PERFORMANCE-BASED EARTHQUAKE-RESISTANT DESIGN BASED ON COMPREHENSIVE DESIGN PHILOSOPHY AND ENERGY CONCEPTS

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ABSTRACT

The main objectives of this paper are: to discuss the need for earthquake-resistant design (EQ-RD) and EQ-resistant construction (EQ-RC) approaches that will result in buildings with more predictable performance under EQ ground motions (EQGMs) than structures built according to current approaches; to present a general conceptual framework for performance-based EQ-RD and EQ-RC; to review a general comprehensive performance-based EQ-RD approach based on the use of energy concepts, performance (or damage) indices, and fundamental principles of structural dynamics and comprehensive design philosophy; to compare the general approach to current code; and to identify research needs for improving the proposed approach and its implementation in practice.

KEYWORDS

Comprehensive design; conceptual design; energy balance equation; energy-based design; damage index; damage spectra; performance-based seismic design; performance design objectives; performance levels; performance objective matrix.

INTRODUCTION

A review of the performance of facilities during the EQs of the last decade, particularly the 1989 Loma Prieta, the 1994 Northridge, and especially the 1995 Great Hanshin, clearly shows the need for EQ-RD and EQ-RC approaches that will result in civil engineering facilities that perform more predictably under EQGMs. The number of people made homeless and the level of economic loss from physical damage to nonengineered houses and engineered facilities, and particularly from functional and indirect damages, are socially and economically unacceptable. This is not surprising in view of the insistence of current seismic codes on EQ-RD approaches that are based on just a life-safety performance level which has no clear quantitative definition, and on following procedures that ***satisfy only strength requirements***. Although the understanding of the basic problems created by EQs and of the behavior of structures subjected to EQGMs has improved significantly, and this improvement has been reflected in the formulation of improved code requirements for the design and particularly the detailing of structural members, current seismic code design approaches fall short of realizing the goals and objectives of the worldwide-accepted philosophy of EQ-RD. Present seismic codes are not transparent, i.e., their regulations do not present in a visible way the basic concepts that govern the EQ performance of civil engineering facilities. Arising from the above need, already pointed out by the 1989 Loma Prieta EQ and emphasized by the 1994 Northridge EQ, the Structural Engineering Association of California (SEAOC), established the Vision 2000 Committee to develop a conceptual comprehensive framework for seismic codes. This framework, which is called ***performance-based seismic engineering***, regulates all areas that a seismic code should: conceptual overall design, preliminary numerical design, acceptability analysis, final design and detailing, quality assurance during

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construction and monitoring of occupancy and maintenance. The authors, based on studies they conducted previously, have developed and incorporated into this performance-based seismic engineering a ***comprehensive EQ-RD approach***. It considers four performance levels (service or fully operational, operational, life safety, and impeding collapse) and four levels of EQGMs (frequent, moderate, rare, and very rare or extreme). The iterative procedure involved in this general comprehensive approach is reviewed below in some detail. The conceptual methodology for the comprehensive performance-based EQ-RD was developed in accordance with the comprehensive design philosophy and in compliance with the worldwide-accepted EQ-RD philosophy, and is based on the use of energy concepts and fundamental principles of structural dynamics. It takes into account from the beginning of the EQ-RD procedure the simultaneous demands for strength (C_y), deformation (δ) and rate of deformation (including torsional effects), and their combined effects on the energy input (E_I), on the demanded and supplied energy capacities [E_E (elastic energy) = E_K (kinetic) + E_S (elastic strain), and the E_D (dissipated energy) = $E_{H\mu}$ (hysteretic plastic deformations) and $E_{H\xi}$ (damping)] of the entire facility system and on the acceptable damage at the different limit states associated with the desired performance levels. Before discussing this comprehensive EQ-RD approach, it is convenient to define and discuss briefly what is understood by performance-based seismic engineering.

PERFORMANCE-BASED SEISMIC ENGINEERING

The Vision 2000 Committee of SEAOC in its 1995 report has defined performance-based seismic engineering as "*a process that begins with the first concepts of a project and lasts throughout the life of the building. It includes identification of seismic hazards, selection of the performance levels and design performance objectives, determination of site suitability, conceptual design, numerical preliminary design, final design, acceptability checks during design, design review, quality assurance during construction, and maintenance during the life of the building.*" A conceptual framework for performance-based seismic engineering has been developed. This framework, which focuses on the case that seismic hazards control the design of building facilities, encompasses the full range of seismic engineering issues to be addressed in the design, construction and maintenance of structures for predictable and controlled seismic performance within established levels of risk. Herein only the methodology of the proposed comprehensive EQ-RD approach will be discussed.

COMPREHENSIVE EQ-RD APPROACH

Selection of Performance Objectives

The first step of the comprehensive design approach is the ***selection of the performance objectives***. These are selected and expressed in terms of expected levels of damage resulting from expected levels of EQGMs. This selection is made by the client in consultation with the design professional based on consideration of the client's expectations, the seismic hazard exposure, economic analysis and acceptable risk. A design performance objective couples expected performance level with levels of possible seismic hazard, as illustrated in the Performance Objective Matrix (Fig.1). Performance levels are defined in terms of damage to the structure and nonstructural components, and in terms of consequences to the occupants and functions of the facility. The performance levels in Fig. 1 are 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*).

Performance objectives typically include multiple goals. For example, for a given site they may be: fully operational in the 43-year event, operational in the 72-year event, life-safe in the 475-year event, and collapse prevention in the 970-year event. Two set of objectives are identified. *(i) Minimum objectives*: within this set, the *Basic Objective* is defined as the minimum acceptable performance objective for typical new buildings, while *Essential/Hazardous Objectives* and *Safety Critical Objectives* are defined as minimum objectives for facilities such as hospitals and nuclear material processing, respectively. These three minimum objectives are illustrated in Fig. 1 as diagonal lines in the *Performance Objective Matrix. (ii) Enhanced objectives*: other objectives providing better performance or lower risk than the minimum objectives may be selected at the client's discretion.

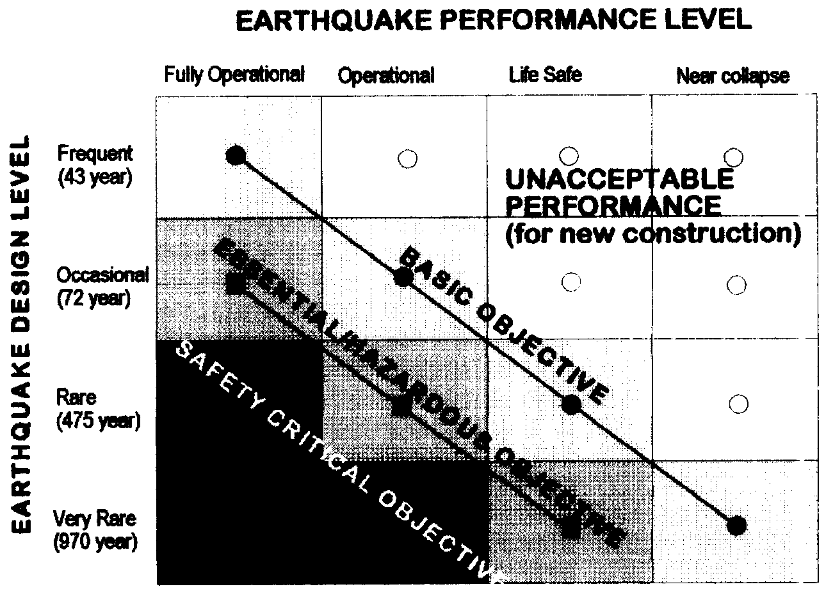

Figure 1. Recommended Seismic Performance Objectives for Buildings (SEAOC, 1995)

These objectives are termed "*enhanced objectives*." The selection of performance objectives sets the acceptability criteria for the design. The performance objectives represent performance levels, or damage levels, expected to result from the selected corresponding design EQGMs. The performance levels are keyed to limiting values of measurable structural response parameters, such as drift, deformation rates, and ductility (monotonic and cumulative) demands. When the performance levels are selected, the associated limiting values become the acceptability criteria to be checked in later stages of the design.

Site Suitability

Before starting the structural design process, site suitability and seismic hazard analysis must be undertaken considering the proposed performance objectives. The EQGM design criteria are established and characterized in a form suitable for the anticipated structural analysis and design methods. Seismic hazard analysis determines the design EQGMs and other significant actions for the specified design events considering all critical seismic sources. It has to be decided whether it is economically possible to build the building on the selected site.

Conceptual Overall Seismic Design

Once the performance objectives are selected and the site suitability and seismic ground motions are established, the structural design process begins with the conceptual overall seismic design of the facility and acceptability checks of the conceptual overall design. Since the conceptual design is closely tied to the desired performance of a building, guidelines specifying appropriate limitations for configuration, structural layout, structural system, structural materials, and nonstructural components and their materials are needed for each performance objective. These must be defined in terms that are usable at the conceptual design stage. The level of restriction should increase in severity with the level of performance objective, and should reflect the excellent historical performance of regularly configured structural systems composed of well-detailed ductile materials properly constructed and maintained. A list of guidelines for overall seismic design of the entire building system is given in Appendix B of the SEAOC 1995 report.

Selection of the structural system and the design strategy to be used in the preliminary EQ-RD and the final sizing and detailing of the structural members should consider application of energy concepts through the use of the energy balance equation, which can be written as:

$$E_I = E_E + E_D = E_K + E_S + E_{H\xi} + E_{H\mu} \tag{1}$$

where E_I is the energy input at the foundation of the building due to the EQGMs, E_E is the stored elastic energy, E_D is the dissipated energy, E_K is the kinetic energy, E_S is the strain energy, $E_{H\xi}$ is the energy dissipated through hysteretic damping and $E_{H\mu}$ is the energy dissipated through hysteretic plastic deformation. The designer analyzes whether it is technically and economically possible to balance the seismic demand (E_I) using only the elastic behavior of the structure (E_E), or whether it is better to attempt to reduce E_E by dissipating the effects of E_I as much as possible using E_D. As shown in Eq. 1, there are three ways to increase E_D: one is to increase the linear viscous damping, $E_{H\xi}$; another is to increase the plastic hysteretic energy, $E_{H\mu}$; and third is a combination of increasing both. It is common practice to try to increase $E_{H\mu}$ as much as possible through inelastic behavior through the use of deformation ductility ratio, which implies damage of structural members throughout the structure. Only recently has it been recognized that it is possible to increase $E_{H\xi}$ significantly and control damage through the use of *energy dissipation devices* at proper locations throughout the structure. Increasing E_D by increasing $E_{H\xi}$ rather than $E_{H\mu}$ has the great advantage of providing control of the structure's behavior through all of its limit states (impending collapse, safety, operational or fully operational performance levels). Increasing $E_{H\mu}$ by just increasing μ will not improve behavior at the service limit state. If it is technically or economically impossible to balance the required E_I by E_E alone or through $E_E + E_D$, the designer has the option of attempting to decrease the E_I to the structure. This can be done by *seismic base isolation techniques.*

Comprehensive Numerical EQ-RD

In accordance with the comprehensive design philosophy, in the comprehensive EQ-RD approach ***an iterative procedure*** that starts with an efficient preliminary EQ-RD is recommended. The preliminary EQ-RD is divided into two main phases: ***establishment of the design EQGMs***, and ***numerical preliminary design procedure***.

First Phase: Establishment of the Design EQGMs. The essential information needed is the time history of the expected EQGMs at the different recurrence periods of the performance levels to be considered. Because of the uncertainties in predicting such EQGMs, it is necessary to specify for each recurrence period a suite of EQGM time histories. With this information, engineers can compute the specified detailed information needed to conduct the preliminary EQ-RD and acceptability analysis. The specific information to be obtained from processing the time history of the EQGMs at each of the recurrence periods are the *Smoothed Inelastic Design Response Spectra (SIDRS)* for strength, total acceleration, velocity, displacement, energy input and energy dissipation corresponding to the predicted or established suite of EQGMs. These spectra have to be computed considering the different levels of displacement ductility ratio, μ_δ , that can be accepted according to the desired performance (damage) at the recurrence period under consideration. These spectra should include as a particular case the *Smoothed Linear Elastic Design Spectra (SLEDRS)* for $\mu_\delta = 1$, and the SLEDRS corresponding to the EQGMs inducing allowable stress.

Second Phase: Numerical Preliminary Design Procedure. 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.***

In the preliminary design, the structural framing elements are sized and checked against selected criteria. The sizing is accomplished using a systematic design approach, which usually involves designing to at least meet two performance design objectives, one for full operation (service) and one for life-safety.

(i) Preliminary analysis. The preliminary analysis can be formulated using an equivalent single-degree-of-freedom (SDOF) system as follows:
GIVEN: Function of building and desired performance design objectives; general configuration, structural layout, structural system, structural materials and nonstructural components and contents; gravity, wind, snow and other possible loads or excitations; and SLERS, SIRS and γ-spectra for frequent minor and rare major EQGMs. The parameter γ is defined by Fajfar (1992) as:

$$\gamma = \left(\sqrt{\frac{E_{H\mu}}{m}}\right) \Big/ \omega\,\delta \tag{2}$$

where ω is the frequency associated with the fundamental period of translation (T_1) of the structure, and **m** is

the reactive mass.
REQUIRED: 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.
SOLUTION: based on a transparent approach that take into account from the beginning that the building structure is a multi-degree-of-freedom (MDOF) system and that there can be important torsional effects even under service EQGMs (i.e., in the linear elastic response), and that for safety EQGMs these effects can be different; and that it is also necessary to consider the desired damage index (control of damage) corresponding to the hysteretic behavior of critical regions of members and connections, and the ductility ratio that can be used, as well as the expected overstrength.

Figure 2 shows a flow chart of the steps involved in the preliminary analysis. As shown in this figure, to initiate the preliminary analysis it is necessary to quantify the performance objectives by setting limits to the maximum value of all relevant response parameters, which for the case illustrated in this paper are: interstory drift demands for the service and safety limit states (IDI_{SER} and IDI_{SAF}, respectively); damage, through the use of damage indexes (DM_{SER} and DM_{SAF}); and probability of failure (PF_{SER} and PF_{SAF}). This quantification is possible through the knowledge of the qualitative definition of the Performance Objectives and through initial estimates of some of the relevant mechanical characteristics of the building, which should be established during the Conceptual Overall Seismic Design. For example, consider the limiting value assigned to IDI_{SER} or IDI_{SAF}: these values depend not only on the performance objectives, but also on the mechanical characteristics of the nonstructural elements and the detailing of their connection to the structure.

Because the preliminary analysis is formulated using an equivalent SDOF system, and this model cannot provide direct estimates of the local seismic demands, it is necessary to consider limits to the maximum value of the relevant response parameters at the global level. Thus, it is necessary to set limits to the global displacement demands in the building for service ($S_{d\ SER}$) and safety ($S_{d\ SAF}$) from their corresponding limits for IDI_{SER} and IDI_{SAF}. As shown, this is done by using the available estimates of the mechanical characteristics of the building to establish: a first mode shape, torsional effects, deviation from first mode shape, and concentration of plastic rotation demands over height.

Next, it is necessary to establish a value of target displacement ductility ratio, $\mu_{\delta TAR}$, defined as the maximum value of μ_δ that the structure can undergo during the safety limit state as limited by the requirement that the demanded $E_{H\mu}$ for this $\mu_{\delta TAR}$ will not exceed the supplied $E_{H\mu}$ capacity. As shown, establishing $\mu_{\delta TAR}$ requires using the available estimates of the mechanical characteristics to consider: concentration of nonlinear deformation demands, ultimate deformation capacity under monotonically increasing deformation, stability of hysteretic cycle, and $E_{H\mu}$ demands. The $E_{H\mu}$ demands can be estimated through the use of a γ-spectra obtained according to the estimated equivalent damping coefficient for safety (ξ_{SAF}) and the return period associated with the design ground motion for safety (T_{R2}). Although in many cases the value of γ is practically independent of the T_1 and μ_δ (provided $\mu_\delta \geq 2$), in some cases, such as EQGMs with very narrow frequency content, it may be necessary to consider the dependence of γ on T_1. Using the Park-Ang damage model (1985) and the parameter γ, estimates of $\mu_{\delta TAR}$ can be obtained from the following equality:

$$\frac{DM_{SAF}\ \theta_{u\ mon}}{\beta_2\ \theta} = 1 + \beta \gamma^2 \mu_{\delta TAR} \tag{3}$$

where θ is the maximum rotation at the critical section during the seismic response; $\theta_{u\ mon}$ the ultimate monotonic rotation for the critical section; β_2 a parameter that quantifies the IDI increase due to concentration of plastic rotations in one story; and β a parameter that characterizes the stability of the hysteretic behavior of the frame members. In general, $\theta \approx$ maximum IDI in multistory frames, so that $\theta \approx IDI_{SAF}$ can be considered as an upper bound for θ. On the other hand, $\theta_{u\ mon}$ and β 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 value of $\mu_{\delta TAR}$ considered in the design of the EQ-resisting structure.

Once the $\mu_{\delta TAR}$ has been established, it is necessary to estimate the value of T_{1TAR}, which is defined as the maximum value of T_1 that the structure can have to limit the deformation demands to values equal to or smaller than those imposed by the performance objectives for structural and nonstructural damage (i.e., according to the values of IDI_{SER} and IDI_{SAF}). As shown in Fig. 2, this is possible through the use of the displacement limits established for the service and safety limit states and their corresponding displacement spectra. Note that the

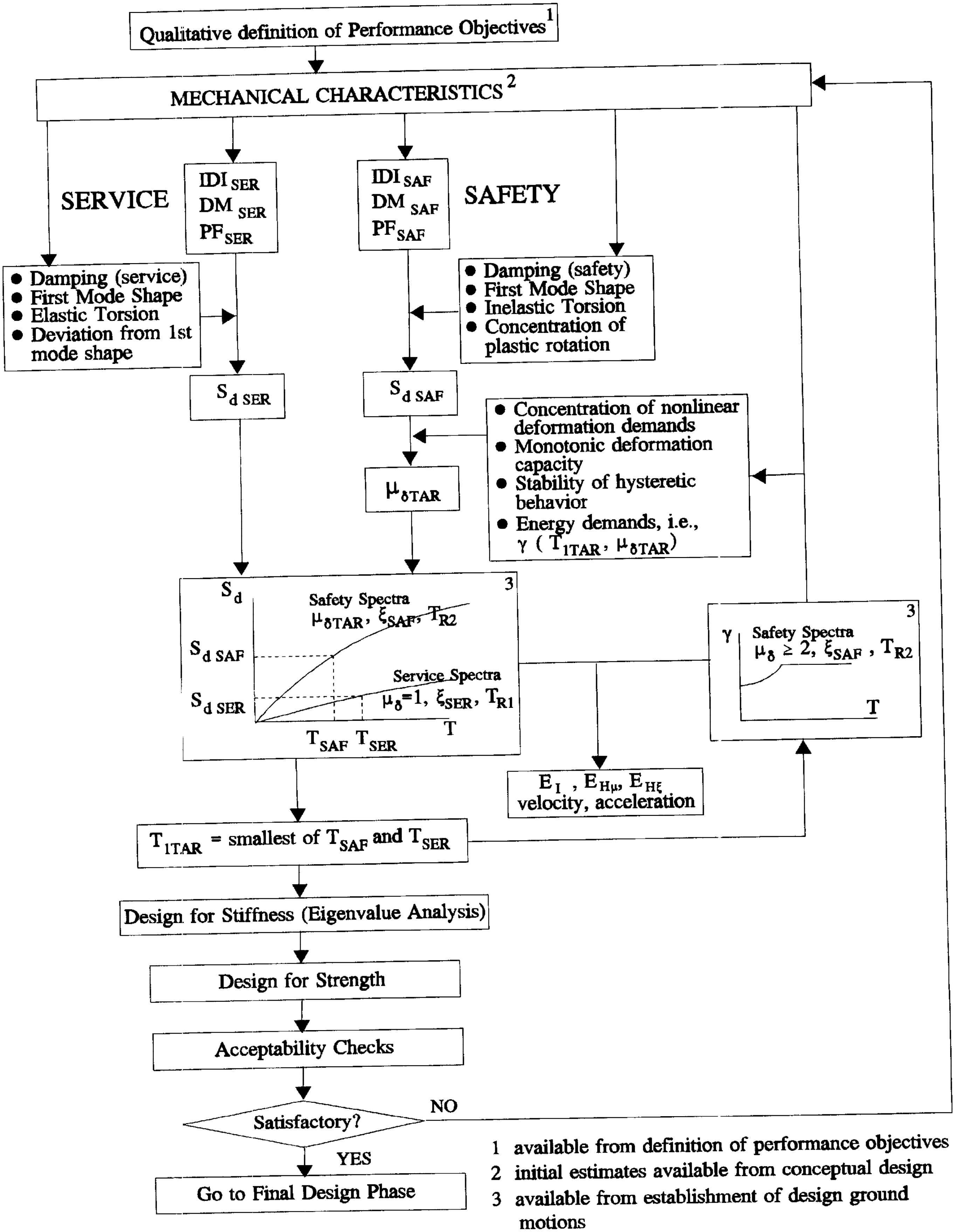

Figure 2 Flow-chart of Preliminary Analysis and Design

service displacement spectra is computed using a μ_δ of 1 and according to the return period associated with the design EQGM for service (T_{R1}). As indicated in Fig. 2, the values of T_1 obtained for service and safety are denoted as T_{SER} and T_{SAF}, respectively, and once they have been established, T_{1TAR} is established in turn as the smaller of the two. Before proceeding to the next step, it is necessary to check if the initially selected value of γ is consistent with the obtained value of T_{1TAR}. If not, the value of γ should be actualized and a new iteration attempted.

In the example shown in Fig. 2, the value of T_{1TAR} is established according to displacement control requirements; nevertheless, in some cases the critical value of T_{1TAR} may arise from the need to control also the maximum velocity and/or total acceleration demands. In the latter cases, it is necessary to establish, besides the displacement design spectra, the spectra for velocity and/or total acceleration. Similar needs arise in the case that energy demands are relevant for the design of the building (e.g., buildings with passive energy dissipating devices). A simple method to estimate these demands for safety can be established if, as suggested by preliminary statistical analysis of the response of SDOF systems to synthetic and real EQGMs, it is possible to estimate the E_I, $E_{H\xi}$ and velocity demands in the equivalent SDOF from the values of T_{1TAR}, $\gamma(T_{1TAR})$, and $S_{d\ SAF}$. Further research needs to be carried to confirm this observation as well as to find a simple way to estimate the total acceleration in the structure.

As will be discussed in detail later, once T_{1TAR} and $\mu_{\delta TAR}$ have been established, the designer may size the frame members (design for stiffness) and estimate their longitudinal reinforcement (design for strength). It should be noted that by carrying out the proposed preliminary analysis procedure, the designer may obtain a fair idea of which are the limit states that control the design, such that early decisions can be made to optimize the design. For example, if the stiffness required to control damage to the partitions of the building is too high, the designer could decide to isolate the partitions from the structure or use other partitions less sensitive to structural distortions; or to use special devices to reduce the displacements of the structure (e.g., energy-dissipating devices). The explicit relationship between selected performance and design can improve the owner's understanding that the expected level of damage and economic losses after an EQ is directly related to his or her initial investment. This can facilitate the communication between the designer and the owner, and the recognition by the owner that an extra small amount of money invested in the initial construction can result in significant savings during the life of the structure.

Note that recently proposed "displacement-based design methods" can be adapted to satisfy the requirements for performance-based design and considered as a particular case of the comprehensive method presented here, in which $\beta = 0$. In this case, damage is assumed to depend on maximum displacement and not on dissipated energy. However, as was clearly shown by the Loma Prieta and Northridge EQs, a minimum strength (or maximum ductility) is needed for most structures to control damage under moderate EQGMs. This minimum strength depends on the hysteretic behavior and energy dissipation capacity of the structure, represented in the comprehensive method given herein by the parameter β.

(ii) Preliminary Sizing and Detailing. The preliminary sizing and detailing step may be stated as follows:
<u>GIVEN</u>: gravity, wind, snow and other possible loads or excitations; minimum stiffness and strength of an equivalent SDOF system required to satisfy the selected seismic performance; critical load combinations; and mechanical characteristics of the structural and nonstructural materials.
<u>REQUIRED</u>: Preliminary sizing and detailing of both the structural elements [beam and columns sizes and their flexural reinforcement (in the case of moment-resisting space frames)], and the unintentional structural (sometimes called nonstructural) components, which can affect the seismic response of the building.
<u>SOLUTION</u>: Select a first period, T_1, that is less than or equal to T_{1TAR}. Using T_1 and the selected first mode shape, to obtain a preliminary sizing for stiffness. Based on these preliminary member sizes, select a minimum equivalent SDOF strength using the service and safety strength design spectra according to the values of T_1 and $\mu_{\delta TAR}$. Consider MDOF and torsional effects, as well as the expected overstrength, to obtain the seismic design loads for service and safety limit states. Based on the application of linear optimization theory and plastic and capacity design, design beams and columns in each story to minimize the volume of flexural reinforcement (in the case of RC), using practical requirements and service forces and moments as constraints so that the preliminary design simultaneously considers the demands for serviceability and safety.

Note that estimation of design forces, overstrength, and elastic moments to be used as constraints in the optimization design are in fact part of the preliminary analysis rather than of the design step. However, since they are intermediate steps between preliminary sizing for stiffness and preliminary sizing and detailing for strength, they are included in the preliminary design of sizes and reinforcement for the sake of simplicity in the discussion.

(iii) Acceptability Checks. An acceptability check is performed to verify that the selected performance objectives are met. The structural response as measured by certain quantifiable parameters must be consistent with the performance objectives and associated acceptability criteria. The acceptability criteria consist of limiting values in structural response parameters, associated with selected performance levels or damage levels for specified levels of EQGM. In a particular building, the design of specific components may be controlled by the same or different response parameters for the same or different performance objectives. Typical response parameters may include: stress ratios, deformation and interstory drift ratios, structural accelerations, ductility demand ratios, damage index, and energy dissipation demand vs. capacity.

Typical limiting values for these response parameters must be established for each performance level through research, including laboratory testing of specific components and calibrating the limiting values by analyzing buildings whose EQGMs and responses, and therefore damages, have been measured (recorded) in past EQs. Then, in a specific design, the appropriate parameters must be checked at the governing performance levels. Typically, the design should at least be checked at the fully operational level and at the life-safety level, after both the preliminary and the final design. In many cases, nonstructural components can be pre-qualified by being prescriptively designed to meet the target parameters of the structural design. Acceptability checks will involve both elastic and inelastic analysis methods. Elastic analysis procedures for checking stress ratios and drift are currently well known and widely used. Several simplified nonlinear inelastic procedures, such as those based on the use of pushover analysis, are also proposed.

CONCLUSIONS

The general comprehensive performance-based EQ-RD approach has been applied successfully to the design of a RC building similar to an existing 30-story building (Bertero *et al.*, 1992) and to a ten-story RC building. The main advantage of the proposed comprehensive performance-based general EQ-RD is that, notwithstanding great uncertainties in the numerical quantification of some of the concepts involved, this quantification can be improved without changing the format of the codified methodology as new and more reliable data are acquired. The relationships between E_I and other relevant seismic demands are usually stable and can be expressed in a simple manner. The use of these relationships simultaneously with performance indices or functions makes it possible to establish a rational and simple EQ-RD procedure that accounts for performance considerations. One such method, which conciliates the analysis and design phases of the overall EQ-RD procedure, is based on energy concepts. This method provides a simple way to estimate the relevant seismic demands in a building while allowing the designer to be an active part of the overall EQ-RD procedure. Nevertheless, at this stage there is still a considerable amount of experimental, field and analytical research that needs to be carried out to create a solid basis on which the proposed method can be properly implemented in practice.

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