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PROFILE OF SEISMIC BEHAVIOR FROM DYNAMIC PARAMETERS: BUILDINGS DESIGNED IN MEXICO CITY

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Abstract

This work presents and discusses the results of the application of criteria or methodologies used to establish an evaluation of the structural health and seismic performance of an important group of existing buildings in Mexico City representing the various types of constructions. It starts from the concept called "Bio-seismic profile of buildings", developed by the Chilean engineers Tomás Guendelman, Mario Guendelman and Jorge Lindenberg. The methodology applied, characterizes the buildings by 13 seismic performance indicators which consider the type of structure, mechanical properties of the elements, parameters that govern the dynamic behavior of structures and the response obtained by seismic actions among others. Based on buildings models results and in the behavior experienced by the existing constructions during recent earthquakes, modifications to the proposed methodology are made to define the limits of the dynamic parameters inherent to the philosophy of seismic design in Mexico, that it considers a ductile behavior of structures.

This work has detailed information of dynamic analyses made to the buildings studied. The results of the application of the methodology of health and structural behavior are congruent with the good response observed in the buildings, in spite of having been designed with earlier regulations which derived later from the earthquake on September 19th, 1985 in Mexico City. It is suggested to extend the application of this methodology to evaluate another type of buildings under the base of steel and masonry structures.

Finally, the results of the application of this methodology are compared and discussed with the evidence of damage caused by intense earthquakes and/or the real response experienced before events of such nature.

1. Introduction

Evaluation of structural safety or vulnerability of a building is intended to estimate the resistant capacity of a building before or after the occurrence of an earthquake. Such resistant capacity can be estimated from different methods that involve assumptions and simplified approximations to analytical methods that provide more detailed information. The type of the selected method of analysis depends on the purpose of the evaluation, availability or existence of data, resources and technology (Aragon, 2013).

There are several classifications of the evaluation methods of seismic vulnerability of buildings, most of them are defined based on the available information of the structures to be evaluated and, it is taken into account the different types and analytical or empirical procedures needed to obtain a good estimate of the condition of the buildings. Calvi y Pinho (Calvi, 2006) propose a classification of these methods into two large groups: empirical and analytical. Empirical methods establish a relationship between structure damage and a damage scale proposed by specialists from the behavior of buildings with similar characteristics. As for analytical methods, the damage scale is obtained from the calculation of the mechanical properties of a building for a determined damage limit calculated from the forces induced by an earthquake (Aragon, 2013).

This article is the continuation of a line of work in the process of development by the authors focused not on the estimation of a structure resistant capacity but on the estimation of a possible behavior that could be experienced by intense seismic effects based on the Bio-Seismic Profile using behavioral parameters established in the Mexican code (NTCDS-RCDF04). Previous studies (Guendelman, 2010) have shown that the application of methodologies focused to review dynamic parameters can detect existing deficiencies on structural systems that allow make corrections to help prevent collapses or damages. The recommendations of the methodology used, can derive the necessity to develop of more detailed complementary studies.

In general terms, the Bio-seismic Profile is a methodology that allows us to evaluate the 'health' of a building through the revision of a series of global indicators associated to its structural conception and its dynamic response before seismic actions that let us detect deficiencies in the resistant structure to propose any corrections or recommended complementary studies with greater analytical rigor.

Hereby, the results of the application of the methodology above mentioned are presented and discussed to establish an evaluation of health and structural behavior for two buildings, one of reinforced concrete of 15 stories and the other of steel of 22 stories subjected to intense earthquakes and its correlation with the actual response experienced before events of such Bio-seismic profile. In addition, there was information of the ocular inspection to buildings.

As a result of the application of the methodology of structural behavior and health, the results obtained were congruent with the good seismic response observed of the structural inspection of the buildings, in spite of having been designed with earlier regulations which derived later from the earthquake on September 19th, 1985 in Mexico City. It is suggested to extend the application of this methodology to evaluate another type of buildings.

2. Bio-seismic profile

The Bio-seismic Profile was first presented at the VII Chilean Conferences on Seismology and Anti-seismic Engineering (ACHISNA) by the engineers Tomás Guendelman, Mario Guendelman and Jorge Lindenberg in 1997.

The good behavior of the buildings before the earthquake on March 3rd in Chile gave the engineers hints for investigation and documentation of the characteristics and dynamic parameters of those buildings starting from a base of 585 real buildings and it was possible to develop a methodology for seismic evaluation of reinforced concrete buildings by the evaluation of indicators that can be compared to satisfactory values. At the beginning, the field of action was limited to buildings up to 30 stories recommending its use in buildings of 20 stories since the shown statistics then concentrated on that height. It is important to mention that the calculation of the indicators is carried out from the results of the normative of the seismic analysis (Latorre, 2010).

Currently, the range of coverage has been extended (Henoch, 2007) up to the so called skyscrapers which are the highest buildings and with the global trend; its use has widespread (Latorre, 2010).

The seismic indicators are clustered in three groups:

a. Stiffness Indicators.

1. Total height/ first period translational mode
2. Effect P – Δ
3. Displacement of the top level
4. Maximum displacement of interstory in gravity center
5. Further maximum displacement of interstory in endpoints

b. Couplings Indicators

6. Rotational period / translational period
7. Coupled rotational equivalent mass / direct translational equivalent mass
8. Dynamic Eccentricity / basal turning radius
9. Coupled translational equivalent mass / direct translational equivalent mass
10. Coupled basal shear / direct basal shear
11. Coupled basal moment / direct basal moment

c. Structural Redundancy and Ductility Demand Indicators

12. Number of structural elements in the seismic resistance
13. Effective Spectral Reduction Factor

Hereafter, each of the indicators above mentioned will be determined. It is worth mentioning that the limits of some of the indicators will be modified in their original value to consider the Mexican code (NTCDS-RCDF04).

a. Stiffness Indicators.

1. Total height/ first period translational mode

$$\frac{H}{T} \tag{1}$$

where:

H: total height of building, (m).
T: translational period, (s).

if $20 > \frac{H}{T}$ $\Rightarrow$ extremely flexible building

if $20 \le \frac{H}{T} < 30$ $\Rightarrow$ flexible building

if $30 \le \frac{H}{T} < 70$ $\Rightarrow$ building with normal stiffness

if $70 \le \frac{H}{T} < 150$ $\Rightarrow$ stiff building

if $150 \le \frac{H}{T}$ $\Rightarrow$ building with excessive stiffness

This indicator represents a very good estimate of a building stiffness. However, an extra discussion derives from the use of this indicator about what period to use. Traditionally, the mode with greater translational mass is used, but this is only a criteria. The modes can also be ordered by magnitude, by participation in the basal shear, by energy, etc. Ordering the modes according to these different conditions will influence the ranges of evaluation of the parameter of the translational stiffness of the structure. For this specific parameter, the period with greater translational mass continued to be determined and it is concluded that there is a new possible parameter, an indicator using the period with greater contribution to the basal shear of the structure (Henoch, 2007).

2. Effect P – Δ

$$\frac{M_{P-\Delta}}{M_{Basal}} \tag{2}$$

Effect P – Δ < 0.05	⇒ Effect P – Δ ignored
0.05 < Effect P – Δ < 0.1	⇒ Effect P – Δ directly added
0.1 > Effect P – Δ	⇒ Effect P – Δ may produce instability

It is suggested to measure this effect through the quotient divided by the basal flector ($M_{P-\Delta}$) generated by the accumulated products of the weight of each story by their respective lateral displacements and the basal moment (M_{Basal}) due to the seismic action (Henoch, 2007).

It should be noted that its use as a stiffness indicator is reduced since in practice the control of movements is restricted by the Chilean norm. Nonetheless, it can be important to the case of high buildings (Henoch, 2007).

Within the NTCDS, the limit 0.08 for the effects P – Δ is used by which the limits of this indicator are changed:

Effect P – Δ < 0.08	⇒ Effect P – Δ ignored
0.08 > Effect P – Δ	⇒ Effect P – Δ directly added

3. Displacement of the top level.

$$1000\,\frac{\delta}{H} \tag{3}$$

$$\text{if } 0{,}2 > 1000\,\frac{\delta}{H} \qquad \Rightarrow \text{excessive stiffness}$$

$$\text{if } 0{,}2 \le 1000\,\frac{\delta}{H} \le 2 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 2 < 1000\,\frac{\delta}{H} \qquad \Rightarrow \text{out of range}$$

where:

δ: Displacement of top level, measured in the center of masses, (m).
H: Total height of building, (m).

Though the Chilean regulation does not restrict this parameter, it does with the interstory displacements (NCh433.Of96). Nonetheless, for many foreign regulations this parameter is restricted or used as a control point of the behavior of the structure (Henoch, 2007).

Since the NTCDS allow a greater displacement of the interstory (0.006 y 0.012), these limits change as follows:

$$\text{if } 2 > 1000\,\frac{\delta}{H} \qquad \Rightarrow \text{excessive stiffness}$$

$$\text{if } 2 \le 1000\,\frac{\delta}{H} \le 12 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 12 < 1000\,\frac{\delta}{H} \qquad \Rightarrow \text{out of range}$$

4. Maximum displacement of interstory in gravity center

$$1000\ \frac{\delta_{cg}}{h} \tag{4}$$

$$\text{if } 0,2 > 1000\ \frac{\delta_{cg}}{h} \qquad \Rightarrow \text{excessive stiffness}$$

$$\text{if } 0,2 \leq 1000\ \frac{\delta_{cg}}{h} \leq 2 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 2 < 1000\ \frac{\delta_{cg}}{h} \qquad \Rightarrow \text{out of range}$$

where:

δ_{cg}: displacement of interstory in gravity center
h: interstory height

This limitation is controlled in the Chilean regulation NCh433.Of96, so the buildings in Chile are forced to comply with it. However, the various forms of high building structures will show if there is something of this philosophy in its design (Henoch, 2007).

As mentioned in indicator 3, the NTCDS allow a greater displacement of the interstory; therefore, the limits are modified as follows:

$$\text{if } 2 > 1000\ \frac{\delta_{cg}}{h} \qquad \Rightarrow \text{excessive stiffness}$$

$$\text{if } 2 \leq 1000\ \frac{\delta_{cg}}{h} \leq 12 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 12 < 1000\ \frac{\delta_{cg}}{h} \qquad \Rightarrow \text{out of range}$$

5. Further maximum displacement of interstory in endpoints

$$1000\ \frac{\delta_{extremo}}{h} \tag{5}$$

This expression can be written like this:

$$1000\ \frac{\left(\delta_e - \delta_{cg}\right)}{h} \tag{6}$$

$$\text{if } 1000\ \frac{\delta_{extremo}}{h} \leq 1 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 1 < 1000\ \frac{\delta_{extremo}}{h} \qquad \Rightarrow \text{out of range}$$

where:

δ_e: Maximum displacement between plant points.
δ_{cg}: displacement of interstory in gravity center.
h: interstory height.

Due to the structural system that have tall buildings in which there are usually two areas resistant to efforts (the core and the plant perimeter), the displacements generated at the endpoints can be very different from the ones of the gravity center.

This explains why, when using the displacement in gravity center, the effects of the plant rotation are not considered but are important in endpoints. In these, the effects of the rotations can considerably increase or decrease the displacements. The parameter allows controlling the displacements in the perimeter elements (Henoch, 2007).

As mentioned in indicator 3, the NTCDS allow a greater displacement of the interstory; therefore, the limits are modified as follows:

$$\text{if } 1000\,\frac{\delta_{extremo}}{h} \le 12 \qquad \Rightarrow \text{normal stiffness}$$

$$\text{if } 12 < 1000\,\frac{\delta_{extremo}}{h} \qquad \Rightarrow \text{out of range}$$

b. Couplings Indicators

6. Rotational period / translational period

$$\frac{T_\theta}{T^*} \tag{7}$$

$$\text{if } 0{,}8 \ge \frac{T_\theta}{T^*} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 0{,}8 < \frac{T_\theta}{T^*} \le 1{,}2 \qquad \Rightarrow \text{acceptable values}$$

$$\text{if } 1{,}2 < \frac{T_\theta}{T^*} \le 1{,}5 \qquad \Rightarrow \text{normal values}$$

$$\text{if } 1{,}5 < \frac{T_\theta}{T^*} \le 2 \qquad \Rightarrow \text{acceptable values}$$

$$\text{if } 2 < \frac{T_\theta}{T^*} \qquad \Rightarrow \text{out of range}$$

where:

T_θ: rotational period, (s).
T*: translational period, (s).

As stated in the Bio-seismic profile of (Guendelman et al., 1997), what is recommended for the structures is that the reason between the translational and the rotational periods in regard to a vertical shaft moves away from the unit. This is due to the phenomenon called modal tuning that can cause dynamic amplifications in the response of the structure (Henoch, 2007).

7. Coupled rotational equivalent mass / direct translational equivalent mass

$$\frac{M_{nx\theta}}{M_{nx}},\ \frac{M_{ny\theta}}{M_{ny}} \tag{8}$$

$$\text{if } 20\% \ge \frac{M_{nx\theta}}{M_{nx}}\ o\ \frac{M_{ny\theta}}{M_{ny}} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 20\% < \frac{M_{nx\theta}}{M_{nx}}\ o\ \frac{M_{ny\theta}}{M_{ny}} \le 50\% \qquad \Rightarrow \text{acceptable values}$$

$$\text{if } 50\% < \frac{M_{nx\theta}}{M_{nx}}\ o\ \frac{M_{ny\theta}}{M_{ny}} \qquad \Rightarrow \text{out of range}$$

It helps complementing the characteristics of coupling that may exist. It is calculated form the ratio of the rotational equivalent mass that occurs in the most important translational mode and thereby the direct translational mass. (Henoch, 2007).

8. Dynamic Eccentricity / basal turning radius.

$$\frac{\left(\dfrac{M_{torsor\ basal}}{Q_{basal}}\right)}{r_{basal}} = \frac{e_{din}}{r_{basal}} \tag{9}$$

$$\text{if } 20\% \geq \frac{e_{din}}{r_{basal}} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 20\% < \frac{e_{din}}{r_{basal}} \leq 50\% \qquad \Rightarrow \text{acceptable values}$$

$$\text{if } 50\% < \frac{e_{din}}{r_{basal}} \qquad \Rightarrow \text{out of range}$$

This is another way of evaluating the dynamic coupling. It is calculated as the quotient by the dynamic eccentricity and the turning basal radius. At the same time, the dynamic eccentricity is calculated as the torsional basal moment divided by the basal shear. On the other hand, the turning radius is equal to the square root of the rotational inertia of the structure plant (Henoch, 2007).

9. Coupled translational equivalent mass / direct translational equivalent mass

$$\frac{M_{nxy}}{M_{nx}}, \frac{M_{nyx}}{M_{ny}} \tag{10}$$

$$\text{if } 50\% \geq \frac{M_{nxy}}{M_{nx}} \text{ o } \frac{M_{nyx}}{M_{ny}} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 50\% < \frac{M_{nxy}}{M_{nx}} \text{ o } \frac{M_{nyx}}{M_{ny}} \qquad \Rightarrow \text{out of range}$$

The same as in point 7, it seeks to relate another way the degree of structure coupling relating the coupled translational equivalent mass with the direct one (Henoch, 2007).

10. Coupled basal shear / direct basal shear.

$$\frac{Q_{0xy}}{Q_{0xx}}, \frac{Q_{0yx}}{Q_{0yy}} \tag{11}$$

$$\text{if } 50\% \geq \frac{Q_{0xy}}{Q_{0xx}} \text{ o } \frac{Q_{0yx}}{Q_{0yy}} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 50\% < \frac{Q_{0xy}}{Q_{0xx}} \text{ o } \frac{Q_{0yx}}{Q_{0yy}} \qquad \Rightarrow \text{out of range}$$

In this case, it relates the coupled basal shear, that is, the basal shear produced in the perpendicular direction to the direction of the seismic application in the model, versus the basal shear that goes in the same direction, that is, direct (Henoch, 2007).

11. Coupled basal moment / direct basal moment.

$$\frac{M_{v0xy}}{M_{v0xx}}, \frac{M_{v0yx}}{M_{v0yy}} \tag{12}$$

$$\text{if } 50\% \geq \frac{M_{v0xy}}{M_{v0xx}} \text{ o } \frac{M_{v0yx}}{M_{v0yy}} \qquad \Rightarrow \text{normal values}$$

$$\text{if } 50\% < \frac{M_{v0xy}}{M_{v0xx}} \text{ o } \frac{M_{v0yx}}{M_{v0yy}} \qquad \Rightarrow \text{out of range}$$

As in the previous point, it seeks to relate the couplings that are produced in the structure between the coupled efforts against the direct ones, in this case, the basal moment (Henoch 2007).

Structural redundancy and ductility demand indicators.

12. Number of structural elements in the seismic resistance

$$N° \tag{13}$$

$$\text{if } 2 > N° \qquad \Rightarrow \text{out of range}$$
$$\text{if } 2 \leq N° < 3 \qquad \Rightarrow \text{acceptable values}$$
$$\text{if } 3 \leq N° \qquad \Rightarrow \text{normal values}$$

This parameter will qualitatively evaluate the structural redundancy that a building has. This is especially important when the seismic loads lead the structure to break into the non-linear range redistributing the efforts in the structure.

Its value will be calculated as follows: it will be equal to the number of basal elements that might accumulate 90% of the basal shear and/or those elements whose shear is greater to 10% of the basal shear (Henoch, 2007).

13. Effective Spectral Reduction Factor.

$$R^{**} \tag{14}$$

$$\text{if } 3 \geq R^{**} \qquad \Rightarrow \text{normal values}$$
$$\text{if } 3 < R^{**} \leq 7 \qquad \Rightarrow \text{acceptable values}$$
$$\text{if } 7 < R^{**} \qquad \Rightarrow \text{out of range}$$

As Guendelman et al 1997 says, the Response Modification Factor is currently an accepted concept worldwide, and helps to generate a seismic design for the linear analysis of a structure (Henoch, 2007).

The regulation in Chile establishes that the design should be obtained out of a dynamic seismic analysis with and elastic spectrum reduced by the R* factor. This is according to the Response Modification Factor and the period of greater translational mass; but at the same time these results have to be modified according to the basal shear amplifying them if the basal shear is lesser than the minimum basal shear, or reducing them if they exceeded the maximum basal shear. In addition, the seismic action must be amplified by 1.4 according to the method of load and resistance factors fort the design of the structure. All this allowed to define an Effective Spectral Reduction Factor R**, that is (Henoch, 2007):

$$R^{**} = \frac{R^{*}}{1.4\, f_{\min}\, f_{\max}} \tag{15}$$

where: R*: reduction factor of spectral acceleration
f_{min}: amplification factor for minimum basal shear
f_{max}: reduction factor for maximum basal shear

Table 1 shows an overview of the recommended limits of each indicator, it is important to mention the respective modification in relation to the Mexican code NTCDS-RCDF04.

Table 1 - Summary of limits for indicators of the Bio-seismic profile

Bio-seismic Profile Indicators	Values within normal ranges (Original)	Values within normal ranges (NTCDS)	Acceptable values slightly away from normal ranges. (Original)	Acceptable values slightly away from normal ranges. (NTCDS)
Rigidity				
1. H / T [m/s]	30 – 70		20 – 30 and 70 – 150	
2. $M_{P-\Delta} / M_{basement}$	0 – 0.1	0 - 0.08		
3. 1000 δ / H	0.2 ‰ – 2 ‰	2 ‰ – 12 ‰	0 ‰ – 0.2 ‰	0 ‰ – 2 ‰
4. 1000 δ_{gc} / H	0.2 ‰ – 2 ‰	2 ‰ – 12 ‰	0 ‰ – 0.2 ‰	0 ‰ – 2 ‰
5. 1000 δ_{end} / H	0 ‰ – 1 ‰	0 ‰ - 6 ‰		
Link translation - rotation and translation - translation				
6. T_θ / T^*	0 – 0.8 and 1.2 – 1.5		0.8 – 1.2 and 1.5 – 2.0	
7. $M_{nx\theta} / M_{nx}$, $M_{ny\theta} / M_{ny}$	0 – 0.2		0.2 or more	
8. $e_{din} / r_{basement}$	0 – 0.2		0.2 or more	
9. M_{nxy} / M_{nx}, M_{nyx} / M_{ny}	0 – 0.5		0.5 or more	
10. Q_{0xy} / Q_{0xx}, Q_{0yx} / Q_{0yy}	0 – 0.5		0.5 or more	
11. M_{v0xy} / M_{v0xx}, M_{v0yx} / M_{v0yy}	0 – 0.5		0.5 or more	
Structural redundancy and ductility demand				
12. N	more than 3		2 – 3	
13. R^{**}	less than 3		3 – 7	

3. Case studies

3.1 Building 1 (Picacho Tower).

Building 1 is meant for offices, it is a reinforced concrete structure of 13 stories and 1 basement. Its structural system is formed by columns and reticular flat slabs, with a concrete core at the center of the plant that houses the elevators and the internal stairs. The basement is a box with perimeter concrete walls to the building which allows the distribution of the loads of the foundation. The concrete wall continues to the ground floor only on the border walls that are perpendicular to the street where the building is located. Figure 1 shows a view of the structural model.

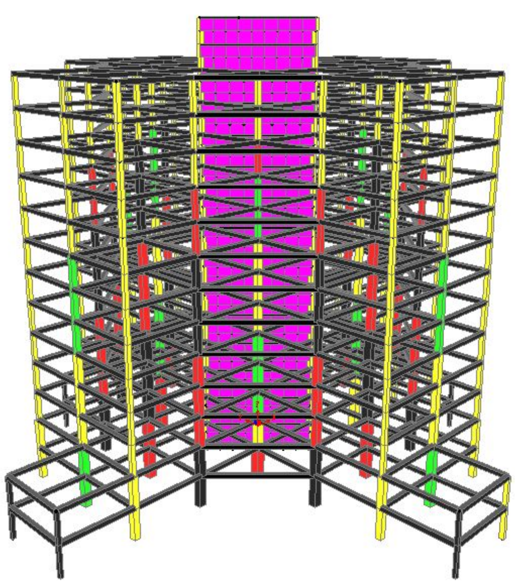

Fig. 1 – Structural model of building 1

The figure is constructed on a site located in area I (firm ground) according to the seismic zoning established by the Complementary Technical Norms for Earthquake Resistant Design NTCDS-RCDF04, with a dominant period of the ground of approximately 0.5 seconds.

A dynamic analysis of the structure based on the corresponding spectrum design to area I (firm ground) was carried out. Conditions of gravitational and lateral (seismic) loading were considered and the corresponding combinations that the NTC for design for earthquake were defined, including the bidirectional effects, fig.2

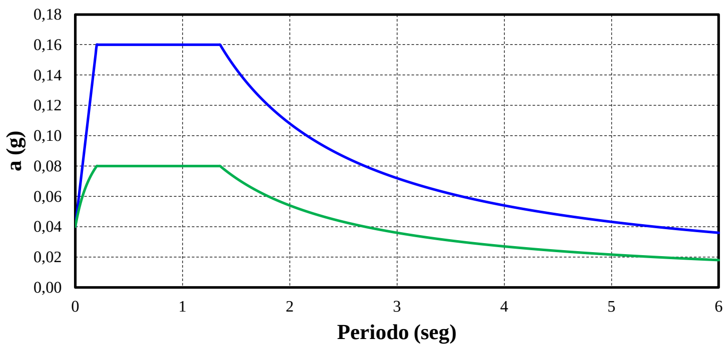

Fig. 2 – Spectrum design of building 1

Based on the results of the structure analysis, the revision of the serviceability limit state and the ultimate limit state was carried out with the outcome that the building reasonably complied even though the building was projected and build before the onset of the current regulation (García y Granados, 2007).

3.2 Building 2 (Executive Tower).

Building 2 is meant for offices. It is a mixed construction of foundation, basements as parking areas and a core of walls in the central area of stairs and elevators solved with reinforced concrete base with a square form of 10 m per side, while surrounding the concrete core walls, 8 metal columns located at 7.5 mm from the corners of the core walls are added and a superstructure is formed in the form of a tower with a square plant that measures 25 m per side that rise 21 floor stories of reinforced concrete slabs supported on metal girders consisting of parallel and diagonal cord truss in V of the P.B. to the first level (13.15 m), there is only the central core of concrete walls, clad with precast with sculptural bas-reliefs which creates a large lobby with a height close to 12.00 m between the floor and the plafond of floor 1.

It is noteworthy that the corners of the tower are in double overhang 7.5 m. In the area of the core of concrete walls, there are restrooms located in between stories with slabs and beams of reinforced concrete that support the walls. The stairs are based on ramps of reinforced concrete loaded with metal beams of 12” standard joist. The in between story heights are uniform from story 1 to story 21, but as mentioned on the above paragraph, it has a lobby with an approximate height of 12 m around the central core. From story 21 (roof) upwards the core of concrete walls continues to structure the machine room of the elevators; on the outer perimeter the 8 steel columns are continued and are connected to the central core on the ceiling of the machine room with metal girders of 7.5 m. Finally, the entire outer perimeter has aluminum works and glasses protecting the area from the roof from the winds which are, at that height, considerably strong, fig. 3.

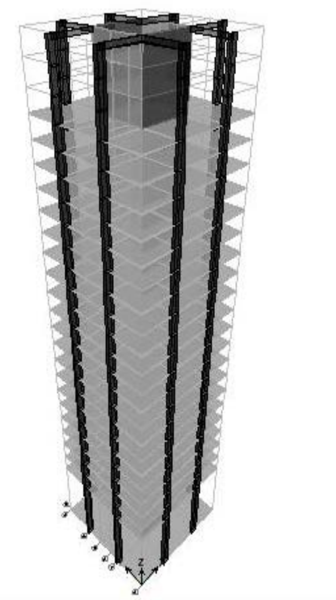

Fig. 3 – Structural model of building 2

According to the seismic zoning, the structure was located in area II and a Q=2 was used for its analysis, fig.4.

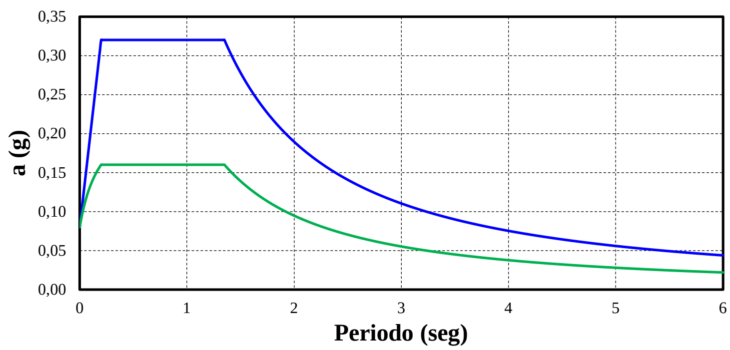

Fig. 4 – Spectrum design of building 2

According to the results of the analysis, the existing structure does not support the lateral seismic forces adequately as the 2004 regulation requires, considering it in group B as in the B1 or A group; these results are presented in detail in reference to a study of (Cid and Montoya, 2007).

4. Inspection results

4.1 Building 1.

Was a general inspection of the building, which was focused to the identification of damage to structural elements and joints of beams and columns that were considered critical to detect possible anomalies in their structural work. Is inspected construction boards, systems of floor, stairs, elevators and buckets in spaces where different facilities are located. With respect to the inspection of Foundation, reviewed the existence of patterns of cracks in the walls and floor of the drawer concrete, without finding any sign of damage. These results were correlated with the topographical survey which indicated that there was no differential settlements suffered by the building which put in evidence a misbehavior.

For this case, it was concluded that the building has a sound structure, with proper maintenance and without structural damage as a result of past earthquakes.

4.2 Building 2.

The inspection found a structure with no obvious damage, but on the walls of concrete of the central core of stairs and elevators opened hole using chisel and pot to spend electrical pipes and/or ducts. They also appreciated by beams of concrete on elevator doors, hidden by panels, some fissures and cracks probably due to contractions of setting.

The behavior of the Foundation has been magnificent, which has been shown not only the ocular inspection but the topographical leveling performed. However, trying to pass the field mechanical elements as high as those who generate hypothetical lateral forces required by the current Regulation (RCDF-04), the Foundation would require major modifications, which so far has not needed.

In summary, of the inspection to the 2 buildings in 2007, he could find that their behavior before intense earthquakes, including the one that occurred in the Mexico City in 1985, has been broadly satisfactory, not being necessary to enable any system of protection or seismic reinforcement.

5. Results of the profile Bio - seismic

Table 2 is the summary of the compliance indicators for all the models of the buildings studied. It can be seen that the buildings designed with the regulation NTCDS-RCDF04 exceed the limits set by the indicators 2, 3, 4, and 5, and it is particularly seen that the models with asymmetric distributions by stiffness are presenting greater seismic vulnerability.

Table 2 – Summary of the limits for the Bio-seismic Profile Indicators

Bio-seismic Profile Indicators	Values within the normal ranges (NTCDS)	Acceptable values slightly away from normal ranges (NTCDS)	Executive Tower		Picacho Tower	
			shaft x	shaft y	shaft x	shaft y
1. H / T [m/s]	30 – 70	20 – 30 and 70 – 150	46,14	36.35	34.24	34.23
2. $M_{P-\Delta} / M_{basement}$	0 - 0.08	–	0.209	0.372	0.268	0.268
3. $1000\ \delta / H$	2 ‰ – 12 ‰	0 ‰ – 2 ‰	5.45	7.37	1.94	1.94
4. $1000\ \delta_{gc} / H$	2 ‰ – 12 ‰	0 ‰ – 2 ‰	6.81	8.84	2.32	2.32
5. $1000\ \delta_{end} / H$	0 ‰ – 6 ‰	–	3.91	0.68	0.77	0.08
6. T_θ / T^*	0 – 0.8 and 1.2 – 1.5	0.8 – 1.2 and 1.5 – 2.0	1.00	0.79	0.94	0.94
7. $M_{nx\theta} / M_{nx}$, $M_{ny\theta} / M_{ny}$	0 – 0.2	0.2 or more	45.62%	0%	0.24%	0%
8. $e_{din} / r_{basement}$	0 – 0.2	0.2 or more	0.44	0.04	0.84	1.46
9. M_{nxy} / M_{nx}, M_{nyx} / M_{ny}	0 – 0.5	0.5 or more	0%	0%	48%	48%
10. Q_{0xy} / Q_{0xx}, Q_{0yx} / Q_{0yy}	0 – 0.5	0.5 or more	1%	2%	75%	75%
11. M_{v0xy} / M_{v0xx}, M_{v0yx} / M_{v0yy}	0 – 0.5	0.5 or more	2%	2%	71%	71%
12. N	more than 3	2 – 3	4	4	7	7
13. R^{**}	less than 3	3 – 7	0.95	0.95	1.43	1.43

This summary chart shows the values in blue as the ones that comply with the original limits and the ones modified by the NTCDS.

Comments on the results of the Bio-seismic Profile

Based on the results of the global indicators presented in previous figures, an opinion is issued about its influence on seismic behavior that the 2 buildings have experienced.

1 H / T.

Both structures have an appropriate balance in their lateral stiffness according to their height.

2 Effect P – Δ.

Effects P – Δ are important in the behavior of buildings.

3 Displacement of the top level

The displacements of both tower roofs do no exceed suggested limits.

4 Maximum displacement of interstory in referred gravity center.

The displacements of interstory referred to the center of mass of both buildings do not exceed suggested limits.

5 Maximum displacement of interstory in endpoints of the plant.

The displacements of interstory in endpoints of the plant of both buildings do no exceed suggested limits.

6 Torsional Stiffness.

Based on the relationship of decoupled rotational period / translational period, building 1 is a torsional stiff building, while building 2 is torsional flexible.

7 Coupled rotational equivalent rotational mass / direct translational equivalent mass.

Building 2 is almost in the recommended exposure limit which is congruent with the result in indicator 6.

8 Dynamic Eccentricity / basal turning radius.

Although the recommended limits do not exceed, according to the results of building 1, they can show torsion effects relatively more important than in building 2.

9 Coupled translational equivalent mass / direct translational equivalent mass.

Although the recommended limits do not exceed, according to the results, building 1 presents a great coupling in its dynamic behavior.

10 Coupled basal shear / direct basal shear.

The results confirm what is found in indicator 9, that is, building 1 presents a major coupling in its dynamic behavior exceeding the recommended limits.

11 Coupled basal moment / direct basal moment.

The results show that building 1 exceeds the recommended values which is attributed to the relatively large mass and, therefore, lateral forces that cause a major moment.

12 Number of structural elements in the seismic resistance

Both buildings have suitable resistant flats.

13 Effective Spectral Reduction Factor.

In both cases, the dynamic seismic forces were lower than the static forces and had to be increased to meet the minimum basal shear.

6. Conclusions

It can be concluded, from the Bio-seismic Profile, that both structures possess appropriate characteristics of stiffness and structural redundancy to satisfactorily control lateral displacements and show a satisfactory behavior of seismic events.

For building 1, there is certain level of translational-torsion coupling, however, it would be expected that it will not cause greater impact since it is a torsionally rigid structure. Although building 2 is torsionally flexible structure, the symmetry it has favors a good seismic behavior.

From the detailed structural evaluations made to the buildings, building 2 could not sustain the level of seismic force requested by the current regulations; however, the results of the inspection reported that its structure is healthy. Building 1 satisfactorily meets with the current regulation and its structure presents no damage at all.

It is concluded that the two buildings were designed and built before the onset of the current regulations; their structural conditions are satisfactory indicating that they have experienced a proper seismic behavior before intense earthquake as the one in Mexico City in 1985, which is consistent with the result of the application of the methodology proposed by the Bio-seismic Profile.

Based on the results, it would be desirable to extend the application of this methodology to evaluate other type of buildings that have been damaged by intense earthquakes.

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