Probabilistic Modeling of Steel Moment Frames with Welded Connections

JIANLIN SONG and BRUCE R. ELLINGWOOD

ABSTRACT

Improved building practices for earthquake-resistant design and evaluation rely on accurate nonlinear analysis procedures and a rational approach to dealing with uncertainties. To address building performance issues and support probability-based evaluation of steel buildings, two steel moment-resisting frames (WSMF) that suffered damage to welded connections in the Northridge Earthquake are evaluated using deterministic and stochastic approaches. Nonlinear dynamic analyses of these two buildings utilize a new degrading hysteretic connection model that incorporates the effects of weld fractures. The role of inherent randomness and modeling uncertainties in ground motion and structural resistance in forecasting or explaining observed building performance is examined for one of the buildings, leading to a probabilistic description of building performance and insights that are useful for condition assessment and performance-based design.

KEY WORDS

Buildings (codes); Design (buildings); Connections; Earthquakes; Frames; Loads (forces); Safety; Steel; Structural Engineering; Welding.

INTRODUCTION

Recent advances in building practices for earthquake hazard mitigation and new trends toward performance-based design increasingly rely on advanced nonlinear analysis procedures. Such analysis procedures, to be credible as design and evaluation tools must be validated by experimental data on building performance during earthquakes. The performance of welded steel moment frame (WSMF) buildings during the Northridge Earthquake of January 17, 1994 and subsequent surveys of building damage provide an excellent opportunity to validate and assess the limitations of current analytical models, and to examine the role of uncertainty when such models are used in forecasting building performance. This study was undertaken to address these issues and the insights gained are believed to be useful for code improvements, building condition assessment, and implementation of policies regarding building rehabilitation.

The Northridge Earthquake caused weld fractures and other damage to a large number of beam-to-column connections in welded steel moment-resisting frames. Subsequent to the earthquake, the National Institute of Standards and Technology managed or was otherwise involved in surveys of a number of WSMF buildings that had sustained damage (Youssef, et al, 1995; Kaufmann, et al, 1997). Four of these buildings have been analyzed in some detail (Song and Ellingwood, 1999a); two of them form the basis for the investigations of building performance reported in this paper.

MODELING STEEL FRAMES SUBJECTED TO EARTHQUAKES

Modern seismic-resistant design of WSMF structures requires that the frame tolerate significant inelastic behavior during a large earthquake (AISC, 1993; FEMA, 1994; AISC,

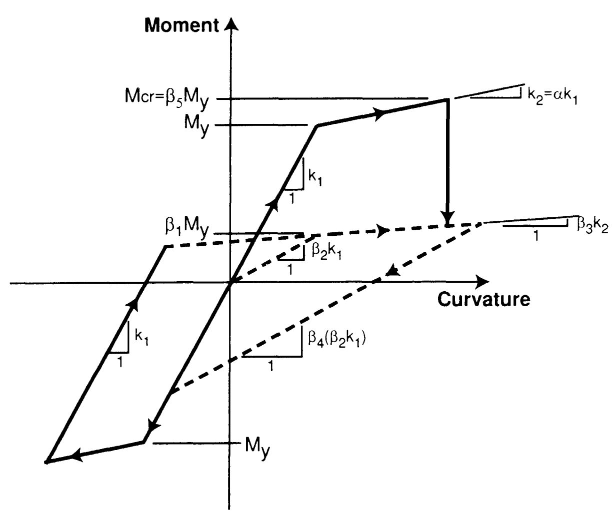

Fig. 1. Hysteresis Model for Damaged Welded Connection.

Jianlin Song is senior research engineer with Applied Insurance Research, Boston, MA.

Bruce R. Ellingwood is professor, department of civil engineering, The Johns Hopkins University, Baltimore, MD.

1997). One common beam-to-column connection in WSMF's is the welded flange-bolted web connection. If the beam flange-to-column weld fractures, the connection cannot fully resist the required moment, the rotational stiffness of the connection deteriorates under subsequent cyclic loading, and the hysteretic energy that can be dissipated through cyclic inelastic deformation decreases.

To model such behavior, a new hysteretic model (Kunnath, 1995; Gross, 1997) that incorporates the effects of damage due to weld fracture and subsequent nonlinear response in the connection region was adopted for this study. The model is illustrated in Figure 1 and is based on recent tests of structural connections (SAC, 1996). The behavior of the undamaged connection is characterized by a bilinear envelope. The moment at weld fracture is denoted by M_{cr}, which is specified as a fraction of the yield moment, M_y. Following weld fracture, the primary envelope is replaced by a degraded bilinear representation with reduced stiffness $\beta_2 k_1$, reduced capacity $\beta_1 M_y$ and post-yield slope $\beta_3 k_2$, and degraded unloading stiffness $\beta_2 \beta_4 k_1$. Since weld fractures that were observed in the damaged buildings surveyed occurred primarily in the bottom flanges, the model was designed to predict fractures that initiated at the weld root of the bottom beam flange-to-column flange groove weld and propagated so as to disconnect the lower beam flange from the column flange. The hysteresis loops on the negative side are assumed to retain their original stiffness and capacity. This new connection hysteresis model was incorporated in an inelastic dynamic analysis program (Kunnath, 1995), which was used to evaluate the frames as systems. The program allows the beam-column panel zone to be modeled, if desired, and takes second-order geometrically nonlinear ($P - \Delta$) effects into account.

DETERMINISTIC RESPONSE OF WSM FRAMES

The two buildings considered are both office buildings and are believed to be representative of WSMF buildings that were damaged in the earthquake. The first is identified as Building C, while the second is the Blue Cross Building (Uang, et al, 1995). Typical elevations of their moment frames are shown in Figures 2 and 3; the frames illustrated are those that sustained the more severe damage. These figures also include indicators of surveyed and predicted damage to the frames, which will be discussed subsequently.

Building C is a four-story building above three levels of below-grade parking. The building was designed about 1985.

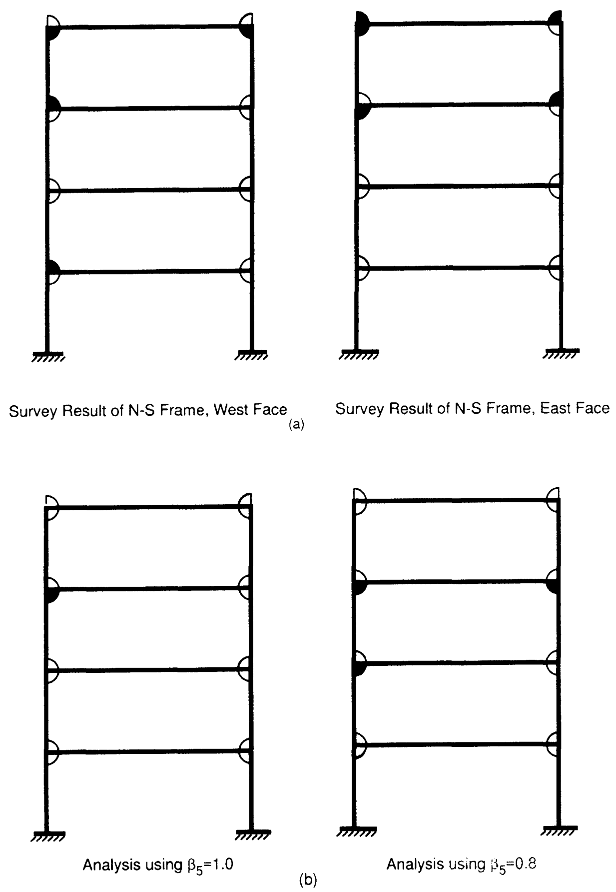

Fig. 2. (a) Surveyed and (b) Predicted Damage for N-S Frames in Building C.

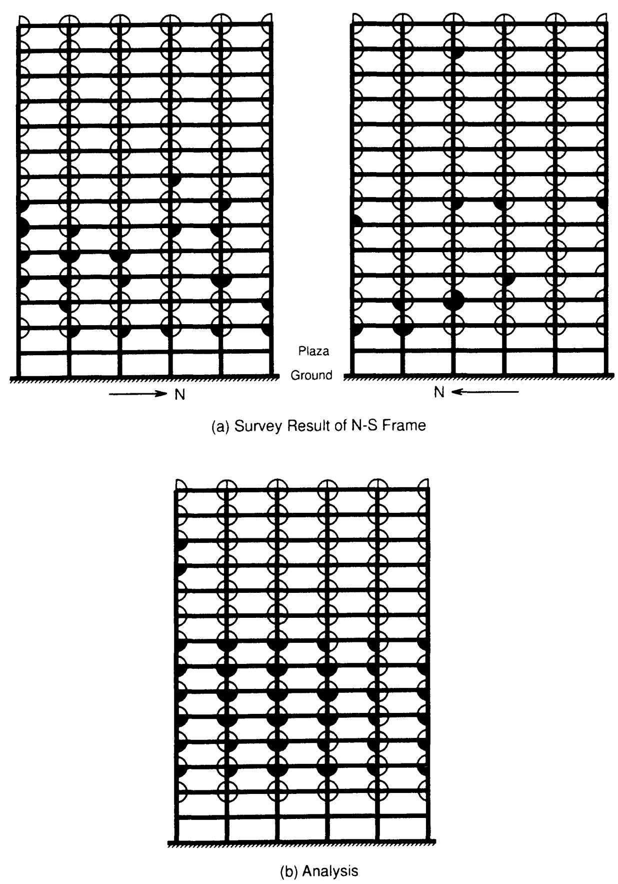

Fig. 3. (a) Surveyed and (b) Predicted Damage for N-S Frames in the Blue Cross Building.

Table 1. Calculated Fundamentals of Building C and the Blue Cross Building						
	Bldg. Fundamental Period(s)					
	This Study with no panel zone		This study with panel zone		Comparison Values	
Bldg.	E-W	N-S	E-W	N-S	E-W	N-S
C	1.63	1.60	1.65	1.62	1.54	1.51
Blue Cross	N/A	N/A	3.05	3.05	2.36	2.66

It is located approximately 9.5 km (5.9 mi.) from the epicenter on a site characterized as "recent alluvium" (Graves, et al, 1995.) The plan dimensions are 44.5 m (146 ft.) by 33.2 m (109 ft.). Typical story heights range from 4.04 m (13 ft. 3in.) to 4.72 m (15 ft. 6 in.) in the office levels and from 3.05 m to 3.73 m (10 ft. to 12 ft. 3 in.) in the parking levels. The building has two single-bay exterior moment frames in both N-S (Figure 2) and E-W directions, with bay widths of approximately 12 m (40 ft.) The Blue Cross Headquarters facility is a thirteen-story (above the plaza) building, which was designed using the 1973 Uniform Building Code and was built in 1975. It is located in the San Fernando Valley, approximately 4.8 km (3 mi.) southwest from the epicenter. The plan dimensions are 48.77 m (160 ft.) by 48.77 m (160 ft.). Typical story heights are 4.01 m (13 ft. 2 in.) The building has five-bay exterior moment frames on all four sides (e.g. Figure 3 for two N-S frames). Details of the framing systems of these four buildings are presented elsewhere (Song, 1998).

Mechanical properties of the beams, columns and weld metal for Building C were determined from tests conducted at Lehigh University (Kaufmann et al, 1997). The beams were ASTM A36 structural shapes, while the columns were of ASTM A572 Grade 50 steel. Since our objective in this phase of the analysis is to obtain a mean-centered estimate of performance, the average yield strength from these tests were used: for the beams, 291 MPa (42 ksi) and for the columns, 393 MPa (57 ksi). The beams and columns in the Blue Cross Building reportedly were ASTM A36 steel, although specific test data were not available. Thus, a yield strength of 276 MPa (40 ksi) was assumed in the analysis of the Blue Cross Building.

The ground motions at the site of Buildings C were developed in Phase 1 of the SAC Joint Venture Project by Woodward-Clyde (Somerville, et al, 1995). A suite of nine time histories for the two horizontal directions (N-S and E-W) and one vertical direction (U-D) was generated at points defined by a 1-km square grid, centered on the site so that the ground motion at the building site itself is surrounded by eight neighboring (and essentially equally likely) ground motions. However, no motions at the site were recorded. In contrast, the Blue Cross Building was instrumented at the basement, 6th-floor, and the roof by the California Division of Mines and Geology (CDMG), and the recorded ground motion at the basement was used directly in its dynamic analysis. These ground motions are displayed elsewhere (Graves, et al, 1995; Uang, et al, 1995).

In the nonlinear dynamic response analyses of the buildings, the steel frames were modeled as planar structures. The effects of torsion were neglected, composite action from floor slabs was not considered, and floor diaphragms were assumed to be rigid in-plane. The masses of the floors were estimated using the dead load of the building plus the live load of furniture and file cabinets, etc, which was assumed to be 0.72 kPa (15 psf). Both frames were assumed to be fixed at the base. Damping was assumed to be 2 percent of critical for Buildings C and 5 percent of critical for the Blue Cross Building, at which the ground motion intensity was higher. The properties of the beam-column panel zone were assumed to be the same as that of the column, with elastic and shear modulus of 200,000 MPa (29,000 ksi) and 77,220 MPa (11,200 ksi), respectively. A bilinear force-displacement model with $k_2 = 0.05k_1$ was assumed for all columns, whereas the beams were modeled by both the bilinear model and degrading model (Figure 1), depending on the purpose of the analysis. Other hysteresis parameters are $\beta_1 = 0.4$, $\beta_2 = 0.2$, $\beta_3 = 0.4$, $\beta_4 = 1.0$, $\beta_5 = 1.0$ (Song, 1998).

The fundamental building periods determined for frames modeled with and without the beam-column panel zone are presented in Table 1. The panel zone model results in a slightly more flexible structure. Note that panel zone yielding can reduce the likelihood of beam fracture by limiting the moment that can develop in the beam. Table 1 also shows periods determined by Kaufmann, et al (1997) for Buildings C from a 3-D analysis using ETABS and periods measured for the Blue Cross Building from the recorded (CDMG) roof displacements. There is reasonable agreement between the periods determined in the current study with those obtained previously. Modal analysis of both buildings showed that their responses were dominated by the first mode (Song and Ellingwood, 1999a). Static pushover analyses of each frame (Song, 1998) revealed that deviations from linearity occur at overall deformations that are approximately 1 percent of the building height.

The results of the time history analyses in the direction of the frame that experienced the stronger ground motion and comparisons of predicted and observed connection damage for these buildings are summarized in the following. Comparisons of predicted and observed connection damage are presented in Figures 2 and 3. Each connection that was inspected is represented by a circle on a sketch of the frame of that building. If the connection experienced damage of the type that can be predicted from the hysteresis model in Figure 1, the circle is darkened at the corresponding location.

Table 2. Random Material Strength Parameters			
Parameter	**Mean**	***COV***	**CDF**
β_1	0.4	0.29	Uniform
β_5	0.95	0.09	Uniform
$F_{y\,col}(MPa)$	393	0.12	Lognormal
$F_{y\,beam}(MPa)$	290	0.12	Lognormal
$F_{y\,pan}(MPa)$	414	0.09	Uniform
$E(GPa)$	200	0.06	Uniform
$G(GPa)$	77	0.09	Uniform
ξ_b	2.33%	0.62	Histogram from Coats (1989)

1. Building C
   While some damage was observed in the building survey [Figure 2(a)], the analysis (using $\beta_5 = 1$ and the "nz-deg" model) predicted only one damaged connection at the third floor. The small level of connection damage leads to predicted absolute roof displacements that are virtually the same with degrading and bilinear connection models [Song and Ellingwood (1999a), Figure 15]. An analysis of variance (Song, 1998) indicated that the moment at fracture (defined by β_5) was the most important hysteresis parameter. Moreover, experimental data (Xue, et al, 1998; Kaufmann, et al, 1997) also indicated that fracture of such connections may occur at nominal stresses less than the elastic limit. Accordingly, additional analyses of Building C were conducted using other plausible values of $\beta_5 \leq 1.0$, holding other structural parameters and ground motions unchanged. The agreement between predicted and surveyed damage in terms of the number of connections damaged was optimal when $\beta_5 = 0.8$ [cf Figure 2(b)].

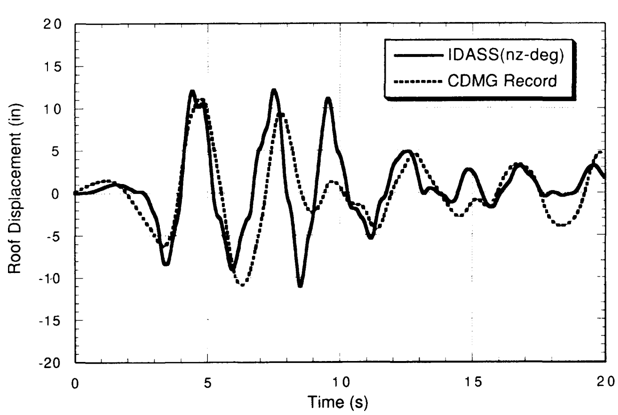

Fig. 4. Roof Displacement of the Blue Cross Building (N-S Direction).

2. Blue Cross Building
   The results of inelastic MDOF time history analysis of the Blue Cross Building are compared to the CDMG measurements in Figure 4. It was found that the panel zone did not need to be modeled in the Blue Cross Building because of its frame flexibility (Song, 1998). The N-S frame roof displacements using model "nz-deg" compare reasonably well to the displacements measured by the CDMG. The analysis predicted a slightly higher roof displacement than the recorded displacement, suggesting that nonstructural or nonlateral force-resisting structural members and components contributed to earthquake resistance. The general pattern of predicted and surveyed connection failures is similar.

The results of the above deterministic analyses [and analyses of other buildings presented elsewhere (Song and Ellingwood, 1999a)] show that predictions of damage in steel frame buildings subjected to strong ground motion using advanced nonlinear dynamic analysis tools may not match what is observed. The lack of agreement may be attributed, in part, to omissions in the modeling process. For example, the contribution to lateral force resistance of vertical load-carrying columns that are not part of the seismic moment-resisting frames was not included; the weight of the building was assumed to be carried only by moment-resisting frames; torsional motions were not considered; the bare-frame analysis neglects the contribution of nonstructural components to lateral force resistance. Moreover, structural system properties, such as stiffness, mass, and damping, actually are random, and there are uncertainties in the members' mechanical properties and in modeling the nonlinear behavior of the connections. Finally, uncertainties in earthquake ground motion are known to be significant. Thus, the lack of agreement may be due as much to inherent variability in the parameters or modeling uncertainties as deficiencies in the structural models. A probabilistic analysis of building response to earthquake ground motion can shed additional light on these comparisons by indicating the quality of agreement between predicted and observed damage that might be expected, given the level of uncertainty in the problem. With this aim, the N-S frames of Building C were selected for more detailed analysis of stochastic response.

STOCHASTIC RESPONSE ANALYSIS OF BUILDING C

Table 2 identifies the structural parameters for Building C that are treated as random variables. Several parameters are modeled by uniform distributions for conservatism in the absence of further data. The mean values 0.4 and 0.95 for β_1 and β_5 are consistent with other experimental data. Other hysteresis parameters are treated as deterministic on the basis of an analysis of variance (Song, 1998). Gravity loads are known to have a small variability compared to the uncertainty in the earthquake loading (O'Connor and Ellingwood, 1987), and the gravity loads (and building mass) are assumed to equal their mean values.

Stochastic analysis of nonlinear building response in the time domain requires an ensemble of ground motions (e.g., Shome, et al, 1997). The ensemble of ground motions simulated by Woodward-Clyde (Graves, et al, 1995) for the 1 km grid surrounding Building C might serve this purpose. Alternatively, one might select accelerograms recorded during actual earthquakes with comparable magnitudes and epicentral distances. With either approach, the peak ground motion intensities vary from record to record. Seismic performance assessment and risk analysis require a common ground motion intensity measure so that the (random) building resistance can be keyed to this measure. Recent NEHRP Recommendations (1994) specify the seismic hazard in terms of spectral acceleration at (or near) the fundamental period of the building rather than peak ground acceleration. Therefore, spectral acceleration, S_a, at the fundamental period of Building C was chosen to characterize the ground motion intensities for those analyses (described subsequently) in which a common intensity is required.

Ground motions were scaled using the ratio of the spectral acceleration (at 2 percent damping) from the center Woodward-Clyde accelerogram at the period (1.6 sec.) of Building C, i.e., $S_a = 0.23g$ (Song, 1998), to the spectral accelerations from the remaining records at the same period and damping. This scaling results in ground motions from which one obtains the same roof acceleration of all (deterministic) structures from a SDOF elastic analysis. To test the sensitivity of predicted building response and connection damage to alternative approaches to ground motion modeling, nine accelerograms also were selected from earthquakes with $5.3 \leq M \leq 6.7$ and 5 km $\leq R \leq$ 24 km (Song, 1998). (Recall that Building C is located approximately 9.5 km from the epicenter of the Northridge Earthquake, which had a magnitude of 6.7.) Sensitivity studies (Song, 1998) have shown that ensembles of nine accelerograms are sufficient for the statistical analyses presented subsequently in this paper.

Four experimental designs for the N-S frames of Building C were considered: (1) Simulated motions, scaled to $0.23g$; (2) Simulated motions, unscaled; (3) Actual motions, scaled to $0.23g$; and (4) Actual motions, unscaled. These four experiments test the impact on building response of alternate ground motion modeling procedures. In each experimental design, the uncertainties in ground motion and in the remaining structural parameters are propagated using a Latin Hypercube sampling technique (O'Connor and Ellingwood, 1987; Song and Ellingwood, 1999b).

Table 3.
Statistical Analysis of Connection Damage in Building C (N-S Direction)

	Experiment Number			
	1 Simulated $S_a = 0.23g$	**2 Simulated Unscaled**	**3 Actual** $S_a = 0.23g$	**4 Actual Unscaled**
Mean	2.0	3.1	2.4	1.6
SD	1.7	1.7	2.8	2.8

Damage patterns predicted in experiments 1 through 4 varied considerably from sample to sample (e.g., Figure 3 of Song and Ellingwood, 1999b). Table 3 summarizes the mean and standard deviation (*SD*) of the number of connections (out of 16) found to be damaged in each experiment. The number of connections predicted as damaged ranged from 0 to 8 (in experiment 3). The *SD* is of the same order as the mean. The observed ratio of damage was 4/16, which falls within one *SD* of the mean in all cases but experiment 1 where it is barely outside that range. This statistical analysis offers a broader perspective on connection damage that is likely to occur during an earthquake of a given magnitude than can be obtained from a single deterministic analysis.

PROBABILISTIC MODEL OF BUILDING C RESISTANCE

Acceptable structural performance requires safety against collapse or other life-threatening damage, and limitations on deformations. Each of these conditions of structural behavior may be termed a "limit state". In this study, limit states are identified with deformations measured by the maximum interstory drift angle, which is consistent with research carried out elsewhere as part of the SAC Joint Venture (Wen and Foutch, 1997; Luco and Cornell, 1997). The maximum interstory drift angle is defined as,

$$ISDA = \max_{i=1}^{4}\left(\frac{\delta_i}{h_i}\right) \tag{1}$$

where δ_i is the maximum inter-story drift for story i and h_i is the story height which in Building C is 4.03 m (159 in.) for stories 2 through 4 and 4.72 m (186 in.) for story 1. Two levels of performance and their corresponding hypothesized limit states are assumed: $ISDA$ = 1 percent (local damage; the point at which the pushover analyses indicated the onset of nonlinear action in the frame); and $ISDA$ = 5 percent (severe damage).

The resistance of Building C as a system can be described probabilistically by its fragility, $F_R(x)$. The fragility is defined as the limit state probability, conditioned on a specific control variable (here, spectral acceleration at the fundamental building period) that is consistent with the specification of the seismic hazard;

$$F_R(x) = P[LS \mid S_a = x] \tag{2}$$

where LS represents the corresponding "limit state" and spectral acceleration, S_a, at the fundamental period of the building is the control variable. Building fragilities can be described by lognormal probability distribution (Song and Ellingwood, 1996b).

The fragility for a limit state is obtained from the cumulative distribution function (CDF) of the *ISDA*. For example, if the limit state is 5 percent *ISDA*, then,

$$P(LS \mid S_a = x) = P[ISDA > 5\% \mid S_a = x] \tag{3}$$

To determine these conditional probabilities, the ground motion ensembles are scaled to different values of S_a at the fundamental period of Building C over the range of interest, the corresponding dynamic responses of the N-S frame to these ensembles are determined, the responses are rank-ordered and plotted on lognormal probability plots, and Equation 3 is used to determine the fragilities for increasing levels of S_a (Song, 1998). To illustrate, four combinations of connection hysteresis and ground motion models were considered: (1) Bilinear structural model with simulated ground motions; (2) Degraded/simulated; (3) Bilinear/actual; and (4) Degraded/actual.

Figure 5 presents the fragilities based on *ISDA* for the two deformation limit states using bilinear and degraded hysteresis models and simulated (based on the Woodward-Clyde study) ground motions. These fragilities are interpreted in the following way (e.g., for the degraded model): the median (50th percentile) spectral acceleration (e.g., the spectral acceleration at which "failure/nonfailure" is equally likely) increases from $0.16g$ for *ISDA* = 1 percent to $1.06g$ for *ISDA* = 5 percent, with increasing severity in the limit state. The corresponding 10 percent exclusion limit fragilities (i.e., those spectral accelerations at which each limit state is only 10-percent probable) are $0.12g$ and $0.75g$. These latter spectral accelerations are comparable, in a sense, to the specified nominal strengths appearing in standards such as the LRFD Specification (AISC, 1993). However, they pertain to the N-S structural frame of Building C as a system rather than to the strength of any one beam or column in that frame.

Figure 5 indicates that the connection hysteresis model apparently has little impact on the fragility when the limit state is defined by *ISDA* equal to 1 percent. The choice of connection hysteresis model affects the building fragilities only when the "severe damage" limit state is approached. The minor role played by connection damage at less severe limit states has been noted by others using different hysteresis models and structural analysis platforms (Wang and Wen, 1998). Moreover, the increasing connection damage at higher S_a causes the variability in *ISDA* fragility to increase. For moderate damage levels, the coefficient of variation is typically on the order of 0.10–0.30, increasing to the order of 0.30–0.60 as damage becomes more severe.

ISDA fragilities using the degraded connection model with simulated or actual ground motions are compared in Figure 6. The median fragility is little affected by the choice of ground motion model if the limit state is 1 percent *ISDA*. However, the 10 percent exclusion limits differ, even at the 1 percent *ISDA* level: $0.13g$ using the simulated ground motions, and $0.09g$ using the actual ground motions. At the 5 percent *ISDA* level, the differences are greater: $0.76g$ vs. $0.56g$. Such differences in fragility may influence the condition assessment of an existing building in a substantial way.

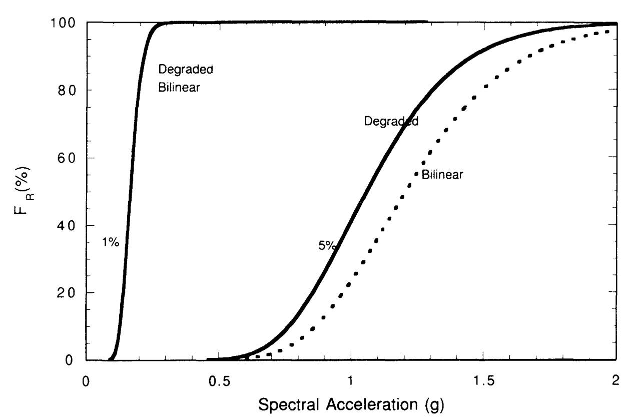

Fig. 5. Fragility for ISDA using Degraded and Bilinear Model (Simulated Ground Motions).

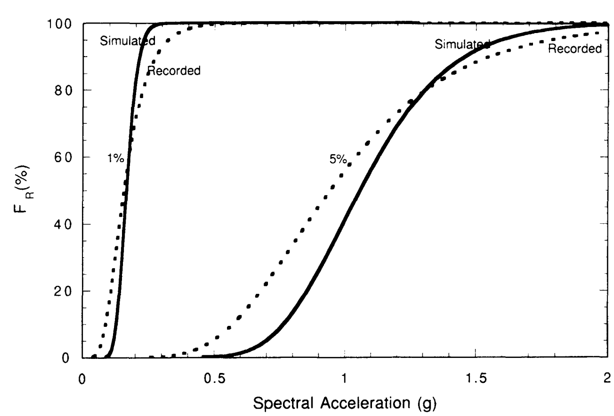

Fig. 6. Fragility for ISDA using Simulated and Recorded Ground Motions (Degraded Hysteresis Model).

We now illustrate with two examples how such fragilities might be used in professional practice.

In the first example, we envision a requirement that an existing building be evaluated to determine its susceptibility to damage or failure during a review-level earthquake of specified magnitude. This earthquake might have been identified as a result of recent seismic hazard analysis showing that previous design requirements were inadequate, or might be specified by the authority having jurisdiction as a result of a proposed change in building occupancy to a higher seismic category. We suppose that the analysis of the building frame has led to a family of fragilities such as those presented in Figure 6. One then might envision a safety requirement for an ordinary office building that "severe damage" will not occur with 90 percent confidence as a result of a earthquake with a 500-year mean recurrence interval (MRI). The spectral acceleration (at the period 1.6 s) associated with this earthquake may be on the order of 0.6*g* (a reasonable value according to the latest 500-yr MRI NEHRP seismic hazard maps in Southern California). On this basis, the building frame would be judged acceptable using the simulated ground motion ensemble (the 10-percentile is 0.76*g* for $ISDA > 5$ percent). However, if the ensemble of actual ground motions were used, the building frame would be judged unacceptable (the 10-percentile is 0.58*g* for $ISDA > 5$ percent).

The second example might arise from an insurance underwriter's need to determine the insurability or premium associated with a specific building. Here the process is somewhat similar, in that an earthquake beyond the original design basis may be of interest in order to answer a question regarding an appropriate underwriting risk or premium. Such questions — How would a particular building respond to an event of Magnitude 7? — frequently arise. Assuming, for illustration, a spectral acceleration associated with the event in question of 0.6*g*, as before, and on the basis of a suite of recorded ground motions selected for conservatism, we find that the probability of minor damage to the building frame is nearly 100 percent. The probability of severe damage to the frame (and, presumably damage to the building contents) is less, on the order of 1.5 percent to 12 percent, depending on the integrity of the connections. Such probabilities might be used to aid in setting a premium to insure the building and its contents.

CONCLUSIONS AND RECOMMENDATIONS

The contribution of uncertainty in hysteresis to building fragility appears to be small in comparison to the contribution due to ground motion. Accordingly, mean or median-centered estimates of the hysteresis parameters should be sufficient for reliability-based condition assessment of buildings. However, additional tests to better define the median connection hysteresis parameters would be desirable. The building fragility is unaffected by the bilinear vs. degraded connection model when the limit states are defined by interstory (or roof) drift angles of 1 percent or less. The difference becomes more apparent when spectral accelerations increase to the point that a severe damage limit state is approached. Moreover, the fragility is more sensitive to the ground motion modeling at limit states defined by *ISDA* of 5 percent (or higher).

Fragilities of building frames can be used in a number of contexts: for evaluating the suitability of alternate design or code proposals; for condition assessment of existing buildings for postulated natural hazards; and for determining risks for underwriting purposes. The framework provided by fragility modeling for the analysis of various sources of uncertainty and the depiction of their contribution to the response of building frames over a range of challenges can be used to establish perspectives on building performance that are not possible with traditional deterministic approaches to evaluation.

ACKNOWLEDGMENTS

We would like to thank Dr. John L. Gross of NIST for offering invaluable comments and suggestions throughout the study, and Dr. Sashi Kunnath of the University of Southern Florida for providing an early version of the IDASS computer program and helping implementing new features into the program. The National Institute of Standards and Technology provided assistance with computing facilities, building models and earthquake ground motions. The comments from reviewers of the manuscript also are gratefully acknowledged.

REFERENCES

American Institute of Steel Construction (1997), *Seismic Provisions for Structural Steel Buildings*, Chicago, IL.

American Institute of Steel Construction (1993), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, Chicago, IL.

Coats, Jr., D. W. (1989), "Damping in Building Structures During Earthquakes, Test Data and Modeling," *NUREGICR-3006*, U.S. Nuclear Regulatory Commission, Washington, D.C.

Federal Emergency Management Agency (1994), *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, Building Seismic Safety Council, Vol. 1 and 2.

Graves, R. W., Somerville, P. G., and McLaren, J. P. (1995), "Ground Motion Time Histories of the 1994 Northridge Earthquake at Three Sites," *Report* prepared to NIST by Woodward-Clyde Federal Services, Pasadena, CA.

Gross, J. L. (1997), "A Connection Model for the Seismic Analysis of Welded Steel Moment Frames," *Engineering Structures*, 20, pp. 4-6, Elsevier Science Ltd.

Kaufmann, E. J., Fisher, J. W., Di Julio, Jr., R. M., and Gross, J. L. (1997), "Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake," *NISTIR 5944*, National Institute of Standards and Technology, Gaithersburg, MD.

Kunnath, S. K. (1995), "Enhancements to Program IDARC: Modeling Inelastic Behavior of Welded Connections in Steel Moment-Resisting Frames," *NIST GCR 95-673*, Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, MD.

Luco, N, and Cornell, C. A. (1997), "Numerical Example of the Proposed SAC Procedure for Assessing the Annual Exceedance Probabilities of Specified Drift Demands and of Drift Capacity," *SAC Report*, SAC Joint Venture, Sacramento, CA.

O'Connor, J. M. and Ellingwood, B. R. (1987), "Reliability of Nonlinear Structures with Seismic Loading," *Journal of Structural Engineering,* ASCE, 113(5).

SAC Joint Venture (1996), "Experimental Investigations of Beam-Column Subassemblages," *Report No. SAC-96-01,* Part 1 and 2, Sacramento, CA.

Shome, N., Cornell, C. A., Bazzurro, P., and Carballo, J. E. (1997), "Earthquakes, Records and Nonlinear MDOF Responses," *Report No. RMS-29,* Reliability of Marine Structures Program, Stanford University, Stanford, CA.

Somerville, P., Graves, R., and Saikia, C. (1995), "Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994," *Report No. SAC-95-03,* Sacramento, CA.

Song, J. (1998), "Seismic Reliability Evaluation of Steel Frames with Damaged Welded Connections," *Ph.D. Dissertation*, The Johns Hopkins University, Baltimore, MD.

Song, J. and Ellingwood, B. R. (1999), "Seismic Reliability of Special Moment Steel Frames with Welded Connections: I," *Journal of Structural Engineering,* ASCE, 125(4).

Song, J. and Ellingwood, B. R. (1999), "Seismic Reliability of Special Moment Steel Frames with Welded Connections: II," *Journal of Structural Engineering,* ASCE, 125(4).

Structural Engineers Association of California (1990), *Recommended Lateral Force Requirements and Commentary*, Seismology Committee, Sacramento, CA.

Uang, C. M., Yu, Q. S., Sadre, A, Bonowitz, D., and Youssef, N. (1995), "Performance of a 13-story Steel Moment-Resisting Frame Damaged in the 1994 Northridge Earthquake," *Technical Report SAC 95-04.* SAC Joint Venture.

Wang, C. H. and Wen, Y. K. (1998), "Reliability and Redundancy of Pre-Northridge Low-Rise Steel Buildings Under Seismic Excitation," *Report No. UILU-ENG-99-2002,* University of Illinois at Urbana-Champaign, Urbana, IL.

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," *Report No. SACIBD-97103*, SAC Joint Venture, Sacramento, CA.

Xue, M, Kaufmann, E. J., Lu, L. W., and Fisher, J.W. (1998), "Fracture and Ductility of Welded Moment Connections under Dynamic Loading," *Proc. Structures Congress,* ASCE, XV, Vol. 1, pp. 607-613.

Youssef, N. F. G., Bonowitz, D., and Gross, J. L. (1995), "A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake," *Report No. NISTIR5625,* National Institute of Standards and Technology, Gaithersburg, MD.