Copyright © 1996 Elsevier Science Ltd
Paper No. 1863. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

STOCHASTIC SEISMIC DAMAGE ESTIMATION IN REINFORCED CONCRETE FRAMES

AJAY SINGHAL

Doctoral Candidate, The John A. Blume Earthquake Engineering Center
Department of Civil Engineering, Stanford University, Stanford, CA 94305-4020

ABSTRACT

This paper presents a systematic approach for estimating damage to reinforced concrete frames subjected to different levels of ground motion. In this paper, damage states describe the physical damage to structures. Fragility curves express the conditional probabilities of reaching or exceeding different damage states at given levels of ground motion. In contrast to previous approaches that used heuristics or empirical data to arrive at the probabilities of different damage states, this paper uses nonlinear dynamic analyses to estimate the fragility curves. The Monte Carlo simulation technique is used to arrive at the probabilities associated with the different damage states. Earthquake time histories are generated by means of the nonstationary autoregressive moving average (ARMA) model. Three different classes of reinforced concrete frames, based on the number of stories, are considered. Park and Ang's damage index is used to identify the different degrees of damage.

KEYWORDS

ARMA; Monte Carlo; damage index; fragility curves; stochastic; seismic damage; ground motion; reinforced concrete; spectral acceleration; simulation.

INTRODUCTION

Information on structural damage is critically important for reliable economic loss evaluation for a structure or a region. Relationships between earthquake ground motion severity and structural damage are most frequently used to characterize the damage distribution over a region. These motion-damage relationships are in the form of probability distributions of damage at specified ground motion intensities and are usually expressed by means of fragility curves or damage probability matrices (DPM). Currently there are only two studies that provide damage probability matrices (ATC-13, 1985), and fragility curves (NIBS, 1995) for a wide variety of structural classes. The damage probability matrices in ATC-13 are based on expert opinion since actual damage data are very limited. The fragility curves in the standardized earthquake loss estimation methodology (NIBS, 1995) are based on test data interpretation and judgment.

The probability of damage is estimated by quantifying the response of a structure subjected to a significant ensemble of ground motion with a wide range of parameter variations. For this purpose, a Monte Carlo simulation approach is used to determine the probabilities of structural damage, and the ensemble of ground motions is generated using an autoregressive moving average (ARMA) model. This method develops fragility curves for three categories of concrete frame structures, including low-rise concrete frames that are 1-3 stories tall, mid-rise frames that are 4-7 stories tall and high-rise frames that are 8 stories or taller. This classification is consistent with that defined in ATC-13 (1985) and is similar to that used in the standardized earthquake loss estimation methodology (NIBS, 1995).

METHODOLOGY

Ground-motion-versus-damage relationships characterize the level of damage to a particular class of structures as a function of a ground motion parameter. In order to represent the variability in earthquake ground motion and uncertainties in structural behavior, these relationships are most frequently described in the form of probabilities of damage conditional on the ground motion parameter. The two most widely used forms of motion-damage relationships are fragility curves and damage probability matrices. A fragility curve describes the probability of reaching a damage state at a specified ground motion level. Thus, a fragility curve for a particular damage state is obtained by computing the conditional probabilities of being in that damage state at various levels of ground motion. A plot of the computed conditional probabilities versus the ground motion parameter describes the fragility curve for that damage state. The conditional probabilities are defined as follows:

$$p_{ik} = P[D = d_i \mid Y = y_k] \tag{1}$$

where

p_{ik} = probability of being in damage state d_i given the ground motion is y_k,
D = damage random variable defined on the damage state vector $\mathbf{D} = \{d_0, d_1, \ldots, d_n\}$,
Y = ground motion random variable.

An alternate representation of fragilities is given by the probabilities of reaching or exceeding a specified damage state given a ground motion level. This definition is used to obtain the fragility curves in this paper. The conditional probabilities can be evaluated from equation 1 as follows:

$$P_{ik} = P[D \ge d_i | Y = y_k] = \sum_{j=i}^{n} p_{jk} \tag{2}$$

While simple fragility formulations have been developed and used extensively for components and mechanical assemblies in nuclear power plant safety analyses, no systematic approach for developing such fragility curves has been presented for complex structural systems. Such an approach is presented in this paper. The major components of this methodology consist of (a) characterization of the structure when subjected to extreme dynamic loads, (b) characterization of the potential ground motions, and (c) quantification of the structural response that includes the variability in the ground motion and the uncertainty in the structural parameters. It is difficult to develop analytical closed form solutions for motion-damage relationships because neither the ground motion nor the nonlinear behavior of the structure can be described in an analytical form. Thus, a Monte Carlo simulation approach is used to estimate the probabilities of damage conditional on different ground motion levels.

Characterization of Ground Motion

In order to characterize earthquake ground motion for the purposes of evaluating structural performance, it is necessary to describe the amplitude, frequency content, and duration of ground motion. Thus, it is difficult to specify a single parameter that captures the above important characteristics of ground motion. Spectral acceleration values are used in this paper to specify different levels of ground motion when developing motion-damage relationships in the form of fragility curves. The spectral values at different periods are frequently represented by means of dynamic amplification factors. The dynamic amplification factors represent the normalized spectral values at specified damping, obtained as a ratio of the spectral acceleration to the peak ground acceleration. Figure 1 shows the dynamic amplification factors obtained at a damping ratio of 5 percent of the critical damping from the firm site records of the Loma Prieta, Whittier Narrows, and Morgan Hill earthquakes. Although the average spectral shapes shown in Figure 1 appear to be smooth in each period band, the individual time histories may have sharp peaks in their spectra. However, it is the average response of the structures over all the time histories that is important Since a structure is likely to be subjected to many different ground motions during its economic life, it is important to consider an ensemble of ground motions that have a wide range of characteristics.

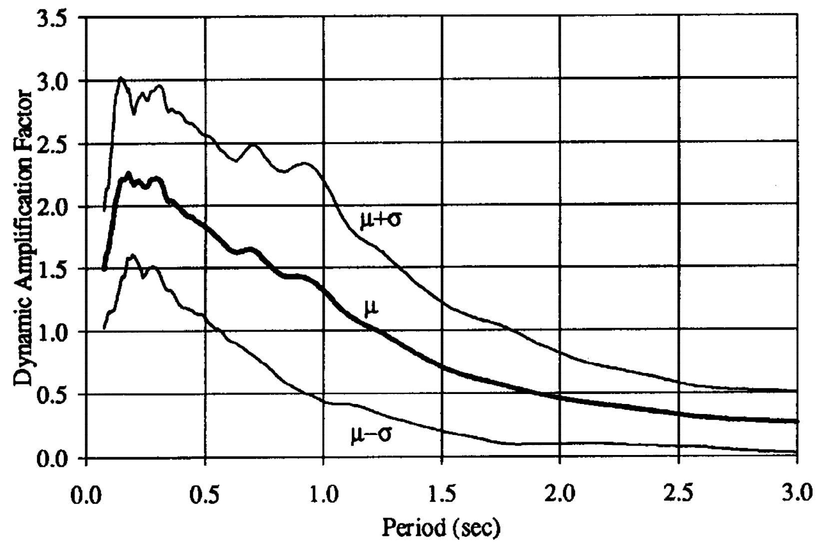

Figure. 1. Dynamic Amplification Factors at 5% Damping for Ground Motions Recorded on Firm Sites

The average spectral acceleration ordinate in the period range corresponding to the three classes of reinforced concrete frames is used to characterize the ground motion for fragility curves. The three period bands used in this paper are 0.1-0.5 seconds, 0.5-0.9 seconds and 0.9-2.5 seconds. These period bands are based on the study reported in FEMA 223 (1992) and are estimated to reflect the natural periods of buildings belonging to the three classes of reinforced concrete frames.

Furthermore, earthquake ground motion time histories are needed for the analysis. Since a consistent ensemble of time histories that covers all the different parameter ranges is not currently available, it is proposed that ensembles of time histories be simulated at each specified ground motion parameter level. Several procedures are available for the generation of artificial time histories. Many researchers including Conte *et al.* (1992) have used ARMA models for the generation of artificial ground motion time histories. In this paper, nonstationary ARMA models are used to generate artificial time histories as the model can account for nonstationarities in both the amplitude and the frequency content.

In this study, the moving time-window technique is used to estimate the ARMA parameters of recorded ground motions from the Loma Prieta, Whittier Narrows and Morgan Hill earthquakes. The moving window assumes that the time history is stationary within a time window. The ARMA parameters estimated for each window are assumed to be representative of the center point of the window. The parameter estimation is repeated for successive equidistant window positions. On the basis of a parametric study, it was found that the ARMA(2,1) model with a window size of 3 seconds gives reasonable results in terms of the spectral acceleration of the simulated time histories. The window is moved by 0.2 seconds between successive window positions. Conte *et al.* (1992) have shown that the ARMA(2,1) processes can be interpreted as the response processes of linear, viscously damped single degree of freedom systems. When generating time histories for a particular level of spectral acceleration, each simulated time history is scaled to match the desired spectral acceleration.

Characterization of Damage

There are several quantitative damage measures that characterize the state of structures after earthquakes. The Park and Ang (1984) model has been widely used in recent years because of its simplicity and because it has been calibrated using data from various structures damaged during past earthquakes. An equivalent form of the Park and Ang index is used in this paper. For a structural component, the index is defined as follows:

$$D = \frac{\theta_m}{\theta_u} + \frac{\beta}{M_y \theta_u} \int dE \tag{3}$$

where

θ_m = maximum positive or negative plastic hinge rotation,
θ_u = plastic hinge rotation capacity under monotonic loading,
β = model parameter (0.15 in this paper),
M_y = calculated yield strength,
dE = incremental dissipated hysteretic energy.

The first term in equation 3 represents the damage due to maximum deformation experienced during cyclic loading. The second term represents the damage due to cumulative hysteretic energy dissipation. The damage index, D, is 0 when there is no damage and is 1.0 for collapse. Cosenza *et al.* (1990) found that the value of $\beta = 0.15$ correlates closely with results based on other damage models. Thus, this value of β is used in this study.

Park and Ang's (1984) global damage index is used in this paper for the identification of the different damage states. It is defined as a weighted average of the local damage indices of each element. The weighting function for each element is proportional to the energy dissipated in the element. The global damage index is given by the following equation:

$$D_T = \sum \lambda_i D_i \tag{4}$$

where

$$\lambda_i = \frac{E_i}{\sum E_i}$$

E_i = energy dissipated at location i.

In order to estimate economic loss or casualties as a result of structural damage, the structural damage must be expressed in terms of discrete damage states. Discrete damage states allow the damage sustained by a structure to be expressed in terms of the nature and extent of the damage suffered by its components. Thus, structural damage which is a continuous function of building response, is quantified by discrete damage states. The five discrete damage states used in this paper are: *none, minor, moderate, severe,* and *collapse.* To obtain these discrete damage states, ranges for the damage measures mentioned above need to be specified.

Park *et al.* (1984) calibrated the damage index with the observed damage to nine reinforced concrete buildings caused by different earthquakes. Gunturi (1992) further investigated these damage states and simplified them according to his findings. Further calibration of the Park-Ang damage index was performed by De Leon and Ang (1993) using data from eight reinforced concrete buildings that sustained different levels of damage during the 1985 Mexico City earthquake. The ranges of the Park and Ang index for different damage states have been established to reflect damage to concrete frames more realistically and are presented in Table 1.

Table 1. Ranges of Park and Ang's Damage Index for Different Damage States

Damage state	Range of the Park and Ang index
Minor	0.1 - 0.2
Moderate	0.2 - 0.5
Severe	0.5 - 1.0
Collapse	> 1.0

Fragility Simulation

In damage analysis, the uncertainties associated with structural capacities and demands need to be modeled. Structural capacities are defined in terms of the capacities of members as part of the structure. Much greater uncertainty is associated with seismic demands than with other demands on the structure. Artificial ground motion simulation using ARMA models is carried out to incorporate this uncertainty. The fragility formulations defined in equations 1 and 2 are determined by the Monte Carlo simulation method. The Monte Carlo simulation technique involves the selection of values of the input random variables required for non-linear dynamic analyses. Examples of input random variables to model capacities for reinforced concrete structures include the strengths of steel and concrete. Latin hypercube sampling provides a good method for selecting the values of the input random variables. In this paper, one hundred Latin hypercube samples are used for the non-linear dynamic analysis at each ground motion level. From the simulations, the means, variances, and distribution functions of the quantitative measure of damage are estimated for an ensemble of time histories corresponding to a given level of ground motion. The probabilities of different damage states are evaluated from the probability distributions of the damage measure.

FRAGILITY CURVES FROM SIMULATION RESULTS

Fragility curves are developed for sample buildings in the three classes of reinforced concrete frames. A typical structure, for the purposes of this paper, is considered to have five bays in the longitudinal direction and one bay in the transverse direction. The sample building for each class of concrete frames was designed according to the 1990 SEAOC Recommendations for special moment resisting frames. The thickness of the floor slab in these buildings is assumed to be 17.78 cm (7 in.). A uniformly distributed dead load of 1.44 kPa (30 psf) is superimposed on the self weight of the structure and used in the design of the members. In addition, reduced live loads for member design were represented by a uniformly distributed load of 1.44 kPa (30 psf). A typical interior frame for each of the three structures was used in the nonlinear time history analysis to estimate damage at different levels of ground shaking. Figure 2 shows the elevations for the three frames used in the analysis.

The uncertainty associated with structural capacities are modeled by taking the compressive strength of concrete and the yield strength of steel as random variables. Following Galambos *et al.* (1982), a normal probability distribution for concrete strength and a lognormal probability distribution for steel strength are used. Concrete strength has a mean of 1.14 times the nominal concrete strength and a coefficient of variation of 0.14. Steel strength has a mean of 1.05 times the nominal strength and a coefficient of variation of 0.11. The uncertainties associated with dead and live loads are considerably smaller compared to the uncertainty in seismic load. In this paper, only the earthquake load is modeled as a nonstationary stochastic process. ARMA models are used for simulating artificial time histories.

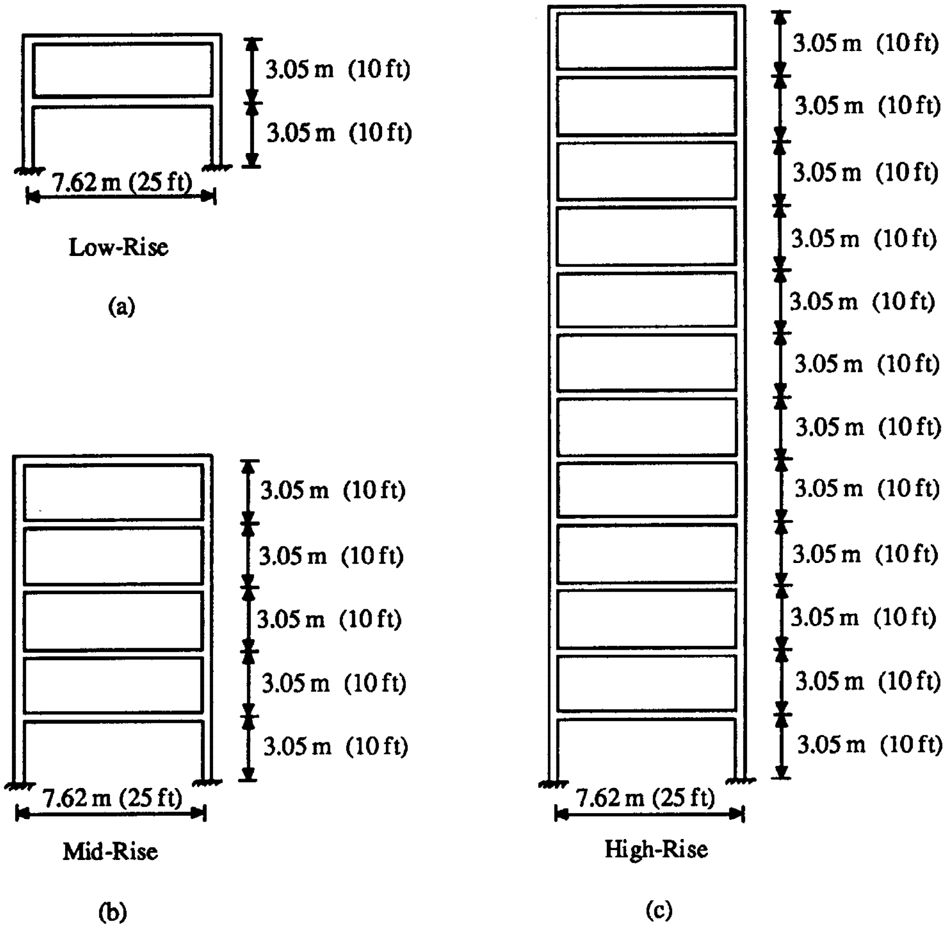

Figure 2. Elevations of the Sample Frames for the Three Classes. (a) Low-Rise; (b) Mid-Rise; (c) High-Rise

Sample Fragility Curves

The computer programs IDARC2D (Kunnath and Reinhorn, 1994) and DRAIN-2DX (Prakash and Powell, 1992) are used for damage analysis. The member properties in terms of moment-rotation relationships are evaluated in IDARC2D. These properties are then used for the nonlinear dynamic analyses performed in DRAIN-2DX. The spread of plasticity along the length of each member is captured by a discretization of the member into smaller elements. On the basis of a parametric study, one small element of length equal to 15 percent of the member length at each end, along with a larger middle element are able to incorporate the spread of plasticity along the member length satisfactorily for most members in a structure. However, more elements are needed for the bottom story columns. It is found that two small elements of length equal to 10 percent of the column length at each end along with a larger middle element are satisfactory in capturing the spread of plasticity.

Nonlinear dynamic analysis is performed for 100 artificial ground motions generated at each value of spectral acceleration. A time step of 0.002 seconds is used in the analysis. The damping matrix was obtained as a linear combination of the mass and stiffness matrices. The coefficients for the mass and stiffness matrices were selected to give 5 percent of critical damping in the first two vibrational modes. The Park-Ang damage index given by equations 3 and 4 is computed from the results of the time history analysis performed in DRAIN-2DX. The statistics of the Park and Ang damage index, obtained at each spectral acceleration value, are used to obtain the parameters of a lognormal probability distribution function at that ground motion level. These lognormal probability functions are then used to obtain the probabilities of the different damage states by computing the probabilities of the damage index being in the ranges given in Table 1. Smooth fragility curves are obtained by arbitrarily fitting lognormal distribution functions to the simulation results. Figures 3 through 5 show the simulation results and the fitted curves as discrete points and smooth curves respectively.

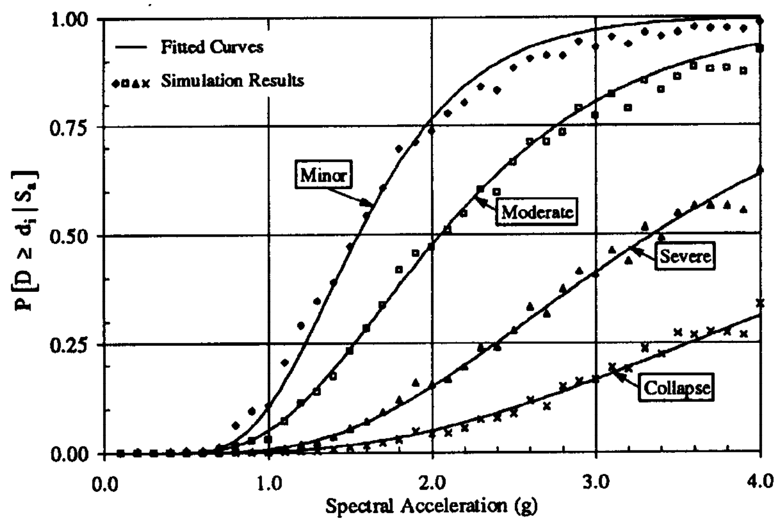

Figure 3. Fragility Curves for the Sample Low-Rise Building

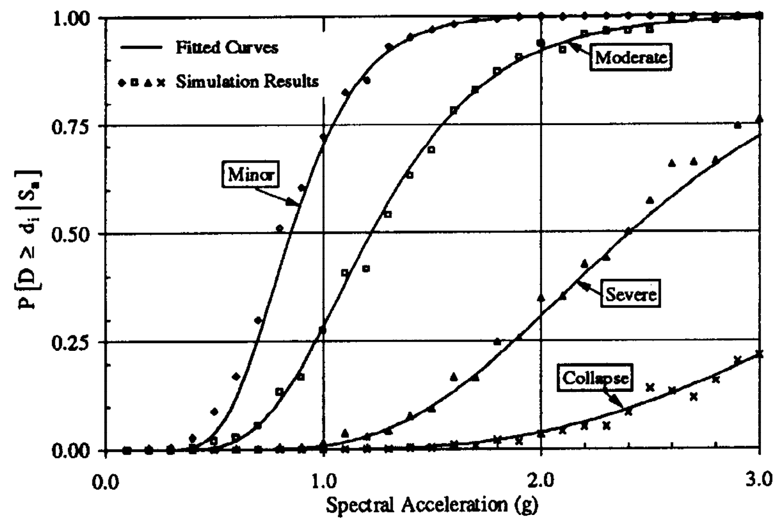

Figure 4. Fragility Curves for the Sample Mid-Rise Building

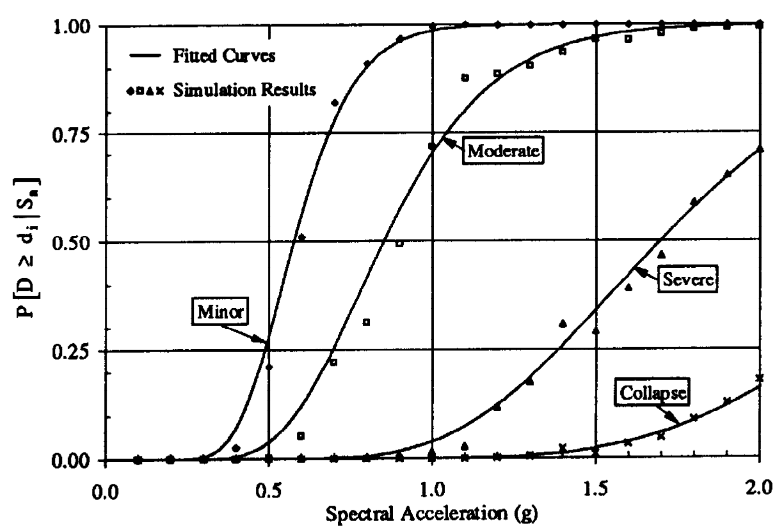

Figure 5. Fragility Curves for the Sample High-Rise Building

CONCLUSIONS

This paper presents a systematic approach for developing damage-motion relationships for reinforced concrete frames. These fragility curves consider the nonlinear properties of the structure and the nonstationary characteristics of the ground motions. Thus, they represent the most consistent set of fragility curves currently available and can be used for estimating damage states for a wide range of reinforced concrete frames. Furthermore, the damage estimates can be used for cost-benefit analysis purposes in retrofit decisions and in the evaluation of potential losses to concrete frames over a region.

ACKNOWLEDGMENTS

The author is deeply indebted to his principal adviser, Professor Anne S. Kiremidjian, for her constant advice, encouragement and trust during the course of this research. This research would not have been possible without her support. This research is currently supported by NCEER Contract 92-4101. Their continued funding is appreciated. The author expresses his gratitude to Dr. Andrei Reinhorn for providing the program IDARC2D, and to Dr. S.K. Kunnath for responding to various queries on IDARC2D. The author would also like to thank Dr. J.P. Conte for providing input on ARMA modeling of ground motions.

REFERENCES

ATC-13 (1985). "Earthquake damage evaluation data for California", *Applied Technology Council*, Redwood City, CA.

Conte, J.P., Pister, K.S., and Mahin, S.A. (1992). "Nonstationary ARMA modeling of seismic motions", *Soil Dynamics and Earthquake Engineering*, Vol. 11, pp. 411-426.

Cosenza, E., Manfredi, G., and Ramasco, K. (1990). "An evaluation of the use of damage functionals in earthquake-resistant design", *Proceedings of the 9th European Conference on Earthquake Engineering*, Moscow, Vol. 9, pp. 303-312.

De Leon, D., and Ang, A.H-S. (1993). "A damage model for reinforced concrete buildings: Further study with the 1985 Mexico City earthquake", *Proceedings 6th International Conference on Structural Safety and Reliability*, Innsbruck, Austria, Vol. 3, pp. 2081-2087.

FEMA 223 (1992). "NEHRP recommended provisions for the development of seismic regulations for new buildings: Part 2 - Commentary", *Earthquake Hazards Reduction Series 65*, Federal Emergency Management Agency, Washington, D.C.

Galambos, T.V., Ellingwood, B., MacGregor, J.G., and Cornell, C.A. (1982). "Probability based load criteria: Assessment of current design practice", *Journal of the Structural Division*, ASCE, Vol. 108, No. ST5, pp. 959-977.

Gunturi, S.K. (1992). "Building specific earthquake damage estimation", *Ph.D. Thesis* submitted to Stanford University.

Kiremidjian, A.S. (1985). "Subjective probabilities for earthquake damage and loss", *Structural Safety*, Vol. 2, pp. 309-317.

Kunnath, S.K., and Reinhorn, A.M. (1994). "IDARC2D : Inelastic damage analysis of RC buildings", Department of Civil Engineering, State University of New York at Buffalo.

NIBS (1995). "Development of a standardized earthquake loss-estimation methodology", Draft Technical Manual 100% Submittal, Prepared by Risk Management Solutions, Inc., for National Institute of Building Sciences.

Park, Y-J., Ang, A.H-S. and Wen, Y.K. (1984). "Seismic damage analysis and damage-limiting design of R.C. buildings", *Structural Research Series, Report No. UILU-ENG-84-2007*, University of Illinois at Urbana-Champaign, Urbana, Illinois.

Prakash, V., and Powell, G.H., (1992). "DRAIN-2DX: user's guide", *Department of Civil Engineering*, University of California at Berkeley.

SEAOC (1990). "Recommended lateral force requirements and commentary", *Structural Engineers Association of California*.