability of the design algorithms associated to those models. That correlation is highest for reasonably symmetric and uniform structures which do not show sharp spatial variations in either mass, stiffness and strength. Otherwise, the prediction of behavior on the basis of simplified response models is characterized by wide uncertainty margins, as well as by some systematic deviations. Both of these have to be covered by adequate corrections to the models applicable to regular structures. As illustrations of these cases some characteristics of the nonlinear response of a) buildings with soft first story and b) buildings with non-symmetric in-plan distribution of mass, stiffness and/or strength are studied in the sequel.

Ref. 15 contains a study about the nonlinear dynamic response of shear systems representative of buildings with excess stiffness and strength at all stories above the first one. This peculiar distribution of mechanical properties arises because of the presence of a significantly larger amount of walls (either designed as structural elements or not) in anyone of two orthogonal directions in the upper stories of a building, as compared to the amount of walls at the first story. A large number of these structures in Mexico City suffered severe damage during the September 1985 earthquake. This fostered the studies described in ref. 15, for the purpose of revising the seismic design code. The variables covered were the number of stories, the fundamental period, the along-height form of variation of story stiffnesses, the ratio of post-yield to initial stiffness, in addition to the variable of primary interest: the factor r, expressing the ratio of the average value of the safety factor for lateral shear at the upper stories to that at the bottom story. The lateral strength at the latter was taken as equal to the nominal value of the corresponding story shear computed by conventional modal dynamic analysis for the design spectrum specified by Mexico City design regulations of 1987 for a seismic behavior (ductility) reduction coefficient of 4.0. The excitation was in some cases the EW component of the accelerogram recorded at the parking lot of the Ministry of Communications and Transport in the same city during the 1985 earthquake, and in some other cases an ensemble of artificial accelerograms with similar statistical properties. It was concluded that the nonlinear seismic response of shear buildings the upper stories of which have lateral strengths and stiffnesses which correspond to safety factors larger than those applied to the first story is very sensitive to the relation between the average of the safety factors at the upper stories and that at the first one, as well as to the ratio of post-yield to initial stiffnesses. The nature and magnitude of the influence of **r** on the peak ductility demands at the first story depend on the low-strain fundamental natural period of the system. The ductility demands computed for elastoplastic systems may in some cases be extremely large. Accounting for P-delta effects leads to an enhancement of the sensitivity of the response with respect to **r**.

It would have proved excessively complicated, and perhaps overly optimistic, trying to provide corrective expressions for the prescribed design shear forces so as to approximate the variability of the influence of the different variables studied on the expected ductility demands and damage indexes at critical members and subassemblages. Therefore, the 1987 version of the Mexico City earthquake resistant design regulations account for the systematic effects and the wide uncertainties associated to the presence of a "soft first story" by requiring that base shear coefficients be increased by a factor of 1.33 when the ratio of the safety factor for lateral shear at the first story is less than 80 percent of the average value of the corresponding safety factors at the stories above the first one.

The nonlinear torsional response of buildings with asymmetric plan is another example of conditions for which the high probability of systematic and random deviations from the predictions of linear response models cannot be overlooked when writing seismic design regulations. The asymmetry in these cases is associated to mass, stiffness, strength, or combinations of them. The possible importance of these variables was emphasized after observing the large proportion of asymmetric structures which failed during the 1985 Mexico City earthquake (16, 17).

Some preliminary studies summarized in ref. 16 consider the nonlinear dynamic response of single-story systems subjected to the EW component of the September 19, 1985 recorded at the parking lot of the Ministry of Communication and Transport, in Mexico City. The lateral resistance of the systems studied was provided by three diaphragms parallel to the excitation. The location of these diaphragms was symmetric in plan, but their stiffnesses and strengths were different in some cases, thus leading to asymmetric stiffness and/or strength for the system as a whole. No transverse resisting elements were considered. In all cases the mass was symmetrically distributed with respect to a vertical section parallel to the excitation.

Results show that very large roof rotations (larger than those derived from linear dynamic analysis), and hence very large ductility demands at the end walls, may be obtained, in particular when strength eccentricities are much smaller than stiffness eccentricities. In the presence of a few results of this type, the 1987 revision of the Mexico City seismic design code established that, at any story, lateral resistances of the structural elements should be such that strength eccentricities were not much smaller than stiffness eccentricities. Allowable differences were made to depend on the response reduction factor adopted in design (18).

Wider studies of the problem are reported in refs. 19 and 20. These studies showed that the nonlinear response is very sensitive to the ratio of strength eccentricity to stiffness eccentricity (described in ref. 16), as well as to other variables, mainly the presence or absence of a pair of transverse resisting elements, capable of contributing significantly to the torsional resistance, and the relative contributions of mass eccentricity and stiffness eccentricity to the asymmetry of an associated linear system. The results of these studies are in agreement with those for the overlapping cases mentioned in ref. 16, but their range of coverage is much wider, and therefore provide a better understanding of the problem. From them, it is concluded that there is a significant influence of nonlinear behavior on the peak deformations of resisting elements, this influence being greater for stiffness-asymmetric than for mass-asymmetric systems. Large nonlinear rotations occur in short-period systems without transverse resisting elements, in particular when the strength eccentricity is much smaller than the stiffness eccentricity. However, because of the interaction between torsional and translational responses, the influence on the peak ductility demands at the resisting elements is in general significantly smaller than in roof rotations. Another consequence of this interaction is that in some cases the ductility demands at the resisting elements are larger for a mass-eccentric system than for its stiffness-eccentric counterpart.

The results also show that both, rotations and ductility demands, are scarcely influenced by the ratio of strength eccentricity to stiffness eccentricity in systems where a substantial fraction of the torsional stiffness and strength are supplied by transverse elements. However, this condition is not necessarily fulfilled by buildings similar to many which suffered severe damage in Mexico City in 1985, the structures of which consisted of two sets of mutually orthogonal beam-and-column frames, stiffened by walls covering two building faces meeting at a corner. The response of this type of systems needs to be studied.

The foregoing paragraphs make evident the difficulties to cover by simple codified rules the wide variability of systematic and random deviations of the torsional responses of actual nonlinear systems with respect to those derived from the analysis of linear models. In addition, some frequently occurring cases are still to be studied. Therefore, significant changes may be expected in design regulations related to this concept.

3.2.4 Non-uniform safety factors

The undesirable influence of some forms of unintended heterogeneity of safety factors on local ductility demands is discussed in section 3.2.3, above. In contrast, the advantages of adopting different safety factors in design, for the purpose of reducing the probabilities of occurrence of brittle failure modes relative to those associated to ductile ones, has been widely recognized. Well known instances of the applications of this concept are the higher values of safety factors recommended by modern building codes for design for local buckling and diagonal tension failure in steel and reinforced concrete beams, respectively, as compared to those applied to bending in the same members; or the practice of designing column and beam sections meeting at a joint in accordance with the strong-column-weak-beam philosophy. The following paragraphs describe the results of two studies dealing with the adoption of non-uniform safety factors: one deals with the variation of the failure modes and safety levels of building frames with the safety factors adopted for the design of beams, while the safety factors for columns are kept constant; the other considers the selection of the optimum combination of safety factors for the design of a structural frame and its foundation.

In the first study (21), a number of three-, fourteen- and twenty-story frames were designed in full accordance with the Mexico City building code of 1987, with the exception of the value adopted for the safety factor for bending of beams, for which three alternate values of that factor were considered: f_{sb} (the value specified in the code), 0.8 f_{sb} and 1.3 f_{sb}. These systems were analyzed under the action of an ensemble of artificial accelerograms (22) simulating the Mexico City, SCT, EW record of September 19, 1985, and plots were obtained of the peak values of the local ductilities at the most severely strained member section for each system. The results are characterized by very large dispersions; their general trend shows that raising safety factors at the beam ends leads to a reduction of the local ductilities at those sections in all systems studied, to an increment of the local ductilities at column ends in systems with moderate and long natural periods, and to a reduction of those effects at column ends in systems with short natural periods (21).

The studies reported in ref. 23 were motivated by the interest in understanding how the uncertainties and safety factors related with both a building structure and its foundation may affect the overall seismic safety, and in applying the knowledge gained to the formulation of recommendations regarding ratios between optimum safety factors to be applied to different members of the system. The study is based on the assumption that seismic design parameters, such as the nominal values of loads and mechanical properties and the safety factors relating them, are established on the basis of a decision formulation aiming at optimizing the expected present value of the utility to be obtained from the operation of the structural systems considered. Following conventional seismic design practice, it is assumed that design criteria are specified in terms of a design response spectrum, a set of load factors for several combinations of external actions, and a vector of strength reduction factors for the relevant failure modes.

The criteria mentioned above are applied to a set of simple systems, the response of which approximately reproduces the general characteristics of the response of tall buildings; in addition, the analysis of response and damage of a full system is carried out for calibration purposes. The variables considered were the height-to-width ratio of the full system, the seismic design coefficient and the strength reduction factors for design corresponding to overturning moment, one for the structure and another for the foundation. The other significant parameters were common to all cases studied; they are summarized in ref. 23. Response indexes, expected damage values and failure probabilities were obtained as functions of ground motion intensity, and were combined with information about an assumed intensity-rate-of-exceedance curve for a given site, in order to obtain curves representing present values of expected costs (initial cost, maintenance, damage and failure) as functions of the design safety factors. For the family of cases studied, the optimum design conditions corresponded to a strength reduction factor of 0.9 for horizontal shear, 0.72 for overturning moment on the structural frame and 0.9 on the foundation, but almost equal expected costs were obtained for the case when all these factors are taken equal to 0.9. However, the corresponding design intensities differ significantly. This points at the need for deeper and wider studies of the problem, for the purpose of improving codified recommendations.

3.2.5 Models of structural behavior

Most dynamic response studies which have served as basis for the estimation of the seismic reliability of multistory frames have assumed the validity of the elastoplastic or the bilinear non-deteriorating constitutive laws; P-delta effects have systematically been disregarded. However, the studies of dynamic response for more general types of constitutive laws show the very pronounced sensitivity of local- and story ductility demands to those laws. In many cases the occurrence of stiffness degradation leads to increased ductility demands as compared to those associated to non-degrading models, but in others the opposite may be true. For instance, ref. 16 shows some shear-beam systems for which the ductility demands including P-delta effects are much larger for the elastoplastic than for a stiffness-degrading Takeda-type constitutive law. Similar results are obtained in ref. 24 for single-degree-of-freedom systems with asymmetric force-deformation law, that is, with different lateral capacities in opposite directions. Therefore, uncertainties about constitutive laws must be explicity recognized when making estimates of seismic reliability of nonlinear systems. The problem of cumulative damage has received very little attention. Some introductory studies using the constitutive model presented in ref. 25 are underway at the National University of Mexico.

3.2.6 Failure criteria

One of the most significant weaknesses of the methods for the calculation of seismic reliabilities of complex systems is the absence of empirically proven criteria and models of failure (collapse, in particular) conditions for those systems when subjected to earthquake excitations. For the case of multi-story building frames, it is often assumed that failure may be associated to the condition that the lateral distortion of a story reaches a critical value, but very little is known about ways to estimate that critical value. Some advocate the use of factors measuring the capacity of ductile deformation, which multiplied by the story yield lateral distortions lead to the critical values of lateral distortions (13). However, there are no uniquely defined ways to determine yield distortions of frame stories and there is no experimental information about the capacity of ductile deformation at a story of a building frame. Failure criteria of complex systems have also been expressed in terms of local ductilities at inelastic regions (often idealized as "plastic hinges") of structural members. However, such criteria neglect the residual capacity available at a subassemblage of a complex system after fracture caused by excessive inelastic deformation has taken place at one or more "plastic hinges" in the subassemblage.

From the foregoing paragraphs it is concluded that for the purpose of developing criteria for the prediction of failure of complex systems subjected to earthquake, significant research efforts are required along the following lines: a) develop refined models of dynamic response, including stiffness and strength degradation related to damage, as well as P-delta effects, and define failure in terms of the occurrence of unbounded deformations; b) express the occurrence of failure as predicted by the above refined model, in terms of the story ductilities predicted by simpler models of nonlinear dynamic response, c) compare failure probabilities estimated on the basis of the story deformations computed in accordance with models a) and b); and d) calibrate theoretical models with observed behavior of actual systems, both in laboratory and under natural conditions (26).

4 DESIGN EARTHQUAKES AND SAFETY FACTORS

A general formulation of the establishment of optimum seismic design criteria was presented in the introduction of this paper. Applications to the simultaneous determination of design intensities and safety factors for various failure modes are presented in ref. 23. As described in the introduction to this paper the possibility of applying the same approach to practical cases is hindered by the difficulties to estimate failure probabilities and expected costs of damage as functions of earthquake intensity. Large efforts must be devoted in the near future to develop accurate and consistent criteria and methods for estimating these quantities. In the meantime, the applicability of formal optimization criteria as described in refs. 4 and 23 will probably be limited to establishing relative values of base-shear coefficients to be adopted for structures of the same type to be built at different sites. One such applications is described in ref. 5, dealing with the adoption of sets of base-shear coefficients which are consistent within an optimum-safety framework.

The consistency criteria proposed in ref. 5 are based on the following simplified assumptions:

a) Earthquake motions at a site occur in accordance with a Poisson stochastic process with random selection of intensities.
b) Each structure has a single failure mode. The failure condition is expressed by the value of the linear-response base shear ratio associated to the earthquake inten-

sity required to produce failure.

c) The initial construction cost (C) of a structure of a given type may be expressed as $A_o+A_1c^{\alpha}$, where A_o, A_1 and α are constants and c is the design base shear ratio.

d) The exceedance rate of base shear ratio c per year at site i may be represented as $\nu_i(c)=K_ic^{-r_i}$, where K_i and r_i are functions of the site of interest.

Consider two sites, A and B, where the exceedance rates of base shear ratio c are respectively ν_A and ν_B, with parameters K_A, K_B, r_A and r_B. Under these assumptions, the optimum design base shear ratios c_{oA} and c_{oB} at sites A and B are related as follows:

$$\frac{c_{oA}^{\,r_A}}{c_{oB}^{\,r_B}} = \left[\frac{r_A c_A^{\,r_A}}{r_B c_B^{\,r_B}}\right]^{1/\alpha} \tag{10}$$

Here, c_A and c_B are base shear ratios corresponding to an arbitrary (but the same at both sites) exceedance rate. This equation expresses the consistency condition tying the optimum design base shear ratios for structures of the same type at two different sites. Its application to formulate structural design requirements for different seismic zones in Mexico is described in ref. 5.

An approach to the calibration of computed failure probabilities and expected damage indexes with observations of the seismic behavior of real systems is presented in ref. 6.

5 CONCLUDING REMARKS: THE FUTURE OF SEISMIC DESIGN CODES

The writer does not have a crystal ball. Therefore, he will not be able to describe the times to come, but only to present his views about the desirable future. If knowledge and rationality are tantamount to best use of society's resources, then future seismic design codes should be expressed in terms of variables and criteria more directly related to the expected performance of structures and systems than according to present practice. This means that design rules based on comparing acting forces with available capacities should leave the way to criteria based on descriptions of expected behavior, probabilities of reaching various limit states, and quantitative measures of the corresponding expected consequences to society. This does not mean that regulations of the type we are used to apply will need to be abolished, at least in the short and intermediate term future, as those regulations serve to guarantee in the average a reliability level acceptable to society, at least for conventional "regular" structures. But even for these structures the discrepancies between the predictions of the models used for design, those derived from more "realistic" models and the observations of actual performance, may be excessively large, as described in the foregoing sections. Uncertainties arising from the use of simplified models are much larger for irregular or non conventional structures.

From the preceding paragraph it is concluded that the next step in the evolution of seismic design codes should imply the coexistence of two formats, either alternate or complementary: one similar to the present one, and the other based on performance, that is, on behavior, failure modes, and consequences. The second format, designated in the sequel as "performance oriented", should include three sets of concepts: quantitative performance requirements (or design decisions criteria), definition of uncertainty sources, and basic rules to evaluate uncertainties and to combine them. All these concepts should be general enough as to warrant their applicability to both conventional and innovative structural systems, and should include sufficient detail as to ensure that the relevant peculiarities are identified and the associated uncertainties are properly evaluated and taken into account.

Design decisions in future seismic design codes should be formulated in terms of expected multi-event time histories, and not only in terms of the expected performance, conditional to the occurrence of a so called "most unfavorable event". The formulations to be adopted should be expressed in life-cycle terms; that is, they should account for the expected performance of a system under the earthquakes that may affect it, as well as for the consequences of that performance and the corresponding maintenance, repair and reconstruction costs. Thus, seismic design will mean much more than providing a structure with the strength required to survive with reasonable damage levels the next large earthquake. It will start from conceiving that structure as an evolutionary system, where cumulative damage, as well as maintenance and repair policies and actions, form part of the initial design process. This means, for instance, that safety factors for the design of individual elements should be determined so as to propitiate the concentration of seismic damage at elements or locations chosen beforehand, in accordance with the corresponding easiness to replace or repair.

Before performance oriented seismic design codes may become a reality, significant efforts will have to be devoted to identifying, evaluating and systematizing uncertainties associated to seismic excitations, response and performance, as well as to consequences of damage and failure. Computational tools will have to be developed, and the new procedures and criteria must be widely disseminated among practicing engineers. But the efforts will be compensated by the progress towards more informed and efficient decisions.

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