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Effect of wind in the design of reinforced concrete buildings

Efeito do vento no dimensionamento de edifícios de concreto armado

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Abstract

This paper presents the results from a parametric study carried in order to quantify how far errors in the design stage related to the consideration of the wind action may put at risk the response and safety of reinforced concrete buildings. Using an architectural model as reference and varying the number of floors of the building, the structural safety was evaluated as a function of the wind action intensity. Results showed that even for low-rise buildings, with 10 floors, ignoring the wind action can significantly jeopardize their behaviour and safety. Yet, for slenderer buildings, up to 30 floors, it can lead to catastrophic results, as the ruin of the structure by progressive collapse.

Keywords: wind, columns, reinforced concrete, buildings.

Resumo

Este artigo apresenta os resultados de um estudo paramétrico realizado com o objetivo de quantificar o tanto que erros na etapa de projeto relacionados com a consideração da ação do vento podem comprometer a resposta em serviço e a segurança de edifícios de concreto armado. Usando-se um modelo arquitetônico como referência e variando-se o número de pavimentos do edifício, a segurança estrutural foi avaliada como uma função da intensidade da ação do vento. Os resultados mostraram que mesmo para edifícios baixos, com 10 pavimentos, a desconsideração da ação do vento pode comprometer significativamente o comportamento e a segurança e que no caso de edifícios mais esbeltos, com até 30 pavimentos, pode levar a resultados catastróficos, como a ruína da estrutura através de colapso progressivo.

Palavras-chave: vento, pilares, concreto armado, edifícios.

1. Introduction

The progressive collapse is a partial or complete ruin mechanism of a structure caused by the located failure of a primary structural element (Li *et al* [1]). This leads to the redistribution of loads to the surrounding elements, which can collapse as well. In national and international literature there are several cases of serious structural accidents in buildings with concrete structure (Kamari *et al* [2], Schellhammer *et al* [3], King and Delatte [4], Gardner *et al* [5]). Souza and Araújo [6] highlight that in Brazil, the amount of structural accidents is increasing and that their origins are mainly related to mistakes in design and construction stages. This recurs in other countries too, and as example it can be cited the work of Kaltakci *et al* [7], that analysing the causes of accidents in buildings under construction in Turkey, got to the same conclusions.

On January 29, 2011, in the city of Belem, Para, occurred one of the greatest structural accidents registered in Brazil in reinforced concrete buildings. On this day collapsed the residential building called Real Class, which was under construction, fatally victimizing three people. Figure 1 shows the building under construction and the landscape after its collapse. This building had constructed area of approximately 13,400 m² and was composed by 01 under-

Figure 1 – Collapse of Real Class building (available in http://compradordeimovel. no.comunidades.net/ edificio-real-class-construtora-real)

ground floor, 01 ground floor and 35 high floors, with about 105 m high. The building's tower had its weight estimated in 9 thousand tons, which were supported by 25 columns.

The building's ruin occurred during a heavy rain, where intense winds were registered in the city. In slender structures, the wind is one of the main actions to be considered (see Dyrbye and Hansen [8]) and there are records of various structural accidents with this action as origin (see Sha and Malinov [9], Klinger *et al* [10] and Rao *et al* [11]). At the time of this accident, three different teams carried out studies about the case and the analyses developed by a professors' team of the Faculty of Civil Engineering from Federal University of Para indicated that the wind load was not properly considered in the design.

This paper presents results of a parametric study done in order to show how errors in the consideration of the wind action during the design stage can jeopardize the behaviour and the safety in the Ultimate Limit State of reinforced concrete buildings. A typical architectural plan was used as reference to develop the structural and foundations designs, having as one of the variables the number of floors, varied between 10, 20 and 30 floors. The other variable of the study was the magnitude of the wind, which was initially assumed as zero, to define the geometry and the amount of steel reinforcement of the structural elements, which were designed according to the recommendations of ABNT NBR 6118 [12]. Later, the wind action was considered as recommended by ABNT NBR 6123 [13] and the internal forces were used to verify the response in service and the safety of the structure and the foundations that were initially designed without wind consideration. The response in service and safety of columns and foundations is discussed.

2. Literature review

2.1 Wind

In the structural design, the wind is physically represented by a speed profile reaching a building. Its characteristics and the effects it generates depend on the velocity of the wind, the geometry of the building, and of the protection caused by the terrain and surrounding obstacles. Rosa *et al* [14] warn that the environment has great influence on the wind loads in a given building and that its accurate consideration is only possible if there are experimental data from tests in wind tunnel. On the other hand, Elsharawy *et al* [15] point out that even low building may suffer significant influence of wind action, as in function of its plan geometry and columns positioning, wind action can induce torsional forces in the spatial frame.

Although ABNT NBR 6123 [13] does not guide the use of automated weather stations data, the ABNT NBR 5422 [16] alludes to this procedure in the recommendations for transmission lines designs. In this case, the currently used values to define the country's wind isotachs can be verified by data from automatic stations, provided by institutions responsible for monitoring meteorological data. Figure 2 shows wind speed data provided to the city of Belem-PA by ICEA [17], in the period of 1951-2010. It is possible to realize that the basic wind speed (V_0) defined for Belem, which is 30 m/s, was exceeded three times in this period and that the average results seems to present increasing trend in function of time.

It is known that pressure made by wind is not static. It suffers fluctuations (gusts) and depends on characteristics of its incidence on

structures (Elsharawy *et al* [18]). These pressure fluctuations not only depend on the gust time, but also on the flow regime (turbulence) and can cause the structure to suffer dynamic oscillations, inducing fatigue. In design situations, standards recommend the adoption of gust speeds in structural design, since structures in general have higher fundamental frequency of vibration than natural wind frequency.

The intensity of wind forces in a structure varies spatially and temporally and, in design, most standards adopt a simplified methodology where wind dynamic action is replaced by an equivalent static load. Through this procedure, it is attempted to represent the peak pressure caused by wind on the structure, being this pressure function of wind basic speed (V_0), and of S1, S2 and S3 parameters, as shown in Equation 1. This speed (V_0) was established for the entire country through probabilistic approach and is defined as the 3-second-speed gust, exceeded on average once in 50 years, measured 10 m above the ground, in open and flat field. Another important consideration on determining wind effects is the drag force, that in multistory buildings, describes the forces induced by the wind and should be calculated according to ABNT NBR 6123 [13] using Equation 2.

Figure 2 – History of wind speed in Belem-PA (ICEA)

$$q=0.613 \cdot (V_0 \cdot S_1 \cdot S_2 \cdot S_3)^2 \qquad (1)$$

$$F_a = C_a q A \qquad (2)$$

Where:

F_a is the drag force,

C_a is the drag coefficient, obtained in function of the height and the plan dimensions of the building,

q is the velocity pressure,
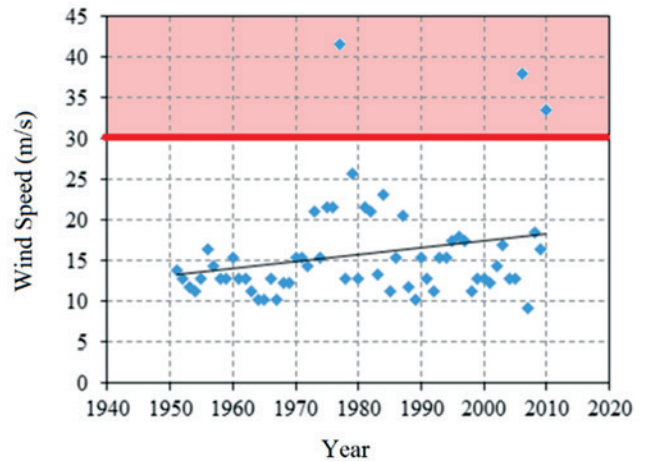
A is the area of the reference surface.

S1, S2 and S3 are parameters defined in ABNT NBR 6123 [13]

The determination of the drag coefficient is made depending on wind turbulence conditions. The ABNT NBR 6123 [13] defines that a building can be considered in high wind turbulence when its height does not exceed twice the average height of buildings in the vicinity, extending these, toward and in the direction of the incident wind, in a minimum distance of: 500 m, for a building up to 40-m-high; 1,000 m, for a building up to 55-m-high; 2,000 m, for a building up to 70-m-high; and 3,000 m, for a building up to 80-m-high.

2.2 Global stability

The verification of reinforced concrete columns in tall buildings is influenced by the overall stability of the building and can be negatively affected by 2nd order effects obtained with the calculation considering the deformed structure. ABNT NBR 6118 [12] presents two approximate procedures for checking the possibility of dispensing the consideration of 2nd order global forces: instability parameter α; and coefficient γ_z. They are used to classify a structure as being composed by fixed or mobile nodes and, in the case of

coefficient γ_z, it is considered that a structure has fixed nodes if $\gamma_z \leq 1.1$. Feitosa and Alves [19] consider that the use of these parameters in design is convenient, since the precise consideration of 2nd order effects may significantly increase analysis complexity. The coefficient γ_z can be determined using Equation 3, and in the case of structures with mobile nodes, since $\gamma_z \leq 1.3$, 1st order forces can be used to calculate the 2nd order ones, being scaled up by $0.95\gamma_z$. If the structure presents $\gamma_z > 1.3$, calculating 2nd order effects must be made using the P-Delta analysis method.

$$\gamma_z = \frac{1}{1 - \dfrac{\Delta M_{tot,d}}{M_{1,tot,d}}} \qquad (3)$$

Where:

$M_{1,tot,d}$ is the 1st order moment, found by Equation 4,

$\Delta M_{,tot,d}$ is the addition of moments after 1st order analysis, determined by Equation 5.

$$M_{1,tot\ d} = \sum \left(F_{hid} h_i \right) \qquad (4)$$

Where:

F_{hid} is the horizontal force applied on *i* floor,

h_i is the floor's height *h*.

$$\Delta M_{tot,d} = \sum \left(P_{id} u_i \right) \qquad (5)$$

Where:

P_{id} is the acting vertical force on *i* floor,

u_i is the horizontal displacement of *i* floor.

2.3 Design of reinforced concrete columns

In buildings a column will usually be subjected to bending moments in addition to the axial compressive forces due to not only asymmetry of spans and loads, but also because of horizontal actions like the wind. In these cases, it is common for both design and verification of load capacity to use bending-axial load interaction diagrams. These curves are obtained assuming points with different strain states in the cross section and computing for each of these points the axial force and the resultant moment in the section, as shown in Figure 3a for combined bending and axial load.
In this figure, a random point A represents a combination of axial force and bending moment that would lead the column to ruin. In it, any combination of forces that results in a point inside the curve represents a safe load state and any point outside shows a combination of forces higher than the element's load capacity. Radial lines as line OA represent the load eccentricity and point B shows the combination of forces for a balanced failure. Above this point, the failure is controlled by compression and below it, controlled by tension. Combination of forces up the line OC indicate critical situations where the ruin can occur without tensile strains of the element (no cracking).
In the case of columns under compression plus biaxial bending it is possible to draw a three-dimensional interaction surface from the interaction diagrams for the two main axes, as illustrated in Figure

3b, for cases a and b. For case c, that combines moments in x and y directions, the resulting eccentricity's direction is defined by the angle λ, calculated with Equation 6. Flexure in this case occurs on an axis defined by the angle θ. In practice, the construction of this three-dimensional surface of interaction can be complicated even using computational methods. In this paper, it was made in a simplified manner, as described by Nilson *et al* [20] and presented in Equation 7.

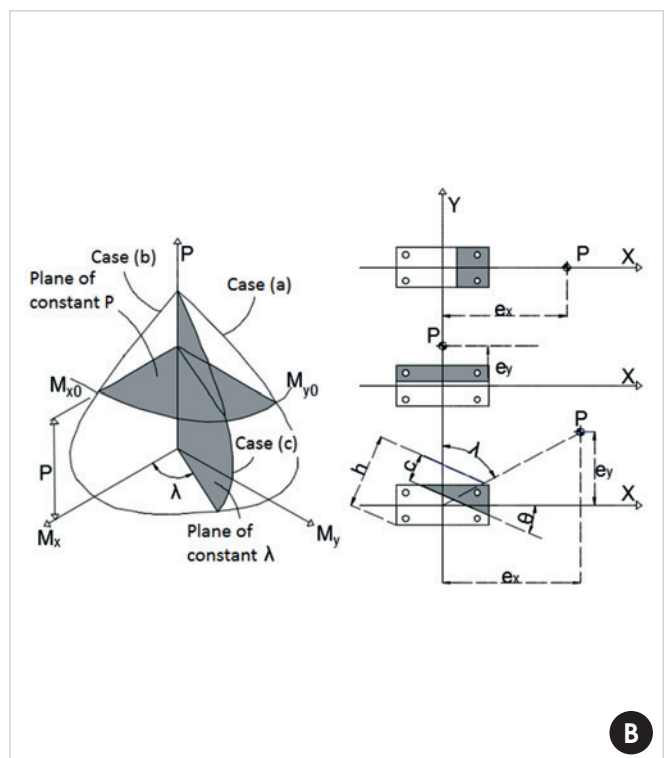$$\lambda = \arctan \frac{e_x}{e_y} = \arctan \frac{M_{nx}}{M_{ny'}} \qquad (6)$$

$$\left(\frac{M_{nx}}{M_{nx0}} \right)^2 + \left(\frac{M_{ny}}{M_{ny0}} \right)^2 = 1 \qquad (7)$$

Where:
$M_{nx} = P_n \cdot e_y$

$M_{nx0} = M_{nx}$ when $M_{ny} = 0$

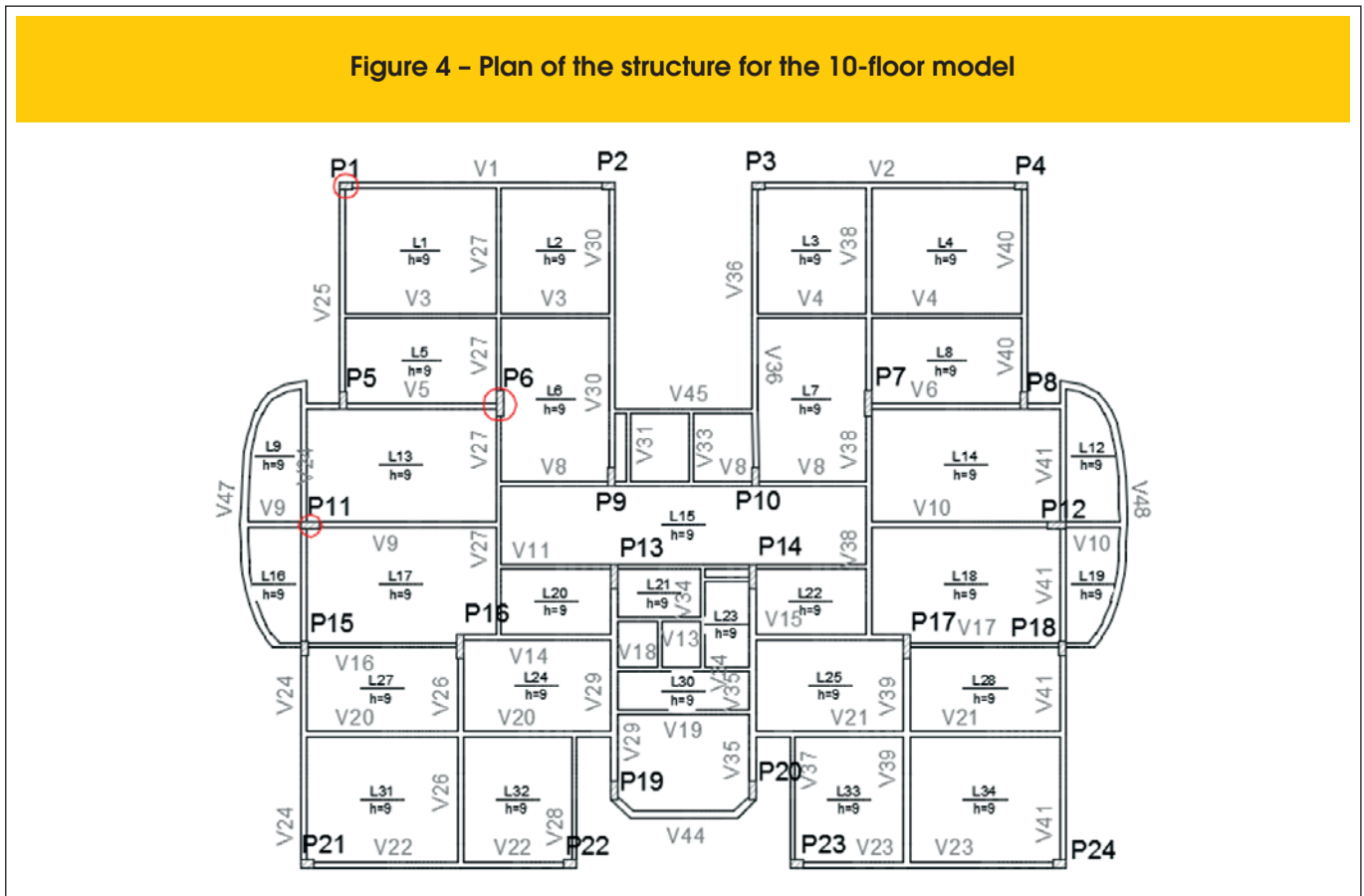**Figure 3 – Iteration diagrams for reinforced concrete columns**
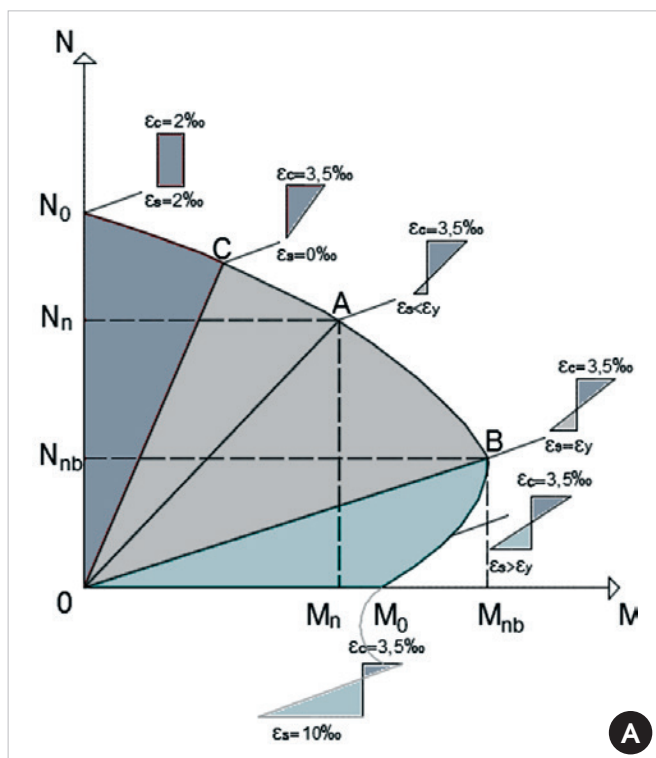
Iteration diagram for combined bending and axial load

Iteration diagram for biaxial bending

Figure 4 – Plan of the structure for the 10-floor model

$M_{ny} = P_n \cdot e_x$

$M_{ny0} = M_{ny}$ when $M_{nx} = 0$

3.   Parametric study

3.1   Computational modelling

The methodology consisted of, based on an architectural plan of a building's typical floor, designing the reinforced concrete structure and the foundations, entirely ignoring the wind action. This was done taking as variable the number of floors, generating spatial frames with 10, 20 and 30 floors, in order to highlight the relevance of the wind action in the design of reinforced concrete buildings. Later, the wind was considered in the computational models, following the recommendations from ABNT NBR 6123 [13], showing how the behaviour and the safety levels of both structure and foundations can be in risk if this horizontal action is not properly considered in the design stage. These analyses were done using the commercial software AltoQi Eberick V9.

In the analyses without wind, the dimensions of the columns were pre-designed considering their influence areas and assuming a constant vertical load on the floor. After this step, the structural design was regularly carried, yielding to the necessary dimensions and reinforcement ratios of the structural elements to safely support the vertical forces. Subsequently, the interaction diagrams of all columns were generated, based on sections and steel rein-

forcement found in the analyses without considering the wind. The structural models were then analysed again, now considering the wind as established by ABNT NBR 6123 [13]. From these analyses the combinations of actions in each of the columns were extracted, for the evaluation of the structural safety. Also, the horizontal displacements of the structure were analysed, in order to evaluate its performance in service, the γ_z coefficient, as an indicative parameter of instability, besides the increase of the 2nd order moments. Figures 4, 5 and 6 present the plans of the structures which were designed for models with 10, 20 and 30 floors, respectively, in the windless analyses. In these figures, columns selected for this article's presentation and discussion of results are highlighted with a red circle. A corner column (P1), a column near the floor's edge (P11) and an inner column (P6) were chosen. Tables 1, 2 and 3 present the dimensions of these columns' cross section and the steel reinforcement designed, for models with 10, 20 and 30 floors, respectively.

3.2   Criteria of ABNT NBR 6123 (1988)

The basic wind speed defined in ABNT NBR 6123 [13] as $v_0 = 30$ m/s to the city of Belem, Para. To calculate the drag coefficient, the flow regime was assumed as being of low turbulence and was calculated according to the Brazilian standard's abacuses knowing that the plan dimensions of the floor are 20.80 m x 19.20 m. The correction coefficients for characteristic speed were:

■ $S_1 = 1.00$, related to flat terrains;

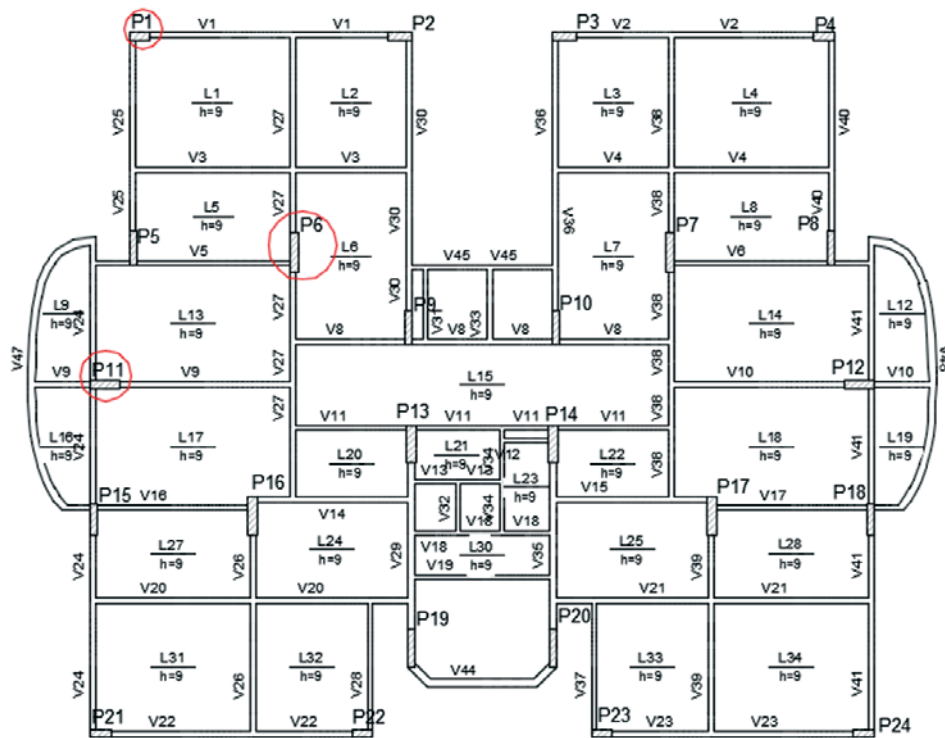Figure 5 – Plan of the structure for the 20-floor model

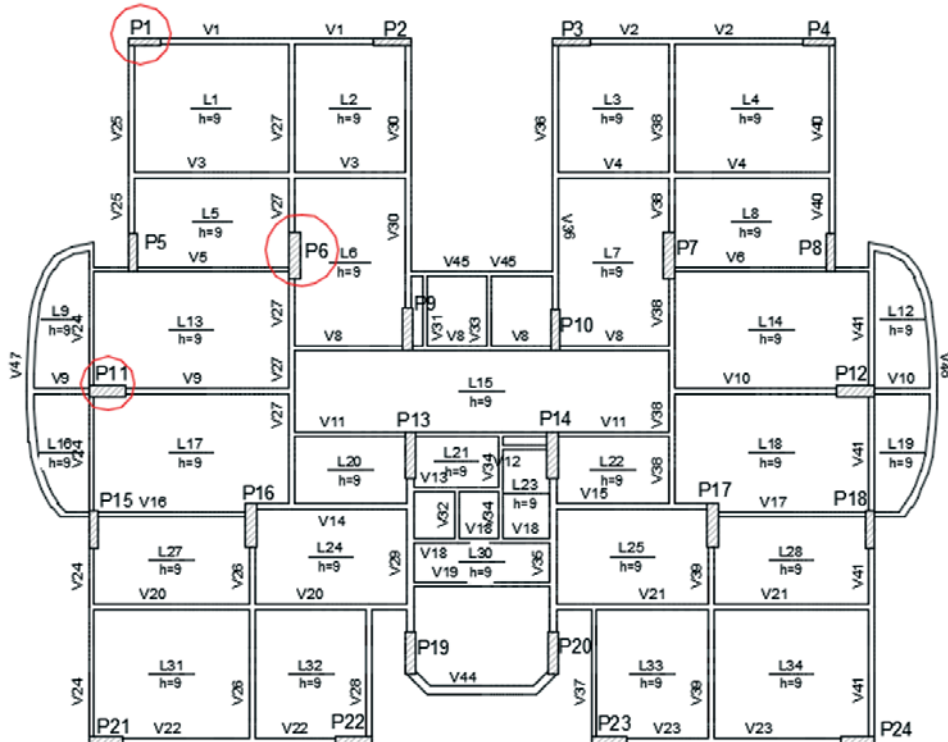Figure 6 – Plan of the structure for the 30-floor model

Table 1 – Dimensions and steel of columns studied for the 10-floor model				
Column	**Steel**	**As (cm²)**	**Section (cm)**	**ρ (%)**
P1	14 Ø 12,5	17,18	20 x 30	2,86
P6	16 Ø 20,0	50,27	20 x 70	3,59
P11	10 Ø 20,0	31,42	20 x 50	3,14

Table 2 – Dimensions and steel of columns studied for the 20-floor model				
Column	**Steel**	**As (cm²)**	**Section (cm)**	**ρ (%)**
P1	18 Ø 16,0	36,19	20 x 50	3,62
P6	18 Ø 25,0	88,36	25 x 100	3,53
P11	36 Ø 16,0	72,38	25 x 80	3,62

Table 3 – Dimensions and steel of columns studied for the 30-floor model				
Column	**Steel**	**As (cm²)**	**Section (cm)**	**ρ (%)**
P1	30 Ø 16,0	60,32	20 x 80	3,77
P6	26 Ø 25,0	127,36	30 x 120	3,54
P11	38 Ø 20,0	119,38	30 x 100	3,98

■ S_2 can be calculated by Equation 8. It is defined by the Brazilian standard as a function of the terrain's roughness category and of the building's dimensions. For the definition of parameters b, p, and Fr it was admitted that the terrain is category IV, characterized by being covered by numerous obstacles. For the 10-floor model, class B was adopted, and for the others, values corresponding to class C were used;

■ S_3 = 1.00.

$$S_2 = b \cdot F_r \cdot \left(\frac{z}{10} \right)^P \tag{8}$$

Where:
b, p and F_r are constants defined in the Brazilian wind standard;
z is the height aboveground.

3.3 Criteria of ABNT NBR 6118 (2014)

According to ABNT NBR 6118 [12], for the determination of the 2nd order overall forces, the physical nonlinearity of materials can be considered in a simplified way to reticulated structures with at least four floors, admitting to the calculation of structural elements the stiffness values presented in Equations 9, 10 and 11.
Slabs:

$$(EI)_{sec} = 0.3 \cdot E_c \cdot I_c \tag{9}$$

Beams:

$$(EI)_{sec} = 0.4 \cdot E_c \cdot I_c \quad \text{for} \quad A_s{'} \neq A_s \tag{10}$$

Columns:

$$(EI)_{sec} = 0.8 \cdot E_c \cdot I_c \tag{11}$$

Where:
$(EI)_{sec}$ is the stiffness of the element;
E_c is the elastic modulus of concrete;
I_c is the moment of inertia of the gross cross section.
For forces' determination, the combination of actions follows the formulas recommended by ABNT NBR 8681 [21], presented in Equation 12.

$$F_d = \sum_{i=1}^{m} \gamma_{gi} \cdot F_{Gi,k} + \gamma_q \left[F_{Q1,k} + \sum_{j=2}^{n} \psi_{0j} F_{Qj,k} \right] \tag{12}$$

Where:
F_d is the calculation value of the action,
γ_{gi} is the coefficient of permanent actions,
$F_{Gi,k}$ is the characteristic value of permanent actions,
γ_q is the weighting coefficient of variable actions,
$F_{Q1,k}$ is the characteristic value of the variable action considered as main action for the combination,
$\psi_{0j} F_{Qj,k}$ is the reduced value of combining each of the other variable

Figure 7 – Variation of gamma z coefficient

Figure 7 – Variation of gamma z coefficient
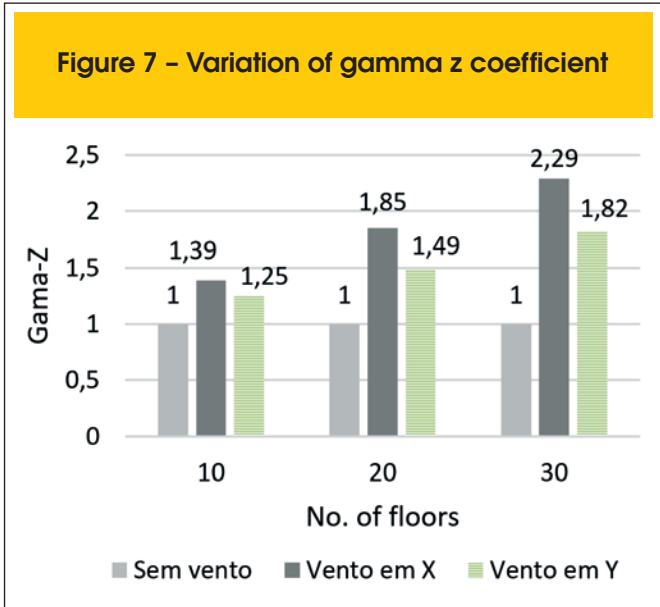
actions.

For columns' safety evaluation, it was considered that the design strength (R_d) should exceed the design load (S_d), as shown in Equation 13.

$$R_d \geq S_d \quad \therefore \quad \frac{R_k}{\gamma_m} \geq \gamma_f \cdot S_k \tag{13}$$

4. Results

4.1 Serviceability Limit State (SLS)

The serviceability limit states are related to people's comfort and to the durability and good working conditions of the structure, considering both users and machines installed in the building. ABNT NBR 6118 [12] recommends for concrete structures that the following serviceability limit states are checked: crack formation limit state; crack opening limit state; excessive deformations limit state; decompression limit state; excessive compression limit state; and excessive vibration limit state.

For this article, it was admitted that the excessive deformations limit state would be dominant in relation to the others and it was evaluated using as analysis parameters the global displacement of the structure and γ_z coefficient, which is used to evaluate the importance of 2nd order global effects. ABNT NBR 6118 [12] admits that in mobile nodes structures with $1.1 < \gamma_z \leq 1.3$ it can be used to approximately determinate the 2nd order global forces.

Figure 7 presents the γ_z variation in x and y directions in function of the number of floors. It is possible to see that ignoring the wind action would cause that even the smaller building, with only 10 floors, had coefficient γ_z of 1.39, which requires that the 2nd order effects are determined by P-Delta analysis method. In the most extreme case, of the 30-floor model, values of γ_z up to 2.29 were found, indicating that the 2nd order effects would be extremely high.

Figure 8 presents the variation of total displacements per floor considering the wind action in x and y directions. ABNT NBR 6118 [12] recommends that displacements are maintained below limit values, to avoid damage to non-structural elements, like masonry walls. It recommends that in case of horizontal displacements of buildings, they are maintained at values in centimetres below
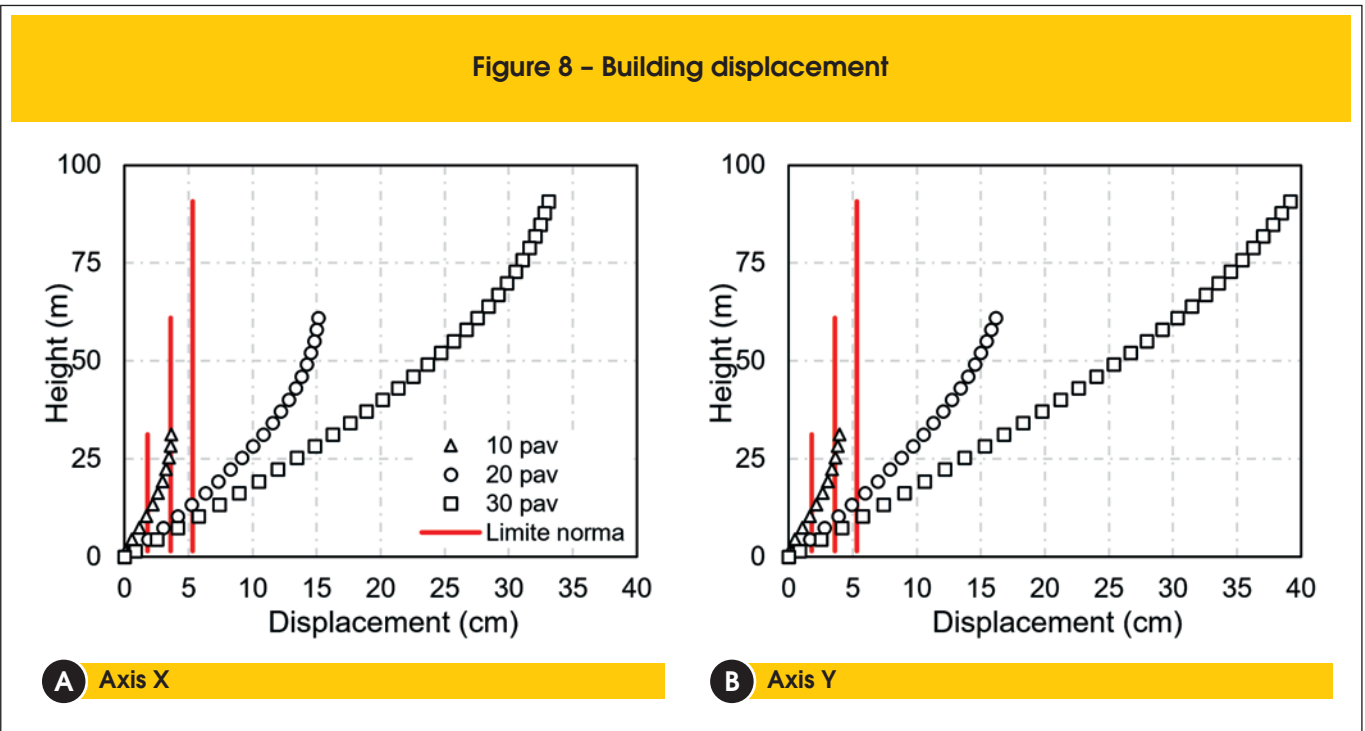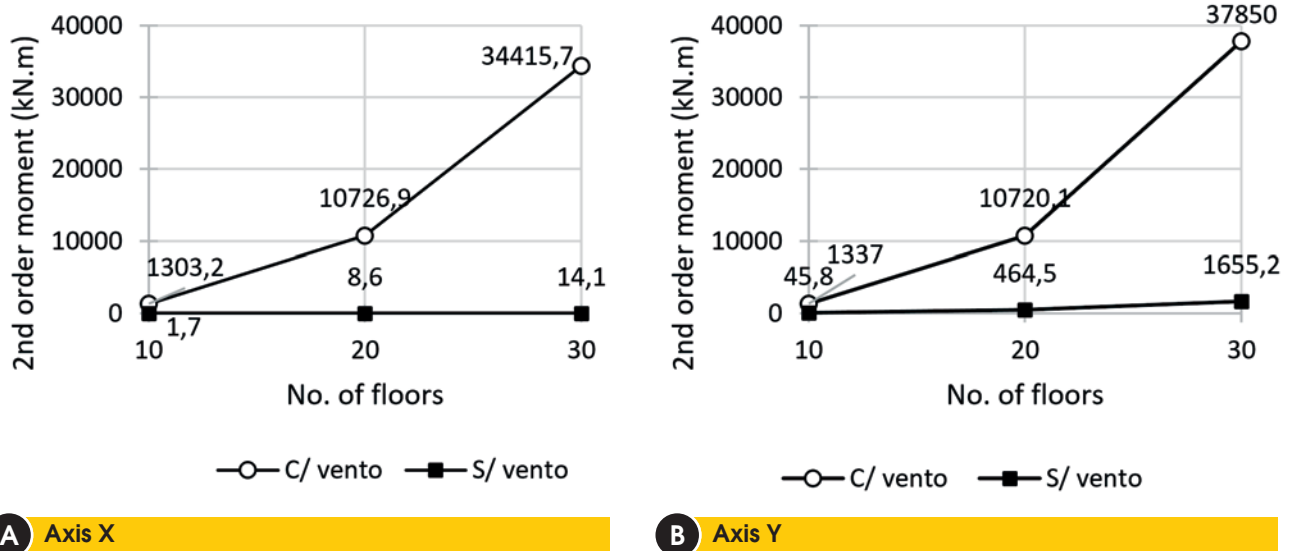
Figure 8 – Building displacement

Figure 8 – Building displacement

A Axis X

B Axis Y

Figure 9 – Variation of 2nd order moments

A — Axis X

B — Axis Y

H/1700, which would result in maximum displacements of 1.8 cm, 3.6 cm and 5.3 cm for the buildings with 10, 20 and 30 floors, respectively. It is possible to see that considering the wind action, the horizontal displacements would be significantly bigger than the maximum values recommended by the Brazilian standard.

4.2 Ultimate Limit State (ULS)

The ultimate limit state is related to the collapse or any other ruin of the structure that leads to the interruption of its use. ABNT NBR 6118 [12] recommends for concrete structures to check the following ultimate limit states: loss of static equilibrium; end of the structure's load capacity, whole or in part, due to axial and tangential forces, assuming the redistribution of internal forces; end of the structure's load capacity, whole or in part, considering the 2nd order effects; ultimate limit state caused by dynamic forces; ultimate limit state of progressive collapse; end of the structure's load capacity, whole or in part, considering exposure to fire; end of the structure's load capacity, considering seismic actions; other ultimate limit states that might occur in special cases.

In the analyses carried in this paper, it was considered that the wind would cause the most significant effects on the ultimate limit state of columns. This way, both the increase in 2nd order moments and the possibility of ending the load capacity of columns under combined biaxial bending were evaluated. Figure 9 shows the increase in 2nd order moments caused by the wind action in x and y directions of the building, as presented in Section 2.2, pointing out how important it is to correctly consider in design the actions caused by wind. Significant increases in 2nd order moments were observed, which can really jeopardize the structure's safety.

Figures 10, 11 and 12 show the load increases in foundations of columns P1, P6 and P11, respectively, caused by the wind action. For column P6, there were no significant changes in the 3 models

possibly because it is an internal column, thereby suffering less influence of the wind. However, for columns P1 and P11, in the 20-floor structure, for example, increases in vertical load of 31.5% and 16.7% were observed, respectively, when compared to cases with and without the wind action. And for the same situation, in the 30-floor building, increases of 75.3% in column P1 and 36.4% in column P11 were observed, which in practice could significantly jeopardize the safety level of foundations.

The structural safety level was evaluated in a simplified manner by checking the load capacity of the columns under biaxial bending.

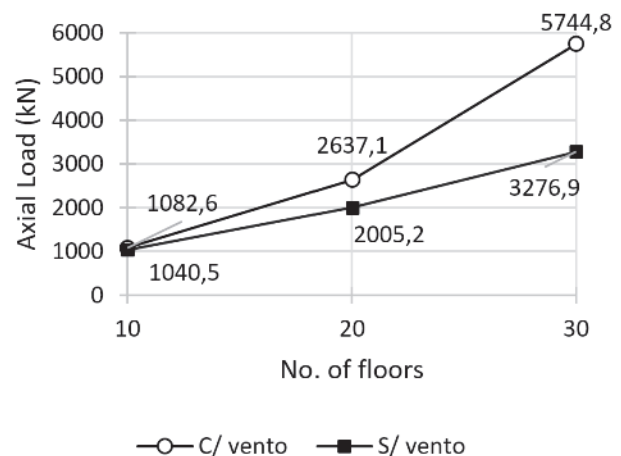**Figure 10 – Addition of load in the foundation of column P1**

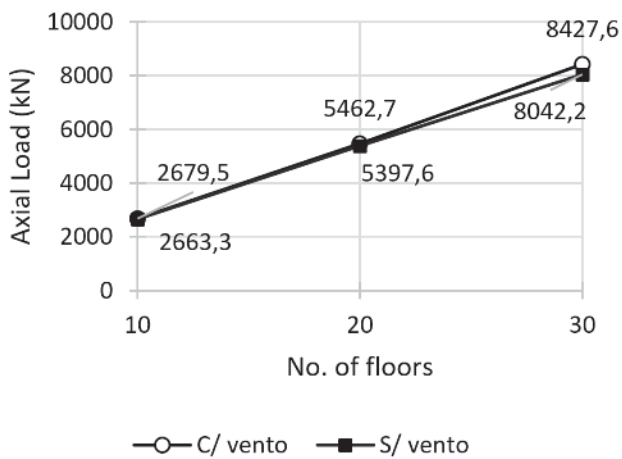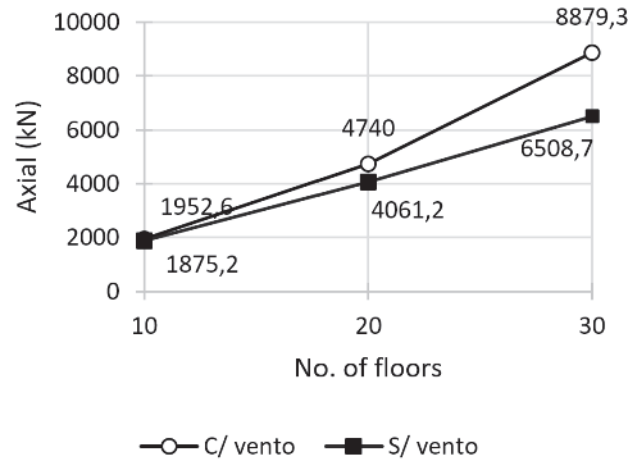Figure 11 – Addition of load in the foundation of column P6

Figure 12 – Addition of load in the foundation of column P11

Columns P1, P6 and P11 were selected to illustrate the global response and for these elements diagrams of iteration were generated, as illustrated in Figure 3. Figure 13 shows the resistance envelopment for P1 in the case of the 10-floor building. In this and in the other figures, the dashed line indicates the design envelop, considering the all safety coefficients and the solid line shows the strength considering the characteristic resistances of steel and of concrete. It is possible to see that for 4 combinations of actions the design load was bigger than the design resistance. This indicates that for these cases of load, the design safety level would not be satisfied.

In the case of the 20-floor building, the diagrams of columns P1 and

P11 are worth mentioning, which presented critical combination of actions, as show in Figures 14 and 15. In x direction of column P1, 4 combinations extrapolated the characteristic resistance of the column and other 2 the design one, whereas for the y direction only 1 combination was higher than the design strength. On the other hand, for column P11, only the y direction presented critical situations, with 4 combinations resulting in higher forces than the characteristic strength and 7 higher than the design one. This indicates that these columns would have a higher probability to ruin. For the 30-floor model, it was observed that the most critical situations would also occur for columns P1 and P11 (see Figures 16

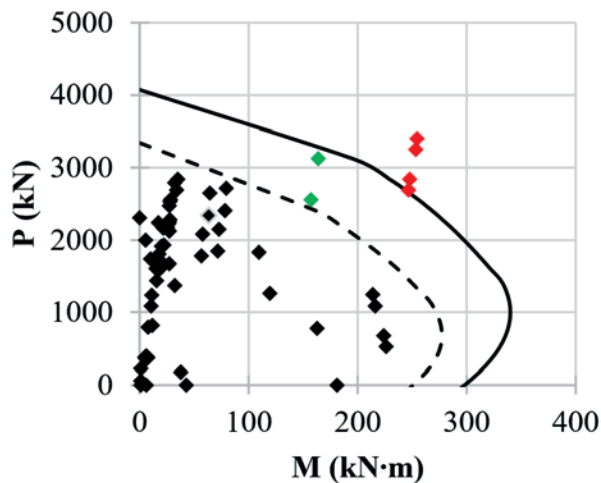Figure 13 – Interaction diagrams for column P1 of 10 floors
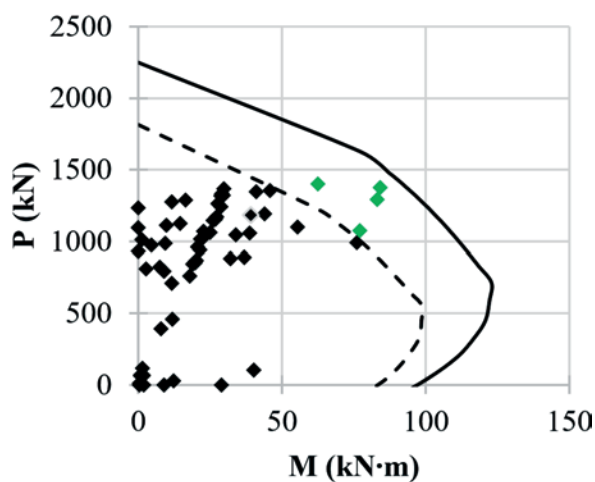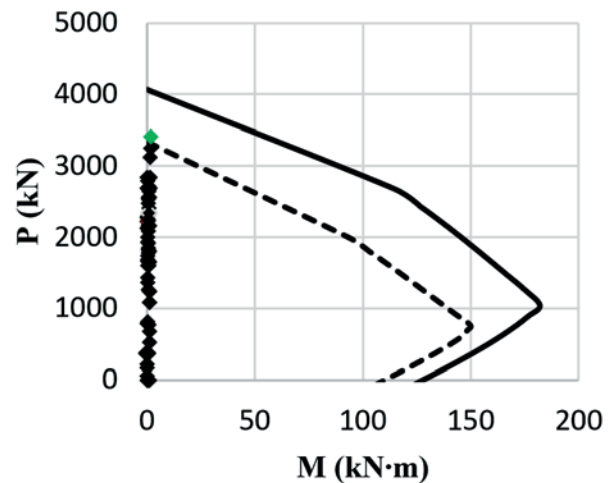
A Axis X
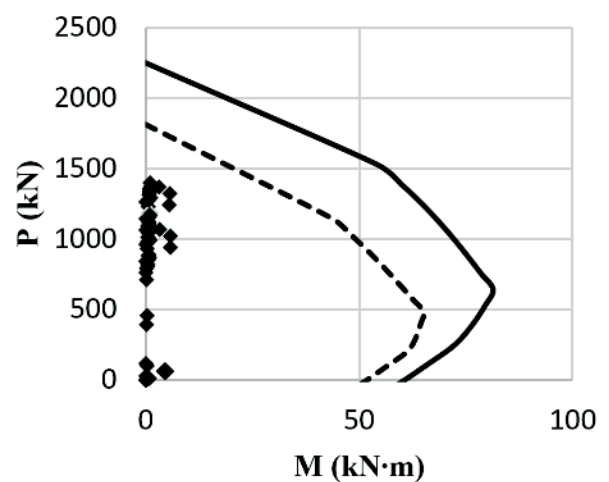
B Axis Y

Figure 14 – Interaction diagrams for column P1 of 20 floors
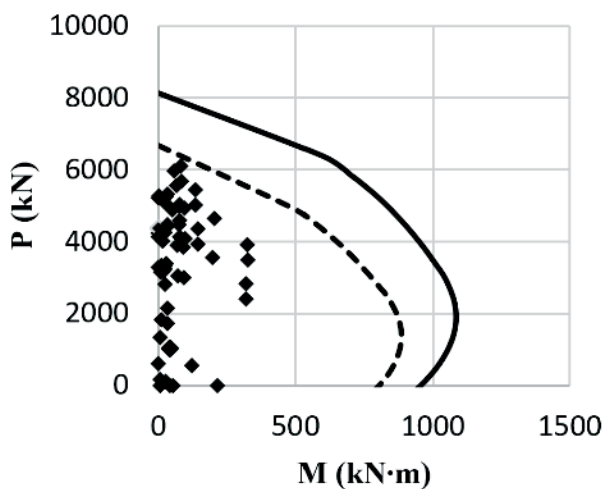
A   Axis X

B   Axis Y

and 18), while for column P6 (see Figure 17) in all models, the iteration diagram showed that the acting force level would be lower than the design strength. The analyses carried out for the 30-floor building showed that also in the case of columns P1 and P11 the probability of ruin would be significant. In these cases, even more seriously, it is possible to realize that many of the critical points

would be related to abrupt ruin modes, governed by crushing of concrete without cracking of the columns.

5. Conclusions

This study shows in an objective way how the wind action affects

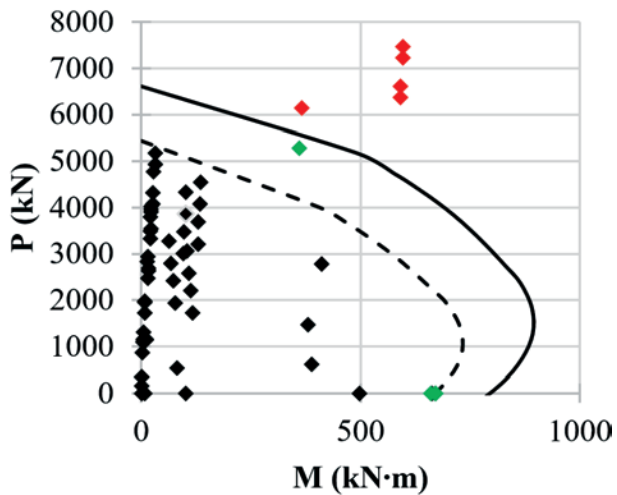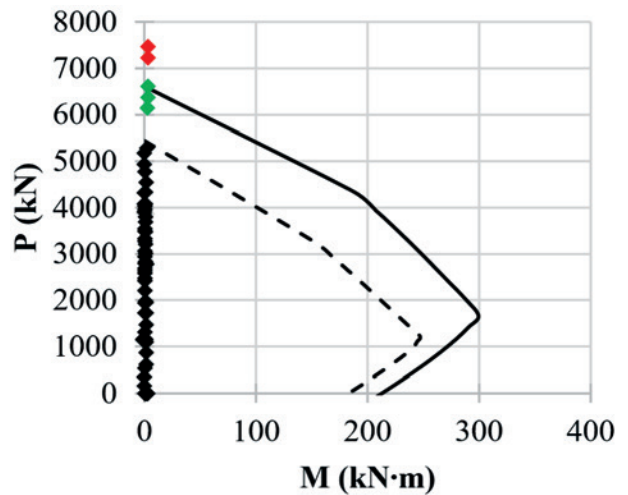**Figure 15 – Interaction diagrams for column P11 of 20 floors**

A   Axis X
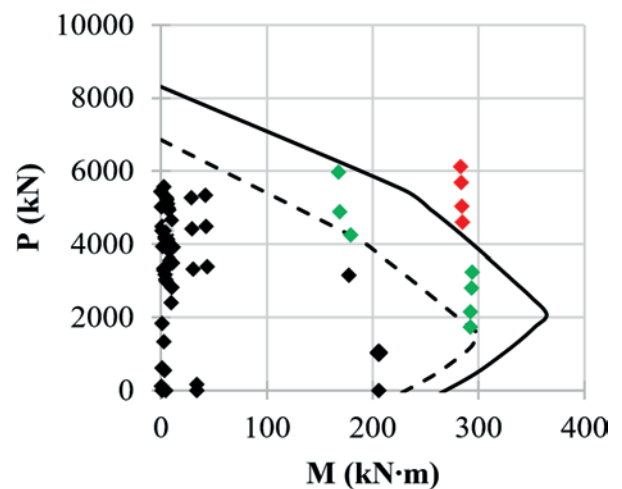
B   Axis Y

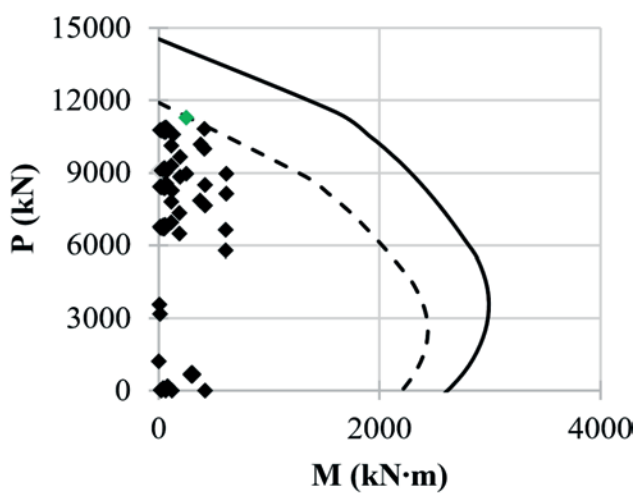Figure 16 – Interaction diagrams for column P1 of 30 floors

A — Axis X
B — Axis Y

the response and the forces in the columns and in the foundations of reinforced concrete buildings. To the Serviceability Limit State, it can be concluded that ignoring the wind action in design stage can significantly jeopardize the stiffness and the stability of the building, even in the case of short building with only 10 floors, where high horizontal displacements and expressive increase in

2nd order effects were observed. This could in practice jeopardize the comfort level of users, as Kwok *et al* [22] highlights, besides generating damage to non-structural elements, as masonry and window frames.

From the Ultimate Limit State point of view, there was expressive increase in foundations loads in edge and corner columns, resulting in

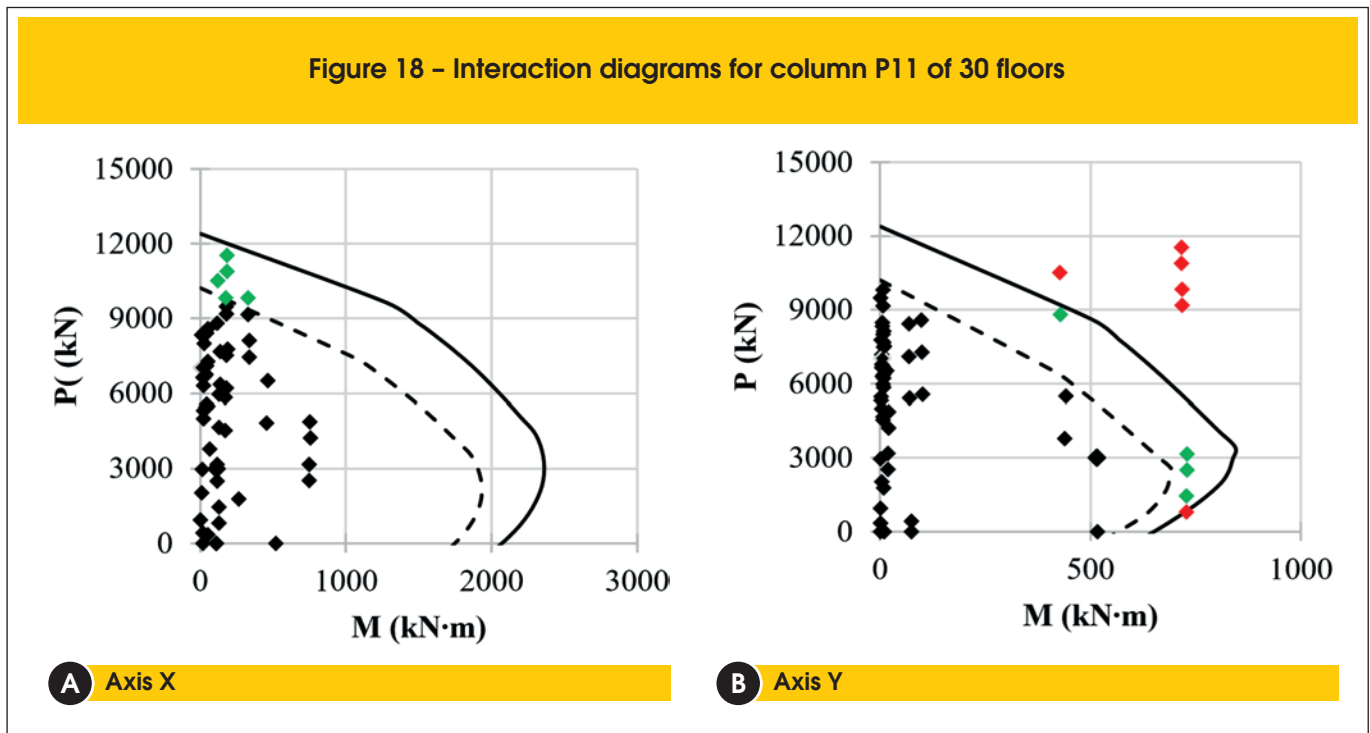Figure 17 – Interaction diagrams for column P6 of 30 floors
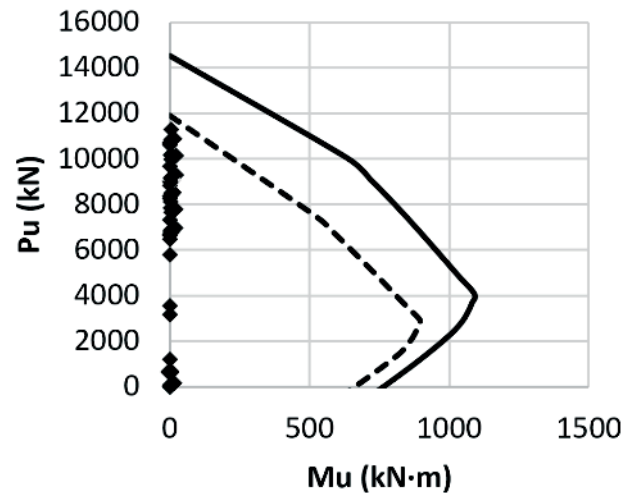
A — Axis X
B — Axis Y

Figure 18 – Interaction diagrams for column P11 of 30 floors

A — Axis X

B — Axis Y

unacceptable levels of ruin probability. In the case of columns from the 30-floor building, it must be pointed out that many of the critical combinations would be related to ruins with the column's section completely compressed, which would not generate any indication of risk to users. The analyses presented in this paper ignore some beneficial effects, such as the increase of stiffness caused by bracing generated by masonry. Still, these results serve as warning to all Brazilian technical community about the importance of wind action consideration in concrete structures design.

6.   Acknowledgments

7.   Bibliographic references

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