APPLICATION OF A COMPREHENSIVE APPROACH FOR THE PERFORMANCE-BASED EARTHQUAKE-RESISTANT DESIGN OF BUILDINGS

Raul D BERTERO[1] And Vitelmo V BERTERO[2]

SUMMARY

Purpose.

After a brief introduction about the importance and definition of Performance-based Seismic Engineering (P-B SE), this paper addressed the following main objectives: (1) To review what are the main features that an Earthquake Resistant Design Approach (EQ-RD) should have to satisfy P-B SE should have; (2) To show why a comprehensive-based design is considered the most suitable approach for P-B SE; (3) To present a proposed comprehensive approach for design of buildings and discuss briefly the application of this approach to the design of two reinforced concrete buildings, one of 30 stories and another of 10 stories.

Outcomes.

From the analysis of the results obtained on the response of the buildings designed according to the comprehensive approach and taking in consideration from the beginning of the design process the probability of the occurrence of different types of earthquake ground motion time histories (from impulsive to periodic types) it is possible to arrive to a final design, whose seismic responses to these different types of ground motions and at the different levels of severity considered in the design are acceptable. The same cannot be stated when other simplified design approaches are used.

INTRODUCTION

A review of the lessons learned from significant earthquakes that have recently occurred regarding the seismic risks in urban areas leads to the conclusion that these risks are increasing rather that decreasing and that one of the most effective ways to reverse such situation in future significant earthquakes that can occur under or near urban areas is through first the development of more reliable seismic standards and code provisions than those currently available, and then their stringent implementation for the complete engineering of new civil engineering facilities and for the evaluation of the seismic vulnerability and upgrading of existing hazardous facilities. It is emphasized the need for a comprehensive approach for development and implementation of the next and more reliable generation of standards and codes which include consideration of the main aspects involved in the engineering of the earthquake-resistant facilities. A promising approach for such needed development is what has been proposed as Performance-Based Seismic Engineering (P-B SE). After a short discussion about why the comprehensive based design is considered the best approach to implement P-B SE, a comprehensive numerical procedure for preliminary seismic design is presented.

DESIGN APPROACH MOST SUITABLE FOR P-B SE

P-B SE is defined as "consisting of the selection of design criteria, structural systems, ..., such that at specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limiting states or other usefulness limits". From the definition it is clear that the ideal design procedure should consider from the beginning that: a)"...at specified levels of ground motion" implies that a multi-level seismic

[1] *University of Buenos Aires. Buenos Aires. Argentina. Email: faber@interserver.com.ar*
[2] *UC Berkeley. California. USA. Email: VBertero@aol.com*

design criteria is needed, b) "... with defined levels of reliability" implies that a probabilistic design approach is needed, and c) "... not be damaged beyond certain limiting states" implies that the main parameters that indicate structural and non-structural damage should be specifically considered. After summarizing the main problems of some of the simplified approaches that have been proposed for P-B SE this paper discusses the application of a comprehensive approach developed to satisfy the above requirements for the performance-based earthquake-resistant design of buildings.

The Need for Multi-Level Seismic Design Criteria

The first step of the comprehensive design approach is the selection of the performance objectives (Table 1). These are selected and expressed in terms of expected levels of damage resulting from expected levels of EQGMs. The client makes this selection in consultation with the design professional based on consideration of the client's expectations, the seismic hazard exposure, economical analysis and acceptable risk. Design performance objective couples expected performance level with levels of possible seismic hazard, as illustrated in the Performance Objective Matrix (SEAOC 1995). Performance levels are defined in terms of damage to the structure and non-structural components, and in terms of consequences to the occupants and functions of the facility. The performance levels can be as follows: Fully Operational or Serviceable (facility continues in operation with negligible damage); Operational or Functional (facility continues in operation with minor damage and minor disruption in non-essential services); Life Safety (life safety is substantially protected, damage is moderate to extensive); and Near Collapse or Impending Collapse (life safety is at risk, damage is severe, and structural collapse is prevented). The seismic hazard at a given site is represented as a set of EQGMs and associated hazards with specified probabilities of occurrence (frequent, occasional, rare and very rare).

Table 1. Example of Performance Objectives

	EQGM'S Level Return Period T_R (years)	Struct. Damage Max. Local damage index DM	Limit State Failure Probability P_f	Non-Struct. Damage Max IDI	Limit State Failure Probability P_f
Serviceable	30	0.2	0.20	0.003	0.30
Operational	75	0.4	0.20	0.006	0.30
Life Safety	475	0.6	0.10	0.015	0.20
Near Collapse	970	0.8	0.10	0.020	0.20

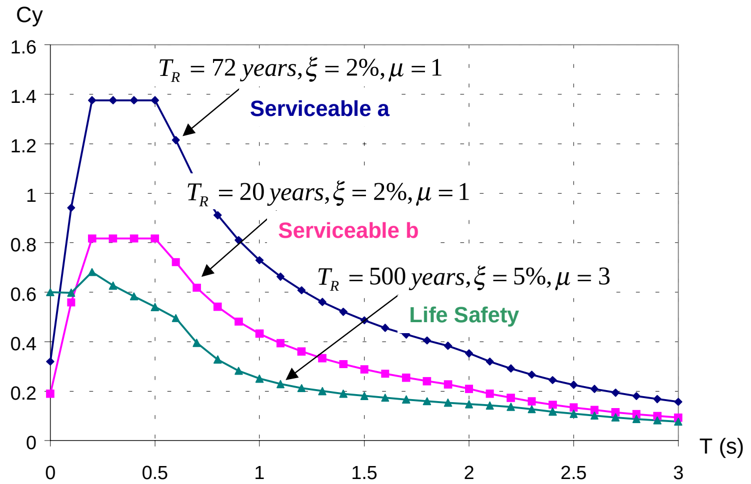

Fig. 1. Strength Spectra For Serviceability And Life Safety

It is very important that the designer recognizes from the very beginning of the design the implications of the selected performance objectives. For example, the strength design spectra for firm soil at San Francisco (USA) (Fig.1) have been obtained using the Newmark-Hall Design Spectra [Newmark and Hall, 1973], with peak horizontal acceleration according to 20, 72 and 500 years of return period [after Der Kiureghian and Ang, 1977], and the strength-reduction factors proposed by Miranda [1993]. It is clear from Fig. 1 that if a serviceability limit state is required for a 72 years of return period (as it has been considered recently for some California projects) not only the serviceability limit state will control the design but also innovative design approaches (base isolation, energy dissipation devices) should be used to economically satisfy the performance objectives. Even if a structure is built as strong as it is required, it will very difficult to avoid content's damage with a conventional design because of the large accelerations at serviceability limit state that are expected for buildings with a fundamental period of about 0.4 sec.

The Need for A Probabilistic Design Approach

The designer must recognize that the design requirements are not completely defined once the maximum damage index or IDI for each of the return period EQGMs are established. It is still needed to establish what is the allowable probability that this limit be surpassed due to the unavoidable uncertainties in the capacities and demands. In order to obtain a guide to develop a probabilistic design approach it is necessary to review the uncertainties of the different variables involved in the EQ design problem.

Means, coefficient of variations (COVs), and probability distributions for structural resistances have been determined from test data on the strength of materials and on dimensions of structural members, from laboratory tests of full-scale members under idealised load conditions and, in some cases, where a clearly defined analytical model exists, through Monte Carlo simulation. A representative sampling of these data, which summarizes results of numerous research programs, can be found in Ellingwood et al. [1982]. The COVs on resistance of structural steel and reinforced concrete members have values between 0.11 and 0.19, depending on material and failure mode. Much less data exists regarding the COVs on displacement capacity of structural members. Experimental data usually shows larger scatter in the ultimate displacement than in the ultimate resistance and therefore a COV slightly larger than 0.2 could be expected in general for the displacement and rotation capacities. On the other hand, seismic hazard studies [Algermissen et al. 1982] indicate that the COV of the peak ground acceleration (PGA) is site dependent ranging from about 138% in the eastern United States, where earthquakes generally can be associated only with seismogenic zones to 56% in the western United States, where ground motions often can be associated with causative faults. The COV of the demand depends no only on the uncertainty in PGA, but also on the uncertainties in the dynamic characteristics of the structure and EQGM time histories. For example, COV of strength demands normalised by PGA for ground motions records have been computed by Miranda (1993). Although the values obtained by Miranda are period dependent, a COV about 0.60 could be used for wide range of periods. Combining the 56% COV in PGA with the 60% COV in the strength demand for a given PGA, a COV about 80% is obtained for the seismic strength demand. Therefore a COV=0.8 could be used to represent the scatter in the EQ strength, plastic hinge rotation, IDI and damage demand.

Since a COV about 0.20 could be expected for the capacities and a COV of about 0.80 could be expected for the EQ demand, a simple probabilistic approach could be used for design. This simple approach is based on the fact that due to the dominant uncertainties in the demand it is possible to consider in the design all the random variables as deterministic (and equal to the mean value) with exception of the EQ demand. Therefore, the design equation for the mean capacity of each parameter X, μ_{Cx} (where X is any design target parameter like yield strength *Cy*, interstory drift index *IDI*, damage index *DM*, etc.) could be reduced to a load factor design as follows [Bertero R., 1997]:

$$\mu_{Cx} \geq \mu_{Dx}\left(1+\beta\delta_{Dx}\right) = \mu_{Dx} + \beta\sigma_{DX} \qquad (1)$$

Where, μ_{Dx} = mean demand for the design parameter X, δ_{Dx}, σ_{Dx} = COV and standard deviation of the demand for the design parameter X, and β = is a parameter to measure the target failure probability so that

$$P_f = \Phi(-\beta) \qquad (2)$$

being $\Phi(\;)$ =cumulative standard normal distribution. For example, for $P_f = 0.20, \quad \beta = 0.84$. Therefore, if a failure probability of 0.20 is specified for one performance objective and a COV of 0.80 is assumed for the

EQ demand parameter under study, the mean value (not the nominal or specified value) of the capacity parameter should be larger than $\left(1+\beta\,\delta_{Dx}\right)=1+0.84\times0.80=1.67$ times the mean demand. If a nominal value of the capacity is used, the demand amplification factor could be reduced according to the ratio between mean and nominal capacity. Note that β=0.84 is very close to the usual rule for designing with mean plus one-sigma spectra. However, it should be remember that no reduction factor is necessary with respect to the mean value of the capacity, and that the standard deviation, sigma, must include all the demand uncertainties (from the EQ magnitude and source to the structural response). The probabilistic analysis allows selecting specific values for the failure probability so that an economic optimization of the total cost of the building (initial construction costs plus repair and other costs due to EQ damages suffered along the life of the building) could be done.

The Need for A Preliminary Design That Consider A Cumulative Damage Index

To satisfy the definition of P-B SE there are a need to numerically compute different levels of structural damage for specific levels of EQGMs. Structural damage during an EQ may be due to excessive deformations, or it may be accumulated damage sustained under repeated load reversals. The earliest and simplest measures of damage were ductility-based, and so failed to take any account of the possible cumulative effect of repeated cycles of deformation [Williams and Sexsmith 1995]. More recently, a number of researchers have proposed damage indexes which take into account of cumulative effects by including the hysteretic energy. Although it can not accurately reproduce all possible load-deformation paths and damage mechanisms, the best know and most widely used of all the cumulative damage indices is that of Park and Ang [1985]. This consists of a simple combination of normalised deformation and plastic energy dissipation:

$$DM=\frac{\delta}{\delta_{u\,mon}}\left(1+b\frac{E_{H\mu}}{F_y\,\delta}\right)=\frac{\delta}{\delta_{u\,mon}}\left(1+b\gamma^2\mu\right) \tag{3}$$

Where, the parameter $\gamma=\sqrt{E_{H\mu}/\left(k\delta^2\right)}$ defined by Fajfar (1992) is a good factor to identify the type of EQGM (i.e., long duration of periodic strong motion or severe pulse), and the deterioration parameter b depends on the designer's decision about the kind of connections, detailing, level of axial load and shear at critical regions (plastic hinges), and aspect ratio of members. Fig. 2 shows the portion of damage due to hysteretic energy dissipated for the SCT (Mexico 1985), Los Gatos (Loma Prieta 1989), and Takatori (Kobe 1995) EQGMs considering $b=0.20$, $\xi=0.05$ and $\mu=3$. According to Fig. 2 the cumulative effect of damage could be responsible for more than 70% of the damage index in the case of a long duration of periodic strong motion like SCT (T=1.8 sec), but also could be responsible for about 40% of the local damage for impulsive type of ground motions like Los Gatos and Takatori for specific values of structural period (T=0.5 sec and T=3.5 sec respectively).

$$DM=\frac{IDI}{\theta_{u\,mon}}\left(1+b\gamma^2\mu\right)$$

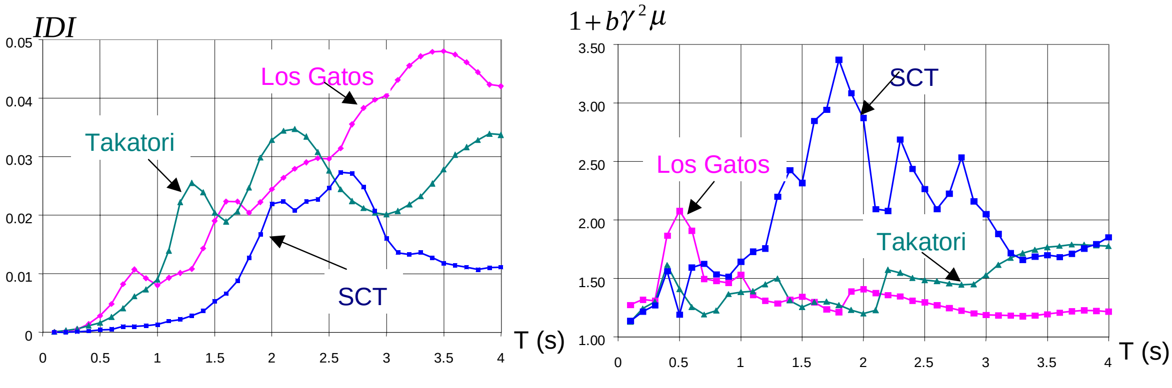

Fig. 2 Damage amplification due to cumulative effects of repeated cycles of deformation

The Need To Control Not Only Displacements But Also Ductility (Minimum Strength) To Limit Damage

From Eq. (3) it is clear that because of the cumulative cyclic damage to control damage is necessary to limit not only displacements but also the displacement ductility, and therefore is necessary to satisfy a minimum strength so that the required ductility be limited. Fig.3 shows the effect of ductility on the damage level for Los Gatos and SCT EQGMs (considering $b = 0.20$, $\xi = 0.05$ and the same $\delta_{u\ mon}$).

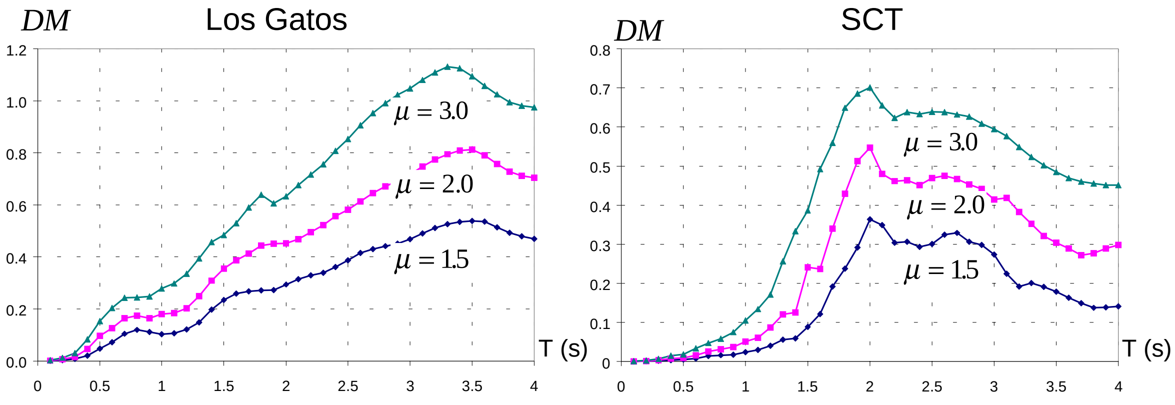

Fig. 3. Effect of the ductility on the damage index for severe pulse and periodic strong motion

The Need for A Comprehensive Preliminary Design

Several simplified design methods has been considered for Performance-Based Seismic Design (P-B SD): a) Strength based design, b) Displacement based design, and c) Energy based design. A discussion about the weakness of strength based design can be found in Priestley [1995]. In short, since P-B SD needs to limit damage, it is clear that displacement-based design should be a better approach because damage is more sensitive to displacement than to strength.

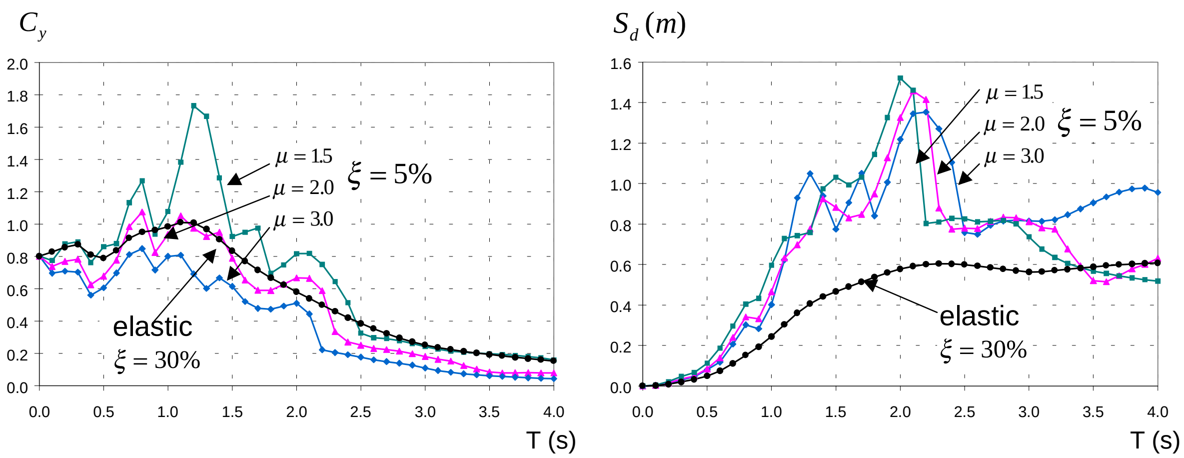

Fig. 4. Comparison of non-linear and "equivalent" linear strength and displacement spectra (Takatori)

Among the procedures that are related to displacement based design there are two groups. In the first group [Mohele 1992, ATC-40 1997], displacements are used mainly to check a preliminary design based on strength using serviceability or code forces. The second group [Priestley 1995] uses a simplified approach to obtain a preliminary design using only displacements. However, this method introduces the concept of equivalent damping that fusing two different properties in one could mislead the designers. Since damping involves dissipation of energy without damage, developing a velocity proportional force without limit, meanwhile ductility involves dissipation of energy with associated damage and a constant force after yielding, it is not possible to define just an unique equivalent damping for all the response parameters that need to be considered. Fig. 4 shows the spectra for strength and displacements for Takatori EQGM using 5 % of damping and 1.5, 2.0

and 3.0 ductilities, and using 30% damping and elastic behaviour. From Fig. 4 it is clear that damping is much more effective in reducing displacements than the ductility (note how important is that the designer have an understanding that ductility and damping are not equivalent in terms of displacement reduction). It is also noted in Fig. 4 that if an equivalent damping is defined using the dissipated energy, this equivalent damping can not reproduce the strength and displacement response.

A few simplified procedures have been developed for using energy-based design [Leelataviwat et al. 1998]. Again, although the input energy and hysteretic energy are probably the best parameters to select the design EQGMs, the energy based design approaches that have been developed until now satisfy the energy equation using basically a monotonic approach that can hardly represent the actual seismic response and damage.

Most of the methods that have been proposed have sacrified some important concept for the sake of simplicity. However, with the amount of specific software, spreadsheets, and mathematical packages available today, simplicity should be redefined. A numerical procedure is not simpler because an equation has fewer terms or some important parameter is ignored. A numerical procedure is simple when is easily understood, when the designer can go from the performance objectives to the design values in an explicit and transparent way. A design procedure, based on a probabilistic multi-level seismic design criteria, that consider explicitly from the beginning the performance objectives in terms of displacement, strength, energy dissipation, and damage for structural and non-structural elements and contents, is called comprehensive based design. A summary of a comprehensive based design procedure that was successfully used for the design of two reinforced concrete buildings, one of 30 stories and another of 10 stories is presented in the following section.

COMPREHENSIVE APPROACH FOR THE PERFORMANCE-BASED EARTHQUAKE RESISTANT DESIGN OF BUILDINGS

A comprehensive EQ-RD involves several steps: 1) Selection of the performance objectives, 2) Site suitability and seismic hazard analysis, 3) Conceptual Overall Seismic Design, and 4) Comprehensive Numerical EQ-RD [Bertero R. et al. 1996]. The last step is divided into two main phases: a) establishment of the design EQGMs, and b) numerical preliminary design procedure itself. In this paper only the comprehensive numerical EQ-RD will be discussed. In order to arrive at the desired final design, it is necessary to start with a preliminary numerical design procedure, whose main objective is a design that is as close as possible to the desired final design. The numerical preliminary design phase consists of three main groups of steps: ***(i) Preliminary analysis***, ***(ii) Preliminary sizing and detailing*** and ***(iii) Acceptability checks of the preliminary design***.

(i) Preliminary analysis. The preliminary analysis can be formulated, using an equivalent SDOF system as follows:

<u>GIVEN</u>: Function of building and desired performance design objectives; general configuration, structural layout, structural system, structural materials and non-structural components and contents (which should have been selected using the guidelines developed for conceptual overall seismic design [Bertero V. 1979, 1980]); gravity, wind, snow and other possible loads or excitations; and displacement, strength and hysteretic energy design spectra for different damping and ductility for frequent minor and rare major EQGMs.

<u>REQUIRED</u>: Establishment of design criteria (acceptable damage levels under the established EQGM levels), minimum stiffness (or maximum period T) and minimum strength of the building capable of controlling the damage, the design seismic forces, and the critical load combinations.

<u>SOLUTION</u>: to use a comprehensive approach that take into account from the beginning that the building structure is a MDOF and there can be important torsional effects even under service EQGMs (i.e. in the linear elastic response); that for safety EQGMs these effects can be different; and that it is also necessary to consider the desired damage index (control of damage), the hysteretical behaviour of critical regions of members and connections, and the ductility ratio that can be used, as well as the expected overstrength.

Using the given data, the designer must prepare design spectra for local structural damage (*DM*) and non-structural damage (*IDI*) like those of Fig. 5. If the use of energy dissipation devices is foreseen, these figures must be prepared for different values of damping. The IDI spectra can be computed assuming shear beam behaviour from:

$$IDI(T,\mu,\xi)=\frac{2}{H}\sqrt{\sum\left[S_d(T_n,\mu,\xi_n)\right]^2}\beta_1\beta_2 \tag{4}$$

Where, $H=$ is the total height of the building, $S_d(\ )$ is the displacement design spectra, β_1 is a coefficient to consider the *IDI* amplification due to torsional effects, and β_2 is a coefficient that quantifies the *IDI* increase due to concentration of plastic rotations in one story (usually also function of the global ductility, Hwang and Jaw 1990).

The local structural *DM* spectra can be computed modifying Eq. (3) for buildings as follows:

$$DM(T,\mu,\xi)=\left(1+b\gamma^2\mu\right)\beta_1\beta_2\frac{\theta}{\theta_{u\,mon}}\approx\left(1+b\gamma^2\mu\right)\beta_1\beta_2\frac{IDI}{\theta_{u\,mon}} \tag{5}$$

Where θ is the maximum rotation at the critical region during the seismic response; and $\theta_{u\,mon}$ the ultimate monotonic rotation for the critical region. In general, $\theta\approx IDI$ in multistory frames, and $\theta_{u\,mon}$ and b depend on the designer's decision about the kind of connections, detailing, level of axial load and shear at critical regions (plastic hinges), and aspect ratio of members. For example, for RC structures the designer could increase the amount of stirrups at critical plastic hinges (increasing $\theta_{u\,mon}$) to increase the acceptable μ considered in the design of the EQ-resisting structure. Using Fig. 5, and considering the specified performance objectives for structural damage (*DMsaf*), and non-structural damage (*IDIser*, *IDIsaf*) for service and life-safety limit states, the designer can select the minimum period and the maximum ductility from an explicit consideration of those performance objectives. Once the design period, ductility, and damping has been selected the design forces can be obtained using the strength spectra.

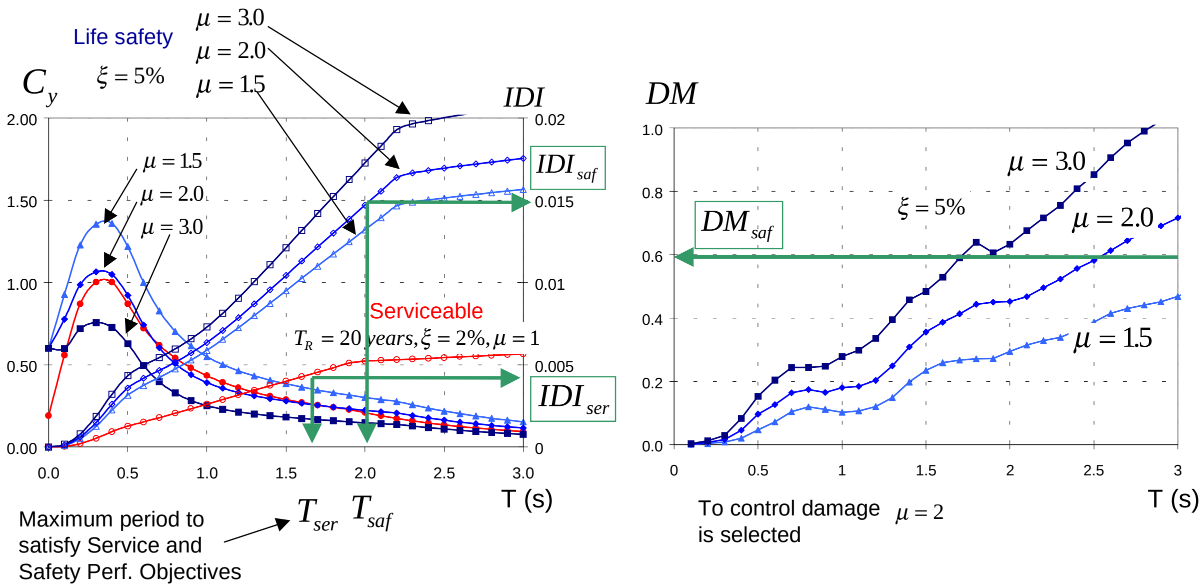

Fig. 5 Explicit selection of minimum stiffness and strength to satisfy performance objectives

(ii) Preliminary sizing and detailing. This step starts with the preliminary design for stiffness that requires the solution of three problems: a) given a target period and first mode shape to obtain the required story stiffness; b) given the required story stiffness to obtain the required member sizes; and c) given the required member sizes to obtain the effective moment of inertia of members. It can be shown that using a stick model of the building, from the selected first mode period, modal shape and story mass, the required story stiffness can be directly obtained from the top of the building to the bottom. Then, using a sub-structure model for each story, problems b) and c) can be solved, and the preliminary sizes of the members to satisfy the required stiffness are obtained [Bertero R. 1999]. Using plastic analysis and linear programming to optimize the amount of reinforcement, the beam flexural reinforcement of a RC structure could be obtained story by story satisfying simultaneously service and life-safety requirements. Then, using capacity design concepts, the columns and shear reinforcement are selected [Bertero R. and Bertero V. 1992].

(iii) Acceptability checks of the preliminary design. A discussion about the use of the different kind of analysis available for the acceptability checks has been presented by Bertero R. and Bertero V. [1992], and a discussion about the reliability aspects and the number of analysis required can be found in Bertero R., 1997.

CONCLUSIONS

From the analysis of the results obtained on the response of the buildings designed according to the comprehensive approach and taking in consideration from the beginning of the design process the probability of the occurrence of different types of earthquake ground motion time histories (from severe pulse to periodic types) it is possible to arrive to a final design, whose seismic responses to these different types of ground motions and at the different levels of severity considered in the design are acceptable. The same cannot be stated when other simplified design approaches are used.

REFERENCES

- Algermissen, S.T. et al. (1982). "Probabilistic estimates of maximum acceleration and velocity in rock in the contiguous United States". Open File Rep. No. 82-1033, U.S. Geological Survey, Reston, Va.
- ATC 40 (1997). "Seismic Evaluation and Retrofit of Concrete Buildings", ATC, Redwood City, CA.
- Bertero, R. (1997). "Acceptability checks for Performance-based Design", Report No.UCB/EERC-97/05, Earthquake Engineering Research Center, University of California, Berkeley.
- Bertero, R. and Bertero, V.V. (1992). "Tall Reinforced Concrete Buildings: Conceptual Earthquake-Resistant Design Methodology", Report No. UCB/EERC-92/16, Earthquake Engineering Research Center, UC Berkeley.
- Bertero, R., Bertero, V.V. and Teran A. (1996). "Performance-based Earthquake-Resistant Design Based on Comprehensive Philosophy and Energy Concepts", Proc. 11WCEE, Acapulco, Mexico, Elsevier Science Ltd.
- Bertero, V.V. (1979). "Seismic Performance of Reinforced Concrete Structures", Anales de la Academia Nacional de Ciencias Exactas, Físicas y Naturales, Buenos Aires, Argentina, Tomo 31, 75-144.
- Bertero, V.V. (1980). "Lessons Learned From Structures Damaged in Recent Earthquakes", Proc. 7 WCEE, Tomo 4, 257-264.
- Der-Kiureghian, A. and Ang, A.H.S. (1977), "A fault-rupture model for seismic risk analysis", Bulletin of the Seismological Society of America, Vol. 67, No.4.
- Ellingwood, B., MacGregor, J., Galambos, T. and Cornell, A. (1982). "Probability based load criteria: assessment of current design practice", Journal of Structural Engineering, Vol. 108, No.5, May 1982, ASCE.
- Fajfar, P. (1992). "Equivalent Ductility Factors, Taking into Account Low-Cycle Fatigue", Earthquake Engineering & Structural Dynamics, Vol. 21, No.10, October 1992, 837-848.
- Hwang, H., Jaw, J. (1990). "Statistical evaluation of deflection amplification factors for reinforced concrete structures", Proceedings Forth U.S. National Conference on Earthquake Engineering, EERI, El Cerrito, California, USA, Vol.2, 1990, 937-944.
- Leelataviwat, S., Goel, S., Stojadinovic, B. (1998). "Drift and yield mechanism based seismic design of structures", Proceedings Sixth U.S. National Conference on Earthquake Engineering, EERI, Oakland, California.
- Miranda, E. (1993), "Site-Dependent Strength-Reduction Factors", Journal of Structural Engineering, Vol. 119, No.12, December 1993, ASCE, 3503-3519.
- Mohele, J.P. (1992). "Displacement-based Design of RC Structures Subjected to Earthquakes", Earthquake Spectra, Vol.8, No.3, August 1992, 403-428.
- Newmark, N.M. and Hall, W.J. (1973), "Procedures and criteria for earthquake resistant design", Build. Res. Series No.46, Nat. Bureau of Standards, U.S. Depts of Commerce, Washington, D. C., 209-236.
- Park, Y.J. and Ang, A.H.S. (1985). "Mechanistic Seismic Damage Model for Reinforced Concrete", Journal of Structural Engineering, Vol. 111, No.4, April 1985, ASCE.
- Priestley, M.J.N. (1995). "Myths and Fallacies in Earthquake Engineering – Conflicts between Design and Reality". Recent Developments in Lateral Force Transfer in Buildings, ACI SP-157.
- SEAOC Vision 2000 Committee (1995), "Performance-Based Seismic Engineering", Report prepared by Structural Engineers Association of California, Sacramento, California.
- Williams, M. and Sexsmith, R. (1995). "Seismic Damage Indices for Concrete Structures: A State-of-the-Art Review", Earthquake Spectra, Vol.11, No2., May 1995, 319-349.