RELIABILITY OF DAMAGED REINFORCED CONCRETE FRAMES

C. Meyer (I)
M. S. L. Roufaiel (II)
Presenting Author: C. Meyer

SUMMARY

An analytical method is described by which the reliability of concrete frames can be determined, which may or may not have been damaged by earlier earthquakes. Ground motions are simulated as a non-stationary random process. A mathematical model is described which permits to simulate the response of reinforced concrete frame elements to large cyclic loads. Using Monte Carlo simulation, damage probability matrices are constructed which take into account the variability of building strength parameters in addition to the randomness of ground motions. The methodology is illustrated with the study of a three-story, two-bay concrete frame building.

INTRODUCTION

The reliability of concrete buildings in seismic zones is of considerable concern. This is particularly true for those buildings which have been damaged by past earthquakes or which simply have deteriorated over time with age. This paper describes an analytical method by which the reliability of damaged concrete frames can be determined. A non-stationary random process is used to generate sample ground motion histories. A mathematical model is described which permits to simulate the response of R/C frame elements to large cyclic loads. Parameters characteristic of building damage and residual strength and stiffness are introduced such that it is possible to perform complete response analyses of damaged concrete buildings. Using Monte Carlo simulation, damage probability matrices are constructed which take into account the variability of building strength parameters and the randomness of ground motions. The methodology is illustrated with the study of a three-story, two-bay R/C frame building.

GROUND MOTION MODEL

The loads resulting from earthquake ground motion are highly random and strongly dependent on the geology and seismology of the building site. In the present study, a ground acceleration $Z(t)$ is used, which consists of a stationary Gaussian process $X(t)$ with zero mean and spectral density function $S_g(\omega)$, multiplied by a deterministic function $\phi(t)$:

$$Z(t) = X(t)\phi(t) \qquad (1)$$

where

$$\phi(t) = r(e^{-\alpha t} - e^{-\beta t}) \qquad t>0 \qquad (2)$$

with r, α and β all positive (Ref. 1), Fig. 1. The spectral density function of $X(t)$ is assumed to be

$$S_g(\omega) = S_I/\{(\omega^2 - \omega_g^2)^2 + 4\xi_g^2\omega^2\omega_g^2)\} \qquad (3)$$

(I) Associate Professor of Civil Engineering, Columbia University, NY, NY
(II) Post-Doctoral Research Associate, Dept. of Civil Engr., Columbia U., NY

where ω_g is a characteristic ground frequency, ξ_g a positive constant, and S_I a constant that, together with $\phi(t)$, controls the acceleration intensity. When a sample function $x(t)$ of $X(t)$ as shown in Fig. 1b is multiplied by the envelope function, Fig. 1a, a sample function $z(t)$ of $Z(t)$ is obtained which looks very similar to that of actual acceleration records, Fig. 1c.

R/C FRAME MEMBER MODEL

The mathematical model of a typical reinforced concrete frame member is different from most others that have been proposed in the past, in that it takes into account the finite size of the plastic region, Fig. 2. The moment curvature relationship of a section is given by a modified Takeda model, Fig. 3. Details of the model can be found in Ref. 2. Some specific features are:

1. The stress-strain curve of plain concrete for a uniaxial state of stress considers the effect of confinement reinforcement on both ultimate strength and strain of failure.
2. The effect of shear on the moment-curvature relationship is included by explicitly introducing an appropriate degree of pinching of the hysteresis loops, Fig. 3.
3. According to test data, the stiffness associated with loading or reloading is a function of maximum curvature attained in any previous loading cycle. Thus, the stiffness degradation associated with large excursions into the inelastic range is accurately reproduced.
4. Experimental evidence suggests that there exists a characteristic curvature ϕ_m beyond which a section experiences not only stiffness deterioration but also strength degradation. This critical curvature has been well correlated with a failure strain ε_m in the extreme compression fibre of the section. With this model enhancement, it is possible to reproduce experimental load-deflection curves up to failure.
5. Axial loads influence a section's moment-curvature relationship by reducing its ductility and energy absorption capacity. The model includes this effect, even though for axial loads exceeding half of the balanced load, the accuracy decreases. For higher axial loads, the ductility of a column is reduced sufficiently so that the inelastic behavior has a minimal impact on the response of a structure to strong ground motion.

Using the refinements listed above it has been possible to reproduce with this member model numerous experimental tests that have been reported in the literature (Ref. 2). Figs. 4 and 5 are typical of the level of agreement obtained. Fig. 4 reproduces the experimental and analytical load-deflection curves of a cantilever beam tested by Ma, Bertero and Popov (Ref. 4), which exhibits rather stable hysteresis loops up to approximately 2 inch tip deflection, at which flexural failure occurs. Fig. 5 illustrates a beam exhibiting strong influence of shear (Ref. 4).

DAMAGE PARAMETERS

In order to analyze damaged reinforced concrete buildings it is necessary to introduce precise definitions of damage. This can be done on the local (element) level and on a global (structure) level. As a measure of local damage, a modified flexural damage ratio is introduced as follows:

$$\mathrm{MFDR} = \frac{M_m/\phi_m}{M_x/\phi_x} \tag{4}$$

i.e. the ratio between the secant stiffness at the onset of failure, M_m/ϕ_m, and the minimum secant stiffness M_x/ϕ_x, Fig. 6. For moments below the yield capacity, deformations remain elastic and there is no damage. In this case, MFDR = 0. The value of MFDR = 1 means that the member has been loaded to the point where the concrete starts to fail, thus indicating initiation of the strength degradation process which may lead to eventual total failure. For practical purposes, this onset of failure is tantamount to failure so that MFDR values between 0 and 1 may serve as measures of element damage.

As a measure of global damage, the following definition is introduced,

$$\mathrm{GDP} = \frac{\delta_R - \delta_Y}{\delta_F - \delta_Y} \geq 0 \tag{5}$$

where δ_R is the maximum roof displacement experienced by the building; δ_Y is the roof displacement amplitude at which the first member in the structure exceeds the yield moment, assuming the building deforms in the fundamental mode shape; δ_F is the roof displacement amplitude at which the building is assumed to fail, again assuming it deforms in the first mode. A value for δ_Y can be determined by statically loading the frame such that it deforms in the first mode shape, until the first member enters the plastic range. To select a corresponding value for δ_F is more difficult. An evaluation of numerous test data (Ref. 2) found that a failure displacement correlates reasonably well with a building failure drift of 0.06. Thus, pending the establishment of a better definition of failure displacement, it will be assummed that

$$\delta_F = .06\ H \tag{6}$$

where H is the building height.

ANALYSIS OF DAMAGED BUILDINGS

The analysis of a damaged R/C building is a difficult undertaking and subject to numerous uncertainties in addition to those associated with undamaged frame analysis. A complete deterministic analysis requires knowledge of past load histories of each member which has undergone inelastic deformations. In a realistic situation none of this information is available and the following approximate analysis is suggested to estimate the damage parameters of all frame members.

1. It is assumed that the time delay effect between earthquakes can be ignored, i.e., the response of a building to n earthquakes is assumed to be the same as if all n earthquakes are concatinated to form a single earthquake of a duration equal to the sum of all individual earthquake durations. In this case, the damaged frame analysis can be viewed as a restart of an analysis interrupted after the first n-1 earthquakes.
2. If no better estimate is available for the maximum past roof displacement, δ_R, which the frame may have undergone during any one of the first n-1 earthquakes, an approximate value can be obtained by

$$\bar{\delta}_R = 1 + 14.2(\bar{\omega} - 1) \qquad \omega \leq \omega_e \tag{7}$$

where $\bar{\delta}_R = \delta_R/\delta_Y$, $\bar{\omega} = \sqrt{\omega_e/\omega}$, ω_e = fundamental frequency of undamaged frame, ω = fundamental frequency after exposure to n-1 earthquakes. Eq. (7) reflects the strong correlation found to exist between maximum roof displacement and fundamental frequency (Ref. 3). If ω is not known, it can be determined in a relatively inexpensive field test.

3. The frame model is statically loaded such that it deforms in the first mode shape, until the roof displacement reaches the value δ_Y, i.e., the first member reaches the yield moment.
4. The static loading is continued into the inelastic range, still maintaining the first mode shape, until the maximum roof displacement of step 2 is reached. From the final displaced structure, all member data can be computed, such as MFDR-values, which are needed for the restart analysis.
5. With all initial conditions established in step 4, a nonlinear dynamic analysis can be performed for the n'th earthquake, and a new maximum roof displacement, $\delta_R^{(n)} \geq \delta_R^{(n-1)}$, computed, as well as a new global damage parameter, according to Eq. (5).

DAMAGE PROBABILITY MATRICES

The building response is a random process, subject to the statistical uncertainties of both the ground motion input and the building strength parameters. In order to avoid the numerous simplifications needed for analytical random vibration techniques, which are necessary for highly nonlinear systems, numerical simulation is used herein to predict the expected building response. The results are most readily presented in the form of damage probability matrices (Ref. 5), in which the global damage parameter is discretized for demonstration purposes into four states of increasing damage, i.e., 0.0-0.25, 0.26-0.50, 0.51-0.75, and 0.76-1.00. A typical element of this damage matrix, DPM_{ij}, indicates the probability that an earthquake of given Mercalli intensity will cause damage state i in the building, given that prior to the earthquake the damage state was j. Obviously, DPM is a lower triangular matrix.

EXAMPLE FRAME

Fig. 7 shows a concrete frame for a typical office building, designed to satisfy the Uniform Building Code (Ref. 6) and the ACI Code (Ref. 7) for seismic zone III. This frame has been analyzed using a modified version of program DRAIN-2D (Ref. 8), on Columbia University's IBM 4341 computer.

When subjected to ground motions of intensities up to MMI = 8, the frame was found to respond either entirely elastically, or the damage parameter did not exceed the lowest category, GDP = 0.0-0.25. Likewise, for any given damage state, ground motions associated with intensities MMI $\leq$ 8 did not increase the initial damage state. As a result, the damage probability matrices for MMI $\leq$ 8 were found to be diagonal identity matrices. The ensemble average of the peak accelerations for MMI = 8 was o.22 g.

Table 1 summarizes the damage probability matrices for earthquake intensities 9, 9.5, and 10. The corresponding ensemble averages of the peak accelerations are, respectively, .46 g, .67 g, and .98 g. The ground motions generated for MMI = 10 are obviously not very realistic. When discounting this case, the remaining results for MMI = 9 and 9.5 lead to the conclusion that even earthquakes of high intensity are not likely to increase significantly the damage sustained during earlier earthquakes. The main exception is the case of the frame with initial damage 0.125, for which the probability that an earthquake of intensity 9.5 causes damage between 0.26 and 0.5 is 46%.

Table 1. Damage Probability Matrices for Nominal Member Strengths

a) MMI = 9

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.98	0	0	0
.26-.50	.02	1	0	0
.51-.75	0	0	1	0
.76-1.0	0	0	0	1

b) MMI = 9.5

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.46	0	0	0
.26-.50	.46	.88	0	0
.51-.75	.06	.12	.92	0
.76-1.0	.02	0	.08	1

c) MMI = 10

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.02	0	0	0
.26-.50	.18	.16	0	0
.51-.75	.42	.36	.52	0
.76-1.0	.38	.48	.48	1

The results of Table 1 were obtained for the frame with member and material properties as summarized in Fig. 7. Actual member properties such as M_y and ϕ_m are subjected to a certain amount of scatter. Specific data on realistic statistical distributions are rare. Some information is given in Ref. 9, according to which the coefficient of variation of beam flexural strengths is approximately 0.1. No such information is available for the ultimate curvature capacity ϕ_m. To investigate the sensitivity of the above results to variations of member properties, the frame has been reanalyzed for MMI = 9.5, once with M_y and ϕ_m for each frame member decreased by a standard deviation, and again for a frame in which M_y and ϕ_m values were increased by a standard deviation for each member. The results are summarized in Table 2. When comparing these results with those of Table 1b, a consistent trend is observed. Weakening the members and reducing their curvature ductilities tends to lead to slightly increased damage states, whereas the opposite is true for the strengthened frame with increased failure curvatures for all members.

It is debatable whether a uniform increase or decrease of member strengths throughout the frame is a realistic assessment of the statistical variations to be expected in the field. An alternative approach is to assign M_y and ϕ_m values to individual members on a random basis. By assuming a normal distribution for both M_y and ϕ_m, mean values equal to the nominal values based on the properties of Fig. 7, and a coefficient of variation of of 0.1, a two-dimensional numerical simulation was undertaken in which strength parameters were assigned to each frame member at random for each sample ground motion history. The results are contained in Table 3 and re-

Table 2. Damage Probability Matrices for Uniformly Modified Member M_y and ϕ_m (MMI = 9.5)

a) Mean values minus std. dev.

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.36	0	0	0
.26-.50	.48	.76	0	0
.51-.75	.10	.16	.76	0
.76-1.0	.06	.08	.24	1

b) Mean values plus std. dev.

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.52	0	0	0
.26-.50	.42	.94	0	0
.51-.75	.04	.02	1	0
.76-1.0	.02	.04	0	1

Table 3. Damage Probability Matrix for Randomly Distributed M_y and ϕ_m

Expected Damage	Initial Damage .125	.375	.625	.875
0-.25	.54	0	0	0
.26-.50	.36	.78	0	0
.51-.75	.08	.08	.96	0
.76-1.0	.02	.14	.04	1

inforce, though to a lesser degree, the previously established pattern to the effect that even with relatively large strength variations within the frame, the previously incurred damage is not very likely to be increased appreciably for MMI = 9.5, unless the initial damage was very small to begin with.

CONCLUSIONS

An ordinary concrete building has been studies for its reliability in withstanding earthquake ground motions, assuming it has been weakened by earlier earthquakes to a specified degree. The stochastic nature of the ground motion has been taken into account, as well as possible variations of member strength and ductility parameters. All results are fairly consistent in predicting that it is unusual for even strong ground motions to appreciably increase previously sustained damage. A possible explanation is that local damage at plastic hinge locations weakens the frame's most highly stressed members and limits the resistance to further seismic loads. It may also shift the building's fundamental frequency into less critical ranges of the load spectrum. It should be noted that the P-Δ effect, which can cause collapse of such weakened frames has been included in the analysis.

The above conclusions should be accepted with caution because of a number of limitations of the study.

1. Because of the high analysis cost (36 sec. computer time for a typical time history analysis) the number of samples for each different case was limited to 50. This number seems to be adequate for most cases listed in Tables 1 and 2, but for the two-dimensional simulation of Table 3, more samples are needed to permit valid conclusions.
2. The definition of failure displacement needs refinement. This is particularly important if more specific predictions regarding the building's survivability rather than lower damage states are to be made.
3. Damage and collapse of real buildings are often due to other failure modes, such as shear or bond failures. Also, gross errors due to workmanship are beyond the scope of this study.
4. The conclusions are also of limited applicability because they were derived from the study of only a single building.
5. To base the conclusions on wider footing, ground motions having different power spectra should be utilized as well, e.g., by replacing the spectrum of Eq. 3 by a Kanai-Tajimi-type spectrum.

ACKNOWLEDGMENT

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REFERENCES

1. Shinozuka, M. and R.Y. Tan, "Seismic Reliability of Damaged Concrete Beams," ASCE, J. of Str. Engr., July, 1983.
2. Roufaiel, M.S.L. and C. Meyer, "Analysis of Damaged Concrete Frame Buil-

dings," Dept. of Civil Engr., Columbia Univ., May, 1983.
3. Meyer, C., M.S.L. Roufaiel, and S.G. Arzoumanidis, "Analysis of Damaged Concrete Frames for Cyclic Loads," Earthq. Engr. & Str. Dyn, 11, 207-228 (1983).
4. Ma, S.M., V.V. Bertero and E.P. Popov, "Experimental and Analytical Studies on the Hysteretic Behavior of R/C Rectangular and T-Beams," Earthq. Engr. Res. Ct.Rep. No. EERC 76-2, Univ. of Calif., Berkeley, May, 1976.
5. Whitman, R.V. et al, "Seismic Design Decision Analysis," ASCE, J. Str. Div., May, 1975.
6. Uniform Building Code, Int. Conf. of Building Officials, 1973.
7. Building Code Requirements for Reinforced Concrete, ACI Standard 318-77, American Concrete Institute, Detroit, Michigan, 1977.
8. Kanaan, A.E. and G.H. Powell, "DRAIN-2D, A Gen. Purpose Computer Program for Dynamic Analysis of Inelastic Plane Structures," Report No. EERC 73-6 & EERC 73-22, Univ. of Calif., Berkeley, April. 1973.
9. MacGregor, J.G., S.A. Mirza and B. Ellingwood, "Statistical Analysis of Resistance of Reinforced and Prestressed Concrete Members," ACI Journal, May/June, 1983.

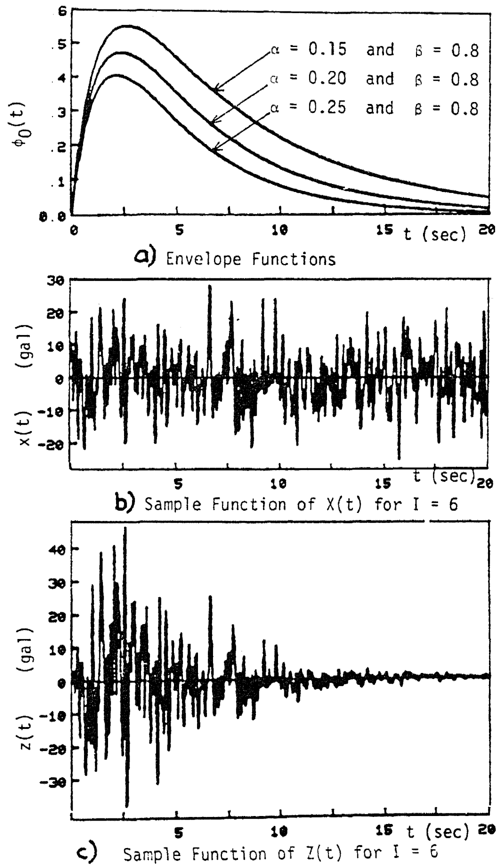


Fig. 1 Ground Motion Model

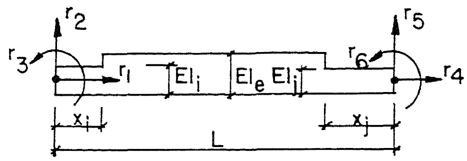


Fig. 2 Frame Element Model

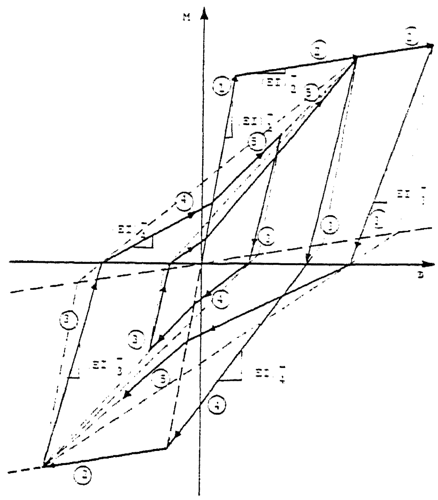


Fig. 3 Moment Curvature Law

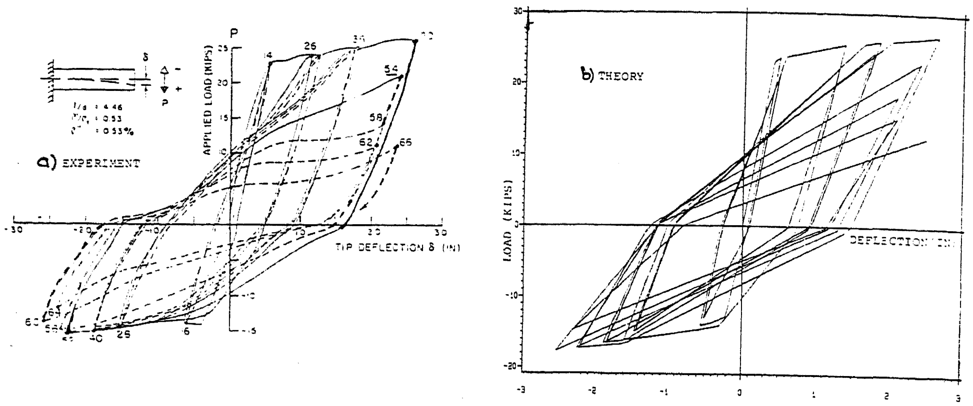


Fig. 4. Flexure Beam - Theory and Experiment (Refs. 2, 4)

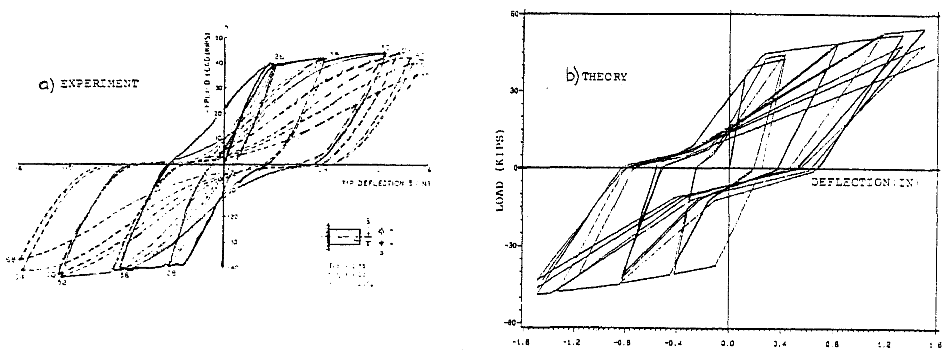
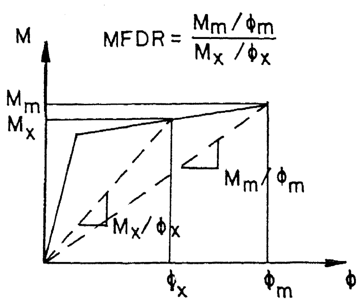


Fig. 5 Shear Beam - Theory and Experiment (Refs. 2, 4)



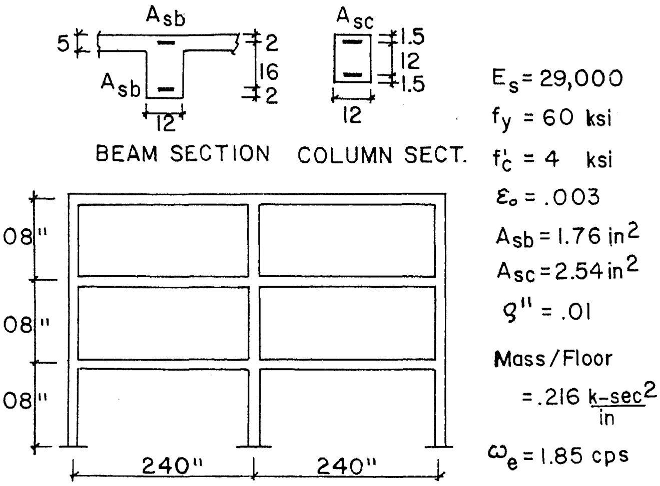


Fig. 6 Modified Flexural Damage Ratio Fig. 7 Example Building Frame