Requirements for performance-based design of buildings

J.M. Bracci, M.B.D. Hueste, and J.M. Roesset
Department of Civil Engineering
Texas A&M University, USA

Abstract

A relatively new and significant trend in earthquake engineering is design based on performance criteria. This approach has the potential to be a significant improvement over current practice. In this paper, we look at the evolution of performance-based seismic design, the basic requirements to satisfactorily carry it out, present procedures and their limitations, and the research and development efforts needed to make it a reality.

1 Introduction

The dimensioning of members of a structure has traditionally been based on the consideration of forces (or stresses) rather than deformations. This was particularly emphasized with the elastic or working stress design approach that was in force for many years. The basic assumption in this approach was that the structure would remain elastic under service loads, which was ensured by imposing a factor of safety on the allowable stresses. The main drawback of this approach was that it did not provide a clear measure of the actual degree of safety of the structure. It was even less meaningful when dealing with extreme loads, when inelastic behavior was accepted and thus the forces would equal the maximum capacities.

For many years now, the trend has been to design structures based on a failure or ultimate limit state, and accounting for the variability in the magnitude of the loads and in the material properties through appropriate scaling factors. This approach is the basis of Load and Resistance Factor Design (LRFD). In this case, one must account for ultimate conditions that may be dictated by
deformation limits, although the code format is still in terms of forces and stresses. Switching from the concepts of stress and strength to that of deformation as a design measure is particularly important for seismic design. In this case, the external excitation (the earthquake) imposes motions to the structure, and the forces are the result of these motions. The damage to a structure during an earthquake is thus related primarily to the induced deformations (both maximum values and number of cycles).

A new and significant trend over the last 15 or 20 years is design based on a performance criteria rather than simply a desired factor of safety with respect to yielding based on load and resistance factors. Because of the uncertainties involved in predicting extreme loads, such an approach must incorporate a probabilistic formulation. A proper set of performance-based criteria would also imply the ability to predict the behavior of the structure not only in the linear elastic range but also as yielding and inelastic action begins to take place. In addition, the ability to identify a limiting serviceability condition or outright collapse is desired. Performance-based design is clearly an improvement over elastic design, which provides information on the level of loads for which yielding would start, or a limit design that would indicate the level of loads associated with failure. However, it also requires more rigorous and sophisticated models and analysis procedures. Present methods for nonlinear dynamic analysis of structures can reasonably predict elastic behavior or failure, but they are not very good at estimating damage.

In this paper, we look at the evolution of performance-based seismic design, the basic requirements to satisfactorily carry it out, present procedures and their limitations, and the research and development efforts needed to make it a reality.

2 Definition of performance-based earthquake engineering

Most building codes in the United States are now based on Load and Resistance Factor Design (LRFD), or ultimate strength design, combined with deterministic serviceability checks. The objective of such an approach is to design building systems to: (1) have an acceptably low probability of failure under “ultimate” (i.e., factored) loading and therefore ensure life safety under these and smaller loads, and (2) satisfy (with an unknown level of confidence) serviceability requirements, such as member deformations and cracking, during unfactored loads and therefore remaining operational under these service-type conditions.

In earthquake engineering, the conventional strength approach has been to design structural systems to resist lateral forces and limit inter-story deformations that might be expected during an earthquake with a return period of about 475 years (defined here as a design basis event [DBE]). This implies that building systems would have a 10% probability of experiencing a large earthquake over a 50-year time span. The intended performance objective was to protect the life safety of the building occupants for the DBE. While these buildings may have sufficient resistance to keep occupants safe under this seismic event level, they may not fare as well as desired under smaller, more frequent seismic activity, or they may not be able to withstand collapse under
more intense seismic activity. In the most current model building code in the United States, IBC [1] has incorporated the requirements from FEMA 302 [2] in which the design basis earthquake is determined from 66% of the response spectral accelerations from a maximum considered earthquake (MCE), an earthquake having about a 2% probability of exceedence in 50 years. The reasoning behind such an approach was to ensure more reliable designs and to eliminate the inherent system overstrength observed in California buildings during past earthquakes. The 66% factor (to account for system overstrength) was established by noting that buildings designed according to the conventional 10% probability of exceedence in the 50 year earthquake were actually capable of resisting 50% larger accelerations. In California, the resulting seismic design forces are similar to past practice because the 10% and 2% probabilities of exceedence earthquakes differ by about 50%. However in other areas, like mid-America, seismic design forces could now be much higher, depending on the geographic location. MCEs in mid-America are generally much larger than earthquakes with a 10% probability of exceedence in 50 years. The latest maps thus show response spectral accelerations for the 2% probability of exceedence seismic intensities.

In the conventional strength design approach, buildings were expected to perform inelastically during the DBE. Some to all of the structural members, which have special detailing to ensure ductility, will have force demands that reach their ultimate strength. Although peak strength can be reached, members may still possess significant deformation capability (ductility). This deformation capability is accounted for by reducing the seismic design force demands depending on the allowable level of ductility (response modification coefficient, R, in IBC [1]). The structure is then designed for strength using these seismic forces and an elastic analysis. The calculated elastic displacements are then magnified by a deflection amplification factor (C\textsubscript{d} in IBC [1]) to account for inelastic behavior. These inelastic lateral displacements are limited to a percentage of the inter-story height (1.0% - 2.5% for frame buildings in IBC [1]). The force reduction factors and displacement amplification factors are primarily based on studies using elasto-plastic single degree of freedom systems and the reference ductility represents therefore a vaguely defined system ductility, which is not easy to compute and has little meaning for a complete building. Krawinkler [3] asserts that present code design procedures are insufficient in that they neither provide the designer with a logical explanation for decision-making rules (i.e., they do not equip the designer to control structural performance through design decisions) nor give the owner a means of weighing costs and benefits of various structural designs. A clear improvement to this situation could be provided by the new performance-based seismic design approach.

The idea of basing structural design on expected damage and its economic consequences was proposed in the early 1970’s in a series of studies conducted at MIT under R.V. Whitman and C.A. Cornell, defining a sequence of five damage states for buildings and establishing probability matrices and reward (or cost) matrices associated with each one of them. The objective of these studies
was to establish a more rational design methodology rather than attempting to define accurately the damage states and the numerical values characterizing them. However, it was not until several years later that these concepts started to gain recognition, still in a simplified form.

Performance-based engineering refers to a design methodology in which various levels of structural performance and probabilistic loading are considered. The desired response is specified in terms of maximum acceptable probabilities of exceeding different damage states under each load type and intensity considered. The engineer's task is to make design choices that result in probabilities of exceedence less than or equal to the desired maximum values. Although performance-based design is applicable to the design of structures for any load type, the current emphases are for fire, wind, and earthquake engineering. When applied to design for seismic loads, the approach is better known as performance-based earthquake engineering (PBEE).

In PBEE, as in the MIT study, various limit states or performance levels must be defined and paired with varying levels of earthquake intensity. For example, three suggested levels of earthquake intensity are those having a 50%, 10%, and 2% probability of exceedence in 50 years (i.e., return periods of 73, 475, and 2475 years). These earthquake intensities have been termed standard frequent, design basis, and maximum considered events. Three limit states generally considered are immediate occupancy, life safety, and collapse prevention. In FEMA 273 [4], immediate occupancy is a situation in which the building is safe to occupy immediately after an earthquake and any repairs are minor. Life safety is described as a condition in which the structure remains stable and has significant reserve capacity. Collapse prevention is considered to be a state in which the building remains standing, but on the verge of collapse. These limit state definitions are obviously very subjective. Inherent in the idea of limit states is the supposition that performance can be measured and either accepted or rejected on the basis of some analytically predicted response values. FEMA 273 [4] mainly offers acceptance criteria for individual components, but it also gives inter-story drift limits as "qualitative descriptions of approximate behavior" for structural systems that meet the various limit states. For concrete frames, for example, these drift limits are 1%, 2%, and 4% for the immediate occupancy, life safety, and collapse prevention limit states, respectively. FEMA 273 [4] suggests the Basic Safety Objective (BSO) as a "desirable goal" when selecting a performance objective. The BSO is achieved by satisfying the acceptance criteria for two performance levels using different earthquake intensities: (1) the life safety performance level for the 10% probability of exceedance in 50 years event, and (2) the collapse prevention performance level for the 2% probability of exceedance in 50 years event.

Adoption of the performance-based design approach requires the selection of appropriate variables, whose values can be obtained from the dynamic analyses, defining damage states or performance levels. The application of the approach requires: (1) selection of ensembles of earthquake motions representing various levels of exceedence probability; (2) analyses of the structure subjected to these different motions to evaluate the degree of damage (or performance level)
associated with each one of them; and (3) integration of the analysis results within a probabilistic framework to obtain the final probabilities of reaching the various performance levels for the different seismic intensities.

3 Performance levels (limit states)

Structural response requirements to ensure various performance limit states must be identified. Immediate occupancy, life safety, and collapse prevention are some of the limit states that are now being considered. These seem to be logical choices. One must, however, identify the structural response variables that directly correlate with the various limit states during earthquake motions and the numerical values defining the transition from one state to another. Such response variables have been historically deformation-based, such as inter-story drifts, various types of ductilities (global, inter-story, or local member ductilities), or plastic rotations. The limitations for some of these measures, which are a function of the maximum values attained during the vibration and do not account for the number of cycles, has been repeatedly pointed out with little success. New approaches based on energy (or fatigue) may be more appropriate to realistically characterize damage or performance levels in seismic design. It is also important to account for the behavior of non-structural elements, which are rarely included in the structural models, in the evaluation of damage.

4 Seismic excitation

The first step of the design process is the definition of the seismic excitation according to probabilistic and statistical measures. Depending on the type of structural analysis to be performed, earthquake ground motions and/or response spectra for varying levels of occurrence probability and earthquake intensity need to be identified for the specific site conditions at the selected geographic location. These intensities should at least include earthquakes with a frequent return period (50% probability of exceedence in 50 years), an infrequent return period (10% probability of exceedence in 50 years), and a rare return period (2% probability of exceedence in 50 years).

To perform the required studies, it is not sufficient to define these earthquake motions by a single variable, such as the effective ground acceleration, or, more importantly, by a single record. To account for the possible variations in frequency content of the earthquakes and in the phases of the different seismic waves arriving at the site, multiple time history excitations must be defined and utilized within a probabilistic context. FEMA 273 [4] requires time history analyses to be performed using a minimum of three scaled ground motion records, each from a different recorded event. Synthetic ground motions are allowed when appropriate records are not available. When three records are used, the resulting maximum response values are used to compare to the
acceptance criteria. Average values of the response parameters may be used when at least seven ground motion records are considered.

5 Demand requirements

The next step is to develop an appropriate numerical model for the system. Such models must be able to adequately reproduce the behavior of the structural system under the selected earthquake ground motion time histories (or earthquake response spectra) in terms of strength and stiffness both in the elastic and inelastic ranges. In addition, the model must be capable of capturing all relevant second order effects. In past design practice, only elastic modeling was required to determine seismic design forces. Inelastic displacements were then determined from the calculated elastic displacements by multiplying by an amplification factor. This approach is not able by itself to produce reliable measures of damage. Inelastic modeling of structural elements requires significantly more sophisticated analyses. In addition to elastic stiffness properties, accurate modeling of strength (yield and post-yield) and hysteretic properties of the elements are crucial.

Most typical buildings are structured on the basis of frames, shear walls, and special, box-type elements, such as enclosures of stairwells or elevator shafts, to resist and transmit vertical and horizontal loads. These elements are tied together by beams and slabs which are supposed to act as rigid diaphragms in their planes. Frames and shear walls are assumed to have stiffness in their own plane, but none in the out of plane direction. This type of building is often modeled as a series of plane elements, whether coupled in some way or acting independently of one another. Models of buildings as fully three-dimensional structures are rarely used, although they would be necessary for some structural types. In addition, there may be a large number of participating non-structural elements such as partitions, false ceilings, staircases, etc. These elements may have little strength, but they will stiffen the structure, contribute to the damage of the main structural elements, and represent an important economic loss on their own when damaged. If it is desired to predict not only failure, but also varying degrees of damage, the selection of an adequate model that will be able to reproduce all significant features and effects is essential.

Once the structural model has been selected, it is necessary to decide on the type of analysis to be performed: static linear or nonlinear analysis; dynamic linear analysis; or full nonlinear dynamic analysis. Most building codes still allow the seismic analysis of typical buildings using equivalent static loads with linear elastic behavior and simple concepts of dynamics as applied to the elastic and inelastic response of single degree of freedom systems. Although these may be acceptable procedures to obtain safe structures in a general sense, the accuracy with which these analyses can predict expected damage and thus be used for a performance-based design is highly questionable. Unfortunately, much research work is still being wasted to derive inelastic response spectra (or variations of these) to be applied within this design approach. Nonlinear static analyses, often referred to as the pushover method, can provide a better
indication of the degree of damage associated with a given level of lateral load. The extrapolation of these measures of damage to the dynamic case (which is the actual case of an earthquake) requires some approximations, assuming in general that the relative importance of the different modes of vibration remain unchanged once yielding takes place and that the distribution of equivalent lateral loads is thus the same in the elastic and inelastic ranges. This is not true and, as a result, the damage predictions will not be very accurate. An alternative approach to the static pushover method is an adaptive spectra-based pushover procedure recently proposed by Gupta and Kunnath [5]. This method uses site-specific spectra to define loading characteristics and the lateral loading pattern over the structure height is adapted during the analysis based on the instantaneous dynamic properties of the structural model. Linear dynamic analyses will better reproduce the possible effects of higher modes in the elastic response and, from that point of view, are perhaps an improvement over linear static analyses. But extrapolation of their results to the inelastic range by applying reduction and magnification factors will again provide inaccurate measures of damage. The most logical approach to obtain reliable measures of performance once yielding takes place and progresses through the structure, until the final collapse condition is reached, is to perform nonlinear dynamic analyses. These analyses must be carried out, however, with realistic structural models. The use of close-coupled, shear beam type models as often used in research studies is not appropriate to model the inelastic behavior of real frames as shown by a large number of studies.

The first model for nonlinear dynamic analysis of frames was proposed by Clough and Benuska [6] as early as 1966. It was based on the assumption that each member could be considered by the superposition of an elastic component with a very small stiffness and an elasto-plastic component. The latter develops hinges at its ends when the moment reaches a limiting value (plastic moment capacity, which may be a function of the axial load through an interaction formula). This model has often been referred to as a dual component point hinge or localized plasticity model. The moment-rotation (instead of moment-curvature) relationship is essentially elasto-plastic with a small hardening (second slope) due to the elastic component. A variation of this model proposed by Giberson [7] is a single component model, where rotational springs with arbitrary nonlinear, inelastic moment rotation relationships, are inserted at the ends of the members that remain otherwise elastic over their entire length. The point hinge model has been implemented in the computer program DRAIN-2D developed at the University of California at Berkeley by Kanaan and Powell [8]. This program has been extensively used and has been the subject of many modifications and enhancements at different universities, leading to a myriad of different versions, some proprietary. DRAIN-2DX is the name of the extended version at Berkeley [9]. In addition, a program is currently being developed by the Pacific Earthquake Engineering Research (PEER) Center for use in nonlinear analysis and performance-based design.

An improvement over the classical point hinge model is provided by the program IDARC [10], developed at the National Center for Earthquake
Engineering Research (NCEER, now named MCEER) of the State University of New York at Buffalo. This program approximately accounts for the spreading of yielding at the ends of a member and models some elements using the fiber idealization. It also incorporates more sophisticated moment-rotation relationships for reinforced concrete sections to account for first cracking, yielding of the steel reinforcement, cyclic degradation of stiffness, etc. The parameters defining these relationships can be selected by the user (normally for posteriori studies trying to improve the match with experimental data) or standard values can be adopted. Additional enhancements to the IDARC program have been made by Kunnath [11] and this version is called IDASS.

A more sophisticated idealization is provided by fiber or filament models. In this case, the time history of stresses and strains is monitored at fibers across selected sections of each member and the tangent stiffness matrix of each element is assembled at each time step from the sectional properties. This model was used initially by LaTona [12], Adams [13], and Mark [14], and more recently by Rubiano [15]. It accounts not only for the spreading of yielding along any part of the members, but also for the coupling between axial and flexural forces and deformations in the inelastic range due to the shifting of the neutral axis. This can have an important effect on the behavior of the columns (more significant in most cases than the P-delta effects). The original model implemented in the 1970’s did not account for shear deformations and did not properly reproduce the behavior of very deep members. The latest version by Rubiano introduced an approximate consideration of inelastic shear deformations, through a truss analogy model. One-dimensional finite element models could be essentially similar to the fiber model if the stresses and strains were monitored at the Gaussian quadrature points. But, this is not a very convenient selection when the yielding is concentrated mostly at the ends.

In two-dimensional finite element models, each member would be modeled using plane stress elements with appropriate nonlinear constitutive equations for the material. These idealizations should be able to reproduce the combined axial force-bending moment-shear coupling, but they have only been used to date for detailed analyses of single members, simple walls with openings, or simple combinations of columns, beams, and walls.

Although the fiber model has been used in a limited number of cases to study the three-dimensional response of reinforced concrete piers, all the above models have been used primarily to analyze plane frames or combinations of plane frames and shear walls. None of them allow in their standard version a way to account for infill walls or partitions. These have been sometimes modeled as equivalent braces, but this model is unable to accurately simulate their cracking and eventual failure or the effect of this on the behavior of the frame.

There are a number of important parameters that can affect the accuracy of the nonlinear dynamic analyses, but receive relatively little attention. Such for instance are the selection of the time step of integration, the use of mass proportional, stiffness proportional, or Rayleigh-type damping to simulate energy losses in the elastic range (to be added to the dissipation of energy
through hysteresis loops under inelastic behavior), the actual values of this additional damping (often excessive), the use of lumped or consistent mass matrices, etc. Finally, most computer programs for nonlinear dynamic analysis consider only a horizontal base excitation, ignoring the vertical component of motion, which can contribute to the amount of damage, if not significantly to the collapse. They are also unable to realistically model soil structure interaction effects (effects of the flexibility of the foundation), which can be important in some cases.

All these limitations should be recognized and should cast some doubt on our present ability to accurately predict the level of damage that a real complete building will experience under a given earthquake. Ignoring them stubbornly may help to sell the concept of performance-based design, but it will not help to achieve real improvement in structural design.

6 Performance evaluation

Once the structural demands under the different earthquake motions have been determined and correlated to the limit state capacities identified, structural performance under various earthquake scenarios must be evaluated in probabilistic terms. For new design, reliability factors to ensure a certain level of confidence must be incorporated. As important, is the issue related to the level of confidence required for earthquake design.

7 Concluding remarks

The concept of performance-based seismic design is an attractive one that represents a significant improvement over the more traditional working stress or load and resistance factor design. Replacing forces by deformations and damage as design goals is a logical step. Yet one must question whether the present state of the art in seismic analysis allows us to realistically carry out this type of design. Changing the name of the design approach, suggesting three performance levels, and specifying three different earthquake intensities are in themselves steps in the right direction. But they will not be very meaningful unless we can define these performance levels in terms of variables that can be reliably predicted through structural analyses, we select representative ensembles of earthquake motions that account for the possible variations in frequency content and phases of the waves, we conduct more rigorous dynamic analyses with better models, and we integrate the results of all these analyses into a total probabilistic model.

References


