Seismic Performance Evaluation for Steel Moment Frames

Seung-Yul Yun, M.ASCE[1]; Ronald O. Hamburger, M.ASCE[2]; C. Allin Cornell, M.ASCE[3]; and Douglas A. Foutch, M.ASCE[4]

Abstract: A performance prediction and evaluation procedure based on nonlinear dynamics and reliability theory is presented. It features full integration over the three key stochastic models: ground motion hazard curve, nonlinear dynamic displacement demand, and displacement capacity. Further, both epistemic and aleatory uncertainties are evaluated and carried through the analysis. A suite of uncertainty analyses are input to the procedure such as period, live load, material properties, damping, analysis procedure, and orientation of the structure. Two limit states are defined instead of the traditional single state. The procedure provides a simple method for estimating the confidence level for satisfying the performance level for a given hazard. The confidence level of a post- and a pre-Northridge nine-story building for a given hazard level is calculated using the procedure described in the paper. New steel moment frame buildings are expected to perform much better during major earthquakes than existing buildings designed and built with older technologies.

DOI: 10.1061/(ASCE)0733-9445(2002)128:4(534)

CE Database keywords: Seismic response; Earthquakes; Steel frames; Nonlinear analysis; Reliability analysis; Performance evaluation.

Introduction

The performance of a building during an earthquake depends on many factors including: the structure's configuration and proportions, its dynamic characteristics, the hysteretic behavior of the elements and joints, the type of nonstructural components employed, the quality of the materials and workmanship, adequacy of maintenance, the site conditions, and the intensity and dynamic characteristics of the earthquake ground motion experienced. Consequently, seismic performance prediction for buildings, either as part of a design or evaluation, should consider, either explicitly or implicitly, all of these factors.

Prediction of seismic response of structure is complex, due not only to the large number of factors that affect performance but also the basic complexity of the physical behavior. In addition, due to impreciseness in our ability to accurately model the physical behavior, as well as inherent lack of knowledge in the precise definition of the structure's characteristics and inherent variability in the nature of future ground shaking, estimation of seismic performance inherently entails significant uncertainty. Clearly the characteristics of future earthquakes can only be approximated leading to very large uncertainties in the structural demands. Structural properties may differ from those intended or assumed by the designer, or may change substantially during the earthquake (e.g., local fracture of connections). Analysis methods may not accurately capture the actual behavior due to necessary simplifications and approximations in the analysis procedure (linear versus nonlinear for instance) and modeling of the structure. Our knowledge of the behavior of structures during earthquakes is not complete which introduces other uncertainties. Consequently, seismic performance prediction must consider the inherent uncertainties and randomness in the process.

These inherent uncertainties in prediction of probable future loading and response are not unique to seismic behavior and many of these issues are covered to a greater or lesser extent in current codes through the use of load and resistance factors. However, in the case of seismic loading, there has not until recently, been any systematic evaluation of the inherent uncertainty and variability and consequently, provision of adequate design margin has largely been judgmental, based on adjustment of various design parameters following observation of unsatisfactory performance in earthquakes. In responding to the problems in steel moment frame buildings after the Northridge earthquakes the Federal Emergency Management Agency/Strategic Air Command (FEMA/SAC) program to reduce earthquake hazards in moment-resisting steel frames (SAC Project) has attempted to develop a comprehensive understanding of the capacity of various moment-resisting framing configurations, connections, and the demands on the frames and components. To achieve satisfactory building performance through design or to evaluate an existing building, one needs to reconcile expected seismic demands with acceptable performance levels while recognizing the uncertainties involved.

A reliability-based, performance-oriented approach has been adopted by the SAC project for design and evaluation. This approach was taken in order to explicitly account for uncertainties and randomness in seismic demand and capacities in a consistent manner and to satisfy with defined reliability identifiable performance objectives corresponding to various occupancies, damage states, and seismic hazards.

[1]Postdoctoral Research Associate, Civil and Environmental Engineering Dept., Univ. of Illinois, 205 N. Mathews Ave., Urbana, IL 61801.

[2]Senior Vice President, EQE International Incorporated, Oakland, CA 94607-5500.

[3]Professor, Civil Engineering Dept., Stanford Univ., Stanford, CA 94305-4020.

[4]Professor, Civil and Environmental Engineering Dept., Univ. of Illinois, Urbana, IL 61801.

Note. Associate Editor: Takeru Igusa. Discussion open until September 1, 2002. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on November 14, 2000; approved on April 10, 2001. This paper is part of the ***Journal of Structural Engineering***, Vol. 128, No. 4, April 1, 2002. ©ASCE, ISSN 0733-9445/2002/4-534–545/$8.00+$.50 per page.

Structural failures observed after the 1994 Northridge and 1995 Kobe earthquakes have exposed the weakness of the prevalent design and construction procedures for steel moment frames and shown the need for new approaches for evaluation of building performance and design. A central issue is proper treatment and incorporation of the large uncertainty inherent in defining seismic demands and building resistance in the evaluation and design process. The state of the art of statistical and reliability methods that can be used for this purpose has been reviewed, and several critical issues directly related to the mission of the SAC project have been discussed in the report ''Critical Issues in Developing a Statistical Framework for Evaluation and Design'' (Wen and Foutch 1997). Based on the review, a statistical and reliability-based framework for the purpose of comparing and evaluating predictive models for structural performance evaluation and design was developed. This was further advanced by Hamburger (1996); Jalayer and Cornell (unpublished 1998, 2000). From this basis, the demand and resistance factor approach described below has been adopted by the SAC project and incorporated into recommended design criteria published by FEMA (SAC 2000a, b, c) as a possible basis for future code provisions. Technical details and justifications of the proposed framework can be found in papers by Luco and Cornell (1998); Hamburger et al. (2000); and Cornell et al. (2001).

Performance Levels and Objectives

Consistent with the FEMA-302 ''1997 Edition: NEHRP recommended provisions for seismic regulation for new buildings and other structures'' [Building Seismic Safety Council (BSSC 1998a)] and the FEMA-273 NEHRP guidelines for seismic rehabilitation of buildings (BSSC 1997), two performance levels are considered. These are termed Immediate Occupancy (IO) and Collapse Prevention (CP). The Immediate Occupancy (IO) level is defined as the post-earthquake damage state where only minor structural damage has occurred with no substantial reduction in building gravity or lateral resistance. Damage in this state could include some localized yielding and limited fracturing of connections. Damage is anticipated to be so slight that if not found during inspection there would be no cause for concern. For pre-Northridge buildings, fewer than 15% of the connections on any floor may experience connection fractures without exceeding the IO level.

The Collapse Prevention (CP) performance level is defined as the post-earthquake damage state in which the structure is on the verge of experiencing either local or total collapse. Significant damage to the building has occurred, including significant degradation in strength and stiffness of the lateral force resisting system, large permanent deformation of the structure, and possibly some degradation of the gravity load carrying system. However, all significant components of the gravity load carrying must continue to function.

A performance objective consists of the specification of a structural performance level and a corresponding probability that this performance level may be exceeded. For example, buildings designed in accordance with SAC (2000a) are expected, with high confidence, to provide less than a 2% chance in 50 years of damage exceeding CP performance. This is similar, but subtly different than the approach taken in BSSC (1998a), in which new buildings are anticipated to be capable of resisting earthquake ground shaking demands with that has less than a 2% chance of being exceeded in 50 years (2/50), while meeting the CP performance level. The important difference is that the approach taken in SAC (2000a) recognizes that there is some potential that ground shaking having a higher probability of occurrence than 2/50 could result in damage exceeding the CP level and similarly, there is some potential that ground shaking that is less probable than 2/50 could result in less damage than 2/50 shaking. SAC (2000a) seeks to provide a total 2/50 probability, considering all levels of ground shaking that may occur, that damage will exceed the CP level. The commentary to the NEHRP Recommended Provisions (BSSC 1998b) suggests that IO performance should be attained for an earthquake that has less than a 50% chance of being exceeded in 50 years (50/50). Under SAC (2000a), this would correspond to a 50% probability of damage more severe than the immediate occupancy level in a 50 year period.

The probability that a building will experience greater damage than desired depends on the vulnerability of the building and the seismic hazard to which it is exposed. Vulnerability is related to the capacity of the building, which may be a function of the global or interstory drift, plastic rotations, or member forces. Ground accelerations associated with an earthquake cause building response resulting in global and interstory drifts and member forces, all of which may be classified as demands. If both the demand over time produced by ground motion and the capacity of the structure to resist this demand could be predicted with certainty, then the design professional could design a building and have 100% confidence that the building would achieve the desired performance objectives. Unfortunately, neither the capacity nor demand can be precisely determined because of uncertainties and randomness inherent in our prediction of the ground motion, the structure's response to this motion, and its capacity to resist damage, given these demands. One of the important advancements in performance evaluation developed under the SAC project is a procedure for associating a level of confidence with the conclusion that a building is capable of meeting a performance objective.

A demand and capacity factor design (DCFD) format is used to associate a level of confidence one might have that a building will satisfy the performance objective. It features full integration over the three key stochastic models: ground motion hazard curve, nonlinear dynamic displacement, and displacement capacity. This process requires the calculation of a confidence parameter λ which may then be used to determine the confidence level that exists with regard to the performance objective. The confidence parameter, λ, is calculated as

$$\lambda = \frac{\gamma \cdot \gamma_a \cdot D}{\phi \cdot C} \tag{1}$$

where C = median estimate of the capacity of the structure. In the FEMA/SAC design criteria this estimate may be obtained from default values specified therein or by a more rigorous direct calculation of capacity described below; D = median demand on the structure for a specified ground motion level obtained from structural analysis [Eq. (2)]; γ = demand uncertainty factor that principally accounts for uncertainty inherent in prediction of demand arising from variability in ground motion and structural response to that ground motion; γ_a = analysis uncertainty factor that accounts for bias and uncertainty associated with specific analytical procedures used to estimate structural demand as a function of ground shaking intensity; ϕ = resistance factor that accounts for the uncertainty and randomness inherent in prediction of structural capacity; λ = confidence parameter from which a level of confidence can be determined by reference to Table 1. [Strictly, the resulting confidence level is ''conditional on the mean esti-

Table 1. Confidence Parameter λ, as a Function of Confidence Level, Hazard Parameter k, and Uncertainty β_{UT}

Confidence	2%	5%	10%	20%	30%	40%	50%	60%	70%	80%	90%	95%	99%
						$\beta_{UT}=0.1$							
$k=1$	1.23	1.18	1.14	1.09	1.06	1.03	1.01	0.98	0.95	0.92	0.88	0.85	0.80
$k=2$	1.24	1.19	1.15	1.10	1.06	1.04	1.01	0.98	0.96	0.93	0.89	0.86	0.80
$k=3$	1.25	1.20	1.15	1.10	1.07	1.04	1.02	0.99	0.96	0.93	0.89	0.86	0.80
$k=4$	1.25	1.20	1.16	1.11	1.08	1.05	1.02	0.99	0.97	0.94	0.90	0.87	0.81
						$\beta_{UT}=0.2$							
$k=1$	1.54	1.42	1.32	1.21	1.13	1.07	1.02	0.97	0.92	0.86	0.79	0.73	0.64
$k=2$	1.57	1.45	1.34	1.23	1.16	1.09	1.04	0.99	0.94	0.88	0.81	0.75	0.65
$k=3$	1.60	1.48	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
$k=4$	1.63	1.51	1.40	1.28	1.20	1.14	1.08	1.03	0.98	0.92	0.84	0.78	0.68
						$\beta_{UT}=0.3$							
$k=1$	1.94	1.71	1.54	1.35	1.22	1.13	1.05	0.97	0.89	0.81	0.71	0.64	0.52
$k=2$	2.03	1.79	1.61	1.41	1.28	1.18	1.09	1.01	0.93	0.85	0.74	0.67	0.54
$k=3$	2.12	1.87	1.68	1.47	1.34	1.23	1.14	1.06	0.98	0.89	0.78	0.70	0.57
$k=4$	2.22	1.96	1.76	1.54	1.40	1.29	1.20	1.11	1.02	0.93	0.82	0.73	0.60
						$\beta_{UT}=0.4$							
$k=1$	2.46	2.09	1.81	1.52	1.34	1.20	1.08	0.98	0.88	0.77	0.65	0.56	0.43
$k=2$	2.67	2.27	1.96	1.64	1.45	1.30	1.17	1.06	0.95	0.84	0.70	0.61	0.46
$k=3$	2.89	2.45	2.12	1.78	1.57	1.41	1.27	1.15	1.03	0.91	0.76	0.66	0.50
$k=4$	3.13	2.66	2.30	1.93	1.70	1.52	1.38	1.24	1.12	0.98	0.82	0.71	0.54
						$\beta_{UT}=0.5$							
$k=1$	3.16	2.58	2.15	1.73	1.47	1.29	1.13	1.00	0.87	0.74	0.60	0.50	0.35
$k=2$	3.59	2.92	2.44	1.96	1.67	1.46	1.28	1.13	0.99	0.84	0.68	0.56	0.40
$k=3$	4.06	3.31	2.76	2.22	1.89	1.65	1.45	1.28	1.12	0.96	0.77	0.64	0.45
$k=4$	4.60	3.75	3.13	2.51	2.14	1.87	1.65	1.45	1.27	1.08	0.87	0.72	0.52
						$\beta_{UT}=0.6$							
$k=1$	4.11	3.21	2.58	1.98	1.64	1.39	1.20	1.03	0.87	0.72	0.55	0.45	0.30
$k=2$	4.91	3.85	3.09	2.37	1.96	1.67	1.43	1.23	1.05	0.87	0.66	0.53	0.35
$k=3$	5.88	4.60	3.70	2.84	2.35	2.00	1.72	1.47	1.25	1.04	0.80	0.64	0.42
$k=4$	7.04	5.51	4.43	3.40	2.81	2.39	2.05	1.76	1.50	1.24	0.95	0.77	0.51

mate of the ground motion hazard,'' e.g., that provided by USGS, because, as will be seen below, only structural response and capacity uncertainties are incorporated in this procedure; see Cornell et al. (2001).]

A simplified performance evaluation procedure is first presented. This procedure employs default values of the above parameters selected from tables. A detailed procedure for deriving these parameters is next outlined which includes a discussion of how the default values were determined. Only the local and global collapse conditions and the CP performance level are discussed here.

The simplified procedure for performance evaluation is described in chapter 4 of FEMA (2000b). This procedure requires only that the design professional calculate the structural demand, D, in the form of interstory drift and column axial forces. The other parameters given in Eq. (1) are selected from tabulated predetermined values. The simplified procedure includes the following steps:

1. Determine the performance objective to be evaluated. This requires the selection of one or more performance levels, that is, either IO or CP, and the appropriate hazard level, that is exceedance probability desired for this performance. The guidelines recommend that design solutions that provide a 90% level of confidence that the building satisfy desired performance from a global perspective and a 50% level of confidence that it satisfy the performance at a local level. For new buildings, consistent with the approach taken in the NEHRP Provisions (BSSC 1998) it is recommended that a minimum design objective of collapse prevention at a 2%/50 year exceedance probability be selected. Selection of hazard level for the immediate occupancy level is optional. For existing buildings, any combination of performance levels and objectives may be selected.
2. Determine the ground motion characteristics for the performance objective chosen. The ground motion intensity for each performance level should be chosen to have the same probability of exceedance as the hazard level of the design objective, e.g., 2/50 for the CP case. Under the NEHRP Provisions (BSSC 1998) ground motion is characterized by two mapped, elastic response spectral ordinates, one for short periods, S_s, and one for a one second period, S_1, at a 2%/50 year exceedance probability. These are modified by factors to account for the soil conditions at the site to define the design response spectrum. FEMA-273 provides an equation for determining S_s and S_1 for other hazard levels.
3. Calculate the structural demand for each earthquake intensity. The demand is computed using standard methods of structural analysis. Either linear methods or nonlinear methods may be used. Once calculated, demand parameters such as the maximum interstory drift, Δ_{max}, are adjusted for bias

Table 2. Default Bias Factors C_B

	Analysis procedure							
	Linear Static		Linear Dynamic		Nonlinear		Nonlinear Dynamic	
Performance level	IO	CP	IO	CP	IO	CP	IO	CP
				Type 1 connections				
Low rise (3 stories or less)	0.90	0.65	1.00	0.80	1.10	0.85	1.00	1.00
Mid rise (4–12 stories)	1.10	0.85	1.10	1.15	1.40	0.95	1.00	1.00
High rise (>12 stories)	1.05	1.00	1.15	1.00	1.30	0.85	1.00	1.00
				Type 2 connections				
Low rise (3 stories or less)	0.75	0.90	1.00	1.20	0.90	1.25	1.00	1.00
Mid rise (4–12 stories)	0.80	1.00	1.050	1.30	1.08	1.35	1.00	1.00
High rise (>12 stories)	0.75	0.70	1.30	1.20	1.30	1.30	1.00	1.00

inherent in the analytical procedure using the equation

$$D=C_B\Delta_{\max} \tag{2}$$

where C_B=analysis procedure-dependent bias coefficient and $\Delta_{\max}$=maximum calculated interstory drift. The bias coefficients are calculated by performing a series of analyses, using representative building structures and the selected methodology, and by comparing the median of the results obtained to the median of results obtained from nonlinear time history analyses of the same structures for the same ground motions. In essence, the bias coefficient when applied to the results of a group of analyses should result in the same median value of demand as obtained from the more accurate, nonlinear time history benchmark method. Table 2 presents the bias factors calculated using this procedure for four different analytical procedures, where Type 1 connections are representative of ductile behavior, similar to those developed following the Northridge earthquake and Type 2 connections are more brittle assemblies, representative of practice prior to the Northridge earthquake.

4. Determination of global and local collapse capacity and resistance factor. Table 3 provides interstory drift capacities and associated resistance factors computed for a series of model buildings representative of regularly configured structures, as limited by global behavior. Capacities were determined using an incremental dynamic analysis approach, reported by Foutch (2000); Lee and Foutch (2000), and Yun and Foutch (2000). Local connection capacities were developed by Roeder (2000) and are provided in Tables 4 and 5 for Type 1 and Type 2 connections. The resistance factors are a product of the integration (Cornell et al. 2001) used to determine the total probability that demand will be greater than capacity. Resistance factors are given by the equation

$$\phi=e^{k/2b\beta^2} \tag{3}$$

where k=logarithmic slope of the hazard curve, i.e., a measure of the rate of change of ground motion intensity with probability of exceedance; b=similar coefficient that represents the change in demand (for example interstory drift) as a function of ground motion intensity (set to unity for the default cases); and b=standard deviation of the natural logarithm of the variation in capacity resulting from variability in ground motion and structural characteristics. These are described in more detail in a later section.

5. Determine the factored-demand-to-capacity ratio λ. Once the demand is calculated and the demand and capacity factors are determined, the factored-demand-to-capacity ratio is calculated using Eq. (1). The demand and analysis uncertainty factors, like the resistance factors, are products of the integration to obtain the total probability that demand is greater than capacity, and are discussed in a later section.
6. Evaluate the confidence level. The confidence in the ability of the building to meet the performance objective is determined, using the λ value determined in accordance with Step 5 above, by a back calculation to obtain K_x from the equation

$$\lambda=e^{-\beta_{\mathrm{UT}}(K_x-k\beta_{\mathrm{UT}}/2b)} \tag{4}$$

where k and b=coefficients previously described; β_{UT} =logarithmic standard deviation of the distribution of both

Table 3. Global Interstory Drift Angle Capacity C and Resistance Factors ϕ for Regular Buildings

	Performance level			
	Immediate Occupancy		Collapse Prevention	
Building height	INTERSTORY DRIFT ANGEL CAPACITY	RESISTANCE FACTOR ϕ	INTERSTORY DRIFT ANGEL CAPACITY	RESISTANCE FACTOR ϕ
		Type 1 connections		
Low rise (3 stories or less)	0.02	1.0	0.10	0.90
Mid rise (4–12 stories)	0.02	1.0	0.10	0.85
High rise (>12 stories)	0.02	1.0	0.085	0.75
		Type 2 connections		
Low rise (3 stories or less)	0.01	1.0	0.10	0.85
Mid rise (4–12 stories)	0.01	0.9	0.08	0.70
High rise (>12 stories)	0.01	0.85	0.06	0.60

Table 4. Default Drift Capacities and Resistance Factors as Limited by Local Connection Response—Ductile Type 1 Connections (Roeder 2000)

Connection type	Strength degradation limit drift angle (rad) θ_{SD}	Immediate occupancy		Collapse prevention	
		Limit Drift Angle (rad) θ_{IO}	Capacity Reduction Factor ϕ	Limit Drift Angle (rad) θ_{CP}	Capacity Reduction Factor ϕ
WUF-B[a]	$0.031-0.0003\ d_b$	0.015	0.9	$0.060-0.0006\ d_b$	0.9
WUF-W[b]	0.051	0.020	0.9	0.064	0.9
FF[c]	$0.077-0.0012\ d_b$	0.020	0.9	$0.10-0.0016\ d_b$	0.9
RBS[d]	$0.060-0.0003\ d_b$	0.020	0.9	$0.08-0.0003\ d_b$	0.9
WFP[e]	$0.12-0.023\ d_b$	0.020	0.9	$0.10-0.0011\ d_b$ except that should used θ_{SD} if w14 or less	0.9
BUEP[f]	$0.071-0.0013\ d_b$	0.015	0.9	$0.081-0.0013\ d_b$	0.9
BSEP[g]	$0.071-0.0013\ d_b$	0.015	0.9	$0.081-0.0013\ d_b$	0.9
BFP[h]	$0.12-0.002\ d_b$	0.015	0.9	$0.10-0.001\ d_b$	0.9
DST[i]	$0.12-0.0032\ d_b$	0.015	0.9	$0.14-0.0032\ d_b$	0.9

[a]WUF-B: welded unreinforced flange-bolted connection.
[b]WUF-W: welded unreinforced flange-welded web connection.
[c]FF: free flange connection.
[d]RBS: reduced beam section connection.
[e]WFB: welded flange plate connection.
[f]BUEP: bolted unstiffened end plate.
[g]BSEP: bolted stiffened end/plate
[h]BFP: bolted flange plate.
[i]DST: double split tee.

demand and resistance, considering all sources of uncertainty; and K_x = standard Gaussian variate associated with probability x of not being exceeded found in conventional probability tables, e.g., if $K_x = 1.28$ then $x = 90\%$.

The values of the uncertainty coefficient β_{UT} used are dependent on a number of sources of uncertainty in the estimation of structural demands and capacities. Sources of uncertainty include, for example, the effective damping, the actual material properties, and the effective structural period and others each contain uncertainties. The uncertainty associated with each source (i) may be identified as β_{Ui}. Then

$$\beta_{UT} = \sqrt{\sum_i \beta_{Ui}^2} \tag{5}$$

The default values of β_{UT} for Type 1 and Type 2 connections are given in Tables 6 and 7, respectively. Further discussion of these uncertainties is presented later.

7. Determine the confidence level. Once the confidence factor λ and the uncertainty coefficient β_{UT} are determined, the confidence level can be found in Table 1. An example is given below.

The procedures used for determining the default values given in the tables are summarized in the remainder of the paper. A more detailed description of the basis for these procedures and the calculation of the default values is reported in Yun and Foutch (2000) and Appendix A of the *Guidelines* (FEMA 2000a, b).

Determination of Hazard Parameters

Two ground motion parameters are required for performance evaluation. These are the intensity as defined by the spectral acceleration S_{aT1} at the first period of the building, corresponding to the hazard level of interest, and the logarithmic slope of the hazard curve, k, at the desired hazard level. The spectral acceleration, S_{aT1}, may be determined using procedures given in FEMA-273 (BSSC 1997). They require values of S_s and S_1 determined from national maps developed by the United States Geological Survey. Alternatively, S_{aT1} and k, may be derived from the mean hazard estimate determined from a site-specific study.

The logarithmic slope k of the hazard curve at the desired hazard level is used in the evaluation of the resistance factors, demand factors, and confidence levels. The hazard curve is a plot of the probability of exceedance of a spectral amplitude value versus the spectral amplitude for a given response period, and is usually approximately linear when plotted on a log-log scale. On

Table 5. Default Drift Capacities and Resistance Factors as Limited by Local Connection Response—Brittle Type 2 (Pre-Northridge) Welded Connections (Roeder 2000)

Connection type	Strength degradation limit drift angle (rad) θ_{SD}	Immediate occupancy		Collapse prevention	
		Limit Drift Angle (rad) θ_{IO}	Capacity Reduction Factor ϕ	Limit Drift Angle (rad) θ_{CP}	Capacity Reduction Factor ϕ
WUF[a] (<1980)	$0.061-0.00013\ d_b$	0.010	0.8	Larger of $0.053-0.0006\ d_b$ or $0.061-0.00013\ d_b$	0.8
WUF[a] (>1980)	0.021	0.010	0.8	$0.053-0.0006\ d_b$	0.8

[a]WUF: welded unreinforced flange connection.

Table 6. Uncertainty Coefficient β_{UT} for Global Interstory Drift Evaluation

	Performance level	
Building height	Immediate Occupancy	Collapse Prevention
Type 1 connections		
Low rise (3 stories or less)	0.20	0.30
Mid rise (4–12 stories)	0.20	0.40
High rise (>12 stories)	0.20	0.50
Type 2 connections		
Low rise (3 stories or less)	0.20	0.35
Mid rise (4–12 stories)	0.20	0.45
High rise (>12 stories)	0.20	0.55

Note: Value of β_{UT} should be increased by 0.05 for linear static analysis and decreased by 0.05 for nonlinear dynamic analysis.

such a scale a straight line fit in the range of hazard levels of interest will have the functional expression

$$H_{Si}(S_i)=k_0 S_i^{-k} \tag{6}$$

where $H_{Si}(S_i)$ = probability of ground shaking having a spectral acceleration greater than S_i; k_0 = constant, dependent of the seismicity of an individual site; and k = logarithmic slope of the hazard curve.

If mapped spectral acceleration values at 10%/50 year and 2%/50 year exceedance probabilities are available, for example as provided with FEMA-273, the value of k may be calculated as

$$k=\frac{\ln\left(\dfrac{H_{s1(10/50)}}{H_{s1(2/50)}}\right)}{\ln\left(\dfrac{S_{1(2/50)}}{S_{1(10/50)}}\right)}=\frac{1.65}{\ln\left(\dfrac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \tag{7}$$

where $S_{1(10/50)}$ = spectral amplitude for 10/50 hazard level; $S_{1(2/50)}$ = spectral amplitude for 2/50 hazard level; $H_{S1(10/50)}$ = probability of exceedance for the 10/50 hazard level = 1/475 = 0.0021; and $H_{S1(2/50)}$ = probability of exceedance for the 2/50 hazard level = 1/2475 = 0.00040.

Default values of k for various regions of the United States are given in Table 8.

Determination of Drift Capacity and Resistance Factors

Local Drift Capacity

The median drift capacities and resistance factors for connection types tested under the FEMA/SAC Project are given in Table 3.

Table 7. Uncertainty Coefficient β_{UT} for Local Interstory Drift Evaluation

	Performance level	
Building height	Immediate Occupancy	Collapse Prevention
Type 1 connections		
Low rise (3 stories or less)	0.30	0.30
Mid rise (4–12 stories)	0.30	0.35
High rise (>12 stories)	0.30	0.40
Type 2 connections		
Low rise (3 stories or less)	0.30	0.35
Mid rise (4–12 stories)	0.30	0.40
High rise (>12 stories)	0.30	0.40

Note: Value of β_{UT} should be increased by 0.05 for linear static analysis and decreased by 0.05 for nonlinear dynamic analysis.

Table 8. Default Values of the Logarithmic Hazard Curve Slope k for Probabilistic Ground Hazards

Region	k
Alaska, California and the Pacific Northwest	3
Intermountain Region, Basin & Range Tectonic Province	2
Other U.S. locations	1

Note: For deterministic ground shaking demands, use a value of k = 4.0.

The values corresponding to the local collapse were determined from cyclic tests of full-size connection specimens. The cyclic tests are used to determine load-deformation hysteresis behavior of the system and the maximum drift for which gravity loads may still be carried by the girders. This gravity-induced drift limit is reached when the shear tab is significantly damaged, a low-cycle fatigue crack develops in the beam web, or the load-deformation behavior of the moment connection has completely deteriorated.

A standard test protocol, based on ATC-24 (ATC 1991) and developed specifically for the SAC Project was used for most of the tests. Instructions on loading sequence and required response measurements are given in Roeder (2000). The moment versus plastic rotation of a beam-column assembly for a reduced beam section (RBS) connection representative of a single test is shown in Fig. 1. The hysteretic behavior is characterized by a gradual strength degradation with increasing plastic rotation. For the specific connection tested, it appeared that the shear-carrying capacity was reached at a plastic rotation of about 0.06. In order to use such data in the reliability framework, it is necessary to have several such tests, from which statistics on the likely distribution of important design parameters, such as plastic rotation at peak load and plastic rotation at loss of capacity, can be obtained. Statistics that must be obtained include the median value of the parameter and the standard deviation of the logarithm of the values obtained from the testing, β. The analytical connection model used for analysis of buildings with RBS connections, representative of median behavior, is also shown in Fig. 1.

Global Drift Capacity

The global drift capacity of a building is determined using the incremental dynamic analysis (IDA) procedure. This is based on the use of nonlinear time history (NTH) analysis. It is important that the analytical model used for determining the global drift demand reproduces the major features of the measured response such as sudden loss of strength. This means that the measured hysteresis behavior must be modeled reasonably well and the model must include all significant components of building stiffness, strength, and damping. Modeling recommendations are given by Foutch (2000) and Lee and Foutch (2000). (See Fig. 2.)

The incremental dynamic analysis (IDA) technique was developed by Luco and Cornell (1998) and is described in detail in Appendix A of the *Guidelines* (FEMA 2000a, b) and Vamvatsikos and Cornell (2001). It consists of a series of nonlinear analyses of a structure for a ground motion that is increased in amplitude, until instability of the structure is predicted. This analysis is repeated for multiple ground motions, so that statistics on the variation of demand and capacity with ground motion character can be attained. A suite of 20 ground motion records (Somerville et al. 1997) was used to determine the global drift capacities given in Table 4. Twenty model buildings (eight 3- and 9-story and four 20-story) designed in accordance with the 1997 NEHRP provi-

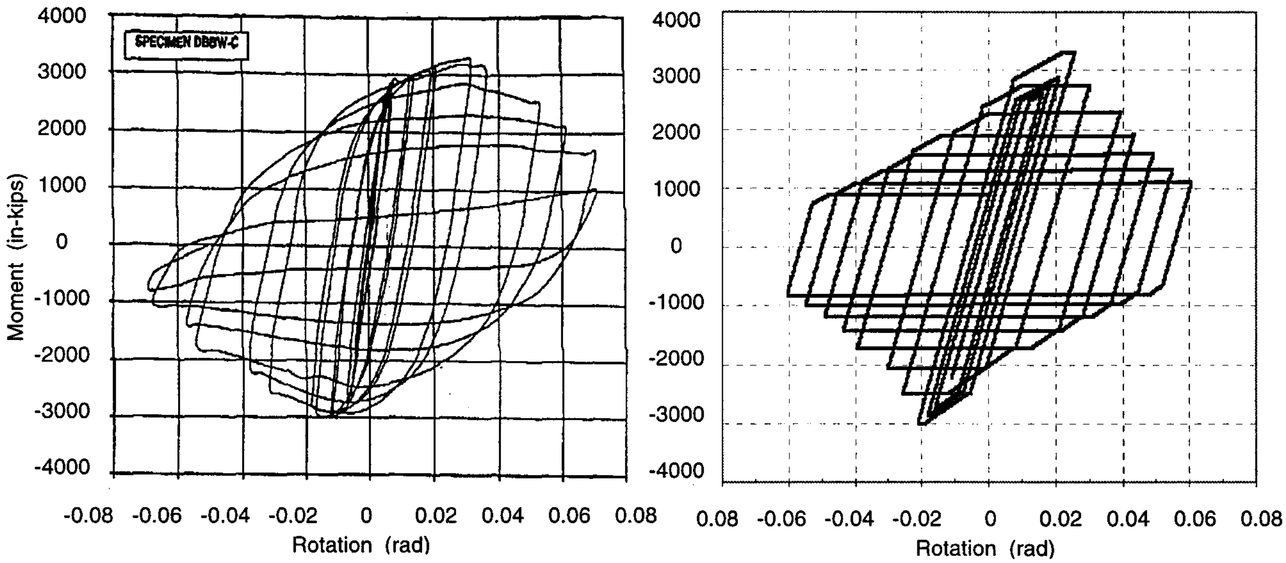

Fig. 1. Measured (Venti and Engelhardt 2000) and modeled (Lee and Foutch 2000) moment-rotation behavior of RBS connection

sions (BSSC 1998) were used for the post-Northridge buildings (Type 1 connections). Nine buildings designed using past UBC provisions (ICBO 1973, 1985, and 1994) were used for the pre-Northridge buildings (Type 2 connections). All of the buildings were very regular. The measured and modeled behavior of the Type 2 connections are shown in Fig. 3.

This procedure that was followed in doing this analysis is as follows:

1. Choose a suite of 10 to 20 accelerograms representative of the site and hazard level. The SAC project developed typical accelerograms for Los Angeles, Seattle, and Boston sites (Somerville 1997). These might be appropriate for similar sites.
2. Perform an elastic time history analysis of the building for one of the accelerograms. Plot the point on a graph whose vertical axis is the spectral ordinate for the accelerogram at the first period of the building and the horizontal axis is the maximum calculated drift at any story. Draw a straight line from the origin of the axis to this point. The slope of this line is referred to as the elastic slope for the accelerogram. Calculate the slope for the rest of the accelerograms using the same procedure and calculate the median slope. The slope of this median line is referred to as the elastic slope, S_e. (See Fig. 4.)
3. Perform a nonlinear time history analysis of the building subjected to one of the accelerograms. Plot this point on the graph. Call this point Δ_1.
4. Increase the amplitude of the accelerogram and repeat step 3. This may be done by multiplying the accelerogram by a constant that increases the spectral ordinates of the accelerogram by $0.1g$. Plot this point as Δ_2. Draw a straight line between points Δ_1 and Δ_2. If the slope of this line is less than 0.2 S_e then Δ_1 is the global drift limit. This can be thought of as the point at which the inelastic drifts are in-

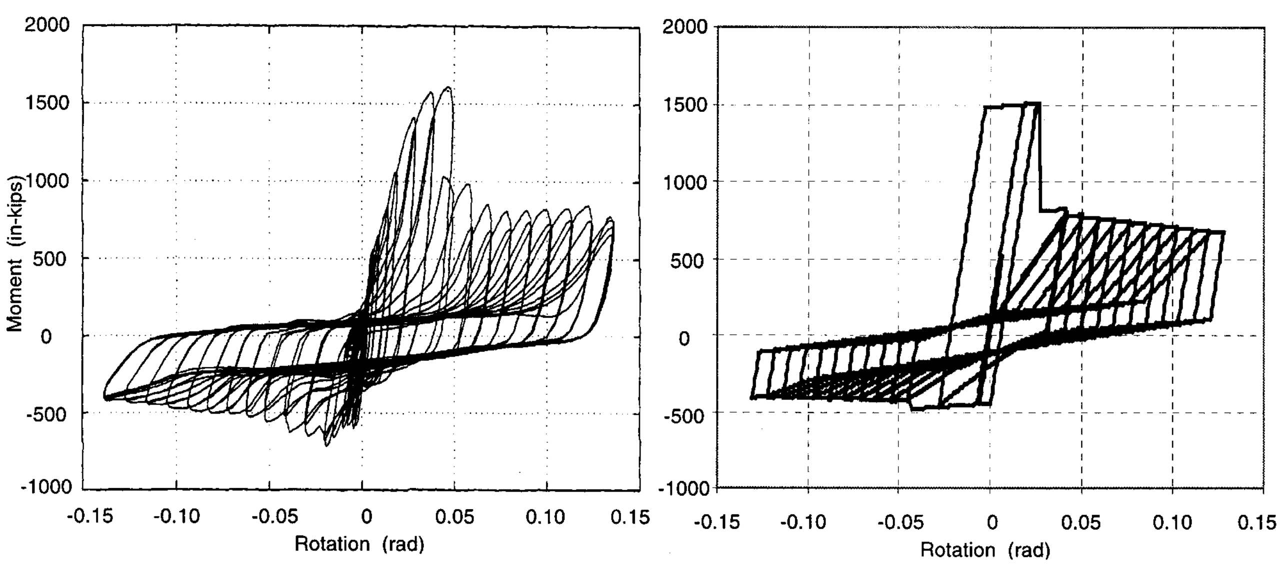

Fig. 2. Measured (Liu and Astaneh-Asl 2000) and modeled (Lee and Foutch 2000) moment-rotation behavior of beams in gravity frame

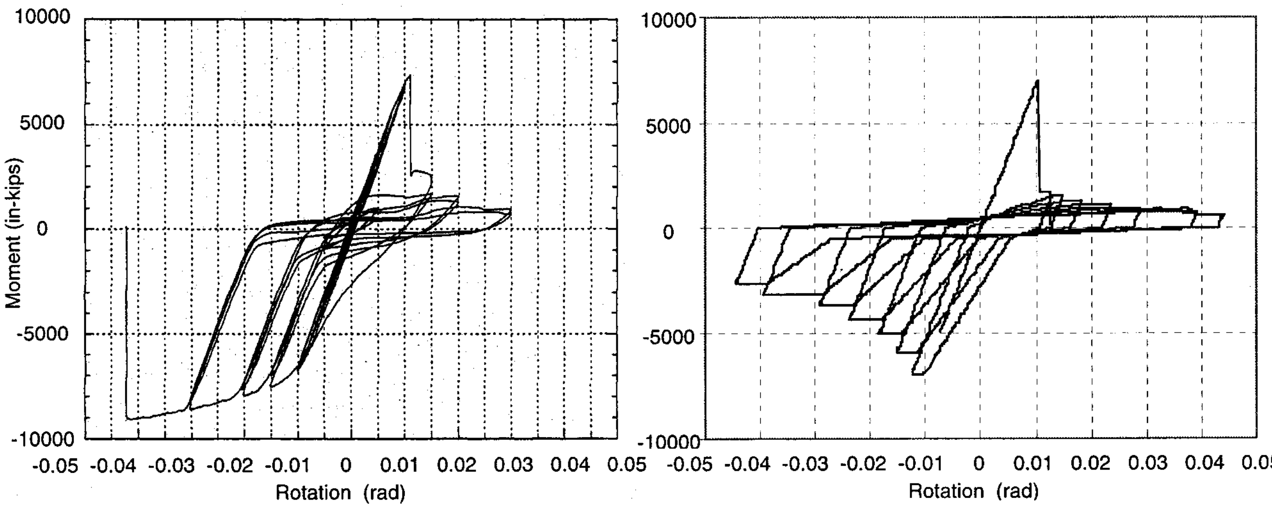

Fig. 3. Measured (Lee et al. 2000) and modeled (Lee and Foutch 2000) moment-rotation behavior of pre-Northridge connection

creasing at five times the rate of elastic drifts. The value of 0.2 is an arbitrary number but calculated collapse drift is rather insensitive to this value.

5. Repeat step 4 until the straight-line slope between consecutive points Δ_i and Δ_{i+1}, is less than 0.2 S_e. When this condition is reached, Δ_i is the global drift capacity for this accelerogram. If $\Delta_{i+1} \geqslant 0.10$ then the drift capacity is taken as 0.10.
6. Choose another accelerogram and repeat steps 3 through 5. Do this for each accelerogram. The median capacity for global collapse is the median value of the calculated set of drift limits. An illustration of an IDA analysis for two accelerograms is shown in Fig. 4. The open circles represent the IDA for an accelerogram where the 0.2 S_e slope determined the capacity. The open triangles represent a case where the default capacity = 0.10 applies.

The factors that affect the curve of the incremental dynamic analysis (IDA) are P-Δ effects, increment used for the analysis, ground motions used, strain hardening ratio, shifting of fundamental period due to nonlinearity, higher mode effects, and shifting of maximum story drift location.

A strain-hardening ratio of 0.03 was used for all of the analysis in this study. Ground motion intensity increment of 0.2g for three-story and nine-story buildings was used, whereas 0.1g was used for the 20-story buildings since sudden increases in drift were observed due to larger P-Delta effects. The ground motion increment must be small enough so that drift increment is relatively small for each step. The values given above should be considered as an upper bound. The use of a larger increment would usually result in smaller drift capacity and larger variation of the capacity. Therefore, it would give conservative results. More discussion of how the global capacity is determined is given in Appendix A of the *Guidelines* (FEMA 2000a,b) and in Yun and Foutch (2000).

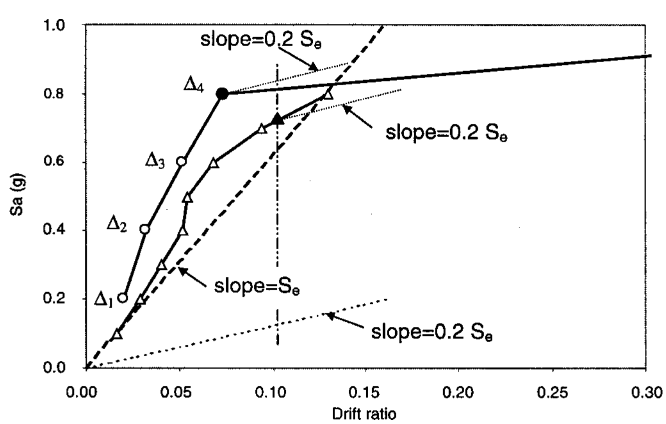

Fig. 4. Two IDA analyses for nine-story building

Determination of Resistance Factor ϕ

The resistance factor, ϕ, accounts for the fact that structural capacity has a distribution of values. In order to determine probabilities and confidence levels the sources of this variation are separated into "randomness" and "uncertainty." Variation due to future factors that cannot be predicted are termed randomness. Variation due to factors that are fixed, but which are uncertain as to actual value, for example, material strength, are termed uncertainty. The principle portion of randomness in global capacity is due to the variation in the earthquake accelerograms the building may experience (as represented by the suite of accelerograms used in the IDA analyses). Estimation of capacity is also subject to uncertainty in the load-deformation behavior of the system, as might in principle be determined from tests. The local collapse value is also affected by uncertainties in the response of the components due to variable material properties and fabrication. Throughout when the distinction is critical the relevant parameters will be subscripted by a R or U for "randomness" and "uncertainty," respectively.

The equation for calculating ϕ is given by (Jalayer and Cornell unpublished internal technical memo, 1998)

$$\phi = \phi_{RC} \cdot \phi_{UC} \tag{8}$$

$$\phi_R = e^{-k\beta_{RC}^2/2b} \tag{9}$$

$$\phi_U = e^{-k\beta_{UC}^2/2b} \tag{10}$$

where ϕ = resistance factor; ϕ_R = contribution to ϕ from randomness; ϕ_U = contribution to ϕ from uncertainties; β_{RC} = standard deviation of the natural logs of the drift capacities due to randomness.

• Global: Based on variability observed in capacities derived from IDA;

• Local: Randomness. Assumed to be 0.20. β_{UC} = the standard deviation of the natural logs of the drift due to uncertainty;

• Global: $\beta_{uc} = \sqrt{3 \cdot \beta_{NTH}{}^2} = \sqrt{3 \cdot}\beta_{NTH}$ is defined below. The $\sqrt{3 \cdot}$ follows from assumed perfect negative correlation between the uncertainties in demand and capacity (Cornell et al. 2001); and

• Local: Uncertainty. Assumed to be 0.25.

For local collapse, β_{UC} accounts for the uncertainty in the median drift capacity. This arises from uncertainties in the representativeness of the testing procedure, in the limited number and scope of tests, material, and weld properties, and other factors. The β_{RC} term accounts for randomness in the drift capacity resulting primarily from record-dependent differences in the story drift (or connection rotation) at collapse.

Determination of Demand Factors

Determination of γ

The demand variability factor *g* is associated with the variation in structural response to different ground motion records, each of the same intensity, as measured by the spectral response acceleration at the fundamental period of the building. This randomness is due to unpredictable variation in the actual ground motion accelerogram and also due to variation in the azimuth of attack, termed orientation, of the ground motion. The orientation component is a significant factor, only for near-fault site. For such sites, located within a few kilometers of the zone of fault rupture, the fault-parallel and fault-normal directions experience quite different shaking. For sites located farther away from the fault, there is no statistical difference in the accelerograms recorded in different directions.

The demand factor γ is calculated as

$$\gamma = e^{k\beta_{RD}^2/2b} \tag{11}$$

where γ = demand variability factor and $\beta_{RD} = \sqrt{\Sigma\beta_i^2}$ where β_i^2 is the variance of the natural log of the drifts for each element of randomness.

The β_i values for each source of randomness as determined for the SAC project are based on studies of twenty building designs using the 1997 NEHRP provisions for the LA site. The notation used is as follows: β_{acc}, accelerogram; β_{or}, orientation. β_{acc} is the standard deviation of the log of the maximum story drifts calculated for each of the 20 accelerograms referred to above.

Determination of γ_a

The demand uncertainty factor γ_a is based on uncertainties related to the determination of median demand, *D*. One significant source of uncertainty is due to inaccuracies in the analytical procedure, termed β_a for analysis procedure. The β_a is nominally composed of five parts as follows: β_{NTH} associated with uncertainties related to the extent that the benchmark, nonlinear time history analysis procedure, represents actual physical behavior; β_{BF} associated with uncertainty in the bias factor; $\beta_{damping}$ associated with uncertainty in estimating the damping value of the structure; $\beta_{live\ load}$ associated with uncertainty in live load; $\beta_{mat.\ prop.}$ associated with uncertainty in material properties.

The β_{NTH} is assumed to be 0.15, 0.20, and 0.25 for 3-, 9-, and 20-story buildings, respectively, based on judgment and on understanding of the relative importance of strength degradation and *p*-delta effects, phenomena not well accounted for in the analyses, to structures in these height ranges. β's computed for the effects on response of damping, live load, and material properties were set to zero since the values were negligible when compared to β_{NTH}.

The bias factor for each analysis procedure is calculated as the ratio of the median drift demand resulting from the nonlinear time history analysis of a building divided by the estimate of the drift demand using a particular analysis procedure. The β_{BF} for the bias factor is the coefficient of variation of the bias factors among buildings of given height.

A number of options are available to the design professional who wants to use other parameters than the default values given in the tables included here. For design of steel moment frames the slope of the hazard curve *k*, for the building site is easily calculated using Eq. (7). If the building is not near a known fault, the β_{or} is not considered so $\beta_{RD} = \beta_{acc}$ and therefore γ value will decrease. Additional reduction in γ value is achievable by calculating dynamic demands using appropriate ground motions for the site. If the connection is not a prequalified connection then testing is required. This will provide a local collapse value *C* and, perhaps, a different value of ϕ. However, usually there will not be enough tests performed to determine ϕ, so the default value of Type 1 connections should be used. In all cases, the demand *D* must be calculated using Eq. (2). Other parameter calculations are outlined above and described in detail in Yun and Foutch (2000). For other structural systems and materials, calculation of all parameters is required. Details may be found in Yun and Foutch (2000).

Performance Evaluation Example

A short example is given here. Two nine-story buildings with perimeter moment frames, one designed for the 1997 NEHRP provisions with Type 1 connections and one designed for the 1994 UBC with Type 2 connections, are used. Each building has five 9.14-m (30-ft) bays in both directions. Both of the buildings have four and a half moment-resisting bays in each perimeter frame. All story heights are 3.96-m (13-ft) except for the first floor and basement which is 5.49-m (18-ft) and 3.66-m (12-ft), respectively. The columns are pinned at the basement level and translation of the first floor is assumed to be restrained by basement walls. The plan and elevation are shown in Fig. 5. For an actual case, the designer would calculate the demand *D* (design drift) using Eq. (2). For this example, the demand is the median drift demand calculated for twenty LA ground motions. The results are given in Table 9.

The results show that the confidence level that the post-Northridge building will satisfy the CP performance level for the 2/50 hazard are 99% for the global collapse and 94% for local collapse. For the 1994 building, the confidence levels are 59 and 19% for the global and local collapse performance, respectively. The main reasons that a lower confidence level is calculated for the 1994 building is that it is more flexible which increases *D*, and has Type 2 connections which decreases *C*. Although the results for the 1994 building are discouraging they are representative of the poor confidence associated with buildings representing pre-Northridge design and construction practice. For buildings designed prior to 1979, the confidence level for satisfying local collapse is particularly low, at less than 10% for 3-, 9- and 20-story buildings. This is because prior to that time, the codes did not specify lateral deflection limits. As a result, such buildings tend to be quite flexible.

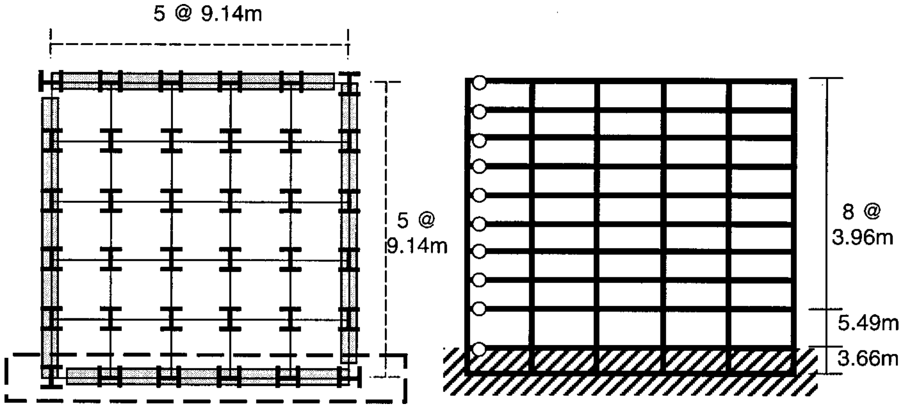

Fig. 5. Plan and elevation view of nine-story building

Table 9. Calculation of Confidence Level for the Post- and Pre-Northridge 9-Story Building

	C	ϕ	D	γ	γ_a	λ	β_{UT}	K_x	C.L. (%)
Collapse prevention against 2/50 hazard level									
1997 NEHRP 9-story (global)	0.10	0.85	0.034	1.2	1.06	0.51	0.40	2.29	99
1997 NEHRP 9-story (local)	0.07	0.90	0.034	1.2	1.06	0.69	0.40	1.54	94
1994 UBC 9-story (global)	0.08	0.70	0.043	1.5	1.06	1.22	0.45	0.23	59
1994 UBC 9-story (local)	0.054	0.70	0.043	1.5	1.06	1.81	0.40	−0.88	19
Collapse prevention against 2/50 hazard level									
1997 NEHRP 9-story (global)	0.02	1.00	0.008	1.4	1.02	0.57	0.20	3.10	99
1997 NEHRP 9-story (local)	0.02	0.90	0.008	1.4	1.02	0.63	0.30	1.97	98
1994 UBC 9-story (global)	0.01	0.90	0.009	1.3	1.02	1.33	0.20	−1.11	13
1994 UBC 9-story (local)	0.01	0.80	0.009	1.3	1.02	1.49	0.30	−0.88	19

The IO performance levels were checked for each building for the 50/50 hazard. IO capacity for Type 1 connections is 0.02 rad, and it is 0.01 rad for Type 2 connections. The confidence levels are 98% for the new building and 13% for the 1994 building.

Summary and Conclusions

A performance-based procedure for seismic performance evaluation of steel moment frames is presented that allows the designer to estimate the confidence level of satisfying the performance objectives. This is an important step forward since it accounts for uncertainties and randomness in the seismic demand and building capacity. A numerical example indicates that buildings designed in accordance with the 1997 NEHRP provisions and constructed with SAC prequalified connections have a confidence level of greater than 90% for satisfying the collapse prevention performance level for a hazard that has less than a 2% probability of being exceeded in 50 years. A 1994 building constructed with pre-Northridge welded connections has only a 59% confidence level for satisfying global collapse performance and 19% for local collapse performance for the same hazard level. The confidence levels for satisfying the immediate occupancy performance level for the 50%-in-50 year hazard level are 98% for the new building and 13% for the 1994 building. This leads to the following conclusions:

1. Randomness and uncertainty in the calculation of seismic demand and structure capacity are important effects that must be accounted for in seismic performance evaluation.
2. The new performance evaluation procedure developed by the SAC project is a powerful tool, but simple to apply.
3. Steel moment frame buildings designed in accordance with the 1997 NEHRP provisions and constructed with SAC prequalified connections are expected to perform much better during major earthquakes than existing buildings designed and built with older technologies.

Acknowledgments

This research was sponsored by the SAC Joint Venture with funds provided by the Federal Emergency Management Agency. This support is gratefully acknowledged. Opinions and advice provided by the Technical Advisory Panel were also very helpful. Any findings, results, or conclusions are solely those of the writers and do not represent those of the sponsors. The writers would like to thank Dr. Norman Abrahamson and Dr. Robert Kennedy for review, discussion, and recommendations regarding categorization and estimation of the components of randomness and uncertainty.

Notation

The following symbols are used in this paper:

b = coefficient relating incremental change in demand to incremental change in ground shaking intensity at hazard level of interest;

C = median estimate of capacity of structure;
C_B = analysis procedure-dependent bias coefficient;
CP = collapse prevention performance level;
D = median demand on structure for specified ground motion level obtained from structural analysis;
$H_{\mathrm{Si}}(S_i)$ = probability of ground shaking having a spectral acceleration greater than S_i;
$H_{S1(10/50)}$ = probability of exceedance for the 10/50 hazard level = 1/475 = 0.0021;
$H_{S1(2/50)}$ = probability of exceedance for the 2/50 hazard level = 1/2475 = 0.00040;
IO = immediate occupancy performance level;
k = slope of hazard curve expressed in log-log coordinates at the hazard level of interest;
k_0 = constant, dependent of seismicity of individual site;
K_x = standard Gaussian variate associated with probability x of not being exceeded as function of number of standard deviations above or below mean;
$S_{1(10/50)}$ = spectral amplitude for 10/50 hazard level;
$S_{1(2/50)}$ = spectral amplitude for 2/50 hazard level;
β_{BF} = associated with uncertainty in bias factor;
β_{damping} = associated with uncertainty in estimating damping value of structure;
$\beta_{\mathrm{live\ load}}$ = associated with uncertainty in live load;
$\beta_{\mathrm{mat.\ prop.}}$ = associated with uncertainty in material properties;
β_{NTH} = associated with uncertainties related to extent that benchmark, nonlinear time history analysis procedure, represents actual physical behavior;
β_{RC} = standard deviation of natural logs of drift capacities due to randomness;
β_{RD} = $\sqrt{\Sigma\beta_i^2}$ where β_i^2 is variance of natural log of drifts for each element of randomness;
β_{UC} = standard deviation of natural logs of drift capacities due to uncertainty;
β_{UT} = uncertainty measure equal to vector sum of logarithmic standard deviation of variations in demand and capacity;
γ = demand uncertainty factor that principally accounts for uncertainty inherent in prediction of demand arising from variability in ground motion and structural response to that ground motion;
γ_a = analysis uncertainty factor that accounts for bias and uncertainty associated with specific analytical procedures used to estimate structural demand as a function of ground shaking intensity;
$\Delta_{\max}$ = maximum calculated interstory drift;
λ = confidence parameter;
ϕ_R = contribution to ϕ from randomness of earthquake accelerogram;
ϕ_U = contribution to ϕ from uncertainties in measured connection capacity;
ϕ = resistance factor that accounts for uncertainty and randomness inherent in prediction of structural capacity; and
ϕ = resistance factor.

References

BSSC. (1997). ''NEHRP guidelines for seismic rehabilitation of buildings.'' *Rep. No. FEMA-273*, Federal Emergency Management Agency, Washington, D.C.

BSSC. (1998a). ''1997 Edition: NEHRP recommended provisions for seismic regulations for new buildings and other structures, part I—provisions.'' *Rep. No. FEMA-302*, Federal Emergency Management Agency, Washington, D.C.

BSSC. (1998b). ''1997 edition: NEHRP recommended provisions for seismic regulations for new buildings and other structures, part 2—commentary.'' *Rep. No. FEMA 303*, Federal Emergency Management Agency, Washington, D.C.

Cornell, C. A., Jalayer, F., Hamburger, R. O., and Foutch, D. A. (2001). ''Probabilistic basis for the 2000 SAC Federal Emergency Management Agency steel moment frame guidelines.'' *J. Struct. Eng.*, 128(4), 526–533.

FEMA. (2000a). ''Recommended seismic design criteria for new steel moment frame buildings.'' *Rep. No. FEMA-350*, SAC Joint Venture, Federal Emergency Management Agency, Washington, DC.

FEMA. (2000b). ''Recommended seismic evaluation and upgrade criteria for existing welded steel moment frame buildings.'' *Report No. FEMA-351*, SAC Joint Venture, Federal Emergency Management Agency, Washington, DC.

Foutch, D. A. (2000). ''State-of-the-art report on performance prediction and evaluation of moment-resisting steel frame structures.'' *Report No. FEMA-355f*, Federal Emergency Management Agency, Washington, D.C.

Hamburger, R. O. (1996). ''Performance-based seismic engineering: the next generation of structural engineering practice.'' *EQE Summary Report*.

Hamburger, R. O., Foutch, D. A., and Cornell, C. A. (2000). ''Performance basis of guidelines for evaluation, upgrade, and design of moment-resisting steel frames.'' *Proc., 12th World Conf. in Earthquake Engineering*, Auckland, New Zealand.

Hart, G. C., and Skokan, M. J. (1999). ''Reliability-based seismic performance evaluation of steel frame buildings using nonlinear static analysis methods.'' *SAC Background Document SAC/BD-99/22*, SAC Joint Venture, Richmond, Calif.

International Conference of Building Officials (ICBO). (1973). *Uniform building code*, Whittier, Calif.

International Conference of Building Officials (ICBO). (1985). *Uniform building code*, Whittier, Calif.

International Conference of Building Officials (ICBO). (1994). *Uniform building code*, Whittier, Calif.

International Conference of Building Officials (ICBO). (1997). *Uniform building code*, Whittier, Calif.

Jalayer, F., and Cornell, C. A. (2000). ''A technical framework for probability-based demand and capacity factor (DCFD) seismic formats.'' *RMS Technical Rep. No. XXX* to the PEER Center, Dept. of Civil Engineering, Stanford Univ. Stanford, Calif.

Krawinkler, H. (2000). ''State-of-the-art report on systems performance of moment-resisting steel frames subject to earthquake ground shaking.'' *Rep. No. FEMA-355c*, Federal Emergency Management Agency, Washington, D.C.

Lee, K., and Foutch, D. A. (2000). ''Performance prediction and evaluation of steel special moment frames for seismic loads.'' *SAC Background Document SAC/BD-00/25*, SAC Joint Venture, Richmond, Calif.

Lee, K.-H., Stojadinovic, B. Goel, S. C., Margarian, A. G., Choi, J., Wongkaew, A., Reyher, B. P., and Lee, D.-Y. (2000). ''Parametric tests on unreinforced connections.'' *SAC Background Document SAC/BD-00/01*, SAC Joint Venture, Richmond, Calif.

Liu, J., and Astaneh-Asl, A. (2000). ''Cyclic tests on simple connections including effects of the slab.'' *SAC Background Document SAC/BD-00/03*, SAC Joint Venture, Richmond, Calif.

Luco, N., and Cornell, C. A. (1998). ''Effects of random connection fractures on the demands and reliability for a three-story pre-Northridge (SMRP) structure.'' *Proc., 6th U.S. National Conf. on Earthquake*

Engineering, Seattle, May 31–June 4, 1998, Earthquake Engineering Research Institute, Oakland, Calif.

Roeder, C. W. (2000). ''State-of-the-art report on connection performance.'' *Rep. No. FEMA-355d*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture. (2000c). ''Recommended postearthquake evaluation and repair criteria for welded steel moment frame buildings.'' *Rep. No. FEMA-352*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture. (2000a). ''Recommended seismic design criteria for new steel moment frame buildings.'' *Rep. No. FEMA-350*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture. (2000b). ''Recommended seismic evaluation and upgrade criteria for existing welded steel moment frame buildings.'' *Rep. No. FEMA-351*, Federal Emergency Management Agency, Washington, D.C.

Somerville, P., Smith, N., Puntamurthula, S., and Sun, J. (1997). ''Development of ground motion time histories for phase 2 of the FEMA/SAC steel project.'' *SAC Background Document SAC/BD-97/04*, SAC Joint Venture, Richmond, Calif.

Vamvatsikos, D., and Cornell, C. A. (2001). ''Incremental dynamic analysis.'' *Earthquake Eng. Struct. Dyn.*, in press.

Venti, M., and Engelhardt, M. D. (2000). ''Test of a free flange connection with a composite floor slab.'' *SAC Background Document SAC/BD-00/18*, SAC Joint Venture, Richmond, Calif.

Wen, Y. K., and Foutch, D. A. (1997). ''Proposed statistical and reliability framework for comparing and evaluating predictive models for evaluation and design, and critical issues in developing such framework.'' *SAC Background Document SAC/BD-97/03*, SAC Joint Venture, Richmond, Calif.

Yun, S. Y., and Foutch, D. A. (2000). ''Performance prediction and evaluation of low ductility steel moment frames for seismic loads.'' *SAC Background Document SAC/BD-00/26*, SAC Joint Venture, Richmond, Calif.