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SEISMIC RELIABILITY OF MULTI-STORY FRAMES, CONSIDERING UNCERTAIN EXCITATIONS AND STRUCTURAL PROPERTIES

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SUMMARY

An approach is developed for the determination of failure probabilities of ductile multi-story frames to earthquake excitation, in terms of the ratio between design forces and expected structural response, and for the use of those results to derive ratios of failure probabilities for given time intervals (for instance, failure rates per year) to rates of occurrence of earthquake intensities higher than the intensity assumed for structural design. Probabilistic models of the responses of non-linear ductile frame structures are derived by means of Monte Carlo simulation, accounting for uncertainties about gravity loads acting on the system and force-deflection curves of its structural members. The variables studied include safety factors, variation coefficients and correlation among the various concepts considered.

INTRODUCTION

Basic criteria and algorithms for selecting seismic design coefficients and spectra on the basis of optimizing present values of expected utilities, including uncertainties about both structural properties and seismic excitations, have been available for a long time. The algorithms developed in detail cover cases where the occurrence of earthquakes of different intensities at a site is modeled either by a Poisson process or by a renewal process. Those algorithms make use of probabilistic descriptions of the seismic-activity process, as well as of concepts such as the probability distribution of the ground motion intensity at which a structure of interest fails, the probability of failure for a given intensity or, more generally, the probability distribution of the cost of damage for that intensity.

Using the argument that uncertainties tied to structural parameters, response and performance, are very small as compared to those attached to the nature and parameters of the seismic processes, it is usually concluded that an uncertain structural strength can be replaced with its expected value when performing studies about the reliability of a structure in a seismic environment; however, the problems still remain of determining the ratio of the expected value of the earthquake intensity resisted by a structure to the nominal value used to express safety-related specifications, and of obtaining $E(\nu_F)$, the expected rate of failure per unit time of a structure with uncertain mechanical properties, in terms of $\nu_Y(y^*)$, rate of occurrence of intensities greater than y^*, the nominal value of the design intensity.

The studies reported in this article aim at assessing the influence of a number of structural parameters on computed failure probabilities of systems designed with the same safety factors for the same nominal intensities. For this purpose, it is assumed that building frames fail in a ductile manner by formation of plastic hinges at those member sections where the acting bending moment reaches the local bending capacity, and that a brittle failure limit state is reached when the ductility demand at any given story, expressed in terms of lateral deformations of that story, reaches the available capacity of ductile deformation. The analytical difficulties implied by the mathematical models adopted are circumvented by applying Monte Carlo simulation.

PROBLEM FORMULATION

The following approach and assumptions will be adopted:

a) Seismic hazard at the site of interest is expressed in mathematical form by a known function, $\nu_Y(y)$, representing the mean number of times per unit time (year), that an intensity greater than y occurs at the site.

b) Under the action of an earthquake of intensity y the structure may fail in n different modes; R_i will designate the structural capacity to resist the i-th failure mode and S_i will be used to denote the maximum amplitude of the response variable governing the occurrence of the i-th failure mode. The ratio S_i/R_i is the reciprocal of a random safety factor and will be denoted with Q_i. Failure in the i-th mode occurs if $Q_i \geq 1$. It is also assumed that failure occurs precisely in the i-th mode and not in any other if $Q_i \geq Q_j$ for all j=1, ..., n.

From the assumptions above, the probability of failure for a given intensity equals the probability that the maximum of all the Q_i's exceeds unity. Thus, if that maximum is called Q, then the probability of structural failure under the action of an earthquake with intensity y is equal to

$$P_F(y) = P(Q \geq 1|y) \tag{1}$$

c) The rate of failure of a structure with deterministically known properties (vector $\underline{R}$) is

$$\nu_F(\underline{R}) = \int_0^\infty - \frac{\partial \nu_Y(u)}{\partial u} \; P_F(u|\underline{R})\,du \tag{2}$$

where $\nu_F(u)$ is the rate of occurrence of an intensity in excess of u, and $P_F(u)$ is given by Eq. 1. If $\underline{R}$ is a vector of uncertain structural properties, then the expected value of ν_F can be obtained by weighing the value given by Eq. 2 with respect to the joint p.d.f. of $\underline{R}$. After this is done and the order of integration is changed, the following is obtained (1):

$$E(\nu_F) = \int_0^\infty - \frac{\partial \nu_Y(u)}{\partial u} \int_0^\infty P_F(u|r) f_{\underline{R}}(r)\,dr\;du \tag{3}$$

The interior integral in Eq. 3 is the failure probability of a system with uncertain properties subjected to an earthquake with intensity Y = u.

BASIC ASSUMPTIONS

Seismic hazard function The intensity y can be expressed in terms of parameters such as peak ground accelerations or velocities, ordinates of response spectra for given period and damping, and expected values of those ordinates. Then the expected rate of occurrence of earthquakes with intensities higher than a given value, y, can be expressed by a function of the form

$$\nu(y) = Ky^{-r}(1 - (y/y_M)^{\varepsilon}), \quad \text{for } y \le y_M \tag{4}$$

$$= 0 \ , \quad \text{for } y \ge y_M$$

where y_M is an upper bound to the intensities that may occur at the site of interest, r and ε are parameters defining the shape of the distribution of intensities and K is a scaling factor. For the applications that follow, y and y_M are measured by peak ground acceleration at the site during an earthquake, and the parameters in Eq. 4 are assumed to take the values K=129.5, r=1.6, ε=1 and Y_M= 1125cm/s^2 for the analysis of cases 1-13 in Table 1. For case 14, K = 80 and Y_M = 500 cm/s^2.

Ground motion time-histories Two sets of simulated ground motion time-histories were used; one is based on the statistical properties of the NS component of the record obtained in 1940 in El Centro, California, and the other represents the most intense portion of the EW component obtained at the parking lot of the SCT building in Mexico City during the earthquake of 19 September, 1985 (2). Twenty sample records belonging to the first set and nine belonging to the second one were generated by means of the algorithm described in Ref. 3. For the first case, the simulated records have a duration of 30sec, and for the second, of 82sec.

Structures studied The studies reported herein cover three families of single-bay frames with one, three and nine stories, respectively. Their nominal dimensions are shown in Fig. 1, and the nominal values of their properties are given in Table 1, together with the ductility-related reduction factors adopted for design and the corresponding seismic design coefficient.

Failure is assumed to occur when the ductility demand at any given story reaches the available capacity of ductile deformation of that story. This capacity is taken as uncertain, and several assumptions about its variation coefficient were considered, as shown in the fifth column of Table 1.

The parameters and the assumed forms of the statistical distributions of members stiffnesses and strengths are given in Table 2, which also includes values corresponding to live loads. Those parameters are: concrete strength, f_c; steel yield stress, f_y; reinforcement cover in girders and columns, r; width and depth, b and h; and live load, W_L.

The expected capacity of ductile deformation, $\bar{\mu}$, at a given story is related with its nominal value, μ^* through the equation $\bar{\mu} = \mu^* \exp(.55 \mathrm{x} 3 \mathrm{x} V_\mu)$, where V_μ is the coefficient of variation of the available ductility. Symbols HC and LC in the sixth column of Table 1 mean "high correlation" and "low correlation". In the first case, each material property or cross-section dimension is assumed to be perfectly correlated throughout the structure, but the different variables at a given member are stochastically independent. In the second case, each material property or cross-section dimension at a given member-end is correlated with its counterpart at any other member-end in accordance with the correlation coefficients of Table 3. All systems studied were assumed to possess a viscous damping of 5 percent of critical.

TYPICAL RESULTS

The values of Q resulting from Monte Carlo simulation for some of the cases studied are depicted in Figs. 2, 3, 5, 7 and 8, together with the expressions fitted for $E(Q|y)$ and the values of σ_Q. From results such as these, and the assumption that Q has a lognormal probability distribution, failure probability functions in terms of intensities were obtained, similar to those plotted in Figs. 4 and 6. Expected failure rates for the different cases considered were obtained in accordance with Eq. 3. Those failure rates are shown as $E(\nu_F)$ in Table 4, which also indicates values of $\nu(y^*)$, the rates of occurrence of intensities higher than the value assumed for seismic design.

CONCLUSIONS

From the studies reported the following was concluded:

1. The influence of the spatial statistical correlation among the mechanical properties of the structural members on the probability of failure is relatively small as compared to the influence of other variables

2. The number of degrees of freedom has a great influence on the probability failure of structures subjected to earthquakes

3. The structural-failure rate decreases when the design ductility factors increase. This occurs because any continuous frame possesses a lateral load capacity even if it has not been specifically designed for that type of load, and because the higher the capacity of the structure to take ductile deformations, the lower the additional lateral strength required to resist an earthquake of specified intensity.

4. Due to the form of the assumed relation between the expected and the nominal values of the available ductility, as a function of the variation coefficient of that variable, the probabilities of failure for a given intensi ty are great er for the cases for which that variation coefficient is lower.

5. Because most systems considered in this study are assumed to develop significant local yielding at several critical sections before a failure limit state is reached, neither the results reported herein nor the conclusions reached are valid if the safety factors with respect to local brittle failure modes are not sufficiently higher than those associated to ductile modes.

6. Finally, the variability of the failure probabilities obtained for the few cases studied is significant enough as to justify the development of new studies trying to understand it. Future investigations should not only widen the ranges of cases studied, but they should also explore better representations of the mechanical behavior of structural members and systems.

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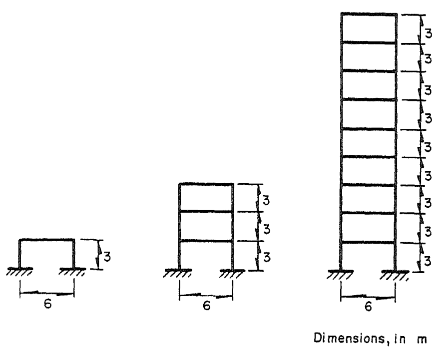

Fig 1 Overall dimensions of cases studied

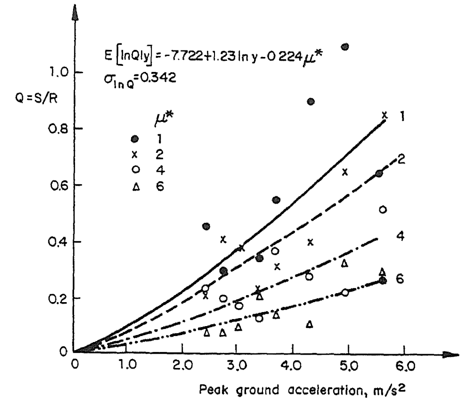

Fig 2 Normalized response of single-story frames; T=0.36s, V_{μ}=0.3

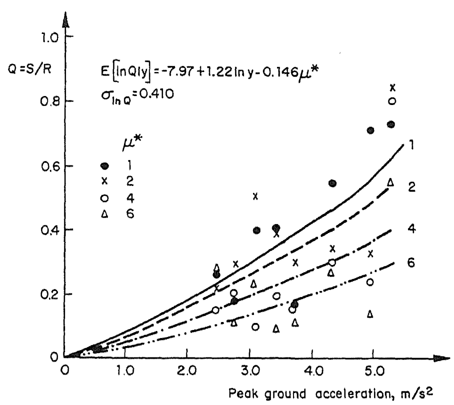

Fig 3 Normalized response of single-story frames; T=0.36s, V_{μ}= 0.5

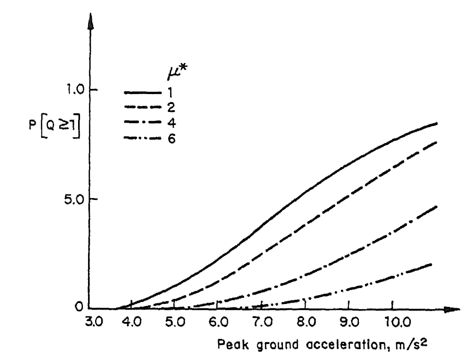

Fig 4 Failure probabilities of single-story frames designed for different ductility factors; V_{μ}=0.5

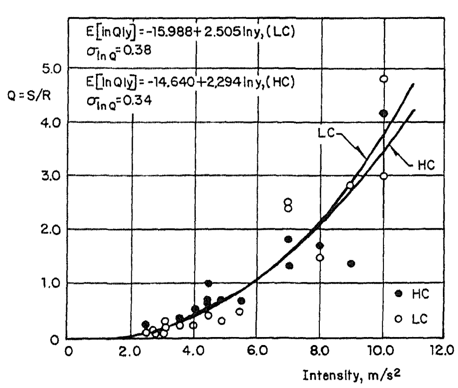

Fig 5 Normalized response of three-story frames; T=0.36s, μ^*=4, V_{μ}=0.3. High (HC) and low (LC) spatial correlation

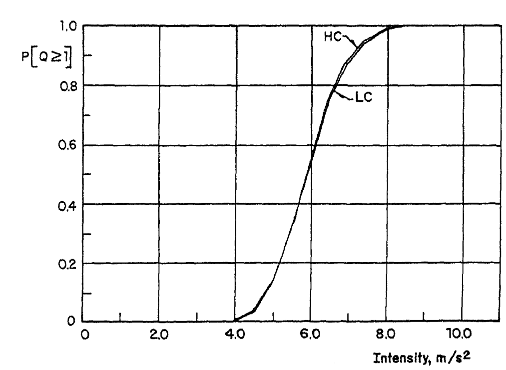

Fig 6 Failure probabilities of three-story frames; T=0.36s, μ^*=4, V_{μ}=0.3. High (HC) and low (LC) spatial correlation

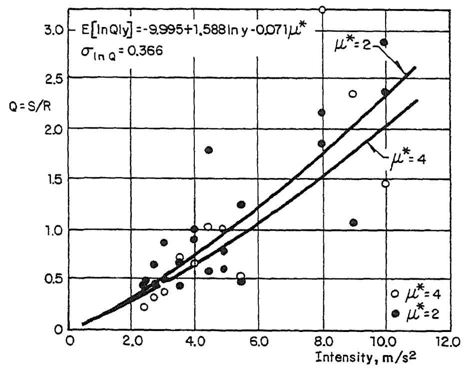

Fig 7 Normalized response of three-story frames; T=0.85 s, μ^* = 2 and 4, V_μ=0.3. Low spatial correlation

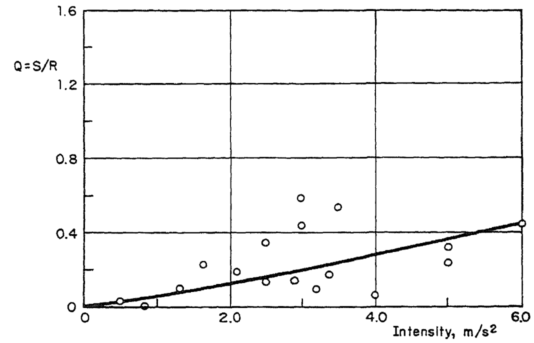

Fig 8 Normalized response of nine-story frames; T= 1.33 s, V_μ=0.3, μ^*= 2.5. Low spatial correlation

TABLE 1. CASES STUDIED

CASE NUMBER (1)	NUMBER OF STORIES (2)	FUNDAMENTAL PERIOD (SEC) (3)	DUCTILITY DESIGN FACTOR (4)	DUCTILITY COEFF. OF VARIATION (5)	SPATIAL CORRELATION (6)	SEISMIC DESIGN COEFFICIENT (7)	EXCITATION (8)
1	1	0.36	1	0.3	HC	0.69	EC
2				0.5			
3			2	0.3		0.35	
4				0.5			
5			4	0.3		0.17	
6				0.5			
7			6	0.3		0.12	
8				0.5			
9	3	0.85	2	0.3	LC	0.25	
10			4			0.12	
11		0.36	4	0.3	LC	0.17	
12					HC		
13				0.6	HC		
14	9	1.32	2.5	0.3	LC	0.115	SCT

HC - High correlation between structural member properties
LC - Low correlation between structural member properties
EC - El Centro, 1940, NS component
SCT - SCT, Mexico City, 1985, EW component

TABLE 3. CORRELATION COEFFICIENTS FOR CASES WITH LOW CORRELATION (LC) BETWEEN MECHANICAL PROPERTIES

VARIABLE	CORRELATION COEFFICIENT, ρ
f_c	0.6
f_y	0.8
b	0.8
h	0.8
r	0.8

TABLE 2. STATISTICAL PARAMETERS OF THE DISTRIBUTIONS OF MATERIAL PROPERTIES AND LOADS

VARIABLE	ASSUMED PROBABILITY FUNCTION	NOMINAL VALUE (kPa)	MEAN VALUE (kPa)	COEFFICIENT OF VARIATION
W_L	Gamma	0.88	0.69	0.480
f_c (field)	Gaussian	17600	19800	0.195
f_y	Gaussian	411600	458600	0.096
b,h,r	Gaussian	similar to those given by Mirza, 1979		

TABLE 4. EXPECTED FAILURE RATES

CASE NUMBER (1)	NUMBER OF STORIES (2)	$E(\nu_F)\times10^3$ (1/year) (3)	$\nu(y^*)\times10^3$ (1/year) (4)	RATIO $E(\nu_F)/\nu(y^*)$ (5)
1	1	2.550	12.00	0.213
2		1.456		0.121
3		1.342		0.112 (*)
4		1.007		0.084 (*)
5		0.332		0.028 (*)
6		0.420		0.035 (*)
7		0.036		0.003 (*)
8		0.137		0.011 (*)
9	3	2.609	12.00	0.217
10		2.060		0.172 (*)
11		2.426		0.202
12		2.527		0.211
13		1.794		0.150
14	9	0.213	13.7	0.0155