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A NEW COMPUTATION OF SEISMIC HAZARD FOR MEXICO AND ITS APPLICATION IN SEISMIC OPTIMAL DESIGN

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Abstract

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. Some 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 accepted worldwide in the last years. These laws seem to be adequate for some earthquakes that occur in Mexico. The formulation to quantify the seismic hazard and their main results, corresponding to exceedance rates of peak ground acceleration and the ones of several structural periods, are shown is this work.

As well, a formulation based on the total cost of construction is applied to determine the optimal seismic design coefficient for the collapse limit state. This cost is the sum of the initial cost and the losses occurred during the life of a structure. This approach yields a continuous descriptor of design seismic intensities along the Mexican territory. As an example, a set of expected spectral accelerations maps are illustrated. These maps show acceleration for typical structure periods. Besides, a seismic regionalization map based on peak ground acceleration values is shown. This map matches very well with the previous one, consisting in four seismic zones.

Keywords: Seismic Hazard; optimal design, seismic spectra

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 Hand-book of Civil Constructions (MDOC-DS 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 de-sign 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 earth-quakes 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 earth-quakes, normal faulting an intermediate depth earth-quakes, shallow earthquakes of the continental crust and earthquakes of the Polochic-Motagua fault sys-tem. 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:

$$\lambda(M) = \lambda_0 \frac{e^{-\beta M} - e^{-\beta M_u}}{e^{-\beta M_0} - e^{-\beta M_u}}; \quad M_0 \le M \le M_u \tag{1}$$

Where M_0 is the minimum relative magnitude and λ_0, β, and M_u are parameters that define the exceedance rate of each one of the seismic sources. These parameters, different of each source, are estimated by statistical Bayesian procedures [1,2], 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.

2.1.2 Sources governed by the modified Gutenberg–Richter relation

The functional form of $\lambda(M)$, given in the equation 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:

$$\lambda(M) = \lambda_0 \frac{\Phi\left[\frac{M_u - E(M)}{\sigma}\right] - \Phi\left[\frac{M - E(M)}{\sigma}\right]}{\Phi\left[\frac{M_u - E(M)}{\sigma}\right] - \Phi\left[\frac{M_0 - E(M)}{\sigma}\right]}; \quad M_0 \le M \le M_u \tag{2}$$

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 [3], intermediate depth earthquakes [4] and superficial earthquakes [5].

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,

$$\Pr(S_A > Sa \mid M, R) = 1 - \Phi\left[\frac{1}{\sigma_{\ln A}} \ln \frac{Sa}{A_m(M,R)}\right] \tag{3}$$

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 rates of exceedance 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 [6]:

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

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, $\Pr(S_A>Sa|M,R_{ij})$ is the probability that the acceleration exceed the value Sa 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 $\Pr(S_A>Sa|M,R_{ij})$ 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:

$$\nu(Sa) = \sum_{i=1}^{N} \nu_i(Sa) \tag{5}$$

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, economic 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 [7], 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:

$$CI(c) = \begin{cases} C_0 & \text{si } c<c_0 \\ C_0 + C_R (c-c_0)^{\alpha} & \text{si } c \geq c_0 \end{cases} \tag{6}$$

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 6 is normalized with respect to C_0, it can be written that

$$\frac{CI(c)}{C_0} = \begin{cases} 1 & \text{if } c < c_0 \\ 1 + K(c-c_0)^{\alpha} & \text{if } c \geq c_0 \end{cases} \tag{7}$$

Where $K=C_R/C_0$

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 would not yield substantial improvements. According to Rosenblueth [8], if it is supposed that the occurrence procedure is Poissonian, and if the updating of the money value is adequately described by an exponential function, the present value of the expected losses, EPV(c), when it is designed for a strength c is:

$$\mathrm{EPV}(c) = \mathrm{CP}(c)\frac{\nu(c)}{\mu} \tag{8}$$

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. It was pointed out by Ordaz [3], that 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

$$\mathrm{CP}(c) = \mathrm{CI}(c)(1 + S_L) \tag{9}$$

Where CI(c) is the initial cost, given in the equation 6 and S_L is a factor that measures the importance of the losses constructions. It is had that,

$$\mathrm{EPV}(c) = \mathrm{CI}(c)(1 + S_L)\frac{\nu(c)}{\mu} \tag{10}$$

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 [9] 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 [10].

It will be assumed that for typical structures of Grupo B (TGB) of short period (T_e<0.3 s), in firm ground, the level of the plateau of the spectrum for zone D of the MDOC-DS [9] 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 [9] are reduced by the effect of the over-strength. Therefore, it is necessary to turn the spectra of the MDOC-DS [9] to real strengths. For this, an over-strength 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 [9] is $c=0.5\times2=1$. 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 [3, 9], it was determined what values of K and α lead 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 α=2, that are not very different from the adopted ones by Ordaz,) [3]for the determination of design spectra in the MDOC-DS [9]. 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 Fig. 1, a map of Peak Ground Accelerations (PGA) associated to the optimal periods of return for this set of structures is shown. These accelerations vary between 0.05 and 0.5 g.

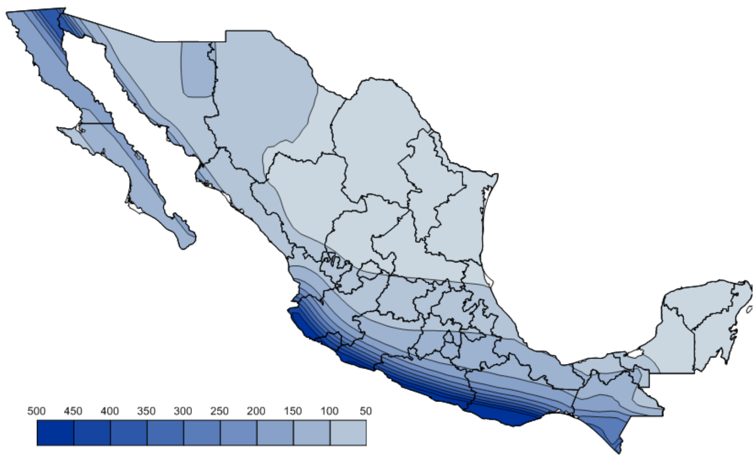

Fig. 1 – PGA associated to the optimal return periods of TGB structures

5 Seismic regionalization of mexico

5.1 Seismic regionalization

As observed, the seismic hazard intensity varies within the country in a continuum way, that is to say, the PGA values and the return periods associated vary throughout the territory. However, to define regional spectra is necessary to have a seismic regionalization map (Fig. 2). Following the history of the seismic design codes in Mexico, four zones are maintained in the map of seismic regionalization, two low seismicity zones and two high seismicity zones [9, 11]. In order to obtain the seismic zones, a criterion is proposed, based on the PGA (a_0^r), Table 1.

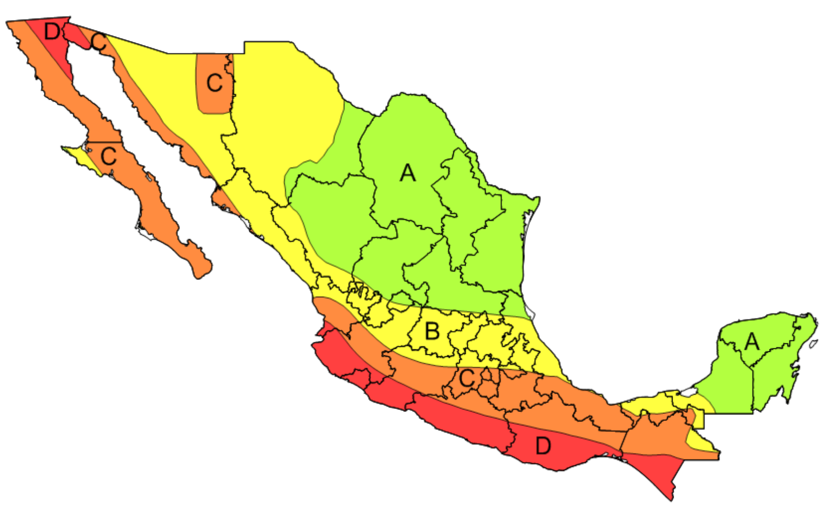

Fig. 2 – Seismic regionalization of Mexico

Table 1 – Regional limits

PGA associated to the optimal return periods of TGB structures, a_0^r (cm/s^2)	Zone	Seismic intensity
$a_0^r \geq 200$	D	Very High
$100 \leq a_0^r < 200$	C	High
$50 \leq a_0^r < 100$	B	Moderate
$a_0^r < 50$	A	Low

6 Soil type classification

The soil foundation is classified as (fig. 3):

Type I: Firm or rocky ground that no dynamic amplifications are presented: Soil deposit with $v_s \geq 720$ m/s or $H_s \leq 2$ m

Type II: Ground consisting in soils with intermediate dynamic amplifications: Soil deposit with $360 \leq v_s < 720$ m/s and $H_s > 2$ m, or $H_s > 30$ and $v_s < 720$ m/s

Type III: Ground consisting in soils with high dynamic amplifications: Soil deposit with $v_s < 360$ m/s and $2 < H_s \leq 30$ m

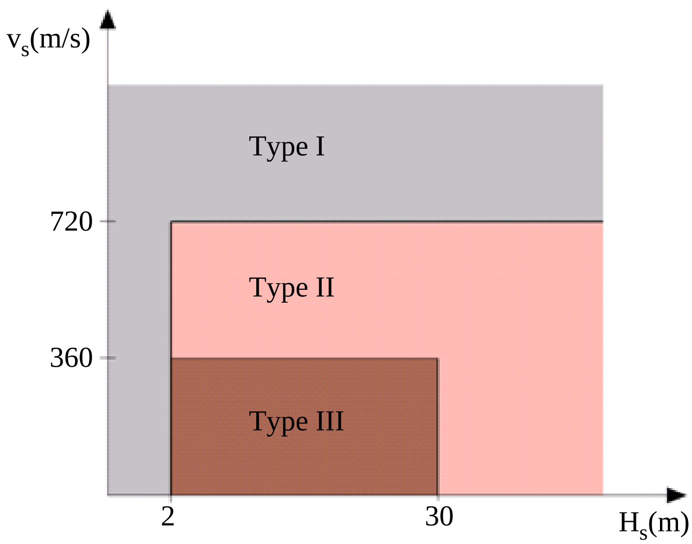

Fig. 3 – Soil classification chart

7 Regional design spectra for different types of soil

7.1 Constant acceleration spectrum

For small structures are not required to specify the type of soil. The recommended spectra for these structures are the most conservative, because they have application only for small structures (less than 13 m). In order to build these spectra is just enough getting the peak ground acceleration from the PRODISIS program (Fig. 1).

7.2 Regional transparent spectrum

In this case, the regional transparent spectrum has a parametric form, which is function of the structural period and the damping factor (Fig. 4). These spectra apply to any kind of structures except those have relation with the energy sector or they require high levels of security, for example, large dams, thermoelectric or nuclear plants. The main parameters of these spectra are the maximum acceleration modified by the site effects, a_0, the maximum spectral acceleration, c, the characteristic periods, T_a, T_b and T_c, and the r and k values that control the fall of the curves.

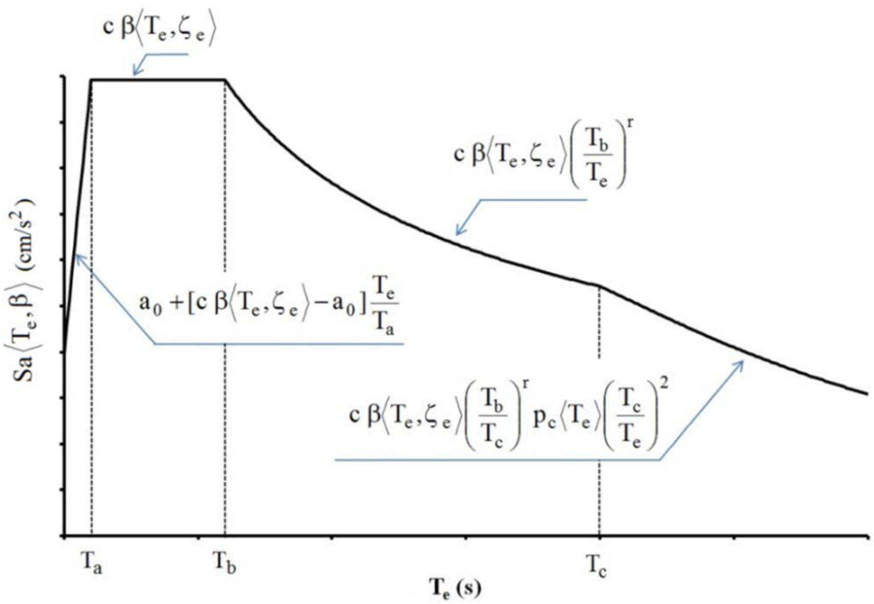

Fig. 4 – Regional transparent spectrum

7.2.1 Parameters for regional transparent spectrum

Based on the response spectra for several sites in the territory, Monte Carlo simulations and statistical analysis, the parameters for building of regional transparent spectra are shown in Table 2 and 3. For soil type I, the maximum ground acceleration, $a_0 = a_0^r$, and the maximum spectral acceleration, c, are obtained from the PRODISIS program. For soils type II and III are determined with:

$$a_0 = F_{Sit}\, a_0^r \tag{11}$$

$$c = F_{Res}\, a_0 \tag{12}$$

Where F_{sit} is the site factor and F_{Res} is the response factor. Both factors depend on the seismic zone, the peak ground acceleration, PGA, and the type of soil (Table 2 and 3).

Table 2 – Site factor for the different seismic zones and soils

	Soil type I	Soil type II	Soil type III
Zone A	$F_{Sit}=1.0$	$F_{Sit}=2.6$	$F_{Sit}=3.0$
Zone B	$F_{Sit}=1.0$	$F_{Sit}=2.6-0.2\left(a_0^r-50\right)/50$	$F_{Sit}=3.0-0.3\left(a_0^r-50\right)/50$
Zone C	$F_{Sit}=1.0$	$F_{Sit}=2.4-0.3\left(a_0^r-100\right)/100$	$F_{Sit}=2.7-0.4\left(a_0^r-100\right)/100$
Zone D	$F_{Sit}=1.0$	$F_{Sit}=2.1-0.5\left(a_0^r-200\right)/290$	$F_{Sit}=2.3-0.6\left(a_0^r-200\right)/290$

Table 3 – Response factor for the different seismic zones and soils

	Soil type I	Soil type II	Soil type III
Zone A	PRODISIS*	$F_{Res}=3.8$	$F_{Res}=4.2$
Zone B	PRODISIS*	$F_{Res}=3.8-0.2\left(a_0^r-50\right)/50$	$F_{Res}=4.2-0.3\left(a_0^r-50\right)/50$
Zone C	PRODISIS*	$F_{Res}=3.6-0.2\left(a_0^r-100\right)/100$	$F_{Res}=3.9-0.3\left(a_0^r-100\right)/100$
Zone D	PRODISIS*	$F_{Res}=3.4-0.5\left(a_0^r-200\right)/290$	$F_{Res}=3.6-0.6\left(a_0^r-200\right)/290$

Table 4 shows the rest of the parameters of the regional transparent spectrum. These spectra are conservative in order to protect the majority of soil conditions for each seismic zone. However, a site specific spectrum can be valid if rigorous methods are applied.

Table 4 – Characteristic periods and exponents that control the descending branches of the design spectra

Zone	Soil type	T_a(s)	T_b(s)	T_c(s)	k	r	Zone	Soil type	T_a(s)	T_b(s)	T_c(s)	k	r
A	I	0.1	0.6	2.0	1.5	1/2	C	I	0.1	0.6	2.0	1.5	1/2
	II	0.2	1.4	2.0	1.0	2/3		II	0.2	1.4	2.0	1.0	2/3
	III	0.3	2.0	2.0	0.5	1		III	0.2	2.0	2.0	0.5	1
B	I	0.1	0.6	2.0	1.5	1/2	D	I	0.1	0.6	2.0	1.5	1/2
	II	0.2	1.4	2.0	1.0	2/3		II	0.1	1.4	2.0	1.0	2/3
	III	0.3	2.0	2.0	0.5	1		III	0.1	2.0	2.0	0.5	1

8 Example

Obtain the regional spectrum for a structure of the Group B, located in Puebla city, in a soil type III. Fig. 5 shows the location of the site and the Peak Ground Acceleration at rock obtained with the PRODISIS program.

For the site of this example, the PGA at rock is $a_0^r = 116.82\ \mathrm{cm/s^2}$. According with table 1, the seismic zone is classified in terms of the PGA at rock, as follows:

$$100 \le a_0^r = 116.82\ \mathrm{cm/s^2} < 200\,\mathrm{cm/s^2}\ \therefore\ \text{This site is in seismic zone C – High seismic intensity}$$

The ground and spectral peak accelerations are determined using equations 11 and 12, respectively. The values of the site factor F_{Sit} and response factor F_{Res} have to be evaluated with the expressions of the tables 2 and 3, for zone C and soil type III, that is

$$F_{Sit} = 2.7 - 0.4\left(\frac{a_0^r - 100}{100}\right) = 2.7 - 0.4\left(\frac{116.82\,cm/s^2 - 100}{100}\right) = 2.6327$$

$$F_{Res} = 3.9 - 0.3\left(\frac{a_0^r - 100}{100}\right) = 3.9 - 0.3\left(\frac{116.82\,cm/s^2 - 100}{100}\right) = 3.8495$$

The values of F_{Sit} and F_{Res} are used in the equations for maximum ground acceleration, a_0, and maximum spectral acceleration, c,

$$a_0 = F_{Sit}\, a_0^r = 2.6327 \times 116.82 \;\; cm/s^2 = 307.5544 \;\; cm/s^2$$

$$c = F_{Res}\, a_0 = 3.8495 \times 307.5544 = 1{,}183.94 \;\; cm/s^2$$

Table 5 summarizes the parameters of design spectrum, which functional form is given in Fig 4.

Table 5 –Parameters of the design spectrum for a site in soil type III in Puebla city, seismic Zone C

Zona	Tipo de terreno	a_o (cm/s^2)	c (cm/s^2)	T_a (s)	T_b (s)	T_c (s)	k	r
C	III	307.6	1183.9	0.2	2.0	2.0	0.5	1

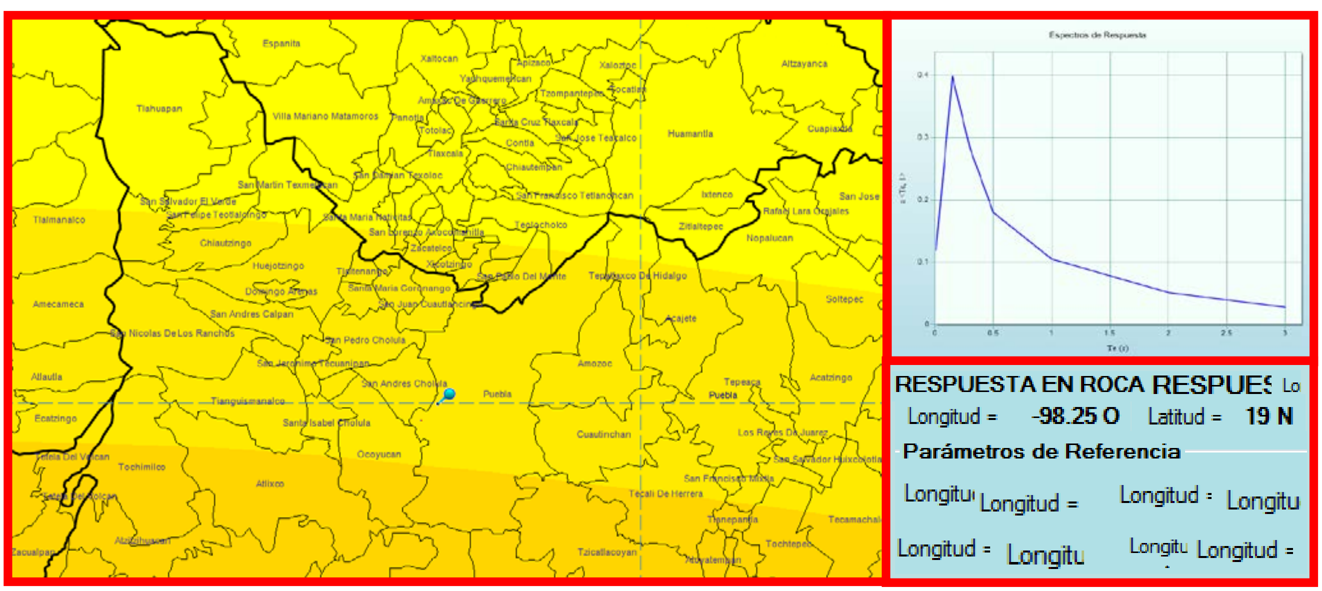

Fig. 5 – Location of the site, response spectrum at rock and reference values (PRODISIS Program)

8. Concluding remarks

This paper summarizes the most relevant changes in the seismic provisions given by the MDOC-2015 code for buildings in Mexico. Seismic hazard is defined as a continuum function where PGA at rock is defined. The PGA is associated with return periods obtained using an optimization design criterion to define the seismic coefficients for the plateaus of the elastic design spectra for standard occupancy structures. There is a great effort to make MOC-2015 seismic guidelines as conceptually transparent as possible in order to (a) clearly state the parameters that were taken into account to assess the earthquake hazard and define the elastic design spectrum and (b) define the sources that can be accounted for reducing the design spectrum for the collapse prevention limit state. Extensive commentary on the recommendations available in MDOC-2015 has been provided, with illustrations and in-depth references to the research studies that were consulted in updating the code. It is expected that the new MOC-2015 guidelines will help improve the seismic safety of new buildings in Mexico.

9. Acknowledgements

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10. Copyrights

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