Displacement-Based Design of Essential Facilities

Gerardo Diaz & Amador Teran-Gilmore
Universidad Autonoma Metropolitana,Mexico

Carlos Reyes
Instituto Tecnologico de Estudios Superiores de Monterrey, Mexico

SUMMARY
Although an importance factor of 1.5 may promote adequate performance of essential facilities built in firm soil; the actual value of this factor corresponding to structures built in soft soil exhibits significant dependence with respect to the dynamic characteristics of the structural system. Because of this, the current use given to the importance factor may result in conservative or unsafe design of essential facilities built in soft soils. One way to improve the seismic design of essential facilities is to formulate displacement-based approaches. Within this context, a displacement-based methodology should aim at controlling simultaneously the level of structural and non-structural damage by limiting the plastic rotation and inter-story drift demands in the structural system of the essential facility.

Keywords: Essential facility, Importance Factor, Displacement-based design, Immediate Occupancy

1. INTRODUCTION

Large socio-economic losses have been the result of the unsatisfactory seismic performance of buildings designed in accordance to current seismic design codes. Examples of this occurred during the Mexico 1985, Northridge 1994, Kobe 1995 and Chile 2010 earthquakes. Recent seismic design approaches, such as performance-based design, have been targeted at reducing the level of loss by making possible the conception of earthquake-resistant structures that are able to adequately control their dynamic response during ground motions of different intensity. In such a context, structural engineers have been forced to improve their knowledge and to update their seismic design procedures in such a manner as to transcend the design of structures that do not collapse under intense ground motion, and to conceive and build structural systems that can satisfy, through adequate damage control, the complex socio-economic needs of modern human societies.

While the level of structural and non-structural damage in a building subjected to strong ground motion depends on the maximum displacement demand; some types of contents are vulnerable to velocity and acceleration. In the particular case of structural and deformation-sensitive non-structural elements, an increase in lateral displacement results in heavier seismic damage. Even though the behavior of different structural and non-structural materials can differ significantly, it can be said that their level of damage increases significantly once they incur in their non-linear range of behavior. Within this context, the structural properties provided to the structural system of an essential facility need to be established in order to control its seismic response within design thresholds that are formulated in terms of what can be considered acceptable structural and non-structural performance.

In spite of the special considerations taken during their design, essential facilities tend to be highly vulnerable to the effects of intense ground motion. Generally speaking and in order to achieve the *Immediate Occupancy* performance level, an essential facility requires a strong and stiff structural system, and damage-free non-structural elements. In order to promote the construction of essential facilities that are able to achieve the *Immediate Occupation* performance level from structural and non-structural points of view, the international seismic design codes usually require the use of an importance factor equal to 1.5. Previous studies suggest that this practice is pertinent for structural systems built on firm soil provided the lateral strength of the essential facility is designed with a pseudo-acceleration spectrum corresponding to a maximum ductility of two (Teran et al. 2010). The same studies suggest that in the case of very soft soil conditions, the current use given to the importance factor can lead to conservative designs for structural systems whose fundamental period of vibration strongly differs from the dominant period of motion, and to unsafe design, from the perspective of *Immediate Occupation*, for systems having a period close to that of the ground motion. Because of the inadequate and highly variable seismic performance observed for essential structures designed for soft soil conditions, a displacement-based methodology aimed at the preliminary seismic design of essential facilities is formulated herein. In terms of scope, the methodology is aimed at the design of structural systems composed by moment-resisting frames, and does not consider explicitly the seismic performance of velocity and acceleration-sensitive contents.

2. SINGLE-DEGREE-OF FREEDOM MODEL

If upper mode effects are irrelevant to seismic response, a single-degree-of-freedom (SDOF) model can be used to assess the structural and non-structural performance of a wide variety of structural systems. Nevertheless, such modeling requires from simplifying assumptions that end up limiting the scope of a given study. Within this context, the simple SDOF 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 essential facilities under consideration herein.

The following information is required to establish the SDOF model of the frames: A) Total height (H); B) Fundamental period of vibration (T); C) Seismic coefficient (c); 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 SDOF model can be defined according to what has been discussed by Teran (2004); that is, the SDOF 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 SDOF model is available, it is subjected to a ground motion of interest, and its ductility (μ) and displacement (δ_{SDOF}) demands estimated through a non-linear dynamic analysis. Then, the maximum roof displacement demand on the frame 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 (Applied Technology Council 1998) and on studies carried out by Teran (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 non-linear behavior characterized by $\mu < 2$.

Table 1. α values for regular moment-resisting frames

Number of stories	α	
	$\mu = 1$	$\mu = 2+$
1	1	1
2	1.20	1.10
3	1.30	1.20
4	1.35	1.20
5+	1.40	1.20

Once δ_{max} has been established, the maximum inter-story drift index (*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 studies carried out by Teran (2004), Table 2 summarizes values of COD for regular frames.

Table 2. *COD* values for regular moment-resisting frames

μ	COD
1	1.2
2+	1.5-1.8

Although the value of IDI_{max} derived from Equation 2 can be used to assess the non-structural performance of the essential facility; 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 significant 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 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 of the frames. As discussed in detail by Bojorquez et al. (2011), the beams located in a particular story of a regular frame 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.

To study the pertinence of using the simple SDOF model, this model was used to evaluate the global and local deformation demands of a family of seven regular framed buildings. The buildings were designed according to the Mexican Building Code to withstand, through the development of ductile behavior, the design ground motion corresponding to the Lake Zone of Mexico City (Teran 1998). Detailed static and dynamic non-linear analyses of the buildings were carried out to estimate their seismic demands. In terms of the dynamic analyses, the refined models were subjected to the East-West component of the motion recorded at the Secretaria de Comunicaciones y Transportes during 1985 Michoacan earthquake (SCTEW).

Figure 1 compares the local and global deformation demands estimated from the SDOF and refined models for the seven buildings. While the refined models considered an elasto-plastic hysteretic behavior for the beams and columns of the frames and $\xi = 0.05$ for the first two modes of vibration; the SDOF model considered an *EPP* behavior and same percentage of critical damping. A *COD* of 1.7 was used to estimate the IDI_{max} demands from SDOF models undergoing $\mu \geq 2$ (see Table 2). The IDI_{max}^{P} demands shown in Figure 1c for the refined models were estimated by averaging the maximum plastic rotation demands at both ends of the beams located in the inter-story with the largest drift demands; and the values of $\Delta\theta_{max}$ shown in Figure 1d, by subtracting IDI_{max}^{P} from the largest value of maximum plastic rotation. It may be concluded that the SDOF model captures in a reasonable manner the maximum local and global deformation demands in the seven buildings, and that the values of $\Delta\theta_{max}$ range from about 0.002 to 0.003.

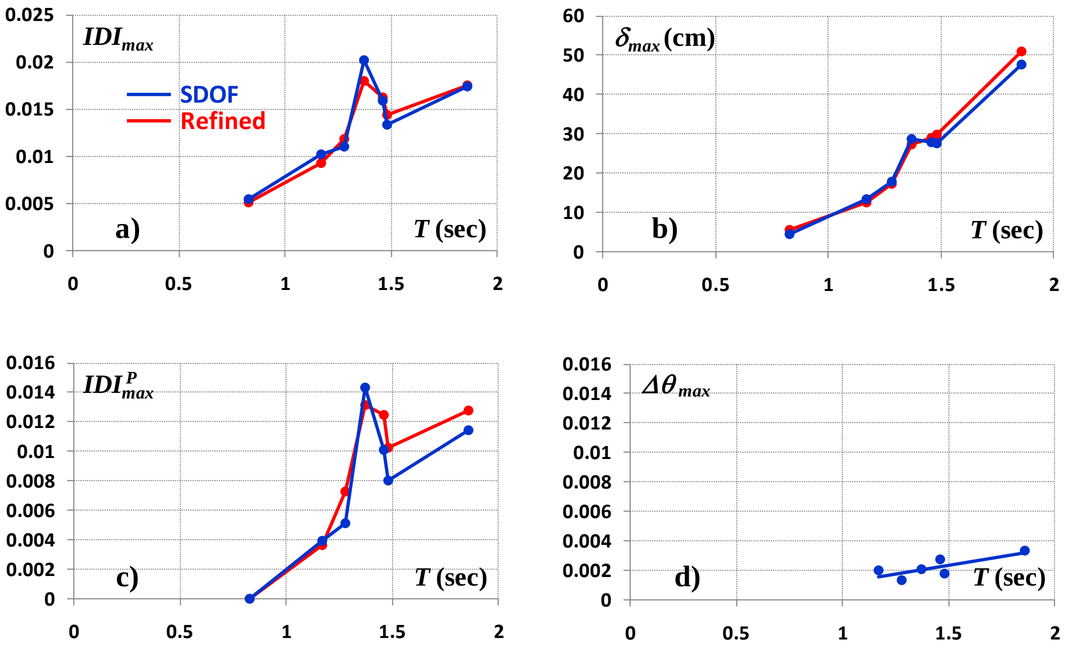

Figure 1. Response envelopes for SCTEW: a) IDI_{max}; b) δ_{max}; c) IDI^{P}_{max}; d) $\Delta\theta_{max}$

As suggested before, the SDOF model cannot capture in a reasonable manner the contribution of upper modes. Because the fundamental period of vibration of the seven buildings under consideration is smaller than the dominant period of motion (T_g), which equals two seconds for the case of SCTEW, the contribution of upper modes is insignificant and the SDOF model provides adequate estimates of global and local maximum response. In the case of buildings having a fundamental period of vibration larger than T_g, the seismic response in the upper stories of the frames may be underestimated by the SDOF model, in such a manner that care needs to be exercised when assessing the seismic demands for this case.

3. CONSTANT PLASTIC INTER-STORY DRIFT INDEX SPECTRA

Through the extensive and careful use of the simple SDOF model, the seismic performance of frames having a wide range of structural and dynamic properties can be studied (Teran et al. 2010). Also, this model can be used to establish design spectra for essential facilities that are able to control their level of structural and non-structural damage. Because of the simplifying assumptions involved in the formulation of the SDOF model, each spectrum derived from it applies to a family of structures that exhibit a particular value of β, where:

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

and T is the fundamental period of vibration of the structural system and N denotes its number of stories.

A constant *maximum* ductility strength spectrum corresponding to ductility μ is defined in such way that the pseudo-acceleration (S_a) evaluated at any value of T will result in a lateral strength that is capable of controlling the *maximum* ductility demand on a single-degree-of-freedom system within a threshold value of μ. Of particular interest is the potential of the simple SDOF model formulated herein to establish constant *maximum* plastic inter-story drift index strength spectra. Unlike a traditional pseudo-acceleration spectrum, a constant IDI^P_{max} strength spectrum establishes, according to the simple SDOF model, the lateral strength required by a regular frame to control its maximum plastic inter-story drift index demand within the value of IDI^P_{max} assigned to the spectrum. Conceptually, a constant maximum plastic inter-story drift index strength spectrum corresponding to β of 0.1 and IDI^P_{max} = 0.003 applies to the design of regular frames having a fundamental period of vibration that is equal to one tenth of its number of stories, and provides the required lateral strength to control their IDI^P_{max} demand within a 0.003 threshold. Within this context, a constant plastic inter-story drift index displacement spectrum is established according to the displacement demands estimated in the *SDOF* systems used to define its corresponding strength spectrum.

4. DISPLACEMENT-BASED SEISMIC DESIGN METHODOLOGY

Because of the limitations of use of an importance factor during the seismic design of essential facilities in soft soils, performance-based tools need to be developed to aid the conception of structural systems that house this type of facilities. This section introduces a displacement-based design methodology aimed at the preliminary design of essential frames that are able to simultaneously control their structural and non-structural damage.

4.1. Methodology

The methodology offered in this paper is based on the conception of an essential building whose seismic resistance is provided by regular moment-resisting frames. In what follows, these moment-resisting frames will be referred just as *frames*.

The methodology, schematically shown in Figure 2, considers the *Immediate Occupancy* performance level. Regarding the qualitative definition of performance, *Immediate Occupancy* is considered to be satisfied if the frames exhibit light structural damage and the non-structural elements remain undamaged. In quantitative terms, the frames are considered to satisfy their structural performance criteria if the θ_p^{max} demand does not exceed a threshold value θ_p^{IO}. Non-structural damage is considered to be adequately controlled if the IDI_{max} demand does not exceed a threshold value associated to initiation of damage (IDI^{IO}_{NS}).

The design process starts with the selection of the structural material and the definition of the structural layout of the frames. At this stage, a definition of the type of non-structural elements is also required. Next and in terms of the structural material and non-structural system under consideration, acceptable values are established for θ_p^{IO} and IDI^{IO}_{NS}; and suitable thresholds are established in terms of these parameters, respectively, for θ_p^{max} and IDI_{max}. Once the value of θ_p^{max} is available, a threshold value for IDI^P_{max} can be estimated with Equation 6.

Because constant maximum ductility spectra lead to inconsistent structural performance, constant IDI^P_{max} spectra need to be defined for design purposes. Within this context, the relation established by the simple SDOF model between the IDI^P_{max} and displacement demands (Equations 1 to 6) requires the availability of a value of β, in such a manner that the methodology requires next the assumption of an initial value for this parameter (e.g., 0.10). Then, an initial estimate for the fundamental period of vibration of the frames can be established as $T = \beta N$.

The value of IDI_{max} can be used to establish a design threshold for the lateral roof displacement:

$$\delta_{max} = \frac{IDI_{max}\,H}{COD} \tag{8}$$

where H is the total height of the frames. Based on the ductility demands estimated by Teran et al. (2010) for regular frames and the values of COD included in Table 2, design aids such as that plotted in Figure 3a can be formulated and used to estimate COD as function of the value of the period of the frames relative to the dominant period of the design ground motion (T_g).

As shown in Figure 2, the target fundamental period of vibration (T_{TAR}) for the frames can be estimated by using a constant IDI^{P}_{max} displacement spectrum and a normalized value of δ_{max}. Particularly, δ_{max} is normalized by α, which takes into consideration multi-degree-of-freedom effects. As was the case for the COD, the value of α can be estimated by using design aids such as that shown in Figure 3b (see Table 1). Note that the design displacement spectrum corresponds to specific values of IDI^{P}_{max} and β; and that the 5% percent of critical damping proposed to formulate the spectrum is considered to be a reasonable lower bound for structural materials that reach yielding.

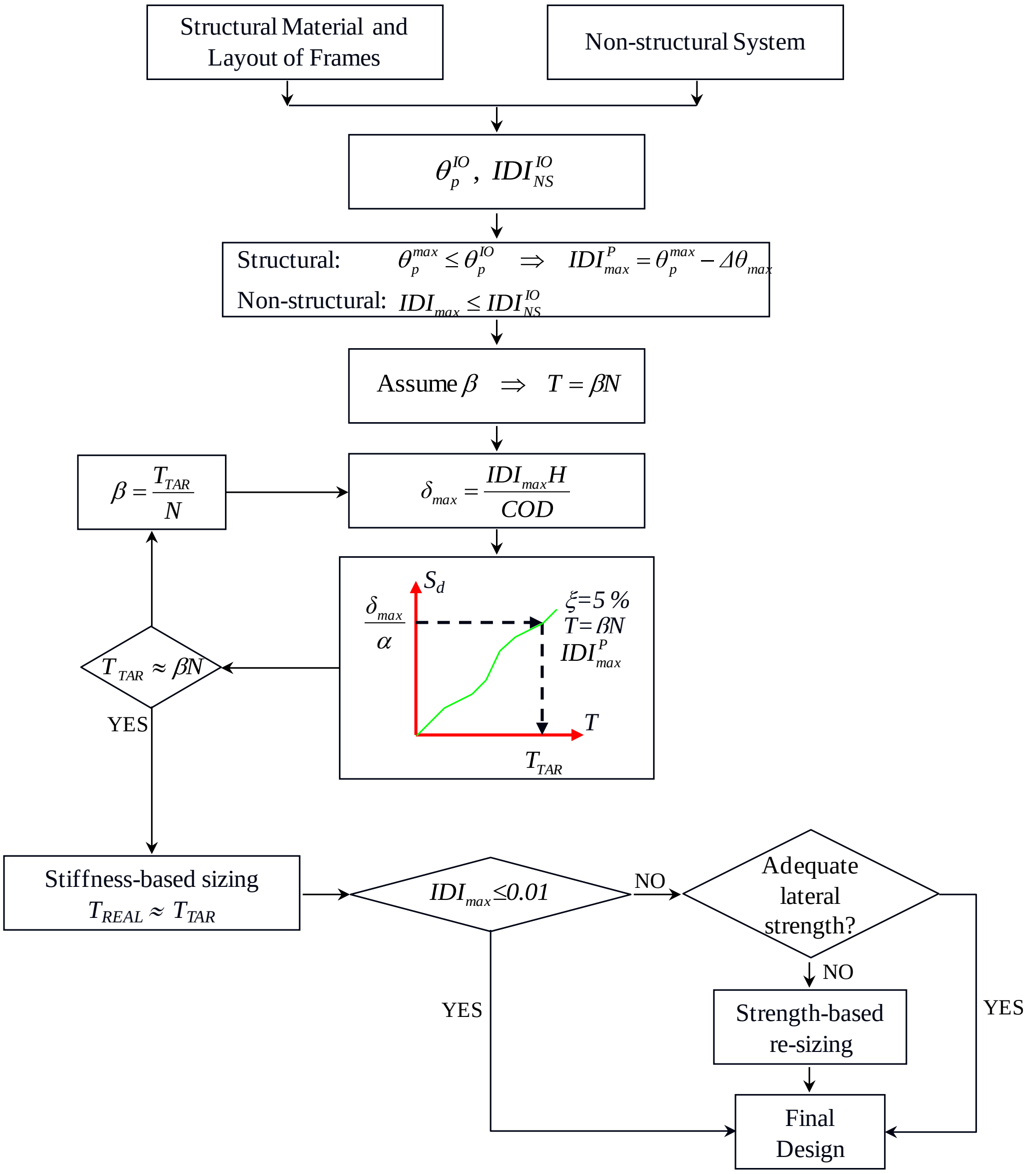

Figure 2. Displacement-based design methodology

Once T_{TAR} is available, it is necessary to check if its value is consistent with that assumed for β. If T_{TAR} is fairly equal to βN, the design proceeds to the stiffness-based sizing of beams and columns. If not, the value of β is actualized and an iteration carried out. In terms of sizing, the transverse sections of beams and columns are deemed adequate if the actual fundamental period of vibration of the frames (T_{REAL}) is sufficiently close to T_{TAR}. Once the beams and columns are sized, the methodology proceeds to its final stage if the design threshold used for IDI_{max} does not exceed 0.010. The final design consists in two tasks: A) The verification of the preliminary design through a series of non-linear time-history analysis and; B) If required, adjustment of the sizes of beams and columns so that the frames can adequately satisfy *Immediate Occupancy*.

In case of frames designed for an IDI_{max} threshold larger than 0.010, it is convenient to check their lateral strength before undergoing the final design. For this purpose, it is necessary to establish a constant IDI^{P}_{max} pseudo-acceleration spectrum corresponding to the final value of β. Within this context, the actual seismic coefficient of the frames, estimated from a static non-linear analysis, should be adequate in relation with the strength ordinate corresponding to the value of T_{REAL}. In case the lateral strength of the frames is insufficient, the beams and columns should be resized to correct the observed deficiencies. The strength revision will be illustrated in detail in the next section.

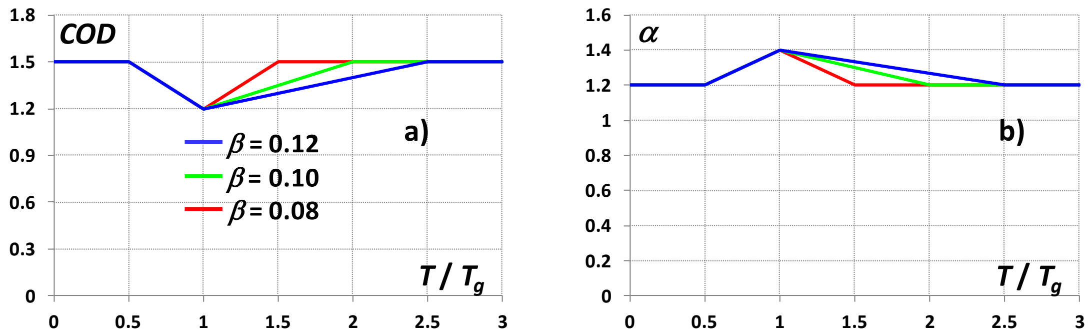

Figure 3. Recommended values for parameters involved in the preliminary design of frames that house essential facilities (θ_{pmax} = 0.005, ξ = 0.05): a) *COD*; b) α corresponding to N = 5+

4.2 Examples

The design methodology is used for the seismic design of two versions of the steel framing system shown in Figure 4, assumed to be located in the Lake Zone of Mexico City. Although the two versions of the building share the same structural layout, they are designed by assuming different values of IDI^{IO}_{NS}. Particularly, while the first version is designed for IDI^{IO}_{NS} of 0.007, the second version considers a threshold of 0.010. In terms of structural performance, both versions are designed so that θ^{IO}_{p} does not exceed 0.005. Also, the frames were designed according to a capacity design approach to meet a weak beam/strong column criteria.

By assuming that $\Delta\theta_{max}$ = 0.002, Equation 6 yields IDI^{P}_{max} = 0.003 for θ^{max}_{p} of 0.005. Note that both versions of the building require the formulation of design spectra corresponding to a constant IDI^{P}_{max} of 0.003. Figure 5 shows design strength and displacement spectra corresponding to different values of β. These spectra were established from the mean + σ spectral ordinates corresponding to a set of seven motions recorded in the Lake Zone of Mexico City and having a dominant period of motion close to 2 seconds. The first iteration considered β = 0.10, in such a manner that the fundamental period of vibration of both versions of the building was initially estimated at 0.10 × 8 = 0.8 seconds. Under the consideration that T_g equals two for the design ground motion, T/T_g = 0.4. According to the design aids included in Figure 3, values of 1.5 and 1.2 were considered, respectively, for *COD* and α. According to Equation 8, δ_{max} values of 11.20 and 16.0 cm were considered for the first and second versions of the frames, respectively. This yielded, respectively, δ_{max}/α of 9.3 and 13.3. By using the approach illustrated in Figure 2, a T_{TAR} of 0.8 seconds was obtained for the first version of the building; and after a single iteration, of 0.96 for the second one (for the latter case, the final value of β is 0.12). Figure 6 shows the estimation of T_{TAR} for both versions of the frames.

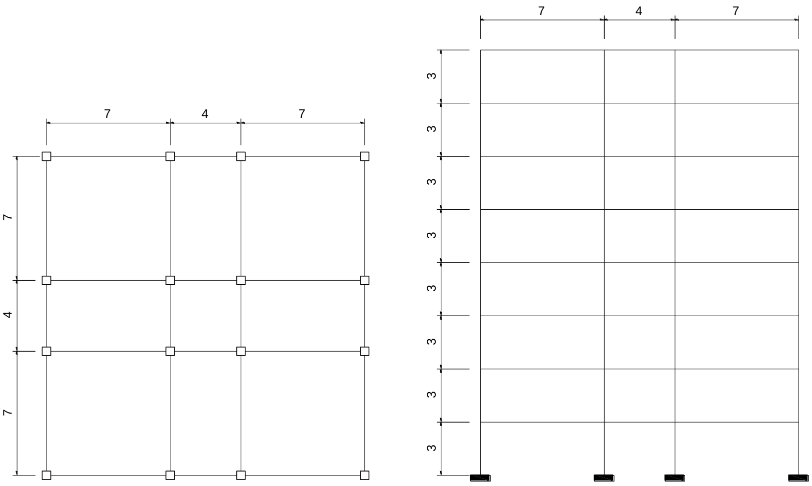

Figure 4. Plan and elevation view of building under consideration (dimensions in meters)

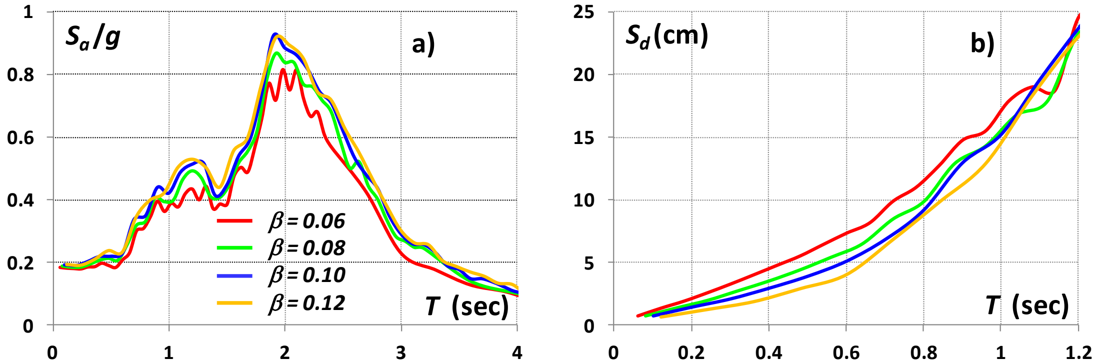

Figure 5. Design spectra for $DI_{max}^{pl} = 0.003$ and $\xi = 0.05$: a) Strength; b) Displacement

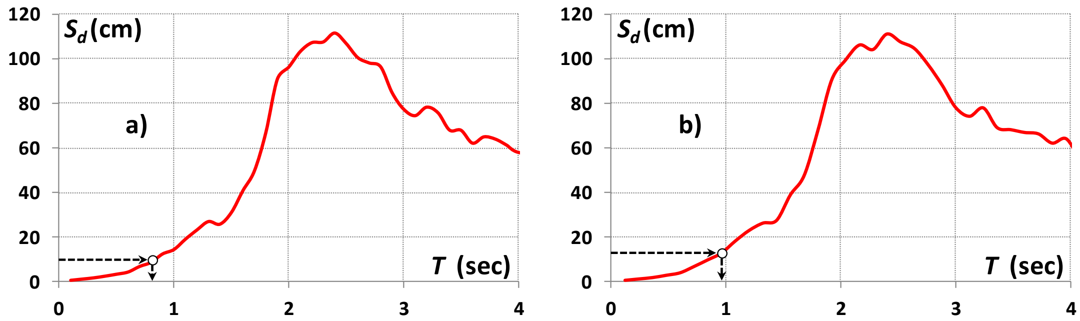

Figure 6. Determination of T_{TAR}: a) First Version ($\beta = 0.10$); b) Second Version ($\beta = 0.12$)

For the stiffness-based sizing of the frames, square box sections were considered for the columns and W-shape sections for the beams. As discussed before, the cross sections are sized in such a manner that the fundamental period of the frames is sufficiently close to the value of T_{TAR}. Table 3 summarizes the sizes of beams and columns for both versions of the frames. While a T_{REAL} of 0.81 seconds was obtained for the first version, the second one exhibited a value of 0.97 seconds. In both cases, A36 steel was considered for the structural elements.

Table 3. Cross sections for beams and columns of both versions of the frame

Stories	Version 1			Version 2		
	Columns		Beams	Columns		Beams
	Side (cm)	Plate thickness (cm)	Section	Side (cm)	Plate thickness (cm)	Section
1-3	70	3.175	W18X97	60	2.540	W18X86
4-6	65	2.540	W18X76	55	1.905	W18X76
7-8	60	2.540	W18X71	50	1.905	W18X60

Detailed two-dimensional non-linear models of both versions of the building were prepared to assess their seismic performance with the DRAIN 2DX program (Prakash et al. 1993). While the beams of the frame 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 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. P-Δ effects were considered through a geometric stiffness matrix, and the base of the columns on the ground story were assumed to be rotationally fixed. In the case of the dynamic non-linear analyses, the non-linear 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.

Figure 7 shows mean + σ maximum inter-story drift index and plastic rotation demands along height for both versions of the building, corresponding to the seven motions used to establish the design spectra. Both versions are able to adequately control their IDI_{max} demands within their design thresholds. Although in terms of their structural performance, both versions also satisfy their design objectives, it is worth noting that the θ_p^{max} demands shown in Figure 7d are closer to the design threshold of 0.005, and significantly larger than those summarized in Figure 7b. On one hand, the θ_p^{max} demands shown in Figure 7b illustrate the unlikeness that the structural elements of frames designed for $IDI_{max} \leq 0.01$ have to be re-sized based on strength considerations. On the other hand, the θ_p^{max} demands shown in Figure 7d illustrate the importance of checking the lateral strength of frames designed for $IDI_{max} >$ 0.01, and the fact that as the allowable drift in the frames increase, the sizing of the beams and columns may be governed by strength rather than stiffness considerations.

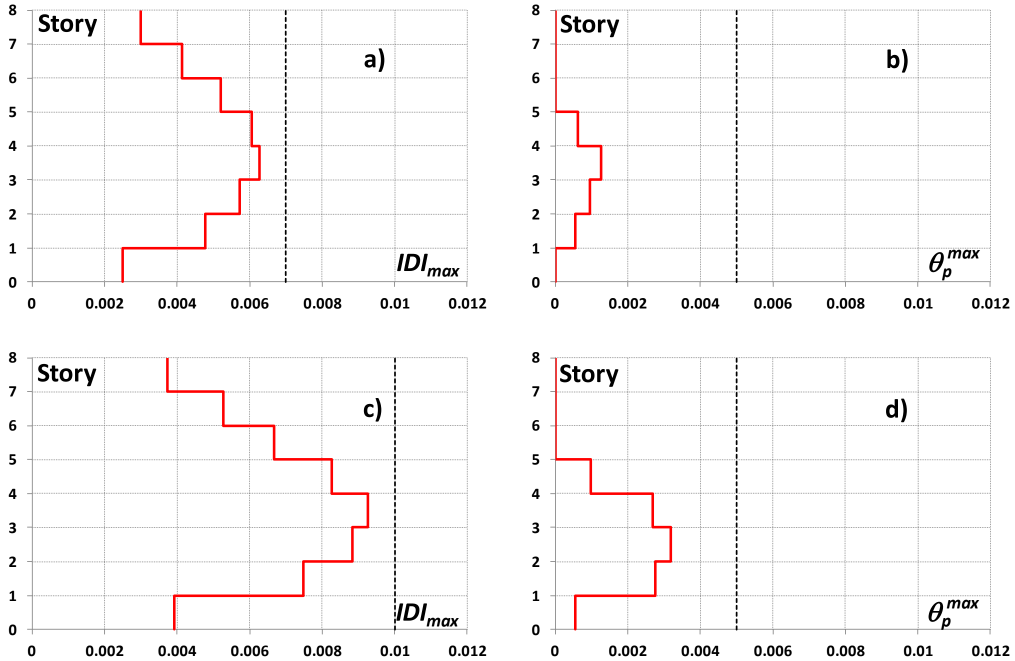

Figure 7. Maximum deformation demands along height for both versions of the building: a) IDI_{max}, Version 1; b) θ_p^{max}, Version 1; c) IDI_{max}, Version 2; d) θ_p^{max}, Version 2;

In terms of the methodology, it can be said that its application has resulted in that the two versions of the building adequately satisfy the *Immediate Occupancy* performance level. Nevertheless and in terms of illustrating the strength revision required for frames designed for IDI_{max} > 0.01, Figure 8 shows the capacity curve and design strength spectrum for the second version of the building. In terms of the required lateral strength, Figure 8a yields a design seismic coefficient of 0.43. In terms of the actual seismic coefficient of the frames, the bilinear idealization of the capacity curve showed in Figure 8b yield a seismic coefficient of about 0.42. Under the consideration of a response fully dominated by the first mode of vibration, and that this first mode moves about 85% of the total mass of the frames, the strength demands in terms of seismic coefficient can be expressed as 0.85 × 0.43 = 0.37. The second version of the building, designed for IDI_{max} = 0.01, exhibits a close supply-demand balance of strength (0.42 versus 0.37). Not surprisingly, the values of θ_p^{max} shown in Figure 7d are close to their design threshold.

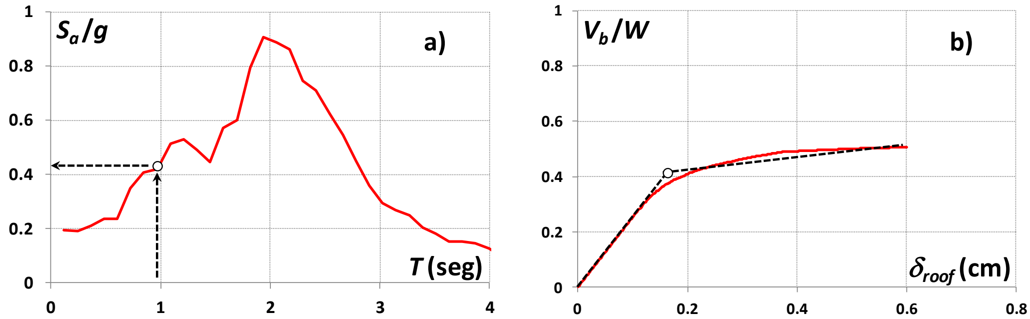

Figure 8. Strength revision for second version of the building, ξ = 0.05 and IDI^{P}_{max} = 0.003: a) Strength demand (β = 0.12); b) Strength capacity

5. CONCLUSIONS

The explicit control of the dynamic response of essential facilities built in very soft soils is instrumental to achieve an adequate seismic performance. Within this context, simple displacement-based formats can be used for the conception and preliminary design of moment-resisting frames that are able to adequately control the level of structural and non-structural damage in essential facilities through adequate control of the lateral drift and plastic rotation demands. The use of one such format for the seismic design of two versions of an eight-story steel building has resulted in structural frames that are able to satisfy the *Immediate Occupancy* performance level during the occurrence of their design ground motion. Further studies have to be carried out to calibrate the use of the proposed methodology for other type of structural material and systems, and to evaluate the effect of structural and mass irregularities on the value currently contemplated for the design parameters.

REFERENCES

Applied Technology Council (1998), "Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual", Reporte FEMA 306.

Bojórquez E., Terán-Gilmore A., Ruiz S.E. y Reyes-Salazar A. (2011), "Evaluation of Structural Reliability of Steel Frames: Inter-story Drift versus Plastic Hysteretic Energy", *Earthquake Spectra*, 27(3), 661-682.

Prakash, V, Powell G H, Campbell S (1993), "Drain-2Dx base program description and user guide.", Manual, University of California.

Terán-Gilmore A. (1998), "Características Mecánicas y desempeño sísmico de marcos dúctiles de concreto reforzado", XI Congreso Nacional de Ingeniería Estructural, I, 564-573.

Terán-Gilmore A. (2004), "On the use of spectra to establish damage control in regular frames during global predesign", Earthquake Spectra, 20(3), 1-26.

Terán Gilmore A., Diaz G. and Reyes Salinas C. (2010), "Estudios sobre el factor de importancia utilizado durante el diseño sismorresistente de hospitales", *XVII Mexican Conference on Structural Engineering (CD).*