

Influence of the concrete structural configuration in the seismic response

Estudo da influência da configuração estrutural na resposta sísmica de uma estrutura de concreto armado

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Abstract

The recent increase in seismic records in Brazil boosted the publication of NBR 15421 (ABNT, 2006) - Design of Earthquake-Resistant Structures- which requires the seismic loads to be considered in the design due to the dissipation of energy in the inelastic regime structures and seismicity of the Brazilian territory. Many buildings built before the publication of this code were not designed to withstand seismic activity, increasing its fragility and vulnerability to earthquakes of any intensity. Some of these buildings were designed and built with irregular distributions of mass, stiffness or resistance over its height, which can lead to insufficient seismic performance. Therefore, it is essential for engineers to understand the seismic response of structures with irregular distributions of mass, stiffness or resistance along its height, which are referred to in this paper as structures with irregularities. In this sense, the aim of this work is to evaluate the seismic behavior of a reinforced concrete structure with horizontal structural irregularities by observing their influence in the loss of the ability to resist lateral loads and energy dissipation. For this purpose, were used the Equivalent Lateral Force (ELF) Method, Modal Response Spectrum (RSA) Method and Pushover Analysis, described in NBR15421: 2006 [1] standard and the American Standard FEMA 356 [2]. Similarly, the seismic behavior of models using the capacity spectrum method defined in the ATC-40 [3] was calculated. All results were compared with those obtained from a regular structure classified as the reference model or base case, in order to view the various changes caused by irregularities. The proposed study was carried out numerically and the simulations were performed using CSI ETABS Nonlinear © v9.5 program. The results show that the ability of the structure to withstand collapse depends on the plasticity of its elements and of the adequate details to permit response without critical loss of strength of the structural system.

Keywords: seismic analysis, seismic resistant structures, irregular structures.

Resumo

Os recentes aumentos dos registros de abalos sísmicos no Brasil impulsionaram a publicação da NBR 15421:2006 [1] – Projeto de Estruturas Resistentes a Sismos – na qual são definidas as cargas sísmicas a serem consideradas no projeto em função da capacidade de dissipação de energia no regime inelástico das estruturas e da sismicidade do território brasileiro. Algumas edificações construídas antes da publicação dessa norma não foram projetadas para resistir a ações sísmicas aumentando a sua fragilidade e vulnerabilidade para terremotos de qualquer intensidade. Observa-se ainda que alguns desses edifícios foram projetados e construídos com distribuições irregulares de massa, rigidez ou resistência ao longo da sua altura que podem levar a um desempenho sísmico inadequado. Assim, torna-se essencial para os engenheiros e projetistas uma melhor compreensão dos efeitos dos abalos sísmicos no comportamento sísmico de edificações, especialmente aquelas com diversas irregularidades. Por simplicidade, utiliza-se neste trabalho o termo “estruturas irregulares” para referenciar estruturas como as descritas. Nesse sentido, o objetivo deste trabalho é avaliar o comportamento sísmico de uma estrutura de concreto armado de oito pavimentos e com diversas irregularidades introduzidas na planta da mesma, a fim de verificar a influência dessas irregularidades na perda da capacidade de resistir cargas laterais e dissipação de energia, as quais são calculadas utilizando o método dinâmico espectral e uma análise pushover, procedimentos descritos nas normas NBR15421:2006 [1] e na norma americana FEMA 356 [2]. Do mesmo modo, calcula-se o desempenho sísmico dos modelos usando o método do espectro de capacidade definido na norma ATC-40 [3]. Todos os resultados são comparados com os obtidos para uma estrutura totalmente regular chamada de caso base ou de referência, com o intuito de visualizar as diversas mudanças provocadas pelas irregularidades. O estudo proposto é realizado de forma numérica e as simulações são feitas no programa CSI ETABS® Nonlinear v9.5 [4]. Os resultados mostram que a capacidade da estrutura dependerá da plasticidade de seus elementos para não colapsar e do detalhamento adequado para garantir o bom funcionamento do sistema resistente.

Palavras-chave: análise sísmica, estruturas resistentes a sismos, estruturas irregulares.

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1. Introduction

The development of cities and their growing population has leads to a shortage of space and the spread mid-rise buildings. This type of construction can be very vulnerable to one of the most destructive natural phenomena: earthquakes. According to Santos et al. [5] in the smaller buildings of 20 stories, the influence of seismic activity may be as important as the effects of wind.

Brazil is largely in the central region of the South American Plate, which is a stable area. However, part of the country near the edges of the plate, where there have been considerable seismic activity and there is still the possibility of intraplate earthquakes. There are about 40 seismically active faults, including faults in the central region of the South American plate, from north to south. Failure BR-47, for example, cuts the state of Minas Gerais and has already recorded a 4.7-magnitude earthquake in the city of Itacarambi in 2007. Therefore, these conditions have favored the development of a Brazilian Code, NBR 15421:2006 - Design of seismic resistant structures - Procedure [1] - to select locations where structures subject to seismic action require specific actions and controls.

Since 2006, research aiming to evaluate the effects of earthquakes on structures has been increasing in Brazil, but there are few studies compared to countries with high seismic activity and high economic growth. In general, these studies make comparisons between different seismic design standards in South America, as well as the main parameters that influence the natural period of the structure, the importance of seismic acceleration and the type of soil at the site where the building is constructed. The work of Santos et al. [6]; Miranda [7] and Parisenti [8] outlined some of the highlights in this area.

Santos et al. [6] published a comparative study of South American seismic design standards with special attention to the following design parameters of the building: return period for the establishment

of the application of seismic forces, seismic zoning, response spectra design, liquefaction effects, among others. Miranda [7] aimed to estimate the seismic vulnerability in some constructions in a qualitative manner using the Hiroswa method and NBR15421: 2006 [1]. Similarly, Parisenti [8] carried out a study of dynamic analysis of reinforced concrete buildings subjected to earthquakes, aimed at assisting engineers in the implementing the NBR15421: 2006 [1] in structural projects.

Many buildings constructed in the country have asymmetrical floor plans and asymmetrical front elevation. In general, these changes in earthquake-resistant standards are called irregularities and influence the ability of the structure to dissipate the energy generated by earthquakes. In the most of the work done in the country is observed that the study of the influence of these changes on the seismic behavior is rare.

In the international context, several researchers have provided useful information on the study of structural irregularities located in the front elevation, such as Chintanapakdee and Chopra [9] Galíndez [10] Michalis et al. [11] Peña [12] Bobadilla and Chopra [13] Jianmeng et al. [14] Le Trung, et al. [15] and Inan et al. [16].

These researchers worked with a symmetrical (regular) structure, introducing along its height different distributions of mass, stiffness and strength, called vertical irregularities. These irregularities were examined one by one through linear dynamic analysis, nonlinear dynamic analysis and nonlinear static analysis or pushover analysis, in order to know the changes in the structural behavior under seismic actions and the possible failure mechanisms that may arise.

However, researches on the effects of earthquakes on structures that present uneven distribution of mass, stiffness or resistance located on the floor plans of the buildings seems to be insufficient. This work aims to study the seismic behavior of a reinforced concrete structure with eight floors and various horizontal structural irregularities located in floor plan, in order to evaluate the influence

Figure 1 - Floor plant a frontal view of the reference model

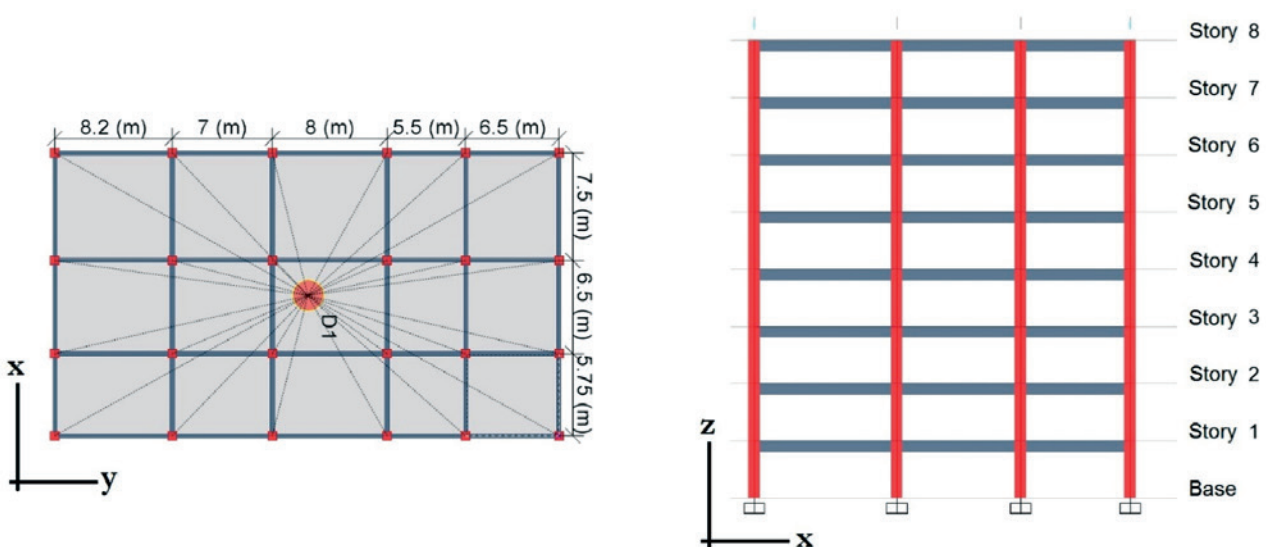
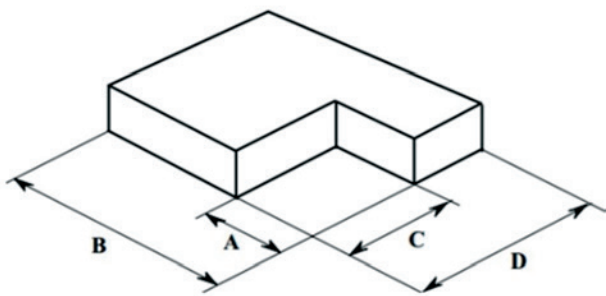
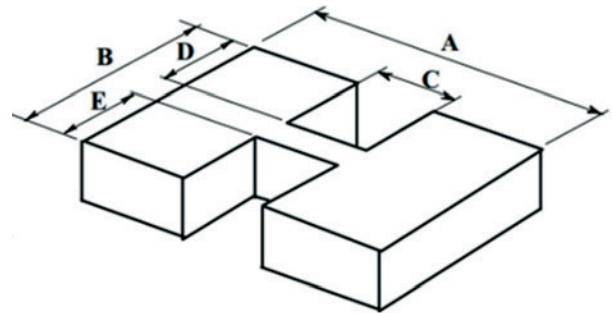


Figure 2 - Horizontal structural irregularities - (a) re-entrant corner irregularity; (b) diaphragm discontinuity irregularity



$$A > 0,15B \text{ y } C > 0,15D$$

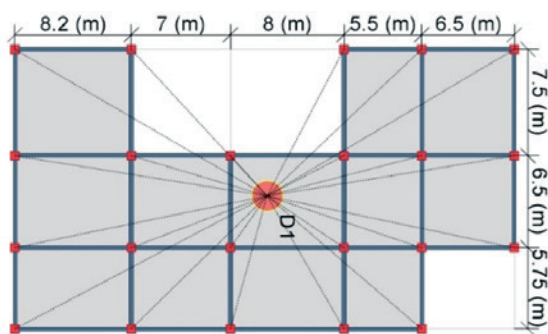
A



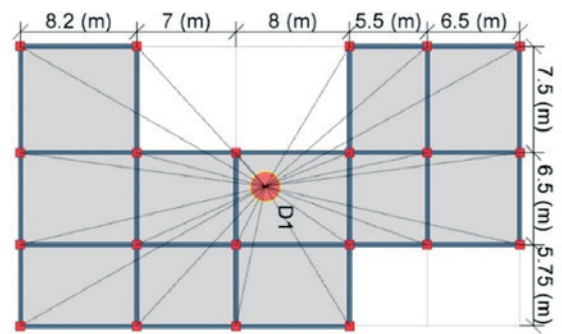
$$(Cx D) + (Cx E) > 0,5Ax B$$

B

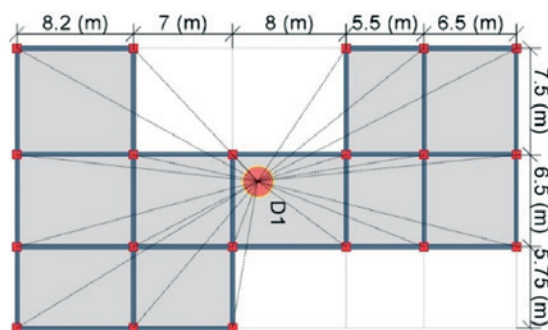
Figure 3 - Models with irregularities



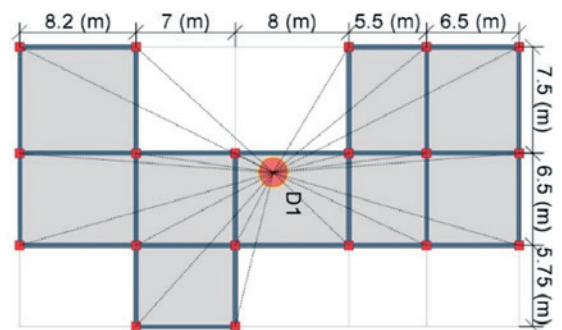
MODEL 1



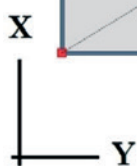
MODEL 2



MODEL 3



MODEL 4



of such irregularities in the loss of the ability to resist lateral loads and the ductility. For this purpose the Equivalent Lateral Force Analysis, Modal Response Spectrum Analysis and Pushover Analysis, described in NBR15421: 2006 [1] standard and the American Standard FEMA 356 [2], are used. In addition, the seismic performance of the models using the capacity spectrum method defined in the American Standard ATC-40 [3] was calculated.

2. Methodology

A symmetrical building as shown in Figure [1] was initially considered. This building has a rectangular plan of 19.75 m x 35.2 m and consists of special reinforced concrete frames, columns of 60 x 60 cm and beams of 28 x 60 cm. The height between floors is 3 meters and the total height is 24 m, hypothetically located in the city of Brasilia. Elevators, escalators, electrical construction, sanitary hydroelectric systems, air conditioning, among others, were not considered in the structural model except the lid of the water tank. The properties of the materials used in this work are provided by NBR 6118: 2014 [17]:

- Compressive concrete characteristic strength (f_{ck}) = 21 MPa;
- Modulus of elasticity of concrete (E_{c1}) = 210.00 MPa;
- Unit weight of concrete (γ) = 25 kN/m³;
- Characteristic strength of steel or yield strength (f_y) = 410 Mpa;
- Modulus of elasticity of steel (E_s) = 200000 MPa.

To assess the response of the model shown in Figure [1], each floor slabs were modeled as rigid diaphragms considering NBR15421:2006 [1], indicating that the rigid diaphragm is considered if the width to depth ratio of the diaphragm is less than three. These diaphragms are horizontal three-dimensional structures, where one of their main functions is to distribute the horizontal forces to the vertical resisting elements in direct proportion to the relative rigidity. The diaphragm is not deformed and likewise causes each vertical member to deflect the same amount.

In the structure of Figure [1] the center of the rigid diaphragm "D1" coincides with the center of mass and the center of rigidity of the floor plan. Therefore, this structure can be considered symmetrical in the horizontal, which is the reference model in numerical analysis. In this reference model were introduced two types of horizontal irregularities defined in NBR 15421:2006 [1] as "re-entrant corner irregularity" and "diaphragm discontinuity irregularity", illustrated in Figure [2]. In Figure [2a], the letters "B" and "D" correspond to the length and width of the structure, "A" and "C" characterize the irregularity. In the case of Figure [2b], "A" corresponds to the length of the structure, "B" indicated the width of the structure while "E", "D" and "C" characterize the irregularity.

For modeling and analysis of these irregularities, four models were defined and are shown in Figure [3], in which the irregularities indicated in Figure [2] were located in many parts of the structure repeating on every floor of the same.

Numerical analysis was conducted in two stages. The first stage, for each model a static and dynamic analysis was performed to calculate the absolute displacements and story drift in order to estimate the lateral stiffness of each model. The methods used in this stage are elucidated in the NBR 15421:2006 [1]: Equivalent Lateral Force Analysis (ELF) and Modal Response Spectrum Analysis (RSA). In the second stage, employed a nonlinear static pushover analysis to estimate the loss of lateral stiffness and dissipation of energy due to the introduction of irregularities in the reference model. FEMA 356 [2] explains this method. Finally, the seismic performance of the models was calculated using the Capacity Spectrum Method defined in in ATC-40 [3] standard.

2.1 Computational tool

In this study, all analyses were performed with ETABS (Extended Three Dimensional Analysis of Building Systems) program. ETABS [4] is a computational program based on the Finite Element

Figure 4 - (a) Frame object; (b) Shell object

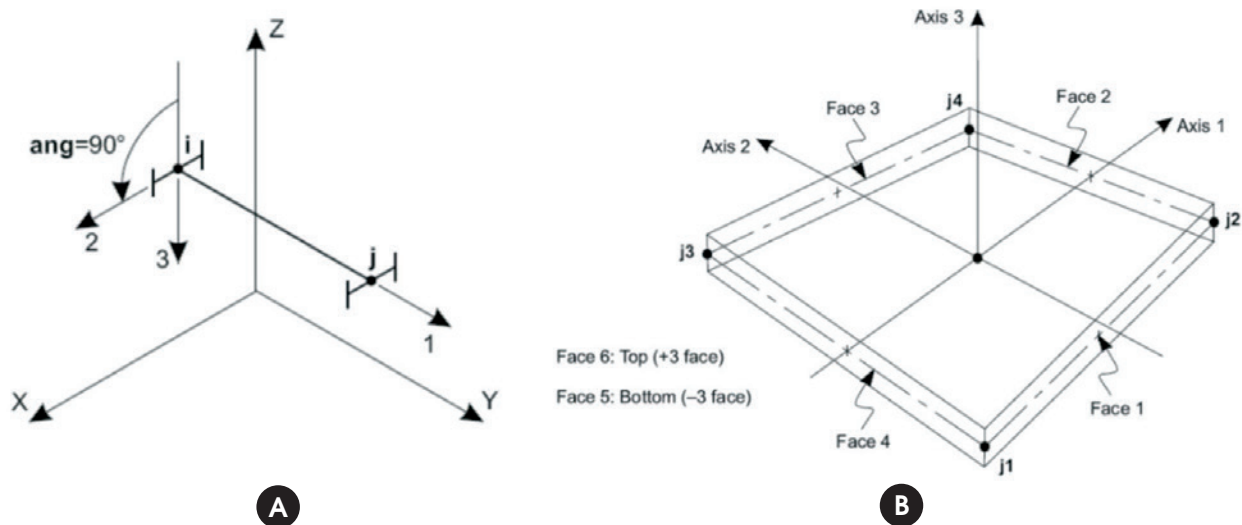
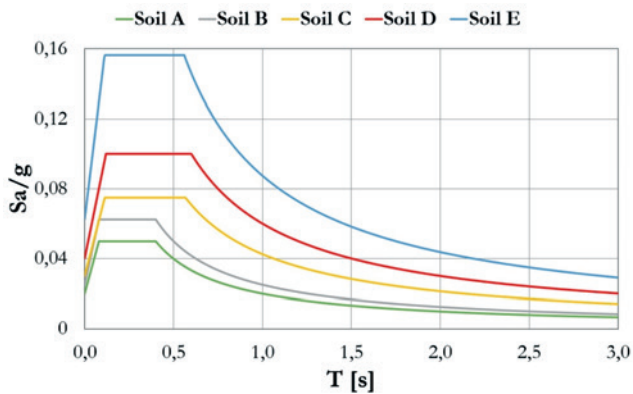


Figure 5 - Brasília response spectra for $a_g < 0,10g$ - NBR 15421:2006 (1)



Method for the analysis and structural design of buildings. In this program, beams and columns are modeled with *Frame* element. This element has two nodes denoted by *I* and *J* with six degrees of freedom per node: three translations and three rotations on the axes 1, 2 and 3. The *Frame* element used is shown in Figure [4a]. To represent the concrete slabs was employed a *Shell* element, illustrated in Figure [4b]. This element has four nodes (*j1, j2, j3, j4*) with six degrees of freedom, three translations called *U1, U2* and *U3* and three rotations denoted by *R1, R2, R3*.

3. Numerical simulations

As mentioned, the numerical analysis was performed in two stages. On the first stage, the Equivalent Lateral Force (ELF) Method and Response Spectrum (RSA) Method (given by NBR 15421:2006 [1]) to calculate the seismic response and story drift were used. These values were calculated on the center of mass of floor plan. On the second stage, the pushover technique (given by FEMA 356 [2]) was used to estimate the capacity of the structure to resist lateral loads as soon as new irregularities are introduced in the models. Torsion occurs under the action of seismic forces when the cen-

Table 1 - Natural frequencies and associated modes of vibration to each model

Reference model				Model 1			
# modes of vibration	T (s)	Γ_x (%)	Γ_y (%)	# modes of vibration	T (s)	Γ_x (%)	Γ_y (%)
1	0,983	77,16	0,78	1	0,935	78,94	0,0011
2	0,958	78,26	79,95	2	0,929	78,96	80,1
3	0,858	80,72	80,97	3	0,855	80,62	80,71
4	0,308	90,47	81,1	4	0,293	90,67	80,71
5	0,301	90,65	90,94	5	0,291	90,67	90,91

Model 2				Model 3			
# modes of vibration	T (s)	Γ_x (%)	Γ_y (%)	# modes of vibration	T (s)	Γ_x (%)	Γ_y (%)
1	0,939	72,76	7,48	1	0,934	79,98	0,3
2	0,934	80,29	80,63	2	0,922	80,28	80,63
3	0,868	80,55	80,67	3	0,87	80,47	80,63
4	0,294	89,72	81,75	4	0,292	90,8	80,66
5	0,292	90,81	90,87	5	0,288	90,83	90,88

Model 4			
# modes of vibration	T (s)	Γ_x (%)	Γ_y (%)
1	0,927	80,32	0,0004
2	0,907	80,33	78,54
3	0,87	80,38	80,58
4	0,289	90,77	80,58
5	0,284	90,77	90,7

ter of mass of a building does not match the center of stiffness. In order to prevent elevated values of torsion on the floor plan of the models of Figures [1] and [3], was verified that the distance between the center of stiffness and the center of mass parallel to each major axis was lower than 5 percent of the greatest width of the floor plan parallel to that axis.

3.1 First stage

It is considered that the building of Figure [1] is used for residence and is located in Brasilia City. The characteristic soil of Brasilia is a porous clay with SPT larger than 40 ($N = 40$) (Alves [18]). From these information and following the NBR 15421:2006 [1], assumed that:

- Seismic zone zero;
- Soil Type D (hard ground) - SPT with $N = 40$;
- The response modification factor of one ($R = 1$) for all models, therefore, the seismic base shear forces obtained are elastic;
- Importance Factor of one ($I = 1$).

According to NBR 15421:2006 [1], the response spectrum for horizontal accelerations corresponds to the maximum elastic response of a system of one degree of freedom with a fraction of critical damping equal to 5 percent. Thus, in the spectral analysis was considered a horizontal seismic acceleration $0.025g$ (zone 0 - GMT) and the spectrum shown in Figure [5], the curve corresponding to soil type D.

In the ETABS [4] program it is possible to make a spectral analysis introducing the response spectra and the damping ratio, but it is necessary to set the number of vibration modes. According to Chopra [19] must be considered three modes of vibration per floor, therefore, used in each model 24-vibration modes.

In the modal spectral analysis must consider all modes that have significant contributions to the to the structure's response. The

number of modes used in the analysis must be larger enough for the modes to represent at least 90 percent of the total mass of the structure. In this work, the first five vibrational modes were sufficient to capture more than 90 percent of the mass of the structure as illustrated in Table [1]. Following, the maximum responses of the structure were obtained from a combination of the response of each of the modes using the CQC (Complete Quadratic Combination) Method available in ETABS [4] program.

According to NBR 15421:2006 [1], the spectrum should be applied in each of the orthogonal directions of models, 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. As mentioned, CQC Method for bidirectional load combination was used.

The results obtained with the two methods described in NBR15421:2006 [1] shown in figures [6] and [7]. These figures illustrate the calculated orthogonal horizontal forces in X and Y directions for each model.

According to NBR 15421:2006 [1], the forces obtained from the Response Spectral Method should be larger than 85 percent of the calculated force to the ELF Method. It is noted in Figure [6] that model 1 has a force on the base in the orthogonal X direction of 4298.85 kN calculated using ELF Method. For the RSA method, the force in the base in the same orthogonal direction was 3814.38 kN, being larger than 85 percent of the force obtained with the ELF method. Therefore, the first model meets the code requirements. Analyzing the results of the other models was concluded that all other designs exhibit the same behavior. Consequently, it proceeds to calculate the drifts using the results from RSA method. The maximum displacements calculated on top of each model are listed in Table [2] and are compared with those obtained in the pushover analysis to assess the seismic demand of the models.

The maximum drift may not exceed 2 percent of the height of the story according to NBR 15421:2006 [1]. In all models, the height

Figure 6 – Base shear in orthogonal X direction

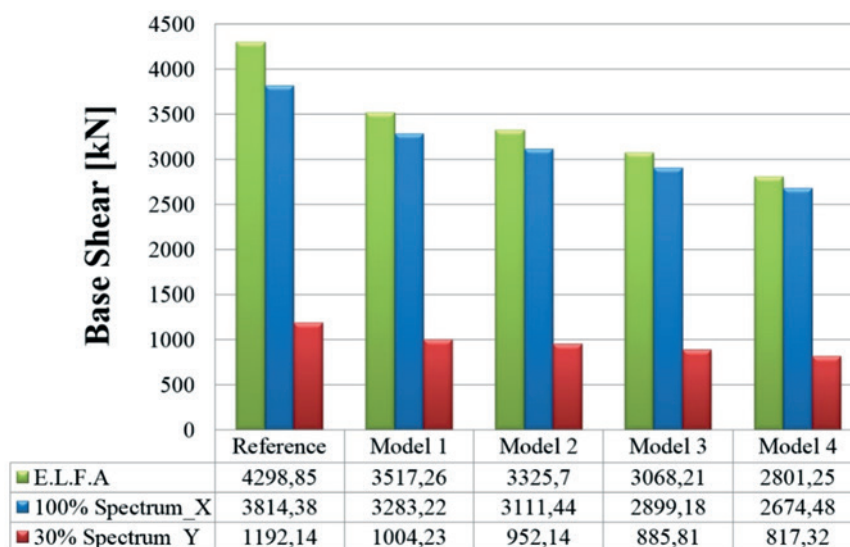
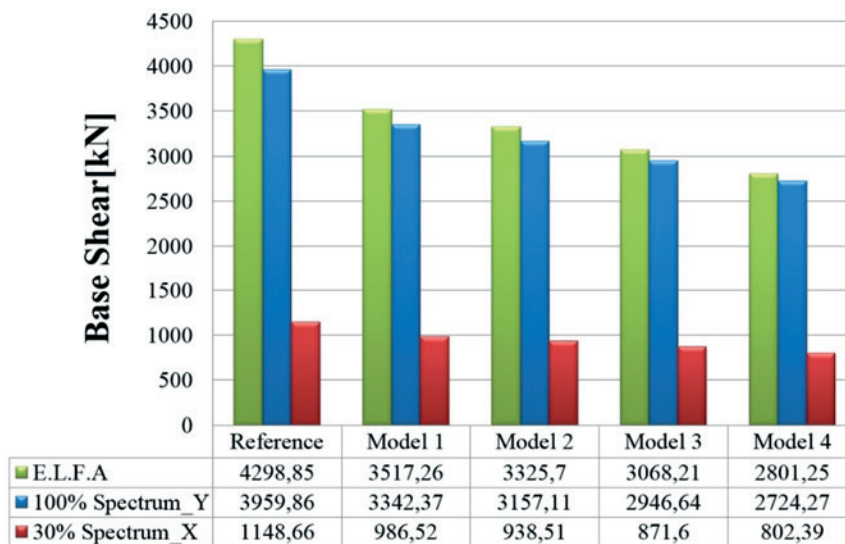


Figure 7 – Base shear in orthogonal Y direction



is 3.0 m between stories; therefore, the maximum drift may not exceed 0.06 m or 6 cm. Figure [8], the maximum drift in the five models was approximately 4.69 cm in the orthogonal direction X and 4.67 cm in the orthogonal Y direction. In both cases, drifts did not exceed the maximum value.

An initial estimative of lateral stiffness of the models could be obtained by analyzing the drifts of Figure [8]. Note that the maximum drifts in the orthogonal X direction were slightly higher in relation to Y direction, so it is understood that in the X direction the lateral stiffness is less by presenting high displacements. It can be seen in figures [1] and [3] that in X direction the structure has fewer connections relative to the Y direction to balance the applied load and to dissipate the energy generated by such forces. Could be say that the elements located in the X direction are less able to withstand lateral loads than the elements located in the other orthogonal direction. Then, a nonlinear static analysis is performed to corroborate these results.

3.2 Second stage

At this stage, a nonlinear static analysis or pushover analysis described in American Standard FEMA 356 [2] was performed. Pushover analysis is a series of incremental static analysis carried out to develop a capacity curve for the building. This technique is available in ETABS program [4].

A Pushover analysis produces a capacity curve that presents the relationship between the base shear and roof displacement (measured in one point named as *control node*). To obtain this curve, the building is subjected to an incrementally lateral load until it reaches the maximum displacement (called target displacement) at a control node. FEMA 356 [2] recommends a target displacement equal to 4 percent of the total building height. In this work, the center of the rigid diaphragm “D1” (Figure [3]) was used as a control node.

The capacity curve is generally constructed by the fundamental vibration mode according to FEMA 356 [2]. This is generally valid for buildings with fundamental periods of vibration up to about one second. It is observed in Table [1] that all models meet this requirement, consequently, this procedure was used.

Pushover analysis requires the development of the force-deformation curve in the critical section of beams and columns using FEMA 356 [2] (Figure[9]), the Mander’s [20] constitutive models for confined concrete (Figure [10a]) and tri-linear model for steel (Figure [10b], Falconi [21]).

In Figure [9], the linear response is depicted between point A (unloaded component) and an effective yield point B. The slope of the BC line represents the strain hardening. Point C indicates the nominal strength of the structural element. The CD line corresponds to an initial failure of the member. Finally, point E indicates final loss of resistance or structural collapse. At deformations greater than point E, the component strength is essentially zero. In ETABS program [4], the nonlinear properties of the

Table 2 – Maximum displacements measured at the top of the models obtained from response spectrum analysis in two orthogonal directions

Model	$\delta_{\text{DIRECTION-X}}$ (cm)	$\delta_{\text{DIRECTION-Y}}$ (cm)
Reference	3,41	3,34
1	2,63	2,62
2	3,14	3,11
3	3,16	3,08
4	3,12	2,98

structural elements are represented by means of plastic hinges and are located at the extremes of the beams and columns.

Therefore, using the method described in FEMA 356 [2] in the symmetric model and considering the criteria described above, the result is the global capacity curve illustrated in Figure [11]. This curve represents the global nonlinear behavior of the structure and the slope at any point indicates the lateral stiffness of the structure. Under incrementally increasing lateral load, the structure component yielded.

In Figure [11], the AB line represents the elastic behavior of the structure, where the point "B" denotes yield point. The slope of the BC curve represents the strain hardening. Point C indicates the nominal strength of the structural element and the final capacity of the structure according FEMA 356 [2]. Therefore, in this paper is only used the AC segment of the capacity curve. Point C represents the final deformation and the whole loss of rigidity of the structure. Beyond point D, the element responds with substantially reduced strength to point E. The seismic design standards

Figure 8 – Story drifts in X directions and Y directions

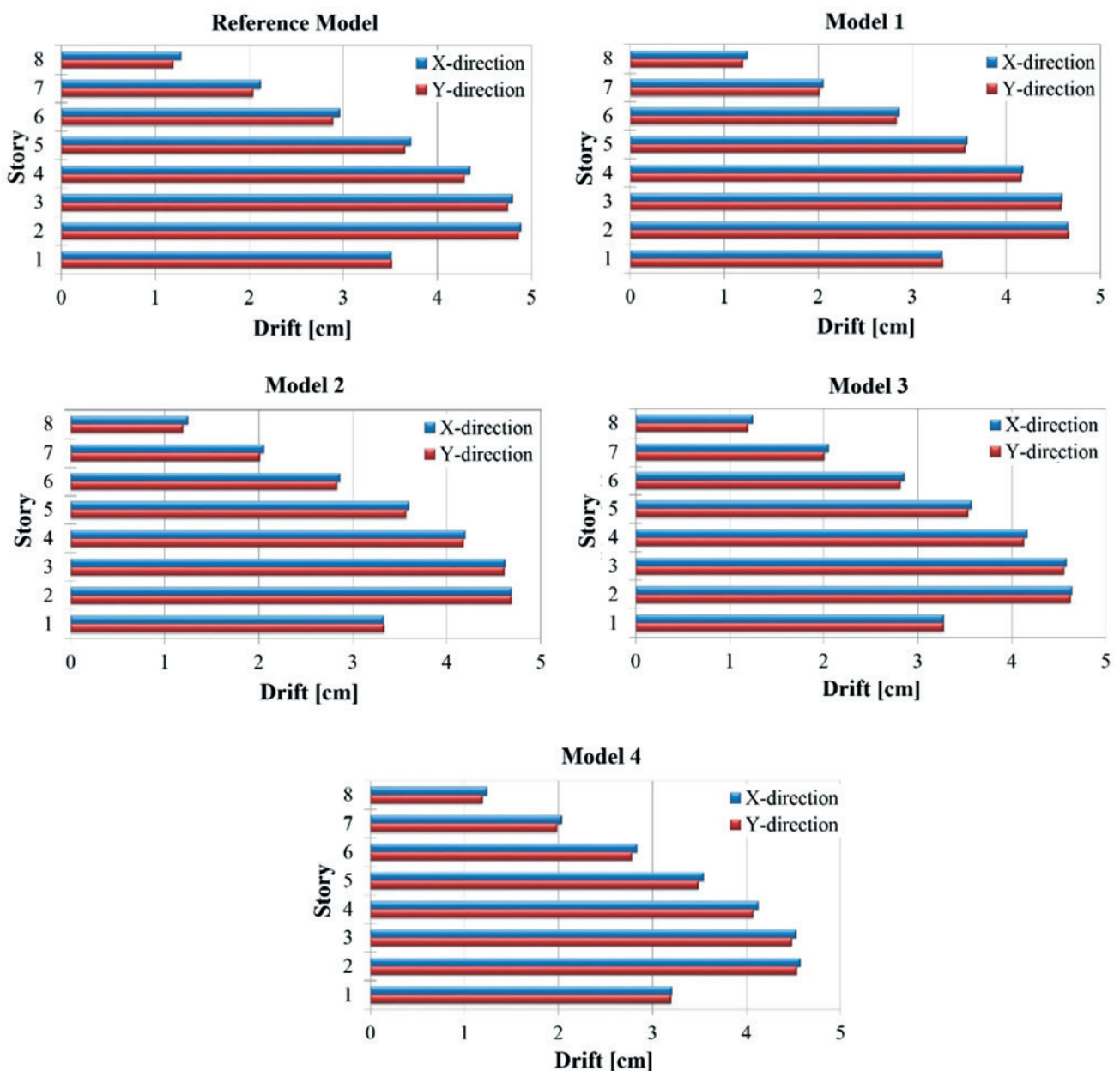
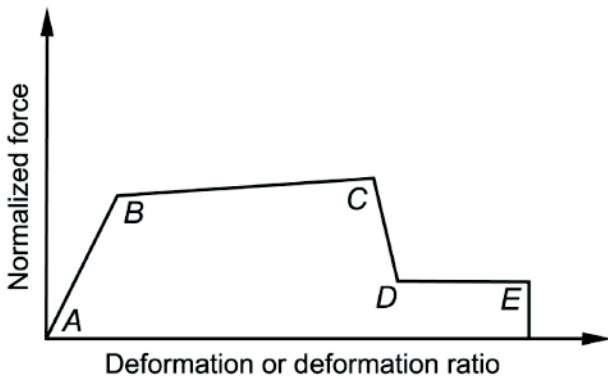


Figure 9 - Generalized Component Force-Deformation - FEMA 356 (2)



recommend that the first plastic hinge should be formed in beams, not in columns.

In order to facilitate the calculation of the available ductility, it is convenient to work with a bilinear model of the global capacity curve such as shown in Figure [12].

For this purpose, has been used an iterative procedure graph which balances the upper and lower sections of the curve as shown in Figure [12]. FEMA 356 [2] and FEMA 440 [22] define this method. In the Figure [12], V_y corresponds to yield strength of the building in the direction under consideration, δ_y is the displacement produced by V_y and called yield displacement, V_u is the base shear that causes the collapse of the first structural component and δ_u the rupture or collapse displacement produced by V_u . The bilinear models for the two orthogonal directions of all the analyzed cases are shown in Figure [13]. The comparison of the bilinear models obtained for structures with irregularities in relation to those obtained for the reference model in two orthogonal directions is shown in Figure [14]. In Table [3] can be observed the displacements δ_y and δ_u .

Figure 10 - Constitutive stress-strain models: (a) for concrete proposed by Mander (20); (b) tri-linear steel (21)

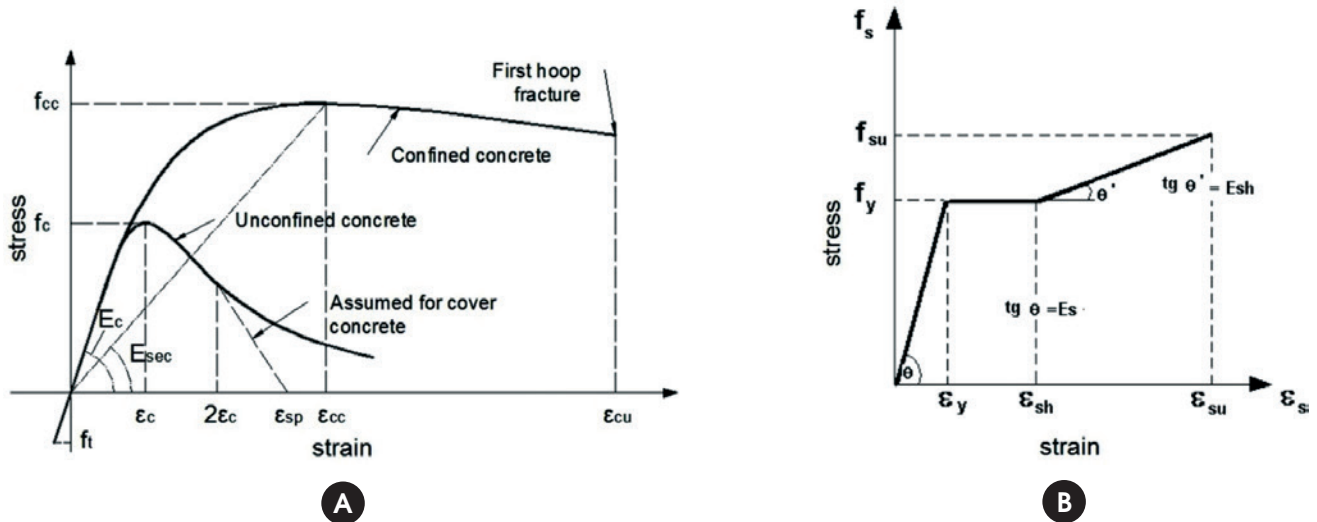


Figure 11 - Capacity curve of the reference model in orthogonal X direction

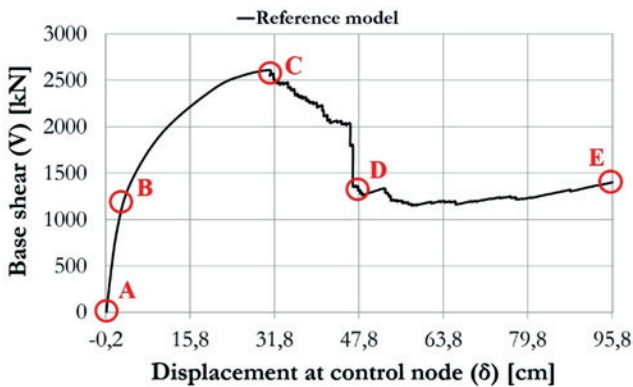


Figure 12 - Idealized capacity curves - FEMA 440 (22)

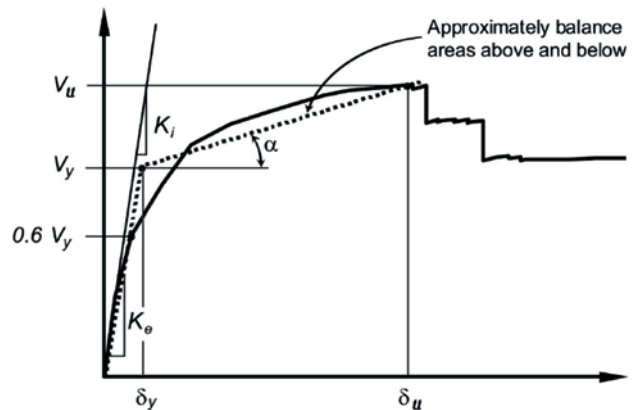


Tabela 3 – Results of bilinear models – yield and collapse displacements; ductilities

Model	X- Direction			Y- Direction		
	δ_y (cm)	δ_u (cm)	μ_x	δ_y (cm)	δ_u (cm)	μ_y
Reference	3,98	24,38	6,13	4,86	25,69	5,29
1	3,51	21,96	6,26	4,53	24,49	5,41
2	3,50	22,45	6,42	4,61	24,36	5,29
3	3,60	22,10	6,13	4,43	23,65	5,34
4	3,66	22,27	6,08	3,88	21,85	5,63

Figure 13 – Bilinear model of the capacity curve to each model

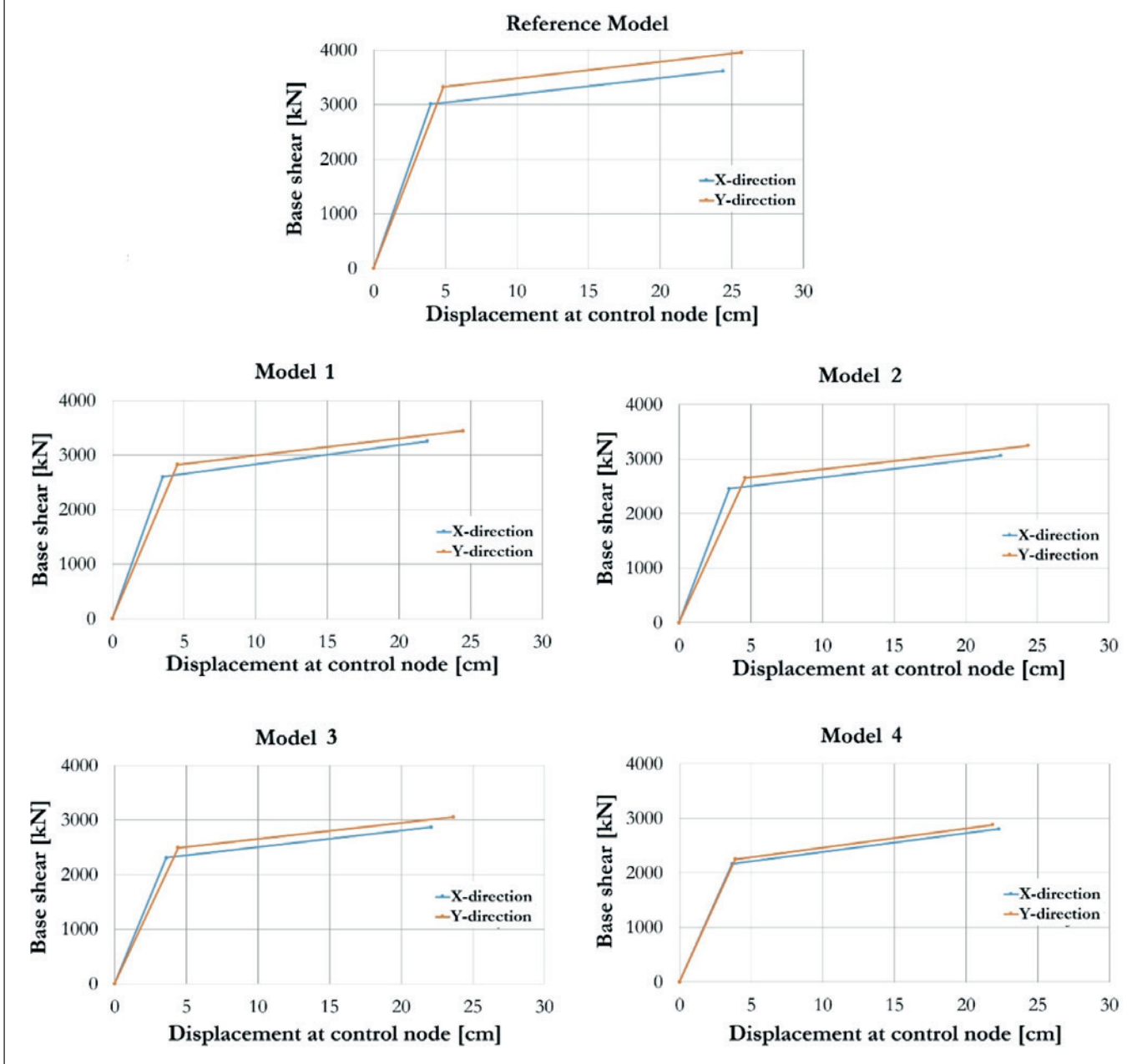
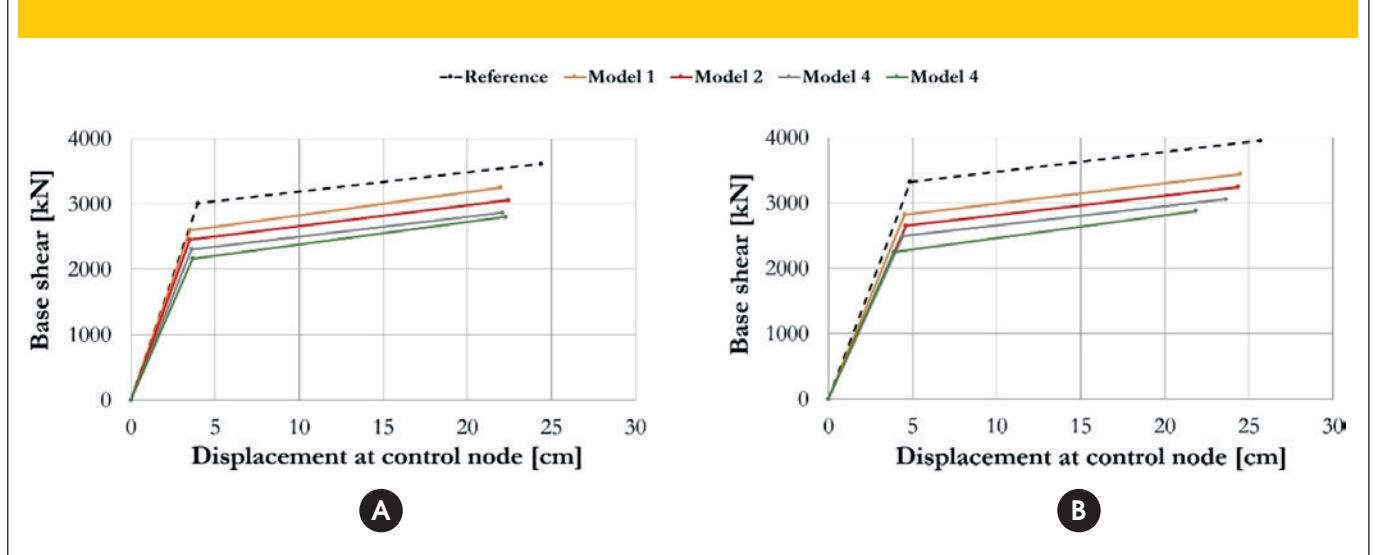


Figure 14 – Comparison of bilinear models of the cases analyzed – (a) X-direction; (b) Y-direction



In the previous figures, it is observed that in all models the forces that generate yield (V_y) and collapse (V_u) decrease as horizontal irregularities are introduced into the reference model. Furthermore, the frames located in orthogonal Y direction require a greater force to generate yield or collapse compared to frames located in the orthogonal X direction. Figure [14] shows that the lateral stiffness of the model 4 varies significantly compared to the reference model in the two orthogonal directions with a reduction in yield strength and the collapse of up to 28 percent in the orthogonal X direction and 38 percent in the orthogonal Y direction.

Using bilinear models is possible to calculate the available ductility of each model analyzed. Ductility was calculated as the ratio between the collapse displacement ($\delta_{u,y}$) and the yield displacement (δ_y). Table [3] presents the ductilities calculated for each model.

In the Table [3], it is observed that in all cases analyzed ductility in the orthogonal X direction was higher compared to the orthogonal Y direction, exceeding the value of the ductility of the reference model and resulting in more flexible structures. The table also shows that the model 4 presented a ductility value in the X direction smaller than the other models. This means that there was a force concentration in some points of this model, which has increased the stresses and strains values causing the collapse with forces with low intensity compared to others models.

A sense of the seismic performance in the models can be obtained by comparing the maximum spectral displacements (Table [2]) with the displacements obtained from nonlinear analysis (Table [3]). In all cases, the maximum spectral displacement was smaller than the yield displacement, which indicates that all analyzed models had a linear elastic behavior and the seismic design was well conceived.

Building performance level can be determined using the Capacity Spectrum Method defined in ATC-40 [3]. The capacity spectrum method allows for a graphical comparison between the structure capacity and the seismic demand. This method aims to estimate the maximum displacement required, called performance point.

The performance point is obtained by superimposing the response spectrum curve (represents the seismic demand) on the global capacity curve (lateral resisting capacity) into spectral coordinate or ADRS (Acceleration Displacement Response Spectrum) format. The capacity curve points (V, δ) are transformed into spectral coordinates (SD, SA) so that the capacity curve and the demand curve are in the same format. The results are shown in Figure [15].

If the elastic response spectrum intersects the linear range of the capacity spectrum, this point is called the Performance Point and represents the maximum displacement of the structure submitted to the proposed action. In Figure [14] the maximum displacement of each model is less than respective yield displacement (Table [3]), therefore the analyzed models had a linear elastic behavior. As expected, as the model becomes irregular, the performance point is near to the yield point. Consequently, the recommendation is to work wherever possible with symmetric structures, as they tend to distribute efforts in a uniform way, reducing the likelihood of damage to structural elements, as well as reducing the effects of rotation (torsion).

4. Conclusions

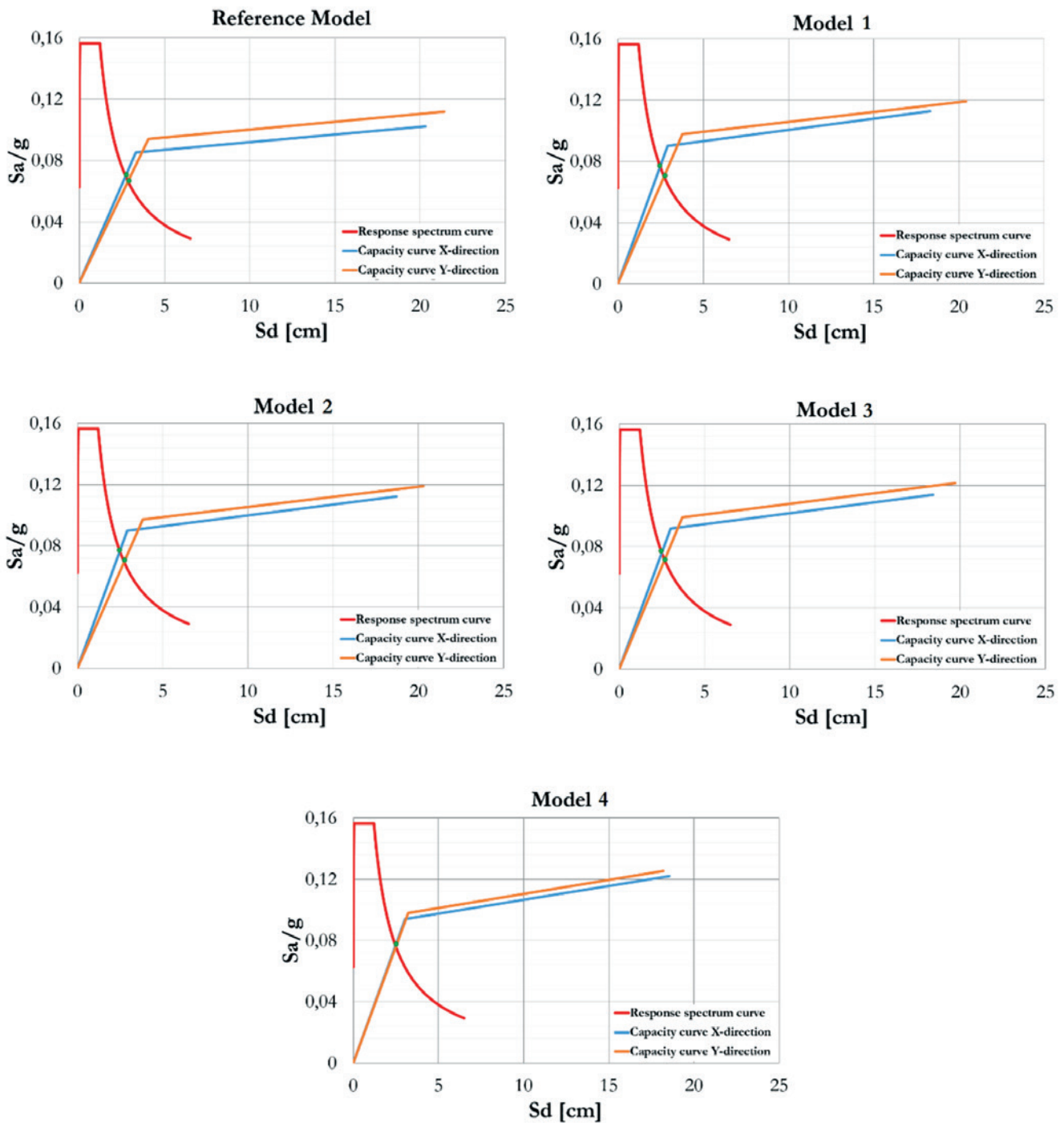
The design of a building structure involves the establishment of an appropriate combination of the various existing structural elements and on the definition of the actions to meet safety requirements, durability, aesthetics, and functionality, among other parameters that buildings must be submitted. A great design defines the structural quality of a property that is crucial for earthquake resistance. Often small or moderate earthquakes cause severe losses if they occur in an area whose buildings were not designed with the necessary calculation considerations.

From the analysis of seismic behavior of a symmetrical structure and others presenting asymmetry or irregularity has been found that the ability of the structure to resist lateral loads in models with irregularities is lower than in the symmetric or reference

model. It was found that an uneven structure could operate in the elastic regime, which is achieved by appropriate detailing, thus ensuring that the energy dissipation mechanism works properly. In this case, the capacity to structural collapse depends on the

plasticity of its elements. It is more important to pay attention to details that create a good structural redundancy, which means increasing the number of to balance the applied loads and dissipate energy generated by earthquakes.

Figure 15 - Acceleration Displacement Response Spectrum (ADRS) of each model - the performance point - ATC-40 (3)



If it is not possible to dissipate seismic energy through the plasticity of structural components, the use of external devices that could help in this role would be necessary.

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