Analysis of the dynamic structural behavior of tall buildings subjected to the transversal action of the wind: considering their interaction with the ground

Análise do comportamento estrutural dinâmico de edifícios altos submetidos a ação transversal do vento: considerando a sua interação com o solo

Análisis del comportamiento estructural dinámico de edificios altos sometidos a la acción transversal del viento: considerando su interacción con el suelo

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Abstract
The remarkable design and construction of constructions in large cities of optimization of engineering methods have as structural models in the increasingly high structural structures and/or stimuli, with low resistance and damping, transforming them into high structures in the increasingly structural structures. Powered by the action of the wind. Thus, it becomes rigorously necessary to study the possible constructions of the buildings in question, mainly as dynamic actions in the most realistic way as their non-deterministic properties. Furthermore, it is important to highlight, no less importantly, the study of the way of individual interaction, considering its influence on the resistance of the (global) structural system. Therefore, the conduct aims to analyze the present with wind action insurance contracts based on human comfort criteria. For the study in question, two “tall” buildings were considered, including the mixed structures (steel-concrete) and the other reinforced concrete structures. The first has a total height of 173 m with 48 floors, with a maximum dimension in the plan of 9 x 29 m. The second has a total height of 140 m with 40 floors, with a maximum dimension in the plan of 45 x 32 m. The second has a total height of 140 m with 40 floors, with a maximum dimension in the plan of 9 x 29 m. With the results obtained, it was noticed the reduction of the mode frequencies of the soil interactions of the buildings and accelerations, as it favored the increase of the reduction of the natural frequencies of the interactions of the buildings and the longitudinal buildings and transversal buildings.

Keywords: Tall buildings; Soil-structure interaction; Non-deterministic wind action; Dynamic analysis of buildings; Human comfort analysis.

Resumo
O notável crescimento de projetos e construções de edifícios altos nas grandes cidades tem estimulado a aplicação da otimização dos métodos de cálculo, por formas a serem projetados sistemas estruturais cada vez mais esbeltos e/ou flexíveis, com baixa rigidez e amortecimento, transformando-os em elementos altamente sensíveis aos efeitos dinâmicos induzidos pela ação do vento. Desse modo, torna-se rigorosamente necessário o estudo das análises estruturais dos edifícios em questão, considerando sobretudo as ações dinâmicas do vento da forma mais realista possível, considerando as suas propriedades não-determinísticas. Para além disso, importa destacar de maneira não menos importante o estudo da interação solo-estrutura, levando em consideração a influência da mesma sobre a rigidez do sistema estrutural (global). Assim sendo, a presente pesquisa tem por objetivo analisar o comportamento dinâmico de edifícios altos submetidos à ação do vento com base em critérios de conforto humano. Para o estudo em questão, foram considerados dois edifícios “altos”, entre os quais um em estruturas mistas (aço-betão) e o outro em estruturas de betão armado. O primeiro possui uma altura total de 173 m com 48 andares, com a dimensão máxima em...
plantas de 45 x 32 m. O segundo possui a altura total de 140 m com 40 andares, com a dimensão máxima em planta de 9 x 29 m. Com base nos resultados obtidos, observou-se a relevância do processo de modelagem interação solo-estrutura, pois contribuiu significativamente para a redução dos valores de frequências naturais das estruturas, e por conseguinte, favoreceu no aumento das acelerações longitudinais e transversais das edificações.

**Palavras-chave:** Edifícios altos; Interacção solo-estrutura; Ação não-determinística do vento; Análise dinâmica de edifícios; Análise de conforto humano.

**Resumen**

El notable crecimiento de proyectos y construcción de edificios altos en las grandes ciudades ha estimulado la aplicación de la optimización de métodos de cálculo, ya que se diseñan sistemas estructurales cada vez más esbeltos y/o flexibles, con baja rigidez y amortiguamiento, transformándolos en elementos estructurales de alta sensibilidad. a los efectos dinámicos inducidos por la acción del viento. Por lo tanto, es estrictamente necesario estudiar el análisis estructural de los edificios en cuestión, especialmente considerando las acciones dinámicas del viento de la manera más realista posible, considerando sus propiedades no-determinísticas. Además, es importante destacar, no menos importante, el estudio de la interacción suelo-estructura, teniendo en cuenta su influencia en la rigidez del sistema estructural (global). Por ello, la presente investigación tiene como objetivo analizar el comportamiento dinámico de edificios altos sometidos a la acción del viento en base a criterios de confort humano. Para el estudio en cuestión se consideraron dos edificios “en altura”, uno de ellos en estructuras mixtas (acero-hormigón) y el otro en estructuras de hormigón armado. El primero tiene una altura total de 173 m con 48 plantas, con una dimensión máxima en planta de 45 x 32 m. El segundo tiene una altura total de 140 m con 40 plantas, con una dimensión máxima en planta de 9 x 29 m. Con base en los resultados obtenidos, se observó la relevancia del proceso de modelado de la interacción suelo-estructura, ya que contribuyó significativamente a la reducción de los valores de las frecuencias naturales de las estructuras, y por lo tanto favoreció el aumento de las aceleraciones longitudinales y transversales de las estructuras de los edificios.

**Palabras clave:** Edificios altos; Interacción suelo-estructura; Acción del viento no determinista; Análisis dinámico de edificios; Análisis del confort humano.

1. **Introduction**

For low-rise, heavy constructions formed by robust walls in terms of thickness, the actions of the wind on them are not characterized as problems. But these wind actions become serious problems when the structures of the buildings are very slender and/or when the structural systems are conformed with low-quality materials (Blessmann, 2013; Quissanga; do Nascimento; Galgoul, 2020). In this context, it is worth noting that the vast majority of tall buildings present significant displacements in the transverse direction of the wind flow (as shown in Figure 1) when they are subjected to this type of load. And when comparing the response of the transverse displacement and the displacement in the direction of the wind, in many cases it comes to a condition to the verification of the service limit states.

**Figure 1** – a) Simplified scheme of the longitudinal and transversal action of the wind; b) Profile view of wind action.

It is important to point out that the phenomenon of the cadenced detachment of vortices in the structure, produces
alternating forces in the transversal direction of the wind, being, therefore, characterized as an aero-elastic action, also known as interaction-fluid-structure that in some cases can contribute to the increased vibrations and consequently high amplitudes (Battista, 2013). In the case of the incidence of wind with constant speed or smooth flow in structures with constant section lengths, the phenomenon in question manifests itself more prominently (Quissanga; do Nascimento; Galgoul, 2019). As much as the problems of vibrations caused by the wind hardly reach the point of causing risks of collapse of the structure, it is important to understand that this type of dynamic action of the wind can cause human discomfort in the users of the buildings.

And, however, given the need for the construction of tall and slender buildings to reduce the free spaces in urban areas to meet, above all, the demand for population growth, it is necessary to constantly update and apply the optimization of the calculation methods by different forms. to solve problems related to excessive vibrations and human discomfort caused by the dynamic effects of the wind. For this reason, according to the different Authors (Barile, 2019; Bashor et al., 2012; Ferrareto, 2017; Jurong, 2006; Rist; Svensson, 2016), the study of soil-structure interaction (buildings high) -wind for ways to improve structural designs, thus avoiding possible service limit states. As for the study of soil-structure interaction, this, like the structure-wind interaction, must be rigorously carried out, given its influence on the vibration control of buildings.

It is known that normally design codes consider the effect of wind load to be static, simplifying them from the average speed calculation without considering the components of the floating part that can somehow induce vibrations in structural systems to the point of making users uncomfortable. Given the aforementioned simplification, it is important to seek to carry out more analyzes that can be carried out by comprehensively considering the dynamic characteristics of the wind (Barile, 2019; Fernández & Elena Parnas, 2017; Liu et al., 2016; Shinozuka & Schüeller, 2012). Thus, according to Franco (1993) and Shinozuka and Schueller (2012), one of the methods that can be used to obtain the most realistic responses possible is the spectral representation method, which is based on the wind series generated with the part wind fluctuation, which is determined from the sum of a finite number of harmonics with randomly generated phase angles.

To maintain the similitude or similarity of the natural wind, the methodology in question (spectral representation) uses the power spectrum and a coherence function to calculate the amplitude of each harmonic (Franco, 1993; Shinozuka; Schüeller, 2012), and then, based on the series of winds considered, simulate the dynamic loads of the wind and evaluate human comfort by checking the values of the accelerations of the structures (Bastos, 2008; Chávez, 2006; Oliveira, 2015; Santos, 2018; Steffen, 2016).

Thus, to analyze the dynamic behaviour of tall buildings subjected to wind action based on human comfort criteria, an analysis methodology based on the work of (Barile, 2019; Shinozuka & Schueller, 2012), for the generation of wind loads considering their non-deterministic dynamic characteristic based on the power spectrum and the coherence function for the calculation of the harmonic amplitude. To this end, three finite element models were designed based on real and/or existing projects of three buildings, where the first one has 48 floors, with a total height of 173 m and a maximum rectangular dimension of 45 x 32 m in the plan, while the second has 40 floors, with a total height of 140 m, with a maximum rectangular dimension in the plan of 9 x 29 m. As for the results of the present research, it was observed that they point to the relevance of modelling the soil-structure interaction, since it contributed significantly to the reduction of the values of natural frequencies of the structures, and consequently to the increase of the longitudinal and transverse accelerations of the structures. buildings. Furthermore, the results show that the calculated peak acceleration exceeds the limits established by the wind standard in buildings (NBR6123, 1988).
2. Methodology

2.1 Input Model Assumptions

2.1.1 Physical and geometric description of the model

In the present research work, two three-dimensional numerical models are performed based on the usual mesh refinement techniques present in the Finite Element Method (FEM) simulations implemented in the computer program ANSYS. The mesh refinement took place to generate finite elements uniformly distributed in the order of 25 by 25 centimetres, demonstrating a good refinement for the models under study. The first numerical model (Model I) is inspired by a real existing building of metallic structures, having plant dimensions of 45 x 32 m (central core of 27 x 9 m), with a span length of 9.0 m in the direction “X” and 11.5 m in the “Y” direction with a total height of 173 m, and 48 floors with a ceiling height of 3.60 m (see Figure 2a). Like the first, the second model (Model II) is also inspired by a real existing building of the reinforced concrete structure, having plant dimensions of 9 x 29 m, with span lengths ranging from 2.80 to 5, 68 m in the “X” direction and 4.20 to 4.80 m in the “Y” direction with a total height of 140 m, and 40 floors with a ceiling height of 3.50 m (see Figure 2b).

Figure 2 – Plan view of Model I and Model II (mixed structural system “steel-concrete” and reinforced concrete).

Source: Authors.

2.1.2 Description of Non-deterministic wind load modelling

As mentioned in item 1 (Introduction) in the present research, the wind load with its non-deterministic dynamic characteristic was developed based on the analysis methodology proposed by the authors, Shinozuka et al., (2012), which considers the portion of fluctuating wind generated from the sum of a finite number of superimposed harmonics, taking into account the phase angles that are randomly perceived, expressed mathematically by Equation 1. Where, \( N_p \) consists of the number of points of the structure to apply the wind load; \( N_o \) is the number of frequencies for the spectrum representation; \( \Delta \omega \) is the frequency increment, \( \omega \) is the frequency (rad/s), \( \phi \) is the uniformly distributed random phase angle in the interval (0 - 2\( \pi \)); \( H \) is the matrix representing the lower part of the cross-spectral density matrix \( S(\omega)S(\omega) = H(\omega)HT(\omega) \). Then, Equation 2 is composed of the spectral density S and the coherence function \( \gamma \).

\[
V_j(t) = \sum_{m=1}^{N_p} \sum_{k=1}^{N_o} |H_{jk}(\omega_k)| \times \sqrt{\Delta \omega} \times \cos(\omega t + \phi_k), \quad j = 1, 2, 3 \ldots N_p
\]
However, in this work, it was considered that the wind speed is divided into two parts, in which the first was for the static part and the second for the turbulent part, as expressed mathematically in Equation 3. Where $V_m$ is the average wind speed (m/s) and $v_f$ is the wind speed of the turbulent part (m/s). It is noteworthy that the interval from 10 minutes to 1 hour is the time considered to obtain the static part.

$$V(t) = V_m + v_f$$

As for the amplitude of harmonics, these, individually, are obtained through the spectral power density of the wind, and the spatial correlation between wind speeds at distant points in the "X" direction (horizontally) and in the "Y" direction (vertically). Therefore, to comply with the methodology in question in the present study, the power spectral density and the spatial correction were considered, based on the mathematical expression of Equations 4 and 5, respectively. Where, $n_k$ is the frequency (Hz); $A_i$ and $A_j$ are the differences between $z_i$ and $z_2$ and between $y_i$ and $y_2$ respectively (m); $C_y$ and $C_z$ are constants ranging from 10 and 7, respectively, and $V_m(z_1)$ and $V_m(z_2)$ are the average wind speeds at $z_1$ and $z_2$ (m/s).

$$S_k(\omega) = \frac{2\pi u^2}{\omega} \times \frac{200m}{(1 \times 50m)^{3/4}} ; \quad n = \frac{\omega z}{2\pi V_m(z)}$$

$$Coh(r, n_k) = e^{-r}, \quad f = \frac{n_k \times \sqrt{C_y^2 \times \Delta y^2 + C_z^2 \times \Delta z^2}}{\frac{1}{2} (V_m(z_1) + V_m(z_2))}$$

In this context, the wind load considered for each section of the building, in the nodes of the finite element models, considered in the present analysis, was obtained through Equation 6, exposed in the NBR 6123 (1988) standard. Where $F_j$ is the wind force and $A_{eff}$ is the effective area for the node in the finite element model selected for analysis. In other words, the dynamic force of the wind applied to the structure is obtained through the equation in question, described immediately below (Equation 6).

$$F_j(t) = 0.61 \times v_f^2 \times C_{a_j} \times A_{eff} , \quad 1, 2, 3, \ldots , N_F$$

Based on the Monte Carlo method (massive random sampling statistical method to obtain numerical results), it was possible to obtain the final structural response by taking into account the non-deterministic characteristics of the natural wind. For this, it was necessary a finite number of analyzes to obtain the structural response that can meet the probabilistic criterion based on statistical analysis. Then, the power spectrum and the spatial correlation for the wind speed signal generated in a time interval of 10 minutes are presented in Figures 3a and 3b.
Next, the typical wind speeds are shown, according to Figures 4a and 4b, where, as they have practically the same behaviour, the action of the same in three different directions of the structural systems was considered. However, based on the figures presented below, it is observed that for the closer distances, in the horizontal and vertical directions, the response signals show or present characteristics very similar to those that were studied based on the data obtained from the natural wind and described by the function of the spatial correlation between wind speeds (Davenport).

**Figure 3** – a) Power spectral density typical of the wind speed generated for a given position of the structure; b) Typical spatial correlation between different points of the structure indicated by the coherence function.

**Figure 4** – Typical wind speeds; a) for horizontal heights; b) horizontal position for vertical alignment.

### 3. Investigated structural model

#### 3.1 Model of metallic structures (Model I)

In the case of Model I, the building consists of 15 cm thick reinforced concrete slabs, main beams with laminated profiles of type W460x106 and secondary beams of type W410x60, all in ASTM A572 steel. The pillars are made of HD-type profiles, in ASTM A913 steel, with gauges varying in height of the building (see Table 1). Regarding the physical characteristics of the materials, reinforced concrete has a characteristic compressive strength of 30 MPa \( (f_{ck} = 30 \text{ MPa}) \), modulus of elasticity of 26 GPa \( (E_c = 26 \text{ GPa}) \), specific weight of 25 kN/m\(^3\) \( (\rho_c = 25 \text{ kN/m}^3) \) and Poisson's ratio of 0.2 \( (\nu_c = 0.2) \); as for steel, the characteristic strength is 345 MPa \( (f_y = 345 \text{ MPa}) \), modulus of elasticity of 205 GPa \( (E_s = 205 \text{ GPa}) \), specific weight of 78.5 kN/m\(^3\) \( (\rho_s = 78.5 \text{ kN/m}^3) \) and Poisson's ratio equal to 0.3 \( (\nu_s = 0.3) \). The Permanent Loads (CP) of coating adopted were 35 kg/m\(^2\) on each floor and the permanent loads considering the windows and frames were 150 kg/m on the facades and an overload of 300 kg/m\(^2\).
Table 1. Laminated profiles of the columns of the structural model (Model I).

| Variation of profiles on floors | Central core pillars | Facade pillars |
|---------------------------------|----------------------|---------------|
| 1º ao 10º floor                 | HD400x990            | HD400x551     |
| 11º ao 20º floor                | HD400x818            | HD400x382     |
| 21º ao 30º floor                | HD400x667            | HD320x245     |
| 31º ao 40º floor                | HD400x421            | HD260x172     |
| 41º ao 48º floor                | HD400x187            | HD260x114     |

Source: Authors.

Regarding the foundation of the evaluated structural model, it is noteworthy that the building model was considered on a terrain that has a detailed geotechnical profile based on the Standard Penetration Test - SPT, according to NBR 6484 (2001). Where the soil has a low bearing capacity in its first 3 m depth, however, sand compactness increases linearly with increasing depth, where it becomes compact medium following layers (3 to 6 m) in depth and acquires high properties. compactness in the following layers.

Then it is presented in Table 2, the values of the support reactions, at the base of each column, are obtained through static analysis to determine the number of foundation piles. Noting that, due to the high load applied to the base of each column, and support reactions, it was necessary to propose 179 piles under a 3-m thick raft to absorb the efforts. Figure 5 presents, as an illustration, the details of the structural model (building-raft) on the reinforced concrete piles. Longitudinal view and isometric view respectively.

Table 2. Loads are considered in the foundation of the structural model (Model I).

| Nº Pillar | PP+CP+SC (kN) | Wind in 0º (kN) | Total (kN) | Nº Pillar | PP+CP+SC (kN) | Wind in 0º (kN) | Total (kN) |
|-----------|---------------|-----------------|------------|-----------|---------------|-----------------|------------|
| P1        | 16940         | 2331            | 19271      | P13       | 17608         | -955           | 16653      |
| P2        | 18207         | 2579            | 20786      | P14       | 28744         | -11125         | 17619      |
| P3        | 20347         | 2698            | 23045      | P15       | 29662         | -10731         | 18931      |
| P4        | 20347         | 2503            | 22850      | P16       | 29662         | -9395          | 20267      |
| P5        | 18207         | 2062            | 20269      | P17       | 28744         | -7562          | 21182      |
| P6        | 16940         | 2148            | 19088      | P18       | 17608         | -792           | 16816      |
| P7        | 17608         | 957             | 18565      | P19       | 16940         | -2329          | 14611      |
| P8        | 28744         | 11123           | 39867      | P20       | 18207         | -2578          | 15629      |
| P9        | 29662         | 10727           | 40389      | P21       | 20347         | -2698          | 17649      |
| P10       | 29662         | 9391            | 39053      | P22       | 20347         | -2503          | 17844      |
| P11       | 28744         | 7561            | 36305      | P23       | 18207         | -2062          | 16145      |
| P12       | 17608         | 796             | 18404      | P24       | 16940         | -2148          | 14792      |
| Total     |               |                 | 526032     |           |               | 0              | 526032     |

PP – Own Weight, CP – Permanent Load; SC – Overload. Source: Authors.
3.2 Model of reinforced concrete structures (Model II)

In Model II, as already mentioned, the columns, beams, piles and diagonals of locking were simulated through the three-dimensional finite element of BEAM type beam (uniaxial) composed of two nodes and each with six degrees of freedom: translation in the x directions, y, and z and rotation in the x, y, and z axes. In the case of solid three-dimensional slabs, these were performed numerically using the shell finite element SHELL. This element considers the effects of bending and membrane, as well as having six degrees of freedom per node, three translations and three rotations in the x, y and z directions. The structural systems investigated were considered that all materials work in the linear-elastic regime and that the plane sections remain plane in the deformed state, to guarantee the compatibility of deformations between the nodes of the beam elements and the shell elements used in the offset type connections in the existing connection between the reinforced concrete slabs and the metal beams.

For vertical loads, for the PP of the structure, reinforced concrete with a specific weight of 25 kN/m³ ($\gamma_c = 25$ kN/m³) is adopted. Masonry loads were calculated as ceramic blocks with traditional coatings, such as roughcast, plaster, plaster and paint, with a total thickness of 13 cm, with a specific weight of 14 kN/m³ ($\gamma_c = 14$ kN/m³). A 1.0 kN/m² (CP =1.0 kN/m²) covering CP for sub-floor and ceramic floor slabs was considered.

About the foundation carried out in Model II, as in Model I, the structure is supported on a soil that has a detailed geotechnical profile based on the Standard Penetration Test - SPT tests, following the NBR 6484 (2001) standard. Then, in Table 3, the compactness and consistency states of the soil are presented, and in Table 4, the values of the support reactions at the base of each pillar that make up the Model II are presented. Figure 6 shows the details of the structural model on the reinforced concrete piles. Longitudinal view and isometric view respectively.

| Soil type and layer | Penetration resistance index | Soil state designation |
|---------------------|-----------------------------|------------------------|
| Sands and sandy silts | < 4 | Fluffy |
| | 5 an 8 | Moderately compact |
| | 9 an 18 | Moderately compact |
| | 19 a 40 | Compact |
| | > 40 | Very compact |
| | < 2 | Very soft |
| | 3 a 5 | Soft |
| | 6 a 10 | Average |
| | 11 a 19 | Hard |
| | > 19 | Very hard |

Source: Authors.
Table 4. Values of the support reactions were obtained at the base of each column (Model II).

| Number pillars | PP+CP+SC (kN) | Wind in 0º (kN) | Total (kN) | Number pillars | PP+CP+SC (kN) | Wind in 0º (kN) | Total (kN) |
|----------------|----------------|-----------------|-----------|----------------|----------------|----------------|-----------|
| P1             | 12,073         | 9,443           | 21,515    | P12            | 4,576          | -1,136         | 3,440     |
| P2             | 6,624          | 6,790           | 13,414    | P13            | 6,594          | 110            | 6,703     |
| P3             | 6,811          | 6,295           | 13,106    | P14            | 8,846          | 332            | 9,177     |
| P4             | 6,692          | 6,911           | 13,603    | P15            | 6,586          | -6,588         | -2        |
| P5             | 26,265         | 42,354          | 68,619    | P16            | 6,464          | -5,213         | 1,251     |
| P6             | 6,664          | 8,140           | 14,804    | P17            | 5,987          | -4,027         | 1,960     |
| P7             | 8,975          | 9,108           | 18,082    | P18            | 5,600          | -10,546        | -4,946    |
| P8             | 9,408          | -7,909          | 1,499     | P19            | 29,595         | -35,887        | -6,291    |
| P9             | 3,349          | -170            | 3,179     | P20            | 6,234          | -9,365         | -3,131    |
| P10            | 3,375          | -45             | 3,331     | P21            | 8,603          | -9,220         | -618      |
| P11            | 3,278          | 623             | 3,901     | ---            | ---            | ---            | ---       |
| Total          |                |                 |           |                |                | 182,596        | 182,596   |

PP – Own Weight, CP – Permanent Load; SC – Overload. Source: Authors.

Figure 6 – Longitudinal and isometric view, detailing the depth of the reinforced concrete foundation (Model II).

Source: Authors.

The two buildings investigated in the present research, as referenced; Model I has plant dimensions of 45 x 32 m (central core of 27 x 9 m), has 48 floors, with a height of 3.60 m, a total height of 173 m, as shown in Figure 7. And Model II has plant dimensions of 9 x 29 m, has 40 floors, with a height of 3.50 m, a total height of 140 m, as shown in Figure 8. The structure of Model I was made of metallic structure and Model II of reinforced concrete.
Figure 7 - Model of the investigated metallic structure (Model I).

Figure 8 - Investigated reinforced concrete structure model (Model II).
4. Finite Element Model of Buildings

The numerical finite element model developed for the dynamic analysis of buildings (Model I and Model II) adopted the usual mesh refinement techniques present in simulations of the finite element method implemented in the ANSYS software. In the case of Model I, it has 59616 shell-type elements - SHELL, 87188 nodes, 55008 beam-type elements - BEAM, 360 COMBIN spring elements and 520908 degrees of freedom. The finite element model referring to Model I, of mixed structures (steel-concrete), is shown below, as shown in Figure 9.

**Figure 9** - Finite element model of the "steel-concrete" mixed structural system (Model I).

In the case of Model II, like the first one, using the usual discretization techniques through the finite element method. The mesh refinement took place in such a way as to generate finite elements uniformly distributed in the order of 25 by 25 cm, demonstrating a good refinement for the building studied (see Figure 10). The model has 232552 nodes, 245880 elements and 1395246 degrees of freedom. The one used in the modal (free vibration) and forced vibration analysis, in which the dynamic behaviour of the structure was evaluated. Next, the isometric and profile views of the finite element model are presented.

**Figure 10** - Finite element model of the reinforced concrete structural system (Model II).
In the case of the transient dynamic analysis performed, it is worth mentioning that simplifying hypotheses were considered, as physical and geometric non-linearity are not considered; models have linear vibration mode.

5. Natural Frequencies and Vibration Modes of the Models

The natural frequencies and vibration modes, and/or the eigenvalues and eigenvectors of the structural systems were obtained based on extraction methods called modal analysis, through free vibration analysis. In Figure 11, the first eight vibration modes of Model I are shown, where the fundamental frequency corresponding to 0.15 Hz ($f_{01} = 0.15$ Hz), obtained through modal analysis, clearly shows the need to perform a dynamic analysis of the building, and the information can be corroborated from the power spectrum (see Figure 3a). To study the direction or situation of the wind that causes greater accelerations and displacements in the “Z” direction of the structure, non-deterministic wind loads were applied, since the lowest frequency corresponds to the bending around the “X” axis.

![Figure 11 - Vibration of the "steel-concrete" mixed structure model (Model I).](image)

It is worth noting that the responses in terms of frequencies obtained considering rigid supports were practically similar to the frequency responses considering the soil-structure or raft-structure interaction, with, therefore, a difference of only 2%. Therefore, it was decided not to place the comparison in the present research. However, the explanation for such behaviour is related to the global rigidity of the structure, which is very flexible even considering the rigid supports. Table 5 presents the parameters a and b used in dynamic analysis with forced vibration.
Table 5. Parameters $\alpha$ and $\beta$ used in forced vibration analysis (Model I).

| Types of support | $f_0_1$ (Hz) | $f_0_5$ (Hz) | $\alpha$ (Mass) | $\beta$ (Stiffness) |
|------------------|--------------|--------------|------------------|---------------------|
| Rigid supports   | 0.153        | 0.552        | 0.015916         | 0.004354            |
| Pile-railer      | 0.145        | 0.174        | 0.010245         | 0.009734            |

Source: Authors.

Figure 12 shows the first eight vibration modes of Model II, under rigid supports, where the fundamental frequency corresponding to $0.29$ Hz ($f_{01} = 0.29$ Hz), obtained through modal analysis. It is important to note that this model (Model II) is very rigid in bending around the “Z” axis (with $f_{04} = 1.28$ Hz) and flexible around the “X” axis (with a frequency of $f_{01} = 0.29$ Hz). As with Model I, if the fundamental frequency clearly shows the need to carry out a dynamic analysis of the building, the information can be corroborated by the power spectrum (see Figure 3a).

Figure 12 - Vibration of the reinforced concrete structure model, under rigid supports (Model II).
Based on the values of the first frequencies related to bending around the “X” axis, it was possible to determine the values of the parameters \( \alpha \) and \( \beta \) that define the damping ratio of the structural system, considering a damping rate of 2\%, as shown in Table 6.

**Table 6. Parameters \( \alpha \) and \( \beta \) used in forced vibration analysis (Model II).**

| Structural system | \( f_{01} \) (Hz) | \( f_{04} \) (Hz) | \( \alpha \) (Mass) | \( \beta \) (Stiffness) |
|-------------------|------------------|-----------------|-------------------|--------------------|
| Model II          | 0.291            | 1.277           | 0.0591160         | 0.0041938          |

Source: Authors.

### 6. Non-deterministic Dynamic Analysis of Models

As already mentioned, the dynamic analyzes were performed using the finite element program ANSYS. Depending on the responses in the modal, dynamic analyzes were carried out with forced vibration in the structural models of the present research. In this sense, in addition to the usual vertical loads of the project, the actions of the wind were applied, taking into account their non-deterministic characteristic on the largest facade or face of the building “Z” direction” (see Figure 9 and Figure 10). It is important to note that initially, the basic wind speed was considered with a recurrence time of 10 years based on the NBR 6123 (1988) standard. In this, the results of the dynamic analyzes were obtained at the top of the buildings; - for Model I at \( h_1 = 173 \) m and for Model II at \( h = 140 \) m, this, when the maximum horizontal displacements of translation were analyzed and also investigated in the floors of the last floors of the buildings \( h_1 - 3.60 \) m = 169.40 m e \( h_2 - 3.50 \) m= 136.50 m) when investigating the maximum accelerations.

Knowing that the dynamic actions of the wind applied to the structures have non-deterministic characteristics, it is consummated that it is almost impossible to predict the responses of the behaviour of the structures at a given instant of time. In this way, the answers reliably were reached through adequate statistical treatment.

In the present research, to meet the recurrence period of the design code NBR 6123 (1988), traditionally used for the evaluation of human comfort, the scenario of wind loading under the structures was considered, with a recurrence period equal to 10 years, based on NBR 6123 (1988). For the scenario in question, 30 wind series with non-deterministic characteristics were considered, generated to facilitate the investigation of the dynamic behaviour of the structural systems in question. Then, in Table 7, the parameters used to generate the wind series with the non-deterministic characteristics for Model I are presented.

It is worth mentioning that the representative basic wind speed in the region of the city of Rio de Janeiro occurs every 50 years, with a duration of 3 seconds, according to the NBR 6123 (1988) standard. And as for the zone, it is an open area with few obstacles resulting, therefore, in category II. The probability factor was obtained to satisfy the recurrence period of 10 years, according to NBR 6123 (1988). A duration time of 10 minutes was considered, which corresponds to 600 seconds, which is usually considered to study the effects of wind on structural systems.
Table 7. Parameters are used to generate the non-deterministic series of events.

| Penetration resistance index | Designation of wind characteristics |
|------------------------------|-------------------------------------|
| Base wind speed              | 35.0 m/s                            |
| Speed adopted at the top of the building | 25.63 m/s     |
| Recurrence time              | 10 years                            |
| Drag coefficient in the “X” direction | 1.25        |
| Drag coefficient in the Z direction | 1.40        |
| Land category                | II                                  |
| Parameters for roughness factor | b = 1 e p = 0.15       |
| Probability factor           | 0.76 (for 10 years)                 |
| Duration                     | 600 seconds                         |
| Time increment               | 0.03 seconds                        |

Source: Authors.

6.1 Non-deterministic dynamic analysis of Model I

In Figure 13a the dynamic behaviour of the structure (Model I) in terms of acceleration, is obtained considering the wind in the direction of 0° applied in the simulated model on a rigid support. It is worth noting that in the dynamic behaviour, the longitudinal peak acceleration is 30% greater or greater than the maximum transverse peak acceleration (a_{pl} = 0.157 m/s², and a_{pt} = 0.148 m/s², respectively). The time-domain response is divided into two phases, transient and permanent. In the transient phase, acceleration peaks are disregarded, since this phase does not accurately represent the real behaviour of the wind action on the structures, since this, in its real configuration, is not suddenly applied to the structure. Therefore, all results will be analyzed in the permanent phase.

The time-domain accelerations due to the wind acting at 90° are illustrated in Figure 13b. In this case, the transverse acceleration was predominant, maximum peak is 103% higher than the maximum peak longitudinal acceleration (a_{pt} = 0.259 m/s² and a_{pl} = 0.115 m/s²). It is important to highlight that the maximum transverse acceleration was even 46% higher than the maximum acceleration obtained when the wind acted at 0° (a_{pt} = 0.259 m/s and a_{pl} = 0.157 m/s²), as shown in Figure 13a.

Figure 13 - Longitudinal and transverse acceleration; a) wind in the 0° and b) wind in 90° direction - rigid support (Model I).

The accelerations in the frequency domain are shown in Figure 14, and it is possible to see that the greatest energy transfers occur at frequencies coinciding with the 1st and 2nd harmonics, related to bending around the “X” and “Z” axis.
respectively. The highest amplitude obtained is highlighted, related to the transversal acceleration when the wind acts at 90º (0.09 m/s²), indicating that this is the critical behaviour of the structure in terms of maximum accelerations.

**Figure 14** - Accelerations in the frequency domain; a) wind in the 0º and b) wind in the 90º direction - rigid support (Model I).

6.2 Non-deterministic dynamic analysis of Model II

Table 8 presents the parameters used to generate the wind series with non-deterministic characteristics for Model I. Noting that, as in Model I, the representative basic wind speed in the region of the city of Rio de Janeiro occurs every 50 years, with a duration of 3 seconds, according to the NBR 6123 (1988) standard. And as for the zone, it is an open area with few obstacles resulting, therefore, in category II. The probabilistic factor was obtained to satisfy the recurrence period of 10 years, according to NBR 6123 (1988). A duration time of 10 minutes was considered, which corresponds to 600 seconds, which is usually considered to study the effects of wind on structural systems.

| Índice de resistência a penetração | Designação das características do vento |
|-----------------------------------|----------------------------------------|
| Base wind speed                   | 45 m/s                                  |
| Speed adopted at the top of the building | 35,79 m/s                              |
| Recurrence time                   | 10 years                                |
| Drag coefficient in the “X” direction | 1,58                                   |
| Drag coefficient in the “Z” direction | 1,00                                   |
| Land category                     | II                                     |
| Parameters for roughness factor   | b = 1 e p = 0,15                        |
| Probability factor                | 0,76 (for 10 years)                    |
| Duration                          | 600 seconds                            |
| Time increment                    | 0,03 seconds                           |

Source: Authors.

Figure 15 presents the dynamic behaviour of the structure (Model II) in terms of acceleration, obtained considering the wind in the 0º direction applied in the simulated model on a rigid support. It is worth noting that in the dynamic behaviour, the longitudinal peak acceleration is 41% higher than the normative limit indicated by NBR 6123 (1988), $[a_{pl} = 0,144 \text{ m/s}^2$ and $a_{lim}=0,10 \text{ m/s}^2]$. 

![Figure 14](image-url)
Figure 15 - Longitudinal acceleration in the time; a) and frequency b) domains; wind in the 0º, rigid support (Model II).

The spectrum referring to the dynamic response in the frequency domain is obtained from the use of the fast Fourier transform (FFT), through the use of the MATLAB program. Then shown in Figure 15b is the frequency domain acceleration for the response discussed in Figure 15b. Note that the highest energy transfer of the system is verified around the first harmonic (f₀₁ = 0,316 Hz).

Figure 16 illustrates the behaviour of the structure when it is subjected to wind loading applied in the 0º direction. It is possible to observe the two accelerations (longitudinal and transversal) being evaluated simultaneously. In the permanent phase, the maximum longitudinal peak acceleration 447% higher than the maximum transverse peak acceleration (aₘₜᵢ₇₉ = 0,185 m/s² and aₘₜᵢ₇₉ = 0,034 m/s²). It is important to remember that the building under study has high slenderness around the “X” axis, hence this significant difference.

The frequency-domain response for this dynamic response is illustrated in Figure 16b. The energy peak referring to the longitudinal response coincides with the harmonic associated with bending around the “X” axis (f₀₁ = 0,316 Hz), while the transverse peak coincides with the harmonic related with the “Z” axis (f₀₃ = 0,48 Hz).

Figure 16 - Acceleration in the time (a) and frequency (b) domains, wind in the 0º direction - rigid support (Model II).

For the wind that acts at 90º (applied to the smallest façade), the transversal response becomes preponderant, as can be seen in Figure 17a. The peak transverse acceleration is 330% greater than the peak longitudinal acceleration (aₘₜᵢ₇₉ = 0,096 m/s² and aₘₜᵢ₇₉ = 0,026 m/s²). It is important to highlight, comparing the maximums obtained for each wind direction considered, that the predominant acceleration is the longitudinal one with the wind in the 0º direction, it is 87% superior to the transversal...
acceleration obtained with the wind at 90º \( (a_{90} = 0.185 \text{ m/s}^2) \). This finding indicates that despite the transverse acceleration being predominant in several cases, for the model under study the longitudinal acceleration with wind acting on the largest facade is the highest value found. The frequency-domain response is shown in Figure 17b.

![Figure 17 - Acceleration in the time (a) and frequency (b) domains, wind in the 90º direction - rigid support (Model II).](source)

Source: Authors.

7. Conclusion

This investigation presents the results of a dynamic analysis of forced vibration performed based on two three-dimensional finite element models developed to represent a real and existing residential building of 173 m and 140 m, respectively, when subjected to wind loads with non-deterministic characteristics. The turbulent wind speed in both cases was calculated by summing a finite number of superimposed harmonics. The evaluations of human comfort in the two buildings were carried out from comparisons between the peak accelerations and the recommended limits proposed by the Brazilian Standard NBR 6123 (1988).

As for the wind speed power spectrum, these are in agreement with the respective mathematical formulation, indicating that the developed analysis methodology leads to a speed signal close to the natural wind speed. On the other hand, using the wind speeds from two different points of the structure, it was possible to obtain the coherence between their speeds, indicating a good agreement with the coherence function formula. Based on the results of Model I of the modal analysis, the value of 0.15 Hz \( (f_{01} = 0.15 \text{ Hz}) \) was obtained for the fundamental frequency of the structural system which, for the design standard NBR 6123 (1988), is necessary to carry out the analysis of forced vibration of the structure, as for Model II, in the same analysis, the value of 0.29 Hz \( (f_{01} = 0.29 \text{ Hz}) \) was obtained for the fundamental frequency of the structure, which is recommended, according to the NBR 6123 (1988) standard, the performance of forced vibration analysis.

As for the dynamic structural analysis, it can be said that to the suction effect that occurs on the facades of buildings, as opposed to the action of the longitudinal wind, the results obtained indicated an increase in the values of accelerations when this effect is considered. The values of maximum longitudinal accelerations are on the order of 25% to 30% higher than the values of accelerations obtained based on wind loading, in which the suction effect is not considered in the mathematical formulation. This dynamic behaviour was verified in the structural designs of the two buildings investigated.

Regarding the transverse responses versus the longitudinal responses, in terms of the values of accelerations, the dynamic analyzes carried out in the “steel-concrete” mixed structure building with 173 m (Model I) and in the reinforced concrete building with a total height of 173 m 140 m (Model II), showed that the transversal response was predominant. When the wind is in the direction of 90º and referring to the first mode of vibration of bending around the global axis “X” \( (f_{01} = 0.15 \text{ Hz}) \), indicating that this is the critical effect for the evaluation of the human comfort of the structure. The highest value
obtained, transverse peak acceleration with the wind in the 90° direction, was 13% higher than the longitudinal peak acceleration value when the wind acts at 0° (\(a_{pl} = 0.40 \text{ m/s}^2\)).

In the investigation of the reinforced concrete model with a height of 140 m (Model I), it is highlighted that the maximum acceleration did not occur in the transverse direction of the structure, but in the direction of the wind at 0° "longitudinal" (\(a_{pl} = 0.24 \text{ m/s}^2\)), relevant to the first mode of flexural vibration around the global axis “X”, considering the modelling of foundations (effect of soil-structure interaction).

In the study of the mixed structure model (steel-concrete) with a total height of 173 m, the modal analyzes showed that the building is very flexible in the two main directions (\(f_{l1} = 0.15 \text{ Hz} \) and \(f_{l3} = 0.19 \text{ Hz}\)) and This finding implied an inadmissible dynamic behaviour since the accelerations obtained in the transient analyzes indicated that the human comfort criteria were not met, with maximum accelerations much higher than the evaluation limits. The dynamic behaviour is evidenced when the wind acts in the direction of 90°, based on the peak accelerations, in which the transverse accelerations had such high values that they practically approached the values that classify the vibrations as very uncomfortable. The highest value obtained is only 6.7% lower than the limit proposed by Hirsch and Bachmann (1995) (\(a_{pl} = 0.46 \text{ m/s}^2\) and 0.49 \(\text{ m/s}^2\)).

Finally, concerning the evaluations of human comfort, it can be stated that meeting the service limit state considering only a static analysis to verify the maximum horizontal translational displacements may not be sufficient. Keeping in mind only the static analysis, Model I met the normative requirements regarding the maximum horizontal translational displacements at the top of the building. On the other hand, the dynamic behaviour in terms of accelerations showed that the human comfort criteria were not met. The dynamic analyses, considering the non-deterministic action of the wind, carried out in Model II, indicated peak acceleration values in the longitudinal and transverse direction above the limits of 0.10 \(\text{ m/s}^2\) proposed by the NBR 6123 (1988) standard (\(a_{pl} = 0.24 \text{ m/s}^2 > a_{lim} = 0.10 \text{ m/s}^2\)).

As suggestions for future work, it is expected that: i) perform an experimental monitoring strategy on real buildings of modes of comparison to dynamic response based on the use of accelerometers with response with finite element appearances, aiming to evaluate the accuracy of the models of structural systems. ii) use research like this, the geometry tunnel equipment to study the relationship between a dynamic and transversal response for a more expressive number of models in order to cover different dimensions and proportions of the system. iii) verification of the possibility of using finite and structure-specific study programs, as well as the intuitive elements of graphic representation of the effect of individual interaction.

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