Structural Design under Seismic Risk Using Multiple Performance Objectives

Thesis by

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Abstract

Structural design is a decision-making process in which a wide spectrum of requirements, expectations, and concerns needs to be properly addressed. Engineering design criteria are considered together with societal and client preferences, and most of these design objectives are affected by the uncertainties surrounding a design. Therefore, *realistic design frameworks must be able to handle multiple performance* objectives and incorporate uncertainties from numerous sources into the process.

In this study, a multi-criteria based design framework for structural design under seismic risk is explored. The emphasis is on reliability-based performance objectives and their interaction with economic objectives. The framework has analysis, evaluation, and revision stages. In the probabilistic response analysis, seismic loading uncertainties as well as modeling uncertainties are incorporated. For evaluation, two approaches are suggested: one based on preference aggregation and the other based on socio-economics. Both implementations of the general framework are illustrated with simple but informative design examples to explore the basic features of the framework.

The first approach uses concepts similar to those found in multi-criteria decision theory, and directly combines reliability-based objectives with others. This approach is implemented in a single-stage design procedure. In the socio-economics based approach, a two-stage design procedure is recommended in which societal preferences are treated through reliability-based engineering performance measures, but emphasis is also given to economic objectives because these are especially important to the structural designer's client. A rational net asset value formulation including losses from uncertain future earthquakes is used to assess the economic performance of a design. A recently developed assembly-based vulnerability analysis is incorporated into the loss estimation.

The presented performance-based design framework allows investigation of various

design issues and their impact on a structural design. It is a flexible one that readily allows incorporation of new methods and concepts in seismic hazard specification, structural analysis, and loss estimation.

Contents

Chapter 1 Introduction

After every destructive earthquake, greater attention is given to the issues associated with structural design. In post-earthquake studies, engineering-based performance evaluations often constitute most of the effort. However, especially after recent earthquakes, the performance of structures, both individually and collectively, has been increasingly expressed in terms of non-engineering measures, such as economic losses. Along with this shift, basic assumptions regarding design objectives have been scrutinized and, as a result, they are receiving heavy criticism. For example, the design premise that has been in force for decades states that structures designed following code specifications "in general, should resist a minor level of earthquake ground motion without damage; a moderate level of earthquake ground motion without structural damage, while possibly experiencing some non-structural damage; and, a major level of earthquake motion without collapse, but possibly with some structural and non-structural damage" (SEAOC 1996). This statement is not very clear because the terms used in expressing the objectives are not well-defined. Furthermore, there is dissatisfaction with the way design codes implement these objectives because it is felt that only the life-safety condition is explicitly addressed in the codes and that they do not adequately treat the other two objectives stated in the design premise and which focus on damage prevention. The choice of life-safety condition has been dominant because it is the minimum requirement of the code, and quite often, structures are designed just to meet that, and no more. Unfortunately, this condition does not necessarily guarantee that the other two objectives would be met by such a designed structure. However, there are reasons why the first two objectives are generally ignored: no clear definitions of the damage terms or explicit damage prevention measures are given. This is exactly where the new paradigm in structural design has its roots.

Damage is almost always expressed in terms of economic losses by individuals, as well as by society, and the codes stop short of converting the engineering design objectives into these economic terms. It is left to the designers to implicitly do the conversion and they often do not have quantitative methods of doing it. However, advances are being made in analytical, computational, and experimental tools in the fields of seismology, earthquake engineering, and economics to fill the gaps. In the meantime, to meet the new higher expectations from clients as well as to address societal concerns, the structural design profession has moved from the vague and qualitative performance objectives to well-defined and quantitative ones (ATC 1995). Professional and governmental organizations, as well as private researchers, have been conducting numerous studies to support this movement.

Vision 2000, a pioneering report prepared by SEAOC (1995), acted as the main document to initiate development of practical performance-based design methodologies and concepts, and the new attitude has been affecting design practice with regard to both existing structures (ATC 1996, 1997a, 1997b) and the new ones (BSSC 1997a, 1997b). The designer can no longer design to meet only the code minimum requirements and claim success. There are now multiple and quantitatively-stated performance objectives to meet.

The investigations by researchers, however, are generally concentrated on the specifics of various issues. One of the more important issues, which is an essential element in this current study, is the reliability of structures. Since it is surrounded by uncertainties, the ability of a structure to meet a limit-state or a design criterion needs to be expressed probabilistically, that is, in terms of a reliability measure, such as the probability of not exceeding a certain limit over a specified amount of time (see Thoft-Christensen 1990 and Schuëller 1998 for extensive lists of references). Lately, there have been attempts to implement such reliability-based limit-state considerations in design in the form of multiple performance criteria (for example, Fu and Frangopol 1990; Wen et al. 1994; Beck et al. 1996a, 1997, 1999a; Collins et al. 1996; Han and Wen 1997; Wen 1999). These implementations often result in defining design as a reliability-based optimization problem for which different optimization methods are studied (for example, Rao 1984; Enevoldsen and Sorensen 1994; Chan 1997; Beck et al. 1999a). However, few studies of this kind are implemented to understand the nature of non-engineering based performance of a structure, such as economics based ones (Ang et al. 1996; Ang and De Leon 1997; Beck et al. 1999b, 1999c; Wen 1999) and even fewer consider other non-engineering based performance criteria that might need to be considered during design (Austin et al. 1985; Takewaki et al. 1991; Beck et al. 1996a, 1996b, 1999a).

However, there are still many challenging conceptual and implementational issues to be addressed before recommended performance-based design approaches receive wide approval in practice (Blockley and Elms 1999). For example, do the performance objectives that are generally expressed in terms of reliability or risk levels at various limit-states relate well to client's, as well as society's, preferences? How do they relate to life-cycle costs? Would the performance objective specifications overconstrain the feasible design space and force a resulting design to be a suboptimal one in some sense? And of course, there is the overriding question: how should an optimal or a best design be defined and how would it be implemented in practice?

In this study, an attempt is made to address the challenges of this new environment, and a formal framework for structural design under multiple performance objectives and in the presence of uncertainties is given. In the framework, reliability-based criteria, and concepts similar to those found in multi-criteria decision theory are used to navigate the uncertain decision-making environment structural designers work in.

In Chapter 2 the general framework is presented. The details of two design evaluation approaches are given in Chapters 3 and 4. Implementation of the framework using these two approaches, and simple illustrative examples are also given in Chapters 3 and 4. The examples are used only to demonstrate various features of the methodology since the current study is an exploratory one. The last chapter contains a commentary on the work presented in this study, and it ends with a brief discussion on possible further research.

Chapter 2 A Reliability-Based Multi-Criteria Framework for Structural Design under Seismic Risk

2.1 Introduction

The design decision-making process is an iterative procedure wherein a preliminary design is cycled through stages of analysis, evaluation, and revision to achieve a design that satisfies various criteria best in some chosen sense. In the design methodology proposed in this study, a rational model of the decision-making process is made through a formal treatment of these three design stages. The development of the framework was initiated as part of a CUREe-Kajima Phase II project (Beck et al. 1996a, 1997, 1999a). The scope of the framework was extended based on research from CUREe-Kajima Phase III (Beck et al. 1999b, 1999c) and PEER projects (Beck et al. 1999d).

The methodology handles the key aspects of decision-making in a design process in a consistent and formal way. Uncertainties associated with the designed structure and the design process itself are incorporated into the framework explicitly and quantitatively. This incorporation allows a better observation of and insight into the effects of uncertainties on the resulting design. The methodology employs a modular approach to the structural design decision-making process. Therefore, it has the flexibility to allow updates and expansions in specific stages, as well as in an overall sense, as improved models of the design process are developed.

The conceptual framework is introduced first. Then, the analysis stage is explained. Emphasis is given to the treatment of uncertainties involved, since in illustrative applications of the methodology, it will be seen that risks surrounding the performance of a structure play an important role in design decisions. A brief overview of the evaluation stage will introduce two approaches. However, extensive studies of these approaches are left to later chapters where their use within the framework will be explained and illustrated by some examples. For the revision stage, only a very general review will be given since the methods used for revision have been treated in detail elsewhere, and the emphasis of this study is not on this aspect of the framework.

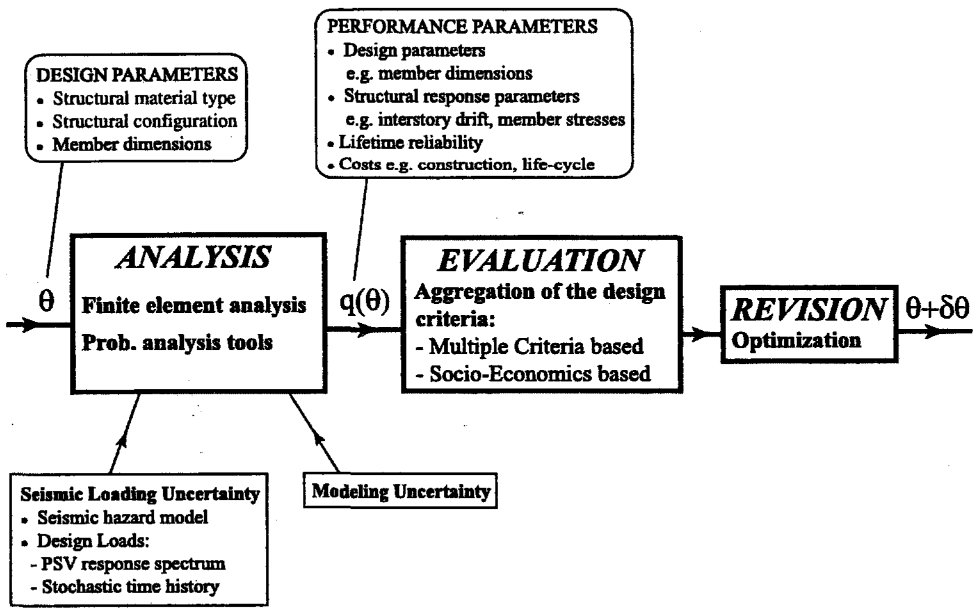

Figure 2.1: Iterative Design Framework for Performance Based Structural Design

Using Fig. 2.1 as an overview, the framework could be summarized as follows. Structural design starts with a description of the design problem as comprehensive as possible. The "design parameters" that are to be varied during the design process need to be specified. These parameters can be, for example, those that specify the structural configuration or construction materials to be used, which might be the case in early stages of the design decision-making process. Or they can be related to the geometric information for the structural members, such as cross-sectional dimensions of the structural elements, which might be the case once the structural system and configuration are chosen. Also, the designer must specify all design requirements, that is, "design criteria," on which each design is to be judged, and list the performance parameters involved in each design criterion. The performance parameters represent quantities related to the "performance" of the design, and can take the form of conventional structural parameters (for example, stress, deflection, interstory drift, or modal frequencies) or other parameters (for example, structural reliability, material cost of the structural system, life-cycle cost of the structure). The designer then chooses a structural system configuration as well as the geometrical and connection information for each structural member to obtain a preliminary design. Furthermore, the designer needs to specify all possible loading cases and associated uncertainties ("loading uncertainties") that the structure might experience during its lifetime. The choice of these cases is of utmost importance since the structural design is greatly affected by them. The "loading" module shown in Fig. 2.1 corresponds to the case of structural design against seismic loads, which is the focus of this work. However, any type of loading, with proper specification of its interaction with the designed structure as well as the associated loading uncertainties, could be included in the loading module.

In the first stage of the design process, the performance parameter values under the specified loading cases are computed through chosen analysis methods. It is important to realize that, whichever response analysis method is used (for example, static, response spectrum or dynamic), there will be an uncertainty in the computed results due not only to the uncertainties in the loads applied to the structure but also to the uncertainties related to the modeling of the structure and the analysis method employed (Beck and Katafygiotis 1998). In other words, there will be "modeling uncertainties" as well as "loading uncertainties." A rational treatment of these uncertainties and their effects on the design can be carried out by using probabilistic models and analysis tools. However, some performance parameters, such as the amount of structural material to be used for the current design, may contain very little uncertainty and so not require any probabilistic structural analysis.

Once the performance parameter values are obtained, the designer must use them to judge how well each design criterion is satisfied. This is the evaluation stage. In general, not every criterion will be satisfied in an optimal manner by the preliminary design. Therefore, the designer must revise the initial design in order to obtain a better one by trying to better satisfy the design criteria collectively. However, usually it is not possible to maximally satisfy all criteria simultaneously since some of them will conflict with each other. Therefore, a compromise, or trade-off, has to be made when seeking a better design, and an overall design evaluation measure is needed to allow such trading off. This approach to the structural design process converts the issue of structural design into a performance-based multi-criteria optimization problem.

This process of analysis, evaluation and revision is repeated iteratively and as long as it is necessary to find a design that is considered to give the best solution to the specified set of design criteria. A detailed study of the methodology developed to solve this problem follows.

2.2 Analysis Stage

In the analysis stage, the design specified by the current values of the design parameters $\boldsymbol{\theta}$ is analyzed to obtain the values of the chosen performance parameters, $\mathbf{q}(\boldsymbol{\theta})$. These values will be used in the evaluation stage. The performance parameters are functions of the current design parameters, albeit not always as explicitly as the symbolic expression implies.

Structural performance parameters under "deterministic" loads, such as those specified by codes if the design is to be based on them, can be computed using a finite-element model of the structure specified by the design parameters. On the other hand, for reliability-based performance parameters, such as the uncertain peak lifetime interstory drift, a probabilistic seismic hazard model and probabilistic analysis tools need to be used. Unfortunately, structural design in the presence of loading uncertainties and modeling uncertainties requires a more involved study and can be computationally expensive. However, in most circumstances, a design that considers these uncertainties models the state of matters more realistically than those designs ignoring them and therefore yields more accurate and informative results. It is the trade-off between completeness and computational cost that sets the extent of the incorporated uncertainties.

The performance of the structure in such uncertain environments is usually judged by safety considerations, and a measure of safety is provided by component and system reliability. For example, the peak interstory drift over the lifetime of the structure due to earthquakes is uncertain. Thus, a performance parameter that directly relates to the interstory drift reliability can be chosen. Available probabilistic analysis tools are then used to calculate the structural reliability or, equivalently, the failure probability, corresponding to a specified interstory drift limit.

The first step in developing an expression for the probability of structural failure, designated by $F(\boldsymbol{\theta})$ for a design with design parameters $\boldsymbol{\theta}$, is to characterize the seismic hazard at the site of the designed structure.

2.2.1 Seismic Hazard Analysis

The objective of a seismic hazard analysis is to obtain a probabilistic description of the ground-motion intensity at the building site over the lifetime of the structure. At a fundamental level, one can express the seismic hazard by a set of ground motion parameters $\boldsymbol{\alpha}$ (for example, response spectrum ordinates, peak ground acceleration, duration of motion, frequency content). For most probabilistic hazard models in use, these parameters depend, through appropriate attenuation relationships, on a set of uncertain seismicity variables $\boldsymbol{\gamma}$ accounting for the uncertain regional seismic environment. For example, $\boldsymbol{\gamma}$ may include variables such as earthquake magnitude, fault dimensions, source parameters, earthquake distance, propagation path properties, and local site conditions. The uncertain values of the seismicity variables $\boldsymbol{\gamma}$ are described by a probability density function $p(\boldsymbol{\gamma})$, which is based on seismological studies.

The required attenuation relationships are often derived by an empirical fit to the observed data. There is uncertainty associated with these attenuation models, even when γ is known, which is reflected by the scatter of the analyzed data about the mean or median model predictions. Therefore, the attenuation relationship should actually give a probabilistic description $p(\boldsymbol{\alpha} \mid \boldsymbol{\gamma})$ of the relation between the ground motion parameters $\boldsymbol{\alpha}$ and the seismicity parameters $\boldsymbol{\gamma}$.

In this study, the parameter chosen to represent seismic hazard is S_V, the pseudo-velocity response of the structure corresponding to its fundamental mode, that is, at its elastic fundamental period and selected damping ratio. It is also possible to consider values of S_V corresponding to more than a single mode in a multi-mode response analysis. However, the fundamental mode in a given direction often dominates the linear response and the contributions of the other modes can be included in the modeling uncertainty. Assuming that a single mode response spectrum analysis is to be used, the task is then to find $p(S_V \mid EQ,\ \boldsymbol{\theta})$, the probability density function of S_V for the structure represented by $\boldsymbol{\theta}$ given that an uncertain earthquake EQ has occurred. Since the earthquake is an uncertain event, a probability distribution over the possible range of event parameters is needed. This can be obtained from a regional seismicity model which represents the seismic environment at the building site. For example, if magnitude M and distance R from the site are chosen as the parameters to represent an earthquake, the formulation for an uncertain event can be given as

$$p(S_V \mid EQ,\ \boldsymbol{\theta}) = \int_{M,\ R} p(S_V \mid \boldsymbol{\theta},\ M,\ R)\, p(M,\ R \mid EQ)\, dM\, dR \qquad (2.1)$$

in which $p(S_V \mid \boldsymbol{\theta},\ M,\ R)$ may be obtained using available ground motion attenuation formulas. The attenuation formula from Boore, Joyner, and Fumal (1993, 1994) is used in this study and is reviewed below.

The probability distributions for M and R depend on the seismic environment. For the illustrative examples in this work, simplifying assumptions are made and the earthquake sources are taken to be point sources located in a circular area with a radius R_{max} centered at the site of the structure. It is assumed that an earthquake is equally likely to occur at any point inside this circular source region, so the probability

$p(R \mid EQ)dR$ is simply the ratio of the area of an annulus of width dR located R distance away from the center to the area of the circle with radius R_{max}. The resulting probability density function for the earthquake distance will be

$$p(R \mid EQ) = 2R/R_{max}^2 \tag{2.2}$$

For modeling the probability distribution of earthquake magnitudes, a truncated form of the Gutenberg-Richter relationship (Gutenberg and Richter 1958; Cornell and Vanmarcke 1969) can be used. In this model, the cumulative number of earthquakes per annum with magnitude up to M is given by the relation

$$log_{10}N(M) = a - bM; \qquad M \in [M_{min},\ M_{max}] \tag{2.3}$$

where M_{min} and M_{max} are, respectively, a chosen lower bound and the regional upper bound for the earthquake magnitude. The expected number of events per annum falling into the magnitude range considered is then given as

$$\nu = 10^{a-bM_{min}} - 10^{a-bM_{max}} \tag{2.4}$$

where ν is also known as the seismicity rate. These relations are consistent with a Poisson model of the occurrence of earthquakes with mean arrival rate ν and each event having a magnitude M distributed according to the probability density function

$$p(M \mid EQ) = b'e^{-b'M}/(e^{-b'M_{min}} - e^{-b'M_{max}}) \tag{2.5}$$

where $b' = blog_e(10)$.

It should be noted that, in the above simple example, the earthquake distance and the earthquake magnitude are assumed to be stochastically independent random variables, although a more refined probability model, in which a correlation between R and M for larger values of M is allowed, could be used. Of course, for more comprehensive probabilistic seismic hazard models, one may need to specify more complex geometries (Cornell 1968; Der Kiureghian and Ang 1977) or choose other parameters to model the seismic environment.

This type of modeling is generally accepted where the seismic sources are spread over a large region and have a large variation in magnitude ("non-characteristic" earthquakes). If there are well-defined sources with a repetitive character around the considered site, uncertain future earthquakes from them could be included as "characteristic" events into the seismic environment modeling (for example, Eliopoulos and Wen 1991).

As noted earlier in this study, the seismic hazard from earthquake ground motions is to be characterized by the pseudo-velocity response spectrum $S_V(T,\ \zeta)$ where T is the period and ζ is the damping ratio of a single degree-of-freedom linear oscillator. For the probabilistic seismic hazard model, the attenuation formula proposed by Boore et al. (1993) is used to model $S_V(T,\ \zeta)$ in terms of earthquake magnitude and distance. This relationship is given as

$$log_{10}(S_V(T,\ \zeta)) = log_{10}(\widehat{S}_V(T,\ \zeta)) + \varepsilon(T,\ \zeta) \tag{2.6}$$

where

$$\begin{aligned} log_{10}(\widehat{S}_V(T,\ \zeta)) &= \widehat{b}_1 + \widehat{b}_2\ (M-6) + \widehat{b}_3\ (M-6)^2 + \widehat{b}_4\ r + \\ &\quad \widehat{b}_5\ log(r) + \widehat{b}_6\ G_b + \widehat{b}_7\ G_c \end{aligned} \tag{2.7}$$

Here, $r = \sqrt{R^2 + h^2}$, where R is the earthquake distance defined as the closest horizontal distance from the site to the rupture's projection on the earth's surface, and h is a fictitious event depth determined by the regression analysis; G_b and G_c are soil type parameters that take a value 0 or 1 depending on the soil classification at the site. The best estimates of the parameters $\widehat{b}_i \equiv \widehat{b}_i(T,\ \zeta)$ appearing in the model for $\widehat{S}_V(T,\ \zeta)$ have been determined by Boore et al. (1993, 1994) by regression analyses over a large database of accelerograms for four different damping values ($\zeta = 2\%$, 5%, 10% and 20%) and at 46 different period values ranging from 0.1 sec to 2.0 sec. Cubic spline fits are obtained for each parameter as a function of the period and the critical damping ratio. A complete description of the variables appearing in the attenuation formula is given in Boore et al. (1993, 1994).

The function $\varepsilon(T, \zeta)$ in Eqn. (2.6) represents the uncertain model error in the actual spectral amplitudes $S_V(T, \zeta)$ compared with the estimated amplitudes $\widehat{S}_V(T, \zeta)$ from the model. The probability density function for $\varepsilon(T, \zeta)$ is assumed to follow a Gaussian distribution over the range of periods analyzed, with zero mean and variance given in Boore et al. (1994).

To illustrate the use of probabilistic seismic modeling, a comparison of the pseudo-velocity response spectra specified by UBC (ICBO 1994) and the results obtained for a chosen seismic environment using the above formulation are presented. The seismic environment is specified as follows: only those earthquakes within a distance of $R_{\text{max}} = 50$ Km are considered; the surrounding seismic region is not capable of generating earthquakes with magnitudes greater than 7.7; and it is assumed that earthquakes with magnitudes less than 5.0 have no structural design consequences. Therefore, $M_{\text{min}} = 5.0$ and $M_{\text{max}} = 7.7$ are chosen. The parameters for the truncated Gutenberg-Richter relationship, Eqn. (2.3), are set as $b = 1.0$ and $a = 5.0$, which result in a seismicity rate of $\nu = 1$ event/yr in the considered magnitude range. In Fig. 2.2, a qualitative comparison of the seismic hazard in the defined environment and the pseudo-velocity response spectrum specified by UBC (ICBO 1994) for seismic zone 4 (with effective peak ground acceleration of 40% of the gravitational acceleration) and site soil type S_2 (medium stiff to stiff soil condition) is given. The response spectrum given by UBC, which assumes a 5% modal damping ratio, is in the form of a "uniform hazard curve," and is generally believed to correspond to a 10% exceedance probability in 50 years. Accordingly, the pseudo-velocity response spectrum with 10% exceedance probability in 50 years in the chosen seismic environment is given in Fig. 2.2, along with those for a few other 50 year exceedance probability levels. The attenuation relationship developed by Boore et al. (1993, 1994) is used to obtain the curves and they correspond to 5% damping ratio case. The site soil type is assumed to be such that G_b=1 in the attenuation formula (in all of the examples in this study, this type of soil will be assumed which corresponds to site class B in Boore et al. (1993); it is equivalent to UBC (ICBO 1994) soil type S_2). It is observed that, depending on the period range, the UBC spectrum falls between the 4% and 15% exceedance levels of the probabilistic response spectrum for the specified seismic environment.

Similar seismic hazard models with various seismicity rates will be used in the examples in later chapters. It should be noted that, without a model for the probabilistic nature of the seismic hazard, no complete analysis of a structural design under seismic risk can be carried out. Therefore, proper specification and modeling of the seismic environment and the hazard it poses to the designed structure are tasks with fundamental importance.

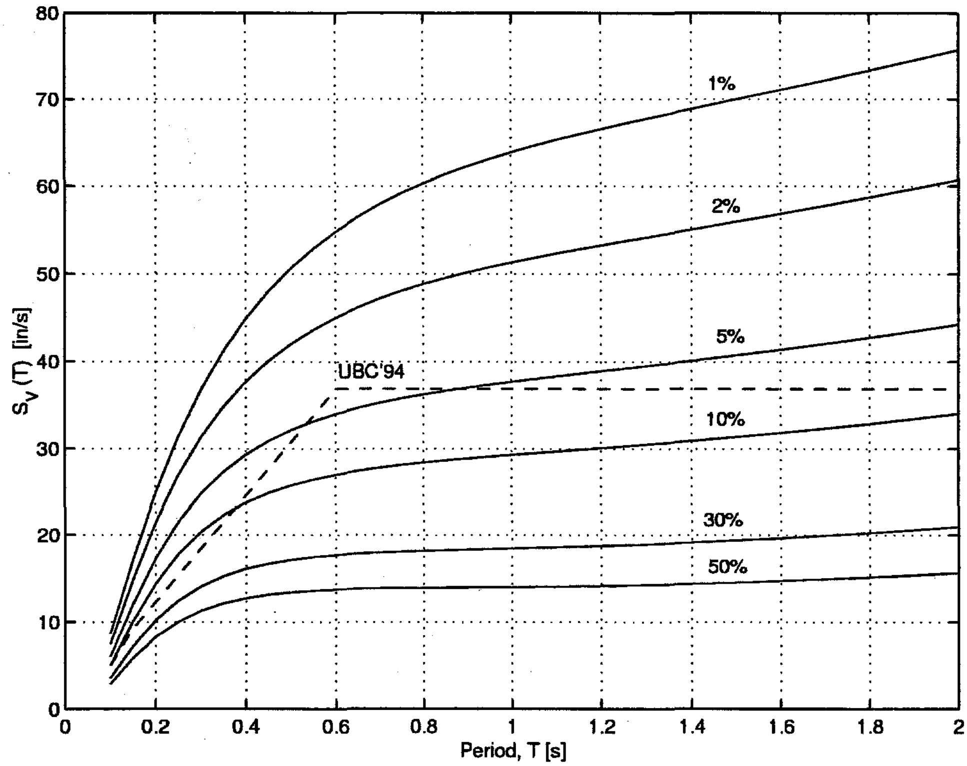

Figure 2.2: UBC (1994) Response Spectrum for Zone 4, Soil Type S_2, and Site-Specific Uniform Hazard Spectra for Various Exceedance Probabilities over 50 Years ($a = 1.0$, $b = 1.0$, $\nu = 1$ event/yr, $M \in [5.0,\ 7.7]$ and $R \leq 50$ Km; Damping Ratio = 5%)

2.2.2 Structural Response Analysis

As mentioned above, response spectra are used to express the seismic hazard and, consequently, a pseudo-dynamic type of structural response analysis based on these spectra will be used in this study. However, it has to be emphasized that the theoretical framework can utilize other methods for the response analysis, such as time-history analyses using some form of Monte Carlo simulation (for example, Au, Papadimitriou, and Beck 1999). Whichever response analysis method is chosen, associated tools that treat relevant uncertainties need to be properly developed and incorporated into the framework.

To be able to perform a response spectrum analysis, first of all, a model with discrete lumped masses is formed for the structural configuration. This model includes the concentrated mass and the stiffness values at all considered degrees of freedom. Expressing the mass and stiffness values in matrix form as $\boldsymbol{M}$ and $\boldsymbol{K}$, respectively, the mathematical formulation for the seismic response becomes

$$\boldsymbol{M}\ddot{\boldsymbol{x}} + \boldsymbol{K}\boldsymbol{x} = \boldsymbol{M}\boldsymbol{f}\ddot{z}(t) \tag{2.8}$$

where $\ddot{z}$ is the ground acceleration in the direction of interest, $\boldsymbol{x}$ is the displacement vector for the degrees of freedom of the structural model, and $\ddot{\boldsymbol{x}}$ is the associated acceleration vector. In the current study, only the lateral motion in one plane of the structural models is taken into consideration, and therefore the components of influence vector $\boldsymbol{f}$ become $f_i = 1$ if i corresponds to a horizontal degree-of-freedom in the plane of interest and $f_i = 0$ otherwise. It should be noted that damping need only be specified later when obtaining the relevant response spectral value.

If $\boldsymbol{M}$ and $\boldsymbol{K}$ are N_mxN_m matrices, the corresponding eigenproblem can be written for each eigenvalue, that is, structural mode, as (Chopra 1995)

$$w_i^2 \boldsymbol{M}\phi_i = \boldsymbol{K}\phi_i; \qquad i = 1, \ldots, N_m \tag{2.9}$$

where w_i is the i-th modal circular frequency which is equal to $2\pi/T_i$, T_i being the corresponding modal period, ϕ_i is the corresponding mode shape, and N_m is the number of degrees of freedom.

The response of the structure is a superposition of the modal contributions which are found using the corresponding modal frequencies and assumed modal damping ratios. However, to be able to combine these modal responses properly, the corresponding modal participation factors need to be computed. For example, the contribution to the displacement at the i-th story by the j-th mode, has a peak value d_{ij} given by

$$d_{ij} = \frac{\Gamma_j}{w_j} \phi_{ij} S_V(w_j, \zeta_j) \tag{2.10}$$

where the modal participation factor Γ_j is given by

$$\Gamma_j = \frac{\phi_j^T \boldsymbol{M} \boldsymbol{f}}{\phi_j^T \boldsymbol{M} \phi_j} = \frac{\sum_{k=1}^{N_m} f_k M_k \phi_{kj}}{\sum_{k=1}^{N_m} M_k \phi_{kj}^2} \tag{2.11}$$

if the mass matrix is diagonal, $\boldsymbol{M} = \mathrm{diag}(M_1, \ldots, M_{N_m})$.

The peak contributions from each mode can be combined in different ways depending on the modal characteristics of the structural system (Der Kiureghian 1981). Structural modeling errors and the uncertainty in an estimate given by the modal combination rule could be included in the analysis. In this study, a single mode analysis based on the response spectra for the fundamental mode of the structure is used.

Modeling uncertainties can be incorporated into the response estimates by assuming a log-normal distribution on the estimates (Shome et al. 1998; Beck et al. 1999c). For example, the response given by the fundamental mode can be taken as the median, and the log-standard deviation can be determined by simulations (Beck et al. 1999c). Such an approach could be used to treat neglected contributions from higher modes at low intensity shaking, and to treat the effects of non-linear response at high intensity shaking. This approach is described later in Chapter 4.

2.2.3 Reliability Computations

Knowing the ground motion parameter S_V for a site does not completely specify the structural excitation. Because of the presence of modeling errors, that is, "parameter uncertainties" resulting from incomplete knowledge of the best values of the model parameters to represent the structure together with "prediction-error uncertainties" resulting from the imperfect analytical models and analysis methods used in response estimation, the response of the structure cannot be predicted exactly. These uncertainties mean that a failure probability corresponding to a design $\boldsymbol{\theta}$ and conditional on the ground motion parameters, designated by $F(\boldsymbol{\theta} \mid S_V)$, must be set up.

In the case of "complete knowledge," that is, if the response of a structure given the S_V value can be predicted precisely, $F(\boldsymbol{\theta} \mid S_V)$ would be a binary function with a value of 0 or 1. Otherwise, the conditional failure probability $F(\boldsymbol{\theta} \mid S_V)$ can be obtained using probabilistic analysis tools and it will be a function spread over the range between 0 and 1.

$F(\boldsymbol{\theta} \mid S_V)$ is also known as the "fragility" of the structure defined by $\boldsymbol{\theta}$ experiencing the seismic attack prescribed by S_V. In a general context, fragilities can be defined at the component or the system levels. Analytic evaluations for component fragilities are possible while the system fragilities are mostly obtained using empirical or simulation techniques.

If the peak interstory drift d_{max} in a building is taken as an example, the corresponding drift risk $F_d(\boldsymbol{\theta},\ t_{life}) = P(d_{max} > d_{allow} \mid \boldsymbol{\theta},\ t_{life})$ over the lifetime t_{life} of the structure can be found as follows. First, the failure probability $F(\boldsymbol{\theta} \mid EQ)$ given the occurrence of an earthquake of uncertain magnitude and location, denoted by EQ, needs to be found. For the peak interstory drift, this can be expressed as

$$F(\boldsymbol{\theta} \mid EQ) = P(d_{\max} > d_{allow} \mid EQ,\ \boldsymbol{\theta}) = P(\bigcup_{i=1}^{n}\{d_i > d_{allow}\} \mid EQ,\ \boldsymbol{\theta}) \tag{2.12}$$

where n is the number of stories in the building and d_i is the peak interstory drift in the i-th story.

The failure probability over the lifetime of the structure is then computed using an occurrence model for earthquake events. Assuming that the occurrences of earthquakes follow a Poisson arrival process, the probability that the structural safety requirements are not satisfied during the lifetime t_{life} years of the structure is given by

$$F_d(\boldsymbol{\theta},\ t_{life}) = 1 - \exp[-\nu\ F(\boldsymbol{\theta} \mid EQ)\ t_{life}] \tag{2.13}$$

where ν is the expected number of events per annum, that is, the annual seismicity rate. How $F(\boldsymbol{\theta} \mid EQ)$ is computed depends on the modeling assumptions.

If the assumption is made that the interstory drifts, for example, are known once S_V and $\boldsymbol{\theta}$ are given, the resulting conditional failure probability $P(d_{\max} > d_{allow} \mid S_V,\ \boldsymbol{\theta})$ is either 1 or 0, depending on whether the safety levels have been exceeded or not. This is the case for the examples in Chapter 3 where the response uncertainty is ignored. In that case, only S_V is uncertain and the range in the S_V parameter space can be divided into safe and unsafe parts where $F(\boldsymbol{\theta} \mid S_V)$ takes the value 0 or 1, respectively.

First, consider the case of component reliability for which there is a single failure point, typically given in the implicit form $g(S_V) = 0$, separating the safe and unsafe parts of the parameter space of S_V: $\mathcal{S} = \{S_V \in \mathrm{R} : g(S_V) > 0\}$ and $\mathcal{F} = \{S_V \in \mathrm{R} : g(S_V) < 0\}$, respectively. The probability of failure then becomes

$$F(\boldsymbol{\theta} \mid EQ) = \int_{g(S_V)<0} p(S_V \mid EQ, \boldsymbol{\theta})\, dS_V \tag{2.14}$$

In this study, a trapezoidal numerical integration scheme in *MATLAB* (MathWorks Inc. 1998) is used to evaluate Eqn. (2.14). This reliability integral can also be evaluated approximately using available first-order or second-order reliability methods (for example, Madsen et al. 1986; Der Kiureghian et al. 1987; Breitung 1989; Polidori et al. 1999).

In the case of multiple components, each with a failure point defining the component's failure and with system failure occurring when any one of the multiple components fails, that is, the system is inside one of the unsafe ranges defined by the component failure points collectively, the formulation can be made as follows. Let $g_i(S_V) = 0$ be the equation for the i-th failure point. The unsafe interval for S_V is the union of the unsafe ranges for each component. The reliability integral for the failure probability given the occurrence of an earthquake is then

$$F(\boldsymbol{\theta} \mid EQ) = \int_{\bigcup_{i=1}^{n} g_i(S_V)<0} p(S_V \mid EQ,\ \boldsymbol{\theta})\ dS_V \tag{2.15}$$

which is in the form of the classical reliability expression for a series system. This type of reliability is encountered in design when, for example, design criteria specify upper limits on the maximum lifetime drift at each story. The limit point $g_i(S_V) = 0$ in this case is the interstory drift requirement for the i-th floor so the failure probability is given by Eqn. (2.15) where $g_i = g_i(S_V) = d_{allow} - d_i(S_V)$.

Simplified approximations to the reliability integral in Eqn. (2.15) can be given in several cases, provided certain conditions apply. One approximation is to replace the reliability integral by the sum of the component reliability integrals, that is, summing the contributions from each component, which works well if the contributions from the overlapping failure ranges are insignificant. Another approximation is, however, to consider the contribution from the component with the highest failure probability while neglecting the contributions from the other components. This will yield a good approximation if the contributions from failures of the other components are insignificant or if the significant parts of the failure ranges of the other components are subsets of the failure range of the component with the highest failure probability. In the numerical results that follow, it is found that considering only the component corresponding to the highest failure probability results in a good approximation of the system reliability for the type of design problem to be discussed therein.

As mentioned earlier, the trapezoidal numerical integration scheme is used to compute the multi-dimensional reliability integrals. However, these integrals can be evaluated numerically only if their dimension is sufficiently low (say, 2 or 3), as it is in this work. Otherwise, efficient importance sampling simulation methods (Schuëller and Stix 1987; Bucher 1988; Papadimitriou, Beck, and Katafygiotis 1997;

Au, Papadimitriou, and Beck 1999) or asymptotic methods (Papadimitriou, Beck, and Katafygiotis 1997) need to be used. An example using the asymptotic method for a multi-mode probabilistic analysis can be found in Beck et al. (1997, 1999a).

In Chapter 4, the response uncertainty given S_V is considered at the expense of increased computational effort. In this case, it is explicitly acknowledged that S_V does not define completely the response of a multi-degree-of-freedom system. Using the total probability theorem, the uncertainties in the seismic environment, ground motion modeling and structural modeling can be combined to determine the total failure probability given the occurrence of an earthquake event as

$$F(\boldsymbol{\theta} \mid EQ) = \int_{S_V} F(\boldsymbol{\theta} \mid S_V)\, p(S_V \mid EQ,\ \boldsymbol{\theta})\, dS_V \tag{2.16}$$

where $F(\boldsymbol{\theta} \mid S_V) = 1 - P(d_{max} \leq d_{allow} | \ \boldsymbol{\theta},\ S_V)$. The cumulative distribution function on the drift response can be determined by assuming a log-normal probability distribution on the drift given S_V, where the median is given by the fundamental mode response and a proper log-standard deviation is determined by simulations (Shome et al. 1998; Beck et al. 1999c).

2.3 Evaluation Stage: Two Approaches

The objective of the evaluation stage of the optimal design methodology is to obtain an overall design evaluation measure for the design specified by the current value of the design parameters. This measure serves as an objective function which, at the revision stage, is used to determine improved, or optimal, designs.

In general, for evaluation of the design, the designer may wish to impose numerous design criteria. These criteria might be related to structural material choice, element dimension, or overall configuration, and therefore, could be classified as architectural or constructional preferences. They might be related to engineering performance objectives, such as the peak responses of the structure under various types and levels of loadings, or they might relate to economic performance objectives, which express preferences related to life-cycle costs, including construction and various life-time costs. Therefore, the problem is fundamentally a multi-criteria decision-making problem in the presence of uncertainties. A methodology is required in which a design can be quantitatively evaluated on the basis of each design criterion individually as well as all of them collectively. Furthermore, since not every design criterion can be satisfied to its maximum extent simultaneously with the other design criteria, the methodology must allow a trade-off to occur between conflicting criteria in the optimization process. To trade off in a controlled manner, the designer should also be given the freedom to set the relative importance of each design criterion explicitly.

Two approaches to perform the overall evaluation have been developed for the design methodology. The first one, an approach based on multi-criteria decision theory (Keeney and Raiffa 1976; Cohon 1978; French 1988), begins by evaluating each design criterion separately and then combines them in a chosen way to obtain an overall design evaluation measure. The second approach is a socio-economics based one, and treats the problem differently. It converts the consequences of various choices and limit-states into monetary terms. However, it still allows explicit imposition of reliability-based preferences. These two approaches give two different ways to look at the problem of structural design decision-making, and the resulting designs from these approaches would in general be different. This is not due to any inconsistency of the design methodology or the evaluation approaches but is due to lack of explicit transformations between the preferences used or implied in the two approaches. Since these multi-criteria based and socio-economics based approaches will be studied in detail in Chapter 3 and Chapter 4, respectively, only brief introductions to each one are given in here.

In the multi-criteria based approach, the current design is evaluated first from many points of view addressing particular concerns, such as those related to the engineering design objectives as well as client's preferences and societal concerns, and then the design is assigned an overall performance value so that it can be compared with other designs. There are various ways to model and implement this approach using preference aggregation of multiple performance objectives. A particular one which is believed to be both intuitive and practical will be studied in detail and illustrated with example applications in Chapter 3.

The second approach to be explored is a socio-economics based one. Despite its engineering and therefore technical nature, the design process has to produce results that satisfy qualitative criteria demanded by society, as well as others expected by the client. It needs to be mentioned that modeling the preferences associated with these criteria is often a very challenging task. A simple but informative approach to evaluate the current design based on socio-economic objectives will be explained and illustrated in Chapter 4.

2.4 Revision Stage

Due to the nature of the engineering design processes, the methodology has two interrelated aspects: modeling the structural decision-making process in which multiple objectives can be specified quantitatively, and optimization of these objectives in some sense. In this study, emphasis is given to the former aspect. Therefore, only an overview of the revision stage is given here.

In this stage, the objective is to find the values for the design parameters that maximize the overall design evaluation measure and thus give the optimal design. Convergence to this design takes place in an iterative manner and the rate of convergence depends not only on the nature of the problem and the complexity of its mathematical formulation but also on the characteristics of the optimization technique employed. In this study, especially since the multi-objective design process has been transformed into a non-constrained mathematical optimization problem, numerous techniques could be used. However, due to the heavy computational effort required at the analysis stage, especially by structural analyses in the case of large structural systems, and by the probabilistic structural response estimations with high dimensional integral evaluations, the type of optimization techniques are limited in practice.

In the design examples of this study, continuous design parameters are considered.

In searching for optimal designs the adaptive random search technique is used (Masri et al. 1980). In practice, the design parameters are generally defined over a discrete set. For example, if these parameters are the dimensions of the structural members, say, I-sections, then a discrete set of sizes needs to be considered, such as the W-shapes listed in the AISC manufactured steel I-sections catalogs (AISC 1989). In such cases, even though the results for an optimal design over continuous parameters could be used as an aid, ultimately, discrete optimization methods need to be used. Genetic algorithms are such methods; the use of genetic algorithms in structural optimization and also within the multi-objective design methodology is given in Beck et al. (1997) and Chan (1997). In some of the examples presented later, results obtained from searches over small discrete sets will be used for comparison with the optimal design based on continuous design parameters.

2.5 Concluding Remarks

Optimal structural design in the presence of uncertainties is a formidable task and requires consideration of many issues and concerns at once. A rational treatment of the design process has been attempted in this study. The three main stages of design, namely, analysis, evaluation, and revision stages, have been formalized. A description of these stages for optimal design and the formulations therein have been given with special emphasis on treating the lifetime reliability corresponding to the various desired performance goals. Extensive treatments of two approaches developed to perform evaluations are covered in the next two chapters, along with illustrative examples to demonstrate various features of the methodology.

Chapter 3 Design Evaluation Using Preference Aggregation of Multiple Performance Objectives

3.1 Introduction

During design, many performance objectives need to be taken into consideration. These objectives may originate from various concerns or preferences. For example, regarding the requirements and preferences by the society, there might be those related to minimum safety requirements as specified by the codes, or concerns about damage states that would hinder the use of the designed structure. There might be preferences of the architects or engineers on the types of construction materials, or configuration and dimensions of the structural elements reflecting the preferences of the architectural and constructional aspects of the design. Also, the client may have preferences or requirements regarding the structure's functionality or costs, such as keeping the construction cost as low as possible. To allow consistent treatment of these often conflicting objectives in an optimal design methodology, their corresponding preferences must somehow be explicitly stated so that a rational trade-off can be made in the design. Even though it might seem rather unusual and possibly tedious to define the preferences for each objective, similar judgmental processes regarding various design objectives take place in every design process, however informally. The first and foremost difficulty is in expressing the preferences explicitly. But it is a requirement of rational decision-making to do so (for example, Keeney and Raiffa 1976; French 1988). The other difficulty is finding a means to combine the individual preferences in an appropriate way to obtain a single measure for the goodness of the design. Such a measure will allow consistent comparison of designs.

In this chapter, an analytical approach to assist in evaluating the performance of a design is presented. It fits into the evaluation stage of the conceptual framework for structural design based on multiple criteria that was explained in Chapter 2. The approach includes specification of preferences on various design criteria (performance objectives), as well as an aggregation rule that can be used to combine the individual preferences to yield an overall design evaluation measure. The fundamentals of the approach were developed by Beck et al. (1996a, 1996b, 1997). The approach is explained in the next section. Integration of reliability-based objectives into this multi-criteria design framework is emphasized. A simple structure is then designed under various circumstances to illustrate the optimal design methodology. Some comments are made on various performance objectives, especially those required or recommended by the structural engineering design profession, based on the results.

3.2 Preference Functions and Preference Aggregation

In order to perform a quantitative evaluation of a design, a set of preference functions μ_i, $i = 1, \ldots, N_c$ corresponding to N_c design criteria is specified where μ_i defines the preference for the various values of each design parameter or performance parameter involved in the corresponding criterion. Within the current context, a preference function is a value function that may simply express a minimum and/or maximum fuzzy bound on a design quantity (design parameter or performance parameter), or it may express a more complex design criterion (such as a function of various design quantities). For a given performance parameter value, a larger preference value at one performance parameter value compared to that for another performance parameter value implies that the decision-maker prefers the first parameter value more than the other. An equivalence of preference values implies an indifference of the decision-maker between the corresponding values of the performance parameter. In other words, a preference function is an ordinal value function that ranks the values of the respective performance parameter using the associated preference values.

To be able to formulate preferences in a consistent way, the values that a preference function can take must lie in a standard range, for example, the unit interval [0, 1]. However, the choice of the range is rather arbitrary since preferences cast in a chosen interval can be transformed into [0, 1] without altering preference attitudes through positive affine transformations, that is, transformations having the form of $\mu' = \alpha\mu + \beta$, $\alpha > 0$, where μ denotes the preference function with the initial value range and μ' is the preference function with the new value range (French 1988).

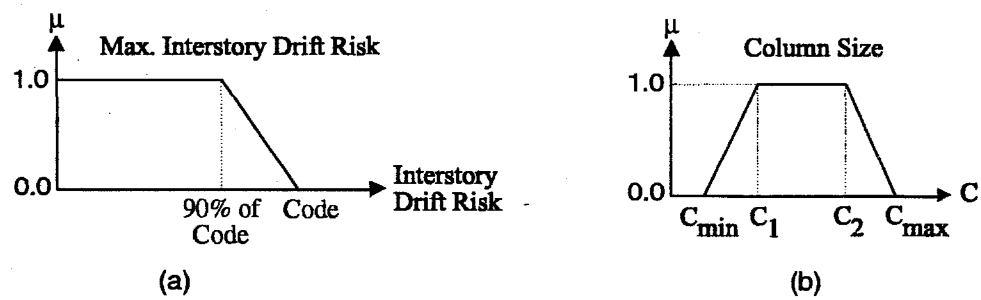

Figure 3.1: Example Preference Functions

Fig. 3.1(a) shows a preference function for the design criterion that the maximum interstory drift should not exceed some code prescribed value. In this case, the user prefers most the maximum interstory drift values that are less than 90% of the code-specified drift value, since the preference function has its greatest possible value there, namely, 1. On the other hand, the user considers values of the maximum interstory drift that exceed the code-specified drift value as unacceptable, as there the preference function has the least possible value, zero. As shown in Fig. 3.1(a), a linear fall-off has been chosen for those values of the maximum interstory drift which lie between 90% and 100% of the code-specified drift value. The forms of the preference functions may differ depending on the decision-maker's attitude to different design criteria. They play the critical role of carrying the subjective attitudes of the decision-maker into the design process. It should be added that, in most codified design requirements, the preferences are stated using "hard" boundaries. In other words, the transition between fully acceptable to totally unacceptable designs over a specified design objective occurs at a single value of the corresponding performance parameter. Such discontinuous preferences are believed to be unrealistic in many engineering designs. Almost always, there exists a fuzzy tolerance region between the totally satisfactory and totally unsatisfactory performance parameter values. Besides, in many cases, the performance parameter involved has associated uncertainty and is not as crisp to justify hard boundaries. In other words, the value obtained from an analysis for that performance parameter is not certain, and rejecting or accepting a design due to some small variations in that parameter can not be justified. Therefore, fuzzy transitions ("soft" boundaries) are recommended for use at the regions where a performance criterion changes from acceptable to unacceptable.

In the current approach, constraints directly imposed on the design parameters, such as geometrical and/or material constraints, are treated as additional design criteria. By treating design parameter constraints in this way, the degree to which the constraint is satisfied can be traded off against other design criteria during the optimization of the design if a preference function is used to express it as a "soft" constraint. Of course, "hard" constraints do not allow trade-off as they are either satisfied or not satisfied and accept no compromise. For example, a preference function similar to the one shown in Fig. 3.1(a) can be used to express a "soft" upper bound on a design parameter. If the designer also wishes to impose a lower bound on the parameter, then a two-sided version of the preference function can be used. For example, if one chooses column sizes as one of the design parameters, upper and lower limits might need to be imposed on them. This might be due to architectural requirements or simply because of limited availability of member sizes. A possible preference function for such a design criterion is given in Fig. 3.1(b). Various preference functions will be utilized in the examples later in this chapter (see Figs. 3.3 and 3.4).

One interpretation of a preference function is that it specifies the degree of satisfaction of a design criterion for each value of the design or performance parameters involved; an extreme value $\mu_i(\mathbf{q}(\boldsymbol{\theta})) = 1$, or $\mu_i(\mathbf{q}(\boldsymbol{\theta})) = 0$, implies that the current design specified by $\boldsymbol{\theta}$ perfectly satisfies, or does not satisfy at all, respectively, the i-th design criterion. $\mathbf{q}(\boldsymbol{\theta})$ denotes the vector of performance parameters associated with the design. Another interpretation is to view the preference function as a membership function for the fuzzy set of "acceptable performance" as judged by the i-th design criterion. In this case, an extreme value $\mu_i(\mathbf{q}(\boldsymbol{\theta})) = 1$, or $\mu_i(\mathbf{q}(\boldsymbol{\theta})) = 0$, implies that on the basis of the i-th design criterion, the current design specified by $\boldsymbol{\theta}$ is definitely acceptable, or definitely unacceptable, respectively. Intermediate values express the degree of performance acceptance given by the design. In this sense, preference functions act as the "weak ordering" functions (French 1988). There are other ways to specify the performance preferences. One example is the approach which uses functions that are complements of the preference functions defined above, namely "dissatisfaction functions" (Austin et al. 1985; Takewaki et al. 1991). As long as certain mathematical requirements for preference ordering and value functions are met, any expression of performance preferences could be used, and the choice becomes a question of convenience and ease of use. The mathematical properties that need to be satisfied to qualify as acceptable preference functions are rather involved, but can be broken into transitivity, negative transitivity, symmetry, antisymmetry, reflexivity, and comparability. The reader is referred to French (1988) for a detailed treatment of these requirements.

It should be noted that the fundamental assumption that allows specification of preference functions is that the decision-maker can assign preferences to the values of the corresponding performance parameters. Of course, this requires that the performance parameters be tangible, that is, that they could be measured or assigned a value on some scale. As a result, once the decision-maker breaks the complex performance objectives space into individual objectives, the objectives can be expressed as preferences over the corresponding performance parameter values.

The final step in the evaluation stage is to compute an overall design evaluation measure $\mu(\boldsymbol{\theta})$ on the basis of the quantitative evaluations, $\mu_i(\mathbf{q}(\boldsymbol{\theta}))$, $i = 1, \ldots, N_c$, of the design for each of the N_c design criteria. This is done by a preference aggregation rule, which must satisfy certain consistency requirements. In general, different aggregation rules give different design strategies for trading off the design criteria, and therefore lead to different optimal designs (see Rao (1984) for comparison of various approaches demonstrated using a simple design problem).

A preference aggregation rule is simply a functional relationship, f, between the individual preference values, $\mu_1, \mu_2, \ldots, \mu_{N_c}$ for all of the design criteria and the overall design evaluation measure, μ (Keeney and Raiffa 1976). An optimal design for a given preference aggregation rule is therefore given by a design parameter vector $\boldsymbol{\theta}$ that maximizes

$$\mu(\boldsymbol{\theta}) = f(\mu_1(\mathbf{q}(\boldsymbol{\theta})),\ \mu_2(\mathbf{q}(\boldsymbol{\theta})),\ \ldots,\ \mu_{N_c}(\mathbf{q}(\boldsymbol{\theta}))) \tag{3.1}$$

where it is understood that some of the preference functions μ_i may correspond to design parameter constraints and, therefore, depend directly on the design parameter values while the rest are functions of performance parameters which are not necessarily analytical functions of the design parameters but nevertheless can be computed, say, through structural response analyses.

By incorporating the design criteria (including design constraints) through the preference aggregation rule into the objective function (the overall design evaluation measure), which is to be maximized by varying the design parameters, the constrained optimization problem is converted into an unconstrained optimization problem.

Axioms of consistency imposed on the preference aggregation rule are (Otto 1992; Scott 1999):

1. The overall design evaluation measure μ lies in the unit interval $[0, 1]$, with $\mu = 1$ for a perfectly acceptable design and $\mu = 0$ for a completely unacceptable design. As mentioned above, this requirement is for the sake of consistency, and the value range could be rearranged.

2. Monotonicity and Continuity: μ is a monotonically increasing continuous function of each μ_i. The monotonicity ensures that if the design is changed to give higher preference for the i-th design criterion, while the preference values for the other design criteria remain unchanged, the overall preference for the design must increase (or possibly remain unchanged). Continuity ensures that a

small change in the preference for a design based on any of the design criteria produces only a corresponding small change in μ.

3. Symmetry: $\mu = f(\mu_1, \ldots, \mu_i, \mu_{i+1}, \ldots) = f(\mu_1, \ldots, \mu_{i+1}, \mu_i, \ldots)$, that is, the aggregation rule should be symmetric with respect to pairs of individual preference functions and their associated importance weights.

4. Idempotency: $\mu_0 = f(\mu_0, \mu_0, \ldots, \mu_0)$, that is, if the individual preferences for a design based on each criterion have the same value, then the overall preference for the design must have this value.

5. Annihilation: $\mu = 0$ if and only if $\mu_i = 0$ for some i. That is, a design is completely unacceptable in the overall sense if and only if it is completely unacceptable on the basis of at least one design criterion.

To be able to combine the multiple design criteria in a preferred manner to obtain a measure for the overall rating of the design, the designer must also specify importance weights, w_i, $i = 1, \ldots, N_c$ with $w_i \geq 0$, which indicate the relative importance of each of the design criteria. Thus, increasing the value of an importance weight for a design criterion gives it more influence in the trade-off that occurs between the various conflicting criteria during optimization of the design. This should hold true irrespective of the rule chosen to combine the multiple design criteria.

Depending on the choice of aggregation rules and the way importance weights are incorporated, there are further axioms that the aggregation rule should satisfy, such as the axiom of self-scaling weights. That is, if the importance weights of all preferences are scaled by the same factor, the resulting overall preference should not change, and the axiom of zero weights, which states that any performance criterion with zero importance weight has no effect on the overall preference value (Scott 1999).

For the optimal structural design, two preference aggregation rules satisfying these axioms have been investigated to combine the multiple performance objectives (Beck, Papadimitriou, Chan, and Irfanoglu 1997):

- Conservative ("weakest link," "non-compensating") strategy:

$$\mu = \min({\mu_1}^{n_1},\ {\mu_2}^{n_2},\ \ldots,\ {\mu_{N_c}}^{n_{N_c}}) \tag{3.2}$$

  where $n_i = w_i/w_{max}$, $i = 1,\ \ldots,\ N_c$, w_i is a positive importance weight assigned to the i-th design criterion and w_{max} is the maximum of w_i over $i = 1, \ldots,\ N_c$.

- Multiplicative trade-off strategy:

$$\mu = {\mu_1}^{m_1}{\mu_2}^{m_2}\ldots{\mu_{N_c}}^{m_{N_c}} \tag{3.3}$$

  where $m_i = w_i/\sum_{j=1}^{N_c} w_j$, $i = 1,\ \ldots,\ N_c$, and w_i is a positive importance weight assigned to the i-th design criterion.

As already described, the importance weight assigned to each design criterion can be used to control its trade-off relative to the other criteria. That is, selected design criteria can be given more influence than others during optimization by assigning larger values to their importance weights. The choice of the values for these weights is subjective. The designer/decision-maker is presumed to develop insight with respect to their selection in any design problem by investigating the influence of different values of the weights on the final optimal design and on the corresponding preference values for each design criterion. For example, if the designer wishes to perform an "aggressive" code-based design that approaches close to the code drift limit (Fig. 3.1(a)), the importance weight for a building cost criterion, which will conflict with the code drift criterion, should be made much larger than the importance weights for the other design criteria. This will give greater emphasis on reducing costs during the trade-off in the optimization at the expense of giving a design that is much closer to the code drift limit. However, this interaction depends very much on the corresponding preference functions and the aggregation rule used to combine them.

The importance weights w_i can be viewed from another perspective. Since there is no natural scale for preferences over all the diverse design criteria, there is a need to be able to independently control their influence during the trade-off that occurs in the optimization process. In the case that the w_i are all equal, the trade-off is governed by the inherent sensitivity of each μ_i with respect to $\boldsymbol{\theta}$. This "natural" trade-off may not satisfy the designer, who may want to give greater influence to selected criteria. In this case, an importance weight, say w_j, can be increased, then the sensitivity of μ_j with respect to $\boldsymbol{\theta}$ will be increased, which will give the j-th criterion more influence during the optimization. As an aside, it should be noted that a sensitivity study of the decision outcome in relation to individual criterion is the primary tool in reducing the dimension of the relevant performance parameter space. Those parameters to which the design is not sensitive may be taken out of the performance parameter list without compromising the final decision outcome. However, some of the parameters might require monitoring as they might be associated with some constraints that need to be satisfied but are expressed in the form of "hard" constraints and therefore might be mistaken as being irrelevant.

Unfortunately, in most practical problems, a sensitivity study can be done only numerically or empirically even though the underlying concept is simple. For a somewhat different but relevant and important stage of the design decision-making process, namely, the "negotiation" stage, there have been some attempts to analytically study the implications of sensitivities (Scott 1999). The relevance of "negotiation" to the structural design process should be clear when one realizes that the decision-makers in structural design do not necessarily have a unique common interest or employ similar approaches in evaluating the design. This is especially true in the multiple criteria based approaches where the criteria and preferences need to be stated explicitly.

Returning to the two aggregation rules considered, that is, the conservative strategy and the multiplicative trade-off strategy, difficulties were encountered in the implementation of the former strategy during the numerical optimization (Beck et al. 1997). On the other hand, the multiplicative trade-off strategy was found not only to allow a flexible environment to trade-off conflicting criteria, but also to have favorable properties from the numerical optimization point of view (Beck, Papadimitriou, Chan, and Irfanoglu 1997). Therefore, the multiplicative trade-off strategy given in

Eqn. (3.3) is used in the examples presented later in this chapter.

Before illustrating the methodology with examples, it should be noted that special cases of the optimal design methodology based on the multiplicative trade-off strategy can be related to existing optimal design concepts. For example, it is easily shown that the optimal solution obtained by maximizing Eqn. (3.3) belongs to the Pareto optimal set corresponding to the multiple "objectives" $\mu_1, \ldots, \mu_{N_c}$ (Chan 1997). But in contrast to an approach based on the full Pareto optimal set, which is not feasible for most practical problems due to the size of the set, the current approach converges to a Pareto optimal design directly. Often, finding one such optimal design is enough.

3.3 Illustrative Design Example: Three-Story SMRF, Case 1

3.3.1 Structural Model, Design Criteria, and Seismic Environment Model

The optimal multi-criteria design methodology using the multiplicative trade-off strategy is demonstrated by applying it to the design of a three-story, single-bay moment-resisting steel frame. The frame members are all taken as I-sections made out of ASTM A36 ($F_y = 36\ ksi$) steel with the length of the floor beams fixed at 20 ft and the height of the story columns fixed at 10 ft. The connections are modeled as being rigid and the steel sections are used in their strong axes. Gravity loads are taken as the sum of 60 lb/ft^2 and 50 lb/ft^2, from the dead and live loads, respectively, for each floor and the roof. An out-of-plane tributary width of 100 inches is used for the gravity load calculations. The gravity loads are assumed to participate in the lateral seismic loading in full.

Both continuous and discrete cases of the design parameter space are considered. The design parameters $\boldsymbol{\theta}$ in the continuous case are member flange width B and web depth D for the beams and columns. Two design parameter sets are considered: (a)

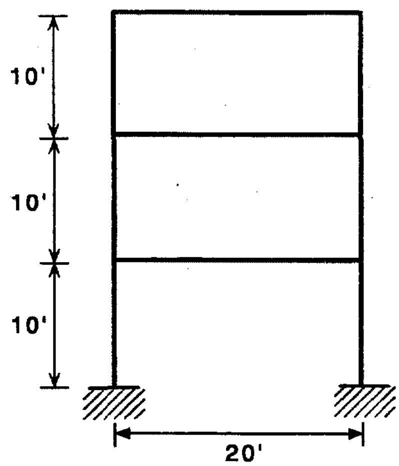

Figure 3.2: Example Three-Story Steel MRF

$\boldsymbol{\theta} = (B,\ D)$, where the beams and columns are required to have the same cross-sectional dimensions, and (b) $\boldsymbol{\theta} = (B_{beam},\ D_{beam},\ B_{col},\ D_{col})$, where the beams and columns are allowed to have different cross-sectional dimensions. The flange and web plate thicknesses are held fixed at 0.25 inches. In the continuous case, the adaptive random search algorithm (Masri et al. 1980) is used to obtain the optimal design. The simplicity of the designs allowed search over a small set of relevant AISC W-shapes (AISC 1989) for discrete optimization and an exhaustive search is performed.

The objective is to determine the best combination of values for the design parameters $\boldsymbol{\theta}$ so that the frame design is optimized according to design criteria involving the following performance parameters: (a) flange width, (b) web depth, (c) building cost, (d) probability of unacceptable peak lifetime interstory drift (drift risk), and (e) code-based maximum interstory drift and (e) maximum allowable stresses in structural members (see Fig. 3.3). The importance weight for each design criterion is set to 1.0 for the aggregation of preference values in equation Eqn. (3.3), unless otherwise stated. The corresponding preference functions are shown in Fig. 3.3.

The first two design criteria shown in Fig. 3.3(a) and (b) involve "soft" constraints on the design parameters. These preference functions define constraints on cross-sectional dimensions of the structural members. Specifically, the forms of the

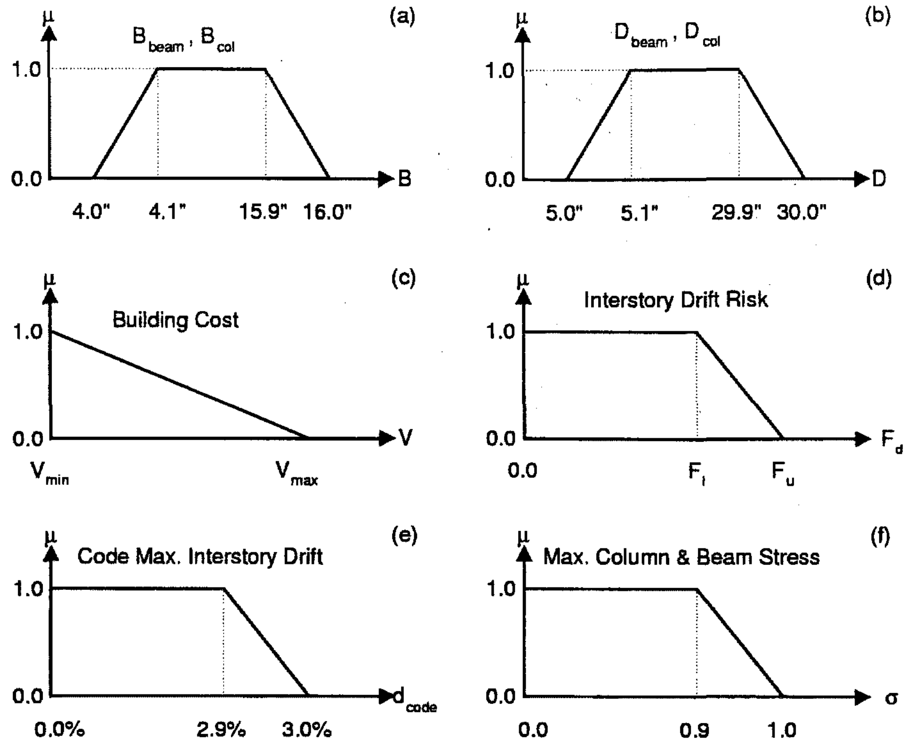

Figure 3.3: Preference Functions for Different Performance Parameters

preference functions show that, for example, in the case of flange width B, members with flange widths shorter than 4.0 inches or longer than 16.0 inches are unacceptable, and therefore are assigned μ values of 0, while those with flange widths between 4.1 to 15.9 inches are favored the most, and therefore are assigned μ values of 1. Flange widths between 4 to 4.1 inches and 15.9 to 16.0 inches have various intermediate degrees of acceptance or satisfaction. One can interpret the preference function for the web depth D of the structural members in a similar way. In real-life applications, the sizes defining the shape of the preference functions related with the cross-sectional dimensions of structural members might be dictated, for example, by the limited availability of structural elements in stock or by architectural restrictions imposed on the structure. It should be noted that, since there is no "uncertainty" about the member sizes once they are chosen, the preference values for them can be computed and directly incorporated in the multiplicative trade-off preference aggregation rule given in Eqn. (3.3).

The preference function for the building cost is given in Fig. 3.3(c). For this example, the building cost C is expressed simply as the sum of a construction (or fabrication) cost C_{con} and a material cost

$$C = C_{con} + c_s V \tag{3.4}$$

where c_s is the material cost per unit steel volume and V is the volume of steel used in the design. The variation in the construction costs for structural members of different sizes is assumed negligibly small, so that C_{con} is taken to be essentially independent of $\boldsymbol{\theta}$. The preference function can then be expressed in terms of a normalized performance parameter

$$q_{cost} = (C - C_{min}) \,/\, (C_{max} - C_{min}) = (V - V_{min}) \,/\, (V_{max} - V_{min}) \tag{3.5}$$

where V_{max}=22,140 in^3 and V_{min}=4,500 in^3 are the steel volumes corresponding to use of the maximum and minimum allowable member section sizes prescribed by the geometric constraints. The preference function for the building cost can therefore be expressed in terms of the steel volume $V(\boldsymbol{\theta})$ for a design given by $\boldsymbol{\theta}$. As shown in Fig. 3.3(c), a linearly decreasing function is used to specify the preference values for the building cost in terms of the steel volume, with $\mu = 1$ at the minimum allowable volume and $\mu = 0$ at the maximum allowable volume. In the tables of results presented later, the building cost is reported as the volume of steel, V.

Three modes of "failure" are considered for the example building in the current study. They are related to lifetime interstory drift risk, code-based maximum allowable interstory drift (ICBO 1994), and code-based maximum allowable column and beam stresses (AISC 1989). The corresponding preference functions are shown in Fig. 3.3(d), (e) and (f), respectively. The code-based maximum allowable interstory drift is calculated using the Uniform Building Code (UBC) response spectrum (ICBO

1994) and employing standard fundamental-mode modal analysis. As it can be seen from Fig. 3.3(e), a computed interstory drift ratio lower than 2.9% is considered perfectly acceptable, while one higher than 3% is considered completely unacceptable. In Fig. 3.3(f), the preference functions for maximum allowable column and beam normalized stresses are given. The performance parameter σ in this figure is the ratio of the maximum induced stress to the allowable stress specified in the AISC Manual of Steel Construction (AISC 1989). Stress calculations are carried out considering the end-of-element stresses resulting from combined axial and bending loads due to full gravity and lateral loadings as given in AISC Allowable Stress Design Manual (AISC 1989). Resulting stresses less than 90% of the code-allowable values are considered perfectly acceptable while those greater than the code-allowables are considered completely unacceptable. It should be noted that in practice other stress checks may need to be performed but these are in this simple illustrative example.

The difference between lifetime interstory drift risk and code-based interstory drift is that the former one gives the failure probability of the structure by considering the uncertainties in future loadings using a site-specific seismic environment explicitly, while the code-based calculations consider the deterministic response spectrum specified in the code. The explicit consideration of the failure probability is of great importance in the design process since it provides flexibility in specifying preferences on the reliability of the structure. However, the code-based requirements can be explicitly included in the design criteria to ensure that the legal requirements are satisfied by the resulting design.

Unacceptable drift performance or "failure" occurs if the maximum interstory drift ratio $d_{\max}$ exceeds a specified allowable drift ratio $d_{allow} = 3\%$ over the lifetime of the structure. The performance parameter is taken as the interstory drift risk, F_d, which is simply equal to the probability of exceeding d_{allow} over the lifetime of the structure. As shown in Fig. 3.3(d), the interstory drift risk F_d is required to be less than a limit value F_u, with greatest preference $\mu = 1$ given to risks (failure probabilities) which are less than a value F_l. In the numerical results, two cases are considered in order to examine their effects on the optimal design: $F_l = 5\%$, $F_u = 10\%$ (the 5% risk case) and $F_l = 1\%$, $F_u = 2\%$ (the 1% risk case). The risk F_d is computed using a probabilistic seismic hazard model and probabilistic structural analysis tools, as described in Section 2.2. For simplicity, a fundamental-mode based linear pseudo-dynamic analysis is used to compute the deformations approximately even though for large drifts the structural response would involve inelastic behavior.

The code criteria, when included, are based on the UBC requirements for seismic zone 4 (effective peak ground acceleration is 0.4 g) and for a structure with reduction factor R_w=12. The requirements on the maximum column and beam stresses are computed under the reduced (by R_w) code forces using the response spectrum given in UBC (ICBO 1994). The maximum interstory drift ratio d_{code} must be less than 3% under forces specified by the code response spectrum with no reduction by R_w.

Based on the seismic hazard model considered in Section 2.2.1, the earthquake distance R and the earthquake magnitude M are treated as the only uncertain seismicity parameters. The source region regarding the distribution of R is assumed to be such that an earthquake is equally likely to occur at any point inside a circular source region centered at the site where the building is located and with a radius $R_{max} = 50$ Km. For the distribution of the earthquake magnitudes, M, the truncated Gutenberg-Richter relationship is used with the parameters $M_{min} = 5.0$, $M_{max} = 7.7$, $b = 1.0$ and a default value of $a = 5.0$, implying a seismicity rate of $\nu = 1$ event per annum in the magnitude range of interest. In the study of the effect of the seismicity rate on the optimal design, the value of a is set to 4.7 and 5.3 to have a seismicity rate of $\nu = 0.5$ and $\nu = 2$ event per annum, respectively.

3.3.2 Numerical Results

For the numerical study, the analysis procedure described in Chapter 2 along with the evaluation procedure explained in Section 3.2 are used. No modeling uncertainty is taken into consideration in the structural response analysis, and the trapezoidal numerical integration method is used for the reliability calculations. The modal damping ratio is chosen to be $\zeta_1 = 0.05$ for the fundamental mode. A lifetime of t_{life}

= 50 years is assumed to allow direct comparison of code-specified and reliability-based performance objectives. Optimization over the design parameters is performed using the adaptive random search algorithm (Masri et al. 1980).

The results in Table 3.1 correspond to a seismicity rate $\nu = 1$ event per annum and the case where beams and columns have the same cross-sectional dimensions, that is, the design parameters are $B_{beam} = B_{col} = B$, $D_{beam} = D_{col} = D$, and so $\boldsymbol{\theta} = (B,\ D)$. Results are shown for both the 5% and 1% risk cases described earlier. As expected, the optimal design for 1% drift risk gives larger member sizes than the 5% risk case does. The code-based drift and strength requirements (ICBO 1994; AISC 1989) are included, but the failure modes related to them do not control the designs here.

Note that in the results, the optimal flange width B is always 4.10 in, which corresponds to the lower corner of the preference function for B shown in Fig. 3.3(a). This occurs because, when the steel plate thickness is fixed, it is more cost-effective to provide the necessary bending stiffness by increasing the web depth D rather than the flange width B. However, if B is reduced below 4.10 in, the rate of reduction in the preference in Fig. 3.3(a) outweighs the improvement in the cost preference in Fig. 3.3(c).

In Table 3.1, $F_{d,i}$, $i = 1, 2, 3$, denote the drift risk for the i-th story over the lifetime t_{life} of the structure. It was found that the interstory drift risk $F_{d,2}$ for the second story governs the design for this example problem. This is because of the rotational constraints at the base of the first story columns. Specifically, it was found that the failure intervals defined by the failure points $g_1(S_V) = 0$ and $g_3(S_V) = 0$ for the first and third stories, respectively, are subsets of the failure interval defined by the dominant failure point $g_2(S_V) = 0$. The notation for the definition of a failure point is taken from classical reliability theory, and for the current case, $g_i(S_V) = 0$ defines the boundary at which $g_i(S_V) = d_{allow} - d_i = 0$ and where $g_i(S_V) < 0$ implies failure for the i-th story (see Section 2.2.3). The dominance of the failure region by the second story's failure mode is because of the fact that the critical value of the fundamental mode pseudo-velocity spectrum at the limit-state is given by S_V, which satisfies, from Section 2.2.2, the relation $d_{max} = 3\% = d_{i,1} - d_{i-1,1} = (\Gamma_1/w_1)(\phi_{i,1} -$

Table 3.1: Optimal Design Values and Their Preferences for $\nu = 1$ event/yr; Case $\boldsymbol{\theta} = (B,\ D)$

5% Risk				
	Continuous Opt.		Discrete Opt.	
Criteria	Value	μ	Value	μ
B (in)	4.10	1.0000	W8x18	1.0000
D (in)	9.67	1.0000		1.0000
Vol (in^3)	6253	0.9007	7574	0.8257
σ^b_{max}	0.2778	1.0000	0.2335	1.0000
σ^c_{max}	0.4506	1.0000	0.3771	1.0000
d_{code}	0.0288	1.0000	0.0287	1.0000
$F_{d,1}$	0.0162	-	0.0160	-
$F_{d,2}$	0.0500	-	0.0494	-
$F_{d,3}$	0.0178	-	0.0175	-
F_d	0.0500	1.0000	0.0494	1.0000
Overall		**0.9852**		**0.9730**
Period	T_1=1.115 s		T_1=1.111 s	

1% Risk				
	Continuous Opt.		Discrete Opt.	
Criteria	Value	μ	Value	μ
B (in)	4.10	1.0000	W14x22	1.0000
D (in)	13.85	1.0000		1.0000
Vol (in^3)	7759	0.8153	9346	0.7253
σ^b_{max}	0.2586	1.0000	0.2180	1.0000
σ^c_{max}	0.3768	1.0000	0.3163	1.0000
d_{code}	0.0188	1.0000	0.0161	1.0000
$F_{d,1}$	0.0025	-	0.0009	-
$F_{d,2}$	0.0100	-	0.0045	-
$F_{d,3}$	0.0029	-	0.0011	-
F_d	0.0100	1.0000	0.0063	1.0000
Overall		**0.9712**		**0.9552**
Period	T_1=0.730 s		T_1=0.623 s	

$\phi_{i-1,1})S_V(w_1,\ \zeta_1)$. The critical value of S_V is a minimum for the second story and therefore the probability of failure is higher for the second story.

As noted earlier, the code requirements (ICBO 1994; AISC 1989) reflected in Fig. 3.3(e) and (f) are also included as design criteria to allow comparison. The values of the code specified performance parameters, that is, maximum column stress, maximum beam stress, and maximum interstory drift, in Table 3.3 are computed using the pseudo-dynamic lateral-load calculation procedure, which is based on the response spectra described in the 1994 UBC (ICBO 1994). It has been found that the drift reliability requirement is more stringent than what the UBC demands, and therefore, the UBC requirements have no influence on the final designs. This can be seen from the fact that the code stress ratios for the beams and columns, σ^b_{max} and σ^c_{max} respectively, are less than 0.9, and the code interstory drift d_{code} is lower than 2.9% (see Fig. 3.3(e) and (f)). Therefore, from the code point-of-view, the obtained designs are fully satisfactory. For comparison, a purely code-based design optimization is also performed. The resulting design, which meets a code-based drift requirement of 3% under the code-specified response spectrum, is found to require a steel volume of 6,133 in^3 (corresponding to 0.25 in thick members with 4.10 in flange widths and 9.34 in web depths; fundamental-mode period is 1.162 s). This structure, when studied in the current specified seismic environment, is found to have a 5.7% risk of exceeding 3% peak drift-ratio over the lifetime of 50 years.

In Table 3.2, the case of four design parameters, $\boldsymbol{\theta} = (B_{beam},\ D_{beam},\ B_{col},\ D_{col})$, is presented. In this case, beam and column cross-sectional dimensions are allowed to be different but all beams must have the same cross-section and so do all columns. Comparing the building costs (steel volumes) in Tables 3.1 and 3.2, it is observed that by treating the sizes of beams and columns independently, the optimal designs are slightly less costly, as expected. However, in both cases, the dynamics of the resulting optimal structures are similar as illustrated by the similar fundamental periods in Tables 3.1 and 3.2.

The last columns of Tables 3.1 and 3.2 give the optimal designs over a set of AISC W-shape steel sections. The simplicity of the designs allowed the use of a small set

Table 3.2: Optimal Design Values and Their Preferences for $\nu = 1$ event/yr; Case $\boldsymbol{\theta} = (B_{beam},\ D_{beam},\ B_{col},\ D_{col})$

5% Risk				
	Continuous Opt.		Discrete Opt.	
Criteria	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	W8x18	1.0000
D_{beam} (in)	11.01	1.0000		1.0000
B_{col} (in)	4.10	1.0000	W8x18	1.0000
D_{col} (in)	7.97	1.0000		1.0000
Vol (in^3)	6188	0.9043	7574	0.8257
σ^b_{max}	0.2683	1.0000	0.2335	1.0000
σ^c_{max}	0.5873	1.0000	0.3771	1.0000
d_{code}	0.0287	1.0000	0.0287	1.0000
$F_{d,1}$	0.0258	-	0.0160	-
$F_{d,2}$	0.0500	-	0.0494	-
$F_{d,3}$	0.0126	-	0.0175	-
F_d	0.0500	1.0000	0.0494	1.0000
Overall		**0.9889**		**0.9789**
Period	T_1=1.140 s		T_1=1.111 s	

1% Risk				
	Continuous Opt.		Discrete Opt.	
Criteria	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	W14x22	1.0000
D_{beam} (in)	15.54	1.0000		1.0000
B_{col} (in)	4.10	1.0000	W14x22	1.0000
D_{col} (in)	11.71	1.0000		1.0000
Vol (in^3)	7677	0.8199	9346	0.7253
σ^b_{max}	0.2510	1.0000	0.2180	1.0000
σ^c_{max}	0.4877	1.0000	0.3163	1.0000
d_{code}	0.0188	1.0000	0.0161	1.0000
$F_{d,1}$	0.0041	-	0.0009	-
$F_{d,2}$	0.0100	-	0.0045	-
$F_{d,3}$	0.0020	-	0.0011	-
F_d	0.0100	1.0000	0.0063	1.0000
Overall		**0.9781**		**0.9649**
Period	T_1=0.742 s		T_1=0.623 s	

of sections with properties similar to those of the results found from the continuous search, so an exhaustive search could be made over the discrete set. In the discrete optimization case, an increase in building cost (corresponding to 20% or so increase in steel volume) occurs compared with the continuous optimization case. This is due to the limited variety of steel-section sizes in the discrete case and the fact that the section combinations cannot come as close to the best compromise design as the section combinations over the continuous set.

In Table 3.3, the effect of the regional seismicity rate on the optimal design is investigated. Results are presented for the three seismicity rates corresponding to $\nu = 0.5$, 1 and 2 events per annum and for 5% and 1% drift risk cases. The seismicity rates are changed by modifying the parameter a in the truncated Gutenberg-Richter relationship that scales the number of events over the considered magnitude range (see Section 2.2.1). However, the ratio of occurrence rates of earthquakes with different magnitudes is unaltered (controlled by the value of b). As expected, higher seismicity or lower risk requirements lead to larger structural members.

In all but the 5% risk for $\nu = 0.5$ event/yr presented in Table 3.3, all of the considered code requirements are perfectly satisfied. This can be seen by observing that the preference function values corresponding to the code-based design criteria are all equal to 1. For the exceptional case, the seismic hazard from the specified environment is much lower than what is implied by the UBC response spectrum. Therefore, for this case, the code-based drift requirement controls the design.

In Table 3.4, the effect on reliability-based optimal designs of increasing the importance weight w_{Vol} for the building cost criterion is illustrated for the 5% and 1% risk cases (the code-based design criteria are not included). At first, as w_{Vol} increases from 1 to 10, the drift risk F_d=1% continues to control the optimal design and the optimal design parameter values are unchanged. However, when w_{Vol} is increased to 50 or 100, the cost criterion becomes influential in the trade-off and so, a more "aggressive" design with lower cost (or steel volume, since it is used as a proxy for cost) but higher risk, F_d, is produced.

As seen from the results, the methodology allows various design criteria along

Table 3.3: Optimal Design for ν = 0.5, 1, 2 events/yr; Case $\boldsymbol{\theta}$ = $(B_{beam}, D_{beam}, B_{col}, D_{col})$

5% Risk						
	$\nu = 0.5$		$\nu = 1$		$\nu = 2$	
Criteria	Value	μ	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{beam} (in)	10.93	1.0000	11.01	1.0000	13.00	1.0000
B_{col} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{col} (in)	7.88	1.0000	7.97	1.0000	9.43	1.0000
Vol (in^3)	6157	0.9061	6188	0.9043	6809	0.8691
σ^b_{max}	0.2686	1.0000	0.2683	1.0000	0.2595	1.0000
σ^c_{max}	0.5909	1.0000	0.5873	1.0000	0.5460	1.0000
d_{code}	0.0290	1.0000	0.0287	1.0000	0.0236	1.0000
$F_{d,1}$	0.0135	-	0.0258	-	0.0245	-
$F_{d,2}$	0.0262	-	0.0500	-	0.0500	-
$F_{d,3}$	0.0066	-	0.0126	-	0.0115	-
F_d	0.0262	1.0000	0.0500	1.0000	0.0500	1.0000
Overall		**0.9891**		**0.9889**		**0.9845**
Period	T_1=1.152 s		T_1=1.140 s		T_1=0.937 s	

1% Risk						
	$\nu = 0.5$		$\nu = 1$		$\nu = 2$	
Criteria	Value	μ	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{beam} (in)	13.56	1.0000	15.54	1.0000	17.43	1.0000
B_{col} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{col} (in)	10.12	1.0000	11.71	1.0000	13.25	1.0000
Vol (in^3)	7034	0.8769	7677	0.8199	8295	0.7848
σ^b_{max}	0.2585	1.0000	0.2510	1.0000	0.2444	1.0000
σ^c_{max}	0.5219	1.0000	0.4877	1.0000	0.4599	1.0000
d_{code}	0.0221	1.0000	0.0188	1.0000	0.0163	1.0000
$F_{d,1}$	0.0045	-	0.0041	-	0.0037	-
$F_{d,2}$	0.0100	-	0.0100	-	0.0100	-
$F_{d,3}$	0.0022	-	0.0020	-	0.0017	-
F_d	0.0100	1.0000	0.0100	1.0000	0.0100	1.0000
Overall		**0.9855**		**0.9782**		**0.9734**
Period	T_1=0.877 s		T_1=0.742 s		T_1=0.644 s	

Table 3.4: Optimal Design for $\nu = 1$ event/yr; Case $\boldsymbol{\theta} = (B,\ D)$

1% Risk								
	w_{Vol}=1		w_{Vol}=10		w_{Vol}=50		w_{Vol}=100	
Criteria	Value	μ	Value	μ	Value	μ	Value	μ
B (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000	4.10	1.0000
D (in)	13.85	1.0000	13.88	1.0000	12.76	1.0000	12.42	1.0000
Vol (in3)	7759	0.8153	7759	0.8153	7364	0.8376	7245	0.8444
$F_{d,1}$	0.0025	-	0.0025	-	0.0041	-	0.0048	-
$F_{d,2}$	0.0100	-	0.0100	-	0.0152	-	0.0173	-
$F_{d,3}$	0.0029	-	0.0029	-	0.0047	-	0.0054	-
F_d	0.0100	1.0000	0.0100	1.0000	0.0152	0.4763	0.0173	0.2717
Overall		**0.9502**		**0.8546**		**0.8343**		**0.8375**
Period	T_1=0.730 s		T_1=0.730 s		T_1=0.805 s		T_1=0.831 s	

with code-based minimum requirements to be included in the design process. The results allow comparison of the effect of individual criterion as well as the effect of various ingredients of the design process, such as the choice of the design parameters, the nature of the seismic hazard, and the designer's preferences.

3.4 Illustrative Design Example: Three-Story SMRF, Case 2

In performance-based structural design, one tries to design a structure to satisfy multiple performance levels under various excitations that it might experience during its lifetime. This design concept is at the heart of all structural design, but until very recent times, some of the performance levels corresponding to different limit-states were not quantitatively specified and were not explicitly taken into consideration. Due to the unsatisfactory economic performance of engineered structures at lower level limit-states during recent earthquakes, there is greater demand for multi-level performance-based design.

Vision 2000, published by the Structural Engineers Association of California in 1995, is one of the key documents that presented the performance-based structural design concept in structural engineering terms. It includes a collection of suggested qualitative performance objectives as well as quantified performance levels in terms of performance criteria. The developed methodology is illustrated below using some of these recommended quantified performance criteria.

3.4.1 Structural Model and Design Criteria

To demonstrate the application of the methodology to performance-based design and how it can be utilized in other ways, the same structure considered in Case 1 is designed using the performance levels given in *Vision 2000* (SEAOC 1995). The same dead load and live load values as in Case 1 are considered for the gravity loads. Similarly, a probabilistic structural analysis based on the response spectrum approach is used in estimating the performance of the structure during its lifetime of 50 years. The uncertain seismic environment is the same as the one considered in Case 1. In other words, earthquakes within a maximum distance of $R_{max} = 50$ Km, and magnitude range [5.0, 7.7] are considered. The associated probability density functions are as given in Section 2.2.1. The seismicity rate is $\nu = 1$ event per annum.

The preference functions for the selected design criteria are shown in Fig. 3.4. The importance weight for each design criterion is set to 1.0 in the aggregation of preference values through the multiplicative trade-off strategy, Eqn. (3.3).

In *Vision 2000*, the expected performance levels for different earthquake design levels vary with the importance of the structure. For the current example, the structure is assumed to be required to follow the "basic objective" requirements of the *Vision 2000*: to be "fully operational" after "frequent" earthquakes; to be "operational" after "occasional" earthquakes; to be "life-safe" after "rare" earthquakes; and to be "near collapse" after "very rare" earthquakes. The descriptions of the four performance levels together with corresponding interstory drift ratios, and the four earthquake design levels are given in Tables 3.5 and 3.6, respectively.

In *Vision 2000*, the qualitative descriptions of Table 3.5 are related to various quantitative response measures and the range of values these measures could take.

Table 3.5: Performance Levels of *Vision 2000* (SEAOC 1995)

Performance Level	*Description*
Fully Operational ($d_{max} < 0.2\%$)	No significant damage in structural and non-structural components. The building can be occupied and is available for its intended use.
Operational ($d_{max} < 0.5\%$)	Light damage in structural elements. Moderate damage in non-structural elements. The building can be occupied for its intended use but some functions may be disrupted
Life-Safe ($d_{max} < 1.5\%$)	Moderate damage in structural and non-structural elements. Loss in lateral stiffness and reduced ability to resist additional lateral loads. The building cannot be occupied, but may be repaired.
Near Collapse ($d_{max} < 2.5\%$)	Extreme damage state. The building cannot be occupied, and it is unlikely to be repaired.

Table 3.6: Earthquake Design Levels of *Vision 2000* (SEAOC 1995)

Earthquake Design Level	*Recurr. Interval*	*Prob. of Exceed.*	*Prob. of Exceed. in* t_{life}
Frequent	43 years	50% in 30 years	68.5% in 50 years
Occasional	72 years	50% in 50 years	50% in 50 years
Rare	475 years	10% in 50 years	10% in 50 years
Very Rare	970 years	10% in 100 years	5.13% in 50 years

For the current study, only the corresponding interstory drift ratio is taken into consideration. In Table 3.6, the earthquake design level exceedance probabilities are given. The third column in Table 3.6 lists the probability of exceedance and time-duration pairs as given in *Vision 2000.* The last column lists the corresponding probabilites normalized for the lifetime of 50 years of the example structure. This normalization can be made using the given recurrence intervals, together with the assumption that the earthquake occurrences follow a Poisson arrival process as implied in *Vision 2000.*

It should be noted that, in *Vision 2000*, the reliabilities, or the exceedance probabilities, are given on the earthquake design level rather than on the performance level, that is, they are specified at the seismic hazard level rather than the seismic risk level. It seems more appropriate to the philosophy of performance-based design to specify the reliabilities (or the failure probabilities) on the performance levels since one has no control on the seismic environment, hence on the level of earthquakes. The concept of performance-based design is based on meeting various levels of performance "expected" from the structure. For example, it is desirable to keep the specified performance levels for similar structures (e.g. structures needed during emergency response) to be the same no matter what the seismic environment is. It is better to focus on directly controlling the seismic risk rather than trying to control it indirectly by specifying probabilities on the seismic hazard levels. Besides, in doing so, one avoids the often confusing and subjective issue of specifying levels for earthquakes.

A possible shortcoming of specifying reliability at the hazard level, that is, obtaining a uniform hazard spectrum and designing a structure based on this particular spectrum, is the difficulty in incorporating structural modeling and response estimation uncertainties into the design process. Especially if these uncertainties vary with the intensity of the seismic loading experienced by the structure, one needs to take the full hazard range with associated probabilities, not just a particular one corresponding to a specified level.

In the current example, since only a fundamental-mode response analysis is performed with no consideration of modeling or response estimation uncertainties, there is no distinction between specifying the reliability at the seismic hazard level or at the seismic risk level. A comparison of designs obtained using the two distinct approaches will be given to illustrate this point. However, further investigation of the consequences of the two design philosophies is most desirable.

3.4.2 Numerical Results

In the second example, for the performance objectives, the following pair of interstory drift ratio and corresponding failure probability F_d (exceedance probability, or risk) over the lifetime t_{life}=50 years, are used (see Tables 3.5, 3.6) : 0.2% with 68.5% risk, 0.5% with 50% risk, 1.5% with 10% risk, and 2.5% with 5.13% risk. The risk levels correspond to F_l in Fig. 3.4(d). To simulate the "hard" boundaries implied by the individual performance objectives, the upper risk value, at and beyond which the design would be unacceptable, is chosen to be $1.01F_l$. F_d is computed for each associated interstory drift ratio.

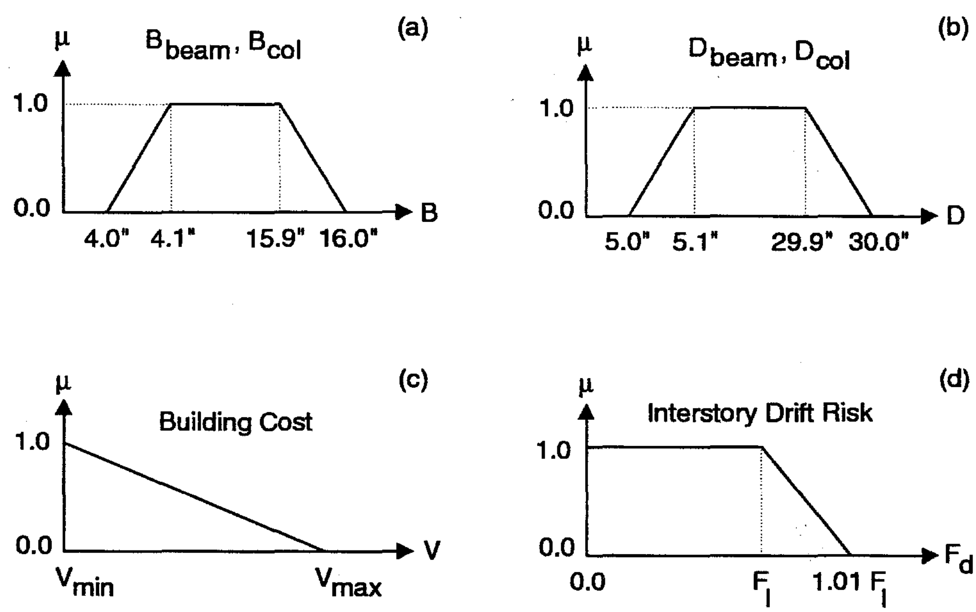

Figure 3.4: Preference Functions for Different Performance Parameters

The chosen design parameters are the member flange width B and web depth D for the beams and columns, that is, $\boldsymbol{\theta} = (B_{beam}, D_{beam}, B_{col}, D_{col})$. As before, the flange and web plate thicknesses are held fixed at 0.25 inches. Continuous optimization over the dimension ranges shown in Fig. 3.4 is performed. The code (ICBO 1994; AISC 1989) requirements on interstory drift ratios and maximum beam and column stresses are not explicitly included in the optimization. However, the resulting designs are found to satisfy these code requirements. No modeling uncertainty is considered in the structural response analysis.

In Table 3.7, the resulting optimum designs for each of the four performance lev-

Table 3.7: Optimal Design for $\nu = 1$ event/yr; Case $\boldsymbol{\theta} = (B_{beam},\ D_{beam},\ B_{col},\ D_{col})$

	Freq./Fully Oper.		Occas./Oper.		Rare/Life-Safe		V. Rare/Coll.(2.5%)	
Criteria	Value	μ	Value	μ	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{beam} (in)	29.90	1.0000	20.38	1.0000	15.09	1.0000	12.44	1.0000
B_{col} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{col} (in)	26.44	1.0000	15.23	1.0000	11.20	1.0000	9.09	1.0000
Vol (in^3)	12913	0.5231	9182	0.7346	7503	0.8298	6648	0.8782
$F_{d,1}$	0.3561	-	0.3010	-	0.0503	-	0.0258	-
$F_{d,2}$	0.6850	-	0.5000	-	0.1000	-	0.0513	-
$F_{d,3}$	0.3402	-	0.1877	-	0.0267	-	0.0129	-
F_d	0.6850	1.0000	0.5000	1.0000	0.1000	1.0000	0.0513	1.0000
Overall		**0.8976**		**0.9499**		**0.9694**		**0.9786**
Period	T_1=0.306 s		T_1=0.539 s		T_1=0.776 s		T_1=0.982 s	

Table 3.8: Lifetime Reliabilities for the Four Performance Levels for the Four Optimal Designs Satisfying Performance Objectives (Tables 3.5 and 3.6)

Design Performance Objective	Performance Level Assessment			
	Fully Oper.	Oper.	Life-Safe	Near Coll.
Frequent—Fully Oper. (0.2%, 31.5%)	**0.3150**	0.9378	0.9996	1.0000
Occasional—Operational (0.5%, 50%)	0.0046	**0.5000**	0.9752	0.9963
Rare—Life-Safe (1.5%, 90%)	0.0001	0.1839	**0.9000**	0.9781
Very Rare—Near Coll. (2.5%, 94.87%)	0.0000	0.0679	0.8046	**0.9487**

els are given. It should be noted that, in order to be able to see the effect on the optimal design of each performance level, the levels are considered one at a time. The case of being "fully operational after frequent earthquakes" required the largest structural member sizes. The second most demanding performance level was "operational after occasional earthquakes," then, "life-safe after rare earthquakes." The last performance level, "near-collapse after very rare earthquakes" required the smallest member sizes.

In Table 3.8, the reliabilities corresponding to different performance levels for each of the four optimal designs are compared. For example, in the first row, results for the optimal design for "fully operational after a frequent event" are presented, showing that for this design, the reliabilities for the other three performance levels are exceedingly high, that is, the corresponding performance objectives are met by a very high margin. However, it must be recalled that a linear structural model is used in obtaining the results. Table 3.8 also shows that an optimal design that is based on the criterion of being "operational after an occasional event" does not meet the "fully operational after a frequent event" criterion. Similarly, using "life-safe after a rare event" for the optimal design does not meet the "fully operational after a frequent event" and the "operational after an occasional event" criteria. An optimal design which is based on the "near collapse after a very rare event" objective does not meet any of the other objectives.

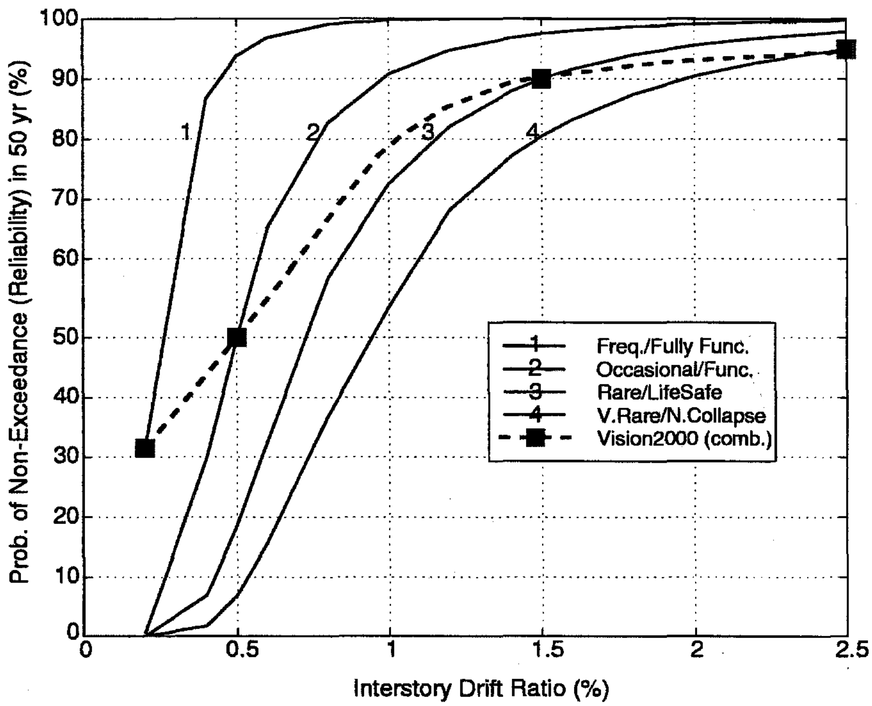

Figure 3.5: Comparison of Reliabilities for Four Optimal Designs

A clearer comparison of the interstory drift reliabilities for each of the four optimal designs can be seen in Fig. 3.5. It should be noted that in order to obtain the reliability curves, the reliabilities of each of the optimal designs are calculated for a multitude of finely separated interstory drift ratios. The *Vision 2000* levels appear as merely four points in this plot (shown as squares). These four points are connected by a smooth curve (dashed) to aid in visual comparison; the resulting reliability curve is artificial, and is neither suggested nor implied in the *Vision 2000* recommendations. From Fig. 3.5, as well as the optimal section sizes given in Table 3.7, it is clear that if all the interstory drift performance criteria of the *Vision 2000* are simultaneously considered, as implied in *Vision 2000*, the optimal design of the structure is governed by the 0.2% interstory drift ratio with a corresponding 68.5% risk over the lifetime, that is, by the condition of being "fully operational after frequent events." It should be noted, however, that in this simple example, all reliability calculations are based on linear dynamics since a response spectrum approach is used, but for higher performance levels (life-safe and near-collapse states), non-linear behavior would be expected. A more rigorous structural response analysis is required to support the above preliminary comments. In any case, it is not expected nor required to have a uniform reliability over multiple performance levels. In other words, it is not necessarily justifiable to require that no single performance objective, that is, a performance level and associated reliability/risk level, should dominate the design. The problem of specifying a performance objective is a very involved process. Seismic hazard and risk studies have to be incorporated with the consequences of various damage states to obtain the vulnerabilities upon which the various performance objectives are built.

In the above results, during the probabilistic analyses to estimate the lifetime peak interstory drift reliabilities, all seismic hazard levels are taken into consideration since a probabilistic description of the seismic environment has been used. However, usually in design practice, the approach is first to choose a uniform response spectrum corresponding to a specified hazard level ("earthquake design level") and then to perform the structural design using that particular spectrum. For example, the commonly used response spectrum specified in UBC (ICBO 1994) is assigned a hazard level of having a "10% probability of exceedance over 50 years." Similarly, in *Vision 2000*, there exist various "earthquake design levels" with associated exceedance probabilities (see Table 3.6).

In Fig. 3.6, uniform seismic hazard curves, expressed in terms of pseudo-velocity S_V contours corresponding to the *Vision 2000* earthquake design levels (in terms of probability of exceedance over 50 years) are given. These curves are obtained by first finding the probability of exceeding specified S_V values at a given period (5% damping ratio is assumed; S_V values between 2 in/s to 80 in/s at every 0.05 in/s, and periods between 0.1 s and 2.0 s at every 0.05 s are considered). Then the S_V values corresponding to particular exceedance probability are computed using linear interpolation between the two closest bounding S_V values. Cubic splines are fit over the resulting S_V values which have identical exceedance probability. The resulting smooth uniform hazard curves are shown in Fig. 3.6.

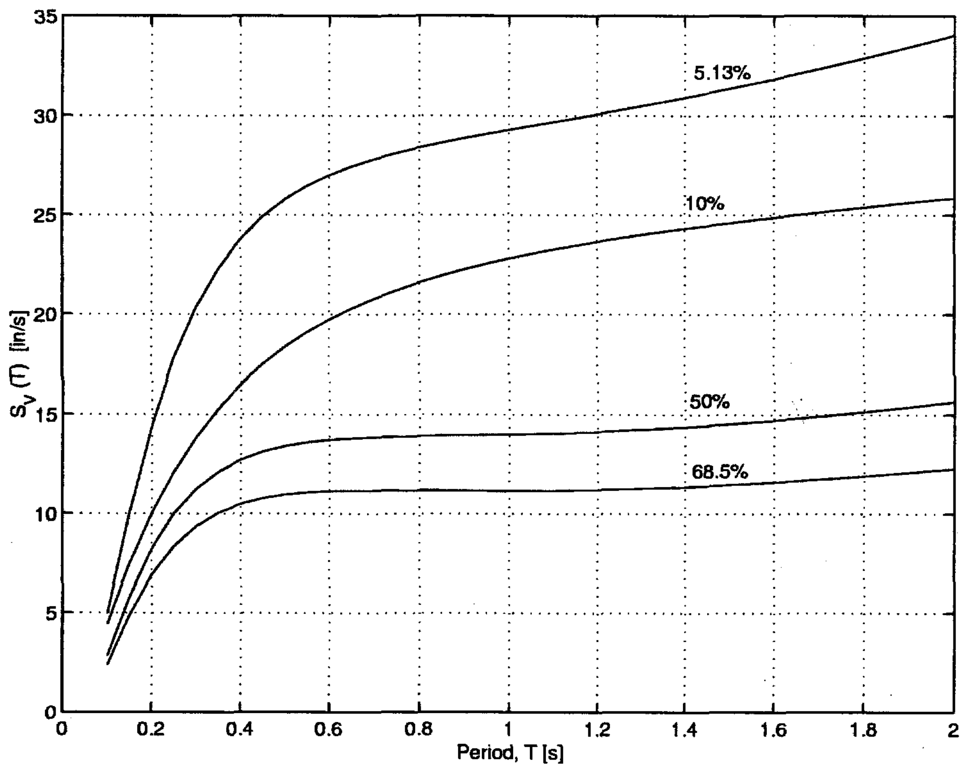

Figure 3.6: Seismic Hazard Levels Corresponding to *Vision 2000* Earthquake Design Levels (SEAOC 1995) in the Specified Seismic Environment

These explicit uniform hazard curves are then used in designing the example three-story single-bay moment-resisting frame at each corresponding performance level. Table 3.9 lists the resulting optimal designs. Comparing these results with those given in Table 3.7, it is seen that practically the same designs as before are obtained. The

Table 3.9: Optimal Design using the Uniform Response Spectra given in Fig. 3.6

	Freq./Fully Oper.		Occas./Oper.		Rare/Life-Safe		V. Rare/Coll.(2.5%)	
Criteria	Value	μ	Value	μ	Value	μ	Value	μ
B_{beam} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{beam} (in)	29.90	1.0000	20.35	1.0000	15.20	1.0000	12.34	1.0000
B_{col} (in)	4.10	1.0000	4.10	1.0000	4.10	1.0000	4.10	1.0000
D_{col} (in)	26.44	1.0000	15.27	1.0000	11.09	1.0000	9.20	1.0000
Vol (in^3)	12913	0.5231	9183	0.7346	7503	0.8298	6648	0.8782
d_1	0.0014	-	0.0039	-	0.0119	-	0.0196	-
d_2	0.0020	-	0.0050	-	0.0150	-	0.0250	-
d_3	0.0014	-	0.0032	-	0.0095	-	0.0159	-
d_{max}	0.0020	1.0000	0.0050	1.0000	0.0150	1.0000	0.0250	1.0000
Overall		**0.8976**		**0.9499**		**0.9694**		**0.9786**
Period	T_1=0.306 s		T_1=0.539 s		T_1=0.777 s		T_1=0.980 s	

small differences between the optimal dimensions obtained here, Table 3.9, and those obtained earlier, Table 3.7, are due to the inaccuracies in the numerical interpolations used in obtaining the uniform hazard curves.

As stated earlier, in a single mode linear response analysis with no consideration of modeling uncertainties, specifying lifetime performance "reliability" at the seismic hazard level or at the response risk level makes no difference in the resulting optimal designs. However, when modeling uncertainties in structural analysis are acknowledged performance "reliability" should be specified at the response risk level.

The simple yet illustrative example considered in this demonstration shows that the design methodology provides a framework capable of considering multiple structural response performance levels. It is also helpful in interpreting various performance criteria recommended for performance-based structural design, especially those classified under reliability-based objectives.

3.5 Concluding Remarks

In this chapter, a consistent and rational approach to evaluate a structural design using multiple design criteria has been presented. Using preference functions on various performance parameters that monitor different aspects of the design, expectations from the designed structure are incorporated into the design process. A multiplicative trade-off strategy, which satisfies a list of axioms of consistency, combines these multiple performance measures together with their associated importance weights to obtain a single design evaluation measure. In doing so, it also acts as the medium for finding the best compromise between conflicting performance objectives. The values of the design parameters that maximize the overall design evaluation measure give the optimal design.

Special emphasis has been given to the reliability-based performance objectives. The uncertain maximum interstory drift and the associated lifetime risk of exceeding various drift limit-states are used to illustrate the development and inclusion of preferences on reliability-based performance objectives.

The methodology is demonstrated through two case studies. In the first one, the design criteria are chosen such that a comparison between reliability-based versus code-specified criteria could be made. The effects of various issues related to the design have been studied as well. In the second case study, the methodology has been utilized to implement the performance-based design concept based on multiple performance objectives (response levels and associated reliabilities) as suggested by *Vision 2000.* It has been found that the specified performance requirements corresponding to diverse objectives and limit-states might not allow as much interaction as one might have hoped for. In the simple examples studied, the design is governed by the performance objective that the structure be fully operational for frequent events. However, for different structural configurations or seismic environments, this conclusion may not hold true. Further study is necessary to explore the opportunities promised by the concept of performance-based structural design. In the second example, two distinct design approaches to specifying lifetime performance reliabilities are considered: one in terms of seismic hazard and the other in terms of response risk. However, for the simple response analysis employed which did not include modeling uncertainties, the same designs are obtained.

Chapter 4 Design Evaluation Using Socio-Economics Based Objectives

4.1 Introduction

Due to the less than favorable performances of structures during recent earthquakes and the resulting extensive economic losses, there has been a vocal demand for, and a conscious effort to develop, performance-based structural design methodologies using multiple design objectives (SEAOC 1995; ATC 1997a). Performance-based design is defined as "[consisting] of the selection of appropriate systems, layout, proportioning and detailing of a structure and its non-structural components and contents such that at specified levels of ground motion and with defined levels of reliability, the structure would not be damaged beyond certain limiting states" (SEAOC 1995). Furthermore, it has been emphasized that not only improvements in structural response estimation are required, but also that the socio-economic consequences of earthquakes need to be taken into consideration more explicitly during the design decision-making process.

Modeling the social impacts of various performance objectives is a formidable task. Conventionally, at least at the code level, the objective of "life-safety ensured after major earthquakes" is used to cover the minimum expectation from a structural design in this aspect. Unfortunately, the vague definition of "major earthquakes" does not allow a quantitative treatment of this objective. The current performance-based design approaches treat the issue in a more tangible way by assigning a minimum reliability for meeting the life-safety condition over the lifetime of the designed structure. Through this interpretation of the objective, the original qualitatively described problem is translated into a reliability-based design problem. Nevertheless, it has not yet been well-studied how designing to meet this minimum objective would affect design when there exist other performance objectives. It has been observed that structures designed to be life-safe under major earthquakes can perform very poorly in terms of other criteria during lower level ground motions.

There are, of course, performance objectives that relate to other societal expectations, such as the requirement to keep essential or emergency facilities functional at all times, even after major earthquakes.

In general, the tendency is to interpret the consequences of a structure's performance in monetary terms, even for the extreme events of casualty and loss of life as done in many regional impact prediction studies (ATC 1992a, 1992b). However, it is believed that such interpretations are not always appropriate, even if possible.

A better way to specify the performance demands of society could be to express them in terms of risk levels. In this way, more realistic specifications could be obtained when society or individuals are able to compare the risks of the structures they live or work in with those from other factors and activities in their day-to-day lives (for example, Reid 1999; Ellingwood 2000). For example, once the statistical risk level of having a fatal car accident over a certain duration is known, an individual might be more comfortable in specifying the risk level of having a near-collapse structural response over the same duration in the structure in which he chooses to live. Such ease or clarity could never be achieved if individuals are asked to specify a certain dollar amount to quantify the worth of their lives. However, when the considered response levels are not related to life-safety issues but are related to economic objectives, such as the objective of keeping a facility operational after frequent earthquakes, the preferences might be better expressed in monetary terms.

Clearly, the issue of specifying performance objectives is a very complicated and highly controversial one as it is of concern to individuals, society, and governing bodies. Earthquakes can be very catastrophic events affecting large areas and populations at once and therefore, the concerns vary from individual losses to societal welfare and to the capabilities of governing bodies. Specifying performances over such a wide spectrum of issues is not easy. Especially for rare but destructive earthquakes, that is, the "low probability-high consequence" ones, there is no universally accepted treatment. It is hoped that multiple performance-based structural design methodologies that consider realistic and accurate socio-economic consequence models will be developed and utilized. The current approaches are restricted by our limited knowledge about the associated uncertainties and by the limited power of the available analytical and computational tools.

The treatment of the economic consequences of earthquakes related to a designed structure and its response is also a complicated issue in itself. However, there are available simple economic models that could be utilized in certain cases to gain insight. In this chapter, a simple yet informative approach to evaluate structural designs using socio-economic performance objectives is developed. The economic consequences receive the main attention, and interpretations are made with an engineering point of view in mind. The social objectives, despite their importance and their possible right to be the governing ones, are treated in a very simplistic manner, and are limited to the implicit life-safety criterion, as mentioned earlier. The approach will be implemented within two contexts and will be illustrated by example applications.

4.2 Socio-Economics Based Approach

4.2.1 Net Asset Value for Decision Making

To evaluate a design from an economic point of view, one may choose the net asset value of the designed structure as a possible evaluation measure. For a given structure, the net asset value is the difference, in monetary terms, of all benefits, that is, the revenues and other gains to be generated from the structure, and all expenses, such as those required to build the structure and maintain its functionality during its lifetime. A proper net asset value formulation should consider both the construction costs and the present value of the consequences of possible future events which might happen during the structure's intended lifetime. This way, combined with the profits, an objective evaluation of the design can be made as both short-term and long-term issues associated with the structure are taken into consideration.

The critical element in obtaining a realistic design evaluation measure is the proper and detailed treatment of the uncertainties surrounding the structure and its use. The attitude of the decision-maker to the risks due to these uncertainties may also need to be considered.

Clearly, the net asset value depends not only on the physical structure itself but also on the type of business it will house as this would specify the income during operational times and indirect losses during downtimes. For the current study, a very simple business model will be assumed and only the loss-of-use of the structure after a destructive earthquake will be considered in the formulation as the source of indirect losses. Profits will also be treated in a simplified manner. The main focus will be on the direct economic consequences of structural response and on developing a methodology to quantify these consequences. Nevertheless, the conceptual framework is flexible enough to allow incorporation of more comprehensive business models if desired.

A possible formulation for the net asset value (NAV) of a new structure can be given as

$$
\begin{aligned}
NAV = \; & Discounted\ net\ income\ stream \\
& - Present\ value\ of\ future\ earthquake\ losses \\
& - Cost\ of\ construction
\end{aligned} \tag{4.1}
$$

The first term is the "discounted net income stream" and this is the value of the structure based on the present value of the gross income expected to be generated over its lifetime were it to be operating at all times minus the regular maintenance costs and other operating expenses. If the income stream is assumed to be independent of the state of the structure, as long as it is operational, the value of this term may be assumed fixed. The second term is the present value of losses due to uncertain future earthquakes. This term is usually the most uncertain one and is probably the most critical element affecting design decisions. Therefore, special attention will be given to computing it. Finally, the "cost of construction" is self-explanatory. The sum of the last two terms plus the present value of the regular maintenance costs, which is implicitly included in the net income stream, constitute what is known as the "life-cycle cost" for the structure. Detailed discussion of the critical term in the life-cycle cost estimation, that is, the present value of the losses due to future uncertain earthquakes is given in Sections 4.2.3 and 4.2.4.

There are various steps in the methodology to compute the net asset value. They can be grouped as seismic hazard analysis, building-specific vulnerability analysis and loss estimation, and lifetime earthquake loss estimation. These steps are described in the subsequent sections and are illustrated, first, using an example of studying alternative designs for a simple new structure, and later, using an example within the context of mitigation analysis.

As noted earlier, the attitude of the decision-maker toward risks associated with the structure is an issue that needs special attention. During the discussion of the methodology to be introduced in this chapter, the attitude of the decision-maker will initially be taken to be a "risk-neutral" one (for example, Hey 1979; French 1988). In general, the risk-neutral assumption should be valid for a large company, especially in the case when the company's net worth is not threatened by a possible loss of the considered structure. A convenient result of risk-neutrality is that only the "expected" values of the uncertain costs are needed in making decisions. A brief discussion of other risk attitudes within the context of Decision Theory is introduced next.

4.2.2 Attitudes to Risk in Decision Making

The net asset value estimate of a structure contains uncertainty because one never has a complete state of knowledge about all the relevant issues. Sources of this uncertainty are a diverse set of matters affecting the structure from the time it is conceived until the end of its lifetime. Some of the more crucial issues are: the unknown seismic excitations the structure is going to experience during its lifetime which gives rise to excitation/loading uncertainty; the uncertainty in the response estimate for the structure to an excitation, and the uncertainty in the damage estimate because there are always modeling uncertainties involved beyond the loading uncertainty; and the uncertainties in the cost and the duration of possible repair or upgrades that might be needed and that are affected by market uncertainties (regarding material and labor) and uncertainty in the precise amount of damage.

As an illustration, consider a decision-maker using the net asset value of a structure on which to base decisions during a design. Since the consequences of the decisions directly or indirectly affect the economic state of the decision-maker, the attitude of the decision-maker to risk becomes an issue. For example, when there exists a high possibility that there could be future losses greatly affecting the net worth of the decision-maker he might give greater importance to those decisions (designs) that mitigate such possibilities at some additional expense. An important question is: how does the existence of uncertainty in the net asset value affect the behavior of the decision-maker? The well-studied von Neumann-Morgenstern utility theory (for example, von Neumann and Morgenstern 1947; Hey 1979; Ang and Tang 1984; French 1988) provides an approach to this question by using utility functions as a measure to express the reactions of the decision-maker to different situations. A utility function quantifies a decision-maker's preference for possible outcomes resulting from taking a chosen action (for example, Turkstra 1970; Lomnitz and Rosenblueth 1976; Keeney and Raiffa 1976; French 1988) and in this sense is similar to the preference function defined in Chapter 3. A discussion of the von Neumann-Morgenstern utility theory applied to the net asset value problem follows.

Let V denote the net asset value, and let $p(V \mid A)dV$ denote the probability that if a recovery or mitigation action A is chosen, the outcome, that is, the net asset value, will be V. Let $\mu(V)$ be the utility function for V. It is a monotonically increasing function to express the decision-maker's preference for a higher net asset value over a lower one. The theory of von Neumann and Morgenstern states that the decision-maker should rank possible actions by their expected utility when making decisions, that is, preference for action A is given by

$$\mu(A) = E[\mu(V) \mid A] = \int \mu(V)\, p(V \mid A)\, dV \tag{4.2}$$

in which the integration is over all possible values of V.

To demonstrate the different possible attitudes to risk, let us consider the three utility functions given in Fig. 4.1, which are over the net asset value V.

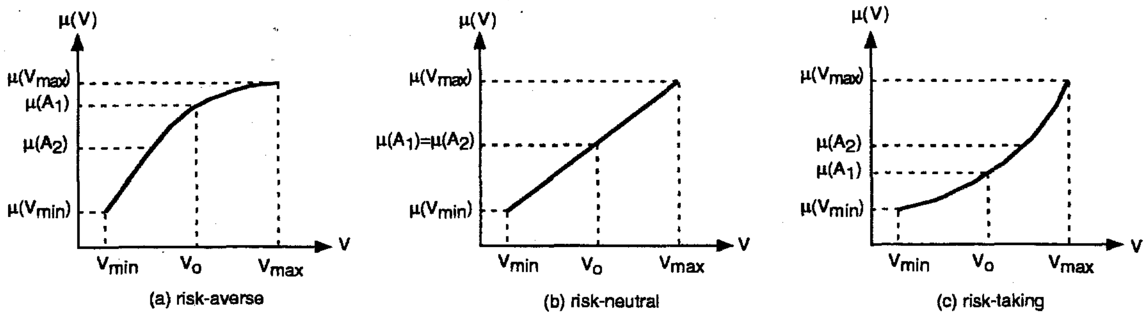

Figure 4.1: Example Utility Functions for Different Attitudes to Risk

Let action A_1 lead to a certain value of $V_0 = \frac{1}{2}(V_{min} + V_{max})$ and action A_2 lead to either V_{min} or V_{max}, each with a probability of $\frac{1}{2}$. Notice that both actions have the same expected value, V_o. The corresponding utilities are $\mu(A_1) = \mu(V_o) = \mu(\frac{1}{2}(V_{min} + V_{max}))$, and $\mu(A_2) = \frac{1}{2}\mu(V_{min}) + \frac{1}{2}\mu(V_{max})$.

The utility functions in Figs. 4.1(a), (b), and (c) may be interpreted as follows:

(a) Concave utility function – *risk-averse*:

$\mu(A_1) > \mu(A_2)$, which implies that the decision-maker prefers A_1, that is, V_o with certainty, to A_2, that is, "V_{min} or V_{max} with fifty-fifty chance," even though the expected net asset value is the same for both actions and equal to V_o.

(b) Linear utility function – *risk-neutral*:

$\mu(A_1) = \mu(A_2)$, which implies that the decision-maker is indifferent between A_1 and A_2. The expected net asset value is the controlling quantity for the decision-maker.

(c) Convex utility function – *risk-taking*:

$\mu(A_1) < \mu(A_2)$, which implies that the decision-maker prefers to take the risk of losing value $(V_o - V_{min}) = \frac{1}{2}(V_{max} - V_{min})$ in the hope of gaining additional value $(V_{max} - V_o) = \frac{1}{2}(V_{max} - V_{min})$ compared with the value $V_o = \frac{1}{2}(V_{min} + V_{max})$ which is given with certainty by taking action A_1.

Since concavity, linearity, or convexity may be expressed in terms of the second derivative of $\mu(V)$ with respect to V, $\mu''(V)$, the attitude to risk can be described as:

1) risk-averse when $\mu''(V) < 0$
2) risk-neutral when $\mu''(V) = 0$
3) risk-taking when $\mu''(V) > 0$

A measure of absolute risk-aversion $R_A(V)$ due to Arrow and Pratt is defined by:

$$R_A(V) = -\frac{\mu''(V)}{\mu'(V)} \tag{4.3}$$

which is independent of a linear transformation of the utility function and therefore is a more appropriate measure of attitude to risk than $\mu''(V)$. Higher values of $R_A(V)$ correspond to a stronger aversion to risk. Since $\mu'(V) > 0$, the three risk attitude cases are, in terms of Arrow-Pratt measure:

1) $R_A(V) > 0$ implies risk-aversion
2) $R_A(V) = 0$ implies risk-neutrality
3) $R_A(V) < 0$ implies risk-taking

The above measure or indicator gives a "local" risk attitude, that is, attitude for a given V value. If it is chosen that the "local" attitudes are to be the same everywhere, for example, for the risk-averse case, the utility function for the special case of "constant absolute risk-aversion," that is, $R_A(V) = R$, constant over $[V_{min}, V_{max}]$ will be obtained.

First, consider the risk-neutral case, for which $R_A(V) = R = 0$. From Eqn. (4.3), the corresponding utility function and the expected utility are

$$\mu(V) = a + bV, \ (b > 0) \tag{4.4}$$

$$E[\mu(V)] = a + bE[V] \tag{4.5}$$

One can obtain the values of a and b by specifying two points on the utility function. For example, if $\mu(V_{min}) = 0$ and $\mu(V_{max}) = 1$, as in Fig. 4.2, one will have

$$\mu(V) = \frac{1}{V_{max} - V_{min}}(V - V_{min}) \tag{4.6}$$

$$E[\mu(V)] = \frac{1}{V_{max} - V_{min}}(E[V] - V_{min}) \tag{4.7}$$

It is important to note that for this risk-neutral attitude, the expected utility depends only on the expected net asset value, $E[V]$, and the limiting values of the range of V.

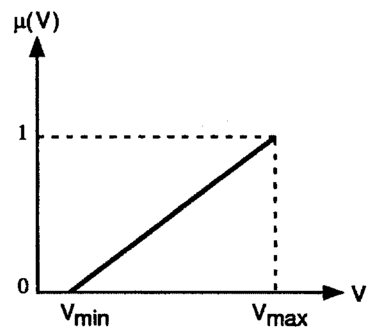

Figure 4.2: Risk-Neutral Utility Function

For the risk-averse and risk-taking cases, however, $R_A(V) = R \neq 0$. Using Eqn. (4.3), the utility function and the expected utility for the constant absolute risk-averse attitude can be found as

$$\mu(V) = a - be^{-RV}, \ (b > 0) \tag{4.8}$$

$$E[\mu(V)] = a - bE[e^{-RV}] \tag{4.9}$$

where

$$E[e^{-RV}] = \sum_{n=0}^{\infty} \frac{(-R)^n}{n!} E[V^n] = M_V(-R) \tag{4.10}$$

which is the value of the moment generating function of V at $-R$. In particular, if it is chosen such that $\mu(V_{min}) = 0$ and $\mu(V_{max}) = 1$, as in Fig. 4.3, for a risk-averse case, then

$$\mu(V) = \frac{1}{e^{-RV_{min}} - e^{-RV_{max}}}(e^{-RV_{min}} - e^{-RV}) \tag{4.11}$$

$$E[\mu(V)] = \frac{1}{e^{-RV_{min}} - e^{-RV_{max}}}(e^{-RV_{min}} - M_V(-R)) \tag{4.12}$$

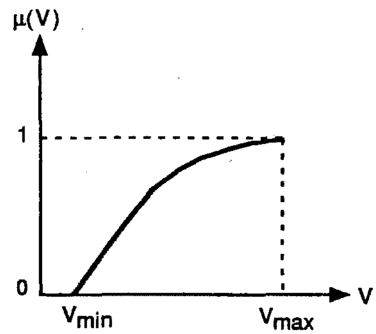

Figure 4.3: Risk-Averse Utility Function

In a rational decision-making model, it is believed that individuals try to maximize their expected utilities. As the attitude of the decision-maker is assumed to be a risk-neutral one in this work, it will be seen that the results will involve only the expected values. In the constant absolute risk-averse case, or risk-taking case, however, the moment generating function for the lifetime losses, $M_V(v)$, is required. Depending on the assumptions regarding various issues, different exact or approximate analytic formulations for this moment generating function for the net asset value might be obtained. It is believed that, however, Monte Carlo simulations would be needed to study realistic cases. As such, quantitative investigations of risk-averse decision-making for performance-based design is a possible future research area. However, qualitative insight into the effect of such attitude can be gained as follows.

Once the preferences over possible values for the net asset value are specified in the form of a utility function, the problem is translated into finding the specific state of design, termed *Design*, which maximizes the expected utility (von Neumann and Morgenstern 1947), that is,

$$\max_{Design} E[\mu(V) \mid Design,\ Env] = \max_{Design} \int_{V_{min}}^{V_{max}} \mu(V)\ p(V \mid Design,\ Env)dV \quad (4.13)$$

where Env denotes the seismic environment surrounding the designed structure, and V_{min} and V_{max} are limiting values for net asset value.

It is clear from Eqn. (4.13) that, irrespective of the risk attitude, the general tendency would be to develop designs which shift the probability distribution for V globally to higher values. For all risk attitudes, the higher the net asset value, the higher the preference. However, a further look into the formulation will help observe special characteristics of each attitude. For example, for a risk-averse attitude, the preference, that is, $\mu(V)$, increases more rapidly at lower V values than it does at higher V values. In other words, $[\mu(V_1+\Delta V)-\mu(V_1)] > [\mu(V_2+\Delta V)-\mu(V_2)]$ where $V_1 < V_2$, $\Delta V > 0$, and $V_1, V_2 \in [V_{min},\ V_{max}]$. In this sense, improvement at lower V values receive greater attention in a risk-averse attitude. In contrast, in a risk-taking attitude, greater attention is given to higher V values as the greater rate of increase in preference is observed there. For the risk-neutral case, there is no such tendency, or bias, if one might call it so.

The above conclusions could be reached mathematically by reformulating Eqn. (4.13) as follows. First, use integration by parts on the integral in Eqn. (4.13). Considering the fact that the cumulative distribution function $P(V \mid Design,\ Env)$, which would result from integration by parts, is equal to 0 at the lower bound for the net asset value, V_{min}, and equal to 1 at the upper bound, V_{max}, the new formulation is

$$\max_{Design} 1 - \int_{V_{min}}^{V_{max}} \frac{d\mu}{dV}\ P(V \mid Design,\ Env)dV \quad (4.14)$$

which can be recast as a minimization problem

$$\min_{Design} \int_{V_{min}}^{V_{max}} \frac{d\mu}{dV}\ P(V \mid Design,\ Env)dV \quad (4.15)$$

Since $\frac{d\mu}{dV}$ is a monotonically decreasing function for a risk-averse attitude, and a monotonically increasing function for a risk-taking attitude, for a given *Design*, greater emphasis is given to lower, and higher, V values, respectively.

Another remark is that, in risk-neutral cases, the preference functions (utility functions) can be recast to consider only the uncertain terms in the net asset value. The terms not affected by the design decisions can be ignored in making comparisons since the preference functions are linear. In the case of other attitudes, however, one has to consider the full formulation and all of the terms since the preference functions are then non-linear. For example, if the decision-maker has a risk-averse attitude and bases decisions on the net asset value of the designed structure, and if the net operating income to be obtained from the structure is unknown, alternative designs cannot be compared properly by considering only the life-cycle costs.

4.2.3 Building Vulnerability Analysis and Loss Estimation

The purpose of a building vulnerability analysis is to provide a probabilistic description of the total earthquake losses for the building for a specified ground motion intensity or seismic hazard level. If S_V is used to represent the intensity, the task is to determine $p(C \mid S_V,\ Design)$ where C represents the total earthquake losses defined to be the sum of the repair costs (direct losses) and the loss-of-use costs (indirect losses); *Design* specifies the structural system; and, S_V is the spectral velocity, which is chosen to represent the seismic hazard level. If a risk-neutral attitude is assumed, the interest is in obtaining the corresponding expected loss, that is, $E[C \mid S_V,\ Design]$.

In this study, building vulnerability analyses are performed through combining finite-element structural response analyses and the recently developed assembly-based vulnerability method (Beck et al. 1999b, 1999c; Porter et al. 2000; Porter 2000) which considers a given structure as a collection of the assemblies, structural and non-structural components and contents, constituting it. Direct and indirect losses are related to the damage state of individual components which are elements in assembly groups. The key difference between the assembly-based vulnerability method and commonly-used heuristic or empirical approaches that give approximate estimates with little or no insight is that, in assembly-based vulnerability method, the relation between the response of a structure and the associated damage potential in various assemblies of the structure is treated formally. The damaging consequences of a structure's response are quantified at every stage.

The building vulnerability analysis must be integrated with the seismic hazard analysis in the loss estimation procedure to obtain the probability distribution for the losses C in the structure represented by $Design$, given that an earthquake EQ has occurred. It should be noted that the event parameters for EQ, such as its magnitude and location, are not known. They are described by probability distributions over the range of possible values. In our study based on a risk-neutral attitude, the final outcome of the loss estimation is $E[C \mid EQ,\ Design]$.

The building vulnerability analysis and loss estimation are carried out as follows. First, one can break the total loss C into direct losses, C_{direct}, due to repairs, and indirect losses, $C_{indirect}$, due to, for example, loss-of-use costs, C_{lou}. The critical issue is then how to relate the seismic hazard expressed in terms of S_V to individual losses, say, C_{ij}, the direct loss for component j in unit i (for example, a suite or a floor) of the structure, as well as the income lost during repairs.

In estimating the direct losses sustained by each component, an intermediate parameter is used, such as the structural response parameter that has the dominant controlling effect on the failure of a particular component. This parameter should correlate best with the possible damage states associated with the component. For example, it could be the peak interstory drift ratio (IDR) for wall partitions and for glazing; peak diaphragm acceleration (PDA) for ceiling panels; and elastic demand-to-capacity ratio (DCR) for welded steel moment-resisting connections. The choices draw upon existing experimental and analytical studies, as well as engineering experience and intuition (Beck et al. 1999c).

The expected repair cost for component j of unit i is expressed using the total probability theorem as

$$
\begin{aligned}
E[C_{ij} \mid S_V,\ Design] &= E[C_{ij} \mid F_{ij}]\ P\{F_{ij} \mid S_V,\ Design\} \\
&\quad +\ E[C_{ij} \mid \sim F_{ij}]\ P\{\sim F_{ij} \mid S_V,\ Design\}
\end{aligned}
\tag{4.16}
$$

where F_{ij} means "failure of the j-th component in unit i" and $\sim F_{ij}$ means "non-failure of the j-th component in unit i." In principle, "failure" could correspond to one of several different limit-states, each corresponding to a different degree of damage. For the applications in this study, "failure" corresponds to one of two limit-states: either repairs on the component are needed or the component must be replaced. In general, the uncertain amount of damage to be repaired is a major source of the uncertainty in the repair cost C_{ij} given F_{ij}. Here, however, only the expected value, $E[C_{ij} \mid F_{ij}]$, is required which may be based on the corresponding values of the costs for various levels of damage. These costs include both local material and local labor costs. Since there will be no cost if the component does not fail, $E[C_{ij} \mid \sim F_{ij}] = 0$.

The expected value of direct losses for unit i with N_i components can then be written as

$$
E[C_i \mid S_V,\ Design] = \sum_{j=1}^{N_i} E[C_{ij} \mid F_{ij}]\ P\{F_{ij} \mid S_V,\ Design\} \tag{4.17}
$$

where $P\{F_{ij} \mid S_V,\ Design\}$, the probability of failure of component j in unit i of the structure under the seismic attack with intensity S_V, can be obtained from

$$
P\{F_{ij} \mid S_V,\ Design\} = \int P\{F_{ij} \mid z_{ij}\}\ p(z_{ij} \mid S_V,\ Design)\ dz_{ij} \tag{4.18}
$$

In Eqn. (4.18), $P\{F_{ij} \mid z_{ij}\}$ is the "fragility" of component j in unit i, that is, the probability of failure of a particular component given that the associated structural response parameter has the value z_{ij}. In determining the probability density function of the response quantity z_{ij}, that is, $p(z_{ij} \mid S_V,\ Design)$, only the spectral velocity corresponding to the small-amplitude ("elastic") fundamental mode of the structure represented by *Design* will be used. Thus, this probability density function should include the uncertainties in the response due to the fact that the ground motion is not completely specified by S_V. For example, it can be modeled as a log-normal distribution to account for these uncertainties, as explained before in this study.

Letting N_{unit} denote the number of revenue generating units in the building, the expected total direct losses for the building is given as

$$E[C_{direct} \mid S_V,\ Design] = \sum_{i=1}^{N_{unit}} E[C_i \mid S_V,\ Design] \tag{4.19}$$

Then, using the total probability theorem, the expected total direct losses given the occurrence of an earthquake in the region of the site can be found by:

$$E[C_{direct} \mid EQ,\ Design,\ Env] = \int E[C_{direct} \mid S_V,\ Design]\ p(S_V \mid EQ,\ Design,\ Env)\ dS_V \tag{4.20}$$

where the probability density function $p(S_V \mid EQ,\ Design,\ Env)$ in Eqn. (4.20) can be found through a probabilistic seismic hazard analysis for the seismic environment *Env* for the site (Reiter 1990). For example, the approach explained in Section 2.2.1 could be used.

For the indirect losses, the expected loss-of-use costs may be determined by a formulation similar to that for the expected direct losses. First, let T_{ij} be the time required to repair component j in revenue-generating unit i. Often, failed components in a unit are repaired in series. For example, all beam/column connection repairs are first done one by one, then the walls are repaired, after which any damaged ceiling panels and glazing are replaced. If this is the case, the expected loss-of-use duration for unit i with a total of N_i components is given by

$$E[T_i \mid S_V,\ Design] = \sum_{j=1}^{N_i} E[T_{ij} + \tau_{ij} \mid F_{ij}]\ P\{F_{ij} \mid S_V,\ Design\} \tag{4.21}$$

where $P\{F_{ij} \mid S_V,\ Design\}$ is given by Eqn. (4.18) and τ_{ij} is the delay time before the crew begins to repair component j. However, Eqn. (4.21) may not apply because the relationship between the total loss-of-use duration T_i and the time T_{ij} for repair for each component depends on how the repair jobs are scheduled among the units. The reader is referred to the discussion of "fast-track" and "slow-track" schedules in Beck et al. (1999b).

The total expected cost due to loss-of-use may be obtained from

$$E[C_{lou} \mid S_V,\ Design] = \sum_{i=1}^{N_{unit}} R_i\, E[T_i \mid S_V,\ Design] \tag{4.22}$$

where T_i is the total loss-of-use duration for unit i and R_i is the gross operating income rate for unit i. Similar to Eqn. (4.20) for the expected total direct losses, the expected total indirect losses for an earthquake is then given by

$$\begin{aligned} &E[C_{lou} \mid EQ,\ Design,\ Env] = \\ &\qquad \int E[C_{lou} \mid S_V,\ Design]\, p(S_V \mid EQ,\ Design,\ Env)\, dS_V \end{aligned} \tag{4.23}$$

In reality, the duration of loss-of-use depends not only on the state of the structure under consideration, but also on the state of the infrastructure surrounding the structure. For example, a structure in an undamaged state may not be habitable because of restrictions on, or loss of, basic amenities, such as power and water, after a destructive earthquake (ATC 1991). Such restrictions should ideally be taken into consideration in computing indirect losses. To this end, the output of regional loss estimation studies (for example, NIBS 1997) may be used to perform more realistic modeling of the loss-of-use duration after a destructive event. Such an analysis is, however, beyond the scope of the present study. Indeed, the costs from loss of use are not considered in the examples presented later.

Finally, once the expected total direct and indirect losses are computed, the expected total loss for an earthquake can be found from

$$\begin{aligned} &E[C \mid EQ,\ Design,\ Env] = \\ &\qquad E[C_{direct} \mid EQ,\ Design,\ Env] + E[C_{lou} \mid EQ,\ Design,\ Env] \end{aligned} \tag{4.24}$$

It should be noted that this formulation could also be used to determine the expected loss for a given event with specified event parameters, in order to study the post-event state of a structure after an earthquake has occurred, or to study a postulated scenario earthquake for a specific loss estimation study.

The major steps in the computer implementation of the developed loss estimation methodology are summarized as follows:

- Specify the seismic environment *Env* and the structural model *Design.*

- Assemble $\boldsymbol{M},\ \boldsymbol{K}$; compute element elastic moment and axial load capacities.

- Find normal modeshapes and periods (*only fundamental mode to be used*).

- Find participation factors for performance parameters z_{ij} associated with the specified components.

- Generate $M,\ R$ mesh for numerical integration and compute corresponding $p(M,\ R \mid Env)$ and median $\widehat{S}_V(M,\ R)$.

- Specify numerical integration parameters such as threshold values, number of integration steps. Compute $S_{V,up}$ such that $p(S_{V,up} \mid EQ,\ Design,\ Env) < \epsilon(S_V)$, where $\epsilon(S_V)$ is a specified cut-off threshold value.

- Find $z_{ij,up}$ for each component to meet chosen numerical integration accuracy; compute $p(z_{ij} \mid S_V,\ Design)$ using associated uncertain response model; integrate over S_V to compute $p(z_{ij} \mid EQ,\ Design,\ Env)$ (*integration over S_V is carried out at this stage for efficiency*).

- Specify fragility model parameters using component list. Compute each component's failure probability $P\{F_{ij} \mid Design,\ Env\}$ by convolving fragility $P\{F_{ij} \mid z_{ij}\}$ with associated $p(z_{ij} \mid EQ,\ Design,\ Env)$ and integrating over z_{ij}.

- Compute $E[C_i \mid Design,\ Env]$ from all $E[C_{ij} \mid F_{ij}]$.

- Compute $E[C \mid EQ,\ Design,\ Env]$.

4.2.4 Estimation of Lifetime Earthquake Losses

To estimate lifetime earthquake losses for a structure, it will be assumed that: for the specified regional seismic environment *Env* for the site, the earthquakes arrive with an average rate of ν and follow a Poisson process; the structure is always "restored" to the same state represented by *Design* after every destructive earthquake; C corresponds to the total loss given an earthquake; and, a continuous discount rate r is used to model the economic environment where the discount rate represents the difference between the interest rate and the inflation rate. The present value of expected losses from future earthquakes over lifetime t_{life} for a building system specified by *Design* can then be written as (Ang et al. 1996)

$$E[C \mid Design,\ Env,\ t_{life}] =$$
$$\sum_{n=1}^{\infty}\left[\sum_{k=1}^{n}\int_{0}^{t_{life}} E[C \mid EQ,\ Design,\ Env]\ e^{-rt_k}\ p(t_k \mid t_k \le t_{life})\ dt_k\right]\frac{(\nu t_{life})^n}{n!}\ e^{-\nu t_{life}} \tag{4.25}$$

where $E[C \mid EQ,\ Design,\ Env]$ is the expected "restoration" cost given an earthquake (EQ) has occurred, and $p(t_k \mid t_k \le t_{life})$ is the probability density function for the time of the k-th event, t_k, given that this event occurs within the lifetime of the structure. The first summation results from considering all possible number of earthquakes during the lifetime of the structure. The term $\frac{(\nu t_{life})^n}{n!}\ e^{-\nu t_{life}}$ gives the probability of having exactly n events during the lifetime of the structure in the specified seismic environment and is due to the Poisson earthquake arrival model. The second summation is over n earthquakes, and the integration considers the distribution of each earthquake's occurrence time within n earthquakes, as well as converting the corresponding expected total loss into its present value by using continuous discounting.

Concentrating on the individual terms, the first term to be considered is the expected value of losses given an EQ,

$$E[C \mid EQ,\ Design,\ Env] = \int_{0}^{C_{replace}} C\ p(C \mid EQ,\ Design,\ Env)\ dC \tag{4.26}$$

which is the outcome of the loss estimation study for a building as developed in Section 4.2.3.

The next term to be considered is the one that represents the event-time distribution for a particular event. For the k-th event over the lifetime t_{life} of the structure, this term is $p(t_k \mid t_k \leq t_{life})$ and is given by

$$p(t_k \mid t_k \leq t_{life}) = \frac{p(t_k)}{Prob\{0 < t_k \leq t_{life}\}} \tag{4.27}$$

for which $p(t_k)$ is the probability density function for the time to the k-th event, t_k. For a Poisson process, with an average arrival rate ν, this probability density function is a gamma probability function and is equal to (Benjamin and Cornell 1970)

$$p(t_k) = \frac{\nu(\nu t_k)^{k-1}}{\Gamma(k)} e^{-\nu t_k} \tag{4.28}$$

where $\Gamma(k)$ is the gamma function of k. Consequently,

$$Prob\{0 < t_k \leq t_{life}\} = \frac{\Gamma(k, \nu t_{life})}{\Gamma(k)} \tag{4.29}$$

which is obtained by integrating $p(t_k)$ in Eqn. (4.28) from 0 to t_{life}. Here, $\Gamma(k, \nu t_{life})$ is the incomplete gamma function of k and νt_{life} is the expected number of earthquakes of interest during the considered lifetime.

Substituting Eqn. (4.27) into Eqn. (4.25), and using Eqns. (4.28) and (4.29), and bringing the constant $E[C \mid EQ, Design, Env]$ out of the summation since it is independent of the summation and integration variables, one obtains

$$E[C \mid Design, Env, t_{life}] = E[C \mid EQ, Design, Env]\ PWF(\nu, r, t_{life}) \tag{4.30}$$

where PWF is the so-called "present worth factor" (Ang et al. 1996). This term is

$$PWF = \sum_{n=1}^{\infty} \left[\sum_{k=1}^{n} \int_0^{t_{life}} e^{-rt_k} \frac{\nu(\nu t_k)^{k-1} e^{-\nu t_k}}{\Gamma(k, \nu t_{life})} dt_k \right] \frac{(\nu t_{life})^n}{n!} e^{-\nu t_{life}} \tag{4.31}$$

As

$$\int_0^{t_{life}} \nu(\nu t_k)^{k-1} e^{-(\nu+r)t_k} dt_k = \Gamma(k, (\nu+r)t_{life}) \left(\frac{\nu}{\nu+r}\right)^k \tag{4.32}$$

Eqn. (4.31) can be rewritten as

$$\begin{aligned}
PWF &= \sum_{n=1}^{\infty} \sum_{k=1}^{n} \frac{\Gamma(k,\ (\nu+r)t_{life})}{\Gamma(k,\ \nu t_{life})} \left(\frac{\nu}{\nu+r}\right)^k \frac{(\nu t_{life})^n}{n!}\, e^{-\nu t_{life}} \\
&= \sum_{k=1}^{\infty} \sum_{n=k}^{\infty} \frac{\Gamma(k,\ (\nu+r)t_{life})}{\Gamma(k,\ \nu t_{life})} \left(\frac{\nu}{\nu+r}\right)^k \frac{(\nu t_{life})^n}{n!}\, e^{-\nu t_{life}} \\
&= \sum_{k=1}^{\infty} \frac{\Gamma(k,\ (\nu+r)t_{life})}{\Gamma(k,\ \nu t_{life})} \left(\frac{\nu}{\nu+r}\right)^k e^{-\nu t_{life}} \sum_{n=k}^{\infty} \frac{(\nu t_{life})^n}{n!} \\
&= \sum_{k=1}^{\infty} \frac{\Gamma(k,\ (\nu+r)t_{life})}{\Gamma(k,\ \nu t_{life})} \left(\frac{\nu}{\nu+r}\right)^k \left[1 - e^{-\nu t_{life}} \sum_{n=0}^{k-1} \frac{(\nu t_{life})^n}{n!}\right]
\end{aligned} \tag{4.33}$$

But using an identity for the incomplete gamma function (Abramowitz and Stegun 1972) which states that

$$\Gamma(k,\ x) = \Gamma(k) \left[1 - e^{-x} \sum_{n=0}^{k-1} \frac{x^n}{n!}\right] \tag{4.34}$$

PWF can be written as

$$\begin{aligned}
PWF &= \sum_{k=1}^{\infty} \left[\ 1 - e^{-(\nu+r)t_{life}} \sum_{n=0}^{k-1} \frac{((\nu+r)t_{life})^n}{n!}\right] \left(\frac{\nu}{\nu+r}\right)^k \\
&= \frac{\nu}{r} - e^{-(\nu+r)t_{life}} \sum_{k=1}^{\infty} \sum_{n=0}^{k-1} \frac{((\nu+r)t_{life})^n}{n!} \left(\frac{\nu}{\nu+r}\right)^k \\
&= \frac{\nu}{r} - e^{-(\nu+r)t_{life}} \sum_{n=0}^{\infty} \sum_{k=n+1}^{\infty} \frac{((\nu+r)t_{life})^n}{n!} \left(\frac{\nu}{\nu+r}\right)^k \\
&= \frac{\nu}{r} - e^{-(\nu+r)t_{life}} \sum_{n=0}^{\infty} \frac{((\nu+r)t_{life})^n}{n!} \sum_{k=n+1}^{\infty} \left(\frac{\nu}{\nu+r}\right)^k \\
&= \frac{\nu}{r} - e^{-(\nu+r)t_{life}} \sum_{n=0}^{\infty} \frac{((\nu+r)t_{life})^n}{n!} \frac{\nu}{r} \left(\frac{\nu}{\nu+r}\right)^n \\
&= \frac{\nu}{r} - \frac{\nu}{r} e^{-(\nu+r)t_{life}} \sum_{n=0}^{\infty} \frac{(\nu t_{life})^n}{n!} \\
&= \frac{\nu}{r} \left[1 - e^{-r t_{life}}\right] \\
&= \nu t_{life} \left[\frac{1 - e^{-r t_{life}}}{r t_{life}}\right]
\end{aligned} \tag{4.35}$$

The result tells that PWF is simply the expected number of events over the lifetime of the structure scaled by a discounting factor that depends on the discount rate times the lifetime. For the special case of no discounting, that is, $r = 0$, one obtains $PWF = \nu t_{life}$, as expected.

A plot of the "present worth" factor normalized with respect to the expected number of events over the lifetime of the structure is given in Fig. 4.4. It is worth noting that the "normalized present worth factor" is independent of the seismic environment and is a purely economic factor.

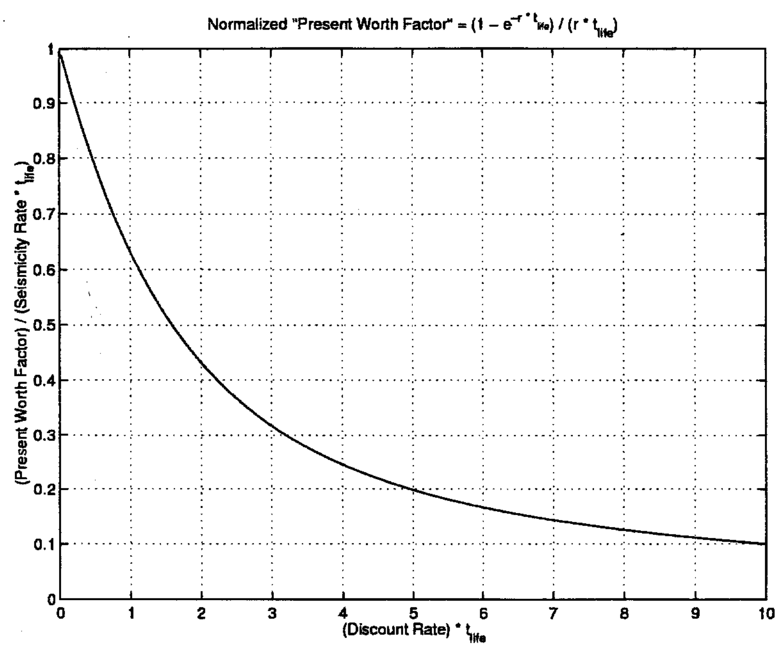

Figure 4.4: Normalized "Present Worth Factor"

In conclusion, the rather complicated looking original formulation given in Eqn. (4.25) for lifetime loss estimation is simplified to

$$E[C_{life} \mid Design,\ Env,\ t_{life}] \ = \ E[C \mid EQ,\ Design,\ Env] \left[\nu t_{life} \, \frac{1 - e^{-rt_{life}}}{rt_{life}} \right] \tag{4.36}$$

This result states that the expected lifetime losses for a structure is the product of the expected total loss incurred given that an uncertain earthquake has occurred in the vicinity of the site and the present worth factor which depends on the seismicity rate ν, discount rate r and the lifetime t_{life} (Beck et al. 1999b).

Once obtained, the lifetime loss estimate can be incorporated into the net asset value estimate given in Eqn. (4.1) which is to be used in evaluating the economic performance of the current design.

It should be noted that the duration of use considered in computing the net asset value changes the contributions from different terms in the formulation. Even though in the above derivation, t_{life}, the life-time of the structure is used, it is possible that a much shorter duration consideration might be preferred. The choice depends on whether the client has a long-term or a short-term point of view. As such, the choice might reflect the type of the client, for example, an owner with long-term objectives versus a developer with short-term ones.

4.3 Implementing Socio-Economic Performance Objectives in Design: a Two-Stage Design Procedure

4.3.1 Introduction

The complex nature of issues surrounding new construction makes design decision-making very challenging. The fundamental task is to find a consensus design balancing the client's preferences and society's expectations. In their essence, modern structural design codes serve primarily to protect society's interests, although new performance-based codes are also putting emphasis on the client's preferences. Elaboration on possible preferences of a client along with a mention of society's expectations was made earlier in this chapter. Even though these preferences and expectations are the ultimate concerns to be addressed, they are generally expressed in terms which can not always be directly translated into the engineering terms that structural designers use. However, a possible approach to perform this translation through the use of preference functions has been given in Chapter 3.

In this section, a two-stage approach to structural design which tries to satisfy all parties involved and which utilizes the methodologies developed earlier will be presented and illustrated. In the first stage, the currently accepted engineering representations ("engineering criteria") of society's demands, as well as simplified versions of the client's preferences, are used to obtain various structural configurations which are equivalent from the point of view of the engineering criteria. For example, each alternative design may be based on the approach obtained using the multiple performance objectives as explained in Chapter 3. In the second stage, the designs are assessed through a detailed socio-economic performance evaluation and the design which has the best performance is selected. However, since the designs are guaranteed from the first stage to meet societal demands, the selection could be carried out based on economic performances only, for example, by comparing the net asset values of the alternative designs.

This two-stage design process is believed to be a simplified version of the way designs are generally carried out in practice. In practice, first, various design options are developed considering the client's preferences as well as the code requirements, representing societal concerns and demands. Then, the design which satisfies the client's preferences in the best way is chosen as the final design.

It should be noted that the recommended approach is geared towards working with current practice and codes without seriously questioning their economic efficiency. A consequence of this "trust" is the possibility of settling with a suboptimal design, that is, a design which is not the actual "best" design. The suboptimality results from the less-than-perfect representation and handling of the interaction of society's and client's preferences using the intermediary "engineering criteria." In a perfect design decision-making environment, the ultimate expectations would be taken into consideration directly –not through some "translations"– and as such, with no artificial constraints over the search process forcing the solution to a suboptimal set. This would be a single-stage design process, similar to the one given in Chapter 3, in which specified performance objectives would be taken into consideration all at once. However, it is believed that neither the current state of knowledge on the issues related to design process, nor the current tools available for structural design and decision-making are sufficiently developed to allow such direct designs. For example, accurate net asset value estimation requires heavy computation which renders its use in the early stages of a design process quite infeasible.

4.3.2 Illustrative Example: Design of a Three-Story Steel Frame

In the first stage, which is design using engineering performance objectives, alternative designs for a three-story single-bay steel frame, similar to the one considered in Chapter 3, will be carried out. To address societal concerns, the life-safety performance objective, which is specified by design codes as the minimum performance requirement, is considered. Furthermore, the client's preferences are incorporated into the design process as a serviceability performance objective. These two performance objectives are related to the reliability of the structure in meeting specified response limit-states, and as such, they require integration of various uncertainties surrounding the designed structure into the process. In the second stage, which is actually an assessment of the designs obtained in the first stage, socio-economic performance of each design is examined in detail. For the current example application, however, this examination will take the form of an economic assessment since the designs were required to properly address the minimum societal concerns in the first stage, and that no further performance criteria related to society's expectations are considered in the second stage. The net asset value of each design will be taken as the evaluation measure. With the assumption that the client's attitude is a risk-neutral one, as well as with some simplifying assumptions regarding some terms in the formulation for the net asset value, the evaluation will be carried out on the basis of expected life-cycle costs. Details of the illustrative application of the two stage design procedure follows.

Three alternative designs meeting identical engineering reliability criterion will be developed for a three-story single-bay steel frame. The frame members are taken as I-sections made out of ASTM A36 steel with member cross-sectional dimensions as the design parameters to be determined. The length of the floor beams are set at 20 feet and the height of the story columns are set at 10 feet. The columns are assumed fixed at the base. Gravity loads are taken as 60 lb/ft^2 and 50 lb/ft^2 for the dead and live loads, respectively, for each floor and the roof. An out-of-plane tributary width of 100 inches is used for the gravity load calculations and full participation of dead and live loads is assumed in both gravity and lateral seismic loadings.

The first design is a bare moment resisting frame which is assumed to have 5% damping ratio for its fundamental mode. The other two alternatives are moment resisting frames with special viscous dampers installed to increase the fundamental-mode damping ratio to 10% and 20%, respectively. These devices are installed into the structural frame through pin connections, and they are assumed to act under lateral loads. The three alternative designs are illustrated in Fig. 4.5.

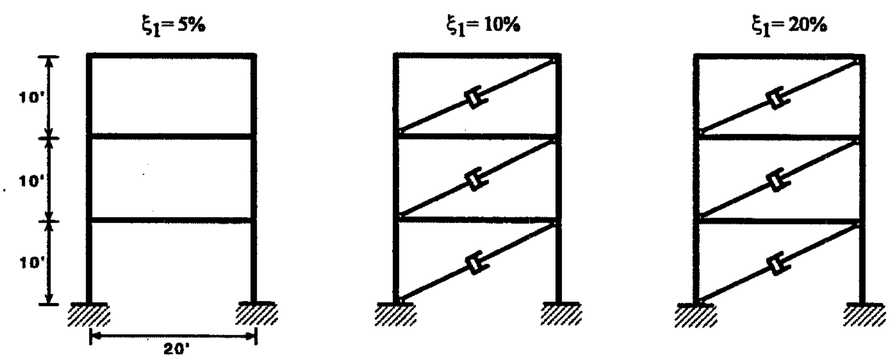

Figure 4.5: Three Alternative Designs

The seismic environment is such that a seismicity rate of $\nu = 1.0$ event per annum for earthquakes with magnitude $M \in [5.0,\ 7.7]$ and within the earthquake distance $R \in [0,\ 50]$ Km is expected. For the distribution of M and R, the probability models explained in Section 2.2.1 with the above seismic parameter ranges are used.

The considered design criteria are as follows:

· *Reliability Criterion*: the frame, first of all, should meet the minimum life-safety requirements implied by the codes to accommodate societal concerns. The "being life-safe after rare earthquakes" performance objective of *Vision 2000* (SEAOC 1995) is used to represent this concern. This "life-safety" objective is translated, in terms of lifetime interstory drift reliability, as "the probability of exceeding 1.5%

interstory drift ratio should not exceed 10% over a lifetime of 50 years." To include the client's preferences into the design process, the "fully-operational after frequent earthquakes" performance objective of *Vision 2000* (SEAOC 1995) is considered. This "serviceability" objective is translated, again in terms of interstory drift reliability, as "the probability of exceeding 0.2% drift ratio over a lifetime of 50 years should not exceed 68.5%." However, as it was observed and explained in Chapter 3, Section 3.4, the chosen "serviceability" objective governs the design of the bare three-story steel frame considered here. Therefore, it is the "serviceability" objective which is explicitly considered as a performance objective during the formal design, and the condition of meeting "life-safety" objective is verified internally.

·*Simplified Cost Criterion*: the frame should be designed to have minimum steel used for each structural configuration while meeting the reliability-based and the geometric criteria.

·*Geometric Criteria*: the dimensions of the I-sections, with 0.25 in plate thickness, used for the frame elements should be limited to the range [4.0 in, 16.0 in] for the flange widths and [5.0 in, 30.0 in] for the web depths.

These criteria are incorporated into the first stage of the design process using the preference functions developed in Chapter 3. The corresponding functions are given in Fig. 4.6. They are identical to those in Fig. 3.4 of Section 3.4 with $F_l = 0.685$.

The structural analyses used during the design cycles are based on the pseudo-velocity response spectra corresponding to the fundamental mode of the frame. The attenuation formula provided by Boore et al. (1993, 1994), which gives a probabilistic model for the spectra, is used. The cross-sectional dimensions of the beams and columns are chosen to be the design parameters over which the optimization is to be carried out. During structural response analyses in the first stage, no uncertainty given the pseudo-velocity response spectra is considered. However, in the second stage, a log-normal probability distribution model, which will be explained below, is assumed while estimating various response parameters of the structure to uncertain future earthquakes.

The three alternative designs obtained at the end of the first stage are given in

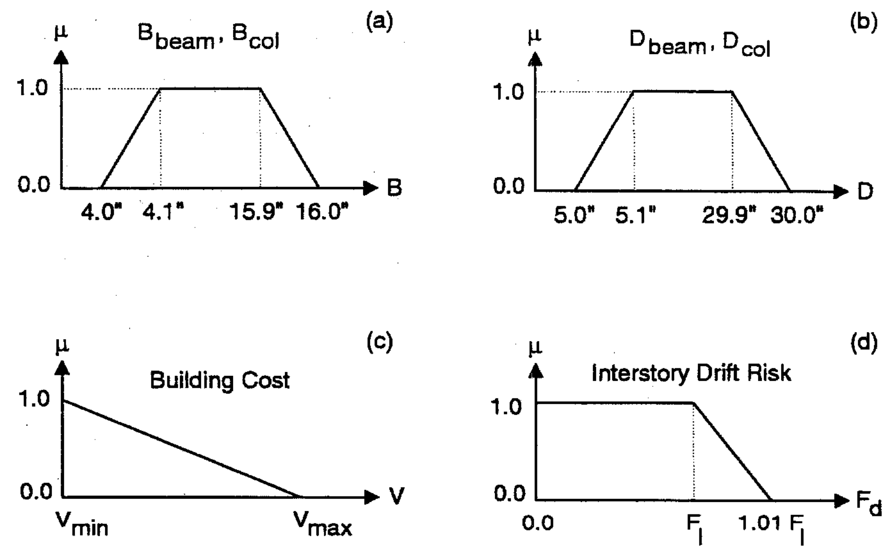

Figure 4.6: Preference Functions for the Considered Design Criteria

Table 4.1. As seen from the results, all three designs have identical reliability regarding the serviceability level and have very similar construction costs. In calculating the construction costs, the cost of steel I-sections is taken as \$5/lb ($\equiv$ \$1.41/in^3) which includes the labor cost. The cost of non-structural components is not explicitly included because it is the same for all designs and therefore, does not influence the comparative results. For the viscous dampers, the total cost of each damper, its accompanying diagonal brace and the installation is taken as \$2,000 per system for the 10% damping ratio case, and \$2,500 per system for the 20% damping ratio case. For the purposes of this illustrative example, these supplemental viscous damping systems are assumed to be robust and no failure of them is taken into consideration.

An important note is that, in this first stage of the design, the only design criterion that takes the lifetime of the structure into consideration is the reliability-based limit-state performance objective. This consideration, however, is done purely in an engineering sense and not translated into appropriate economic terms. The importance of considering the economic implications of events to take place during the lifetime of the structure will be seen in the next stage.

Table 4.1: Three Alternative Designs from First Stage

	MRF 5%	MRF 10%	MRF 20%
Criteria	Value	Value	Value
B_{beam} (in)	4.10	4.10	4.10
D_{beam} (in)	29.90	26.91	21.92
B_{col} (in)	4.10	4.10	4.10
D_{col} (in)	26.44	20.81	17.05
Vol (in^3)	12,913	11,362	9,788
F_d	0.6850	0.6850	0.6850
$E[C_{steel}]$ (1,000\$)	18.2	16.0	13.8
$E[C_{visc.\ damps}]$ (1,000\$)	0.0	6.0	7.5
$E[C_{cons}]$ (1,000\$)	18.2	22.0	21.3
Period	T_1=0.306 s	T_1=0.376 s	T_1=0.482 s

In the second stage, the three alternative designs are evaluated on the basis of their net asset value. A risk-neutral attitude for the final decision-maker is considered. It is assumed that the revenue to be obtained from the structure which these frames would go into is independent of the structural configuration of the frames. Therefore, the net asset value comparison of the designs is reduced to that of the expected life-cycle costs in which the regular maintenance costs are also ignored since they too can be assumed to be independent of the structural configuration. In other words, the sum of the expected construction costs and the expected lifetime losses will be used for comparison. The smaller the sum of these costs are, the better the design is.

Some assumptions similar to those used earlier in this chapter during the formulation for lifetime loss estimation are made: the earthquake arrivals follow a Poisson process; the frames are always "restored" to the same state after every destructive earthquake; and a continuous discount rate $r = 5\%$ is used to model the economic environment over a lifetime of $t_{life} = 50$ years, resulting in a present worth factor value $PWF = 18.36$.

The cost data used in computing the initial costs, that is, the construction costs, were given above. In calculating the expected future costs, only the direct costs are considered. No indirect losses or content damage are taken into consideration. The viscous damper installations are assumed to be robust enough not to require any maintenance or repair throughout the life-time of the structure.

The direct costs are related to any damage sustained by the structural and non-structural assemblies associated with the frames. The assembly information is as follows (Beck et al. 1999c):

· *Beam/column connections*: their fragility is related to the elastic demand-to-capacity ratio (DCR) at the end of each beam element. DCR is the sum of the ratios of the applied bending moment to the elastic moment capacity and the applied axial force to the elastic axial load capacity (SAC 1995). The fragility of the connections is taken to be log-normal distribution with a median DCR capacity $DCR_{cap} = 1.65$ and log-standard deviation $\beta = 1.72$. The relation is such that $P\{Connection\ failure \mid DCR\} = \Phi(log(DCR/DCR_{cap})/\beta)$ where Φ is the standard Gaussian cumulative probability function with zero mean and unit variance (Beck et al. 1999b). The expected cost of repairing a failed connection is taken as $\$28,200$. There are 6 beam/column connections in the frames.

· *Gypsum wallboards on metal studs*: their fragility is related to the peak interstory drift ratio (IDR) at the corresponding story of the frame. The fragility of wallboards is modeled as $LN(0.39\%,\ 0.17)$. This fragility is assumed to consider both repair and replacement cases. The corresponding expected cost of repairing/replacing the wallboards is $\$3.2/\text{ft}^2$. The total area of gypsum wallboards is 3x10x20 = 600 ft^2.

· *Ceiling panels*: their fragility is related to the peak diaphragm acceleration at the corresponding floor. The fragility of ceiling panels is modeled as $LN(2.9g,\ 0.8)$ where g is the gravitational acceleration. The expected cost of repairing ceiling panels is $\$2.21/\text{ft}^2$. The total area of ceiling panels is 3x20x(100/12) = 500 ft^2.

Fragility curves for the considered assemblies are given in Figs 4.7, 4.8 and 4.9. A couple of observations could be made by studying these curves. For example, it can be seen that there is large uncertainty in the beam/column connection fragility and that even for very small DCR values there exist a relatively high failure probability. This is due to the nature of the damage database on which regressions for the fragility curve was made (Beck et al. 1999b). The fragility curve for gypsum wallboards have a well-defined region which dominates the transition from no-failure to failure state. The fragility curve for the ceiling panels also indicates the existence of relatively large uncertainty in the estimates. However, this curve also indicates that the ceiling panels are very robust and are not likely to fail.

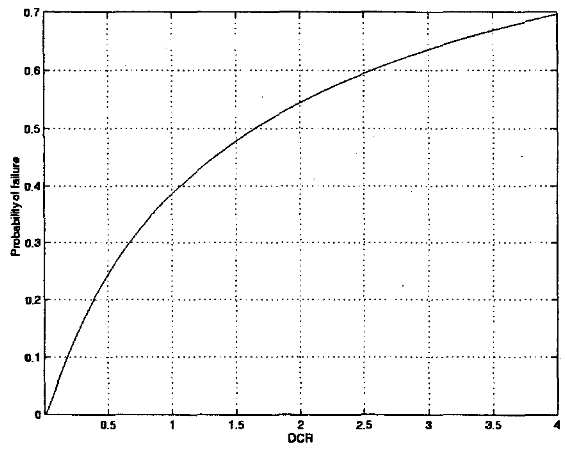

Figure 4.7: Fragility Curve for Beam/Column Connections

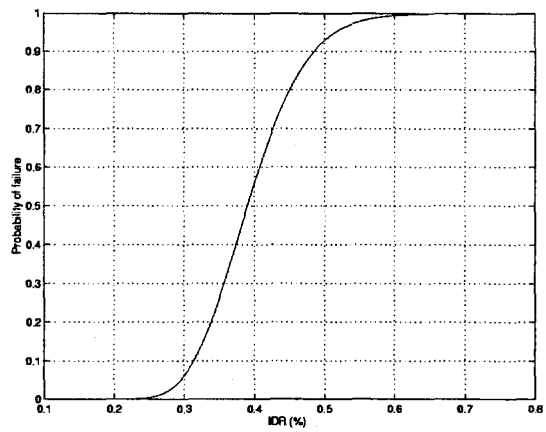

Figure 4.8: Fragility Curve for Gypsum Wallboards on Metal Studs

In the response estimation, only the fundamental elastic mode characteristics are used. However, modeling uncertainties are included in the response by assuming

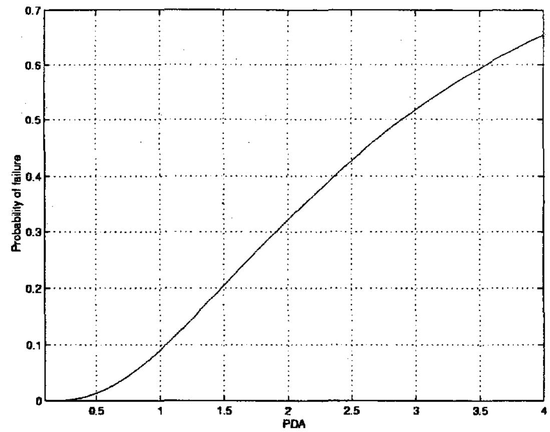

Figure 4.9: Fragility Curve for Ceiling Panels

that for a given seismic hazard S_V the response estimates have log-normal probability distributions. The fundamental mode linear analysis estimates are taken as the median response and the log standard deviations, which are measures of response dispersion due to non-linearities and higher mode contributions, are taken to be 0.15 for the demand-to-capacity ratio, and 0.35 for the interstory drift ratio and the peak diaphragm acceleration. These results are based on simulations performed using *DRAIN-2D* analysis program (Beck et al. 1999b, 1999c). So, for example, for the demand-to-capacity ratio (DCR) estimate at each S_V value, the DCR given by the fundamental-mode linear response analysis is taken as DCR_{median}. Then, the cumulative probability distribution of DCR is modeled as $P(DCR \mid Design,\ S_V) = LN(DCR_{median},\ 0.15)$ which is equivalent to $\Phi(log(DCR/DCR_{median})/0.15)$ where Φ is the standard Gaussian probability distribution function with zero mean and unit variance. The derivative of the considered distribution function with respect to DCR gives the corresponding probability density function. Similarly, the interstory drift ratio and the peak diaphragm acceleration estimates are modeled as $LN(IDR_{median},\ 0.35)$ and $LN(PDA_{median},\ 0.35)$, respectively.

The assessment results are given in Table 4.2. It is seen that the life-cycle cost estimates for the three designs are considerably different despite the fact that all three have identical reliability for being serviceable throughout their lifetime as obtained

Table 4.2: Comparison of the Three Alternative Designs in Second Stage

	MRF 5%	MRF 10%	MRF 20%
$E[C_{cons}]$ (1,000\$)	18.2	22.0	21.3
$E[C_{future}]$ (1,000\$)	133.2	101.5	76.7
$E[C_{life-cycle}]$ (1,000\$)	151.4	123.5	98.0

in the first design stage. The use of viscous dampers are found to be quite effective in reducing the damage-proneness. Of the three alternative designs, the design with a fundamental-mode damping ratio of 20% is found to be the most cost-effective one. It is also observed that the initial cost, that is, the construction cost, of a structure is not necessarily a good indicator of the expected future costs over the lifetime of the structure. The results illustrate the importance of considering the effect of uncertain future events when making design decisions. Basing design decisions on immediate costs might give improper emphasis on short-term concerns and might lead to poor overall performance in the long run. It should be noted that this example is for illustration purposes only, and the observations should not be taken as recommendations regarding design of structural systems.

Next, the issue of specifying the optimal reliability level for a chosen performance level is investigated. This is done by designing the bare moment resisting frame to meet various reliability levels at the 0.2% maximum interstory drift ratio, the serviceability limit-state, over its lifetime of 50 years and then comparing the corresponding expected life-cycle costs.

In Fig. 4.10, the expected life-cycle costs of designs with different lifetime serviceability reliability levels are given. The curve labeled "size constrained" corresponds to the cases that have the preferences on structural member dimensions as specified earlier (see Fig. 4.6(a), (b)). This curve shows that the optimal reliability level to be specified for 0.2% interstory drift ratio is 22.5% since the lowest expected life-cycle cost is given by the design corresponding to this reliability level. Reliability levels below or above this value yield higher life-cycle costs. To understand why the

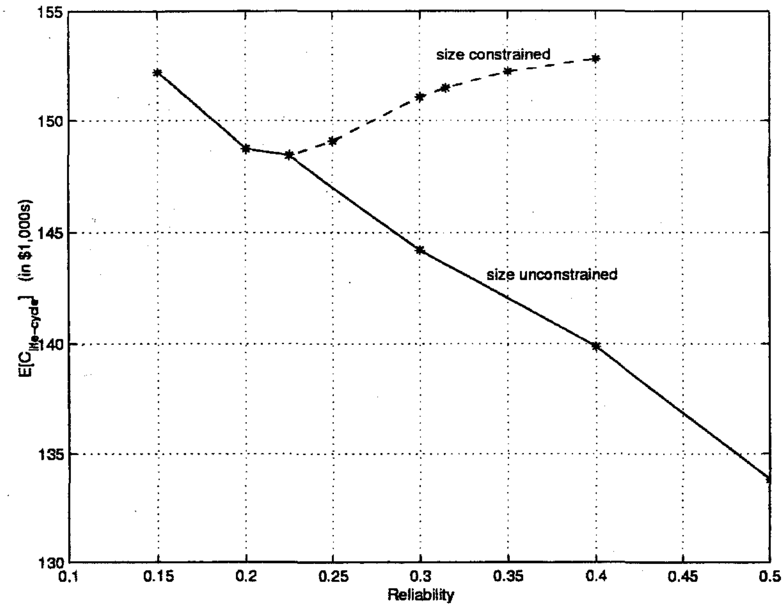

Figure 4.10: Reliability versus Expected Life-Cycle Cost

life-cycle costs do not decrease consistently with increase in the reliability level, it is necessary to study the optimal structural member dimensions obtained. The results at reliability levels used to plot the "constrained" curve in Fig. 4.10 are given in Table 4.3.

Table 4.3: Constrained Optimal Designs; Costs are in 1,000 $

Reliability	15%	20%	22.5%	25%	30%	31.5%	35%	40%
B_{beam} (in)	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
D_{beam} (in)	28.4	29.5	29.8	29.9	29.9	29.9	29.9	29.9
B_{col} (in)	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
D_{col} (in)	22.0	22.8	23.4	24.2	26.0	26.5	27.7	29.4
Vol (in^3)	11,831	12,194	12,357	12,514	12,826	12,919	13,137	13,453
$E[C_{cons}]$	16.7	17.2	17.4	17.6	18.1	18.2	18.5	19.0
$E[C_{future}]$	135.5	131.5	130.9	131.4	133.0	133.2	133.7	133.8
$E[C_{life-cycle}]$	152.2	148.7	148.3	149.0	151.1	151.4	152.2	152.8

It is seen that at the 25% reliability level, the web depth of the beams, D_{beam}, reaches the maximum value, 29.9 in, above which the corresponding preference value for this geometric design criterion decreases rapidly (see Fig. 4.6(b)). Thereafter, during the design optimization, the beam cross-sectional dimensions become fixed and therefore, the columns are required to make up for the extra demand from the increasing reliability level. It should be noted that the flange widths of the beams and columns, B_{beam} and B_{col}, are always fixed at 4.10 in. The reason for this is that the trade-off between member volumes and cross-sectional moment of inertia demands the lowering of the flange widths to their practical minimum and adjusts the web depths to take the load. The fixing of beam cross-sectional dimensions above 22.5% reliability level means that up to that particular reliability level, the search for the best combination of beams and columns in the design space is not restricted by the limits on the member sizes. But for the designs with reliability levels just over 22.5% or higher, the search becomes constrained by the maximum allowable beam dimensions. The less than perfect combinations of beams and columns result in higher expected future losses even though the structures have higher lifetime interstory drift reliability. The main reason for the increase in the expected losses is found to be due to the increase in failure probabilities of the beam/column connections at upper stories.

To verify the conclusion regarding the deleterious effect of "constrained" sizes, the upper limit for D_{beam} is increased to 39.9 in and the frame is redesigned for various reliability levels. The results from these "unconstrained" designs are given in Table 4.3.2, and the corresponding reliability level (probability of non-exceedance) versus expected life-cycle costs are plotted in Fig. 4.10. It is observed that once the search for optimal beam and column dimensions combination is set free of upper geometric constraints, the expected future costs decrease with increasing design reliability level. Besides, the decrease in the expected future losses is greater than the increase in the initial construction costs and therefore, the expected life-cycle costs are also found to decrease with increasing reliability level.

In conclusion, finding the means to quantitatively represent the expectations and preferences of all parties associated with a designed structure is the first challenge in design. Furthermore, these representations have to be integrated in such a way that trade-off between conflicting preferences may be performed. The two-stage approach

Table 4.4: Unconstrained Optimal Designs; Costs are in 1,000 $

Reliability	15%	20%	22.5%	30%	40%	50%
B_{beam} (in)	4.10	4.10	4.10	4.10	4.10	4.10
D_{beam} (in)	28.4	29.5	29.8	31.1	32.5	34.2
B_{col} (in)	4.10	4.10	4.10	4.10	4.10	4.10
D_{col} (in)	22.0	22.8	23.4	24.6	26.2	27.5
Vol (in^3)	11,831	12,194	12,357	12,804	13,350	13,882
$E[C_{cons}]$	16.7	17.2	17.4	18.1	18.8	19.6
$E[C_{future}]$	135.5	131.5	130.9	126.2	121.1	114.2
$E[C_{life-cycle}]$	152.2	148.7	148.3	144.3	139.9	133.8

to structural design suggested in this section builds upon the strategies developed earlier in this chapter and the previous chapters. In the first stage of the approach, the design problem is cast as a multi-criteria based optimization using engineering design criteria to represent performance expectations and preferences. These criteria are chosen to reflect the minimum requirements, such as society's demands as conveyed by code regulations, as well as constraints due to more obvious restrictions such as architectural or constructional limitations and simplified versions of client's preferences. The result of the first stage is a set of alternative designs. These designs have different structural configurations optimized to satisfy the specified design criteria in their respective best way. In the second stage, the alternative designs are evaluated through comprehensive economic and loss-estimation studies, for example, through estimating the life-cycle costs due to uncertain future earthquakes, or other evaluation measures requested by the client. The expensive computational demand is the reason why such detailed evaluations are not integrated into the first stage of the design process but carried out as an assessment over a smaller set of alternative designs in the second stage. It is believed that the two-stage approach to design is an intuitive and straightforward way to incorporate different expectations and preferences into the design decision-making process.

4.4 Implementing the Economic Performance Objectives in Mitigation Analysis for a Braced Frame

The formulation for net asset value, NAV, given in Eqn. (4.1) in Section 4.2.1 can be extended for an existing structure where possible strengthening strategies are being considered as part of a mitigation effort, or where the structure has been damaged by a recent earthquake and upgrading is being considered as a part of the recovery process. Considering both of these cases, the NAV may be restated as

$$\begin{aligned} NAV = \ & Discounted\ net\ income\ stream \\ & -\ Present\ value\ of\ future\ earthquake\ losses \\ & -\ Present\ value\ of\ current\ earthquake\ loss \\ & -\ Present\ value\ of\ upgrade/rebuild\ cost \end{aligned} \tag{4.37}$$

The additional terms in Eqn. (4.37) need some explanation. The "current earthquake loss" is taken to be the cost of bringing the building back to its pre-earthquake state plus the income loss during the repairs, or the cost and income loss for demolition, depending on the recovery action. Similarly, the "cost of upgrade/rebuild" is the cost of upgrading the structure beyond its original state, or the cost of rebuilding it if it is demolished, plus the income loss during the upgrade or reconstruction. However, depending on the case and type of action considered, various terms in Eqn. (4.37) might be inactive. For example, if the intention is to perform a "mitigation" study only, the third term, "present value of current EQ loss" is not relevant. For a post-earthquake study, when the case is a "recovery-only" action, the present value of future earthquake losses and the cost of upgrade are not considered when computing the net asset value. But these terms should be considered if the case is to perform a "recovery-and-upgrade" action (Beck et al. 1999b, 1999c).

The use of the proposed socio-economic performance objective approach given earlier in this chapter will be illustrated through an example of finding an optimal design in a mitigation study, which corresponds to the upgrading of a three-story single-bay moment-resisting steel frame by adding steel cross-braces. The frame, for which a sketch is given in Fig. 4.11, has identical column members (each 10 feet high) and identical beam members (each 20 feet long). The columns are fixed at the foundation level. The tributary width of the frame is 20 feet; the dead-load is 60 lbs/ft^2 and the live-load is 50 lbs/ft^2.

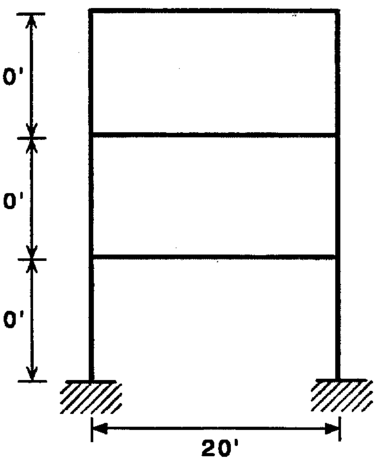

Figure 4.11: Example Three-Story Steel MRF

First, the frame members are designed using the general framework with the design evaluation approach which considers multiple performance objectives as developed in Chapter 3. The design objective is to have the minimum steel volume in the members while satisfying a life-safety condition. This condition is interpreted to require the maximum risk of exceeding 2% interstory drift ratio in any story over 50 years must be limited to 10%.

The seismic environment is such that a seismicity rate of $\nu = 0.5$ events per annum for earthquakes with magnitude $M \in [5.0,\ 7.7]$ and within the earthquake distance $R \in [0,\ 50]$ Km is expected. For the distribution of M and R, the probability models explained in Section 2.2.1 with the above seismic parameter ranges are used. For the truncated Gutenberg-Richter relationship for M, the model parameters a and b are set as 4.7 and 1.0, respectively.

The optimal member cross-sectional dimensions for the base-frame are obtained using the approach explained in Chapter 3. These results have been verified using the *CODA* software which was developed for a CUREe-Kajima Phase II project (Beck et al. 1996, Beck, Papadimitriou, Chan, and Irfanoglu 1997). The beams and columns are restricted to have dimensions in the range [4.0 in, 16.0 in] for the flange widths and [5.0 in, 30.0 in] for the web depths. The associated preference functions are kept same as the ones used throughout this study (see, for example, Fig. 3.3(a), (b)). The optimal member dimensions over the continuous design parameter space using 0.25 in thick ASTM A36 steel plates are found as:

·*Beams*: Flange width = 4.10 in and web depth = 15.14 in; cross-sectional area A = 5.71 in^2, second moment of area I = 179 in^4, and, section modulus S = 23.6 in^3

·*Columns*: Flange width = 4.10 in and web depth = 10.23 in; cross-sectional area A = 4.48 in^2, second moment of area I = 70.2 in^4, and, section modulus S = 13.7 in^3

With these section dimensions, the amount of steel used in construction is minimized while the reliability-based performance objective is met right at its limit. That is to say, the resulting frame has a risk of 10% of exceeding 2% interstory drift ratio over 50 years. However, the member dimensions obtained from the continuous optimization are translated into available steel I-sections in the market (AISC 1989) with closest section properties, resulting in the following sections:

·*Beams*: W14x22 [A = 6.49 in^2, I = 199.0 in^4, S = 29.0 in^3]

·*Columns*: W10x17 [A = 4.99 in^2, I = 81.9 in^4, S = 16.2 in^3]

The base frame with these members has a 50-year risk of 8.4% having maximum drift ratio exceed 2% in the specified seismic environment.

Since it is assumed that a base structure that meets the simplified reliability requirement already exists, the decision problem is cast as a mitigation analysis study of the given frame within the context of life-cycle costs, where a lifetime of 30 years is considered. In other words, it is assumed that 20 years from its lifetime has already passed, and the frame is considered to have the same characteristics as it was first built. However, an upgrading action is requested to improve it. The problem of finding the best upgrading strategy is addressed by minimizing the expected life-cycle cost over the specified lifetime of 30 years, that is, minimizing the present value of uncertain future earthquake losses plus the installation cost of the upgrading.

The chosen upgrading scheme is installation of square tubular steel cross-braces that are made of ASTM A36 steel and have a wall thickness of 0.4 in. The braces are to be installed in one or more stories and they will be attached to the existing frame through pin connections. Should it be decided to install a brace in a story, its section dimension is limited to be within [5.86 in, 7.33 in] due to a global buckling concern and the maximum allowable width-to-thickness ratio due to a local buckling concern as specified in the UBC design code (ICBO 1994). The brace material cost is set as \$0.70/lb (\$0.20/in^3) and a fixed labor cost for installation, fire proofing, etc. of \$2,000 per pair of braces in a story is chosen.

The structural and non-structural assemblies considered for the frame are similar to those considered in the previous example and are as follows (Beck et al. 1999c):

· *Beam/column connections*: their fragility is related to the elastic demand-to-capacity ratio (DCR) at the end of each beam element. The fragility of the connections is taken to be a log-normal distribution $LN(1.65,\ 1.72)$ with a median DCR capacity of 1.65, and a log-standard deviation of 1.72. The expected cost of repairing a failed connection is taken as \$28,200. There are 6 beam/column connections in the frame.

· *Gypsum wallboards on metal studs*: their fragility is related to the peak interstory drift ratio at the corresponding story of the frame. The fragility of wallboards is modeled as $LN(0.39\%,\ 0.17)$. This fragility is assumed to consider both repair and replacement cases. The corresponding expected cost of recovering the wallboards is \$3.2/ft^2. The total area of gypsum wallboards is 3x10x20 = 600 ft^2.

· *Ceiling panels*: their fragility is related to the peak diaphragm acceleration at the corresponding floor. The fragility of ceiling panels is modeled as $LN(2.9g,\ 0.8)$ where g is the gravitational acceleration. The expected cost of repairing ceiling panels is \$2.21/ft^2. The total area of ceiling panels is 3x20x20 = 1,200 ft^2.

The fragility functions are same as the ones used for example in the previous implementation, and plots of them are given in Figs. 4.7, 4.8 and 4.9. As before, no content damage is taken into consideration during the designs. Same approach to probabilistic response analysis as in the last example is used. The modeling uncertainty in the associated response parameters are treated identically. That is, the response estimates given by the fundamental-mode linear response analysis is taken as the median of the considered parameters (such as DCR_{median}, IDR_{median}, PDA_{median}). The corresponding log-normal probability distributions for the elastic demand-to-capacity ratio, peak interstory drift ratio, and peak diaphragm acceleration estimates are, respectively, $LN(DCR_{median}, 0.15)$, $LN(IDR_{median}, 0.35)$, $LN(PDA_{median}, 0.35)$.

For the lifetime earthquake losses, only the direct losses, that is, cost of repairing the damaged assemblies, are considered. It is assumed that the duration to perform the repairs is either minimal or that the repairs do not interfere with building operations. Therefore, the indirect losses due to loss-of-use are not be considered.

A discount rate of 5% per annum is assumed to represent the economic environment over the $t_{life} = 30$ years of the structure. Taking the seismicity rate of $\nu = 0.5$ events per annum as stated above, the present worth factor PWF to be used in Eqn. (4.36) is equal to 7.75 as found by direct substitution from Eqn. (4.35).

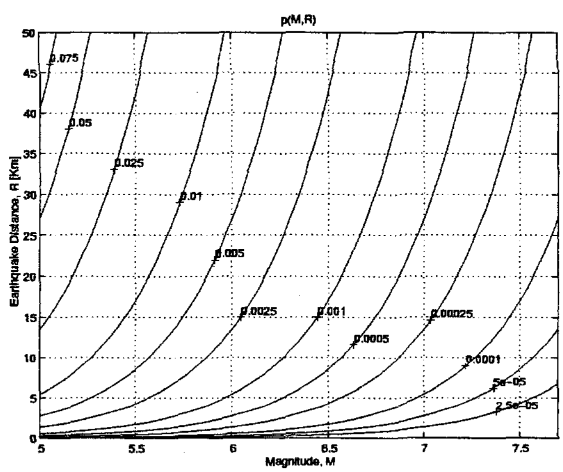

Figure 4.12: Probability Density Function, p(M,R)

Fig. 4.12 gives the probability of having a particular earthquake magnitude M and distance R pair. The median pseudo-acceleration spectrum value at the fundamental period of the base frame, 1.187 s for each M and R pair is given in Fig. 4.13. In Fig. 4.14, the contour plot of the expected direct losses for each (M, R) pair is given. It is interesting to note that Fig. 4.14 indicates existence of relatively high expected direct losses even for small earthquakes occurring at large distances. The main source for these high estimates is found to be the expected losses from damaged beam/column connections. As mentioned earlier, the fragility curve for the beam/column connections contains relatively high probabilities of failure even at low elastic demand-to-capacity ratio values. The result of this feature of the connection fragility is the existence of relatively high expected losses even from small earthquakes at large distance. Fig. 4.15 is the convolution of Figs. 4.12 and 4.14, and it shows the contribution of each earthquake (M and R pair), scaled by the associated probability, in the overall loss estimation. The integral sum over M and R of the values shown in Fig. 4.15 gives the expected direct loss given the occurrence of an uncertain earthquake. This expected direct loss is equal to \$10,630. Multiplying this value with the PWF gives the present value of the expected future losses for the original base frame as \$82,440.

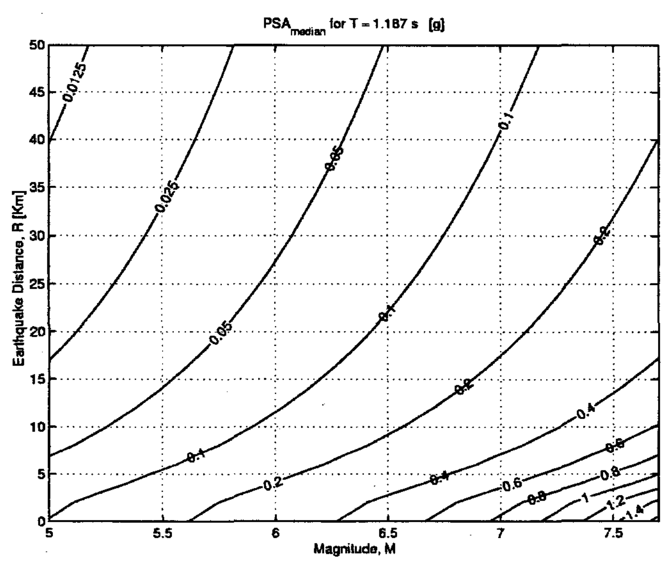

Figure 4.13: Median Pseudo-Acceleration Spectral Value, $PSA_{median}(T_1 \mid M,\ R)$, for 1.187 s Period, and 5% Damping Ratio

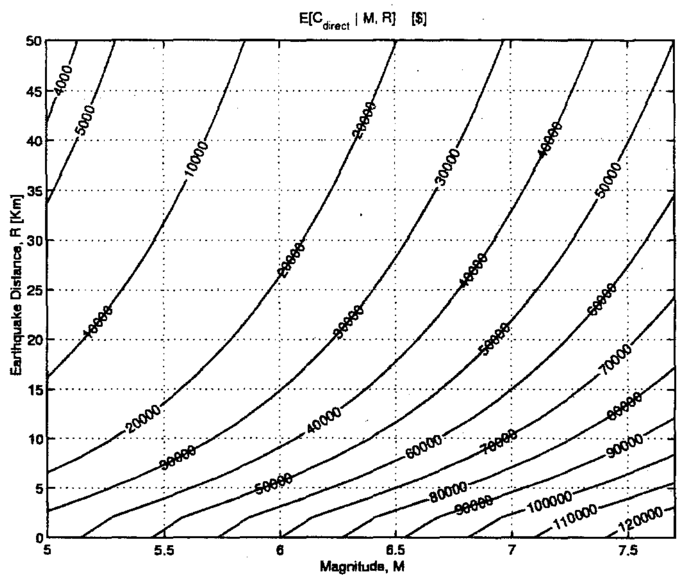

Figure 4.14: Expected Direct Losses, $E[C_{direct} \mid M,\ R]$

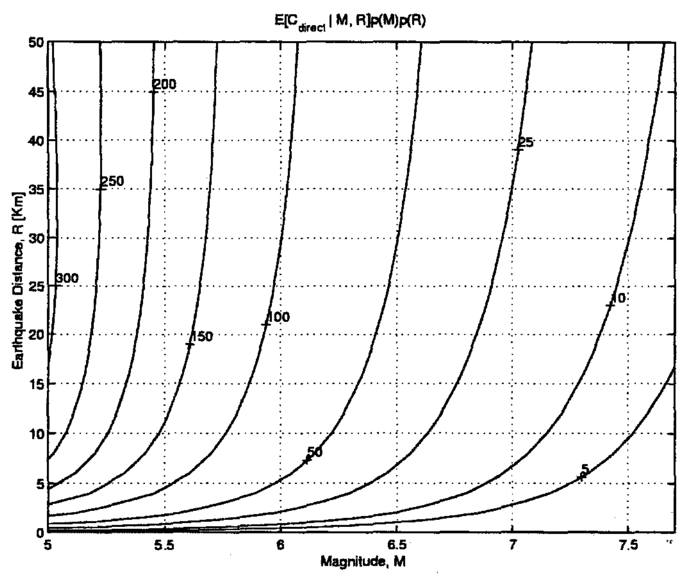

Figure 4.15: $E[C_{direct} \mid M,\ R]p(M)p(R)$

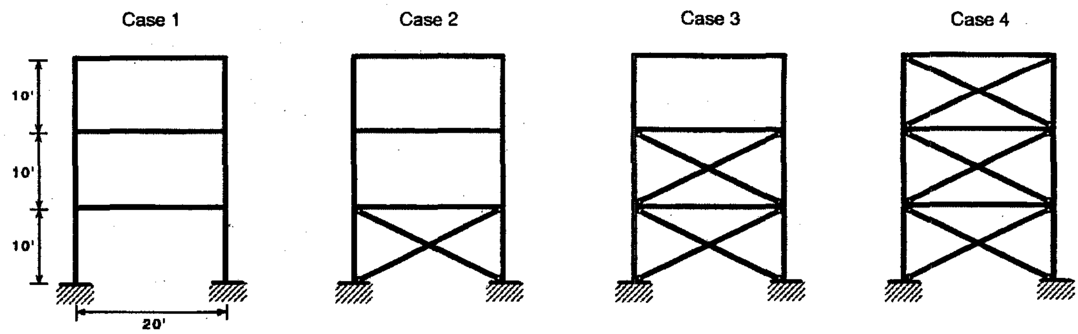

Figure 4.16: Considered Upgrading Strategies

For comparison of various upgrading strategies with the frame as it is, various combinations of cross-bracing are studied. The upgrading strategies are numbered as shown in Fig. 4.16.

Table 4.5: Upgrade Example with Minimum Brace Dimensions per UBC

	1	2	3	4
Story 1 Brace Size (in)	0	5.9	5.9	5.9
Story 2 Brace Size (in)	0	0	5.9	5.9
Story 3 Brace Size (in)	0	0	0	5.9
Fundamental Period (s)	1.187	0.869	0.504	0.203
$E[C_{wall} \mid EQ]$ (1,000$)	0.2	0.1	0.1	0.0
$E[C_{conn} \mid EQ]$ (1,000$)	10.5	9.1	5.6	0.0
$E[C_{ceil} \mid EQ]$ (1,000$)	0.0	0.0	0.0	0.0
$E[C_{future}]$ (1,000$)	82.4	71.5	43.6	0.3
$E[C_{brace}]$ (1,000$)	0.0	2.9	5.9	8.8
$E[C_{life-cycle}]$ (1,000$)	82.4	74.4	49.5	9.1

The tabulated results for the cases using the minimum and the maximum brace dimensions specified by the UBC requirements are given in Tables 4.5 and 4.6. Only the extreme dimensions of the feasible brace dimension space are considered as it has been found that the results are bounded by them and the trend is of monotonic nature in between these bounds.

The results show that installation of minimum-size braces in all three-stories is

Table 4.6: Upgrade Example with Maximum Brace Dimensions per UBC

	1	2	3	4
Story 1 Brace Size (in)	0	7.3	7.3	7.3
Story 2 Brace Size (in)	0	0	7.3	7.3
Story 3 Brace Size (in)	0	0	0	7.3
Fundamental Period (s)	1.187	0.867	0.502	0.196
$E[C_{wall} \mid EQ]$ (1,000\$)	0.2	0.1	0.1	0.0
$E[C_{conn} \mid EQ]$ (1,000\$)	10.5	9.1	5.5	0.0
$E[C_{ceil} \mid EQ]$ (1,000\$)	0.0	0.0	0.0	0.0
$E[C_{future}]$ (1,000\$)	82.4	71.4	43.4	0.1
$E[C_{brace}]$ (1,000\$)	0.0	3.2	6.4	9.6
$E[C_{life-cycle}]$ (1,000\$)	82.4	77.8	49.8	9.7

the best upgrading strategy in terms of minimizing expected life-cycle costs. Such a cross-brace installation stiffens every story so much that it practically eliminates damage in the structure. However, it should be noted that in this example, simplified response and loss estimation analyses have been performed. For example, only the fundamental mode is considered in the linear response analyses. However, for braced frames, especially for irregular ones as in the Cases 2 and 3 in the upgrade schemes considered, contributions from higher modes can be very significant and should be taken into consideration in a more rigorous study. Furthermore, in the current simple study, various issues associated with braced frames, such as the possibility of column base uplift and failure of brace-frame connections have not been considered because analytic fragility curves associated with these failures are not readily available. Therefore, it is likely that the losses to be suffered in the considered braced frames are underestimated. In any case, the above results are for illustration purposes only and should not be taken as implying any recommendation about upgrading using braces.

Another observation from the results is that the direct losses are governed by the damage to beam/column connections, the only structural components considered. The non-structural components are found to be either much less costly to repair or replace (the gypsum wallboards) or very robust (ceiling panels), and therefore, their contribution to the life-cycle cost estimates are minimal. It should be recalled that one basic assumption was that the structure would be recovered to its pre-event state after every damaging earthquake. As such, this assumption ignores the possible design improvements in, for example, beam/column connections which often occur with accumulated research and increased understanding after such events. It has been found that the braces are not likely to be damaged. The brace stresses are checked against the stress requirement in compression and tension elements per AISC Allowable Stress Design (AISC 1989) and under the response spectrum specified by UBC (ICBO 1994) for seismic zone 4 (effective peak ground acceleration 0.4 g) and reduced by a factor R_w=12. A probabilistic study also found the probability of a brace buckling over the lifetime of 30 years to be very small. In any case, the pseudo-dynamical response spectrum approach as used in this study does not allow any detailed study of the consequences of a possible brace failure. If more realistic and accurate damage estimates are sought, simulations with fully non-linear, dynamic time-history input and analyses that consider progressive damage need to be considered (for example, Hall 1998; Carlson 1999).

4.5 Concluding Remarks

In this chapter, a rational approach to evaluate a design using socio-economics based objectives is developed with an emphasis on economic considerations. A "net asset value" formulation is developed to compare different designs. In obtaining the expression for net asset value, both the revenues to be generated from the structure over its intended lifetime and the life-cycle costs expected over this duration are considered. Expressions for both new structures and existing structures are given. The approach is demonstrated, for the case of new construction, through developing alternative designs for a three-story single-bay steel structure using a two-stage design procedure, and for the case of an existing structure, through evaluation of various cross-bracing upgrading options for a simple moment-resisting steel frame within the context of a mitigation study.

The ultimate goal of a structural design process is to obtain an optimal design that meets all the diverse social, economic, and engineering performance objectives demanded by various parties related to the designed structure. Unfortunately, many of these objectives conflict with one another, and there is no design that satisfies perfectly all of the objectives at once. The difficult task of finding a best-compromise design is hindered not only by the wide range of objectives, but sometimes with issues at much more fundamental levels, such as the lack of quantified models for various objectives, especially those related to societal expectations. Converting the qualitative performance expectations into analytical and quantitative forms requires more than the single-handed effort of the design engineers involved in the decision-making process. With increasing interaction between social and economic aspects of life, the task has not only been made more complex but also become more essential. Tools being used to estimate lifetime consequences of a design are still in their early stages of development. Since interpretation of structural damage in terms of economic losses is actually adding another layer with uncertainty into the evaluation process, care must be taken not to make the matters worse. Common assumptions regarding the economic environment or attitude to risk might help reduce some of the difficulties encountered during the formulation. However, validity of these assumptions can not be guaranteed for all circumstances. Improving the extent and accuracy of performance estimation tools is of utmost importance to understand and reduce uncertainties surrounding a designed structure, be they social, economic, or other.

Chapter 5 Conclusion

5.1 Conclusions

In this study, an optimal structural design framework that rationally treats multiple performance objectives and that allows incorporation of uncertainties associated with a designed structure is given. Various design examples have been presented to illustrate the use of the framework. The implementations have been made for the case of structural design against uncertain seismic loads. However, the framework has the flexibility to allow consideration of other types of loads, as long as proper probabilistic models for the loads are provided.

There are various sources of uncertainties that need to be considered during structural design and therefore, to be included in any rational decision-making framework. The uncertain loads are often considered as the primary source of uncertainty. Probabilistic seismic hazard modeling can be used to incorporate this kind of uncertainty into a design framework.

The second source of uncertainty is the modeling uncertainty and it is generally overlooked. The imperfect state of knowledge regarding the best parameter values to represent the structure in a chosen structural model, and the imperfect models and methods used in the design process to estimate, for example, various structural response quantities add to the modeling uncertainty. A probabilistic response analysis approach which borrows from ongoing research is used in an attempt to incorporate the modeling uncertainty into the design process in a simple way.

The third general source of uncertainty in a performance-based structural design is associated with economics. Simplifying assumptions regarding the economic environments have been made and the question of attitude towards risk within the context of performance-based structural design is studied only briefly.

The methodology builds upon the conventional structural design process where the design iterates through analysis, evaluation, and revision stages. In the analysis stage, probabilistic response analysis tools are used to compute the reliability-based performance parameters. For the evaluation stage, two approaches have been considered: a multi-criteria one using concepts similar to those found in multi-criteria decision theory, and a socio-economics one. Both approaches allow the design of a structure to be converted into an optimization problem and revisions are made to find the design which is best in some chosen sense.

In the first approach, preference functions expressing the degree of satisfaction of multiple design criteria and performance objectives are aggregated to evaluate the overall performance of a structure. Special attention has been given to the treatment of reliability-based performance parameters. This first approach is implemented in a single-stage design procedure.

In the socio-economics approach, performance objectives are quantified through reliability-based evaluation measures for societal preferences, and through a net asset value formulation for the economic objectives. These objectives allow interaction of societal demands and client preferences, short-term and long-term, in a formal and consistent way. A recently developed assembly-based vulnerability approach is used to estimate building-specific losses from uncertain seismic events over the lifetime of a structure. These uncertain future losses address the long-term concerns and play a critical role in the net asset value of a structure. However, it is only recently that they have started to be considered in performance-based design applications. The socio-economics approach to design evaluation has been implemented through a practical two-stage design procedure which allows interaction of conventional engineering-based design evaluation measures with socio-economics based performance criteria.

Even though attention has been primarily given to the application of the socio-economic objectives for the design of new structures, a brief example in the form of a simple mitigation analysis has been given to point to its possible use for existing structures. In fact, some of the tools mentioned and used in this study have been implemented in a project to develop decision support tools for business recovery after earthquakes or for mitigation actions (Beck et al. 1999b, Beck et al. 1999c). In that implementation, the socio-economics based performance parameters and objectives are used to carry out cost-benefit and feasibility studies on possible rehabilitation and upgrading actions.

The assumptions made in this study during the implementation and examples should not be taken as assumptions rooted in the framework. Particular choices for various methods and models have been made in this exploratory work to avoid heavy computational effort and complicated examples that do not readily illustrate the essential features of the methodology. The modular nature of the framework allows individual aspects to be modified and improved.

As it stands, the developed methodology creates a medium through which various issues associated with structural design can be studied. For example, it could be used to compare performance of different structural configurations or materials. Sensitivity analyses to find the appropriate design parameters and the criteria which control the design could be carried out. Sensitivities to different uncertainty sources and levels could also be investigated.

Overall, the general methodology and the framework explained in this study provides a rational means to carry out structural design in the presence of uncertainties and with consideration of multiple performance objectives.

5.2 Future Work

A number of challenging issues need to be addressed to improve the current implementation of the methodology, and to further its application area.

Clearly the analysis stage in a performance-based structural design framework contains the most critical part of the whole machinery, especially for reliability-based performance criteria which involves a probabilistic response analysis. In this study, simple linear dynamics have been utilized to do this analysis. For some examples, modeling uncertainties are included in the analysis using results from simulations. It is not yet known whether these results can be generalized. For low level seismic excitations, which are related more with serviceability limit-states, the approach may work well since the structure would behave linearly at those shaking levels. However, for critical limit-states such as the ones associated with life-safety criteria, clearly the structural response goes into non-linear inelastic range, and linear dynamics based methods are likely to give poor response estimates. To improve accuracy in the estimates, and accuracy is crucial in performance-based design, one needs to use realistic models and analysis methods. For example, full non-linear time-history analyses of the structural response using extensive finite-element methods might be needed. Such an approach should allow modeling of progressive failures leading to structural deterioration. However, these methods require heavy computational effort, too, and therefore, they are not feasible for cases where the dimension of the uncertainty space is large. In design problems with realistic uncertainty considerations, the interactions between uncertain variables in the conceptual design are too complex to allow any analytical treatment. In this case, some type of Monte Carlo simulation may be used. In each such simulation, computationally expensive time-history analyses need to be carried out. Therefore, it is important to use an advanced variance-reducing simulation method to greatly reduce the required number of simulations (for example, Au and Beck 1999). Currently, the main limitation for realistic practical applications of reliability-based multi-criteria design frameworks is the computational effort required to obtain accurate results.

The treatment of the modeling uncertainties have been carried out in a rather simplified manner. The parameter and prediction-error uncertainties which are classified under modeling uncertainties have been incorporated into the applications as a single combined uncertainty. For every response parameter, one simple probability distribution is considered. Certain assumptions have been made regarding the probability models and their parameter values. Modeling the uncertainties in this way may not be acceptable for all types of structural design configurations or performance objectives, and further study in this area is needed. Another issue which will benefit from a detailed analysis including modeling uncertainty is related to choosing the proper approach in specifying performance reliabilities. Currently, in practice, the reliabilities are usually defined at the seismic hazard level ("design earthquake level") and not at the performance reliability level. However, when modeling uncertainties are present, which is always the case due to the fact that analytical models of physical phenomena are imperfect, it is not clear how accurate prediction of performance reliabilities could be made using a specified hazard curve, such as response spectra specified in current design codes and recommendations. It is believed that consideration of a complete probabilistic description of the seismic hazard is necessary to predict performance reliability of a structure in a realistic and accurate manner. Further study of this issue is highly desirable.

There is yet another area that needs to be studied further, and it is related to the performance objectives that are based on economics. In the development of a methodology to address economic objectives, the decision-maker's attitude to risk is assumed to be a risk-neutral one. It is believed that this assumption is valid when the ultimate decision-maker, that is, the designer's client, does not have a high risk exposure, in the economic sense, from losses or damage of the designed structure. However, it is possible that the client might have a risk-averse attitude. The useful simplifications that arise from the risk-neutral assumption would then no longer be possible. In fact, even for risk-neutral attitude cases, a quantitative assessment of the economic uncertainties might be desirable. This would be the case, for example, when the client does not want to base decisions only on point-estimates, such as the expected value of the economic performance measure, but also requires some measure of the associated uncertainty. A higher expected performance value is not always a better one if it is achieved at the expense of increased uncertainty that is not quantified. Detailed studies of uncertainties stemming from economic performance as well as the affect of risk attitude on design decisions would be of great interest for performance-based design frameworks.

On a more general tone, it is believed that the depth and the scope of the methodology could be greatly increased if it could be connected with regional impact and loss-estimation methodologies (NRC 1989; EERI 1997). These methodologies would provide crucial information, especially regarding the socio-economics based objectives used in this study. Therefore, such an interaction should at least improve the accuracy of the loss estimates within the current framework.

In conclusion, the presented methodology for structural design under seismic risk using multiple performance objectives contains concepts applicable to a wide range of problems. Since it is a general framework, in return, it can accept research from all the diverse fields related to performance-based structural design.

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