SEISMIC RISK, DESIGN SPECTRA AND STRUCTURAL RELIABILITY

by

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SYNOPSIS

A formulation of seismic structural reliability is presented that analyzes the probability distribution of the maximum seismic response of a system with imperfectly known properties. The various sources of uncertainty in the parameters describing seismicity and in those related to the structural properties are analyzed within a framework intended to permit formulation of design requirements. Simplified reliability models are introduced, having in mind practical applications.

INTRODUCTION

Structural reliability analysis for seismic effects requires assessment of the probability distribution of the maximum seismic response of a given structure during a given time interval. This assessment is based on a set of different concepts, including analysis of statistical information on intensities at the site and on magnitudes of earthquakes originating at neighbouring seismic provinces, geotectonic studies, corrections for the influence of local soil, and probability distribution of the structural parameters affecting the response.

This paper aims at formulating a unified treatment of the seismic risk problem that puts together, from the viewpoint of the engineer who has to make design decisions, the various uncertainties that are found along the line from earthquake magnitudes near the site to the influence of local soil and the distribution of the maximum response of structures with imperfectly known values of period and damping.

Wherever feasible, simplifications have been introduced in order to maintain the analysis of seismic structural reliability within a framework simple enough to permit its application to formal decision making in the selection of seismic strength requirements in practice. Some of the concepts described here are being applied to actual problems in Mexico, while others are still in the phase of development and implementation.

SEISMIC RELIABILITY OF SIMPLE SYSTEMS

It is assumed that seismic design decisions will be made on the basis of maximum expected actualized utilities as described elsewhere,[1,2] and exemplified by eq.21. For a simple structure, the expected actualized cost of failure is a function of the rate of occurrence of seismic excitations of intensity higher than that necessary to produce structural failure, and this rate, in turn, depends on the probability of structural failure under given intensities and on the rate of exceedance of those intensities.

Since in most cases rates of exceedance of given intensities at a site have to be derived from estimates of rates of exceedance of given magnitudes at the seismic sources near the site, use has to be made of attenuation laws, similar to eq. 6. For

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a given magnitude and focal distance, computed values of peak absolute amplitudes of ground acceleration, velocity and displacement are obtained by means of such equations, and from them the ordinates of the average response spectrum are estimated. For an uncertainty analysis of the spectral response as a function of natural period for a given magnitude and focal distance, let Y be the peak absolute value of ground acceleration (or velocity), Y_c its value computed in terms of magnitude and focal distance, Q(T) the spectral response for period T, and S(T) the ordinate of the average spectrum (or smoothed, eliminating local peaks and valleys) for the same period. Thus, Q and Y_c are related as follows:

$$Q = \phi_Y \, \phi_S \, \phi_Q \, Y_c \qquad (1)$$

Where $\phi_Y = Y/Y_c$, $\phi_S = S/Y$ and $\phi_Q = Q/S$. Since for most structures S is more significant than Q in determining their seismic performance, the former variable will be used as the response measure. As shown later, the rate of exceedance of given values of S can be obtained in terms of the rate of exceedance of given values of Y_c and of the probability distributions of S/Y_c. If an assumption is made concerning the shape of that distribution, only its first and second moments will have to be obtained. Hence, after some simplifcations

$$\bar{S} = \bar{\phi}_Y \, \bar{\phi}_S \, Y_c \qquad (2)$$

$$V_S^2 = V_{\phi_S}^2 + V_{\phi_Y}^2 + \rho_{\phi_S \phi_Y} V_{\phi_S} V_{\phi_Y} \qquad (3)$$

where the bars above the variables identify their mean values, $V_{\phi S}$ and $V_{\phi Y}$ are respectively the coefficients of variations of ϕ_S and ϕ_Y, and $\rho_{\phi S \, \phi Y}$ is their correlation coefficient.

An analysis of the distribution of error of eq. 6 has been made previously. If ϕ_S and ϕ_Y were stocastically independent, V_S^2 would equal $V_{\phi S}^2 + V_{\phi Y}^2$. However, it is reasonable to expect that to abnormaly high values of ϕ_Y (for instance, a single peak of ground acceleration much larger than expected) should correspond relatively low values of ϕ_S. This would imply a negative correlation coefficient and hence a value of V_S^2 lower than that obtained under the independence assumption. This hypothesis was confirmed by the studies described below concerning the derivation and the error analysis of attenuation laws for directly predicting ordinates of the average response spectrum.

In the computation of structural reliability under an event of a given intensity, or of a given computed intensity, or given magnitude and focal distance, uncertainty in structural properties has to be accounted for. For perfectly brittle simple structures with perfectly known period and damping, only the distribution of R, the lateral strength, is required. For an elastoplastic simple system with uncertain ductile capacity, lateral strength and natural period, let $S_A(T)$ denote the ordinate of the average acceleration spectrum for period T, $\bar{S}_A(T)$ its expected value (predicted by an equation similar to 6), T and T_o respectively the actual and the nominal natural periods, a the ratio of peak elastoplastic to average linear relative displacement response, $D_{(T)}$ the peak elastoplastic relative displacement for period T, D_E the yield deformation of the structure, and μ its ductile capacity (ratio of failure deformation to yield deformation). For the nominal period, $D(T_o) = a \, S_A(T_o) \, (T_o/2\pi)^2$. For a given $S_A(T_o)$, D(T) is obtained as follows:

$$D(T) = \frac{S_A(T_o)\, T_o^2}{4\pi^2}\, a\, \frac{\bar{S}_A(T)}{\bar{S}_A(T_o)} \left(\frac{T}{T_o}\right)^2 \qquad (4)$$

where $\bar{S}_A(T)/\bar{S}_A(T_o)$ was written instead of $S_A(T)/S_A(T_o)$, in view of the strong correlation between numerator and denominator when $T/T_o \cong 1$. Since the deformation capacity is given by $D_E\,\mu$, failure occurs if

$$S_A(T_o) > \frac{\mu}{a} D_E \left(\frac{2\pi}{T}\right)^2 \frac{\bar{S}_A(T_o)}{\bar{S}_A(T)} \left(\frac{T_o}{T}\right)^2 \qquad (5)$$

Thus, the second member of this equation can be identified as the structural capacity, measured in terms of spectral acceleration for period T_o. Its probability distribution can be calculated, or a simple second moment formulation may be adopted, in order to estimate the failure probability for a given $S_A(T_o)$.

INTENSITIES AND RESPONSE SPECTRA

Previous publications[1] contain expressions relating intensities with magnitudes and hypocentral distances. Such expressions were found proccessing the information contained in references 8 and 9, and from the data derived from MM intensities of mexican earthquakes. They were found to be of the form

$$Y_c = b_1\, e^{b_2 M}\, R'^{-b_3} \qquad (6)$$

where M is magnitude, R hypocentral distance, R' a function thereof, and b_1, b_2 and b_3 are constants. The following parameters produced the best fit to the data used. The last two columns show the mean and the standard deviation of the natural logarithm of observed to computed intensities.

Expression for:	b_1	b_2	b_3	R'	m	σ
Peak ground acceleration = a (cm/sec^2)	1 230	0.8	2.0	R + 25	0.14	1.02
Peak ground velocity velocity = v (cm/sec)	15	1.0	1.7	$R+0.17e^{0.59M}$	0.14	0.84

In the process of deriving expressions to predict maximum values of average spectral ordinates the former expressions were actualized, incorporating new data available.[10 – 13] From these new sources, the two horizontal components of each shock were considered. Their response spectra were smoothed in order to eliminate local peaks and valleys. The following set of parameters resulted. The corresponding data, the resulting expressions and the relative frequencies of the natural logarithm of the ratio of observed and computed intensities are plotted in figs. 5 – 12.

Expression for:	b_1	b_2	b_3	R'	m	σ
Peak ground velocity = v	32	1.0	1.7	R + 25	0.124	0.74
Peak ground acceleration = a	5 600	0.8	2.0	R + 40	0.04	0.64
Maximum Average Spectral Velocity = $\bar{V}$	250	1.0	1.7	R + 60	0.058	0.64
Maximum Average Spectral Acceleration = $\bar{A}$	69 600	0.8	2.0	R + 70	0	0.75

Comparison of new and previous expressions (see fig 5,6 and 10) shows that the new ones give predicted values about twice as larger as the former ones. The dispersion is now much lower, and the combined effect of both changes, when introduced in eq. 13 is not very important. From figs. 5 – 12 it is clear that maximum ordinates of average response spectra are predicted with the same degree of uncertainty as peak ground acceleration or velocity.

SEISMIC RISK ON FIRM GROUND

Let $\lambda(M)$ be the mean annual number of earthquakes having magnitude equal to or greater than M per unit volume of the earth's crust. This function can be approximated as follows:[1,7]

$$\lambda(M) = \lambda_0 \, g(M) \, e^{-\beta(M - M_0)} \quad , \text{ for } M \leqslant M_1 \qquad (7)$$

$$= 0 \quad , \text{ otherwise}$$

Here, λ_0 is the value of $\lambda(M_0)$, M_0 is an arbitrary reference magnitude, and M_1 is the upper bound to the magnitudes that can be generated in a seismic province, and λ_0, β and M_1 are functions of the location of the elementary volume being considered. The function g(M) serves to produce a better fit of eq. 7 in the ranges of very low and very high magnitudes. For simplicity, it will be taken as 1 in the following.

Let us now define the contribution of an elementary volume of a seismic source to the seismic risk at a site. Let $\nu(y)$ denote the mean annual number of earthquakes whose intensity[II] at the site of interest exceeds y. Let ν_c (y) be the same mean number, but with intensities computed in terms of magnitudes and focal distances, instead of being actually measured intensities. The contribution of the seismicity of the elementary volume ΔV to $\nu_c(y)$ at a site distant R kilometers is

$$\Delta\nu_c(y) = \lambda(M(R, y) \, \Delta V \qquad (8)$$

where M(R, y) is the magnitude whose computed intensity at a distance R is equal to y. From eqs. 6 and 7,

II In this paper, *intensity* means any measure of the ground motion which is significant for the purpose of estimating peak structural response. Thus, according to the problem at hand, it will be taken to mean peak ground velocity, peak ground acceleration, ordinates of response spectra for given periods, etc.

$$\Delta\nu_c(y) = Ky^{-r}, \quad \text{for } y \leq y_1 \qquad (9)$$
$$= 0, \quad \text{otherwise}$$

Here,

$$K = \lambda_0 \, e^{\beta M_0} \, b_1^r \, R'^{-\rho} \Delta V \qquad (10)$$
$$r = \beta/b_2 \qquad (11)$$
$$\rho = \beta b_3/b_2 \qquad (12)$$

and y_1 is the computed intensity produced by an earthquake of magnitude M_1 at the distance R.

It has been shown[3,4] that if f(x), the probability density function of the ratio of observed to computed intensity is independent of the latter variable, $\nu(y)$ and $\nu_c(y)$ are mutually related as follows:

$$\nu(y) = \int_0^\infty \nu_c \, (y/u) \, f(u) \, du \qquad (13)$$

According to a previous paper f(.) is the lognormal probability density function. The mean and the standard deviation of the natural logarithm of the ratio of actual to computed intensity are denoted by m and σ, respectively. From eqs. 9 and 13

$$\Delta\nu(y) = h \, Ky^{-r} \qquad (14)$$

Where

$$h = e^{K_0} \, [1 - \Phi(a)] \qquad (15)$$

$\Phi(.)$ is the normal cumulative probability distribution function with mean = 0 and standard deviation = 1,

$$a = (\log a_1 - u_0)/\sigma \qquad (16a)$$
$$K_0 = 0.5 \, \sigma^2 r^2 + mr \qquad (16b)$$
$$u_0 = m + \sigma^2 \, r \qquad (16c)$$
$$\phi_1 = y/y_1 \qquad (16d)$$

Integration of the contribution of all sources close to the site leads to:

$$\nu(y) = y^{-r} \int_V \lambda_0 \, e^{\beta M_0} \, b_1^r \, R'^{-\rho} \, h \, dV \qquad (17)$$

where β, λ_0, and h are functions of position. Using this expression, $\nu(y)$ was evaluated for a site in the southeast of Mexico, near the coast of the Pacific Ocean and the border with Guatemala. The area is characterized by very frequent earthquakes originated either at small depths (about 20 km) in a zone where the crust is subjected to tensile stressess or at larger depths (100 km) in the zone of contact between the continental mass and the subducting material resulting from underthrusting of the Pacific Ocean bottom plate. Epicenters of recorded motions in the area do not show any clear pattern following known surface faults. Thus, local seismicity was idealized as uniformly distributed throughout horizontal planes at 20 and 100 km respectively. Values of λ_0 for both seismic sources were inferred from statistical information, while β was taken as 2.16, on the basis of the value it adopts for a wider area that includes that of the greatest contribution to seismic risk at the site. According to geologists, magnitudes as high as 8 can be expected to originate at 100 km (or in what will be called zone 2 in the sequel), whereas at 20 km (zone 1) no magnitudes greater than 6.5 were considered possible. Peak ground accelerations were computed with eq. 6, and the curve of frequency vs. computed intensities was corrected

by means of eq. 14, with $\sigma = 1.0$ and $m = 0.14$. Fig. 1 permits a comparison to be made of the influence of various assumptions concerning M_1, the upper bound assigned to magnitudes at each zone, on $\nu(y)$. As seen in the figure, the assumption of relatively low values for M_1 reduces significantly the intensity that corresponds to a given return period only for high values of the latter. It is also seen that $\nu(y)$ may be expressed as

$$\nu(y) = K(1 - g(y))\, y^{-\tau} \qquad (18)$$

where g(y) is a function that increases monotonically with y, within the bounds 0 and 1.

Fig. 2 depicts in a different manner the same information contained in fig. 1. Here, probabilities of exceedance of given intensities in 10, 100 and 1 000 years are compared with the corresponding probabilities for a single occurrence of the worst event, i.e. magnitude M_1 at the shortest possible distance. It is seen that, either if we consider given time intervals or if we are concerned only with the worst possible event, there are significant probabilities of exceedance of the computed intensities that correspond to the worst possible event.

UNCERTAINTY IN SEISMIC RISK PARAMETERS

Both the form and the parameters of the stochastic models used for the evaluation of seismic risk stem from estimates based on imperfect knowledge of nature. The following is an analysis of some factors that contribute to the uncertainty on the mentioned form and parameters, and of the manner in which they may affect seismic design decisions.

Under the assumption that in a given volume of the earth's crust the generation of earthquakes having a magnitude greater than a given value can be represented by a Poisson process, evaluation of seismic risk at a site is affected by uncertainties tied to the values of λ_0, β and M_1 for the seismic provinces near the site, to the accuracy of the attenuation laws predicting intensity in terms of magnitude and distance, and to the estimation of the response of local soil formations. Previous works[3,4] have dealt with the nature and influence of the uncertainty in λ_0 and in β. Decisions for the seismic design of ordinary structures are usually sensitive to λ_0 and β_1, but practically insensitive to M_1. For some structures whose failure might have exceptionally high consequences the opposite may be the case. According to the problem at hand, the design earthquake may be selected either on the basis of a geologist's assessment of the maximum magnitude that could be generated in the vicinity of the site or on the basis of curves of intensity vs return period obtained by statistical analysis of recorded events. In very few cases is seismic risk estimated on the basis of an analysis that makes rational use of both sources of information and of both points of view. When the second approach is followed, a decision has to be made as to the acceptable failure probability for a given time interval. The first approach is apparently consistent with a minimax decision maker, who would act as though he were sure of the future occurrence of the worst possible natural event. This is not true, however, as a consequence of the very significant, although generally ignored, uncertainties tied to the maximum possible magnitude to be expected in a given province and to the attenuation laws. Thus, the first approach leaves the sometimes wrong impression that a sufficiently safe decision has been made, even though no formal reliability analysis has been performed.

In areas of high seismicity λ_0 and β may be estimated with great accuracy, and thus only M_1 has to be dealt with as an uncertain parameter. Following the case studied in figs. 1 and 2, bayesian probabilities were assigned to the various assumptions concerning M_1, in accordance with the geologist's degree of belief on them. Thus, bayesian probabilities of 0.15, 0.80 and 0.05 were assigned to the hypotheses that M_1 in zone 1 equals 6.0, 6.5 and 8.0, respectively.

Consequences of this uncertainty in the design decision are presented below.

A more significant source of uncertainty, generally neglected, is that tied to the validity of the assumption about the statistical stability, within intervals of several hundred years, of the stochastic process representing seismicity. In other words, what is put in doubt here is the Poisson process model and the possibility of estimating its parameters on the basis of statistical information, even in areas of high seismicity, where samples with a large number of events are available. Consider for instance fig. 3, which represents a site in the southern coast of Mexico. Using information about magnitudes and coordinates of earthquakes recorded in the vicinity of the site since the beginning of the century, intensities were obtained for each of them by means of eq. 6, and curve A was drawn, representing estimates of return periods of computed intensities. It was then realized that for a given computed intensity the corresponding return period was much longer than for practically any other point in the southern coast of Mexico. A second evaluation of seismic risk was performed, under the assumption that λ_0 and β for the earthquake souces near the site were equal to the corresponding mean values for wider seismic provinces including those sources. The corresponding intensity-return period curve is shown by the solid line B. According to geotectonic studies[5] there is no reason to consider that potential activity near the site should be lower than in the rest of the mentioned seismic sources, but that, on the contrary, the relative quiescense of a given area of potentially dangerous tectonic features might conceal gradual accumulation of energy. If this is so, seismic risk would be highest where statistics say it is lowest. Thus, studies on the laws governing the possible migration of seismic activity would be of utmost importance. Collaboration of geophysicists and statisticians in an effort to incorporate study of the physical problem and statistical analysis of records of seismic activity in different regions would then be mandatory.

SEISMIC RISK ON NONLINEAR SOIL FORMATIONS

Given $\nu_c(y)$ or $\nu(y)$ on rock it is possible to obtain the corresponding expressions for the surface of soil deposits, provided it is possible to determine the distribution of the significant parameters of the surface motion in terms of the intensity of the rock base motion As a consequence of nonlinearity an equation similar to 13 ceases to be applicable, and the following has to be used instead:

$$\nu_{S_1(T_o)}(y) = \int - \frac{\partial \nu_{S_0(T_o)}(u)}{\partial u} P(S_1(T_o) > y \,|\, u)\, du \qquad (19)$$

Here, $S_1(T_o)$ and $S_o(T_o)$ are respectively the ordinates of the average response spectrum for a given period T_o at the surface and at the base rock, and $P(A\,|u)$ denotes conditional probability of A given that $S_o(T_o) = u$. In practical applications this probability may be obtained by Monte Carlo simulation at several levels of $S_o(T_o)$. Through use of interpolation functions the distributions that correspond to other levels of T_o can be obtained. A relatively small number of samples at each of three or four levels may suffice for this purpose, since the largest uncertainty in the seismic reliability problem is associated with the maximum $S_o(T_o)$ for a given time interval rather than with the dispersion in the soil response to an earthquake of a given intensity.

In a separate paper[6] an approximate procedure was presented for estimating the ordinates of the average response spectra of stratified non linear soil formations in terms of the corresponding spectra on bedrock. The procedure starts from the assumption of unidimensional propagation of shear waves, and states that the statistical description of the motion at the surface can be obtained by assuming that the effects of nonlinearity can be accounted for by assuming an equivalent transfer function of bedrock accelerations to surface relative displacements.

The probability P(A|u) in this case is obtained as follows:

$$P(S_1(T_o) > y|u) = P\left(\frac{S_1(T_o)}{\bar{S}_1(T_o)} > \frac{y}{\bar{S}_1(T_o)} \mid u\right). \qquad (20)$$

where the conditional distribution of $S_1(T_o)/\bar{S}_1(T_o)$ is obtained from an analysis of error distribution of the approximate procedure mentioned.

DESIGN SPECTRA

A decision criteria based on the maximum expected value of actualized utilities is adopted here. Rosenblueth and Mendoza[2] have shown that when the policy of reconstructing the structure under the original specifications is followed after each failure the expected actualized cost of failure is equal to $D_o p^*$, where p^* is called the equivalent failure probability and is given by

$$p^* = E_S[\nu_F/(\nu_F + \delta)]/\left\{1 - E_S[\nu_F/(\nu_F + \delta)]\right\} \qquad (21)$$

Here, D_o is the cost of each failure, ν_F is the failure rate, which is imperfectly known as a result of uncertainty in structural parameters, δ is the actualization rate (interest+inflation) and E_S is an operator meaning expectation with respect to the distribution of the structural properties. Uncertainty in the seismicity parameters (i.e. in parameters of the $\nu(y)$ curve) can be accounted for by taking the expected value of p^* with respect to their probability distribution.

Results of applying this criterion to a structure having deterministic strength are shown in fig. 4 for the site whose seismicity under various assumptions concerning M_1 is shown in fig. 1. Bayesian probabilities of 0.15, 0.80 and 0.05 were assigned respectively to M_1 equal to 6.0, 6.5 and 8.0. Initial cost was supposed to vary with the design intensity in the proportion $1 + ky^2$, with y in cm/sec^2. Optimum design accelerations are presented in terms of $B/\delta k$ and of B alone for $\delta = 0.10$ and $k = 1.1 \times 10^{-6}$. Here, B is the ratio of the expected cost of failure to the initial cost. It is clear from the figure that for high values of B design accelerations should be higher than their expected value for the most unfavorable assumption concerning magnitude and location, but that those design accelerations grow at a slower rate for extremely high B values.

For random structural properties the distribution of ν_F can be obtained from that of the structural parameters: let $S(T_o)$ be the response spectral ordinate for a structure with natural period T_o. Let $S_F(T) = 4\pi^2 \mu D_E/aT^2$ be the failure spectral ordinate for the actual structure, whose capacity μD_E and natural period T are imperfectly known. Failure occurs every time that $S(T_o) > S_F(T)\,(\bar{S}_A(T_o)/\bar{S}_A(T))\,(T_o/T)^2$. The second member in this inequality will be called structural resistance and will be denoted by R. The natural logarithm of the ratio of this variable to its mean value will be assumed to possess a lognormal distribution $f_{R/\bar{R}}(.)$ with mean m_R and dispersion σ_R. If $\nu_S(y)$ is the rate at which $S(T_o)$ exceeds y, $\nu_F = \nu_S(R)$. For small values of $\delta/\bar{\nu}_F$, p^* may be approximately obtained as $\bar{\nu}_F/\delta$. Thus, if $\nu_S(R) = K_S\,(1 - g(R))\,R^{-r}$, for a given $\bar{R}$

$$\bar{\nu}_F = K_S \bar{R}^{-r} \int (1 - g(u\bar{R}))\, u^{-r} f_{R/\bar{R}}(u)\, du \qquad (22)$$

After the approximation is made of taking $g(u\bar{R})$ in the integral as a constant equal to $g(\bar{R})$, this equation leads to

$$\bar{\nu}_F = \nu_S(\bar{R}) \exp\left(\frac{1}{2} r^2 \sigma_R^2 - r m_R\right) \qquad (23)$$

In the formulation of seismic design recommendations uncertainty in the natural period is ordinarily accounted for by increasing the ordinates of the response spectrum for a given return period in the ranges adjacent to the peaks, while uncertainty in lateral structural strength is accounted for by specifying reduced or characteristic strength values. Following this idea, it is of interest to compute the spectral response S_1 of the structure of uncertain period but deterministically known strength that corresponds to the same return period as $S(T_o)$. This response is obtained from the condition.

$$S_1^{-r}\,[1 - g(S_1)]\,\exp\left(\frac{1}{2}r^2\,\sigma_R^2 - rm_R\right) = S^{-r}(T_o)\,[1 - g(S(T_o))] \qquad (24)$$

Thus, for a given return period the design spectrum should be obtained by multiplying by $S_1/S(T_o)$ the spectral ordinate for each nominal natural period.

CONCLUDING REMARKS

Evaluation of seismic reliability for given structures requires analysis of uncertainty stemming from those sources related to estimation of seismicity and from those proper of the structures themselves. Since most of the uncertainty is associated with seismicity and attenuation laws, at least rough reliability analyses are convenient when specifying design requirements on the basis of local geology and statistical data. Extensive efforts in the quantitative analysis of the physical processes related to accumulation and liberation of energy in the earth's crust are a primary need in the development of more reliable criteria for seismic risk estimation.

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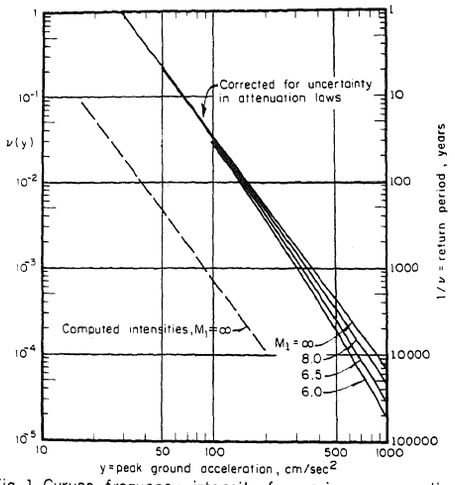
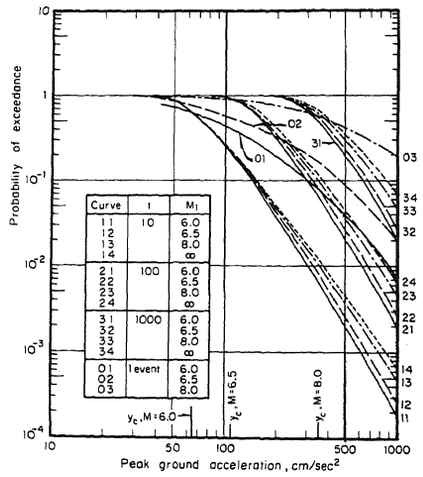


Fig 1 Curves frequency-intensity for various assumptions about M_1



Curve	I	M_1
11	10	6.0
12		6.5
13		8.0
14		∞
21	100	6.0
22		6.5
23		8.0
24		∞
31	1000	6.0
32		6.5
33		8.0
34		∞
01	1 event	6.0
02		6.5
03		8.0

Fig 2 Probabilities of exceedance of different intensities

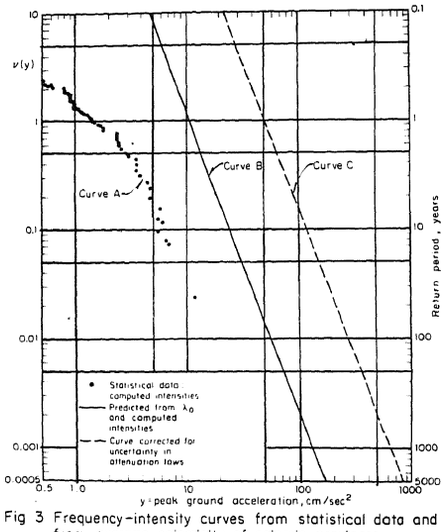


Fig 3 Frequency-intensity curves from statistical data and from average seismicity of seismic provinces

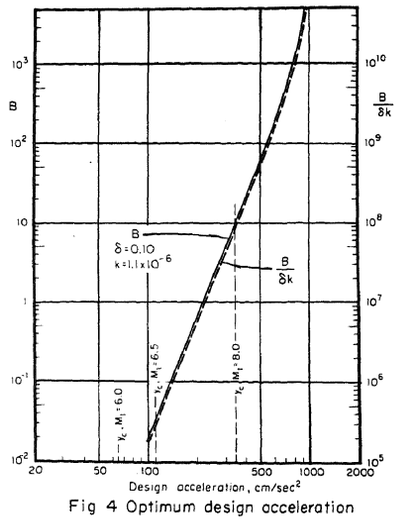


Fig 4 Optimum design acceleration

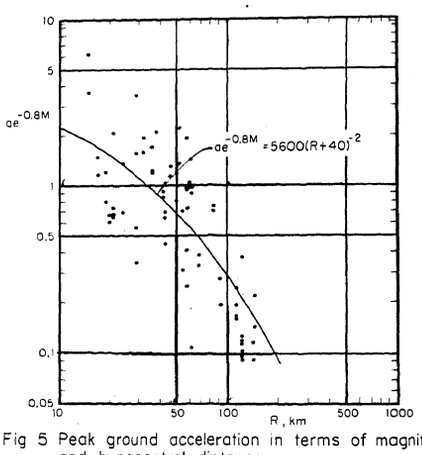


Fig 5 Peak ground acceleration in terms of magnitude and hypocentral distance

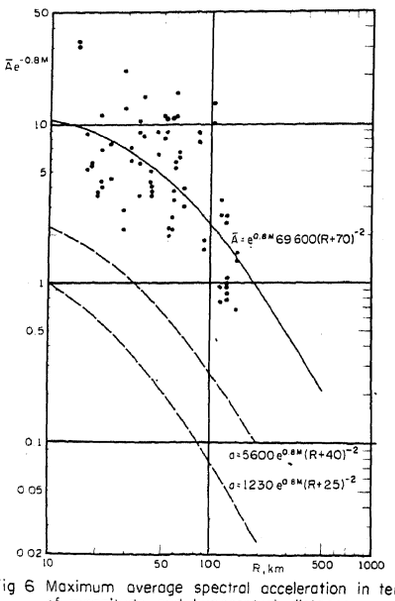


Fig 6 Maximum average spectral acceleration in terms of magnitude and hypocentral distance

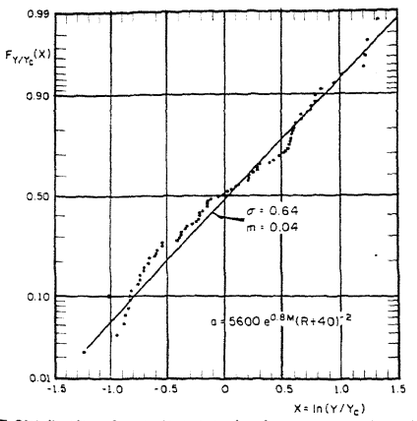


Fig 7 Distribution of error in expression for peak ground acceleration

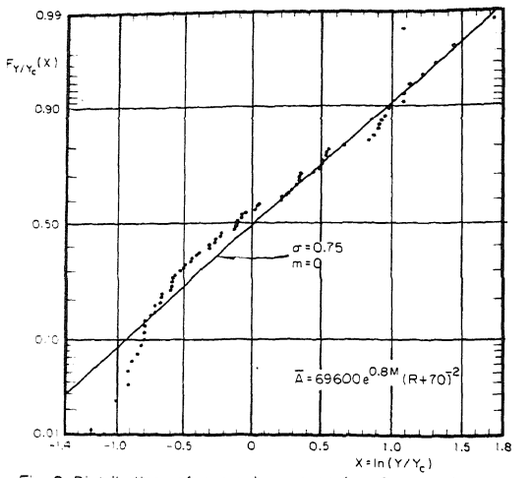


Fig 8 Distribution of error in expression for maximum average spectral acceleration

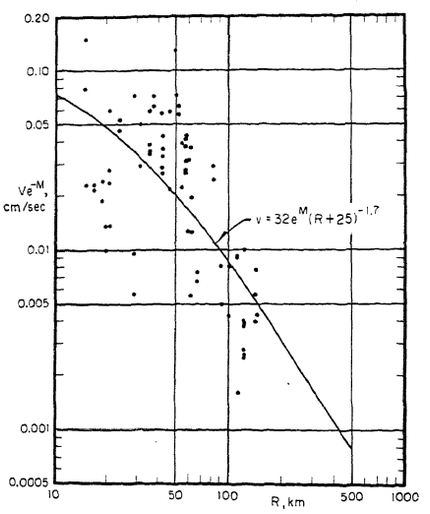


Fig 9 Peak ground velocity in terms of magnitude and hypocentral distance

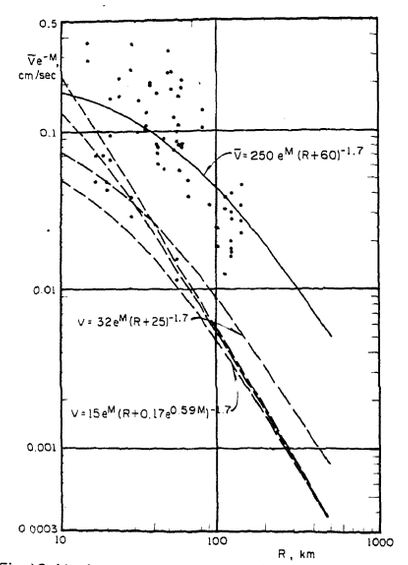


Fig 10 Maximum average spectral velocity in terms of magnitude and hypocentral distance

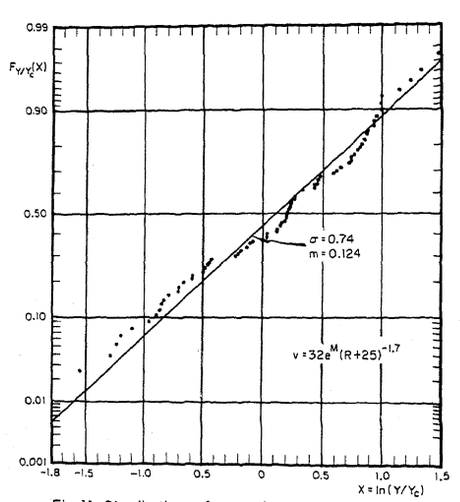


Fig 11 Distribution of error in expression for peak ground velocity

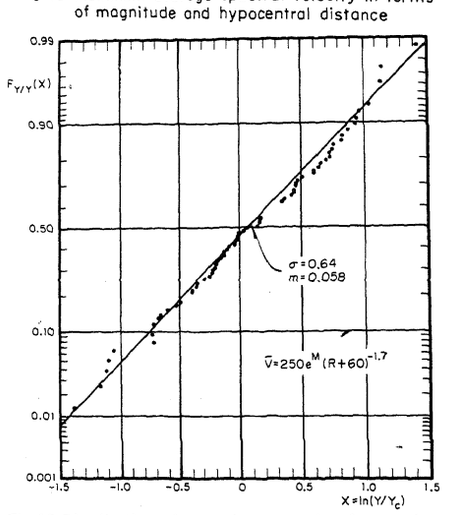


Fig 12 Distribution of error in expression for maximum average spectral velocity