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This report presents reliability-based criteria for the seismic design of reinforced concrete, moment-resisting frame buildings. The authors develop these criteria in the LRFD format by considering structural strength explicitly and ductility implicitly. The proposed design criteria are established on the basis of the seismic performance of the entire frame system, since the collapse limit state employed here is determined by the flexural failure mechanism of a structural system rather than the failure of structural members. Acceptable risk levels for various categories of buildings are investigated here, and seismic load factors for ordinary buildings in various seismic zones of the United States are determined. It is shown that, in low seismicity zones and when detailed as an IMR frame as specified by ACI 318-89, frame structures designed only for dead and live loads can provide enough seismic resistance for ordinary and high risk buildings. For essential buildings, the seismic design requires use of both a seismic load factor and an importance factor of 1.0.

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A Study of Reliability-Based Criteria for Seismic Design of Reinforced Concrete Frame Buildings

by

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August 10, 1991

Technical Report NCEER-91-0023

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PREFACE

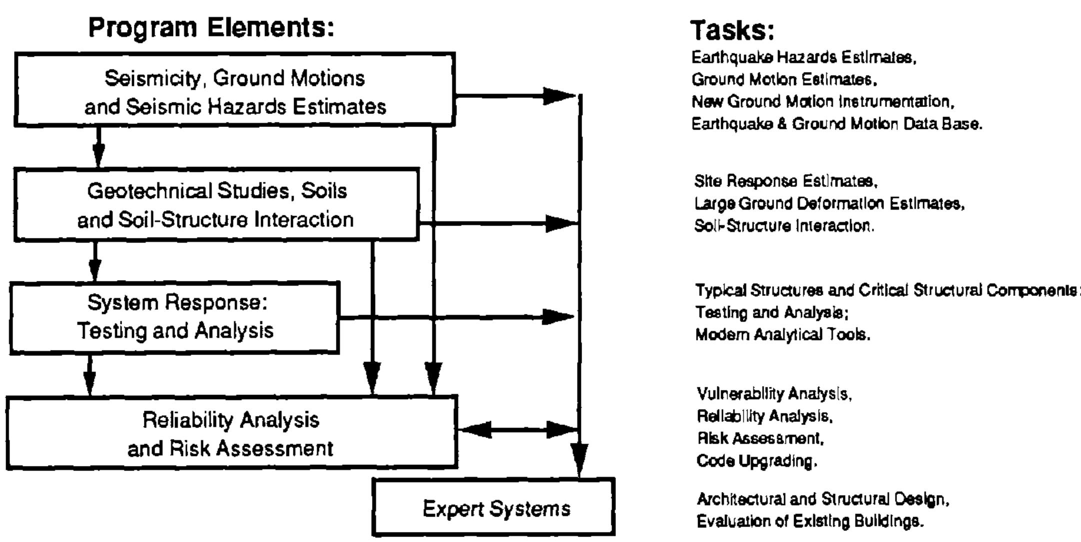
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to reliability analysis and risk assessment.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. This work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



Reliability analysis and risk assessment research constitutes one of the important areas of Existing and New Structures. Current research addresses, among others, the following issues:

1. Code issues - Development of a probabilistic procedure to determine load and resistance factors. Load Resistance Factor Design (LRFD) includes the investigation of wind vs. seismic issues, and of estimating design seismic loads for areas of moderate to high seismicity.
2. Response modification factors - Evaluation of RMFs for buildings and bridges which combine the effect of shear and bending.
3. Seismic damage - Development of damage estimation procedures which include a global and local damage index, and damage control by design; and development of computer codes for identification of the degree of building damage and automated damage-based design procedures.
4. Seismic reliability analysis of building structures - Development of procedures to evaluate the seismic safety of buildings which includes limit states corresponding to serviceability and collapse.
5. Retrofit procedures and restoration strategies.
6. Risk assessment and societal impact.

Research projects concerned with reliability analysis and risk assessment are carried out to provide practical tools for engineers to assess seismic risk to structures for the ultimate purpose of mitigating societal impact.

This report examines and compares the strengths of the three categories of reinforced concrete frames (ordinary, high-risk, and essential) using the capacity-spectrum method. The first yield and the collapse stages are studied for selected frames that are assumed to be detailed to have sufficient ductility. Although the collapse level is based on static nonlinear analysis, the ratios of elastic force levels to collapse load levels will be useful in developing improved response modification factors in Codes. The study of risk levels has lead to recommendations for consistent importance factors for the three types of frames.

ABSTRACT

This report presents a study to establish reliability-based criteria for seismic design of reinforced concrete moment-resisting frame buildings. The seismic criteria in the LRFD format are developed on the basis of structural strength being considered explicitly and ductility considered implicitly. The criteria are applicable for three categories of buildings (ordinary, high risk, and essential) in various seismic zones. The developed seismic LRFD criteria have a well-established rationale and will produce risk-consistent structures under various design conditions.

Two types of limit states, first yielding and collapse of a structure, are considered. It concludes that the collapse limit state controls the design and evaluation of buildings. The collapse of structures is determined on the basis of the flexural failure mechanism of a structural system rather than the failure of a structural member. Thus, the proposed seismic design criteria are established from the seismic performance of the entire frame system. The intermediate moment-resisting (IMR) frame designed according to the ACI code 318-89 is used to represent the frame system considered in this study. The IMR frame has enough strength and reasonable ductility; thus, it can be used in the entire United States. The IMR frame may be specially suitable for the eastern United States where no seismic requirement is currently enforced.

The acceptable risk levels for three categories of buildings have been investigated. For collapse of a structure as the limit state, the acceptable (target) limit-state probability is 1 in 1000 per year for ordinary buildings,

1 in 2000 per year for high-risk buildings, and 1 in 5000 or 1 in 10,000 per year for essential buildings. It seems that the target probability of 1 in 10,000 per year for essential buildings is too stringent to be accepted in view of the current practice.

The seismic load factors for ordinary buildings in various seismic zones of the United States have been determined. The seismic load factor for the area with high seismicity such as California is 1.3, which is larger than the value for the area with low seismicity. These seismic load factors determined for ordinary buildings are also used for high-risk and essential buildings. To meet the more stringent acceptable risk level for high-risk and essential buildings, the importance factor I is used to increase the design strength. The I factor of 1.2 is recommended for high-risk buildings, while 1.5 is recommended for essential buildings, if the acceptable collapse probability is chosen as 1 in 5000 per year.

For low seismicity area such as the design earthquake less than 0.1 g, gravity loads instead of seismic load dominate the design of structures. It has been shown that if the frame structures are designed only for dead and live loads and are detailed as the IMR frames, then the frame structures can provide enough seismic resistance for ordinary and high-risk buildings. For essential buildings, seismic design with both the seismic load factor and the importance factor as 1.0 is required to satisfy the acceptable risk level specified for essential buildings.

ACKNOWLEDGMENTS

This report is prepared as a doctoral dissertation by Mr. Hui-Mi Hsu and submitted to the Graduate School of the Memphis State University in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering. The dissertation is completed under the supervision of Professor Howard H. M. Hwang.

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SECTION 1

INTRODUCTION

Buildings in the United States are designed according to local building codes, which usually refer to a model building code with or without modifications. Three major model building codes are the Uniform Building Code (1988), the Standard Building Code (1988), and the BOCA National Building Code (1990). In addition, the ASCE 7-88 Standard (formerly ANSI A58.1) (1988) and the NEHRP Recommended Provisions (1988) are also widely referenced.

The current earthquake-resistant design philosophy implied in most building codes is that a building designed according to the seismic provisions will (1) resist a moderate earthquake without structural damage, and (2) resist a large earthquake without collapse. This design philosophy allows a structure to undergo inelastic deformation during a large earthquake. Thus, the inelastic design procedure is implemented in building codes. To facilitate the design process, however, the nonlinear design base shear is usually specified approximately. For example, the NEHRP Provisions use the response modification factor R, one effect of which is to reduce the base shear from an elastic force level to an inelastic force level, and the R factor for each structural system is empirically determined. The structures designed in such a fashion are expected to satisfy the design philosophy. Nevertheless, the relationship between the design procedure and the design philosophy cannot be clearly demonstrated.

Structural design of earthquake-resistant buildings is complicated by large uncertainty in predicting the spatial and temporal characteristics of future earthquakes. Uncertainties are also caused by limited ability of analytical models to properly describe nonlinear response of structures. In addition, structural capacity cannot be determined precisely because of variation in material strengths, workmanship, etc. Therefore, earthquake loading, structural response, and structural capacity are probabilistic in nature. Probability-based (reliability-based) design criteria are needed to include all the uncertainties in building codes.

Currently, building codes account for uncertainties in loads and structures by means of load and resistance factors (or safety factors). However, these factors specified in most building codes are mainly determined by code committees on the basis of members' expertise and judgment. The subjective manner in which the load and resistance factors are determined tends to result in nonuniform reliability of structures under various design conditions. To improve building codes, probabilistic methods can and should be used. Probabilistic methods could, in principle, be used directly in structural design; however, designers require extensive familiarity with reliability analysis methods and statistical models for loads and structural resistance. Thus, the direct use of probabilistic methods is not suitable for routine design of structures.

In probability-based limit states design, for example, the load and resistance factor design (LRFD) criteria (Ravindra and Galambos 1978), probabilistic methods are used to guide the selection of load factors and resistance factors by taking into account variabilities in individual loads and resistances. Therefore, the LRFD criteria, which have a deterministic format yet reflect the probabilistic nature of design parameters, are more appropriate for routine design purposes. In the United States, the LRFD criteria have been implemented in the ASCE 7-88 (1988) and the AISC LRFD manual (AISC 1986). In general, structural codes are moving toward the adoption of LRFD criteria as a basis for structural design. The LRFD criteria have been established for nuclear power plant structures (Hwang et al. 1987). In addition, the LRFD criteria are being developed for bridges (Kulicki and Mertz 1991), wood structures (Gromala and Cheung 1991), and electrical transmission line structures (Mozer and DiGioia 1991).

1.1 Objectives of Research

The objective of this study is to develop the reliability-based seismic LRFD criteria for reinforced concrete (RC) moment-resisting frame buildings. The criteria are applied to three categories of buildings (ordinary, high risk and essential) in various seismic zones. The LRFD criteria are developed on the basis of the structural strength being considered explicitly and ductility considered implicitly. For an RC moment-resisting frame designed according to the ACI code 318-89 (1989), brittle failure modes such as shear and bond failures are avoided through careful detailing of members, and the ultimate flexural capacity is usually less than the shear capacity. Therefore, the flexural failure mode is taken as the only failure mode in the development of the LRFD criteria. Other limit states such as shear failure, instability, buckling, and drift limit are not included in this study.

1.2 Organization of Report

The procedure for establishing the reliability-based LRFD criteria is as follows (Ellingwood et al. 1980; Hwang et al. 1987; Shinozuka et al. 1989):

1. select a load combination format,
2. establish representative frame structures,
3. design structures according to the proposed design criteria,
4. evaluate seismic performance of structures,
5. determine load and resistance factors by optimization with respect to the acceptable risk.

In this report, Section 2 discusses the code design philosophy. The generic seismic hazard curves for the United States, moderate and large earthquakes, limit states, and acceptable risk levels are established. Section 3 presents the LRFD load combination format and the selection of several load and resistance factors. Representative samples of reinforced concrete frame structures are established in Section 4, while Section 5 presents the proposed seismic design procedure for reinforced concrete frame structures. The seismic design is mainly accomplished by means of the equivalent lateral force procedure. The reliability analysis method for estimating the annual limit-state probabilities of buildings is established in Section 6, while Section 7 describes the optimization technique for determining the optimum value of the seismic load factor. Section 8 investigates the influence of three parameters, limit states, seismic zones, and site conditions, on the seismic performance of structures. The seismic load factors for designing ordinary buildings in various seismic zones are determined in Section 9. The importance factors for high-risk and essential buildings are also determined. Section 10 compares the proposed criteria with the NEHRP Provisions and the Tri-Services Guidelines. Finally, Section 11 presents the conclusions of this study.

SECTION 2

CODE DESIGN PHILOSOPHY

As mentioned in introduction, the earthquake-resistant design philosophy implied currently in most building codes is that a building designed according to the seismic provisions will (1) resist a moderate earthquake without structural damage, and (2) resist a large earthquake without collapse. This is, however, a qualitative statement. For engineering applications, a quantitative statement is needed. In this study, seismic hazard curves, moderate and large earthquakes, limit states, and acceptable risk levels are established.

2.1 Seismic Hazard Curves

Seismic hazard at a site is usually expressed in terms of a seismic hazard curve, which displays the peak ground acceleration (PGA) versus the annual probability of exceedence. The seismic hazard curve of a site is established on the basis of geology, seismicity, path attenuation, and local site conditions. The characteristics of seismic hazard vary a great deal across the United States. Algermissen and Perkins (1976) of the U.S. Geological Survey (USGS) evaluated seismic hazards for the contiguous 48 states and produced the generic seismic hazard curves corresponding to various levels of the design earthquake E_D ranging from 0.05 g to 0.4 g. These generic seismic hazard curves (figure 2-1) are shown in the commentary of the NEHRP Provisions. Since the seismic hazard maps

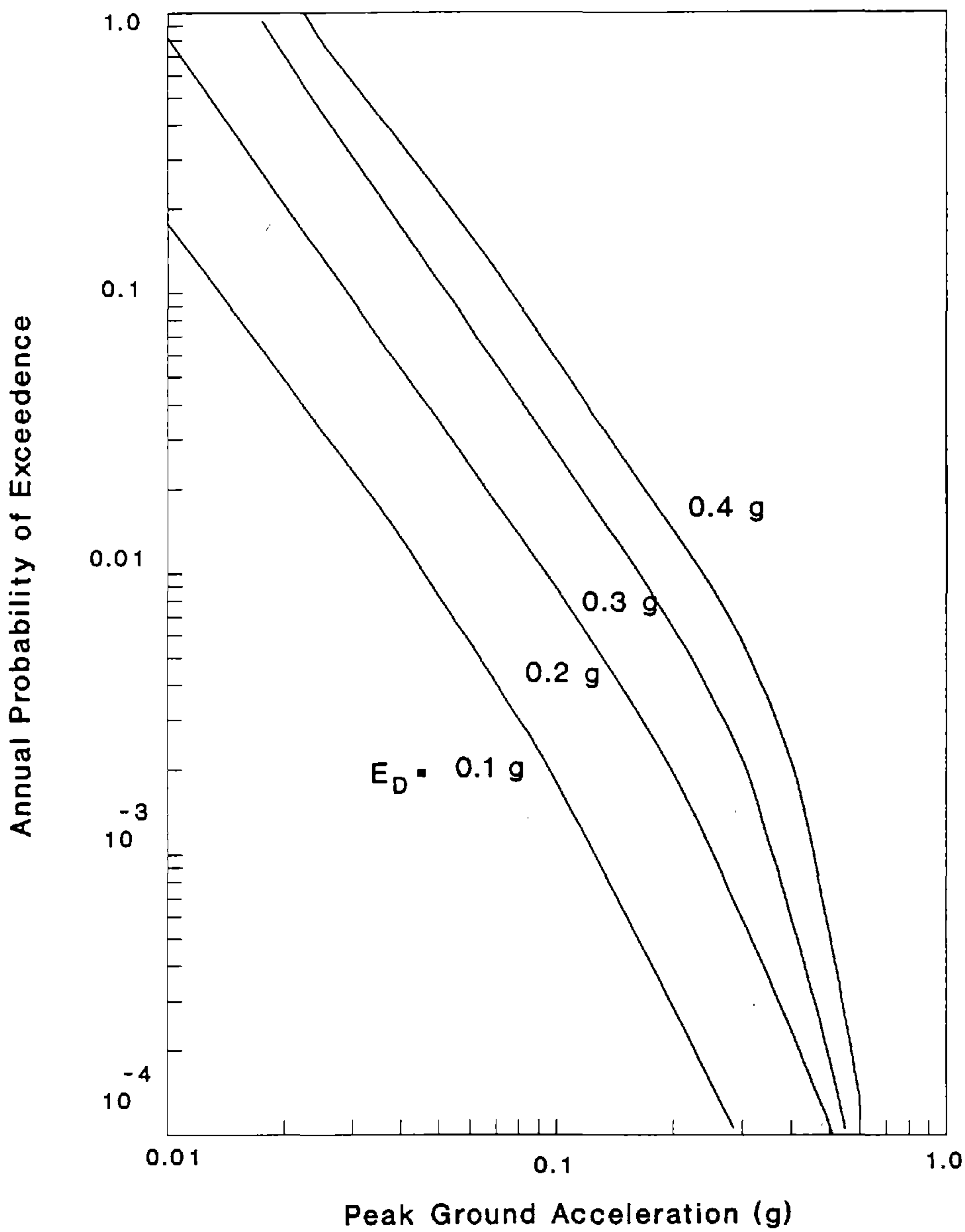


FIGURE 2-1 Seismic Hazard Curves (NEHRP Recommended Provisions 1988)

specified in the building codes are based on the USGS studies, these generic seismic hazard curves are adopted for this study.

2.2 Moderate and Large Earthquakes

In current model building codes, the design earthquake E_D is usually defined as an earthquake with a 10% probability of exceedence in 50 years. It is denoted as a 475-year earthquake, since the mean recurrence interval (return period) of such an earthquake is 475 years. This design earthquake is neither a moderate earthquake (EQ) nor a large earthquake mentioned in the design philosophy. In fact, most building codes do not specify a moderate or a large earthquake. Hwang and Hsu (1991) investigated the definitions of the moderate and large earthquakes. They suggested a 100-year earthquake as the upper bound for moderate earthquakes and a 2000-year earthquake as the upper bound for large earthquakes. In this study, the upper bounds of moderate and large earthquakes are also defined as 100-year and 2000-year earthquakes, respectively. From the seismic hazard curves (figure 2-1), the peak ground accelerations (PGA) corresponding to these two earthquakes are determined and shown in table 2-I.

2.3 Limit States

A limit state represents a state of undesirable structural behavior. In general, a building should be designed by considering all possible limit states such as collapse, buckling, instability, drift limit, and serviceability. In this study, first yielding and collapse of a structure are the two types of

TABLE 2-I Moderate and Large Earthquakes

Design EQ (g)	PGA (g)	
	Moderate EQ (100-year)	Large EQ (2000-year)
0.1	0.05	0.16
0.2	0.09	0.32
0.3	0.16	0.42
0.4	0.24	0.51

limit states considered. For a moment-resisting frame structure, the first yielding is defined as the formation of first plastic hinge anywhere in the structure. If a structure subject to an earthquake does not reach the first yielding, then the structural response remains in the elastic range and the structure does not sustain any structural damage. Thus, the first-yielding limit state can be considered as a serviceability limit state. The collapse of a structure is defined as the formation of a failure mechanism. The collapse limit state represents an ultimate strength limit state.

2.4 Acceptable Risk Levels

Most model building codes do not explicitly state the acceptable risk level. In general, risk can be defined in terms of annual limit-state probability, fatalities, and damage cost. The relationship between fatalities and magnitudes of earthquakes is not clearly defined. The damage cost includes direct and indirect losses. Indirect loss is very difficult to estimate in most cases. Therefore, the annual limit-state probability is used to represent the acceptable risk.

Ellingwood et al. (1980) used the reliability index β to express the level of acceptable risk. For a structure subject to dead and live loads, they found that the β value (for 50-year design lifetime) corresponding to the ultimate strength limit state implied in building codes is 3.0. By assuming that the structural capacity and response are normally distributed, the reliability index of 3.0 is equivalent to the annual limit-state probability PF_T of 2.6×10^{-5}. However, for a structure subject to earthquakes, the β value is only about 1.75, which is equivalent to the annual limit-state probability of 8.0×10^{-4}.

Galambos (1990) proposed the target reliability indices β_T (for 50-year design lifetime) for various limit states. As shown in table 2-II, $\beta_T = 4$ is assigned to the complete damage (loss of life), which is equivalent to the collapse of structure. For such a limit state, the equivalent target limit-state probability PF_T is equal to 6.4×10^{-7} per year. For the slight damage, which is similar to the first yielding, β_T is equal to 2 (the equivalent PF_T is 4.0×10^{-4} per year).

Hays (1985) discussed the annual probability of occurrence and common perception of risks (figure 2-2). If risk is related to economic loss, the annual probability of occurrence from 1 to 0.1 is regarded as high risk, 0.1 to 0.01 as moderate risk, 0.01 to 10^{-3} as low risk, and 10^{-3} or smaller as negligible risk. On the other hand, if risk is related to fatality (loss of life), the annual probability of occurrence from 1 to 5×10^{-3} is regarded as high risk, 5×10^{-3} to 5×10^{-5} as moderate risk, and 5×10^{-5} or smaller as low risk.

In regard to safety of nuclear power plants, the U.S. Nuclear Regulatory Commission stated that in order to have reliable performance of containment systems, the overall mean frequency of a large release of radioactive materials to the environment from a reactor accident should be less than 10^{-6} per year of reactor operation (Okrent 1989). This annual probability of 10^{-6} can be considered as the extreme value for establishing the acceptable risk level for buildings.

TABLE 2-II Target Reliability Indexes (Galambos 1990)

Limit State	β_T
Slight damage	2.0
Moderate damage	2.5
Serious damage	3.0
Complete damage (no loss of life)	3.5
Complete damage (loss of life)	4.0

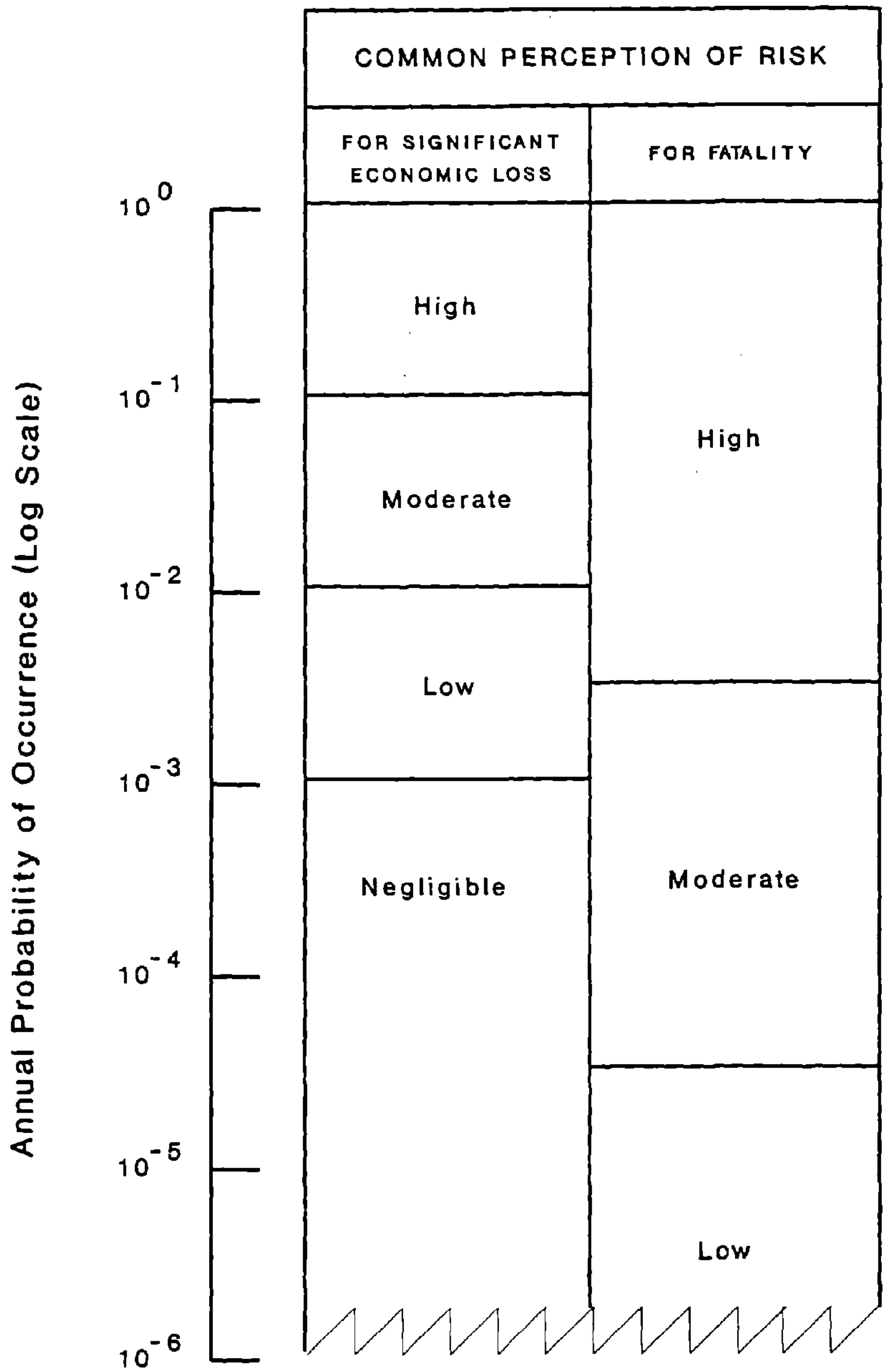


FIGURE 2-2 Acceptable Risk (Hays 1985)

The level of acceptable risk shall be established by an authority on the basis of the function of structure, characteristics of limit states, consequences of reaching limit states, and damage cost. Hence, the acceptable risk level or the target limit-state probability may not necessarily be the same for different limit states. In addition, the target probability may be subjected to change in accordance with the perception of acceptable risk by the general public. On the basis of foregoing discussions, acceptable risk levels are set for three categories of buildings, i.e., essential, high-risk, and ordinary buildings ("Seismic" 1982). Essential buildings are defined as structures housing critical facilities such as hospitals and fire stations that are required to be functional during and after an earthquake. High-risk buildings are those structures 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 buildings, the acceptable probability of reaching the first-yielding limit state is set as 1/50 per year if a moderate earthquake occurs. The acceptable probability of reaching the collapse limit state is set as 1/1000 per year if a large earthquake occurs. For essential buildings, the acceptable risk level is set as the annual probabilities of 1/100 for the first yielding as a limit state. For the collapse limit state, the acceptable risk level is set as the one between 1/5000 and 1/10,000 per year. The acceptable risk levels are more stringent because the consequence of reaching the limit states for essential buildings is more serious.

For collapse of high-risk buildings, the acceptable risk level should lie between those set for essential and ordinary buildings and is, therefore, selected as 1/2000 per year. For the first-yielding limit state, the acceptable risk level is set as the annual probability of 1/100, which is the same as the risk level for essential buildings. The acceptable risk levels for all three categories of buildings are summarized in table 2-III, in which $PF_{Y,T}$ and $PF_{C,T}$ represent the acceptable (target) first-yielding and collapse limit-state probabilities, respectively.

TABLE 2-III Acceptable Risk Levels for Buildings

Building Category	Acceptable Limit-State Probability	
	First Yielding ($PF_{Y,T}$) (/yr)	Collapse ($PF_{C,T}$) (/yr)
Ordinary buildings	1/50	1/1000
High-risk buildings	1/100	1/2000
Essential buildings	1/100	1/5000 - 1/10,000

SECTION 3

LOAD COMBINATION FORMAT

The design of a building requires that the structural resistance should be larger or equal to the design load effects. This requirement expressed in the LRFD format is

$$\phi_i R_i \geq \gamma_{j1} L_1 + \ldots + \gamma_{jn} L_n \quad (i = 1, 2 \ldots k;\ j = 1, 2 \ldots m) \qquad (3.1)$$

where

$L_1, L_2 \ldots$ = design load effects

$\gamma_{j1}, \gamma_{j2} \ldots$ = load factors in the j-th load combination

R_i = nominal structural resistance for the i-th limit state

ϕ_i = resistance factor for the i-th limit state

k = total number of limit states considered

n = total number of loads to be combined

m = total number of load combinations

Three loads, dead load D, live load L, and seismic load E, are considered in this study. Thus, equation (3.1) becomes

$$\phi_i R_i \geq \gamma_D D + \gamma_L L \pm \gamma_E E \qquad (3.2)$$

The dead and live load factors are preset to simplify the optimization. The mean value of dead load is approximately equal to the nominal value and variability of dead load is quite small. Ellingwood et al. (1980) have found that the dead load factor of 1.2 (or 0.9 when dead load has a stabilizing effect) is more than adequate to account for uncertainty in dead load.

The design live load specified in building codes is usually close to the maximum possible value. It is very conservative, because the floor loads measured in a live-load survey usually are well below the design values (ASCE 7-88 1988). If live load is not a principal load in a load combination, the use of maximum live load is not reasonable. Instead, the so-called arbitrary-point-in-time live load should be used. For the load combination with seismic load as the principal load, Ellingwood et al. (1980) suggested that the live load factor of 0.5 is adequate when seismic load has the same effect as dead and live loads. When live load has a stabilizing effect, the live load factor is taken as zero.

The described dead and live load factors have been adopted in the ASCE 7-88 (1988). These factors are also adopted in this study; thus, the right-hand side of equation (3.2) can be written as

$$1.2\,D + 0.5\,L \pm \gamma_E\,E \tag{3.3}$$

$$0.9\,D \pm \gamma_E\,E \tag{3.4}$$

The left-hand side of equation (3.2) represents the nominal capacity multiplyed by the resistance factor. In general, resistance factors depend on the material properties and limit state under consideration. For RC frame structures, the resistance factors (strength reduction factors) specified in the ACI code 318-89 are used in engineering practice.

Therefore, these resistance factors are also used in this study. The resistance factors for various conditions are summarized below.

1. Axial tension or flexural with axial tension, $\phi = 0.9$
2. Axial compression or flexural with axial compression, $\phi = 0.7$
3. Shear, $\phi = 0.85$
4. Bearing on concrete, $\phi = 0.7$

It is noted that for f_y not exceeding 60 ksi and sections with symmetrical reinforcement, the ϕ value increases from 0.7 to 0.9 as axial compression decreases from $0.1f'_cA_g$ to zero.

SECTION 4

REPRESENTATIVE FRAME STRUCTURES

The structural system considered in this study is RC moment-resisting frames. Three categories of RC moment-resisting frames, special, intermediate, and ordinary, are used in current practice. The ordinary moment-resisting frame is usually not used as an earthquake-resistant system because of its low ductility during an earthquake. Therefore, ordinary moment-resisting frames are not considered in this study.

The special moment-resisting (SMR) frame employs the strong-column—weak-beam concept and specifies elaborate detailing of joints. Thus, the SMR frame is expected to form the sway mechanism and possesses a high degree of ductility. However, the design and construction of the SMR frame is more complicated. Therefore, the special moment-resisting frame may be suitable for buildings constructed in regions of high seismicity such as California. For the eastern United States where no seismic requirement is currently enforced, the use of the SMR frame to resist earthquakes may not be easily accepted by professionals. On the other hand, the intermediate moment-resisting (IMR) frame has enough strength as well as reasonable ductility and can be used throughout most of the United States. Thus, the frame referred to in this study hereinafter is the intermediate moment-resisting frame, unless stated otherwise. Furthermore, it is assumed that the detailing of the IMR frame according to the ACI code 318-89 is adequate so that the plastic hinges formed on the beams or columns can hold up until a failure mechanism is reached.

An important requirement for codified structural design is that all the structures designed according to a building code should meet the performance objectives of the code. To test if this requirement is satisfied, a set of representative (sample) structures must be selected for evaluating the code. To select a set of representative structures, the following two conditions must be considered: (1) the selection should be made in such a manner that the conclusions drawn from the code evaluation, with the aid of these structures, will be valid for all structures that fall within the scope of the code; (2) the number of representative structures should be sufficient but not too large, since the design and reliability analysis of each structure is expensive and time-consuming.

To satisfy the above two conditions, first of all, the parameters affecting the design of RC frame buildings are identified and shown in table 4-I. The range of each parameter is established by considering the current practice. For each parameter, representative values are selected within the defined range. For example, the maximum number of stories for an RC moment-resisting frame system is about 15 stories for office and apartment buildings (Fintel 1985; Coleman 1983). If a building is more than 15 stories, frame action usually can provide adequate lateral resistance; however, the stiffness of frame may not be sufficient to satisfy the drift requirement and other lateral load resisting systems such as shear walls need to be added to the frame system. Thus, the maximum number of stories is taken as 15 stories and six values (3, 5, 7, 9, 11, and 13) are selected as the representative numbers of stories. Other design parameters such as story height, number of span, and span length are investigated in a

TABLE 4-I Representative Values of Design Parameters

Items	Range	Representative Values
No. of stories	1-15	3, 5, 7, 9, 11, 13
Story height (ft) (1st story)	11-15 (+3)	12, 13, 14 (15, 16, 17)
No. of spans	2-5	3, 4
Span length (ft)	20-30	20, 25, 30
Trans. spacing (ft)	20-30	25
RC weight (pcf)	145-155	150
Live load (psf)	40-50	40, 50
Roof live load (psf)	12-20	16
f_y (ksi)	60	60
f_c' (ksi)	3-5	4, 5
Site condition (Site coefficient)	S_1, S_2, S_3 (1.0-1.5)	S_1, S_2, S_3 (1.0), (1.2), (1.5)
Design earthquake (g)	0.1-0.4	0.1, 0.2, 0.3, 0.4

similar manner. The ranges and selected representative values for each parameter are listed in table 4-I.

In most building codes, the site condition is usually represented by a site coefficient S, which takes 1.0, 1.2, 1.5, 2.0 for rock (S_1), dense alluvium (S_2), medium to soft clay (S_3), and very soft clay (S_4), respectively (UBC 1988). For the site of very soft clay, the equivalent lateral force procedure may not be applicable; instead, dynamic analysis procedure with site-specific ground motion as input is usually required. Thus, S_4 is excluded from this study. As a result, 1.0, 1.2, and 1.5 are selected as the representative S factors.

The representative values listed in table 4-I can be used to construct the samples of frame structures by using the Latin hypercube sampling technique (Iman et al. 1980). The Latin hypercube sampling technique is a systematic and efficient technique of selecting random samples (Hwang et al. 1987). A sample structure is identified by a sample vector, which consists of one of the representative values of each parameter. Since the design earthquake plays an important role in seismic design of a building, the design earthquake will be considered explicitly. Hence, all the representative values of the design parameters except design earthquake (table 4-I) are chosen to make up six samples by means of the Latin hypercube sampling technique (table 4-II). These six samples are then used with each of the four design earthquakes; thus, yielding a total of 24 samples.

TABLE 4-II Representative Frames

Item	Frame					
	1	2	3	4	5	6
No. of stories	5	11	3	13	7	9
Story height (ft) (1st story)	12 (15)	14 (17)	13 (16)	14 (17)	13 (16)	12 (15)
No. of spans	3	4	4	3	3	4
Span length (ft)	30	25	20	20	30	25
Trans. spacing (ft)	25	25	25	25	25	25
RC weight (pcf)	150	150	150	150	150	150
Live load (psf)	40	50	40	40	50	50
Roof live load (psf)	16	16	16	16	16	16
f_y (ksi)	60	60	60	60	60	60
f_c' (ksi)	4	5	4	5	5	4
Site coefficient	1.5	1.0	1.2	1.5	1.0	1.2

SECTION 5

SEISMIC DESIGN OF FRAME STRUCTURES

For the representative frames such as those shown in table 4-II, each frame is designed according to the proposed design criteria with design loads, nominal resistance, resistance factors, and trial load factors. For nominal resistance, the current values specified in the ACI code 318-89 are used. For design dead and live loads, the values specified in model building codes are adopted. On the other hand, the design earthquake is specified using the same format as the 1988 Uniform Building Code except the R_μ factor is used instead of the R_w factor. The design base shear V is expressed as

$$V = I\left(\frac{ZC}{R_\mu}\right)W \tag{5.1}$$

where

I = importance factor

Z = seismic zone factor

C = spectral acceleration coefficient

R_μ = elastic-to-inelastic response factor

W = total seismic dead load

5.1 Importance Factor

In the NEHRP Provisions, the seismic hazard exposure group together with the seismicity index is used to define the seismic performance category of a building. In general, a building classified as a high-risk or an essential building will be in a higher seismic performance category. The NEHRP Provisions attempt to increase the safety of an essential building or a high-risk building by placing stricter requirements on drift, requiring a more detailed analysis, placing restrictions on the type and height of framing to be used, and imposing more restrictive detailing requirements. Thus, the NEHRP Provisions mainly rely on the ductility rather than the strength to protect high-risk and essential buildings. This concept may provide protection against collapse of structures; however, it may not result in better structural performance with respect to the first yielding as a limit state. For example, for a hospital built in a region of high seismicity, the SMR frame rather than the IMR frame is required as specified in the NEHRP Provisions because of the higher ductility requirement. However, if the IMR frame is used, the yielding capacity of the IMR frame will be higher than that of the SMR frame because a smaller response modification factor is used for the IMR frame. Consequently, the chance of a hospital with the IMR frame being damaged by an earthquake is less than that of a hospital with the SMR frame. Thus, increasing the ductility without increasing or even decreasing the strength does not improve the serviceability of a building. The concept employed in the current NEHRP Provisions may not be adequate to protect the essential and high-risk buildings against all possible earthquakes.

The Uniform Building Code (1988) attempts to increase safety of high-risk and essential buildings by using the importance factor coupled with independent design review and appropriate program of inspection. As stated in the SEAOC Commentary (1990), the I value raises the level at which inelastic behavior may occur and thereby increases the level at which the function or operability of high-risk or essential facilities is compromised. In addition, the probability of collapse will also be reduced. Thus, the use of importance factor may be a suitable way to protect high-risk and essential buildings. In this study, the importance factor is assigned as 1.0 for ordinary buildings. For high-risk and essential buildings, the importance factors will be determined later in this study.

5.2 Elastic Design Response Spectrum

In this study, the elastic design response spectrum is considered as the product of the spectral acceleration coefficient (normalized response spectrum) and the seismic zone factor, which is equivalent to the peak ground acceleration in g. In reality, the response spectrum is affected by seismic source, epicentral distance, and site conditions; thus, the response spectrum cannot be scaled by a factor such as the peak ground acceleration. However, until the response spectra for various parts of the Untied States are produced directly by the USGS, the current practice is to use the normalized response spectrum scaled by the seismic zone factor.

The peak ground acceleration of the design earthquake can be determined from the seismic hazard analysis. Algermissen and Perkins (1976) evaluated the seismic hazards for the contiguous 48 states and produced the contour maps of the peak ground acceleration. The map was modified by the code committee and then adopted in building codes such as the NEHRP Provisions (1988). Figure 5-1 shows the contour map of the effective peak acceleration A_a of the design earthquake. It is noted that this map was prepared by using two kinds of rock, hard rock and soft rock, as the reference site condition. The hard rock is the one with shear wave velocity close to 3.5 km/sec and it can be found in the eastern United States. On the other hand, the soft rock is the one with shear wave velocity about 750 m/sec and it exists in the western United States, for example, California. The site coefficients in the current building codes are specified on the basis of the earthquake data related to the soft rock. Therefore, the soft rock is used as the reference site condition in this study.

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

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad (5.2)$$

where

S = site coefficient

T = structural period

The normalized response spectra calculated from equation (5.2) are shown in figure 5-2. The upper limit of C is equal to 2.75 and is independent of the soil type and structure period. This upper-limit value is also specified

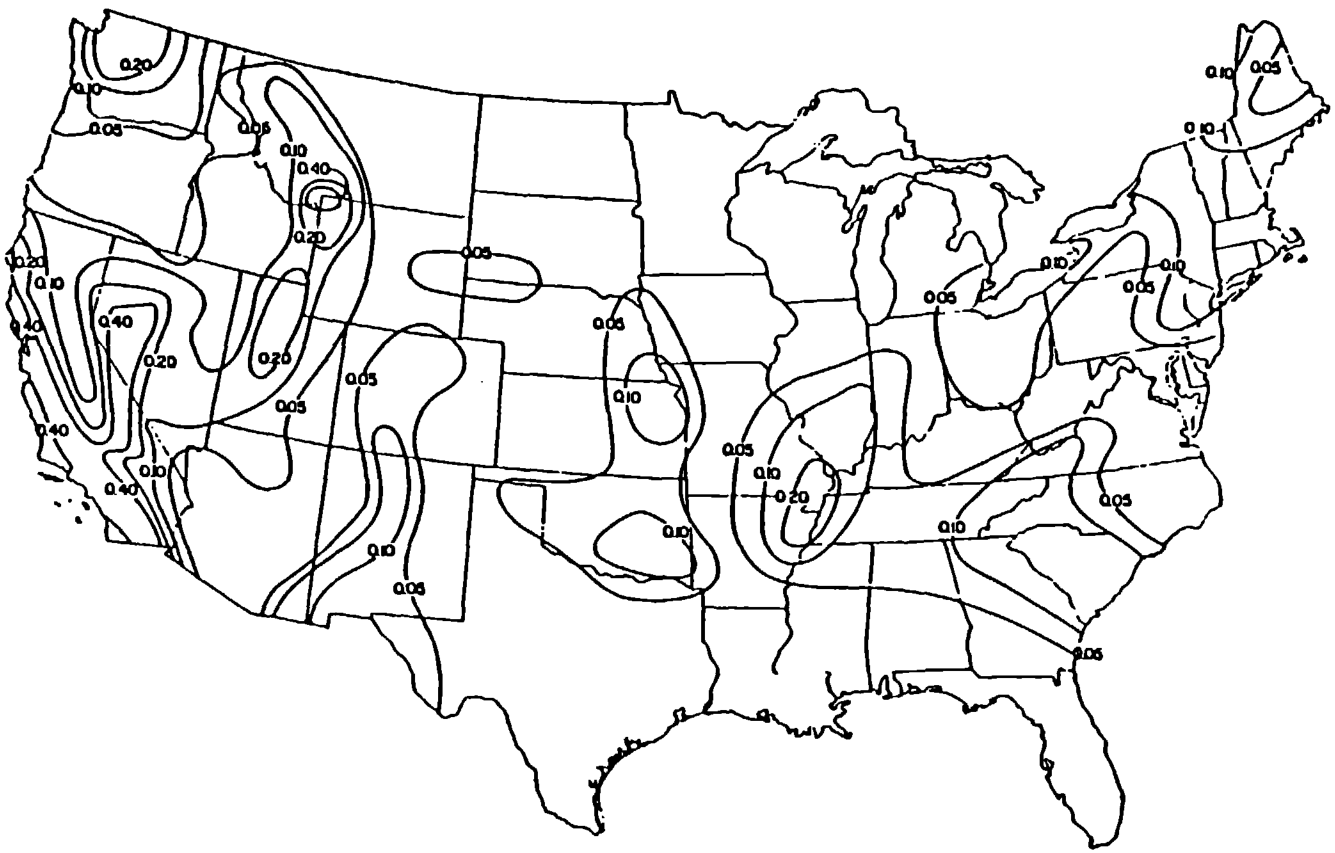


FIGURE 5-1 Contour Map for Effective Peak Acceleration (NEHRP Commentary 1988)

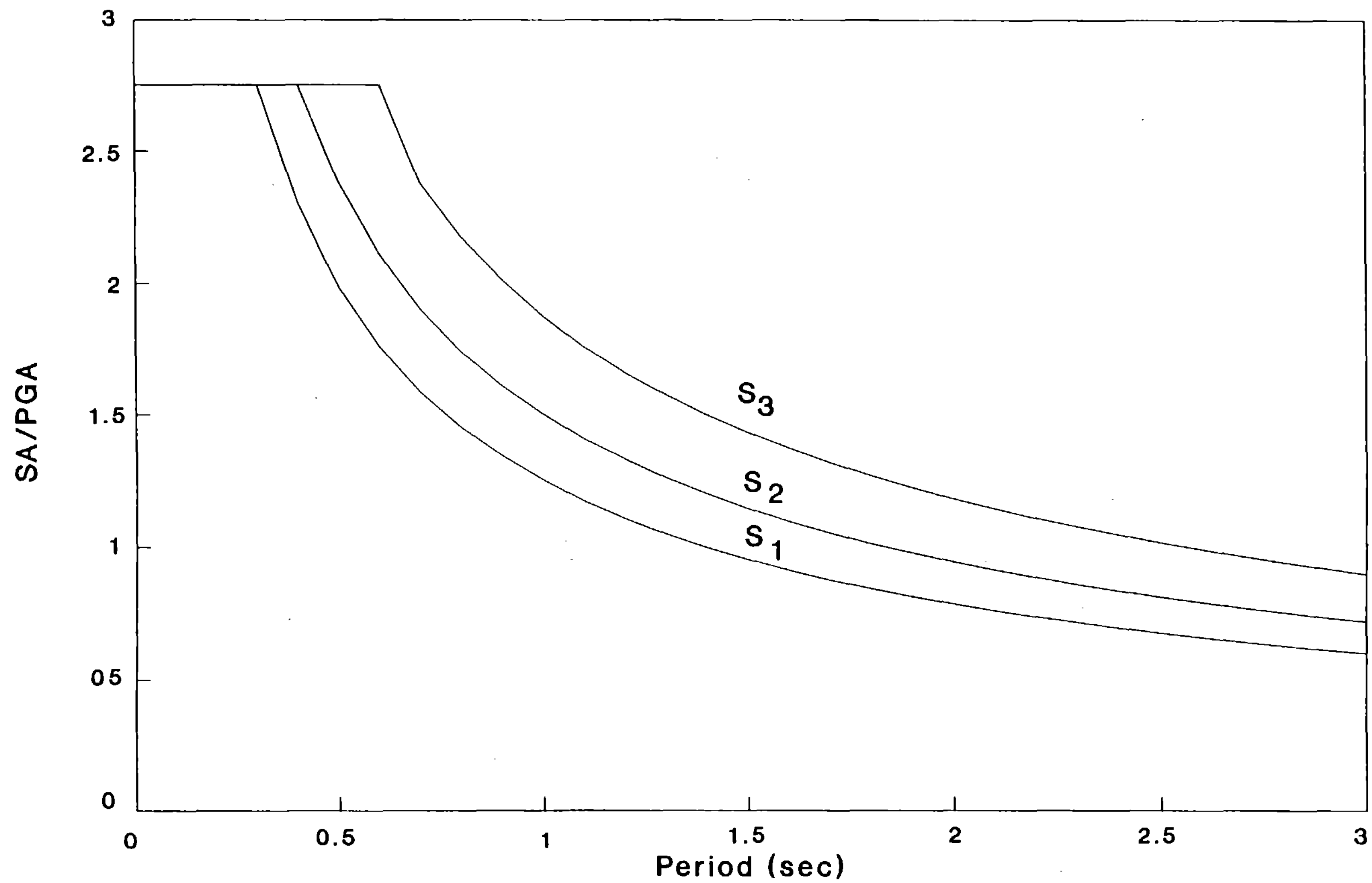


FIGURE 5-2 Normalized Response Spectra

in the UBC, while a smaller value (i.e., 2.5) is specified in the NEHRP Provisions.

The site condition is classified into four categories: rock (S_1), dense alluvium (S_2), medium to soft soil (S_3), and very soft soil (S_4). The corresponding S factors are 1.0, 1.2, 1.5, and 2.0, respectively. However, the equivalent lateral force procedure may not be applicable for the site with very soft clay. Thus, the S_4 site condition is not included in this study. Furthermore, the site with hard rock actually does not belong to S_1 site condition. In the proposed seismic code for New York City (Jacob 1990), the S_0 site condition has been created for the site with hard rock (table 5-I). The S_0 site condition is also not included in this study.

The structural period T for an RC frame structure is determined by using the empirical formula specified in the NEHRP Provisions:

$$T = 0.03h_n^{3/4} \tag{5.3}$$

in which h_n is the height in ft above the base to the top level of the building. This formula is commonly used in current practice to determine the fundamental period of an RC frame structure.

5.3 Elastic-to-inelastic Response Factor

The current seismic design criteria for buildings in the United States allow structures to behave in a nonlinear manner in the event of a large earthquake. Thus, the NEHRP Provisions use the response modification

TABLE 5-I The Proposed Site Coefficients for the New York City Building Code (Jacob 1990)

Type	Description	S-Factor
S_0	A profile of rock materials.	2/3
S_1	A soil profile with either: (a) soft rock or hardpan or similar material characterized by shear wave velocities greater than 2500 ft/sec, or (b) medium compact to compact sands and gravels or hard clays, where the soil depth is less than 100 ft.	1.0
S_2	A soil profile with medium compact to compact sands and gravels or hard clays, where the soil depth exceeds 100 ft.	1.2
S_3	A total depth of overburden of 75 ft or more and containing: more than 20 ft of soft to medium clays or loose sands and silts, but not more than 40 ft of soft clay or loose sands and silts.	1.5
S_4	A soil profile containing more than 40 ft of soft clays or loose sands, silts or uncontrolled fills, where the shear-wave velocity is less than 500 ft/sec.	2.5

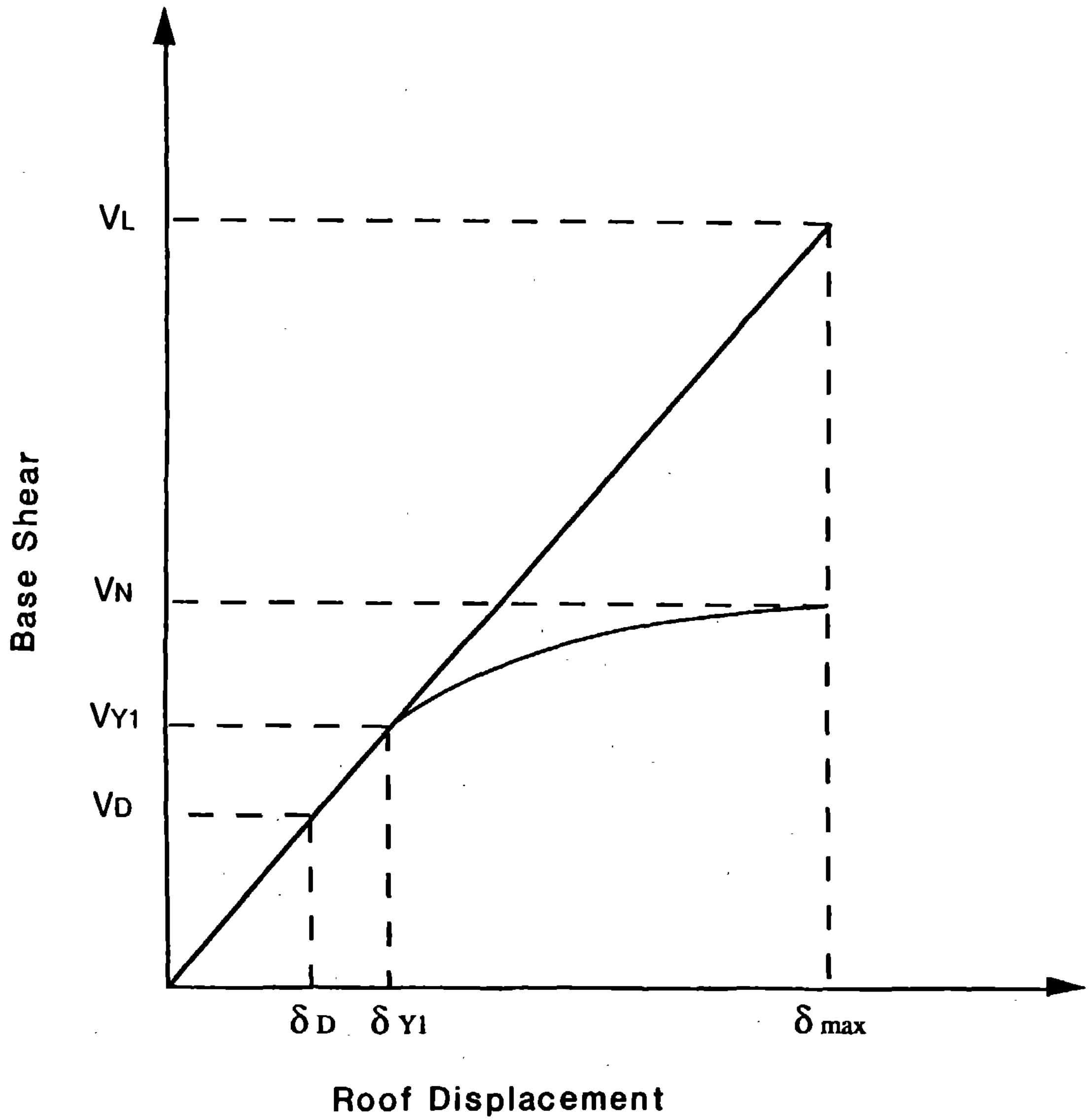
factor R to derive the design base shear from the corresponding elastic base shear. However, the definition of the R factor is not clearly defined. In the NEHRP Provisions, the R values for various structural systems are specified without a definition. According to Chapter 4 of the 1988 NEHRP Commentary, the R factor is an empirical factor intended to account for both damping and ductility inherent in the structural system at displacements great enough to surpass initial yield and to approach the ultimate load displacement of the structural system. On the other hand, Chapter 3 of the same commentary states that the R factor is the ratio of the linear elastic forces to the prescribed design forces specified at the significant yield level.

Several studies of the R factor for single-degree-of-freedom (SDF) and multi-degree-of-freedom (MDF) systems have been performed. Because the definition of the R factor used in these studies is different, the resulting R values are not consistent. Figure 5-3 shows the base shear versus the roof displacement of a structure. V_D is the design base shear; V_{Y1} is the base shear corresponding to the actual first yielding of the structure; V_N is the nonlinear base shear corresponding to the collapse of the structure; and V_L is the linear elastic base shear, if the structure behaves elastically until it reaches the collapse limit state. To discuss the definition of the R factor, the following three ratios are defined:

$$R_\mu = V_L/V_N \tag{5.4}$$

$$R_{OS} = V_N/V_{Y1} \tag{5.5}$$

$$R_{DS} = V_{Y1}/V_D \tag{5.6}$$



$$R_{\mu} = V_L / V_N$$

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

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

$$R_1 = R_{\mu}$$

$$R_2 = R_{\mu} R_{OS}$$

$$R_3 = R_{\mu} R_{OS} R_{DS}$$

FIGURE 5-3 Possible Definitions of the R Factor

where R_μ is the ratio of the elastic base shear to the collapse base shear; R_{OS} is the ratio of the collapse base shear to the first-yielding base shear; and R_{DS} is the ratio of the first yielding to the design value. If the R factor is to reduce the base shear from the elastic level to the collapse level, then the R factor (denoted as R_1 for this case) can be determined as

$$R_1 = R_\mu \tag{5.7}$$

If the R factor is to reduce the base shear from the elastic level to the actual first-yielding level, then the R factor, denoted as R_2, is

$$R_2 = R_\mu R_{OS} \tag{5.8}$$

Furthermore, if the R factor is used to reduce the base shear from the elastic level to the design level, then the R factor, denoted as R_3, is

$$R_3 = R_\mu R_{OS} R_{DS} \tag{5.9}$$

Hwang and Hsu (1991) evaluated the seismic performance of an RC special moment-resisting frame and an intermediate moment-resisting frame. The values of three possible R factors for these two frames are determined and listed in table 5-II. In addition, the values of the three R factors determined by Bertero (1986) on the basis of test results of a seven-story RC frame-wall structure are also shown in table 5-II. From this table, it can be seen that the R values are quite different depending on the definitions used.

TABLE 5-II The R Values for Various Structural Systems

Structure	R_μ	R_{OS}	R_{DS}	R_1	R_2	R_3
SMR frame (Hwang & Hsu 1991)	2.9	1.6	2.6	2.9	4.5	12
IMR frame (Hwang & Hsu 1991)	1.7	1.7	1.4	1.7	2.9	4
Frame-wall (Bertero 1986)	2.7	2.8	1.3	2.7	7.8	10

The R_μ and R_{OS} factors are affected by the hysteretic model, ductility, damping ratio, structural period, site period, and redundancy of structure. On the other hand, the R_{DS} factor is affected by the conservative design requirements including load and resistance factors (safety factors), actual material strength, member over-size, code minimum requirements, and deflection limits. In addition, nonstructural elements also have an effect on the R_{DS} factor.

If the design earthquake is selected at the level of small or moderate earthquake, the structure is expected to remain elastic under the design earthquake. If an appropriate elastic analysis of structure is performed, the structural response does not need to be modified by the R factor. Thus, the R factor should be 1.0. However, if R_{DS} is included in the definition of R, then the R factor will not be 1.0, since safety factors (or load and resistance factors) and minimum code requirements, etc. used in the design will always produce the safety margin represented by R_{DS}. Thus, R_{DS} is not a part of the R factor.

If the design earthquake is selected as a large earthquake and a nonlinear time history analysis of structure is performed, then nonlinear response of structure can be obtained directly without using the R factor. In this case, safety factors or load and resistance factors are still used in design and these factors will again produce R_{DS}, even though the R factor is not used. Thus, R_{DS} is not a part of the R factor. Furthermore, R_{OS} is also not a part of the R factor. It is not realistic to divide the results of nonlinear time history analysis by an arbitrary value so that the design base shear is obtained at the first-yielding level instead of at the collapse level; that is the level at which the structure should be protected against a large earthquake. Therefore, the R factor should be taken as R_1 or R_μ; that is, the R_μ factor is used to reduce the base shear from the elastic level to the collapse level. To avoid the confusion, the R_μ factor is denoted as the elastic-to-inelastic response factor.

Riddell and Newmark (1979) conducted a statistical analysis of the response data of SDF systems with various hysteretic models subject to actual earthquake records. The ratio of the linear elastic response to the nonlinear response, i. e., the R_μ factor, is a faction of the ductility ratio μ and viscous damping ratio ζ. Riddell and Newmark proposed the empirical formula for the R_μ factor for three different regions. In the acceleration region, the empirical formula is

$$R_\mu = [(q+1)\mu - q]^\gamma \tag{5.10}$$

and the coefficient q and γ are determined as

$$q = 3.00\ \zeta^{-0.3}\ ;\ \ \gamma = 0.48\ \zeta^{-0.08} \tag{5.11}$$

In the velocity region, the empirical formula has the same form as the above equation with the following expressions for coefficients q and γ.

$$q = 2.70\ \zeta^{-0.4}\ ;\ \ \gamma = 0.66\ \zeta^{-0.04} \tag{5.12}$$

In the displacement region, the empirical formula is given as

$$R_\mu = p\mu^\gamma \tag{5.13}$$

and the coefficients p and γ are determined as

$$p = 1.15\ \zeta^{-0.055}\ ;\ \gamma = 1.07 \tag{5.14}$$

As an example, by using equations (5.10) and (5.11), i.e., for the acceleration region, the R_μ value is 2.6 for the case of ζ = 5% and μ = 4.

Hwang and Jaw (1989) evaluated the R_μ factor for multi-degree-of-freedom RC structures. The R_μ factor is shown as a function of the structural period T_s, site period T_g, viscous damping ratio ξ, and system ductility ratio μ_m, which is taken as the maximum story ductility ratio. From the simulation results, the R_μ values exhibit significant scatter and the empirical formula for the median R_μ factor is

$$\ln R_\mu = [e^{-0.1857Ts/Tg} - e^{-2.1673Ts/Tg} - 0.0276\xi]\ \ln\mu_m \tag{5.15}$$

The median R_μ values are shown in figure 5-4. For the case of μ_m = 4, the R_μ factor is about 2; for μ_m = 6, the R_μ factor is less than 3.

In the Japanese code for seismic design of buildings ("Earthquake" 1988), the structural coefficient D_s is used to derive the ultimate lateral seismic shear. D_s is numerically equal to the reciprocal of the R_μ factor. The D_s values (table 5-III) depend on types of frame as well as on the ductile behavior of members. For ductile moment frames with the fair ductile members, D_s is equal to 0.4, and thus the equivalent R_μ factor is 2.5. For

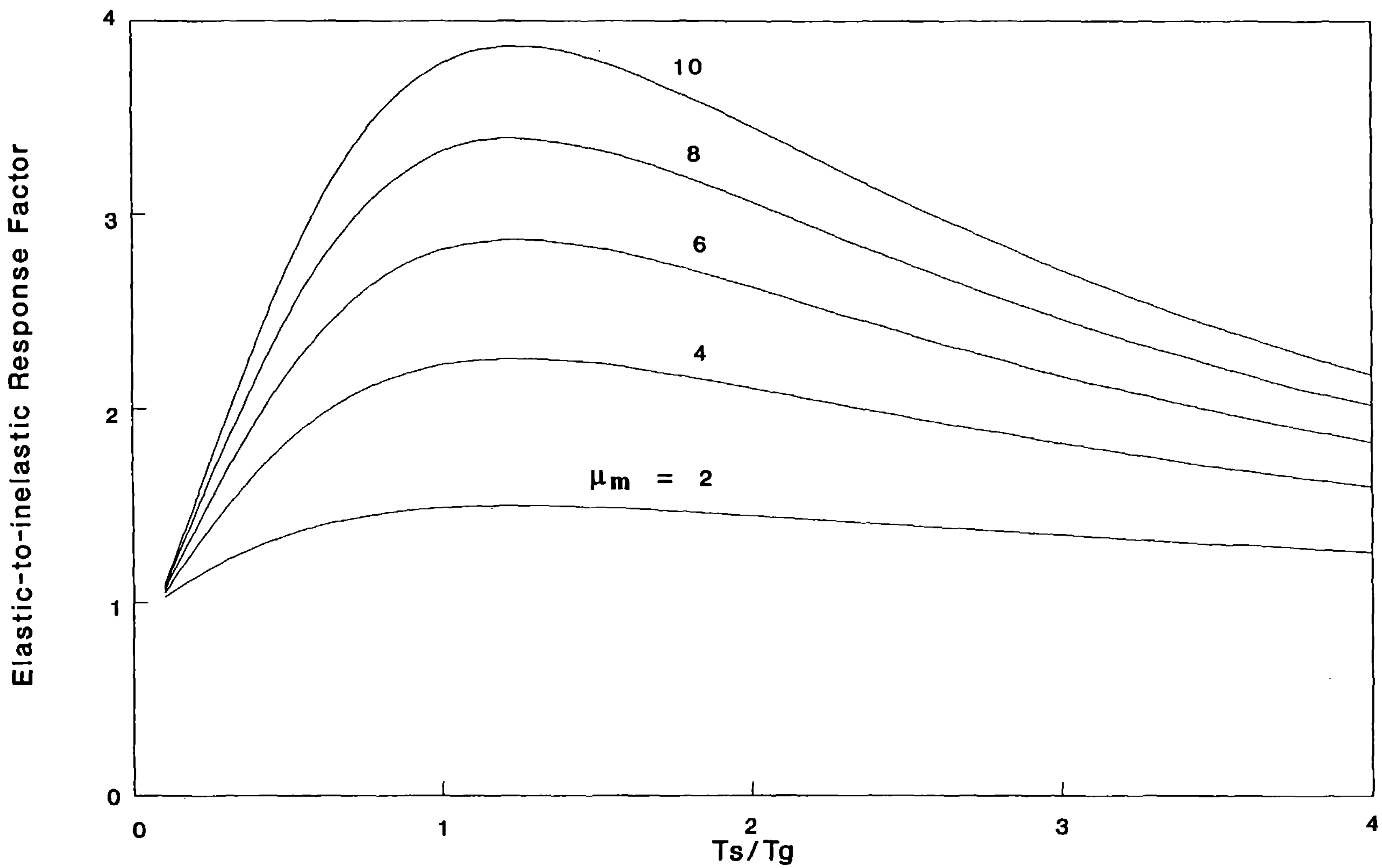


FIGURE 5-4 Median R Factor for RC Structures (Hwang and Jaw 1989)

TABLE 5-III Structural Coefficient D_s for Ductile Moment Frame Specified in Japanese Building Code

Behavior of Members	D_s
Excellent ductility	0.3
Good ductility	0.35
Fair ductility	0.4
Poor ductility	0.45

the same frames but with the excellent ductile members, the D_S is equal to 0.3 and the R_μ factor is 3.3.

Hawkins (1986) defined the ductility index F_i as the ratio of the i-th story shear yielding strength to the corresponding elastic shear force of a multi-story building. From the calibration with the seismic performance of frame buildings in Japan and China, Hawkins recommended the F_i values for RC buildings as shown in table 5-IV. The F_i values are equivalent to the R_μ values. For a column with flexural capacity less than shear capacity and with shear reinforcements satisfying the code requirements, the R_μ (or F_i) value is 2.5. Furthermore, for the same column with hoops satisfying the special confinement requirements adjacent to connections, the R_μ factor is 3.5.

In principle, the R_μ factor is a function of the structural period and other parameters (Riddel and Newmark 1979; Bertero 1986; Hwang and Jaw 1989). However, the R_μ factor specified in seismic provisions such as the NEHRP Provisions and the Uniform Building Code is taken as a constant to simplify the seismic design process. In this study, on the basis of the foregoing discussions, the R_μ factor is taken as 2.5 for the IMR frame and 3.5 for the SMR frame.

5.4 Distribution of Lateral Force

The lateral force F_x acting on the x floor (level) of the structure is calculated from the base shear by using the following formula as specified in the NEHRP Provisions.

TABLE 5-IV Ductility Index (Hawkins 1986)

Member Type	F_i
1. Column with clear height-to-depth ratio less than 2.0 and shear capacity less than flexural capacity (brittle failure)	0.8
2. Column with shear capacity less than flexural capacity but $h/d > 2.0$	1.0
3. Column with flexural capacity less than shear capacity but inadequate hoops and high axial stress	1.5
4. Column as in 3 and with hoops satisfying code requirements for shear	2.5
5. Column as in 3 and with hoops satisfying special confinement requirements adjacent to connections	3.5

$$F_x = C_{vx} V \tag{5.16}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{5.17}$$

where

w_i and w_x = portion of W located at or assigned to level i or x

h_i and h_x = height above the base to level i or x

k = distribution coefficient

= 1.0 for T < 0.5 sec

= 2.0 for T > 2.5 sec

n = number of story

5.5 Determination of Required Reinforcement

The member forces caused by the design earthquake are calculated from the lateral forces. These member forces are combined with forces resulting from gravity loads according to equations (3.3) and (3.4). By using these member forces, structural members are designed according to the ACI code 318-89. In this study, the design of a structure is considered complete after determining the required area of reinforcement instead of the detailed arrangement of reinforcing steel. It is expected that an experienced designer can provide such an arrangement based on the required area of reinforcement.

5.6 Design Example

The above-mentioned design procedure is illustrated by using the first representative frame indicated in table 4-II. Figure 5-5 shows the elevation of the illustrative frame. The sizes of beams and columns are also shown in the figure. The analyses of frame for dead and live loads are performed by means of the computer program PC-STRESS (Wang 1980), which can perform the linear analysis of elastic, statically loaded structures composed of prismatic slender members in two or three dimensions.

The building is assumed to be an ordinary building (I = 1.0) located in a high seismicity area with Z = 0.4. The IMR frame is used and the elastic-to-inelastic response factor R_μ is 2.5. The site condition is assumed to be medium to soft soil S_3; thus, the S factor is 1.5. The total dead load of the building W is calculated as 2262.7 kips (table 5-V). The building height h_n is 63 ft and the fundamental period of the building is determined as

$$T = 0.03 \times h_n^{3/4} = 0.03 \times 63^{3/4} = 0.671 \text{ sec}$$

From equation (5-2), the spectral acceleration coefficient C is calculated as

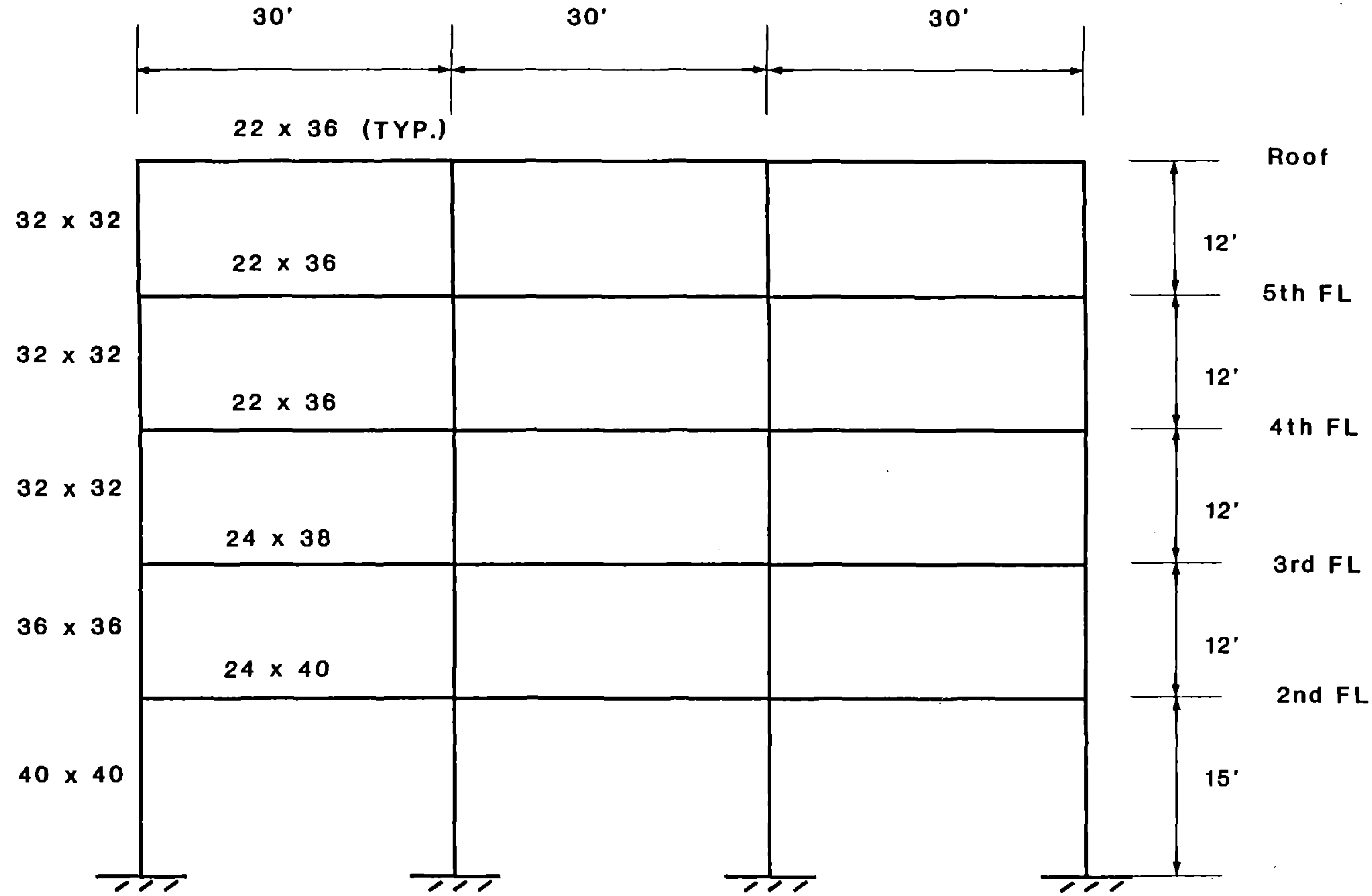


FIGURE 5-5 Elevation of the Illustrative Frame

TABLE 5-V Total Dead Load of the Illustrative Frame

		(kip)
Roof		
Slabs	0.15 x 8.5/12 x (3 x 30) x 25	239.1
Columns	0.15 x 32/12 x 32/12 x 12/2 x 4	25.6
Beams	0.15 x 22/12 x (36-8.5)/12 x (3 x 30)	56.7
Walls	0.015 x 2 x 25 x 12/2	4.5
Partitions	0.040 x (3 x 30) x 25	90.0
Subtotal		415.9
5th & 4th floor		
Slabs	0.15 x 8.5/12 x (3 x 30) x 25	239.1
Columns	0.15 x 32/12 x 32/12 x 12 x 4	51.2
Beams	0.15 x 22/12 x (36-8.5)/12 x (3 x 30)	56.7
Walls	0.015 x 2 x 25 x 12	9.0
Partitions	0.040 x (3 x 30) x 25	90.0
Subtotal		446.0
3rd floor		
Slabs	0.15 x 8.5/12 x (3 x 30) x 25	239.1
Columns	0.15 x (32 x 32 + 36 x 36)/2/144 x 12 x 4	58.0
Beams	0.15 x 24/12 x (38-8.5)/12 x (3 x 30)	66.4
Walls	0.015 x 2 x 25 x 12	9.0
Partitions	0.040 x (3 x 30) x 25	90.0
Subtotal		462.4
2nd floor		
Slabs	0.15 x 8.5/12 x (3 x 30) x 25	239.1
Columns	0.15 x [(36 x 36) x 6 + (40 x 40) x 7.5] /144 x 4	82.4
Beams	0.15 x 24/12 x (40-8.5)/12 x (3 x 30)	70.9
Walls	0.015 x 2 x 25 x (12 + 15)/2	10.1
Partitions	0.040 x (3 x 30) x 25	90.0
Subtotal		492.5
Total dead load	415.9 + 2 x 446.0 + 462.4 + 492.5 =	2262.7

$$C = \frac{1.25S}{T^{2/3}} = \frac{1.25 \times 1.5}{0.671^{2/3}} = 2.45$$

The design base shear V is

$$V = I\left(\frac{ZC}{R_\mu}\right) W = 1.0\left(\frac{0.4 \times 2.45}{2.5}\right) 2262.7 = 885.8 \text{ kips}$$

By using equations (5.16) and (5.17), the design base shear V is distributed over the height of the structure to establish the lateral forces F_x as shown in table 5-VI. By taking the seismic load factor as 1.1, the member forces are combined by using the following load combinations.

$$1.2\,D + 0.5\,L \pm 1.1\,E$$
$$0.9\,D \pm 1.1\,E$$

Following the design procedure specified in the ACI code 318-89, the required flexural reinforcing steel areas of beams and columns are determined and shown in figure 5-6.

TABLE 5-VI Distribution of Lateral Forces

Location	w_x (kip)	h_x (ft)	C_{vx}	F_x (kip)
Roof	415.9	63	0.314	278.3
5th floor	446.0	51	0.268	237.3
4th floor	446.0	39	0.200	177.4
3rd floor	462.4	27	0.139	123.4
2nd floor	492.5	15	0.078	69.4

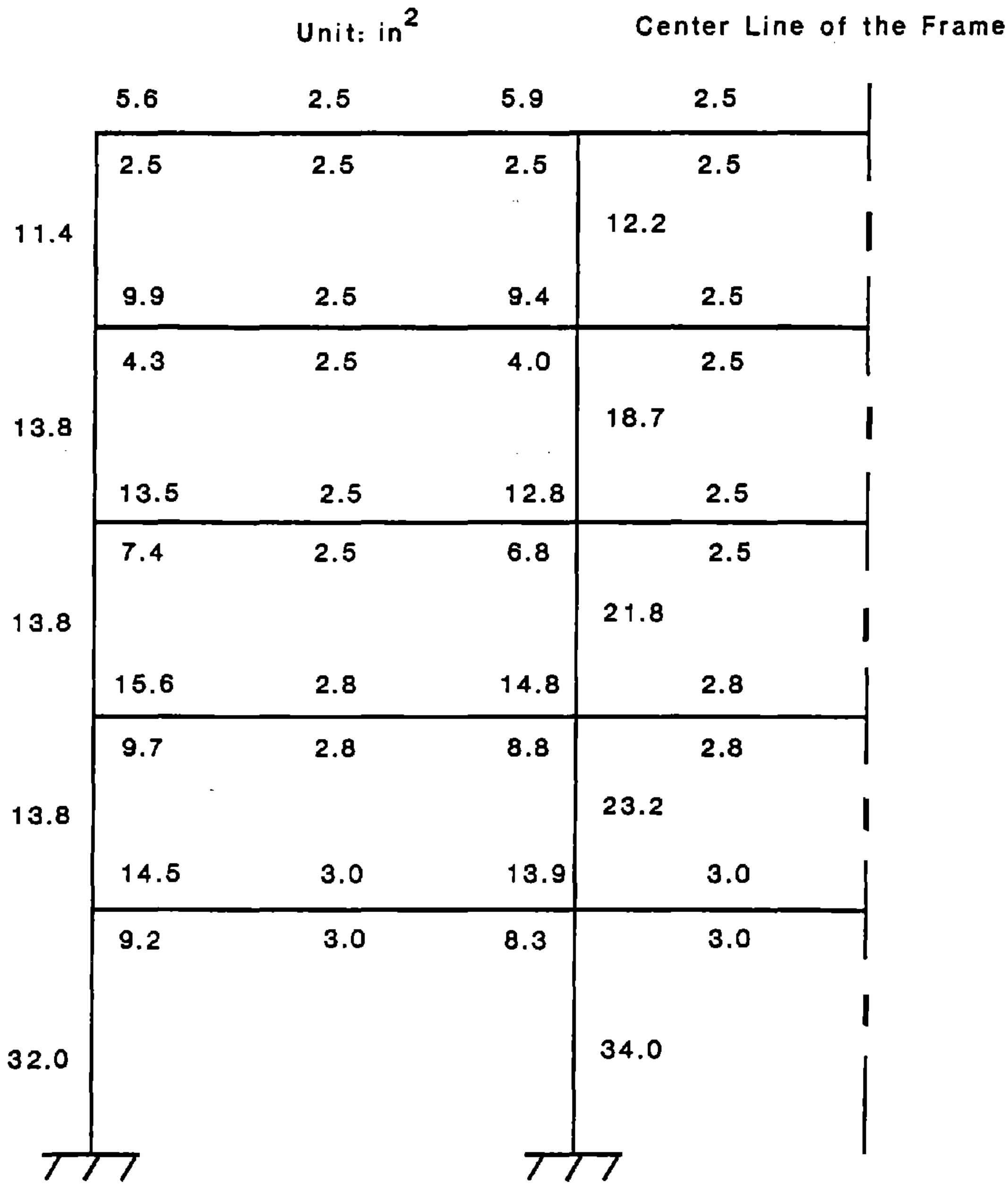


FIGURE 5-6 Required Steel Areas of the Illustrative Frame

SECTION 6

EVALUATION OF SEISMIC PERFORMANCE

A reliability analysis method for evaluating the performance of moment-resisting frame structures subject to earthquake ground motions has been developed (Hwang and Hsu 1991). The evaluation procedure is illustrated in figure 6-1. This method uses a systematic approach to evaluate the seismic performance of a structure. In this method, seismic hazard curve, limit states, nonlinear behavior, structural failure mechanism, and acceptable risk level are integrated to provide an overall view of the structural performance in the event of an earthquake. By using the illustrative frame in Section 5.6 as an example, the evaluation procedure is discussed below.

6.1 Probabilistic Structural Capacity

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

$$SA_C = Ln(\widetilde{SA}_C, \beta_C) \tag{6.1}$$

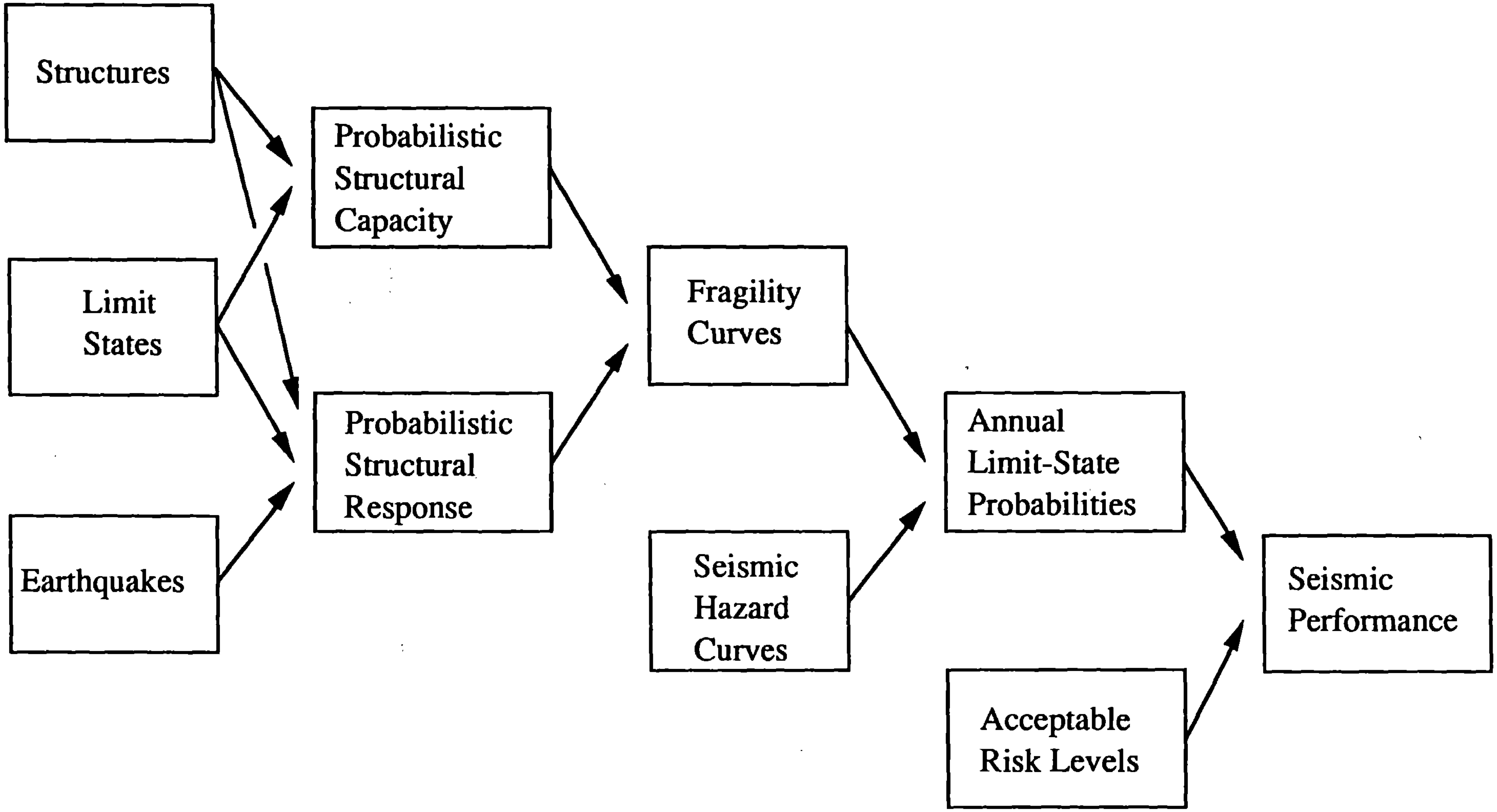


FIGURE 6-1 Evaluation of Seismic Performance of Structures

The median capacity in terms of the spectral acceleration is determined from a capacity curve established by means of the capacity spectrum method proposed by Freeman (1978). In this method, the structural lateral force capacities of a frame structure are determined from the sequential formations of plastic hinges on structural members by gradually increasing the lateral forces applied to the structure. Figure 6-2 shows the capacity curve of the illustrative frame structure.

In the capacity spectrum method, the actual ultimate capacity of a member (beam or column) is used to determine the formation of plastic hinges. To determine actual member ultimate capacity, the best-estimated (mean) values of material strengths instead of the nominal strengths are used in the formulas for capacities specified in the ACI code 318-89. For well-designed structures, the shear capacity is greater than the flexural capacity, and the structural members are expected to fail in the flexural failure mode. Therefore, the flexural capacities of beams and columns are used to represent the actual capacities of structural members. In this study, the flexural capacities of beams are determined on the basis of rectangular cross-sections; that is, the contribution of slabs is not included. The column capacities are affected by the presence of axial force which is a result of the combination of gravity and seismic loads. Thus, the column capacities vary, while the beam capacities remain the same at various stages of structural behavior.

The statistics of gravity loads show that the average of gravity loads acting on a structure is about 100% of the design dead load and 25% of the design live load. Thus, these gravity loads are used to evaluate the seismic

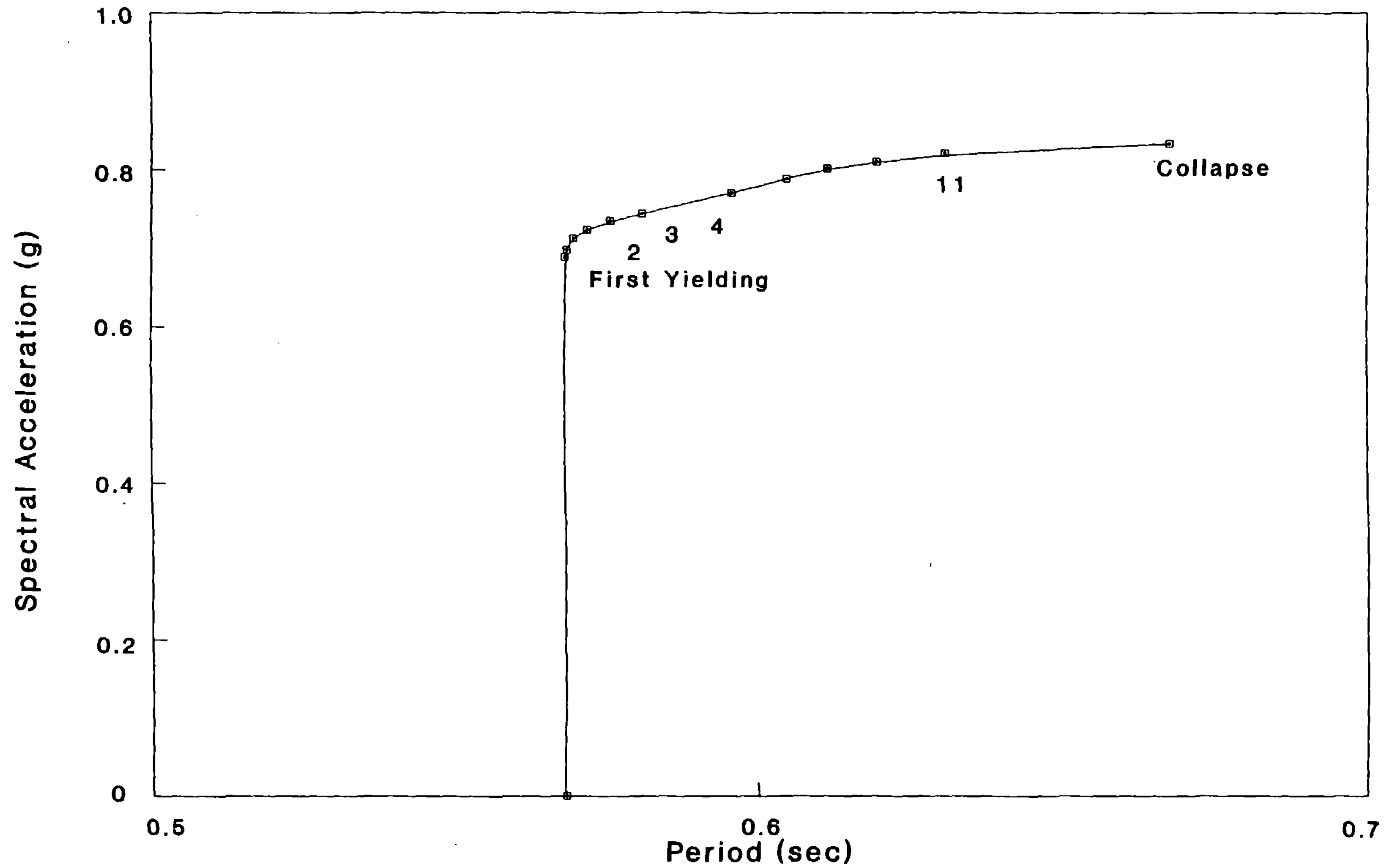


FIGURE 6-2 Capacity Curve of the Illustrative Frame

capacity of a frame. Furthermore, the lateral force acting on each floor is taken as the code-specified lateral-force distribution as shown in equations (5.16) and (5.17) to approximately account for the contribution of higher modes (NEHRP Commentary 1988).

The logarithmic standard deviation of capacity β_C in the range of 0.28 to 0.34 was reported by an ASCE publication ("Uncertainty" 1986). Hwang and Jaw (1990) used β_C as 0.3 to evaluate the capacity of shear wall structures. In this study, the β_C is taken as 0.3 for both first-yielding and collapse limit states. The probabilistic structural capacity of the illustrative frame structure is summarized in table 6-I.

6.2 Probabilistic Structural Response

The seismic structural response is mainly affected by seismic source, path attenuation, local site conditions and structural properties such as damping and structural period. Hwang and Jaw (1990) calculated the structural responses of a shear wall from 81 earthquake-structure samples and indicated that the lognormal distribution fits the data very well. Other researchers also proposed to use the lognormal distribution ("Uncertainty" 1986; McCann, Jr. 1983; Kameda et al. 1982). Therefore, the probabilistic structural response SA_R in this study is described by using a lognormal distribution.

$$SA_R = Ln(\widetilde{SA}_R , \beta_R) \qquad (6.2)$$

TABLE 6-I Probabilistic Structural Capacity

Limit State	$\tilde{SA}_C$ (g)	β_C
First Yielding	0.689	0.3
Collapse	0.833	0.3

where $\tilde{SA}_R$ is the median spectral acceleration corresponding to a limit state and β_R is the logarithmic standard deviation of response. In this study, the response spectrum is represented by a peak ground acceleration times a normalized response spectrum (figure 5-2). As a consequence, the median structural response $\tilde{SA}_R$ is also expressed in a similar manner.

$$\tilde{SA}_R = A_p \times \tilde{SA}_n \tag{6.3}$$

where A_p is the peak ground acceleration, which is used as a parameter in the fragility analysis, and $\tilde{SA}_n$ is the median value of the normalized spectral acceleration that can be determined from the normalized design response spectrum with a 5% damping ratio. If the damping ratio is different from 5%, the normalized response spectrum is modified by the damping adjustment factors (table 6-II) specified in the Seismic Design Guidelines for Essential Buildings (Tri-Services Guidelines 1986). For reinforced concrete structures, the damping ratios at first-yielding and collapse limit states are taken as 5% and 10%, respectively; thus, the damping adjustment factors are 1.0 and 0.8, respectively. From the multiplication of the normalized response spectrum and the damping adjustment factors, the normalized response spectra for the first-yielding and collapse limit states are constructed and shown in figure 6-3. The structural period at the stage of first yielding is 0.568 sec. The normalized spectral acceleration at this period is 2.733. If the PGA is 0.15 g, then spectral acceleration is 0.15 times 2.733 and equal to 0.41 g.

TABLE 6-II Damping Adjustment Factors

Damping Ratio (%)	Damping Adjustment Factor
2	1.25
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60

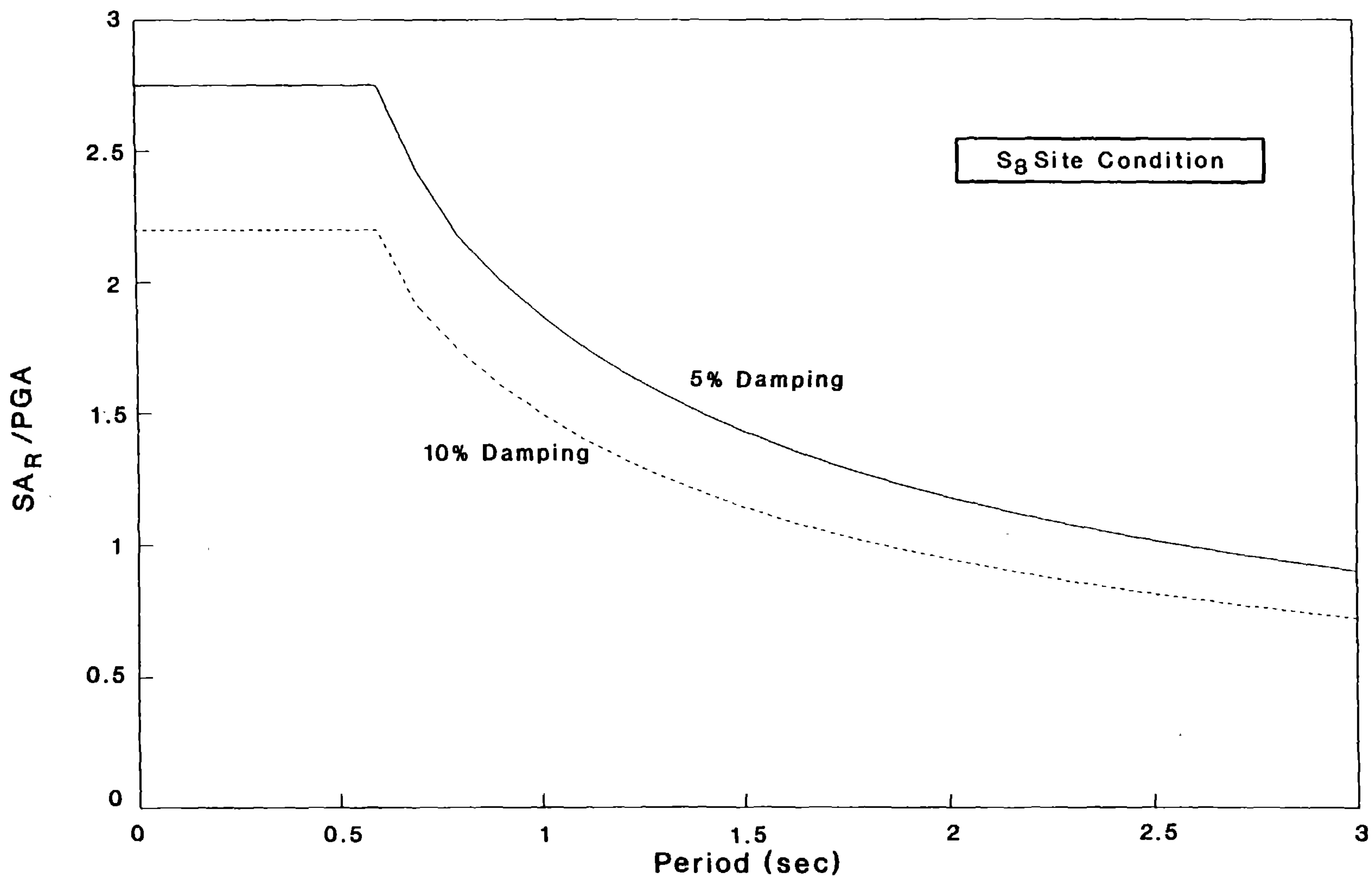


FIGURE 6-3 Response Spectra Corresponding to Different Damping Ratios

Seed et al. (1976) evaluated the site-dependent response spectra using 104 actual earthquake records. From the mean and mean plus one standard deviation response spectra, the β_R is estimated to be about 0.34. The Tri-Services Guidelines reported that the coefficient of variation (COV) of the spectral acceleration is about 0.3 - 0.5. The corresponding β_R is 0.29 - 0.48. Hwang and Lee (1990) evaluated the site-specific response spectrum for the Sheahan pumping station, Memphis, Tennessee. Uncertainties in three seismic parameters and three site parameters were included in the analysis. The COVs of the spectral accelerations were determined as 0.25 to 0.49 (the corresponding β_R is 0.25 to 0.46). The β_R of the response spectra for nuclear power plant facilities in the range of 0.25 - 0.40 was reported ("Uncertainty" 1986). On the basis of the above discussions, the β_R of response spectra is taken as 0.35 in this study.

The major structural parameters affecting the structural response are damping ratio and structural period. Ang and Hu (1983) collected test results from cyclic loading tests and damage records for reinforced concrete buildings. The COV of the damping ratio was evaluated to be 0.2 (β_R = 0.2). The β_R of 0.09 - 0.2 was also estimated for nuclear power plant facilities ("Uncertainty" 1986). In this study, the β_R for damping ratio is taken as 0.2. Sources of uncertainty in structural period are quite complex, but the combined uncertainties can be determined on the basis of the observed and computed periods of structures. Haviland (1976) collected available data on periods and reported that the COV is about 0.336 (β_R = 0.33). In this study, the β_R for structural period is taken as 0.3. Thus, the β_R for structural properties including the damping ratio and structural period is

$$\beta_R = \sqrt{0.2^2+0.3^2} = 0.36$$

The overall uncertainty in structural response can be determined by combining uncertainties in response spectra and structural properties.

$$\beta_R(\text{overall}) = \sqrt{0.35^2+0.36^2} = 0.5$$

Ang and Newmark (1977) reported that the COV associated with the prediction of structural response is about 0.56 (β_R = 0.52) on the basis of uncertainties in various components of the response analysis. In addition, the report ("Uncertainty" 1986) indicated that the β_R for building response is 0.40 - 0.59. Thus, the β_R = 0.5 used in this study is very close to the values indicated in these two studies. Table 6-III summarizes the probabilistic structural response at each limit state for the illustrative frame structure.

6.3 Fragility Analysis

For a given level of peak ground acceleration, the fragility of a structure with respect to a limit state is defined as the conditional probability that the structural response SA_R exceeds the structural capacity SA_C. The conditional limit-state probability P_f is determined as:

$$P_f = P_r(SA_C/SA_R \leq 1) \tag{6.4}$$

TABLE 6-III Probabilistic Structural Response

Limit State	$\widetilde{SA}_R$ (g)	β_R
First Yielding	2.733 A_p	0.5
Collapse	1.964 A_p	0.5

Since both SA_R and SA_C are lognormally distributed, P_f can be determined as:

$$P_f = \Phi\left[\frac{-\ln(\tilde{SA}_C/\tilde{SA}_R)}{(\beta_C^2 + \beta_R^2)^{1/2}}\right] \tag{6.5}$$

For the illustrative structure, the conditional limit-state probabilities with respect to both first-yielding and collapse limit states at various levels of PGAs are calculated and the corresponding fragility curves are shown in figure 6-4. The limit-state probability increases as the PGA level becomes higher (an expected trend); as the earthquake becomes more severe, the chance of the building reaching its limit state increases.

6.4 Annual Limit-State Probability

The annual (unconditional) limit-state probability PF of a structure with respect to a specified limit state is determined from the integration of seismic hazard and fragility curves.

$$PF_Y \text{ or } PF_C = \sum_{j=1}^{N} \lambda a_j \times P_f(a_j) \tag{6.6}$$

where PF_Y and PF_C stand for the the first-yielding and collapse annual limit-state probabilities, respectively; P_f is the conditional limit-state probability expressed as a fragility curve; and λa_j is the annual occurrence of an earthquake with a specified peak ground acceleration a_j. Given a

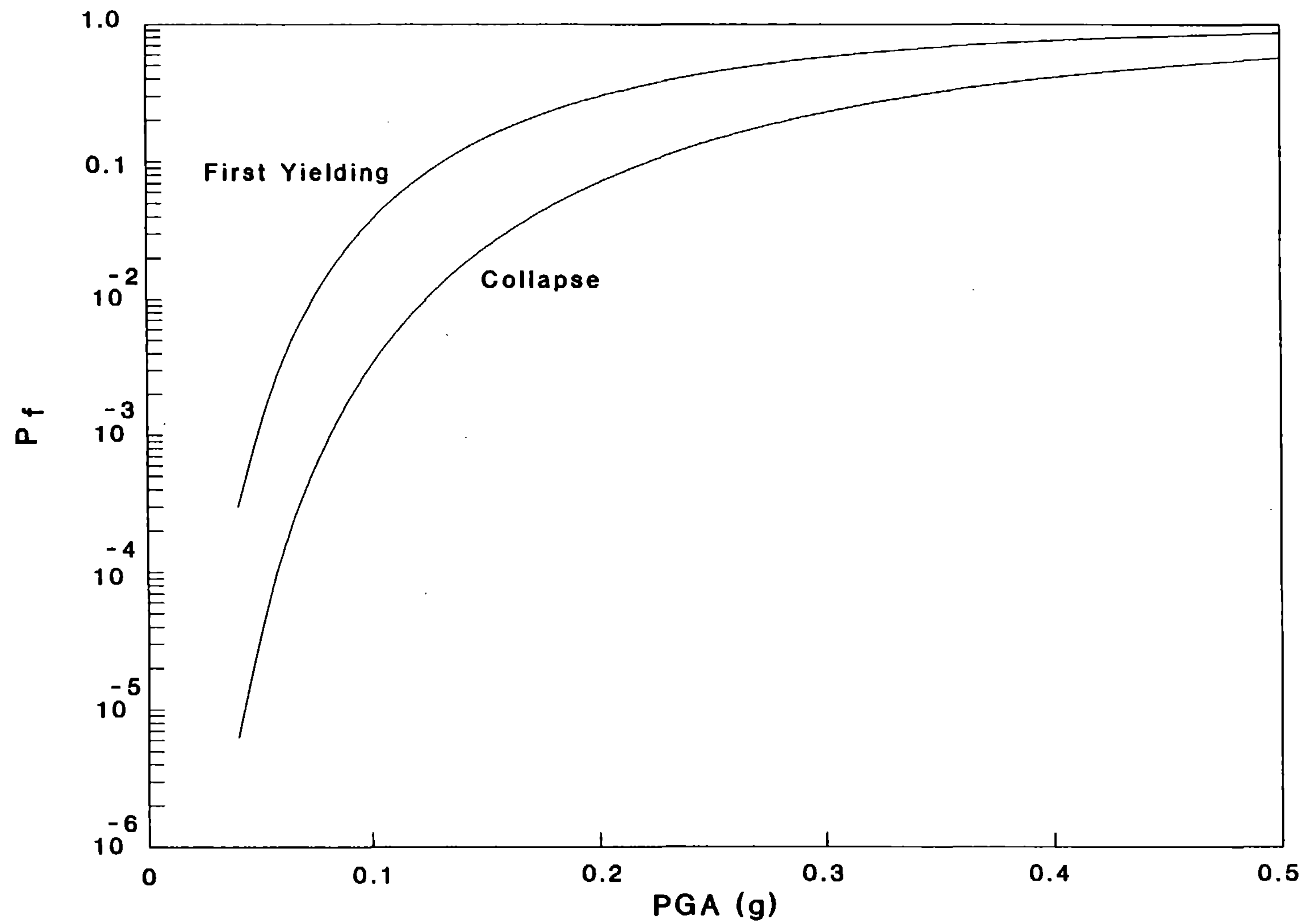


FIGURE 6-4 Fragility Curves

seismic hazard curve as shown in figure 2-1, λa_j can be determined as follows:

$$\lambda a_j = F_A(a_j + \frac{\Delta a}{2}) - F_A(a_j - \frac{\Delta a}{2}) \qquad (6.7)$$

where $\Delta a = (a_{max} - a_o)/N$ 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 a_{max} is the maximum PGA possible at a site. For the first-yielding and collapse limit states, a_{max} is determined from the earthquake with the return periods of 100 and 2000 years, respectively.

The illustrative frame is designed for PGA of 0.4 g; thus, the hazard curve with E_D = 0.4 g (figure 2-1) is used to evaluate seismic performance of the structure. This seismic hazard curve is replotted in figure 6-5. The fragility curves with respect to both first-yielding and collapse limit states are displayed in figure 6-5. From the integration of the seismic hazard curve and the fragility curve, the annual limit-state probability can be determined. For the first-yielding limit, PF_Y is 0.0077 per year and for the collapse limit state, PF_C is calculated as 0.0038 per year.

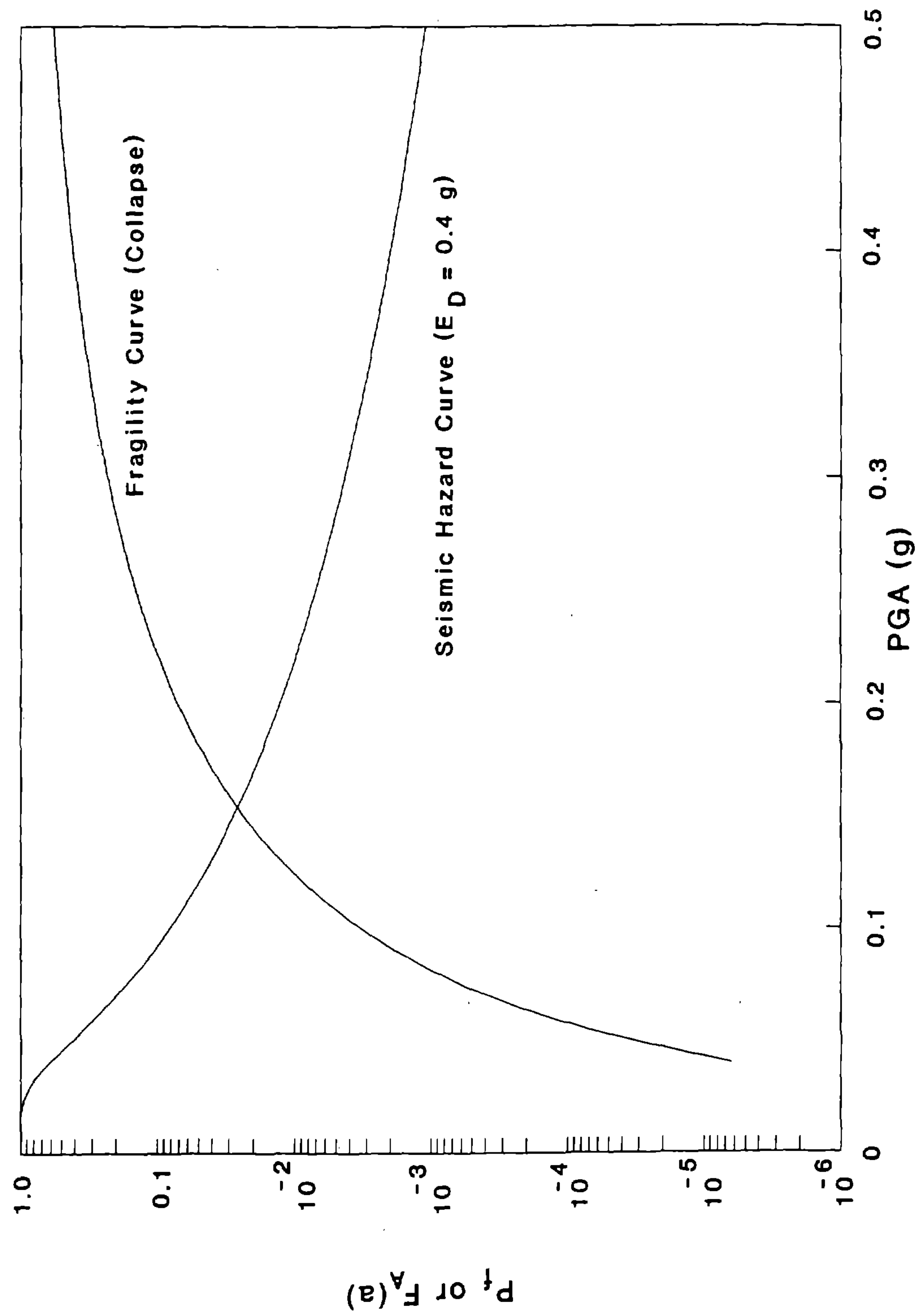


FIGURE 6-5 Seismic Hazard and Fragility Curves

SECTION 7

DETERMINATION OF OPTIMUM LOAD FACTOR

The load and resistance factors in the seismic LRFD criteria for buildings can be determined so that the limit-state probabilities of the representative structures are sufficiently close to the acceptable risk level (target limit-state probability) defined in Section 2. The closeness of these two probabilities can be measured by an objective function (Ellingwood et al. 1980; Hwang et al. 1987; Shinozuka et al. 1989). For collapse of a structure as the limit state, the objective function $\Omega(\gamma, \phi)$ is defined as follows:

$$\underset{\min}{\Omega(\gamma, \phi)} = \sum_{j=1}^{N} w_j \left\{ \frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})} \right\}^2 \qquad (7.1)$$

where $PF_{C,T}$ 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 and w_j represents a weight factor for the j-th sample structure. On the basis of the Latin hypercube sampling technique, each sample structure in table 4-II is equally representative and thus w_j is equal to 1.0.

The selected LRFD format includes the gravity load factors, seismic load factor, and resistance factors. Since both the gravity load factors and resistance factors are preset, the above optimization can be performed for the seismic load factor γ_E only. Thus, equation (7.1) can be written as

$$\Omega(\gamma_E)_{min} = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{7.2}$$

The optimum seismic load factor determined above is for ordinary buildings. For high-risk and essential buildings, the importance factor I is used to increase the design strength. A similar optimization technique can be used to determine the optimum I value as shown in equation (7.3).

$$\Omega(I)_{min} = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{7.3}$$

SECTION 8

PARAMETRIC STUDIES

8.1 Limit States

According to the current code design philosophy, structural performance in the event of an earthquake should be checked for both first-yielding and collapse limit states. In reality, one of the limit states will be reached first. For the comparison of structural performance with respect to these two limit states, the yielding index YI and the collapse index CI are defined as follows:

$$YI = \frac{PF_Y}{PF_{Y,T}} \tag{8.1}$$

and

$$CI = \frac{PF_C}{PF_{C,T}} \tag{8.2}$$

where

PF_Y = the first-yielding limit-state probability

PF_C = the collapse limit-state probability

$PF_{Y,T}$ = the target first-yielding limit-state probability

$PF_{C,T}$ = the target collapse limit-state probability

When PF_Y is larger than $PF_{Y,T}$, the value of YI is greater than 1.0 and the structural performance is not satisfactory with respect to the first-yielding limit state. Similarly, if the value of CI is greater than 1.0, the structural performance is not satisfactory with respect to the collapse limit state.

A nine-story frame structure is used to investigate which limit state is reached first. The frame configuration and design parameters are listed in table 8-I. The site condition is assumed as S_2 and thus the site coefficient is 1.2. The frame is designed according to the proposed design procedure with the seismic load factor of 1.0 and the design earthquake of 0.1 g, 0.2 g, 0.3 g, and 0.4 g, respectively. The annual limit-state probabilities with respect to first-yielding and collapse limit states are calculated and shown in table 8-II. The frame structure is designed as an ordinary building; thus, the target limit-state probability is 1/50 per year for the first-yielding limit state and 1/1000 per year for collapse limit state. From equations (8.1) and (8.2), the yielding index and collapse index are computed and also shown in table 8-II. It is noted that the collapse index for each design condition is much larger than the corresponding yielding index. This may be due to two factors. First, the upper bound of a large earthquake (2000-year earthquake) is much larger than the design earthquake (475-year earthquake); thus, the upper bound of the large earthquake has significant effects on the collapse limit-state probability. Second, the target limit-state probability for the first yielding is larger than the target probability for the collapse limit state; thus, the collapse limit state is more difficult to be satisfied. Table 8-II demonstrates that the collapse of a structure is the critical limit state to be designed and checked in the event of an earthquake. Thus, in the following study, only the collapse limit state is considered.

TABLE 8-I Design Parameters for a Nine-story Frame Structure

Item	Value
No. of stories	9
Story height (ft)	12
1st story height (ft)	15
No. of spans	4
Span length (ft)	25
Transverse spacing (ft)	25
Live load (psi)	50
Roof live load (psi)	16
RC weight (psf)	150
f_y (ksi)	60
f_c' (ksi)	4

TABLE 8-II Yielding and Collapse Indexes for a Nine-story Frame Structure

Frame	E_D (g)	PF_Y	PF_C	YI	CI
1	0.1	4.58E-04	1.75E-04	0.02	0.17
2	0.2	1.51E-03	9.24E-04	0.08	0.93
3	0.3	4.08E-03	2.06E-03	0.20	2.08
4	0.4	7.91E-03	2.98E-03	0.40	2.94

8.2 Seismic Zones

For the four frame structures, the annual collapse limit-state probabilities PF_C shown in table 8-II have significant variations. At the collapse stage the safety ratio SR is defined as follows:

$$SR = SA_C/SA_R$$

where SA_C and SA_R are the deterministic structural capacity and response, respectively, at the collapse stage. Table 8-III shows SA_C, SA_R, and SR for the four frames with the design earthquake ranging from 0.1 g to 0.4 g. It is noted that when the ratio of the design earthquake increases four times, the collapse structural capacity is only increased three times because of the constant gravity load. On the other hand, the ratio of the structural response is increased 6.3 times, because the structure becomes much stiffer and the fundamental period is shorter; thus, the structural response becomes larger as shown in figure 8-1. Consequently, the safety ratio is reduced to about one half of frame 1. Because the safety ratio is not uniform under various conditions, the seismic load factor cannot be the same. Therefore, the seismic load factors will be determined according to different levels of the design earthquake.

8.3 Site Conditions

In building codes, the site coefficients (S factor) are used to represent various site conditions. As discussed in the previous section, the hard-rock and very soft soil conditions are not included in this study. The nine-story

TABLE 8-III Deterministic Safety of a Nine-story Frame Structure

Frame	E_D (g)	$\frac{E_{D,i}}{E_{D,1}}$	SA_C (g)	$\frac{SA_{C,i}}{SA_{C,1}}$	SA_R (g)	$\frac{SA_{R,i}}{SA_{R,1}}$	SR	$\frac{SR_{,i}}{SR_{,1}}$
1	0.1	1.0	0.155	1.00	0.063	1.00	2.46	1.00
2	0.2	2.0	0.230	1.48	0.150	2.38	1.53	0.62
3	0.3	3.0	0.343	2.21	0.273	4.33	1.26	0.51
4	0.4	4.0	0.461	3.00	0.397	6.30	1.16	0.47

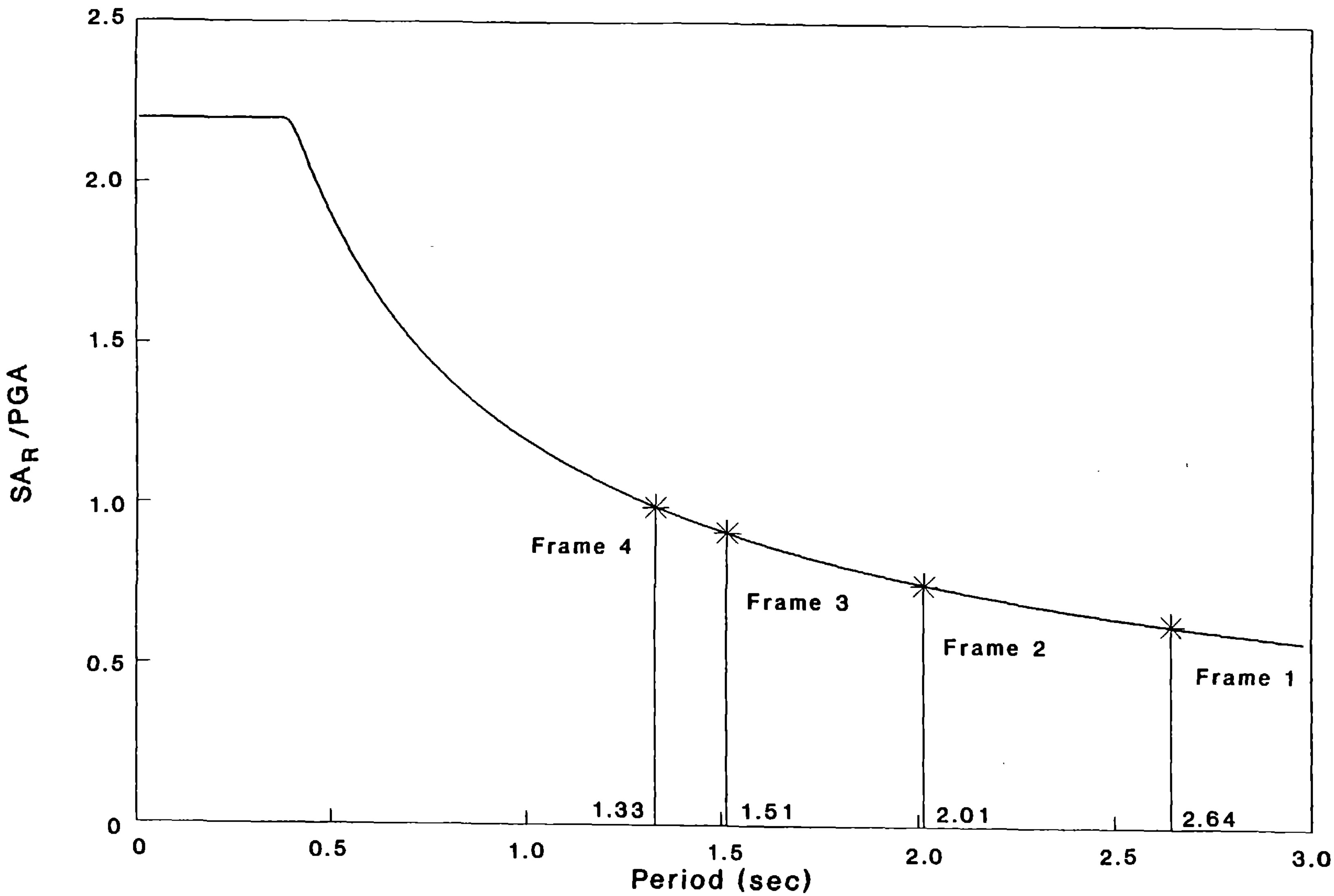


FIGURE 8-1 Structural Response of a Nine-Story Frame Structure

frame structure as described in table 8-I is designed for three soil conditions: S_1, S_2, and S_3. These three frames are designed for a 0.4 g earthquake by using the proposed design criteria with the seismic load factor of 1.0. The limit-state probabilities are determined and shown in table 8-IV. The variation is small by comparison with the one shown in the zone factor. Thus, the seismic load factors will not be determined for each site condition. In other words, all site conditions will have the same seismic load factor.

TABLE 8-IV Effect of Soil Conditions

Soil Category	Site Coefficient	PF_C
S_1	1.0	2.40E-03
S_2	1.2	2.98E-03
S_3	1.5	2.35E-03

SECTION 9

RELIABILITY-BASED SEISMIC DESIGN CRITERIA

The reliability-based seismic design criteria for ordinary, high-risk, and essential buildings are developed by using the procedure described in Sections 3 through 7. The procedure includes five steps and each step is briefly summarized below.

1. Select a load combination format

The load combination format selected for this study is the LRFD format. Thus, the load combinations involving earthquakes are:

$$\phi R \geq 1.2\, D + 0.5\, L \pm \gamma_E\, E \tag{9.1}$$

$$\phi R \geq 0.9\, D \pm \gamma_E\, E \tag{9.2}$$

where γ_E is the seismic load factor to be determined. The dead and live load factors are preset on the basis of past experience in dealing with the reliability-based seismic design criteria. The resistance factors (strength reduction factors) ϕ used in this study are the same as those specified in the ACI code 318-89.

2. Establish representative frame structures

By using the Latin hypercube sampling technique, six representative moment-resisting frames are constructed (Section 4). These frames are then used with each of the four levels of the design earthquake to derive the seismic load factor and the importance factor.

3. Design structures according to the proposed criteria

Each of the representative structures is designed according to the criteria proposed in this study (Section 5).

4. Evaluate reliability of structures

The annual collapse limit-state probability PF_C for each of the representative structures is evaluated by using the reliability analysis method described in Section 6.

$$PF_C = \sum_{k=1}^{n} \lambda a_k \times P_f(a_k) \tag{9.3}$$

where P_f is the conditional limit-state probability estimated on the basis of the probabilistic structural capacity and response and λa_k is the annual occurrence of an earthquake a_k, estimated from a seismic hazard curve.

5. Determine seismic load factor

For ordinary buildings, the acceptable collapse limit-state probability $PF_{C,T}$ is 1/1000 per year. The seismic load factor γ_E is determined by the optimization of the following objective function (Section 7).

$$\Omega_{min}(\gamma_E) = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{9.4}$$

For high-risk and essential buildings, the acceptable risk levels are more stringent, that is, $PF_{C,T}$ of 1/2000 per year for high-risk buildings and 1/5000 or 1/10,000 per year for essential buildings. Since the seismic load factor is determined for ordinary buildings, the importance factor I is used to increase the design strength to meet the more stringent acceptable risk levels. The importance factor can also be determined by the optimization.

$$\Omega_{min}(I) = \sum_{j=1}^{N} \left\{\frac{\log(PF_{C,j}) - \log(PF_{C,T})}{\log(PF_{C,T})}\right\}^2 \tag{9.5}$$

9.1 Ordinary Buildings

For the design earthquake E_D = 0.4 g, six representative frames (table 4-II) are first designed according to the proposed design criteria with the trial seismic load factor 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 (table 9-I). To obtain the optimum value of γ_E, these limit-state probabilities are substituted into equation (9.4) to compute the value of the objective function $\Omega(\gamma_E)$ for each trial value of γ_E. Figure 9-1 shows the curve fitting the data of γ_E versus $\Omega(\gamma_E)$. In the figure, the fitted line passes through each data point by means of the Steinman interpolation (Steinman 1980). The optimum γ_E corresponds to the lowest point of the curve; thus, the optimum γ_E is determined as 1.3 for

TABLE 9-I Annual Collapse Limit-State Probabilities of Ordinary Buildings Designed for E_D = 0.4 g

Frame	PF_C (/yr)				
	$\gamma_E = 1.1$	$\gamma_E = 1.2$	$\gamma_E = 1.3$	$\gamma_E = 1.4$	$\gamma_E = 1.5$
1	3.83E-03	2.90E-03	2.41E-03	2.03E-03	1.65E-03
2	1.50E-03	9.16E-04	7.63E-04	5.08E-04	3.56E-04
3	3.37E-03	2.63E-03	1.95E-03	1.58E-03	1.29E-03
4	1.83E-03	9.83E-04	6.08E-04	3.64E-04	2.21E-04
5	2.18E-03	1.48E-03	1.34E-03	8.53E-04	6.31E-04
6	2.05E-03	1.54E-03	1.80E-03	7.28E-04	8.17E-04

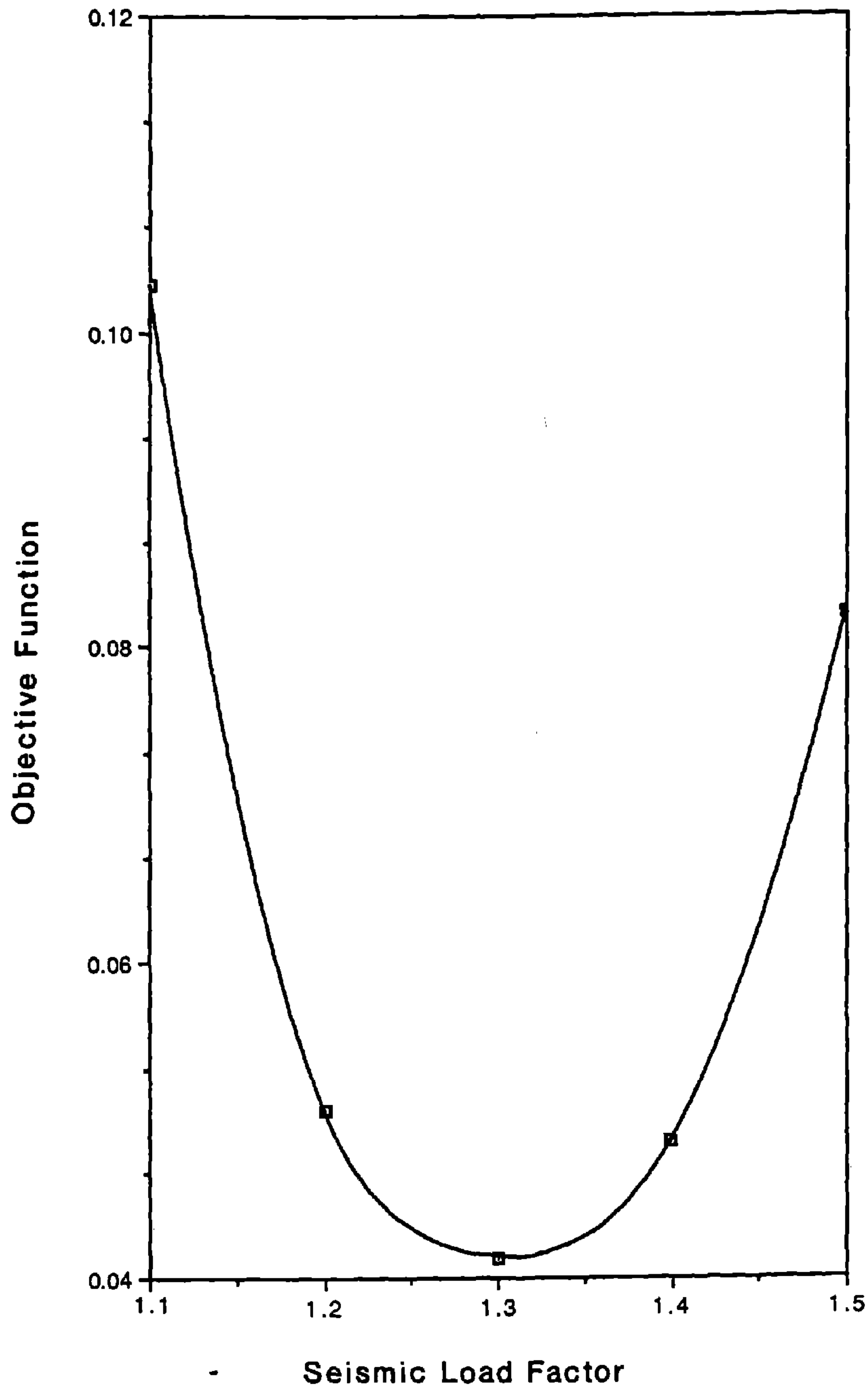


FIGURE 9-1 Determination of Seismic Load Factor for Ordinary Buildings (E_D = 0.4 g)

for ordinary buildings located in high seismicity zone with the Z factor of 0.4 (E_D = 0.4 g).

Similarly, six representative frames are designed with the design earthquake E_D = 0.3 g and the trial γ_E = 0.9 - 1.3. Table 9-II shows the annual collapse limit-state probabilities and figure 9-2 displays the plot of the objective function. The optimum γ_E is determined as 1.15 for the seismic zone with the Z factor of 0.3 (E_D = 0.3 g). For the seismic zone with the Z factor of 0.2 (E_D = 0.2 g), the trial seismic load factors γ_E = 0.6 - 1.0 are used. The annual collapse limit-state probabilities are summarized in table 9-III and the values of the objective function are plotted in figure 9-3. The optimum γ_E is determined as 0.8. This value is less than 1.0 because dead load effects have significant contribution to the load combinations and provide relatively higher seismic resistance. For the region with the design earthquake of 0.2 g, the seismic load factor of 1.0 is recommended for the design of buildings. The increase of the seismic load factor will give the structures extra protection against earthquakes.

For low seismicity zone such as the design earthquake equal to 0.1 g, it is apparent that the load combinations including earthquake do not govern the design of structure. Thus, the representative frames are designed by using the load combination including only dead and live loads as specified in the ASCE 7-88.

$$\phi R \geq 1.4D \tag{9.6}$$

$$\phi R \geq 1.2D + 1.6L \tag{9.7}$$

TABLE 9-II Annual Collapse Limit-State Probabilities of Ordinary Buildings Designed for E_D = 0.3 g

Frame	P_{FC} (/yr)				
	$\gamma_E = 0.9$	$\gamma_E = 1.0$	$\gamma_E = 1.1$	$\gamma_E = 1.2$	$\gamma_E = 1.3$
1	3.15E-03	2.52E-03	2.21E-03	1.74E-03	1.42E-03
2	1.32E-03	1.15E-03	9.90E-04	6.20E-04	4.62E-04
3	3.32E-03	2.41E-03	1.81E-03	1.35E-03	1.08E-03
4	1.32E-03	1.14E-03	7.83E-04	5.14E-04	2.88E-04
5	1.54E-03	1.18E-03	8.63E-04	6.75E-04	5.23E-04
6	2.20E-03	1.86E-03	1.40E-03	1.04E-03	6.73E-04

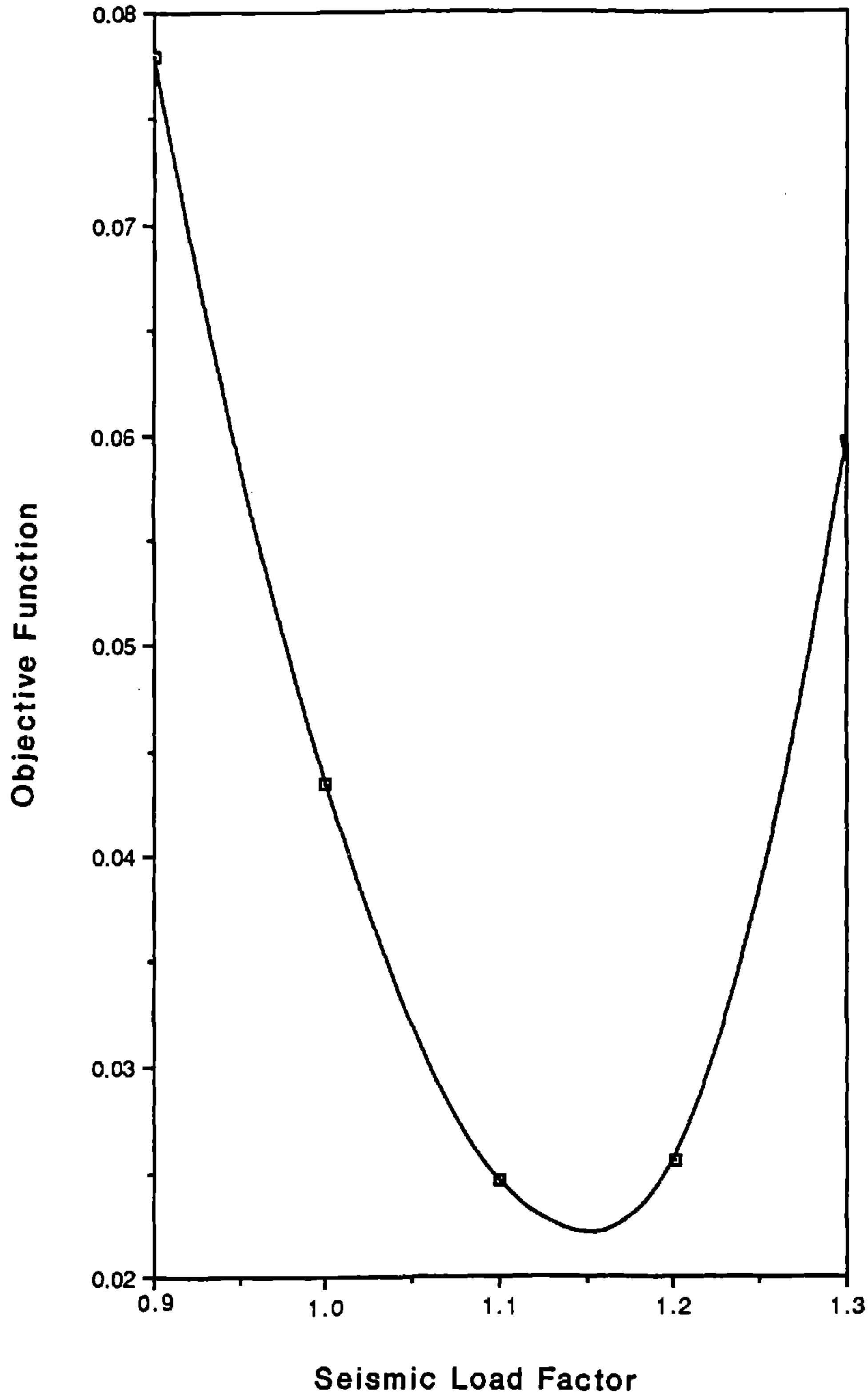


FIGURE 9-2 Determination of Seismic Load Factor for Ordinary Buildings (E_D = 0.3 g)

TABLE 9-III Annual Collapse Limit-State Probabilities of Ordinary Buildings Designed for E_D = 0.2 g

Frame	PF_C (/yr)				
	$\gamma_E = 0.6$	$\gamma_E = 0.7$	$\gamma_E = 0.8$	$\gamma_E = 0.9$	$\gamma_E = 1.0$
1	2.08E-03	1.59E-03	1.30E-03	1.14E-03	7.39E-04
2	5.66E-04	5.13E-04	5.47E-04	4.97E-04	4.11E-04
3	2.50E-03	1.82E-03	1.35E-03	1.24E-03	9.37E-04
4	1.12E-03	6.90E-04	6.31E-04	4.57E-04	4.13E-04
5	6.08E-04	6.20E-04	6.06E-04	5.14E-04	4.05E-04
6	1.67E-03	1.48E-03	1.31E-03	9.58E-04	7.60E-04

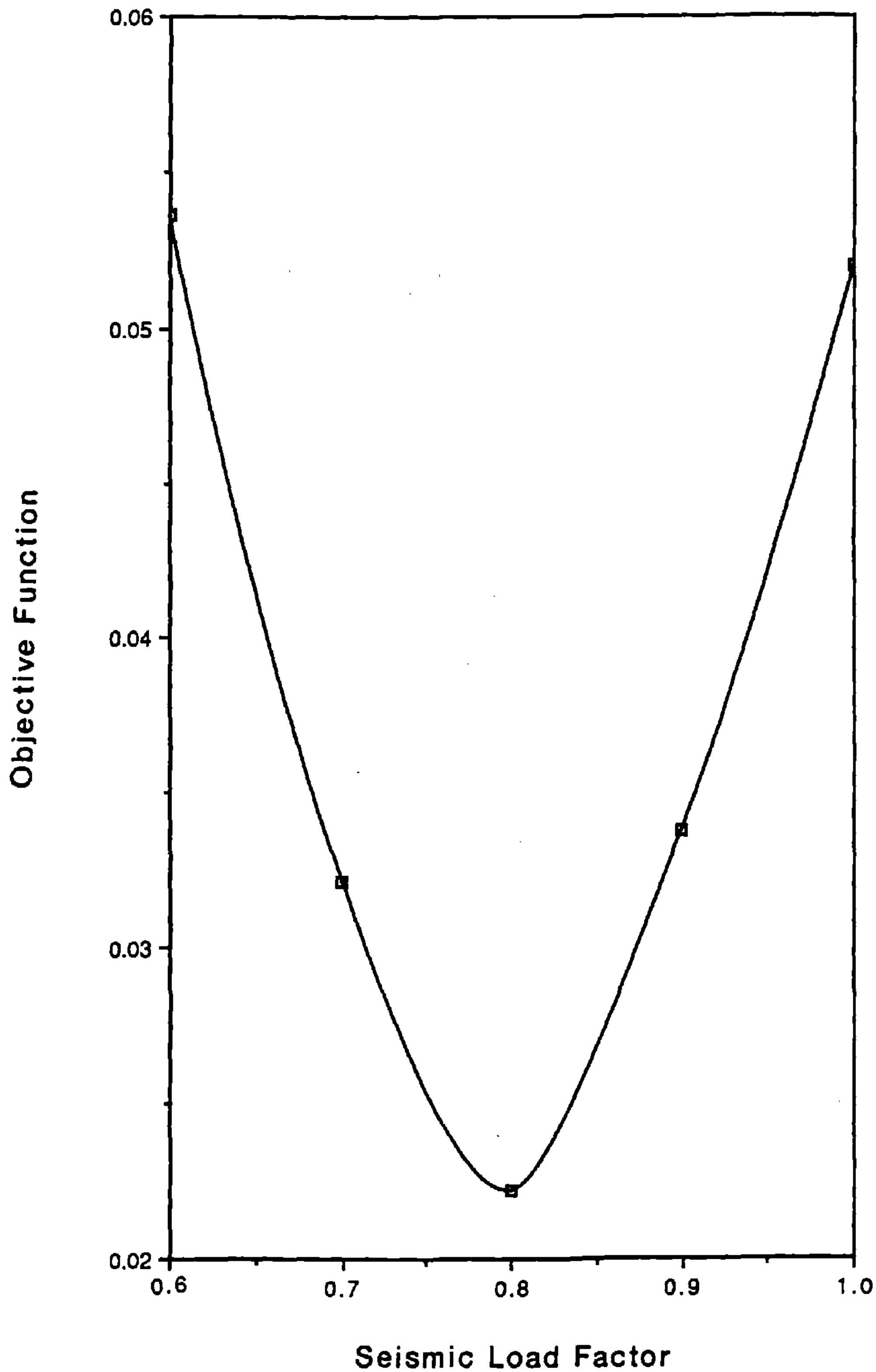


FIGURE 9-3 Determination of Seismic Load Factor for Ordinary Buildings (E_D = 0.2 g)

The annual collapse limit-state probabilities of these six frames are evaluated by using the same reliability analysis method and the results are indicated in table 9-IV. The annual collapse limit-state probabilities are all less than the acceptable probability, that is, 1/1000 per year. Thus, the design for gravity loads is sufficient for low seismicity region such as $E_D \leq 0.1$ g. It is noted that even though the design is only based on gravity loads, the detailing of frame elements (beams, columns, and joints) should be adequate to form the failure mechanism when the structure is subject to an earthquake.

9.1.1 Ordinary Buildings Designed with a Different R_μ Value

In the proposed design criteria, the elastic-to-inelastic response factor R_μ is set equal to 2.5 for the intermediate moment-resisting frames. However, the R factor of 4 is specified in the NEHRP Provisions for the same type of frame. To investigate the effects of the higher R_μ value on the seismic load factors, six representative frames are designed with $E_D = 0.4$ g by using the proposed design criteria, except the R_μ factor is set as 4.0. The reliability analyses of these frames are carried out and the annual collapse limit-state probabilities are summarized in table 9-V. Substituting these limit-state probabilities into the objective function, the optimum γ_E is determined as 2.05 (figure 9-4). Thus, the larger R_μ factor used in the design is compensated by the larger seismic load factor. This clearly demonstrates that the seismic load factor and the elastic-to-inelastic response factor (or the response modification factor) are interrelated, since the proposed design criteria are based on the systematic approach and take all the essential components into consideration. On the basis of the studies

TABLE 9-IV Annual Collapse Limit-State Probabilities of Buildings Designed for Gravity Loads

Frame	PF_C (/yr)
1	2.20E-04
2	7.72E-05
3	6.37E-04
4	6.75E-04
5	6.75E-05
6	2.06E-04

TABLE 9-V Annual Collapse Limit-State Probabilities of Ordinary Buildings Designed for E_D = 0.4 g (R = 4)

Frame	PF_C (/yr)				
	$\gamma_E = 1.8$	$\gamma_E = 1.9$	$\gamma_E = 2.0$	$\gamma_E = 2.1$	$\gamma_E = 2.2$
1	3.75E-03	2.99E-03	2.54E-03	2.37E-03	2.11E-03
2	1.62E-03	9.84E-04	8.69E-04	7.35E-04	6.04E-04
3	3.17E-03	2.72E-03	2.28E-03	1.91E-03	1.67E-03
4	1.66E-03	1.12E-03	7.41E-04	4.47E-04	1.99E-04
5	1.96E-03	1.54E-03	1.44E-03	1.29E-03	1.00E-03
6	1.82E-03	1.47E-03	1.16E-03	9.37E-04	7.85E-04

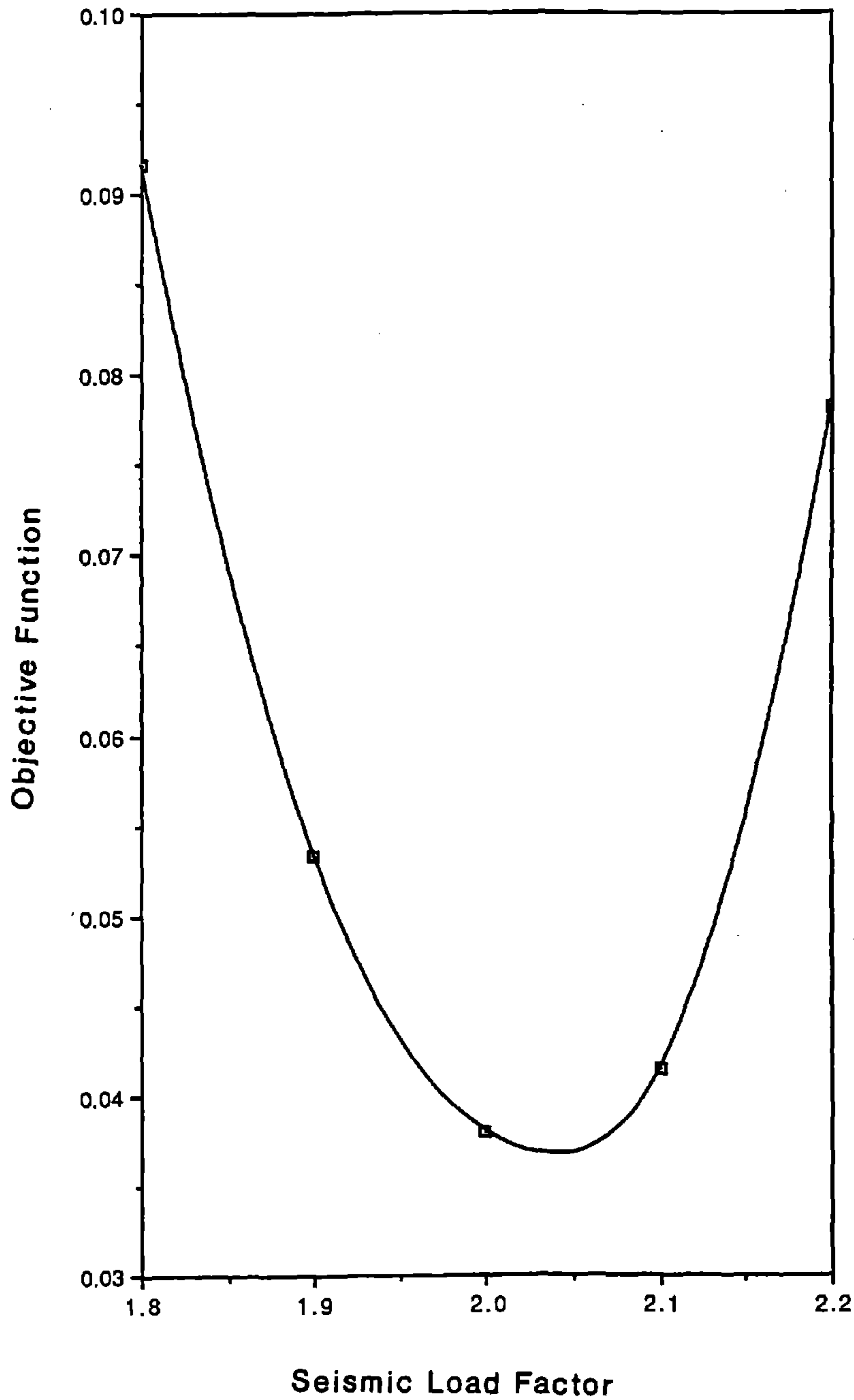


FIGURE 9-4 Determination of Seismic Load Factor for Ordinary Buildings (E_D = 0.4 g, R = 4)

conducted by Newmark and Hall (1973), Hwang and Jaw (1989), Aoyama (1981), and Hawkins (1986), the R_μ factor of 2.5 instead of 4.0 is appropriate for the IMR frame.

9.1.2 Use of Other Frame System for Ordinary Buildings

In this study, the intermediate moment-resisting frame with enough ductility capacity to form plastic hinges in structural members is used. According to the NEHRP Provisions and the Uniform Building Code, the special moment-resisting (SMR) frame is required for high seismicity area. For the purpose of comparison, the seismic load factor is determined by using the SMR frames in the high seismicity area with E_D = 0.4 g.

Six representative frames are designed as the SMR frame by using the R_μ factor of 3.5 and the design provisions for the SMR frame in the ACI code 318-89, such as the strong-column—weak-beam concept and the joint design requirement. The reliability analyses of these SMR frames are carried out in the same way without any modifications. The resulting annual collapse limit-state probabilities are summarized in table 9-VI, and the objective values with respect to γ_E = 1.7 - 2.1 are plotted in figure 9-5. The optimum value of γ_E is determined as 1.9 (figure 9-5). It appears that the design provisions for the SMR frame enforce a sway failure mechanism and do not increase the level of safety because the larger R_μ factor is used.

TABLE 9-VI Annual Collapse Limit-State Probabilities of Ordinary Buildings Designed for E_D = 0.4 g (SMR Frame)

Frame	PF_C (/yr)				
	$\gamma_E = 1.7$	$\gamma_E = 1.8$	$\gamma_E = 1.9$	$\gamma_E = 2.0$	$\gamma_E = 2.1$
1	3.08E-03	2.68E-03	2.63E-03	2.30E-03	2.05E-03
2	9.96E-04	6.38E-04	8.27E-04	9.74E-04	3.14E-04
3	2.76E-03	2.46E-03	2.15E-03	1.82E-03	1.62E-03
4	3.84E-04	3.27E-04	2.88E-04	2.19E-04	1.72E-04
5	1.14E-03	1.23E-03	1.03E-03	8.29E-04	7.74E-04
6	1.84E-03	1.39E-03	1.12E-03	8.57E-04	7.11E-04

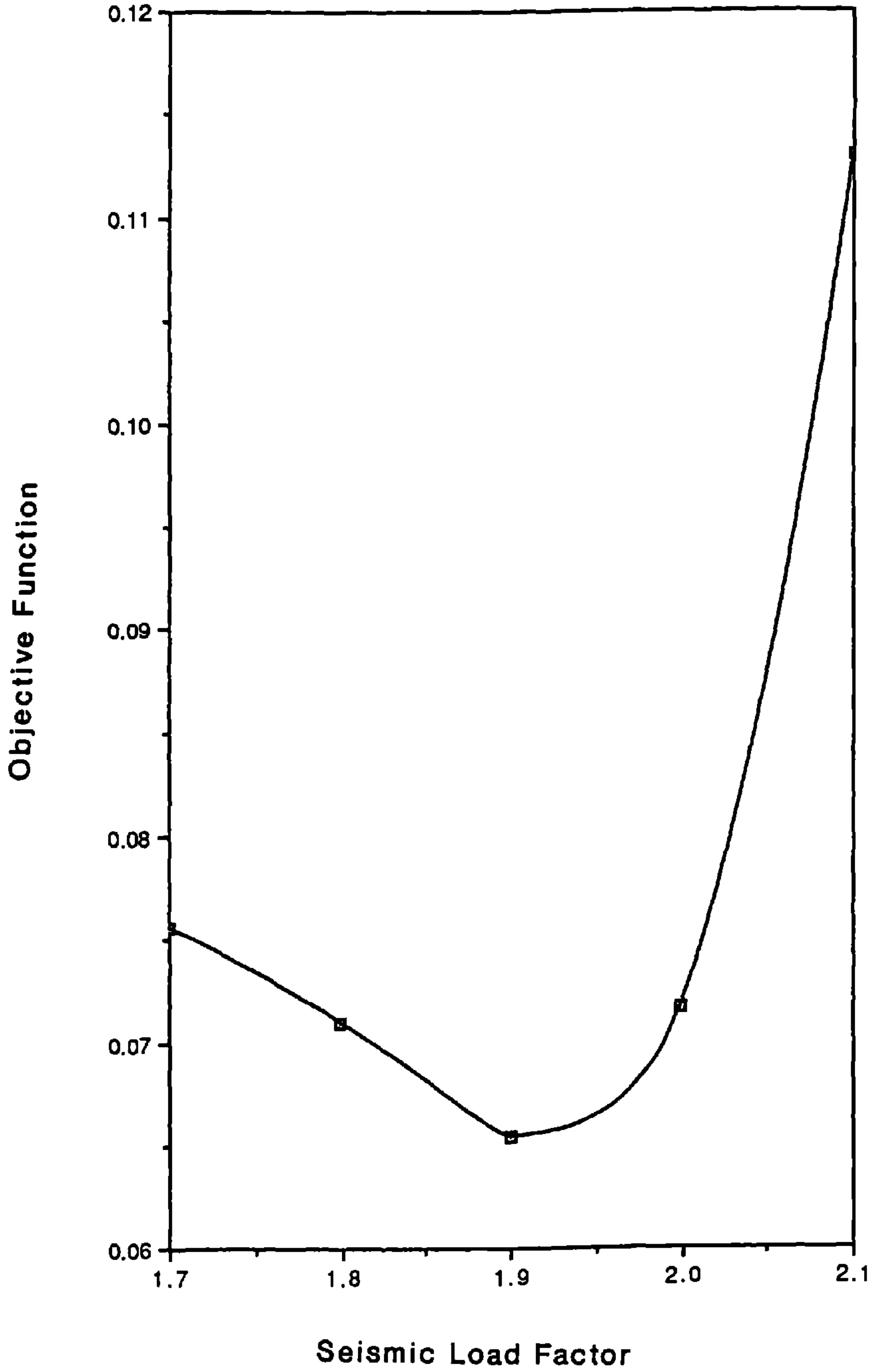


FIGURE 9-5 Determination of Seismic Load Factor for Ordinary Buildings (E_D = 0.4 g, SMR Frame)

9.2 High-risk and Essential Buildings

For high-risk and essential buildings, the acceptable collapse probabilities are 1/2000 and 1/5000 to 1/10,000 per year, respectively. It is apparent that these target probabilities are less than the one set for ordinary buildings. Since the seismic load factors determined for ordinary buildings are also used for high-risk and essential buildings, the importance factor is employed for increasing the design strength to meet the more stringent target probabilities.

For the case of $PF_{C,T}$ = 1/2000 per year, six representative frames are designed for the design earthquake E_D = 0.4 g according to the proposed design criteria with trial I values = 1.0, 1.1, 1.2, 1.3, and 1.4, respectively. Then, the annual collapse limit-state probabilities of all the sample frames are determined and shown in table 9-VII. By substituting these limit-state probabilities into equation (9.5), the values of the objective function can be determined and are plotted in figure 9-6. The optimum I value is determined as 1.2. By using the same procedure, the optimum I values are determined as 1.2 and 1.1 for E_D = 0.3 g and 0.2 g, respectively (figures 9-7 and 9-8).

Similarly, for the case of $PF_{C,T}$ = 1/5000 per year, the optimum I values are determined as 1.6, 1.5, and 1.4 for E_D = 0.4 g, 0.3 g, and 0.2 g, respectively (figures 9-9, 9-10 and 9-11). For the case of $PF_{C,T}$ = 1/10,000 per year, the optimum I values are determined as 1.8, 1.9, and 1.7 for E_D = 0.4 g, 0.3 g, and 0.2 g, respectively (figures 9-12, 9-13 and 9-14). Table 9-

TABLE 9-VII Annual Collapse Limit-State Probabilities of High-Risk Buildings Designed for E_D = 0.4 g

Frame	PF_C (/yr)				
	I = 1.0	I = 1.1	I = 1.2	I = 1.3	I = 1.4
1	2.41E-03	2.19E-03	1.85E-03	1.85E-03	1.43E-03
2	7.63E-04	5.24E-04	3.33E-04	1.89E-04	1.39E-04
3	1.95E-03	2.30E-03	1.74E-03	1.36E-03	1.41E-03
4	6.08E-04	3.61E-04	1.42E-04	1.21E-04	9.50E-05
5	1.34E-03	1.00E-03	6.65E-04	4.83E-04	3.51E-04
6	1.80E-03	9.16E-04	5.74E-04	4.58E-04	3.08E-04

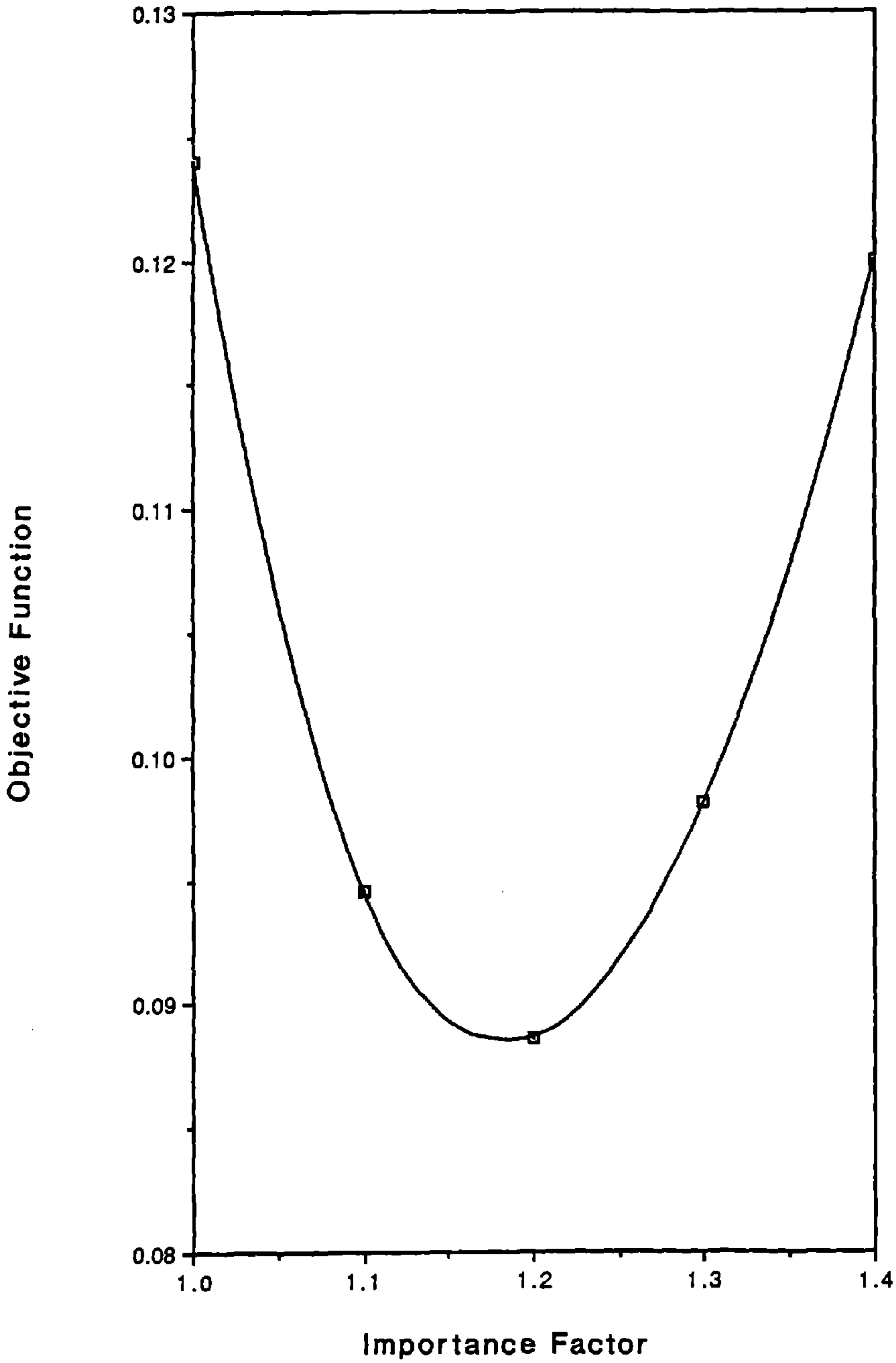


FIGURE 9-6 Determination of Importance Factor for High-risk Buildings (E_D = 0.4 g)

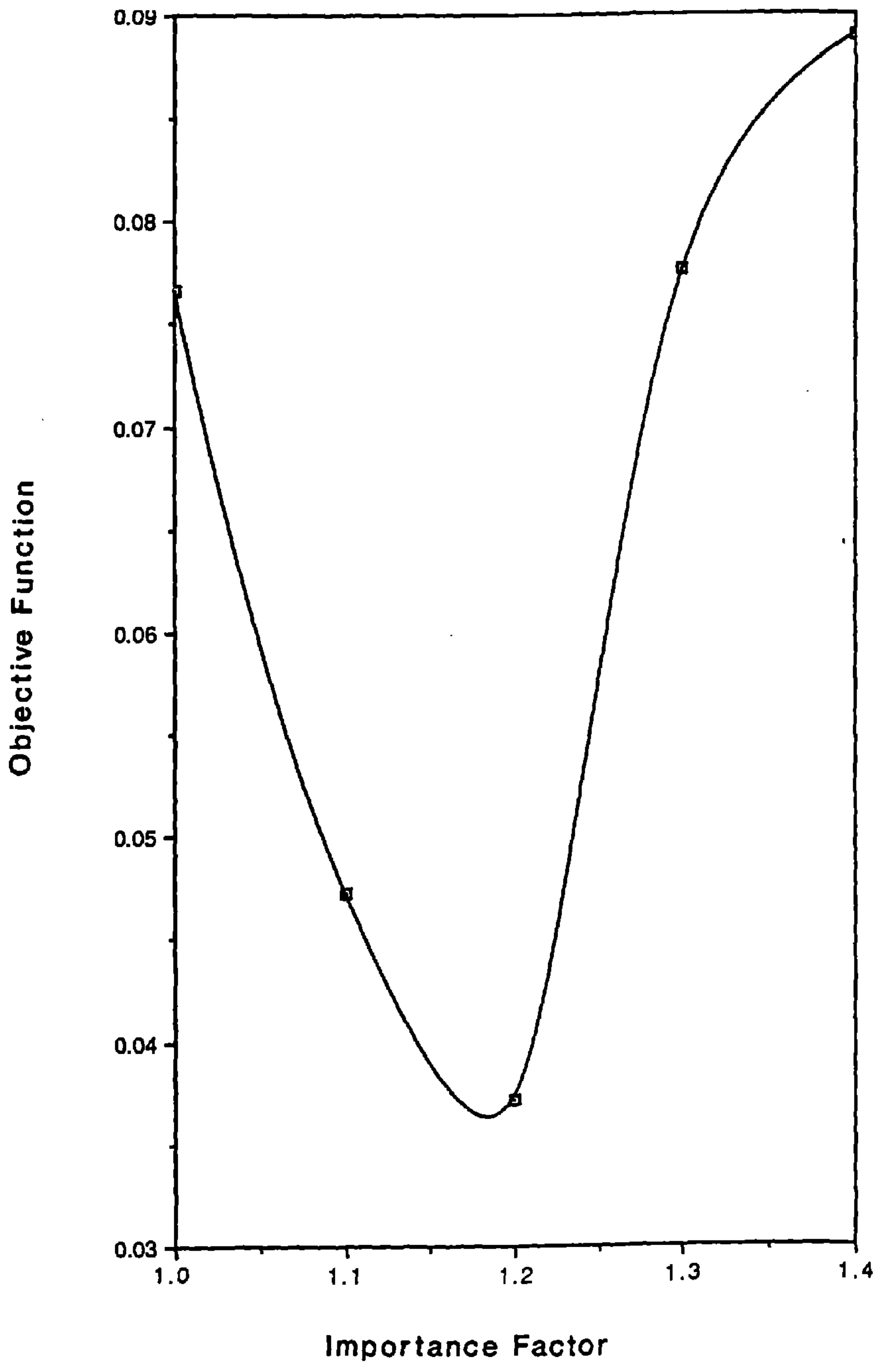


FIGURE 9-7 Determination of Importance Factor for High-risk Buildings (E_D = 0.3 g)

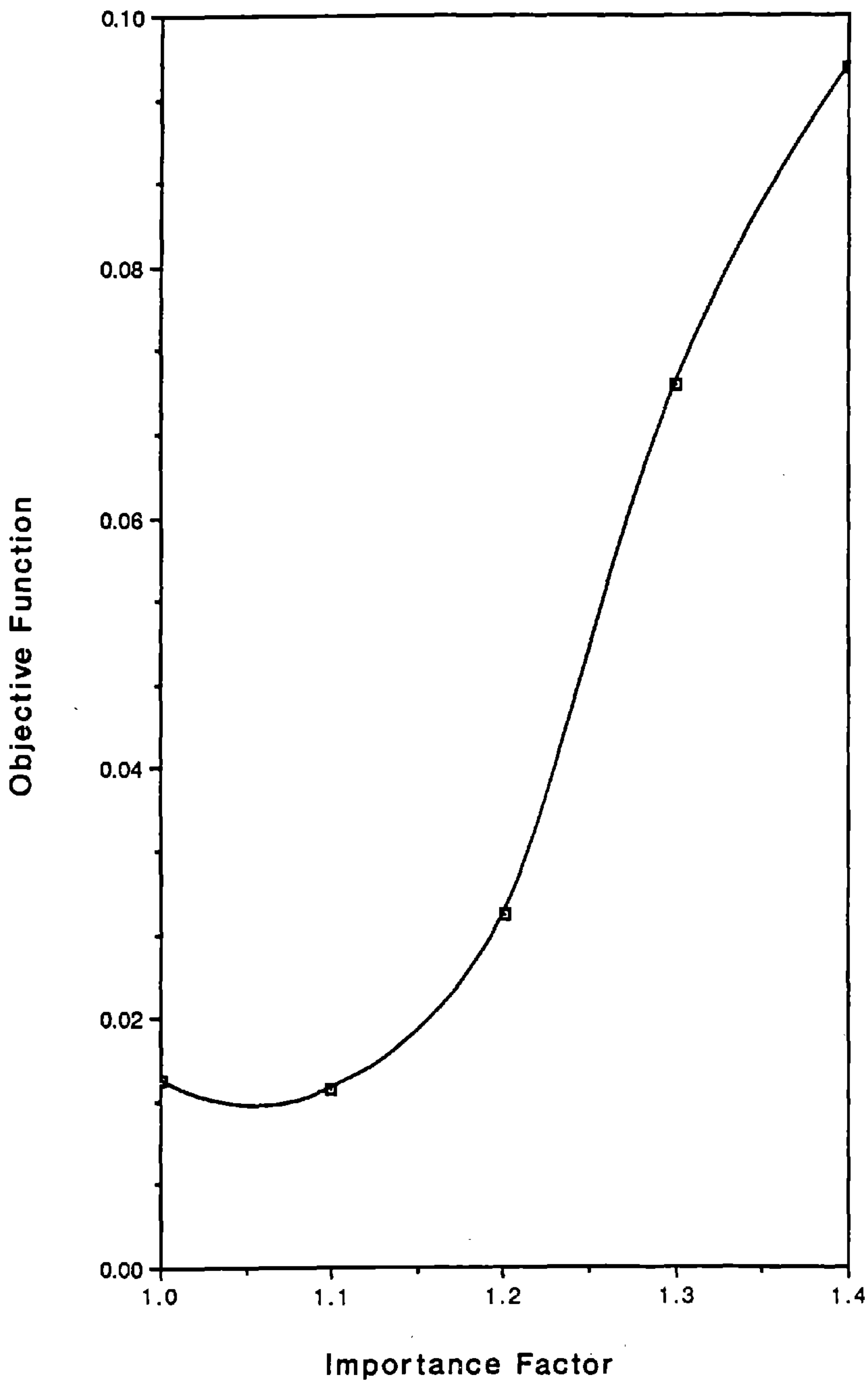


FIGURE 9-8 Determination of Importance Factor for High-risk Buildings (E_D = 0.2 g)

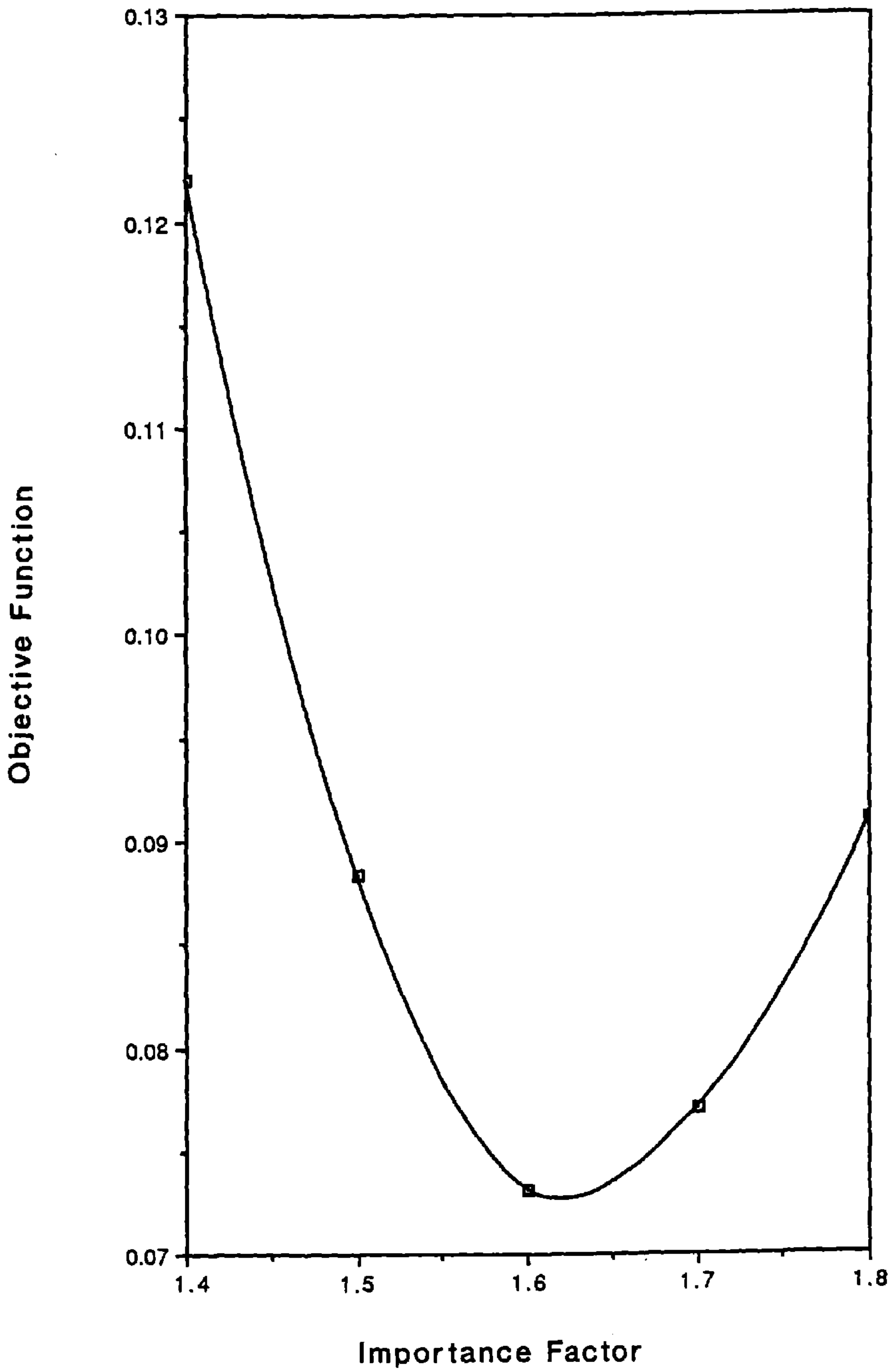


FIGURE 9-9 **Determination of Importance Factor for Essential Buildings (E_D = 0.4 g, $PF_{C,T}$ = 1/5000 per year)**

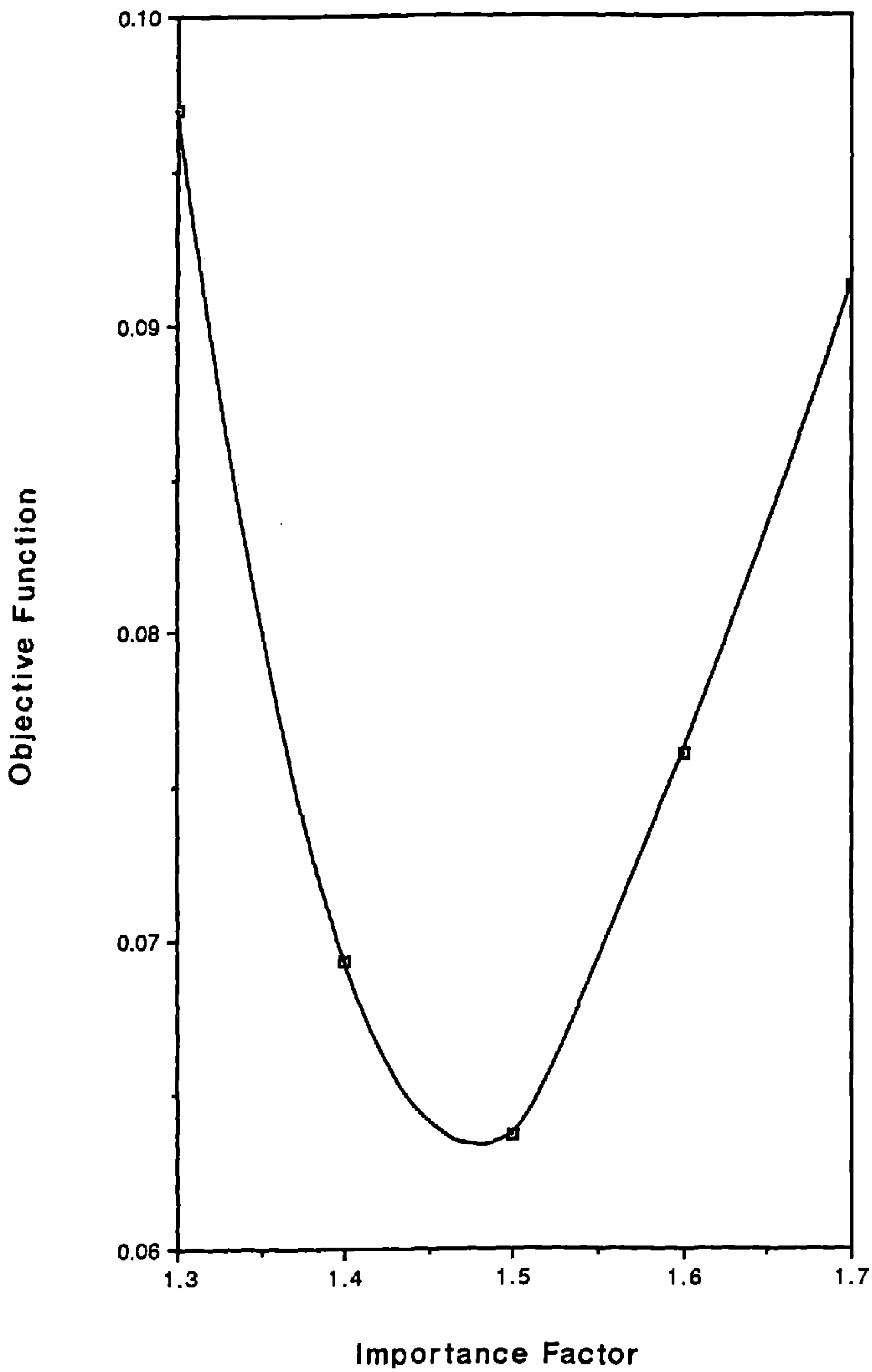


FIGURE 9-10 **Determination of Importance Factor for Essential Buildings (E_D = 0.3 g, $PF_{C,T}$ = 1/5000 per year)**

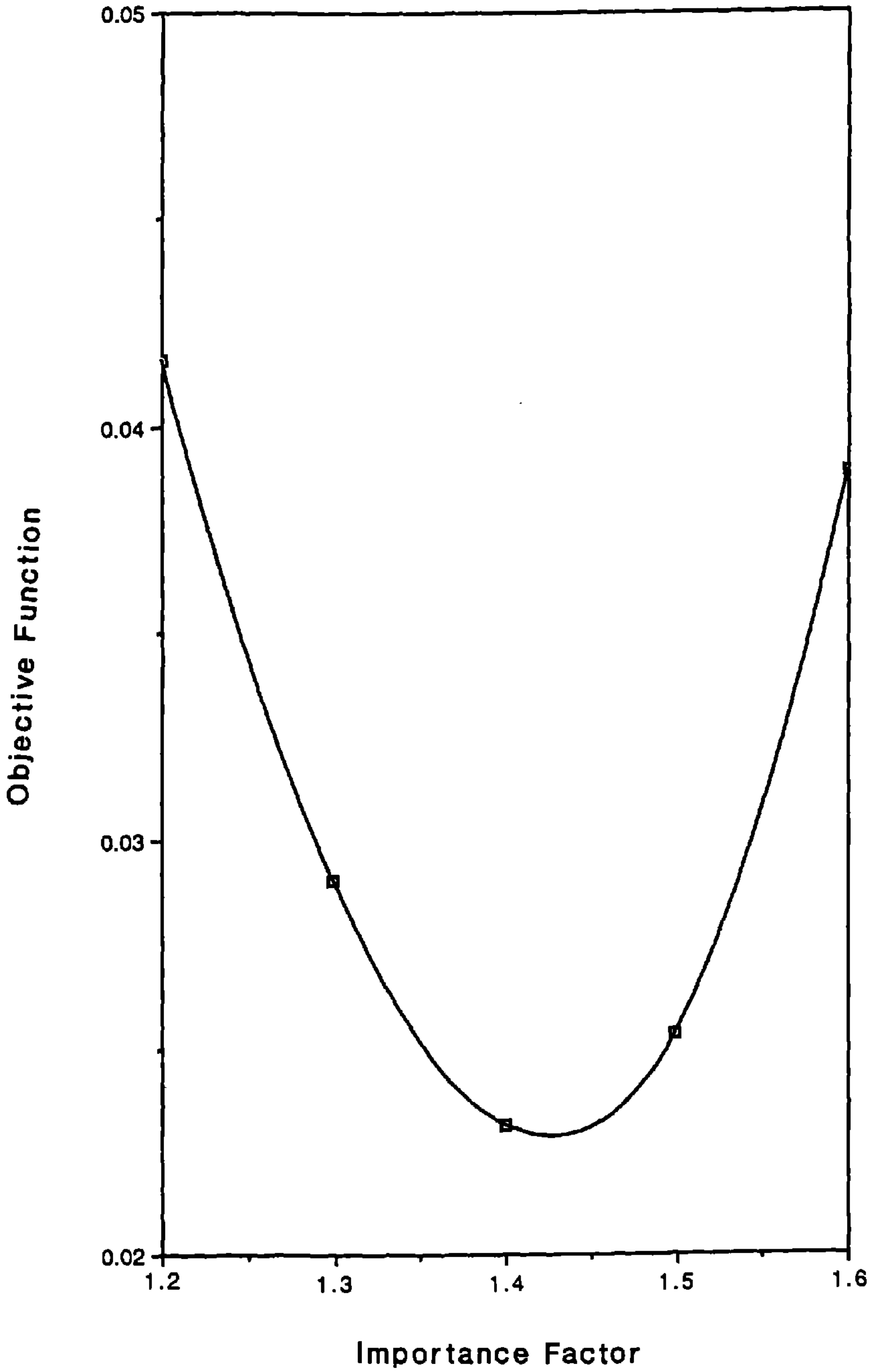


FIGURE 9-11 Determination of Importance Factor for Essential Buildings (E_D = 0.2 g, $PF_{C,T}$ = 1/5000 per year)

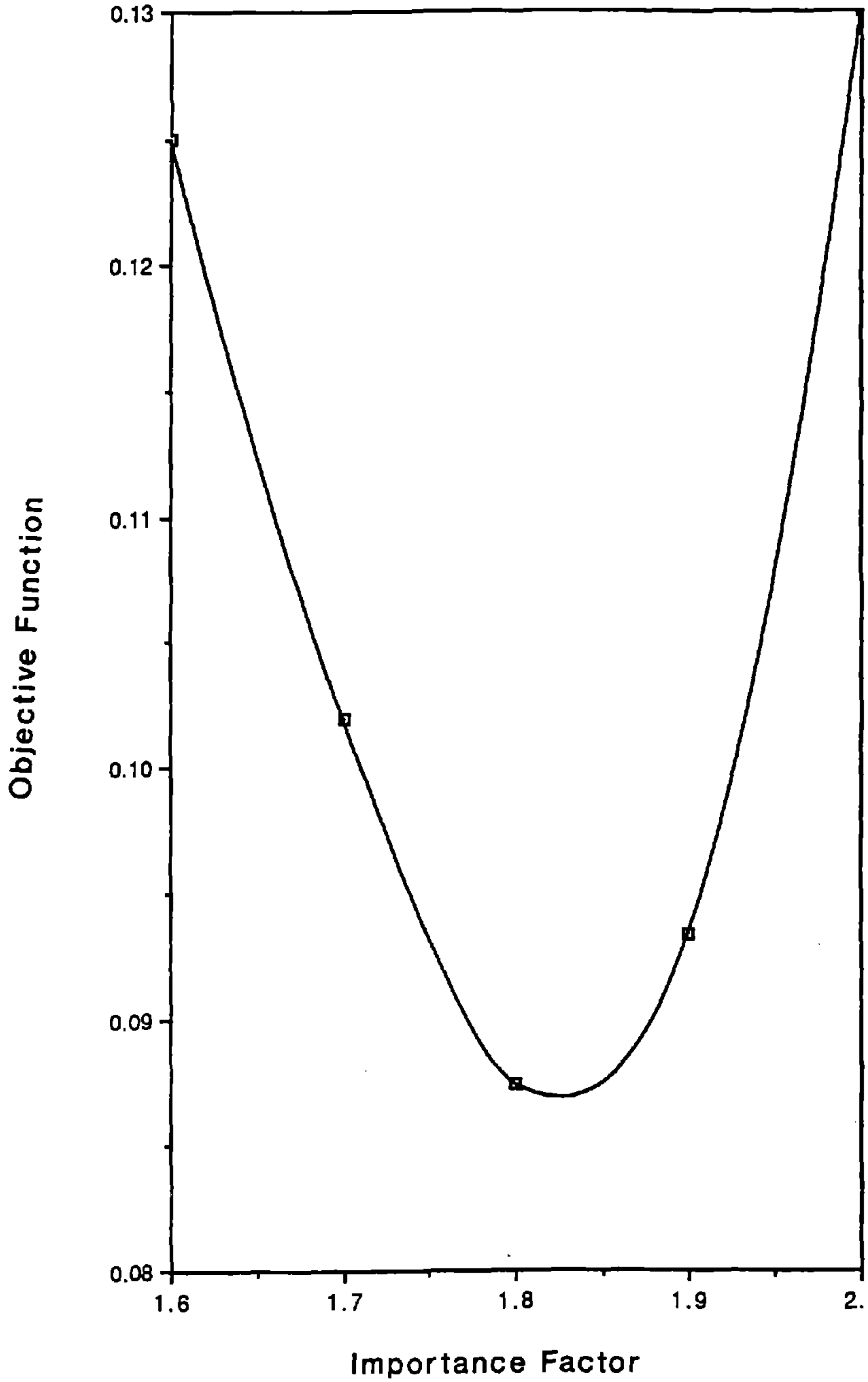


FIGURE 9-12 **Determination of Importance Factor for Essential Buildings (E_D = 0.4 g, $PF_{C,T}$ = 1/10,000 per year)**

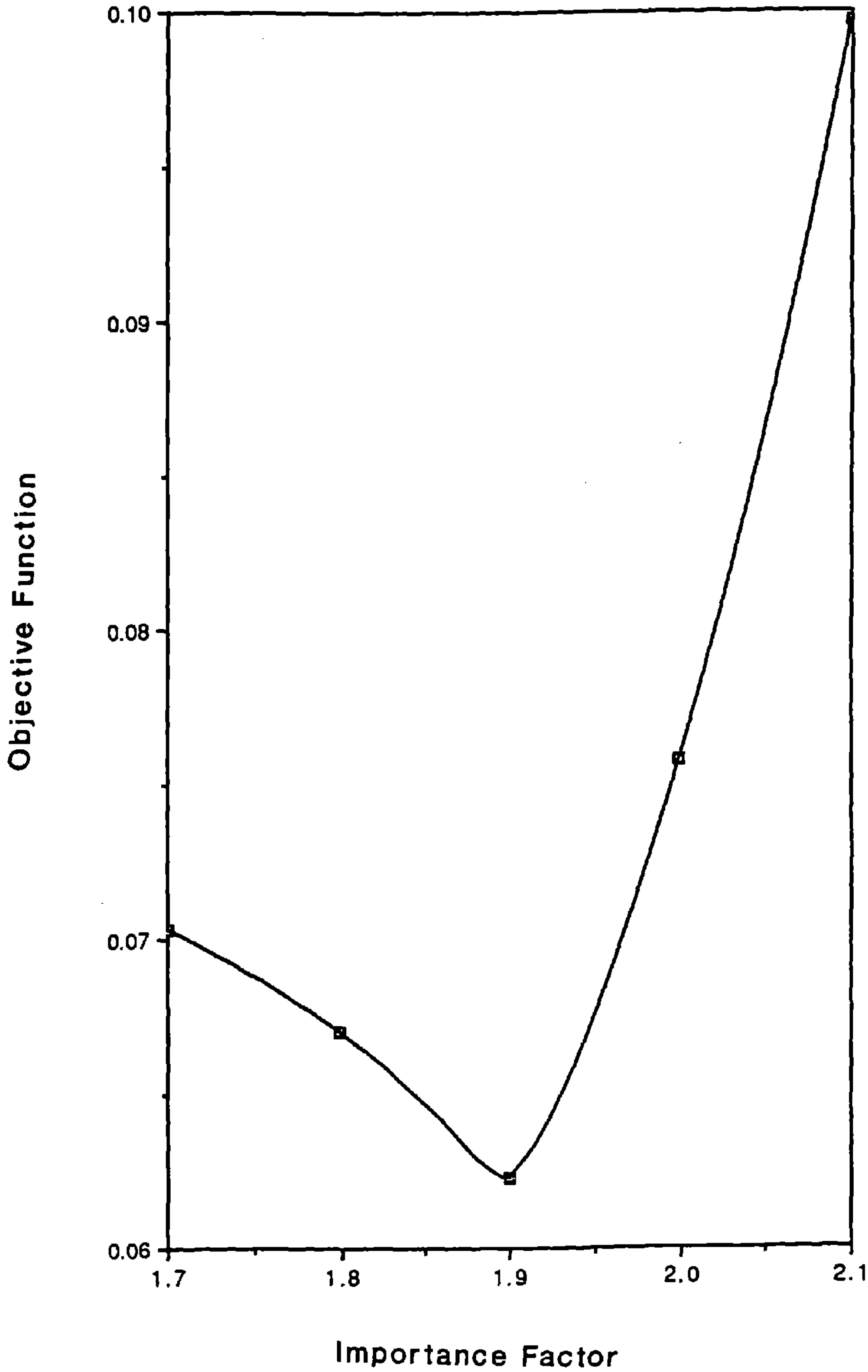


FIGURE 9-13 **Determination of Importance Factor for Essential Buildings (E_D = 0.3 g, $PF_{C,T}$ = 1/10,000 per year)**

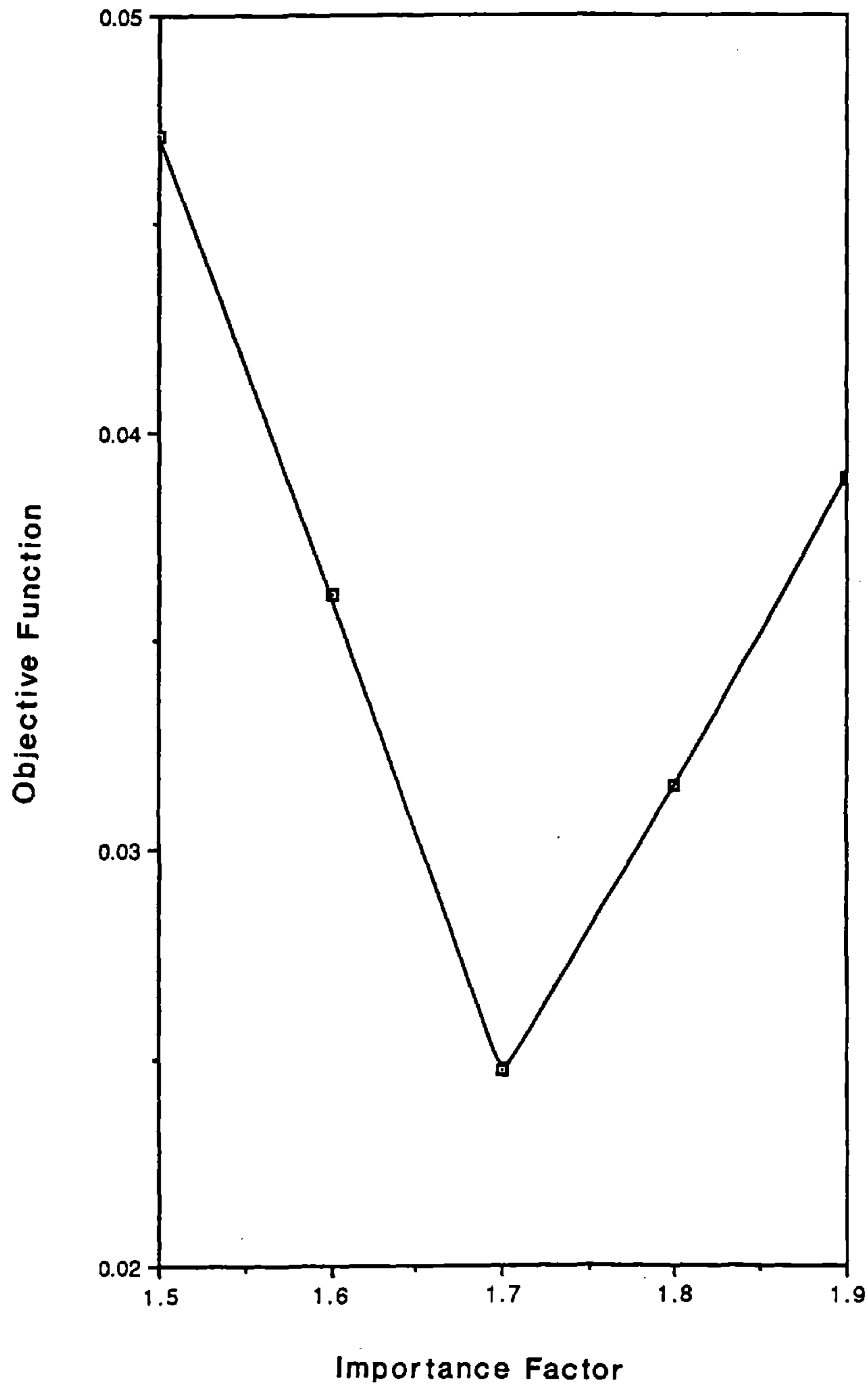


FIGURE 9-14 Determination of Importance Factor for Essential Buildings (E_D = 0.2 g, $PF_{C,T}$ = 1/10,000 per year)

VIII summarizes the results of the importance factors. The recommended value for each acceptable risk level is also shown in table 9-VIII.

For high-risk buildings in the low seismicity region, for example, $E_D \leq 0.1$ g, the annual collapse limit-state probabilities determined for ordinary buildings are still less than the target probability of 1/2000 per year. Thus, design for gravity loads is still adequate. For essential buildings in the low seismicity region, the design based on gravity loads no longer satisfies the target probability, 1/5000 to 1/10,000 per year. Thus, the seismic load combinations with $\gamma_E = 1.0$ and $I = 1.0$ are used to design the structure. The limit-state probabilities (table 9-IX) indicate that $I = 1.0$ is sufficient to provide the required strength to meet the target probability.

9.3 Summary of the Proposed Design 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 proposed reliability-based design criteria for reinforced concrete frame buildings are summarized as follows:

The design base shear V is determined as

$$V = I \left(\frac{ZC}{R_\mu}\right) W \qquad (9.8)$$

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad (9.9)$$

TABLE 9-VIII Summary of Importance Factor

Building Category	$PF_{C,T}$ (/yr)	Importance Factor			
		$E_D = 0.4$ g	$E_D = 0.3$ g	$E_D = 0.2$ g	Recommended Value
High-risk	1/2000	1.2	1.2	1.1	1.2
Essential	1/5000	1.6	1.5	1.4	1.5
Essential	1/10,000	1.8	1.9	1.7	1.8

TABLE 9-IX Annual Collapse Limit-State Probabilities of Essential Buildings Designed for E_D = 0.1 g

Frame	PF_C (/yr)
1	1.60E-04
2	6.25E-05
3	1.96E-04
4	1.37E-04
5	5.43E-05
6	9.60E-05

where I is the importance factor and the recommended value is given in table 9-X for each category of buildings and seismic zone factor. Z is the seismic zone factor, wcich is equivalent to the peak ground acceleration in g with soft rock as a reference site. S is site coefficient and the S values for various site conditions are given in table 9-XI. R_μ is the elastic-to-inelastic response factor. For the intermediate moment-resisting frame constructed according to ACI code 318-89, the R_μ factor is set as 2.5. W is the total seismic dead load and T is the structural period determined by using the formula as specified in the NEHRP Provisions. The base shear is distributed over the height of the building by using the same formula specified in the 1988 NEHRP Provisions.

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

$$\phi R \geq 1.2 D + 0.5 L \pm \gamma_E E \qquad (9.10)$$

$$\phi R \geq 0.9 D \pm \gamma_E E \qquad (9.11)$$

where $\gamma_E = (0.7 + 1.5 Z) \geq 1.0$. The values of γ_E are tabulated in table 9-XII for various Z values. ϕ is the resistance factor which applies to a particular structural action such as flexure, shear, axial compression, etc. The ϕ values specified in the ACI code 318-89 are used in this study.

For the seismic zones of $Z \leq 0.1$, the above-mentioned load combinations are required only for essential buildings. Ordinary and high-risk buildings

TABLE 9-X 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
Essential	1/10,000	1.0	1.8

TABLE 9-XI Site Conditions and Site Coefficients

Type	Description	S-Factor
S_1	A soil profile with either: (a) soft rock or hardpan or similar material characterized by shear wave velocities greater than 2500 ft/sec, or (b) medium compact to compact sands and gravels or hard clays, where the soil depth is less than 100 ft.	1.0
S_2	A soil profile with medium compact to compact sands and gravels or hard clays, where the soil depth exceeds 100 ft.	1.2
S_3	A total depth of overburden of 75 ft or more and containing: more than 20 ft of soft to medium clays or loose sands and silts, but not more than 40 ft of soft clay or loose sands and silts.	1.5

TABLE 9-XII Recommended Seismic Load Factors

Z	γ_E
0.1	NA
0.2	1.00
0.3	1.15
0.4	1.30

in this seismic zone need to be designed by using the following load combinations including only gravity load effects.

$$\phi R \geq 1.4\, D \tag{9.12}$$

$$\phi R \geq 1.2\, D + 1.6\, L \tag{9.13}$$

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

SECTION 10

COMPARISONS OF PROPOSED CRITERIA WITH OTHER STANDARDS

The proposed reliability-based LRFD criteria are compared with the NEHRP Provisions (1988) and the Tri-Services Guidelines (1986). For ordinary and high-risk buildings, a five-story RC frame building is designed according to the proposed criteria and the NEHRP Provisions, separately. Then, the seismic performance of these two frames is evaluated and compared. For essential buildings, the Tri-Services Guidelines is added as the third design standard. Once again, the seismic performance of these three frames is evaluated and compared. Furthermore, uniform reliability among structures designed from the proposed criteria for various design conditions is also demonstrated by comparing the results with those obtained from structures designed according to the NEHRP Provisions.

10.1 Ordinary Buildings

10.1.1 Design Using the NEHRP Provisions

The five-story frame building used for the comparative study is the same as frame 1 in table 4-II. The building is assumed to be located in a moderate seismic zone with A_V and A_a equal to 0.2. According to the NEHRP Provisions, the seismic hazard exposure group for an ordinary building is group I. From these two conditions, the seismic performance category D is assigned to this building. Thus, the special moment-resisting frame is required to provide seismic resistance. The design base shear V is calculated from the following formula:

$$V = C_S W = \frac{1.2 A_v S}{R T^{2/3}} W \tag{10.1}$$

where C_S is the seismic base shear coefficient and has 2.5 A_a/R as the upper bound. The site condition is classified as S_3; thus, the S factor is equal to 1.5. For the SMR frame, the response modification factor R is specified as 8 according to the 1988 NEHRP Provisions. The fundamental period of the building is determined as 0.67 sec and the total dead load W of the building is 1920 kips. From equation (10.1), the design base shear V is calculated as 112.7 kips. For dead, live, and earthquake loads, the load combinations specified in the NEHRP provisions are

$$(1.1 + 0.5 A_v) Q_D + 1.0 Q_L \pm 1.0 Q_E \tag{10.2}$$

$$(0.9 - 0.5 A_v) Q_D \pm 1.0 Q_E \tag{10.3}$$

where

Q_D = dead load effect

Q_L = live load effect

Q_E = seismic load effect

For A_v = 0.2, the above load combinations can be rewritten as

$$1.2 Q_D + 1.0 Q_L \pm 1.0 Q_E \tag{10.4}$$

$$0.8 Q_D \pm 1.0 Q_E \tag{10.5}$$

The structural members are designed according to the provisions for the SMR frame in the ACI code 318-89. The required sections and flexural reinforcements of beams and columns are listed in tables 10-I and 10-II, respectively. The structural members designed according the ACI code 318-89 are expected to fail in the flexural mode; thus, the shear reinforcements of beams and columns are not actually determined and are assumed to satisfy the requirement of the ACI code.

10.1.2 Design Using the Proposed Criteria

The proposed criteria are summarized in Section 9.3. The building is located in the moderate seismic zone with $A_a = 0.2$; thus, the Z factor is also equal to 0.2. The intermediate moment-resisting frame is used to provide seismic resistance. For the IMR frame, the R_μ factor is set as 2.5. The design base shear V is calculated as

$$V = I\left(\frac{ZC}{R_\mu}\right)W \qquad (10.6)$$

where

$$C = 1.25\ S/T^{2/3} \leq 2.75 \qquad (10.7)$$

The site coefficient S and the fundamental period T of the building are 1.5 and 0.67 sec, respectively; thus, the spectral acceleration coefficient C is determined as 2.45. Since the building is an ordinary building, the importance factor I is equal to 1.0. The total dead load W of the building is determined as 2000 kips. From equation (10.6), the design base shear V is calculated as 391.4 kips. The member forces are calculated from the base

TABLE 10-I Beam Sizes and Required Steels of the Ordinary Building (NEHRP Provisions)

FL	Exterior Beams: Section (in)	A_s and A_s' (in^2) Left	Mid.	Right	Interior Beams: Section (in)	A_s and A_s' (in^2) Left	Mid.	Right
		6.6	1.6	7.3		7.3	1.5	6.9
1	16 x 26	3.0	2.4	3.3	16 x 26	3.3	2.1	3.1
		6.8	1.6	7.2		7.2	1.5	6.9
2	16 x 26	3.1	2.3	3.2	16 x 26	3.2	2.1	3.1
		6.9	1.5	7.2		7.2	1.5	7.1
3	14 x 24	3.1	2.5	3.2	14 x 24	3.2	2.3	3.2
		6.5	1.4	6.7		6.7	1.4	6.6
4	14 x 24	2.9	2.5	3.0	14 x 24	3.0	2.4	2.9
		4.3	1.2	5.6		5.6	1.2	5.3
5	14 x 24	2.0	2.4	2.5	14 x 24	2.5	2.1	2.4

TABLE 10-II Column Sizes and Required Steels of the Ordinary Building (NEHRP Provisions)

Story	Exterior Columns		Interior Columns	
	Section (in)	A_s (in^2)	Section (in)	A_s (in^2)
1	24 x 24	8.8	24 x 24	14.0
2	24 x 24	8.8	24 x 24	14.0
3	22 x 22	8.8	22 x 22	14.0
4	22 x 22	8.6	22 x 22	13.3
5	22 x 22	8.6	22 x 22	12.5

shear and combined with those obtained from the gravity loads by the following proposed load combinations.

$$1.2D + 0.5L \pm 1.0E \tag{10.8}$$

$$0.9D \pm 1.0E \tag{10.9}$$

The structural members are designed according to the provisions for the IMR frame in the ACI code 318-89. The required sections and flexural reinforcements of beams and columns are listed in tables 10-III and 10-IV, respectively.

10.1.3 Result Comparisons

The design base shear required by the proposed criteria is about three times larger than the one required by the NEHRP Provisions (391.4 kips vs. 112.7 kips). However, the strong-column—weak-beam requirement for the SMR frame specified in the NEHRP Provisions usually will increase the column size of the SMR frame determined by the design base shear. In other words, the column size of the SMR frame is not completely determined by the design base shear. Thus, the required member size and flexural reinforcement determined by the proposed criteria are larger than those determined by the NEHRP Provisions; however, the required member size is not proportional to the design base shear.

To further compare the behavior of both structures in the event of an earthquake, the capacity curves of these two structures are determined as shown in figure 10-1. For the first-yielding capacity, the structure

TABLE 10-III Beam Sizes and Required Steels of the Ordinary Building (Proposed Criteria)

FL	Exterior Beams				Interior Beams			
	Section (in)	A_s and A_s' (in^2)			Section (in)	A_s and A_s' (in^2)		
		Left	Mid.	Right		Left	Mid.	Right
		10.9	1.9	11.0		11.0	1.7	10.4
1	18 x 30	4.5	2.1	3.5	18 x 30	3.5	1.7	3.1
		10.8	1.8	10.7		10.7	1.7	10.2
2	18 x 30	4.1	2.0	3.4	18 x 30	3.4	1.7	3.1
		9.8	1.6	9.7		9.7	1.6	9.4
3	18 x 28	2.8	2.1	2.8	18 x 28	2.8	1.8	2.7
		7.7	1.5	7.6		7.6	1.5	7.5
4	18 x 28	2.3	2.0	2.3	18 x 28	2.3	1.8	2.2
		4.5	1.5	5.5		5.5	1.5	5.2
5	18 x 28	1.5	2.1	1.7	18 x 28	1.7	1.7	1.6

TABLE 10-IV Column Sizes and Required Steels of the Ordinary Building (Proposed Criteria)

Story	Exterior Columns		Interior Columns	
	Section (in)	A_s (in^2)	Section (in)	A_s (in^2)
1	26 x 26	20.5	26 x 26	21.1
2	26 x 26	10.2	26 x 26	11.9
3	24 x 24	10.2	24 x 24	11.8
4	24 x 24	10.2	24 x 24	10.1
5	24 x 24	10.1	24 x 24	6.3

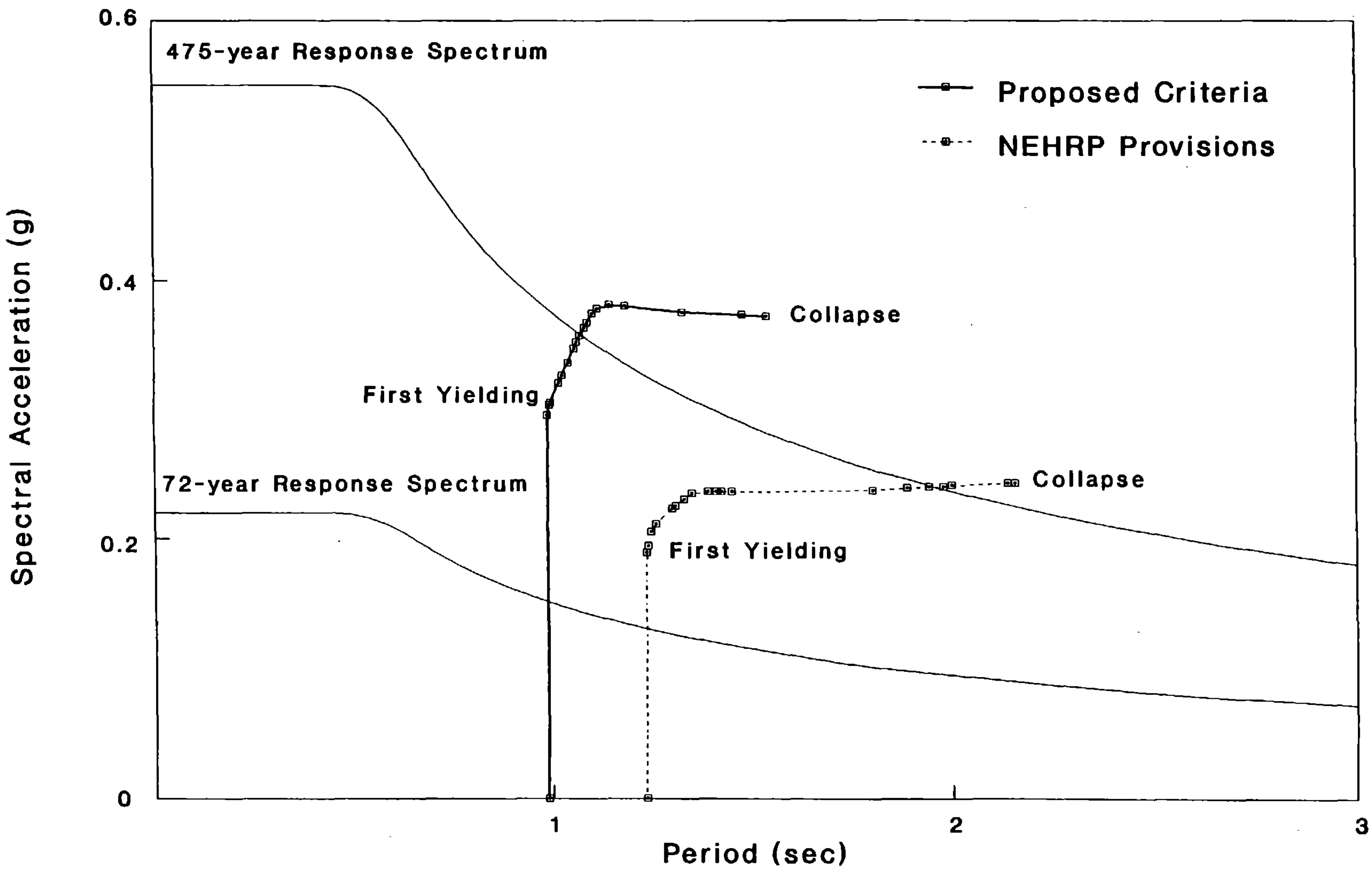


FIGURE 10-1 Spectral Capacity Curves and Response Spectra

designed by the proposed criteria has a larger value. This is apparently caused by the larger design base shear. The first-yielding capacities of the two structures are not at the same periods. Consequently, the demand forces are different at these two periods. To evaluate the seismic performance at the stage of first-yielding, the response spectrum of a moderate earthquake (for example, an earthquake with a return period of 72 years) is also plotted together with the capacity curves (figure 10-1). The first-yielding capacities of both structures are larger than the corresponding demand response; thus, if a 72-year earthquake occurs, both structures will perform satisfactorily with respect to the first yielding as the limit state.

To evaluate the structural performance at the collapse stage, the response spectrum of a 475-year (design) earthquake as plotted in figure 10-1 is used to represent a large earthquake. Even though the collapse capacities of both structures are larger than the corresponding demand response, the collapse capacity of the structure designed by the proposed criteria is much larger than the demand force; thus, the safe margins of this structure are larger. Furthermore, the NEHRP Provisions heavily rely on ductility to provide the structural resistance. This behavior can be observed from the longer capacity curve between the first-yielding and collapse limit states. On the other hand, the proposed criteria show the balance between strength and ductility for seismic resistance.

The above comparison is based on an earthquake with a specific return period. To take all the possible earthquakes into consideration, the reliability analysis method developed in this study is used. The annual limit-state probabilities for both first-yielding and collapse limit states are shown in table 10-V. The corresponding yielding and collapse indexes are also shown in the table. For the first yielding as the limit state, the annual limit-state probabilities of both structures are less than the target probability. For the collapse limit state, the annual limit-state probability of the structure designed by the proposed criteria is less than the target probability (collapse index is 0.74), while the probability of the structure designed by the NEHRP Provisions is slightly larger than the target probability (collapse index is 1.32).

10.2 High-risk Buildings

For high-risk buildings, the proposed criteria use the importance factor of 1.2 to increase the design base shear. Thus, the design base shear of the five-story frame as a high-risk building is increased as 470 kips. Following the same design procedure for the ordinary building, the required sections and flexural reinforcements of beams and columns are listed in tables 10-VI and 10-VII, respectively. On the other hand, according to the NEHRP Provisions, the seismic performance category D is also assigned to the high-risk building; thus, the required sections and flexural reinforcements of structural members for the high-risk building are the same as those for the ordinary building (tables 10-I and 10-II).

The annual limit-state probabilities of both structures are listed in table 10-VIII. For the collapse of structure as the limit state, the collapse index for the structure designed by the proposed criteria is 1.06, which is only slightly larger than 1.0; thus, the structure will perform well in the event

TABLE 10-V Comparison Between Ordinary Buildings

Standard	PF_Y (YI) (/yr)	PF_C (CI) (/yr)
Proposed Criteria	1.54E-03 (0.08)	7.39E-04 (0.74)
NEHRP Provisions	4.48E-03 (0.22)	1.32E-03 (1.32)

TABLE 10-VI Beam Sizes and Required Steels of the High-risk Building (Proposed Criteria)

FL	Exterior Beams				Interior Beams			
	Section (in)	A_s and A_s' (in^2)			Section (in)	A_s and A_s' (in^2)		
		Left	Mid.	Right		Left	Mid.	Right
1	18 x 30	11.9 5.4	2.0 2.1	11.9 4.5	18 x 30	11.9 4.5	1.9 1.9	11.3 4.1
2	18 x 30	12.0 5.4	2.0 2.1	11.8 4.5	18 x 30	11.8 4.5	1.9 1.9	11.4 4.1
3	18 x 28	10.9 3.8	1.8 2.2	10.7 3.1	18 x 28	10.7 3.1	1.8 1.8	10.4 3.0
4	18 x 28	8.7 2.5	1.5 2.0	8.5 2.5	18 x 28	8.5 2.5	1.5 1.8	8.3 2.4
5	18 x 28	4.9 1.5	1.5 2.1	5.9 1.8	18 x 28	5.9 1.8	1.5 1.7	5.5 1.7

TABLE 10-VII Column Sizes and Required Steels of the High-risk Building (Proposed Criteria)

Story	Exterior Columns		Interior Columns	
	Section (in)	A_s (in^2)	Section (in)	A_s (in^2)
1	28 x 28	23.6	28 x 28	23.4
2	26 x 26	12.0	26 x 26	15.5
3	24 x 24	12.0	24 x 24	15.5
4	24 x 24	11.9	24 x 24	13.0
5	24 x 24	11.1	24 x 24	8.1

TABLE 10-VIII Comparison Between High-risk Buildings

Standard	PF_Y (YI) (/yr)	PF_C (CI) (/yr)
Proposed Criteria	7.53E-04 (0.08)	5.29E-04 (1.06)
NEHRP Provisions	4.48E-03 (0.45)	1.32E-03 (2.64)

of a large earthquake. On the other hand, the collapse index for the structure designed according to the NEHRP Provisions is 2.64, which is much larger than the acceptable value of 1.0. Therefore, the performance of the high-risk building designed according to the NEHRP Provisions is not satisfactory in the event of a large earthquake. It is apparent that the proposed criteria are better than the NEHRP Provisions. One significant contributor to this observation is the use of the importance factor in the proposed criteria.

10.3 Essential Buildings

The acceptable risk level for the essential building is set as 1/5000 per year to 1/10,000 per year in Section 2. For this comparison, the acceptable risk level of 1/5000 per year is used and the importance factor of 1.5 is used to increase the design base shear. Thus, the design base shear for the five-story frame structures is increased as 589.2 kips. Following the design procedure, the required sections and flexural reinforcements are determined and shown in tables 10-IX and 10-X, respectively. According to the NEHRP Provisions, the seismic performance category E is assigned to the five-story building used as the essential building. However, the RC frame structure assigned to category D or E has the same design criteria. Therefore, the required sections and reinforcements are the same as those listed in tables 10-I and 10-II.

The Tri-Services Guidelines (1986) is the standard for the design of essential buildings for military facilities. Thus, it is added as the third

TABLE 10-IX Beam Sizes and Required Steels of the Essential Building (Proposed Criteria)

FL	Exterior Beams: Section (in)	Exterior Beams: A_s and A_s' (in^2) Left	Mid.	Right	Interior Beams: Section (in)	Interior Beams: A_s and A_s' (in^2) Left	Mid.	Right
		13.3	2.3	13.2		13.2	2.2	12.6
1	18 x 30	6.9	2.3	5.9	18 x 30	5.9	2.2	5.5
		13.8	2.4	13.5		13.5	2.2	12.9
2	18 x 30	7.6	2.4	6.4	18 x 30	6.4	2.2	5.8
		12.4	2.2	12.2		12.2	2.0	11.8
3	18 x 28	5.6	2.2	4.7	18 x 28	4.7	2.0	4.2
		10.1	1.7	9.9		9.9	1.6	9.7
4	18 x 28	2.9	2.1	2.8	18 x 28	2.8	1.8	2.8
		5.5	1.5	6.5		6.5	1.5	6.1
5	18 x 28	1.7	2.2	2.0	18 x 28	2.0	1.7	1.9

TABLE 10-X Column Sizes and Required Steels of the Essential Building (Proposed Criteria)

Story	Exterior Columns		Interior Columns	
	Section (in)	A_s (in^2)	Section (in)	A_s (in^2)
1	30 x 30	29.2	30 x 30	30.2
2	26 x 26	15.0	26 x 26	21.4
3	24 x 24	15.0	24 x 24	21.0
4	24 x 24	14.6	24 x 24	17.6
5	24 x 24	12.5	24 x 24	10.6

design standard for the comparison. Using the Tri-Services Guidelines, the design of the five-story frame is summarized below.

10.3.1 Design Using the Tri-Services Guidelines

According to the Tri-Services Guidelines, an essential building shall be designed to resist the following two levels of earthquakes: EQ-I and EQ-II. EQ-I is the maximum probable earthquake, likely to occur during the lifetime of a building, and is defined as an earthquake with a 50% probability of being exceeded in 50 years (a 72-year earthquake). On the other hand, EQ-II is the maximum theoretical earthquake that can occur at a site, but has a low probability of occurrence during the lifetime of a building. EQ-II is defined as an earthquake with a 10% probability of being exceeded in 100 years (a 950-year earthquake).

The design response spectrum is defined as

$$S_a = \frac{1.22\ A_v\ S_i\ D_f}{T} \tag{10.10}$$

and S_a has 2.5 A_a as the upper bound. For the five-story frame building located in the seismic zone with A_v and A_a equal to 0.2 corresponding to the return period of 475 years, A_v and A_a of EQ-I are equal to 0.08. For the S_3 soil condition, the soil factor S_i is equal to 1.5. According to the Tri-Services Guidelines, the damping adjustment factor D_f is 1.00 for the reinforced concrete structure with a damping ratio of 5%. Substituting A_v,

S_i, D_f, and A_a into equation (10.10), the design response spectrum of EQ-I can be determined.

The Tri-Services Guidelines require an essential building to be analyzed dynamically even though the building is a regular building. In this study, the five-story building is idealized as a multi-degree-of-freedom stick model with a fixed base. From the free vibration analysis, the nature periods, mode shapes, and modal participation factors can be obtained. The story lateral forces F_{jm} for mode m at level j can be computed as

$$F_{jm} = PF_{jm} S_{am} w_j \tag{10.11}$$

where S_{am} is the spectral acceleration to be computed from the EQ-I response spectrum at the natural period of mode m; PF_{jm} is the modal participation factor for mode m at level j and w_j is the weight assigned at the level j. From these story lateral forces, the member forces caused by EQ-I can be determined and combined with those from gravity loads by using the following equations.

$$1.2\, Q_D + 1.0\, Q_L \pm 1.0\, Q_E \tag{10.12}$$

$$0.8\, Q_D \pm 1.0\, Q_E \tag{10.13}$$

The combined member forces are used to design structural members in accordance with the design procedure specified in the ACI code 318-89.

In level-two design, the structure is analyzed to determine its ability to resist the forces and deformations caused by EQ-II. For A_v and A_a equal to

0.2 corresponding to the return period of 475 years, A_v and A_a of EQ-II are equal to 0.25. For a reinforced concrete structure, a 10% damping ratio is used for post-elastic analyses; thus, the damping adjustment factor D_f is equal to 0.80. From equation (10.10), the design response spectrum of EQ-II is established. The elastic member forces are calculated by means of modal analysis using the EQ-II response spectrum. The member forces from gravity loads and EQ-II are combined by using the following load combinations.

$$1.0\ Q_D + 0.25\ Q_L \pm 1.0\ Q_E \tag{10.14}$$

$$1.0\ Q_D \pm 1.0\ Q_E \tag{10.15}$$

The combined member forces are allowed to be larger than the design ultimate capacity. However, the inelastic demand ratio (IDR) is implemented to control the overstress within an acceptable limit for each member. Because the shear capacity is usually greater than the flexural capacity for a properly designed moment-resisting frame, the flexural capacities of beams and columns are used to represent the capacities of structural members. Hence, IDR is defined as the ratio of the elastic demand moment M_D to the design ultimate moment capacity M_C.

$$IDR = M_D/M_C \tag{10.16}$$

For essential buildings, the Tri-Services Guidelines specify that the IDR of the beams shall not be greater than 2, and the IDR of the columns shall not be greater than 1.25. The smaller IDR for the columns is to ensure the strong-column—weak-beam behavior, that is, to ensure that plastic hinges will form on the beams first rather than on the columns.

By using the cross section and reinforcement determined by EQ-I, IDRs of several columns and beams exceed the allowable limit. Thus, the structure must be redesigned by increasing the capacity of these critical members. By a trial-and-error process, IDRs of all the members are within the allowable limits set forth in the Tri-Services Guidelines (figure 10-2). The required sections and flexural reinforcements of beams and columns are listed in tables 10-XI and 10-XII, respectively.

10.3.2 Result Comparisons

Table 10-XIII lists the the annual limit-state probabilities of the five-story frame buildings resulting from the the Tri-Services Guidelines, the NEHRP Provisions, and the proposed design criteria with the target probability of 1/5000 per year. With respect to the collapse limit state, the collapse indexes resulting from the proposed criteria and the Tri-Services Guidelines are quite close to each other (1.73 vs. 1.54). On the other hand, the collapse index from the NEHRP Provisions is 6.6, which is too large to be acceptable. This reflects that the sections and reinforcements determined according to the the NEHRP Provisions are not adequate to protect the essential building in the event of a large earthquake.

The Tri-Services Guidelines incorporate the strong-column—weak-beam concept to resist earthquakes. On the other hand, the proposed criteria use the importance factor to increase the design base shear without the

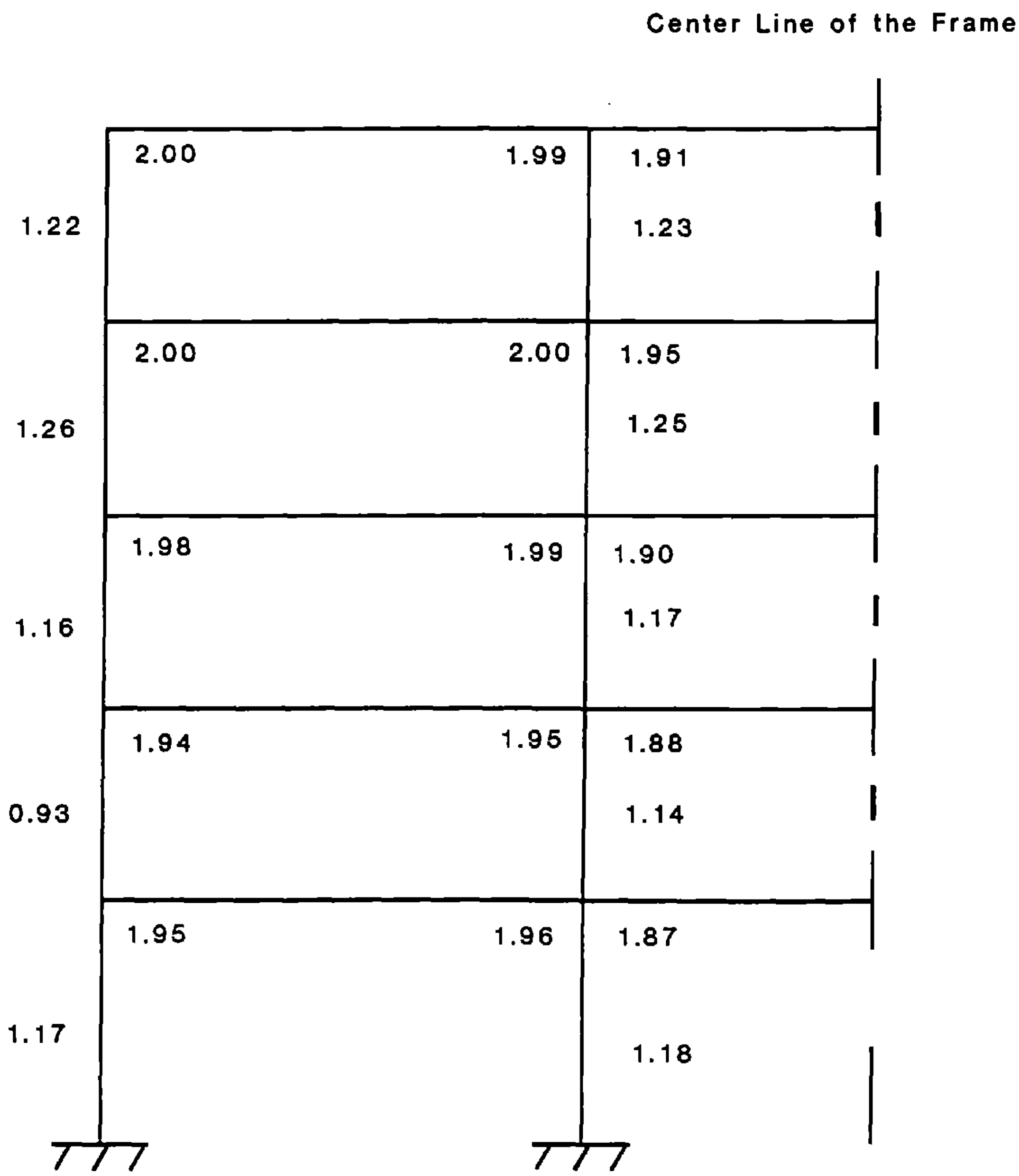


FIGURE 10-2 Inelastic Demand Ratios (Tri-Services Guidelines 1986)

TABLE 10-XI Beam Sizes and Required Steels of the Essential Building (Tri-Services Guidelines)

FL	Exterior Beams				Interior Beams			
	Section (in)	A_s and A_s' (in^2)			Section (in)	A_s and A_s' (in^2)		
		Left	Mid.	Right		Left	Mid.	Right
		8.5	1.6	8.3		8.3	1.6	7.9
1	18 x 30	5.3	1.6	4.9	18 x 30	4.9	1.6	4.6
		8.8	1.6	8.5		8.5	1.6	8.2
2	18 x 30	5.5	1.6	5.1	18 x 30	5.1	1.6	4.9
		7.3	1.5	7.0		7.0	1.5	6.9
3	18 x 28	3.9	1.5	3.7	18 x 28	3.7	1.5	3.5
		5.3	1.5	5.1		5.1	1.5	5.1
4	18 x 28	2.2	1.5	2.0	18 x 28	2.0	1.5	2.0
		3.2	1.5	3.4		3.4	1.5	3.2
5	18 x 28	1.5	1.5	1.5	18 x 28	1.5	1.5	1.5

TABLE 10-XII Column Sizes and Required Steels of the Essential Building (Tri-Services Guidelines)

Story	Exterior Columns		Interior Columns	
	Section (in)	A_s (in^2)	Section (in)	A_s (in^2)
1	32 x 32	38.1	32 x 32	40.7
2	32 x 32	12.6	32 x 32	20.5
3	28 x 28	12.6	28 x 28	20.5
4	28 x 28	12.6	28 x 28	17.1
5	28 x 28	9.4	28 x 28	10.9

TABLE 10-XIII Comparison Between Essential Buildings

Standard	PF_Y (YI) (/yr)	PF_C (CI) (/yr)
Proposed Criteria	2.69E-04 (0.03)	3.46E-04 (1.73)
NEHRP Provisions	4.48E-03 (0.45)	1.32E-03 (6.60)
Tri-Services Guidelines	6.13E-03 (0.61)	3.08E-04 (1.54)

implementation of the strong-column—weak-beam concept. As a consequence, the beam sizes determined by the proposed criteria are larger than those determined by the Tri-Services Guidelines; however, the reverse is true for column sizes. Since the first yielding usually occurs at beams, the yielding index from the proposed criteria is therefore smaller.

10.4 Uniform Reliability

The load combinations and seismic load factors resulting from this study represent an attempt to achieve a more uniform level of structural reliability among the structures designed according to the proposed criteria for various design conditions. To investigate whether or not the uniform reliability is achieved, two frame structures, one 5-story (frame 1 in table 4-II) and one 11-story (frame 2 in table 4-II), are used. These two frames considered as ordinary, high-risk, and essential buildings, are designed for four levels of design earthquakes ranging from 0.1 g to 0.4 g. Thus, 24 frames are designed from the proposed criteria and the NEHRP Provisions, separately. The annual collapse limit-state probabilities of these 24 structures are determined by using the reliability analysis method employed in this study. The values of collapse index for these 24 structures are listed in table 10-XIV. The collapse index for the proposed criteria is in the range of 0.08 to 4.64, while the index for the NEHRP Provisions is in the range of 0.07 to 56.42. It is apparent that the proposed criteria result in the more uniform reliability among the structures designed under various conditions. This result is expected because the load and important factors in the proposed criteria are obtained by using an optimizing technique to achieve consistent reliability.

TABLE 10-XIV Collapse Indexes Determined from Proposed Criteria and NEHRP Provisions

Frame	Category	Proposed Criteria				NEHRP Provisions			
		$E_D = 0.1g$	$E_D = 0.2g$	$E_D = 0.3g$	$E_D = 0.4g$	$E_D = 0.1g$	$E_D = 0.2g$	$E_D = 0.3g$	$E_D = 0.4g$
5-story	Ordinary	0.22	0.74	1.96	2.41	0.27	1.32	4.29	11.28
	High-risk	0.44	1.06	2.21	3.70	0.54	2.65	8.58	22.57
	Essential	0.80	1.73	3.40	4.64	1.36	6.61	21.45	56.42
11-story	Ordinary	0.08	0.41	0.73	0.76	0.07	0.64	2.28	4.21
	High-risk	0.15	0.43	0.68	0.67	0.14	1.28	4.57	8.42
	Essential	0.31	0.46	0.64	0.44	0.35	3.20	11.41	21.05

SECTION 11

CONCLUSIONS

Reliability-based seismic LRFD criteria for reinforced concrete moment-resisting frame buildings have been developed in this study. The seismic LRFD criteria are developed based on structural strength being considered explicitly and ductility considered implicitly. The proposed seismic LRFD criteria summarized in Section 9.4 are applicable for three categories of buildings (ordinary, high-risk, and essential buildings) in various seismic zones. The major observations and conclusions are as follows:

1. Reliability-based LRFD criteria, which have a deterministic format yet reflect the probabilistic nature of design parameters, are appropriate for routine design of buildings. The reliability-based design criteria established in this study have a well-established rationale. Furthermore, the design criteria will produce risk-consistent structures under various design conditions.

2. Two types of limit states, first yielding and collapse of structure, are considered in this study. It concludes that the collapse limit state controls the design and evaluation of the buildings. It implies that if the design satisfies the requirement for life safety in the event of a large earthquake, it will also satisfy the requirement for no structural damage in the event of a moderate earthquake. This is especially true for the eastern United States, where the large earthquake is severe but infrequent.

3. The collapse of a structure is determined from the failure mechanism of a structural system rather than the failure of a structural member. Thus, the proposed seismic design criteria are established on the basis of the seismic performance of the entire frame system.

4. The intermediate moment-resisting (IMR) frame designed according to the ACI code 318-89 is used to represent the frame system considered in this study. The IMR frame has enough strength and reasonable ductility; thus, it can be used throughout the entire United States. The IMR frame may be specially suitable for those parts of the eastern United States where no seismic requirement is currently enforced, since the complicated design and detailing of the special moment-resisting (SMR) frame may not be easily accepted by professionals in the eastern United States. Furthermore, the SMR frame as specified in the NEHRP Provisions mainly relies on ductility rather than strength to resist earthquakes and thus does not provide enough seismic damage protection for high-risk and essential buildings.

5. In the proposed seismic design procedure, the elastic-to-inelastic response factor is used to determine the design base shear. For the IMR frame, the R_μ factor is set as 2.5. In this study, the limit state considered is the collapse of a structure; thus, the R_μ factor is used to obtain the equivalent nonlinear base shear at the collapse stage from the elastic base shear. Furthermore, this study demonstrates that if a larger R_μ factor is used in the design, it requires a larger seismic load factor to achieve the same acceptable risk level. Thus, there is no

advantage to using the R_μ factor larger than the value justified by the structural behavior.

6. The acceptable risk levels for three categories of buildings have been investigated. For the collapse of a structure as the limit state, the acceptable (target) limit state probability is 1 in 1000 per year for ordinary buildings, 1 in 2000 per year for high-risk buildings, and 1 in 5000 to 1 in 10,000 per year for essential buildings. It seems that the target probability of 1 in 10,000 per year for essential buildings is too stringent to be accepted in view of the current practice.

7. The seismic load factors for ordinary buildings have been determined for various seismic zones in the United States. The seismic load factor for the area with high seismicity such as California is determined as 1.3, which is larger than the value for the area with low seismicity. This is due to the fact that the structural capacity and response do not increase proportionally to the increase of the design earthquake because of the constant gravity loads involved.

8. The seismic load factors determined for ordinary buildings are also used for high-risk and essential buildings. To meet the more stringent acceptable risk level for high-risk and essential buildings, the importance factor I is used to increase the design strength. The I factor of 1.2 is recommended for the high-risk building, while 1.5 is recommended for the essential building, if the acceptable collapse probability is chosen as 1 in 5000 per year.

9. For low seismicity area such as the design earthquake less than 0.1 g, gravity loads rather than seismic load dominate the design of structures. It has been shown that if the frame structure is designed only for dead and live loads and the detailing of structural members follows the requirement for the IMR frame, then the frame structure can provide enough seismic resistance for ordinary and high-risk buildings. In other words, if the frame is tied together to provide the required ductility, the frame with the strength required by gravity loads can provide enough strength to resist earthquakes. For essential buildings, the seismic design with both the seismic load factor and the importance factor as 1.0 is required to satisfy the acceptable risk level specified for essential buildings.

The seismic LRFD criteria for IMR frame buildings are established on the basis of ultimate strength corresponding to the collapse limit state. Further research is needed to implement other limit states such as instability and drift limit. Furthermore, the proposed seismic design criteria are developed for reinforced concrete intermediate moment-resisting frame buildings. The general approach established in this study can be used to develop the seismic design criteria for other types of buildings such as shear wall and steel buildings.

SECTION 12

REFERENCES

Algermissen, S.T. and Perkins, D. M. (1976), "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States," USGS Open File Report 76-416. Reston, Virginia, U.S. Geological Survey.

Ang, A. H.-S. and Newmark, N.M. (1977), "A Probabilistic Seismic Safety Assessment of the Diablo Canyon Nuclear Power Plant," Report to the U.S. Nuclear Regulatory Commission, Washington, D.C..

Ang, A. H.-S. and Hu, Y.-X (1983), "Development of Probability Bases for Earthquake Resistant Design," Technical Report, Department of Civil Engineering, University of Illinois, Urbana, Illinois.

Aoyama, H. (1981), "A Method in the Evaluation of the Seismic Capacity of Existing Reinforced Concrete Buildings in Japan," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14, No. 3, pp. 105-130.

Bertero, V.V. (1986), "Lessons Learned from Recent Earthquakes and Research and Implications for Earthquake-Resistant Design of Building Structures in United States," Report No. UCB/EERC-86/03, Earthquake Engineering Research Center, University of California, Berkeley, California.

"Building Code Requirement for Reinforced Concrete," (1989), ACI 318-89, American Concrete Institute, Detroit, Michigan.

Coleman, R.A. (1983), "Structural System Design," Prentice-Hall International, Inc., Englewood, New Jersey.

"Design Handbook in Accordance with the Strength Design Method of ACI 318-89," (1991), ACI Publication SP-17A (91), American Concrete Institute, Detroit, Michigan.

"Earthquake Resistant Regulations: A World List - 1988," (1988), International Association for Earthquake Engineering, Tokyo, Japan.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1980), "Development of Probability-Based Load Criteria for American National Standard A58," National Bureau of Standards Special Publication No. 577, Washington, D.C..

Ellingwood, B. and Hwang, H. (1985), "Probabilistic Descriptions of Resistance of Safety-Related Structures in Nuclear Power Plants." Nuclear Engineering and Design, Vol. 88, pp. 169-178.

Fintel, M. (1985), "Handbook of Concrete Engineering," Second Edition, Van Nostrand Reinhold Company Inc., New York, New York.

Freeman, S.A. (1978), "Prediction of Response of Concrete Buildings to Severe Earthquake Motion," Proceedings of the Douglas McHenry International Symposium on Concrete and Concrete Structures, ACI Publication SP-55, American Concrete Institute, Detroit, Michigan, pp. 589-605.

Galambos, T.V. (1990), "System Reliability and Structural Design," Journal of Structural Safety, Vol. 7, Nos. 2-4, pp. 101-108.

Gromala, D.S. and Cheung, C.K. (1991), "Design of Wood Structures: LRFD and ASD," Compact Papers of the Ninth Structures Congress, Indianapolis, Indiana, American Society of Civil Engineers, New York, New York, pp. 331-334.

Haviland, R. (1976), "A Study of the Uncertainties in the Fundamental Translational Periods and Damping Values for Real Buildings," Technical Report, Department of Civil Engineering, Massachusetts Institute of Technology, Boston, Massachusetts.

Hawkins, N.M. (1986), "Seismic Evaluation Procedure for Existing Structures," Convention Seminar for Seismic Design Concerns for Existing Structures, American Concrete Institute, Detroit, Michigan.

Hays W.W. (1985), "An Introduction to Technical Issues in the Evaluation of Seismic Hazards for Earthquake-Resistant Design," ICSSC TR-6, U.S. Geological Survey, Reston, Virginia.

Hwang, H., Ellingwood B., Shinozuka M., and Reich, M. (1987), "Probability-Based Design Criteria for Nuclear Plant Structures," Journal of Structural Engineering, American Society of Civil Engineers, Vol. 113, No. 5, pp. 925-942.

Hwang, H. and Hsu, H.M. (1991), "Seismic Performance of RC Special vs. Intermediate Moment Frame Structures," Compact Papers of the Ninth Structures Congress, Indianapolis, Indiana, American Society of Civil Engineers, New York, New York, pp. 215-218.

Hwang, H. and Jaw, J.W. (1989), "Statistical Evaluation of Response Modification Factors for Reinforced Concrete Structures," National Center for Earthquake Engineering Research, Buffalo, New York.

Hwang, H. and Lee, C.S. (1990), "Site-Specific Response Spectra for Memphis Sheahan Pumping Station," Technical Report NCEER-90-0007, National Center for Earthquake Engineering Research, SUNY, Buffalo, New York.

Hwang, H. and Jaw, J.W. (1990), "Probabilistic Damage Analysis of Structures," Journal of Structural Engineering, American Society of Civil Engineers, Vol. 116, No. 7, pp. 1992-2007.

Iman, R.L. and Conover, W.J. (1980), "Small Sample Sensitivity Techniques for Computer Models, With an application to Risk Assessment," Communications in Statistics, Theory and Methods, Vol. A9, PP. 1749-1842.

Jacob, K.H. (1990), "Seismic Hazards and the Effects of Soils on Ground Motions for the Greater New York City Metropolitan Region," Proceedings of the Continuing Education Seminar of the NYC Metropolitan Section of the American Society of Civil Engineers, New York, New York.

Kameda, H., Sugito, M., and Goto, H. (1982), "Microzonation and Simulation of Spatially Corrected Earthquake Motions," Proceedings of the 3rd International Earthquake Microzonation Conference, Vol. III, pp. 1463-1474.

Kulicki, J.M. and Mertz, D.R. (1991), "Development of the AASHTO LRFD Bridge Design Code: Overview and Live Load Considerations," Ninth Structures Congress of the American Society of Civil Engineers, Indianapolis, Indiana, pp. 470-473.

"Manual of Steel Construction--Load and Resistance Factor Design," (1986), American Institute of Steel Construction, Inc., Chicago.

McCann, Jr., M.W. (1983), "Uncertainty in Ground Motion Predictions," Proceedings of Conference XXII, A Workshop on Site-Specific Effects of Soil and Rock on Ground Motion and the Implications for Earthquake-Resistant Design, United States Geological Survey, Reston, Virginia, pp. 331-356.

"Minimum Design Loads for Buildings and Other Structures," (1988), ASCE Standard ASCE 7-88 (Formerly ANSI A58.1), American Society of Civil Engineers, New York, New York.

Mozer, J.D. and DiGioia, Jr., A.M. (1991), "Overview of the New ASCE T/L Structure Loading Guide," Ninth Structures Congress of the American Society of Civil Engineers, Indianapolis, Indiana, pp. 112-115.

"NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings," (1988), Earthquake Hazard Reduction Series 17 and 18, Building Seismic Safety Council and Federal Emergency Management Agency, Washington, D.C..

Newmark, N.M. and Hall, W.J. (1973), "Procedures and Criteria for Earthquake Resistant Design," Building Practice for Disaster Mitigation, Building Science Series 46, National Bureau of Standards, pp. 209-236.

Okrent, D. (1989), "Safety Goals, Uncertainties, and Defense in Depth," in Risk Analysis and Management of Natural and Man-Made Hazards, Yacov Y. Haimes and Eugene Z. Stakhiv (eds), American Society of Civil Engineers, New York, New York, pp. 268-282.

Ravindra, M.K. and Galambos, T.V. (1978), " Load and Resistance Factor Design for Steel," Journal of Structural Engineering, American Society of Civil Engineers, Vol. 104, No. ST9, pp. 1337-1353.

Riddell, R. and Newmark, N.M. (1979), "Statistical Analysis of the Response of Nonlinear Systems Subjected to Earthquakes," Civil Engineering Studies, Structural Research Series No. 468, University of Illinois, Urbana, Illinois.

SEAOC Seismology Committee (1990), "Recommended Lateral Force Requirements and Commentary," Structural Engineering Association of California, San Francisco, California.

Seed, H.B., Ugas, C., and Lysmer, J. (1976), "Site-Dependent Spectra for Earthquake-Resistant Design," Bulletin of Seismological Society of American, Vol. 66, No. 1, pp. 221-244.

"Seismic Design for Buildings (Tri-Services Manual)," (1982), Departments of the Army, the Navy, and the Air Force, Washington, D.C..

"Seismic Design Guidelines for Essential Buildings (Tri-Services Guidelines)," (1986), Departments of the Army, the Navy, and the Air Force, Washington, D.C..

Shinozuka, M., Furuta, H., Emi, S., and Kubo, M. (1989), "Reliability-Based LRFD for Bridges: Theoretical Basis," Proceedings of the 5th International Conference on Structural Safety and Reliability, San Francisco, American Society of Civil Engineers, New York, New York, pp. 1981-1986.

"Standard Building Code," (1988), Southern Building Code Congress International, Inc., Birmingham, Alabama.

Steinman, R.W. (1980), "A Consistently Well-Behavior Method of Interpolation," Creative Computing, pp. 54-57.

"The BOCA National Building Code," (1990), Building Officials and Code Administrators International, Country Club Hills, Illinois.

"Uncertainty and Conservatism in the Seismic Analysis and Design of Nuclear Facilities," (1986), American Society of Civil Engineers, New York, New York.

"Uniform Building Code," (1988), International Conference of Building Officials, Whitter, California.

Wang, P. C. (1980), "PC-Stress Structural Engineering System Solver," Garden City, New York.

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