Global stability of buildings' structural systems considering beam-wall connections

Estabilidade global de sistemas estruturais de edifícios considerando a ligação viga-pilar parede

C. W. S. DINIZ [a]
carlos.diniz@sinfra.ufal.br
https://orcid.org/0000-0002-0776-4789

E. N. LAGES [b]
enl@lccv.ufal.br
https://orcid.org/0000-0001-6704-4057

A. S. R. BARBOZA [b]
aline@lccv.ufal.br
https://orcid.org/0000-0001-6296-927X

Abstract

Several factors influence the stability of concrete structures, such as building height, structural components' rigidity, bracing system, beam-wall connection rigidity, among others. Global structural stability is evaluated according to NBR 6118/2014 by the α e γ_z parameters. This article aims to evaluate the global structural stability concrete structures considering the eccentricity of the beam-wall connections. With the aid of software Abaqus© and TQS©, reticulated models were analyzed to evaluate the contribution of the eccentricity of the beam-wall connections on the global stability and the rigidity of the connections. The geometric nonlinear effects were disregarded in these analyses. The stability parameters from NBR6118/2014 were evaluated in the frame models, as well as the flexibilization criteria from the TQS© software, LEPMOL e REDMOL. The results of the displacement analysis show an increase of 26.45%, comparing the centered connection to the more eccentric connection.

Keywords: global stability, beam-wall connection, TQS©, flexibility criteria.

Resumo

Diversos fatores influenciam a estabilidade das estruturas de concreto, como a altura das edificações, a rigidez dos elementos estruturais componentes, o sistema de contraventamento, a rigidez das ligações viga-pilar entre outros. A estabilidade global das estruturas é avaliada segundo a NBR 6118/2014 pelos parâmetros α e γ_z. Este trabalho objetiva avaliar a estabilidade global de estruturas de concreto considerando a influência da excentricidade na ligação viga-pilar parede. Com o auxílio dos softwares Abaqus© e TQS© foram analisados modelos reticulados, avaliando-se a contribuição da excentricidade da ligação viga-pilar parede para a estabilidade global e para rigidez da ligação. Os modelos foram analisados sem a consideração da não linearidade geométrica. Nos modelos de pórtico foram avaliados os parâmetros de estabilidade da NBR 6118/2014, assim como, os critérios de flexibilização da ligação do pórtico do TQS©, LEPMOL e REDMOL. Os resultados das análises de deslocamentos indicaram um crescimento de 26,45%, comparando-se a ligação centrada com a mais excêntrica.

Palavras-chave: estabilidade global, ligação viga-pilar parede, TQS©, flexibilidade.

[a]   Federal University of Alagoas, Superintendence of Infrastructure, Maceió. AL, Brazil;
[b]   Federal University of Alagoas, Laboratory of Scientific Computing and Visualization, Center of Technology, Graduate Program in Engineering, Maceió, AL, Brazil.

1. Introduction

The Brazilian population has suffered, over the years, a process of intense urbanization. According data available from IBGE [1], Brazil, currently, has its population concentrated in urban centers. The great concentrated populations end up limiting the horizontal spaces for construction, which increases the need for optimization of vertical spaces for this objective. Because of this, the typical structures of residential and commercial buildings are being conceived more frequently of greater heights. According to Fonte [2], the growing valorization of urban areas causes the big cities to concentrate the greatest quantities of tall edifications.

Another important factor that makes the construction of taller buildings possible is the technological advance over the years. According to Zumaeta Moncayo [3], the advance in the technology of materials produced more resistant concretes, which are available in the market, resulting in constructions with narrower structural elements. NBR 6118 [4] included in its texts concretes with the resistance of 55 MPa up to 90MPa, which are the concretes of high resistance.

One of the great concerns with the structure of a tall building is due to the effect of the wind over these edifications, because, depending on the geometry and stiffness of the structural elements of the building, these effects become even more pronounced. The structure of a tall building must be conceived in a much more careful way because of the effects of the horizontal actions. NBR 6118 [4], in item 11.4.1.2, strictly recommends the consideration of resulting effects on the edification caused by the action of the wind. The consideration of these effects implies in complex analysis of the stability of the edification. It is the horizontal displacements referent to these efforts that cause the appearance of effects of second order in the structure.

The verification of global stability of an edification has become more recurrent. This happens due to the great number of tall and narrow edifications, or still, according to Bueno [5], due to the necessity of verification of simple but narrow structures. However, with the advance of computer technology, the analysis may be done quickly and with satisfactory precision.

The global stability of the edification is influenced by diverse factors, such as the external actions (forces due to the wind, the actual weight of the structure and overloads) its height and narrowness, the stiffness of its elements and the connections between them, etc.

The framed structure, constituted by the association/connection between columns and beams, is one of the main means responsible for the rigidity of the structure. The combination of frame-shaped beams and columns provides s a greater load-bearing capacity and rigidity to displacements. Its efficiency in combating the effects of instability is in the rigidity of the connection between the two elements that constitute the structure (beams and columns) and in the rigidity of the transversal section of the actual elements. The beam may be connected to the column in different points of its width. According to Araújo [6], the rigidity between the beam and the column is altered when the eccentricity of the beam in relation to the axis of the column is modified. When analyzing the growth of the width of the column Araújo [6] verified that as the dimensions of the transversal section of the column become close to a large-scale column, the parameters used to determine the rigidity suffer more intense variations.

The analysis of stability is done usually adopting simplified models of the structure. It is common to represent beams and columns with bars in the spatial framed structure and the slabs with a weave of bars (grid). However, there is the possibility of the structure to be braced by large-scale columns, which are a bi-dimensional element of the sheet and that usually are not adequately represented by a single bar.

The large-scale columns are introduced in the structural systems for analysis and dimensioning as linear elements. According to NBR 6118 [4], in the item 15.9.1, for the large-scale columns to be included as linear elements in the resistant set (structural system), it is necessary that its horizontal section has its form maintained by adequate clamping in the several floors and that the effects of local second order and localized be conveniently evaluated.

In reference to the eccentric connections between beams and pillars, NBR 6118 [4] considers rigid parts to enforce the condition that beams transmit its efforts to the axis of the column, however, it does not consider the possibility that the rigidity of the eccentric connection is different in each position. The eccentricity of the loads may introduce effects in only parts of the columns, increasing only the possibility of the occurrence of these effects when referring to large-scale columns.

In the study of Araújo [6], there are signs that the beam determined in the region of the beam-wall connection suffers alterations when the beam is eccentric in relation to the column. This fact implies in alterations in the rigidity of the whole structure, and, consequently, alterations in the ability of dislocation of the edification. The less
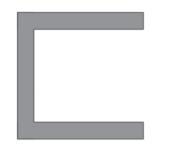

Figure 1
Exemplification of geometries of the transversal section of the large-scale columns
Source: Diniz [8]

rigid the connection, less rigid is the structure and greater the possibility of dislocation of the edification.

The study of the stability considering the eccentricity of the beam in relation to the axis of the column, especially in large-scale columns, may offer guidelines for the consideration of the flexibility of the beam-large scale column connection for each position of eccentricity of the beam.

In this study, the global stability of structural models of concrete is evaluated taking into consideration the influence of the eccentricity in the beam-large-scale column connection by verifying the normative parameters of stability and the absolute displacements of the reticulate structures. The global stability is verified using the absolute displacement on the top of the columns, in all of the models idealized modifying the eccentricity of the connections of the beam-large-scale column. Analysis of the normative parameters of stability of the models adopted and of the criteria of the flexibility of the reticulated model of TQS© will also be analyzed. The effects of the non- geometric linearity were not considered, once the connections were analyzed in structural models of relatively low heights compared to the buildings classified as of great height.

2. Considerations of large-scale columns

The columns are responsible for the reception of loads of the edifications and transferring them to the elements of the foundation. The columns are characterized by their longitudinal dimension being greater than the dimensions of their horizontal section. However, large-scale columns present a geometry that approximates its behavior to a bi-dimensional element.

According to Pascual [7], a large-scale column, or simply a wall, is a plane panel, continuous, without horizontal rigidity, predominantly solicited in a bending moment and supposedly extremely rigid to cutting force. NBR 6118 [4], in item 18.4.1, characterizes columns as the structural elements where the greater dimension of its hori-

zontal section does not exceed five times its smallest dimension. They may be composed by one or more associated surfaces (Figure 1) and in one of these surfaces, the smallest dimension of the transversal section should be smaller than 1/5 of the greatest one. The analysis of the large-scale columns is more complex than their most simple form, which is the idealized columns as linear elements of bars. However, NBR 6118 [4], in items 14.8.1 e 15.9.1, imposes conditions, which, when attended, allow that the large-scale columns be analyzed within the structural system as linear elements. Large-scale columns usually are used as a nucleus of rigidity in the edifications. According to Zumeta Moncavo [3], the use of nuclei formed by large-scale columns of great dimensions (usually in a "U" shape) contribute significantly to the increase of the rigidity of the edification.

3. Stability of edifications according to NBR 6118

The stability of the edifications is a recurring theme in scientific studies of engineering and also among the projectors of structures. The elevated number of tall and narrow buildings propels the need of verifications of global stability of the structure. For the project of structures, there are many softwares disseminated within the technical community for this objective, in which some verifications of stability of common buildings in reinforced concrete are noted, usually verifications expressed in NBR 6118 [4].

The verification of the stability of the concrete structures is a complex analysis due to the considerations of the non-linearities that exist in these structures: geometric non-linearity and physical non-linearity. The first is present when the balance of the structure is associated with a deformed configuration significantly different from the non-deformed one. The second is characterized by constitutive non-linear forces presented by the materials components of the structure.

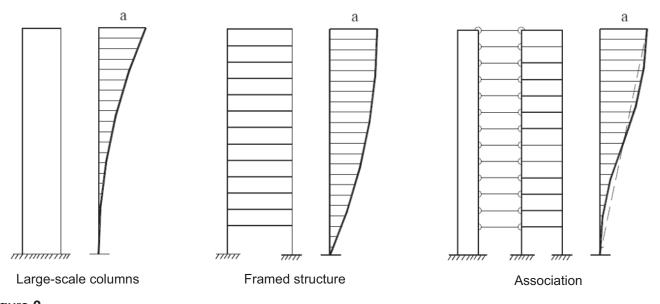By considering the real balance of the structure in the deformed

Large-scale columns   Framed structure   Association

Figure 2
Braced structures
Source: Lima [11]

geometric configuration, additional loads to the ones of geometrically linear analysis emerge in the system, denominated from effects of second order [9]. There are basically three types of effects of second order that may occur in the structures of concrete: global, local, and localized. The effects of the second order, where the non-linear behavior of materials should be considered, may always be discarded when they do not present an increase superior to 10% in the reactions and in the relevant loads in the structure (NBR 6118 [4]).

The analysis of the global stability of structures aims to classify them related to their lateral ability of displacement of knots, or in other words, tries to analyze their sensibility to the effects of second order [5]. Due to the ability of displacement of knots of the geometrical model of the structure, the actual structure may be classified as of fixed knots or mobile knots. For NBR 6118 [4], in item 5.4.2, the structures of fixed knots are those in which the global effects of second order are inferior to 10% of the effects of the first order, otherwise, they are considered mobile knots.

Global stability is a condition which is reached through the structures. The intensity of the horizontal displacements and of the deformations define if the structure is stable or not. According to NBR 6118 [4], in item 15.2, the limit-state of instability in the structures of concrete is always reached when, the growth of the intensity of the load grows and, therefore, of the deformations also; there are elements submitted to the bending compression where the increase of the resistant capacity becomes inferior to the increase of the loads.

The objective criteria given by NBR 6118 [4] used to classify the structure related to global stability are the parameters α (*alfa*) e γ_z (*gama Z*). These parameters classify the structure as of fixed or mobile knots, according to the values defined in the norm. When the structure is classified as of fixed knots, the dimensioning is realized considering the analysis of first order. In case of being classified as of mobile knots, an analysis of second order should be conducted, which is more complex, for the determination of the efforts to be added to the ones of the first order.
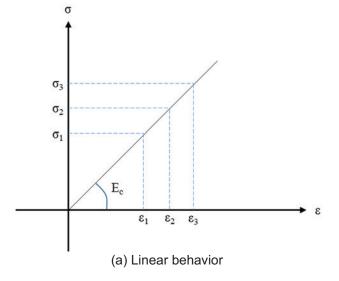
The vertical elements (columns and large-scale columns), or the association of these with the horizontal elements (beams), due to their elevated rigidity, are responsible for guaranteeing the structural stability, when adequately disposed of. According to Wordell [10], these sub-structures are defined as braced structures and the other elements that don't participate in this sub-structure are denominated unbraced. The braced structures may be large-scale columns, framed structures and the association of large-scale columns and framed structures, as illustrated in Figure 2.
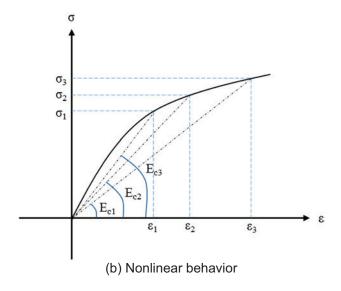
4.   Instability parameter (α)

The coefficient of instability α was developed by authors Hubert Beck and Gert Koning in 1967. CEB incorporated the coefficient in its text and posteriorly other regulating codes also started to incorporate it, such as Brazilian norm NBR 6118 [4]. This coefficient only classifies the structure as of fixe or mobile knots, evaluating if the efforts of second order need to or not be considered.

Item 15.5.2 of NBR 6118 [4] is about the parameter of instability α. Formulations and limit values of reference for the evaluation of the structure are presented. A symmetric reticulate structure may be considered of fixed knots if its parameter of instability α is smaller than the value α_1, as indicated in the norm.

The physical non-linearity of the constituent materials of the structure is considered in the determination of value α_1. The fissure of the elements is not considered; however, the physical non-linearity of the concrete also emerges through the action of loads of compression on the structural elements [3].

5.   Coefficient γ_z

This coefficient is one of the most used ones for analysis of the stability of the effects of second order. Besides classifying the structure related to the necessity or not to consider the effects of

(a) Linear behavior   (b) Nonlinear behavior

Figure 3
Diagram of the tensile deformation of the concrete
Source: Diniz [8]

the second the coefficient γ_z estimates such efforts. The γ_z is determined by a linear analysis of first order, for each case of loading, using the expression given by NBR 6118 [4], item 15.5.3.

The classification of the structure of fixed knots occurs when $\gamma_z \leq 1.1$. For values of $\gamma_z > 1.1$, the structure is classified as mobile knots, with the consideration of the effects of second order being mandatory.

The effects of second order may be estimated by applying the coefficient γ_z, as long as the limits of application are respected. There isn't a consensus between authors as to the definition of this limit. Franco and Vasconcelos (1991 apud [3]) e [12] established the limit of 1.2 for the validity of γ_z as an estimator of the efforts of second order. Carmo [13] points out that the limit may be advanced from 1.2 to 1.3. Pinto et al. [14] indicate an intermediate value of 1.25 as an acceptable limit. However, according to Vieira et al. [15], in studies with the coefficient γ_z and the buckling load factor, the employment of coefficient γ_z is inefficient when the most critical manner of instability is of torsion.

The quantification of efforts of second order in an approximate manner, through coefficient γ_z, consists in the evaluation of the final horizontal efforts of the combination of loading considered by 0.95 γ_z. This procedure is in NBR 6118 [4] in item 15.7.2 and is applicable when the limit of $\gamma_z \leq 1.3$ is established.

It is pointed out that the γ_z can only be applied to structures of buildings with four or more floors. In cases where the building has less than four floors, it is recommended that the global stability be evaluated by the parameter of instability α and the calculus of the global efforts of second order be evaluated by process P-Delta [3].

6. Non-linear physics

Non-linear physics is related to the mechanical properties of the materials. It consists of the loss of proportionality between the tension applied to the material and the resultant deformation [11]. Concrete is a typically non-linear material because factors such as the formation and opening of fissures that occur with the increase of the loads and its own constitution of materials of different behaviors imply in a non-linear tension formation curve. The module of rigidity to the longitudinal bending of concrete (E) does not maintain itself constant for the levels of tension applied, like the inertia of the section (I_c) is reduced by the fissuring of the material.

In Figure 3, the idealizations of the diagrams' tension versus deformation of the concrete are illustrated. In part (a) the graphic of the linear behavior of the concrete is observed. The physical non-linearity of concrete comes from the fact of that it is constituted of materials with varied physical properties, forming an anisotropic, heterogeneous material, with a module of elasticity that varies in each point of the curve of the diagram of the tension-deformation [16].

There is a necessity to determine with precision the horizontal displacements that occur in the structures of buildings of reinforced concrete because the efforts of second order are well evaluated when the deformed configuration of the structure is taken into consideration. It is known that the rigidity of the structural elements interferes in the displacements suffered by the structure, and, when seeking better answers related to the ability of displacement of the structure, the rigidity must be determined through processes that consider the physical non-linearity of the constituent materials of the structure [17].

In item 15.7.3 of NBR 6118 [4], an approximated process for the consideration of the non-linear physics in the global analysis of second order is described, which consists in the reduction of the rigidity of the structural elements.

The rigidities of beams and pillars may assume the value The rigidities of beams and pillars may assume the value $EI_{sec} = 0.7\, E_{ci}\, I_c$ equal for both, as long as the braced structure of the edification is composed exclusively by beams and columns, without the presence of rigid nuclei, usually composed by large-scale columns and the limit of $\gamma_z < 1.3$ [11] is respected.

7. Criteria of flexibilization of the spatial portico of the TQS©

The spatial framed structures generated in the TQS© software is a reticulate model, composed by bars of beams and columns, in which the connections between the bars of the beams and columns may or not be flexible. The criteria of the software allow establishing the rigidity of these connections, transforming them in settings, articulations or semi-rigid connections. The flexibilizations of the framed structures are imposed by the use of "springs" inserted between the bars of the beams and of the columns. The rigidities of these springs are calculated taking the coefficient of the rigidity of a bi-enclosed bar when mobilized by a rotation of unitary value in the setting. This way, the spring has a rigidity K equal to:

$$K = \frac{4EI}{L} \tag{1}$$

where E is the module of longitudinal elasticity of the material of the column, I is the moment of inertia of a section of the column which effectively will be considered in the rigidity of the connection and L is the length of the flight of the column.

Through the software, it is possible to calculate the rigidity of these "springs' through two criteria:

LEPMOL and REDMOL. The first is the fact that multiplies the width of the beam resulting in the width of the equivalent section of the column for the calculus of the moment of inertia I_y. The second is a factor that modifies the rigidity K of the spring. These two parameters allow the users to make considerations about the flexibilization of the connections. Figure 4 illustrates the process of use
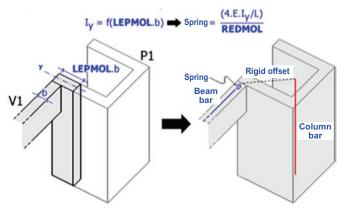

Figure 4
Illustration of the procedures of consideration of the criteria LEPMOL and REDMOL
Source: TQS© [18]

Table 1
Identification of the models used

Models	Description
MFS3DWSL1	Model of tri-dimensional framed structure without slab with 1 floor
MFS3DWSL2	Model of tri-dimensional framed structure without slab with 2 floors
MFS3DWSL3	Model of tri-dimensional framed structure without slab with 3 floors
MFS3DWSL4	Model of tri-dimensional framed structure without slab with 4 floors
MFS3DWSL5	Model of tri-dimensional framed structure without slab with 5 floors
MFS3DWSL6	Model of tri-dimensional framed structure without slab with 6 floors
MFS3DSL1	Model of tri-dimensional framed structure with slab with 1 floor
MFS3DSL2	Model of tri-dimensional framed structure with slab with 2 floors
MFS3DSL3	Model of tri-dimensional framed structure with slab with 3 floors
MFS3DSL4	Model of tri-dimensional framed structure with slab with 4 floors
MFS3DSL5	Model of tri-dimensional framed structure with slab with 5 floors
MFS3DSL6	Model of tri-dimensional framed structure with slab with 6 floors

Source: Diniz [8]

of LEPMOL and REDMOL. The moment of inertia I_y and the rigidity of the "spring" considered in the TQS© software are given by:

$$I_y = \frac{(\text{LEPMOL } b)b^3}{12} = \frac{\text{LEPMOL } b^4}{12} \tag{2}$$

$$K_{mola} = \frac{\frac{4EI_y}{L}}{\text{REDMOL}} \tag{3}$$

where b is the width of the beam involved in the connection for which the rigidity is being calculated.

8.   Characteristics of the models analyzed

In this study, 12 models of tridimensional framed structures were analyzed, which are divided in models of framed structures without the presence of slabs and framed structures with the presence of slabs, as presented in Table 1. The bases of the geometric models are found in Figure 5. In part (a) of the figure, there is the base structure for the models without slabs, as well as in part (b) for the models with slabs. The models (with or without slabs) with more than one floor are composed of vertical repetitions of the respective bases presented, according to the number of floors of the model.

The models adopted are constituted by slabs, beams and large-scale columns. In the computer system Abaqus©, the elements are connected with the help of the *Constraints/connections* tool for the assembly of the framed structures. The system of coordinates adopted is orthogonal and referenced as indicated in Figure 6. The axis *Y* represents the vertical direction and the axis *X and Z*

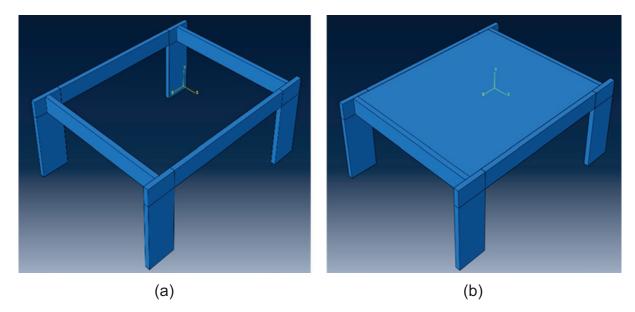(a)   (b)

Figure 5
Presentation of the models: (a) base structure for the composition of the models without slabs and (b) base structure for the composition of the models with slabs
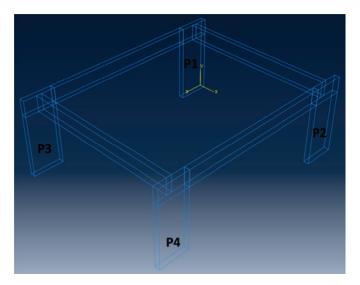**Source:** Diniz [8]

Figure 6
The system of coordinates adopted and
identification of the large-scale columns
Source: Diniz [8]

compose the plane in which the large-scale columns are set into. In this same figure, the nomenclature of the large-scale column is indicated, numbered from 1 to 4.

The geometrical characteristics of the structural elements used in the composition of the models are defined similarly to the typical structures of concrete. The beams are defined with a transversal section of 0.20 m x 0.60 m and length of 6.00 m. The large-scale columns are defined with a transversal section of 0.20 m x 1.20 m and length of the flight of 3.00 m. The dimensions of the slabs in the plane are of 6.00 m x 7.00 m and a thickness of 0.15 m.

The models are discretized in weaves of finite elements for the simulations in the no Abaqus©. The beams are discretized with weaves generated by subdivisions of 0.05 m and the slabs with subdivisions of 0.10 m. The level of refinement of the weave is defined considering the test of conversion associated to the use of the computer capacity of the machine of processing and contribution of each structural element for the connection of the beam-large-scale column. The finite elements used in all of the models are linear cubic elements of 8 knots with complete integration (C3D8 do Abaqus©), with no other types of finite elements being tested. The machine used in the processing was a computer with 8 GB of RAM memory, processor Intel core i7, video card NVIDEA GEFORCE GT de 2 GB, HD de 750 GB e 7500 rpm and operational system Windows 10.

The material of the constitution of the models is the concrete with characteristic resistance to compression at 28 days $f_{ck} = 25.0$ MPa, a longitudinal elasticity module $E_{ci} = 28,000.0$ MPa and coefficient of Poisson $v = 0.20$. The non-linearity of the material is considered of approximate shape according to the normative recommendations [4].

The loads applied to the models are the ones used in a common edification of reinforced concrete, besides the actual weight of the structure, both obtained considering NBR 6120 [19]. Considering the horizontal loads of wind, obtained through TQS© that uses the recommendations of NBR 6123 [20], acting over the large-scale columns and permanent and accidental vertical loads, according to NBR 6120 [19], about beams and slabs. The pattern of distribution of loads of wind to the columns used by TQS© is illustrated in Figure 7.

The surfaces of applications of the vertical and horizontal loads are indicated in Figure 8, in which the model MP3DL1 is the reference. In parts (a) and (b) of Figure 8, the surfaces of application of vertical loads are observed, corresponding to the surfaces of the tops
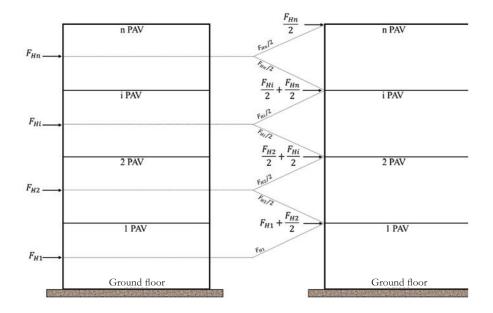

Figure 7
The pattern of the load distribution of wind on the columns in each pavement
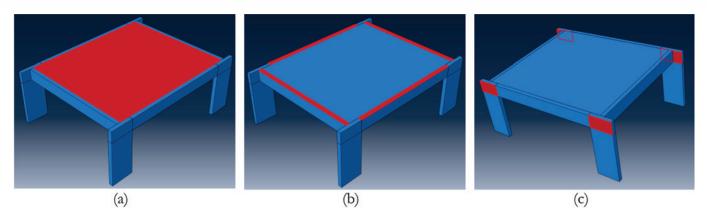**Source:** Adapted from TQS© [18]

Figure 8
An indication of action of the surfaces of application of vertical and horizontal loads over the structural elements of the models: (a) vertical over the slabs, (b) vertical over the beams and (c) horizontal on the large-scale columns
Source: Diniz [8]

of each slab and beam of the models. In part (c) the surfaces of application of the horizontal loads of wind over the large-scale columns are observed. It is perceived that the surfaces of application of the horizontal loads of wind correspond to those in reference to the superior part of each flight of column, the rectangular region of the intersection with the beams. In the modules with more floors, it is only necessary to apply the same vertical loads in all of the floors

and adapt loads of wind according to the height. In the models without slabs, loads of the removed slabs are summed to the loads of the same category already existent in the beams in a uniform manner, or, in other words, the total loads applied to the models with slabs are maintained in the models without slabs.
The magnitude of the vertical loads applied over the surfaces indicated in Figure 8, parts (a) and (b), in each floor of the models

Table 2
Magnitude of the vertical loads used in the models

Surfaces of the elements	Model with slabs			Models without slabs		
	PP (kN/m²)	PERM (kN/m²)	ACID (kN/m²)	PP (kN/m²)	PERM (kN/m²)	ACID (kN/m²)
Slabs	3.750	1.500	2.000	–	–	–
Beams	15.000	23.400	–	47.813	36.525	17.500

Source: Diniz [8]

Table 3
Magnitude of the horizontal loads applied over the surfaces of the large-scale columns

Floors	MFS3DSL1 and MFS3DWSL1 WIND (kN/m²)	MFS3DSL2 and MFS3DWSL2 WIND (kN/m²)	MFS3DSL3 and MFS3DWSL3 WIND (kN/m²)	MFS3DSL4 and MFS3DWSL4 WIND (kN/m²)	MFS3DSL5 and MFS3DWSL5 WIND (kN/m²)	MFS3DSL6 and MFS3DWSL6 WIND (kN/m²)
1° FL	3.083	6.958	7.333	7.667	7.917	8.042
2° FL	–	2.542	5.500	5.750	5.917	6.000
3°FL	–	–	2.833	6.042	6.208	6.125
4° FL	–	–	–	3.083	6.458	6.542
5° FL	–	–	–	–	3.292	6.750
6° FL	–	–	–	–	–	3.417

Source: Diniz [8]

Table 4
Factors for the reduction of loads in the combination ELS/CFREQ

Loads	PP	PERM	ACID	VENT
Factor of multiplication	1.00	1.00	0.30	0.30

Source: Diniz [8]

are presented in Table Table 2, where PP, PERM e ACID, mean loads due to their own weight, permanent loads and accidental loads, respectively.

Table 5
Eccentricity in the axis of the beam related to the large-scale column in each simulation

Simulation	e (m)
1	0.0
2	0.05
3	0.10
4	0.15
5	0.20
6	0.25
7	0.30
8	0.35
9	0.40
10	0.45
11	0.50

Source: Diniz [8]

The horizontal loads of wind have different magnitudes in each floor and also vary with the number of floors of the model. The horizontal load is applied in the direction of the axis X, which is the direction in which the columns present smaller moments of inertia, positive direction, over the surfaces of the four large-scale columns indicated in Figure 8, item (c). The magnitudes of the horizontal loads are organized in Table 3. Despite the intensity of the strengths of the wind grow as they go away from the base of the edification, it is observed in the table, that, due to the pattern of the load presented in Figure 7, they result in greater loads in the first floor and smaller ones in the last. It is pointed out that the horizontal loads of wind are equal for the models with and without slabs. The magnitude of these loads takes into consideration the Combination of the Limit State of Frequent Service (ELS/FREQ), according to Table 4, once the displacement obtained in the simulations of the Abaqus© are compared to the ones obtained in the CAD/TQS© with the reticulate models considering the same combination. The procedure of the simulations in each model is conducted as follows: for each coordinate presented in Table 5 simulations are conducted on the models with slabs and without slabs from 1 to 6 floors of the axis X on top of the axis of the column of greatest

Table 6
Results of the simulations of the models MFS3DWSL1 and MFS3DSL1

Simulation	e (cm)	MFS3DWSL1			MFS3DSL1		
		U_1 (m)	DPRDA	DPA	U_1 (m)	DPRDA	DPA
1	0.0	0.00076	0.00%	0.00%	0.00051	0.00%	0.00%
2	0.05	0.00076	-0.12%	-0.12%	0.00052	0.23%	0.23%
3	0.10	0.00076	0.08%	-0.05%	0.00052	0.08%	0.31%
4	0.15	0.00077	0.31%	0.26%	0.00052	-0.11%	0.20%
5	0.20	0.00077	0.63%	0.90%	0.00051	-0.25%	-0.06%
6	0.25	0.00078	1.08%	1.98%	0.00051	-0.23%	-0.29%
7	0.30	0.00079	1.58%	3.60%	0.00051	-0.08%	-0.37%
8	0.35	0.00081	2.08%	5.75%	0.00051	0.11%	-0.26%
9	0.40	0.00083	2.56%	8.45%	0.00051	0.34%	0.08%
10	0.45	0.00085	3.01%	11.71%	0.00052	0.60%	0.68%
11	0.50	0.00088	3.20%	15.29%	0.00052	1.20%	1.89%

Source: Diniz [8]

Table 7
Results of the simulations of the models MFS3DWSL2 and MFS3DSL2

Simulation	e (cm)	MFS3DWSL2			MFS3DSL2		
		U_1 (m)	DPRDA	DPA	U_1 (m)	DPRDA	DPA
1	0.0	0.00204	0.00%	0.00%	0.00154	0.00%	0.00%
2	0.05	0.00204	0.21%	0.21%	0.00155	0.43%	0.43%
3	0.10	0.00205	0.36%	0.57%	0.00156	0.53%	0.97%
4	0.15	0.00206	0.53%	1.10%	0.00157	0.63%	1.60%
5	0.20	0.00207	0.76%	1.87%	0.00158	0.75%	2.36%
6	0.25	0.00210	1.05%	2.94%	0.00159	0.94%	3.32%
7	0.30	0.00213	1.40%	4.38%	0.00161	1.18%	4.54%
8	0.35	0.00216	1.78%	6.24%	0.00163	1.46%	6.06%
9	0.40	0.00221	2.21%	8.59%	0.00166	1.77%	7.94%
10	0.45	0.00227	2.69%	11.51%	0.00170	2.13%	10.23%
11	0.50	0.00235	3.47%	15.38%	0.00174	2.78%	13.30%

Source: Diniz [8]

displacement. The axis pillar-wall is of greatest displacement. The axis of the large-scale column is represented by the coordinate 0.0 in Table 5 and coordinate 0.50m the position in which the face of the beam is aligned with the face of the large-scale column. In Figure 9 the variation of the of the eccentricity of the beam in the relation to the axis large-scale column is illustrated. In part

Table 8
Results of the simulations of the models MFS3DWSL3 and MFS3DSL3

Simulation	e (cm)	MFS3DWSL3			MFS3DSL3		
		U_1 (m)	DPRDA	DPA	U_1 (m)	DPRDA	DPA
1	0.0	0.00468	0.00%	0.00%	0.00360	0.00%	0.00%
2	0.05	0.00471	0.45%	0.45%	0.00363	0.66%	0.66%
3	0.10	0.00474	0.65%	1.09%	0.00366	0.85%	1.51%
4	0.15	0.00478	0.86%	1.96%	0.00370	1.04%	2.57%
5	0.20	0.00483	1.10%	3.09%	0.00374	1.25%	3.85%
6	0.25	0.00490	1.39%	4.52%	0.00380	1.48%	5.39%
7	0.30	0.00498	1.71%	6.31%	0.00386	1.77%	7.25%
8	0.35	0.00508	2.07%	8.51%	0.00394	2.06%	9.46%
9	0.40	0.00521	2.47%	11.19%	0.00404	2.41%	12.09%
10	0.45	0.00536	2.94%	14.46%	0.00415	2.79%	15.22%
11	0.50	0.00557	3.95%	18.99%	0.00430	3.54%	19.30%

Source: Diniz [8]

Table 9
Results of the simulations of the models MFS3DWSL4 and MFS3DSL4

Simulation	e (cm)	MFS3DWSL4			MFS3DSL4		
		U_1 (m)	DPRDA	DPA	U_1 (m)	DPRDA	DPA
1	0.0	0.00910	0.00%	0.00%	0.00700	0.00%	0.00%
2	0.05	0.00915	0.58%	0.58%	0.00706	0.81%	0.81%
3	0.10	0.00922	0.80%	1.38%	0.00713	1.05%	1.88%
4	0.15	0.00932	1.04%	2.43%	0.00722	1.28%	3.18%
5	0.20	0.00944	1.29%	3.76%	0.00733	1.53%	4.76%
6	0.25	0.00959	1.58%	5.40%	0.00746	1.79%	6.64%
7	0.30	0.00977	1.91%	7.41%	0.00762	2.09%	8.87%
8	0.35	0.00999	2.26%	9.84%	0.00780	2.40%	11.48%
9	0.40	0.01026	2.67%	12.78%	0.00802	2.77%	14.57%
10	0.45	0.01058	3.15%	16.33%	0.00827	3.16%	18.19%
11	0.50	0.01104	4.27%	21.30%	0.00860	3.98%	22.90%

Source: Diniz [8]

Table 10
Results of the simulations of the models MFS3DWSL5 and MFS3DSL5

Simulation	e (cm)	MFS3DWSL5			MFS3DSSourceL5		
		U_1 (m)	DPRDA	DPA	U_1 (m)	DPRDA	DPA
1	0.0	0.01542	0.00%	0.00%	0.01187	0.00%	0.00%
2	0.05	0.01552	0.66%	0.66%	0.01198	0.92%	0.92%
3	0.10	0.01566	0.89%	1.56%	0.01213	1.18%	2.11%
4	0.15	0.01584	1.14%	2.72%	0.01230	1.42%	3.57%
5	0.20	0.01606	1.41%	4.16%	0.01251	1.70%	5.33%
6	0.25	0.01633	1.70%	5.93%	0.01275	1.96%	7.40%
7	0.30	0.01666	2.02%	8.07%	0.01304	2.29%	9.86%
8	0.35	0.01706	2.39%	10.65%	0.01338	2.60%	12.71%
9	0.40	0.01754	2.80%	13.75%	0.01378	2.99%	16.08%
10	0.45	0.01812	3.29%	17.49%	0.01425	3.39%	20.01%
11	0.50	0.01892	4.46%	22.73%	0.01486	4.28%	25.14%

Source: Diniz [8]

Table 11
Results of the simulations of the models MFS3DWSL6 and MFS3DL6

Simulation	e (cm)	MFS3DWSL6			MFS3DSL6		
		U₁ (m)	DPRDA	DPA	U₁ (m)	DPRDA	DPA
1	0.0	0.02349	0.00%	0.00%	0.01800	0.00%	0.00%
2	0.05	0.02366	0.71%	0.71%	0.01818	0.99%	0.99%
3	0.10	0.02389	0.96%	1.68%	0.01841	1.26%	2.27%
4	0.15	0.02418	1.21%	2.91%	0.01869	1.52%	3.82%
5	0.20	0.02454	1.48%	4.44%	0.01902	1.80%	5.69%
6	0.25	0.02497	1.78%	6.30%	0.01942	2.08%	7.89%
7	0.30	0.02550	2.11%	8.53%	0.01988	2.40%	10.48%
8	0.35	0.02613	2.47%	11.22%	0.02042	2.72%	13.49%
9	0.40	0.02688	2.89%	14.43%	0.02106	3.11%	17.02%
10	0.45	0.02779	3.39%	18.30%	0.02180	3.52%	21.13%
11	0.50	0.02907	4.58%	23.72%	0.02276	4.39%	26.45%

Source: Diniz [8]

(a), there is the axis of the beam aligned to the axis of the large-column. In part (b) there is the axis in an intermediate position, between the axis and the face of the large-scale column. And, last, in part (c), there is the alignment of the borders of the beam and the large-scale column.

9. Comparative analysis of the models without and with slabs

The results of the simulations are the horizontal displacements in the direction and positive sense of the axis X (U_1) captured on top of the axis of the large-scale column of greatest displacement of each model. The procedure adopted for the simulations implied in greater displacements always for the large-scale column C1, indicated in Figure 6. The models lose symmetry when the beams and slabs are dislocated in a direction to the borders of the columns.
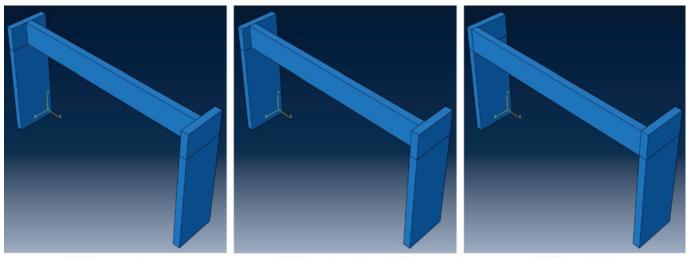
This way, the displacements in C1 and C3 are only the same in the initial position, just like in C2 and C4.

The data obtained in the simulations are in the sequence of Table 6 and Table 11, in which the acronyms DPRDA e DPA, mean Percentage Difference in Relation to the Previous Displacement and Accumulated Percentage Difference, respectively, and are calculated according to the following expressions:

$$\text{DPRDA}_i = \frac{U_{1,i} - U_{1,i-1}}{U_{1,i-1}} \qquad \text{DPA}_i = \frac{U_{1,i} - U_{1,1}}{U_{1,1}} \tag{4}$$

where $U_{1,i}$ is the displacement of the current line, displacement to the current eccentricity of the beam; $U_{1,i-1}$ is the displacement of the previous line, displacement to the previous eccentricity of the beam and $U_{1,1}$ is the displacement of the initial line, displacement to the beam in the initial position, without eccentricity.

In Table 6, the model of 1 floor with slab presented an anomalous

(a) Viga no centro   (b) Viga entre centro e borda   (c) Viga na borda

Figure 9
Illustration of the eccentricity of the beams related to the large-scale columns
Source: Diniz [8]

Figure 10
Curves of the accumulated percentage differences of the 11 simulations of the models MFS3DWSL1 to MFS3DWSL6
Source: Diniz [8]

Figure 11
Curves of the accumulated percentage differences of the 11 simulations of the models MFS3DSL1 to MFS3DSL6
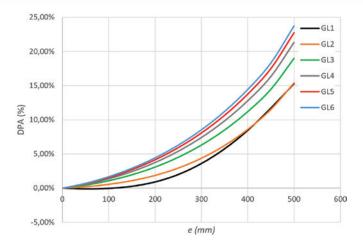**Source:** Diniz [8]
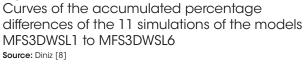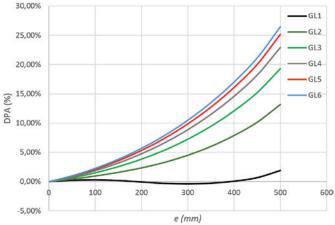
behavior in relation to the others, presenting growth of the displacement for the first eccentricities, decrease for the intermediate positions and, for the final positions, it starts to grow again. In the analysis conducted, it was observed that the slab, when it dislocates in the normal direction to its plane, causes torsion in the beams and in the parts of the columns connected to it, what accentuates the displacements of the part of the columns in the sense of the load and part in the contrary sense. In this model, the vertical load of the columns is inferior to the ones of the other models, facilitating the occurrence of the effects cited previously. Besides this, the pattern of distribution of a load of wind for structures with 1 floor is different (results in inferior load) from the correspondent one for structures with 2 or more floors, according to Figure 7.

In each model analyzed, the influence of the flexibility generated by the eccentricity of the beam in the horizontal displacements is verified. In the columns referring to the DPAs of the Tables 6 to 11, it is observed that the growth of the percentage of the displacements is observed while the beam becomes more eccentric in relation to its initial position. The greater the quantity of floors of the model more the growth of the displacements is intensified. The differences between U_1 measured with the beam in the initial position e = 0.0 and in the final position e = 0.50 m come from 15.29%/1.89% (without slab/with slab) in the models with 1 floor and arrived at 23.72%/26.45% (without slab/with slab) in the models with 6 floors. The graphics of Figures 10 and 11 present the curves of growth of the displacements obtained for the 6 models without slabs and 6 models with slabs. It is pointed to that the growth is not linear and intensifies as the position of the beam comes close to the border of the large-scale column.

The displacements increase in a non-linear manner when compared to the displacement of the previous simulation. It is observed that the columns referring to the DPRDA of Table 6 and Table 11 contain more elevated values in the final simulations, implying in increases of more elevated displacements when the eccentricities of the beams get close to the borders of the large-scale columns.

The inclusion of slabs contributes in a relevant manner for the hardening of the models, increasing the rigidity of the beams, constituting beams in the shape of an inverted "L". The reduction of the horizontal displacements on the top of the axis of the large-scale columns is significant, as observed in the sequence of Table 6 and Table 11. The absolute displacements U_1 are always inferior for the models with slabs, indicating that the models without slabs are less rigid that the models with slabs.

The braced structure in the models with slabs is more robust because they count on the rigidity of the bending of the slab, which although small when compared to the rigidities of beams and pillars, contributes a lot to the stability of the models in the simulations conducted. Another contribution of the slabs for the rigidity obtained in the simulations is the effect of the rigid diaphragm presented by Corrêa [21], according to which the pavement behaves like an element infinitely rigid in its plane, what distributes the horizontal actions between the several panels of braced structures (columns
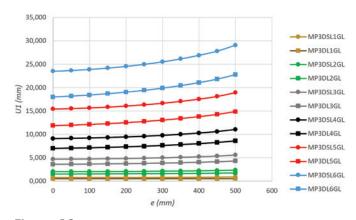

Figure 12
Displacements of all of the models
Source: Diniz [8]

Table 12
Percentage of the reduction of the displacements of the models with slabs related to the models without slabs

e (m)	PR (%)					
	1FL	2FL	3FL	4FL	5FL	6FL
0.0	32.6	24.3	23.1	23.1	23.0	23.4
0.05	32.4	24.1	22.9	22.9	22.8	23.2
0.10	32.4	24.0	22.8	22.7	22.6	23.0
0.15	32.7	23.9	22.6	22.5	22.4	22.7
0.20	33.3	23.9	22.5	22.3	22.1	22.5
0.25	34.1	24.0	22.5	22.2	21.9	22.2
0.30	35.2	24.2	22.4	22.0	21.7	22.0
0.35	36.4	24.4	22.4	21.9	21.6	21.8
0.40	37.8	24.8	22.5	21.8	21.4	21.7
0.45	39.3	25.2	22.6	21.8	21.3	21.6
0.50	40.4	25.7	22.9	22.0	21.5	21.7

Source: Diniz [8]

and/or framed structures). Goulart [22] verified the significant contribution of the slabs for the stabilities of the edifications, being able to reduce the coefficient γ_z significantly when the contribution of the slabs is considered, even with the reduction of the rigidity for the consideration of the non-linear physics.

In the graphic of Figure 12, there are the curves of the displacements of the models with and without slabs of 1 to 6 floors. With this, it is possible to verify that, comparing the models with the same number of floors, the displacements of the models without slabs distance themselves more intensely related to the models with the most elevated numbers of floors.

Table 12 groups the data of the reductions of the displacements when the slabs are inserted in the models. In this table, the Percentage of Reduction (PR) of the displacements of the models related to the models without slabs is compared for each eccentricity of the beam. It is observed that the reductions are greater for models with fewer floors, reducing gradually with the increase of the floors of the models.

Another important result taken from the data of Table 12 is the tendency of smaller reductions of displacements when the eccentricity of the beam increases related to the axis of the large-scale column, reinforcing the idea of the influence of the increase of the flexibility of the connection generated by the eccentricity of the beam. It is observed, for example, that the PR for the beam with eccentricity $e = 0$ is 23.4% and reduces to 21.7% for the final eccentricity of the beam $e = 0.50$ m. Besides this, it is verified that there is a tendency of displacements to always increase as the number of floors of the models increase, however, the increase of the displacement is decreasing, or, in other words, the relation between the displacement with a number i+1 of floors and the displacement of the of the model with a number i of floors decreases with the increase of the number of floors, according to Table 13. It is pointed out that the displacements used for this verification are of the models without slabs, but that the same pattern occurs for the model with slabs. Once more, the effect of the instability generated by the eccentricity of the beam is perceived, because of the relation $\dfrac{U_{1,i+1}}{U_{1,i}}$ is greater when the beam is more eccentric, indicating a slower decrease for the more eccentric positions of the beam.

Table 13
Relation between the displacements of the models without slabs with i+1 floors e i floors

e (m)	$\dfrac{U_{1,2}}{U_{1,1}}$	$\dfrac{U_{1,3}}{U_{1,2}}$	$\dfrac{U_{1,4}}{U_{1,3}}$	$\dfrac{U_{1,5}}{U_{1,4}}$	$\dfrac{U_{1,6}}{U_{1,5}}$
0.0	2.67	2.30	1.94	1.69	1.52
0.05	2.68	2.31	1.94	1.70	1.52
0.10	2.69	2.31	1.95	1.70	1.53
0.15	2.69	2.32	1.95	1.70	1.53
0.20	2.69	2.33	1.95	1.70	1.53
0.25	2.69	2.34	1.96	1.70	1.53
0.30	2.69	2.34	1.96	1.71	1.53
0.35	2.68	2.35	1.97	1.71	1.53
0.40	2.67	2.36	1.97	1.71	1.53
0.45	2.66	2.36	1.97	1.71	1.53
0.50	2.67	2.37	1.98	1.72	1.54

Source: Diniz [8]

Table 14
Values of α, γ_z and and LEPMOL for the model MFS3DWSL6

e (m)	Non-linear geometric analysis				
	U_1A (m)	LEPMOL	U_1T (m)	α	γ_z
0.0	0.0235	6.00	0.0245	0.651	1.102
0.05	0.0237	6.00	0.0245	0.649	1.102
0.10	0.0239	6.00	0.0245	0.651	1.102
0.15	0.0242	6.00	0.0245	0.651	1.102
0.20	0.0245	6.00	0.0245	0.651	1.102
0.25	0.0250	5.85	0.0250	0.654	1.103
0.30	0.0255	5.60	0.0255	0.659	1.105
0.35	0.0261	5.35	0.0261	0.664	1.106
0.40	0.0269	5.01	0.0269	0.672	1.109
0.45	0.0278	4.70	0.0278	0.680	1.112
0.50	0.0291	4.30	0.0291	0.691	1.116

Source: Diniz [8]

10. Parameter α, coefficient γ_z and criteria of flexibilizing of the portico TQS©

The parameters of stability are obtained with the help of the TQS© UNI-PRO12 VERSION 19.10 considering the model IV, in which only beams and columns contribute to the rigidity of the reticulate model. The parameters of stability are noted after the calibration of the criteria of flexibilization LEPMOL and REDMOL of the TQS© using the results of the Abaqus©. Both criteria, in separate, are modified until the displacements in the TQS© equal to the displacements obtained with the Abaqus©. The model used for the calibration of the criteria is model MFSP3DSL6 with GLs simulations, by the fact of the greater quantity of the floors and also because it has only columns and beams contributing to the rigidity, similar to the model IV do TQS©.

The criteria of the flexibilization of the connections between beams and columns, LEPMOL and REDMOL, are connected to the mo-
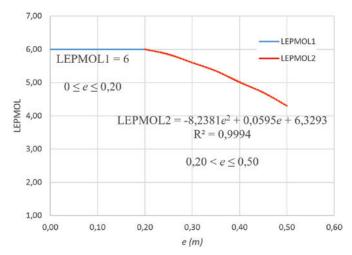
ment of inertia of the column that really contributes to the rigidity of the connection and a factor that reduces the rigidity of the spring that represents the beam-column connection, respectively, as already presented in Figure 4.

In light of the foregoing, in Table 14 the values obtained of LEPMOL for each eccentricity of the beam in the model MP3DSL6 with the linear geometric analysis is presented. In the same table, there are also the values of α and γ_z noted for each calibration of LEPMOL. The values of LEPMOL are obtained maintaining all of the other criteria of the TQS© invariable, including REDMOL, which is maintained with a fixed value of 1.0. U_{1A} and U_{1T} are, respectfully, horizontal displacements obtained in the Abaqus© and horizontal displacement obtained in the TQS©.

According to Table 14, the calibration of LEPMOL is not sufficient to equal the displacements of the TQS© to the ones of the Abaqus© in the first eccentricities of the beam. It is perceived that even using the maximum value for LEPMOL, which is 6 times the
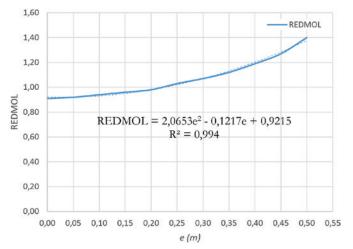

Figure 13
The curve of the LEPMOL in function of the eccentricity of the beam
Source: Diniz [8]

Figure 14
The curve of the REDMOL in function of the eccentricity of the beam
Source: Diniz [8]

Table 15
Values of α, γ_z and REDMOL for the model MFS3DWSL6

e (m)	Non-linear geometric analysis				
	U_1A (m)	REDMOL	U_1T (m)	α	γ_z
0.00	0.0235	0.91	0.0235	0.641	1.099
0.05	0.0237	0.92	0.0237	0.642	1.100
0.10	0.0239	0.94	0.0239	0.644	1.100
0.15	0.0242	0.96	0.0242	0.647	1.101
0.20	0.0245	0.98	0.0245	0.649	1.102
0.25	0.0250	1.03	0.0250	0.654	1.103
0.30	0.0255	1.07	0.0255	0.658	1.105
0.35	0.0261	1.12	0.0261	0.664	1.106
0.40	0.0269	1.19	0.0269	0.671	1.109
0.45	0.0278	1.27	0.0278	0.679	1.112
0.50	0.0291	1.40	0.0291	0.692	1.116

Source: Diniz [8]

width of the beam (equal to the width of the large-scale column), the minimum displacement in the TQS© is of 0.0245 m.

Using the data of Table 14, the graphic of Figure 13 was built, in reference to the curve of LEPMOL. The graphic is composed of two parts: the first is constant and receives the denomination of LEPMOL1 and the second receives the denomination of LEPMOL2. There is also an estimative equation of LEPMOL for each part due to the eccentricity of the beam. It is pointed out that the equation is an adjustment obtained through the Excel 2013 software with satisfactory approximation, as coefficient R², indicating that the curve of adjustment is close to the real curve when its value becomes close to 1.0.

The criteria of flexibilization of REDMOL is also calibrated for each eccentricity of the beam. REDMOL is obtained considering the value of LEPMOL, that would use the whole section of the column, LEPMOL equal to 6, and the other criteria of the invariable TQS©s. The values of REDMOL, α e γ_z obtained for the model MP3DSL6 with simulations GLS are presented on Table 15.

In the graphic of Figure 14, there is the curve of REDMOL and an equivalent equation obtained also on Excel 2013 through an adjustment of a polynomial of the second level.

The decrease of the equation of LEPMOL indicates that its values tend to decrease as the displacements increase. This way, the tendency is that for framed structures with more floors - even smaller values are found for these criteria. Whereas the values of REDMOL increase with the growth of the displacements, indicating a tendency of growth for the framed structures with a greater number of floors.

Another observation related to LEPMOL is that it influences only the rigidity of the connection, not reducing the inertia of the bar that represents the column in the framed structure, or, in other words, the integral rigidity of the horizontal section of the column in the reticulate model of the TQS© is considered. The section of the column is modified by LEPMOL only for the deliberation of the rigidity of the "spring".

The equation of LEPMOL and REDMOL are approximations and have several limitations because the displacements and stability of the model in the TQS© depend on several other criteria that may be calibrated for a more real representation of the structure.

The parameters of stability for the model analyzed are in Tables 14 and 15, for calibration of LEPMOL and REDMOL, respectively. Parameter α is always superior to the value of reference for braced structures by framed structures of large-scale column, which is α = 0.6. This parameter indicates that the structure is always of mobile knots, for any eccentricity of the beam in the model considered (MP3DSL6).

Whereas for parameter γ_z indicates a structure of mobile knots for any eccentricity of the beam when the calibration is done by LEPMOL. When calibrating by the REDMOL criteria, γ_z presents, for the local eccentricities, values below the limit for structures of mobile knots. Based on the eccentricity e = 0.15 m the values of γ_z, if they already indicate that the structure is of mobile knots. Despite the geometrically linear analysis, in structural models of relatively low height, the effect of the large-scale columns connections on the flexibility and global stability was achieved in a satisfactory manner for the classification of the conditions of displacement of the knots and posteria or estimate of the effects of second order.

11. Conclusions

In this study, the global stability of structural models of concrete was evaluated considering the influence of the eccentricity in the beam large-scale column by verifying the parameters of normative stability and displacements of reticulate models. The analysis of stabilities of the influence of the eccentricity was realized in geometric models and offer an indication of the influence of the eccentricity of the large-scale column connection in the stability of these models.

In the simulations of the models idealized in this work, some composed by large-scale columns and beams and others by large-scale columns, varied in the eccentricities of the beam-large-scale column trying to determine its influence in the stability. The displacements were captured on the top of the large-scale columns, using as a reference to the longitudinal axis, in each model for each eccentricity of the beam.

In the simulations of the models idealized in this study idealized in this work, some of the ones composed by large-scale columns and

beams and others by large-scale columns, beams, and slabs, the eccentricities of the connections large-scale columns varied when trying to determine its influence in the stability. The displacements were captured on the top of the large-scale columns, using as a reference to the longitudinal axis, in each model for each eccentricity of the beam.

The horizontal displacements obtained demonstrate that the eccentricity of the beam related to the axis of the large-scale column has a considerable effect on the rigidity of the connection and consequently in the global stability because the displacements grow as the eccentricity of the connection increases. The difference of displacements obtained with the connection beam-large-scale column and the most extreme connection in the column is significant, reaching 26.45% in the model MP3DL6. Another important factor is that the displacements grow faster for the eccentricities of the close connection of the extremity of the large-scale column.

The models with the presence of the slab were more rigid than the models composed only by large-scale columns and beams. The percentage of reduction in the displacements of the models with slabs related to the models without slabs is significant, reaching 21.7% among the models of 6 floors. It is also perceived that the reduction decreases with the increase of the eccentricity and with the increase in the number of floors of the models.

The analysis of the flexibility criteria of the spatial framed structure of the TQS© resulted in important information. The LEPMOL tends to be reduced, the reduction implying on the rigidity of the connection, for more eccentric positions of the beams and models with a greater number of floors, once the displacements grow with the increase of the number of floors. Whereas the REDMOL tend to increase with the growth of the displacements, indicating also a reduction of the rigidity of the connection of the beam-large-scale column with a greater number of floors and for more eccentric positions of the beams.

The parameters of stability evaluated in the models analyzed remained within the limits established by NBR 6118/2014. The values of the parameters, in general, indicate that the model evaluated is classified as a structure of mobile knots.

At last, it is pointed out that the results obtained in this work have the capacity to aid in the considerations of global stability of structures constituted by large-scale columns of the simple section. It also offers indications about the consideration of the criteria of flexibilization of the framed structure TQS©, LEPMOL, and REDMOL, when the structures have eccentric connections of beams with large-scale columns. It is pointed out that the results presented in this work are of simple models and considering simplified conditions of application of loads and of the disposition of the columns in the plant.

12. Acknowledgments

13. References

[1]   INSTITUTO BRASILEIRO DE GEOGRAFIA E ESTATÍSTI-CA. Sinopse do censo demográfico 2010. Disponível em: http://www.censo2010.ibge.gov.br/sinopse/index.php?dados=8 Acessado em: 15/09/2016.

[2]   FONTE, A. O. C. Análise não linear geométrica de edifícios altos, 1992, Tese (Doutorado em Ciências em Engenharia Civil) – Universidade Federal do Rio de Janeiro, Rio de Janeiro – RJ, 223p.

[3]   ZUMAETA MONCAYO, W. J. Análise de segunda ordem global em edifícios com estrutura de concreto armado, 2011, Dissertação (Mestrado em Estruturas) – Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 221 p.

[4]   ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. Projeto e execução de obras de concreto armado. - NBR 6118, Rio de Janeiro – RJ, 2014.

[5]   BUENO, M. M. E. Avaliação dos parâmetros de instabilidade global em estruturas de concreto armado, 2009, Dissertação (Mestrado em Estruturas e Construção Civil) – Faculdade de Tecnologia, Departamento de Engenharia Civil, Universidade de Brasília, Brasília - DF, 105p.

[6]   ARAÚJO, T. J. F. Estudo da ligação viga-pilar em sistema estrutural de concreto considerando a variação da semi-rigidez por meio de parâmetros geométricos da ligação, 2013, Dissertação (Mestrado em Engenharia Civil: Estruturas) – Centro de Tecnologia, Universidade Federal de Alagoas, Maceió-AL, 111p.

[7]   PASCUAL, C. A. Q. Parâmetros característicos para a definição de um modelo estático: "pilar de pórtico" ou "pilar-parede", 1987, Dissertação (Mestrado em Estruturas) - Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 228p.

[8]   DINIZ, C. W. S. Estabilidade global de estrutura de edifícios considerando a influência da excentricidade na ligação viga-pilar parede, 2017, Dissertação (Mestrado em Engenharia Civil) – Centro de Tecnologia, Universidade Federal de Alagoas, Maceió – AL, 128p.

[9]   FEITOSA, L. A.; ALVES, E. C. Estudo da estabilidade global de edifícios altos com lajes protendidas. Revista Ibracon de Estruturas e Materiais, v.8, n.2, 2015; p. 196 - 224.

[10]   WORDELL, F. Avaliação da instabilidade global de edifícios altos, 2003, Dissertação (Mestrado em Engenharia Civil) – Universidade Federal do Rio Grande do Sul, Porto Alegre – RS, 94p.

[11]   LIMA, J. S. Verificações da punção e da estabilidade global em edifícios de concreto: desenvolvimento e aplicação de recomendações normativas, 2001, Dissertação (Mestrado em Engenharia de Estruturas) – Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 249p.

[12]   PINTO, R. S. Não-linearidade física e geométrica no projeto de edifícios usuais de concreto armado, 1997, Dissertação (Mestrado em Engenharia Civil) – Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 128p.

[13]   CARMO, R. M. S. Efeitos de segunda ordem em edifícios usuais de concreto armado, 1995, Dissertação (Mestrado em Estruturas) – Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 135 p.

[14]   PINTO, R. S.; CORRÊA, M. R. S.; RAMALHO, M. A. Utilização do parâmetro γ_z para estimar esforços de segunda ordem em edifícios de concreto armado. Revista Ibracon de Estruturas. v.1, n.2, 2005, p. 145 - 165.

[15] VIEIRA, V. V. S.; RODRIGUES JUNIOR, S. J.; VELOSO, L. A. C. M. Análise de estabilidade global de edifícios de concreto armado utilizando o coeficiente γ_z. Revista Ibracon de Estruturas. v.10, n.5, 2017, p. 1113 - 1140.

[16] PASSOS, V. M.; FEITOSA, L. A.; ALVES, E. C.; AZEVEDO, M. S. Análise da instabilidade de edifícios altos com lajes protendidas e lajes nervuradas. Revista Ibracon de Estruturas. v.9, n.2, 2016, p. 244 - 262.

[17] PINTO, R. S.; RAMALHO, M. A. Não-linearidade física e geométrica no projeto de edifícios usuais de concreto armado. Cadernos de Engenharia de Estruturas, São Carlos, 2002, n.19, p. 171 - 206.

[18] TQS INFORMÁTICA (2018). Manual do usuário: dominando os sistemas CAD/TQS – análise estrutural. TQS Informática, v.3, São Paulo, 2018.

[19] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. Cargas para o cálculo de estruturas de edificações. - NBR 6120, Rio de Janeiro – RJ, 1980.

[20] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. Forças devidas ao vento em edificações. - NBR 6123, Rio de Janeiro – RJ, 1988.

[21] CORRÊA, M. R. S. Aperfeiçoamento de modelos usualmente empregados no projeto de sistemas estruturais de edifícios, 1991, Tese (Doutorado) – Escola de Engenharia de São Carlos, Universidade de São Paulo, São Paulo – SP, 331p.

[22] GOULART, M. S. S. Contribuição da rigidez à flexão das lajes para a estabilidade global de edifícios, 2008, Dissertação (Mestrado em Ciências em Engenharia Civil) – Universidade Federal do Rio de Janeiro, Rio de Janeiro – RJ, 115p.