Earthquakes and Structures, *Vol. 7, No. 6 (2014) 1241-1258*
DOI: http://dx.doi.org/10.12989/eas.2014.7.6.1241

Response of structures to seismic sequences corresponding to Mexican soft soils

Gerardo Díaz-Martínez*[1], Jorge Ruiz-García[2] and Amador Terán-Gilmore[1]

[1]*Depto. de Materiales, Universidad Autónoma Metropolitana-Azcapotzalco, 02200 México D.F., México.*
[2]*Facultad de Ingeniería Civil, Universidad Michoacana de San Nicolás de Hidalgo, Edificio C, Planta Baja, Cd. Universitaria, 58040 Morelia, México*

(Received February 20, 2014, Revised October 1, 2014, Accepted October 8, 2014)

Abstract. This is paper presents the results of an analytical study aimed at evaluating the effect of narrow-banded mainshock/aftershock seismic sequences on the response of structures built on very soft soil sites. Due to the scarce availability of recorded seismic sequences in accelerographic stations located in the lake-bed of Mexico City, artificial narrow-banded sequences were employed. In the first part of this study, a parametric investigation was carried out to identify the mainshock/aftershock ground motion features that have detrimental effects in the seismic performance of equivalent single-degree-of-freedom systems representative of framed-buildings that house standard and essential facilities. In the second part of this work, the seismic response of two (8- and 18-story) steel-moment resisting frames that house essential facilities is examined. It is concluded that buildings with fundamental periods of vibration longer than the dominant period of the mainshock can experience a significant increment in their inter-story drift demands due to the occurrence of an aftershock.

Keywords: seismic sequences; displacement-based design; soft soils; performance-based design

1. Introduction

On September 19, 1985, the wave trains of a strong subduction interface earthquake (M_w = 8.0) struck Mexico City leading to a very large stock of damaged buildings that were located in soft soil sites of the former lake-bed (Rosenblueth and Meli 1986). The following day, a strong aftershock (M_w = 7.6) shook the city, and caused further damage and increased the level of lateral permanent displacements of buildings previously hit by the mainshock. As a consequence of both earthquakes, several dozen damaged buildings had to be demolished because of the technical difficulties involved in straightening and repairing them. In addition to standard buildings, some hospitals, schools and other essential facilities exhibited unexpectedly high levels of structural damage (it is understood herein that an essential facility should satisfy the *Immediate Occupancy* performance level after the occurrence of an intense ground motion). In spite of the 1985 experience, very few studies have been conducted to investigate the effect of aftershocks in the seismic performance of buildings located at sites, such as locations in the bay zones of San

*Corresponding author, Professor, E-mail: diazmgerardo@hotmail.com

http://www.techno-press.org/?journal=eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

Francisco and Tokyo, and the lake-bed of Mexico City, that can develop narrow-banded long duration ground motions with long dominant period of motion.

The effects of aftershocks, or mainshock/aftershock seismic sequences, on the seismic response of structures, either idealized as single-degree-of-freedom (SDOF) systems or multi-degree-of-freedom (MDOF) systems, have been the topic of several investigations. Most of them have examined the dynamic response of structures when subjected to either real (as-recorded) or artificial seismic sequences.

Real seismic sequences can be assembled from available strong motion databases, such as the *Pacific Earthquake Engineering NGA Database* (PEER NGA 2014). As an alternative, artificial sequences can be generated using the following approaches: 1) back-to back, 2) randomized, and 3) stochastic. While the first approach consists on repeating the real mainshock, at scaled or identical amplitude, as an artificial aftershock (Amadio *et al*. 2003; Hatzigeorgiou and Beskos 2009; Hatzigeorgiou 2010; Hatzigeorgiou and Liolios 2010); the second approach consists on assembling a set of real mainshocks, and generating artificial sequences by selecting a mainshock and simulating the remaining aftershocks by repeating the mainshock waveformat repeatedly, at reduced or identical amplitude, with no change in spectral content, as an artificial aftershock (Lee and Foutch 2004; Li and Ellingwood 2007; Ruiz-García *et al*. 2008; Ruiz-García and Negrete-Manriquez 2011; Goda and Taylor 2012). The third approach utilizes a sequence of stationary Gaussian random processes modulated by an envelope function (Moustafa and Takewaki 2011; Moustafa and Takewaki 2012).

Although previous studies have provided valuable conclusions in terms of the response of structural systems to seismic sequences, most of them employed motions recorded in firm soil conditions (i.e., high-frequency waves with relatively short duration). Nevertheless, there is ample evidence that the occurrence of sequences does not depend on the soil type or the site-source distance, but on the source properties (Moustafa and Takewaki 2011); and that structures built at sites that develop narrow-banded motions (such as those located in the lake-bed of Mexico City) can be subjected to seismic sequences. Within this context, it is important to understand that unlike earthquake ground motions recorded at firm soil sites, narrow-banded motions usually have long dominant period, long duration and high energy content, in such a manner that studies on the effects of aftershocks should specifically address the latter type of motions. A detailed discussion on seismological information and the characteristics of ground motions generated in the lake-bed zone of Mexico City can be found in Quiroz-Ramírez *et al*. (2014).

Recent research on the effects of sequences in structures located in the lake-bed of Mexico City point out to the lack of seismological information to characterize such sequences. Within this context, Díaz-Martínez (2013) used artificial sequences and concluded that the effect of an aftershock on the structural performance of a structure strongly depends on the ratio of its dominant period of motion to that of the mainshock. Later, Ruiz-García *et al*. (2014) studied the response of 4 reinforced concrete (RC) buildings to the action of artificial seismic sequences having features corresponding to the lake-bed of Mexico City, and found that their response strongly depends on the ratio of their *damaged period* (i.e., period at the end of the mainshock) to the dominant period of motion of the aftershock.

This paper offers a general discussion on the effects of mainshock/aftershock seismic sequences in the response of regular framed-buildings located in very soft soil sites, and identifies the range of periods (spectral regions) for which this type of sequences can be detrimental to seismic performance. Rather than carrying out a statistical study on the properties of actual narrow-banded sequences generated in the lake-bed of Mexico City; the general aspects of the response of structural systems to sequences having different properties are identified and discussed, and the features of the most damaging sequences identified from an engineering perspective. Buildings that house standard and essential facilities were considered (note that to promote that essential facilities satisfy the *Immediate Occupancy* performance level, current codes usually use an importance factor larger than 1 to increase the lateral forces used during their design). In order to cover a wide range of structural properties, equivalent single-degree-of-freedom (ESDOF) systems were employed to estimate the response of the buildings. Particular objectives of the study reported in this paper are to: a) Study the influence of the ground motion features of artificial seismic sequences on the response of regular frames; b) Evaluate the effects of using an importance factor of 1.5 in the seismic performance of an essential facility; and c) Assess the response of buildings that house essential facilities and that were designed according to the 2004 edition of the Mexico City Building Code.

2. Structural systems

2.1 Equivalent single-degree-of-freedom systems

The first part of this study evaluated the response of ESDOF systems to narrow-banded seismic sequences. If upper mode effects are not significant to seismic response, an ESDOF model can be used to assess the structural and non-structural performance of a wide variety of structural systems (Qi and Mohele 1991; Teran-Gilmore *et al*. 2013). Nevertheless, such modeling requires from simplifying assumptions that end up limiting the scope of a given study. Within this context, the simple ESDOF model under consideration herein can only be used to assess the dynamic response of moment-resisting frames that exhibit regular distributions, in plan and height, of stiffness, strength and mass. In the next paragraphs, the word *frames* will refer to the structural system of the standard and essential facilities under consideration herein.

The following information is required to establish the ESDOF model of the frames used herein: a) Total height (H); b) Fundamental period of vibration (T); c) Seismic coefficient (c); defined, as illustrated in Fig. 1, as the ratio of the base shear corresponding to the idealized yield point to the total weight (W) of the building; d) Percentage of critical damping (ξ); and e) Nature of the hysteresis loops (e.g., elasto-perfectly-plastic, *EPP*). Once these properties have been established, the ESDOF model can be defined according to what has been discussed by Teran-Gilmore (2004); that is, the ESDOF system used to evaluate the dynamic response of a frame is assigned its values of T, c and ξ, and an hysteretic behavior that is consistent with the expected overall response of the frame (e.g., *EPP* behavior for regular steel frames). Once the ESDOF model is available, it is subjected to a ground motion of interest, and its ductility (μ) and displacement (δ_{SDOF}) demands estimated through a nonlinear dynamic analysis. Then, the maximum roof displacement demand on the frame, δ_{max}, can be estimated as:

$$\delta_{max} = \alpha \delta_{SDOF} \tag{1}$$

where α is a factor that takes into consideration multi-degree-of-freedom effects. Based on the recommendations made by FEMA 306 (FEMA 1998) and on studies carried out by Teran-Gilmore (2004), Table 1 provides values of α for regular frames. While the values for $\mu = 1$ should be applied to elastic behavior; α should be interpolated from the values included in both columns of the table for systems developing nonlinear behavior characterized by $\mu < 2$.

Once δ_{max} has been established, the maximum *IDI* demand can be estimated as follows:

$$IDI_{max} = \frac{COD \times \delta_{max}}{H} \tag{2}$$

where *COD* quantifies the ratio of IDI_{max} to the average inter-story drift index along height. Based on the discussions made by Qi and Moehle (1991) and Bertero *et al*. (1991), and on the studies carried out by Teran-Gilmore (2004) and Aschheim *et al*. (2007), Table 2 summarizes values of *COD* for regular frames.

Although the value of IDI_{max} derived from Eq. 2 can be used to assess the non-structural performance of a building; plastic demands need to be estimated to assess the level of structural damage. Within this context, a $\mu \leq 1$ implies elastic behavior and thus, the absence of structural damage. If $\mu > 1$, it becomes necessary to determine the elastic and plastic components of IDI_{max}. To accomplish this, the elastic component of inter-story drift can be estimated as:

$$IDI_{max}^{EL} = \frac{COD \times \delta_y}{H} \tag{3}$$

where δ_y is, as illustrated in Fig. 1, the roof displacement at yield. Once the elastic component is available, the plastic component of inter-story drift (IDI_{max}^{P}) can be estimated as:

$$IDI_{max}^{P} = IDI_{max} - IDI_{max}^{EL} \tag{4}$$

Under the consideration that the frames should be designed following a *weak beam/strong column* approach, the plastic demands should concentrate in plastic hinges at the ends of the beams. If the frames have a geometric regularity in plan and height, the beams located in a particular story develop similar plastic rotations, in such a manner that it is possible to state that:

$$\theta_p^{mean} \approx IDI_{max}^{P} \quad \Rightarrow \quad \theta_p^{mean} = IDI_{max}^{P} \tag{5}$$

where θ_p^{mean} is the mean plastic rotation at the ends of the beams located in the inter-story that develops IDI_{max}. In spite of the similarity of the plastic rotations in all the beams, the maximum plastic rotation in the inter-story will necessarily be greater than the average:

$$\theta_p^{max} = \theta_p^{mean} + \Delta\theta_{max} = IDI_{max}^{P} + \Delta\theta_{max} \tag{6}$$

where $\Delta\theta_{max}$ is an incremental rotation that can be evaluated from non-linear dynamic analyses.

Although the ESDOF model cannot capture in a reasonable manner the contribution of upper modes, it can be said that its use provides a reasonable idea of the local and global deformation demands in regular buildings (Teran-Gilmore *et al*. 2013). In fact, the ESDOF model has been successfully used to formulate displacement-based methodologies for different types of structural systems (Teran-Gilmore and Virto-Cambray 2009; Teran-Gilmore and Coeto 2011; Díaz-Martínez 2013). In the case of buildings having a fundamental period of vibration much larger than the corner period of the ground motion, the dynamic response in the upper stories of the frames may be underestimated by the ESDOF model, in such a manner that care needs to be exercised when assessing the seismic demands for this case.

2.2 Building models

In the second stage of this study, the response of 2 steel moment-resisting framed buildings to narrow-banded seismic sequences was studied. The buildings, which house essential facilities, have 8 and 18 stories, and were assumed to be located in the lake-bed zone of Mexico City. Figs. 2 and 3 show the plan and elevation views of the case-study buildings. To design their lateral strength, the elastic design spectrum established according to the Mexico City Seismic Provisions (MCSP-2004) for *sub-zone IIIb* of the *Lake Zone* of Mexico City was used. This spectrum was reduced by response modifications factor of 2 and 3, which requires designers to provide standard and ductile detailing to the structural members of the 8 and 18-story buildings, respectively. A force-based design approach, which is customarily in Mexican design practice, was employed for preliminary sizing of the structural members of the frames. Final sizing was determined to satisfy the lateral drift requirement of 1.2% imposed by the MCSP-2004. A detailed description of the design process of the buildings can be found in Díaz-Martínez (2013).

Square box sections were considered for the columns and *W*-shape sections for the beams. *A36* steel was considered for the structural members. In addition, the frames were designed to meet a *weak-beam/strong-column* criterion according to a capacity design approach. Tables 3 and 4 summarize the sizes of beams and columns of the frames. From a modal analysis, it was found that the fundamental periods of the frames were 0.97 and 1.58s, respectively, for the 8- and 18-story buildings. A detailed two-dimensional nonlinear model of each building was prepared to assess its seismic performance with the DRAIN 2DX program (Prakash *et al*. 1993). While the beams of the frames were assigned a bilinear behavior with 2% strain-hardening, the model of the columns considered the combined effect of bending and axial load and a bilinear behavior with no strain

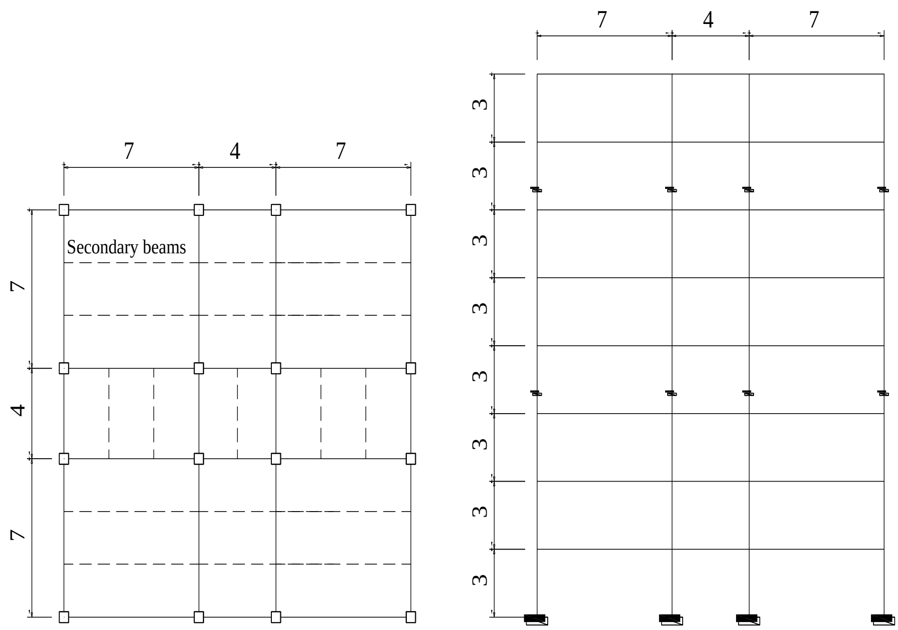

Fig. 2 Plan and elevation view of the 8-story building (dimensions in meters)

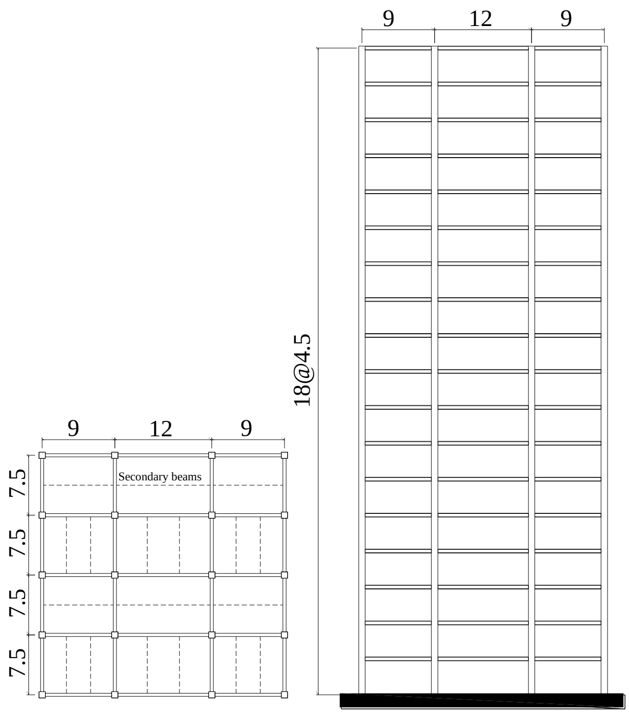

Fig. 3 Plan and elevation view of the 18-story building (dimensions in meters)

Table 3 Cross sections for beams and columns of the 8-story frame

Stories	T = 0.92 sec		
	Columns		Beams
	Side [cm]	Plate thickness [cm]	Section
1-3	60	2.540	*W*18X86
4-6	55	1.905	*W*18X76
7-8	50	1.905	*W*18X60

Table 4 Cross sections for beams and columns of the 18-story frame

Stories	T = 1.58 sec		
	Columns		Beams
	Side [cm]	Plate thickness [cm]	Section
1-3	110	5.08	*W*36X485
4-6	105	5.08	*W*36X439
7-9	100	5.08	*W*33X424
10-12	95	5.08	*W*33X387
13-15	85	5.08	*W*27X368
16-18	80	5.08	*W*24X306

hardening. Expected material strengths were used to estimate the structural properties of beams and columns. Particularly, the expected yield stress of the steel was considered to be 20% larger than its nominal value (Wong 2009). P-Δ effects were considered through a geometric stiffness matrix, and the bases of the columns on the ground story were fixed. The model of the frames considered 5% of critical damping through a Rayleigh matrix that assigned the indicated damping to the first two modes of vibration.

3. Seismic sequences

Real (as-recorded) mainshock/aftershock acceleration time histories (hereafter denoted *sequences*) are needed to perform nonlinear dynamic analyses and subsequent statistical studies. However, it was found that only two real mainshock/aftershock acceleration time-histories recorded at the Central de Abastos (CDAF) station during the September 19 and 20, 1985 earthquakes were available from the Mexican Database of Strong Motions (Sociedad Mexicana de Ingeniería Sísmica 1999). Because of the existing limitations in terms of information, this investigation employed a set of artificial seismic sequences. For this purpose, the 7 earthquake ground motions included in Table 5 and recorded during historical earthquakes in the lake-bed of Mexico City, were selected from the Mexican Database for Strong Motions. All motions have a dominant period of motion (T_g) close to 2.0s. Before generating the sequences, all mainshock accelerograms were scaled to reach the maximum peak ground acceleration of the *EW* component of the motion recorded at the *Secretaría de Comunicaciones y Transportes* station during the mainshock of the 1985 events (*MX08*).

The frequency content and amplitude of the individual motions that compose a sequence can be significantly different, in such a manner that it may not be accurate to consider acceleration sequences with identical frequency content (Li and Ellingwood 2007, Moustafa and Takewaki 2011, Ruiz-García 2012, Goda and Taylor 2012). In terms of the lake-bed of Mexico City, only two sequences have been recorded, and the available information is contradictory at best. On one hand, aftershocks recorded at firm soil sites on the Mexican Pacific Coast exhibit smaller amplitude and dominant periods than those of their mainshocks (Ruiz-García *et al*. 2008, Ruiz-García 2012); and, in general, motions recorded at the lake-bed exhibit larger contents of higher frequencies as their amplitude of motion decreases. On the other hand, Moustafa and Takewaki (2012) discuss the existence of low-frequency content in secondary sequences. Within a context in which high-and low-frequency contents have been observed in actual aftershocks, it is important to use artificial sequences with different properties. Particularly, to explore the influence of the frequency content of the aftershock, thirty-five artificial sequences having T_A/T_M ratios of 1.2, 1.0, 0.9, 0.8 and 0.7, were generated by modifying the original frequency content of the records used as aftershocks. While T_A denotes the dominant period of the aftershock; T_M denotes that of the mainshock. To modify the original frequency content of a record used as aftershock, its sampling time (Δt) was modified without changing its acceleration ordinates. For example, if the original dominant period of a motion had to be increased 50%, the value of Δt of the original record was also increased by 50%. To ensure that the systems reach their resting position after free vibration upon the action of the mainshock, 50.0s of zero acceleration was inserted between the main and aftershock motions (which was found to be enough time gap for systems subjected to narrow band earthquake ground motions). Table 6 summarizes the value of dominant periods of motion under consideration for the seismic sequences. Another parameter under consideration for the studies was the ratio of the peak ground acceleration of the aftershock, A_A, to that of the mainshock, A_M. Within the context of limited information discussed before, it was decided to use A_A/A_M ratios of

Table 5 Earthquake ground motions

Nomenclature	Record	Date	Component	T_g (s)	A_{max} (cm/s^2)
MX01	Alameda	04/25/89	*NS*	2.1	45.83
MX02	Alameda	04/25/89	*EW*	2.1	37.25
MX03	Garibaldi	04/25/89	*NS*	1.9	52.24
MX04	Tlahuac	09/19/85	*EW*	2.0	117.63
MX06	Tlahuac	09/21/85	*NS*	2.0	49.26
MX07	Tlahuac	09/21/85	*EW*	1.9	51.47
MX08	SCT	09/19/85	*EW*	2.0	167.26

Table 6 Artificial Seismic Sequences

Sequence	T_M (s)			T_A (s)		
MX01_03	2.1	2.5	2.1	1.9	1.7	1.5
MX01_07	2.1	2.5	2.1	1.9	1.7	1.5
MX02_03	2.1	2.5	2.1	1.9	1.7	1.5
MX02_07	2.1	2.5	2.1	1.9	1.7	1.5
MX04_02	2.1	2.5	2.1	1.9	1.7	1.5
MX06_04	2.1	2.5	2.1	1.9	1.7	1.5
MX08_06	2.1	2.5	2.1	1.9	1.7	1.5

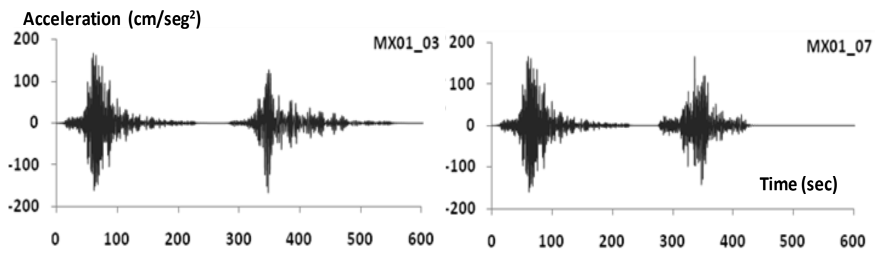

Fig. 4 Artificial seismic sequences ,A_A/A_M = 1.0 and T_A/T_M = 1.2

1.0, 0.9, 0.8 and 0.7 for each artificial sequence with a given T_A/T_M ratio. To provide the different values of the A_A/A_M ratio, the motions used as aftershocks were linearly scaled. All in all, 140 artificial seismic sequences were considered in this study. Fig. 4 shows two artificial seismic sequences.

Moustafa and Takewaki (2010) have observed that if the information available for an earthquake ground motion is limited to its energy content and peak ground acceleration, the critical input has a narrow-banded nature with a dominant period of motion close to that of the structural system; and, that within such resonant scenario, the displacement and energy demands are maximized. The motions considered herein to define the sequences exhibit a similar narrow-banded nature; and thus, it can be anticipated that the response of those systems with a period close to the dominant periods of the main and/or aftershock (resonant scenarios) will likely be more susceptible to change during the occurrence of an aftershock.

4. Inter-story drift demands in ESDOF systems

A parametric study was performed to assess the effects of seismic sequences and, in particular, of aftershocks, on the response of ESDOF systems that model the response of buildings that house standard and essential facilities.

ESDOF systems having 5% of critical damping and elasto-perfectly-plastic behavior were subjected to the set of artificial sequences under consideration. The following relationship was used to characterize the lateral stiffness of the regular frames modeled through ESDOF systems:

$$T = \beta N \tag{7}$$

where T is the fundamental period of vibration, N the number of stories, and β a dimensionless coefficient. Frames having from 1 to 50 stories were considered in this study. While the inter-story height was considered equal to 4.0 meters for all stories, β was assigned values of 0.08 and 0.1. Note that once the total number of stories of the building is defined (N), its fundamental period of vibration can be established from the value of β under consideration. As regards to the lateral stiffness of the frames it will be considered that β equal to 0.1 represents in a reasonable manner the lateral stiffness of "typical" earthquake-resistant frames. Under the consideration that the use of an importance factor of 1.5 results in essential facilities that exhibit a lateral stiffness that is about 50% larger than that of a "typical" frame, β of 0.08 can be used to characterize their fundamental period of vibration ($0.08 \approx 0.10/\sqrt{1.5}$

Fig.5 shows mean-plus-one-standard-deviation constant-ductility lateral strength spectra, S_a, corresponding to displacement ductilities (μ) of 2 and 4. These spectra were computed for the mainshocks listed in Table 5.

Two sets of structural systems will be considered next: A) Standard systems, with a lateral strength that is established directly from the spectral ordinates shown in Fig. 5, and B) Essential systems, with a lateral strength capacity that is 50% larger than that under consideration for the standard systems. These levels of lateral strength were assigned to the ESDOF systems with the purpose of estimating and comparing inter-story drift index demands of standard and essential facilities. In consistency with the statistical characterization of the spectra shown in Fig. 5, the mean-plus-one-standard-deviation inter-story drift demands will be considered next.

IDI_{max} spectra corresponding to standard and essential systems subjected to artificial sequences with A_A/A_M = 1.0 and three T_A/T_M ratios are shown in Fig. 6. It can be seen that the influence of the aftershocks tends to increase with an increase in the value of T_A/T_M. Note that the effect of the aftershocks in terms of IDI_{max} is slightly larger for a μ of 4 than that for a μ of 2; fact that can be explained by the smaller levels of lateral strength assigned to the former ductility level. The spectral region where aftershocks affect in a significant manner the seismic response of the ESDOF systems starts at about 2.0s (note that the mainshock motions have a T_g of 2.0s). Furthermore, the effect of aftershocks is more evident in systems that model standard structures than in those that model essential facilities; fact that can be explained by the larger lateral strength of the essential facilities. In spite of this, there is a noticeable effect of the aftershocks for the essential systems having a fundamental period of vibration similar or longer than T_M.

Fig. 7 shows IDI_{max} spectra corresponding to standard and essential systems designed for μ of 4, and subjected to the set of sequences having T_A/T_M of 0.9 and three values of A_A/A_M. This case was considered relevant since most mainshock/aftershock sequences recorded in Mexico exhibit

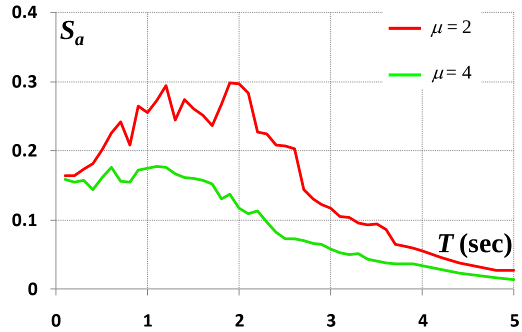

Fig. 5 Pseudo-acceleration spectra for mainshocks recorded in the lake-bed of Mexico City, 5% of critical damping

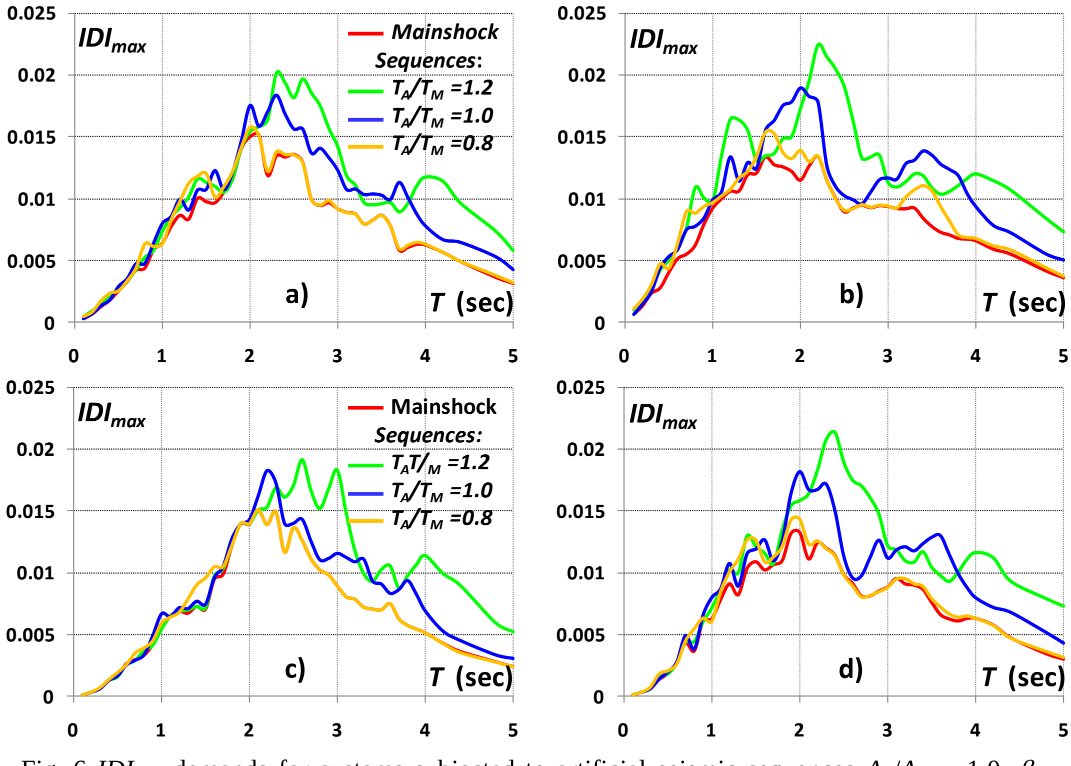

Fig. 6 IDI_{max} demands for systems subjected to artificial seismic sequences A_A/A_M = 1.0, β = 0.10: a) Standard facilities, μ = 2, b) Standard facilities, μ = 4, c) Essential facilities, μ = 2, d) Essential facilities, μ = 4

smaller predominant periods for their aftershocks (Ruiz-García 2012). In general, the A_A/A_M ratio does not have a significant influence in the response of the systems under consideration. Note that the use of an importance factor of 1.5 tends to better constraint the increments in the IDI_{max} demands in the essential facilities, in such a manner that the occurrence of an aftershock does not have a noticeable impact on their seismic performance.

Fig. 8 shows IDI_{max} spectra corresponding to standard and essential systems designed for $\mu = 4$, that have fundamental periods of vibration corresponding to $\beta = 0.08$ (see Eq. 7), and subjected to sequences having $A_A/A_M = 0.8$ and three levels of T_A/T_M. A β of 0.08 implies that the structural systems are about 50% stiffer than those analyzed for the case of $\beta = 0.1$. In spite of their increased lateral stiffness and strength, and that A_A is smaller than A_M, the seismic sequences still have the potential to trigger large IDI_{max} demands on essential systems having periods of vibration longer than 2.0s. Once again, it can be observed that the T_A/T_M ratio has an important influence in the response of the systems. Particularly, seismic sequences having T_A larger than T_M have an increased damage potential for the frames vulnerable to aftershocks. Under these circumstances, the occurrence of the aftershock will be translated into an increase in the level of damage of systems having a fundamental period of translation that is longer than T_M.

Fig.9 shows IDI^P_{max} spectra corresponding to essential systems designed for μ of 2 and 4, and subjected to sequences having different features. The IDI^P_{max} demands for systems subjected to the action of the mainshocks increase in a linear fashion from a value close to zero for small periods, until it peaks at a period close to T_M. A further increase in period with respect to T_M results in IDI^P_{max} to decrease for large values of period. In terms of essential systems having $\beta = 0.10$, the mainshocks have peak IDI^P_{max} demands close to 0.006 and 0.010 for μ of 2 and 4, respectively. In terms of the effect of the aftershocks, it can be said that the value of T for which the IDI^P_{max} demands peak tends to increase with an increase in T_A/T_M; and that these demands increase significantly for systems having a fundamental period of vibration larger than T_M, particularly as T_A/T_M increases. In general, it can be said that the IDI^P_{max} demands can be strongly influenced by the occurrence of an aftershock, and that they tend to increase with an increase in the value of μ used during the design. The levels of plastic inter-story drift under consideration illustrate the potential of aftershocks to induce larger plastic rotation demands, and thus structural damage and permanent drift, in regular frames.

From the results under consideration until now, it can be concluded that narrow-banded seismic sequences can affect the dynamic response of standard and essential structures. In general terms, it can be said that as the value of T_A/T_M and A_A/A_M increase, larger is the effect of the aftershock in the seismic response of regular frames; and that the effect of T_A/T_M is larger than that of any other parameter under consideration. Fig. 10 schematically illustrates the effect of T_A/T_M and A_A/A_M in the response of regular frames subjected to the action of seismic sequences generated in soft soils. It should be emphasized that the importance factor used during the strength-based design of the essential systems has less effect on their dynamic response than the values of T_A/T_M. Note that the worst-case scenario is represented by the dark red color, while green represents cases in which an aftershock has no significant influence in the response of the structural systems.

The results obtained by Moustafa and Takewaki (2010, 2011, 2012) can be used to contextualize the conclusions offered in the previous paragraph. In a similar fashion to what they observed, the effects of sequences on the seismic performance of the systems under consideration herein strongly depend on their strength and initial period. While these effects decrease as the lateral strength of the systems increase, systems having a T equal or slightly larger than T_M exhibit important increases in their drift demands. It is interesting to note that Moustafa and Takewaki

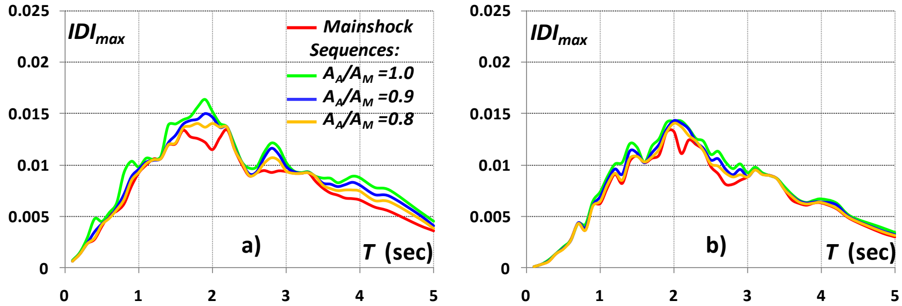

Fig. 7 IDI_{max} demands for systems subjected to artificial seismic sequences, T_A/T_M = 0.9, β = 0.10, μ = 4: a) Standard facilities, b) Essential facilities

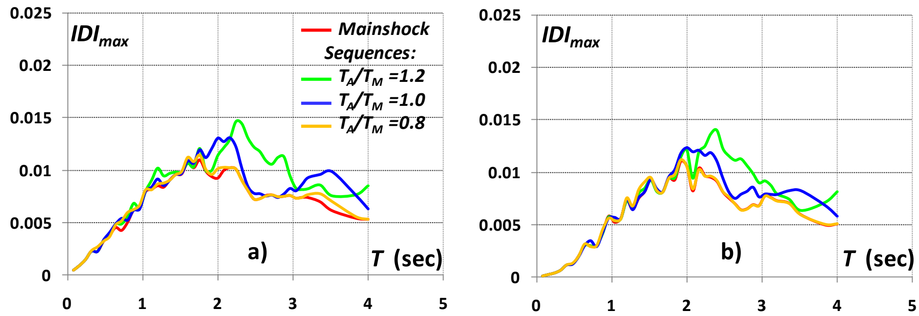

Fig. 8 IDI_{max} demands for systems subjected to artificial seismic sequences, A_A/A_M = 0.8, β = 0.08, μ = 4: a) Standard facilities, b) Essential facilities

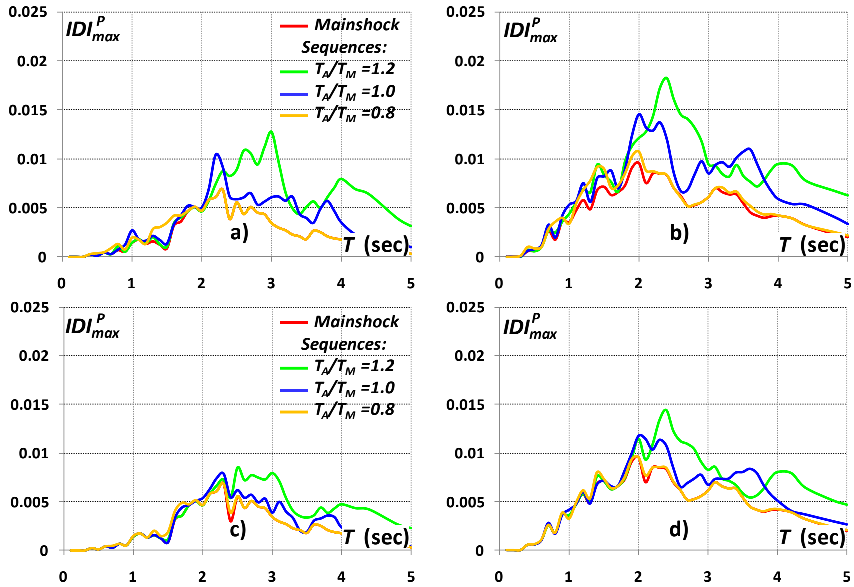

Fig. 9 IDI^P_{max} demands for essential systems subjected to artificial seismic sequences, β = 0.10: a) A_A/A_M = 1.0, μ = 2; b) A_A/A_M = 1.0, μ = 4;c) A_A/A_M = 0.8, μ = 2; d) A_A/A_M = 0.8, μ = 4

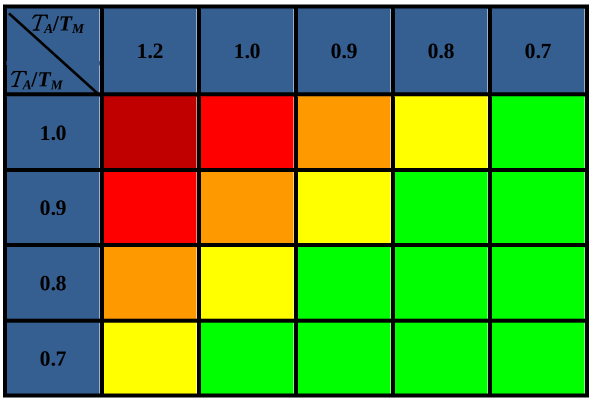

Fig. 10 Color diagram representing influence of parameters T_A/T_M and A_A/A_M on response of systems studied herein when subjected to seismic sequences

have discussed that the critical input for a particular structure has a narrow-banded nature with a dominant period of motion close to that of the structural system; and, that within such resonant scenario, the displacement demands are maximized. Another observation that can be made from the results presented herein is that the effects of aftershocks increase as the value of T_A/T_M increases. This is consistent with the following observation made by Moustafa and Takewaki (2012): "*Moreover, the low-frequency content in the secondary sequences may cause resonance in lower modes of the damaged structure leading to further damage*." Although the ESDOF systems under consideration have elasto-perfectly-plastic behavior, their damage is reflected by large residual displacements that result in important increases in the drift demands for systems with T slightly larger than T_M. In spite of the consistency of the results obtained herein with those reported in previous papers, it is important to note that the T_A/T_M ratio has the largest influence in the effects of aftershocks, and that this conclusion was difficult to anticipate.

Finally, and although not illustrated, the results obtained from the nonlinear dynamic analyses discussed herein can be used to conclude that the levels of strain hardening and damping ratio have little impact on the observations offered so far. That is, while an increase in the value of these two parameters results in a decrease in the drift demands of the structural systems, there are no changes in terms of the spectral zone and conditions under which a structural system becomes susceptible to the occurrence of a narrow-banded aftershock. This is consistent with what has been observed by Moustafa and Takewaki (2010) in terms of the small influence that these parameters have on the critical earthquake input for a particular structural system.

5. Inter-story drift demands in essential systems

Although the results derived from ESDOF offer important insights in terms of the response of regular frames to narrow-banded sequences, it is important to validate them through the use of MDOF systems.

5.1 Response of 8-story building

To help understand the effect of aftershocks on the response of the 8-story building, two sets of artificial sequences were considered: a) T_A/T_M = 1.2 and A_A/A_M = 1.0; and b) T_A/T_M =1.0 and A_A/A_M = 1.0. Fig.11 shows the mean-plus-one-standard-deviation height-wise distribution of IDI_{max} and maximum plastic rotation demands (θ_p^{max}) corresponding to the mainshocks and mainshock/aftershock sequences. Although the frame is able to adequately control its IDI_{max} and θ_p^{max}) demands, both sets of demands show a slight increase as a consequence of the aftershocks. Particularly, while the IDI_{max} demands increase in 5%; the θ_p^{max}) demands increase in about 10%. Note that the small increases in demand evaluated for the 8-story building are consistent with the results shown in Figs. 6 and 9. In the case of the 8-story building, its fundamental period of vibration is 0.92s; and the T_M of the seismic sequences equal to 2.0s. As was discussed before, systems having a fundamental period shorter than T_M tend to exhibit small increases in their seismic demands when subjected to the effect of the after-shocks.

5.2 Response of the 18-story building

To help understand the effect of aftershocks on the response of the 18-story building, three sets of artificial sequences were considered: a) T_A/T_M = 1.2 and A_A/A_M = 1.0; b) T_A/T_M =1.0 and A_A/A_M = 1.0; and c) T_A/T_M = 0.8 and A_A/A_M = 1.0. Fig. 12 shows the mean-plus-one-standard deviation height-wise distributions of IDI_{max} and θ_p^{max} corresponding to the mainshocks and mainshock/aftershock sequences. Again and although the frame is able to adequately control its IDI_{max} and θ_p^{max} demands, both demands show a slight increase as a consequence of the

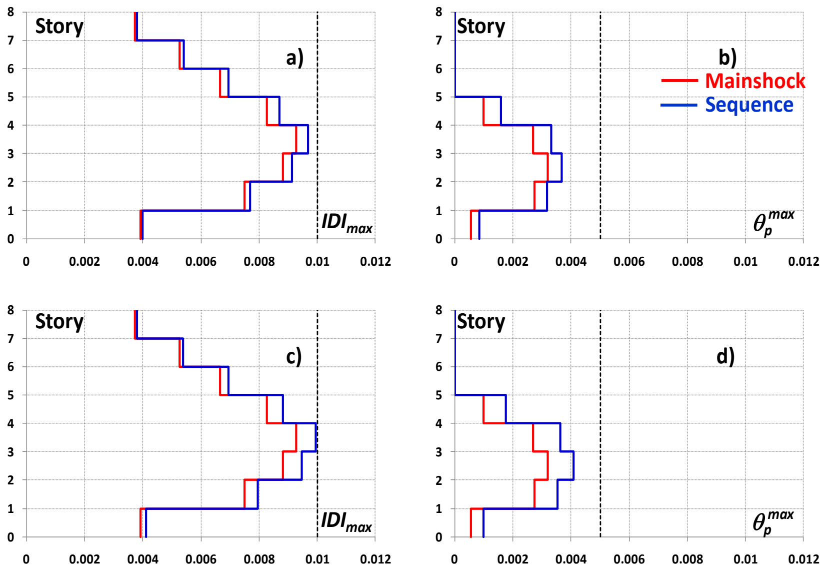

Fig. 11 Maximum deformation demands along height for the 8-story building, A_A/A_M = 1.0: a) IDI_{max}, T_A/T_M = 1.2; b) θ_p^{max}, T_A/T_M = 1.2; c) IDI_{max}, T_A/T_M = 1.0; d) θ_p^{max}, T_A/T_M = 1.0

aftershocks. In the case of the 18-story building, the largest increase in demands occurs for sequences in which $T_A/T_M = 0.8$. Although this result may appear atypical, note that the demands evaluated for the 18-story building are consistent with the results shown in Figs. 6c and 9a. Particularly, for systems having a fundamental period of vibration in the vicinity of 1.5 (that of the 18-story building is equal to 1.58s), the sequences having $T_A/T_M = 0.8$ impose larger seismic demands than those having $T_A/T_M = 1.0$ and 1.2. To explain this, it is necessary to understand that for sequences having $T_A/T_M = 0.8$; the dominant period of the aftershocks ($T_A = 1.7$s) is close to the fundamental period of vibration of the 18-story building.

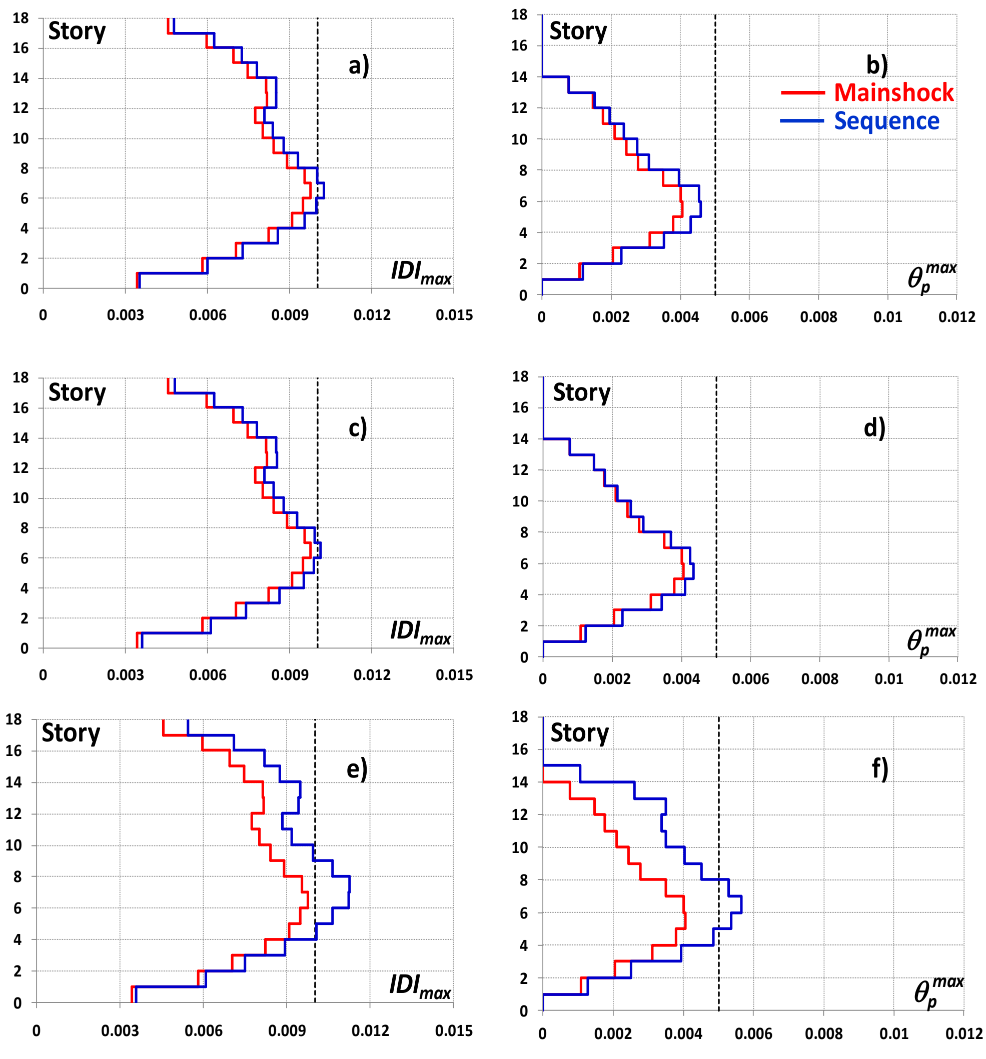

Fig. 12 Maximum deformation demands along height for 18-story building, $A_A/A_M = 1.0$: a) IDI_{max}, $T_A/T_M = 1.2$; b) θ_p^{max}, $T_A/T_M = 1.2$; c) IDI_{max}, $T_A/T_M = 1.0$; d) θ_p^{max}, $T_A/T_M = 1.0$; e) IDI_{max}, $T_A/T_M = 0.8$; f) θ_p^{max}, $T_A/T_M = 0.8$

6. Summary and conclusions

Two strong earthquakes struck Mexico City on September 19 and 20, 1985, causing large damage to buildings. As a consequence of both earthquakes, several dozen damaged buildings had to be demolished because of the technical difficulties involved in straightening and repairing them. In spite of these historical events, limited investigation has been devoted to understand the effect of seismic sequences in the response of buildings located in soil sites capable of developing narrow-banded long duration motions with long dominant periods. Given the aforementioned, the work reported in this paper presents the results of an analytical study aimed at evaluating the effect of narrow-banded mainshock/aftershock seismic sequences on the response of structures built on the lake-bed of Mexico City.

In this study, inter-story drift demands for frames that house standard occupancy and essential facilities were estimated through the use of an ESDOF model. Due to the lack of information, a wide range of values was considered for parameters that characterize the seismic sequences. In terms of the ground motion features, attention was focused on the impact that parameters T_A/T_M and A_A/A_M have on the response of the frames. The effect of aftershocks becomes more significant as the value of these two ratios increases. Nevertheless, the influence of parameter T_A/T_M is larger. A range of values was also considered for the structural properties of the steel frames. Particularly, while a wide range was considered for the fundamental period of vibration; four different levels of lateral strength took into account two occupancy types (standard and essential) and two values of maximum ductility. The effects of an aftershock become more significant as the strength of the structural system decreases, and for systems having a fundamental period of vibration longer than the dominant period of motion of the mainshock. Even for the stronger frames under consideration (essential facilities designed for a maximum ductility of 2), aftershocks with T_A/T_M of 1.0 or larger resulted in significant increments in the inter-story drift demands of buildings having a fundamental period of vibration longer than the dominant period of motion of the mainshock.

The results obtained by using detailed nonlinear analytical models of 8 and 18-story buildings were used to validate those obtained with ESDOF systems; and confirm that as long as the fundamental period of vibration of a structure is smaller than the dominant period of motion of the mainshock, the effect of the aftershock will have little impact on its seismic demands.

Acknowledgements

While the first and third authors would like to express their gratitude to *Universidad Autónoma Metropolitana-Azcapotzalco*, the second author would like to thank *Universidad Michoacana de San Nicolás de Hidalgo*. The authors acknowledge the Mexican Council for Science and Technology (CONACYT) for the financial support provided to develop the research reported in this paper.

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