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The collapse of Real Class building

O colapso do edifício Real Class

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Abstract

This paper describes the scene of the Real Class building collapse, a residential building with reinforced concrete structural system and located in the urban area of the city of Belem / PA, occurred in 2011. The unconformities found in the building are displayed using data extracted from reports and verification of structural and architectural designs. The data was compared with the Brazilian code for reinforced concrete structures, NBR 6118 (2007), valid at the time of the accident. The security of the building was evaluated through a computer model with linear analysis with the software used by the designer. The conditions of the structural system designed and as built was evaluated with parameters of global stability and load capacities of columns and foundations. The results showed that the structure of the building was subject to large displacements and the sections of columns were unable to resist the stresses produced by regional wind actions.

Keywords: reinforced concrete structures, strctural safety, global stability, collumns.

Resumo

Neste texto são descritos o cenário do colapso do edifício Real Class, edifício residencial com sistema estrutural de concreto armado, situado na zona urbana da cidade de Belém/PA, ocorrido em 2011. As inconformidades encontradas no edifício são exibidas através de dados extraídos dos laudos e da verificação dos projetos estrutural e arquitetônico perante a norma de estruturas de concreto armado, NBR 6118 (2007), vigente na época do acidente. A segurança do edifício foi avaliada através da modelagem computacional com análise linear do edifício pelo software utilizado pelo projetista. Foram analisadas as condições do projeto e da estrutura executada diante da estabilidade global, capacidade de carga das fundações e resistência dos pilares. Os resultados obtidos demonstraram que a estrutura do edifício estava sujeita a grandes deslocamentos e as seções dos pilares projetadas eram incapazes de resistir às solicitações das ações de vento incidentes na região da edificação.

Palavras-chave: estruturas de concreto armado, segurança estrutural, estabilidade global, pilares.

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1. Introduction

Concrete structures are the most built in Brazil, being used both in small and tall buildings with residential destination, because of the facility that this structural system presents in its construction without great technical investment in its execution. Tall buildings have been used as lucrative solution in the occupation of large urban centers suffering from lack of habitable areas, exaggerated growth rate and high population density. The possibility of constructing tall and slender structures using this constructive system resides in the possibility of using larger resistances for the structural elements and in the facilities brought by the use of computational structural models.

According to CTBHU (Council on Tall Buildings and Urban Habitat), a building is classified as tall if the number of floors is greater than 14 or its height exceeds 50 m. Slenderness (relationship between height and smallest dimension in the building plan) can also be considered as impacting since structures with this coefficient greater than 6 may present great dynamic sensitivity [1]. The structural modeling of tall buildings requires the evaluation of effects that are generally neglected in smaller structures such as: global stability, wind actions and soil structure interaction [2]. The feasibility of the execution of slender buildings is also linked to the adoption of specific structural systems, being very important the evaluation of the impact of wind, as this increases in proportion to the height of the building [3]. The adoption of the rigid frame system with beams and columns is recommended for buildings up to 25 floors, and the rigid frame system associated with shear walls is recommended for edifices of up to 40 floors [1], the most used systems in Brazilian buildings.

Although collapses of tall buildings are rare, when they occur can cause great commotion by impacting large numbers of people socially and economically. Examples include the collapse of the buildings Raimundo de Farias (Belem - 1987), Palace II (Rio de Janeiro - 1998), Areia Branca (Recife - 2004), Liberdade (Rio de Janeiro - 2012) Real Class, occurred in the city of Belem in 2011. These accidents assessments are hampered by the fact that the structures had reached the ultimate limit state and, in some cases, had very poor documentation and records [4].

Figure 1
Real Class building

2. Description of the building and scenario of collapse

The residential building Real Class (Figure 1) was located in an urban area in the central part of Belem city. The description for the region of the building, according to the code regarding the wind speedy [5], is of a flat urban terrain with vertical obstacles not exceeding 10 m. The building had 35 floors distributed in four areas of common use (underground, ground, level 1 and level 2), 30 pattern floors and a roof, totaling a height of 104.7 m, with maximum values of 26.8 m in length and 13.9 m wide in plan, resulting in a projection area of 298.4 m².

The collapse of the building occurred around 13:45 p.m. on January 29, 2011, under a tropical storm with wind gusts of 39.4 m/s [6] in some parts of the city, although the disclosed measurement data only reported the maximum value of 10 m/s [7]. The debris reached the building area and were projected for neighboring buildings (Figure 2) and part of a busy street. This accident, whose consequences were not only worse due to the time of its occurrence, killed three people (two workers and one resident of a house adjacent to the building), generating doubts in the population about the quality of the structures produced in the state of Para.

The structural system adopted in the design of the building was the one of reinforced concrete rigid frame system with conventional slabs. The dimensions of peripheral beams were 120 mm x 650 mm and the internal beams dimensions were 120 mm x 500 mm, with these dimensions varying for other floors of the structure. The dimensions of the building elements are shown in the plan view of the pattern pavement (Figure 3) and cross section views (Figure 4 and Figure 5). The foundations were of spread footings over pile caps with 600 mm in diameter and 9 m in length. The concrete was

Figure 2
House damaged by the Real Class building debris

specified with compressive strength of 35 MPa for the superstructure and 20 MPa for the foundations.
Two committees were established to assess the causes of the accident and to report to society: one of a technical-scientific nature, represented by the Structures and Materials Analysis Group (GAEMA) of the Federal University of Para [8], and another established

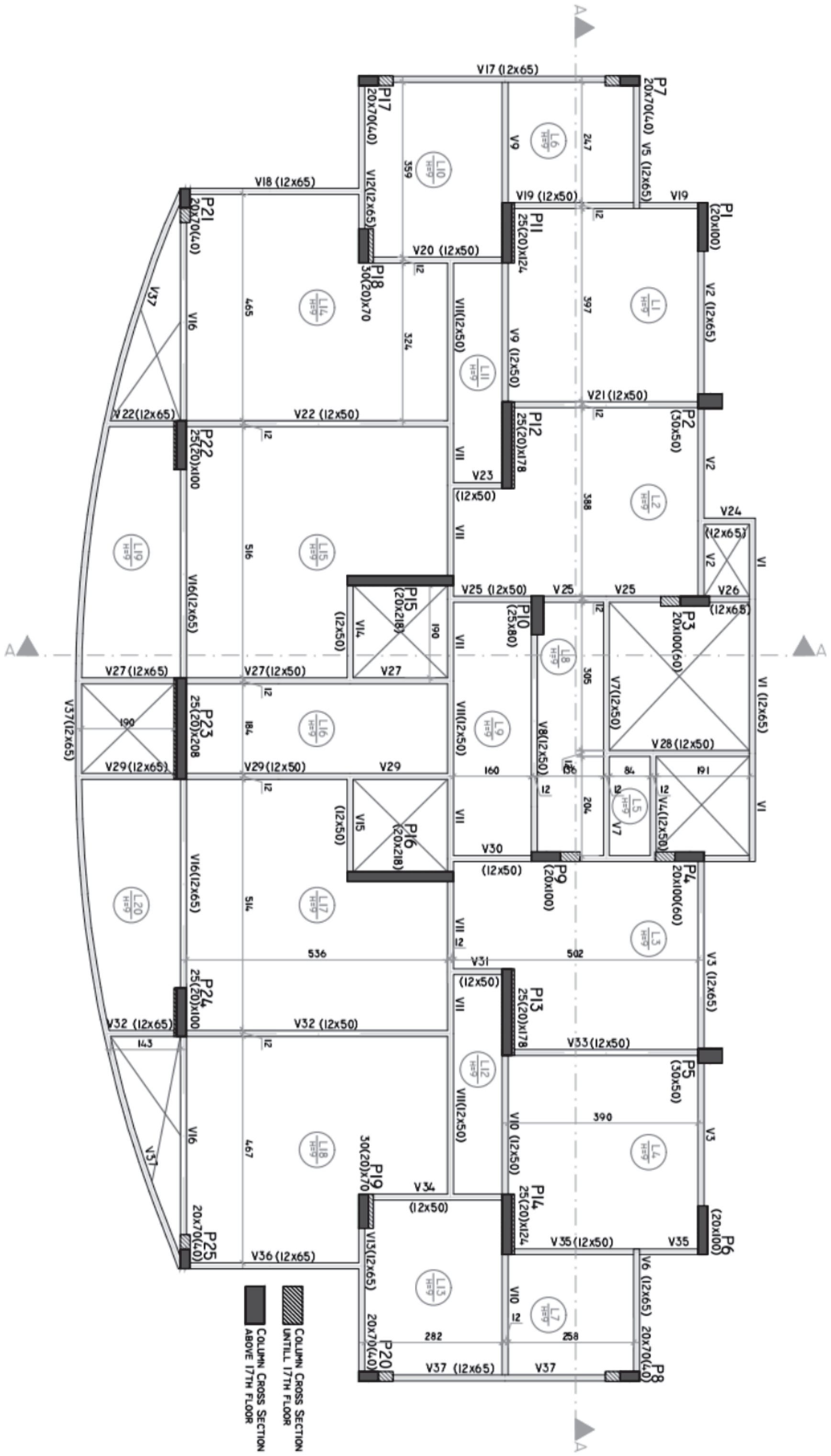


Figure 3
Plan view of floors 01 - 30

by the Para State government through the Renato Chaves Institute (IRC) [9]. The reports of the two committees were based on the analysis of the materials of the structure, as well as the architectural, structural and foundations designs.

The two committees have adopted similar strategies for assessing the collapse. The IRC expert team used the verification of the compliance of the foundations and structural design, in face of the current code prescriptions and laboratory tests of the materials used in the structure [9]. In addition to the previously mentioned approach, GAEMA used the computational modeling of the building to verify undeclared or non-detectable characteristics in a preliminary analysis of structural design, both for ultimate limit state (ULS) and the service limit state (SLS) [8].

2.1 General design unconformities from reports

The Brazilian code for reinforced concrete structures [10] recommended for the region of construction of the building a concrete cover value of 20 mm for slabs and beams and 25 mm for columns. These values were not fulfilled in the design, once the value of 15 mm for all structural elements was used. Concrete cover values, according to the recommendation of the same design code, would limit the maximum diameter used in the longitudinal reinforcement of the elements by 12.5 mm. Instead there were reinforcement bars with 16.0 and 20.0 mm diameter in the beams of the underground floor, engine room, roof elements and columns of the first five stories of the structure mainly. The stirrups adopted in the structural design (φ_s = 4.2 mm) had a diameter lower than the design code recommended (φ_s = 5.0 mm), as well as the transverse reinforcement ratio, with 78% of the elements presenting spacing values higher than the maximum allowed.

The values of the maximum longitudinal reinforcement ratio of the columns were incompatible with the design code recommendations. According to Figures 6, 7 and 8, the maximum reinforcement rate values exceeded the code value of 8% of the cross sections area of the columns P03, P04, P10, P22 and P24. Also, according to the report [9], the longitudinal reinforcement presented discrepancies between the structural design and the *as built*. The difference between the designed steel area and the executed is shown in Table 1, in which the columns P04, P10 and P21 exceeded 30%. According to the results presented by the two reports, the steel used in the building proved to be suitable, since tests indicated a

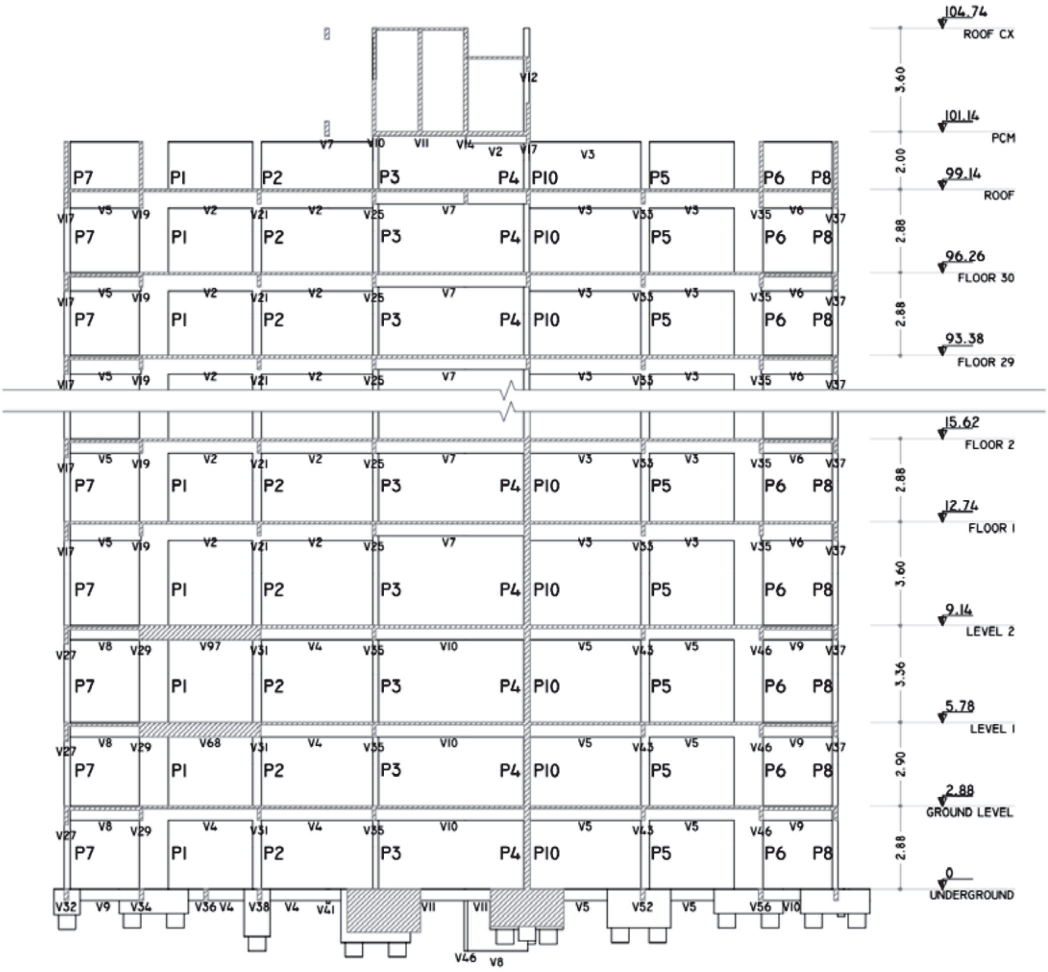


Figure 4
AA cross section view

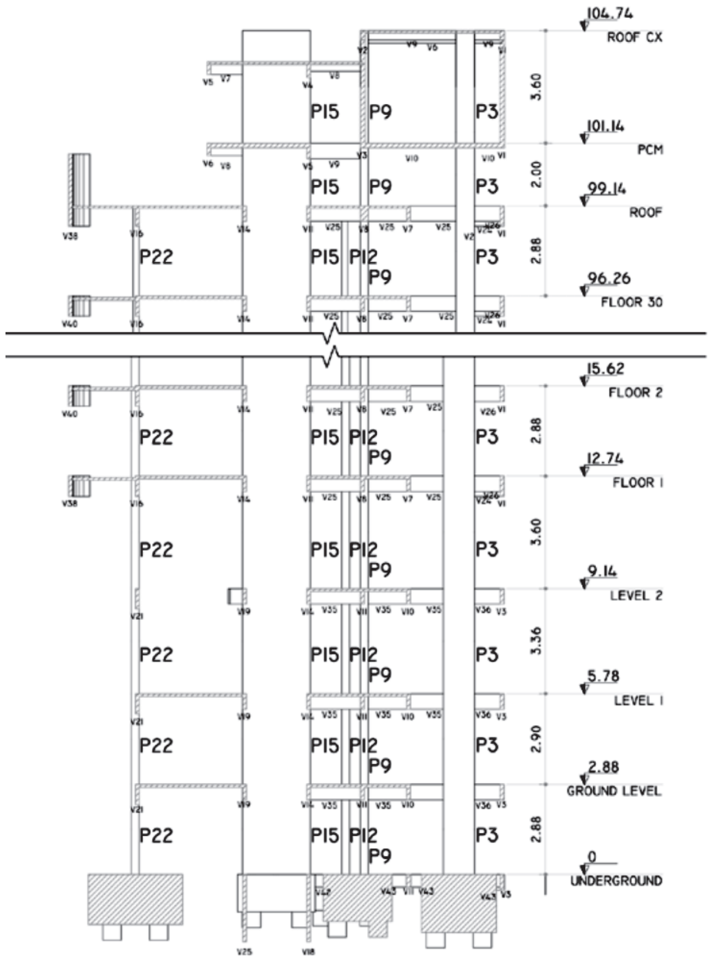


Figure 5
AA cross section view

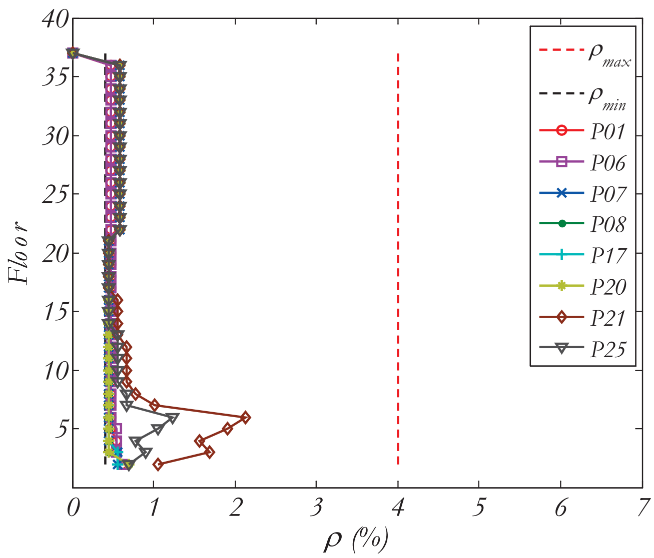


Figure 6
Corner columns reinforcement ratios

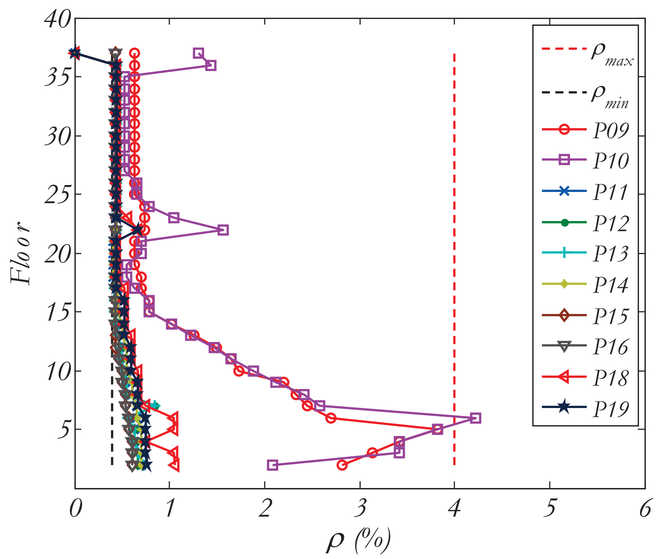


Figure 7
Internal columns reinforcement ratios

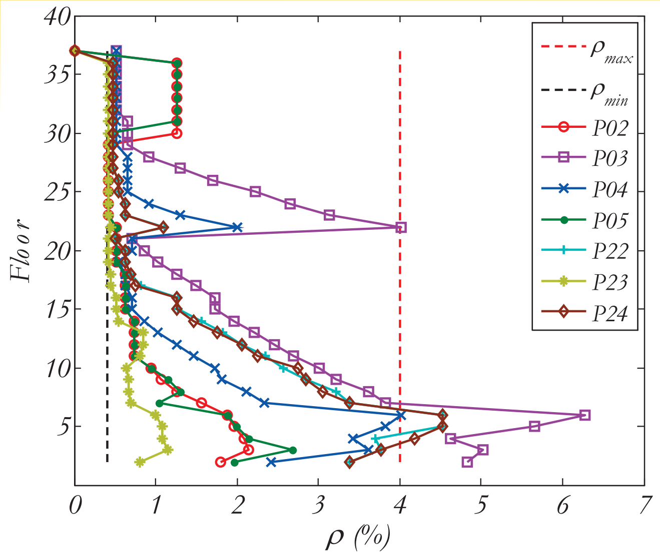


Figure 8
Peripheral columns reinforcement ratios

yield stress above 500 MPa for 10 mm CA-50 steel samples taken from the columns. The specific code for characterization of this material [11] considers the yield stress referring to a strain of 2 ‰ and the minimum tensile stress of 1.08 f_y.

The concrete compressive strength (f_c) was differently checked in each report through sclerometry tests and extraction of proofs. Nine sclerometry tests were performed randomly on non-damaged elements length, three from slabs, three from beams and three from columns, with average results of 42 MPa, 40 MPa and 45 MPa, respectively [8]. In the tests with extracted proofs the elements were identified and the respective resistances were found [9]. Table 2 summarizes the test results on the concrete, which was considered satisfactory.

2.2 Structure and foundations assessment

Computational modeling using the *software* CAD TQS, a software widely used in Brazil in the design of reinforced concrete structures, was carried out by GAEMA to assess the structural behavior. This team made use of the rigid frame system with all floors modeled using grid analogy and according to the design code prescribed loads. The wind speed adopted in the model was 32 m/s, i.e. the

Table 1
Designed and built columns' reinforcements [9]

Column	As built		Design		Difference	
	Quantity	Diameter (mm)	Quantity	Diameter (mm)	Area (mm²)	ρ (%)
P 01	10	12.5	14	10.0	-128	-12%
P 02	22	12.5	16	16.0	517	16%
P 03	48	16.0	50	16.0	402	4%
P 04	24	16.0	36	16.0	2413	33%
P 05	24	12.5	18	16.0	674	19%
P 06	10	12.5	14	10.0	-128	-12%
P 07	10	10.0	10	10.0	0	0%
P 08	8	12.5	8	10.0	-353	-56%
P 09	28	16.0	20	20.0	653	10%
P 10	34	12.5	34	16.0	2664	39%
P 11	18	12.5	28	10.0	-10	0%
P 12	38	10.0	38	10.0	0	0%
P 13	16	16.0	38	10.0	-232	-8%
P 14	18	12.5	28	10.0	-10	0%
P 15	34	10.0	34	10.0	0	0%
P 16	34	10.0	34	10.0	0	0%
P 17	10	10.0	10	10.0	0	0%
P 18	18	12.5	28	10.0	-10	0%
P 19	8	16.0	20	10.0	-38	-2%
P 20	8	12.5	8	10.0	-353	-56%
P 21	12	12.5	30	10.0	884	38%
P 22	42	16.0	30	20.0	980	10%
P 23	34	12.5	76	10.0	1797	30%
P 24	42	16.0	34	20.0	2237	21%
P 25	8	12.5	16	10.0	275	22%

average speed prescribed for the area in which the city is located, according to the specific code for wind action on structures [8].
The results obtained by the GAEMA team demonstrated that the building had large displacements for both the ULS and the SLS. Under a second order analysis, the structure was considered flexible. According to this technical report, only 4 of the 25 columns presented geometric characteristics that could be dimensioned according to the design code requirements, and the reinforcement was found to be significantly superior [8]. The columns responsible for the collapse are shown in Figure 9, with the columns P16, P17 and P22 showing instability under the action of wind at 32 m/s [8]. However, the P04, P07 and P08 columns would collapse under the action of the wind 23 m/s [8]. From these results, the report of GAEMA concludes that the building was poorly designed without considering the actions of the wind, exposing it to loads of intensity greater than those predicted in the design.

Table 2
Concrete's mechanical properties

Element	f_{cd} (MPa)	f_c (MPa)	Report
P 06	35	30	IRC
P 11	35	37	
P 15	35	41	
P 16	35	29	
Pile cap P13	20	26	
Retaining walls	20	31	
Columns	35	45	GAEMA
Beams	35	40	
Slabs	35	42	

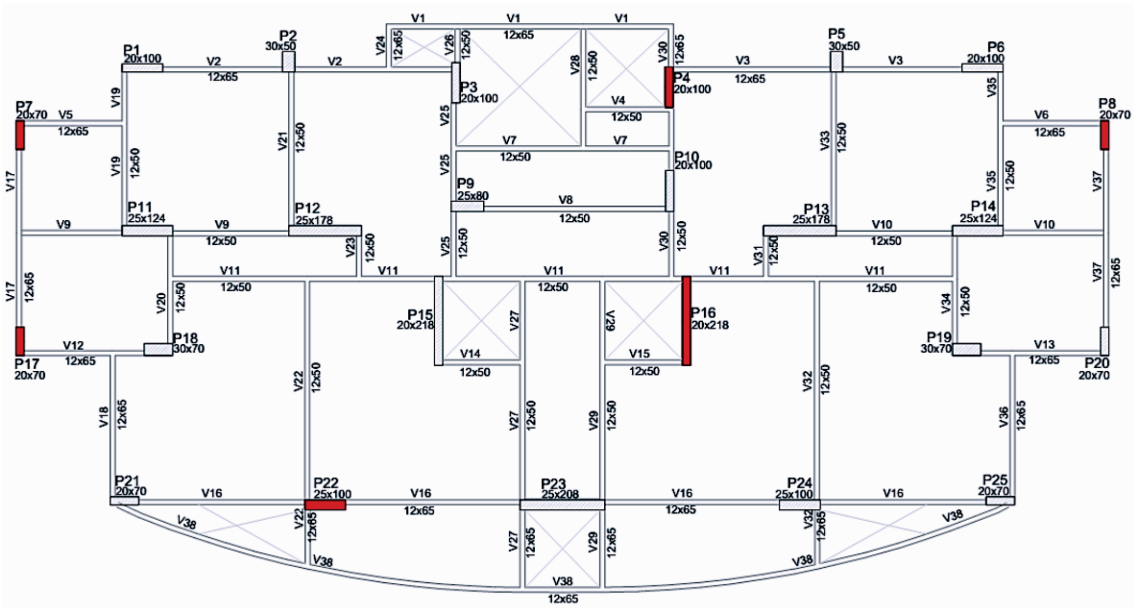


Figure 9
Supposed columns responsible for collapse

The foundations design was considered adequate to the loads declared in the executed structural design, as well as the detailing of the spread footings and the pile caps. It was verified that, under the collapse scenario, the stresses were lower than the characteristic resistant capacity of the spread footings and the pile caps, which led the report to discard the possibility of foundations failure in the moment of collapse, with P17 column being the worst case with maximum lading corresponding to 57% of its design resistance. Visual inspections reported by the IRC also did not mentioned foundations damages [9].

2.3 Hypotheses for current computational model and building reliability assessment

To develop the current analysis the calculation and dimensioning software for reinforced concrete structures AltoQI Eberick v.6 was used. This software is also widely used in design offices in Brazil and was used by the building designer. The software performs a second-order linear static analysis for the evaluation of the loads and dimensioning of the structural elements (Figure 10), and the second order effects are calculated through *P-Δ* process.

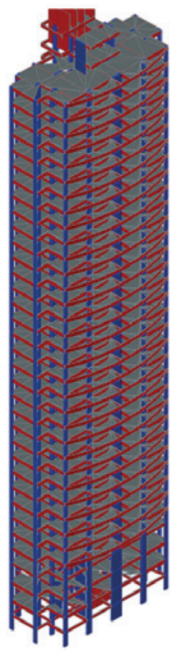


Figure 10
Real Class building computational model

Table 3
Loads adopted in the computational model

Ambient	NBR 6120 [12] Load (kN/m²)
Bedrooms, living rooms, kitchens and bathrooms	1.50
Storerooms, service areas and laundries	2.00
Ceilings without access to people	0.50
Stairs without access to people	2.50
Corridors with access to people	2.00
Garages	3.00
Balconies without access to people	2.00
NBR 6123 [5]	
Velocity of gust (m/s)	30
Drag coefficient	Low turbulence

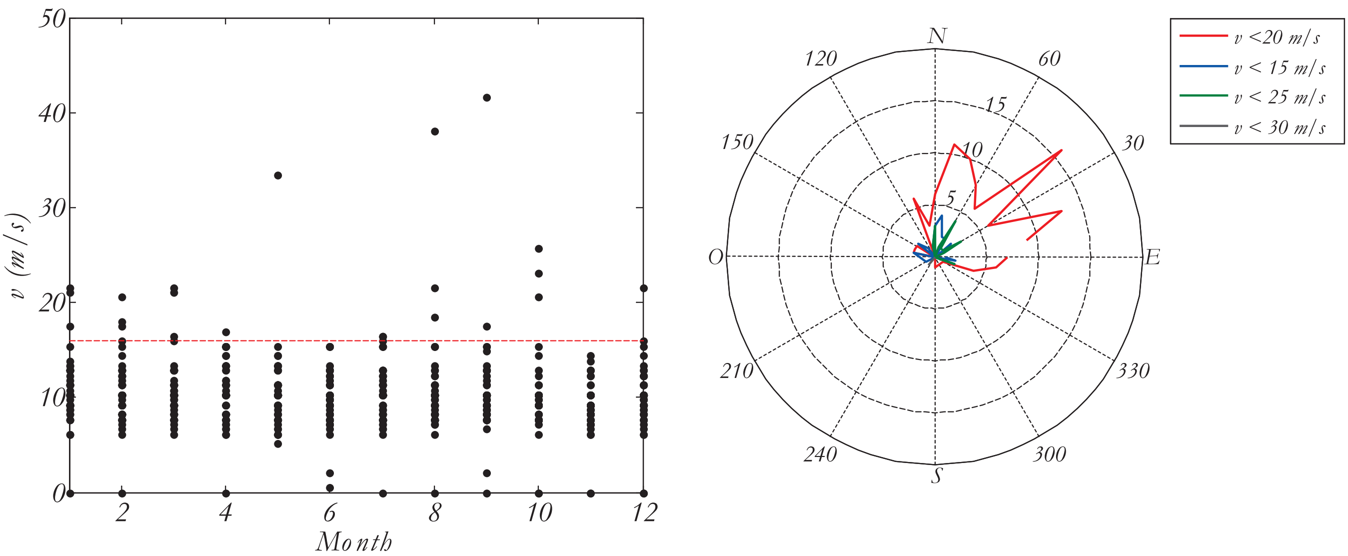
The effects of physical non-linearity were taken account through simplifications suggested by the Brazilian design code for reinforced concrete structures with stiffness reduction for structural elements. The slabs were modeled considering the grid analogy and beams and columns as frame elements.
For the steel constitutive properties, the model of NBR 6118 [10] considering different strain limits for compression (ε_{su} = 2,0 ‰) and tension (ε_{su} = 10,0 ‰) was used. The steel yielding stress was taken from GAEMA technical report [8] because the 10.0 mm diameter of the sample was representative of the reinforcement bars from the original design. The concrete used regionally in this type of building is a normal weight concrete with compressive resistance ranging from 25 to 35 MPa. The concretes constitutive model and its modulus of elasticity were those prescribed by NBR 6118 [10]. The compressive resistance *(fc)* adopted by the designer was 35 MPa and the value found by the IRC expertise at the time of collapse was 29 MPa [9].

2.4 Vertical loads

The formulation of Brazilian code design for reinforced concrete structures considers normal, construction and exceptional combinations for evaluation of a possible occurrence of an ELU (Equation 1). The design loads (F_d) in the structure are defined according to the combinations of actions, whose load enhancement coefficients (γ) and actions simultaneity (ψ) are normatively specified [10]. The gravitational actions (F_g) used in the models followed the design code recommendations [12], as well as the occupation loads (Table 3), with the actual weight of the reinforced concrete considered as 25 kN/m³ [10]. The loads of indirect actions such as retraction ($F_{\varepsilon gk}$) and temperature ($F_{\varepsilon qk}$) were not considered.

$$F_d = \gamma_g \cdot F_{gk} + \gamma_{\varepsilon gk} \cdot F_{\varepsilon gk} + \gamma_q \cdot \left(F_{q1k} + \Sigma\psi_{0j} \cdot F_{qik}\right) + \gamma_{\varepsilon q} \cdot \psi_{0\varepsilon} \cdot F_{\varepsilon qk} \tag{1}$$

For the ULS, only the results regarding the of the building's columns design were initially analyzed. Such approach was adopted due to the fact of these elements be fundamental to keep the stability of the building, and from witness reports that the



A Wind gust speed in the period 1951 - 2011

B Direction of wind gusts in the period 2000 - 2011

Figure 11
Characterization of wind gust velocity in Belem city (ICEA)

building collapse was abrupt, without any structural element punctual failure. Aiming to estimate the loadings responsible for the occurrence of the ULS, 61 combinations were generated to check the conformity of the structural design (*software* standard configuration) and 23 construction combinations to evaluate the structure at the moment of collapse distributed in favorable situations (γ_g = 1.00) and unfavorable (γ_g = 1.30).

2.5 Wind load

In Brazil, the main horizontal action is the result of wind gusts, whose design code prescriptions adopt two approaches: static and dynamic. In this work, the equivalent static force approach was used, whose model prescribed by code [5] is equivalent to a force produced by a 3-second wind gust of basic velocity (v_0) that is likely to be exceeded once in 50 years. Wind gust velocity is measured at a height of 10 m above ground in open and flat areas. The mathematical formulation also adopts modifiers according to the type of terrain (s_1), its slope and the type / use of the building (s_2), as well as the probability of occurrence of the gust wind and importance of the structure (s_3).
A velocity of 30 m/s was used to determine the design loads, as recommended by the wind design code for the building region [5]. To estimate the velocity at which the collapse occurred, the value for which the *software* did not dimension the columns cross sections was checked and this value (25 m/s) was used as the upper limit. The velocity initially used as the lower interval for analysis was recorded by the local meteorological service (weather data bank of the Air Space Control Institute) at the time of the accident (v_0 = 10 m/s). According to the data (Figure 11), the average speed of the gust wind is 12 m/s and the average of the annual maximum values corresponds to 17 m/s.

2.6 Displacements and second order effects

The second order effects were calculated by the *P-Δ* process. For better representation of the ULS, the design code uses the reduction of the stiffness of the elements $E \cdot I = 0,4 \cdot E_{ci} \cdot I_c$ for the beams, $E \cdot I = 0,3 \cdot E_{ci} \cdot I_c$ for the slabs and $E \cdot I = 0,8 \cdot E_{ci} \cdot I_c$ for the columns which, according to Oliveira [13], are satisfactory, where E_{ci} is the initial tangential modulus of elasticity of concrete and I_c is the moment of inertia of the concrete section. The limit displacements (δ_{lim}) considered in the analysis were those, which according to code design indication, cause effects on non-structural elements due to the lateral movement of the building (Equation 2). This is due to the total height of the building (H).

$$\delta_{lim} = \frac{H}{1700} \tag{2}$$

For the evaluation of the second order effects, the coefficient γ_z (Equation 3) was used. This coefficient is widely used in the design offices of the country. This coefficient correlates the effects of the moments produced by all the gravitational forces ($\Delta M_{tot,d}$) with the moment produced by all the horizontal forces ($M_{1tot,d}$) in a first order analysis.

$$\gamma_z = \frac{1}{1 - \frac{\Delta M_{tot,d}}{M_{1,tot,d}}} \tag{3}$$

2.7 Interaction diagram

The safety of the columns was assessed through interaction diagram constructed using the assumptions of strain compatibility with concrete and steel with limitation of concrete strain (ε_{cu}) at 3,5 ‰ and steel strains (ε_{su}) at 10 ‰, according to the recommendation of NBR 6118 [10]. Figure 12 shows the simplifications for the calculation of the cross sections strength and for compression stress rectangle. The construction of such diagram can be seen in the literature [14], and admitting the height of the compression rectangle *(a)* equivalent to 0.80 (β_1) of the height of the neutral line (*c*) [10].
One way to evaluate the safety of the cross sections in a

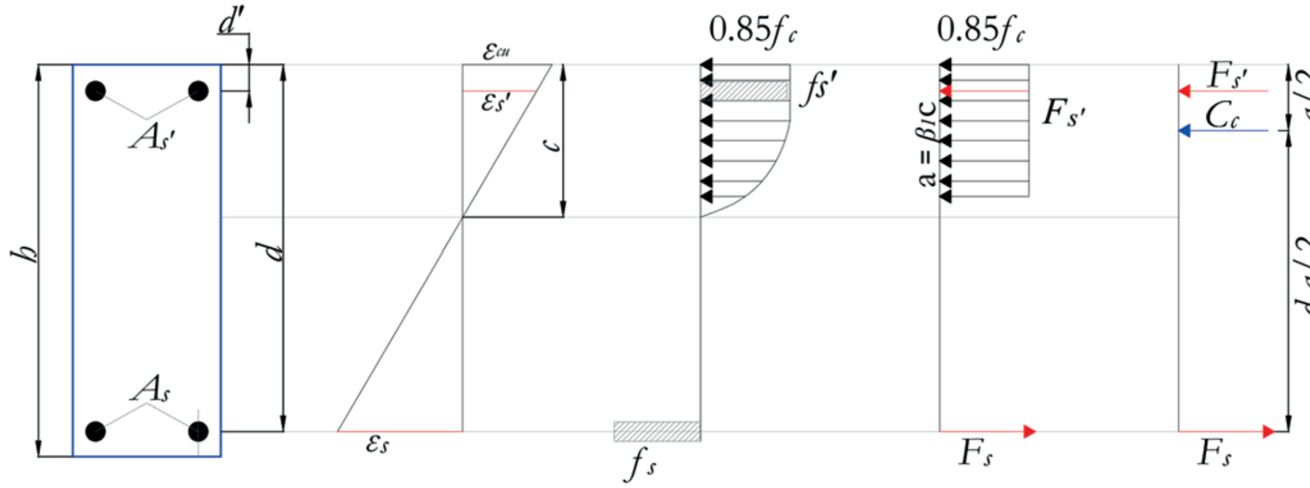


Figure 12
Simplifications for cross sections check

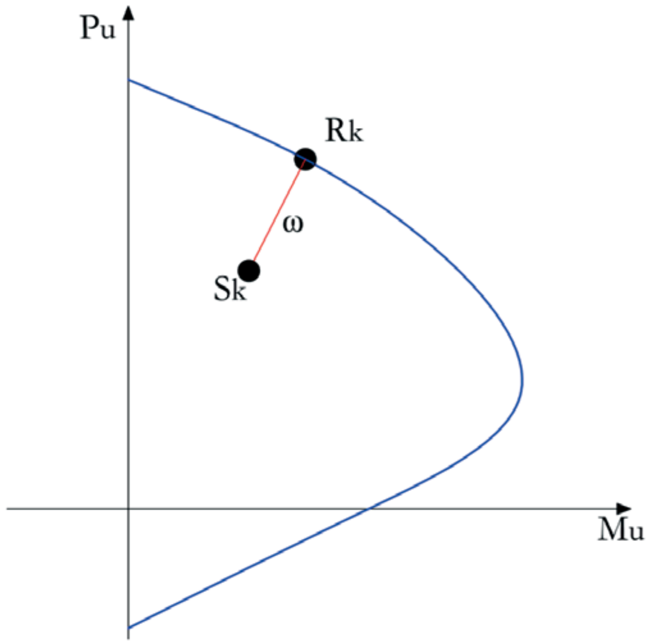


Figure 13
Safety margin for columns cross sections

qualitative way was to adopt the safety margin (Equation 4), which in the case of the columns (Figure 13) represents the resistance reserve of the element (ω) under a load (S_k) for a resistance (R_k) [15]. The diagrams prepared for conformity assessment of the structural design used values of 35 MPa for concrete compressive strength (f_c) and 500 MPa for the yielding stress (f_y) of CA - 50 steel. For the diagram developed to evaluate the moment of collapse the IRC results for concrete (f_c =29 MPa) and the GAEMA - UFPA technical report on steel strength (f_y = 573 MPa) were used. To obtain the calculation resistance (R_d), the strength reduction coefficients used for steel (γ_s = 1,15) and concrete (γ_c = 1,40) were used for normal combinations and the coefficients γ_s = 1,15 and γ_c = 1,20 for the construction combinations.

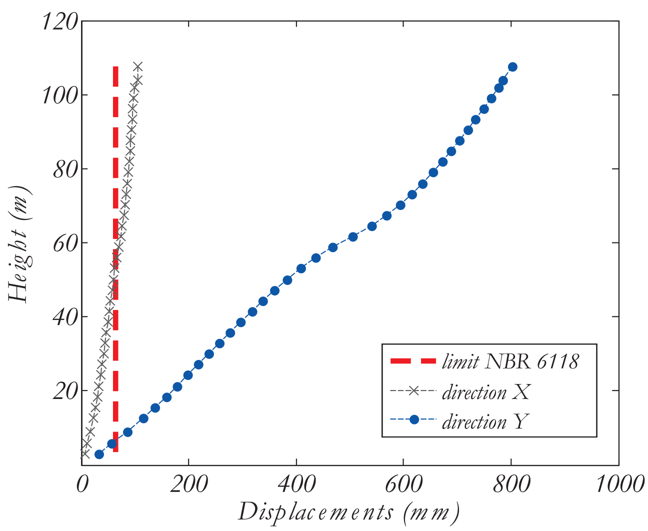
$$\omega = \frac{R_k - S_k}{R_k} \tag{4}$$

3. Results

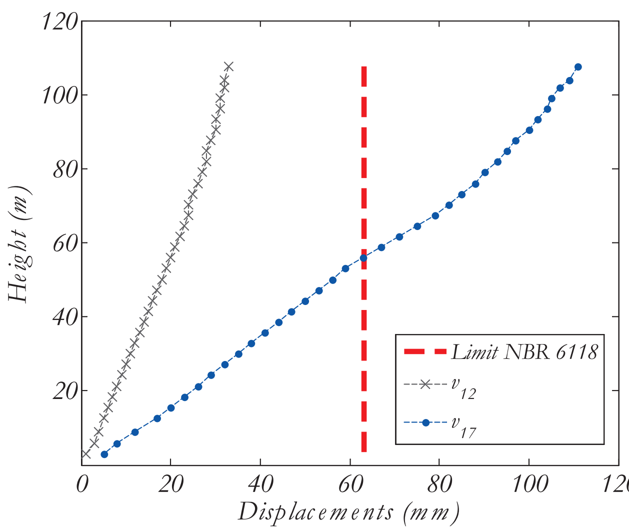
The results obtained from the computational model were analyzed for the normative compatibility checks, i.e. evaluation of the ultimate limit state and building safety. These results showed that in addition to an error in the design of the structural system due to lack of redundancy (with excessive displacement, even without horizontal forces), there was negligence in not considering wind actions over the structure.

The second order moment on the structure corresponded to 24% (direction x - greater stiffness) and 37% (direction y - less stiffness) of the total bending moment of the structure when the conformity of the project was verified. The effects of second order corresponded, under construction combinations, to 17% and 27% of the total moment of the structure, showing its flexibility. The values of the coefficient γ_z came to correspond to the value of 1.7, exceeding the design code recommended values [10]. In figure 14, the structure displacements predicted for the design situation (a) and for the collapse scenario (b) are shown for the annual average wind gust velocity value (v_0 = 12 m/s) and the mean values of maximum annual wind gust velocity (v_0 = 17 m/s). In both cases the structure presented large displacements incompatibles with design code recommendations [10].

The safety of the foundations was evaluated through the loads obtained in the models (S_k) compared to those presented in the structural design (S_d) of the building. The characteristic resistance value (R_k) did not make use of the strength reduction factor, ac-



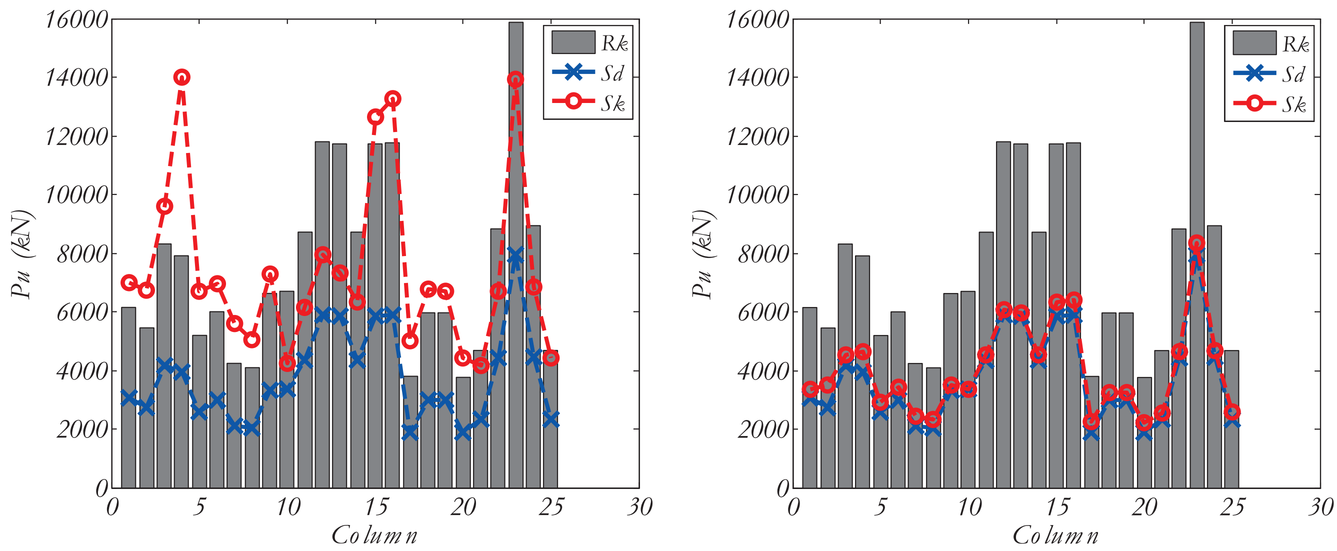
A Projection under code wind gust velocity



B Projection under frequent wind gust velocities

Figure 14
Building's horizontal displacements

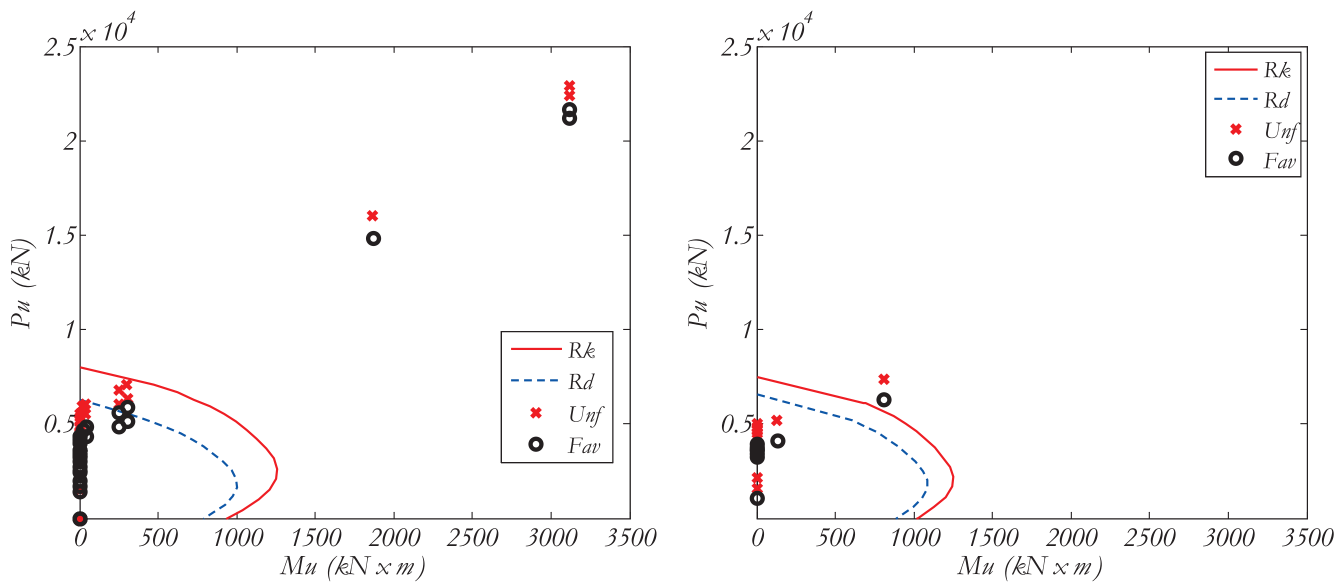
cording to item 6.2.1.2 of NBR 6122 [16]. The total loads in the foundations presented in the design presented a difference of 46% when compared with the model. In the evaluation of the collapse scenario, they presented a difference close to 5%, a result that was in agreement with the reports [8] [9] that there was no failure of the foundations at the moment of collapse. Figure 15 (a) shows the differences between founded loads (S_k) according to the requirements of NBR 6118 [10] and the designed (S_d) ones. The values of ultimate strength (R_k) are exceeded in 60% of the elements, indicating an under sizing and the hypothesis of possible rupture if they were put into service. For the moment of collapse of the building (b) the loads estimated by the model (S_k) were very close to those of the designed (S_d), not exceeding in any case the ultimate resistance of the assessed elements.



A Design loads

B Loads at the moment of collapse (v = 23 m/s)

Figure 15
Foundations loads

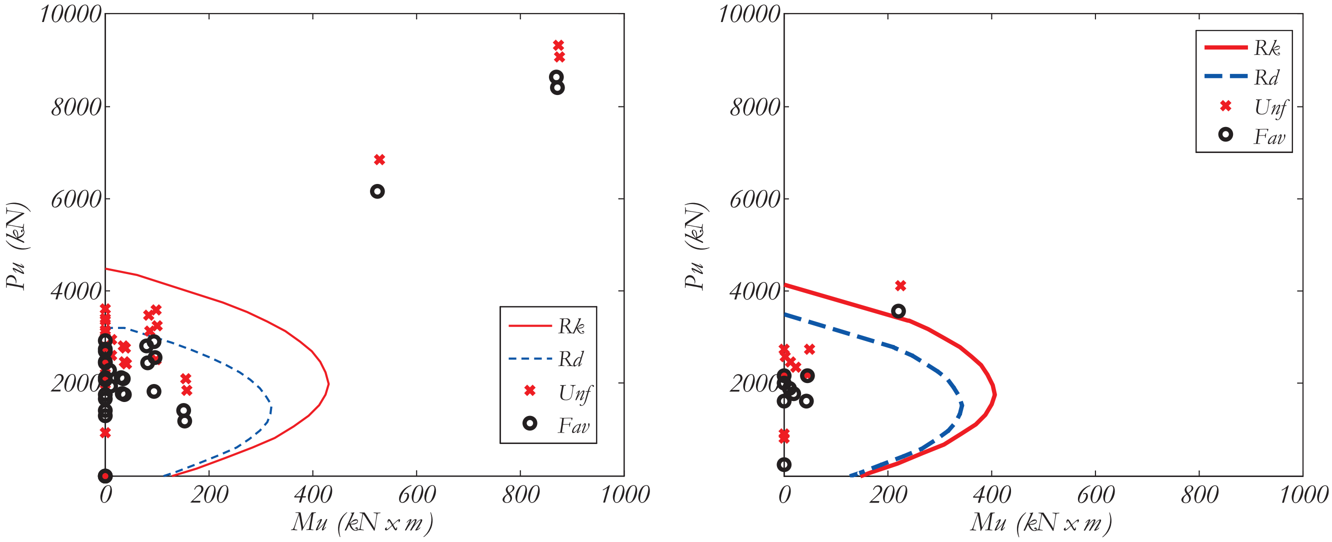


A Design loads

B Loads at the moment of collapse (v = 23 m/s)

Figure 16
Interaction diagram of column P04

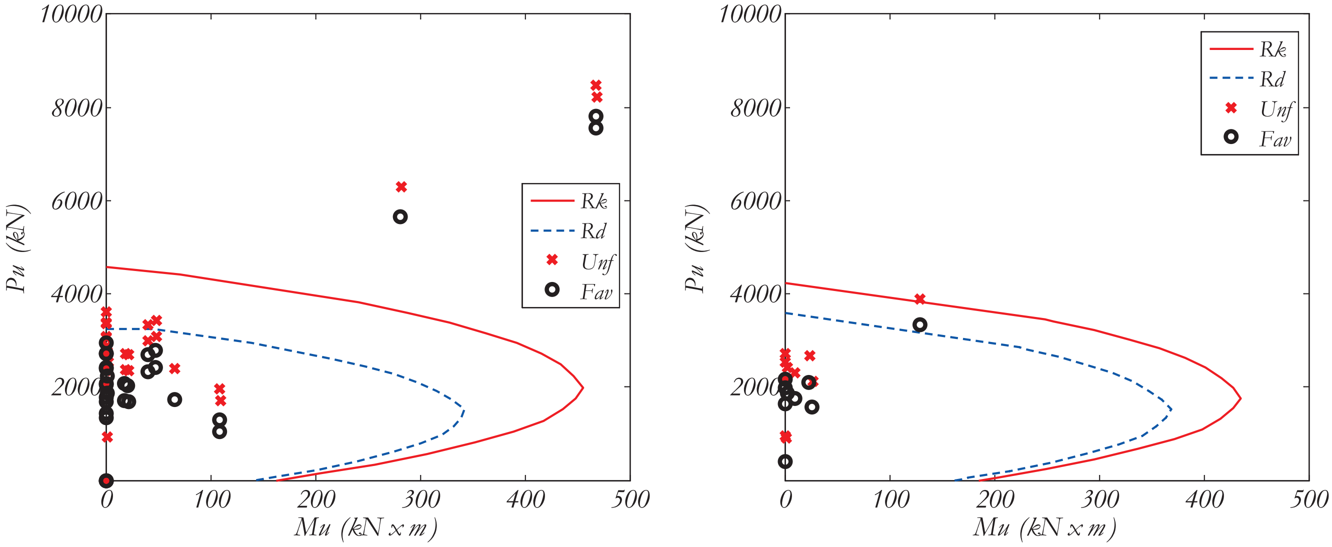
The safety of the columns was verified through diagrams of interaction with the reinforcement configuration used in the construction process. The *software* in its default configuration generated a total of 61 combinations for ULS evaluation, being arranged in the diagrams as favorable (*Fav*) and unfavorable (*Unf*). In order to evaluate the collapse scenario the total number of combinations (23) for ULS analysis was lower due to non-occurrence of accidental loads (*Q*) and non-consideration of water loads (*A*), soil (*S*). Figures 16 to 21 show the diagrams for the abovementioned columns as being responsible for the collapse [8], being shown: the conformity of the elements with the NBR 6118 (a) code and the loadings configuration in the collapse scenario (b). The stress distribution in the diagrams shows that a large number of columns had a reasonable number of combinations very close to the design resistance limits (R_d) when evaluating design compliance. In tables 4 and 5 are presented the margin of safety (ω) and the estimated probability of failure (p_f) of the columns in the computational models. As shown in Table 4, the P15, P16 and P17 columns were



A Design loads

B Loads at the moment of collapse (v = 23 m/s)

Figure 17
Interaction diagram of column P07



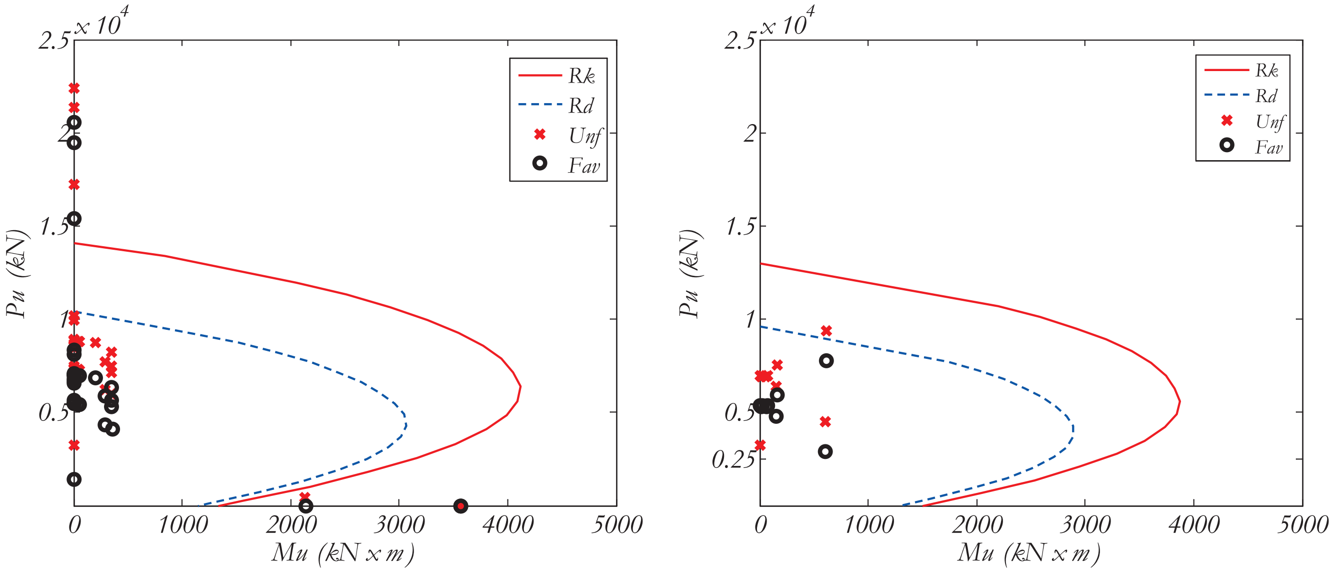
A Design loads

B Loads at the moment of collapse (v = 23 m/s)

Figure 18
Interaction diagram of column P08

more likely to fail with a safety margin far below that needed to maintain the structural stability. It is also worth mentioning that the combinations indicated as more unfavorable to the elements presented the wind as main variable action.

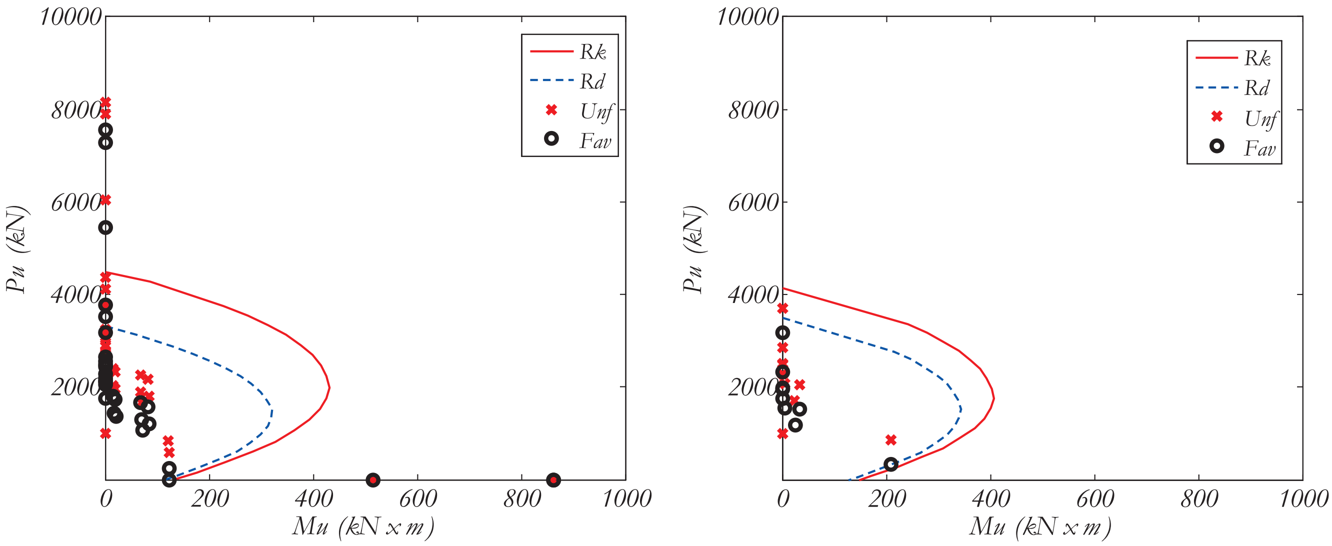
Table 5 shows the state of all columns at the time of collapse. The results show that for the computational model loads all the columns were working with a small safety margin (on average 20% of the characteristic resistance (R_k)), with the probable failure of the P02, P04, P07, P19 and P25. As the P04 and P07 columns had the lowest safety margin and the highest probability of failure for the combinations used (9%) and they could be considered as the first to failure. Due to the complexity of the wind forces acting on the buildings the loads for the collapse scenario of the P04 column for the abovementioned wind gusts (Figure 22) are shown as: frequent occurrence in the region (12 and 17 m/s), that used in the technical report (23 m/s) and the probable responsible



A Design loads

B Projection of loads at the moment of collapse (v = 23 m/s)

Figure 19
Interaction diagram of column P16



A Design loads

B Loads at the moment of collapse (v=23 m/s)

Figure 20
Interaction diagram of column P17

for the sections failure (19 m/s), according to the model results. Analyzing the data from ICEA, velocities greater than 20 m/s show a return period of 10 years and the highest value recorded in the region was of 42 m/s in the year of 1977. Also, the column P04 would withstand twice the shear stresses generated by 23 m/s winds, even with stirrups of 4.2 mm in diameter.

4. Conclusions

From the considerations obtained through the reports, as well as the use of the computational analysis, it was possible to verify that the building was designed in disagreement with the design code instructions, resulting in a structure unable to meet the requirements necessary to avoid the ULS.

- The results of the model indicated a deficiency of the structural arrangement, with a lack of redundancy (increase of the degree of hyperesticity of a rigid frame) and design errors in the consideration of the loads acting on the building, being the original design very close to a model computing only permanent gravitational loads;

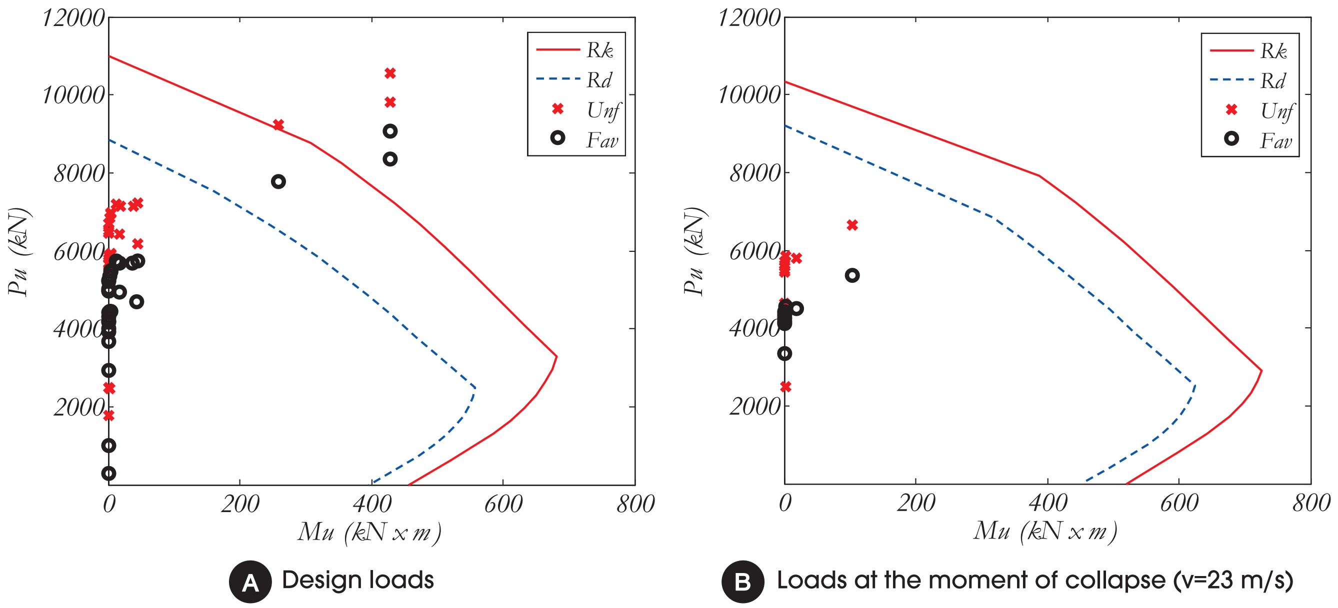


Figure 21
Interaction diagram of column P22

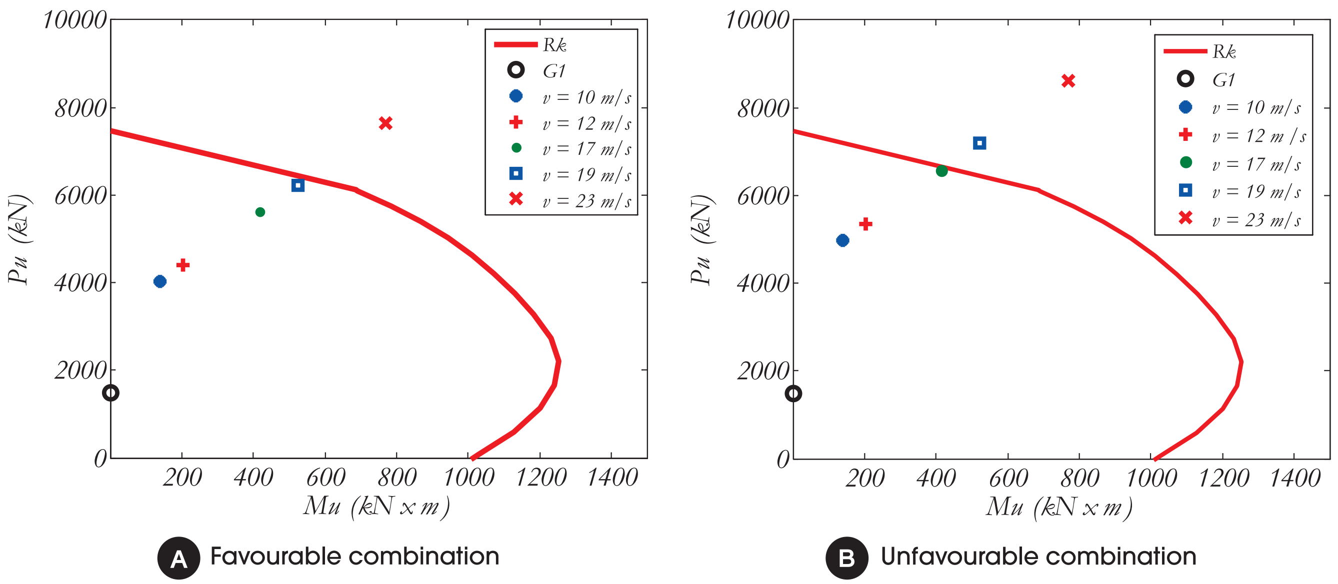


Figure 22
Interaction diagram of column P04 under frequent wind gust velocities

- The designed structure had great flexibility, which can be verified by the displacements described in the model, as well as the parameter γ_z much higher than that recommended by the Brazilian design code;
- Sections designed for the columns were unable to resist to combinations of actions in the construction region, presenting, according to the design code, a safety margin far below that necessary to ensure the structural stability;

Table 4
Evaluation of the columns under NBR 6118's prescriptions

Column	Combination	p_f	ω
P04	1.3G1+1.4G2+0.98Q+1.4V3	10%	-3.9
P07	1.3G1+1.4G2+0.98Q+1.4V3	10%	-2.7
P08	1.3G1+1.4G2+1.4V3	10%	-3.6
P15	G1+G2+S+1.4V3	34%	-58.2
P16	G1+G2+S+1.4V3	15%	-33.4
P17	1.3G1+1.4G2+0.98Q+1.4V4	18%	-27.2
P22	1.3G1+1.4G2+0.98Q+1.4V4	8%	-0.7

Table 5
Columns' situation at the collapse moment

Column	Combination	p_f	ω
P01	1.3G1+1.3G2+V3	0%	0.0
P02	1.3G1+1.3G2+V3	4%	-0.1
P03	1.3G1+1.3G2+V3	0%	0.2
P04	1.3G1+1.3G2+V3	9%	-0.2
P05	1.3G1+1.3G2+V3	4%	0.0
P06	G1+G2+V4	0%	0.3
P07	1.3G1+1.3G2+V3	9%	-0.2
P08	1.3G1+1.3G2+V3	4%	0.0
P09	1.3G1+1.3G2+V3	0%	0.3
P10	1.3G1+1.3G2+V4	0%	0.3
P11	1.3G1+1.3G2+V4	0%	0.4
P12	1.3G1+1.3G2+V4	0%	0.4
P13	1.3G1+1.3G2+V4	0%	0.3
P14	1.3G1+1.3G2+V4	0%	0.3
P15	1.3G1+1.3G2+V4	0%	0.3
P16	1.3G1+1.3G2+V4	0%	0.2
P17	G1+G2+V3	0%	0.1
P18	1.3G1+1.3G2+V4	0%	0.1
P19	1.3G1+1.3G2+V4	4%	-0.1
P20	1.3G1+1.3G2+V4	0%	0.1
P21	1.3G1+1.3G2+V4	4%	0.0
P22	1.3G1+1.3G2+V4	0%	0.3
P23	1.3G1+1.3G2+V4	0%	0.1
P24	1.3G1+1.3G2+V4	0%	0.3
P25	1.3G1+1.3G2+V4	4%	-0.1

- It was verified through computational model that the wind speed of 19 m/s could lead to failure the P04 and P07 columns. This speed, according to the local meteorological data, presents a high probability of occurrence in a period of 10 years;
- Although there was difference between designed and *as built* reinforcement rates, this difference was not significant to impact on the overall resistant capacity, as well as the sections in order to avoid the building's collapse;
- The loading on the foundations, considering the limitations of the model, did not influence the collapse scenario, being at that moment with loads lower than those required for suspicion. The performance of the designed foundations could be questioned if the building was put into service, since its design was linked to the load of the building´s structural design.

5. Acknowledgements

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