MAXIMA EARTHQUAKES FOR SEISMIC DESIGN OF STRUCTURES

L E Pérez Rocha [1] and M Ordaz Schroeder [2]

[1] *Researcher, Instituto de Investigaciones Eléctricas, Cuernavaca, Mor, México.* lepr@iie.org.mx

[2] *Researcher, Instituto de Ingeniería,UNAM, México* mors@pumas.iingen.unam.mx

ABSTRACT :
A use of optimal design was done to determine the level of security that has to be yielded in the new version on the Seismic Design Chapter of the Mexican Handbook of Civil Constructions. The test of the optimization was done for essentially short-period conventional building structures (called typical of the Group B or TGB). Optimal design coefficients, as well as the correspondent return periods, were obtained for the whole Mexican territory. It was found that in low seismicity zones, these coefficients can be associated to very long return periods, with low probabilities of occurrence. Maxima earthquakes were invoked in order to examine the intensities. Both probabilistic and deterministic maxima earthquakes were useful to identify the maxima intensities that is possible to obtain in a classical hazard approach. Attenuation laws with truncated distribution were used to account for the physical inherent saturation.

KEYWORDS:
Seismic hazard, Optimal design, maximum earthquake, seismic zone, return period, peak ground acceleration

1. INTRODUCTION

It was made use of the optimal design to establish the lateral strength levels of design in the new version of the Seismic Design Chapter of the Mexican Handbook of Civil Constructions (MDOC by its name in spanish Manual de Diseño de Obras Civiles). The optimization exam was done for Typical structures of the Group B (TGB), that essentially correspond to short period buildings. Optimal design coefficients, as well as the correspondent return periods, were obtained for the whole Mexican territory, and were provided by means of maps in the MDOC. It was found that, in low seismicity zones, these coefficients are associated to very long return periods. For structures located in low seismicity zones, some undesirables consequences can be pointed out: For redundant and robust structures, such as the TGB, the gravity design would usually lead to seismic strength higher than the recommended design values, therefore designing for actions with very long return period would not have economic implications of importance (it would be buying security for lateral load at very low cost). However, for structures with less reserve than the TGB (i.e. pendular structures or flexible bottom floor structures) vertical loading design would yield low seismic resistance, so it would have to make seismic design by using optimal coefficients associated with very long return periods. Its economic consequences could be important. In these low seismicity zones, the optimal design coefficients match with unrealistic earthquakes.

2. SEISMIC HAZARD IN MEXICO

The recent advance in seismology and engineering has significantly contributed to the knowledge of seismic hazard in Mexico, especially in the next topics: The geometry of the Cocos Plate in its portion subducted beneath the continental plate of North America was improved. Better attenuation laws for intermediate depth earthquakes have been made due to great activity of this kind of earthquakes in the last few years. New attenuation laws for crustal earthquakes that include data of a great number of recorded events in several places of USA, especially in California, have been developed in the last years. These laws seem to be adequate for some earthquakes that occur in Mexico. The procedure to compute the seismic hazard is described in what follows.

2.1. Local Seismicity Models

The Mexican Republic has been divided in 43 sources of earthquakes. There are four possible origins for earthquakes in Mexico: subduction earthquakes, normal faulting an intermediate depth earthquakes, shallow earthquakes of the continental crust and earthquakes of the Polochic-Motagua fault system. These sources obey the regional tectonic process and are reflected by the instrumental history of the recorded earthquakes. Each one of these sources generates earthquakes with a constant rate per unit area. The activity of the i-th seismic source is specified in terms of the exceedance rate of magnitudes there generated, $\lambda_i(M)$. The exceedance rate of magnitudes measures how frequent earthquakes whit magnitude greater than a given one are generated.

2.1.1 Sources governed by the modified Gutenberg–Richter relation

For the most of the seismic sources, the function $\lambda(M)$ is a modified version of the Gutenberg and Richter relation. In these cases, the seismicity is described by the next form:

$$\lambda(M) = \lambda_0 \frac{e^{-\beta M} - e^{-\beta M_u}}{e^{-\beta M_0} - e^{-\beta M_u}}; \qquad M_0 \le M \le M_u \tag{2.1}$$

where M_0 is the minimum relative magnitude and λ_0, β, and M_u are parameters that define tha exceedance rate of each one of the seismic sources. These parameters, different of each source, are estimated by statistical Bayesian procedures (Rosenblueth y Ordaz, 1989), that include information about regions tectonically similar to Mexico, and expert information, specially about the values, the maximum values that can be generated in each source.

2.1.1 Sources governed by the characteristic earthquake model

The functional form of $\lambda(M)$, given in the equation 2.1, is used for the most of the seismic sources. However, it has been observed that the distribution of magnitudes of great subduction earthquakes (M>7) is far from the one predicted by the Gutenberg and Richter relation, causing the characteristic earthquake concept. For great, subduction earthquakes, $\lambda(M)$ is defined as:

$$\lambda(M) = \lambda_0 \frac{\Phi\left[\frac{M_u - E(M)}{\sigma}\right] - \Phi\left[\frac{M - E(M)}{\sigma}\right]}{\Phi\left[\frac{M_u - E(M)}{\sigma}\right] - \Phi\left[\frac{M_0 - E(M)}{\sigma}\right]}; \qquad M_0 \le M \le M_u \tag{2.2}$$

where λ_0, □ $E(M)$, and σ are parameters that have to be obtained statistically for the Mexican subduction zone, and $\Phi(\cdot)$ is the normal standard distribution function.

2.2. Attenuation laws

After the activity rate of each one of the seismic sources is determined, it is necessary to evaluate the effects (in terms of seismic intensity) that each source produces in a specific site (supposedly located at firm ground). For this purpose, knowing what intensity would take place in the site if an earthquake with a given magnitude occurs in the i-th source, is required. Equations that relate magnitude, site-source distance and intensity are known as attenuation laws. It is considered that the seismic intensities of interest are the ordinates of the response spectrum Sa (pseudoaccelerations, 5% of critical damping), quantities that are approximately proportional to the lateral inertial forces generated in the structures during earthquakes and depend on the natural period of vibration.

Three attenuation laws dependent of the path from the source to the site are used in this study in a spectral scheme to account for the fact that the attenuation is different for waves of different frequency, so there are parameters for each vibration period. These laws are: costal earthquakes (Ordaz et al, 1989), intermediate depth earthquakes (García et al, 2005) and superficial earthquakes (Abrahamson and Silva, 1997).

2.3. Seismic hazard assessment

Given magnitude and epicentral distance, the seismic intensity cannot be considered deterministic because it is not free of uncertainties. it is often to suppose that, given magnitude and distance, the intensity Sa is a random variable lognormally distributed, with median $A_m(M,R)$, given by the attenuation law and typical deviation of the natural logarithm equal to $\sigma_{\ln A}$. For instance, if an earthquake with magnitude M and distance R has occurred, the probability that the spectral acceleration S_A would be greater than a given value, Sa,

$$\Pr(S_A > Sa \mid M, R) = 1 - \Phi\left[\frac{1}{\sigma_{\ln A}} \ln \frac{Sa}{A_m(M,R)}\right]; \qquad A \geq 0 \tag{2.3}$$

Once the seismicity of the sources and the attenuation patterns of the generated waves in each one of them are known, the seismic hazard can be computed regarding the sum of the effects of all the sources and the distances between each source and the referred site. The hazard, $\nu(Sa)$, expressed in terms of the exceedance rates of intensities Sa is computes as is indicated in what follows:

2.3.1 Basic Equations

In this study, seismic sources are areas, where a spatial integration process to account for all possible focal locations is performed. Generally it is assumed that, in a seismic source, all point have the same probability to be an epicenter (constant seismicity per unit area). In this case, the rates of exceedance of acceleration due to one seismic source (the i-th) are computed with the next equation (Esteva, 1967):

$$\nu_i(u) = \sum_j w_{ij} \int_{M_0}^{M_u} \left(-\frac{d\lambda(M)}{dM}\right) \Pr(S_A > Sa \mid M, R) dM \tag{2,4}$$

where j is the index for each one of the sub-elements in what the source has been divided M_0 and M_u are the minimum and maximum magnitudes considered in the analysis, $\Pr(S_A > u \mid M, R_{ij})$ is the probability that the acceleration exceed the value u in the site, given that an earthquake with magnitude M is generated at the distance R_{ij}, and R_{ij} are the distances between the site and the sub-element j of the source i. A weight w_{ij} for each sub-element is assigned, which is proportional to its size. The term $\Pr(S_A > u \mid M, R_{ij})$ is computed as pointed in equation 3. Finally, the contributions of all sources - N - are summed to the seismic hazard of the site. This analysis is performed for several structural periods:

$$\nu(u) = \sum_{i=1}^{N} \nu_i(u) \tag{2.5}$$

3. OPTIMAL DESIGN

In the last decades, in Mexico as in other parts of the World, it has been stipulated design criteria in which, besides the hazard, economical issues are taken into account. It has given place to the optimal design. A design value is optimal if minimize the sum of the present value of the expected losses due to earthquakes and the initial cost of construction. It is supposed that the expected losses due to earthquakes as well as the initial cost of construction depend on a single parameter: the nominal design strength, expressed in terms of the base shear force. As a consequence, the optimal values are no associated to a constant return period. In fact, the optimization leads to a situation that is instinctively correct: in low seismicity zone, where the design value to resist lateral load is relatively cheap, designing to return periods greater than the ones used in the highest seismicity zones is optimal. According to Esteva (1970), it is considered that a design coefficient is optimal if it minimizes the sum of the expected cost of the decision of having used precisely this design value. The expected costs are formed by two components: the initial cost, that grows as the adopted design value grows, and the updated to the present value of the cost of the all losses due to earthquakes that can be occur in the future.

3.1. Initial cost

The next variation of the initial cost of construction $CI(c)$, with the design coefficient c is adopted:

$$CI(c) = \begin{cases} C_0 & si\, c < c_0 \\ C_0 + C_R(c - c_0)^\alpha & si\, c \geq c_0 \end{cases} \tag{3.1}$$

where C_0 is the cost that it would have even when it does not be designed to resist lateral loads, c_0 is the lateral strength that it would have in this case and C_R and α are coefficients. If equation 3.1 is normalized with respect to C_0, it can be written that

$$\frac{CI(c)}{C_0} = \begin{cases} 1 & si\, c < c_0 \\ 1 + K(c - c_0)^\alpha & si\, c \geq c_0 \end{cases} \tag{3.2}$$

3.2. Present value of the expected losses due to earthquakes

As initial model, it is supposed that each time that the design strength, c, is exceeded, it will have a total loss of the structure, This model is, evidently, very simple. The real strength of a structure is, in general terms, uncertain but with a median higher than the nominal of design. So, when the nominal design strength is exceeded, it is not necessary that a total loss would be presented, only can be given probabilistic asseverations about the value of the loss. On the other side, it is conceivable that partial failures would be presented even when the demand does not exceed the nominal strength. This would make mandatory the formulation of vulnerability relations and their formal inclusion in the losses assessment.

However, as it will be shown, the optimization asses will be done just to determinate relative levels of expected costs between buildings in different parts of the country. This is why it was believed that the use of a more detailed model was would not yield substantial improvements.

According to Rosenblueth (1976), if it is supposed that the occurrence procedure is Poissionian, and if the updating of the money value is adequately described by an exponential function, the present value of the expected losses, $EVP(c)$, when it is designed for a strength c is:

$$EVP(c) = CP(c)\frac{\nu(c)}{\mu} \tag{3.3}$$

where $CP(c)$ is the cost of the loss due to earthquake, μ is the discount rate of the value and $\nu(c)$ is the exceedance rate of the demand that produce the failure when it has been designed for a strength c. s it was pointed out by Ordaz et al (1989), the cost of the loss is not only the value of the damaged constructions, because the loss of the buildings affects the work of the economy such that, in general, the total losses are greater than the purely material losses. To account for this effect, it is proposed in the cited work that

$$CP(c) = C_I(c)(1 + S_L) \tag{3.4}$$

where $C_I(c)$ is the initial cost, given in the equation 3.1 and S_L is a factor that measures the importance of the losses constructions. It is had that

$$EVP(c) = C_I(c)(1 + S_L)\frac{\nu(c)}{\mu} \tag{3.5}$$

4. **OPTIMAL SPECTRA FOR STRUCTURES OF THE GROUP B**

The intention of this phase of the study was not to carry out meticulous calculations to rigorously determine the values of the optimal coefficients of design. The objectives of the calculations were the following: Supposing that the coefficients of design in zone D of the MDOC-DS (1993) are optimal for structures of located group B for sites in the Pacific Coast, how much the coefficients of optimal design would have to value in the rest of the country? This it is, in essence, the approach adopted by Esteva and Ordaz (1988).

It will be assumed that for Typical structures of Grupo B (TGB) of short period ($<0.3\text{seg}$), in firm ground, the level of the plateau of the spectrum for zone D of the MDOC-DS (1993) leads to optimal design in the Pacific Coast. It can be appreciated that the resistance to which the simplified model for the present value of the expected losses by earthquake is the real strength, whereas the ordinates of the spectrum of the MDOC-DS (1993) are reduced by the effect of the overstrength. Therefore, it is necessary to turn the spectra of the MDOC-DS (1993) to real strengths. For this, an overstrength factor of 2 has been used, that is reasonable for a wide group of structures. In view of this, the implicit real strength in the plateau of the spectrum of zone D of the MDOC-DS (1993) is $c = 0.5 \times 2 = 1$. With these adoptions and using the seismicity of a representative point of the Pacific Coast (the selected site was Acapulco and the spectral ordinate was measured in $T_e = 0.3$ s) and the value of $S_L = 12$ used in the MDOC-DS (1993) (Ordaz, et al, 1989), it was determined what values of K and α ead to the conclusion that the value $c = 1$ is optimal in that site. It was done by iterations, obtaining the following values: $K = 1.6$ and $\alpha = 2$, that are not very different from the adopted ones by Ordaz, et al (1989) for the determination of design spectra in the MDOC-DS (1993). Once the values of K and α were determined, the values of the optimal coefficients in the rest of the country were determined.

A consequence of the application of optimization criteria as the one described herein is that the optimal values are not associate to a constant return period. The optimization leads to a situation that is correct: in low seismicity zones, where the design to resist lateral load is relatively cheap, it is optimal to design for longer return periods than those that would be used in zones of greater seismicity. For the zones of greater seismicity of Mexico, the calculated optimal return periods are between 250 and 500 years, whereas after the zones of smaller seismicity, the values reach the 3.000 years. Said in other words, in low seismicity zones, where seismic coefficient c is associated to very great return periods, the security is cheap and it must be bought. In figure 1, a map of Peak Ground Accelerations (PGA) associated to the optimal periods of return for this set of structures is shown. These accelerations, which vary between 0.06 and 0.5 g, will serve as reference for the examination of the accelerations produced by maxima earthquakes. The values for five cities are compared more ahead.

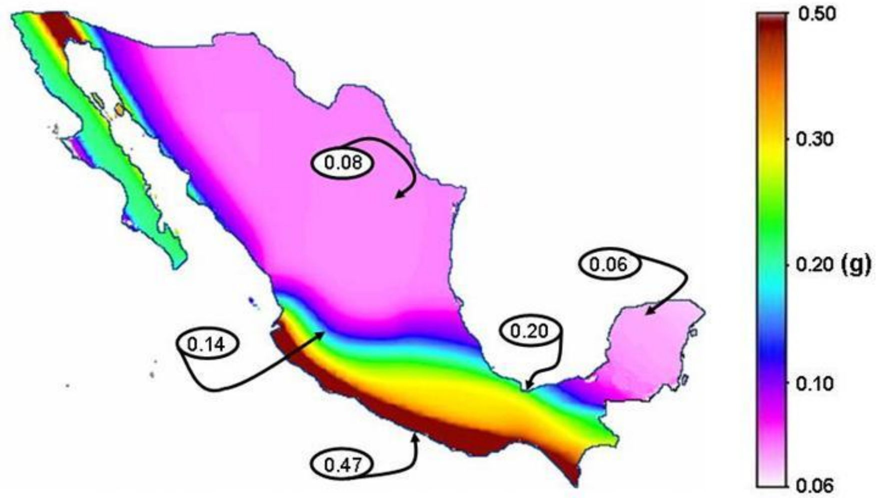

Figure 1. PGA associated to the optimal return periods of TGB structures

5. MAXIMA EARTHQUAKES

As it was commented, it is not optimum to design for a constant return period in zones of high and low seismicity; at least partially, seismic regionalizations are an approach towards the optimal design. Nevertheless, it is in these zones of low seismicity in which the optimal design provides design coefficients that can be result excessively conservative for structures that are more vulnerable to lateral lading that those that served as reference to determine the optimal coefficients, in this case structures type building of short period (TGB, 0.3 s).

Why it is considered that the optimal coefficients in zones of low seismicity can excessively be conservative? First, because the fact that they are associated to extremely long return periods (several thousands of years). Second, because it is difficult to imagine a realistic earthquake that would produced them. The following question fits: if it is difficult to imagine an earthquake that produces them, why its period of return is of several thousands of years and not infinite? The reason of this, as it is seen in the section corresponding to maximum probabilistic earthquakes, is the way in which the uncertainty in the attenuation laws is modeled. In figure 1 it is indicated that for low seismicity zones, the peak ground acceleration is 0.08 g. Since the optimal design leads to buy security in low seismicity zones, then it is probable that a paradoxical situation would be had: as the observer moves away of the high seismicity zones, the intensity of the optimal design is reduced more slowly than the one due to the maximum earthquake that is reasonable to imagine. It is expected that in high seismicity zones, the optimal coefficients would be minors than those corresponding to maxima earthquakes. In low seismicity zones, there is the opposed situation. So far, we have referred about the maximum earthquake or the maximum earthquake that is reasonable to imagine without defining it. It will be illustrated in the following paragraphs the difficulties that there are to define the maxima earthquakes.

5.1 Maximum determinist earthquake

It is possible to be asserted that the maximum possible earthquake or maximum determinist earthquake is that one that is located to the minimum possible distance of the interest site and with the maximum possible magnitude. Even supposing that the minimum distance as well as the maximum possible magnitude can be known in a determinist way (supposition very doubtful), is impossible to ignore the uncertainty in the intensity that will appear given to magnitude and distance. It is assumed that given these two amounts, the intensity Sa is a variable lognormally distributed, with median $A_m(M,R)$ - given by the law of attenuation that is being used and standard deviation of natural logarithm equal to $\sigma_{\ln A}$. For the maximum possible earthquake, it is common to suppose that the intensity that will appear is not the median, but a high percentile, defined almost always of arbitrary way. The intensity associated to the maximum determinist earthquake can be calculated as:

$$\mathrm{Sa} = \mathrm{A_m}\left(\mathrm{M},\mathrm{R}\right)\exp(\varepsilon\sigma_{\ln \mathrm{A}}) \tag{5.1}$$

where ε determines what percentile is being used. For example, for a lognormal distribution, percentile 84 is associated to ε=1. As it can be appreciated, the intensity associated to the maximum possible earthquake cannot be associated to any period of return.

5.1 Maximum probabilistic earthquake

The maximum determinist earthquake is, according to the authors, an inadequate way to characterize the earthquake greater than it is reasonable to imagine. Let consider, two seismic sources able to produce the same maximum possible magnitude, located both to the same possible minimum distance of the interest site. In agreement with the given definition of maximum possible earthquake, both would generate the same maximum possible intensity, independently of the frequency whereupon earthquakes with magnitudes near to the maximum possible one are generated. In other words, although one source could be 100 times more active than the other, both would generate the same maximum possible intensity, which seems absurd for design purposes.
In order to correct this deficiency of the maximum determinist earthquake, we will prefer the definition of the maximum probabilistic earthquake, or maximum probable earthquake. We will say that the maximum probable intensity is that one that, as a result of a formal examination of seismic hazard, is associated to a specified return period, sufficiently long. Nevertheless, in the formulation that has presented in this work, that is the one that was used in the calculations for the MDOC-CFE, still outlandishly great intensities have finite return periods, since, in view of which a lognormal distribution was assigned to the intensity (given M and R), the probability of having an unreally great intensity, given any pair of values of, M and R, it is not null. To correct this problem, it is set out to use truncated lognormal distributions to the intensities (conditional to M. and R). In other words, given M and R, it will be assumed that the probability of having intensities higher than the given one in equation 5.1 is zero. For finite return periods, as the ones used for seismic design of structures, the definition of maximum probabilistic earthquake takes into account the effect of different activity rate of the sources.

6. RESULTS FOR PEAK GROUND ACCELERATION

In order to quantify the intensities produced by maximum determinist earthquakes, the maximum intensity produced by each source in each node of the proposed mesh was determined using equation 5.1. For it, the value

ε=1 was fixed and it was made use of the attenuation law corresponding to each examined source, using the minimum distance and the maximum magnitude. The maximum value was taken after evaluating the intensity of all the considered sources. The map of PGA due to maximum deterministic earthquakes appears in figure 2. In this figure the accelerations that would be had in five cities are indicated (in parts of the gravity - g). These cities are: Acapulco (1.0), Coatzacoalcos (0.29), Guadalajara (0.43), Monterrey (0.13) and Mérida (0.033).

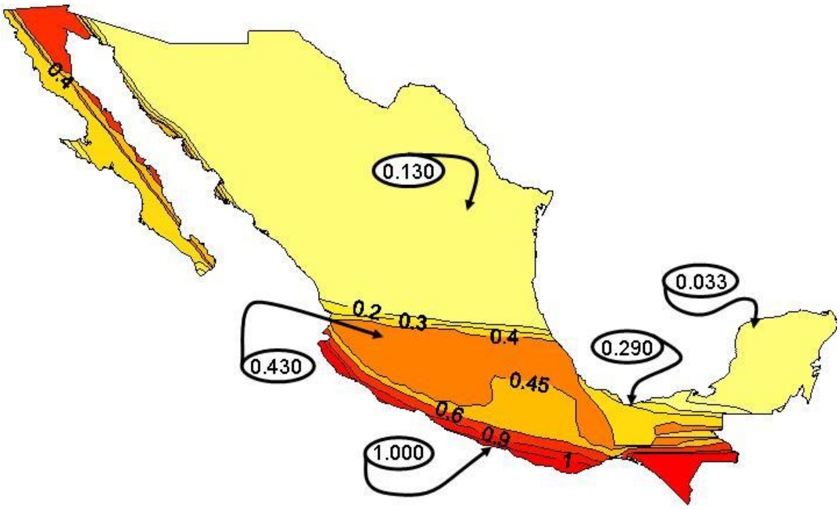

Figure 2. PGA produced by the maximum deterministic earthquake with truncated lognormal distribution (ε= 1)

For the probabilistic earthquakes it was specified that the distributions of the intensities predicted by the attenuation laws are lognormals, truncated with ε=1. PGA due to maximum probabilistic earthquakes for a return period of 6500 years appears in figure 3. For the selected cities the following intensities are had: Acapulco (0.601), Coatzacoalcos (0.129), Guadalajara (0.121), Monterrey (0.040) and Merida (0.012).

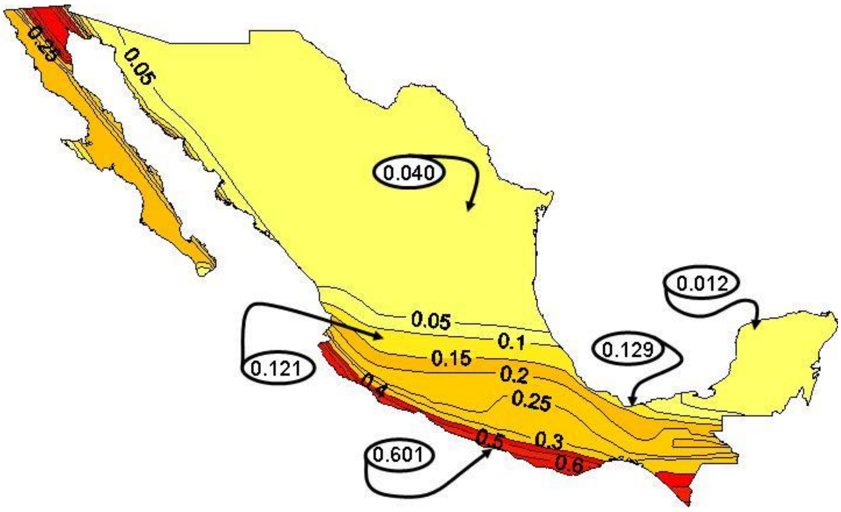

Figure 3. PGA produced by the maximum probabilistic earthquake with truncated lognormal distribution (ε= 1)

Finally, figure 4 shows PGA corresponding to optimal return periods obtained from seismic hazard curves that were constructed with attenuation laws with truncated distribution (ε= 1). Compare the values that are reported in figures 1, 2 and 3 with the ones reported in figure 4. For high seismicity zones, the peak values correspond to the optimal design, whereas in low seismicity zones, the peak values correspond to values lower than the ones displayed in figure 1, obtained with attenuation laws with lognormal distribution without truncation.

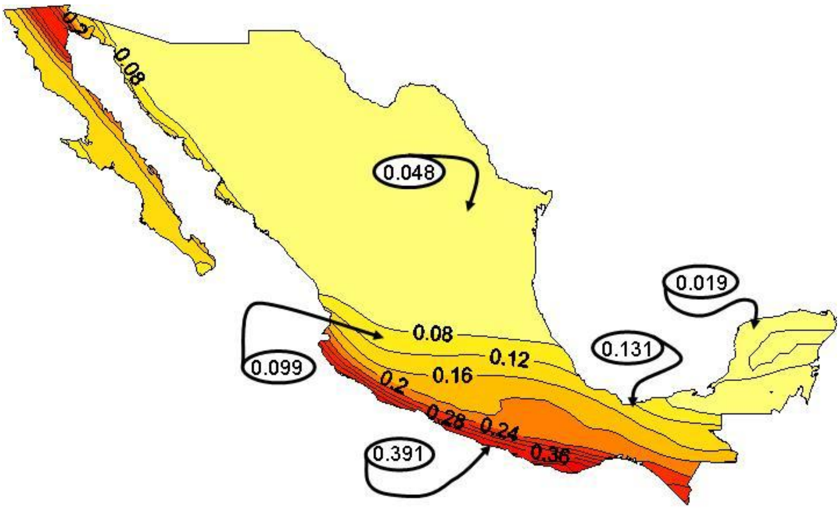

Figure 4. PGA associated to the optimal return periods (as fig 1) using truncated lognormal distribution (ε= 1)

7. CONCLUSIONS

It has been made to see that the optimal design leads to coefficients that are excessively conservative for certain structures in low seismic zones. The reason that we consider them excessively conservative is that they correspond to earthquakes difficult to imagine. In order to correct this problem, maximum deterministic and probabilistic earthquakes were defined. It was shown that in two sites in identical conditions, that only differ in the seismicity, the same maximum determinist earthquakes are present, and that are the maximum probabilistic earthquakes those that differ. It was discussed that the maximum determinist earthquake is the higher level and it is not related to any return period. Also, it was shown that the maximum probabilistic earthquake, calculated with truncated distribution, tends asymptotically to the value corresponding to the maximum deterministic earthquake, when the return period tends to infinite. Maps computed with attenuation laws with truncated lognormal distribution were used to show PGA for deterministic, probabilistic and optimal earthquakes. The last is close to the one that will be used in the next version of the Seismic Design Handbook

REFERENCES

MDOC-DS (1993). Capítulo de Diseño por Sismo del Manual de Obras Civiles, Comisión Federal de Electricidad.

Rosenblueth E. y Ordaz M. (1989). Maximum earthquake magnitude at a fault, *Jour of the Eng Mech Div*, ASCE, **116**, 204-216.

Rosenblueth, E. (1976), Optimum design for infrequent disturbances, *Jour Struct Div*, ASCE, **102**, 1807-1825.

Ordaz M., Jara J.M. y Singh S.K. (1989). Riesgo sísmico y espectros de diseño en el estado de Guerrero. Joint report of the II-UNAM and the Seismic Investigation Center AC of the Javier Barros Sierra Foundation to the Government of the state of Guerrero, Instituto de Ingeniería, UNAM, projects 8782 y 9745, Mexico.

García, D., Singh, S.K., Herráiz, M., Ordaz, M. y Pacheco, J. (2005). Inslab earthquakes of Central Mexico: peak ground-motion parameters and response spectra, *Bull. Seism. Soc. Am.*, **95**, 2272-2282.

Abrahamson, N.A. and Silva W.J. (1997). Empirical response spectral attenuation relations for shallow crustal earthquakes, *Seismological Research Letters* 68, 94-127.

Esteva, L. (1967). Criterios para la construcción de espectros para diseño sísmico, *3er Panamerican Simposium of structures*, Caracas, Venezuela.

Esteva, L. (1970), Regionalización sísmica de México para fines de ingeniería, *Serie Azul* de II, 246.

Esteva L and Ordaz M. (1988) Riesgo sísmico y espectros de diseño en la República Mexicana. *Proc Nat Simp on Earthquake Engineering*, Guadalajara, México.