NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

Development of Reliability-Based Design Criteria for Buildings Under Seismic Load

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NOTICE

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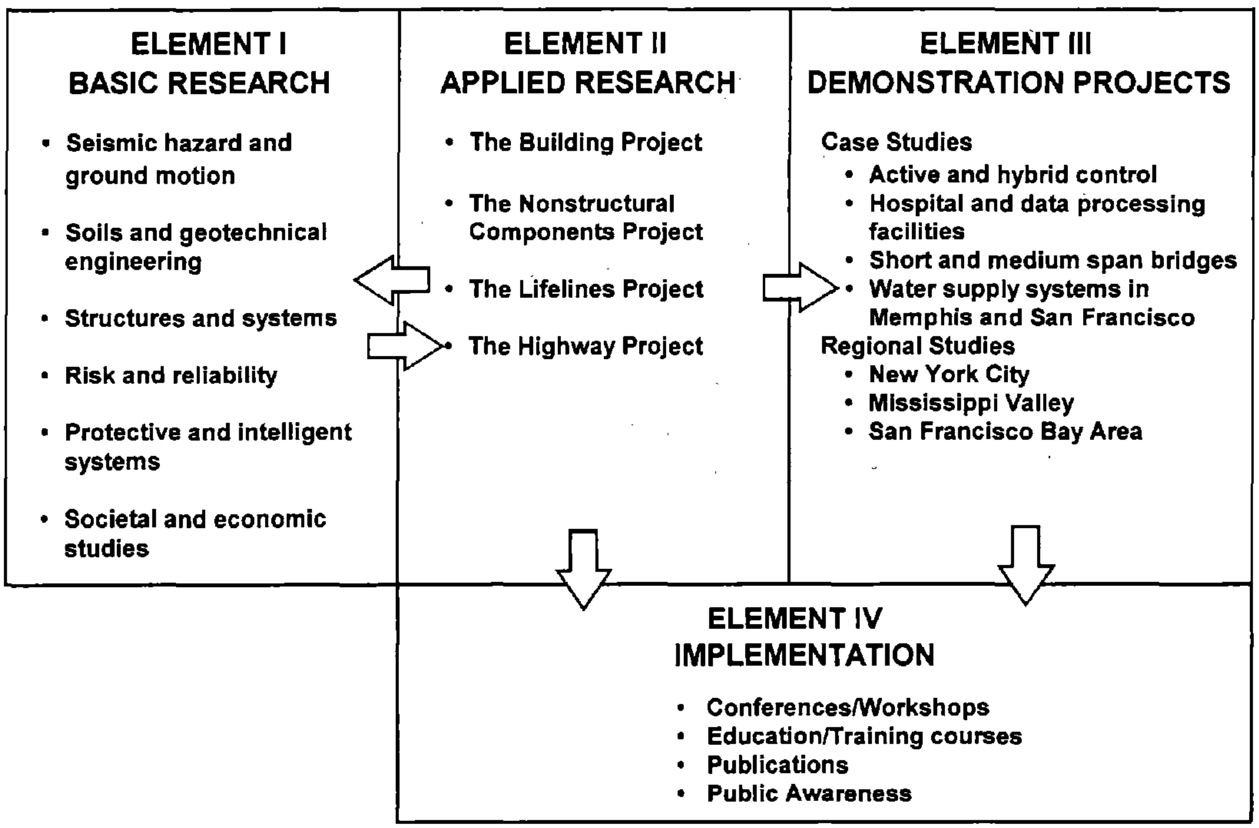
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **risk and reliability program** constitutes one of the important areas of research in the **Building Project**. The program is concerned with reducing the uncertainty in current models which characterize and predict seismically induced ground motion, and resulting structural damage and system unserviceability. The goal of the program is to provide analytical and empirical procedures to bridge the gap between traditional earthquake engineering and socioeconomic considerations for the most cost-effective seismic hazard mitigation. Among others, the following tasks are being carried out:

1. Study seismic damage and develop fragility curves for existing structures.
2. Develop retrofit and strengthening strategies.
3. Develop intelligent structures using high-tech and traditional sensors for on-line and real-time diagnoses of structural integrity under seismic excitation.
4. Improve and promote damage-control design for new structures.
5. Study critical code issues and assist code groups to upgrade seismic design code.
6. Investigate the integrity of nonstructural systems under seismic conditions.

This report is a good contribution to the application of structural safety and reliability theory. A reliability-based design procedure, which considers both serviceability and ultimate-state failures, is proposed so that structures can be designed to meet the prescribed target reliability levels. Such procedures can be incorporated to design provisions.

ABSTRACT

Designs of buildings and structures for seismic loads are traditionally based on experience of the performance of structures in past earthquakes. Although the large uncertainty in the earthquake loadings has long been recognized by engineers, it has not been fully accounted for in the code procedures other than in the selection of a design earthquake. Since the design earthquake is used in conjunction with a series of factors to account for effects of structural period, site soil condition, inelastic behavior, importance of the structures etc., the reliability and safety of the final design remains unknown and undefined. The recent sentiment of the research community and design professionals is that there is a need for development of design procedures based on consideration of the physics of the problem and explicit treatment of the uncertainties. Such procedures may be used as the basis for development of the next generation of buildings codes. In this report, the theory and methodology that can be used to formulate such a design procedure is presented. A brief review is given of the theoretical background of reliability analysis and reliability-based design, followed by an examination of the safety considerations in representative current code procedures as well as the reliability of buildings designed in accordance with such procedures in different countries. Finally, a bi-level, performance-based design procedure is proposed in which desirable reliabilities can be implemented against both unserviceability and ultimate failure.

ACKNOWLEDGEMENTS

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INTRODUCTION

1.1 Contributions of The Reliability Method to Structural Safety

Civil engineers have always been aware of the uncertainties (natural and manmade) that they must negotiate within their practice of planning, design, analysis and construction. In particular, structural engineers have dealt with this problem by making allowances for them through safety factors. While the safety factor approach to structural engineering has worked remarkably well in practice, it is not entirely without problems. One of these problems stems from the fact that the process to specify key design parameters such as design loads and allowable stresses is based primarily on collective professional judgment of a subjective nature. Among other things, this makes it rather difficult to evaluate structural safety in quantitative terms. Obviously, the safety factor itself can be used for comparison purposes. Such a comparison, however, makes sense only in extremely simple situations.

In civil engineering, particularly in structural engineering, probabilistic concepts were first introduced in the 1940's in an attempt to develop a quantitative measure of structural safety. Over the last four decades or so, they gradually evolved into what is currently known as the structural reliability analysis method. More recently, some of the existing design codes were reexamined and modified on the basis of these probabilistic concepts, as exemplified by the introduction of reliability-based load and resistance factor design (LRFD) codes in various countries.

The emphasis of structural reliability analysis has been placed on the estimation of structural safety in terms of the probability that a structure subjected to loads and other adverse environments will perform its specified mission without failure. In the classical approach, this probability is defined as the structural reliability. It is classical in the sense that the reliability is estimated under the following assumptions: All possible failure mechanisms under the projected operational conditions and all the pertinent parameters involved are known and at the same time, the probabilistic characteristics of all these parameters are also known. Indeed, the theory of reliability analysis in this context is often referred to as the full-distribution theory. The full-distribution theory, however, is unrealistic not only because of its requirements for a substantial database, but also because of the enormous numerical chore that could entail. Nevertheless, it permits a sensitivity analysis with respect to the specific probability models assumed for design variables and therefore provides an analytical base for the engineering application of reliability concepts.

At an early stage of the development of reliability analysis methods, the necessity to examine the level of confidence of such a reliability estimate was recognized only implicitly; it was usually implied that the highest level of confidence could be obtained by taking advantage of all available data and by making use of state-of-the-art probabilistic and statistical techniques. More recently, the task of establishing such a confidence level in terms of a confidence interval became more of a routine, however crudely that might have been done. Reinforcing a reliability estimate with a confidence statement represents active recognition of uncertainties other than those arising from randomness. Reliability theories that recognize this are no longer classical. In estimating the reliability and associated confidence level, we must keep in mind that the degree of analytical sophistication should be consistent not only with the quality and quantity of the pertinent information available but also with the current analytical and other capabilities of the profession in the following areas: (a) structural and stress analysis - linear, nonlinear, static and dynamic; (b) failure analysis for various modes of structural unserviceability and collapse; (c) environmental and load analysis; (d) durability analysis considering the effects of in-service inspections and repairs; and (e) quality assurance procedures covering the entire spectrum of planning, design, analysis, construction and maintenance. It is precisely in this context that we often recognize the results of first-order and second-moment analyses as credible as those obtained by applying the full distribution theory.

Reliability analysis methodology has made and can further make genuine contributions toward enhancing the structural safety and integrity of constructed facilities. One might add that these contributions have so far been made primarily through such conferences as ICOSSAR and ICASP. Indeed, through these contributions, we have made it possible to (a) establish the correlation between structural safety and design parameters such as safety factors, stress allowable and inspection periods, (b) achieve balanced designs among structures with differing degrees of importance, (c) allocate the desired reliability performance to each component within an individual structure, (d) identify the additional information needed to upgrade the confidence of reliability estimates, and (e) develop a consistent and systematic procedure in which a safety analysis can be made logically. In accomplishing all these, the sensible and well-disciplined use of subjective engineering judgment in the Bayesian framework is considered beneficial in bringing about the compromise required and even desired for a reasonable blending between analytical rigor and availability of pertinent information.

Recently, reliability analysis has become an integral part of the risk assessment and management procedures for a wide variety of structures including such risk-sensitive structural systems as nuclear power plants. This demonstrates an added dimension of the usefulness of reliability concepts beyond their applicability to traditional engineering structures. Since the perception of risk stems from the recognition of the possible occurrence of undesirable events with grave consequences, a risk assessment and management procedure is usually built around a reliability methodology with one or more analytical component, i.e., consequence analysis, integrated into it. Parenthetically, one might add this is precisely where the cost-effectiveness issue should be addressed. The acceptable level of risk is correlated to acceptance reliability levels of the components of the system for which the risk is to be evaluated and managed. In this sense, the difficulty in arriving at a consensus on the acceptable level of risk, translates into the same difficulty in determining acceptable reliability levels, although the latter can be somewhat lessened by means of calibration at least for traditional civil engineering structures.

Having made these observations, it is appropriate to point out a number of major issues that have not really been resolved in the structural reliability analysis methodology. First, it is by no means easy to obtain a consensus on the target levels of reliability even for traditional civil engineering structures. For example, when we attempt to develop the Load and Resistance Factor Design approach, a question still remains as to how we can specify target reliability levels for various load combinations. Second, the reliability analysis methodology developed so far presumes that we can somehow formulate everything in terms of probability. Obviously, not everything is always probabilistic. In this respect, fuzzy set concepts are advocated by some to provide an alternative interpretation of uncertainty. Third, even if we somehow agree that we can interpret everything as probabilistic, the casual fashion in which the source of the variability is often divided into that arising from "randomness" and that from "uncertainty" could give the false impression that such a division is easy, while it certainly is not. Fourth, we often get carried away in constructing the simplest possible analytical model out of a structure for the sake of wide applicability of reliability analysis methodology. The case in point is severely nonlinear structural behavior that must be dealt with for the analysis of structural integrity against collapse, say, under earthquake acceleration. A reliability analysis using too simplistic models in such a situation will not only produce a grossly wrong answer but also cost us credibility in such a way that even those credible structural reliability analysis results we endeavored to derive on the basis of carefully constructed models will be placed under suspicion. Two more items should be added to this list. Fifth, the confidence interval we evaluate for the reliability estimate is often too wide to be useful. Sixth, so far the reliability methodology has been unable to properly incorporate human factors, managerial as well as technical. This is particularly important in view of such unfortunate events as the Three Mile Island accident, the Chernobyl accident, the Challenger explosion and commercial airplane crashes. At least for these accidents, managerial factors, rather than technical factors, appear to be more crucially responsible. In many cases like these, however, engineers are also guilty at least to the extent that they have not asserted themselves strongly enough to change managerial decisions or improve managerial procedures, on the basis of their technical knowledge.

1.2 Quality Assurance

The issue of quality assurance is also important. It is particularly so to the medical and civil engineering profession, and, to a lesser extent, for that of architects, although the implied commonology may appear farfetched. The medical profession deals primarily with the physical nature of human bodies, whereas the civil engineering profession with mother nature itself. In either case, nature challenges the profession with its unpredictability. Also common to these professions is the fact (well known even before the current craze for often frivolous liability suits lodged against them particularly in the United States) that they are both highly vulnerable to poor judgment, incompetence and mismanagement. The issue of quality assurance has always been at the heart of civil engineering, particularly of structural engineering. However, the recent concern for potential liability problems has made the profession even more acutely aware of the importance of quality assurance. These days, quite extensive efforts are probably needed to assure the delivery of high-quality products. This is particularly so because the profession must currently operate in an environment where excessive competition, tighter fiscal maneuverability, inferior workmanship, and a less productive labor force are likely to prevail. A good example is the performance of steel frame buildings in the Northridge earthquake. Steel moment frame buildings have been considered to be one of safest designs for seismic loads. The discovery of widespread fractures in moment connections of steel frame buildings after the earthquake has sent shock waves throughout the engineering community who have been content with current practice in steel design and construction for seismic loads. It also has the national attention focused on the seriousness of the consequence that poor quality control in design and construction workmanship can cause.

Moreover, the profession at large appears to command less prestige and fewer financial rewards than other professional groups, and as a consequence, suffer from a decline in the quality of the human resources it must depend on. These contemporary non-technical issues certainly influence the quality of the overall performance of the profession, of which the technical quality assurance issue is possibly a small part. Therefore, the profession, and in particular, its leadership are well advised to address themselves to these non-technical issues and map out strategies for improving the environment in which they must survive and prosper.

1.3 Need for a Reliability-Based Earthquake-Resistant Design

Engineers traditionally design based on past experience. In design of buildings and structures against seismic loads, code requirements and provisions have been periodically revised and upgraded based on lessons learned after each disastrous earthquake in the last century. Since earthquake occurrence time, intensity and ground motion are all random in nature, an overriding concern and challenge to the engineer therefore is proper treatment of the uncertainty such that the structural performance will be satisfactory under the loadings during its lifetime. Or more specifically, how to provide service and life safety in the design at a reasonable cost. It requires a careful consideration of the uncertainty in the demand (loadings) on the structural system as well as the capacity (structural resistance) that the system has to satisfy the demand. The uncertainty is used here in a wide sense that it includes both natural randomness (variability) and modeling error. As the seismic loading uncertainty almost always is a major contributor among all the uncertainties, the modeling and analysis of seismic loading and load effects on buildings and structures have received increasing attention and great progress has been made. Probability and random process theory have been used for this purpose and methods have been developed for evaluating the risk of seismic loads and in combination with other time varying loads.

The applications of the results to characterization of seismic loads for design purpose and development of probabilistic design criteria, however, have been slow and limited in scope. The challenges in developing such a procedure include (1) correct probabilistic interpretation of limited data base, (2) modeling of the nonlinear and inelastic dynamic response behavior of the soil-structural system, (3) enforcement of satisfactory performance of the structural system from serviceability under minor to moderate earthquakes to life safety under major earthquakes, and (4) consideration of the effect of other time varying loads such as live, snow and wind loads.

Most current US code procedures define a single "design earthquake" as an earthquake which has a given (e.g. 10%) probability of exceedance in a given period (e.g. 50 years). Corresponding estimates of peak (or effective) ground acceleration or related ground motion parameters are given for the various regions of the US. These ground motion parameters are then used in conjunction with a series of factors to account for the effect of the natural period of the structures, site soil condition, inelastic behavior, relative importance of the structures, e.t.c.

These factors are determined largely based on judgment and experience and often calibrated such that the resultant designs do not deviate significantly from the acceptable practice at the time, thus they may contain much uncertainty. Therefore despite their simplicity and ease of use, the current seismic design provisions oversimplify a complex problem; there are many inherent assumptions built into the approach which often are not easily understood by or " transparent" to the designers. Another significant shortcoming is the inability to quantify the reliability of the final design; i.e. against either unserviceability or ultimate failure. In other words, the reliability of the structural system so designed is unknown and undefined. In view of the foregoing , it is obvious that there is a need for developing a more direct, reliability-based design procedure where the uncertainty in the problem is properly considered.

SECTION 2
THEORECTICAL BACKGROUND FOR RELIABILITY-BASED DESIGN

2.1 Introduction

In this section, the state of progress is reviewed of reliability analysis and reliability-based design for both time-invariant and time-variant problems. In the time-invariant reliability analysis, it is assumed that the uncertainty in loadings and structural resistance can be adequately modeled by random variables; it is inherently an approximation since most loadings on structure fluctuate in time but it has the advantage of being readily applicable to codified design. It has been the method used in most reliability-based code procedures thus far and for design with a single target limit state probability in mind. In the more realistic time-variant reliability analysis the time varying loads and structural response are modeled as random processes and therefore the loading modeling and load combination problem are major considerations and the satisfactory treatment of these problems would put the reliability-based design on solid theoretical foundation. Based on the time-variant reliability formulation, a design procedure is then proposed which properly considers the time varying nature of the loadings and uncertainty in the problem. It aims at satisfying a set of target reliabilities from serviceability to ultimate structural limit states with proper weighing of the risk and consequence of the limit states. Therefore, in contrast to most current code procedures, the emphasis is on developing a multi-level, reliability and performance based design procedure.

2.2 Reliability and Reliability-Based Design

2.2.1 Time-Invariant Problem

Reliability analysis under time-invariant loads and reliability-based design has been given mathematically rigorous treatments by researchers in the last two decades. Representative works can be found, for example, in Veneziano [1976] and Shinozuka [1983] . When the loadings and structural properties do not change with time or can be idealized as time-invariant, the uncertainties in the problem can be described by a set of basic random variables. The reliability of the structure against a limit state can be then described in terms of a performance function of these random variables. In other words the probability of limit state is given by:

$$P_f = P\,[\,g(\mathbf{x}) < 0\,] \qquad (2.1)$$

in which g is the performance function; **x** is the vector of the basic random variables describing the loadings and resistance. $g(\mathbf{x}) > 0$ indicates successful performance; $g(\mathbf{x}) < 0$ indicates limit state has been reached; and $g(\mathbf{x}) = 0$ is the limit state function which divides the basic variable space into safe and unsafe regions. P_f can be evaluated according to the well known First Order Reliability Method (FORM) or Second Order Reliability Method (SORM) [e.g. Ang and Tang 1984, Madsen et al 1986]. These methods are based on respectively first or second order approximation of the limit state function at the point where the contribution to the failure probability is highest, commonly referred to as the "design point". In other words the design point is on the surface described by the limit state function where the joint probability density function of the basic random variables reaches a maximum value. They are theoretically superior to the earlier method which approximates the limit state function at the point of the mean values [e.g. Ang and Cornell 1976] because: (1) that they incorporate all relevant information of the random variables including the moments as well as type of distribution and (2) they give results which are invariant under different mathematical formulations one may use to describe the same limit state. It has been proved that these methods are generally robust, accurate and computationally efficient. Depending on the complexity of the structural systems or limit state under consideration one may also use Monte-Carlo method or any of its variations for the same purpose.

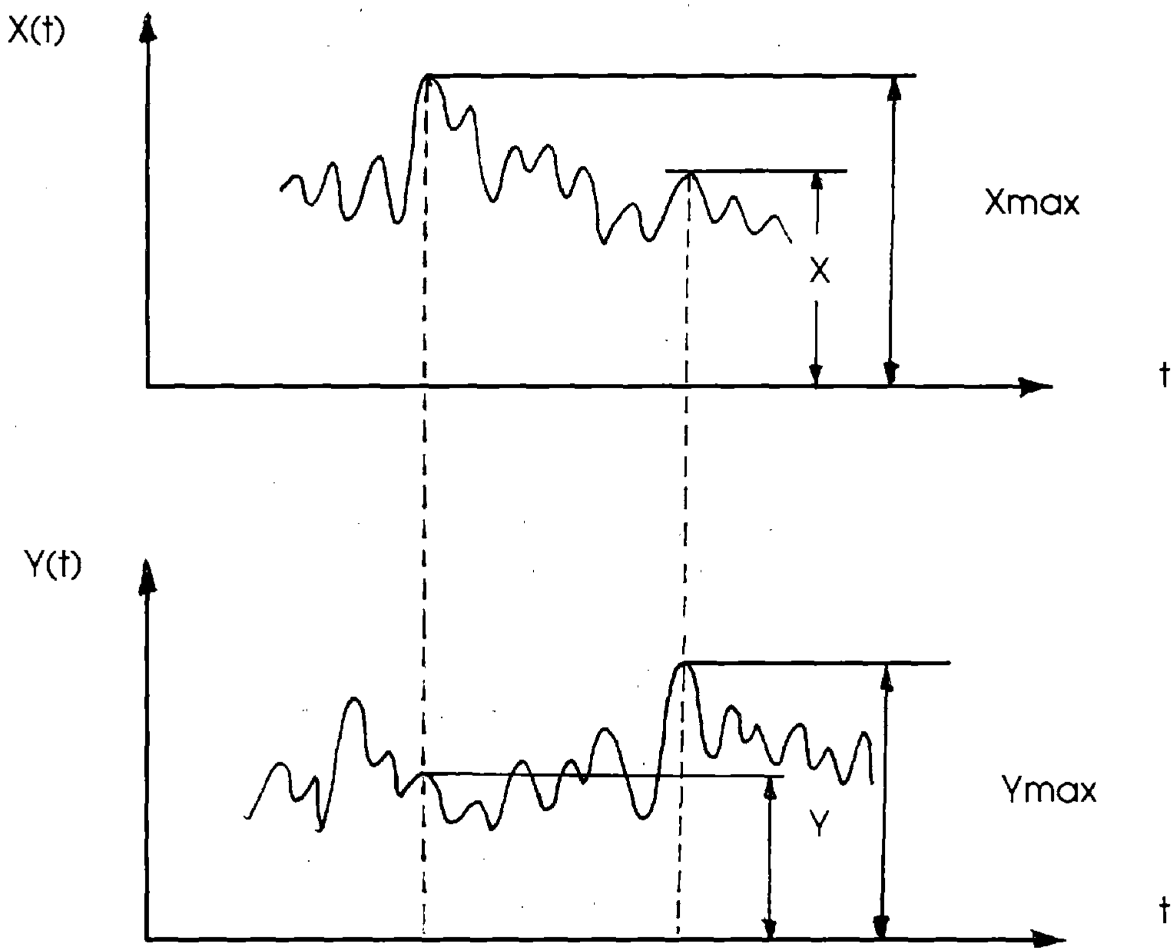
The FORM also allows one to derive the partial safety factors for design according to a prescribed reliability using the solution of the design point. In other words, one can use the coordinates of the (basic) random variables at the design point to express the required design resistance in terms of the moments (means and standard deviations) of the basic random variables and the target reliability level. This is also the theoretical basis for the Load and Resistance Factor Design (LRFD) format used in most reliability-based design procedures.

2.2.2 Time-Variant Problem

When the loadings are time-varying, the foregoing time-invariant procedure can be still used if one can characterize each load by a set of random variables. For example the commonly used procedure is to characterize the time varying loading by the arbitrary-point-in-time (APIT) value and lifetime maximum (LM) value. A simple empirical rule is then used for load combination (see Fig. 2-1) since in consideration of limit state over a given time period (e.g. time life of the structure) the life time maximum combined load effect is of primary concern. The so called

"Turkstra's Rule" [Turkstra 1970] is often used for this purpose which simply combines the LM value of one time varying load with the APIT values of all other time varying loads and select the largest values among all the combinations to approximate the lifetime combined maximum value. Therefore characterization of each time varying load by an APIT value and a LM value would be sufficient. The underlying assumption is obviously that the maximum value of one load or load effect occurs at the time that can be regarded as an arbitrary point in time for other loads. The accuracy of this approximate method has been widely discussed. It obviously works best when the loads or load effects are statistically independent. It generally leads to underestimates of the probability of the combined load effect since it ignores the possibility of combined maximum occur when none of the processes reaches a maximum. In many cases the error may be tolerable in comparison with the large uncertainty arising from our limited knowledge of actual loading and structural behavior. In other cases it may not. The approximation of time varying loads by random variables was basically the procedure used in many recent code calibration efforts. Notable examples are American National Standard Institute (ANSI) effort for development of probability-based load criteria [Ellingwood et al 1980] and American Petroleum Institute (API) calibration of load and resistance factor design [Moses and Larrabee 1990]. The results have been since adopted in ANSI/ASCE 7-88 Standard [1990] and API Recommended Practice 2A-LRFD [1991]. An LRFD procedure for design of highway bridges is also currently under study by AASHTO. It should be pointed out that these calibration efforts are restricted to checking of probability of a given limit state of the structural members, e.g. first yield occurrence; also linear elastic and equivalent static structural response behavior are assumed in converting loads to load effects to utilize the time-invariant reliability formulation.

In the case of seismic resistance design, however, the performance and safety of the overall structural system is of primary concern and the structural response is dynamic and nonlinear that a linear, equivalent static response treatment may become quite inadequate. Also, the load effects on the structure due to earthquake and other time varying loads may be correlated in occurrence time and intensity to which simple empirical rules for load combination may no longer be adequate. The reliability analysis requires a more detailed treatment of the time domain fluctuation of the load and structural response under this circumstances and so does the inverse problem of reliability-based design.



$$\mathrm{Max}[X(t) + Y(t)] \approx \mathrm{Max}[X_{max} + Y, Y_{max} + X]$$

FIGURE 2-1 Turkstra's Rule for Combining Time Varying Loads

2.3 Modeling of Time Varying Loads

The time domain fluctuation of loads and load effects can be taken into consideration explicitly by modeling the loads and load effects as random processes. In theory the reliability problem can be formulated as a random vector process outcrossing a safe domain described by

the limit state under consideration. The solution to this problem, however, is generally difficult. In the past decade, efforts have been concentrated on developing load and load effect processes and approximate solutions to the combination problem based on engineering considerations [Wen 1990, Larrabee and Cornell 1979]. For structural response analysis under environmental loads it proves to be convenient to describe the fluctuation of the loading according to a time scale corresponding to the structural period. For loadings with a macro-scale fluctuation (much larger than the structural period) such as live loads, snow loads and thermal load, they can be modeled by pulse processes with random occurrence time, random duration and random intensity. Fig. 2-2 shows the time histories of various loadings with macro-scale variability modeled by the pulse process. The structural response under such loading is primarily static and follows the fluctuation pattern of the loads. Only a few parameters are needed to characterize the pulse processes, i.e. mean occurrence rate, mean duration and intensity distribution. For loadings such as those due to winds, waves, and earthquakes which in addition have micro-scale fluctuation (in the vicinity of the structural period), the dynamic response is almost always important. Such loadings and the responses that they generate can be described by intermittent continuous random processes. This process is the result of superposition of a continuous random process on the pulse process given a set of the macro-time scale parameters, e.g. in the case of a storm, the occurrence, intensity and duration. Fig. 2-3 illustrates the structure of an intermittent continuous random process. Another important advantage of these models is that it is relatively easy to introduce dependencies within each load and among different loads, such as occurrence clustering, correlation in intensity and duration commonly observed in environmental loadings. Details can be found in Wen [1990].

These models require a small number of parameters thus allow easy estimation of parameters from load data. For example, recent analyses of wind and snow records of a large number of cities in the U.S. [Belk and Bennett, 1991, Bennett and Gilley 1991] have indicated that the pulse processes are valid models and that there is occurrence clustering in both loads and intensity correlation in snow load. These correlation and dependencies can be easily included in the pulse processes.

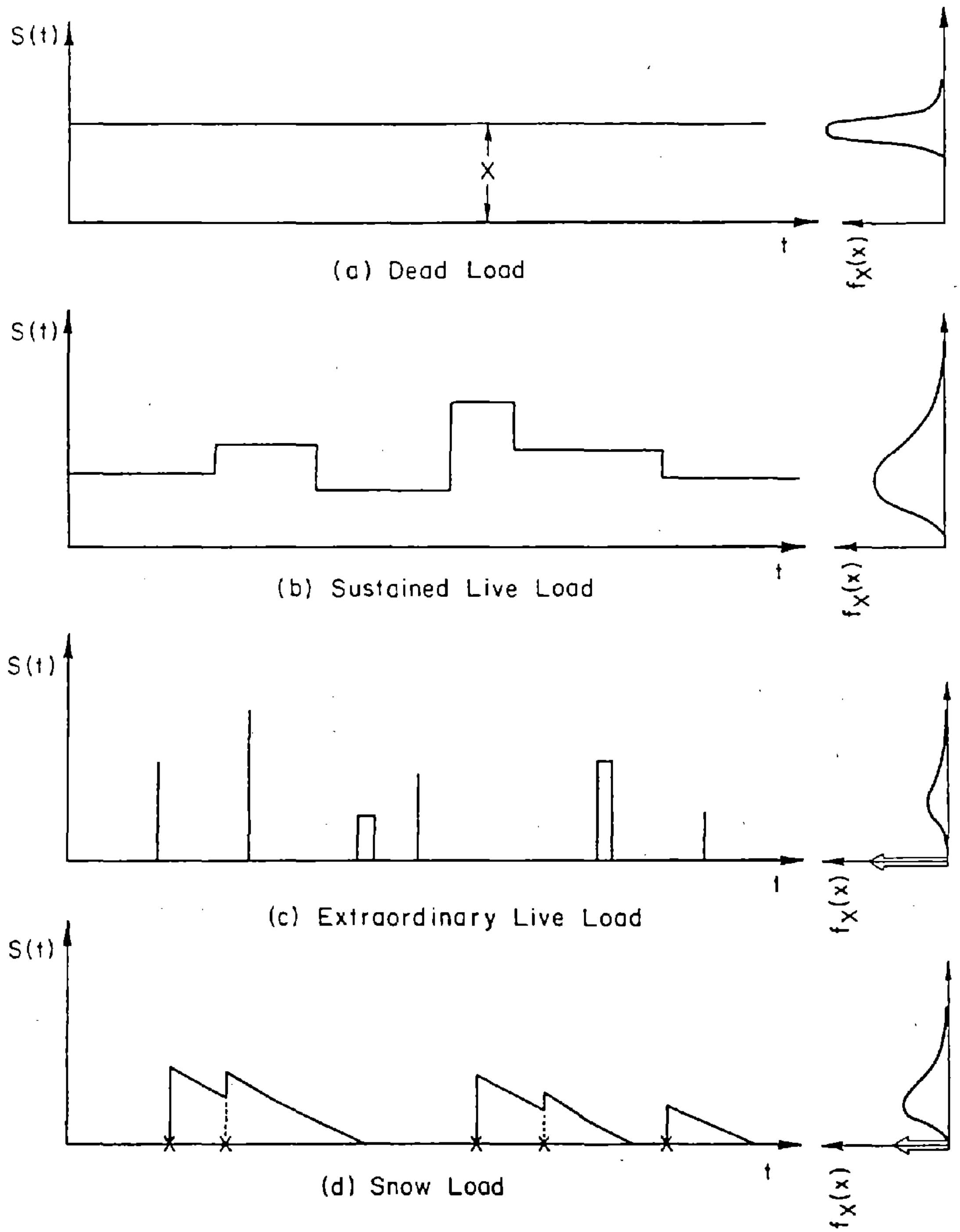


FIGURE 2-2 Loading with Macro-Scale Variability

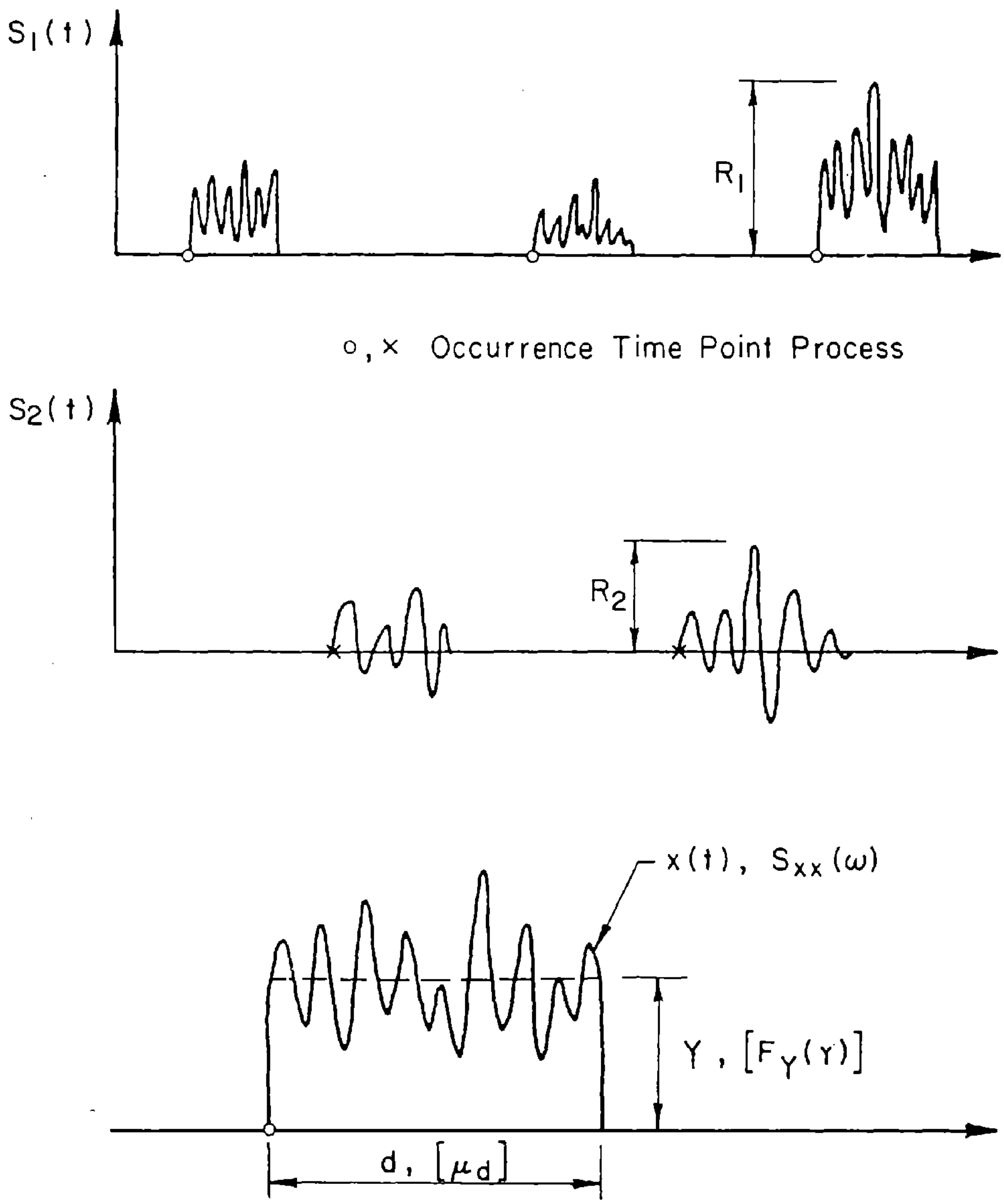


FIGURE 2-3 Load with Macro- and Micro-Scale Variability (Intermittent Continuous Random Process)

2.4 Reliability Under Combined Time Varying Loads

A major advantage of the foregoing load and load effect models is that they allow tractable solutions to the difficult problem of reliability of structures under the simultaneous actions of different time varying loads over a long period of time such as the lifetime of the structure. Extensive research efforts in this area in the last decade have resulted in methodologies which can be used for both linear and nonlinear combination of processes with macro-and micro-scale fluctuations. While the complexity and mathematical rigor of these methods vary, there has been a good convergence of the results. The event-based Load Coincidence Method [Wen 1990] and the Point Crossing Method [Larrabee and Cornell 1979] are most versatile and show comparable accuracy's and computational efforts required. Both are approximate methods; while the former is synthetic in nature thus suitable for nonlinear, dynamic problems and consideration of load dependencies, the latter is mathematically more rigorous. The Load Coincidence Method (LCM) is briefly outlined in the following.

Consider a limit state " E " involving combination (linear or nonlinear) of a number of loads or load effects modeled by either pulse or intermittent continuous processes or both. For example, E can be the ultimate limit state of interstory drift exceeding the code limit of 1.5 % of the story height under the combined action of dead, live, wind and earthquake loads. Let S_i (t) indicate the i-th load or load effect which is a time varying function treated as a random process. One is interested in the probability of the limit state, E, occurring over a time interval (0, t). In a LCM formulation, one considers the contributions to this probability from individual processes and various combinations of two or more processes. The solution can be approximated by:

$$P(E, t) \approx 1\text{-}\exp \{ - [\sum_{i=1}^{n} \lambda_i p_i + \sum_{i=1}^{n-1} \sum_{j=i+1}^{n} \lambda_{ij} p_{ij} + \sum_{i=1}^{n-2} \sum_{j=i+1}^{n-1} \sum_{k=j+1}^{n} \lambda_{ijk} p_{ijk} + \ldots]\, t \} \qquad (2.2)$$

in which λ_i is the occurrence rate of the process $S_i(t)$; P_i is the conditional probability of E , henceforth referred to as conditional probability of failure given the occurrence of $S_i(t)$ only; λ_{ij} is the mean rate of coincidence of $S_i(t)$ with $S_j(t)$; P_{ij} is the conditional probability of failure given the coincidence of these two processes; triple subscripts indicate coincidence of three processes e.t.c. Methods have been developed for evaluation of the mean rates of coincidence as function of the individual process occurrence rates, mean duration's and correlation parameters for processes that are not statistically independent. The conditional probabilities of limit state can be evaluated by reliability methods most suitable to the problem under consideration. For example, state-of-the-art system reliability analyses may used for static problems whereas random vibration method or time history/simulation method can be used for dynamic problems including those caused by seismic loads. Therefore for reliability analysis of structures under seismic and other time varying loads the most time consuming part of the analysis is the evaluation of the conditional probability of failure. A simplified analysis procedure with good accuracy and computational efficiency for evaluating the conditional probability of failure would expedite the reliability analysis. Such a procedure would be even more critical for developing a reliability-based design criteria as will be covered in later sections.

It is seen in Eq. 2.2 that the contribution to the overall failure probability depends on the conditional failure probability as well as the occurrence rate. For example, although wind and earthquake loads represent a serious combination that would certainly give high probability of failure, the rate of simultaneous occurrence (coincidence), $\lambda_{W,e}$, of these two loads is generally so small that this combination can be neglected. In most building code provisions the foregoing observation is invoked and the combination of wind and earthquake loads is not considered in load combination. Other combination may not be so obvious that the contribution can be discounted entirely. In the formulation given in Eq. 2.2, the importance of each combination is fully accounted by the coincidence rate and conditional probability therefore does not require any decision by the engineer on whether a particular combination needs to be considered. The formulation is also general enough that it is applicable to nonlinear systems and the load or load effect processes which are not statistically independent. The accuracy of this method is generally good and on the conservative side compared with other methods and Monte-Carlo simulations.

2.5 Code Calibration

The objective of design is to proportion the structure such that it will perform satisfactorily during its lifetime; in other words, have adequate reliability against all loadings and to achieve this at a reasonable cost. Implied in this design philosophy is that there is a target reliability level that one should try to achieve which strikes a balance between benefit and cost. Determination of a desirable target reliability is the age-old "how safe is safe enough" question whose answer requires an optimization considering cost-benefit trade-off with proper consideration of social-economical implications. To adjust the code parameters such that the requirement will be satisfied is commonly referred to as "code calibration" [Madsen et al 1986]. A widely used procedure in code calibrations [Ellingwood et al 1980, Moses and Larrabee 1988] is to assume that the target reliabilities can be inferred from what is acceptable in current practice. Therefore the satisfactory performance of the structure can be described in terms of a set of target reliabilities for various limit states and the design objective is then to satisfy these target values as much as possible.

One way to formulate this reliability-based design problem is to determine a set of design variable vector **X** such that the difference between the reliabilities of the design and target reliabilities for all limit states under consideration is minimized. In code calibration, **X** is the vector of the factors and coefficient in the provisions that govern the design. To include the wide spectrum of structural types, loading conditions and limit states into consideration, minimization of the following objective function has been proposed [Shinozuka et al 1989]:

$$\Omega = \sum_i \sum_j \omega_{ij} \left[\frac{\ln P_{ij}(x) - \ln P_j^*}{\ln P_j^*} \right]^2 \tag{2.3}$$

in which i refers to limit state and j refers to type of structure under consideration; ω_{ij} is the weight assigned to each case; $P_{ij}(\mathbf{x})$ is the probability of i-th limit state of structure type j resulted from the design with the design variable vector $\mathbf{X} = \mathbf{x}$; and P_j^* is the target probability of the j-th limit state. A slightly different formulation based on the safety index β has been also proposed as follows [Siu et al 1975]:

$$\Omega = \sum_i \sum_j \omega_{ij} [\beta_{ij}(x) - \beta^*_j]^2 \tag{2.4}$$

In both formulations the minimization is with respect to the design variable vector **X.**

The weight ω_{ij} in the above optimization problem is intended to reflect the importance of a given limit state of a particular type of structure. It should be in direct proportion to the consequence of the limit state for each type of structure. The square terms in the equations function as penalties for deviation from the target value. Since the expected consequence of the limit state is measured by the product of the consequence and the failure probability, the deviation term should be in probability, which obviously is not the case in either formulation. To incorporate the above consideration, an alternative mathematically convenient formulation has been proposed in the following [Wen 1994] :

$$\Omega = \sum_i \sum_j \omega_{ij} [p_{ij}(x) - p_j^*]^2 / p_j^* \qquad (2.5)$$

in which the penalty term is now proportional to the target failure probability. Figure 2-4 shows the comparison of the penalty terms divided by P* in the three formulations for the case that p_{ij} is equal to 60 % of P_j^* (or -40% deviation) for various target reliability levels. For a fair comparison the penalty term in Eq. 2.4 has been divided by β_j^*. It is seen that limit state contributions in Eqs. 2.3 and 2.4 are distorted depending on the target reliability level. This distortion is particular serious when P* is either very high or very low. Therefore these two formulations are not appropriate for problems in which limit states of different target probability levels are combined. In the case of design against seismic force, the generally accepted philosophy in design is that the structure should suffer no damage in minor earthquakes and no structural damage in moderate earthquakes and no collapsed in major earthquakes. In view of the different probability levels associated with the occurrence of earthquakes of different intensities the criteria therefore can be interpreted as different acceptable risk levels for limit states of various degree of seriousness. Therefore if a single design requirement is needed, in view of the multiple limit states and various target reliability levels formulation given in Eq. 2.5 should be used.

Note that in the above formulations, the probabilities of various limit states over a given time period control the design and these probabilities can be calculated according to the foregoing load combination methods, e.g., Load Coincidence Method. In other words, the various terms in Eq. 2.2 represent the various possible load combinations and the contribution of each combination in the final design. For instance, if a particular load combination dominates it would have high occurrence rate and high conditional probability of failure that would dominate Eq. 2.2 and hence play a dominant role in the resulting design criteria. Therefore there is no need to assign target reliabilities for various stipulated load combinations for which there is no rational basis.

In checking the safety of the design, it is most convenient to express the limit state function in terms of the load effect and structural resistance variables; or in other words the reliability problem is formulated in the load effect space. In formulating the design criteria, however, it is more convenient to directly specify the criteria in terms of loads. This is particularly true in code calibration where the conventional design formats are usually followed, e.g. the Load and Resistance Factor Design (LRFD) format used in most recent code calibrations. In this case the design variables **X** takes the form of load and resistance factors and the above minimization

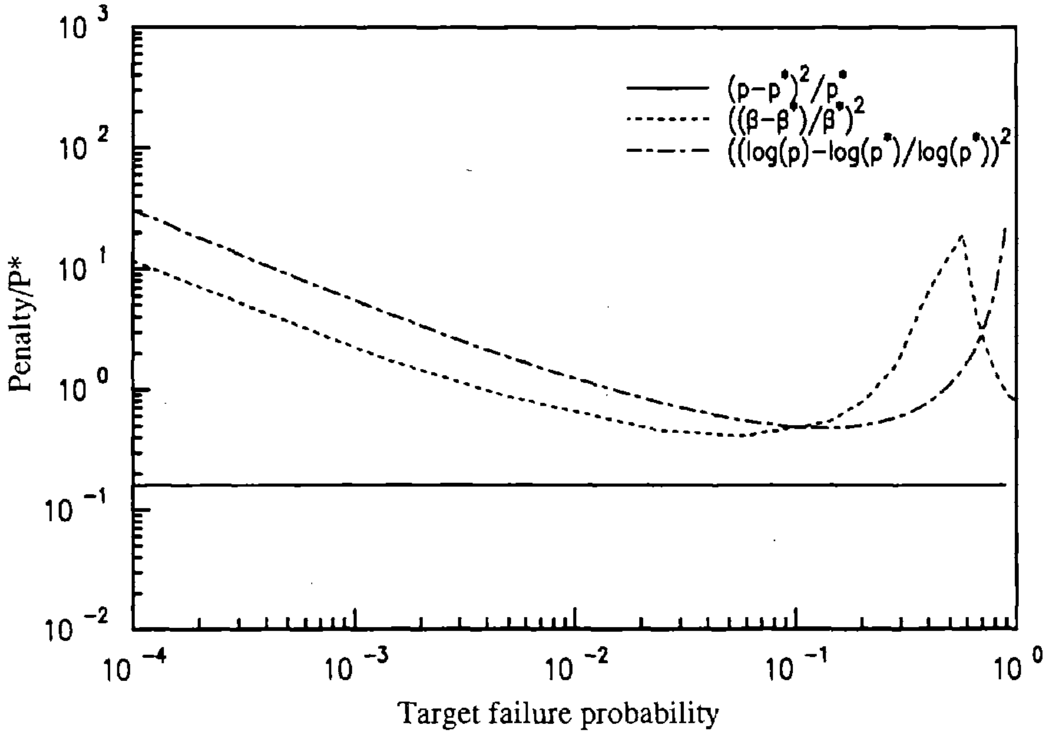


FIGURE 2-4 Comparison of Penalty Terms Devided by Target Failure Probability

would assure that the resulting load and resistance factors lead to designs which satisfy the prescribed target reliabilities for all limit states and types of structure under consideration as much as possible.

The solution for the design variables in the above formulation requires minimization of the objective function. Since the failure probability p_{ij} is in general a rather complicated nonlinear function of the design variables, the search for the minimal point is a nonlinear programming problem. It is pointed out that each iteration in the search requires a different design for all structural systems under consideration and a reliability analysis of the corresponding systems, the computational effort can easily become excessive. Under this circumstance, efficient approximate methods for both reliability analysis and search for the minimal point are highly desirable. Such methods have been developed and are given in Appendix A and Appendix B.

In the following, the current procedure is reviewed for developing probability-based load combination for structural design in a load and load resistance factor design (LRFD) format. In this format, the load and resistance factors associated with a particular load combination are determined for use in a limit state design on the basis of a target limit state probability. Critical comments are provided on the validity of the methodology by which these load and resistance factors are derived.

2.6 Load Combination and Load Resistance Factor Design

As mentioned in the foregoing, in the practice of structural design, both extreme and abnormal loading conditions must be considered. This requirement possibly results in a large number of load combinations in the design criteria. Furthermore, the load and resistance factors specified in the code are usually determined by the code committee primarily on the basis of collective judgment and experience. Hence, a rational procedure is needed to justify the number of load combinations and to determine appropriate load and resistance factors. The procedure of probability-based load combination criteria is reviewed here for structural design and critical comments are also provided. While the discussion primarily centers on the LRFD methodology, its implementations extend to a re-examination of the basic issues associated with the concept of structural safety and design. In this connection, the notion of the limit state probability curve (surface) is introduced.

Procedure for Establishing Load Combination Criteria

While many assert that the current probability-based LRFD criteria are rational, the way in which particular load combinations are chosen for design purposes appears to be rather arbitrary. To be on the safe side, one tends to cover all the possible combinations of all the conceivable loads. Indeed, this appears to be the case for nuclear power plant design where the grave consequences of failure warrants the utmost care in selecting such load combinations. Probabilistically speaking, one can obviously enumerate all the possible load combinations that are mutually exclusive. For simplicity, consider a structure subjected to a primary load D/L (dead and live load), earthquake load E and wind load W.

Recognizing that highly frequent micro-tremors and constantly present breezes do not really constitute an earthquake and wind loads, respectively, the structure will be subjected to a set of mutually exclusive load combinations consisting of D/L, D/L+E, D/L+W and D/L+E+W:

D/L:	dead and live load only; no other load acting;
D/L+E:	dead/live load and earthquake load but not wind load;
D/L+W:	dead/live load and wind load but not earthquake load;
D/L+E+W:	dead/live load, earthquake and wind loads

However, identifying a certain load combination as part of such a mutually exclusive set does not necessarily warrant that this combination be considered for structural design. Indeed, for the choice of load combinations to be considered for design, limit state probabilities must be taken into consideration.

Structural limit states represent various states of undesirable structural behavior. For example, the allowable stress σ_a , a state of stress corresponding to the yield stress σ_y divided by a (material) safety factor, represents a limit state. Similarly, the yield stress σ_y and ultimate stress σ_u represent limit states which, however, have more physical significance than the allowable stress. Note the words "allowable", "yield" and "ultimate" stress are used for simplicity of discussion. In the present study, they conceptually represent the following states of structural behavior: allowable stress = threshold of undesirability, possibly with respect to serviceability or to stress history dependent failure such as fatigue, yield stress = threshold of permanent deformation, and ultimate stress = threshold, possibly leading to structural collapse.

The limit state probability associated with σ_a is then given by $P_a = P\{\sigma > \sigma_a\}$ where $\{\sigma > \sigma_a\}$= the event that the state of stress at some location in the structure exceeds σ_a at least once in the structure's lifetime. Since the events D/L, D/L+E, D/L+W and D/L+E+W are mutually exclusive, this limit state probability can be written as

$$\begin{aligned} P_a = {} & P\{\sigma > \sigma_a \mid D/L\} P\{D/L\} + P\{\sigma > \sigma_a \mid D/L+E\} P\{D/L+E\} \\ & + P\{\sigma > \sigma_a \mid D/L+W\} P\{D/L+W\} \\ & + P\{\sigma > \sigma_a \mid D/L+E+W\} P\{D/L+E+W\} \end{aligned} \tag{2.6}$$

where $P\{\sigma > \sigma_a \mid A\}$ = limit state probability conditional to A and $P\{A\}$ = probability of A. Similarly,

$$\begin{aligned} P_y = {} & P\{\sigma > \sigma_y \mid D/L\} P\{D/L\} + P\{\sigma > \sigma_y \mid D/L+E\} P\{D/L+E\} \\ & + P\{\sigma > \sigma_y \mid D/L+W\} P\{D/L+W\} \\ & + P\{\sigma > \sigma_y \mid D/L+E+W\} P\{D/L+E+W\} \end{aligned} \tag{2.7}$$

and

$$\begin{aligned} P_u = {} & P\{\sigma > \sigma_u \mid D/L\} P\{D/L\} + P\{\sigma > \sigma_u \mid D/L+E\} P\{D/L+E\} \\ & + P\{\sigma > \sigma_u \mid D/L+W\} P\{D/L+W\} \\ & + P\{\sigma > \sigma_u \mid D/L+E+W\} P\{D/L+E+W\} \end{aligned} \tag{2.8}$$

The target limit state probabilities P_a^*, P_y^* and P_u^* are then introduced, being respectively associated with σ_a σ_y, and σ_u and the design must satisfy

$$P_a < P_a^*, \qquad P_y < P_y^*, \qquad P_u < P_u^* \tag{2.9}$$

Note that the formulation given in Eq. 2.6 to 2.8 is equivalent to that in Eq. 2.1 except for the joint probabilities of occurrence which is used in place of the joint occurrence rates in Eq. 2.1.

Limit State Probability Diagram

The notion of a limit state probability diagram is introduced at this point as shown in Figure 2.5. The diagram plots the common logarithm of the probability $P_f^{(i)}(x) = P\{\sigma_i > x\}$ that the response state σ_i will exceed x at least once in the structure's lifetime as a function of x. Curve B_i in Fig. 2.5 indicates $P_f^{(i)}(x)$ for structure i. Note that such a curve depends on the structure, thus the super-or subscript i. When x assumes specific limit state values such as $x = \sigma_a$ σ_y, or σ_u, $P\{\sigma_i > x\}$ represents the corresponding limit state probabilities.

The target limit state probabilities P_a^*, P_y^* and P_u^* are indicated respectively by points A_I, A_{II} and A_{II} in Figure 2-5. While it is not a well-recognized notion, we suggest that conceptually the safety of a class of structures, for which the design code is intended to be used, should be specified by a target limit state probability curve $P_f^*(x)$, as designated by A in Fig. 2.5. If the state of structural behavior is to be described by more than one variable, say by x and y, the safety should be specified by a target limit state surface $P_y^*(x,y)$.

Since it is impractical to prescribe the entire curve $P_f^*(x)$ as a safety requirement and, even if one could do that, it is impractical to verify if $P_f^{(i)}(x) < P_f^*(x)$ or curve B_i is below curve A for all values of x, one chooses a few values of x to perform such a check. In the present paper, $x = \sigma_a$ σ_y, and σ_u are chosen as an example. Curve I_i in Figure 2-5 represents

$$P_{fI}^{(i)}(x) = P\{\sigma_i > x \mid D/L\}\, P\{D/L\} \qquad (2.10)$$

Similarly, curves II_i, III_i and IV_i represents respectively

$$P_{fII}^{(i)}(x) = P\{\sigma_i > x \mid D/L+E\}\, P\{D/L+E\} \qquad (2.11)$$

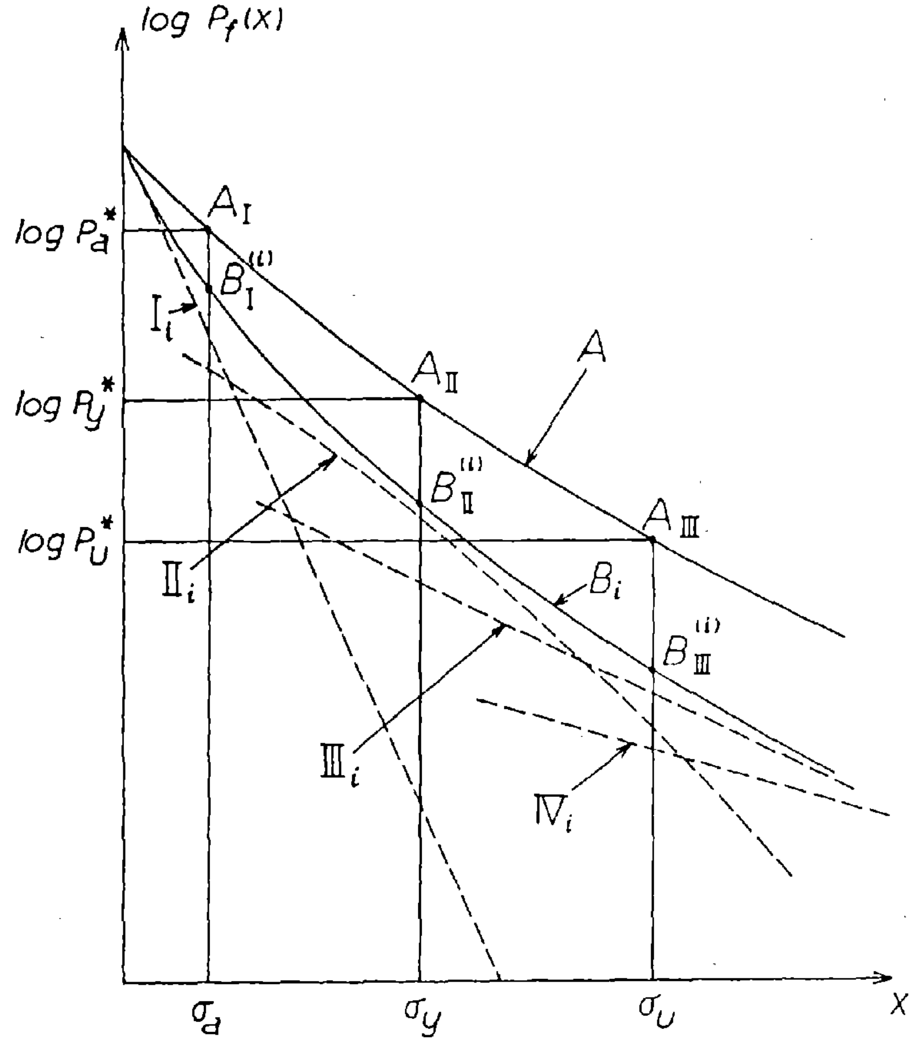


FIGURE 2-5 Limit State Probability Diagram

$$P_{fIII}^{(i)}(x) = P\{\sigma_i > x \mid D/L+E\}\, P\{D/L+E\} \tag{2.12}$$

$$P_{fIV}^{(i)}(x) = P\{\sigma_i > x \mid D/L+E+W\}\, P\{D/L+E+W\} \tag{2.13}$$

Curve B_i in Figure 2-5 is the sum of the probability values represented by curves I_i, II_i, III_i and IV_i.

If the curves I_i, II_i, III_i and IV_i indeed take the relative positions sketched in Figure 2-5, then the D/L combination controls the limit state probability P_a, D/L+E the limit state probability P_y and D/L+E+W the limit state probability P_u. Note that, in this case, the combination D/L+E+W does not really control any of the limit state probabilities. If all the structures to be designed under the design code exhibit this trend, then the combination D/L+E+W does not have to be considered in the design, and the combination D/L should be considered only for I_i, II_i, III_i and IV_i , D/L+E for σ_y and D/L+W for σ_u. In fact, this can be interpreted as the conceptual basis for allowing the allowable stress to be increased when combinations of primary and secondary loads are considered in the classical allowable stress design.

Obviously, the dominance of a particular combination of loads for a particular limit state does not necessarily materialize in reality and therefore the above interpretation is most probably too simplistic.

The limit state probability diagram nevertheless clearly indicates the interrelationship among the limit states, limit state probabilities, target limit state probabilities and load combinations. More importantly, the limit state probability diagram as introduced here provides a much more global interpretation of the safety of a structure. Finally, it is pointed out that the state of structural response σ_i may take a most undesirable value at different structural locations, depending on the load combinations and therefore the limit state probability diagram may not necessarily be constructed with respect to a specific point in the structure.

Load and Resistance Factor Determination

The currently practiced procedure for determining load and resistance factors can then be extended to deal with a more general interpretation of the safety as introduced above. For example, consider the following LRFD format:

$$\phi_I R_n = \gamma_{DI} D_n + \gamma_{LI} L_n \tag{2.14}$$

$$\phi_{II} R_n = \gamma_{DII} D_n + \gamma_{LII} L_n + \gamma_{EII} E_n \tag{2.15}$$

$$\phi_{III} R_n = \gamma_{DIII} D_n + \gamma_{LIII} L_n + \gamma_{WIII} W_n \tag{2.16}$$

where R_n, D_n, L_n, E_n and W_n are the nominal values of resistance, dead load, live load, earthquake load and wind load, and the γ's and ϕ's represent the load and resistance factors, respectively.

Consider, then, a set of N representative structures (i = 1, 2, , N) and assign initial values to all the load and resistance factors, design each representative structure, develop an objective function which measures the difference between the target limit state probabilities and the computed limit state probabilities, determine a new set of load and resistance factors in the direction of maximum descent with respect to the objective function, and repeat these steps until a set of load and resistance factors that minimizes the objective functions is found.

According to Eq. 2.5, the objective function $\Omega = \Omega$ (ϕ_I , ϕ_{II} , ϕ_{III}, γ_{DI}, γ_{LI}, γ_{DII}, γ_{LII}, γ_{EII}, γ_{DIII}, γ_{LIII} ,γ_{WII}) is

$$\Omega = \omega_I \sum_{i=1}^{N} \frac{\left[P^i{}_a - P^*{}_a\right]^2}{P^*{}_a} + \omega_{II} \sum_{i=1}^{N} \frac{\left[P^i{}_y - P^*{}_y\right]^2}{P^*{}_y} + \omega_{III} \sum_{i=1}^{N} \frac{\left[P^i{}_u - P^*{}_u\right]^2}{P^*{}_u} \tag{2.17}$$

where $P_a{}^i = P_f{}^{(1)}(\sigma_a)$, $P_y{}^i = P_f{}^{(1)}(\sigma_y)$ and $P_u{}^i = P_f{}^{(1)}(\sigma_u)$, and ω_I , ω_{II} , ω_{III} ,are the weights that are assigned to the limit states σ_a , σ_y , σ_u , respectively. In principle, the optimum values of the load and resistance factors can be obtained from

$$\frac{\partial \Omega}{\partial \delta} = 0 \qquad (\delta = \phi_I, \phi_{II}, \ldots\ldots\ldots \gamma_{WII}) \tag{2.18}$$

The practical difficulties in solving such equations and computationally efficient methods that have been developed for this purpose have been discussed in Section 2.5.

In summary, the LRFD format is considered from a more global point of view then that currently prevailing. In this connection, the notion of the limit state probability diagram is introduced to conceptually clarify the interrelationships among the limit state probability, target limit state probability and load combinations. A method consistent with the limit state probability diagram concept introduced here is suggested to determine the load and resistance factors.

2.7 Multi-Level, Reliability and Performance Based Design

It is clear from the foregoing that to reflect the uncertainties in the structural loading and resistance, the performance of the structure need to be described in terms of risk (probability) of the limit states being exceeded over a given period of time. Figure 2-6 illustrates such performance requirement curves. The vertical axis indicates the risk of limit state being exceeded and the horizontal axis shows the gradation of seriousness of the limit states from loss of serviceability to life safety being endangered. In Figure 2-6 two curves are shown; one for important and one for other structures. A family of such curves may be used as target performance curves for structures of various importance. The objective of design is then to ensure that the target performance curves are satisfied at least at the key points indicated by a_1, a_2, and a_3 and b_1, b_2, and b_3 which correspond to elastic limit, repairable damage, and significant damage. Such a procedure may be called multi-level, reliability-based design.

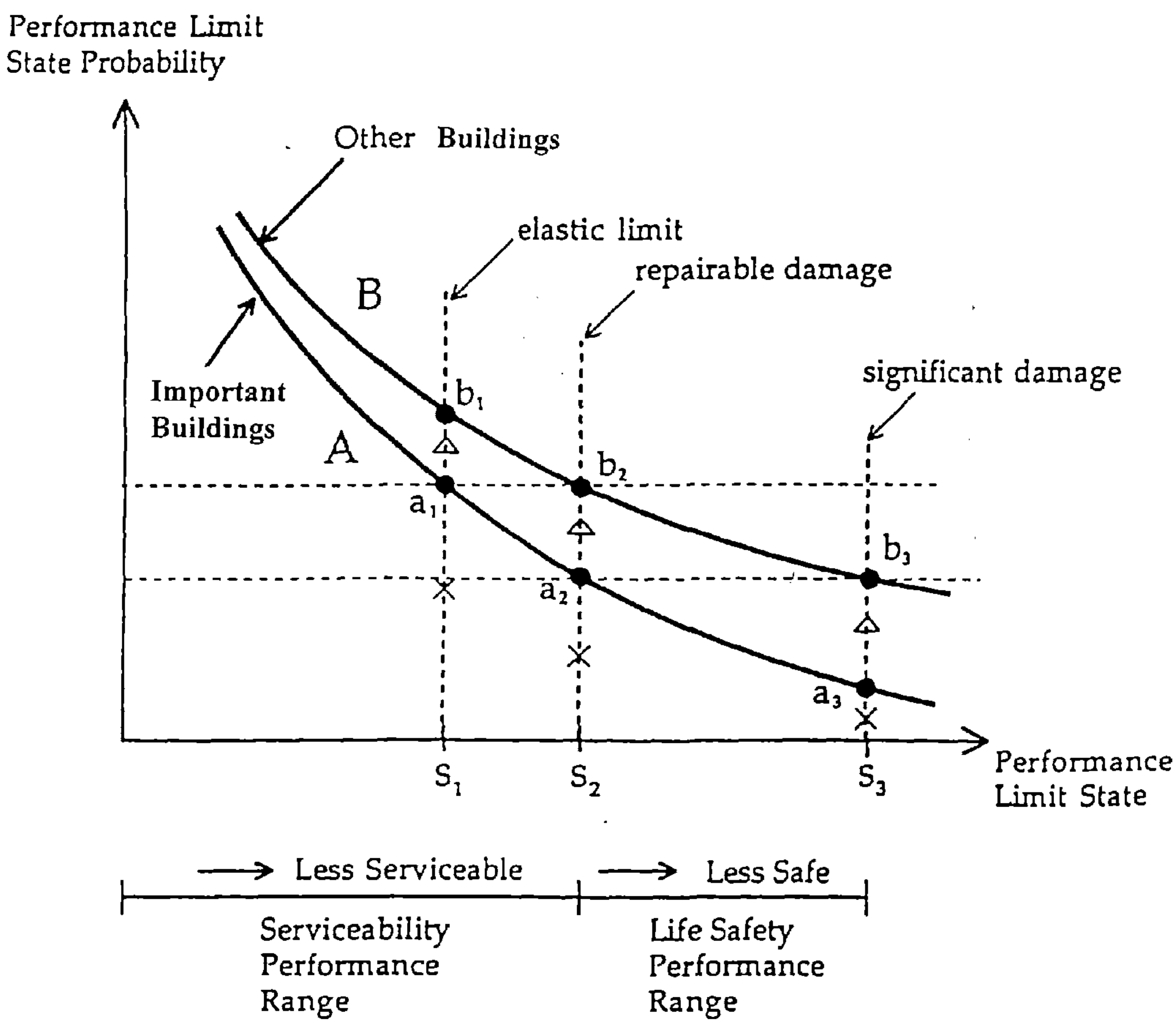


Figure 2.6 Structural Performance Curve

SECTION 3
REVIEW OF CURRENT DESIGN PROCEDURES FROM RELIABILITY POINT OF VIEW

3.1 Introduction

Code design procedures evolve with time and most have undergone periodical upgrading based on lessons learned in past earthquakes. The uncertainties in the seismic loadings and structural resistance have long been recognized by engineers and accounted for to some extent by using conservative values in selecting design load and assessing structural resistance capacity. Since judgment and experience play an important role in the process, it is difficult to quantify the margin of safety or the reliability implied in the procedures. Most recent code procedures select the design earthquakes based on a prescribed level of probability of occurrence but generally they do not set risk goal for the structural performance. Still in others even the probability levels for the design earthquake are not given. Since in addition to design earthquake, various other factors and performance limits are used in the design, the exceedance of a design earthquake by no means indicates that failure (or limit state) of the structure will occur. The reliability of the codified design procedures against future earthquakes therefore is still largely an open issue.

In this section, some representative current code procedures are reviewed from reliability point of view - namely Uniform Building Code [UBC, 1991], Department of Energy Design (DOE) Guidelines [1990], Architectural Institute of Japan (AIJ) Design Guidelines [1990], and New Zealand Code of Practice [1984]. The first two are similar in concept; both are single-level design procedures. DOE guidelines adopt UBC provisions for certain categories of structures, therefore the UBC format is discussed in more detail. The last two are two-level design procedures. The AIJ Guidelines represent the most recent effort of modernizing building design with explicit provisions for enforcement of proper structural inelastic response behavior; therefore are also discussed in more detail. Throughout the section though, the emphasis is on the risk criteria used in selection of design earthquakes, how safety and satisfactory performance of the structure is incorporated in the procedure and how uncertainties are accounted for. Discussion of fine details of design provisions in these documents and comparisons of the design procedures can be found in recent literature devoted to this purpose, e.g. Bertero et al [1991]. Finally, reliability implied is discussed in performance of structure designed according to the code procedures based on results of recent studies.

3.2 UBC and DOE Procedures

3.21 UBC Procedures

Base Shear V

In the most recent Uniform Building Code provisions [UBC 1991], the design earthquake base shear is determined from:

$$V = ZICW/R_w \tag{3.1}$$

in which Z is the design earthquake ground acceleration according to the zone in which the structure is located. Z is based on the criterion that the design earthquake has a probability of 10% being exceeded in 50 years. Assuming independence in earthquake occurrence time and intensity a Bernoulli sequence model gives the relationship between 50 -year exceedance probability, P_{50}, and annual exceedance probability, P, as follows:

$$P_{50} = 1 - (1 - P)^{50} \tag{3.2}$$

One can solve from the above equation to determine the annual probability to be 0.002105, or a corresponding return period of 475 years, that an earthquake of an intensity greater than the design earthquake will occur. Note that the above relationship or the return period description is only approximate and for purpose of easy reference and comparison since seismological studies at most locations show that neither the occurrence time nor the intensity is strictly statistically independent from occurrence to occurrence. Note also that the annual probability of the exceedance of the design earthquake provides little risk information on the performance of the structure since a string of factors is used in the above equation and all of them affect the structural resistance and hence the reliability. Furthermore, the structure needs also to satisfy drift and other limits which generally have an impact on the final design.

Importance Factor I

I is the importance factor which varies from 1 to 1.25 according to the importance of the type of occupancy. Therefore I is a factor to account for the safety or performance standard required of the structure. The basis for assigning I has not been clearly indicated. Such a small range of I has also been criticized [e.g. Bertero et al 1991], for structures of the most stringent safety requirement a small factor of 1.25 would fall far short of keeping the structure in the linear elastic range under a severe earthquake. From a probabilistic point of view, this small range is also hard to justify since the uncertainty in the seismic excitation is generally so large that different reliability levels required of the structure would lead to a much larger range of the structural resistance. To determine the importance factor rationally and quantitatively, a calibration of this value needs to be performed according to the performance goal required of the structure in terms of acceptable risks of limit states.

Response Spectrum Factor C

C is a function of the structural period and site soil condition; i.e., basically it is the elastic response spectrum multiplied by a factor according to four classifications of soil condition. As far as the characterization of soil condition is concerned, there are two significant shortcomings with this approach. First, the effect of soil deposit at the site cannot adequately be accounted for by a constant soil factor; the structural ductility and dynamic characteristics of the soils need also to be taken into consideration [Krawinkler and Rahnama 1992]. Second, response spectra have distinct regional shapes and should be specified accordingly [Algermissen and Leyendecker 1992, NEHRP provisions in BSSC 1992]. A risk consistent procedure therefore needs to take the above into consideration and develop a response spectrum with explicit information on the probability of exceedance of the spectral values.

Structural Quality Factor R_W

R_W is the " structural system quality factor". Value of R_W varies from 4 to 12, therefore this factor alone can cause a factor of two to three difference in the design base shear. It is also a factor causing most controversy. It is intended to account for the ability of the structure to resist the seismic force when it goes into the inelastic range due to structure's ductility, additional damping and overstrength which is not included in the linear elastic response spectrum analysis. The basis of assigning R_W , however, is largely empirical and judgmental and not to deviate from accepted practice; it fails to account for the important effects such as structural period and ground motion characteristics. The shortcomings of this factor is well documented [Bertero 1989]. Its risk implication is of most importance and yet difficult to assess in the current procedure. There is much recent sentiment of moving away from the R_W factor approach and toward an alternative procedure based directly on inelastic response spectra that may be more appropriate for developing direct and risk explicit design procedure.

Drift Limits

In UBC the design is required to satisfy the drift limits under the design base shear given by d defined as the interstory displacement divided by the story height:

$$d = 0.03/R_w \text{ and } < 0.004 \tag{3.3}$$

which often turns out to be the governing provision except for low-rise buildings. Therefore in addition to the base shear the drift limits oftentimes play a crucial role in determining the reliability of the design. The maximum interstory drift that is expected during the earthquake is estimated by:

$$d_{max} = 3 R_w d/8 \tag{3.4}$$

Combining Eqs 3.3 and 3.4 one obtains a d_{max} of 1.5 % independent of structural system quality factor. The maximum drift is obviously a good measure of the performance of the design against future earthquakes. The actual maximum drift under future earthquakes over the lifetime of the structure is a random quantity and there is no guarantee that this 1.5 % drift limit will be satisfied for all future earthquakes. A rational criterion for d_{max} should be established based on acceptable risk. In other words, in a reliability-based design format, the performance of the design is governed by acceptable risks for drift limits at the serviceability as well as at the ultimate failure levels.

3.2.2 DOE Procedure

As in UBC the DOE provisions also focus on designing the structure to resist a prescribed set of equivalent lateral forces. Drift limits are imposed to control structural and nonstructural damage. A R factor is used to reduce the design base shear. It uses a single-level seismic design criterion, i.e. one "design earthquake level" for which the structure must be designed. The format and risk implications are therefore the same as in the UBC provisions and not repeated here. There are some major differences, however, in definition of design level earthquake and performance goals and use of R factors which are summarized in the following.

DOE guidelines assign a different level of earthquake hazard to each of the various categories of buildings. For example, the annual probability of exceedance of the design earthquake for " General use (GU) " facilities is set at 2×10^{-3}. For " Important (I) or Low Hazard (LH) " facilities (such as hospitals, fire stations, e.t.c.) and moderate Hazard (MH) facilities, this annual exceedance probability is set at 10^{-3}. For High Hazard (HH) facilities, the probability is 2×10^{-4}. Therefore the exceedance probabilities for GU, LH, MH, and HH are of the 10 : 5 : 5: 1 ratio. In addition an importance factor 1.25 is also applied to Important or Low Hazard Facilities. The last two categories (MH and HH) generally refer to nuclear power plant facilities. In comparison, UBC has only one level of 2.1×10^{-3} for all structures.

DOE guidelines clearly identify the performance goals for each category of building in terms of probability of exceedance of some measure of damage. For GU facilities, for which the primary goal is to avoid major damage or collapse, the annual probability of failure to achieve the performance goal is 10^{-3}. For I and LH facilities, for which the goal is to ensure occupant safety and to ensure that the building can still perform its intended function following an earthquake, the annual probability of failure is 5×10^{-4}. For MH and HH facilities, for which the goal is occupant safety, continued function and hazard confinement the corresponding probabilities are 10^{-4} and 10^{-5} respectively. The target risk levels for the design earthquake and performance goal (limit state) of different classes of structures are summarized in Table 3.1. Note also that for ordinary (non-nuclear, GU and LH) structures the ratio of the risk of design earthquake to that of failure to achieve the performance goal is 2 and for nuclear structures (MH and HH) the ratio is from 10 to 20. As mentioned in the foregoing , UBC does not explicitly identify target performance goals in terms of probability.

For GU and LH facilities, R_w as recommended in UBC is used. For MH and HH facilities a F_μ factor which is much lower than R_w is used to account for inelastic response behavior and achieve the much lower risks of failure associated with the performance goal.

In summary, although DOE guidelines follow a similar approach as in UBC, they aim toward a more risk explicit procedure where design earthquakes and performance goals for buildings of different hazard categories are associated with specific annual probability levels.

Table 3-I
Target Risk Levels for Design Earthquake and Structural Performance
(DOE Procedure)

Structural Category	Annual Risk of Design Earthquake	Performance Goal	Annual Probability of Failure
General Use (GU)	$2x10^{-3}$	Occupant Safety (OS)	10^{-3}
Low Hazard (LH)	10^{-3}	OS, Continued Operation	5×10^{-4}
Median Hazard (MH)	10^{-3}	OS, Continued Function Hazard Confinement	10^{-4}
High Hazard (HH)	2×10^{-4}	OS, Continued function Hazard confinement	10^{-5}

3.3 Japan and New Zealand Design Procedures

Unlike the UBC and DOE procedures, the Japan the New Zealand procedures consider more than one level of design earthquake and explicitly consider the inelastic response behavior and assurance of safe post-yielding structural performance such as prevention of column mechanism. On the other hand, the probabilities of the design earthquakes and the performance goal probabilities are not clearly spelled out as in UBC and DOE procedures. Some details of the two procedures in terms of its risk implications are given in the following.

3.3.1 Building Standard Law (BSL) and Architectural Institute of Japan (AIJ) Design Guidelines

Since 1981, Building Standard Law (BSL) Enforcement Order, adopted a two-level design procedure in Japan; i.e., (a) traditional allowable stress design and (b) examination of ultimate lateral load resistance of each story. For allowable stress design under moderate earthquake, the standard base shear coefficient is 0.2 with no plastic deformation. For examination of ultimate lateral load resistance under severe earthquake, the standard base shear is 0.3 for ductile structures with no collapse but accepting yielding. BSL is the current design code for building not taller than 60 m. For structural taller than 60 m, the design is subjected to the review of the Structural Review Committee for High-rise Buildings of the Building Center of Japan. The probability levels for these two design earthquakes correspond to several times during the use of the buildings for moderate earthquake to a low probability of occurrence for severe earthquake. As in UBC, the service level design base shear is equal to the standard base shear modified by factors depending on zone, structural period, soil type and distributed over the height of the building according a nonlinear formula. Similarly for ultimate level design earthquake, there is a period dependent factor and a structural coefficient that reduces the base shear to take inelastic behavior into consideration. The net reduction, however, is much smaller than those in UBC and as a result the base shear could be of a factor of 2 to 3 that of UBC for comparable structural frames [Bertero et al 1991]. It is pointed out here that the higher base shear by itself does not necessarily mean that the requirements in BSL is proportionally larger since the design also has to satisfy the drift limits which are less strict in BSL, i.e., from 0.005 to 0.0083 under moderate earthquake ground motions and no explicit limit for maximum drift under severe earthquake ground motions.

In 1990 Design Guidelines for Earthquake Resistance Reinforce Concrete Buildings Based on Ultimate Strength Concept was proposed by Architectural Institute of Japan (AIJ).
The refinement of the guidelines is still in progress [US-Japan PRESSS Project 1992, Otani et al 1992]. They provide a design procedure for R.C. moment-resisting frame or wall-frame structures, regular in shape and not taller than 45 m. As in BSL, the new guidelines provide two performance criteria; one based on serviceability limit state under moderate earthquake and the other based on an ultimate limit state under strong earthquake. The design earthquake story shear at i-th story is calculated by:

$$Q_i = Z\, R_t\, A_i\, C_B\, W_i \tag{3.5}$$

in which Z = the seismic zone factor;

R_t = the vibration characteristics factor taking into account types of soil;

A_i = vertical distribution of seismic story shear; and

C_B = standard base shear coefficient.

A structure should be serviceable after a moderate earthquake motion. The standard base shear coefficient for the serviceability limit state is 0.2. The shear distribution along the structural height is the same as in BSL. No member is allowed to yield in this limit state and the story drift ratio must be less than 0.005. A structure should not collapse during a strong intensity earthquake motion. The standard base shear coefficient for the ultimate limit state is 0.3 for moment resisting frames. At a story drift ratio of 0.01, which is called the design limit deformation, the story shear at any story must be more than 0.9 times the design story shear for the ultimate state. At story drift ratio of 0.02, which is called the design proof deformation, the story shear must be more than the design story shear for the ultimate limit state. This strength versus drift relationship is shown in Fig. 3.1. Therefore the idea is to ensure proper inelastic response behavior under seismic load of the structure by closely monitoring the performance of the structure under an equivalent static force well into the inelastic range.

The building is designed to form a strong-column weak-beam failure mechanism. Regions other than the specified yield hinges must be designed not to yield. The design of these non-yielding regions must take into consideration factors which might increase the member design forces such as the dynamic effect and the bi-directional response. Therefore the basic philosophy of this design procedure was to avoid large plastic deformation, concentration of damage in limited locations, and brittle failure [Otani et al 1992]. Note also that no importance factor is used and the performance of the structure is strictly enforced by the explicit consideration of adequate strength of the structure at various drift levels after the structure becomes inelastic. On the other hand, as in the BSL, moderate and strong earthquake are defined by occurrence rate of several times and once respectively in a building's lifetime and no specific performance goals in terms of probability are given.

3.3.2 New Zealand Code of Practice

As in AIJ guidelines, the New Zealand Code of Practice for General Structural Design and Design Loading for Buildings (NZS 4203: 1984) is a two-level design; limit states design is used to proportion members and then "capacity" design is used to ensure that the structural frame will behave according to the preferred mode of energy dissipation. In other words, as in AIJ, the design ensures inelastic response at intended critical sections to avoid brittle failure elsewhere. As in UBC, an importance factor related to the risk to life of failure is used which can be as high as 2. As in UBC a structural period independent factor called structural type factor is used to reduce the design base shear taking into account the ability of the structural to dissipate energy and its degree of redundancy. There is a maximum drift ratio limit of 0.01 and general deformation limit ratio of 0.002. As far as risk implication is concerned, the design earthquake corresponds to extreme earthquake ground motion but no specific risk level is mentioned; neither is there any requirement of performance goal in terms of probability.

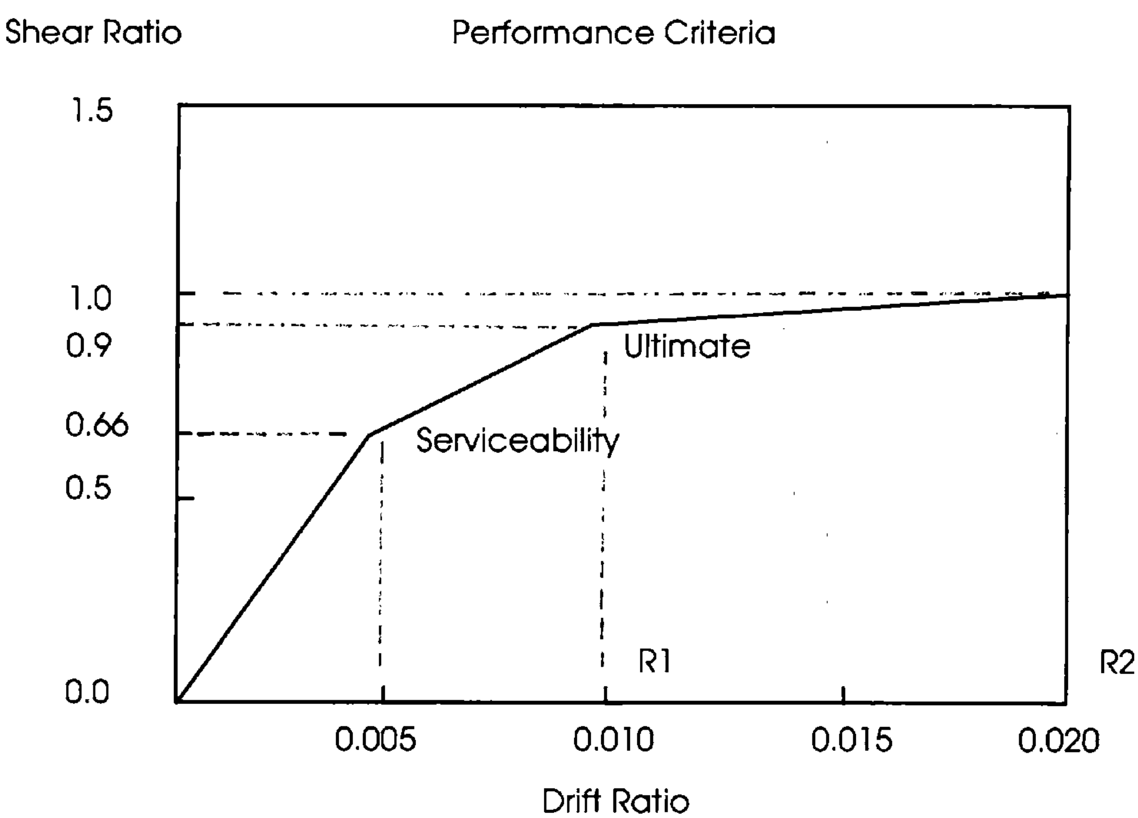


FIGURE 3-1 US/Japan PRESSS Design Guidelines Structural Performance Criteria

3.4. Reliability Implied in Codified Design

From the brief review of the code procedures in the previous sections, it is clear that although the procedures are similar in concept and for some even in format, the base design base shears vary greatly for the same limit state, be it for serviceability or life safety. Also the reduction factors for structural ability to dissipate energy and ductility capability vary widely (by a factor as high as 3) for different structural frames in the same building code as well as for the same type of frame in different building codes. The risk implication is obviously important but difficult to fathom in the code provisions. A larger base shear by itself does not necessarily mean safer design, neither does a stricter drift limit. Since seismic environments and construction practice in these countries are so different and there are so much uncertainties associated with seismic loads, simple comparison of design base shears or drift limits may lead to erroneous conclusions. The only rational method of assessing adequacy of code procedures is to take the loading environments and uncertainties into consideration and measure the degree of satisfactory performance of codified designs by the probability of the buildings against specified limit states over a given time period. Serious investigations in this direction have been started and some recent results are summarized in the following.

3.4.1 Reliability of Steel Buildings Designed According to UBC

A study of the reliability of steel frame buildings designed for seismic loads in accordance with UBC has been recently carried out [Wen et al 1992, Foutch et al 1992]. Six frame types and two sites in California are considered. The emphasis of the study is realistic modeling of the seismic environments of the site, the ground motion as processes with time-varying intensity and frequency content, and the inelastic response behavior of the structures. It represents a serious effort of investigation of the reliability of codified design. The methodology used in the investigation and the results are summarized in the following.

Site Seismic Risk Analysis

Two sites are considered both in Southern California. One of them is at Imperial Valley, 5 km form the Imperial Fault, and the other is at downtown Los Angeles, 60 km from the Mojave Segment of the Southern San Andrea fault (Figure 3-2). The potential future earthquakes that present a threat are characterized as either characteristic or non-characteristic according to recent results of seismological studies. The former are major seismic events which occur along the major fault and with relatively better understood magnitude and recurrence time behavior

(USGS Working Group Report 1988), therefore, treated as a renewal process. The latter are local events that their occurrences collectively can be treated as a Poisson process (Cornell and Winterstein 1988). The major source parameters of the characteristic earthquakes (CE) are magnitude (M), epicentral distance (R) and attenuation, whereas parameters for non-characteristic earthquakes (NE) are local (MMI) intensity, I, and duration, t_d. The duration is defined as the significant duration associated with the strong phased of the ground motion (Trifunac and Brady 1975). The duration is a random variable dependent on M and R for CE and I for NE with additional uncertainty modeled by lognormal variates. Past earthquake records and regression analyses are used to establish the functional relationship and the statistics and probability distributions of the these source parameters.

Modeling of Ground Motion

The future ground motions at the site are modeled as nonstationary random processes whose intensity and frequency content vary with time. The model by Yeh and Wen (1990) is used. The ground motion is obtained by passing a white noise through two linear filters in cascade with time varying coefficients. The general frequency content of the process is controlled by the filter parameters. It generates random processes with a Clough-Penzien (CP) spectral density. Such a spectral form has four parameters allowing modeling of the ground frequency and damping and approaches zero according to ω^4 as ω goes to zero which is in agreement with physics of wave propagation. It has been shown that the instantaneous power spectral density function of the ground motion has a shape of that of the CP spectrum but with the frequency content vary with time. This approach allows easy identification of the model parameters. Figure 3-3 shows the instantaneous power spectral density function of the model identified for non-characteristic earthquake at the Los Angeles site. Note the clear frequency content change with time which is most important when the structure becomes inelastic and the structural period changes. Details can be found in Yeh and Wen (1990).

The above procedure is used for the Imperial Valley site where past records (1940 El Centro and 1979 Imperial Valley Array records) are available and are assumed to be statistically representative of future earthquakes. Also, since the Imperial Valley site is very close to the fault, the directivity effect is important and is considered in the ground motion model which is known to affect significantly the frequency content and duration of the ground motion. For the L. A. site that these conditions do not apply, the model parameters are identified from sites close-by in San Fernando earthquake of 1971 and Whittier earthquake of 1987 are used. The CP spectrum is determined from the ground acceleration Fourier amplitude spectrum as an empirical function of

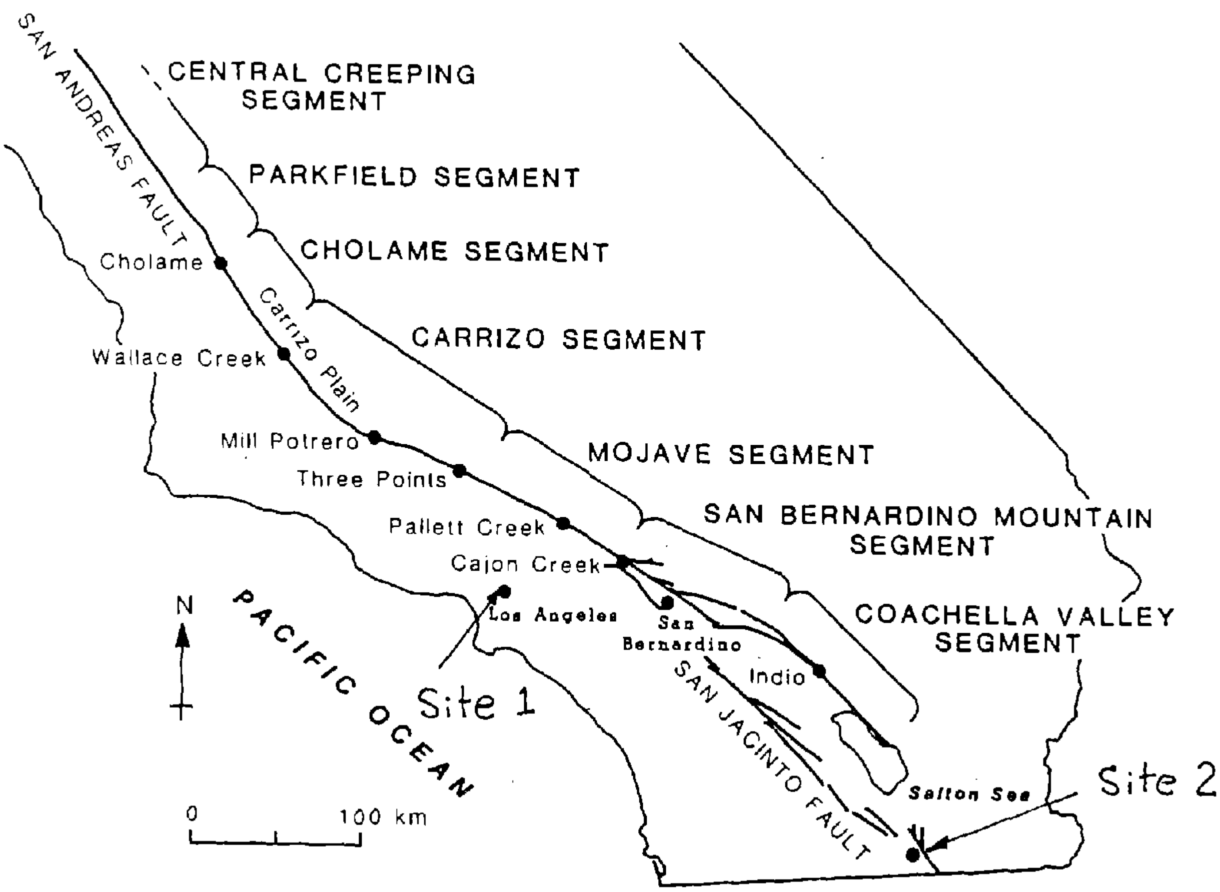


FIGURE 3-2 Two Site at Southern California Segment of Central and Southern Andreas Fault.

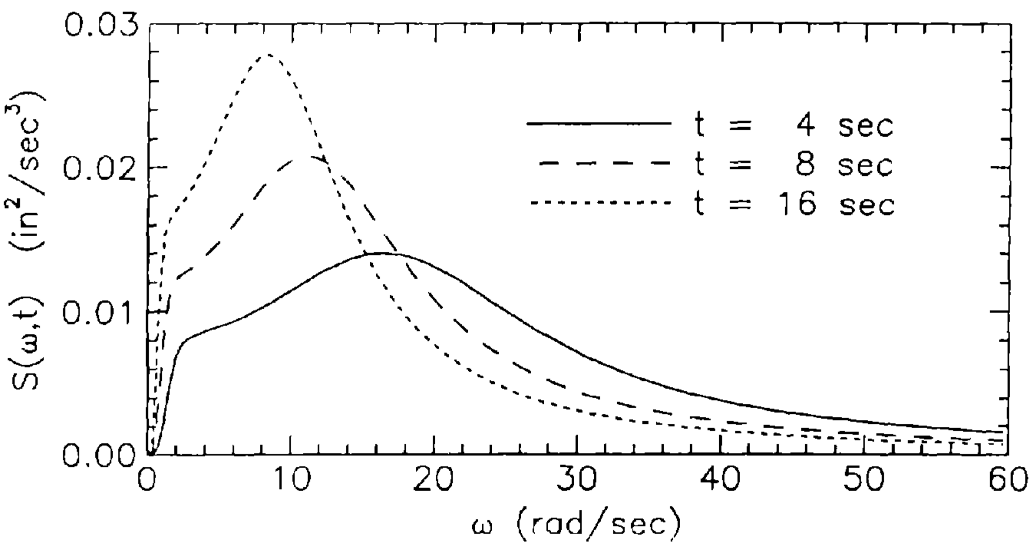


FIGURE 3-3 Instantaneous Ground Acceleration Power Spectral Density at the Los Angeles Site due to Non-Characteristic Earthquakes

the source, path, and site parameters (Trifunac and Lee 1989). This relationship has been established based on regression analyses of a large number of earthquake records in Southern California. The major contribution to the uncertainty is from the attenuation law. There have been a large body of literature dealing with the uncertainty in attenuation. The value used in this study is based on result of survey of recent literature and consideration of excluding variability due to site-to-site variation in attenuation. Details can be found in Wen and Eliopoulos (1991). Therefore knowing the source, path, and site parameters the ground motion random process model parameters can be determined accordingly and the time histories of ground motion can be generated.

It is pointed out that the source and path parameters of future earthquakes are in general random variables, therefore so are the ground motion model parameters. Time histories can be simulated by first generating the source parameters from which the ground motion parameters are obtained and then passing the white noise through the above filters. The resultant time histories are samples of nonstationary random processes in which the uncertainties in intensity, duration and frequency content are properly considered. They, therefore, represent realistically what is going to occur at the site in future earthquakes.

Figure 3-4 shows identified intensity, frequency modulation, and spectral density functions of the ground motion model for the L.A. site due to NE. Note that I(t) depends on the duration and attenuation. Figure 3-5 shows sample ground motion time histories at the two California sites. For the Imperial Valley site, the rupture propagation toward site is assumed in this time history which is known to give rise to long duration pulses most damaging when the structure becomes inelastic.

Building Design

At the two California sites, six low-rise steel buildings types are designed according to UBC; namely, (i) ordinary moment-resisting space frame (OMRSF), (ii) special moment-resisting space frame (SMRSF), (iii) concentric braced frame (CBF), (iv) eccentric braced frame (EBF), (v) dual system with CBF, D/CBF, and (vi) dual system with EBF, D/EBF. The R_W value varies from 6 (OMRSF) to 12 (SMRSF). R_W is the response modification factor and is used to reduce the elastic design forces to account for inelastic behavior. The design should also satisfy the drift limit of $0.03/R_W$ and less than 0.004. One five story building using each of the above framing systems was designed for Zone 4 in accordance with the 1988 UBC. Structural engineers were consulted to ensure that the floor plan and the design loads and procedures would be consistent with those used in practice. A plan view of the building is shown in Figure 3-6(a). Lateral loads are carried by the perimeter frames. All beam-to-column connections at interior joints are assumed to be pinned. An elevation view for the SRMSF and OMRSF frame are shown in Figure 3-6(b) and for the CBF frame in Figure 3-6(c).

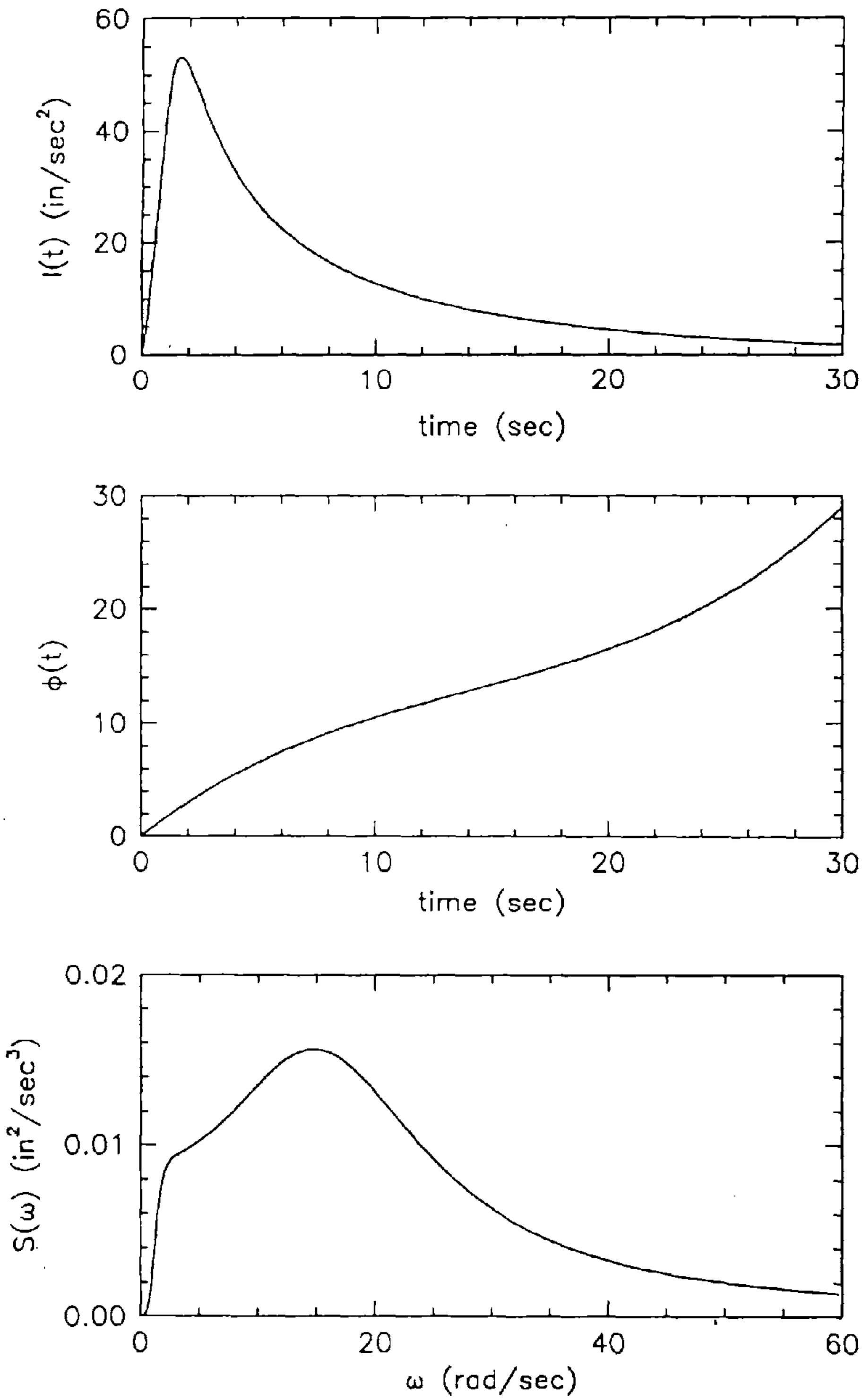
Response and Damage Analysis

Given the occurrence of an earthquake, the response of the building is calculated by both time history and random vibration methods. The forgoing ground motion model provides the ground excitation either in the form of time histories or nonstationary random processes in which the effect of the source parameters and their uncertainties have been properly accounted for. The responses of interest are: (1) story drift, (2) damage to non structural elements, (3) energy dissipation demand, and (4) damage index. In the time history method, the well known finite element program DRAIN 2DX is modified and used for the analysis of the steel frames. Examples of the story shear force-displacement relationship for four steel frames and damage index calculation for the SMRSF frame under the most severe sample ground excitation generated at the Imperial Valley site are shown in Figure 3-7. A damage index of 1.0 at any location indicates failure of the connection by low-cycle fatigue. These results indicate that failures of well-constructed connections of a steel frame are not likely to be a problem during earthquakes.

In the random vibration analysis, the time domain approach for an inelastic system (Wen,1989) is used. It gives response statistics of interest such as maximum interstory displacement and hysteretic energy dissipation. For the SMRSF frame a strong-column and weak-beam (SCWB) is developed which localizes inelastic behavior at the base and the floor level at the beams. It allows lateral displacement and floor rotation. The hysteretic restoring moments are described by the smooth differential equation model which allows solution by the equivalent linearization method. Comparison between the results for different acceleration records by the SCWB and DRAIN 2DX indicate that the former reproduces well the inelastic response behavior of the SMR frame.

Limit State Risk Evaluation

The random vibration analyses of the structural response provide the conditional statistics and probability of limit states being reached given the occurrence of the earthquake and the parameters of the source, path, site and ground motion. These parameter are known to have



Intensity Function Parameters

$A = 996.74$
$B = 2.867$
$C = 0.124$
$D = 4.274$
$E = 4.119$

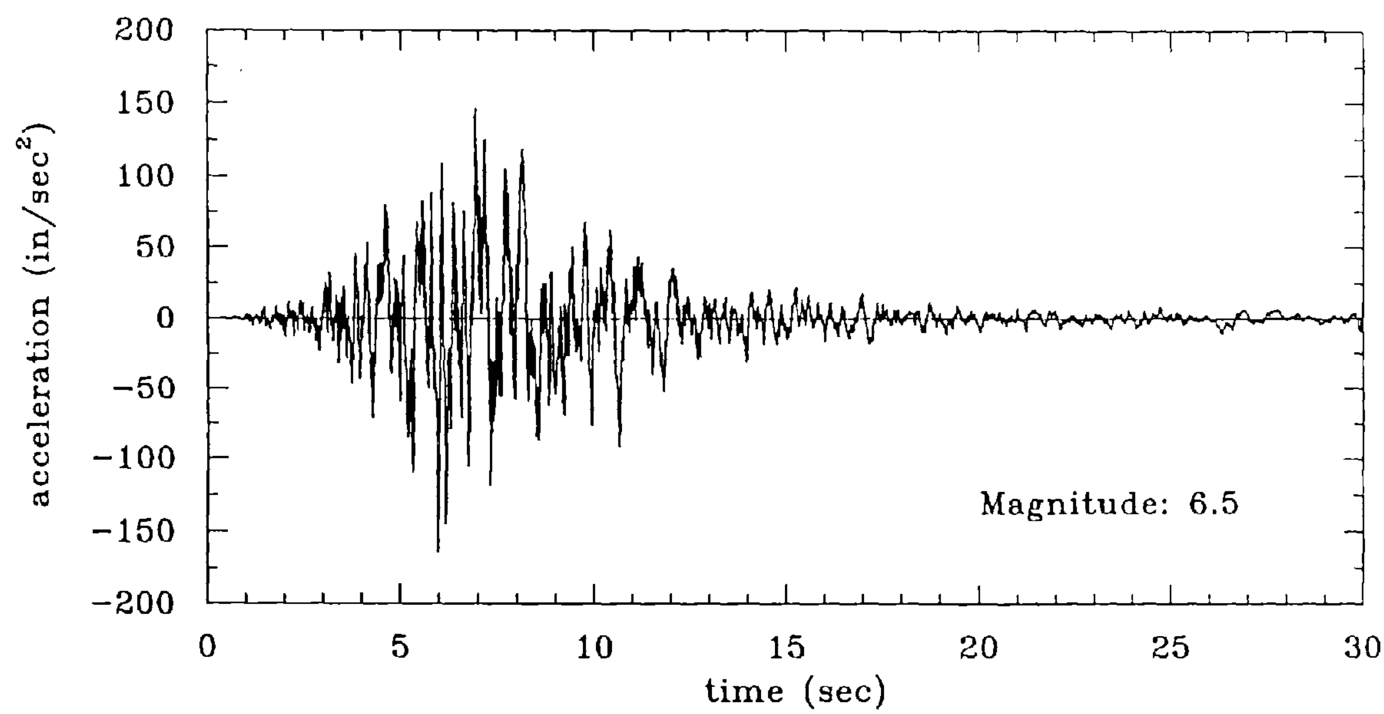
Frequency Modulation Function Parameters

$r_1 = 1.659$
$r_2 = -0.079$
$r_3 = 0.0019$

Normalized PSD Function Parameters

$S_0 = 0.0087$
$\omega_g = 18.16$
$\zeta_g = 0.62$
$\omega_f = 1.35$
$\zeta_f = 0.66$

FIGURE 3-4 Parameters and Functions of the Ground Motion Model for The Los Angeles Site due to Non-Characteristic Earthquakes.



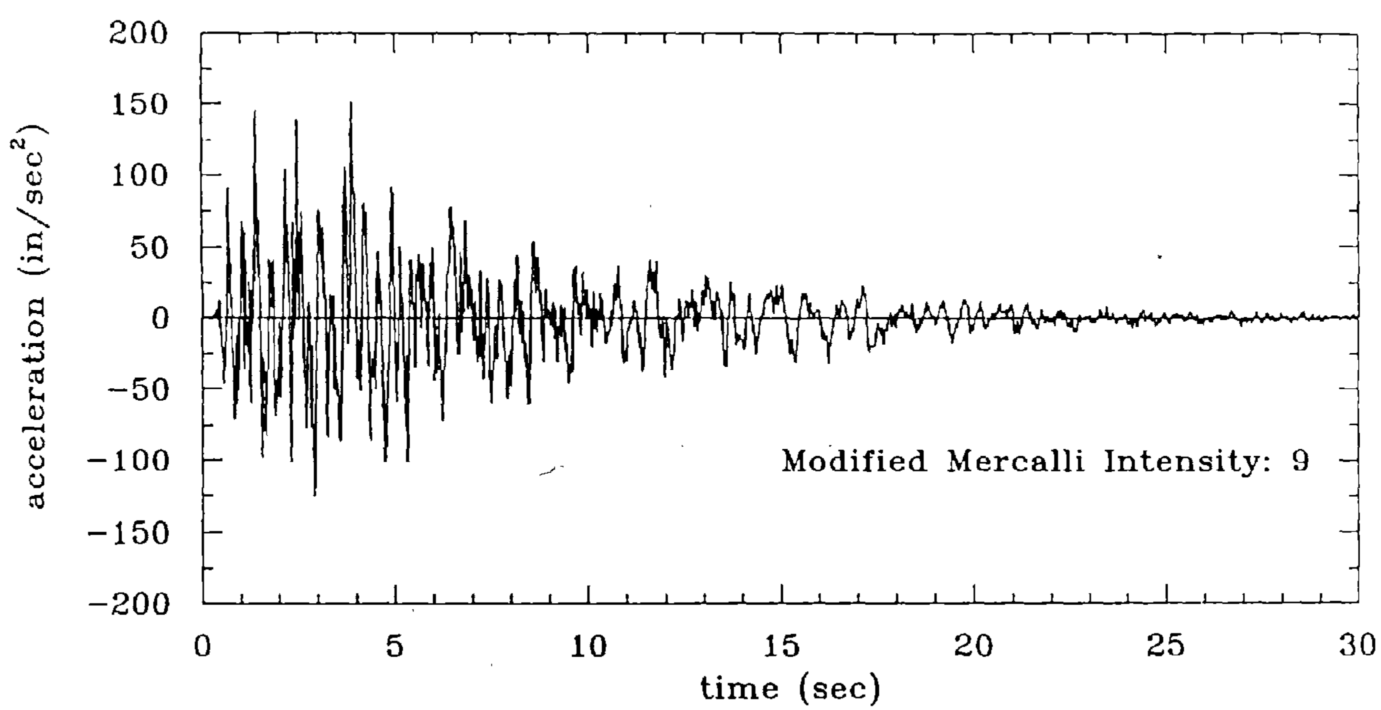


FIGURE 3-5 Ground Motion Time Histories Generated by The Ground Motion Model: Characteristic Earthquake at The Imperial Valley Site (top) and Non-Characteristic Earthquake at the Los Angeles Site (bottom).

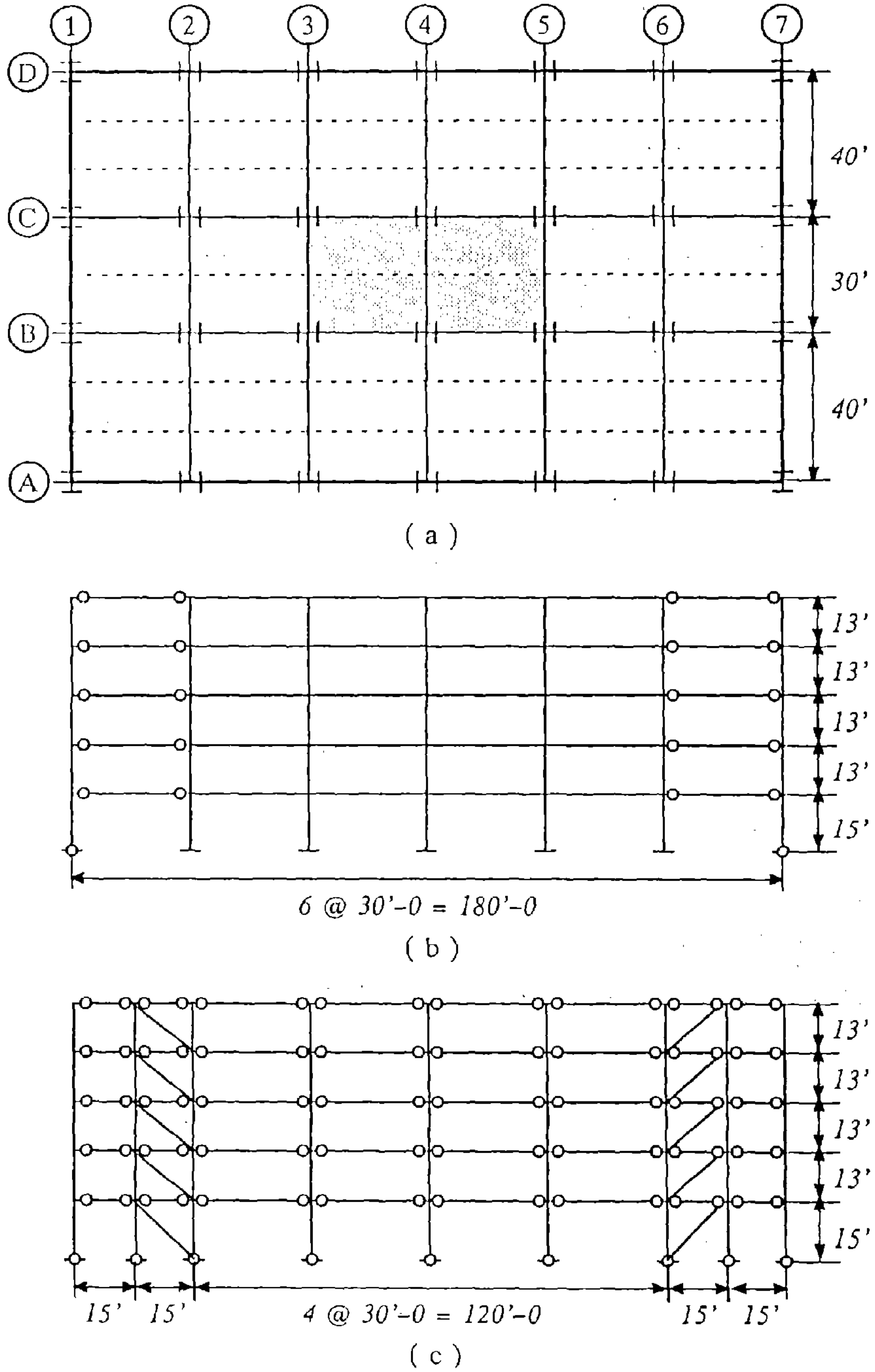


FIGURE 3-6 Design of Steel Frame Buildings: (a) Plan View of Building, (b) Elevation of SMRSF and OMRSF, and (c) Elevation of CBF.

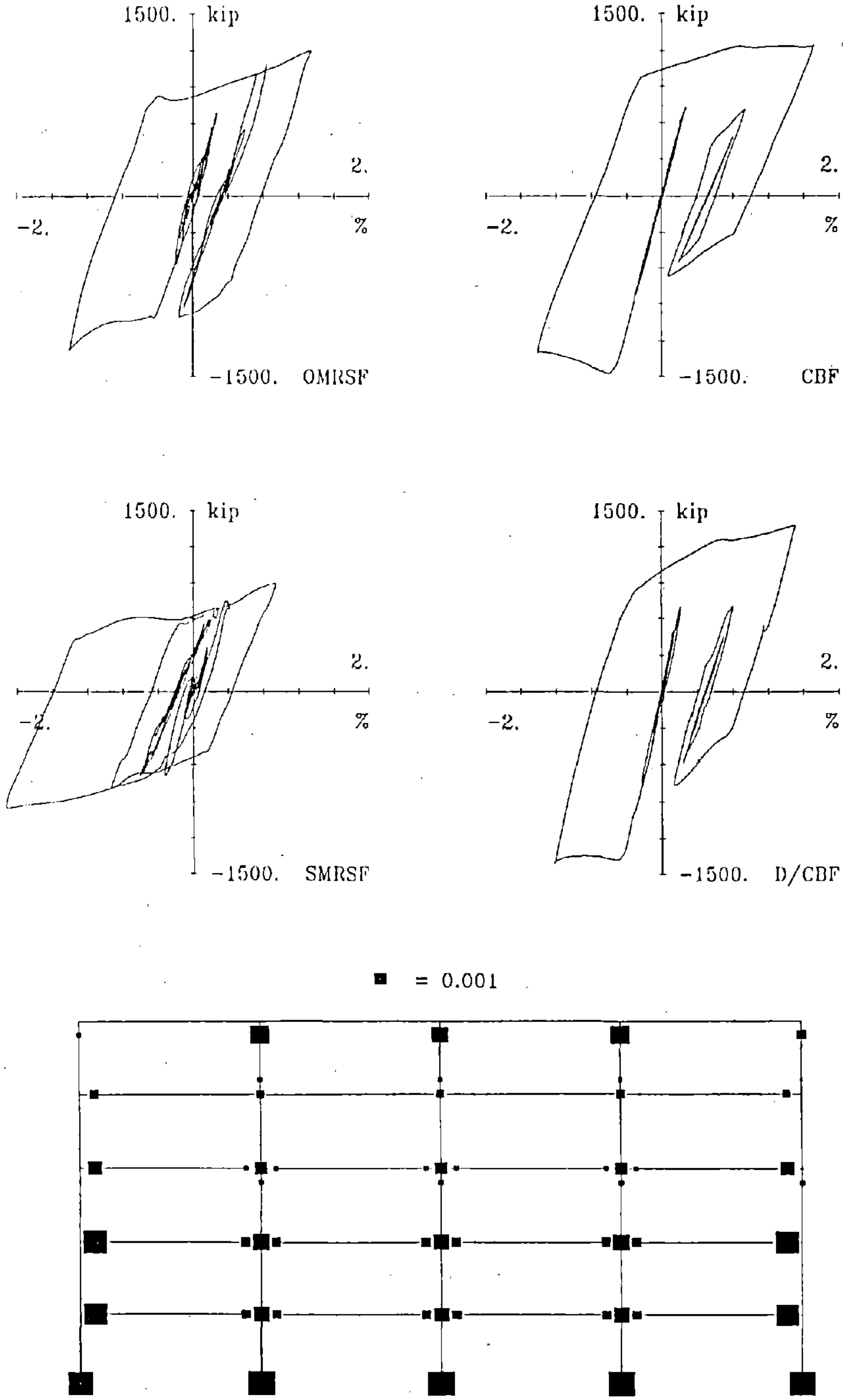


FIGURE 3-7 First Story Shear Force-Displacement Relationship for Four Steel Frames (top) and Damage Index of Connections of SMRSF (bottom) under the Most Severe Ground Motion Generated at the Imperial Valley Site.

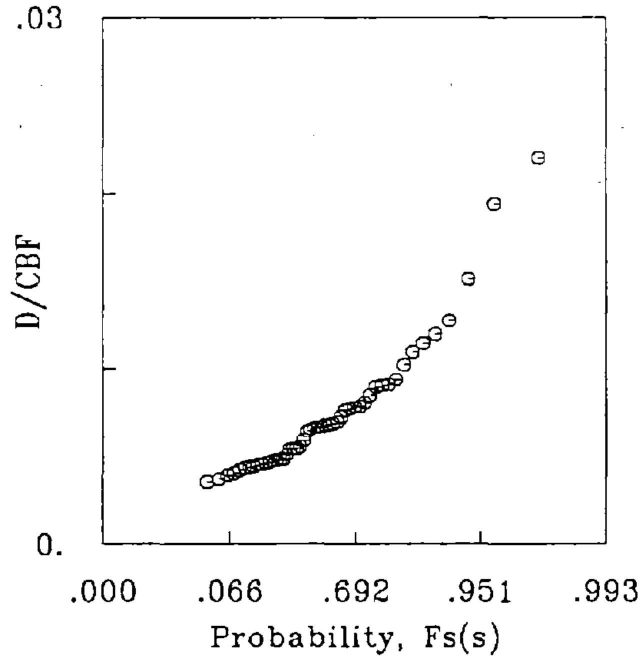
large variability's and often are correlated. These parameters generally play an important role in the overall risk evaluation; are therefore treated as random variables. A literature survey has been carried out (e.g. USGS Report 1988; Joyner and Boore 1988) and statistics and distribution models developed based on the survey results. The uncertainties in the structural resistance such as those in yielding strength are assumed to be small compared to the uncertainties in ground motion therefore were not considered in this study. To include these parameter uncertainties into the risk analysis, a fast integration technique (Wen and Chen, 1986) based on the first order reliability method is used. In the time history/simulation approach, these parameters have been randomized according to their distributions and incorporated into the simulated ground motions time histories. Figure 3-8 shows a plot of maximum first story drift for OMRSF and CBF frames the Imperial Valley site on type I extreme values probability papers given the occurrence of a characteristic earthquake.

The risks of limit states in terms of interstory drift thresholds being exceeded are evaluated for a time window of 50 years. At the Imperial Valley Site, the major threat is characteristic earthquakes of magnitude 6.5. A 50 % probability of fault rupture propagation toward or away from the site is assumed. The results are found to be not particularly sensitive to this assumption. At the L.A. site, both types of earthquake contribute. The CE, though of a larger magnitude of 7.5, contribute less than the NE primarily due to the distance (60 km) from the site to the Mojave segment. The random vibration / fast integration method gives comparable results. The earthquake occurrence probabilities are then evaluated from either a renewal process or a Poisson process. The results are then combined to arrive at the risk of limit states being exceeded as function of the length of the time window considered and the dormant period since the last CE. The last event occurred in 1979 at the Imperial Valley fault and in 1867 at the Mojave segment of the San Andrea fault.

Table 3-II shows the drift ratio (% of story height) being exceeded corresponding to various probability levels for the next 50 years at the two California sites for the four steel buildings. If the time dependence in CE occurrences is neglected, the drifts in first and last rows corresponds to approximately annual probabilities of 10^{-2} and 10^{-3} respectively. Comparisons of performance curves of these steel buildings in terms of probability of the local maximum interstory drifts are also shown in Figures 3-9 and 3-10. Note that although both sites are in Zone 4, at equal probability levels, response are much higher at the Imperial Valley site. On the other hand , the much larger spreads of the drift levels for different probability levels at the L.A. site indicate that the variability in the response is much larger. It is mainly due to the large

Table 3-II
Interstory Drift (% of story height) Level of Steel Buildings According to UBC Corresponding to an Exceedance Probability in 50 Years at Two Sites in California.

Site	Frame	Prob (%)	Story 1	2	3	4	5
Los Angeles (LA)	OMRSF	50	.18	.29	.31	.31	.34
		25	.28	.44	.48	.47	.52
		10	.45	.69	.75	.77	.87
		5	.66	1.01	1.11	1.18	1.34
(LA)	SMRSF	50	.22	.34	.39	.39	.39
		25	.35	.52	.60	.59	.59
		10	.63	.89	1.02	1.01	1.00
		5	1.00	1.38	1.60	1.59	1.54
(LA)	CBF	50	.17	.21	.25	.29	.30
		25	.24	.30	.36	.43	.45
		10	.39	.47	.57	.70	.78
		5	.58	.69	.84	1.06	1.22
(LA)	D/CBF	50	.14	.24	.25	.26	.25
		25	.21	.34	.36	.39	.36
		10	.33	.54	.55	.62	.60
		5	.48	.79	.81	.93	.91
Imperial Valley (IV)	OMRSF	50	.64	.90	.96	1.03	1.20
		25	.89	1.15	1.22	1.28	1.42
		10	1.16	1.43	1.51	1.54	1.65
		5	1.36	1.63	1.71	1.72	1.81
(IV)	SMRSF	50	.76	.97	1.07	1.10	1.4
		25	1.04	1.25	1.36	1.38	1.70
		10	1.34	1.56	1.69	1.70	2.03
		5	1.55	1.78	1.93	1.92	2.26
(IV)	CBF	50	.59	.64	.77	.98	1.00
		25	.87	.90	1.01	1.26	1.23
		10	1.26	1.21	1.27	1.57	1.47
		5	1.54	1.44	1.45	1.78	1.63
(IV)	D/CBF	50	.47	.75	.82	.88	.82
		25	.70	1.02	1.09	1.09	.98
		10	1.01	1.36	1.39	1.33	1.16
		5	1.23	1.59	1.60	1.49	1.28



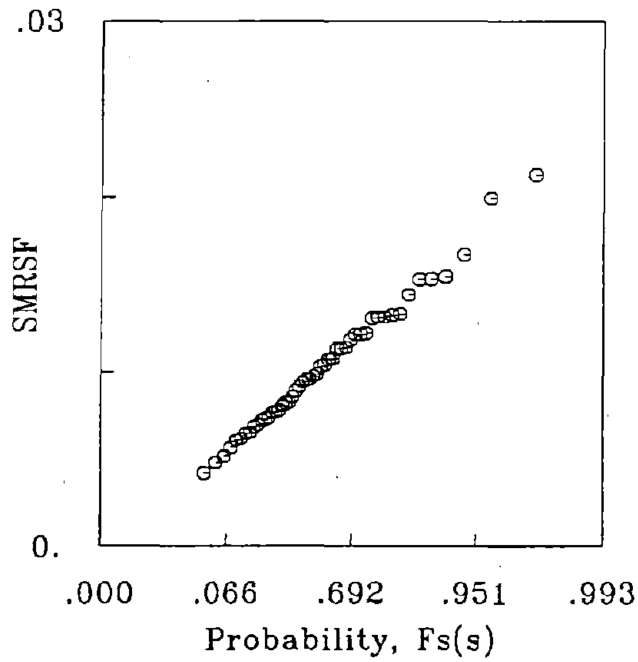


FIGURE 3-8 Maximum First-Story Drift for 50 Imperial Valley Accelerograms Generated by the Ground Motion Model.

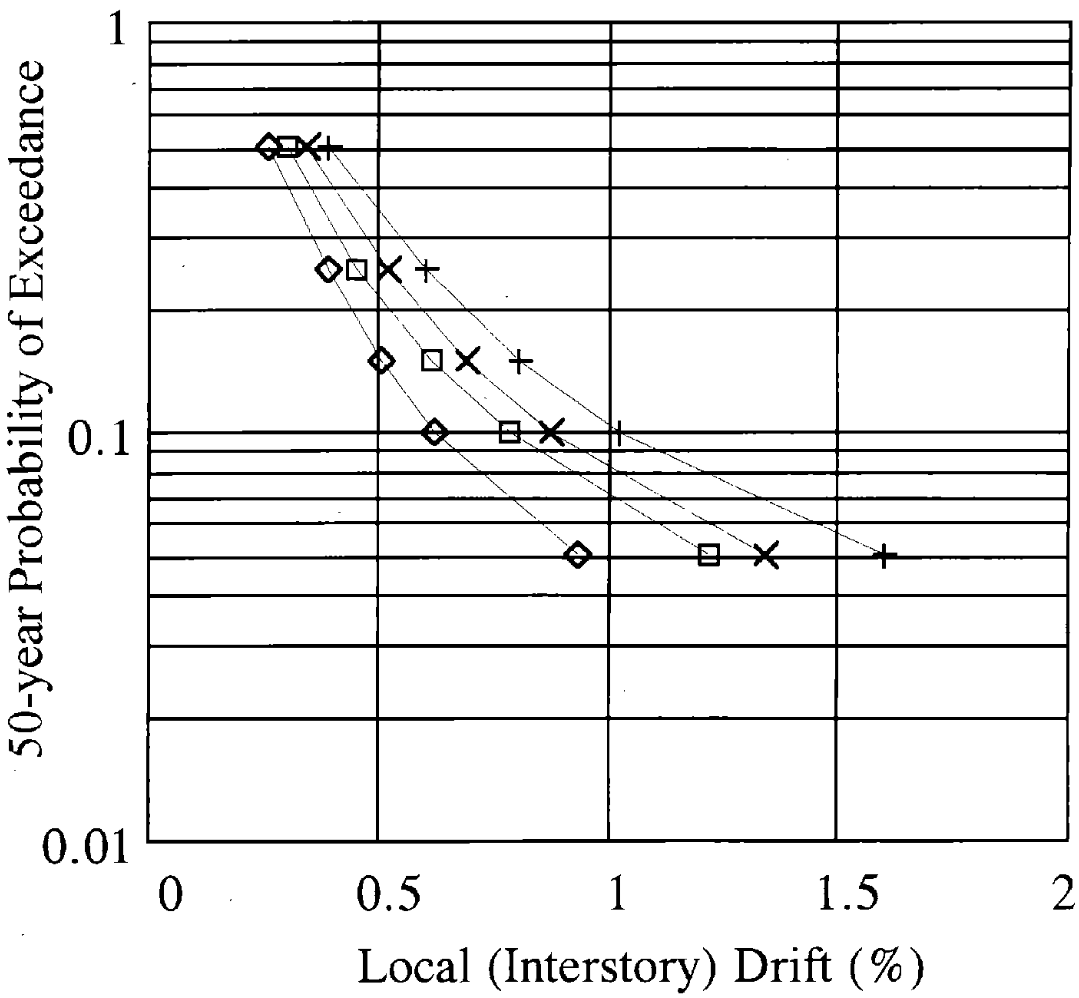


FIGURE 3-9 Comparison of Performance Curves of 5-Story Steel Buildings of Different Designs at Downtown Los Angeles: SMRSF(+), OMRSF(x), CBF(box), D/CBF(diamond)

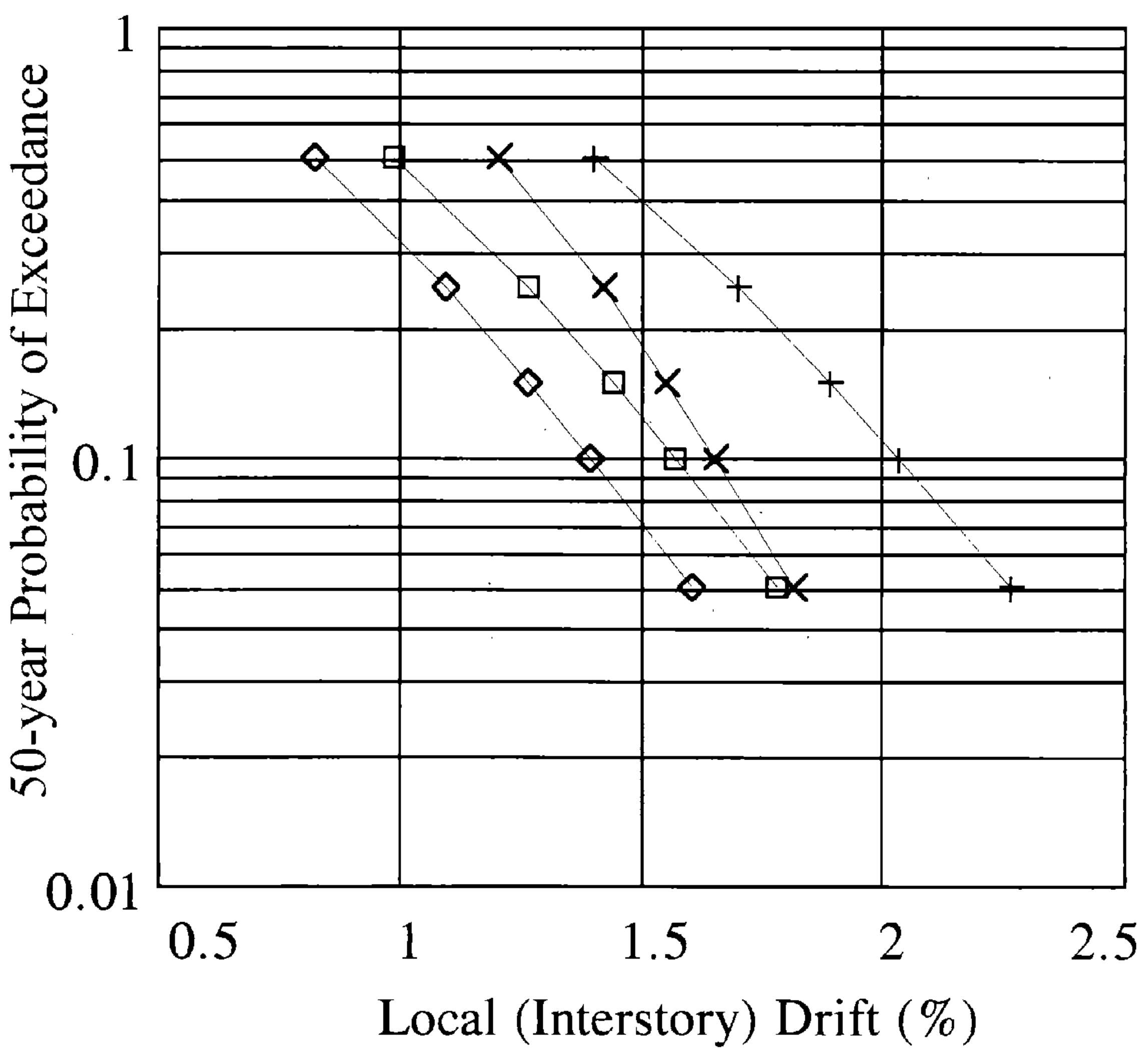


FIGURE 3-10 Comparison of Performance Curves of 5-Story Steel Buildings of Different Designs at Imperial Valley Site: SMRSF(+), OMRSF(x), CBF(box), D/CBF(diamond)

uncertainty in the intensity of NE and the attenuation for CE at the L.A. site. As expected the braced frame and dual system give lower responses; particularly the dual system, in spite of the large R_w factor value (12) used in design for such system. The differences in performance curves can be attributed more to the difference in structural frame systems than the different R_w values used in design. It is seen from Table 3-II that there is a trend of increasing story drift with height. At the level of an annual probability of 10^{-3}, some exceed 2 % at the upper stories. For steel frames, such high drift levels may not yet cause concerns of collapse but would certainly mean significant damage to contents, claddings, or even structural frame.

Concluding Remarks

The results obtained are based on the assumption that moment connections will function as ductile members as designed. The recent Northridge earthquake findings have not been incorporated in this study. The results indicate the steel frame buildings according to UBC will provide adequate life safety against major earthquakes. These buildings, however, are quite flexible particularly at upper stories. For example, the 0.4 % interstory drift limit for serviceability will be exceeded with an annual probability of about 3×10^{-3} for all frames at the Los Angeles site and with an annual probability of 10^{-2} for all frames at the Imperial Valley site. The 1.5 % limit specified in UBC for maximum interstory drift has a risk of 10^{-3} of being exceeded per year for the special moment frame at the upper stories at the Los Angeles site and for all frames at the Imperial Valley site. They reflect the reliability implied in the current version of UBC. An interstory drift of 2 % may still be far from collapse for steel frame buildings but serious content damage, nonstructural and structural damage may result from such large distortion. The results also indicate that the risks of different drift limit states show variation among the four different types of steel frames which are consistent with the design philosophy and appear to be reasonable. There is, however, greater discrepancy in the risks and response levels for the two California sites considered, both in Zone 4 of the UBC designation indicating a dominance of the effect of site seismicity. Additional uncertainties which need to be investigated are those in structural resistance, in particular, those due to nonstructural components (partition walls and claddings) (e.g. Foutch et al 1986).

3.4.2 Reliability of Reinforced Concrete Buildings Designed According to NEHRP Provisions

In this study, two four-story reinforced concrete (RC) frame structures, one special moment-resisting (SMR) frame and one intermediate moment-resisting (IMR) frame, are designed according to the NEHRP Provisions (1988) and ACI code 318-83 (1983). The reliability analysis method developed by Hwang and Hsu (1990) is then used to estimate the reliability of these two structures.

Design of Four-Story Building

The building selected for this study is a four-story school building. A typical floor plan and an elevation of the building are shown in Figures 3-11 and 3-12, respectively. This study focuses on the design of a typical interior frame in the north-south direction. The moment-resisting frames are used to resist both gravity (dead and live) and earthquake loads.

Special Moment-Resisting Frame

For the first case, special moment-resisting frames are used to provide seismic resistance. The beam size is 12 in. by 18 in. and the column size is 14 in. by 14 in. throughout the building. The design base shear V specified in the 1988 NEHRP Provisions is:

$$V = C_S W \tag{3.6}$$

and

$$C_S = \frac{1.2 A_V S}{R T^{2/3}} \tag{3.7}$$

where C_S = seismic design coefficient; W = total seismic dead load; A_V = effective peak velocity-related acceleration coefficient; S = soil coefficient; R = response modification factor and T = fundamental period of a building. The building is assumed to be located in a moderate seismic hazard area where $A_V = 0.2$. For SMR frame structures, the response modification factor R is 8 according to the NEHRP provisions. The site condition is assumed to be rock; thus, the S factor is 1.0. The total seismic weight of the building W is calculated as 3153.5 kips, and the fundamental period is estimated as 0.547 sec. Using these values, the design base shear V is determined as 141.6 kips. The design base shear is then distributed over the height of the structure and divided equally among six frames in the N-S direction to determine lateral forces acting on the floor levels and member forces caused by these lateral forces.

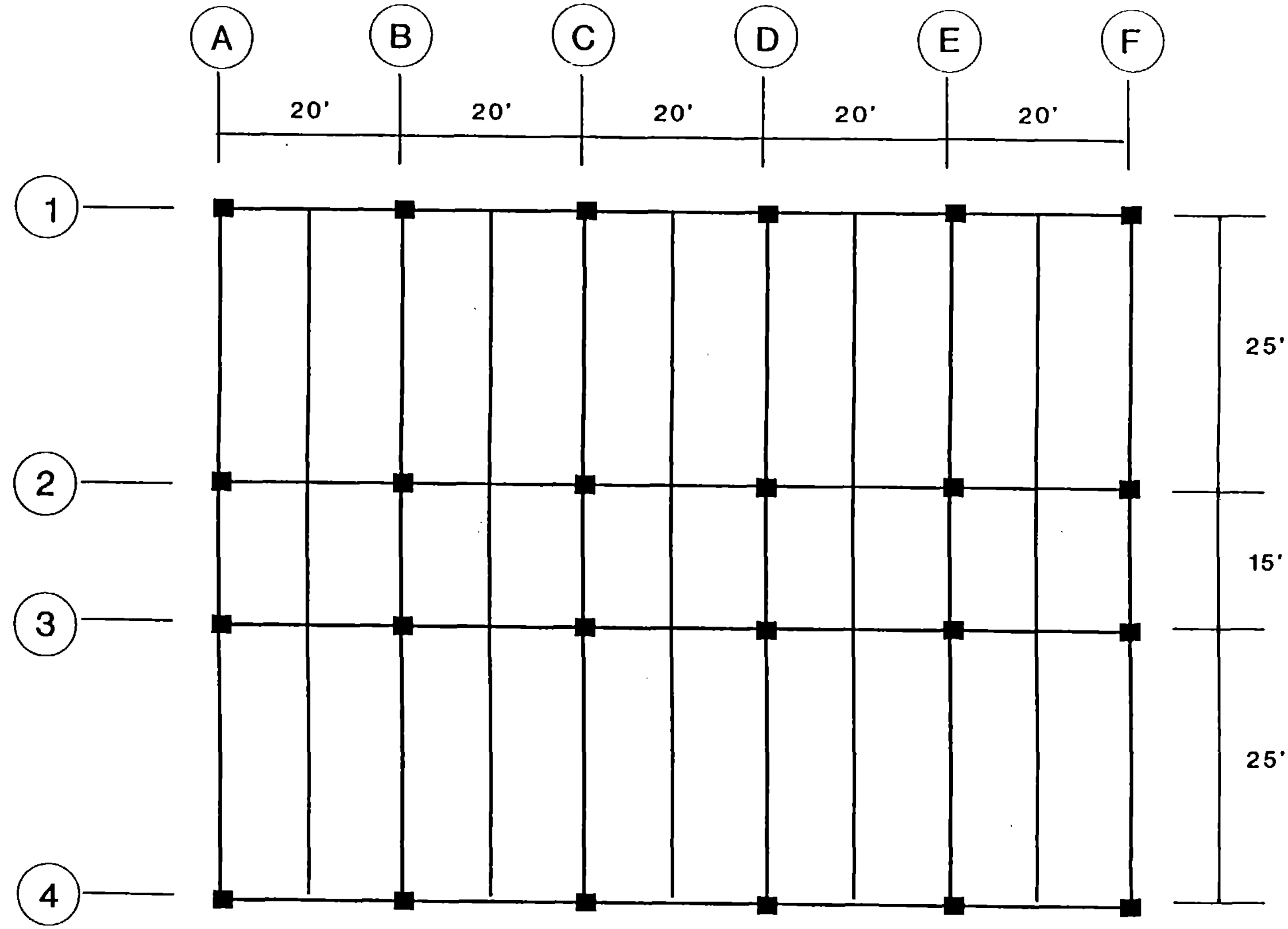


FIGURE 3-11 Typical Floor Plan of a 4-Story Reinforced Concrete Building

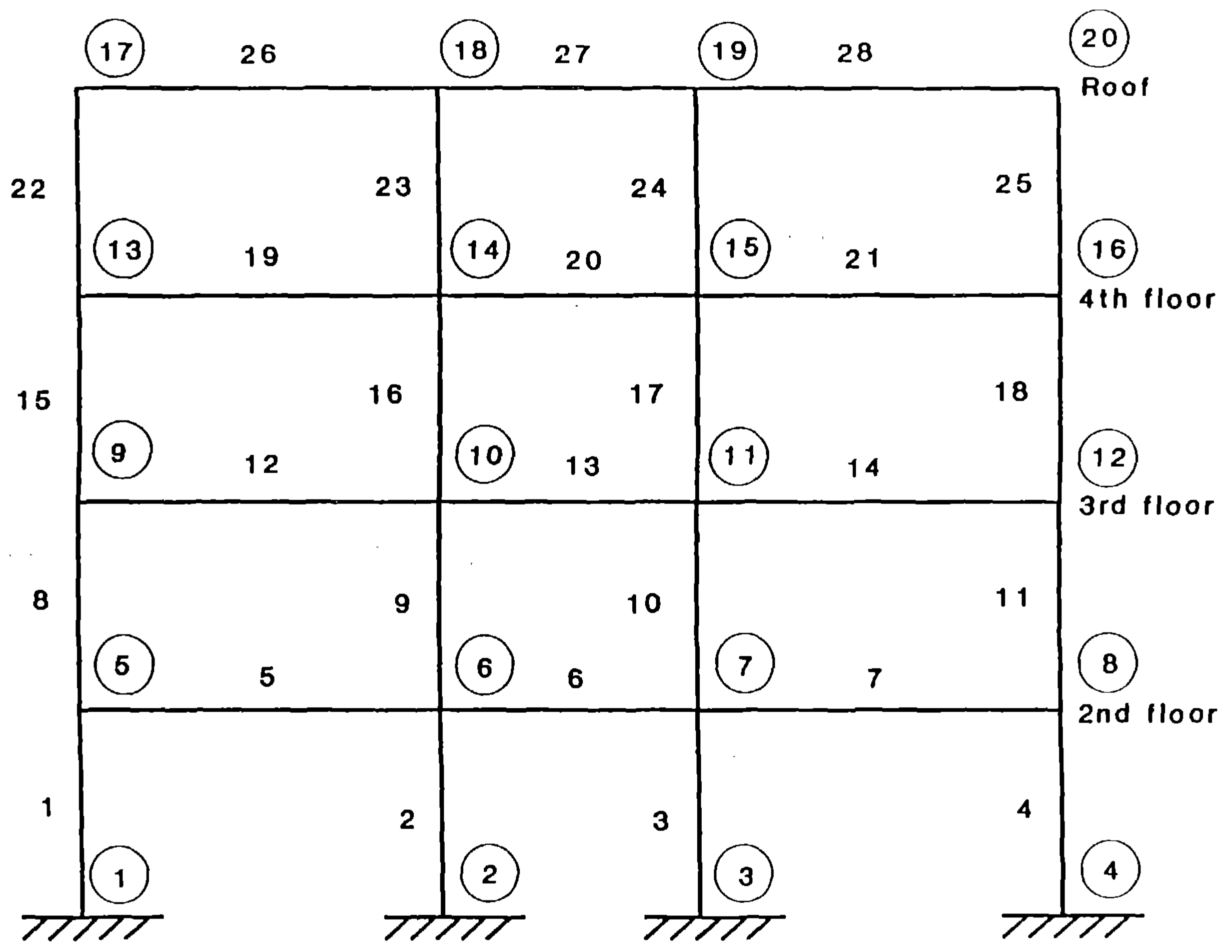


FIGURE 3-12 Elevation of an Interior Frame (N-S Direction)

For $A_v = 0.2$, the following combinations of load effects are used for the design of buildings:

$$1.2\,Q_D + 1.0\,Q_L + 1.0\,Q_E \tag{3.8}$$
$$1.2\,Q_D + 1.0\,Q_L - 1.0\,Q_E \tag{3.9}$$
$$0.8\,Q_D + 1.0\,Q_E \tag{3.10}$$
$$0.8\,Q_D - 1.0\,Q_E \tag{3.11}$$

where Q_D = dead load effect; Q_L = live load effect and Q_E = earthquake load effect. Using member forces from combined load effects, the SMR frame is designed in accordance with Appendix A and Chapters 1-19 of ACI code 318-83. The detail of design including strong-column-weak-beam requirement is shown in Hwang and Hsu (1990).

Intermediate Moment-Resisting Frame

The same building is also designed using IMR frames. The beam size is 12 in. by 16 in. and the column size remains the same as 14 in. by 14 in. The design base shear V is also calculated according to Eqs. (3.6) and (3.7). For IMR frames, the response modification factor R is 4 as specified in the NEHRP Provisions. The total seismic dead load W of the building is calculated as 3041.5 kips. Besides R and W, other parameters are the same as those used for the SMR frame. From Eqs. (3.6) and (3.7), the design base shear is determined as 273.2 kips. For design of the IMR frame, Chapters 1-19 and Appendix A-9 of ACI code 318-83 shall be considered. The detail of design is also shown in Hwang and Hsu (1990).

Reliability Analysis of Frame Structures

A reliability analysis method for evaluating the reliability of frame structures has been developed. (Hwang and Hsu 1990). In this method, seismic hazard, limit state, nonlinear structural response, etc. are integrated to provide an overall view of seismic performance of structures.

Seismic Hazard Curve

According to Hwang et al. (1987), a representative seismic hazard curve for the eastern United States can be expressed as:

$$G_A(a) = 1 - \exp[\,-(a/\mu)^{-\alpha}\,] \tag{3.12}$$

where α and μ are two parameters. α is estimated as 2.7 and μ is determined as 0.0204 for the case where the PGA corresponding to a 10% probability of exceedance in 50 years is 0.2g. Figure 3-13 shows the representative seismic hazard curve for the eastern United States.

Limit States

Two limit states are considered in this study, first yielding and collapse of a structure. For a frame structure, the first yielding is defined as the formation of first plastic hinge anywhere in the structure. If a structure subject to earthquakes does not reach the first yielding, the structure will not sustain any structural damage. The collapse of a structure is defined as the formation of a failure mechanism. The collapse limit state represents an ultimate strength limit state.

Probabilistic Structural Capacity

For a structure properly designed in accordance with ACI code, the shear capacity is greater than the flexural capacity and thus the members are expected to fail in flexure. In this study, the flexural and shear capacities of all the members are calculated and the shear capacities are greater than the flexural capacities, as intended. Therefore, the flexural capacities of beams and columns are used to represent the actual capacities of structural members.

The structural capacity is affected by variation in material strength, structural geometry, quality of workmanship, etc. Thus, a probabilistic model is used to describe the actual capacity of a structure. In this study, the probabilistic structural capacity is taken to be lognormally distributed, which is defined by two parameters: a median value $\widetilde{SA}_C$ and a logarithmic standard deviation β_C. In other words, SA_C is expressed by:

$$SA_C = LN\,(\,\widetilde{SA}_C,\ \beta_C\,) \qquad (3.13)$$

The median value in terms of the spectral acceleration is determined by means of the capacity spectrum method (Freeman 1978). In this method, the structural capacity of a frame structure is determined from the sequential formations of plastic hinges on structural members by gradually increasing the lateral forces applied to the structure. To obtain the actual strength of a structure, the mean material strength instead of the nominal strength is used. The capacity curves for both SMR and IMR frames are shown in Figure 3-14. Table 3-III summarizes the probabilistic structural capacities of the SMR and IMR frame structures.

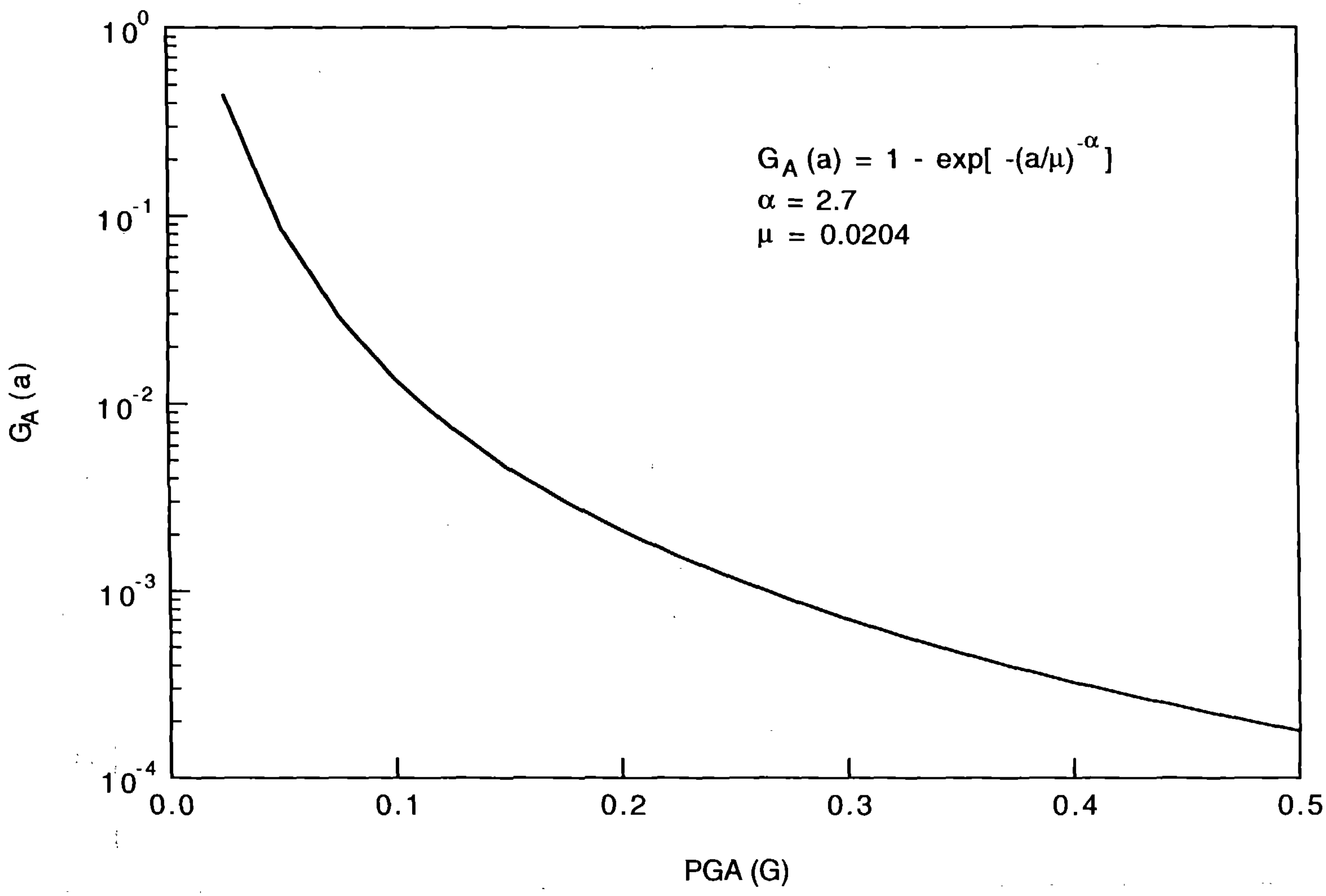


FIGURE 3-13 Seismic Hazard Curve

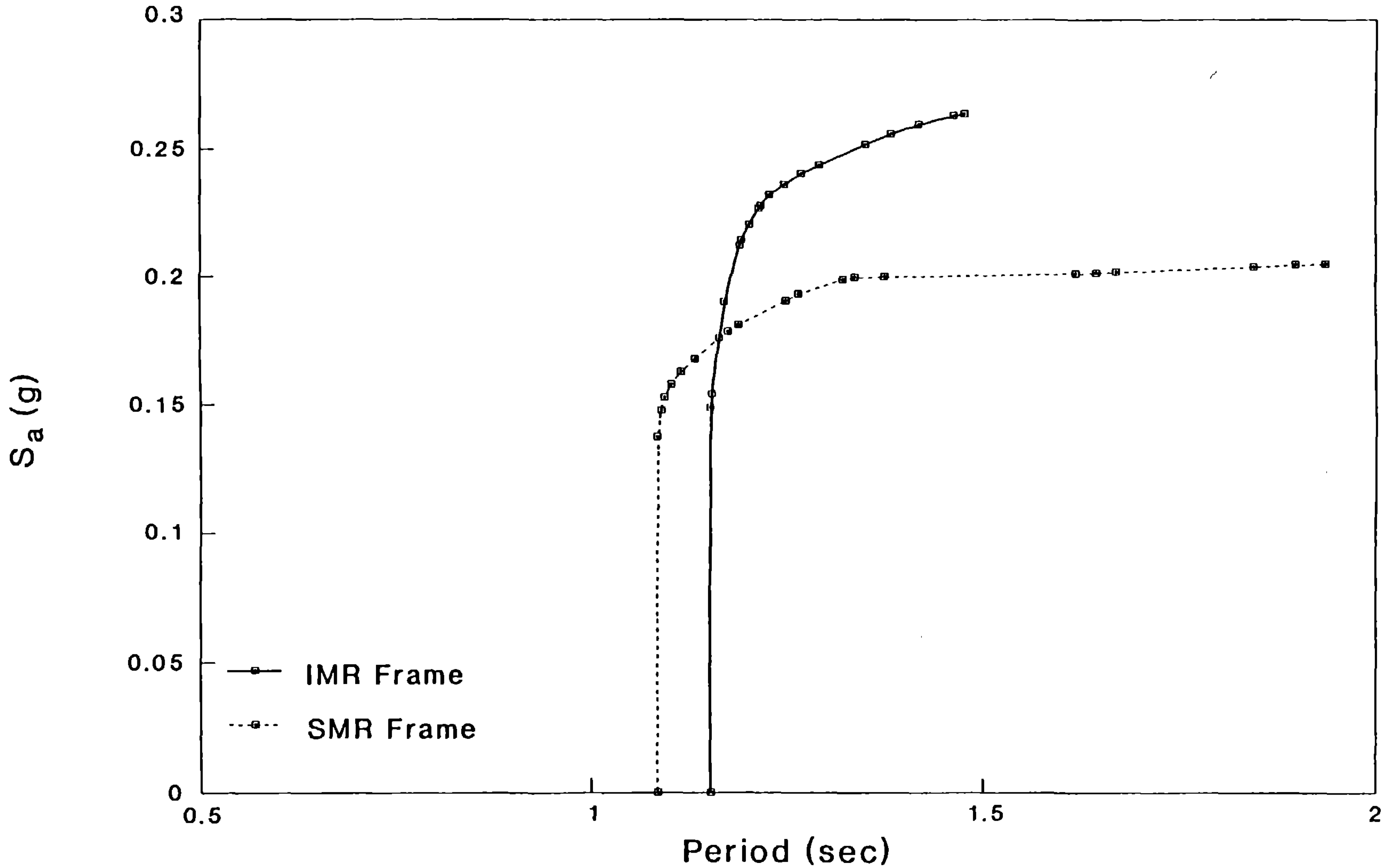


FIGURE 3-14 Capacity Curves of SMR and IMR Frames

Probabilistic Structural Response

The seismic structural response is mainly affected by seismic source, path attenuation, soil conditions, and structural properties such as damping and structural period. The probabilistic structural response SA_R is also described using a lognormal distribution:

$$SA_R = LN(\widetilde{SA}_R, \beta_R) \tag{3.14}$$

where β_R is the logarithmic standard deviation of structural response and $\beta_R = 0.5$ is used in this study; $\widetilde{SA}_R$ is the median value of the spectral acceleration and it is further expressed as:

$$\widetilde{SA}_R = A_p \times \widetilde{SA}_n \tag{3.15}$$

where A_p is the value of the peak ground acceleration (PGA), and $\widetilde{SA}_n$ is the median normalized spectral acceleration at the period corresponding to a limit state and is determined from the response spectra specified in the Tri-Services Guidelines (1986). Table 3-IV summarizes the probabilistic structural response for both SMR and IMR frame structures.

Fragility Analysis

For a given level of peak ground acceleration, the conditional limit-state probability P_f is the probability that the structural response SA_R exceeds the structural capacity SA_C. If both SA_R and SA_C are lognormally distributed, P_f can be determined as:

$$P_f = \Phi\left[\frac{-\ln(\widetilde{SA}_C/\widetilde{SA}_R)}{(\beta_C^2 + \beta_R^2)^{1/2}}\right] \tag{3.16}$$

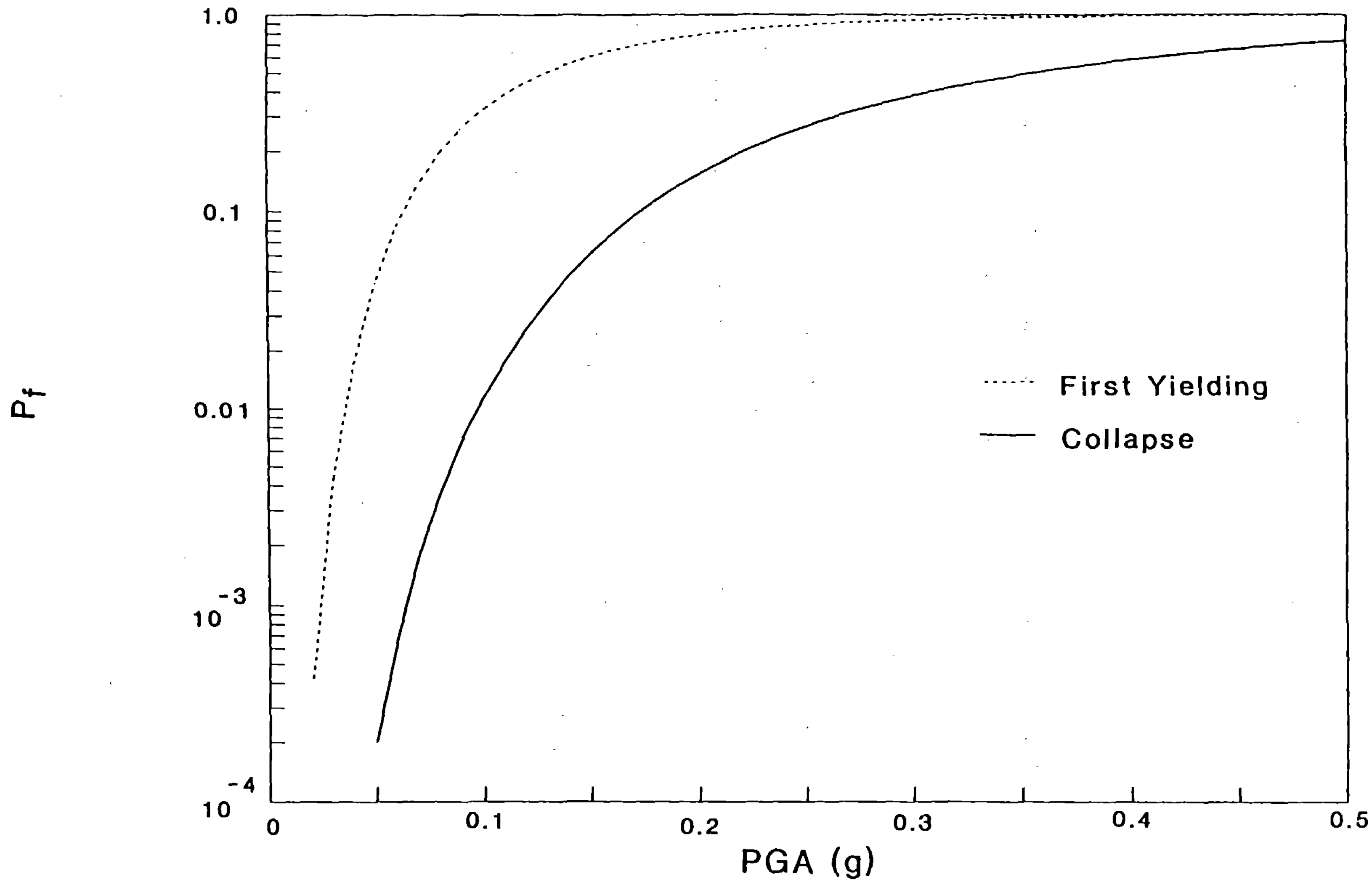
where $\Phi[\cdot]$ is the standard normal distribution. For the SMR and IMR frame structures, the conditional limit-state probabilities corresponding to both first-yielding and collapse limit states at various PGA levels are calculated and plotted as fragility curves. Figure 3-15 shows the fragility curves for the SMR frame structure. The limit state probability increases as the PGA level becomes higher (an expected trend).

Table 3-III
Probabilistic Structural Capacity

Frame	Limit State	$\tilde{SA}_C$ (g)	β_C
SMR	First Yielding	0.137	0.3
	Collapse	0.205	0.3
IMR	First Yielding	0.149	0.3
	Collapse	0.263	0.3

Table 3-IV
Probabilistic Structural Response

Frame	Limit State	$\tilde{SA}_R$ (g)	β_R
SMR	First Yielding	$1.137A_p$	0.5
	Collapse	$0.618A_p$	0.5
IMR	First Yielding	$1.092A_p$	0.5
	Collapse	$0.741A_p$	0.5



FIGURES 3-15 Fragility Curves of SMR Frame

Annual limit-state probability

The annual (unconditional) limit-state probability PF of a structure with respect to a limit state is determined as :

$$PF = \sum_{j=1}^{N} \lambda a_j \times P_f(a_j) \tag{3.17}$$

where λa_j is the annual occurrence of an earthquake with a specified peak ground acceleration a_j. Given a seismic hazard curve, λa_j can be determined as follows:

$$\lambda a_j = F_A(a_j + \frac{\Delta a}{2}) - F_A(a_j - \frac{\Delta a}{2}) \tag{3.18}$$

where $F_A(\cdot)$ is the probability distribution of PGA; $\Delta a = (a_{max} - a_o)/N$ and N is the number of intervals between a_o and a_{max}. a_o is the minimum PGA for any ground shaking to be considered as an earthquake and is taken as 0.02g. a_{max} is the maximum PGA possible at a site. For first yielding as the limit state, a_{max} is taken as the PGA value corresponding to 100-year return period. For collapse of a structure as the limit state, a_{max} is set as the PGA value corresponding to 2000-year return period. The annual limit-state probabilities for both the SMR and IMR frames are determined and shown in Table 3-V. It is pointed out that the collapse probability is based on static spectral analysis of the structural capacity and not dynamic response analysis.

Table 3-V
Annual Limit State Probabilities

Earthquake	Limit State	SMR (/yr)	IMR (/yr)
Moderate	First Yielding	1.38×10^{-2}	9.28×10^{-3}
Large	Collapse	1.05×10^{-3}	8.26×10^{-4}

3.4.3 Reliability of Reinforced Concrete Buildings in Japan in Accordance with 1993 PRESSS Guidelines

Parallel to the effort on reliability evaluation of steel frame buildings designed in accordance with UBC, an investigation of reliability of R.C. structures in Japan designed according to the 1993 PRESSS Guidelines (Otani et al 1993), which are similar in concept to the 1990 AIJ Guidelines with slight modifications, has been also carried out [Saito and Wen 1994]. The methodology follows that of Wen et al [1992] with the same emphases on uncertainty in the seismic excitation. It provides a basis for comparison of the implied safety in the recently developed codified design procedures in US and Japan. It is noted that the UBC is a building code whereas the 1993 PRESSS Guidelines are still a proposal under review and revisions. The Building Standard Law is still the "law" for designing buildings in Japan. The study and results are summarized in the following.

Selection of Site and Risk Analysis

Two sites are chosen, both in Zone A with a zone factor Z=1.0. One site is at Sendai and the other is at Tokyo as shown in Figure 3-16 where the distribution of large events for the period of 1605 to 1992 is also shown. For both sites, seismicity data recorded between 1900 and 1992 are used and seismic events with epicentral distance (R) less than 350 km, magnitude (M) greater than 5.5, and hypocentral depth less than 100 km are considered to be representative. A total of 974 records are available for the Sendai site and 721 for the Tokyo site. No classification into characteristic earthquakes and noncharacteristic earthquakes is done and the occurrence of future earthquakes is treated as Poisson process. An upper bound of M = 8.5, which is consistent with record, is assumed in the Gutenberg-Richter magnitude frequency equation. Fig. 3.17 shows the comparisons with data of the G-R equation and Fig 3.18 shows the comparison of distribution of R used in the seismic risk analysis with data. The correlation between M and R are found to be small; ρ = -0.026 for the Sendai site and ρ = -0.016 for the Tokyo site, therefore, M and R are assumed to be independent variables. The duration is modeled by the empirical relationship commonly used in Japan:

$$\log t_d = 0.31\,M - 0.774 \tag{3.19}$$

It is assumed that the uncertainty in the above equation is much smaller than that in the attenuation relation and therefore not considered.

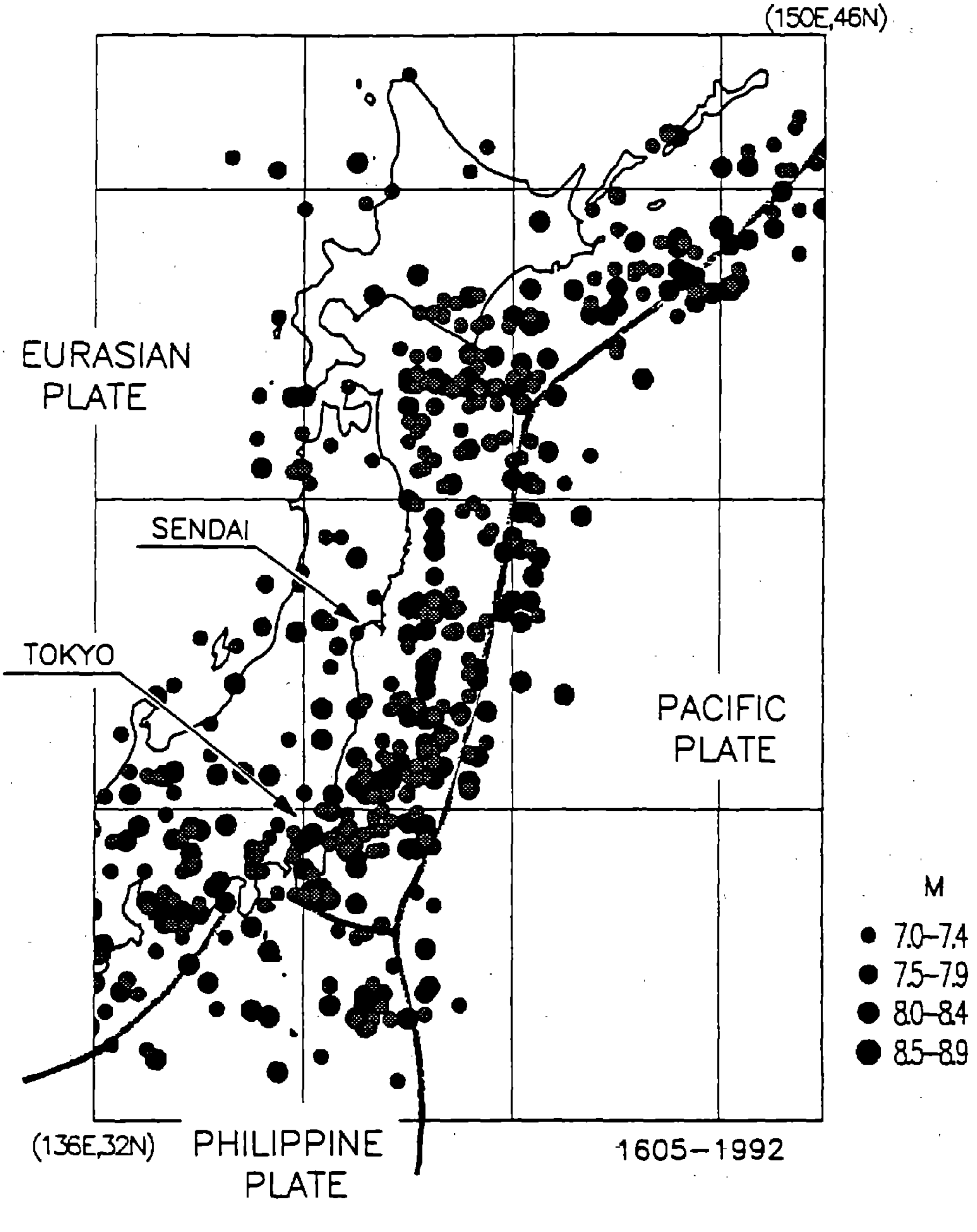


FIGURE 3-16 Two Sites and Distribution of Earthquake Sources in Eastern Japan

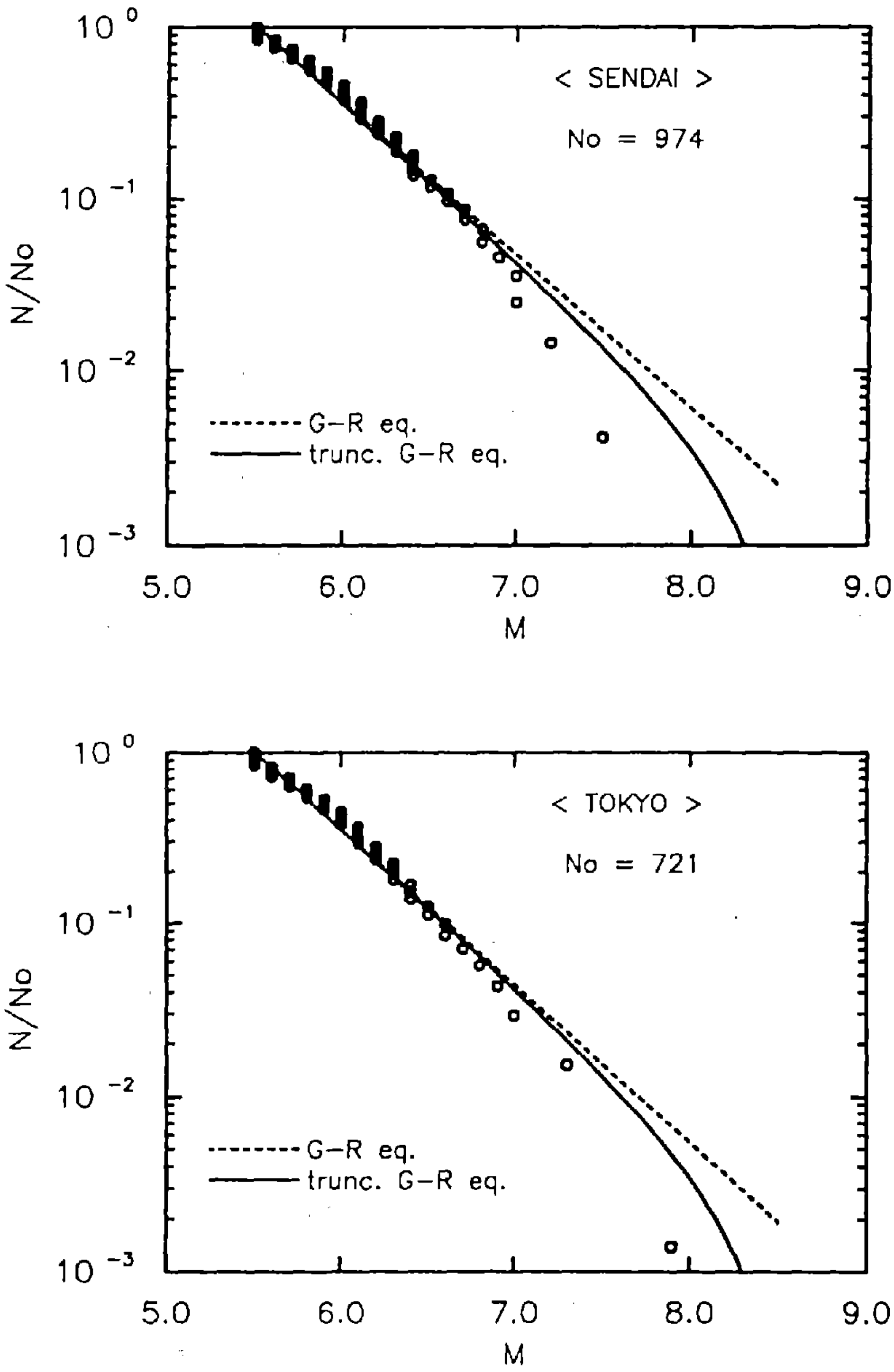


FIGURE 3-17 Comparison of Gutenberg-Richter Magnitude Frequency Relationship with Data at The Sendai Site

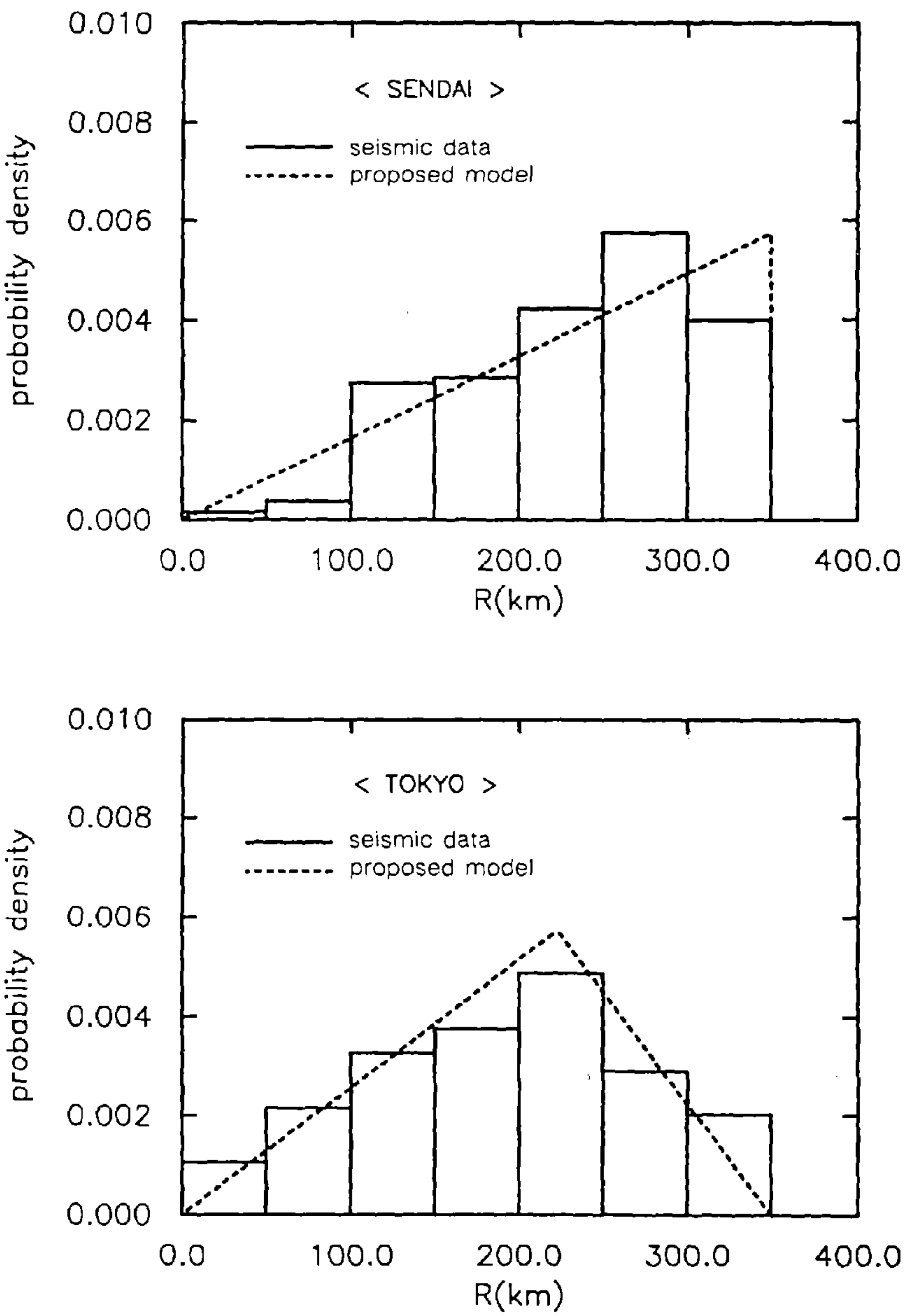


FIGURE 3-18 Frequency Diagram of Epicentral Distances

Modeling of Ground Motion

The attenuation equation proposed by Katayama (1982) for acceleration response spectra at a damping value of 5 % is used. The response spectra are dependent on source magnitude, epicentral distance, and site soil condition. Table 3-IV is used for determining the response spectra for 18 different periods. An uncertainty term is incorporated which is modeled by a lognormal variate with a mean value of 1.267 and standard deviation of 1.025.

At the Sendai site, the parameters in the intensity and frequency modulation functions are identified by assuming that the 1976 Miyagiken-Oki earthquakes time histories are representative and used in the regression analysis. For the Tokyo site the acceleration record of the 1956 earthquake measured at Tokyo University is used. To identify the Clough-Penzien (CP) spectrum parameters, an iterative procedure proposed by Shinozuka et al (1988) is used so that the response spectrum of the time history generated by the random process model matches the target (Katayama) response spectrum determined by the foregoing procedure. For a future earthquake in the region where the site is located, the source parameters random variables are first generated from computer. With the aid of Table 3-VI the response spectrum at the site and hence the power spectral density function consistent with the response spectrum can be determined and the time histories of the ground motion can be simulated using the nonstationary random process model described in the foregoing. Therefore each time history generated will have a different intensity, frequency content, and duration which are consistent with the source, path, and site parameters; hence it represents a sample of what is going to occur in the future at the site. Figure 3-19 shows the CP spectrum density identified for the two sites.

Table 3-VI
Acceleration Response Spectra Parameters (Katayama 1982)

T(sec)	fM Magnitude(M)					fR Epicentral distance(R:km)					fGC Ground condistion(GC)			
	4.5-5.3	5.4-6.0	6.1-6.7	6.8-7.4	7.5-7.9	6-19	20-59	60-119	120-199	200-405	Type 1	Type 2	Type 3	Type 4
0.1	0.218	0.278	0.296	0.399	1.00	5.10	2.67	2.05	1.00	1.00	126	107	120	106
0.15	0.225	0.274	0.297	0.448	1.00	4.85	3.01	2.15	1.00	1.00	155	130	141	125
0.20	0.185	0.280	0.288	0.499	1.00	5.48	3.24	2.07	1.05	1.00	169	149	161	129
0.25	0.171	0.254	0.283	0.534	1.00	6.86	3.65	2.33	1.21	1.00	135	129	143	129
0.30	0.164	0.269	0.280	0.548	1.00	6.59	3.51	2.25	1.27	1.00	109	130	147	131
0.35	0.161	0.274	0.302	0.588	1.00	5.74	3.05	2.13	1.24	1.00	92.8	126	149	142
0.40	0.152	0.268	0.311	0.557	1.00	5.45	3.01	1.92	1.33	1.00	83.0	122	145	144
0.50	0.108	0.237	0.309	0.593	1.00	6.35	2.91	1.60	1.36	1.00	76.6	113	140	156
0.60	0.0889	0.246	0.321	0.618	1.00	5.88	2.79	1.46	1.32	1.00	62.1	101	134	159
0.70	0.0730	0.222	0.315	0.644	1.00	6.77	2.96	1.56	1.37	1.00	50.0	88.8	118	148
0.80	0.0683	0.214	0.294	0.595	1.00	5.89	2.73	1.54	1.28	1.00	47.9	91.0	115	145
0.90	0.0672	0.214	0.285	0.581	1.00	5.13	2.38	1.48	1.20	1.00	46.4	90.5	113	136
1.00	0.0653	0.204	0.284	0.636	1.00	4.62	2.15	1.40	1.16	1.00	43.3	89.3	107	125
1.50	0.0503	0.138	0.204	0.534	1.00	4.40	2.20	1.44	1.00	1.00	33.0	56.5	68.5	84.6
2.00	0.0605	0.148	0.215	0.585	1.00	3.66	1.99	1.29	1.00	1.00	24.7	36.8	44.1	46.2
2.50	0.0587	0.136	0.183	0.405	1.00	3.50	1.95	1.34	1.00	1.00	21.9	32.7	35.8	33.0
3.00	0.0660	0.138	0.194	0.391	1.00	3.26	1.79	1.35	1.00	1.00	18.8	26.6	28.5	26.6
4.00	0.0704	0.144	0.187	0.395	1.00	2.81	1.61	1.27	1.00	1.00	15.7	20.3	24.1	19.1

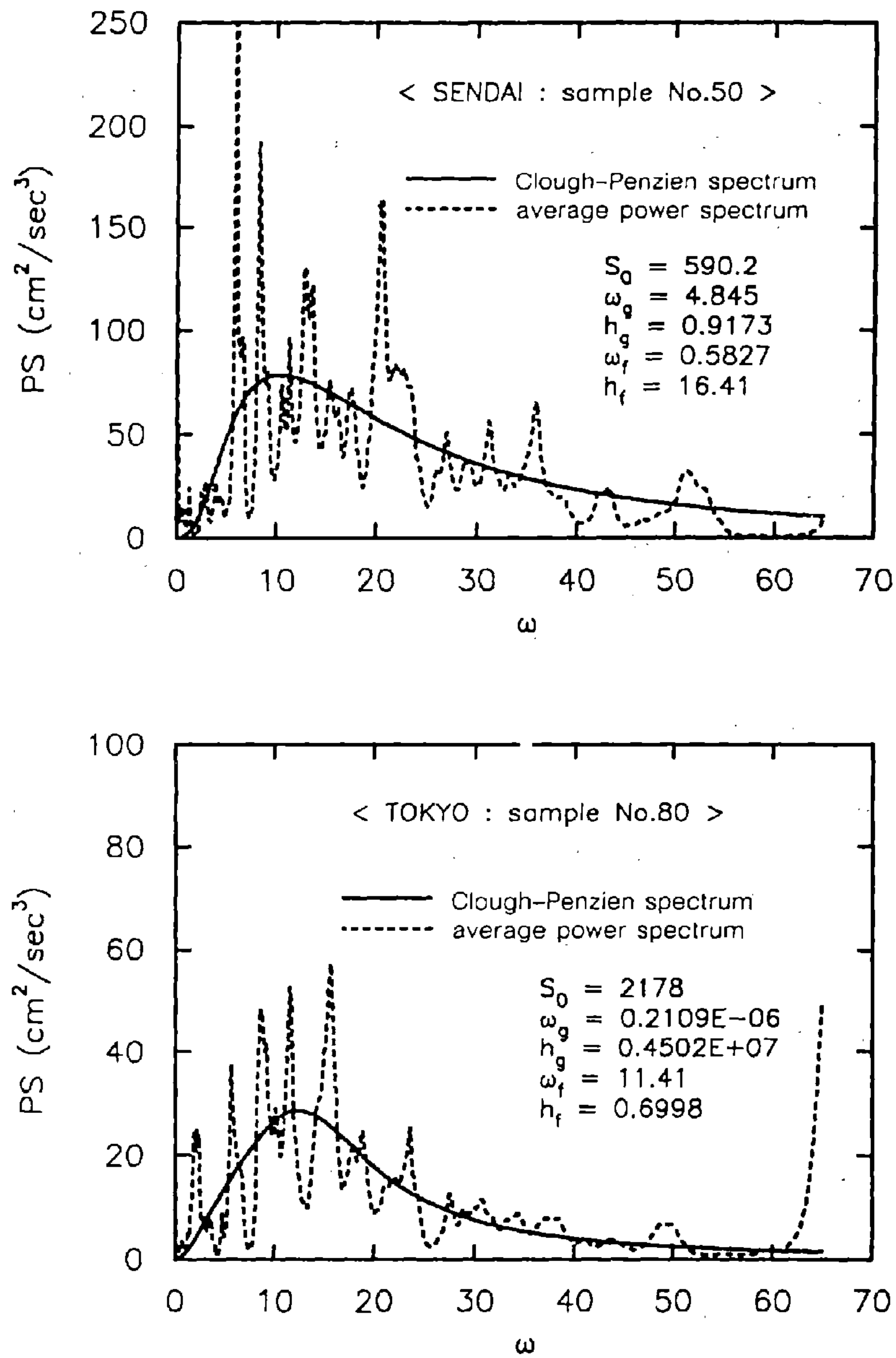


FIGURE 3-19 Comparison of Average Power Spectra and the Identified Clough-Penzien Spectra

Figure 3-20 shows a sample time history at the sites and Figure 3-21 compares the response spectrum of this simulated time history with the target (Katayama) spectrum.

Building Design

Two buildings, one 7-story and one 12-story reinforced concrete moment-resisting frames are designed according to the 1993 PRESSS Guidelines. The Plan and elevation view of the frames are shown in Figure 3-22. As indicated in Section 3.3.1, the design satisfies two performance criteria; one based on consideration of serviceability and the other on ultimate limit states. The buildings are designed to form a strong-column-weak-beam mechanism under a severe earthquake load. The performance of the frames is checked by a static nonlinear (push-over) analysis to ensure right amount of resistance provided at various levels of drift. Details of the design can be found in Saito and Wen (1993).

Response and Damage Analysis

The response of the reinforced concrete structure under the excitation of the ground motion time histories generated as described in the foregoing is analyzed by the computer program " Frame-D " developed at Tohoku University reinforced concrete frames. It is a 2-D program which takes the inelastic response behavior of RC frames into consideration. The hysteresis of the restoring force is modeled by the well known modified Takeda hysteresis rule for RC structures in which the degradation in the system is properly considered. The interstory drifts of the building are calculated and their statistics and probability distribution functions are obtained by repeating the process for a large number of times. Because of the domination of the uncertainties in the attenuation equation which is modeled by a lognormal random variable, it is found that the story drifts given the occurrence of an earthquake are best modeled by a lognormal random variable. Figure 3- 23 shows the probability plot of the story drift of the R.C. frames on a lognormal probability paper where a sample size of 100 is used.

Limit State Risk Evaluation

Based on these simulated records of future earthquakes at the Sendai site and the Poisson process model for the occurrence, the response spectrum of 5 % damping corresponding to a probability of 0.632 of exceedance in 75 years is plotted and compared with the results by Katayama (1982) in Figure 3-24. This response spectrum corresponds to an annual probability of exceedance of 1/75. The agreement is very good. Tables 3-VII shows the drift levels for the 7-story

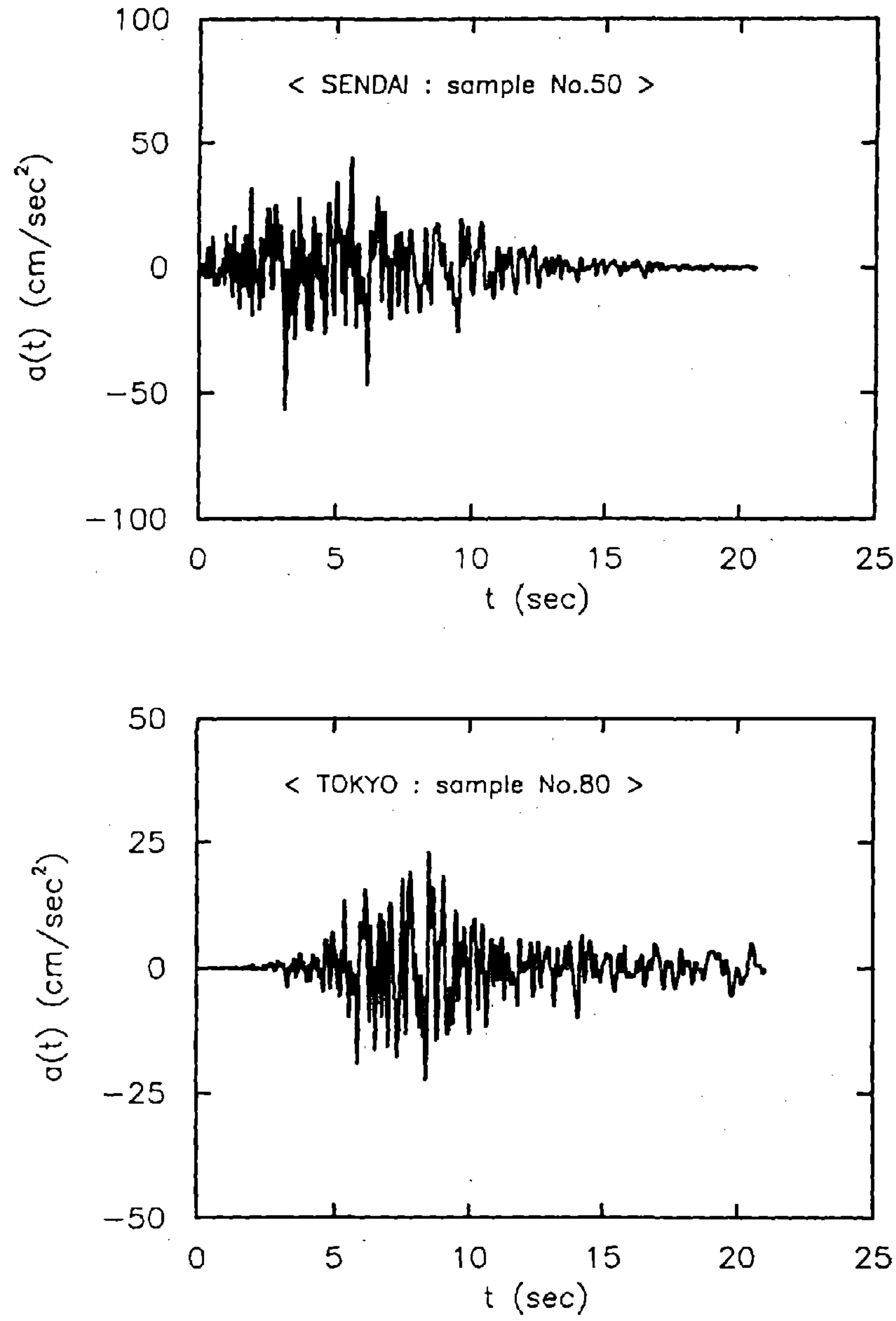


FIGURE 3-20 Sample Time History at The Sendai and Tokyo Site.

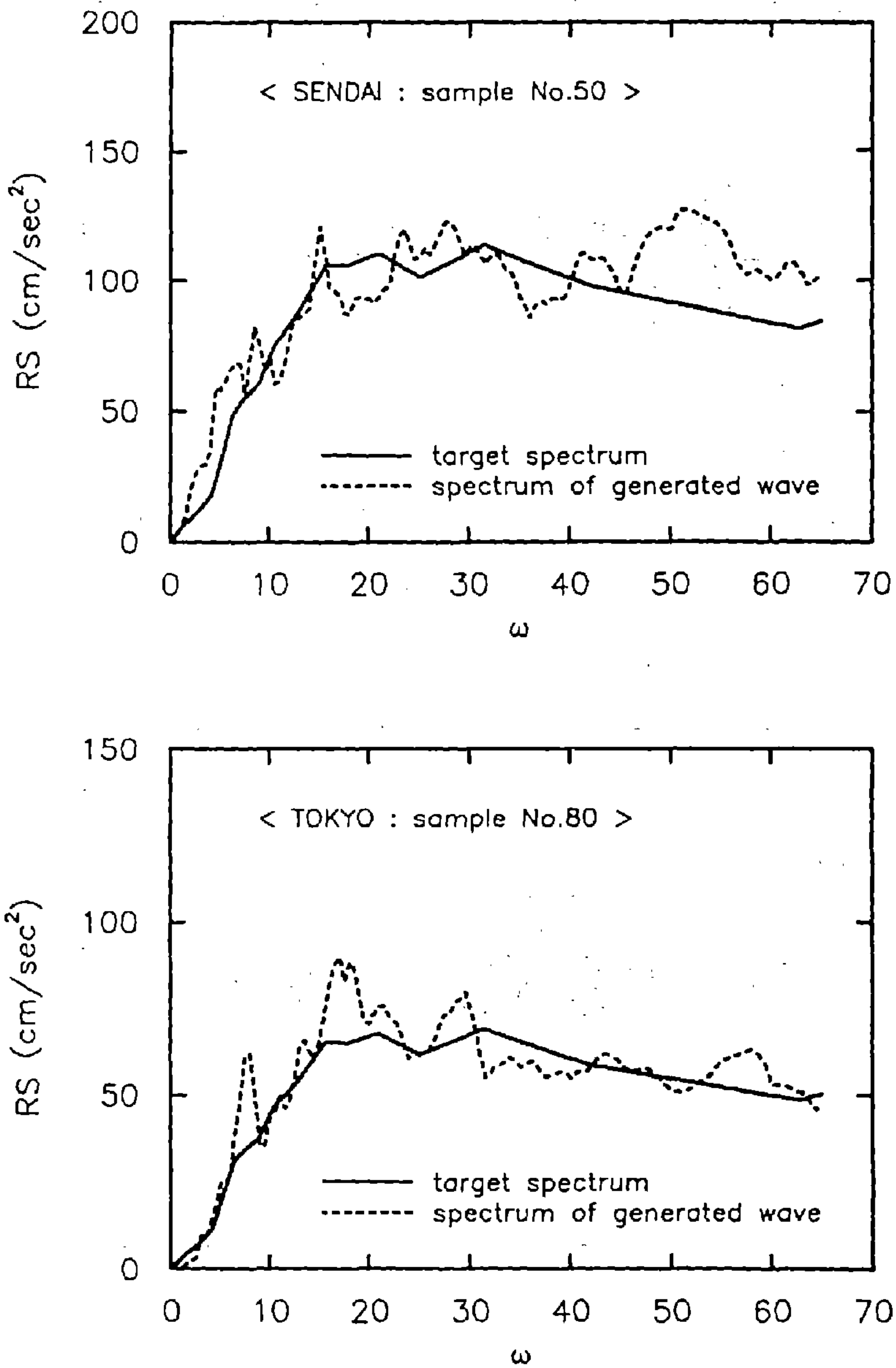


FIGURE 3-21 Comparison between Target and Average Response Spectra

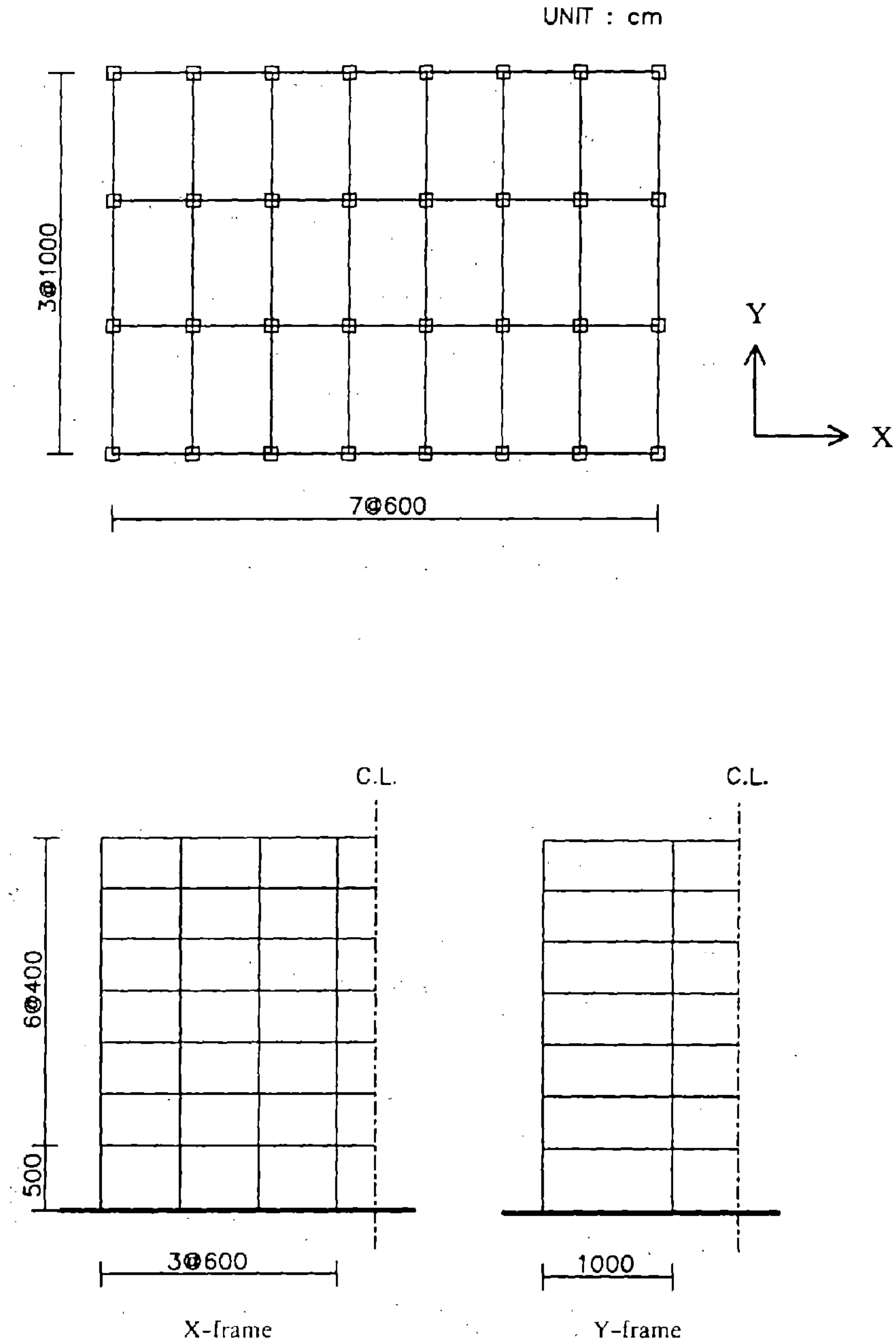


FIGURE 3-22 Floor Plan and Elevation of The 7-Story Reinforced Concrete Frame Designed According to The 1993 PRESSS Guidelines

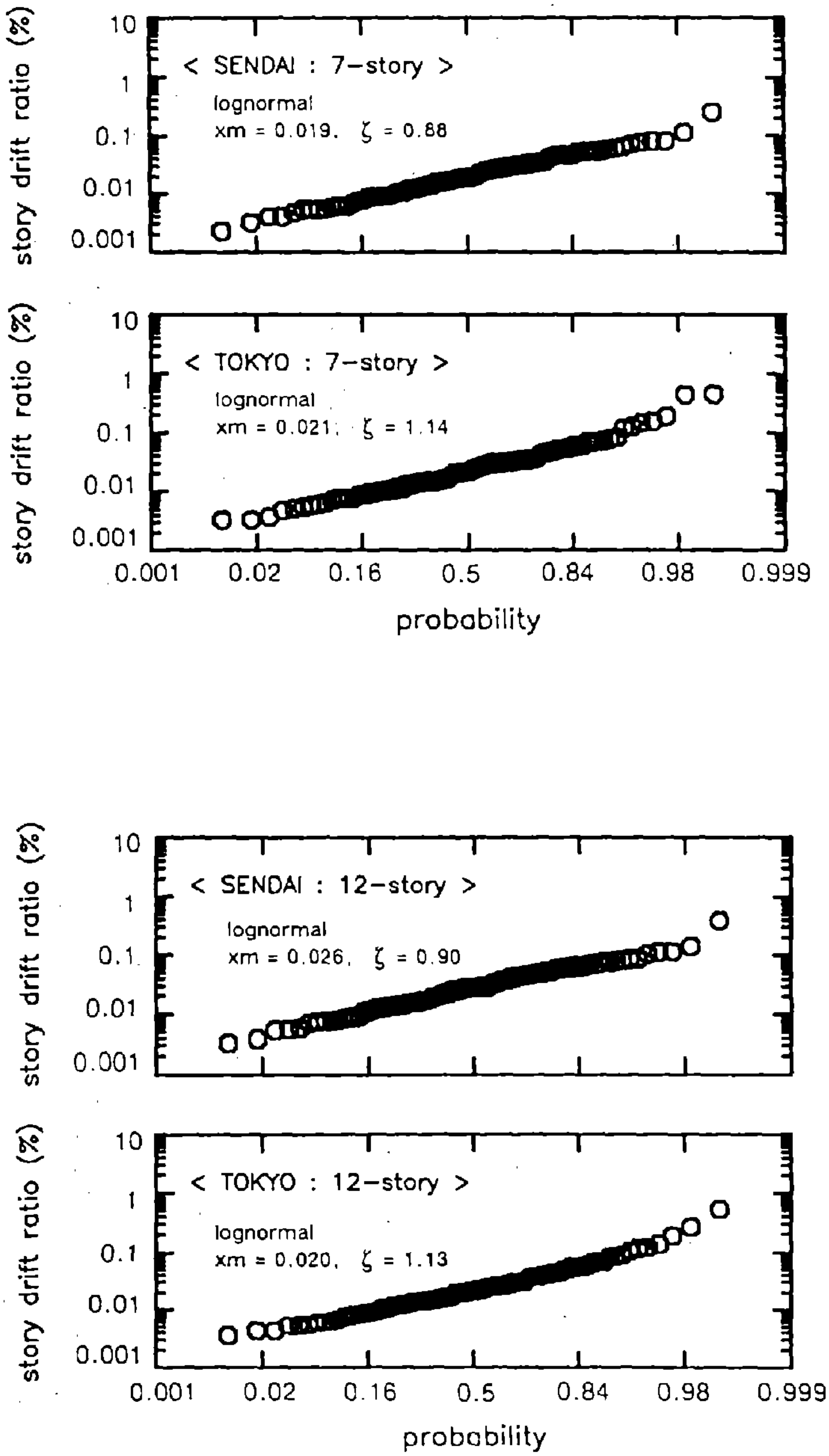


FIGURE 3-23 Lognormal Probability Plot of the Story Drift of The RC Frames.

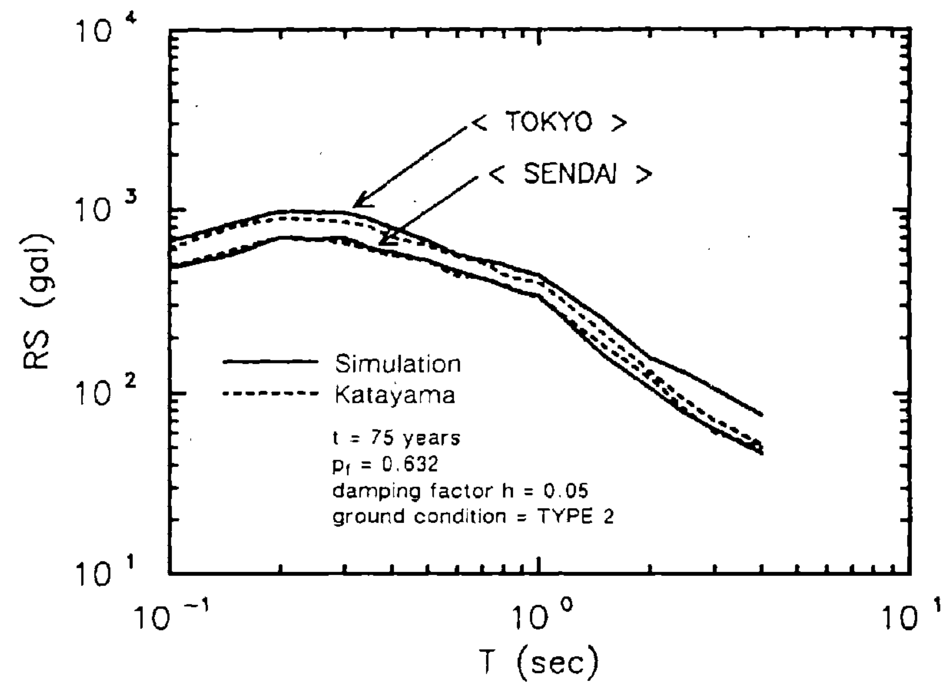


FIGURE 3-24 Comparison of Acceleration Response Spectra Corresponding to an Exceedance Probability of 0.632 in 75 Years

R.C. frame at both sites. The drifts in the first and last row correspond to an annual exceedance probability of 10^{-2} and 10^{-3} respectively. The drifts reach a maximum at the second floor and then decrease uniformly with height. At the Sendai site, at an annual exceedance probability of 10^{-3}, the maximum interstory drift is 0.56 % of the story height, which is just slightly higher than the first level (serviceability) drift limit. The design limit deformation of 1% and the design proof deformation of 2 % are out of the range of the response of the simulations. The risks of such high level drifts therefore are not calculated; they would be much lower than 10^{-3}. At the Tokyo site, although the occurrence of earthquake is less frequent, the intensities are generally higher resulting in larger interstory drifts; i.e., generally larger by a factor of 2 or more. For example, at the lower stories, the serviceability drift limit of 0.5 % will be exceeded with a probability 10^{-2} per year; and at an annual probability of 10^{-3}, the drifts are close to 1.5 %.
The performance curves of the two structures in terms of the probability of maximum interstory drifts are compared in Figure 3-25.

The serviceability probabilities are quite comparable to those of first yield given in Table 3-V for reinforced concrete buildings in eastern United States. In the 1993 PRESSS Guidelines the design limit deformation is set a 1 % and the design proof deformation at 2 %. Therefore a 1.5 % drift may mean some or even significant damage but not yet collapse. By comparison, probability of collapse of 10^{-3} for reinforced concrete structures in eastern United States given in Table 3-V seems high. It may be attributable to the conservatism in the estimate of capacity against collapse by a static spectral analysis.

Similar results were found for the 12-story building as shown in Table 3-VIII. Compared with results of 7-story frame, at the Sendai site the drifts are slightly higher and at the Tokyo site the drifts are comparable.

3.4.4 Reliability of UBC Buildings Against Damage

The above reliability evaluation of buildings designed according to current codes are based on analysis and calculation. Owing to the extremely complex behavior of both excitation and response of the structures during earthquakes, these estimates represent results form one approach to this difficult problem. These estimates certainly are subjected to refinements and

Table 3-VII
Story Drift Ratio (% of story height) Corresponding to Various Exceedance Probabilities in 50 Years (7-story building)

(a) SENDAI

p_f	story drift ratio (%) (T=50 years)						
	1F	2F	3F	4F	5F	6F	7F
0.50	0.272	0.293	0.262	0.238	0.185	0.136	0.077
0.25	0.342	0.369	0.330	0.302	0.233	0.170	0.097
0.15	0.394	0.425	0.381	0.349	0.268	0.196	0.111
0.10	0.435	0.471	0.422	0.388	0.297	0.217	0.123
0.05	0.513	0.556	0.499	0.460	0.351	0.256	0.144

(b) TOKYO

p_f	story drift ratio (%) (T=50 years)						
	1F	2F	3F	4F	5F	6F	7F
0.50	0.586	0.639	0.592	0.543	0.415	0.277	0.153
0.25	0.793	0.867	0.807	0.741	0.562	0.370	0.203
0.15	0.954	1.046	0.976	0.897	0.676	0.442	0.242
0.10	1.095	1.201	1.123	1.034	0.776	0.504	0.275
0.05	1.358	1.494	1.401	1.292	0.964	0.620	0.336

Table 3-VIII
Story Drift Ratio (% of story height) Corresponding to Various Exceedance Probabilities in 50 Years (12-story building)

(a) SENDAI

p_f	story drift ratio (%) (T=50 years)											
	1F	2F	3F	4F	5F	6F	7F	8F	9F	10F	11F	12F
0.50	0.398	0.494	0.459	0.434	0.394	0.339	0.315	0.284	0.223	0.168	0.117	0.059
0.25	0.502	0.627	0.581	0.549	0.497	0.426	0.397	0.360	0.281	0.210	0.145	0.073
0.15	0.579	0.726	0.671	0.634	0.574	0.490	0.458	0.416	0.323	0.241	0.166	0.082
0.10	0.642	0.807	0.745	0.704	0.636	0.542	0.507	0.462	0.358	0.266	0.183	0.090
0.05	0.759	0.957	0.882	0.834	0.752	0.639	0.600	0.548	0.422	0.312	0.215	0.105

(b) TOKYO

p_f	story drift ratio (%) (T=50 years)											
	1F	2F	3F	4F	5F	6F	7F	8F	9F	10F	11F	12F
0.50	0.556	0.676	0.616	0.568	0.537	0.491	0.442	0.384	0.300	0.229	0.165	0.082
0.25	0.750	0.914	0.828	0.761	0.720	0.657	0.588	0.507	0.390	0.294	0.212	0.104
0.15	0.902	1.100	0.993	0.911	0.862	0.786	0.700	0.600	0.458	0.343	0.247	0.120
0.10	1.033	1.261	1.137	1.040	0.985	0.897	0.796	0.680	0.516	0.385	0.276	0.133
0.05	1.280	1.566	1.405	1.283	1.215	1.105	0.976	0.829	0.622	0.461	0.330	0.157

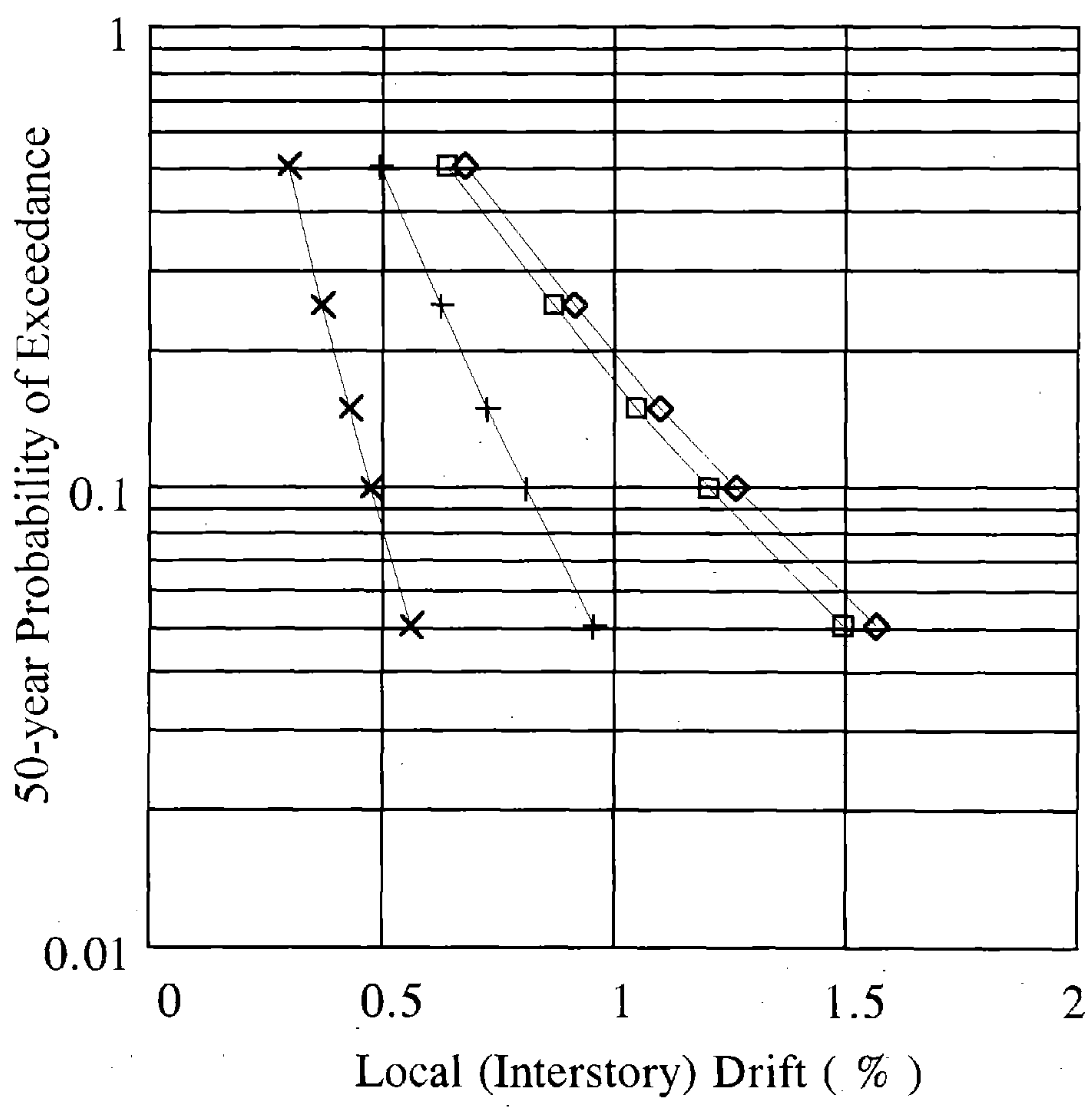


FIGURE 2-25 Comparison of Performance Curves of RC Moment Frame Buildings Designed According to 1992 Japan PRESSS Design Guidelines: Sendai, 7-Story (x), 12-Story (+);Tokyo, 7-Story (box), 12-Story (diamond).

modifications as more information becomes available regarding seismic environments as well as the accuracy of the methods of modeling used in the analysis. The reliability of the codified design can be also estimated from an entirely different approach based on the expected building performance estimated by experts. These experts are experienced structural engineers who can give their opinions regarding building performance in future earthquakes based on performance and damage surveys of buildings in past earthquakes such as given in ATC-13 (1985). Such an survey has been recently conducted by EERI for buildings designed according to 1991 UBC and the results are used herein for a reliability estimate of buildings against damage and collapse. Details of this survey can be found in Holmes and Tubbesing (1993), the salient features of this survey results and how they are used in combination with seismic risk analysis to arrive at an estimate of the reliability is described in the following. The purpose is to present a balanced view of the reliability of codified design.

Table 3-IX shows the survey results of building performance in terms of damage states against earthquakes of specified MMI intensity or magnitude and distance. All buildings are assumed to be located on an intermediately hard soil. The designated damage states from no damage to complete collapse are described in Table 3-X The table values are the estimated percentage of buildings in various damage states given the occurrence of an earthquake of the specified intensity or magnitude and distance. Therefore the results can be used as the conditional probability of performance of building given the occurrence of the earthquake. For a given site, the conditional probability is then combined with the probability of occurrence of earthquakes of various intensities and magnitudes over a given period of time to arrive at the risk of damage and collapse as follows:

$$P_f = 1 - \exp[-\nu t P] \qquad (3.20)$$

in which ν is the occurrence rate of earthquake with intensity $I > I_{min}$, t is the time window, and P is the probability of the structure in a given damage state given the occurrence of the earthquake. With the probability distribution of the intensity, P can be calculated by combining Table 3.IX with intensity distribution as follows:

$$P(X = x) = \sum P(X = x|I = i)P(I = i) \qquad (3.21)$$

in which X is the damage state of the building (from A to E in Table 3-IX), I is the intensity of the earthquake.

At downtown Los Angeles, based on Algermissen et al (1982), the MMI intensity may be modeled by the following truncated exponential distribution

$$F_I(i) = \frac{1}{1 - e^{-b(I_{max} - I_{min}) \ln 10}} [1 - e^{-b(i - I_{min}) \ln 10}] \tag{3.21}$$

in which I_{max} and I_{min} are the upper and lower bounds for I and are set at 5 and 11 respectively for the LA site. b = 0.37. The occurrence rate for earthquake with I larger than 5 is 0.136 per year. Since no damage estimates are given for intensity outside VII and X in Table 3-IX, it is assumed that damage outside this range can be approximated by extrapolation. The resulting annual risks of the five damage states for buildings designed according to 1991 UBC are shown in Table 3-XI The probability of each damage state and the probability of each damage state or higher are shown. It is seen that the damage state E (collapse and life threatening situation) has approximately a risk of 10^{-4} per year and damage state D (extensive damage) or higher has a risk of approximately 10^{-3} per year. These numbers correlate well with the interstory drift statistics for steel buildings given in Table 3-II of Section 3.4.2 . For example at an annual risk level of 10^{-3}, drifts of 1.5 % can occur for moment frames which may lead to extensive nonstructural and some structural damage although not quite life threatening situation.

Table 3-IX
Expected Damage to Buildings (in percent of buildings)
Designed in Accordance with the 1991 UBC

MMI	EQ Size in Richter Magnitude		A	B	C	D	E
	6.0-6.5	7.5-8.0					
	Distance						
VII	30 mi.	50 mi.	60-90	10-40	1-5	< 1	0
VIII	3 mi.	40 mi.	35-60	35-45	10-30	1-5	0-1
IX	1 mi.	30 mi.	25-40	25-40	20-40	3-10	0-2
X	——	3 mi.	7-25	7-25	40-70	10-30	0-5

Table 3-X
Description of Damage States

A	No Damage, only incidental hazard.

B	Minor Damage to nonstructural elements.

C	Primarily nonstructural damage; also could be minor but non-threatening structural damage.

D	Extensive structural and nonstructural damage; localized, life threatening situation would be common.

E	Complete collapse or damage that is not economically repairable. Life threatening situation.

Table 3-XI
Estimate of Annual Risk of Damage of Buildings at Downtown Los Angeles Designed According to 1991 UBC

Damage State	A	B	C	D	E
Risk of Damage State	9.6×10^{-2}	2.03×10^{-2}	5.5×10^{-3}	1.0×10^{-3}	1.4×10^{-4}
Risk of Damage State or Higher	0.13	3.0×10^{-2}	6.7×10^{-3}	1.1×10^{-3}	1.4×10^{-4}

Results given in Table 3-XI are shown in Figure 3-26 in the form of performance curve of 50-year exceedance probability versus increasing damage level.

3.5 Conclusions

Current practice of design of buildings for seismic load is reviewed. Emphasis is on the treatment of uncertainty in the loading and the reliability implied in the current procedures. Provisions for base shear, drift limit, ductility reduction factor are found to vary significantly for different building types in a given country and for the same type of building in different countries. They can differ by a factor of three or more. The reliability implied in various code procedures is evaluated and compared by considering the seismicity and the uncertainties in the ground excitation. Different methods from analytical to empirical are used for this purpose. Performance curves in terms of probability of exceedance of interstory drift limits and a gradation of damage are calculated and compared. The results show variation in the reliability which can be attributed to differences in design philosophy, site seismicity, structural frame system and coefficient values in code provisions such as the R_w factor. The risk of serviceability limit state (approximately 0.5 % interstory drift) being exceeded is generally of the order of 10^{-2} per year. Ultimate limit state (2 % drift or higher) has a risk of 10^{-3} per year or lower. Risk of collapse limit state is difficult to evaluate due to our limited understanding of and capability to model the complex structural collapse behavior during an earthquake. Crude estimates based on expert opinion and empirical evidence seem to indicate that it is of the order of 10^{-4} or lower for buildings which designed in accordance with current code procedures. These performance curves may be used as basis in calibration of multi-level code design procedures based on reliability.

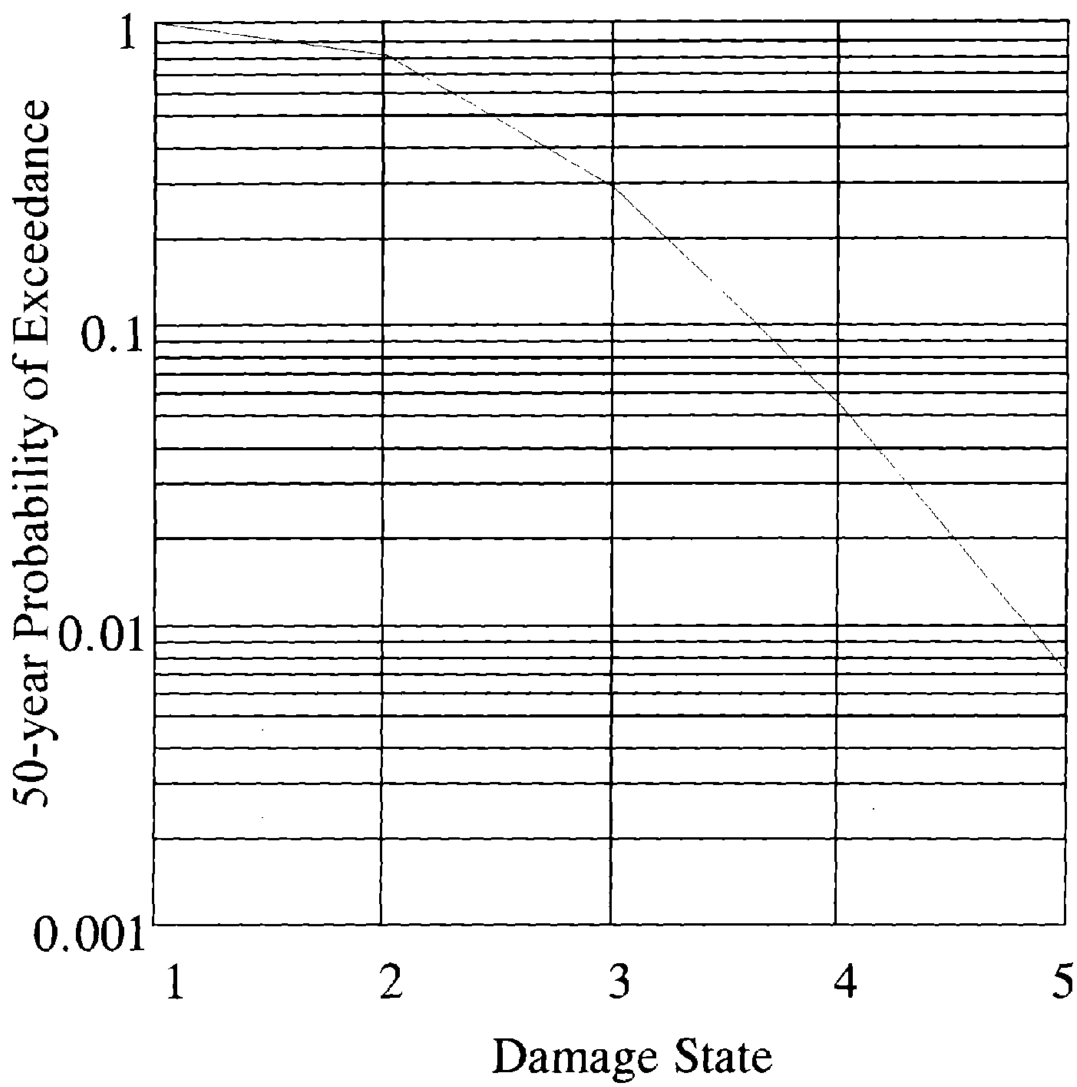


Figure 3-26 Performance Curve of Buildings Designed According to 1988 UBC Against Damage: 1 (A, No Damage), 2 (B, Minor Damage), 3 (C, Moderate Damage), 4(D, Extensive damage), and 5 (E, Complete Damage) (refer to Table 3.10 for details of damage definition)

SECTION 4
RELIABILITY-BASED BI-LEVEL DESIGN PROCEDURE

4.1 Introduction

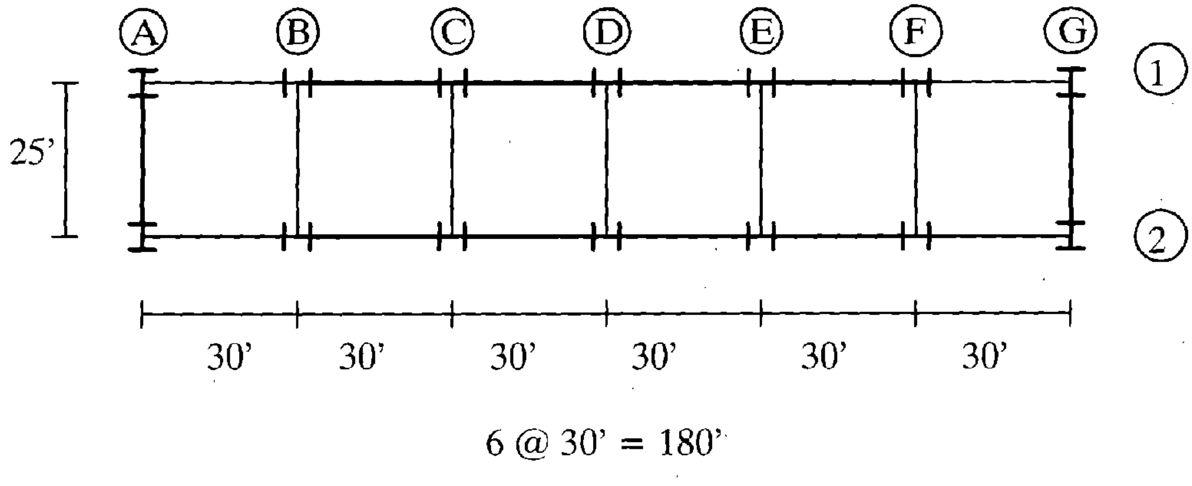
As indicated in previous sections, most current code procedures for design against seismic loads recognize the importance of the uncertainty in the seismic loads but stop at only using a design earthquake intensity based on probability. In many codes even this information is not provided. The determination of load factors, factors for base shear and performance check standard such as drift limit are largely based on experience, judgment, compromise, and conformity to current practice that the reliability implied of the resulting design, e.g., against a given limit state is undefined. With the exception of DOE guidelines, code procedures generally do not set building performance goal in terms of probability. Though in view of the large uncertainty normally associated with seismic load, the performance of buildings and structures designed according to the code procedures against future earthquakes can be assured only in terms of probability of performance goal being exceeded. The review of the code procedures in the Section 3 indicates that: (1) the reliability levels of buildings designed according to building codes varies greatly among different building types in a code and same building type in different countries; (2) there is a need for developing design procedure in which the reliability is explicitly used in the calibration of the various code factors and coefficients. Also, although the majority of code procedures design only for life safety, more recent studies have shown the importance of potential large loss due to content damage and interruption of service. This point is further accentuated by the aftermath of the Northridge earthquake. Therefore, a bi-level or multi-level design with consideration of both the serviceability and ultimate limit states is needed. In the following such a reliability-based, bi-level design procedure is proposed. The theoretical basis for developing such a procedure has been given in Section 2. The concept and procedure are illustrated in the following by simple design examples.

4.2 Calibration of Current Procedures Based on Reliability

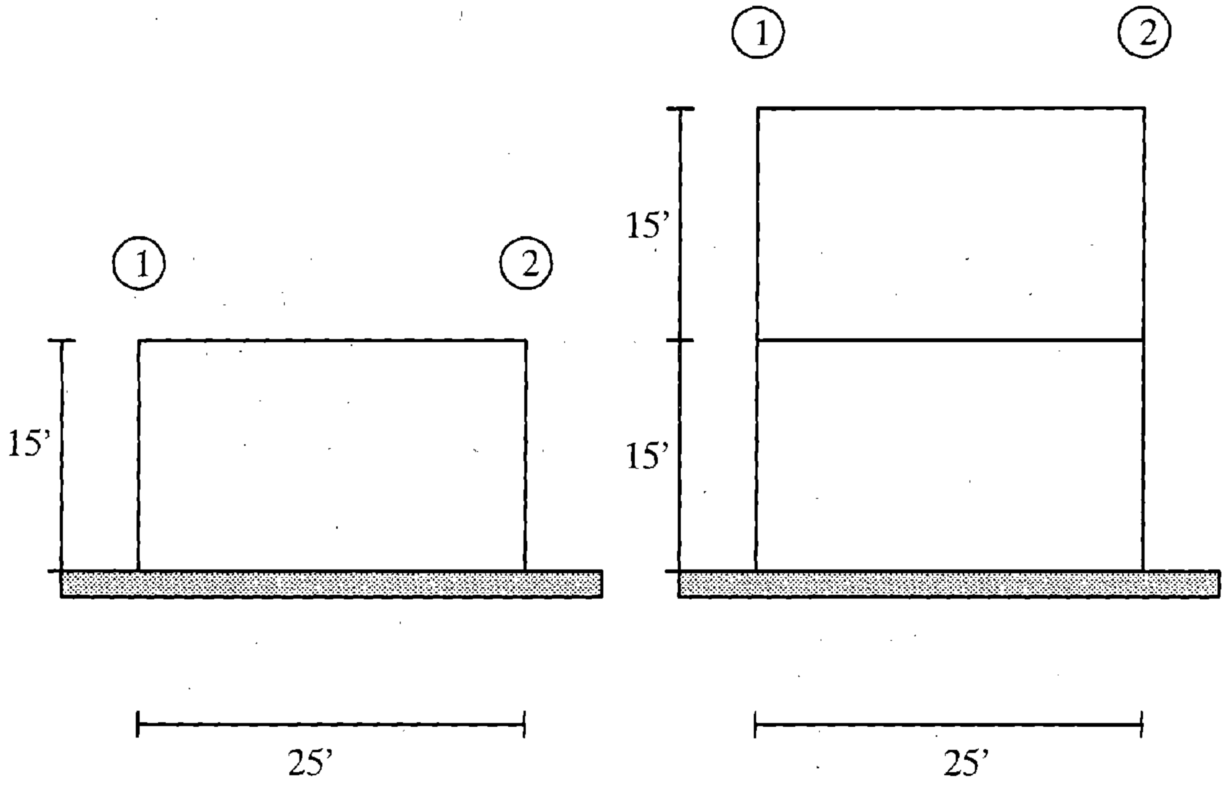
Within the context of the current code format, one can adjust the factors and coefficients in the provisions in such a way that the resultant design will have desirable (target) reliabilities against specified limit states at the serviceability and ultimate failure levels. It is commonly referred to as "code calibration". In the current code procedures, the design earthquake is determined based on probability which naturally has a direct impact on the reliability of the design. In addition, the load factors which account for the overall uncertainty in the loading and the importance factor which accounts for the different levels of performance required of the buildings will also directly affect the reliability of the design. Another safety related design parameter is the drift limits which in many cases control the design. Therefore these design parameters are the target for calibration. They are also interconnected as far as the implication of overall reliability of the structure is concerned. For example, an design earthquake based on a long return period in combination with a small load factor and a small importance factor may lead to a design which is less safe than using a design earthquake based on a shorter return period but larger load and importance factors. Therefore selection of one of these design values without considering the others may lead to inconsistent reliability. The only rational method of determining the design values is calibration according to explicit target reliabilities against specified limit states. Factors based on consideration of structural dynamics, soil condition, ductility capacity and so on should be risk neutral and not part of the calibration process. In summary, within the current code format, one can select key reliability-related factors as design variables in the code calibration process as given in Chapter 2.

4.3 Bi-level, Reliability-Based Design of Steel Structures

The design earthquake according to the provisions recommended by NEHRP (1992) is used; i.e., the design earthquake has a 10 % probability of being exceeded in 50 years, or a return period of 475 years. The importance is taken to be 1.0. Calibration of the importance factor is demonstrated in section 4.4. The design problem is to determine the load factors for dead, live and earthquake loads such that the resultant design will have reliabilities equal to the target values against serviceability and ultimate limit states respectively. For demonstration purpose, the calibration problem is simplified and concentrated on load factors; otherwise the 1992 NEHRP provisions are followed. To emphasize the effect of load factors, the drift limit is not considered in this example but its role in reliability-based design will be investigated in the next phase of study. For simplicity, consider one and two-story, five-bay Special Moment Resisting Steel Frame (SMRSF) structures of as shown in Figure 4-1 The site is at downtown Los Angeles where the risks and ground motions for future earthquakes have been investigated in [Wen et al 1992] and the results are used in this study. For the purpose of demonstrating the procedure, the dead load factor of 1.3 as recommended in the 1992 NEHRP provisions is used in this study and the attention will be concentrated on factors for live and seismic loads. It is pointed out that the objective is to demonstrate the proposed design method. In a real code calibration effort, the buildings selected should be representative of the building population for which the code procedures are developed, i.e. buildings of different designs and located in different seismic zone as will be demonstrated in Section 4.4.



Floor Plan



Elevation

FIGURE 4-1 Special Moment Resisting Steel Frame

4.2.1 Loading Models

The live load is assumed to consist of sustained live load (L_s) and transient live load (L_t) modeled by Poisson square wave and Poisson pulse process respectively [Wen 1990]. The statistics of the dead and live load parameters are given in Table 4-I. The potential future earthquakes that present a threat to the site are modeled as either characteristic or non-characteristic earthquakes [USGS 1988]. The former are major seismic events which occur along the major fault and with relatively better understood magnitude and recurrence time behavior therefore treated as renewal processes. The latter are local events that their occurrences collectively can be treated as a Poisson process. Because of the close proximity to the site and more frequent occurrence, the noncharacteristic earthquakes prove to be more damaging at this site. The ground motion is modeled by a nonstationary random process whose intensity and frequency content vary with time [Wen et al 1992]. The parameters of the ground motion such as intensity, duration, and frequency content depend on the source parameters and local site condition.

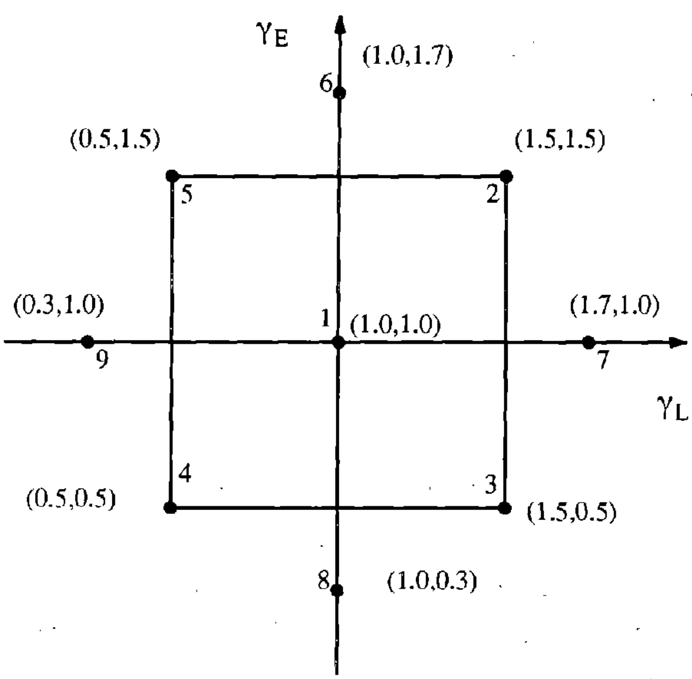
4.2.2 Target Reliability and Response Surface Method

The bi-level design criterion dictates that the formulation in Eq. 2.5 in Section 2 should be used since the target reliability levels for the two limit states are quite different. The serviceability limit is chosen to be an interstory drift of 0.5 % of story height and the ultimate limit is 1.5 % of story height. These limits correspond approximately to the UBC requirements. Results given in Wen et al [1992] on 5-story steel SMRSF buildings designed according to Uniform Building Code 1988 in Ref.18 indicate that the probability of exceedance of a drift of 0.5 % for the 50-year time window after 1991 varies between 0.15 to 0.35; and exceedance of 1.5 % varies between 0.03 and 0.07. Target reliability levels at these levels are used in this study. As indicated in Eq. 2.5 the live and seismic load factors are determined by minimization of the deviations of the limit state probabilities from the target values. Strictly speaking, a nonlinear programming solution procedure is required since the limit state probabilities are generally nonlinear function of the load factors. At each step in the search for the minimization point, however, two frames need to be designed according to the load factor values at that particular step and responses and limit state probabilities need to be calculated. Depending on the number of steps required in the search, the computation required could become excessive. In view of this potential computational difficulty, an alternative procedure is proposed based on method of experimental design and response surface method in the following.

Table 4–I Sustained and Transient Live Load Parameters

Load \ statistics	Distribution	mean	c.o.v	mean occur.rate	mean duration
Dead Load	deterministic	105 psf	–	–	–
Sustained–live load $S_1(t)$	Lognomal	34.5 psf	0.55	0.125	8 yr.
Transient–live load $S_2(t)$	Lognomal	23 psf	0.54	0.25	3.2×10^{-2}yr.

Table 4–II Central Composite Design for Load Factors



Load Factor Plane

Set No.	Load format
1	1.3D+1.0L+1.0E
2	1.3D+1.5L+1.5E
3	1.3D+1.5L+0.5E
4	1.3D+0.5L+0.5E
5	1.3D+0.5L+1.5E
6	1.3D+1.0L+1.7E
7	1.3D+1.7L+1.0E
8	1.3D+1.0L+0.3E
9	1.3D+0.3L+1.0E

In this procedure nine combinations of load factors are selected according to a central composite design procedure[Box et al 1978]. Table 4-II shows the combinations. For each combination two frames, one single-story and one two-story buildings, are designed according to the provisions in the 1992 NEHRP. The responses of the each frame under future earthquakes are calculated and the limit state probabilities evaluated accordingly. With assigned target reliabilities and weighting matrix, Eq. 2.5 is then approximated by a response surface consisting of second order polynomials for which the minimum point can be determined without difficulty. The central composite design assures that the second-order polynomials represents the most efficient approximation of the original response, in this case the objective function, as function of the design variables. Well established criteria can be also used to judge the lack-of-fit error versus errors of other sources such as the noise type due to numerical calculation [Yao and Wen 1993]. The advantage of using this method is that in this procedure the numbers of designs and response analyses are fixed enabling one to have control over the computational effort. Some details of the response surface methodology are given in Appendix A.

4.3.3 Structural Design

In the design, the nominal dead load is 100 psf and nominal live load is 50 psf. The equivalent static base shear is calculated according to NHERP (1992):

$$V = C_S W$$

$$C_S = \frac{1.2 A_V S}{R T^{2/3}} \tag{4.1}$$

in which $A_V = 0.4$ for zone 4, $R = 8$ for SMRSF, $S = 1.0$ for dense and stiff soil, and T = structural period. Table 4-III shows the dimensions and loads for the frames studied. Table 4-IV shows, for example, the resultant designs of member size of the two-story building for the nine combination of load factors. Normally, for one and two-story buildings the designs are controlled by base shear. In this design example the drift limits are not considered for the reasons stated earlier and as a result the resultant frames are slightly on the flexible side. The member sizes of the one and two-story buildings are shown in Tables 4-IV and 4V respectively for the nine design cases for calibration. Note that No.6 corresponds to an earthquake load factor of 1.7 (Table 4-II) and is the strongest among the nine cases and No.8 is the weakest.

Table 4–III Structural Dimension and Loading

Frame \ Prop.	D(k/ft)	L(k/ft)	E(kip)	height(ft)	Span(N–S)	Span(E–W)
Frame1	1.5	0.75	62	15	25	30
Frame2	1.5	0.75	102	30	25	30

Table 4–IV Structural Member Sizes and Limit State Probabilities
(All members are W section)

	1	2	3	4	5	6	7	8	9
1st col.	21x44	24x55	16x40	18x35	24x55	24x55	21x50	18x35	18x40
1st beam	18x40	21x50	18x35	16x31	21x44	21x50	21x44	16x31	18x35
T	0.68	0.54	0.83	0.87	0.55	0.54	0.61	0.87	0.78
Uy	1.25	1.10	1.50	1.40	1.08	1.10	1.20	1.38	1.25
P_{l1}	0.5456	0.4155	0.7156	0.7598	0.4350	0.4155	0.4645	0.7593	0.6555
P_{l2}	0.0766	0.0609	0.1355	0.1517	0.0622	0.0609	0.0693	0.1512	0.1194

Table 4–V Structural Member Sizes and Limit State Probabilities
(All members are W section)

	1	2	3	4	5	6	7	8	9
1st col.	21x62	24x76	16x50	16x45	24x76	24x84	21x62	18x35	18x60
2nd. col.	24x55	24x62	16x45	18x35	24x55	24x62	24x55	18x35	21x50
1st beam	24x62	24x76	18x46	21x44	24x76	24x76	24x62	18x40	24x55
2nd beam	21x50	24x55	18x40	18x35	24x55	24x55	24x55	18x35	21x44
T	1.01	0.86	1.44	1.47	0.87	0.84	0.99	1.59	1.15
Uy	2.6	2.55	3.25	3.15	2.6	2.6	2.6	3.07	2.92
P_{l1}	0.5586	0.3707	0.7385	0.7504	0.3886	0.3304	0.5256	0.8108	0.6327
P_{l2}	0.0883	0.0581	0.1536	0.1540	0.0610	0.0480	0.0851	0.2251	0.1056

1st col., 1st. Beam : Member of 1st story column and Beam
2nd. col., 2nd. Beam : Member of 2nd. story column and Beam
T : Structural Period Uy : Yield Displacement
P_{l1} : Limit State Probability Excceding 0.5% of Story Height
P_{l2} : Limit State Probability Excceding 1.5% of Story Height

4.3.4 Limit State Probability Evaluation

In calculating the limit state probabilities according to the method given in Eq. 2.2 of Section 2. The combinations required are that of sustained live and transient live ($L_s + L_t$), sustained live and earthquake ($L_s + E$), and all three time varying loads ($L_s + L_t + E$). The dead load is always present and assumed to be deterministic in these combinations for simplicity. As the occurrence behaviors of characteristic and non-characteristic earthquakes are different, the contribution to the overall limit state probability from these two sources are treated separately. The response and conditional probabilities of limit state being exceeded are evaluated by time-history/simulation method. Time histories of the ground motions due to these two type of earthquakes are generated according to the source, path, and site characteristics. The inelastic response behavior of the structural frame is included in the consideration. The responses are evaluated and the limit state probabilities estimated based on response statistics obtained and an extreme value distribution that fits the data. Details can be found in Wen et al [1992].

As expected, the combination of sustained live with earthquake contributes the most. The combination of all three time varying loads although causes a slightly higher limit probabilities, it also has a small joint occurrence rate because of the very brief duration's of both earthquake and transient live load resulting in a very small contribution in the limit state probability. Due to the distance of 60 km from the site to the Mojave Segment of the San Andrea Fault, the contribution of the characteristic earthquakes is much less that of the local events. The calculated limit state probabilities for the one and two-story buildings are also shown in Tables 4-IV and 4-V respectively for the nine combinations of load factors. Note that case No. 6 is the most conservative design and No. 8 the least, well reflected by the limit state probabilities.

It is pointed out that the limit state probability evaluation is most computationally intensive since repeated response analyses are required for each frame for nine combinations of load factors. For large and complex systems, more efficient methods are needed. For this purpose, approximate methods are being developed in which inelastic response statistics of MDOF system under seismic excitation can be approximated by those of an equivalent nonlinear systems (ENS) multiplied by appropriate response modification factors considering the characteristics of the MDOF system and inelastic restoring force behavior. The computation required in the reliability analysis in the above optimization therefore can be significant reduced and becomes manageable even for large systems. Comparison of the probabilities of limit states corresponding to various drift thresholds for buildings of different heights using the approximate method with those based on detailed MDOF response analyses shows the accuracy of the approximate method is quite satisfactory. An example is shown in Figure 4-2 for SMRSF (Special Moment Resistance Space Frames) of one to twelve stories designed in accordance with UBC provisions. The site is at downtown Los Angeles as described in Section 3.4.1. 50-year exceedance probabilities are compared of various threshold levels of building global (top) drift from 0.5 to 3 % of building height based on the ENS and response modification factors. The accuracy is quite satisfactory for purpose of design calibration. The method is summarized in Appendix B, details can be found in Han and Wen (1994).

4.3.5 Results and Conclusions

To see the sensitivity of load factors to prescribed reliability, the target serviceability limit state (0.5 % interstory drift) probability for the next 50 years is chosen at 0.30, 0.50 and 0.70; and that for the ultimate limit state (1.5 % interstory drift) at 0.05, 0.08 and 0.10. In the analysis, the two frames have been assigned equal weights and the ultimate limit state assigned a weight ten times that for the serviceability limit state reflecting the seriousness of the consequence of the limit states. In actual code calibration, the weights may be decided based on consensus among professionals experienced in assessing consequences of different limit states. The second order response surface fit to the objective function is carried out based on a least square procedure. A typical comparison of the fit with data points is shown in Figure 4-3. The data points of the Ω in Eq. 2.5 show that it is not a very smooth function indicating that a nonlinear programming approach in searching for a global minimum may encounter computational difficulty. The response surface method proves to be an valuable alternative.

The load factors obtained from the minimization of the response surface are obtained and shown in Table 4-VI for various combinations of target 50-year limit state probabilities. Each combination represents two key check points in the performance curve required of the design procedure. The corresponding load factors will ensure that this performance objective will be achieved at least for the majority of the building population for which the calibration is intended; in this example, one to two story steel SMRSF buildings of the general configurations shown in Figure 4-1. As expected, the live load factor has little influence on the reliability level compared with seismic load and may be taken a constant value in future study to expedite the calibration process. The load factors are seen to be more sensitive to the change in ultimate limit state probability though it is also somewhat influenced by the serviceability limit state probability. This sensitivity also depends on the relative weights chosen for the limit states considered. Note that to achieve target reliability

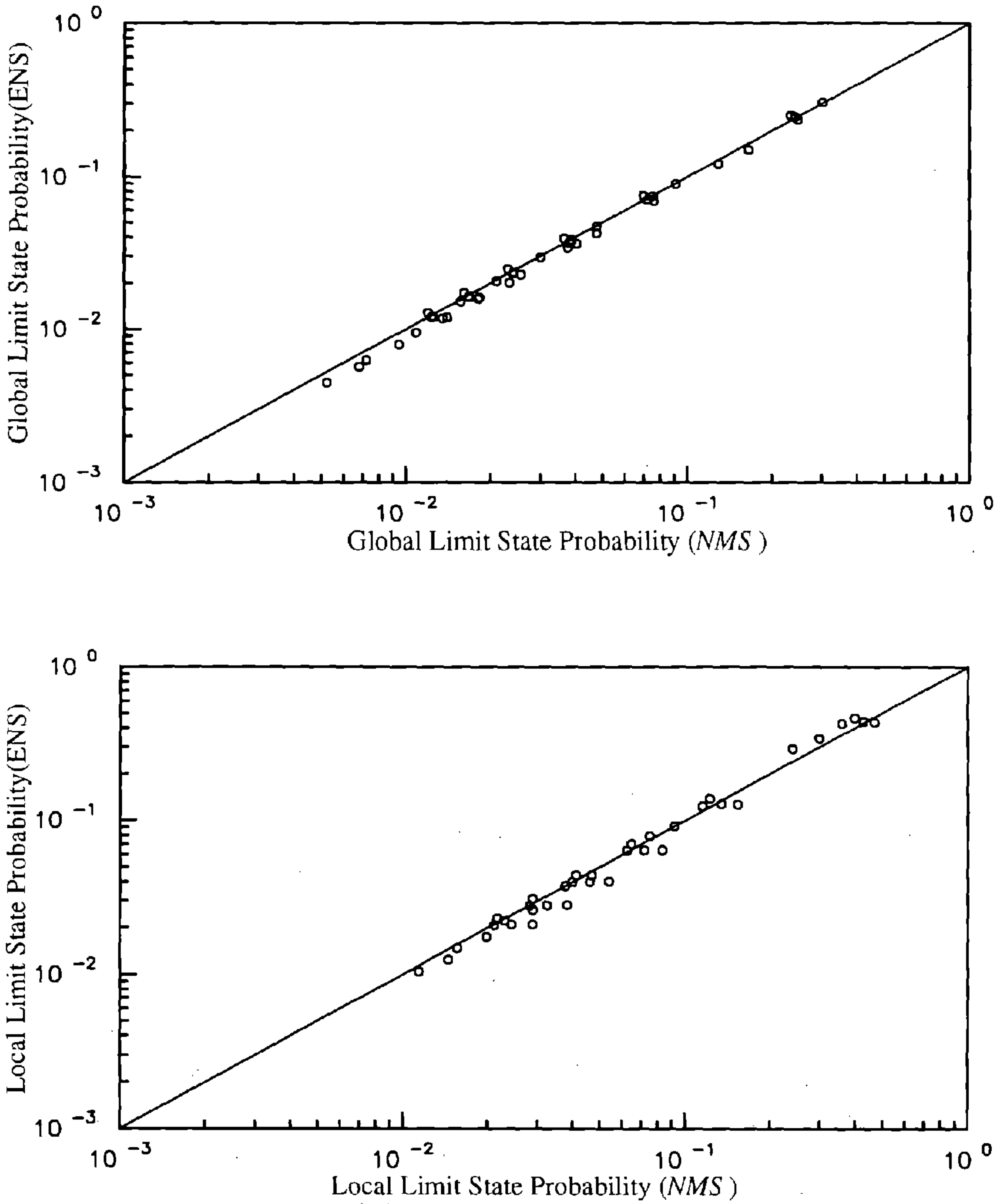


FIGURE 4-2 Comparison of 50-year Global (Building) and Local (Interstory) Drift Exceedance Probabilities of Equivalent Nonlinear Systems (ENS) with Those of Nonlinear MDOF Systems (NMS) at Los Angeles Site

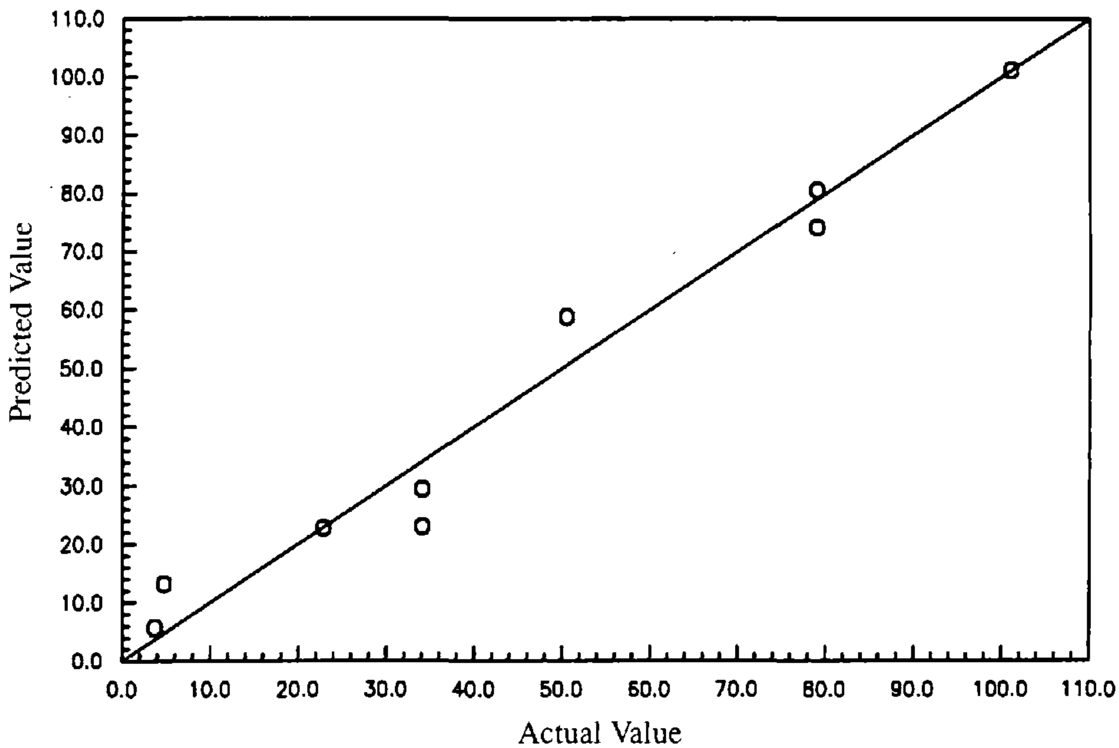


FIGURE 4-3 Comparison of Objective Function with Predicted Value by Response Surface Method

Table 4-VI Optimal Load Factors

(Serviceability : interstory drift of 0.5 % of story height; Ultimate: 1.5 % of story height)

Target for Service	0.30	0.30	0.50	0.50	0.70	0.70
Target for Ultimate	0.05	0.10	0.05	0.08	0.05	0.10
γ_L	1.00	1.00	1.00	1.10	1.00	1.00
γ_E	1.35	1.18	1.28	1.20	1.24	0.68

levels comparable to those for 5-story steel buildings according to 1988 UBC reported in Wen et al (1992), the earthquake load factor needs to be around 1.35 instead of 1.00 given in NEHRP. It is due to mostly the fact that the drift limits are not enforced in the design and to a lesser extent the configurations of the one and two-story buildings chosen in this design example.

The results of these simple design examples serve to demonstrate the bi-level, code calibration process based on reliability and performance. The methodology can be applied to calibration for next generation code procedures. Further study is needed in areas of selection of target limit state probabilities, assignment of the weights for limit states and structural type, inclusion of drift limits in calibration process, and using efficient method for selecting structural frame and configuration representing the building population. Research in some of the above area is currently in progress.

4.4 Reliability-Based Design of RC Structure

Design of RC frame building based on reliability is demonstrated. As shown in previous section the live load factors are not sensitive to the changes in the target reliability level. To concentrate on seismic load factor, the dead load and live load factors are kept constant and the current code values are used. Also, as shown in the previous section, the ultimate limit state dominates the calibration, therefore, for simplicity calibration will be with regard to ultimate limit state only. The emphases are on: (1) specific consideration of risks of performance goals of different levels (ordinary, high-risk, and essential), (2) effect of nonlinear response behavior, uncertainty in structural resistance, and (3) calibration of importance factor and load factor, including dependence of load factor on seismic zone. The results are based on a study by Hwang and Hsu (1991, 1993) to develop reliability-based seismic design criteria for RC intermediate moment resisting frames and summarized in the following.

4.4.1 Seismic Hazards

In model building codes such as the Uniform Building Code (1988), the design earthquake is usually defined as an earthquake with a 10% probability of exceedance in 50 years. It is denoted as a 475-year earthquake, since the return period of such an earthquake is 475 years. Algermissen and Perkins (1976) of the U.S. Geological Survey (USGS) evaluated seismic hazards for the contiguous 48 states and produced generic seismic hazard curves corresponding to four levels of the design earthquake E_D ranging from 0.1g to 0.4g (NEHRP Provisions 1988). Since the seismic hazard map specified in model building codes are based on the USGS study, these generic seismic hazard curves (Figure 4-4) are used in this study. This design earthquake defined in model building codes is neither a moderate earthquake nor a large earthquake mentioned in the design philosophy. In this study, the upper bound of a moderate earthquake is set as a 100-year earthquake, while a 2000-year earthquake is used as the upper bound of a large earthquake.

4.4.2 Limit States and Acceptable Risk Levels

A limit state represents a state of undesirable structural behavior. At the beginning of this study, two limit states, first yielding and collapse of a structure, are considered. For a moment-resisting frame structure, the first yielding is defined as the formation of the first plastic hinge anywhere in the structure. The collapse of a structure is defined as the formation of a failure mechanism. The results of this study indicate that the collapse limit state controls the performance of buildings in the event of an earthquake. Thus, the discussion hereafter will focus on the collapse of a structure as the limit state.

Most model building codes do not explicitly specify the acceptable risk level. The acceptable risk level shall be established based on the usage of a structure, characteristics of a limit state, and consequences upon reaching that limit state. Hence, the acceptable risk level (target limit-state probability) may not necessarily be the same for different limit states. In this study, the acceptable risk levels are set for three categories of buildings, essential, high-risk, and ordinary buildings. Essential buildings are defined as structures housing critical facilities such as hospitals and fire stations that are required to remain functional during and after an earthquake. High-risk buildings are those structures used for assembly of a large number of people, for example, schools. All structures not covered by the above two categories are ordinary buildings. For ordinary, high-risk, and essential buildings, the acceptable collapse probabilities are set as 1/1000, 1/2000, and 1/5000 per year, respectively.

4.4.3 Procedure for Establishing Seismic Design Criteria

To ensure that a structure designed according to seismic criteria will achieve a specified acceptable risk level, the procedure for establishing the reliability-based seismic design criteria is as follows (Ellingwood et al. 1980; Hwang et al. 1987; Shinozuka et al. 1989):

1. select a load combination format,
2. select representative frame structures,
3. design structures according to the proposed design format,

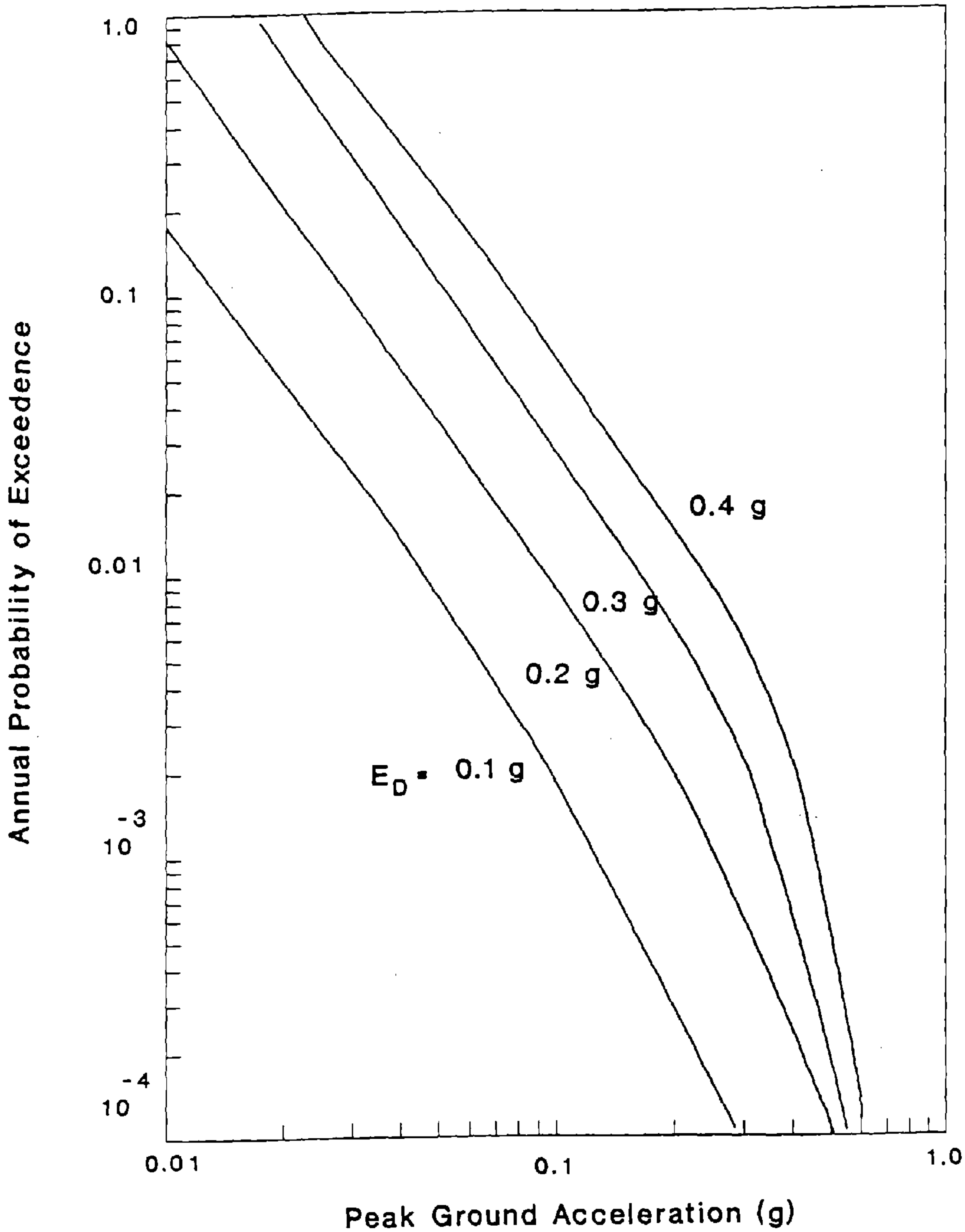


FIGURE 4-4 Seismic Hazard Curves ("NEHRP" 1988)

4. evaluate seismic performance of structures,
5. determine load and resistance factors by optimization with respect to the acceptable risk level.

Load Combination Format

The design of a building requires that the structural resistance should be larger than or equal to the design load effects. For a structure subject to three types of loads, dead load D, live load L, and seismic load E, this requirement expressed in the load and resistance factor design (LRFD) format (Ravindra and Galambos 1978) is as follows:

$$\phi R \geq 1.2 D + 0.5 L \pm \gamma_E E \tag{4.2}$$

$$\phi R \geq 0.9 D \pm \gamma_E E \tag{4.3}$$

where γ_E is the seismic load factor. The dead load and live load factors are preset according to ASCE7-88. The nominal structural resistance R and the resistance factor ϕ are those specified in ACI code 318-89.

Representative Frame Structures

The structural system considered in this study is reinforced concrete intermediate moment-resisting (IMR) frames. To establish a set of representative structures, first of all, parameters affecting the design of RC frame buildings are identified. For each design parameter, a range of the parameter value is established from the current practice; then representative values are selected within the range. The representative parameter values are then used to construct six samples of frame structures (Table 4-VII) by using the Latin hypercube sampling technique (Iman and Conover 1980). Since the design earthquake plays an important role in seismic design of a building, the design earthquake is considered explicitly. The six samples are then combined with each of four design earthquakes (0.1g, 0.2g, 0.3g, and 0.4g), thus yielding a total of 24 samples.

Seismic Design of Structures

Each representative frame structure is designed according to the proposed design criteria with the design base shear V expressed as:

Table 4-VII Representative Frame Structures

Item	Frame					
	1	2	3	4	5	6
No. of stories	5	11	3	13	7	9
Story height (m) (1st story) (m)	3.7 (4.6)	4.3 (5.2)	4.0 (4.9)	4.3 (5.2)	4.0 (4.9)	3.7 (4.6)
No. of spans	3	4	4	3	3	4
Span length (m)	9.2	7.6	6.1	6.1	9.2	7.6
Trans. spacing (m)	7.6	7.6	7.6	7.6	7.6	7.6
RC weight (kN/m^3)	23.6	23.6	23.6	23.6	23.6	23.6
Live load (kN/m^2)	1.9	2.4	1.9	1.9	2.4	2.4
Roof live load (kN/m^2)	0.8	0.8	0.8	0.8	0.8	0.8
f_y (MPa)	414	414	414	414	414	414
f_c' (MPa)	28	35	28	35	35	28
Site coefficient	1.5	1.0	1.2	1.5	1.0	1.2

$$V = I\left(\frac{ZC}{R_\mu}\right)W \qquad (4.4)$$

where I is the importance factor; Z is the seismic zone factor; C is the spectral acceleration coefficient; R_μ is the elastic-to-inelastic response factor, and W is the total seismic dead load. The importance factor is assigned 1.0 for ordinary buildings. For high-risk and essential buildings, the importance factors will be determined later in this study. The seismic zone factor Z is equivalent to the PGA value in g with soft rock as a reference site. The seismic zone factor Z for a site can be determined from a seismic hazard map showing the contours of horizontal peak ground accelerations in the United States.

The spectral acceleration coefficient C is determined from the following equation:

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad (4.5)$$

where S is the site coefficient and T is the structural period. In model building codes, the site condition is classified into four categories: rock site (S_1), dense or stiff soil site (S_2), deep site with medium to soft soil (S_3), and site with soft clay (S_4). The corresponding S factors are 1.0, 1.2, 1.5, and 2.0, respectively. However, the equivalent lateral force procedure specified in building codes may not be applicable for the site with soft clay. Thus, the S_4 site condition is not included in this study.

Building structures are expected to behave in a nonlinear manner in the event of a large earthquake. In the NEHRP Recommended Provisions, the effect of inelastic deformation is introduced into the equivalent lateral force by means of the response modification factor R, which is used to reduce the base shear from an elastic response level to design strength level. An evaluation of the R factor is presented in Appendix III. In this study, the R_μ factor is used to reduce the base shear from the elastic level to the collapse level. On the basis of studies conducted by Riddell and Newmark (1979), Hwang and Jaw (1989), and Hawkins (1986), the R_μ factor is taken as 2.5 for reinforced concrete IMR frame structures.

The lateral forces acting at the floor levels are calculated from the design base and then used to determine member forces caused by the design earthquake. The seismic member forces are combined with gravity forces, and structural members are designed according to ACI code 318-89.

Evaluation of Seismic Performance

Figure 4-5 shows the main steps of a reliability analysis method for evaluating seismic performance of moment-resisting frame structures. In this method, the seismic hazard, limit state, nonlinear response, structural failure mechanism, structural capacity, and acceptable risk level are integrated to provide an overall view of seismic performance. The actual structural capacity in terms of spectral acceleration is taken to be lognormally distributed, which is defined by two parameters: the median $\widetilde{SA}_C$ and the logarithmic standard deviation β_C. The median capacity is determined from a capacity curve established by using the capacity spectrum method (Freeman 1978). The mean values of material strengths instead of nominal strengths are used in the formulas specified in the ACI code 318-89 to estimate the actual ultimate capacity of beams and columns. These actual capacity are consequently used to determine the formation of plastic hinges. In this study, β_C is taken to be 0.3. The probabilistic structural response SA_R is also described using a lognormal distribution. The median spectral acceleration $\widetilde{SA}_R$ is determined at the structural period according to a specified limit state, and the logarithmic standard deviation of response β_R is taken as 0.5.

For a given PGA level, the fragility of a structure is defined as the probability P_f that the structural response SA_R exceeds the structural capacity SA_C. If both SA_R and SA_C are lognormally distributed, P_f can be determined as:

$$P_f = \Phi\left[\frac{-\ln\left(\frac{\widetilde{SA}_C}{\widetilde{SA}_R}\right)}{(\beta_C^2 + \beta_R^2)^{1/2}}\right] \tag{4.6}$$

The P_f values are calculated at various PGA levels and then displayed as a fragility curve. The annual limit-state probability of a structure is determined from the integration of the seismic hazard curve for the site and the fragility curve for the structure. For the collapse limit state, the upper bound of a large earthquake is taken as an earthquake with return period of 2000 years.

Optimization with Respect to Acceptable Risk

The seismic load factors for ordinary buildings can be determined by means of optimization so that the annual limit-state probabilities of the representative structures are sufficiently close to the target limit-state probability. For the collapse of a structure as the limit state, the following

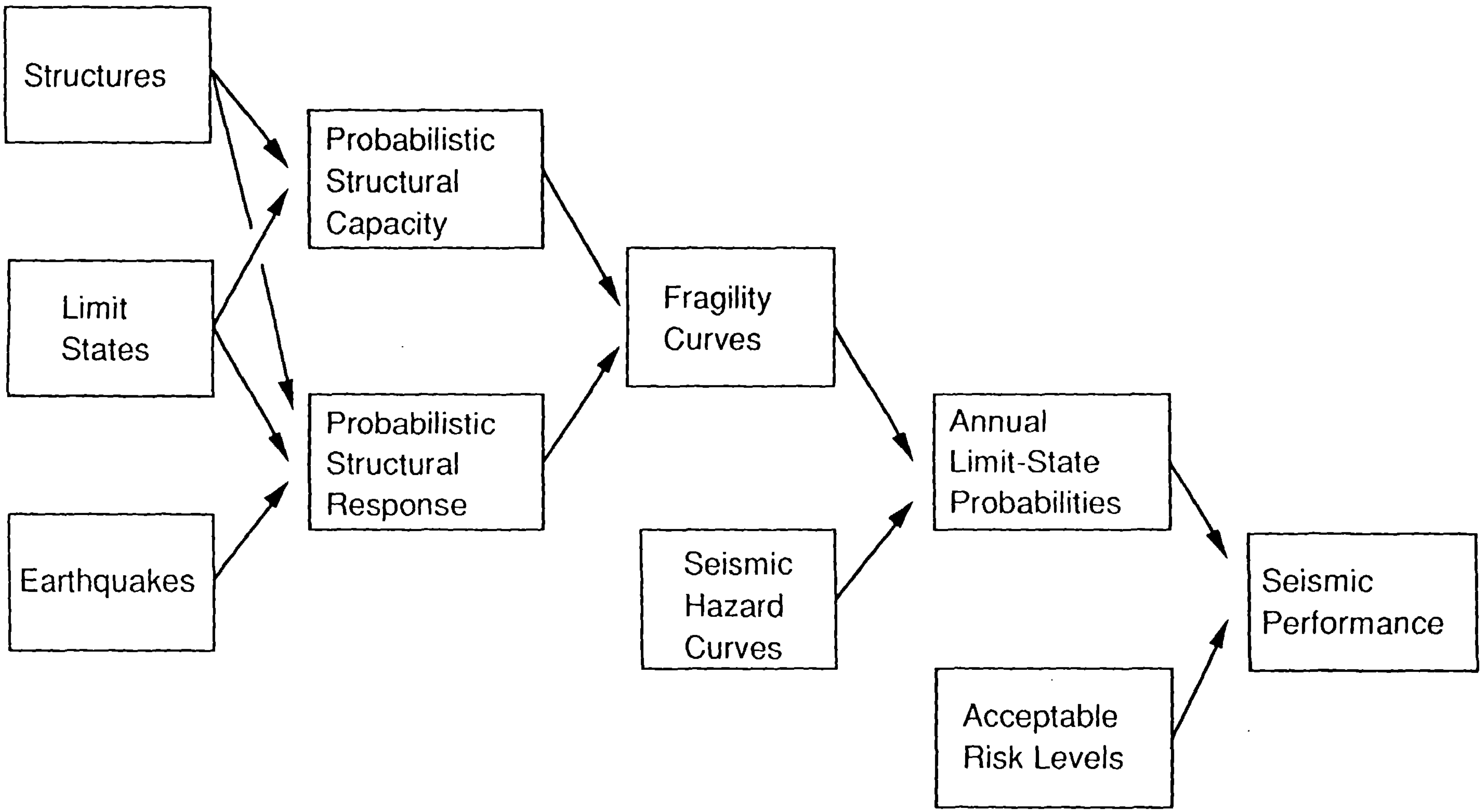


FIGURE 4-5 Main Steps for Evaluating Seismic Performance of Structures

objective function is used to measure the closeness of these two probabilities (Ellingwood et al. 1980; Hwang et al. 1987; Shinozuka et al. 1989):

$$\underset{\min}{\Omega(\gamma_E)} = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{4.7}$$

where $\Omega(\gamma_E)$ is the objective function to be minimized; $PF_{C,T}$ is the target collapse limit-state probability; $PF_{C,j}$ is the collapse limit-state probability computed for the j-th representative structure; N is the total number of representative structures. A similar optimization technique can be used to determine the optimum value of the importance factor.

$$\underset{\min}{\Omega(I)} = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{4.8}$$

4.4.4 Determination of Seismic Load Factors

For a design earthquake E_D = 0.4g, six representative frames (Table 4-VII) are designed according to the proposed design criteria with trial seismic load factors γ_E equal to 1.1, 1.2, 1.3, 1.4, and 1.5, respectively. Reliability analyses of these frames are carried out to determine the annual collapse limit-state probabilities. These limit-state probabilities are then used to compute the $\Omega(\gamma_E)$ value for each trial value of γ_E. Figure 4-6 shows the curve fitting the data of γ_E versus $\Omega(\gamma_E)$. The optimum γ_E value corresponds to the lowest point of the curve; thus, the optimum γ_E is determined to be 1.3. Similarly, six representative frames are designed with a design earthquake E_D = 0.3g and trial γ_E = 0.9 - 1.3. The optimum γ_E is then determined to be 1.15. For the seismic zone with a Z factor of 0.2 (E_D = 0.2g), trial seismic load factors γ_E = 0.6 - 1.0 are used. The optimum γ_E is determined to be 0.8. This value is less than 1.0 because dead load effects have significant contribution to the load combinations in this case and thus provide relatively higher seismic resistance. For the region with the design earthquake of 0.2g, a seismic load factor of 1.0 is recommended for the design of buildings.

For a low seismicity region where the design earthquake $E_D \leq 0.1$g, the representative frames are designed by using the load combinations with only dead and live loads as specified in the ASCE 7-88 :

$$\phi R \geq 1.4 D \tag{4.9}$$

$$\phi R \geq 1.2 D + 1.6 L \tag{4.10}$$

The annual collapse limit-state probabilities of these six frames are evaluated using the same reliability analysis method. All these annual probabilities are less than the acceptable probability, that is, 1/1000 per year. Thus, the design for gravity loads is sufficient to provide seismic resistance in a low seismicity region. It is noted that even though the design is only based on the gravity loads, detailing of frame elements (beams, columns, and joints) should follow the requirement for an IMR frame as specified in the ACI code 318-89 so that the structure has enough ductility to resist earthquakes.

4.4.5 Determination of Importance Factors

For high-risk and essential buildings, the acceptable collapse probabilities are 1/2000 and 1/5000 per year, respectively. Since the seismic load factors determined for ordinary buildings are also used for high-risk and essential buildings, the importance factor is employed to increase structural strength and stiffness so that the structure can achieve the more stringent acceptable risk level in the event of an earthquake. For the case of $PF_{C,T}$ = 1/2000 per year, six representative frames are designed according to the proposed design criteria for a design earthquake E_D = 0.4g and trial I values = 1.0, 1.1, 1.2, 1.3, and 1.4, respectively. Their annual collapse limit-state probabilities are substituted into Eq. 4.5 to determine the values of the objective function. The optimum I value is determined to be 1.2. By using the same procedure, the optimum I values are determined to be 1.2 and 1.1 for E_D = 0.3g and 0.2g, respectively. Since these values are very close to each other, the I factor is recommended as 1.2 for all levels of design earthquakes. Similarly, for the case of $PF_{C,T}$ = 1/5000 per year, the optimum I values are determined to be 1.6, 1.5, and 1.4 for E_D = 0.4g, 0.3g, and 0.2g, respectively. An I factor of 1.5 is recommended in this case.

For high-risk buildings in a low seismicity region, $E_D \leq 0.1$g, the annual collapse limit-state probabilities of buildings designed using only gravity loads are still less than the target probability of 1/2000 per year. Thus, design using only gravity loads is still adequate to provide seismic resistance. For essential buildings in a low seismicity region, a design based on gravity loads no longer satisfies the target probability, 1/5000 per year. Seismic load combinations with γ_E = 1.0 and I = 1.0 are used to design the structure. The resulting limit-state probabilities indicate that I = 1.0 is sufficient to provide the required strength.

Table 4-VIII Recommended Importance Factor

Building Category	$PF_{C,T}$ (/yr)	I Factor	
		Z = 0.1	Z = 0.2, 0.3 & 0.4
Ordinary	1/1000	NA	1.0
High-risk	1/2000	NA	1.2
Essential	1/5000	1.0	1.5

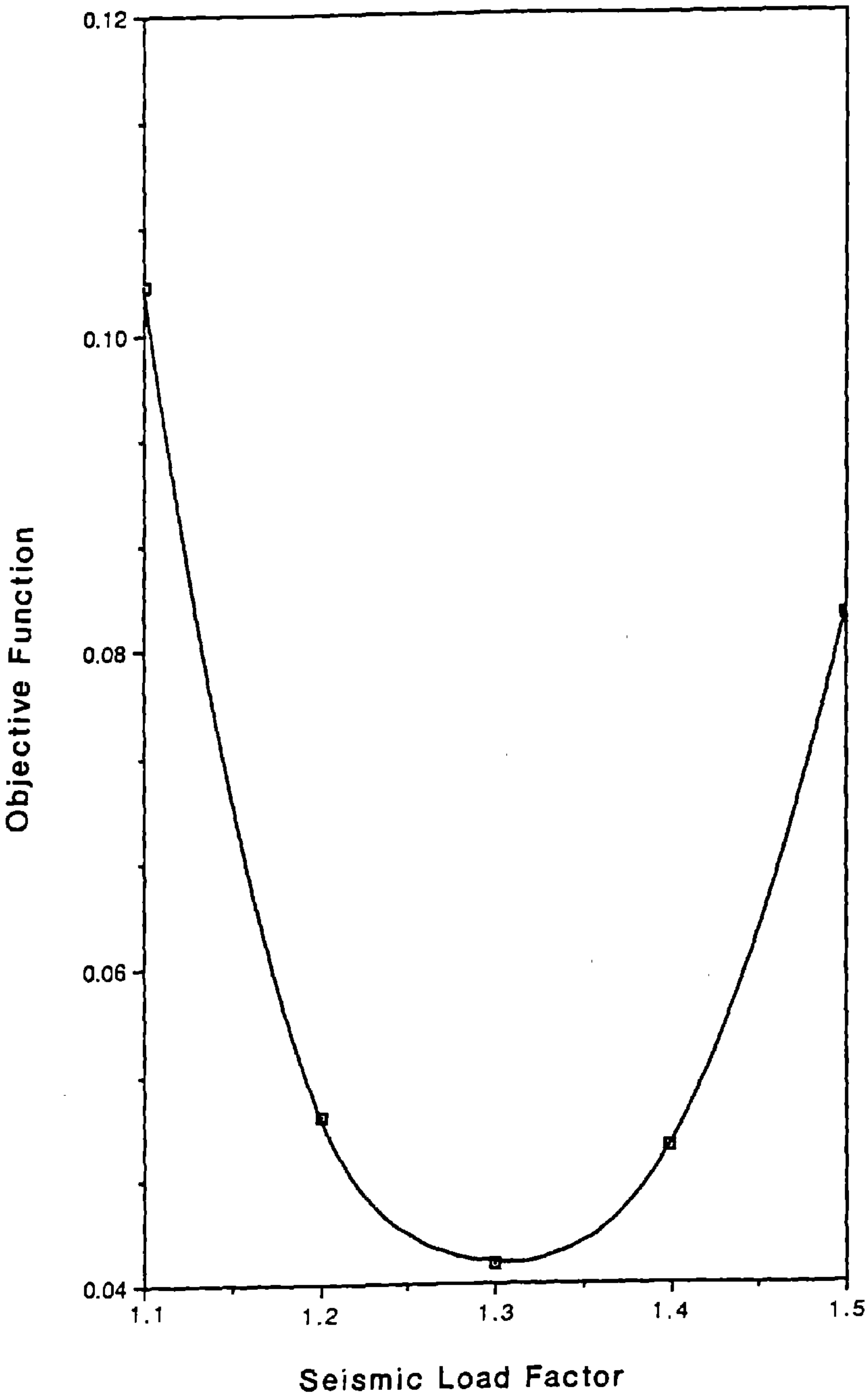


FIGURE 4-6 Determination of Seismic-Load Factor for Ordinary Buildings (E_D= 0.4 g)

4.4.6 Summary of Proposed Seismic LRFD Criteria

In this study, buildings are classified into three categories: ordinary, high-risk, and essential buildings. The intermediate moment-resisting frame as specified in the ACI code 318-89 is used to provide seismic resistance. The design base shear V is determined as :

$$V = I \left(\frac{ZC}{R_\mu}\right) W \tag{4.11}$$

and

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \tag{4.12}$$

where Z is the seismic zone factor, which is equivalent to the peak ground acceleration in g with soft rock as a reference site. S is the site coefficient and its values for S_1, S_2, and S_3 site conditions are 1.0, 1.2, and 1.5, respectively. I is the importance factor and the recommended value is given in Table 4-VIII for each category of building and seismic zone factor. R_μ is the elastic-to-inelastic response factor. For the IMR frame constructed according to the ACI code 318-89, the R_μ factor is set as 2.5. T is the structural period determined by using the formula as specified in the NEHRP Provisions, and W is the total seismic dead load.

For seismic zones of $Z > 0.1$, the frame structures need to be designed according to the following load combinations, including seismic load effects:

$$\phi R \geq 1.2 D + 0.5 L \pm \gamma_E E \tag{4.13}$$

$$\phi R \geq 0.9 D \pm \gamma_E E \tag{4.14}$$

where $\gamma_E = (0.7 + 1.5 Z) \geq 1.0$. The resistance factor ϕ is that specified in the ACI code 318-89. For seismic zones of $Z \leq 0.1$, the above-mentioned load combinations are required only for essential buildings. Ordinary and high-risk buildings need to be designed using the following load combinations, including only gravity load effects:

$$\phi R \geq 1.4 D \tag{4.15}$$

$$\phi R \geq 1.2 D + 1.6 L \tag{4.16}$$

It is noted that even though the structure is not designed for the seismic load in this case, detailing of members should follow the requirement for the IMR frame specified in the ACI code 318-89.

4.5 Conclusions

Reliability-based design procedures for seismic load are proposed which take into consideration explicitly the uncertainties in the load and resistance and satisfy the requirement that probabilities implied of limit states of concern meet prescribed target levels. The methods are demonstrated by design of low-rise steel buildings in Los Angeles and reinforced concrete buildings in four different earthquake zones. The design generally follows that of the 1992 NEHRP recommendations. The load factors and importance factor which have a direct bearing on the reliability of the systems, however, are calibrated according to the target limit state probabilities. Both serviceability and ultimate limit states are considered for this purpose; therefore, it is a bi-level reliability-based design. The results show that the calibration can be done using such a procedure. It is feasible, therefore, to use such a methodology to incorporate reliability into the provisions and develop a performance-assurance building code for seismic load.

SECTION 5

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APPENDIX A
RESPONSE SURFACE METHOD IN RELIABILITY BASED DESIGN

Response Surface Methodology (RSM) is a collection of statistical analysis methods and techniques which examines the relationship between experimental response and variations in the values of input variables. It was developed by research scientists performing experiments in biology and in agriculture (Box, 1978), although its use in other scientific fields has grown considerably in recent years. Sir Ronald Fisher is credited as the principal innovator to use statistical methods in experimental design. RSM's initial development in agriculture and in biology are reflected in its terminology; e.g., a configuration of input variable values is referred to as a "treatment," since early experiments often dealt with the effects of chemical treatments on plant growths. Similarly, input values of each independent variable are referred to as "levels." Principal among the techniques developed in RSM are ANOVA (analysis of variance) and the many classes of experimental designs which optimize the fitting of a response surface model using relatively few sampling points.

The motivation underlying RSM is to create and analyze statistical models of processes which are difficult to study directly, using data which is generally expensive to produce. This motivation is applicable to the problems encountered here in this study, i.e. to find the probability of failure of complex structural systems exposed to a variety of load combinations. Many civil engineering researchers have developed or adapted reliability methods to utilize the efficiency of RSM. RSM is also used to optimize a response quantity which is influenced by several independent variables, because it provides simple models of complicated processes. This Appendix provides a quick overview of the components of RSM which influence the proposed method. Details of this method can be found in Yao and Wen (1993).

The part of RSM which is of greatest value is the response surface design. There are several classes of experimental design which attempt to create predictive models from available information. Given specified variations in the system parameters and their corresponding system response values, each class of experimental design attempts to find the best coefficients of a specific model formula. In the context of the problem of this study, the system parameters are the load factors and the response surface is the objective function given in Eq. 2.5, a quadratic function of the limit state probabilities which depend on the load factors. Petersen (1985) lists six considerations for selecting the appropriate design. These are:

- estimate the coefficients of the equation of the surface
- assess the reliability of the estimates
- minimize variance
- minimize bias
- measure lack of fit
- add a minimum number of points, if necessary, to estimate higher order coefficients

One of the conceptually simpler designs is the factorial design. Factorial designs are conceptually simple, brute–force designs where response values are sampled at equal intervals for each variable. They are called factorial designs since they take all possible combinations of the levels selected from each factor. For example, a 2^k factorial design basically samples each of k factors (system variables) at two levels. Figure A.1 displays a 2^k factorial design for $k = 3$ variables. The resulting response surface model for a 2^k factorial design is hyperplanar.

In order to properly fit a second–order surface for k input variables, experimental designs must have at least three levels for each variable. One such design is the 3^k factorial design which requires 3^k points (response observations) to fit the $1 + 2k + \frac{1}{2}k(k - 1)$ coefficients of the response surface polynomial. For k=5 variables, this means that 125 points are required to fit 21 coefficients. Figure A–2 shows the configuration of a 3^k factorial design for $k = 3$.

In 1951, G. E. P. Box and K. B. Wilson introduced a much more efficient class of designs for fitting second–order surfaces (Figure A–3). These central composite designs consist of a 2^k factorial design (with each factor at two levels: -1, $+1$) augmented by n_2 center points and 2k axial or "star" points (where each factor, in turn, is set to $-\alpha$ and $+\alpha$). The number of center points, n_2, and the value of α are chosen by the experimenter to attain certain design properties.

The central composite class of designs is used in this study. The central composite design adds to the 2^k design several star points and center points. The center points make it possible to measure the pure error in the system response. The star points allow the design to fit higher order surfaces without the cost associated with a 3^k factorial design; each factor is measured at three levels by sharing the information of the star points and center points.

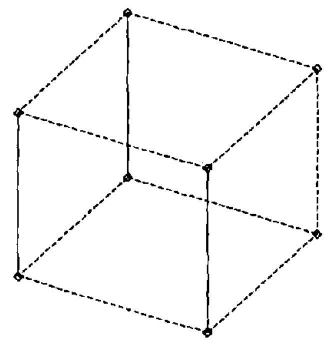


FIGURE A−1 2^k **factorial design for k = 3 variables**

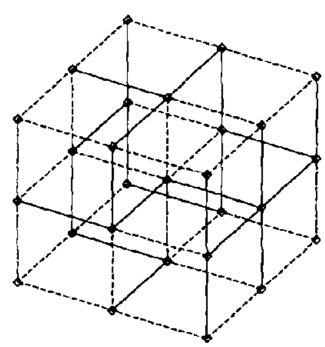


FIGURE A−2 3^k **factorial design for k = 3 variables**

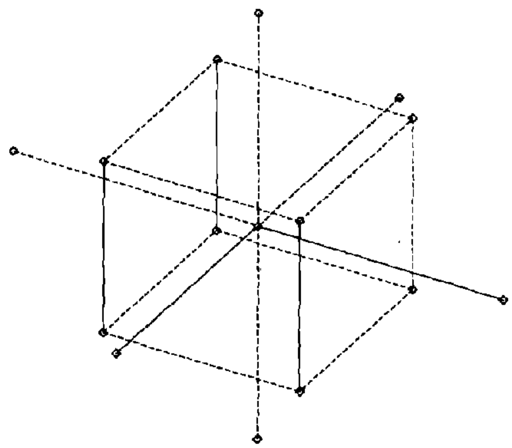


FIGURE A−3 Central composite design for $k = 3$ variables

The general expression for the central composite design is described by the model:

$$E(y) = \beta_0 + \sum_{i=1}^{k} \beta_i x_i + \sum_{i=1}^{k} \beta_{ii} x_i^2 + \sum_{i=1}^{k} \sum_{j=1}^{i-1} \beta_{ij} x_i x_j \quad \text{(A.1)}$$

where

y = response

$E(y)$ = expected value of the response

x_i = i^{th} variable of k variables

Let F = number of factorial points

Let $T = 2k + n_2$ = number of axial points + number of center points

Let $c = \dfrac{F + 2\alpha^2}{F + T}$

A matrix **X** may then be formed (Table A–I) such that the least squares estimates, **b**, of the parameters, $\boldsymbol{\beta}$, of the model may be calculated by $\mathbf{b} = (\mathbf{X}^T\mathbf{X})^{-1}\mathbf{X}^T\mathbf{y}$. **X** (of dimension $n \times n_b$, where $n_b = 1 + 2k + \frac{1}{2}k(k-1)$ and n=number of points used to fit the response surface) consists of the experimental design matrix (of dimension $n \times n$) which is used to find the vector **y** (of dimension n x 1) of the response plus additional terms associated with the other coefficients of the fitted polynomial and based upon the terms of the design matrix.

The three design properties which are available with the proper combination of n_2 and α are orthogonality, rotatability, and uniformity of precision. With orthogonality, $\mathbf{X}^T\mathbf{X}$ becomes diagonal and the response surface coefficients are uncorrelated. In order to achieve orthogonality, α, the coordinate of the "star" points in the normalized parameter space, must be selected so that $FT - 4F\alpha^2 - 4\alpha^4 = 0$.

TABLE A–I General representation of the X matrix for central composite design (Petersen, 1985)

x_0	x_1	x_2	...	x_{kk}	x_{11}	x_{22}	...	x_{kk}	x_{12}	x_{13}	...	$x_{k-1,k}$
1	−1	−1	...	−1	1−c	1−c	...	1−c	1	1	...	1
1	1	−1	...	−1	1−c	1−c	...	1−c	−1	−1	...	1
1	−1	1	...	−1	1−c	1−c	...	1−c	−1	1	...	1
:	:	:		:	:	:		:	:	:		:

1	0	0	...	0	$-c$	$-c$	...	$-c$	0	0	...	0
1	$-\alpha$	0	...	0	(α^2-c)	$-c$	...	$-c$	0	0	...	0
1	α	0	...	0	(α^2-c)	$-c$	...	$-c$	0	0	...	0
1	0	$-\alpha$	...	0	$-c$	(α^2-c)	...	$-c$	0	0	...	0
1	0	α	...	0	$-c$	(α^2-c)	...	$-c$	0	0	...	0
:	:	:		:	:	:		:	:	:		:
1	0	0	...	$-\alpha$	$-c$	$-c$	...	(α^2-c)	0	0	...	0
1	0	0	...	α	$-c$	$-c$	...	(α^2-c)	0	0	...	0

Rotatable designs ensure that the variance of the estimated response at a point is a function only of the distance of that point from the center of the design and does not depend on the orientation of the design to the true response surface. For central composite designs incorporating a full 2^k factorial set of points, α must be $2^{\frac{k}{4}}$ to achieve a rotatable design.

If the variance of the response is constant for a distance of 1 (in the normalized factor space) from the center of the design, then that design is considered to have uniform precision. By judicious selection of the number n_2 of center points used, a rotatable CCD (central composite design) may gain uniform precision or orthogonality.

The advantage of fitting the objective function with Eq. A.1 is obvious. The minimization problem becomes trivial since it is with respect to a second order polynomial. Also only a fixed number of data points are required whereas in a nonlinear programming algorithm the search for the minimum may require a large number of iterations and in each iteration redesign and reanalysis of reliability of the structure are needed.

APPENDIX B

AN EFFICIENT METHOD FOR RELIABILITY EVALUATION OF MDOF INELASTIC STRUCTURES

Introduction

In the reliability-based design procedure as proposed in Eq. 2.5, it is necessary to search for the optimal design parameters and in the process repeated solutions for the reliability under seismic and other loads are required. For multi-degree-of-freedom (MDOF), inelastic structures, the computation can become excessive since a large number of response time history analyses are necessary for each combination of building type (story height and structural frame system) and design parameters (e.g. load factors, drift limit) and the number of such combinations needed can be also large. To alleviate this computational difficulty, an approximate method is developed in which the MDOF inelastic system is replaced by a simple equivalent nonlinear system (ENS) which retains the most important properties of the original system, i.e. the dynamic characteristics of the first two modes and the global yielding behavior of the MDOF system. The system response is described by the maximum global (building) and local (interstory) drifts. The equivalency is achieved by the use of two response modification factors, a global response factor R_G , and a local response factor R_L , applied to the responses of the ENS to match those of the original MDOF system. These response modification factors are obtained by extensive regression analysis based on results of responses of the MDOF system and the ENS to actual ground accelerations recorded in past earthquakes. The method is summarized in the following. Details can be found in Han and Wen (1994).

Equivalent Nonlinear System (ENS)

The ENS is defined as a system consisting of two SDOF systems which have dynamic properties (natural frequency, mode shape, and modal participation factor) same as those of the first two modes of the original system. It is assumed that both SDOF systems have an yield displacement equal to the global yield displacement associated with the MDOF structure. The global yield displacement is determined by a static pushover analysis in which a linear vertical distribution of the lateral force is assumed and the displacement at the top of the building is monitored. The well known and well-tested DRAIN -2DX is used in the pushover analysis. Since for MDOF systems the yielding of the system occurs incrementally, the point of intersection of the pre-yielding and post-yielding slopes in the force-deflection curve is taken as the yield point and based on which the global yield displacement is determined. Figure B-1 shows an example of such an analysis for a two-story steel special moment frame. A restoring force model (e.g. bilinear system) is then used for the inelastic response analysis of the two SDOF systems under the excitation of ground acceleration. Again the displacement at the top of the building is monitored. The analysis is otherwise the same as a modal response analysis and the displacements of the two SDOF systems are then combined by modal superposition. It is fully recognized that modal analysis is strictly valid only for linear systems. For many structures, however, it has been observed that deviation from linear behavior may not be severe and one can still take advantage of the modal analysis to obtain an approximate solution which is what we need in this study.

Response Modification Factors

A parallel dynamic analysis of the original MDOF system under the same ground acceleration is carried out using DRAIN-2DX. The global response modification factor, R_G , is defined as the ratio of the maximum displacement at of top of the ENS (G_e) to that of the original MDOF structure (G_m) as follows:

$$R_G = \frac{G_e}{G_m} \tag{B.1}$$

The local response modification factor, R_L , is defined as the ratio of the maximum global ductility to the local (interstory drift) ductility of the original MDOF structure as follows:

$$R_L = \frac{G_m / Y}{d^i_{max} / y^i} \tag{B.2}$$

in which d^i is interstory drift at the i-th story; y^i is the i-th story yield displacement; and Y is the global yield displacement. The local yield displacement is obtained by the same procedure as for the global yield displacement. Figure B-2 illustrates how the local yield displacement is determined for a two-story building. In general, R_G and R_L depend on parameters such as the structural properties, characteristics of the excitation and the severity of the response. Since considerable amount of uncertainty exists in the seismic excitation, exact functional relationships of these two factors in terms of the parameters would be difficult to obtain; instead, approximate relationships are established by extensive regression analyses as follows.

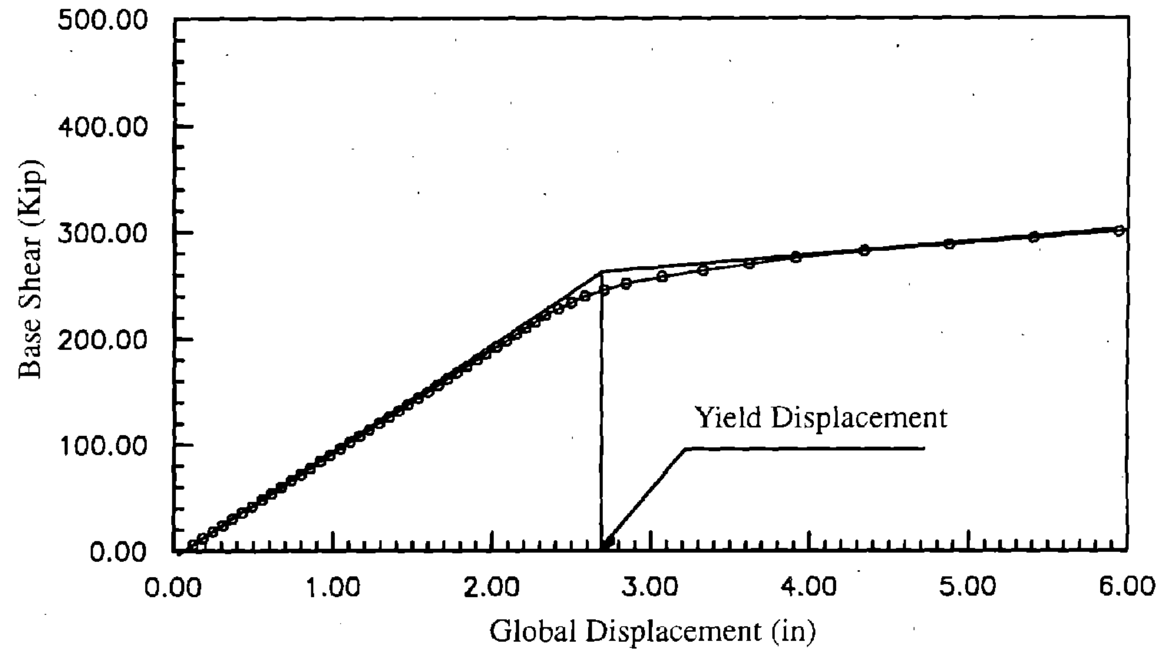


FIGURE B−1 Global Yield Displacement for Two Story SMRSF

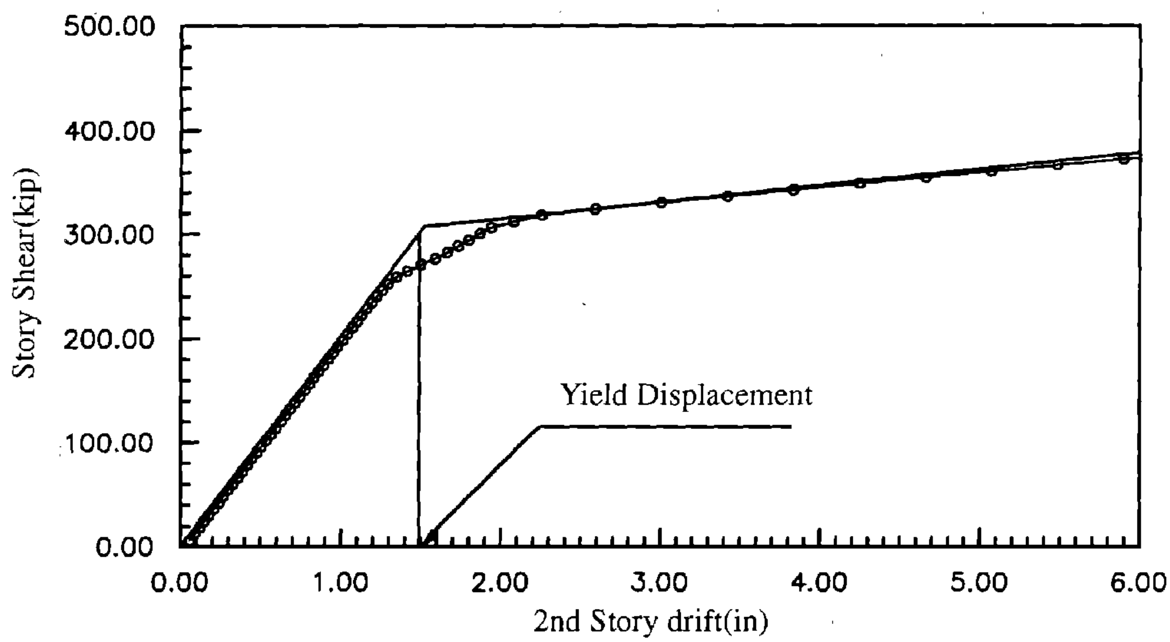


FIGURE B−2 Local Yield Displacement for Two Story SMRSF

Regression Analysis of the Response Modification Factors

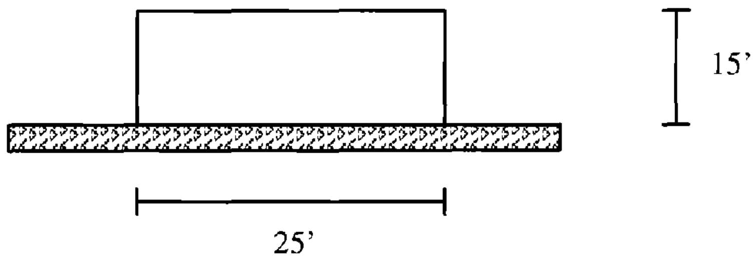
It is assumed that the response modification factors can be modeled by polynomial functions of the global ductility factor (μ) and the modal participation factors (P_i). In this study only steel Special Moment Resistant Space Frames (SMRSF) are considered. Seven typical office buildings located in UBC Zone 4, ranging from one to four bays and one to twelve stories are designed according 1988 UBC and the AISC Allowable Stress Design Manual. Figures B-3 to B-9 show the perimeter frames of these structures. In the dynamic analysis, a 5 % damping ratio is used according to the values in the current codes and provisions for buildings such as SEAOC , UBC and NEHRP. A strain hardening ratio of 5 % is assumed for individual members in the dynamic analysis of the MDOF and a 10 % value is used in the ENS to represent the hardening effect at a global level which may be on the conservative side (Osteraas and Krawinkler 1990). A suite of eighty eight real earthquake records are used in the dynamic response time history response analyses for the calibration of R_G , and R_L . The records consist of moderate to severe ground acceleration time histories in North America since 1933 including those of the recent North Ridge earthquake, 6 earthquakes in Japan, Chilean, and Mexico earthquakes. The magnitude of the earthquakes range from 4.4 to 8.1, peak ground acceleration from 0.03 to 1.17 g , and the source distance from 0 to 400 km. The characteristics of the excitations considered, therefore, cover a range which is wide enough for this calibration study. The global response modification factor is expressed as function of global ductility factor μ as follows:

$$R_G = C_0(\gamma) + C_1(\gamma)\mu + C_2(\gamma)\mu^2 \qquad \text{(B.3)}$$

in which C_0 , C_1 , and C_2 are assumed to be linear functions of the modal participation factor ratio γ defined as

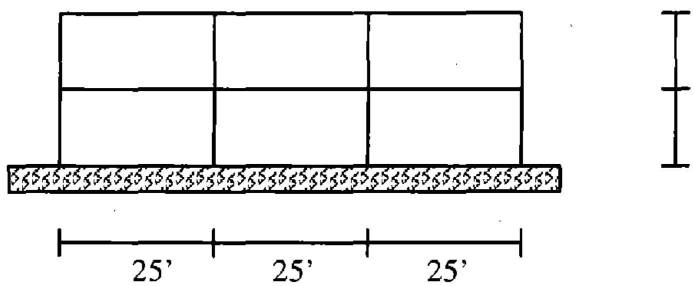
$$\gamma = \frac{|P_1| + |P_2|}{\sum_{i=1}^{n} |P_i|} \qquad \text{(B.4)}$$

in which P_i is i-th modal participation factor. To determine the above coefficients, the empirical values of the R_G for the above seven buildings under the excitation of all the 88 acceleration records are first calculated using the program DRAIN-2 DX. A two-stage regression analysis is then performed; in the first stage, coefficients C's are determined for each of the seven structures



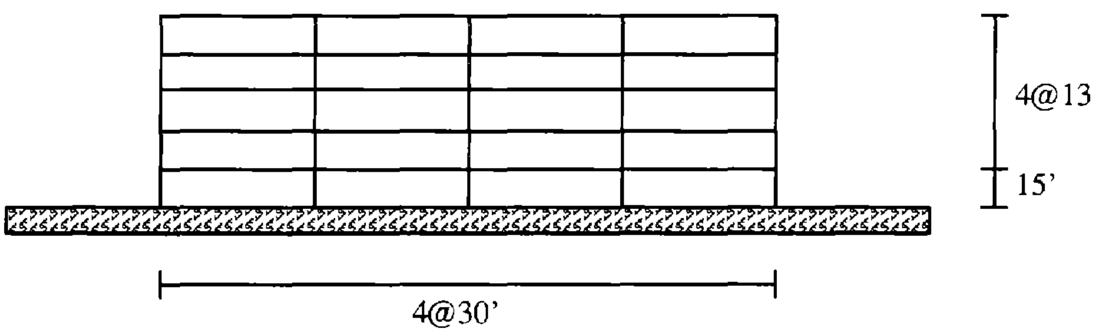
Weight of each node : 150 kip

FIGURE B−3 1 Story SMRSF



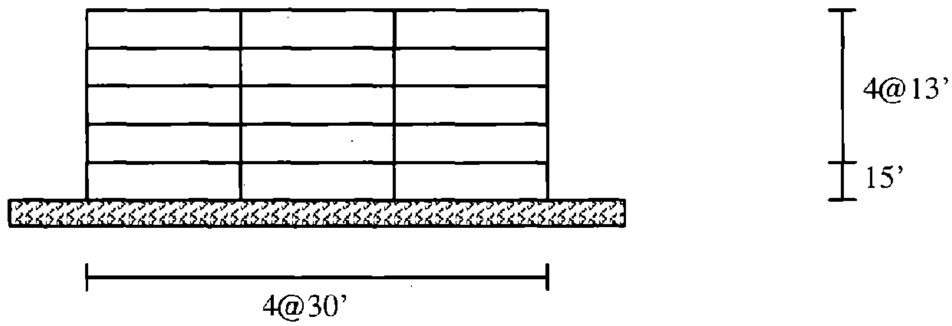
Weight of each node : 163,12 kip (1st floor)
128.25kip (2nd floor)

FIGURE B−4 2 Story SMRSF



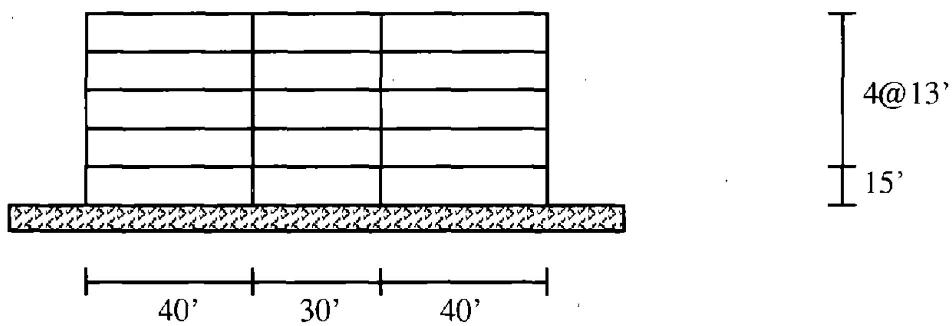
Weight of each node : 156kip(1st fl.), 154kip(2nd–4th fl.),133kip(5th fl.)

FIGURE B−5 5 Story SMRSF 1



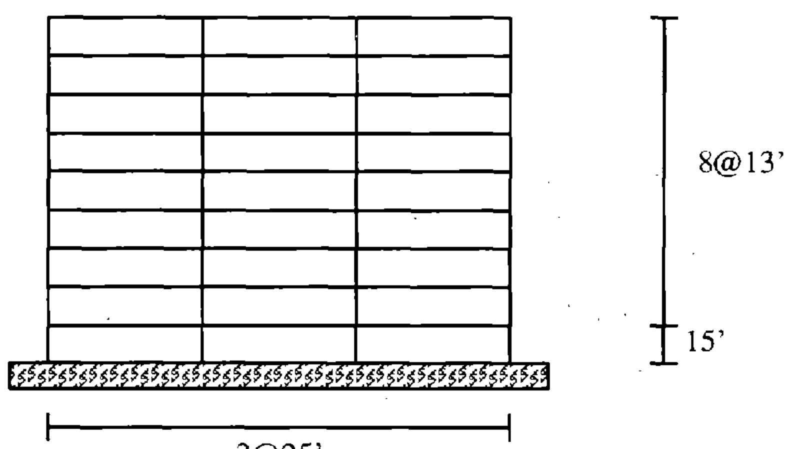
Weight of each node : 273kip(1st fl.), 271kip(2nd–4th fl.),233kip(5th fl.)

FIGURE B−6 5 Story SMRSF 2



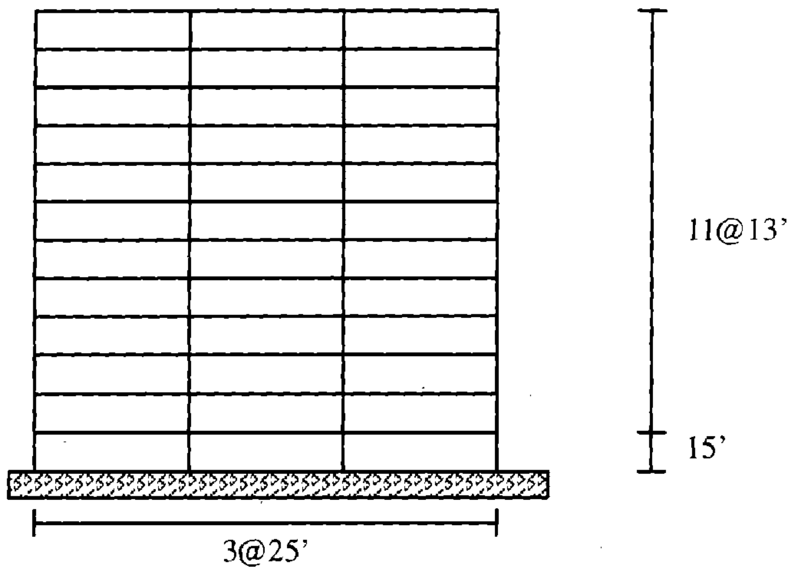
Weight of each node : 235kip(1st–4th fl.), 198kip(5th fl.)

FIGURE B−7 5 Story SMRSF 3



Weight of each node : 163kip(1st fl.),162kip(2nd–8th fl.),128kip(9th fl.)

FIGURE B−8 9 Story SMRSF



Weight of each node : 163kip(1st fl.),162kip(2nd–11th fl.),128kip(12th fl.)

FIGURE B−9 12 Story SMRSF

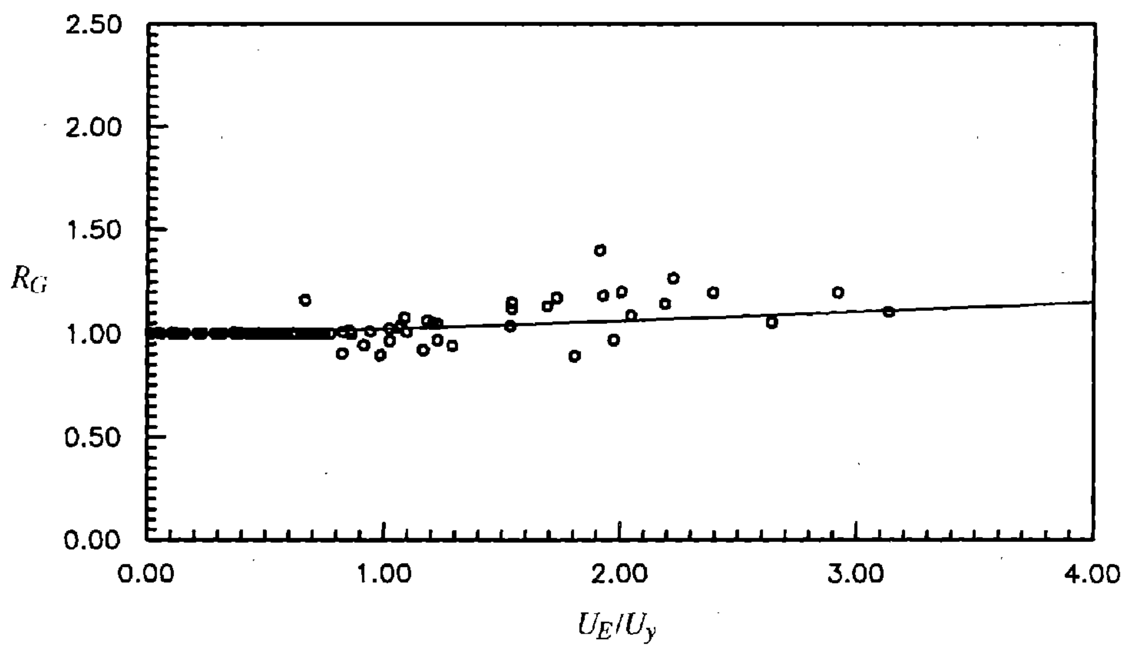


FIGURE B−10 Global Response Scaling Factor R_G For Two Story SMRSF

with a specified γ value; and in the second stage, a regression analysis of the coefficients on γ is carried out. The results of the two-stage regression analysis are:

$$C_0 = 0.9695 + 0.0178\,\gamma \tag{B.5}$$

$$C_1 = -0.1664 + 0.2016\,\gamma \tag{B.6}$$

$$C_2 = 0.1473 - 0.1467\,\gamma \tag{B.7}$$

Figures B-10 to B-13 show the comparison of the data points with the regression results given by Eqs. B-3 to B-7. It is seen that for μ less than 0.5, the variability of R_G is small and can be neglected. For μ greater than 0.5 the coefficient of variation of the R_G for the 1,2,5,9, and 12 story structures are 5 %, 6 %, 10 %, 11 %, and 10 % respectively. A constant coefficient of variation of 10 % is therefore assumed in the evaluation of limit state probability based on the use of the response modification factors.

A similar two-stage regression analysis is carried out to calibrate the local response modification factor R_L. It is found that the variation of R_L with μ is small and can be neglected. R_L is hence given by linear function of γ only as follows:

$$R_L = 0.3627 + 0.4774\,\gamma \tag{B.8}$$

Figure B-14 shows the comparison of regression equation of R_L with data points. The variability of R_L is neglected in the limit state probability evaluation.

Reliability Evaluation Using ENS

The performance of buildings under seismic excitation can be described by limit states such as yielding, excessive deflection, instability, buckling , damage, and collapse. Among these measures the drift limits are the most commonly used and most amenable to analysis. The performance of the building can be described in terms of drift limits being exceeded. For example, in many code provisions limit states have been expressed in terms of local (interstory) and global (system) drifts. To include the uncertainties in the problem into the consideration, the performance can be described in terms of the probability of certain drift limits being exceeded over a given time period as follows:

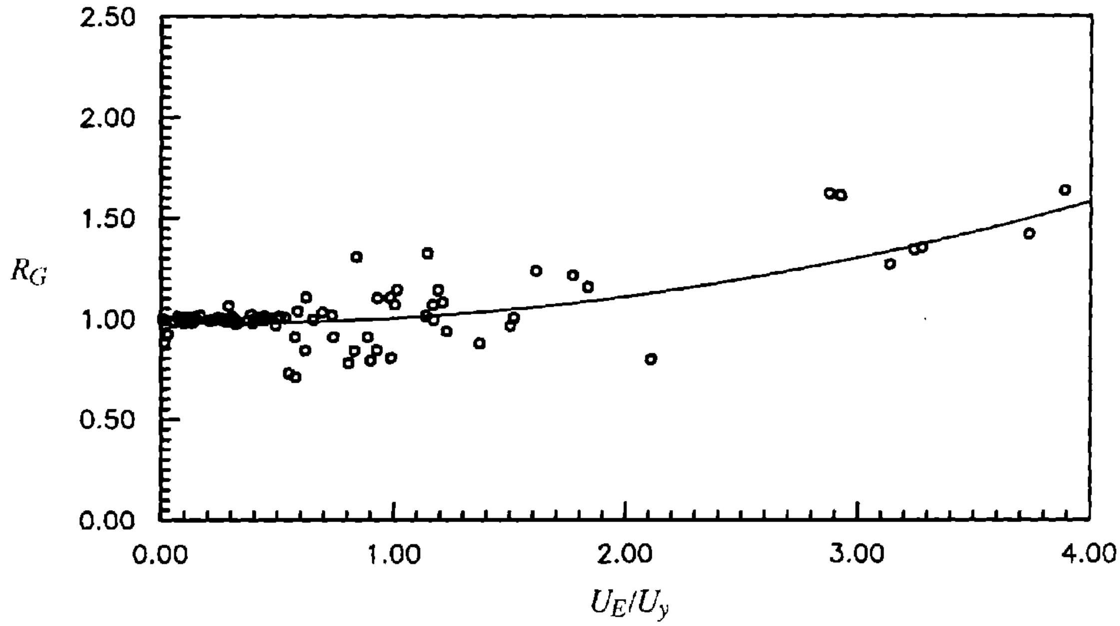


FIGURE B–11 Global Response Scaling Factor R_G For Five Story SMRSF 1

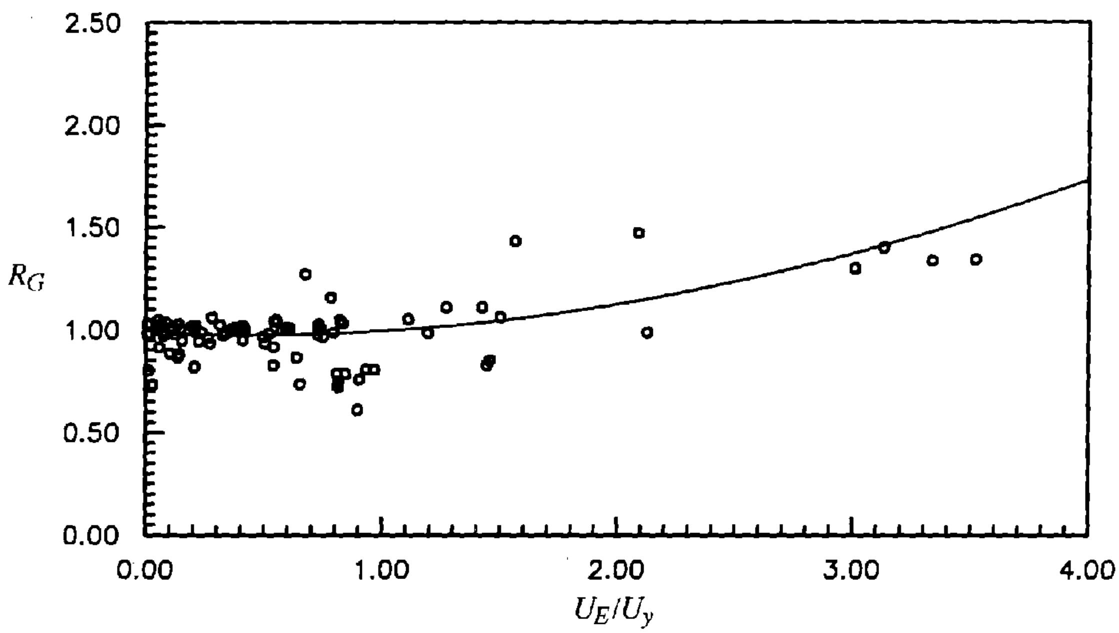


FIGURE B–12 Global Response Scaling Factor R_G For Nine Story SMRSF

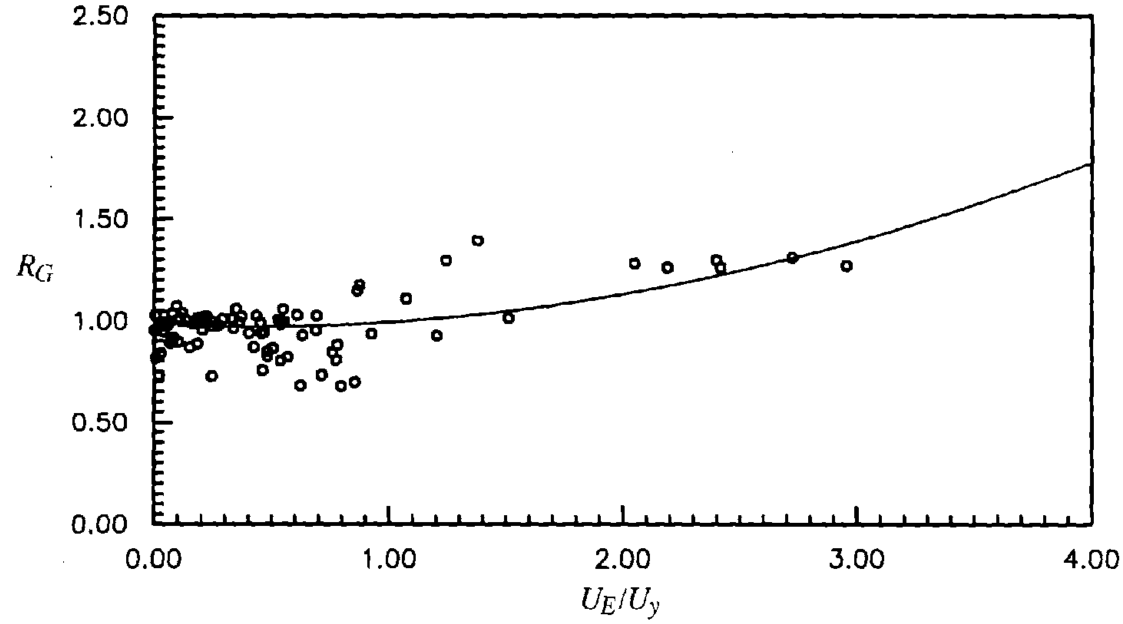


FIGURE B−13 Global Response Scaling Factor R_G For Twelve Story SMRSF

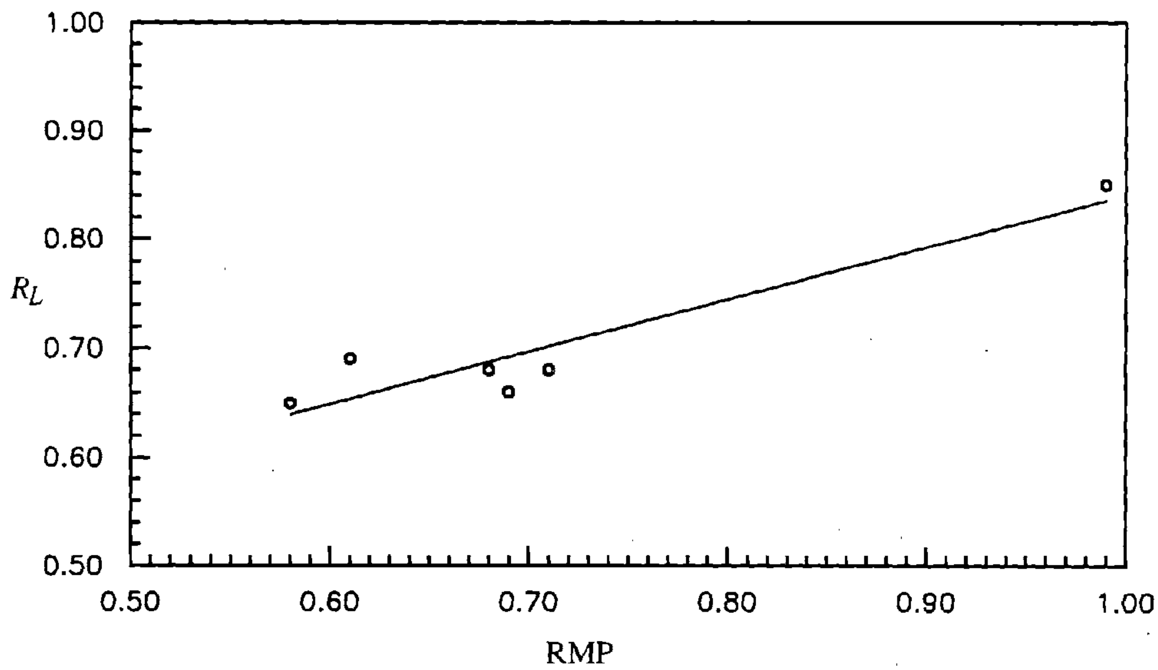


FIGURE B−14 Regression of R_L w.r.t. RMP

$$P_G = P\left(\frac{G_m}{H} > G_0\right) \tag{B.9}$$

$$P_L = P\left(\frac{d^i_{max}}{h^i} > L_0\right) \tag{B.10}$$

in which h^i and H are the story height and the height of the building respectively; G_0 and L_0 are the limits for global and local drifts. For example, a value of 0.005 has been suggested for G_0 and 0.015 for L_0 as ultimate limit in 1988 UBC. One can evaluate the response of the ENS which is computationally easy to do and convert it to that of the MDOF structure using the response modification factors. The global limit state probability can be expressed in terms of the response of ENS as follows:

$$P_G = P\left(\frac{G_e}{R_G H} > G_0\right) \tag{B.11}$$

Similarly, with aid of Eqs. A-3 and A-11, the local limit state probability can be expressed as

$$P_L = P\left(\frac{G_e}{R_G R_L H} > L_0\right) \tag{B.12}$$

in which the following is assumed

$$\frac{y^i}{Y} \approx \frac{h^i}{H} \tag{B.13}$$

Eq. B-13 is an approximate relationship which simplifies the calculation since the story yield and story height need not be considered. The advantage of formulation through Eqs. B-11 and B-12 is of course that one needs to do large number of response time history analyses of the ENS only. Since the method proposed is approximate, the accuracy is verified before it can be applied to reliability-based design in the following.

The accuracy of the proposed method is verified by comparison of the limit state probabilities with those for the original nonlinear multi-degree-of-freedom systems, hereafter referred to as NMS. The seven steel special moment frames in the foregoing are used for this purpose. The sites chosen are downtown Los Angeles (Santa Monica Blvd) and Imperial Valley as in the previous study (Wen et al, 1992). The Santa Monica Blvd site is 60 km from the Mojave segment of the Southern San Andrea fault and the Imperial Valley site is 5 km from the Imperial fault. Future earthquakes are modeled either as characteristic or non characteristic event. The seismic risk analysis and ground motion modeling follow that of Wen et al (1992). A large number of ground acceleration time histories are simulated according to the risk and ground motion models and from which the response time histories of both the NMS and ENS are calculated. The limit state probabilities in terms of drift levels being exceeded in the 50-year time window after 1994 are then evaluated based on the proposed procedure and compared with those based directly on the NMS. Both local and global limit state probabilities are calculated. Drift levels up to 3 % of story or building height are considered. For a steel frame building, at 3 % drift level it will probably suffer severe nonstructural or even structural damages. The results are shown in Figures B-15 to B-18. It is seen that the agreements are generally satisfactory, the results for the LA site are specially encouraging.

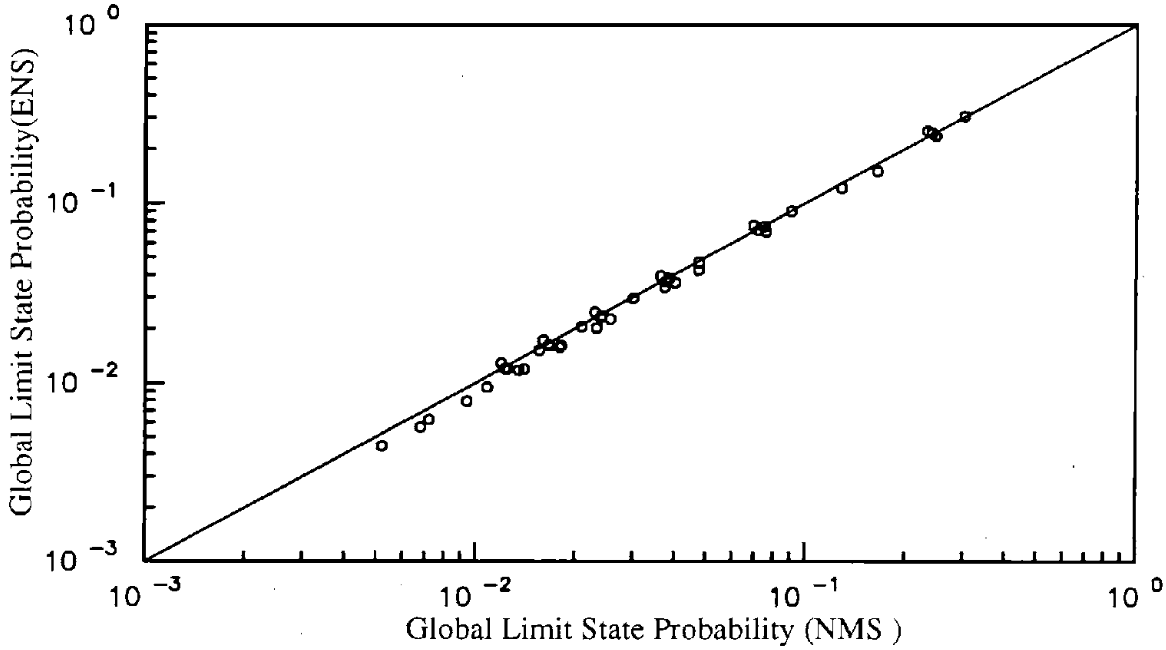


FIGURE B−15 Comparison of Global Limit State Probabilities of ENS and NMS (L.A. site)

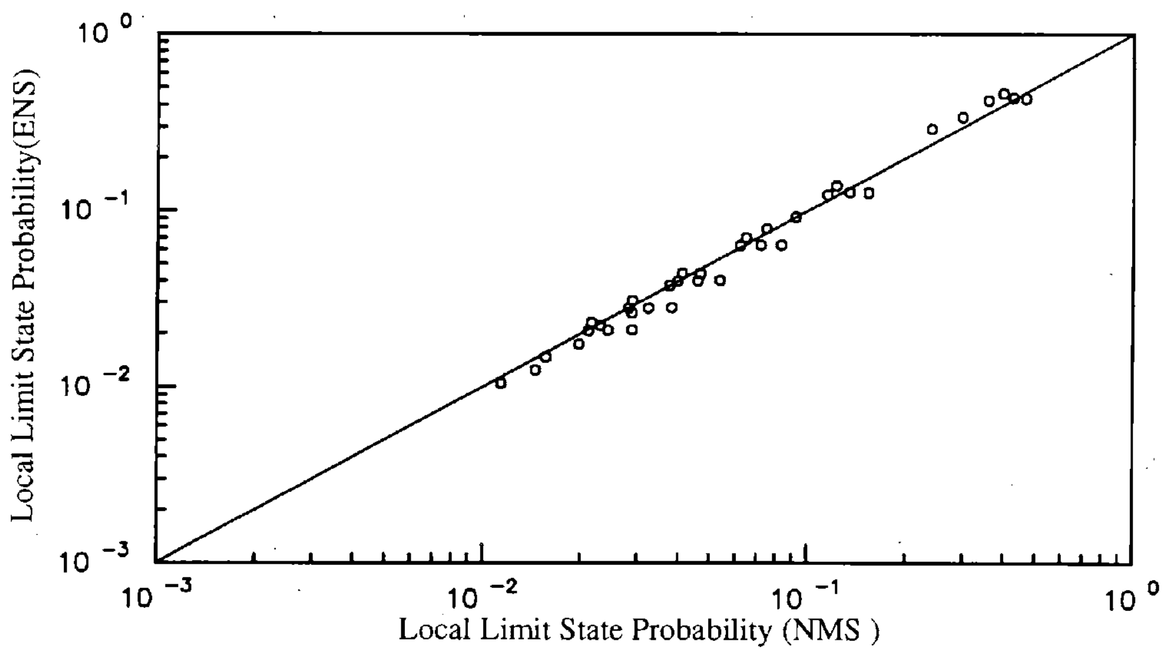


FIGURE B−16 Comparison of Local Limit State Probabilities of ENS and NMS (L.A. site)

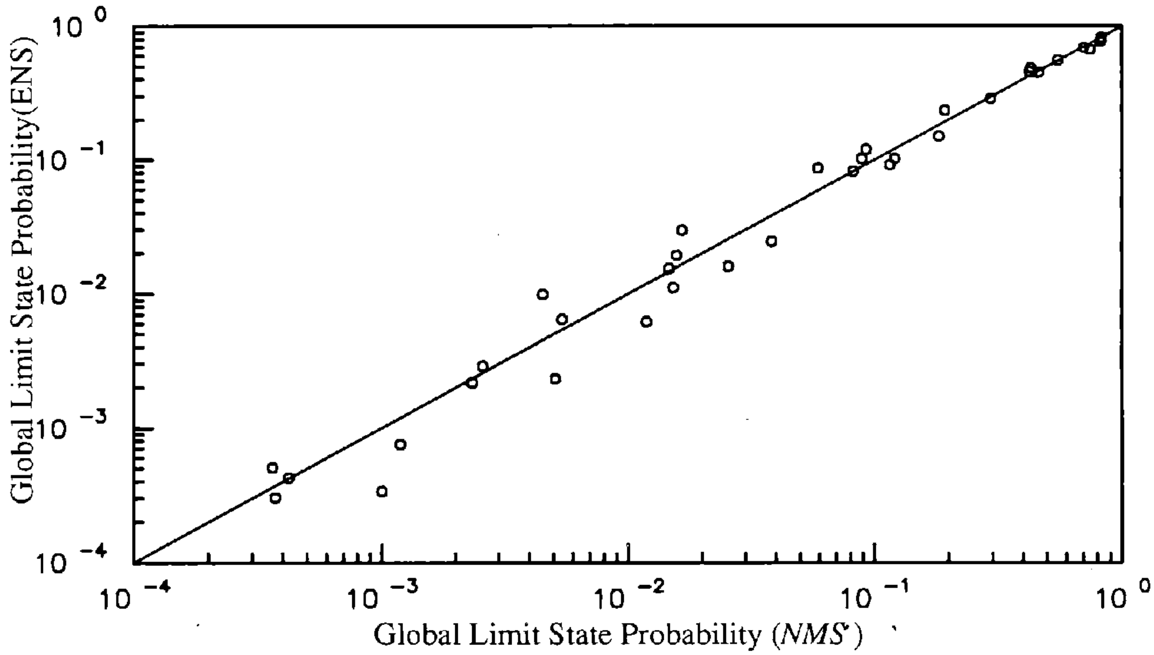


FIGURE B−17 Comparison of Global Limit State Probabilities of ENS and NMS (Imperial Valley site)

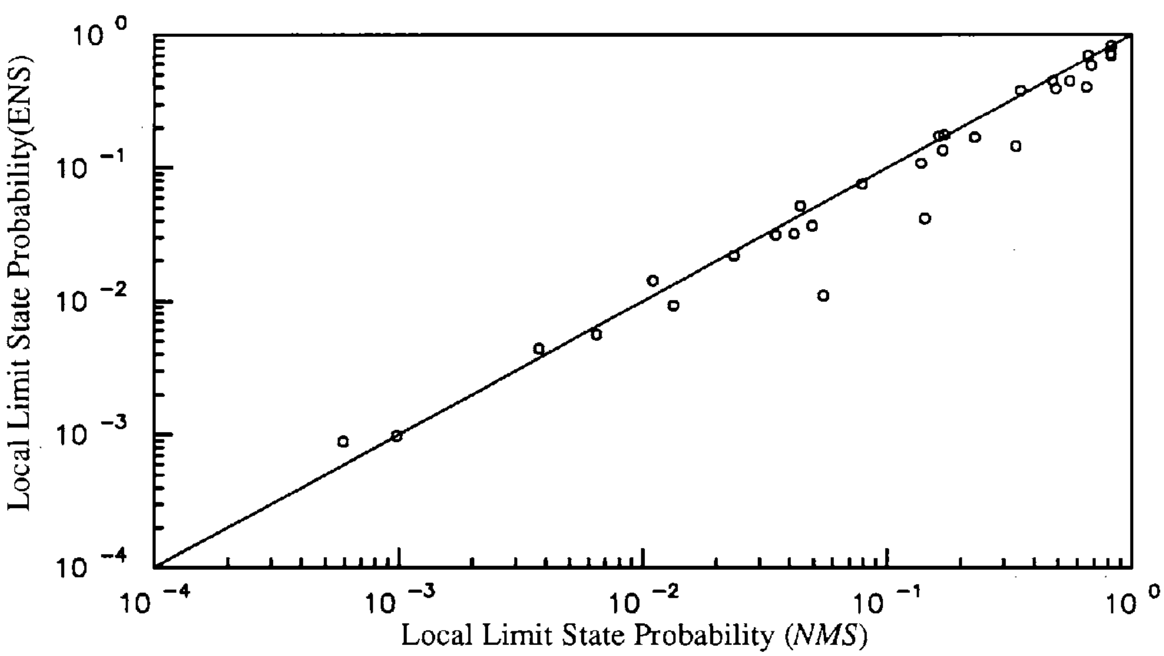


FIGURE B−18 Comparison of Local Limit State Probabilities of ENS and NMS (Imperial Valley site)

APPENDIX C
AN EVALUATION OF THE RESPONSE MODIFICATION FACTOR

The buildings designed in accordance with current seismic building codes, such as the NEHRP Recommended Provisions (FEMA 1992), are expected to behavior in a nonlinear manner in the event of a large earthquake. The effect of inelastic deformation is introduced into the equivalent lateral force procedure by means of the response modification factor R and the deflection amplification factor C_d. The R factor is used to reduce the base shear from an elastic response level to a design strength level, while the C_d factor is used to estimate the nonlinear interstory displacement (story drift) from the corresponding linear value. In this appendix, an evaluation of the R factor is presented.

A building in general is considered as a multi-degree-of-freedom (MODF) system. For a MODF system, the response modification factor R is defined as the ratio of the absolute maximum elastic (linear) base shear to the design base shear. Following Hwang and Hsu (1991), the R factor is expressed as follows:

$$R = R_\mu R_{OS} R_{DS} \tag{C.1}$$

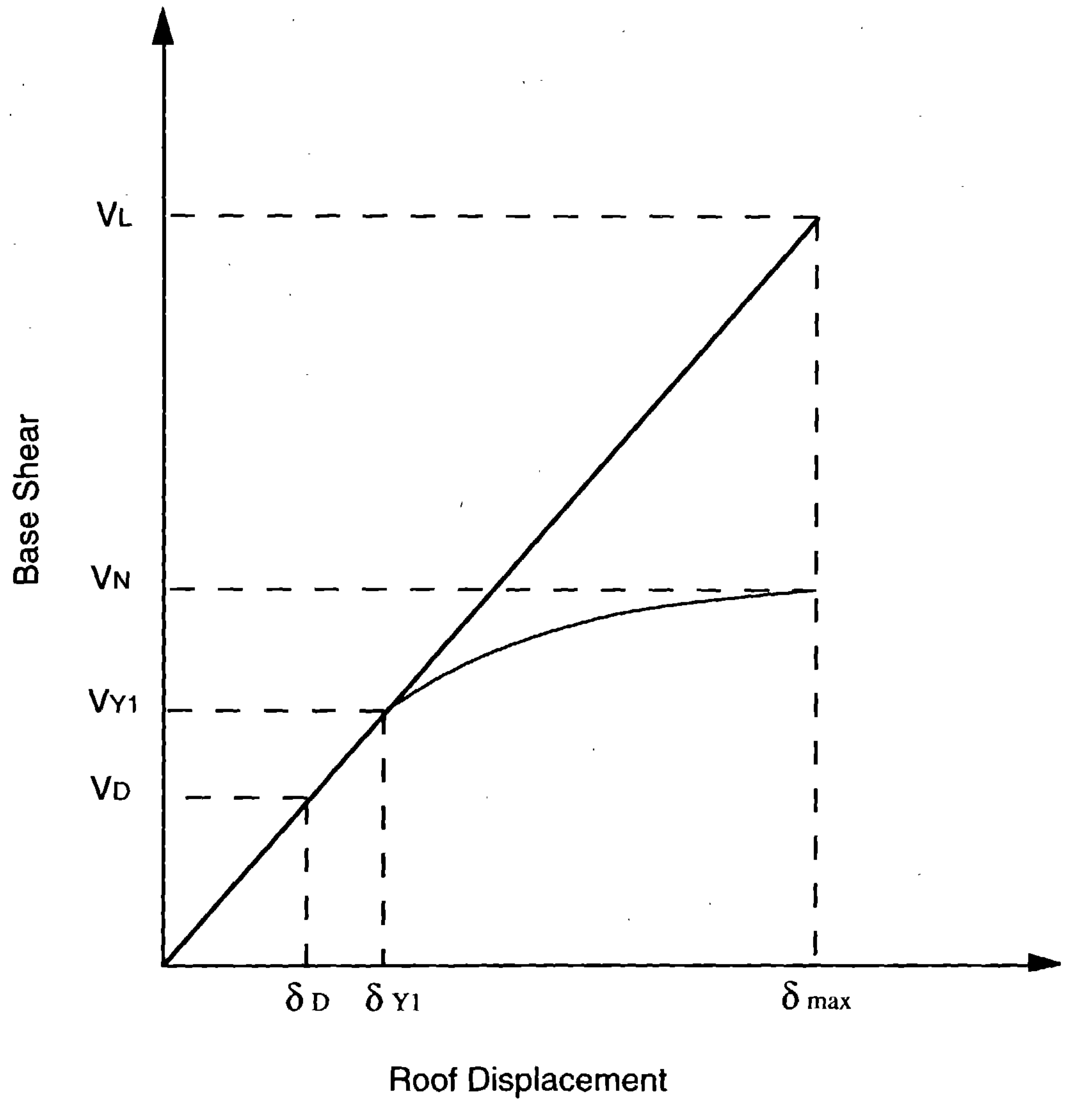
where R_μ is the elastic-inelastic response factor, which is defined as the ratio of the absolute maximum elastic base shear V_L to the absolute maximum nonlinear base shear V_N, when a structure is subject to the same earthquake. R_{OS} is the ratio of the maximum nonlinear base shear to the actual first-yielding base shear V_{Y1}, and R_{DS} is the ratio of the actual first-yielding base shear to the design base shear V_D. Figure C-1 illustrates the components of the R factor in Equation (C.1).

Assuming the R_μ factor is a lognormal variable and the R_{OS} and R_{DS} factors are deterministic, then the R factor is also a lognormal variable. The mean value of the R factor, i.e. $\bar{R}$, is determined as

$$\bar{R} = (R_{OS} R_{DS}) \bar{R}_\mu \tag{C.2}$$

where $\bar{R}_\mu$ is the mean value of the R_μ factor. The standard deviation of the R factor σ_R is determined as

$$\sigma_R = (R_{OS} R_{DS}) \sigma_{R_\mu} \tag{C.3}$$



$R_\mu = V_L/V_N$

$R_{OS} = V_N/V_{y1}$

$R_{DS} = V_{y1}/V_D$

FIGURE C-1 Components of the R Factor

where σ_{R_μ} is the standard deviation of the R_μ factor.

Hwang and Jaw (1989) investigated the elastic-inelastic response factor R_μ by simulating nonlinear and corresponding linear responses of multistory shear buildings subjected to synthetic earthquake time histories. From a regression analysis of the simulated data, the elastic-inelastic response factor R_μ is obtained as follows:

$$Ln(R_\mu) = [e^{-0.1857T} - e^{-2.1673T} - 0.0276\zeta] \, Ln(\mu_m) + \varepsilon \tag{C.4}$$

where ζ is the viscous damping ratio, μ_m is the maximum story ductility ratio, and T is the earthquake-structural period ratio, which is defined as

$$T = \frac{T_s}{T_g} \tag{C.5}$$

in which T_s is the fundamental period of structure and T_g is the dominant period of the earthquake motion.

The random variable ε in Equation (C.4) is used to represent the dispersion of data about the regression curve. Following Hwang and Jaw (1989), ε is modeled as a normal variable with zero mean and the standard deviation σ_ε as follows:

$$\sigma_\varepsilon = 0.113 \, Ln(\mu_m) \tag{C.6}$$

Since the R_μ factor is lognormally distributed, its standard deviation can be obtained as follows:

$$\sigma_{R_\mu} = \bar{R}_\mu \sqrt{\exp(\sigma_\varepsilon^2) - 1} \tag{C.7}$$

where $\bar{R}_\mu$ is the mean value of R_μ and can be obtained from Equation (C.4).

Hwang and Hsu (1991) evaluated the response modification factor for a RC special moment-resisting frame and an intermediate moment-resisting frame. The factors R_{OS} and R_{DS} for these two frames are listed in Table C-I. In addition, these two factors obtained by Bertero (1986) from the test results of a seven-story RC frame-wall structure are also shown in Table C-I.

TABLE C-I Values of R_{OS} and R_{DS} for Various Structural Systems

Structural system	R_{OS}	R_{DS}
SMR frame (Hwang & Hsu, 1991)	1.6	2.6
IMR frame (Hwang & Hsu, 1991)	1.7	1.4
Frame-wall (Bertero, 1986)	2.8	1.3

Using the values listed in Table C-I and Equations (C.2) and (C.3), the mean values $\bar{R}$ and standard deviations σ_R of the response modification factors for these three types of structural systems are obtained and shown in Table C-II. The mean values $\bar{R}$ in Table C-II are comparable to the response modification factor R specified in the NEHRP recommended provisions.

TABLE C-II Response Modification Factors for Various Structural Systems

Structural system	μ_m	2			4			6		
	$\frac{T_S}{T_g}$	0.5	1	2	0.5	1	2	0.5	1	2
SMR frame	$\bar{R}_\mu$	1.35	1.49	1.45	1.83	2.23	2.11	2.18	2.82	2.63
	$\bar{R}$	5.62	6.21	6.04	7.60	9.27	8.78	9.07	11.7	10.9
	σ_R	0.44	0.49	0.47	1.20	1.46	1.38	1.86	2.40	2.23
IMR frame	$\bar{R}_\mu$	1.35	1.49	1.45	1.83	2.23	2.11	2.18	2.82	2.63
	$\bar{R}$	3.22	3.55	3.46	4.35	5.30	5.02	5.19	6.70	6.25
	σ_R	0.25	0.28	0.27	0.69	0.84	0.79	1.06	1.37	1.28
Frame-wall	$\bar{R}_\mu$	1.35	1.49	1.45	1.83	2.23	2.11	2.18	2.82	2.63
	$\bar{R}$	4.92	5.43	5.29	6.65	8.11	7.68	7.94	10.3	9.56
	σ_R	0.39	0.43	0.42	1.05	1.28	1.21	1.62	2.10	1.96

Notes:

μ_m = maximum story ductility ratio of structure

T_S = fundamental period of structrure

T_g = dominant period of earthquake motion

$\bar{R}_\mu$ = mean vale of elastic-inelastic response factor R_μ

$\bar{R}$ = mean value of response modification factor R

σ_R = standard deviation of R

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