CHAPTER 1

Introduction

1.1 General overview

Every now and then a big earthquake shatters a part of our planet and sends seismic waves to the built-environments across the world. The level of seismic awareness in the society surges and a myriad of questions are raised as to how well the structures dealt with the earthquake and what can be done to improve their future performance.

The history of formation of seismic building codes is marked by these big earthquakes; often serving as landmarks for the evolution of seismic codes. As time passes, the design criteria become more and more articulate and ambitious, just as the design objectives evolve from collapse prevention only to also ensuring the possibility of immediate occupancy after an earthquake. Loma Prieta Earthquake 1989, Northridge Earthquake 1994, and Kobe Earthquake 1995 are distinct landmarks in this sense, marking the point where the term *Performance-Based Design* (PBD) found its way into seismic guidelines such as, VISION 2000, ATC-40, FEMA 273, FEMA 350-352. The definition of the desirable performance for a structure was more-or-less qualitative and based on the engineering judgment and intuition before then, whereas, the PBD objectives are quantified based on life-cycle cost considerations. Life cycle considerations encompass various parameters that affect the structural performance, such as, structural damage, non-structural content damage, and human casualties. Hence, quantified performance objectives are among the fundamental qualities distinguishing PBD from traditional earthquake engineering.

Apart from being rare events and having large consequences, earthquakes events also have large uncertainty associated with them[1]. This makes the quantification of performance objectives even more challenging and complicated. Application of the theory of probability in modeling the uncertainty in earthquake hazard or *Probabilistic Performance-Based Design* has been the subject of ongoing research in the past years (see Bazzurro and Cornell, 1994, Wen 1995, Beck 1999). Probabilistic PBD is hardly a new concept; it has been first implemented in specialized design guidelines such as the nuclear power plant procedures (see, e.g., Kennedy and Short, 1994, and,

[1] "Earthquakes are low-probability, large-consequence, and large-uncertainty hazards." Quoting Y.K. Wen, from the paper: "Reliability and Performance-Based Design", Structural Safety 23 (2001).

DOE, 1994). The performance objectives in the context of probabilistic PBD can be defined as the probability of exceeding a performance level (see Krawinkler, 1996, 1999).

Various factors such as, structural damage, non-structural content damage, human casualties and life cycle costs, are instrumental in measuring the structural performance under seismic excitations. Nonlinear time-history analyses of structural response can be implemented in order to make probabilistic estimations of the structural damage and to facilitate estimation of non-structural damage and life cycle costs. On the ground motion side, there has always been the need for variables that are going define and quantify the intensity level of the ground motion records as they are applied to the structure. This is going to affect the selection of the ground motion records to be used in the time history analyses and also the assessment of structural performance.

This dissertation resembles a journey through probabilistic performance-based design equipped with common nonlinear dynamic analysis tools. The itinerary for this journey includes the development of a theoretical foundation for assessment of the performance objectives using the theory of probability, shaping the results into design formats that can be (indeed are) implemented by seismic guidelines, using the nonlinear dynamic procedures within the developed design format(s), and observing the validity of such formats for the case where the dynamic analysis tools indicate that global instability is imminent. The final stop in the journey elaborates on the selection of the ground motion records that are going to be applied in the non-linear dynamic analyses. This is the place for testing the validity of some of the major assumptions that were implicitly or explicitly made in order to make the performance assessments stated in the thesis.

1.2 This thesis in the context of probabilistic PBD

Most of the current seismic design procedures (FEMA 356, ATC-40) are based on defining "design earthquakes" with prescribed probabilities of exceedance in a given time period. One of the main attributes of these procedures is that the probabilistic nature of the seismic excitations are only implicitly "accounted for" in the definition of the design response spectra (see Bazzurro et. al, 1998), and not in the statement of the performance objectives. In the recent years, specifically after the Northridge Earthquake, considerable research effort has been focused on defining probabilistic performance objectives (e.g., Collins et. al., 1995) that balance desirable structural performance and life cycle costs (e.g., Wen, 2001). A performance objective can be expressed in terms of the (mean) annual frequency of exceeding a specified limit state (limit state

frequency in short), λ_{LS}[2]. The limit state frequency can calculated by summing up the contributions from all the events that can be generated from the faults that surround the site:

$$\lambda_{LS} = \sum_i v_i \cdot P_{LS|event_i} \qquad (1-1)$$

where $P_{LS|event_i}$ is the probability of exceeding the limit state given an event on a specific fault, *i*, and v_i is the mean annual rate[3] of activity of events generated by fault, *i*. Estimation of the term, $P_{LS|event_i}$ directly would entail performing several (e.g., thousands of) non-linear dynamic analyses in order to represent all of the events and ground motions that can be generated by a given fault. Y.K. Wen and his co-workers (Collins et. al., 1996, Han and Wen 1997) have followed this approach by employing simple "equivalent" non-linear systems in order to reduce the computational effort.

The limit state frequency could also be evaluated, in a potentially more efficient manner, by applying the total probability theorem (see Appendix B and Benjamin and Cornell, 1970) and regression analysis in order to predict the variation of the conditional probability of exceeding the limit state given the event and motion parameters such as moment magnitude, *M*, and source-to-site distance, *R*:

$$\lambda_{LS} = \sum_i v_i \cdot \iint P_{LS|M,R}(m,r) \cdot f_{M,R}(m,r) \cdot dr \cdot dm \qquad (1-2)$$

where $P_{LS|M,R}(x)$ is the conditional probability of exceeding the limit state for a given magnitude and distance, and $f_{M,R}(m,r)$ is the joint probability density function (PDF) for magnitude and distance.

The evaluation of the limit state probability can be greatly simplified by decoupling the structural analysis and the ground motion predictions by means of introducing a site and structure-specific interface variable referred to as the ground motion intensity measure (IM):

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[2] In this thesis, we have used the notation $H(.)$ and $\lambda(.)$ interchangeably to refer to the mean annual frequency of exceedance.

[3] Note the way we have used the terms "probability" and "frequency" in Equation 1-1, where the product of the "probability" of exceeding the limit state times a "mean annual rate" results in "mean annual frequency" of exceedance. We may sometimes alternate between the two terms, "probability" and "frequency", in this thesis, since, for the type of rare events we are studying, the corresponding numerical values are very close (and small).

$$\lambda_{LS} = \int P_{LS|IM}(x) \cdot f_{IM}(x) \cdot dx = \int P_{LS|x} \cdot \left| d\lambda_{IM}(x) \right| \qquad (1\text{-}3)$$

where $P_{LS|IM}(x)$ is the conditional probability of exceeding the limit state for a given ground motion intensity level, x, $f_{IM}(x)$ is the PDF for the intensity measure, and $\lambda_{IM}(x)$ is the mean annual frequency of exceeding (also known as the hazard function) a given intensity level x. The *practical implementation* of Equation 1-3 is usually (implicitly or explicitly) based on the assumption that the probability of exceeding the limit state for a given ground motion intensity level is conditionally independent of ground motion characteristics such as magnitude and distance (Shome et al., 1998). The term $P_{LS|IM}(x)$ can be estimated by employing a very limited number of non-linear structural analyses (see Shome et. al. 1998), whereas the $\lambda_{IM}(x)$ term can be calculated by performing conventional probabilistic seismic hazard analysis (PSHA),

$$\lambda_{IM}(x) = \sum_i v_i \cdot \iint G_{IM|M,R}(x \mid m, r) \cdot f_{M,R}(m, r) \cdot dr \cdot dm \qquad (1-4)$$

(where $G_{IM|M,R}(.)$ is the conditional complementary cumulative distribution function for the intensity measure given magnitude and distance). Such hazard information is routinely generated by earth science community (e.g., the hazard maps available on USGS website). This approach has been developed by Cornell and co-workers in the recent years (e.g., Bazzurro and Cornell, 1994, Shome et. al. 1998, Luco and Cornell 2003, Vamvatsikos and Cornell 2002, and, Jalayer and Cornell 2003).

In the present work, an analytic framework will be derived for the evaluation of limit state probability based on this last approach. An analytic closed-form solution for the limit state probability in Equation 1-3 will be derived based on a set of simplifying assumptions. This analytic expression forms a technical basis for developing design and assessment formats suitable for implementation in seismic guidelines. It will be shown how non-linear dynamic procedures can be implemented in order to provide parameter estimates for this analytic closed-form solution. The performance objective(s) underlying the limit state probability calculations in this work is purely based on structural response parameters and does not take into account life cycle cost considerations. The methodology presented in this work could also be applied to alternative interpretations of the limit state such as, for example, the life cycle cost of exceeding a specified level, provided that appropriate procedures for defining the probability of the limit state given $IM = x$ (as a function of x) can be established.

1.3 The organization of this thesis

This thesis is organized in the same order it has been formed. Chapter 2 with the deserving title of "Foundation Development" is the central piece that connects the other chapters. It is intended to serve as a step-by-step derivation of a closed-form expression for the annual probability of exceeding structural performance level(s) based on some simplifying assumptions. The formulation of every piece is explained in detail in order to provide an insight into probabilistic assessments for the interested reader, including those with limited experience with such probabilistic derivations. The derivations start with hazard estimations for the intensity measure of choice, which is the first-mode spectral acceleration. The next level of complexity is to derive the hazard expressions for structural displacement response based on the available expression for spectral acceleration hazard and expressions for the displacement probability distribution given the intensity level. In the final step, structural limit state capacity information is added in order to derive the expression for annual probability of exceeding a structural limit state, which also serves as the primary product of this chapter. Another layer of complexity is added by considering the uncertainty due to limited knowledge (epistemic uncertainty) in the formulation of the limit state frequency.

Chapter 3 or "Format Development" starts from where Chapter 2 ends. This chapter discusses several of the many alternative design formats that can stem from the expression for probability of exceeding a limit state. Demand and Capacity Factored Design (DCFD) is a closed-form design and assessment format that directly results form the original formulation for probability of exceeding a limit state derived in the previous chapter. This format has been implemented in FEMA 350, 351, and, 352 and in an ISO offshore structure guideline (Banon et. al., 2001). The Fragility-Hazard design format is another way of transforming the closed-from expression for probability of failure from Chapter 2 into a potentially graphical design format. A variation of this format has been implemented in Department of Energy Guidelines (DOE 1020) for nuclear power plants (PRA 19383). Similar to the previous chapter another level of complexity is added by including the epistemic uncertainty in the formulations. The consideration of this type of uncertainty may manifest itself in the form of a confidence statement about the performance objective being met (which may in effect modify the demand and capacity factors in DCFD format), or in the <u>mean</u> estimate for the limit state frequency (as in DOE 1020, 1994).

Chapter 4, "Alternative Non-Linear Demand Estimation Methods for Probability-Based Seismic Assessment" was formed around the question of how the design formats developed are going to

be practically implemented, as the previous two chapters introduced many equations and parameters that need to be estimated in practice. It began as a brief two or three page outline of alternative ways to estimate the parameters in the DCFD formulation developed in Chapter 3. It became important however, to explore the various ways that non-linear dynamic analyses can be implemented in the design format for varying levels of analysis effort. The division of Chapter 4 into narrow-range and wide-range methods reflects a variety of possibilities in terms of the number of analysis runs performed. The basic idea was to be able to estimate the parameters in the DCFD format locally while performing the fewest number of nonlinear dynamic analyses at a ground motion intensity level that lies in the region of interest, which is defined by the allowable probability of exceeding a limit state. The wide-range methods are discussed in the context of multiple performance levels. Although they require a comparatively large amount of analysis effort, the wide range methods can provide a great deal of information about the structural response to wide ranges of seismic excitation. These methods are also useful for assessing the onset of global dynamic instability, also referred to as global "collapse" in the structure.

One of the underlying assumptions for the derivation of DCFD design format is that the distribution of response for a given level of intensity can be modeled by a lognormal distribution. However, for the displacement-based response in the highly non-linear region of global dynamic instability in the structure this assumption may well be unrealistic. Hence, the question rises as to how effective the DCFD format would be in the region of displacements close to the dynamic instability in the structure. Chapter 5 is written as a sequel to Chapter 4 aiming to explore structural response prediction in this region. It is devoted to the prediction of response when the displacements are so large as cause non-convergence of the structural analysis computer algorithm used and/or not explained properly by a common two-parameter distribution such as the lognormal. In this manner, a modified version of DCFD format is introduced based on a three-parameter distribution proposed by Shome (1999).

The probabilistic assessments coupled with non-linear dynamic time history analyses in the context of DCFD format are typically based (at least implicitly) on the assumption that the structural response given the intensity measure is conditionally independent of the ground motion record characteristics such as magnitude and source-to-site distance. In simple words it is assumed that the intensity measure is adequate for conveying the ground motion record characteristics for the purpose of response predictions. Based on the assumption that this was true for first-mode spectral acceleration and the structure under investigation, we took the liberty, in

Chapters 4 and 5, of choosing more-or-less arbitrary selections of ground motion records, and also of scaling them whenever we needed multi-level assessments. Chapter 6 investigates whether such assumptions are realistic for the two extreme non-linear cases of a 20-story long-period structure and an SDOF system with a very short period. It is observed in both these illustrations that first-mode spectral acceleration does indeed render the displacement response effectively independent of record characteristics such as magnitude, source-to-site distance and the normalized residual of attenuation prediction, ε. Also, in the case where the response *is* dependent on one or more record characteristics, it is demonstrated how the output of non-linear dynamic analyses can be post-processed in order to enhance the resulting demand estimations. This is done by coupling the actual distribution of the ground motion characteristics for a particular level of intensity, also know as the PSHA disaggregation results (e.g., Bazzurro and Cornell, 1998), with the structural response results for that level.

Chapters 4,5 and 6 are (in part) based on the anticipation that assessment (and even design) will be more and more based on systematic non-linear dynamic analysis procedures in the future. Most common use of these procedures today is in the assessment of existing and/or damaged buildings (e.g., FEMA 351, 352 and 356). These chapters show how to organize the ground motion records, intensity levels, number of analyses, and processing of results. One can envision that this could all be one day automated for the engineer once she has modeled the structure.