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# Damage and Risk Considerations for Selecting Seismic Design Requirements

Réflections sur les risques et les dommages pour le choix des prescriptions sismiques

Betrachtungen über Schaden und Risiko zur Wahl von Erdbebenbemessungsanforderungen

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## SUMMARY
This paper discusses some problems related with the determination of the failure probability of structural systems with uncertain properties subjected to random earthquake histories. The main purpose is to show how the relation between the expected failure rate of a structural system and the rate of occurrence of earthquakes with intensities greater than the design value is affected by variables such as uncertainty about system properties, P-Δ effects, number of potential failure modes and safety factors with respect to brittle and ductile failure modes.

## RESUME
Cette communication traite de certains problèmes liés à la détermination de la probabilité de rupture de systèmes structuraux à propriétés incertaines et soumis à une succession aléatoire de séismes. Son objectif principal est de déterminer l'influence de facteurs tels que l'incertitude sur les propriétés du système, les effets P-Δ, le nombre de modes de rupture potentiels et les facteurs de sécurité face aux modes de rupture fragile et ductile, sur la relation entre l'espérance de vie d'un système structural et le nombre de séismes dont l'intensité est supérieure à l'intensité de projet.

## ZUSAMMENFASSUNG
Dieser Artikel behandelt einige Probleme der Bestimmung der Ausfallwahrscheinlichkeit von Bauwerken, die durch Erdbeben beansprucht werden. Im Speziellen befasst sich der Artikel mit der Beziehung zwischen der erwarteten Versagenswahrscheinlichkeit des Bauwerkes oder eines Teils desselben und der Eintretenswahrscheinlichkeit von Erdbeben, deren Intensität grösser als die Intensität des der Bemessung zugrunde gelegten Bebens ist. Diese Betrachtungen berücksichtigen Unsicherheiten der Eigenschaften des Bauwerkes, P-Δ Effekte, Anzahl mögliche Versagensarten und Sicherheitsfaktoren im Hinblick auf sprödes oder elastisches Versagen.

## 1. INTRODUCTION

The main purpose of this paper is discussing a link which is missing in conventional seismic risk studies.

Significant research efforts have been devoted during the last few years to problems such as the probabilistic analysis of seismic hazard, the establishment of reliability based criteria for structural design and the development of cost-benefit and risk-cost-benefit criteria and methods for selecting optimum design values and safety factors. The results of this research have enabled structural engineers to produce designs for which the values of safety factors and failure probabilities for different critical members, sections or modes are such that the degrees of protection that each design is expected to provide with respect to the different potential failure modes are consistent with the corresponding expected costs, both those due to initial construction and those which may arise from failure and damage. In other words, the present development of the theory of probabilistic structural safety has permitted the establishment of adequate ratios, or relative values, of the mentioned safety factors or failure probabilities. However, if we talk about the absolute values of those variables we must recognize that neither the optimum safety levels nor the corresponding safety factors and design parameters have been derived for specific practical cases on the sole basis of the probabilistic theory of structural reliability: when reaching the point of stating desirable safety factors or β-values we resort to calibration with conventional design practice.

The probabilistic theory of structural safety has enabled code writers to decide which structure should be safer than other, and how safer, and therefore to make statements about the relative values that should be adopted for the corresponding safety factors; but establishing the desirable absolute values of those factors is something that has not escaped a comparison with or an adjustment to values that engineers have semi-intuitively arrived at after many years of trial and error. And those who try to make quantitative studies about the relations between the return periods of earthquake intensities adopted for design, the implicit failure probabilities and the observed failure rates bewilder at the numerous hindrances and apparent discrepancies; and yet, an extremely small amount of research efforts has been oriented to overcoming those hindrances and understanding those discrepancies.

For very important structures, such as large dams, large efforts and bitter arguments are spent in the establishment of seismic hazard curves (intensities *vs* return periods), and also in deciding what return period should be considered as the basis of design specifications. Nevertheless, the last link in the chain is overlooked: in general no attention is paid to a quantitative analysis of the influence that on failure probabilities have the probability density functions of the structural parameters and of the relationship between nominal design values, mean values and dispersion measures of those parameters. As a consequence, no clear understanding has been reached of the ratio of expected failure rates of given systems to rates of exceedence of design intensities. The need to understand this relation motivated the studies described herein.

What are the main reasons for the engineering profession and the participants in seismic-risk-related decisions to have overlooked this missing link? We can mention at least the following: the difficulties attached to handling of uncertainties associated with modelling of seismic hazard, the complexities involved in obtaining failure probabilites of systems with uncertain properties subjected to random earthquake excitation and the difficulties that arise when trying to evaluate failure consequences as well as when trying to express consequences of different types in the same scale, in order to build utility (objective) functions to be optimized. This paper is centered on the second of the above points -which does not mean that the other two are not at least equally relevant and worthy of study.

In the following, a very brief review will be presented of the basic concepts related to descriptions of seismic hazard and risk and their application to decision making. Then the influence of uncertain structural parameters and seismic design criteria on seismic reliability will be discussed in detail.

## 2. SEISMIC HAZARD AND SEISMIC RISK

Seismic hazard relates to the likelihood of occurrence of earthquakes of different sizes and intensities at given sources, regions or sites. In quantitative terms, it can be expressed by joint probability distributions of magnitudes, intensities, times of occurence and locations of seismic events. Seismic risk involves the likelihood of different degrees of damage. Typical (but not necessarily complete) descriptors of it are failure probabilities for given intensities or for given time intervals, probability distributions of time to failure, expected failure rates or expected costs of damage per unit time. Significant progress has been lately attained in the development of relatively sophisticated stochastic process models of seismic hazard [1-4] but their use for the development of seismic risk descriptors other than expected failure rates or expected costs of damage -which suffice as descriptors of risk when seismic activity is modelled as a homogeneous Poisson process with stocastically independent selection of intensity- is virtually unexplored.

If the seismic resistance of a structural system can be deterministically measured by the earthquake intensity causing the system to fail, the failure rate, $\nu_F$, is equal to the rate of occurrence of earthquakes having intensities Y greater than the resistance, y*. For many applications the latter rate can be expressed as $\nu(Y) = KY^{-r}$ [5], where K and r are site dependent parameters and y* should be substituted for Y. Under the assumptions that the system can only remain in the zero-damage state or suffer total collapse, that any time that the latter state is reached the system is instantaneously rebuilt in accordance with the same specifications, the present value of the expected cost of failure is equal to $D = D_0\nu_F/\gamma$ [1, 6], where D is the cost of each failure and $\gamma$ is the applicable discount rate (that is, a number such that a given cost or benefit $U_1$ suffered or perceived at instant t is equivalent to a utility $U = U_1 \exp(-\gamma t)$ at instant t = 0).

The seismic strength of a real structure to be built in accordance with a given design is not deterministically known in advance. The uncertainty is made up from the contributions of that associated to the material properties and of the randomness in the detailed ground motion characteristics for a given intensity. To these must be added the systematic or random errors arising from the inaccuracies of the structural response analysis algorithms. In ref. 1 these uncertainties are grouped according to whether they can be handled in terms of random variables indpendent from some seismic event to the other (denoted as type 1 uncertainty) or as dependent on the properties of the structural system, and changing only every time that the system is rebuilt (type 2 uncertainty). The same reference deals with the case when earthquakes take place in accordance with a Poisson process with mean rate of occurrence $\nu_0$ and random selection of intensities and of variables of type 1, and the structural properties are uncertain. If the system is repaired every time it is damaged so as to return it to its previous state, and in case of collapse it is replaced with a system (with uncertain properties) built in accordance with the same specifications as the original one -and therefore with the same distribution of structural properties- the present value of the expected cost of failure and damage is shown to be D as given by the following equation:

$$D = \frac{E_2\left[\left(A_c + \frac{A_d}{P}\right)\mu\right]}{1 - E_2(\mu)} \tag{1}$$

Here, $\mu = \nu_o P/(\gamma + \nu_o P)$, $A_c$, $\nu_o$ and $\gamma$ were defined above, $E_2(\cdot)$ is an expected value taken with respect to the probability density function of the variables in group 2, and P and $A_d$ are respectively the failure probability and the expected cost of damage other than collapse of a deterministic system for each earthquake of random intensity. The latter quantity is computed as follows

$$A_d = \int_0^R \delta(u/R) f_Y(u)\, du \tag{2}$$

In this equation, $\delta$ is a damage function which depends on the ratio of earthquake intensity to structural strength R, and $f_Y$ is the probability density function of earthquake intensities.

When the uncertainties associated to the variables of group 2 are not too large, eq. 1 may be approximated with the following

$$D \cong E_2[(A_c + \frac{A_d}{P}) \frac{\nu_o P}{\gamma}] \tag{3}$$

An important part of this paper is devoted to assessing the influence of uncertainty about system properties on $E_2(\nu_o P)$ -the expected failure rate- and its relation to $\nu^*$ - the rate of occurrence of earthquakes with intensities greater than the design value.

## 3. INTENSITY, RESPONSE AND FAILURE CONDITION

Fig. 1 shows a set of linear response pseudovelocity spectra for the El Centro earthquake of 1940 on a four-log plot, which also shows peak values of ground acceleration, velocity and displacement (a, v, d). The ordinates of the pseudovelocity spectra are assumed to be the sum of the expected values shown by the dashed lines, which are determined by a, v and d, and random deviations with respect to the expected value. The latter can be obtained from a and d as follows

$$\frac{\bar{S}_v}{\sqrt{ad}} = \frac{\chi}{[(1 - \chi^m + e\chi^{m/z})^{zn} + D(\chi)]^{1/mn}} \tag{4}$$

Here, $\chi = \omega/\omega_o$, $\omega$ is natural frequency, $\omega_o = \sqrt{a/d}$, $e = 0.15$, $m = 0.5$, $n = 2$ and $D(\chi) = \alpha_1\chi^{0.7} + \alpha_2\chi^2$, where $\alpha_1$ and $\alpha_2$ are functions of the damping ratio. If the second member in eq. 4 is denoted by $G(\chi)$, the linear response spectra of other earthquakes can be scaled with respect to a and v as follows

$$\frac{\bar{S}_v}{a} = G(\chi)\,\frac{v}{a}\,\frac{\sqrt{ad}}{v} \quad \text{(5a)} \quad , \quad \frac{\bar{S}_v}{v} = G(\chi)\,\frac{\sqrt{ad}}{v} \quad \text{(5b)}$$

Using information available for the earthquake record being considered, simple equations have been obtained for the ordinates of the elastoplastic displacement spectra D, in terms of the yield ratio $\varepsilon_y = A_D \bar{S}_d^{-1} \omega^{-2}$, where $A_D$ is the ratio of lateral strength to vertical load expressed in units of gravity and $\bar{S}_d$ is the expected linear displacement spectrum, equal to $\bar{S}_v \omega^{-1}$. The results can be expressed as

$$D = \bar{S}_d \frac{Q + 8TQ}{1 + 8TQ} \xi_{D_e} \tag{6}$$

where Q is the required ductility factor, T is the natural period (equal to $2\Pi/\omega$) and $\xi_{D_e}$ is an uncertainty factor which depends on T and Q.

The failure condition for a simple elastoplastic system is obtained by equating response and deformation capacity, that is

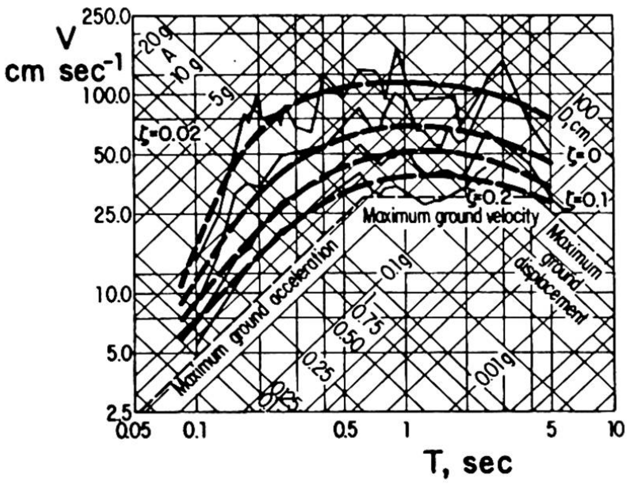



Fig. 1 Linear response spectra. El Centro, 1940

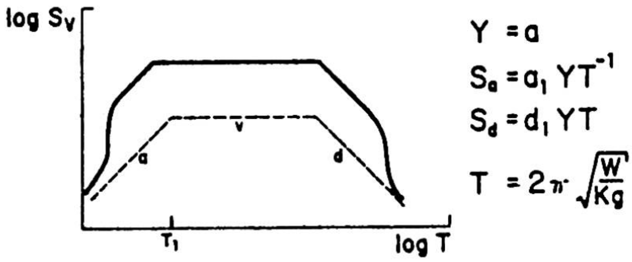
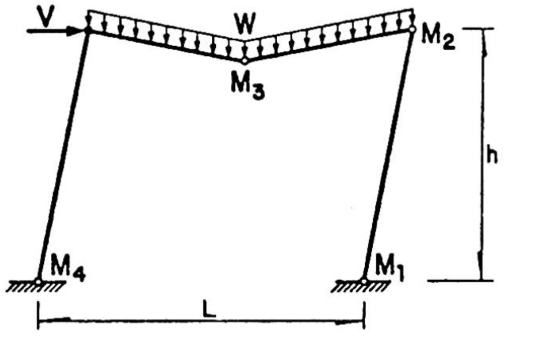



Fig. 2 Simplified expected spectrum




$$R = \frac{1}{h}\left(M_1 + 2M_2 + 2M_3 + M_4 - \frac{WL}{4}\right)$$

Fig. 3 Lateral resistance of single-story structure

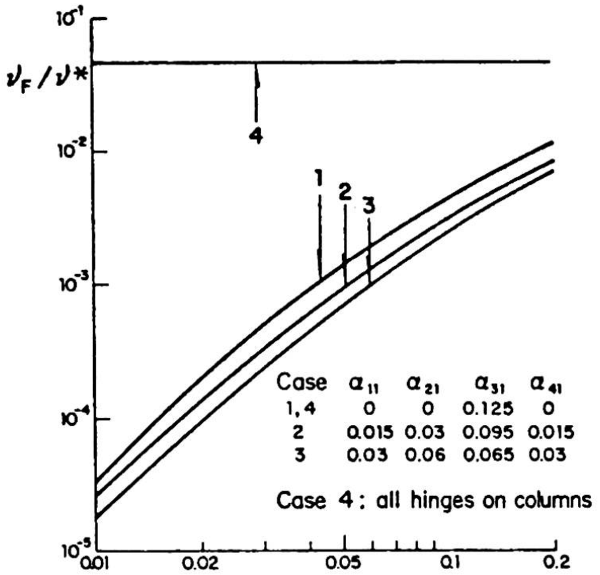



Fig. 4 Failure rates for single-story frames

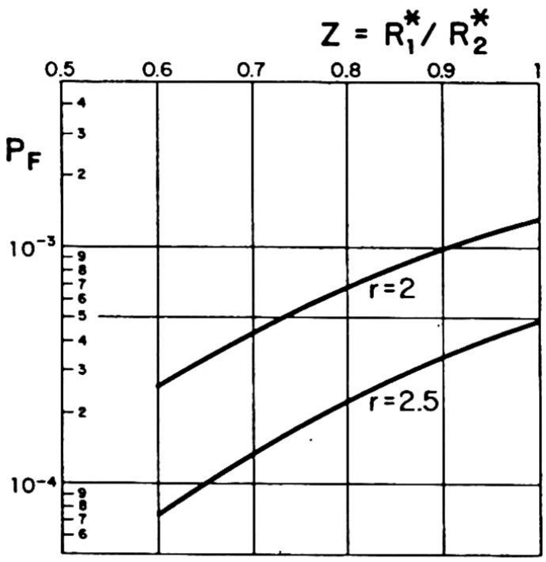



Fig. 5 Simple systems with ductile and brittle failure modes

$$v\,G\,(\chi)\,\frac{\sqrt{ad}}{v}\cdot\frac{1}{\omega}\frac{Q+8TQ}{1+8TQ}\,\xi_{D_e}=QR/K \tag{7}$$

where K is lateral stiffness and R is lateral strength. The probability of failure is the probability that the first member is greater than the second. The failure intensity v can be obtained solving for v in eq. 7. The first two moments of its probability density function can then be obtained by straight forward application of first-order second-moment analysis (7).

A simplified condition is used in the following for the purpose of making a parametric study of the expected failure rates of simple and multi-story elasto-plastic building frames: instead of adopting for $G(\chi)$ the form defined by eq. 4 and depicted in fig. 1, use is made of the conventional form shown in fig. 2, under the assumptions that intensity is measured by peak ground acceleration a and that the natural periods of all systems studied lie well within the constant pseudovelocity branch. Linear spectral acceleration and displacement are related to Y in accordance with the expressions shown in fig. 2, where $a_1$ and $d_1$ are transformation coefficients. Thus, instead of eq. 7 the following failure condition is obtained,

$$Y\,d_1\,(2\Pi\sqrt{\frac{W}{Kg}})\,\xi_{D_e}=\frac{QR}{K} \tag{8}$$

and the ratio of the random failure intensity Y to its nominal design value Y* equals

$$\frac{Y}{Y^*}=\frac{\sqrt{k^*w^*}}{q^*\rho^*}\,\frac{q\rho}{\xi_{D_e}\sqrt{kw}} \tag{9}$$

where $k=K/\bar{K}$, $k^*=K^*/\bar{K}$, $q=Q/\bar{Q}$, $q^*=Q^*/\bar{Q}$, $\rho=R/\bar{R}$, $\rho^*=R^*/\bar{R}$, $w=W/\bar{W}$, $w^*=W^*/\bar{W}$; the asterisks denote nominal design values and the bars denote mean values.

## 4. EXPECTED FAILURE RATES

Under the assumptions that Y in eq. 9 has a lognormal distribution and that $\nu(y)=Ky^{-r}$, the expected failure rate $\nu_F$ and the rate of exceedence of the design intensity, $\nu^*$, are related as follows

$$\frac{\nu_F}{\nu^*}=\left(\frac{y^*}{\bar{Y}}\right)\,(1+V_Y^2)^{\frac{r(r-1)}{2}} \tag{10}$$

Here, $\bar{Y}$ and $V_Y$ are mean value and variation coefficient of Y, and $y^*$ is the nominal value of the design intensity. If $y^*$ is related to $\bar{Y}$ through the expression $y^*=\bar{Y}\exp(-2V_Y)$ [8], one obtains that $\nu_F/\nu^*$ equals 0.26 and 0.15 if r equals respectively 2.5 and 4 and $V_Y=0.3$.

In order to study the variation of $\nu_F/\nu^*$ for the most frequent ranges of values of $y^*/\bar{Y}$ and $V_Y$, a parametric study was carried out for a single-story frame depicted in fig. 3 in its yielding mechanism. The lateral strength R is as shown in the same figure. The vertical load W and the resisting moments $M_i$ at critical sections were considered as random variables. The nominal design values of those moments are $M_i^*=\psi W^*L(\alpha_{i1}+\alpha_{i2}\varepsilon)$, where $\varepsilon=V^*h/(W^*L)$, $\psi$ is a load factor (taken here as 1.1, in accordance with Mexico City seismic code), $\alpha_{i1}$ and $\alpha_{i2}$ are influence coefficients for vertical and lateral load respectively and $V^*$ is nominal value of design lateral load. Fig. 4 shows the results in terms of $\varepsilon$ for the sets of values of $\alpha_{i1}$ listed in the same figure and the following values of the other parameters: $\alpha_{12}=\alpha_{42}=0.3$, $\alpha_{22}=0.2$, $\alpha_{32}=0$, $h/L=0.5$, $V_W=0.25$, $w^*=1.65$, $V_{M_i}=0.25$, $m_i^*=0.61$, $\rho_{ij}=\delta_{ij}+0.25(1-\delta_{ij})$, $V_K=0.15$, $k^*=0.74$, $V_Q=0.3$, $q^*=0.55$, $\bar{\xi}=1$, $V_\xi=0.25$. Here, V means variation coefficient, $m_i^*=M_i^*/\bar{M}$, and $\rho_{ij}$ is the correlation coefficient between $M_i$ and $M_j$.

Curves 1, 2 and 3 in fig. 4 show a strong dependence (orders of magnitude) of $\nu_F/\nu^*$ on the ratio of the nominal design shear force to the design vertical load. This is due to the lateral strength which results from continuity considerations when designing for vertical load. A constant value is obtained for case 4, which assumes that the yielding mechanism is produced by hinges at the column ends and that $\alpha_{i1} = 0$ for all i. This constant value is an upper bound to the other curves.

An approximate analysis was carried out in order to study the possible influence of multiple, imperfectly correlated, failure mechanisms, on the reliability of a multistory system, as compared to that of a simple system. Shear buildings having 1, 2, 5, 10 and 20 stories were studied. They were assumed to have been designed so as to provide uniform safety factors for all story shears. The objective was to obtain the first two moments of the probability density function of the ratio of the failure intensity (that is, of the minimum of the failure intensities $Y_i$ determined for the individual stories) to the design value. This was achieved by means of a Monte Carlo approach, using the following parameters

| | | | |
|---|---|---|---|
| $q^* = 0.53$ | $V_q = 0.3$ | $\rho_{ij} = 0.5$ | q, r and $\xi$ mutually independent |
| $r^* = 0.72$ | $V_r = 0.2$ | $\rho_{ij} = 0.5$ | |
| $\xi^* = 1$ | $V_\xi = 0.25$ | $\rho_{ij} = 0.5$ | $\nu = Ky^{-r}$ , $r = 2.5$ |
| $k^* = 0.74$ | $V_k = 0.15$ | $\rho_{ij} = 1$ | |
| $w^* = 1.65$ | $V_w = 0.25$ | $\rho_{ij} = 1$ | $\psi = 1.1$ |

The results are summarized in the following table, where $\lambda = \min(\ln(Y_i/Y^*))$ and $y_n^*/y^*$ is the ratio of the design intensity for n stories to that for 1 story required for obtaining equal failure rates. The last two columns are obtained under the assumpltion that $\lambda$ has a lognormal distribution.

| n | $\bar{\lambda}$ | var $\lambda$ | $\nu_n^*/\nu_1^*$ | $y_n^*/y^*$ |
|---|---|---|---|---|
| 1 | 1.11 | 0.21 | 0.055 | 1.0 |
| 2 | 0.96 | 0.17 | 0.082 | 1.17 |
| 5 | 0.79 | 0.15 | 0.126 | 1.40 |
| 10 | 0.69 | 0.14 | 0.165 | 1.55 |
| 20 | 0.56 | 0.14 | 0.228 | 1.77 |

The influence of multiple failure modes is obvious.

## 5. SLENDERNESS EFFECTS (P-Δ)

Ref. 9 presents a plot of the failure probabilities of deterministic simple bilinear hysteretic systems subjected to segments of duration 15 sec of stationary Gaussian white noise. Each system is determined by its initial stiffness $K_1$, the negative stiffness $K_2 = -W/H$ of the second portion of the force-deflection curve, the damping ratio $\zeta = 0.03$ and the yield deflection $y_o$, equal to 0.25 times the expected maximum linear response displacement. The table that follows summarizes failure probabilities obtained from fig. 8 of ref. 9

| T | $K_1/\lvert K_2\rvert$ | | | |
|---|---|---|---|---|
| | 100 | 80 | 40 | 20 |
| 0.8 | $1.13 \times 10^{-4}$ | $3.30 \times 10^{-3}$ | 0.157 | 0.555 |
| 1 | $7.31 \times 10^{-5}$ | $2.33 \times 10^{-3}$ | 0.114 | 0.479 |
| 1.5 | $4.54 \times 10^{-5}$ | $1.10 \times 10^{-3}$ | 0.082 | 0.368 |
| 2 | $4.54 \times 10^{-5}$ | $1.01 \times 10^{-3}$ | 0.062 | 0.287 |
| 2.5 | $4.54 \times 10^{-5}$ | $1.01 \times 10^{-3}$ | 0.040 | 0.230 |
| 3 | $4.54 \times 10^{-5}$ | $1.01 \times 10^{-3}$ | 0.036 | 0.199 |
| 3.5 | $4.54 \times 10^{-5}$ | $1.01 \times 10^{-3}$ | 0.032 | 0.184 |
| 4 | $4.54 \times 10^{-5}$ | $1.01 \times 10^{-3}$ | 0.028 | 0.168 |

This table shows that, as $K_1/|K_2|$ and the natural period increase, the failure probability decreases, and that beyond a given period the failure probability remains constant. Values of $K_1/|K_2|$ of about 20 are not infrequent in practical cases.

The above results were used to obtain estimates of failure probabilities of deterministic multistory buildings including P-Δ effects. It was assumed that the dynamic response is the product of a function of time by the shape of the fundamental mode, and that P-Δ effects give place to reduced lateral stiffness but do not affect the shape function; in other words, the response of the multistory systems was obtained from that of a simple system by assuming the generalized mass and initial stiffness valid for a linear modal analysis, and extending the concept of generalized stiffness to the range of negative stiffness values. It was also assumed that the fundamental natural period T in seconds varies as H/30 where H is the building height in meters. A summary of the results follows. The last column shows the ratio of the design intensities required to give place to the failure probability corresponding to a system with T = 1 sec

| T | $P_F$ | $y_o(T)/y_o(1)$ |
|---|---|---|
| 1.0 | $1.93 \times 10^{-3}$ | 1.0 |
| 1.5 | $1.98 \times 10^{-2}$ | 1.25 |
| 2.0 | $6.22 \times 10^{-2}$ | 1.58 |
| 2.5 | $9.25 \times 10^{-2}$ | 1.76 |
| 3.0 | $1.46 \times 10^{-1}$ | 1.92 |
| 3.5 | $1.62 \times 10^{-1}$ | 2.12 |
| 4.0 | $1.68 \times 10^{-1}$ | 2.22 |

## 6. SAFETY WITH RESPECT TO DUCTILE AND BRITTLE FAILURE MODES

Let $R_1$ be the lateral force required to make a simple system reach its yield strength in a ductile failure mode and $R_2$ the lateral force required to make that system reach its maximum capacity with respect to a perfectly brittle failure mode. The system fails in a ductile manner if the internal force $S_1$ giving place to ductile failure is reached before the internal force $S_2$ giving place to brittle failure. If this happens, $S_2$ is controlled by the facts that it is correlated with $S_1$ and that the latter cannot grow above the value that gives place to ductile failure. This can be expressed by the condition that $S_2 \leq \gamma_{21} R_1$, where $\gamma_{21}$ is random. If the design value $R_2^*$ with respect to the brittle mechanism is kept constant and the design value $R_1^*$ with respect to the ductile mode is made to grow, one does not obtain a safer structure, but one for which the probability of reaching the brittle failure condition is greater. This is shown in fig. 5 for a set of values of the relevant parameters which are representative of usual practical cases. the probability of failure corresponds to the occurrence of one earthquake of random intensity for two cases of the parameter r in the equation $\nu = Ky^{-r}$.

## 7. CONCLUSIONS

An analysis has been presented of the influence of the uncertainty about structural parameters as well as of design criteria on the expected failure rates of structures subjected to earthquakes. On the basis of approximate estimates of the probabilistic dynamic response, the following can be concluded:

a) Failure rates of structures with uncertain parameters may be orders of magnitude smaller than rates of occurrence of earthquakes with intensities greater than the design values.

b) A very important variable contributing to this difference is, as should have been expected, the lateral strength already available in continuous frame structures subjected to vertical loads.

c) Due to the possible occurrence of multiple failure modes, multistory buildings may be less safe --and significantly so-- than single-story structures nominally designed for the same spectrum and the same safety factors.

d) Accounting for P-Δ effects may drastically affect failure probabilities, even in ranges of parameters usual in real structures. The influence of these is very sensitive to the natural period. This is true also for multistory systems.

e) Overdesign with respect to ductile failure modes may be harmful, rather than beneficial, if safety factors with respect to brittle modes are not correspondingly raised. The results shown are based on crude estimates of the probabilistic dynamic response of nonlinear systems. In view of the very high sensitivity of failure probabilities to the variables studied, more detailed studies along the same lines, but using more refined models, are strongly recommended.

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