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SEISMIC ANALYSIS AND DESIGN: CURRENT PRACTICE AND FUTURE TRENDS

R. RIDDELL and J.C. DE LA LLERA

Universidad Católica de Chile
Casilla 306, Correo 22, Santiago, Chile

ABSTRACT

A brief summary of the available methods of analysis and design, and a review of the state of practice in earthquake resistant design is presented. Although significant advance has been made in this field, the available information has not been synthesized and conveyed to general practice. A change in format of our seismic design codes is proposed, aimed to guarantee structural quality consistent with a predetermined earthquake performance objective. The concept of *validation ground motion* is introduced as opposed to *design ground motion*, and an integrated *design-validation* procedure proposed. The new design procedure shall include evaluation of the relation between cost and different levels of protection to enable owners choose an acceptable risk.

KEYWORDS

Seismic codes, methods of design, design philosophy, validation ground-motion, deterministic analysis procedures, probabilistic analysis procedures

INTRODUCTION

At the present time, the generally accepted philosophy of earthquake resistant design is that nonessential structures should: a) resist without damage minor earthquakes that might occur several times during the life of the structure, and b) do not collapse or cause loss of human lives even in the event of a major earthquake of small probability of occurrence. Intermediate design levels may also be defined for which some degree of damage may be accepted, as long as the structure remains operational and/or human life is protected. Further, the design levels also depend on the importance of the structure, its use, and the consequences of its failure.

Looking toward the future, various observations and questions arise when confronting this philosophy with today's practice:

a) Codes have not implemented explicitly the two (or three) design levels in terms of operational and quantifiable performance objectives. Furthermore, codes do not necessarily satisfy the important objective of survivability; of course one should not expect codes to *guarantee* that structures will not collapse, but rational procedures should be available to ensure safety against collapse according to the current state of knowledge.

b) Owners are not aware that the structures they acquire may result in total loss or even collapse during a severe earthquake. Wouldn't they be willing to pay for additional protection if an explicit cost-performance relationship was offered to them?
c) Earthquake design standards do not seem to be compatible with a maximum economic benefit criterion. Given that the cost of the structure is only a fraction of the total cost of the project, say 20%, and the cost of providing seismic resistance is a fraction of the structural system's cost, can't a few percental points of additional cost be economically justified to provide better seismic protection? Of course, a cost study should include the eventual cost of structural repair; however, accepting structural damage also means accepting deformations which result in nonstructural damage, damage to contents, and loss of service, which usually add up to losses that significantly exceed those due to structural damage.

Although answering the previous questions is not intended herein, a general earthquake analysis and design framework will be proposed for the future. A challenge for the earthquake engineering academic and professional community will be to implement such a framework or another with similar goals. For this purpose, the current methods of analysis and design are first reviewed, pointing out their strengths and weaknesses, as well as their necessary improvements. Finally, aiming to correct several inconsistencies implicit in current codes and practice, a new procedure is proposed for future earthquake resistant design.

GROUND MOTIONS

A great deal of information on ground motion records has been gathered since the days the classic El Centro 1940 record was the paradigm of strong motion. Important and abundant information has been obtained in large earthquakes in different conditions, such as Chile 1985, Mexico 1985, Northridge 1994, and Kobe 1995, among others, so that numerous records are available for direct use or as a reference for the generation of synthetic motions.

Although selecting the level of intensity of motion for design is still a difficult task, the frequency and amplification characteristics can be specified in terms of average normalized spectra with considerably less uncertainty. Comparison of spectral shapes from different studies and conditions reveals remarkable good agreement, providing confidence on the methodologies used and the general nature of the findings. Elastic and inelastic design spectra can then be presented in the Newmark-Hall form or other alternative shapes (Riddell, 1995).

Another useful representation of earthquake ground motions may be obtained by the so-called Power Spectral Density function (PSD). In simple terms, the PSD, which is defined for a random process, is the probabilistic equivalent of the power spectrum of a real signal. The PSD is defined in frequency domain and its integration over frequency leads to the mean square value of the process. Therefore, by using peak factors (Davenport, 1963) it is simple to obtain a response or design spectrum that is consistent with a PSD and vice versa. One of the advantages of using a PSD is that formally represents a ground motion process, i.e., with infinite possible realizations, as opposed to design spectra which are obtained for a small number of realizations.

Because of the need of using ground motion histories for inelastic structural analysis, or problems where ground motions vary spatially, it is becoming more common nowadays to generate synthetic ground motions. There exist a large number of procedures used for constructing synthetic ground motions, ranging from simple ones such as response spectrum compatible, or Kanai-Tajimi filtering, evolutionary models, to more sophisticated ones where the source, travel path, and incoherency models of the ground are incorporated (Irikura and Aki, 1985).

There are still some problems that need considerable improvement, like obtaining reliable ground displacements from strong motion records. However, the rapidly increasing worldwide ground motion database and techniques available for the generation of synthetic motions enable us to say that considerable advance has been attained in the knowledge of ground motions, so that the subject is not anymore the weakest link in the earthquake analysis and design procedure. Truly, it is the knowledge of the behavior of materials and the capabilities for nonlinear analysis which need a faster development.

CURRENT METHODS OF ANALYSIS

Depending on the nature of the variables considered, the methods of analysis can be classified as: deterministic or probabilistic. In deterministic analyses, specific values of the demand and properties of the system are used, while probabilistic methods recognize the uncertain nature of such. In the latter, probability distributions of the variables are used, thus permitting to quantify the reliability of the results.

Probabilistic methods in earthquake analysis and design of structures, as well as in other areas, provide an extra dimension in the interpretation and solution of the problem. By their use, computed element forces, stresses, deformations or any other response quantities not only assume a value but also a probability associated to it. This becomes essential, especially in the earthquake problem where important uncertainties exist in the input ground motion and properties of the soil, materials, and structure. However, in spite of the great theoretical and practical value of such methods, their use today in practice is limited. Several reasons might explain this: a) probabilistic procedures are more complex than their deterministic counterparts, b) engineers are not comfortable designing for values associated to probabilities since building codes do not establish a definite probability of exceedance for their designs, and c) the computational tools for stochastic analysis in practice are more scarce than those for deterministic analysis. Although some of these arguments are strong, the truth is that probabilistic methods in structural engineering should be used more frequently as a tool for assessing the reliability of a design (e.g., Wen et al, 1994).

Based on the level of response considered for the system, elastic (usually also linear) or inelastic models are used. The various methods can be grouped in each category depending on the form of presentation of the earthquake loading and modeling assumptions (see Figure 1).

It should be noted that for a selected analysis different levels of sophistication are always possible. For instance, a model may consider three components of ground motion and their spatial variations, or, as it is in most analyses in practice, just a single component. Another example are *true* three-dimensional (3D) structural models, which are perfectly possible nowadays; however, models that represent resisting planes independently are still commonly used. Even the 3D models may consider six degrees of freedom per node or just be a pseudo-3D analysis, in which, an assemblage of resisting planes is imposed by connecting the planes to rigid floor diaphragms and neglecting the compatibility of common nodes belonging to two or more planes.

Deterministic Linear Elastic Methods

Equivalent Lateral Force Analysis (ELFA)

ELFA is the oldest and simplest analysis procedure still being used for the analysis of low rise buildings. In this procedure, equivalent static lateral loads are used with magnitudes based on an estimate of the fundamental period of the building and the design spectrum. Their heightwise distribution is done according to standard code formulas. The method is deemed adequate for low and medium rise buildings with regular heightwise and planwise distribution of mass and stiffness.

Mode Superposition Method (MSM)

This method is applicable to any structure in its linear range of behavior. The method is based on decomposing the dynamic response of a structure into uncoupled modal responses (corresponding to single degrees of freedom with the same frequencies as the modal frequencies), which are then combined to compute the total response. Modal analysis leads to the response history of the system to a specified ground motion; however, the method is usually used in conjunction with a response spectrum. In such case, the modal maxima are computed and combined by available modal superposition rules such the CQC to obtain estimates of the expected mean-maximum response of the structure. If an elastic spectrum is used the method is exact; however, the method is often used together with a design spectrum whose ordinates have been reduced to account for inelastic behavior. The latter interpretation has led to two alternative analysis procedures, which use:

a) A design spectrum obtained by applying a constant reduction factor R, based on the fundamental mode, to an elastic design spectrum. Some scientists and engineers feel comfortable in such situation arguing that at least it corresponds exactly to the response of a structure to a smaller, service type, earthquake. Naturally, there is a fallacy in such a belief since the code is essentially reducing the elastic forces to account for energy dissipation, which has nothing to do with a representative motion for serviceability or elastic response. Indeed the fact that a constant R factor is used should be regarded as an imperfect way of representing the ratio of elastic to inelastic strength for a given ductility.

b) A true inelastic design spectrum associated with a period dependent ratio relative to the elastic spectrum. The use of such spectrum is intrinsically inconsistent with the linear response assumption of the MSM; however, the response thus computed may have the merit of keeping the contribution of the higher modes of vibration, which would be, otherwise, wiped out if a constant R factor was used.

Elastic Response History Analysis (ERHA)
It is a linear-elastic dynamic response analysis to a base acceleration excitation performed by direct integration in time of the equations of motion (rather than using modal decomposition). One interesting advantage of such procedure is that the relative signs of response quantities are preserved in the response histories, as opposed to MSM analysis in which the signs are lost. This is particularly important when interaction effects are considered in design among stress resultants.

The principal disadvantage of the linear procedures is that they give limited insight into the actual inelastic response of the structure under severe earthquake loading, in terms of the real displacement and ductility demands, and stress distributions. This obvious fact, however, should not disqualify these methods since a large number of structures which have been analyzed by these means have successfully survived major earthquakes. It is therefore believed that these analyses will continue to be used in the future since they are simple and well understood. Structures thus analyzed must be designed and detailed for adequate deformation (ductility) capacity. As it will be discussed later, the proposed integrated analysis-design approach combined with proper nonlinear response verification should guarantee the desired performance.

Deterministic Inelastic Methods

Plastic Analysis (PA)
This procedure was initially developed for the analysis of steel frame structures to take advantage of the redistribution of stresses that takes place in ductile redundant structures for loads beyond the elastic limit. Plastic analysis is a well developed and useful technique for the evaluation of structures in earthquake engineering. Based on principles of plasticity theory, plastic analysis provides a rational tool to establish collapse mechanisms in a structure when subjected to a loading pattern. Such analysis can be done very efficiently in practice using tools for linear programming such as the "simplex" method. Besides, plastic analysis provides a conceptual framework to understand not only the ultimate behavior of a resisting plane but also the complete three-dimensional behavior of a building.

Push-Over Analysis (POA)
Push-over is the analysis of a structure subjected to an increasing lateral load of fixed pattern in which the structural members plastify sequentially until a local or global structural failure is achieved. The analysis provides information on the strength and deformation of the building, and the distribution of demands, thus permitting to identify the critical members likely to reach limit states during the earthquake, which should, therefore, be given more attention during the design and detailing process. The strengths of this method are its simplicity relative to actual nonlinear dynamic response analysis, and its potential to expose weak links in the structure, i.e. give some insight on its performance. On the other hand, the principal shortcoming is the questionable validity of a fixed loading pattern. Discussion of advantages an limitations of the method and additional references are given by Lawson et al (1994).

Nonlinear Response History Analysis (NLRHA).
A complete three-dimensional NLRHA is possible today although important processing time and data storage

are required. Since these hardware limitations reduce every day, this procedures, which have had mostly academic use, should soon become customary in engineering practice. Yet, the great difficulty for nonlinear analysis is the limited understanding of the inelastic properties of structures, so that there are no generally accepted mathematical models for structural members, specially for reinforced concrete members in their various forms (walls, columns, beam-columns, joints) and predominant types of behavior such as shear and its interaction with flexure and compression. Besides modeling the hysteretic load-deformation behavior, limit or collapse states need to be identified and defined including disruption of concrete members due to crushing, punching shear, bond or anchorage failure, and of steel members due to fracture of welds and local buckling. Also, a quantification of the energy absorption capacity of structural members is essential. Furthermore, a realistic nonlinear structural model should include the foundations and surrounding soil. Extensive discussion of these topics can be found elsewhere (Fajfar and Krawinkler, 1992; Okada, 1993; and Fillipou and Issa, 1988).

Simplified Nonlinear Analysis (SNA)
Another competing alternative to full three-dimensional inelastic analysis is the use of simplified nonlinear analysis procedures such as the one recently developed by De la Llera and Chopra (1995a). In this procedure a single element is used to model the elastic and inelastic properties of a building story. The inelastic properties are defined using ultimate story shear and torque surfaces, each representing combinations of story shears and torques that lead to static collapse of the story. The basis for this model as well as its accuracy in different cases has been studied elsewhere (De la Llera and Chopra, 1995a). Among the most important advantages of the method are its simple use and interpretation, relative to a complete NLRHA, as well as the conceptual understanding of the earthquake behavior of the system that can be developed by using the model even prior to any dynamic analysis.

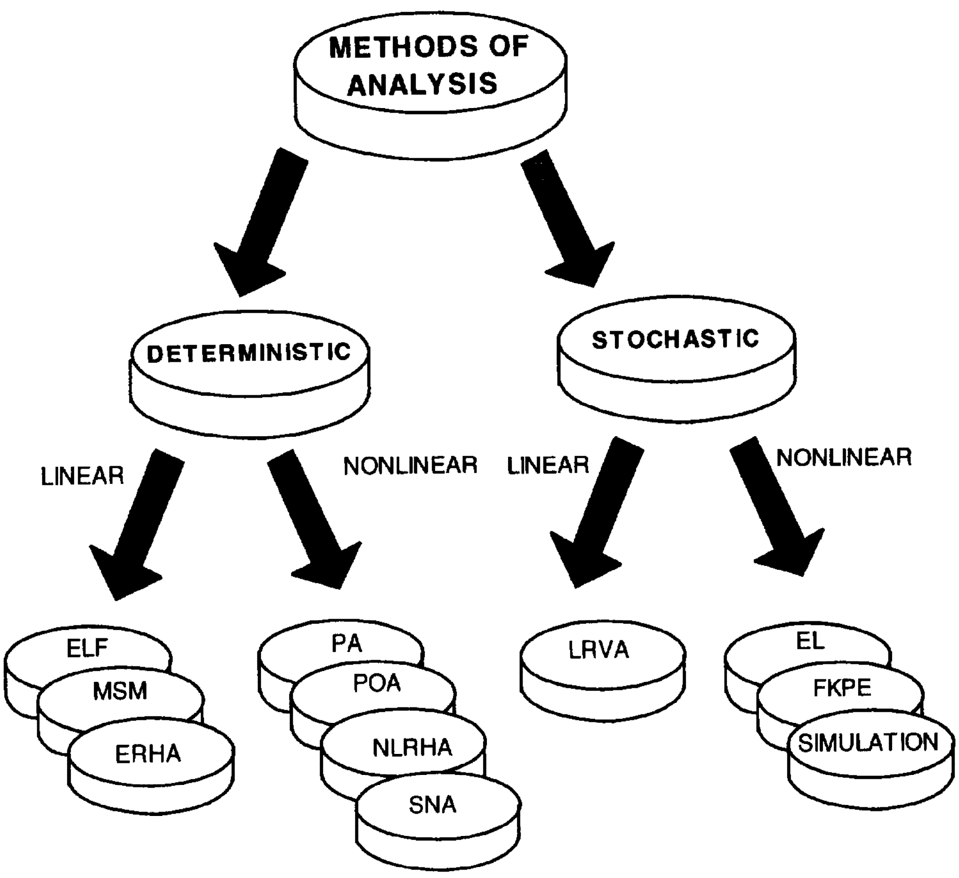

Figure 1. Classification of methods of analysis

Probabilistic Methods

As mentioned earlier, probability methods belong to a completely different family of methods which enable a richer understanding of the earthquake behavior of structures. A brief description of these methods is presented next.

Linear Random Vibration Analysis (LRVA)
Linear random vibration analysis is a technique to compute statistical properties of the response of a linear system subjected to inputs modeled as random processes. Although the technique is a powerful tool on its own, its use in earthquake engineering is better known as a theoretical justification for modal superposition rules, such as the SRSS and CQC procedures. Such procedures are certainly important results of the use of the technique; however, LRVA leads to equally important results in situations where conventional procedures are not straight forward. Examples of this situation are: (1) structures subjected to several inputs correlated in time and space (e.g., multiple support excitations), (2) structures with linear systems not amenable to "standard" modal decompositions (e.g., linear viscoelastic elements), (3) non-stationary responses, (4) non-gaussian input random processes, etc. Most frequently, LRVA is accomplished under the assumptions of wide sense stationarity and gaussianity of responses; such analysis is at reach of the engineering profession today and it could be easily implemented in earthquake analysis. The limitations of LRVA are similar to any other linear analysis procedure, in particular, its inability to predict accurate responses for inelastic systems. Nevertheless, linear systems lead in some cases to sufficiently accurate representations of their inelastic counterparts, especially if the inelastic excursions of the system are few and small.

Nonlinear Methods
Three methods frequently used to study the statistical properties of the nonlinear dynamic response of systems subjected to random inputs are: (1) Equivalent linearization, (2) Fokker-Planck-Kolmogorov Equation, and (3) Montecarlo Simulation. The three approaches are fundamentally different. In Stochastic Equivalent Linearization (SEL) the objective is to represent the nonlinear constitutive relationship of the elements through a linear spring-dashpot model and then solve the problem as a sequence of linear random vibration analyses. The parameters of the equivalent linear model of the elements are obtained through minimization of the variance of the error between the linear and nonlinear constitutive relation (Skinner et al, 1993); their values depend also on the variances of the deformation and deformation rate that are unknown at the time of linearization. Because of this the analysis becomes iterative, by first guessing the variances of deformation and deformation rates, then performing a linear vibration analysis, and finally correcting the initial guess of the response variances. The process continues until convergence is achieved. By using this procedure very complex systems can be analyzed with reasonable accuracy.

The Fokker-Planck-Kolmogorov equation (FKPE), also called the diffusion equation, is used to describe Markovian processes continuous in time (Piszczek and Niziol, 1986). The FKPE is a partial differential equation of the diffusion type on the probability density function of the response. For some special cases this equation may be solved in closed form, otherwise numerical integration techniques could be used. Because of the complexity of the formulation this procedure has been confined mainly to research.

Another very powerful procedure is Montecarlo simulation (Ross, 1990). The main advantage of this method is its proximity to the procedures used in deterministic analyses. Indeed, Montecarlo simulation is just a repetition or collection of deterministic analyses, each of them for a realization of the random input process (in the case of random vibrations). This collection of responses is analyzed to obtained mean, variance, and other statistics of the response quantities. The implementation of this process is straight forward and does not involve any additional difficulty than that associated to solving the problem in the deterministic case. Its main drawback is the higher computational cost since many analyses have to be repeated in order to obtain a good representation of the statistics and probability density functions of the response. However, several sampling techniques exist to reduce the number of simulations required to achieve minimum variance on the results.

Evaluation of Analysis Procedures: Some Results

The evaluation of the accuracy of the analysis procedures in use today has been the subject of several studies. Although it is impossible to make general statements, results presented in a recent study (De la Llera and Chopra, 1995b), in which the measured elastic responses of eight buildings during the Northridge earthquake were compared with the "blindly" predicted elastic responses using linear models, shows that discrepancies in peak responses may be vary from few percents to as large as 100 percent. This, considering that the models developed have carefully accounted for different modeling aspects such as elements not intended to resist lateral loads, principal non structural components, flexibility of joints, and so forth. It is apparent from the results that there is a direct correlation between the inaccuracy of the model and the complexity of the structure. Consequently, it is not the analysis procedures that lead to inaccurate results but the uncertainty in the model, especially in the stiffness properties of the resisting elements.

A similar analysis was also done using a fixed base building and two base isolated buildings that experienced, during the Northridge earthquake, inelastic behavior in the superstructure and isolators, respectively. For the fixed base building elastic and inelastic responses were computed and compared with the recorded motions. Improvements in the accuracy were observed in the inelastic prediction relative to the elastic one. In the case of the two base isolated buildings, the predictions obtained using inelastic models were remarkably better that the predictions obtained for conventional fixed base models. Such is the case because the behavior of the isolated buildings is controlled by the behavior of the isolators, which may be accurately modeled.

CURRENT METHODS OF DESIGN

Discarding very complex structures, building analysis usually reveals that the preliminary design is satisfactory and only minor changes to the size of members contributing to seismic resistance are necessary. This fortuitous outcome, together with the fact that codes do not require verification of the performance of the structure under a severe excitation, leads to the dissociation in practice of the analysis and design procedures. The latter then becomes a process of stress checking (determination of required steel reinforcement in R/C construction) and detailing; thus, the design consists only in applying a code for the material used. The most frequently used design procedures are summarized next.

Lateral Strength Design (LSD)
This is the most common seismic design approach used today. It is based on providing the structure with a minimum lateral strength to resist seismic loads. The seismic loads are specified in the seismic design code in conjunction with the ELFA and the MSM methods of analysis. Once the seismic internal forces have been computed and combined with other loads, e.g., gravitational and live loads, the structural members are sized using Allowable Working Stress Design (AWS) or Ultimate Strength Design (USD). In the case of AWS, the members are designed so that the materials do not exceed their allowable stresses. In the case of USD, the load is meant to represent the maximum probable load on the element; the critical section of which is designed to have a reduced nominal limit strength that shall not exceed the factored loads. Load factors and strength reduction factors depend on the material code (ACI for concrete and LRFD-AISC for steel). Both AWS and USD assume that the elements behave elastically at service loads; the advantage of USD is that it applies the safety factor to the section ultimate capacity (combining the contribution of steel and concrete to determine the ultimate strength in R/C) while AWS considers the materials stresses independently. The LSD approach is essentially based in providing strength, assuming that the structure will behave adequately in the nonlinear range. Checking of deformations is merely a mean to protect nonstructural elements rather than a way to check expected inelastic deformation in the structure.

Displacement Based Design (DBD)
Recognizing that damage in structures subjected to earthquakes is the result of excessive deformations, the DBD approach has been proposed (Moehle, 1992) to operate directly with deformation quantities, and therefore give better insight on expected performance of R/C structures, rather than simply providing strength as in the LSD approach. Based on some simplifications, an estimate of the displacement demands

is made to define element deformations wherefrom the required ultimate curvatures and material strains can be calculated and compared with the corresponding available capacities. Thus, the design procedure explicitly focuses on strain demands, and hence, provides some insight on the expected performance (damage) and required details.

Capacity Design (CD)
Based on the current seismic codes principles that accept severe excursions into the inelastic range, the method seeks protection of resisting planes against collapse by providing adequate ductility capacity to critical plastic hinge locations. By this approach (Park 1986), a collapse mechanism, for a given code type lateral force distribution, is chosen in order to *ensure* that yielding will occur only in the selected elements. Such elements are then designed and detailed so that ductility demands are met, while all other structural elements are provided with sufficient overstrength as to remain elastic; thus, yielding is confined to the predetermined locations.

The concepts involved in DBD and CD are a valuable contribution to seismic design because attention is placed on the performance structural members and structure. However, designers must be aware of the uncertainties involved in the selection of either a collapse mechanism or an ultimate displacement configuration. Indeed, a collapse or plastic analysis is done using as variables the internal forces at critical sections, e.g., the bending moments in frame analysis. By introducing the possible plastic hinge rotations at critical sections as a set of additional degrees of freedom, an attempt can be made to state the problem in terms of displacements (in very much the same way as in the standard stiffness method (Vasquez, 1996)); however, such approach which could be very convenient to determine required ductilities, invariably fails. If mathematical programming is used to solve collapse analysis stated in terms of displacements, the solution is found to be unbounded. Of course, this mathematical instability is due to the fact that infinitely many displacement configurations correspond to the same distribution of internal forces over the structure. Thus interpreting collapse analysis in terms of displacements is futile; displacements are entirely dependent on loading history, and different loading schedules can lead to widely different resulting displacements and also to substantially different internal force distributions. Hence, the independence of loading history inherent to the collapse state should not be directly extrapolated to infer characteristics of displacement configuration and ductility requirements at impending collapse.

Energy Based Design (EBS)
Energy concepts in seismic design have been recently emphasized as a mean toward better appraisal of the earthquake demands on structures (Uang and Bertero, 1988), and structural damage has been related to a linear combination of maximum deformation and energy dissipated by hysteresis (Park et al., 1985). The basic equation of interest is the energy balance $E_I = E_D + E_H$, which is valid at the end of motion when both the kinetic and strain energy vanish, where E_I is the total energy input per unit mass imparted by the ground to the structure during the earthquake, E_D is the total energy per unit mass dissipated by viscous damping, and E_H is the total hysteretic energy per unit mass dissipated by inelastic deformations during the response history (i.e. the total area accumulated under the load-deformation relationship).

The importance of the energy balance equation is that permits to visualize various design strategies with the purpose of minimizing E_H which is related to damage. One option is to minimize E_I by means of base isolation techniques, and/or maximize E_D providing the structure with energy dissipation devices. Alternatively, in conventional structures, E_I and E_H can be reduced by shifting the period of vibration of the structure; as a matter of fact, the input energy is not constant for all structures. Figure 2 shows average energy spectra (in terms of $\sqrt{E_H}$) for stiffness degrading systems with various response ductility levels (Garcia and Riddell, 1995); it can be seen that energy dissipation demands in the intermediate frequency range ($0.5 < f < 5$ cps) are considerably larger, by several hundred times, than the energy demands on very flexible or very stiff systems. Figure 2 corresponds to the average of spectra for 52 strong earthquake records selected from various locations in the world including the U.S. West Coast (24 records), Chile (15), Japan (9), Argentina (1), Mexico (1), Peru (1), and Rumania (1). The spectra shown in Figure 2 correspond to records normalized to peak ground acceleration equal to 1g. Normalizations to $v^{2/3}$ and d (with v and d equal to peak ground velocity and displacement respectively) are more appropriate in the intermediate and low frequency regions respectively.

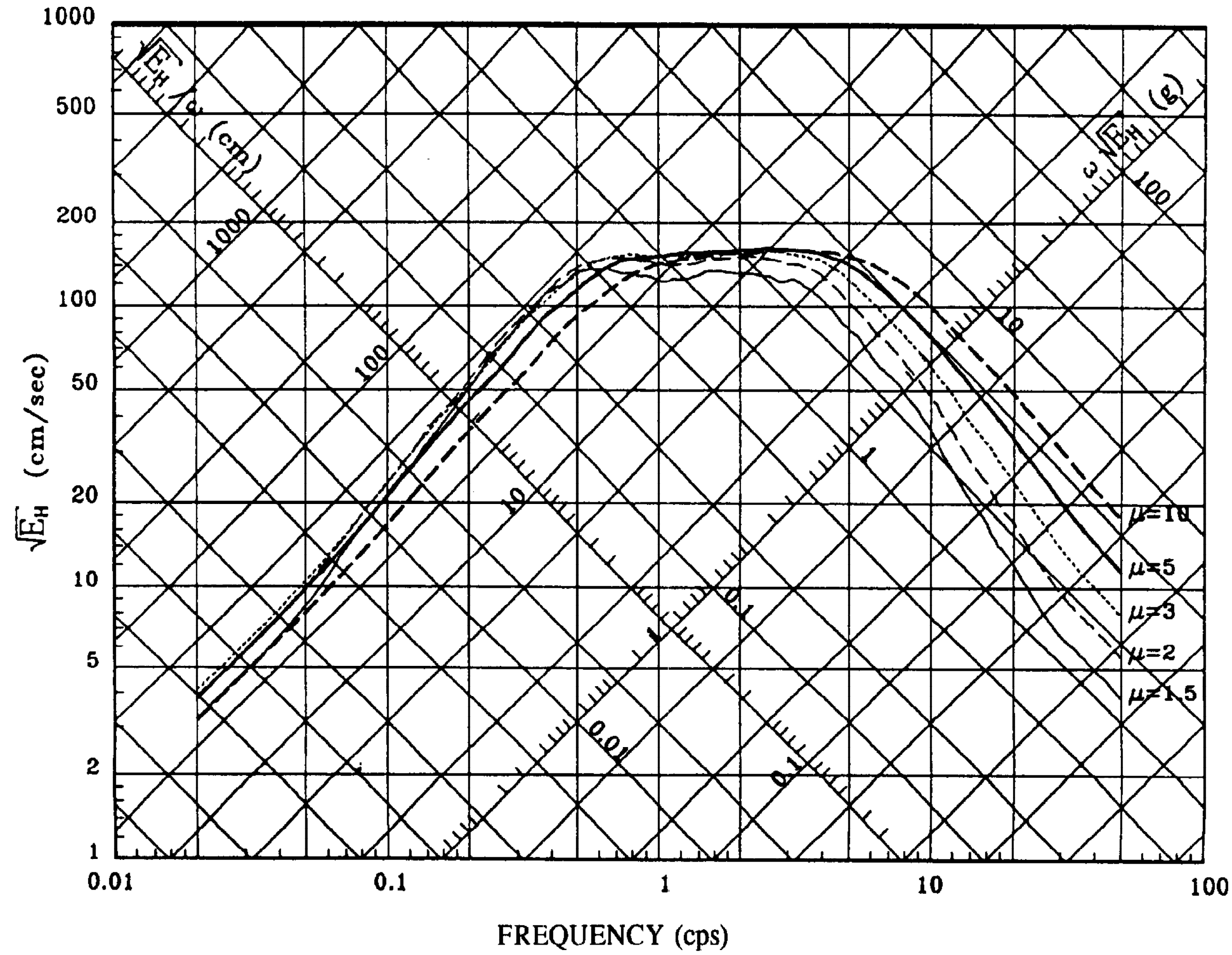

Figure 2. Average hysteretic energy spectra per unit of mass normalized to 1g peak ground acceleration for stiffness degrading systems with 5% damping and ductilities of 1.5, 2, 3, 5, and 10.

FUTURE TRENDS OF ANALYSIS AND DESIGN

A change in philosophy and format of our seismic design codes seems necessary for the progress of earthquake engineering. The principal characteristic of future codes is that they should provide an objective standard for validating earthquake performance of structures. Today, seismic codes are mostly a set of formulas and guidelines for earthquake resistant design, and efforts are continuously made to improve them, with the belief that a most comprehensive code should lead to safer designs. However, the intent is betrayed by the need to achieve simplicity, so that complex aspects of earthquake motions and response are represented by crude factors. As a consequence, during a moderate or severe earthquake a structure competently designed to comply with all provisions of the code may still have an unpredictable behavior.

A change in current practice is proposed herein. The proposed procedure, schematically illustrated in Figure 3, consists of:

i) two ground motion levels corresponding to serviceability and maximum demand are specified; ii) total freedom for the engineer regarding the analysis and design methods used (e.g. current methods are acceptable); iii) validation of the earthquake resistant design accomplished in the design step by satisfying performance criteria at the two response levels defined in (i); and iv) iteration between the design and validation steps until performance criteria is achieved.

In summary, the procedure guarantees a certain quality of design (or validation of a design). As mentioned, it gives the engineer total freedom regarding the procedures to produce a design, but specifies precise elements to validate the design in conformity with a predetermined performance criteria. In order to achieve the goals of the proposed procedure, the following basic requisites should be implemented in future codes:

- *Provide specific definition of validation earthquakes*
- *Give freedom to the designer and privilege good engineering*
- *Establish explicit performance objectives associated to various levels of protection*
- *Standardize acceptable models for nonlinear response validation*

in turn, the designer should:

- *Privilege high-performance structures*
- *State explicitly to the owner the quality of the design (level of protection) and provide the owner a cost-protection relationship*

Provide specific definition of validation earthquakes
The validation earthquake for checking given performance criteria is defined as a specific demand level for which the design is to be verified. The validation earthquake shall not be regarded as a *design ground motion* but as a *ground motion to test the design*. Two excitation levels are necessary: a maximum credible earthquake which has a small probability of occurrence during the life span of the structure, and a service type earthquake with a much larger probability of occurrence, e.g., the former may correspond to a return period of 500 years and the latter to 50 years.
The specification of the validation motions should include several compatible characterizations such us spectral shapes, power spectral density functions, and acceleration histories for the two excitation levels; each of these levels involve not a single acceleration record but infinite realizations (samples). The code shall specify the algorithm to generate the validation samples, using as a reference actual recorded ground motions for specific sites.
Although it is beyond the general scope of this paper, it is worth mentioning that extreme earthquake events larger than the maximum credible earthquake may not be of practical significance for design; in fact, very large magnitude earthquakes of very rare occurrence are primarily related to total energy and length of rupture, but do not necessarily imply a more severe motion at any given point. (Actually, seismic moment magnitude is proportional to the rupture area, but in various tectonic environments the rupture surface has limited width, thus justifying the previous statement of primarily dependence on length of rupture).

Give freedom to the designer and privilege good engineering
The code shall not specify a design spectrum, or lateral forces for design, or specific methods of analysis. The designer may freely use the various available techniques for design (among them LSD, DBD, CD) and others that may be proposed in the future. The designer may of course use as a guide elastic or inelastic design spectra appropriate for the required performance level, material, and structural configuration, but such spectra are different from the validation motion. In the new scheme, the designer will be free to produce a structure with some reasonable distribution of strength and deformation capacity, but the design shall be later checked with the standard validation earthquake to comply with the predetermined performance objective. If it does not pass the validation step, the design will have to be modified until it does. This form of operation results in an integrated analysis-design process, requiring the designer to be well aware of the consequences in the behavior of the structure that various alternative modifications lead to. It is expected that the designer skills and methods used shall progressively improve to produce better preliminary design solutions closer to the desired performance objective. This process is a radical departure from current practice in which analysis and design procedures are almost in cascade: given certain lateral forces, the analysis provides some internal forces, then the structural members are sized.

Establish explicit performance objectives associated to various levels of earthquake protection
A performance objective is the desired behavior of the structure for a specific validation earthquake level. For instance, for the maximum credible earthquake the predetermined performance level can be labeled by global concepts like *fully operational*, *operational*, *life safe*, and *near collapse* (SEAOC, 1995); in turn, for

the serviceability level earthquake only the fully operational global state is admissible. Each state must be defined by criteria in terms of limiting values of specific response parameters (displacements, stresses, drifts, plastic-hinge rotations, strains, etc.) and limiting load deformation configurations that may result in local collapse (member instability, joint disruption, punching failure, etc.).

Standardize acceptable models for nonlinear response validation
Except for the cases controlled by serviceability, which can be handled with available analysis capabilities for linear-elastic behavior, the verification for the maximum credible earthquake requires 3D nonlinear response-history capabilities. Although there is a great deal of information on the subject of material nonlinearity and inelastic response, this information has not been synthesized so far in generally accepted models and computer programs oriented to engineering practice. The quality-check character of the proposed procedure requires a standard validation of the design, it is therefore expected that the inelastic models for the different materials, the modeling techniques implicit in the software, and the software itself will need to be accredited by the standardization institution.

Privilege high-performance structures
In the design effort to pass the performance criteria, not only modifications of member stiffnesses and strengths shall be considered, but also including high performance elements like energy dissipators or even seismic isolation. The earthquake design of these systems today is an example of the performance oriented procedures proposed here.

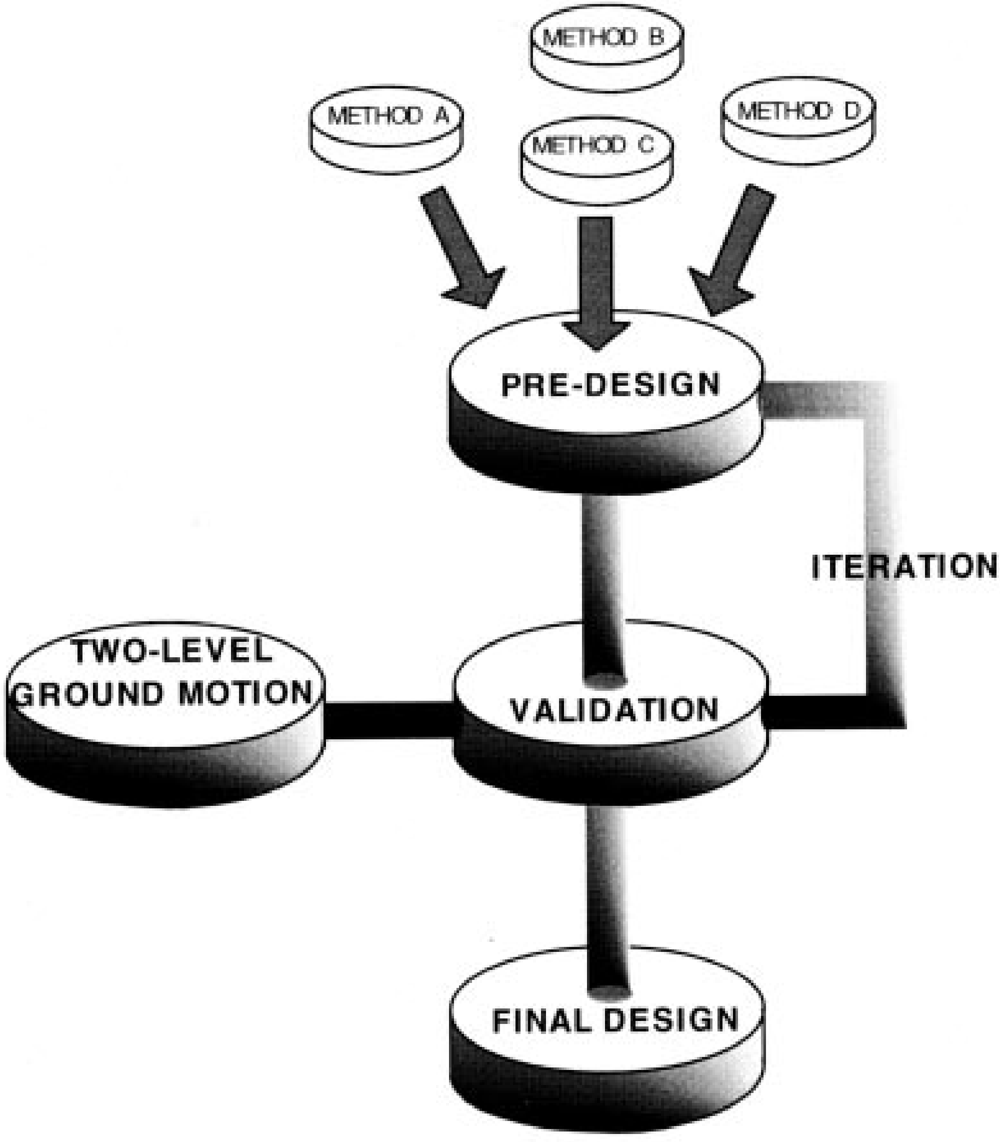

Figure 3. Schematic illustration of the proposed design procedure

State explicitly to the owner the quality of the design
It is becoming common that after a large earthquake owners of severely damaged structures, and society in general, are surprised when they learn that the affected structures have experienced the expected behavior according to our design philosophy. In the future, the owner should be aware of the level of protection, i.e., performance level, of the structure he is acquiring. For this purpose, the design process shall include evaluation of the relationship between cost and different levels of protection. Such a relationship must not only include the cost of preventing damage to meet a certain performance level, but also the cost of repair of probable structural and nonstructural damage, contents damage, and loss of service in the future.

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