Updated Seismic Design Guidelines for Model Building Code of Mexico

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The Manual of Civil Structures (MOC), a model design code in Mexico, has been in the process of being updated, and the new version of this code was published in 2008. A major update from the 1993 version was performed in the chapter for the seismic design of building structures. This paper summarizes the most relevant changes of this building code and their relation to research efforts conducted within Mexico and worldwide to improve the seismic design of building structures. One goal is to make the guidelines as transparent as possible to users, so that the design process will be clearer to structural engineers. [DOI: 10.1193/1.3240413]

INTRODUCTION

The previous version of the Manual of Civil Structures (MOC) was published in 1993 (MOC-93 1993, Tena-Colunga 1999), so an in-depth review was mandatory in order to update the document for 2008. Like ASCE-7 (2005), MOC-2008 (2008) is a very comprehensive code that specifically addresses the design of several structural systems (buildings, bridges, dams, power stations, industrial facilities, chimneys, silos, pipelines, tanks and deposits, vessels, inverted pendulums, retaining walls, etc.) to such hazards as earthquakes and winds. Modern technologies, such as base isolation and passive energy dissipation, are now addressed, along with the use of modern materials like carbon fibers and composites. Specialized topics, including soil-structure interaction, the monitoring of structures, and the evaluation and rehabilitation of existing structures, are also covered.

The following sections summarize only some of the most important updated provisions that affect the seismic design of building structures. Some background in the bases and design philosophy of the previous seismic provisions of the MOC-93 code in English can be found elsewhere (Tena-Colunga 1999).

SEISMIC ZONATION

One of the major changes in the MOC-2008 with respect to previous MOC-93 code is the concept of the seismic zonation. In the MOC-93 code, Mexico was divided into

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Figure 1. MOC-93 seismic zone map of Mexico (courtesy of Servicio Sismológico Nacional).

four seismic zones (A, B, C, and D; Figure 1), for which there were three different soil profile types: I (firm soils), II ("transition" soils), and III (soft soils), as explained in greater detail elsewhere (MOC-93 1993, Tena-Colunga 1999).

In the MOC-2008 code, seismic hazard in Mexico is defined as a continuum function where peak accelerations in rock are defined (Figure 2a). These peak accelerations are associated with return periods (Figure 2b) that were obtained using an optimization design criterion to define the seismic coefficients for the plateaus of the elastic design spectra for standard occupancy structures, as explained in detail elsewhere (Ordaz et al. 2007, Pérez-Rocha and Ordaz 2008, MOC-2008 2008). All known earthquakes sources for the different regions of seismic risk in Mexico, as well as their maximum credible earthquake (MCE) scenarios expected using updated information, were taken into account. The seismic hazard was evaluated using both deterministic and probabilistic approaches (Pérez-Rocha and Ordaz 2008).

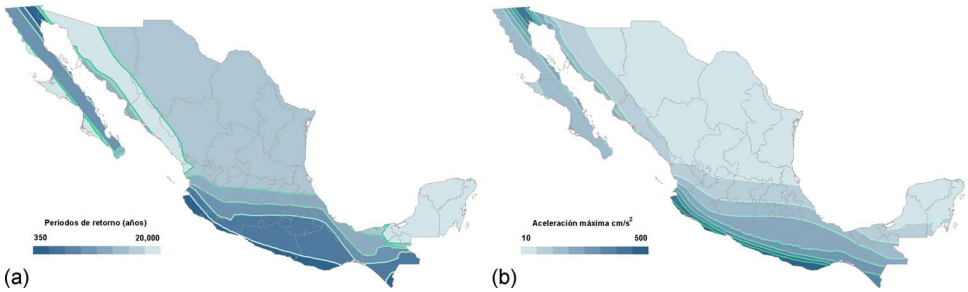

Figure 2. Peak ground accelerations for MOC-2008 associated to return periods obtained using optimal design criteria.

ELASTIC DESIGN SPECTRUM

The elastic acceleration design spectrum for MOC-2008 code consists, in theory, of an infinite number of discrete functions within the Mexican Territory as a direct consequence of deciding to define the seismic hazard as a continuum, as described above.

This major conceptual change was made for the following reasons: (1) important progress has been made in the fields of seismology and seismicity, where more reliable information is available, (2) practicing engineers and researchers in Mexico often noted that the definition of seismic forces for design for different structures across Mexico cannot be done in a rational and transparent way using the collection of 12 design spectra in MOC-93 because relevant information about site effects and structural dynamics are lost, unless site-specific design spectra were allowed for design, and (3) the rapid development in computer technology and its availability to practically anyone in the workplace now allows a new approach using user-friendly software to define the design spectrum for any given site, as planned for MOC-2008.

In essence, the proposed elastic acceleration design spectrum is transparent as modification factors are defined exclusively in terms of the seismic hazard and site effects. Spectral amplifications and nonlinear effects due to the characteristics of the soil profile and its relation to the seismic intensity incidence are considered in site-effect modeling (Mena-Hernández et al. 2006, Pérez-Rocha et al. 2007, Pérez-Rocha and Avilés 2008, MOC-2008 2008). A soil model based on a homogeneous layer with nonlinear behavior supported by an elastic half space was used for such purpose (Pérez-Rocha et al. 2007).

ACCELERATION DESIGN SPECTRUM FOR MCE (COLLAPSE PREVENTION)

In order to define the elastic acceleration design spectrum for a given site for the maximum credible earthquake (MCE) related to the collapse-prevention performance level, the following steps must be taken (MOC-2008 2008, Pérez-Rocha et al. 2007, Pérez-Rocha and Avilés 2008):

1. Assess the expected peak acceleration in the bedrock a_0^r (Figure 2a), a parameter that takes into account the seismic hazard.
2. Compute the distance factor as $F_d = a_0^r/400 \leq 1$, which is equal to unity near the subduction earthquake source. This parameter not only expresses the seismic wave attenuation with distance, but also the filtering of the high-frequency components of the earthquake excitation.
3. From geotechnical information in the soil profile, compute the dominant site period T_s as follows:

$$T_s = 4\sqrt{\left(\sum_{n=1}^{N} \frac{h_n}{G_n}\right)\left(\sum_{n=1}^{N} \rho_n h_n (w_n^2 + w_n w_{n-1} + w_{n-1}^2)\right)} \tag{1}$$

where G_n and ρ_n are the shear modulus and mass density of the nth layer of thickness h_n; $w_o = 0$ at the bedrock and

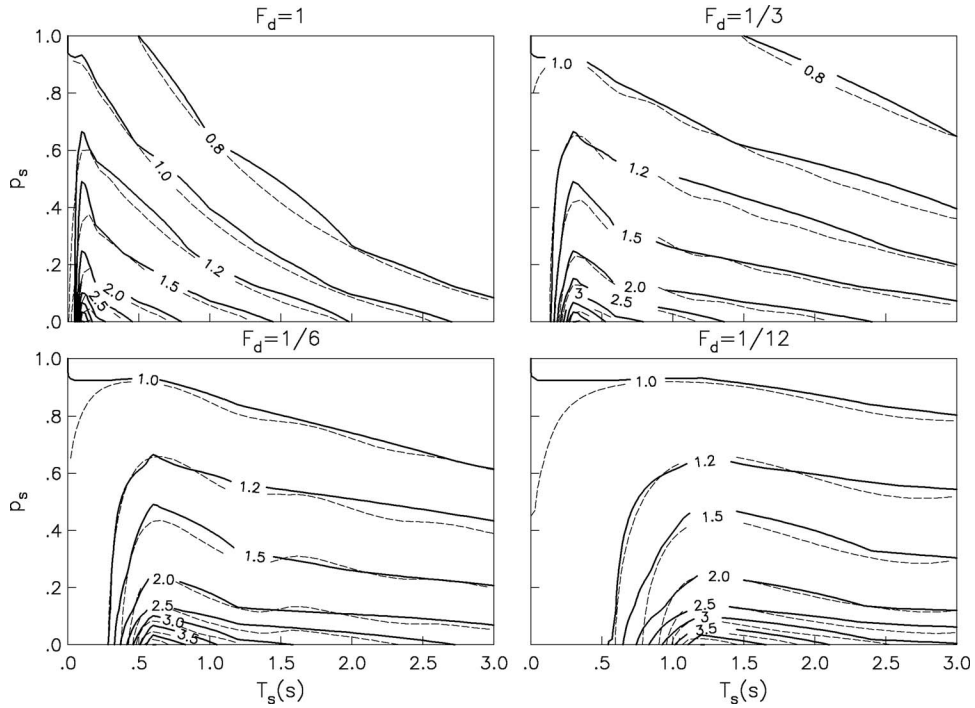

Figure 3. Contours of F_s derived from free-field ground-response analyses (dashed line) and by linear interpolation of data in Table 1 (solid line).

$$w_n = \frac{\Sigma_{i=1}^{n} h_i/G_i}{\Sigma_{i=1}^{N} h_i/G_i}; \quad n = 1,2, \ldots ,N \tag{2}$$

is a static approximation for the fundamental mode of soil vibration. With T_s known, the effective shear-wave velocity $V_s = 4H_s/T_s$ is computed over a depth H_s of at least 30 m. This novel approach is found to yield more accurate results than those obtained by using the average shear-wave velocity of surficial soils, which ignores the sequence of the layers in the soil deposit.

4. Assuming linear behavior for the soil formation, site (F_s) and structural amplification (F_r) factors are then obtained. The site-response factor F_s is based on the theoretical results shown in Figure 3 for the ratio of peak accelerations measured at the surface and base of the soil deposit. The analysis was made using as input ground motion, the power spectrum of the MCE, and through application of the random vibration theory to predict peak responses. The discrete values specified by the code for this factor are tabulated in Table 1 as a function of $T'_s = T_s F_d^{1/2}$ and the impedance contrast, p_s, between soil and bedrock.
5. As an option, to account for the nonlinear soil behavior, the additional factors, $F_{nl}^a \leq 1$ and $F_{nl}^T \leq 1$, can be calculated. While F_{nl}^a expresses the amplitude reduc-

Table 1. Values of the site amplification factor F_s

	$T'_s(s)$							
p_s	0.00	0.05	0.10	0.20	0.50	1.00	2.00	3.00
1.000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.625	1.00	1.08	1.23	1.12	1.00	1.00	1.00	1.00
0.250	1.00	1.18	1.98	1.60	1.40	1.12	1.00	1.00
0.125	1.00	1.20	2.64	2.01	1.69	1.32	1.00	1.00
0.000	1.00	1.22	4.51	3.17	2.38	1.75	1.19	1.00

tion of the site response due to a damping increase, $1/F_{nl}^T$ expresses the site period shift because of a stiffness decrease. Both factors are dependent on the level of shaking and are equal to unity for linear elastic strains.

6. The peak ground acceleration for the site, a_0, is obtained from the expected peak acceleration in the bedrock affected by the site and nonlinear factors:

$$a_0 = F_s F_{nl}^a \frac{a_0^r}{g} \tag{3}$$

where g is the acceleration of gravity.

7. The seismic coefficient that defines the plateau of the design spectrum, c, is computed from the peak ground acceleration for the site and the response factor as follows:

$$c = F_r a_0 \tag{4}$$

where F_r is the ratio of the peak structural acceleration to the peak ground acceleration. This structure-response factor is based on the random vibration analysis of a single degree of freedom oscillator subject to a base excitation given by the power spectrum of the MCE passed through the soil deposit. The theoretical results so obtained are shown in Figure 4 and the discrete values specified by the code are presented in Table 2 as a function of T_s and p_s only, since the distance factor has little influence.

8. The control periods, T_a and T_b, that define the plateau of the design spectrum are computed from the fundamental site period T_s as follows:

$$T_a = 0.35 \frac{T_s}{F_{nl}^T} \geqslant 0.1s \tag{5}$$

$$T_b = 1.2 \frac{T_s}{F_{nl}^T} \geqslant 0.6s \tag{6}$$

For a rock site, T_a=0.1 s, and T_b=0.6 s. The values of these control periods are intended to cover not only the peak response at the first soil period (T_s), but also that

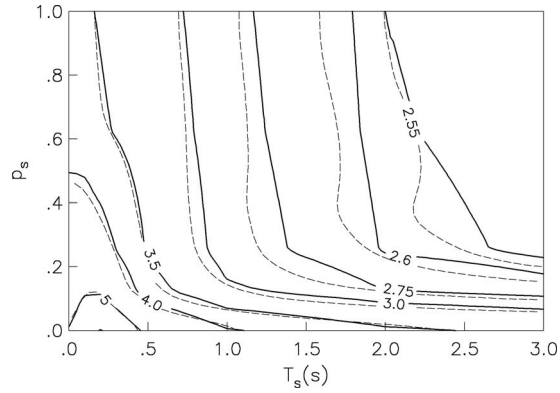

Figure 4. Contours of F_r derived from site-structure response analyses (dashed line) and by linear interpolation of data in Table 2 (solid line).

at the second soil period ($\approx T_s/3$). The upper period is taken 20% greater than the site period to account for differences between computed and actual values of T_s.

9. Finally, the elastic acceleration design spectrum for an equivalent viscous damping of 5% is defined for the site.

Then, the elastic acceleration design spectrum for 5% equivalent viscous damping for structures of group B (standard occupancy) for the MOC-2008 code, schematically depicted in Figure 5a, is defined with the following general expressions:

$$a = \frac{S_a(T_e)}{g} = \begin{cases} a_0 + (\beta c - a_0)\dfrac{T_e}{T_a}; & \text{if } T_e < T_a \\ \beta c; & \text{if } T_a \leqslant T_e < T_b \\ \beta c\left(\dfrac{T_b}{T_e}\right)^r; & \text{if } T_b \leqslant T_e < T_c \\ \beta c\left(\dfrac{T_b}{T_c}\right)^r\left[k + (1-k)\left(\dfrac{T_c}{T_e}\right)^2\right]\left(\dfrac{T_c}{T_e}\right)^2; & \text{if } T_e \geqslant T_c \end{cases} \tag{7}$$

where a is the spectral acceleration ordinate for the design spectrum (S_a) expressed as a fraction of the acceleration of gravity (g), a_0 is the ground acceleration coefficient, c is the seismic coefficient that defines the plateau, T_e is the structural natural period of interest, T_a and T_b are control periods that define the plateau of the spectrum, T_c is a control period that defines the descending branch of the acceleration spectrum in order that the displacement design spectrum computed from the acceleration design spectrum will converge to the ground displacement at long periods, r is the parameter that defines the descending branch of the acceleration spectrum in the period range $T_b \leq T_e < T_c$, k is the

Table 2. Values of the structural amplification factor F_r

$T_s(s)$	0.00	0.05	0.10	0.20	0.50	1.00	2.00	3.00
p_s								
1.000	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50
0.625	2.50	3.80	3.74	3.57	3.26	2.81	2.56	2.51
0.250	2.50	4.36	4.41	4.27	3.45	2.85	2.59	2.53
0.125	2.50	4.74	4.91	4.90	3.70	3.06	2.75	2.65
0.000	2.50	5.27	5.66	6.02	4.81	4.05	3.58	3.40

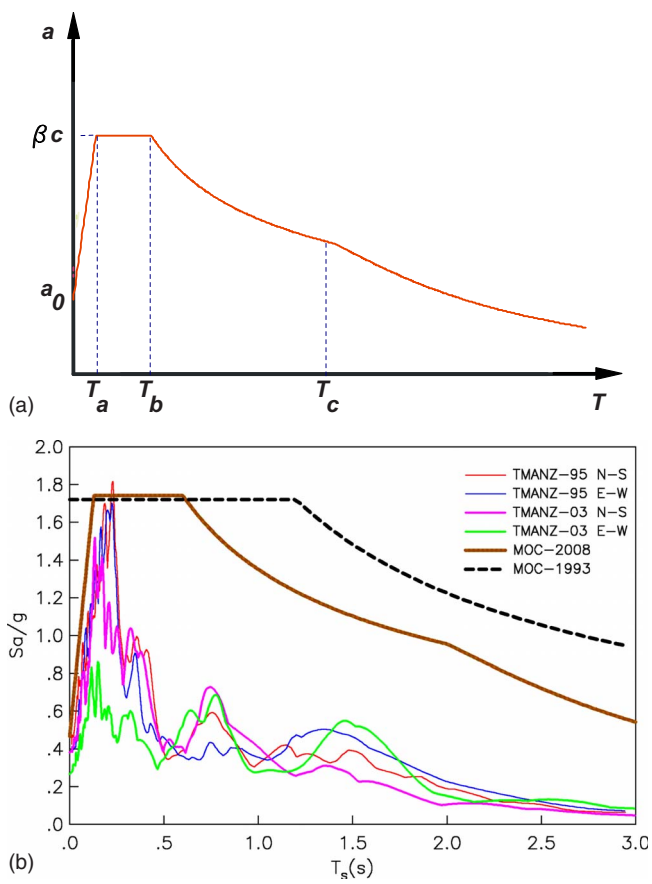

Figure 5. Schematic representation of elastic acceleration design spectrum for MOC-2008.

parameter that defines the descending branch of the acceleration spectrum when $T_e \geq T_c$ and β is a damping factor. The control period T_c and the parameters r and k that define the descending branch of the acceleration spectrum are defined as follows:

$$T_c = \begin{cases} 2s & if\ T_b < 2s \\ T_b & if\ T_b \geqslant 2s \end{cases} \tag{8}$$

$$r = T_s; \quad 0.5 \leqslant r \leqslant 1.0 \tag{9}$$

$$k = \begin{cases} min\{1.5, 2 - T_s\} & if\ T_s \leqslant 1.65s \\ max\{0.35, \beta/F_r\} & if\ T_s > 1.65s \end{cases} \tag{10}$$

where all terms have been already defined.

The damping factor β allow modifying the spectral ordinates for damping ratios different from 5% to account primarily for soil-structure interaction effects and/or supplemental damping and is defined by the following expressions:

$$\beta = \left(\frac{0.05}{\zeta_e}\right)^{\lambda} \ where\ \lambda = \begin{Bmatrix} 0.35 & if\ T_e < T_c \\ 0.35\left(\frac{T_e}{T_c}\right) & if\ T_e \geqslant T_c \end{Bmatrix} \tag{11}$$

where ζ_e is the effective (target) damping of interest for the structural system. This proposal is based on a study conducted by Ruiz et al. (2008) for structural systems that may develop a reduced to moderate nonlinear response.

For important facilities (e.g., public schools and hospitals, structures in Group A), the spectral acceleration ordinates (a) given in Equation 7 should be multiplied by an importance factor I = 1.5. For essential facilities (for example, nuclear power plants, structures of group A^+) an importance factor I $\geq$1.5 should be used depending on the hazard of the site (MOC-2008 2008).

The elastic design spectrum for the MCE obtained from MOC-2008 for Manzanillo Powerplant site (TMANZ) is compared in Figure 5b with the elastic design spectrum obtained from MOC-93 and with the elastic response spectra obtained for that site during the 9 October 1995 Manzanillo Earthquake (M_w=8.0) and the 21 January 2003 Tecomán Earthquake (M_w=7.8). One can observe that the design spectrum obtained with MOC-2008 is more realistic and less conservative for periods $T_e > 0.7$ s than the design spectrum previously defined by MOC-93.

ACCELERATION SPECTRUM FOR SERVICEABILITY LIMIT STATE

The acceleration design spectrum to check the serviceability of the damage state was obtained using probabilistic cost-benefit analyses, where the fundamental period of the structure is optimized by measuring damage through story drifts. Once the optimized fundamental period (T_k) is obtained, the spectral acceleration required to reach the allowable story drift (γ_0) is assessed, as well as its corresponding return period.

It was found that, for practical purposes, the acceleration design spectrum to check

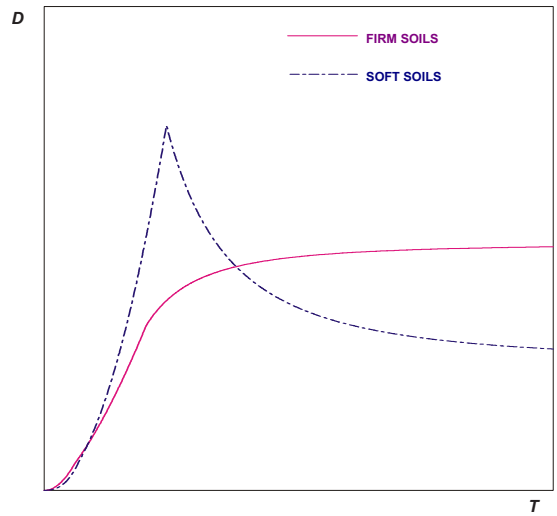

Figure 6. Schematic representation of elastic displacement design spectra for MOC-2008.

for the serviceability performance level can be obtained indirectly from the one defined for the collapse prevention level divided by a factor of 5.5 and assuming linear behavior for the soil profile, therefore $F_{nl}^{a}=F_{nl}^{T}=1.0$. Then, a_0, T_a and T_b are computed as:

$$a_0 = \frac{F_s a_0^r}{5.5g} \tag{12}$$

$$T_a = 0.35T_s \geqslant 0.1s \tag{13}$$

$$T_b = 1.2T_s \geqslant 0.6s \tag{14}$$

The remaining parameters used to define the acceleration design spectrum remain unchanged. The described spectrum should be used to review damage prevention (linear behavior) for the structural system for both essential and standard occupancy building structures. The importance factor is neglected for important and essential facilities (no amplification for this concept).

DISPLACEMENT DESIGN SPECTRUM

Displacement design spectrum $S_d(T_e)$ is obtained indirectly from acceleration design spectrum based upon standard relation from structural dynamics:

$$S_d(T_e) = \frac{T_e^2}{4\pi^2} S_a(T_e) \tag{15}$$

It can be demonstrated that when $T_e \rightarrow \infty$, the maximum spectral displacement converges to the peak ground displacement D_{max}, as schematically depicted in Figure 6.

When $k<1$, the maximum spectral displacement occurs when $T_e=T_c$ and is given by:

$$\mathrm{S}_{\mathrm{d}_{\max}}=\beta\frac{\mathrm{c}T_c^2}{4\pi^2}\left(\frac{T_b}{T_c}\right)^{1/2}g \tag{16}$$

If $k\geq 1$ the maximum spectral displacement occurs when $T_e\rightarrow\infty$ and converges to the peak ground displacement D_{max}, that it is independent of the damping coefficient β, and is given by:

$$\mathrm{D}_{\max}=\mathrm{k}\frac{\mathrm{c}T_c^2}{4\pi^2}\left(\frac{T_b}{T_c}\right)^{1/2}g \tag{17}$$

Therefore, from Equations 16 and 17 it is clear that the parameter k has a physical meaning as it is the ratio between the peak ground displacement and the maximum spectral displacement modified by the damping coefficient β:

$$k=\frac{D_{\max}}{S_{d_{\max}}/\beta} \tag{18}$$

The shape of the displacement design spectrum depends on several parameters that define the absence or presence of site effects, but three of them are particularly important: the k parameter, the site factor F_s and the fundamental site period T_s. For relatively firm to firm soils or rocks ($T_s<0.8$ s, $F_s<1.5$), design displacement spectrum converges to the peak ground displacement in an asymptotic manner ("firm soils," Figure 6), whereas for relatively soft to very soft soils ($T_s>1.3$ s, $F_s>1.5$), design displacement spectrum reaches a peak value when $T_e=T_c$ and decay to converge to the ground displacement ("soft soils," Figure 6).

REDUCTION OF ELASTIC RESPONSE PARAMETERS FOR DESIGN

For the sake of clarity in the design process, there is an important conceptual adjustment in the reduction of elastic response parameters for design in MOC-2008 with respect to that in MOC-93. In the earlier code, the elastic design spectra were reduced by dividing the spectral ordinates by a somewhat obscure reductive seismic force factor Q' that accounted for everything (ductility, redundancy, overstrength, etc.).

In MOC-2008, it is established that for the collapse prevention limit state, the reduced spectral ordinates a' should be computed as (Figure 7):

$$a'=a(\beta)/Q'R\rho \tag{19}$$

where Q' is a seismic reduction force factor that accounts primarily for ductility (deformation) capacity, R is an overstrength factor that depends on the structural system and the structural period, and ρ is the redundancy factor; at this time it is essentially a correction factor of Q' and R to account for is the redundancy of the lateral-load structural system in a given direction of analysis.

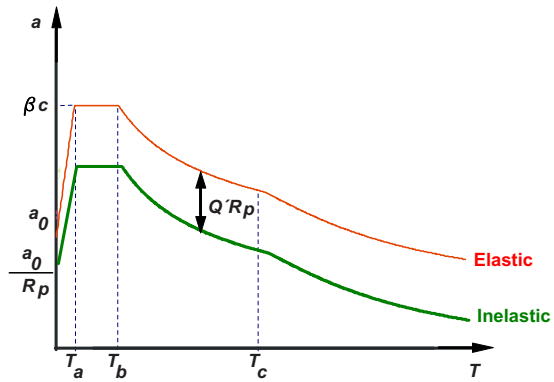

Figure 7. Schematic representation of inelastic acceleration design spectrum for MOC-2008.

For structural systems with stiffness and/or strength degrading characteristics under cyclic loading located in soft soils, the reduced spectral ordinates a' should be computed as

$$a' = a(\beta)A_{cd}/Q'R\rho \tag{20}$$

where A_{cd} is a modification factor to account for stiffness and/or strength degradation in soft soils.

All these parameters are explained in greater detail below.

SEISMIC RESPONSE MODIFICATION FACTOR *Q*

The definition, requirements, and proposed values for the seismic response modification factor Q remain practically unchanged in MOC-2008. The values for Q established by all modern Mexican codes are 1, 1.5, 2, 3, and 4, and they depend on the selected structural system (Tena-Colunga 1999). For example, in order to use Q=3 or Q =4 for dual systems, the designer has to demonstrate that the dual system satisfies specific requirements related to the strength and stiffness balances of frames with respect to shear walls and/or braced frames. The Q factors of Mexican codes account primarily for the deformation capacity of the structural system and its relation with its displacement ductility, redundancy and overstrength.

DUCTILITY REDUCTION FACTOR *Q′*

In the MOC-2008 code, the seismic reduction force factor Q' stands now only for the approximate ductility deformation capacity of the selected structural system, given in terms of the seismic response modification factor Q. For any given structural system, Q' should be computed as follows:

$$Q' = \begin{cases} 1 + (Q-1)\sqrt{\dfrac{\beta}{k}\left(\dfrac{T_c}{T_b}\right)^r}\dfrac{T_e}{T_c}; & \text{if } T_e \leqslant T_b \\ 1 + (Q-1)\sqrt{\dfrac{\beta}{k}\left(\dfrac{T_c}{T_e}\right)^r}\dfrac{T_e}{T_c}; & \text{if } T_b < T_e \leqslant T_c \\ 1 + (Q-1)\sqrt{\beta p/k}; & \text{if } T_e > T_c \end{cases} \tag{21}$$

where p is a factor to define the descendent curve of the inelastic response spectrum given by:

$$p = k + (1-k)\left(\frac{T_c}{T_e}\right)^2 \tag{22}$$

and all remaining terms have already been defined.

Therefore, it can be observed from Equation 21 that the proposed Q' factor is not constant and depends on the structural period T_e and the site period T_s (in terms of parameters T_a, T_c and k). In fact, the proposal for Q' mostly coincides with the proposal available in Appendix A of the seismic provisions for current Mexico's Federal District Code (NTCS-2004 2004). This proposal is based on the study of SDOF systems with elastoplastic hysteretic behavior (for example, Krawinkler and Rahnama 1992, Miranda 1993, Miranda and Bertero 1994, Ordaz and Pérez-Rocha 1998), where Q' is the ratio between the minimum strength required to limit a structural system to an elastic response $C(T_e, 1)$ and the strength required for a structural system to limit its ductility capacity to a given Q value $\mathrm{C}(T_e, Q)$, this is:

$$Q'(T_e, Q) = \frac{C(T_e, 1)}{C(T_e, Q)} \tag{23}$$

Ordaz and Pérez-Rocha (1998) showed that, in general terms, Q' depends on the ratio between the spectral displacement $Sd(T_e)$ and the peak ground displacement D_{max} as:

$$Q'(T_e, Q) = 1 + (Q-1)\left(\frac{Sd(T_e)}{D_{\max}}\right)^{\alpha} \tag{24}$$

where $\alpha \approx 0.5$.

The proposed Equations 21 and 22 are a simplified version of Equation 24. A detailed explanation on how these expressions were derived are presented elsewhere (MOC-2008 2008) and briefly summarized here.

For $T_e = 0$, $Q'(T_e, Q) = 1$ independently of the proposed Q value. For the sake of simplicity, a linear variation is taken between $Q' = 1$ for $T_e = 0$ and $Q' = Q'_{max}$ for $T_e = T_a$. Then, Q'_{max} is obtained when $Sd(T_e)$ is maximum that occurs when $T_e = T_c$. Therefore, it can be demonstrated from Equations 18 and 24 that if $\alpha = 0.5$, then

$$Q'_{\max} = 1 + (Q-1)\sqrt{\frac{\beta}{k}} \tag{25}$$

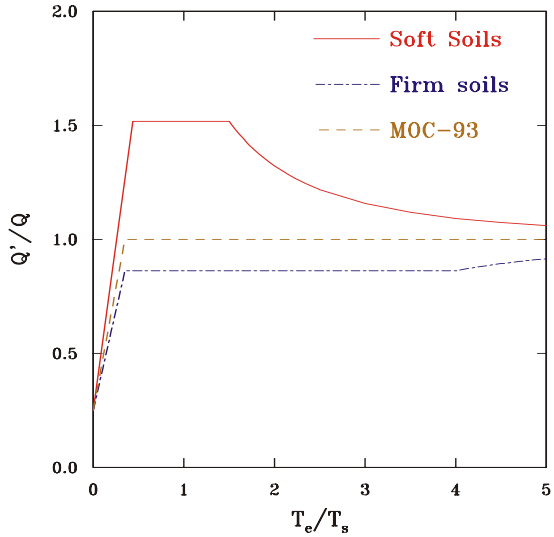

Figure 8. Typical normalized Q'/Q curves for MOC-2008 and MOC-93.

The equation for the descending branch ($T_e \geqslant T_c$) is obtained taking into consideration that, from basic structural dynamic principles, for long periods ($T_e \rightarrow \infty$) the associated elastic displacement spectrum converges to the peak ground displacement D_{max} and also Q' should converge to Q.

As it can be deducted from Equations 21 and 22, several Q' curves can be obtained for MOC-2008. Typical normalized Q'/Q ratio vs the normalized T_e/T_s ratio curves for soft soil sites and firm soil sites are depicted in Figure 8 and compared with that one defined by MOC-93. These curves were computed considering Q=4 for all curves; in addition, T_s=2 s was taken for soft soils and T_s=0.6 s for firm soils for MOC-2008.

In contrast to the proposal in MOC-93 (i.e., Tena-Colunga 1999), it is observed in Figure 8 that Q' can be larger than Q, this is, $Q'/Q > 1$ in a given period range. These higher values are obtained for soft soil sites ($k < 1$); this has been reported before in previous studies that considered a large number of acceleration records typical of soft soils (Miranda 1993, Ordaz and Pérez-Rocha 1998). In contrast, for firm soils, Q' is usually smaller than or equal to Q, this is, $Q'/Q < 1$ in a wide period range. For all soil profile types, Q' converges to Q as $T_e \rightarrow \infty$, this is, $Q'/Q \rightarrow 1$ as $T_e/T_s \rightarrow \infty$ (Figure 8).

OVERSTRENGTH REDUCTION FACTOR *R*

The introduction of an overstrength reduction factor R in MOC-2008 is a new concept for this manual and was not included in MOC-93. However, the R factor was first introduced in the seismic codes of Mexico in Mexico's Federal District Code (NTCM-2004 2004). In fact, the proposal of the R factor for MOC-2008 is an improved version of the one presented in NTCM-2004.

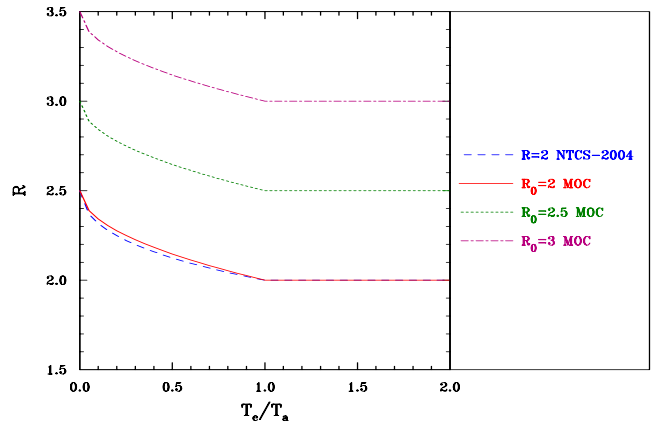

Figure 9. Overstrength reduction factors R for MOC-2008.

The proposal for R in MOC-2008 is given by the following equations:

$$R = \begin{cases} R_0 + 0.5(1 - \sqrt{T_e/T_a}); & \text{if } T_e \leqslant T_a \\ R_0; & \text{if } T_e > T_a \end{cases} \tag{26}$$

where R_0 is an overstrength index value that depends on the structural system. For example, $R_0=2$ for ordinary and intermediate moment-resisting frames, ordinary moment-resisting braced frames and confined masonry wall structures made with hollow units (ungrouted or partially grouted); $R_0=2.5$ for special moment-resisting frames, intermediate moment-resisting braced frames, and confined masonry wall structures made with solid units; $R_0=3.0$ is for dual systems built with special moment-resisting frame connections.

The proposed R curves for MOC-2008 are depicted in Figure 9, where they are compared with the R curve proposed in NTCM-2004. It can be observed that the overstrength reduction factor R in Mexican codes is period dependent. This is done because it is recognized that for squatty, short period structures ($T_e/T_a < 1$), the impact of gravitational load combinations in the design provides structures with additional lateral strength.

In NTCM-2004, R is independent of the structural system (Figure 9). This conceptual shortcoming is fixed in MOC-2008, where it is also recognized that the overstrength that a structure can develop under earthquake loading strongly depends on the structural system, as it is done in other modern seismic codes (i.e., ASCE-7 2005, IBC-2006 2006).

The proposed values for R_0 are based on the following: (1) analytical studies conducted in Mexico for some structural systems such as ordinary and special moment-resisting RC and steel frames (i.e., Tena-Colunga et al. 2008) and special moment-resisting concentric braced frames (i.e., Godínez-Domínguez and Tena-Colunga 2008); (2) experimental studies (shaking table tests) conducted for confined masonry structures

(i.e., Barragán et al. 2008); and (3) proposed values of NTCM-2004 and U.S. codes, such as ASCE 7-05 and IBC-2006. Of course, the current proposal for MOC-2008 has room for improvement as more reliable data regarding the assessment of overstrength for different structural systems will be available in the future.

REDUNDANCY FACTOR ρ

The introduction of a redundancy factor ρ in MOC-2008 is a new concept for Mexican seismic design codes, not only for MOC-2008. The purpose of this "new" factor is to recognize directly that structural systems are able to develop more strength and increase their deformation capacity as they become more redundant. This fact is well-known by the structural engineering community worldwide. However, it seems some seismic codes have come up short before, by not recognizing that a more redundant structural system under lateral loading should be allowed to be designed with higher reductions and that weakly-redundant systems should be penalized and be designed with smaller reductions.

In MOC-2008, ρ is a factor that basically corrects the previous assessment of the overstrength factor R, as most of the available studies where R has been computed have been mostly done in 2-D models with different degrees of redundancy. In addition, this factor takes into account unfavorable performances of weakly-redundant structures in strong earthquakes occurred worldwide in the last 30 years.

The proposed values for ρ in MOC-2008 are the following:

- $\rho = 0.8$ for structures with at least two earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames are one-bay frames (or equivalent structural systems).
- $\rho = 1$ for structures with at least two earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames have at least two bays (or equivalent structural systems).
- $\rho = 1.25$ for structures with at least three earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames have at least three bays (or equivalent structural systems).

As one can observe, one-bay framed buildings are now penalized in the design because they are weakly redundant, and their observed performances during strong earthquakes have been poor; some collapses or partial collapses have been documented in reconnaissance reports (e.g., Figure 10). In addition, numerical collapses of such structures designed according to modern building codes have also been reported (i.e., Tena-Colunga 2004). Finally, smaller R factors have been reported in the literature for such frames ($R = 1.5$, Terán-Gilmore 2005).

The structural systems where $\rho = 1$ is proposed correspond to those considered in most of the consulted studies to define target values for the overstrength factor R. The proposal for $\rho = 1.25$ is based in some recent studies where parallel frames of these characteristics have been studied and where higher R factors were obtained (i.e., Tena-Colunga et al. 2008). It is also worth noting that the value of ρ may vary in each main orthogonal direction.

Figure 10. One-bay framed building severely damaged during 1995 Kobe earthquake (http://www.eqe.com/publications/kobe/building.htm).

The assessment of the ρ factor for a given structure is straight-forward and it is illustrated with the buildings which plans are depicted in Figure 11. For the building plan depicted in Figure 11a, $\rho = 0.8$ should be taken in the Y direction as it has eight parallel one-bay frames, whereas in the X direction, $\rho = 1$ because it has two parallel seven-bay frames. In contrast, for the building plan depicted in Figure 11b, $\rho = 1$ should be taken in the Y direction as it has eight parallel two-bay frames, whereas in the X direction, $\rho = 1.25$ because it has three parallel seven-bay frames.

This simple example illustrates the philosophy behind the new ρ factor. *A-priori*, most engineers would agree that the building plan depicted in Figure 11b is more redundant than the building plan depicted in Figure 11a. Former Mexican codes did not recognize directly this fact for their seismic design, now MOC-2008 does. It is hoped that this approach would help structural engineers to promote the use of more redundant structural systems in zones of high earthquake hazard and to limit or avoid the use of weakly-redundant structures (i.e., Figure 10 and 11a).

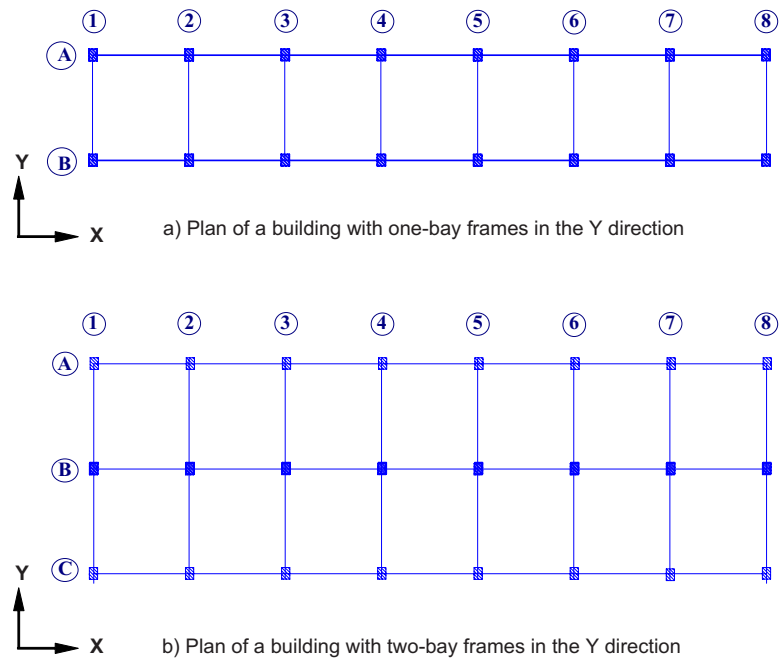

Figure 11. Sample buildings to illustrate the assessment of the ρ factor.

Although the values proposed for the ρ factor are based on some studies, they are also based on past experiences and intuition. Therefore, there is room for improvement in assessing these values with specific-oriented research studies for future revision of this manual.

FACTOR FOR DEGRADING HYSTERETIC BEHAVIOR A_{cd}

The introduction of a correction factor A_{cd} for structures with degrading hysteretic behavior (stiffness and/or strength) located in soft soils is also a new concept for the seismic design codes of Mexico, not only for MOC-2008. The A_{cd} factor is computed as (Figure 12):

$$A_{cd} = 0.8 + \frac{1}{2 + 3\left|2\frac{T_e}{T_s} - 1\right|^5} \tag{27}$$

The A_{cd} factor is now introduced as it has been shown that low-cycle fatigue is very important in the seismic behavior of stiffness and strength degrading systems such as RC and masonry structures located in soft soils where long durations of the earthquake motions are observed, such as in the lakebed zone of Mexico City. There are also other soft soil sites in zones of high seismic risk in Mexico besides Mexico City—for example, Ciudad Guzmán in the state of Jalisco. The proposal is based in the study pre-

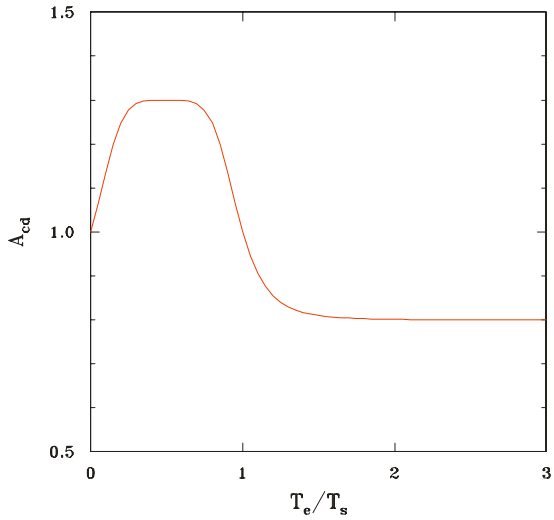

Figure 12. Correction factor for degrading hysteretic behavior A_{cd} for MOC-2008.

sented by Terán-Gilmore (2005) that incorporates the findings of previous studies conducted in Mexico (i.e., Terán-Gilmore and Espinoza 2003) and worldwide (i.e., Fajfar 1992).

CONDITIONS OF STRUCTURAL REGULARITY

As in previous versions (MOC-93 1993, Tena-Colunga 1999), MOC-2008 defines 11 conditions of regularity that building structures must satisfy to directly use the reductive seismic force factor Q'. However, some adjustments were made in the definition and design of irregular structures, mostly coinciding with what is currently proposed in Mexico's Federal District Code (NTCS-2004 2004). The modifications were made based on a comprehensive review of research studies reported worldwide and on specific studies conducted in Mexico to review the original design strategy for irregular buildings proposed by Mexico's Federal District Code (i.e., Tena-Colunga 1997, 2003, and 2004).

Mostly, the original 11 regularity conditions remain almost the same (Tena-Colunga 1999), but the statement devoted to prevent a soft-story condition (Condition # 10) was redefined and now is more conservative than in previous versions, taking into account, among other material, recent research findings summarized in Tena-Colunga (2003). The new definition of regularity Condition # 10 is the following:

"10. The lateral shear stiffness or strength of any story shall not exceed more than 50% the shear stiffness or strength of the adjacent story below the one in consideration. The top story is exempt from this requirement."

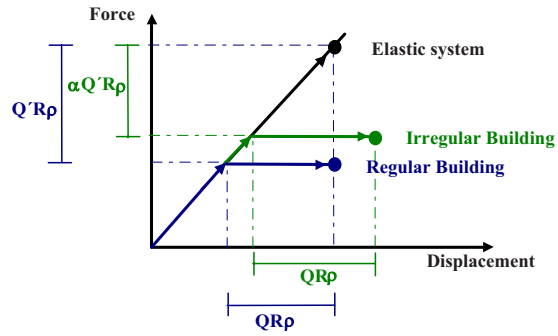

Figure 13. Schematic illustration of the design procedure for regular and irregular buildings according to MOC-2008.

If a building structure satisfies all 11 conditions of structural regularity, it is defined as a regular structure, so Q' remains unchanged. However, if at least one condition of structural regularity is not satisfied, the building is defined as irregular structure, and then Q' is reduced for design purposes as follows:

$$Q'_{irregular} = \alpha Q'_{regular} \tag{28}$$

where α is a corrective reduction factor that depends on the degree of irregularity according to MOC-2008. If a building does not satisfy one regularity condition (from those numbered 1 to 9, i.e., Tena-Colunga 1999), then $\alpha = 0.9$. If a building does not satisfy regularity condition 10 (soft story) or 11 (torsion), or two or more of the remaining regularity conditions (1 to 9) are not satisfied, then $\alpha = 0.8$. If a building has a strong irregularity, then $\alpha = 0.7$.

Strong irregularity conditions are defined as follows: (1) If conditions 10 (soft story) and 11 (torsion) are not satisfied simultaneously, (2) a strong torsional irregularity is met, evaluated in terms of a static eccentricity greater than 20 percent of the plan dimension in the given direction of analysis ($e_s > 0.20L$), (b) a strong soft story condition is found, where the lateral shear stiffness or strength of any story exceeds more than 100 percent the shear stiffness or strength of the adjacent story below the one in consideration.

The conceptual adjustment for the design of irregular buildings in Mexican seismic codes, MOC-2008 included, is illustrated in Figure 13. For design purposes, irregular buildings must be designed for higher forces but required to comply with the lateral story drift criteria specified for regular buildings.

METHODS OF ANALYSIS

As in previous versions, three methods of seismic analysis are formally described in MOC-2008: a) the simplified method, b) the static method and, c) dynamic methods. The

general description of the methods is available in English language elsewhere (Tena-Colunga 1999). In the following sections the more important updates for each method will be briefly described.

SIMPLIFIED METHOD

The simplified method is allowed for low-rise (up to five stories or less than 13 m in height), bearing-wall, shear-wall structures with no mass or stiffness eccentricities, diaphragm flexibility, and/or slenderness effects, where seismic forces are determined using reduced seismic coefficients specified according to the soil profile, the height of the structure and the type of bearing wall used. Seismic forces are distributed among structural elements according to their shear stiffnesses. A more detailed description of the simplified method and its theoretical background is presented elsewhere (Tena-Colunga and Cano-Licona 2007).

The simplified method has been extensively reviewed recently and based upon these parametric studies (i.e., Tena-Colunga and Cano-Licona 2007, Tena-Colunga and López-Blancas 2006), important adjustments were made. In particular, new, improved effective shear area factors F_{AE} are proposed for two different limit states for the structure.

For elastic behavior related to the service limit state:

$$F_{AE} = \begin{cases} 1.5 + \dfrac{h'}{L} - 1.5\left(\dfrac{h'}{L}\right)^2 & \text{if } \dfrac{h'}{L} \leqslant 1 \\ 2.2 - 1.5\dfrac{h'}{L} + 0.3\left(\dfrac{h'}{L}\right)^2 & \text{if } 1 \leqslant \dfrac{h'}{L} \leqslant 2.5 \end{cases} \tag{29}$$

and for the collapse prevention limit state:

$$F_{AE} = 0.6 + 0.6\frac{h'}{L} - 0.3\left(\frac{h'}{L}\right)^2 + 0.05\left(\frac{h'}{L}\right)^3 \quad \text{if } \frac{h'}{L} \leqslant 2.5 \tag{30}$$

where h' and L are respectively the height and the length of the wall under consideration.

STATIC METHOD

In the static force procedure the seismic forces are obtained assuming that mass accelerations vary linearly with height. However, a correcting procedure for the lateral load distribution to account for higher mode effects is established for structures where the fundamental period T_e is greater than T_b. Adjustments were done to define the design base shear to account also for the newly introduced modification factors R, ρ and A_{cd}, besides Q'.

Special provisions are specified to account for P-Δ effects, directional effects, torsional and overturning moments and asymmetric strength capacity in the two principal axes of the building. In this regard, there are important updates to account for torsion,

for directional effects and asymmetric strength capacity, based upon a comprehensive review of studies conducted in Mexico and worldwide, as briefly described in following sections.

Torsional Effects

The amplified design static eccentricities to account for torsional effects are now given by the following equations:

$$e_n^+ = 1.5e_n + 0.05b_n \tag{31}$$

$$e_n^- = 0.5e_n - 0.05b_n \tag{32}$$

where e_n is the computed static eccentricity between the center of mass ant the center of rigidity at interstory n in the direction of interest and b_n is the maximum plan dimension of interstory n of the building perpendicular to the direction under consideration.

Therefore, some changes were made with respect to MOC-93. The most notable one is that the coefficient proposed in MOC-2008 to account for accidental torsion is 0.05, instead of 0.10 that have been used in Mexican codes since 1987 (i.e., Tena-Colunga 1999). This adjustment was done taking into account that results of several research studies conducted worldwide suggested that the 0.10 coefficient proposed in previous Mexican codes was too conservative and that the 0.05 coefficient used in other codes might be more adequate (i.e., De la Llera and Chopra 1995, Wong and Tso 1995, Chandler and Duan 1997, Harasimowicz and Goel 1998, Tso and Smith 1999, De la Colina 1999 and 2003). The second adjustment is the new proposal for the secondary eccentricity (Equation 32), because the dynamic factor is reduced from 1.0 to 0.5, taking into account the works presented by Wong and Tso (1995) and De la Colina (2003).

Directional Effects

Buildings should be analyzed for three orthogonal components of the ground motions: two horizontal and one vertical. This requirement is not new, as it is also established in MOC-93. However, the combination for directional effects is completely different in MOC-2008 from the one outlined in MOC-93 (Tena-Colunga 1999).

Instead of using the 100% +30% combination rule for the two horizontal orthogonal components and take the vertical component of the ground motion as two-thirds of the largest horizontal component, in MOC-2008, all response quantities of interest S (displacements, internal forces, etc.) should be obtained by combining each orthogonal response using the square root of the sum of the squares in 3-D, this is:

$$S = \sqrt{S_x^2 + S_y^2 + S_z^2} \tag{33}$$

where S_x, S_y and S_z are respectively the response quantity of interest associated to the largest horizontal, the smallest horizontal and the vertical components for the ground motions. This adjustment was done taking into account, among other studies, the one presented by Hernández and López (2002).

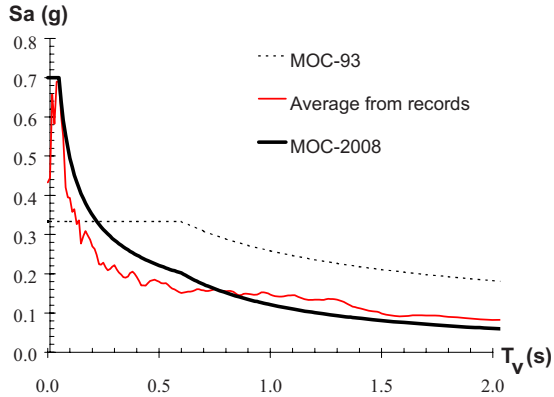

Figure 14. Comparison of vertical spectra obtained for zone D-I of former MOC-93 (adapted from Perea and Esteva 2005).

In MOC-2008, the vertical component of the ground motion E_v is now defined as a vertical spectrum computed from the largest horizontal ground motion E_h as follows:

$$E_v = \begin{cases} 1.4E_h & if\ T_v < 0.05s \\ 1.4\left(\dfrac{0.05}{T_v}\right)^{2/3} E_h & if\ T_v \geqslant 0.05s \end{cases} \tag{34}$$

where T_v is the natural period of the structure in the vertical direction.

The proposed vertical spectrum is based on a comprehensive study conducted by Perea and Esteva (2005) and takes care of previous shortcomings, mainly: (1) the natural periods for the structure in horizontal and vertical directions are different, then, they are uncoupled and, (2) vertical ground motions usually have a richer high frequency content than horizontal ground motions, and these differences are more evident as soil profiles becomes softer.

The new vertical spectrum proposed for MOC-2008 is compared in Figure 14 with the former one obtained for zone D-1 of MOC-93 and with the average spectrum for a family of records studied by Perea and Esteva (2005) that are compatible with former zone D-I of MOC-93 (i.e., Tena-Colunga 1999). It is observed that the new proposal is more rational and conservative enough. Former provisions in MOC-93 were unconservative for structures with $T_v < 0.2$ s, but excessively conservative for structures with $T_v > 0.2$ s.

The action of the vertical ground component can be neglected for buildings founded in soft soils located more than 80 km from an active fault.

Asymmetric Strength

Updated provisions are given to account for the design of structures with strength asymmetries, that is, their strength-deformation capacity envelope curves in positive and

negative directions are different. The new proposal is based in the study presented by Terán-Gilmore and Arroyo (2005) and provides improved equations for firm soils and soft soil sites.

DYNAMIC METHOD

According to MOC-2008, the following options can be used for dynamic analysis: (1) response-spectrum analysis and (2) time-history analysis. The general recommendations to use both methods remain practically unchanged from previous MOC-93 version (i.e., Tena-Colunga 1999). However, there are some fine adjustments for both procedures.

In the response spectrum analysis, accidental torsional effects are accounted in the design by translating $\pm 0.05b_n$ as the centers of mass at each level for each horizontal direction of analysis. This recommendation would require the use of four additional models to assess the impact of the modal coupling due to accidental torsion. As an option, one can use a single model if the line of action of the lateral forces obtained from the response spectrum analysis is translated $\pm 0.05b_n$ at each level, this is, a static torque is applied as an approximation of the modal coupling due to accidental torsion. In addition, SRSS or CQC combination procedures are specified; however, it is clearly stated that SRSS method can only be used if the natural periods for the building in each given direction differ in 10% or more.

For time-history analysis, it is clearly specified that the acceleration ground motions to include in the analyses must be fully compatible with the seismic hazard for the site of interest, as outlined in an specialized section of the manual. At least four representative trios of representative ground motions should be included in the analyses. The nonlinear characteristics of the structural system and their associated uncertainties shall be taken into account.

For either option of dynamic analysis, the corresponding design base shear shall not be less than 80% of the base shear determined by the static force procedure.

REVIEW OF LIMIT STATES

In MOC-2008, four limit states have to be reviewed for seismic loading: (1) story drift limits for the service earthquake, (2) story drift limits for collapse prevention under the maximum credible earthquake (MCE), (3) glass gaps under the MCE and, (4) building separations under the MCE.

The recommendations for glass gaps and buildings separations (i.e., Tena-Colunga 1999) remain unchanged from the previous code.

The review of drift limits for the service earthquake is new in MOC-2008, but not in Mexican codes, as this review is specified in NTCS-2004. In fact, the proposal in MOC-2008 is based upon what it is defined in NTCS-2004. For the service earthquake, buildings should remain elastic, so the proposed story drift limits are $\Delta_{ser} \leq 0.002$ if non-structural elements are not properly separated from the structural system and $\Delta_{ser} \leq 0.004$ if non-structural elements are properly separated from the structural system.

For the collapse prevention state, story drifts are obtained from the displacements

Table 3. Story drift limits of MOC-2008 for collapse prevention under the MCE

STRUCTURAL SYSTEM	DRIFT LIMIT
Special moment-resisting (ductile) reinforced concrete (RC) frames (Q = 3 or 4)	0.030
Special moment-resisting (ductile) steel frames (Q = 3 or 4)	0.030
Ordinary or intermediate moment-resisting RC or steel frames (Q = 1 or 2)	0.015
Flat slab frame systems without walls or bracing	0.015
Eccentric braced steel frames	0.020
Concentric braced RC or steel frames	0.015
Dual system: RC walls with ductile RC frames (Q = 3)	0.015
Dual system: RC walls with ordinary or intermediate moment-resisting RC frames (Q = 1 or 2)	0.010
Masonry infill panels	0.006
Confined masonry wall system made with solid units and with horizontal steel reinforcement (joint reinforcement or wire mesh)	0.004
Confined masonry wall systems: (a) walls made with solid units and, (b) walls made with hollow units and with horizontal steel reinforcement (joint reinforcement or wire mesh)	0.003
Combined and confined masonry wall systems	0.003
Confined masonry wall system made with hollow units and without horizontal steel reinforcement (joint reinforcement or wire mesh)	0.002
Unreinforced and unconfined masonry wall systems	**0.0015**

from linear analysis for the reduced spectrum multiplied by $QR\rho$ (Fig. 13). In contrast with MOC-93, where the story drift limits were not defined in terms of the structural system (i.e., Tena-Colunga 1999), the story drift limits defined in MOC-2008 for collapse prevention under the MCE are a function of the structural system. The proposed drift limits are given in Table 3. Note that the proposed drift limits mostly coincide with what is recommended in U.S. codes (e.g., ASCE-7-05 2005 and IBC-2006 2006, NTCS-2004 2004). However, it is worth noting that the proposed values for confined masonry structures are based upon experimental studies conducted in Mexico and coincide with the masonry guidelines (NTCM-2004 2004) of Mexico's Federal District Code.

SOIL-STRUCTURE INTERACTION

If a site-specific design spectrum is to be used for computing the lateral loads and the corresponding displacements of the building, the effects of soil-structure interaction (SSI) should be accounted for using the additional provisions specified by the code, which are based on the studies presented by Avilés and Pérez-Rocha (2003, 2005a, 2005b). The use of the recommended SSI provisions will increase or decrease the required strength with respect to the fixed-base value, depending primarily on the relation between the structure and site periods. The lateral displacement will undergo additional changes due to the contribution of the foundation's rotation.

The base-shear coefficients $\tilde{a}'$ and a' with and without SSI are computed in the following way:

$$\tilde{a}' = \frac{a(\tilde{T}_e, \tilde{\zeta}_e)}{R(T_e)Q'(\tilde{T}_e, \tilde{Q})} \tag{35}$$

$$a' = \frac{a(T_e, \zeta_e)}{R(T_e)Q'(T_e, Q)} \tag{36}$$

The two coefficients $\tilde{a}'$ and a' are used to emphasize the fact that the former should be evaluated for the effective, period $\tilde{T}_e$, damping $\tilde{\zeta}_e$ and ductility $\tilde{Q}$ of the system, whereas the latter should be evaluated for the fixed-base values of T_e, ζ_e and Q. Specific expressions are given in the code to compute the effective parameters of the system. Notice that the overstrength reduction factor R is independent of SSI.

MODIFIED BASE SHEAR

The recommended SSI provisions may be used either with the static analysis procedure or with the modal analysis procedure. In the first case, the base shear modified by SSI can be determined as follows:

$$\tilde{V}_0 = a' W_0 - (a' - \tilde{a}') W_e \tag{37}$$

where W_0 is the total weight of the structure and $W_e = 0.7 W_0$. This expression is similar to that used in the ATC and FEMA codes, except that it incorporates the effects of SSI on the structural ductility, a subject ignored so far in all building codes worldwide.

The interaction factor $\tilde{V}_0 / V_0$, with $V_0 = a' W_0$ being the fixed-base shear, should be used to modify the design earthquake forces computed without SSI to obtain the corresponding forces with SSI. In view of many uncertainties in the model and its parameters, the value of $\tilde{V}_0 / V_0$ cannot be taken less than 0.75, nor greater than 1.25. In general, the former condition occurs when the structure period is longer than the site period, while the latter when the structure period is shorter than the site period.

As it is common practice, the effects of SSI are accounted for only on the fundamental mode of vibration. Thus, when applying the modal analysis procedure, the base shear associated to the first mode can be modified by SSI as follows:

$$\tilde{V}_1 = \tilde{a}' W_1 \tag{38}$$

where W_1 is the effective weight of the structure when vibrating in its fixed-base fundamental mode. The contribution of the higher modes and the combination of the modal responses are performed as for structures fixed at their base.

MODIFIED LATERAL DISPLACEMENT

In terms of the ratio $\tilde{V}_0/V_0$, the displacement of the structure relative to the ground can be expressed as

$$\tilde{X}_m = \frac{\tilde{V}_o}{K_e}Q + \frac{\tilde{V}_o(H_e+D)^2}{K_r} = \frac{\tilde{V}_o}{V_o}\left(X_e + (H_e+D)\frac{M_o}{K_r}\right) \tag{39}$$

where $X_e=(V_0/K_e)Q$ is the lateral displacement and $M_0=V_0(H_e+D)$ the overturning moment of the fixed-base structure, with K_e being the lateral stiffness of the structure, H_e its effective height and D the foundation depth; K_r is the rocking stiffness of the foundation.

CONCLUDING REMARKS

This paper summarizes the most relevant changes in the seismic provisions for buildings of the Manual for Civil Structures (MOC-2008), a model design code in Mexico, and their relations to research efforts conducted within Mexico and worldwide to improve the seismic design of building structures. One goal was to make the guidelines as transparent as possible to users, so that the design process will be clearer and enriching to structural engineers.

In the MOC-2008 code, seismic hazard in Mexico is defined as a continuum function where peak accelerations in rock are defined. These peak accelerations are 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. All known earthquakes sources for the different regions of seismic risk of Mexico and their maximum credible earthquake (MCE) scenarios expected to use updated information were taken into account. The seismic hazard was evaluated using both deterministic and probabilistic approaches.

As a result, the proposed design spectrum consists in an infinite number of discrete functions within Mexico, and it is proposed in such a way that is both acceleration and displacement compatible, this is, for long periods, the displacement design spectrum converges to the expected peak ground displacement, whereas for a zero period, the acceleration design spectrum converges to the expected peak ground acceleration.

Seismic reduction force factors for displacement ductility (Q') were reviewed and modified. Overstrength (R) and redundancy (ρ) reduction factors are now included, and they depend on the structural system. The new guidelines also include a proposal for modifying the spectral ordinates for reinforced concrete structures in soft soils due to stiffness and/or strength degradation in their hysteretic behavior because of low-cycle fatigue. The design of buildings with structural irregularities was reviewed and updated and a more stringent design is now set for structures with soft story and torsional irregularities. New rules for the combination of vertical and horizontal ground motions are

proposed. A new vertical spectrum is defined. All methods of analysis were reviewed and updated, incorporating new findings from recent research studies. Design drift limits were reviewed and now they depend on the structural system. Recommendations to account for soil-structure interaction were also reviewed and updated to incorporate new research findings, for example, the effects of SSI on the structural ductility, a subject ignored to date in most building codes worldwide.

As a result, MOC-2008 is a much-improved seismic code. Steps have been taken to make MOC-2008 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 MOC-2008 have been provided, with illustrations and in-depth references to the research studies that were consulted in updating the code. It is also recognized in this discussion that seismic codes should continuously evolve, so there is always room for improvement.

It is expected that the new MOC-2008 guidelines will help improve the seismic safety of new buildings in Mexico.

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