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SOME NEEDS AND SUGGESTIONS CONCERNING THE DEVELOPMENT OF DESIGN REGULATIONS

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To the memory of Emilio Rosenblueth

ABSTRACT

The paper is intended to summarize some needs of improving the consistency of code provisions and to contribute to the implementation of such improvements. The two main directions dealt with are related to the specification of seismic input and to the control of risk. The considerations on hazard assessment and on specification of seismic input are concerned with the consistent use of Cornell's method, with the suitable formats of zonation maps, with the specification of design accelerograms and with the specification of seismic input for the case of arbitrary ground-structure interface. The considerations on risk control are concerned with the format of control criteria, with the estimate of deformation demands, with the capacity design in relation to 3D behaviour, with the consistent consideration of multi-parameter internal forces, with the specification of floor design conditions and with the evaluation of existing structures.

KEYWORDS

Capacity design; codes; design accelerograms; design criteria; existing structures; floor design conditions; hazard assessment; Monte-Carlo analyses; risk control; seismic input.

INTRODUCTION

The development of design regulations, witnessed during last decades, was aimed to improve design from various viewpoints. One of them is represented by the improvement of control of safety or, conversely, of risk. The paper is devoted to this particular field of work.

Achieving a satisfactory degree of risk control involves at its turn efforts in several directions, related to modelling of physical phenomena during one seismic event, modelling of the expectancy of events, using appropriate tools for risk estimates etc. Moreover, in case one considers risk at a wider scale, not only in relation to the mechanical performance of an individual structure, but in relation to complex systems, like urban systems or lifelines, and in case one wants to express risk in terms of expectancy of losses of various kinds, more generally of overall impact, important additional aspects which are qualitatively different as compared with mechanical phenomena are to be considered too.

Like any complex field, earthquake engineering and, more generally, earthquake protection, involve numerous intangibles and aspects that are not prone to fully rational treatment. It is the author's belief that, in spite of this fact, rational approaches are bound to gradually conquer new fields in practice. This paper is an attempt to put together some needs of improving the consistency of codes and practice, implementing more of the knowledge and understanding already at hand. The author believes that this attempt is in the spirit of the legacy of Emilio Rosenblueth, whose so widely recognized contribution to earthquake engineering and vision of the future, as so brilliantly presented in (Rosenblueth), besides his human warmth, brought him the author's deep esteem.

CONSIDERATIONS ON THE SPECIFICATION OF SEISMIC INPUT

General

The specification of seismic input should provide essentially input parameters:

a) along all *DOF* of the ground-structure interface;

b) with appropriate consideration of the frequency content and of the amplitude of expected, relevant, motions.

One may use here various possible representations (or models) of ground motions, referred to as:

R.1: stochastic representations;

R.2: design accelerograms (more precisely, systems of ...);

R.3: design spectra.

The representation *R.3* is generally used in codes and knowledge related to it is of obvious importance. Design spectra have nevertheless some important shortcomings too. They cannot provide by themselves any information on the inter-relation between the components of motion along different *DsOF* of the ground-structure interface (item (a) above). On the other hand, they are inherently compatible with the semi-probabilistic approach to safety control, which, at its turn, is not well suited for genuine probabilistic risk estimates. The representation *R.2*, which is increasingly used in engineering analyses, may be used for linear or non-linear analyses, and it can be specified for a desired number of *DsOF* of a ground-structure interface, provided a corresponding model is adopted. This basic model should be of category *R.1*. In general, the representations *R.1* may be used for various analytical purposes. The sequence *R.1*, *R.2*, *R.3* was adopted due to the logical filiation of the representations referred to.

Hazard Assessment

Considerable hazard underestimates can be witnessed unfortunately quite often. A striking example is the case of the North Los Angeles area, where the reference basic acceleration of 0.4 g, stated in design regulations (ATC) to correspond to a return period of 475 years, was considerably exceeded twice in a quarter of a century (San Fernando, 1971, Northridge, 1994). More must be done to avoid such situations.

The expected recurrence of seismic events (at source level, or at site level) is assumed further on to be modelled by a poissonian process. In case an event is characterized by a scalar severity measure, denoted q, the main recurrence characteristic considered is represented by the expected number of events of severity not less than q, to occur during a time interval of duration T, $N^{(h)}{}_q(q, T)$. The parameter q may be any severity characteristic of an event, at source level or at site level. The function referred to has a structure

$$N^{(h)}{}_q(q, T) = \int_q^{\infty} n^{(h)}{}_q(q')\, dq' \qquad (1)$$

where $n^{(h)}{}_q(q)$ is the density of frequency of events of severity q.

The use of Cornell's method (Cornell) as the basic way to derive local (or site) hazard characteristics by means of a convolution of the source hazard characteristics and of the attenuation law is widely accepted. Unfortunately, when using this method, one neglects as a rule the attenuation randomness, which is high

(Esteva, Sandi & Floricel). The use of a more consistent approach requires the explicit consideration of attenuation randomness. In case the average attenuation law (considered, e.g., for intensities I, dealt with as real numbers) is given by a function $I^*(M, r, h)$ (M: magnitude; r: epicentral distance; h: source depth; asterisk: expected value) and the conditional intensity distribution for an expected value I^* is $f(I, I^*)$, it turns out that, in case of a lumped source, Cornell's method is to be applied for the frequency densities $n^{(h)}{}_I(I)$ and $n^{(h)}{}_M(M)$,

$$n^{(h)}{}_I(I) = \int f[I, I^*(M, r, h)]\, n^{(h)}{}_M(M)\, dM \tag{2}$$

In case one assumes some spatial source distribution, the expression (2) can be generalized as in (Sandi & Stancu). A direct consequence of the use of convolutions in the sense presented is the lack of consistency of the concept of "design magnitude" (since this implies the disregard of attenuation randomness).

The importance of an explicit consideration of attenuation randomness was made obvious e.g. in (Sandi & Stancu) where it turned out that the assumption on the r.m.s. attenuation deviation strongly influences the outcome, i.e. the output on site hazard characteristics. It turned out also that the return period $T^{(h)}{}_q(q)$ of a site severity characteristic q (that may be I, or log PGA, or log EPA etc.) can be well approximated by a function

$$\lg T^{(h)}{}_q(q) = a_q/(b_q - q) - c_q \tag{3}$$

which can be of course reversed (q as a function of $T^{(h)}{}_q$). The systems of values $a_q = 17.5$, $b_q = 14.$, $c_q = 1.2$ for $q = I$ and $a_q = 14.$, $b_q = 5.8$, $c_q = 1.2$ for $q = \log_2 EPA$ (m/s^2) were obtained for Bucharest. These results appear to be in good agreement with the recurrence observed during the current century and seem to be credible for longer periods too (centuries, if not millennia).

In case one disposes of recurrence characteristics like those referred to previously, it is most suitable to reconsider the formats of seismic zonation and to specify, for the seismic zones of a territory, explicit recurrence characteristics (possibly, in terms of parameters like those of expression (3)).

Specification of Design Input

Following considerations are related essentially to the representations $R.1$ and $R.2$. Several codes (ATC, UBC, EC8, MLPAT) allow, or even recommend at present, the use of accelerograms and of time-history analyses in design. Unfortunately, there are very few code provisions on how to perform such analyses. Forgetting at this place about the most difficult problems of adopting appropriate models and complete constitutive laws for structural constituents (especially in case of combined internal forces), one may remark a gap between the ways in which one explicitly selects in practice this kind of input and the needs of relevance for the analysis of structural safety. According to the views of the author, a consistent specification of design accelerograms should rely on two bases: use of stochastic ground motion models ($R.1$) and compatibility with the requirements of risk analyses (even if, in practice, as a rule, one does not explicitly reach this goal). Without contending about the importance of natural accelerograms, it may be stated that their use should be related essentially to two main goals: explaining the performance of structures during past earthquakes (at sites where such records exist) and development of models intended to be used for a predictive specification of accelerograms. The models referred to should be stochastic ($R.1$), given the impossibility of deterministic prediction of accelerograms, and they should explicitly account for such features like spectral content and non-stationarity. These models should be used to generate systems of sample accelerograms, conditional upon the value of a scalar parameter q, like overall intensity of ground motion, or some reference acceleration (PGA, EPA) etc. or upon some vector Q to account also for frequency content, non-stationarity, possibly space correlation too. The definition of a space of accelerograms, to be represented in a condensed way by vectors Q, requires specific analytical work. Some discretization will be required for practical reasons. In this case, the hazard characteristics $N^{(h)}{}_q(q, T)$ are to be generalized as $N^{(h)}{}_Q(Q, T)$. The use of vectors Q and of corresponding hazard characteristics $n^{(h)}{}_Q(Q)$ promises to considerably reduce the scatter of vulnerability characteristics determined for structures, contributing thus directly to the accuracy and certainty of risk estimates. This may be a concern for the more remote future, yet it is promising a considerable return.

The use of conditional sample accelerogram systems in the frame of a Monte-Carlo approach will make it possible to perform conditional statistical analyses upon some relevant output parameters, like internal forces, drift displacements etc. The suitable outcome of such analyses is represented by full vulnerability characteristics of structural types or categories dealt with. The availability of conditional statistical data corresponding to various values of the scalar q (or of the vector Q), i.e. of full vulnerability characteristics, will make it possible to perform full risk analyses on the basis of convolution with hazard characteristics like $n^{(h)}{}_q(q)$, or its generalization $n^{(h)}{}_Q(Q)$.

The non-synchronousness of ground motion along parallel directions at relatively remote points is generally recognized, yet the features of non-synchronousness are poorly known to date. In spite of the poor direct instrumental information, this aspect, which leads among other to oscillations of "overall torsion" even for dynamically symmetrical structures, cannot be neglected. A consistent input specification appears to require here the use of appropriate stochastic models, to account for the features of non-synchronousness. Space correlation characteristics could be best expressed in terms of coherence factors, depending upon similitude criteria characterizing wave propagation. Since it is difficult to calibrate all parameters involved, a solution to be suggested is to perform parametric analyses for alternative values of some characteristic ground motion parameters (specifically, equivalent wave propagation velocities). In case such analyses are combined with representation *R.3*, the participation factors derived for spatial motion make it possible to perform *3D* analyses with corresponding explicit full input. Some developments of this kind were implemented in the Romanian code (MLPAT).

Summary on Specification of Seismic Input

Summarizing the needs emphasized and the suggestions presented in relation to the development of design regulations, one may state that:

1. The hazard assessment should rely on the use of Cornell's method, yet with explicit consideration of attenuation randomness in corresponding convolutions.
2. Zonation maps should explicitly specify recurrence functions for intensities or for reference accelerations (preferably: *EPA*) etc., for any of the zones defined. Unless other analytical expressions fit better the outcome of hazard analysis, an expression like (3) is proposed.
3. Design accelerograms should be specified in the spirit of full risk analyses, even if such analyses are seldom performed in practice. This appears to be a way to ensure appropriate consistency.
4. The specification of seismic input for *NDOF* ground-structure interfaces should rely explicitly on the use of appropriate stochastic ground motion models (representation *R.1*).

CONSIDERATIONS ON THE CONTROL OF RISK

General

The control of risk affecting various components of structures (more generally, various categories of elements at risk, like people, property, function etc.) means basically the determination of the risk of exceeding some definite thresholds, like limit states, or some loss thresholds. For the classical domain of earthquake engineering, which is related to the protection of structures, this means adopting a system of criteria and checking in some way that they will be fulfilled (with sufficiently high probabilities). While in the field of structural engineering the need and importance of using a probabilistic approach to safety verification problems is widely accepted, it is known that the application of probabilistic concepts and methods can be performed at present at three different levels:

P.1: the semi-probabilistic approach, which by now is widely implemented in codes in force;

P.2: the simplified probabilistic approach, based on the use of lower order moments of some standard distributions assigned to the random parameters dealt with;

P.3; the complete probabilistic approach.

A first precondition to perform meaningful risk analyses is to make sure that probabilistic modelling adopted is related to significant aspects and parameters. Qualitative aspects may be therefore much more important than the accuracy of calculations.

Control of Risk of Exceedance with Respect to Various Limit States

The tendency to consider simultaneously and systematically several limit states is increasingly visible in codes. Quantitative control criteria are specified, using alternative values of design accelerations, of reduction factors etc. Yet, more consistency would be welcome in this field too. A suggestion that seems to be appropriate for providing more consistency, is to specify a matrix of return periods of basic design factors (reference ground motion accelerations). This will be expressed in terms of return periods differentiated for different categories of limit states and classes of importance of structures. To illustrate this suggestion, three categories of limit states are considered

L.S.1: apparent damage to non-structural components, functionality moderately affected;

L.S.2: heavy damage to non-structural components, apparent damage to structural components, functionality heavily affected;

L.S.3: collapse, or at least building condemned;

and one considers four classes of importance (most residential buildings: class III), one could consider, for the conditions of Romania, the matrix of Table 1.

Table 1

Illustrative calibration of return periods related to design criteria

Class of importance	Return periods (years) for limit states: *L.S.1*	*L.S.2*	*L.S.3*
I	50	200	1000
II	20	100	500
III	10	50	200
IV	5	20	100

The return periods proposed may seem rather short in comparison with what is theoretically accepted as suitable return periods (e.g., the well known 475 years). In case one keeps in view that, according to recent hazard estimates (Sandi & Stancu) the values of *EPA* would be in Bucharest of some 0.7 m/s^2, 2.0 m/s^2 and 3.5 m/s^2 for the return periods of 10, 50 and 200 years respectively, and that a return period of 475 years would correspond to an *EPA* of some 0.45 m/s^2, it turns out that it may be impractical to think of return periods longer than suggested. Ultimately, a table like Table 1 is subject to negotiation.

One final remark in this connection is that classes of importance are to be defined not only with consideration of what could happen to an individual building (consequences for occupancy included), but also with consideration of the potential consequences of poor performance (damage, accidents etc.) for the environment (consider structures housing facilities with a potential of heavy pollution. fire, explosions etc.).

Control of Deformation and of Ductility Demands

Attention is increasingly and correctly given to the control of deformation of structures. This is related to several aspects, like absolute displacements (possibly generating severe second order, or P-Δ, effects), relative displacements between neighbouring structures (possible pounding), drift (in most cases, damage to non-structural components, but in some cases even possible brittle shear failure due to excessive ductility demand, leading at its turn to reduction of the compressed area able to transmit shear forces), ductility demand, control of deformation of isolators, and control of relative displacements in supporting devices of bridge floors. It is known that, in some cases, excessive deformation may lead to heavier consequences than excessive (nominal) loading. Thus, there is increasing agreement that control of deformation should govern design, but there are considerable difficulties to be overcome to reach this goal, since to date control of deformation is poorer than that of nominal loading. In cases when the required resources are available, the proper way to answer this challenge is to perform Monte-Carlo analyses of non-linear behaviour, in agreement with the conditions discussed in previous section, and to conduct on this basis at least sensitivity analyses, if not full risk estimates.

Capacity Design in Relation to 3D Analysis

Capacity design, aimed essentially to prevent brittle failure, is widely accepted at present as a philosophy of paramount importance for providing a satisfactory structural performance. Unfortunately, the quantitative requirements are illustrated in codes as a rule for *2D* cases of frame joints, where the condition that the sum of ultimate bending moments of columns incident in a joint should exceed the sum of ultimate bending moments of beams incident in the same joint is formulated. This condition, which would be correct in case of a plane frame loaded exclusively in-plane, can nevertheless in no way prevent by itself column failure in case of *3D* frames, where beams oriented along different horizontal directions can easily mobilize simultaneously their ultimate bending moments to produce column failure. It is thus of first importance to draw the attention of code users to the need of considering the capacity design philosophy in explicit connection with the *3D* performance of structures.

Checking Sections and Members for Combined Internal Forces

Critical sections of structural members are subjected as a rule to combined internal forces generated by the seismic action. This situation leads in some cases to biases in formulating checking, or design, criteria. It is reasonable to prescribe in this direction too some more consistent rules. A proper way to answer this question is to analyze the random walk of vectors representing systems of internal forces, to determine boundaries with controlled probabilities of exceedance by the random walk and to compare these boundaries with homologous boundaries corresponding to the bearing capacity.

Specification of Floor Design Conditions

Codes do seldom provide information or specifications on floor design conditions. Some improvement in this connection is of course increasingly important, given the increasing endowment of structures with valuable, often highly sensitive, equipment. The needs referred to consist essentially of drafting guidelines on how to specify floor design conditions or criteria. These may be related, according to the specific features of the problem, to floor accelerations, velocities or displacements (possibly relative displacements in separation zones), to floor response spectra etc.

In case one determines floor accelerations, velocities etc., starting from usual code formats for floor seismic loads, accelerations etc., it is necessary to consider values of reduction factors that are closer to 1.0 than the values used for determining design internal forces. Capacity overstrengths may be important and any potential, credible, source of overstrength (including the contribution of "non-structural" components) is to be considered, in order to avoid unconservative estimates of floor design conditions (according to limited data provided by floor accelerographic records in Romania, some buildings were able to withstand, with little damage, floor accelerations that exceeded by a factor of 2. or 3. the conventional design ones).

In case one determines floor spectra, it is suitable to do this for several directions in a horizontal plane, building thus roses of spectra. This is necessary, because the features of spatial natural modes of structures may lead to situations for which the most severe spectral values can correspond to directions that are not parallel to the main (or reference) axes considered in structural analysis.

The use of programs developed for linear behaviour of bearing structures and equipment may not correspond to the real behaviour and thus lead to results that are considerably more severe than what can be withstood, primarily by the bearing structures. There occur thus cases when floor spectra are to be developed relying on non-linear models and analysis of systems dealt with. When building non-linear models, the overstrength reserves referred to above should be never neglected, in order to avoid unconservative estimates.

Considerations on the Evaluation of Existing Buildings

A parameter used in several codes (ATC, MLPAT) in order to characterize the capability of existing buildings to withstand earthquakes is a ratio R, defined as

$$R = [\textit{available strength}]/[\textit{code demands of strength for new buildings of same category}] \qquad (4)$$

This ratio is apparently simple and attractive, but its use raises several questions, which are extensively discussed in (ATC). The Romanian experience permits to emphasize in this connection some aspects like:

1. The expected additional exposure duration for an existing building, up to its replacement, may be much shorter than the exposure duration for a new one. In cases when this difference is important, a less than 1.0 value of R is rationally acceptable, given the lower expectancy of strong earthquakes and its implications for the risk of damage or failure.
2. A new building is currently designed in many countries in a way to provide satisfactory ductility properties, while older buildings do not fulfill in most cases such requirements. It turns out that, due to this fact, the two conventional strength parameters considered according to relation (4) may not have the same relevance for the earthquake resistance capacities considered for comparison.
3. The strength demand for new buildings, according to codes, corresponds to some design material strengths which include the contribution of some partial safety factors. The evaluation of existing buildings may rely on the determination of some actual strength values, which may be considerably different. A problem on how to establish the material strengths to be considered in the use of relation (4) is raised.
4. The problem on how to define the strength of a building is raised in this connection too. There will be wide acceptance of some (limited) plastic deformation to be considered in order to estimate the capacity of a structure as a whole. It must be thus specified how much post-elastic deformation can be accepted in evaluating capacity, in order to make this comparable with code demands.

An attempt to answer at least partially the problems raised in this connection (assuming for some of the suggestions presented that only conventional static analyses are performed) is:

1. To specify the demand, or code requirements (essentially in the form of a reference elastic response spectrum) as an explicit function of return period.
2. To define structural models in agreement with the information on actual material strength characteristics, considering essentially expected values in relation (4). The structural models developed should account (possibly, alternatively) also for components which are not considered in structural design of new buildings, but which could nevertheless contribute to overstrength.
3. To perform repeated analyses of structural performance under alternative (static) loading distributions. The outcome of these analyses should be represented by force-deflection relationships, derived for alternative assumptions on the direction of loading, possibly also for alternative assumptions on the loading distribution along height and, what may be very important, for alternative loading amplitudes. It is suitable to determine also hysteresis loops for each of the alternative assumptions referred to and to estimate on this basis the equivalent damping characteristics, to be used in specifying the seismic input.
4. To combine the demands referred to at item 1 and the outcome of analyses referred to at item 3, to lead to assessments of the stages of performance corresponding to various return periods.

Summary on the Control of Risk

Summarizing the needs and suggestions related to the control of risk, one may state that:

1. The control of risk should be performed in a consistent way with respect to various specific limit states and the protection level required should be expressed in terms of return periods of the seismic loading, differentiated according to limit states and classes of importance.

2. Increased attention should be provided to the control of deformation and of ductility demands and appropriate techniques to estimate these demands, going sometimes up to full risk estimates, should be used.

3. The formulation of capacity design criteria should consider the 3D nature of structural behaviour.

4. Checking criteria for sections, members etc. should be related to the consideration of the random walk of vectors representing combined internal forces etc.

5. Codes should provide information on how to specify, in appropriate terms, floor design conditions (alternatively accelerations, displacements, spectra etc. with consideration of the expected characteristics of structures, non-linearity properties and overstrength included).

6. The quantitative evaluation of the existing building stock should be made in more relevant terms. The expected stages of performance, ductility demands etc. should be derived as explicit functions of the return periods of seismic loading.

CONCLUDING REMARKS

The paper was intended to emphasize some main needs of improvement and development of the regulatory basis of structural design, in order to provide better control of the seismic risk affecting structural performance. Some main points of the developments presented were related to the use of more appropriate representation of physical phenomena, as well as of some specific problems raised by risk estimates. In relation to the latter ones, it is the belief of the author that the semi-probabilistic method (referred to as *P.1* level in the paper) tends to become a major obstacle for the improvement of practical approaches in structural engineering (earthquake engineering included) and that surpassing this obstacle is dependent upon the gradual introduction of *P.3* level approaches. It is obvious that a brutal introduction of *P.3* level approaches in practice is beyond question at present. Codes may, nevertheless, be formulated in a way to suggest users to think beyond the limits artificially involved by conventional, fixed, values of design parameters (first of all, those related to loading) and to educate also users to gradually ask for questions whose ultimate response can be given by a *P.3* level approach.

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