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SEISMIC DESIGN AND CODES IN MEXICO

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SUMMARY

A brief description is given of the origins and evolution of seismic design codes and practices in Mexico. The Mexico City Building Code is more deeply examined since, for many years, it has been a reference document in Mexico for the drafting of most of the Mexican codes, which, by law, are of municipal competence. Problems related to the implementation of building codes in Mexico are discussed, and measures needed to foster a better construction quality and a higher seismic safety are proposed. Later, the newest (early 2004) version of the Mexico City Building Code is discussed, and what the authors believe to be its most important contributions are highlighted.

INTRODUCTION

Seismic design codes in Mexico are more than 60 years old. At several moments of their history, Mexican codes have contributed with new ideas and methods, some of which have later been adopted in codes elsewhere (Fukuta [1]). Some examples: in 1942, the importance factors; in 1957, the linear distribution of seismic forces with height, the dynamical method of analysis, the first limits to lateral displacement of structures, and higher seismic coefficients for soft-soil sites (in fact, the first compulsory seismic microzoning); in 1976, new strength-reduction factors.

In the following paragraphs, a brief description of the evolution of Mexican building codes and seismic design practice will be presented, along with some comments on the codes' enforcement and compliance. In the final part, we will discuss some of the most important innovations, regarding seismic design, contained in the latest version of the Mexico City Building Code, issued in early 2004.

THE EVOLUTION OF MEXICO CITY BUILDING CODE

In Mexico, building codes are to be issued by each of the more of 2400 municipalities. Regarding requirements for structural design, local building codes, when available, usually refer to technical

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standards typically issued by other parties, and most often to the Technical Norms of the Mexico City Building Code (MCBC). Several attempts to implement a National Model Code for structural design have insofar failed. However, the Mexico City Building Code is, in fact, used as a model code because it is the most widely recognized technical document for structural design in the country.

The first MCBC was issued in 1942; since 1966, contains a complete set of regulations for structural design. In 1976, the code adopted a coherent format for all materials and structural systems, based on limit states design philosophy. Only the general criteria have remained in the main body of the code and the specific requirements have been separated in a set of Technical Norms (Criteria and Loading, Seismic Design, Wind Design, Foundation Design, Concrete Structures, Steel Structures, Masonry Structures and Timber Structures). The design philosophy and the specific values for intensity of loads and load factors are the same for all structural materials. Strength reduction factors have been developed independently for each material and mode of failure.

Great importance is given to the seismic design, which is performed with the same procedure for all materials; only reduction factors to account for non-linear behavior, as well as detailing requirements for ductility, are associated to specific structural systems. As in most present international regulations, structures are required to not exceed lateral drifts that could cause non structural damage under moderate, frequent earthquakes, and are allowed to undergo significant post-elastic displacements under severe, rare events. For this last condition, forces determined for elastic behavior are significantly reduced according to the ductility that could be developed by each particular structural system, for which detailed requirements are defined in order to guarantee the highest possible level of ductility.

The MCBC has been updated approximately every ten years, the process being coordinated by a technical committee integrated by academics and practitioners, aimed directly at updating the code, and especially the requirements for seismic design; the city government has sponsored a significant amount of research. In February 2004 a new version of the Code and of its technical norms has been issued. We will show later some of the most important innovations that this new version contains.

CODE ENFORCEMENT AND COMPLIANCE

It can be safely stated that a significant wealth of knowledge on seismic design has been gathered in the country in the last 50 years, and especially after the 1985 Mexico earthquake, and that this knowledge has led to the development of updated and refined seismic codes and standards. Nevertheless, it must be admitted that the available knowledge has not thoroughly reached the common design and construction professional. (Meli and Alcocer [2]). It is also apparent that the design codes are often incorrectly understood or misinterpreted, and are often not complied with by lay practitioners. The lack of building code compliance shall not be regarded merely as a legal issue to be addressed only through enforcement actions. One significant reason for the lack of compliance with construction codes is that requirements are not understood and correctly applied by all designers and contractors. There is a vast difference between the level of expertise and quality of practice of a relatively small group of well-informed specialists and academics, and that of most professionals and construction workers. This is true in Mexico City itself, but is especially evident when the large disparity of knowledge and of consciousness about seismic problems along the country is considered. It could be concluded that to attain a reasonable safety level, it is essential to have consistency between the regulations, the level of expertise of most design and construction professionals, and local materials and construction systems. Given that the level of expertise and quality of practice of design and construction professionals in the country is quite diverse, one way to reach this goal is to implement codes with procedures and requirements of different levels of complexity. The most complex and comprehensive rules should be aimed at large, important structures; simple yet conservative approaches would be followed for most common structures limited to certain size, geometry and complexity. As it will be seen later, this is the case of the new version of the Mexico City building code, which includes two design procedures with different levels of complexity.

For the case of common structures designed through simple approaches, it would be wise to implement a series of safety checks or limits based on critical features of the structure to avoid undesirable performance. Such limits can best be related to geometric rules that are used to establish member dimensions and basic percentages of reinforcement. One successful example of a simplified method, allowed for several decades in the MCBC, is aimed at verifying the seismic safety of low-rise walled buildings with a regular and symmetric structural layout. After few calculations, the required wall area in the two principal plan directions can be readily determined. No similar procedures are available for other kinds of structures

Considering that the critical point for the success of a code is not the quality of the code itself, but rather its correct understanding and application, commentaries, figures and design aids that facilitate its correct use have been produced and intense dissemination programs have been implemented by professional associations and colleges; nevertheless, misunderstanding of code requirements is common, thus showing a need for more efficient mechanisms to promote and assure code compliance.

All enforcement mechanisms developed and put into practice by building authorities in different parts of the country have had their drawbacks and limitations. The most obvious one is that local offices, generally at the municipal or county level, typically do not have a technical group with proper qualifications and enough size to thoroughly review structural designs, grant building permits, and inspect the general quality of construction of all buildings. This situation is particularly prevalent in small municipalities, where local budgets are commonly too scarce to support this very necessary group. Furthermore, specialized technical boards in public offices have substantially diminished in recent years. To overcome this problem, responsibilities for inspection and quality assurance have been assigned to private professionals certified for these purposes; the mechanism has partially benefited building quality, but its implementation must be greatly improved.

For structures of high importance the participation of highly qualified professionals in the design and construction processes has been fostered through the development of a registry of specialists, strictly rated by their peers. Furthermore, a “liability statement on structural safety” must be issued by one of these qualified specialists, who must participate during the whole process (preliminary studies, design and construction. In Mexico City, such “liability statement” is mandatory for critical facilities, as well as for large buildings, and should be issued prior to occupancy and every three years or after an intense earthquake.

Non-engineered construction is common in Mexico. As a consequence, a large percentage of the building stock (and in some parts of the country, the vast majority) is built without construction permits, without compliance with codes, and without the participation of qualified professionals. Although this phenomenon is prevalent in rural zones, it has also become characteristic of large urban areas, mainly in poor neighborhoods. As an example, it has been estimated that from the 700,000 housing units built in Mexico every year, at least 300,000 fall in this category.

Because in this type of construction builders are not aware of, or disregard the importance of design codes and regulations, mere code enforcement cannot be considered a viable solution. Increase in construction quality should come from improvements in the skills of builders, and in the strength and durability of construction materials. Nonetheless, this is the most difficult group of the construction industry to reach using common technology-transfer mechanisms. First, quality improvement of non-engineered construction becomes more complicated because this construction practice evolves with little or no influence of specialists and organized boards. In this evolution, cultural and economic aspects play a very significant role, and in some cases, act as an obstacle to improvement. Second, an important drawback is that structural safety is not usually a primary concern of those living in non-engineered buildings, and that is difficult to "sell" to a population with serious unmet needs in their everyday lives.

The most significant success case for the improvement of the structural safety of non-engineered construction in Mexico has been de development and dissemination of confined-masonry construction. Walls confined with vertical and horizontal reinforced concrete elements, bond beams and tie-columns around the perimeter, were adopted in Mexico City in the 1940's to control the wall cracking caused by large differential settlements in the soft soil of the central portion of the city. Several years later, after examining its excellent seismic performance, this system became popular, even outside the soft soil area of the city, and in other zones of high seismic hazard. It must be pointed out that confined masonry has evolved essentially through an informal process based on experience, and that it has been incorporated in formal construction through code requirements and design procedures that are mostly rationalizations of the established practice, even after been validated by structural mechanics principles and experimental evidence. In non-engineered construction, the system is of general use in seismic prone urban areas of the country, and is slowly but steadily disseminating also in rural areas. The lesson that could be extracted from this case is that structural solutions akin to the local practice, but with superior performance based on their improved layout, materials and structural features must be promoted and disseminate to potential beneficiaries of this program, case studies of success attained in similar areas and conditions.

MEXICO CITY TECHNICAL NORMS FOR SEISMIC DESIGN, VERSION 2004

The new version of the Mexico City building code is the result of research advances made in the last 15 years in Mexico, both related to structural behavior and to strong-motion estimation in Mexico City. The code is composed of technical norms for all materials, plus design norms for earthquake and wind loads. The norms are written in a limit state format and, although an increasing trend towards an explicit performance based design has been perceived, especially for seismic design, this approach has not been fully incorporated into the code.

Regarding the technical norm for earthquake design, it includes two possible approaches. The first one, contained in the main body of the norm, is a conventional design procedure, very similar to the one that has been present in Mexican codes in the last three decades. This procedure admits three different levels of complexity in structural analysis and design, depending on the size and location of the structure. These three levels are, in order of complexity: the simplified, the static and the dynamic methods.

The second possible approach accepted by the new MCBC includes some innovations that will be described in this paper. This new approach is contained in an appendix to the earthquake design norm; although its use is optional, it is expected to become the standard design method for important structures. As it will be seen, it is a first step towards a performance-based code.

In the following paragraphs we will discuss the new approach, with emphasis on the new ways to specify earthquake design spectra for locations in Mexico City and some of the new seismic design criteria to be applied together with these spectra.

Elastic Pseudoacceleration Spectra

Elastic pseudoacceleration response spectra (5% damping) are the starting point to evaluate both design forces and lateral deformations. It is recognized that a reasonable way to specify earthquake loads starts with the construction of uniform-hazard spectra (UHS), that is, spectra in which all ordinates have the same probability of being exceeded in a given period of time. UHS at firm ground were determined by means of a probabilistic seismic hazard analysis (Esteva [3], Cornell [4]) to calculate exceedance rates for spectral values associated to structural periods between 0 and 5 s for the Ciudad Universitaria (CU) accelerometric site. This site is located in the firm zone of Mexico City and, as it will be discussed later, it has been used as a reference site to estimate strong motion and seismic hazard at other sites of the city.

Figure 1 shows UHS (average between NS and EW components) at site CU for a return period of 125 years. The thick line corresponds to the spectrum obtained accounting for the effect of all seismic sources that affect Mexico City. Figure 1 also shows hazard de-aggregation results, in terms of UHS obtained considering only subduction events, intermediate-depth events or earthquakes from local and crustal sources.

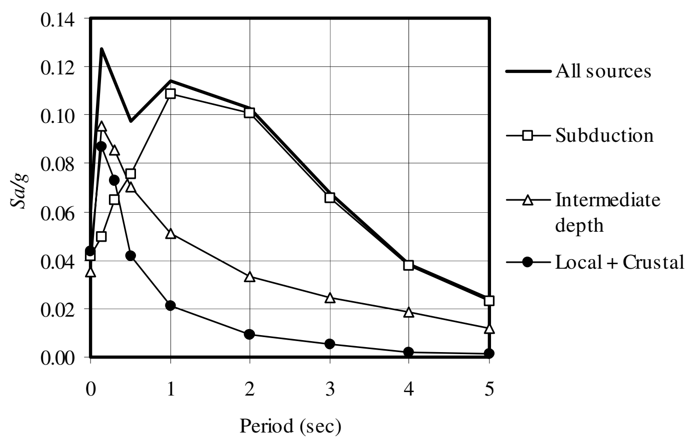

Figure 1. Uniform hazard spectra (pseudoacceleration, 5% damping) at CU site for a return period of 125 years, associated to the various types of earthquake sources that affect Mexico City.

From previous research studies (*e.g.*, Ordaz *et al.* [5]), spectral amplification functions *Fj(T)* for more than 100 instrumented sites at Mexico City were available. These functions are defined as the average ratio, for several recorded events, between spectral values at the *j*th instrumented site and at the CU reference station, both values for the same period, *T*.

With the values of *Fj(T)* for about 100 sites and using the interpolation procedure devised by Pérez-Rocha [6], spectral amplification functions were computed for 1600 points forming a rectangular grid of 40 by40 points covering most of the populated portion of the Federal District. With the functions of amplification relative to CU and the exceedance rates at this site, it is possible to compute, with reasonable assumptions, UHS for each of the 1600 points.

As previously noted by Miranda [7], the shape of elastic and inelastic response spectrum at soft soil sites is strongly influenced by the predominant periods of the ground motion, which, for very soft soil sites, in most cases coincides with the fundamental period of the soil deposit. Figure 2 shows UHS associated to a return period of 125 years, computed for several sites in Mexico City characterized by the predominant ground period T_s. Sites in Mexico City with predominant periods of vibration shorter than about 0.7s are characterized by wide spectra, while sites with predominant periods between 1 and 3 s are characterized by narrow spectra. Meanwhile, sites with deep deposits of very soft clay are characterized by having two peaks associated with the first and second mode of vibration of the soil deposit. The amplitude of these two peaks is often similar.

Smoothed uniform hazard spectra

As it can be noted in figure 2, UHS have very irregular shapes, inadequate to be incorporated in the building code. Therefore, it is necessary to simplify their forms. For this reason, the following functional form was chosen:

$$\frac{Sa(T)}{g}=\begin{cases} a_o+(\beta c-a_o)\dfrac{T}{T_a}; & \text{if } T<T_a \\ \beta c; & \text{if } T_a\le T<T_b \\ \beta c\left[k+(1-k)\left(\dfrac{T_b}{T}\right)^2\right]\left(\dfrac{T_b}{T}\right)^2; & \text{if } T\ge T_b \end{cases} \tag{1}$$

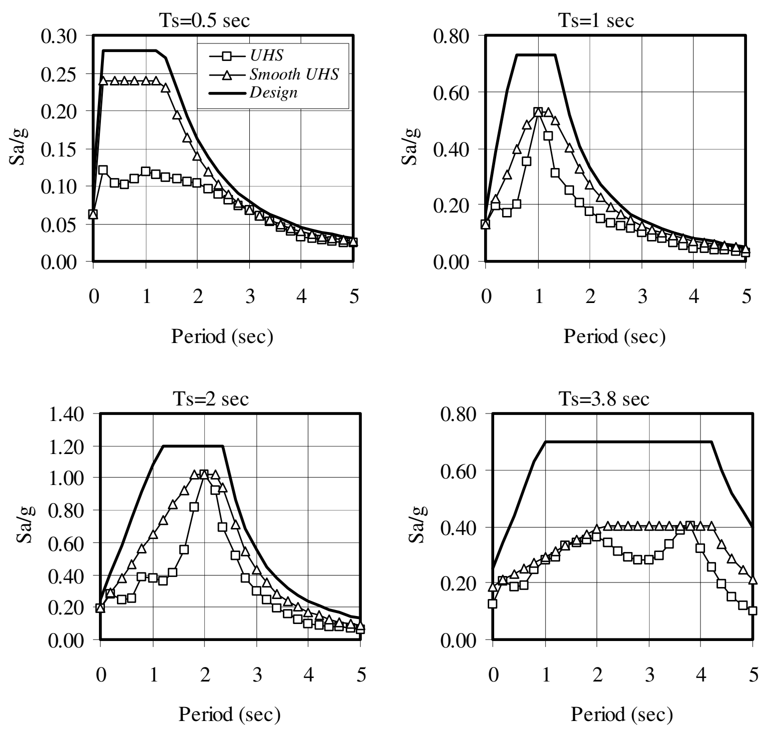

Figure 2. Uniform hazard spectra (UHS), smoothed UHS and design spectra for sites in Mexico City with the indicated predominant ground periods, *Ts*.

In the expression above, the smoothed spectrum depends on five parameters: a_0, the peak ground acceleration; *c*, the peak spectral value; T_a and T_b, which are the lower and upper periods of the flat part of the spectrum; and *k*, which, as it will be shown later, controls the descending branch of the spectrum. The remaining parameter β is used to account for the supplemental damping due to soil-structure interaction; β=1 when the effects of interaction are neglected. The resulting spectral shapes can be appreciated in figure 3, along with their corresponding displacement spectra, *Sd(T)*. Note the variation of spectral values for *T>Tb* depending on parameter *k*.

The spectral shapes for $T<T_b$ have been in use for many years in Mexican building codes. For $T>T_b$, a new shape is proposed in order to have a better description of the spectral displacement in this period range.

As it is well known, for long period the spectral displacement tends to the peak ground displacement, D_{max}. In view of the relationship between pseudoacceleration and displacement ($Sd=SaT^2/4\pi^2$), this long-period limit can only be achieved if the pseudoacceleration spectrum decays as T^2 for $T>Tb$. The shapes stipulated by many codes in the world indicate a slower decay, which produces a displacement spectrum that tends to infinity as T grows. This is inconvenient, especially at soft sites, where very large spectral displacements can occur for natural periods around the predominant site period, and considerably smaller displacements for $T >> T_s$. As an example, the accelerogram recorded during the 1985 Michoacán event at station SCT (located on soft soil) exhibits a peak spectral displacement (5% damping) of 1.2 m but a peak ground displacement of only 0.2 m.

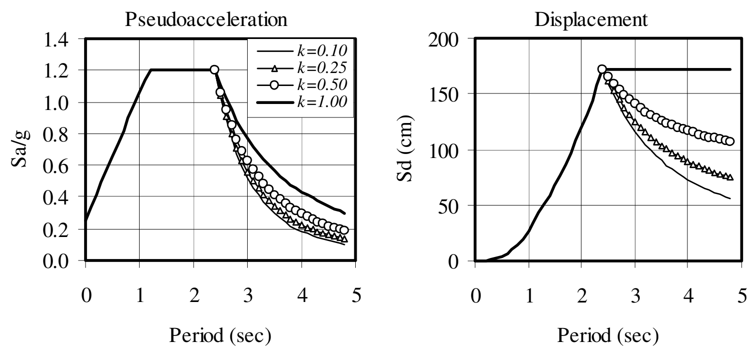

Figure 3. Characteristic shapes of the pseudoacceleration and displacement smoothed UHS.

As it can be seen in figure 3, the proposed spectral shape for $T>T_b$ yields more realistic structural displacement spectra and it is flexible enough as to represent a wide variety of site conditions, from firm (k=1) to very soft soils (k=0). It can be shown that parameter k has a physical meaning, because it is proportional to the ratio between peak ground displacement and peak spectral displacement.

With the chosen spectral shape given by equation 1 we proceeded to determine, for the 1600 grid points, values for the 5 parameters in such a way that the UHS at every point was safely enveloped in the whole period range. Some examples of the resulting smoothed uniform hazard spectra are depicted in figure 2.

Figure 4 shows the computed values of the five parameters defining the smoothed UHS for the 1600 grid point, as functions of predominant ground period. It can be noted that, although the correlation between these parameters and T_s is not perfect, clear tendencies are observed, which are safely covered by the proposed lines also shown in figure 4. These envelope lines have the following algebraic expressions:

$$a_o = \begin{cases} 0.1 + 0.15\left(T_s - 0.5\right); & \text{if} \quad 0.5 \le T_s \le 1.5 \text{ sec} \\ 0.25; & \text{if} \quad T_s > 1.5 \text{ sec} \end{cases} \tag{2}$$

$$c = \begin{cases} 0.28 + 0.92\,(T_s - 0.5); & \text{if} \quad 0.5 < T_s \le 1.5 \text{ sec} \\ 1.2; & \text{if} \quad 1.5 < T_s \le 2.5 \text{ sec} \\ 1.2 - 0.5\,(T_s - 2.5); & \text{if} \quad 2.5 < T_s \le 3.5 \text{ sec} \\ 0.7; & \text{if} \quad T_s > 3.5 \text{ sec} \end{cases} \tag{3}$$

$$T_a\,(\text{sec}) = \begin{cases} 0.2 + 0.65\,(T_s - 0.5)\,; & \text{if } \; 0.5 < T_s \le 2.5 \text{ sec} \\ 1.5\,; & \text{if } \; 2.5 < T_s \le 3.25 \text{ sec} \\ 4.75 - T_s\,; & \text{if } \; 3.25 < T_s \le 3.9 \text{ sec} \\ 0.85\,; & \text{if } \; T_s > 3.9 \text{ sec} \end{cases} \tag{4}$$

$$T_b\,(\text{sec}) = \begin{cases} 1.35\,; & \text{if } \; T_s \le 1.125 \text{ sec} \\ 1.2\,T_s\,; & \text{if } \; 1.125 < T_s \le 3.5 \text{ sec} \\ 4.2\,; & \text{if } \; T_s > 3.5 \text{ sec} \end{cases} \tag{5}$$

$$k = \begin{cases} 2 - T_s\,; & \textit{if } \; 0.5 < T_s \le 1.65 \textit{ sec} \\ 0.35\,; & \textit{if } \; T_s > 1.65 \textit{ sec} \end{cases} \tag{6}$$

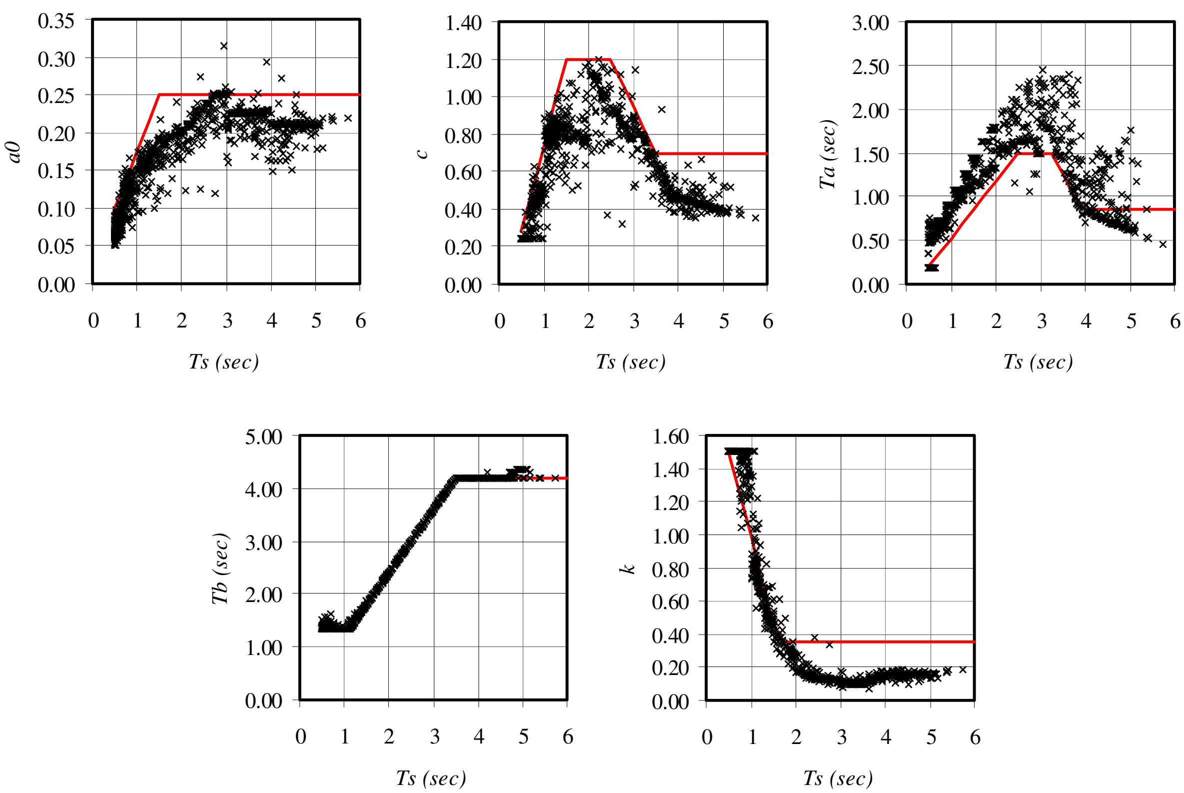

Figure 4. Points: values of the corresponding parameters computed for the 1600 grid points, as a function of predominant ground period, T_s. Straight lines: proposed expressions (equations 2-6) to specify parameters of elastic design spectra.

With equations 2-6 it is possible to determine the five parameters that define the site-specific design spectrum if T_s is known. It is proposed that the value of the site period be taken from a map included in the code and displayed in Figure 5. Elastic design spectra resulting from this approach are shown in figure 2 for some sites in Mexico City.

More precise shapes and sizes of design spectra could have been obtained if the site characterization had been made with more than one parameter. For instance, along with *Ts*, the depth of the clay deposits could have been used to characterize a site; however, this would have resulted in more complex rules to determine design spectra. Finally, after a balance between simplicity, precision and safety, the overestimations were considered acceptable.

Ductility reductions

Contemporary design criteria admit inelastic excursions when the structure is subjected to the earthquake characterizing the collapse prevention limit state. This situation limits the force demands in the structural elements, hence allowing the use of smaller design strengths, at the cost of certain -limited- levels of structural damage due to yielding of some portions of the structure.

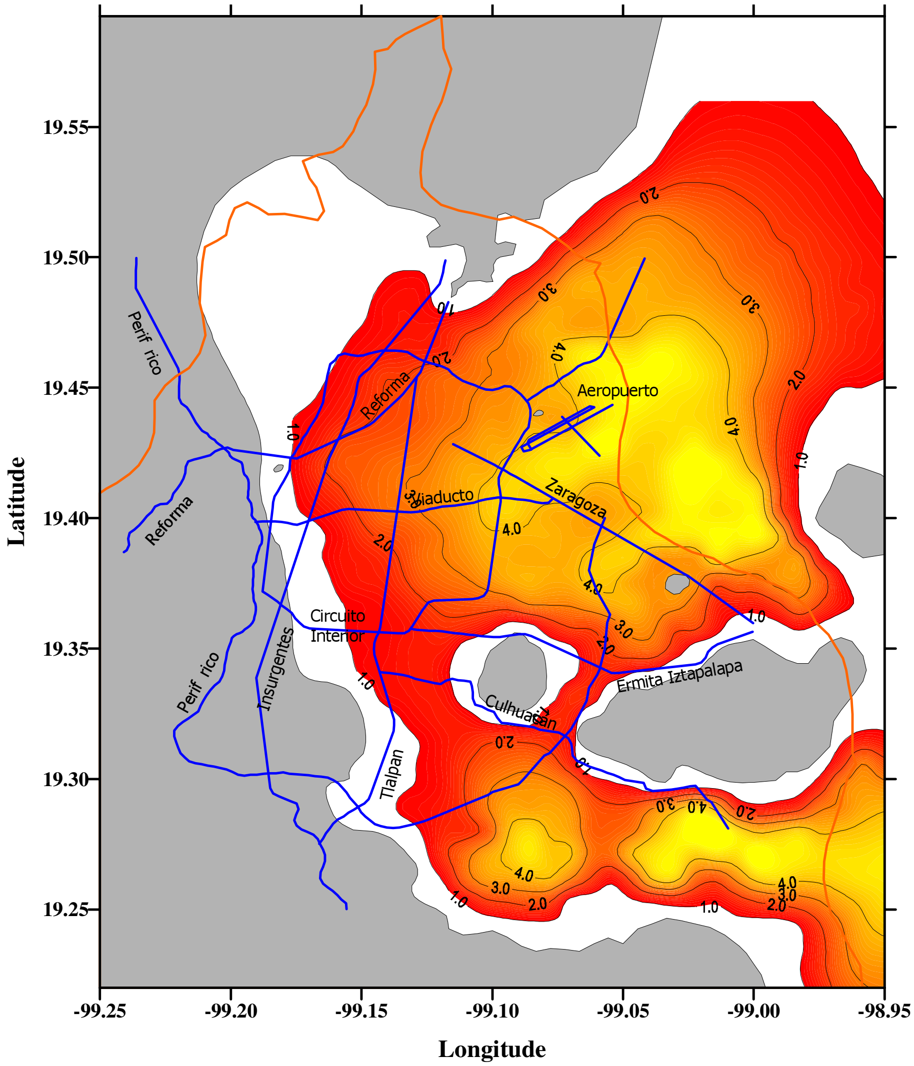

Figure 5. Curves of predominant ground period (sec) in Mexico City

In order to construct simple models, useful for regular structures, of the non-linear structural behavior, most codes in the world are based on a single-degree-of-freedom oscillator with elastoplastic behavior. It is with this model that the required strength to limit ductility demand to the specified ductile capacity is determined. It is common to express the strength required for the allowable ductility, under a certain earthquake, $Fy(T,\mu)$, as a fraction of the required strength to have elastic behavior for the same earthquake, $Fy(T,1)$. Let R be this ratio, often referred to as the force reduction factor.

This factor depends on both the structural period T and the global ductility capacity of the structure μ. The form of R has been widely studied in the last years (e.g., Krawinkler *et al*. [8], Miranda [9], Miranda and Bertero [10]). In particular, Ordaz and Pérez-Rocha [11] observed that, under very general circumstances, the shape of R for elastoplastic systems depends on the ratio between the spectral displacement, $Sd(T)$ and the peak ground displacement, D_{max}, in the following way:

$$R(T,\mu) = 1 + (\mu - 1)\left(\frac{Sd(T)}{D_{max}}\right)^{\alpha} \tag{7}$$

where $\alpha \approx 0.5$. It is apparent that the dependency of R with period and damping is implicit in $Sd(T)$. A simplified version of equation 7, when applied to the pseudoacceleration spectra defined by equation 1, is the following:

$$R = \begin{cases} 1 + (\mu - 1)\sqrt{\beta / k}\,\dfrac{T}{T_a}\ ; & \text{if } T \le T_a \\ 1 + (\mu - 1)\sqrt{\beta / k}\ ; & \text{if } T_a < T \le T_b \\ 1 + (\mu - 1)\sqrt{\beta p / k}\ ; & \text{if } T > T_b \end{cases} \tag{8}$$

where

$$p = k + (1 - k)\left(\frac{T_b}{T}\right)^2 \tag{9}$$

It can be shown that the force-reduction factor defined in the previous equation has the correct limits for short and long periods. Also, it is interesting to note that, contrarily to what happens in many building codes, in this proposal the force reduction factor R can be larger than the ductile capacity, μ. This fact was observed for the first time by Meli and Ávila [12] after analyzing recordings obtained in Mexico City's lakebed zone during the September 19, 1985 Michoacán earthquake. Later, their observations have been verified studying hundreds of accelerograms (Miranda [9], Ordaz and Pérez-Rocha [11]). An example of R computed with equations 8-9 is shown in Figure 6.

Overstrength reductions

Existence of structural overstrength has been explicitly recognized in some building codes in the world. There are several sources of overstrength and a deep discussion on the subject is beyond the scope of this paper (see, for instance, Miranda [13]). One important source of overstrength in many structures is the design procedure itself. The structure must be analyzed using forces reduced with a factor that depends on the structure's global ductility capacity. Then, enough strength must be furnished to avoid yielding under the reduced forces. Therefore, if a structural member yields under these reduced forces, its strength must be enlarged until it remains elastic. In consequence, it is assumed that the nominal strength of the structure is that for which all members remain elastic. In reality, some members will yield under the design motion, and the code provisions are oriented to limiting the ductility demand to the ductile capacity. However, the global behavior of the structure is not, in general, elastoplastic; it would only be so if all structural members had elastoplastic behavior and they yielded at the same time. This consideration implies that, in many cases, the real strength is higher than its nominal value.

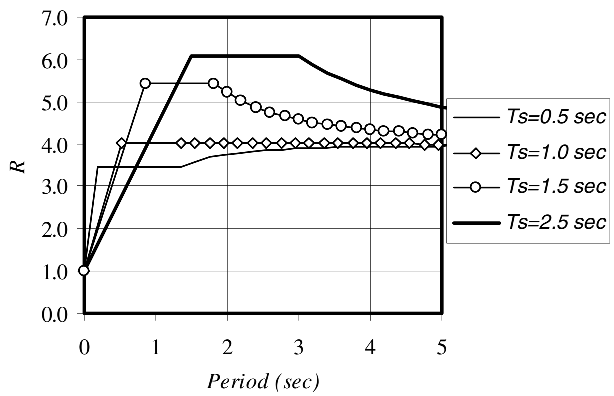

Figure 6. Force-reduction factor, R, as a function of period, for μ=4 and several predominant ground periods.As Loera [14] has pointed out, the effect of overstrength should be accounted for when evaluating members' strength and not as a reduction factor applied to the loads. Indeed, this would be the most rational approach to deal with this problem. However, this would bring major changes in the earthquake analysis and design criteria and would demand the compulsory use of more advanced non-linear analysis techniques of which, at present, the most promising alternative is the pushover method. Although these methods have been deeply investigated in recent years, in our opinion they are still not commonly used by the practicing engineers. In view of these limitations, it is proposed to continue applying the effect of overstrength as a reduction factor to the loads.

Overstrength depends on many factors. In particular, it depends on the level of force redistribution that can take place in the structure. Unfortunately, there are not many studies that allow computation of overstrength as a function of a few, simple structural parameters. For this reason, it is proposed to use overstrength factors that yield, roughly, to the same strengths required by the 1987 Mexico City code for ductile capacities between 3 and 4. In consequence, it is proposed that the over-strength reduction factor, Ω, be given by

$$\Omega = \begin{cases} \dfrac{10}{4+\sqrt{T/T_a}}\,; & if\ \ T \le T_a \\ 2\,; & if\ \ T > T_a \end{cases} \tag{10}$$

There are no sound theoretical or empirical bases to explain the variation of Ω presented in equation 10. However there is evidence that short-period structures have larger overstrength than long-period structures.

Inelastic design spectra

In view of what has been discussed in the previous sections, the required base-shear coefficient for design is computed as follows:

$$C(T,Q) = \frac{Sa(T)/g}{\Omega(T)R(T,Q)} \tag{11}$$

with $Sa(T)/g$ given by equations 1 and 2-6, $R(T,\mu)$ by equations 8-9 and $\Omega(T)$ by equation 10. Figure 7 depicts comparisons, for several sites in Mexico City, between the required strengths proposed in this paper and those required in the 1987 Mexico City building code.

It can be seen that, for long periods, the proposed strengths are systematically smaller than those presently required by the Mexico City building code. To avoid structures with excessively small strengths, a minimum reduced strength C_{min}=0.03 is proposed for firm and transition sites (T_s< 1s) and C_{min}=0.05 for soft soils ($T_s \geq$ 1s).

Computation of displacements
Actual lateral displacements are computed multiplying those obtained under reduced loads by certain factors. The code proposes revision of displacements for two limit states: collapse prevention and service. The limit states and the way to compute displacement in each case will be discussed in the following paragraphs.

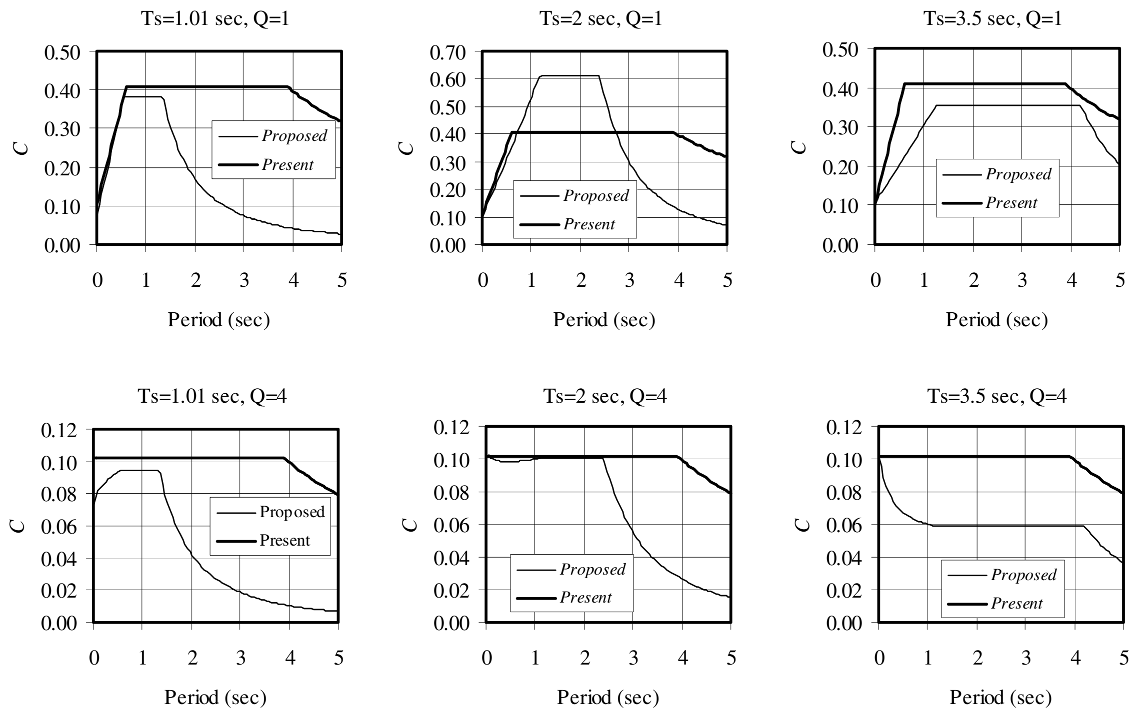

Figure 7. Present and proposed required strengths (base-shear coefficient, *C*) for sites in Mexico City with predominant ground period T_s=1.01, Ts=2 sec and T_s=3.5 sec. for ductile capacities of 1 and 4.

Collapse-prevention limit state
For this limit state, inter-story drifts obtained under the collapse earthquake (whose spectrum is described by $Sa(T)$ in equation 1 must be compared with allowable values stipulated in Table 1 for diverse structural systems. The displacements for the collapse limit state, D_C, will be computed, as it is common practice, multiplying the reduced displacements, D_R, by the ductility factor μ. But also, they must be multiplied by the overstrength reduction factor, Ω. The reason for this is that, if indeed the structure has such overstrength, the acting forces will not be limited by the design strength, but by a larger strength, that is exactly the nominal strength times Ω. In view of this we have

$$D_C = D_R \mu \Omega \tag{12}$$

Service limit state

As a novelty in the Mexico City building code, it was proposed to have a clearly specified service limit state, to limit displacements for earthquakes that occur much more frequently than the collapse event. The problem of determining the return period for the service earthquake is not trivial and, although research has been made in this direction (see, for instance, Reyes [15]), there are not clear answers. However, in the work by Reyes there is some evidence in the sense that a return period of 10 years is reasonable for common structures in Mexico City. To this return period are associated spectral ordinates similar to those produced by the April 25, 1989, M=7 event. Also there was agreement among the review committee members that damage to non-structural members should not be tolerated for an earthquake like this one.

In view of this, it was decided that the service event should be one with a pseudoacceleration spectrum like that given by equation 1 but divided by a constant factor of 7, to have demand levels similar to those imposed by the 1989 event. Under the service earthquake, the resulting elastic inter-story drifts must be less than 0.002 when non-structural elements are linked to the structural system, or less than 0.004 if they are separated.

Table 1. Allowable inter-story drifts for the collapse limit state.

Structural system	**Inter-story drift**
R/C ductile frames (μ=3,4)	0.0300
Steel ductile frames (μ=3,4)	0.0300
Frames with limited ductile capacity	0.0150
Flat slabs without shear walls or bracing	0.0150
Steel frames with eccentric bracing	0.0200
Frames with concentric bracing	0.0150
Ductile dual systems (walls + frames)	0.0150
Limited-ductility dual systems (walls + frames)	0.0100
Diaphragm walls	0.0060
Confined and reinforced masonry walls	0.0050
Masonry walls type I	0.0040
Masonry walls type II	0.0020
Simple masonry walls	0.0015

It must be pointed out that, in a multi-level design approach, design spectra should not necessarily have the same shape for all design levels. The reason for this, which is of special importance in soft soils (Ordaz *et al* [16]) is that the frequency content of the design events, service and collapse prevention, can be substantially different depending on the chosen return period. For the sake of simplicity, however, it was decided to have identical spectral shapes for the two design stages. This aspect, undoubtedly, should be modified in the future, as well as those design criteria related to the service earthquake demanding elastic structural behavior, as it happens for instance with the static eccentricity amplification to account for the effects of torsion.

Computation of displacements for the minimum-strength condition
As it has been stated, when the computed required strength is less than a given amount, the resulting seismic forces must be scaled so that the base shear coefficient is exactly C_{min}. This correction, however, must not affect the computed displacements, because scaling them implies, roughly, to have a constant acceleration spectrum for the period interval in which the minimum-strength condition applies. With such an acceleration spectrum, the associated displacement spectrum would start growing as T^2, yielding to a very unrealistic spectral shape.

CONCLUSIONS

After a brief description of the evolution of seismic design codes and practice in Mexico, a novel procedure that forms the basis of the new seismic design criteria in the Mexico City building code has been presented. This procedure allows determination of design strengths and displacements in a more rational way, more in accordance both with the present state of knowledge concerning dynamic response of soft soils as well as with the contemporary tendencies in building codes. In our opinion, the most important proposed modifications are the following:

1. The starting point of the analysis is the assessment of elastic acceleration and displacement spectra that have realistic sizes and shapes.
2. Design spectra vary depending on site conditions, which are characterized by the predominant ground period. The city is not divided into microzones anymore. Instead, a continuous variation of predominant ground period is used
3. An empirical overstrength reduction is explicitly applied.
4. Ductility reductions are made with approximate equations based on observed reductions in elastoplastic oscillators subjected to narrow-band earthquakes.
5. More rational criteria are stipulated for the computation of lateral displacements.
6. The existence of two limit states (service and collapse prevention) is clearly defined, along with allowable inter-story drifts that better reflect the expected structural performance

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