PROBABILISTIC MODELING OF SPECIAL MOMENT-RESISTING FRAMES UNDER EARTHQUAKE

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SUMMARY

Two steel moment-resisting frames experienced connection fracture damage in 1994 Northridge earthquake were analysized using both deterministic and stochastic approaches. Deterministic approach seems to match damage pattern from damage survey. Stochastic approach can shed more lights on the possible pattern of the future damage. Fragility curve was convolved with seismic hazard to estimate reliability of the buildings.

INTRODUCTION

Many of the connection failures in 1994 Northridge earthquake observed a weld fracture that initiated at the root of the groove weld connecting the bottom flange of the beam to the column flange, and that subsequently propagated in such a way as to disconnect the bottom flange of the beam from the column. The new degrading hysteresis model, which captures the effects of weld fracture and subsequent nonlinear response of the connection region at the end of beam, was developed by Kunnath (Kunnath, 1995; Gross, 1998). The features of the model are shown in Figure 1. The model prior to weld fracture is characterized by a bilinear envelope with yield capacity My. Weld fracture is presumed to occur at a moment denoted by Mcr. Subsequent to weld fracture, a reduced stiffness and strength bilinear regime is followed. Unloading from this regime results in a further reduction in the stiffness in Figure 1. This paper uses IDASS (Inelastic Damage Analysis of Structural System, Kunnath,1995), which incorporates this degrading connection model, as an analytical tool.

DETERMINISTIC MODELING OF FRAMES

The two office buildings, identified as Building B and Building BC, were under consideration. Typical elevations of their moment frames are shown in parts (a) of Figures 2 and 3. Building B is a four-story building with two levels of parking below ground. Designed according to the 1980 Los Angeles City Building Code, the building is located about 19.3 km (12 mi) from the epicenter. Plan dimensions are 43 m (140 ft) by 26 m (86 ft). Typical story heights are 4.27 m (14 ft). Constructed in1975, Building BC is a 13-story building located about 5 km from the epicenter. Its plan dimensions are 48 m by 48 m; each bay is 9.75 m and the story heights are 4 m. Details of their framing systems are presented elsewhere (Song and Ellingwood, 1999a).

Mechanical properties of beams, columns and weld metal for Buildings B were determined by tests conducted at Lehigh University (Kaufmann et al, 1997). Yielding strength of 283 Mpa (41.0 ksi) and 259 Mpa (37.5 ksi) were used for beam and column, respectively. A value of 276 Mpa (40 ksi) was assumed for Building BC since no tests were available for this building. Damping was assumed to be 2% and 5% of critical for Building B and BC, respectively. The fundamental building periods determined for the two N-S frames of 2-D models were 1.7 s and 3.1 s for Building B and Building BC, respectively.

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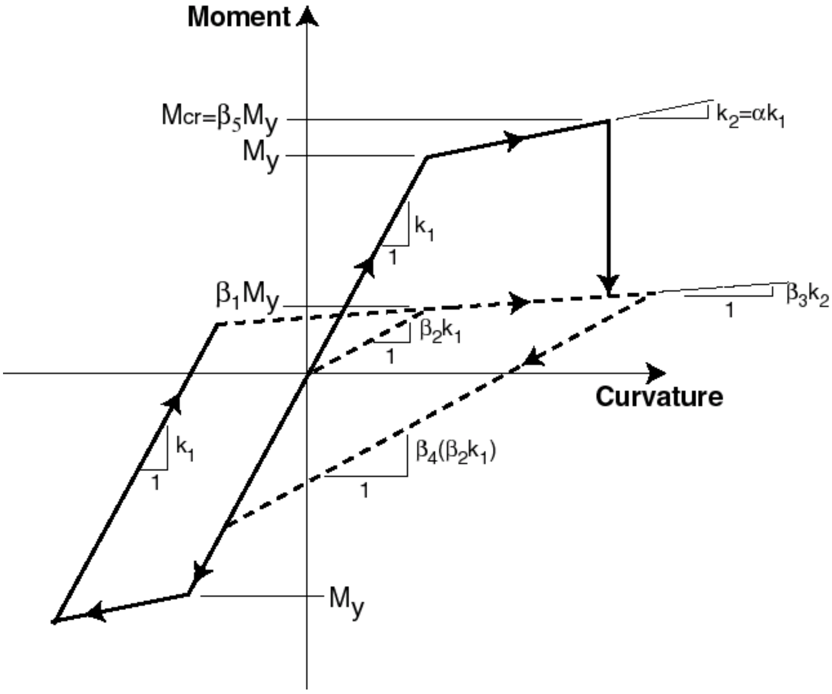

Figure1: Hysteresis Model for Damaged Welded Connection

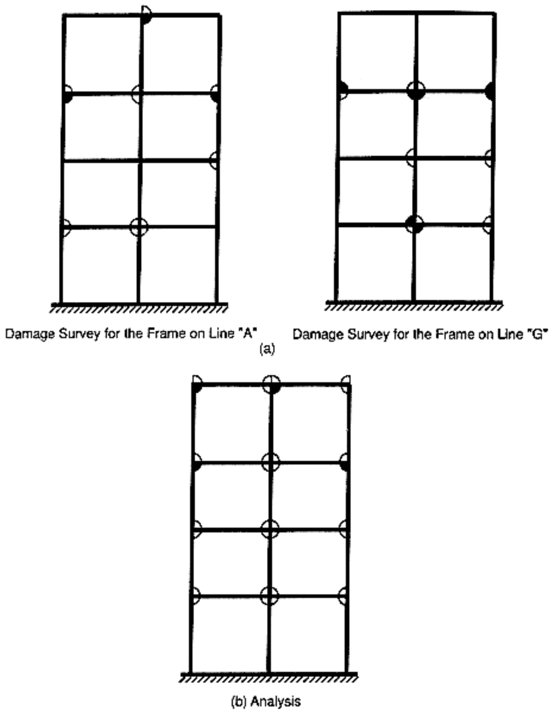

Figure 2: (a) Surveyed and (b) Predicted Damage for N-S Frames in Building B

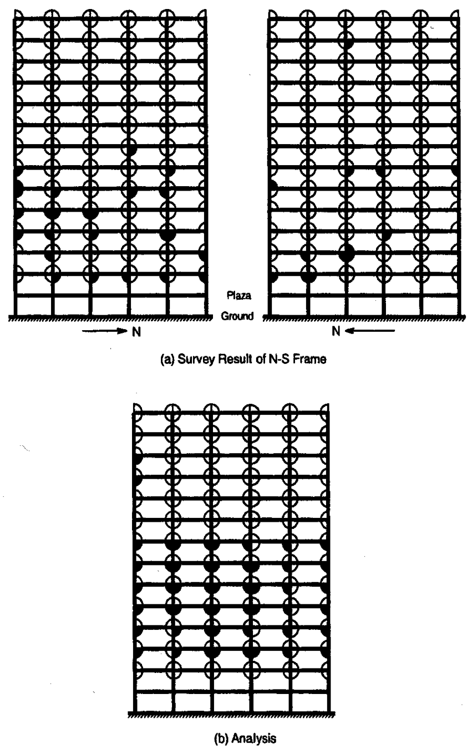

Figure 3: (a) Surveyed and (b) Predicted Damage for N-S Frames in Building BC

Ground motions from the Northridge earthquake were unavailable at Building B site. Therefore, the ground motions were those developed in Phase 1 of the SAC Joint Venture by Woodward-Clyde (Somerville, et al, 1995). For Building BC, the ground motion that had been recorded in its basement by California Division of Mines and Geology was applied to the model. Comparisons of predicted and observed connection damage are presented in Figure 2 and 3. Each connection that was inspected is represented by a circle (or a portion of a circle, depending on the extent of inspection) on a sketch of the frame of that building. If the connection experienced damage of the kind that can be predicted by the model in Figure 1, the circle is darkened at the corresponding location. For Building B (Figure 2), although there are some differences in the predicted and observed patterns of damage, e.g., some damage was surveyed on the first floor that is not predicted by the analysis, reasonable agreement can be found between the overall levels of observed and computed damage. The predicted response of Building BC (Figure 3) to the earthquake indicates the occurrence of failures in mid-lower floors, This is consistent with the damage survey.

STOCHASTIC RESPONSE ANALYSIS

Predictions of damage in steel frames subjected to strong ground motion using even 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 and uncertainties in structural system properties, members' mechanical properties, nonlinear behavior of the connections, and earthquake ground motion. A probabilistic rather than deterministic analysis of building response to earthquake ground motion can place such comparisons in better perspective by indicating the agreement between predicted and observed damage that might be expected, given the level of uncertainty in the problem.

The structural parameters that are treated as random variables and their distributions are listed in Table 1. An ensemble of ground motions is required for a stochastic analysis of building response in the time domain (e.g., Shome and Cornell, 1998). The ensembles of nine ground motions simulated by Woodward-Clyde (Graves, et al, 1995) for the 1-km grid surrounding Buildings B were used. In contrast, nine accelerograms from historic earthquakes with magnitudes range from 5.3 and 6.7 and epicentral distances ranging from 5 km to 24 km were used for Building BC. The peak ground motion intensities vary from record to record. Spectral acceleration was chosen to characterize the ground motion intensities. Ground motions were scaled using the ratio of the spectral acceleration from the center Woodward-Clyde accelerogram (S_a=0.17 g for Building B) at the fundamental period (1.70 s), to the spectral accelerations from the remaining eight records at site for the same period and damping. In other words, the ground motions are scaled so that the same roof acceleration would be achieved from a SDOF elastic analysis of a deterministic model of the structure. Two separate experimental designs were considered for each building: (1) ground motions scaled to 0.17 g for Building B and 0.09 g for Building BC as described above; and (2) ground motions unscaled. The uncertainties in ground motion and in the remaining structural parameters are treated using a Latin Hypercube sampling plan (O'Connor and Ellingwood, 1987).

Table 1

Parameter		Mean	*COV*	CDF
Bldg B	$F_{y,col}$ (MPa)	259	0.12	Lognormal
	$F_{y,beam}$ (MPa)	283	0.12	Lognormal
	$\hat{\imath}_b$	2.33%	0.62	Histogram from Coats (1989)
Bldg BC	$F_{y,col}$ (MPa)	276	0.12	Lognormal
	$F_{y,beam}$ (MPa)	276	0.12	Lognormal
	$\hat{\imath}_b$	5%	0.62	Histogram from Coats (1989)
$\hat{a}_1$		0.4	0.29	Uniform
β_5		0.95	0.09	Uniform
E(Gpa)		200	0.06	Uniform
G(GPa)		77	0.09	Uniform

In this study, limit states are identified with two simple measures of deformation, 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 inter-story drift angle is defined as,

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

where δ_i is the maximum inter-story drift for story i and h_i is the story. Roof displacement angle (RDF) is similarly defined. Table 2 shows the statistics from these experiments. Note that scaling reduces the coefficient of variation (COV) in response. This reduction in COV is more pronounced if ensembles of ground motion are constructed from the general earthquake catalog for Building BC than those constructed from the simulated ground motion for Building B. This is due to large uncertainty in frequency contents in actual earthquake accelerograms. Using scaled ground motions has been done in some concurrent SAC Joint Venture studies (e.g., Shome and Cornell, 1998).

Table 2

	Building B				Building BC			
	Scaled, Sa=0.17 g		Unscaled		Scaled, Sa=0.09 g		Unscaled	
	RDA	ISDA	RDA	ISDA	RDA	ISDA	RDA	ISDA
μ(%)	0.82	1.78	0.74	1.70	0.35	0.78	0.38	0.90
COV	0.11	0.19	0.21	0.21	0.63	0.53	0.16	0.32

RELIABILITY EVALUATION

Earthquake-resistant structural design must consider building performance during a range of earthquakes, ranging from loss of serviceability or impaired function during or subsequent to moderate earthquakes to life-threatening damage or incipient collapse during or following great earthquakes. Three levels of performance ISDA=1%, 2% and 5% and their corresponding hypothesized limit states are presented. The resistance of a building as a system can be described probabilistically by its fragility, $F_R(x)$, which is defined as the limit state probability, conditioned on spectral acceleration;

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

where LS represents the corresponding limit state and spectral acceleration, S_a, at the fundamental period of the building, is the control variable. The fragility for any limit state is obtained from the cumulative distribution function (CDF) of the ISDA. For example, if the limit state is 2% ISDA, then,

$$P(LS \mid S_a = x) = 1 - P[ISDA < 2\% \mid S_a = x] \tag{3}$$

To determine these conditional probabilities, the ground motion ensembles are scaled so that S_a at the fundamental period of the building increases over the range of interest, the corresponding dynamic responses of the frame to these ensembles are determined, the responses are rank-ordered on lognormal probability plots, and Equation 3 is used to determine the fragilities for increasing levels of ISDA (Song and Ellingwood, 1999b).

This fragility concept pertains to the structural frame as a system rather than to any one beam or column. Figures 4 presents the fragilities for Building B and BC based on ISDA for the three deformation limits identified above. The median (50th percentile) spectral accelerations at progressively more severe limit states (e.g., the spectral acceleration at which "failure/nonfailure" is equally likely) are 0.11g for 1% ISDA, 0.21 g for 2% and 0.62 g for 5%. Notice that Building B and BC have the same median fragility. However, for 2% and 5% ISDA limit states, Building B is more vulnerable than Building BC at spectral accelerations less than median values and less vulnerable than BC at spectral accelerations higher than median values. During a building safety evaluation process, the 5% ISDA (incipient collapse) fragility should be evaluated at 10% level in stead of median level for the sake of conservatism. For Building B, the 10-percentile fragility is 0.36 g. If the review level earthquake in the above example with Building B were to be set at S_a=0.6 g, the N-S frame in Building B would be judged acceptable. However, the 2% ISDA would be reached with almost 97% probability at S_a=0.6 g.

The probability of failure or limit state probability can be estimated by the convolution of the derivative of building fragility and seismic hazard which was modeled by Type II distribution of largest values (Cornell, 1968).

$$P_f = \int_0^\infty H(x)\frac{dF_R(x)}{dx}dx \approx H(m_{Sa})\exp\left[(\sigma_{\ln Sa}k)^2/2\right] \tag{5}$$

The first term is simply the seismic hazard evaluated at the median capacity. The second term is a correction factor that accounts for randomness in capacity. The second equation is an approximation for large ground motion. Assuming k=2.38, u=0.045 in H(x), annual probability of failure are estimated in Table 3 for the two buildings.

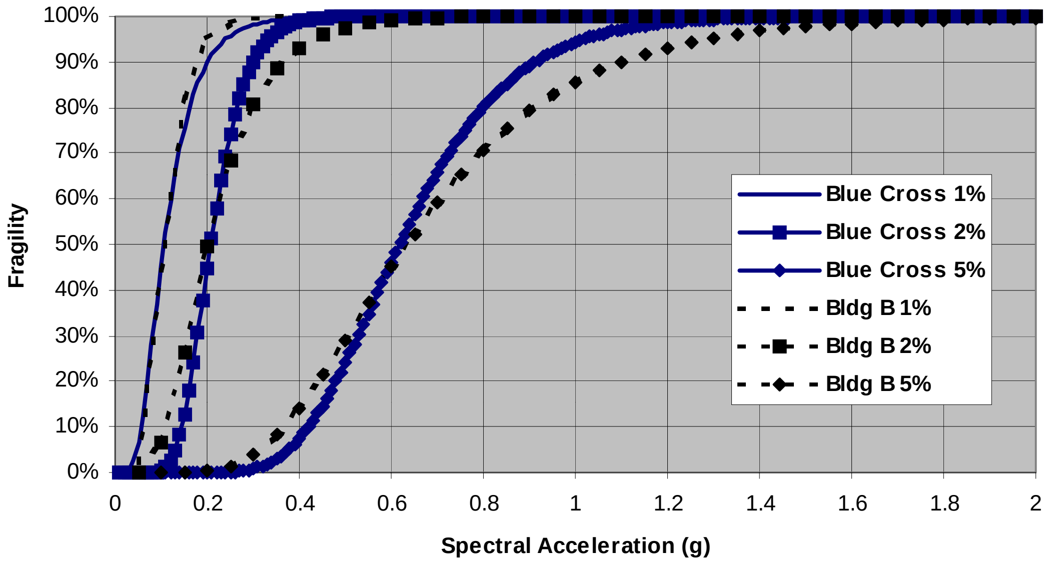

Figure 4: Comparison of Fragility for ISDA

Table 3

	2% ISDA		5% ISDA	
Building	B	BC	B	BC
Pf	0.0515	0.0319	0.00316	0.00251

Using these results, insurance company can determine the approriate premium level for different target building performance. For severe damage level (e.g., 2% ISDA), about 4% premium can be applied to these steel buildings. However, when life safety (e.g., 5% ISDA) is concerned, premium should be approximately 0.3%. This estimation procedure can also be applied to earthquake mitigation and other engineering decision process, such as building design and retrofit.

CONCLUSIONS

Advanced structural finite element model is able to predict welded connection failure with nonlinear time domain analysis with reasonable accuracy. However, using a relatively simple random sampling procedure, a properly-designed stochastic model can offer a broadened perspective on the likely performance of existing buildings during earthquakes. A fragility provides a simple depiction of the likelihood of unseccessful building frame performance and can be used for preliminary condition assessment. It is one of several tools for improving earthquake-resistant building practices. Annual probability of failure for different limit states can be estimated by convolving building fragility with seismic hazard. The reliability estimate would be useful for risk management for the insurance industry.

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