XLVI CILAMCE
VITÓRIA - ES - BRAZIL - 2025
XLVI IBERO-LATIN AMERICAN CONGRESS ON COMPUTATIONAL METHODS IN ENGINEERING

Assessment of high buildings human comfort subjected to wind actions based on simplified design formulations and a nondeterministic dynamic structural analysis

George Lucas S. Quintanilha[1], Augusto Cesar S. Campos[1], Glauco José O. Rodrigues[1], José Guilherme S. da Silva[1]

[1]*Civil Engineering Postgraduate Programme (PGECIV). State University of Rio de Janeiro (UERJ).*
São Francisco Xavier St., Nº 524, Maracanã, 20550-900, Rio de Janeiro/RJ, Brazil.
eng.georgequintanilha@gmail.com, augusto_micro@hotmail.com, glauco.jose.rodrigues@uerj.br, jgss@uerj.br

Abstract. The requirements for space in major urban centres have driven the development of structural designs for high-rise buildings. Consequently, these structures are more susceptible to wind-induced effects, which may lead to excessive vibrations and occupant discomfort. This way, this investigation aims to evaluate the human comfort levels in a 48-storey steel-concrete composite building with total height of 172.8 m, when subjected to wind loads. The mean and fluctuating wind responses were determined in accordance with the recommendations of the Brazilian standard NBR 6123, considering the design wind velocity of 35 m/s (126 km/h). The numerical model of the building was developed using the Finite Element Method (FEM), implemented through the ANSYS computational software. The assessment of the investigated building human comfort was based on the maximum accelerations values, in time and frequency domain, considering several design standards and simplified formulations, and also a nondeterministic dynamic structural analysis. The investigated scenario allows a general comparative assessment of the results calculated based on the use of simplified formulations and through a nondeterministic structural response. Finally, it must be emphasized that the structural engineers have to be alerted about the relevant quantitative differences associated to the peak accelerations values determined by simplified formulations and based on a traditional nondeterministic dynamic structural analysis.

Keywords: High-Rise Buildings; Dynamic Structural Analysis; Human Comfort Assessment.

1 Introduction

Global population expansion and the need to accommodate billions of people in densely populated urban areas have long driven the architectural exploration of the high-rise building typology. Consequently, when this construction methodology is adopted, a reduction in the natural frequencies of the structures is consistently observed, thereby making them more susceptible to induced vibrations. This inherent characteristic increases the structural vulnerability to wind-induced dynamic effects, frequently elevating human comfort to the prevailing criterion during the assessment of serviceability limit states [1, 2].

The accurate characterization of wind dynamic loads is a crucial parameter in the assessment of building human comfort. The ongoing construction of tall and slender buildings, driven by population growth and diminishing urban free spaces, has led to structural issues related to excessive vibrations and human discomfort. These problems are a direct consequence of the dynamic characteristics of wind loads. Consequently, it has become imperative to thoroughly investigate the interaction between wind actions and tall buildings, aiming to enhance structural design guides and mitigate potential future serviceability limit state problems [1, 2].

Various design codes and guides have been developed to provide directives for representing buildings modal characteristics, defining wind actions, and quantifying the resulting structural responses. A more in-depth examination of these recommendations is necessary. This is due to the potential for discrepancies in the results, particularly when comparing the human comfort assessment criteria adopted against more refined analyses, such as those derived from nondeterministic dynamic analysis [3].

Considering these diverse approaches, this study aims to assess the human comfort of a steel-concrete building. The structure stands at 172.8 metres in height, comprising 48 storeys, and features floor dimensions of 45 m x 32 m. Numerical modelling of the building will be performed utilising the Finite Element Method (FEM), and analyses will be developed based on the application of the ANSYS program [4]. This study concluded that the use of simplified design formulas for evaluating the dynamic structural response due to wind action can yield different results when compared to those obtained numerically. It is highlighted that maximum accelerations exhibited a reduction of 19% to an increase of up to 168%, depending on the standard or design guide adopted.

2 Modelling of the wind loads

Accurate consideration of wind characteristics is essential for structural design, as wind is an action that varies in intensity over time. This variation in speed induces oscillations, which can cause discomfort for occupants of these buildings. In this research, the wind flow is assumed to be unidirectional, stationary, and homogeneous. This assumption dictates that the primary flow direction remains constant across both temporal and spatial domains, and that the wind statistical characteristics are invariant throughout the simulation period [1].

In this research work, the wind pressure acting on the building's façades was assumed to be a direct function of wind velocity, aligning with the Davenport classic model adopted in the Brazilian standard NBR 6123 [5]. Subsequently, with the dynamic wind pressure (q) acting on the structure, the dynamic wind load (F), in N, was calculated at each investigated building structural section through eq. (1). In this equation, C_{Di} represents the drag coefficient in the 'i' direction, and A_i denotes the influence area in m². The drag coefficient (C_D) is contingent upon the relationships between the structure dimensions and can be determined via NBR 6123 [5].

$$F = C_{Di} q A_i \tag{1}$$

The dynamic wind pressure can be determined by the sum of two components: a turbulent parcel (nondeterministic dynamic load) and a static parcel (mean wind force). Its magnitude over time can be calculated according to eq. (2), where q(t) is the dynamic wind pressure in N/m², and $\overline{V}$ is the mean wind velocity in m/s.

$$q(t) = 0.613[\overline{V} + v(t)]^2 \tag{2}$$

The turbulent component of the wind is decomposed into a finite series of harmonic functions, each characterised by a randomly determined phase angle. The amplitude of each harmonic is derived using a Kaimal Power Spectrum function, considering the influence of building height on the dynamic response [1]. The energy spectrum is calculated using eq. (3) and (4), where f represents the frequency in H, S^V denotes the spectral density of the wind turbulent longitudinal component in m^2/s, x is a dimensionless frequency, $\overline{V}_z$ signifies the mean wind velocity relative to the height in m/s, and z is the height in metres.

The friction velocity u* is determined according to eq. (5), in m/s, with the Karmán constant (k) set at 0.4 and z_0 denoting the roughness length in metres. The turbulent component of wind velocity v(t) is modelled as a random process, derived from the summation of a finite number of harmonics, as delineated in eq. (6), where N corresponds to the number of power spectrum divisions, f is the frequency in Hz, Δf represents the frequency increment, θ is the random phase angle uniformly distributed within the range of [0-2π], and t is the time in seconds.

$$\frac{f S^V(f,z)}{u^{*2}} = \frac{200x}{(1+50x)^{5/3}} \tag{3}$$

$$x(f,z) = \frac{f\,z}{\overline{V}_z} \tag{4}$$

$$u^* = \frac{k\overline{V}_Z}{ln\,(z/z_0)} \tag{5}$$

$$v(t) = \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f}\cos\,(2\pi f_i t + \theta_i) \tag{6}$$

Considering the formulation of the turbulent wind portion, eq. (1) can be rewritten to account for wind action over time. In this context, V_0 is the basic wind velocity and p is the exponent of the potential law of variation of the S_2 factor as defined in the NBR 6123 [5]. The dynamic loads are applied to the building façade for 10 minutes, subsequently yielding the respective nondeterministic responses.

$$F(t) = 0.613 c_D A_i \left[V_0 \left(\frac{z}{z_0}\right)^p + \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f} \cos(2\pi f_i t + \theta_i) \right]^2 \quad (7)$$

In contrast, NBR 6123 [5] presents two methods for calculating the dynamic response: the simplified method and the discrete method. This study will utilise the simplified method. Within this approach, the variation of dynamic pressure with height is defined by eqs. (8) to (10) where $\overline{q}_0$ represents the dynamic pressure, z denotes the height above ground level, z_r is the reference height, h is the total height in metres, ξ corresponds to the dynamic amplification coefficient, p and b are parameters contingent on the terrain roughness, $\overline{V}_p$ is the wind design velocity, S_1 is the topographic factor, and S_3 is the statistical factor associated to the probability of destruction.

$$q(z) = \overline{q}_0 \, b^2 \left[\left(\frac{z}{z_r}\right)^{2p} + \left(\frac{h}{z_r}\right)^p \left(\frac{z}{h}\right)^g \frac{1+2g}{1+g+p} \, x \right] \quad (8)$$

$$\overline{q}_0 = 0.613 [\overline{V}_p]^2 \quad (9)$$

$$\overline{V}_p = 0.69 V_0 S_1 S_3 \quad (10)$$

The initial parcel within the square brackets in eq. (8) represents the mean response, while the second parcel signifies the maximum amplitude of the fluctuating response. Based on the dynamic wind pressure, eq. (1) can be rewritten to calculate the static wind loads. This load is applied along the building facade, which yields the maximum displacement. Finally, the maximum acceleration at a given height can be calculated according to eq. (12), where f_j represents the fundamental frequency of the building and u_j denotes the displacement at that height. In this research, both the total responses and those associated solely with the fluctuating part will be defined.

$$F(z) = c_D A_i \overline{q}_0 \, b^2 \left[\left(\frac{z}{z_r}\right)^{2p} + \left(\frac{h}{z_r}\right)^p \left(\frac{z}{h}\right)^\gamma \frac{1+2\gamma}{1+\gamma+p} \, \xi \right] \quad (11)$$

$$a_j = 4\pi^2 f_j^2 u_j \quad (12)$$

3 Investigated steel-concrete composite building

The investigated building comprises 48 floors, each with a height of 3.6 m, resulting in a total structural height of 172.8 m. The plan dimensions are 45 m x 32 m, incorporating a central core measuring 27 m x 9 m. W460x106 steel profiles form the main beams, while W410x60 profiles are utilised for the secondary beams. The steel employed for the beams is the conventional ASTM A572. The concrete slab has a thickness of 15 cm, and the steel columns, made from HD profiles (ASTM A913 steel), have their geometric characteristics detailed in tab. (1). Figure 1 illustrates the floor plan of the examined steel-concrete building, with dimensions in metres.

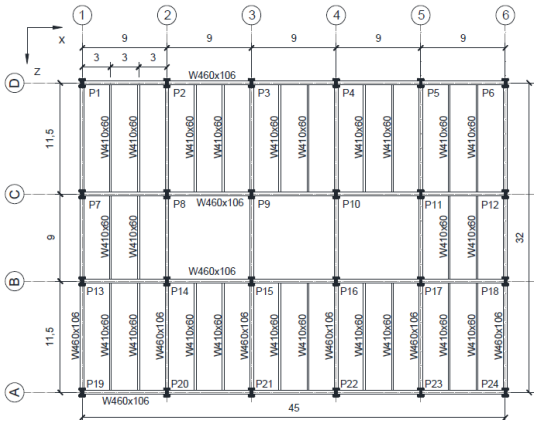


Figure 1. Structural project of the steel-concrete building: H = 172.8m

The concrete exhibits a compressive strength (fck) of 30 MPa, modulus of elasticity (Ecs) of 26 GPa, Poisson's ratio (ν) of 0.2, and specific weight (γc) of 25 kN/m³. The steel presents a characteristic strength (fy) of 345 MPa, modulus of elasticity (Es) of 205 GPa, Poisson's ratio (ν) of 0.3 and specific weight (γs) of 78.5 kN/m³.

Table 1. Investigated structural model: steel profiles

Floor	Centre core columns	Façade columns
1º to 10º	HD400x990	HD400x551
11º to 20º	HD400x818	HD400x382
21º to 30º	HD400x667	HD320x245
31º to 40º	HD400x421	HD260x172
41º to 48º	HD400x187	HD260x114

4 Finite element modelling

The steel-concrete composite building was investigated utilising the ANSYS program [5]. This involved the application of standard discretisation techniques inherent to the Finite Element Method (FEM). Furthermore, the finite element model satisfies the mesh convergence study previously conducted [1]. Three-dimensional BEAM44 finite elements were utilised to represent the steel beams, columns, and piles, thereby considering both bending and torsional effects. The building's concrete slabs were simulated using SHELL63 shell finite elements. The foundation block was represented by SOLID45 solid elements. Furthermore, COMINB14 elements were employed to model the soil spring coefficients. The building foundation was specifically modelled to incorporate the soil-structure interaction effect within the dynamic analysis. Subsequently, fig. (2) illustrates the developed steel-concrete composite building finite element model.

In this study, full interaction between the concrete slabs and steel beams was assumed. The finite element nodes were interconnected to prevent any slippage. Both steel and concrete materials exhibit linear elastic behaviour, ensuring all structural sections maintain planarity in their deformed configuration. The resultant numerical model of the building incorporated 689,700 nodes and 164,274 elements, culminating in a finite element model possessing 3,120,888 degrees of freedom.

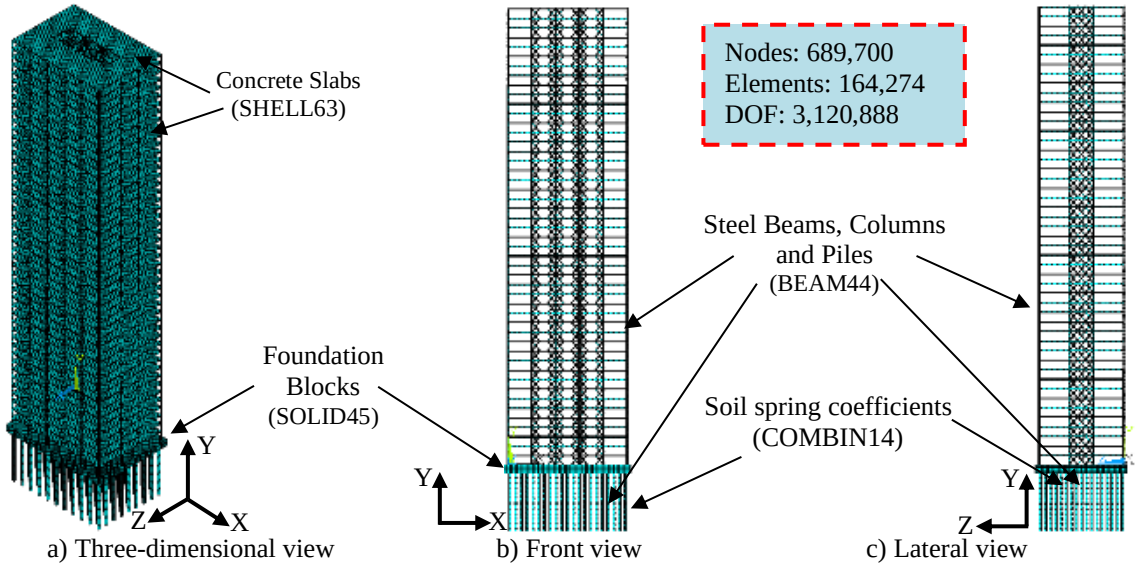


Figure 2. Steel-concrete building finite element model: H = 172.8m

5 Modal analysis: eigenvalues and eigenvectors

The building natural frequencies (eigenvalues) and associated vibration modes (eigenvectors) were determined through numerical extraction methods (modal analysis). This was accomplished via a free vibration analysis performed in the ANSYS program [4]. Figure 3 presents the first four natural frequencies and corresponding vibration modes of the structure. The mode shapes provide an indication of the building vibrational tendencies. Maximum modal amplitude is denoted by red colour and the minimum amplitude is depicted in blue.

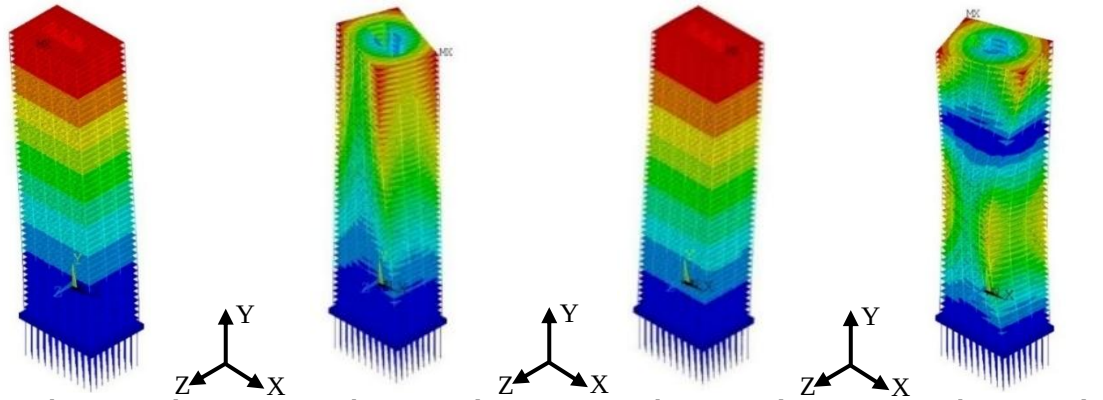


Figure 3. Vibration modes of the investigated steel-concrete building

The fundamental frequency of the steel-concrete building is determined to be 0.161 Hz (f_{01} = 0.161 Hz). This finding holds significant relevance, as per the Brazilian standard NBR 6123 [5], structures exhibiting natural frequencies below 1 Hz, particularly those with low structural damping, are susceptible to considerable along-wind fluctuating dynamic responses, indicating potential excessive vibrations. In addition to the frequencies and vibration modes derived using ANSYS [4], tab. (2) provides equations recommended by the NBR 6123 [5], Hirsch and Bachmann [6], and Eurocode 1-4 [7] for estimating a building fundamental frequency. This enables a comparison of different design recommendations with the numerical results.

Table 2. Fundamental frequency according to design standards

Reference	Abbreviation	Equation or methodology	Frequency
ANSYS [4]	ANS	Numerical model	0.161 Hz
NBR 6123 [5]	NBR	$f_1 = \frac{1}{0.29\sqrt{h} - 0.4}$	0.293 Hz
Hirsch & Bachmann [6]	H&B	$f_1 = 0{,}4\left(\frac{100}{h}\right)^{1.6}$	0.167 Hz
Eurocode 1-4 [7]	ECD	$f_1 = \frac{46}{h}$	0.266 Hz

6 Dynamic structural analysis

To evaluate the structural response, the load was applied against the building façade. Firstly, a nondeterministic analysis was performed based on sixty dynamic load series. Following, a statistical treatment was conducted to obtain the mean maximum displacements and accelerations. The parameters used to determine the wind series are the wind basic velocity (V_0) of 35 m/s (126km/h); terrain category IV; recurrence time of 1 years; topographic factor (S_1) 1; probability factor (S_3) 0.54; roughness Factor (S_2) b = 0.84, p = 0.135 and Fr=0.69.

The assessment of structural response according to design standards was carried out based on the equations of the simplified method presented by NBR 6123 [5]. Static loads were obtained through eq. (11) and horizontal displacements were acquired through a static analysis performed in ANSYS [4]. Subsequently, maximum accelerations were calculated according to eq. (12), which considered the fundamental frequencies proposed in tab. (2). The parameters used in the definition of the wind loads consist of the basic wind velocity (V_0) of 35 m/s (126km/h); terrain category IV, p = 0.23 and b_m = 0.71; recurrence time of 1 years; topographic factor (S_1) 1; probability factor (S_3) 0.54; dynamic amplification coefficient (ξ) 3; and dynamic effects parameter γ = 1.2.

The maximum peak accelerations were recorded at the penultimate floor (H = 169.2 m), encompassing responses for both the total load and the fluctuating wind load. Human comfort assessment was conducted based on the limit acceleration values specified by NBR 6123 [5], which are calculated according to eq. (13). The k_c

value of 4.08 is applicable for residential buildings and 6.12 for commercial buildings, with f representing the fundamental frequency. Additionally, the perception levels stipulated by Hirsch and Bachmann [6] and ISO 2631 [8] were verified, these limits are presented in tab. (3) and (4) respectively.

$$a_{lim} = 0.01 k_c f^{-0.445} \tag{13}$$

Table 3. Human perception according to Hirsch and Bachmann [7]

Human perception	Acceleration limit values (m/s²)
Imperceptible (I)	a < 0.049
Perceptible (P)	0.049 < a < 0.147
Uncomfortable (UN)	0.147 < a < 0.49
Very uncomfortable (VU)	0.49 < a < 1.47
Intolerable (IN)	1.47 < a

Table 4. Comfort assessment criteria according to ISO 2631 [8]

Acceleration limit values	Human perception
Less than 0.315m/s^2	Not uncomfortable (NU)
0.315 m/s^2 to 0.63m/s^2	A little uncomfortable (LU)
0.5m/s^2 to 1m/s^2	Fairly uncomfortable (FU)
0.8m/s^2 to 1.6m/s^2	Uncomfortable (UN)
1.25m/s^2 to 2.5m/s^2	Very uncomfortable (VU)
Greater than 2m/s^2	Extremely uncomfortable (EU)

According to the data presented in tab. (5), the results from the nondeterministic analysis were established as reference values (RV). Firstly, the fundamental frequencies derived from NBR 6123 [5] and Eurocode 1-4 [7] were greater than those obtained numerically, with increases of 82% and 65%, respectively. Regarding maximum horizontal displacements due to total wind action, the analytical results were 29% lower. However, the influence of the fluctuating component was more significant, constituting a more substantial proportion of the total response. Finally, maximum accelerations induced by the total load, considering the NBR 6123 [5] method, exhibited reductions of 19% and 13% when natural frequencies obtained from ANSYS [4] and Hirsch and Bachmann [6] were employed. Conversely, the response was considerably higher when derived via NBR 6123 [5] and Eurocode 1-4 (168% and 121% increase). The fluctuating component contribution was greater in all cases, reaching 654% difference compared with the nondeterministic analysis.

Table 5. Peak values of the dynamic structural response

Analysis method	Frequency (Hz)	Displacement (m)		Acceleration (m/s²)	
		Total	Fluctuating	Total	Fluctuating
Nondeterministic	ANS: 0.161[RV]	0.11[RV]	0.02[RV]	0.102[RV]	0.023[RV]
NBR 6123 [5]	ANS: 0.161[RV]	0.08[-29%]	0.05[+183%]	0.082[-19%]	0.053[+127%]
	NBR: 0.293[+82%]	0.08[-29%]	0.05[+183%]	0.272[+168%]	0.174[+654%]
	H&B: 0.167[+4%]	0.08[-29%]	0.05[+183%]	0.091[-13%]	0.056[+143%]
	ECD: 0.266[+65%]	0.08[-29%]	0.05[+183%]	0.225[+121%]	0.144[+161%]

Considering human comfort assessment based on peak acceleration values, tab. (6) demonstrates that the structural response derived from nondeterministic analysis is not acceptable (NA) by the criterion adopted by NBR 6123 [5] for residential buildings (a_{lim} = 0.092 m/s²) and acceptable (A) for office building (a_{lim} = 0.138 m/s²). This response is classified as perceptible (P) by Hirsch and Bachmann [6] and not uncomfortable (NU) according to ISO 2631 [8]. When applying the NBR 6123 method [5], if the standard's own frequency (f_{01} = 0.293 Hz) or the Eurocode 1-4 [7] frequency (f_{01} = 0.266 Hz) is utilised, the results exceed the recommendations for both residential (a_{lim} = 0.07 m/s²) and commercial (a_{lim} = 0.11 m/s²) buildings, classified as uncomfortable (UN) by Hirsch and Bachmann [6]. When the frequency is defined through modal analysis (f_{01} = 0.161 Hz) or by Hirsch and Bachmann [6] (f_{01} = 0.167 Hz), the accelerations satisfy the analysed criteria.

Table 6. Assessment of human comfort: peak acceleration

Analysis method		Total acceleration (m/s^2)	NBR 6123 [$a_{lim}(m/s^2)$] Residential		Office		Hirsch and Bachmann	ISO 2631
Nondeterministic		0.102	0.092	NA	0.138	A	P	NU
NBR 6123 [5]	Frequency ANS	0.082	0.092	A	0.138	A	P	NU
	Frequency NBR	0.272	0.070	NA	0.106	NA	UN	NU
	Frequency H&B	0.088	0.091	A	0.136	A	P	NU
	Frequency ECD	0.225	0.074	NA	0.110	NA	UN	NU

7 Conclusions

The conclusions of this research paper relate to the assessment of human comfort in tall buildings, considering different approaches to defining wind dynamic action. Comparisons were drawn between established design standards and the adoption of nondeterministic loads. The following conclusions can be stated, based on the results associated with the investigated building. (H=172.8 m; total mass: 4.56×107 kg; stiffness: 1176 kN/m):

1. The definition of the fundamental frequency through design standards or recommendations can yield overestimated values when compared to numerical analysis. In this research, the frequency derived from NBR 6123 [5] and Eurocode 1-4 [7] exhibited discrepancies reaching up to 82% when compared to numerical results.

2. The responses obtained from the nondeterministic analysis and from the analytical procedure prescribed in NBR 6123 [5] exhibited notable discrepancies. The maximum horizontal displacement was 29% lower when the static loadings defined by the design standard were considered. In relation to maximum accelerations, their estimation is governed by the fundamental frequency; consequently, the method adopted for its definition has a direct impact on the results. Therefore, the acceleration obtained through numerical analysis was 0.102 m/s^2, while under the simplified procedure the values ranged from 0.082 m/s^2 (f_{01} = 0.161 Hz; frequency defined through modal analysis) to 0.272 m/s^2 (f_{01} = 0.293 Hz; frequency defined according to NBR 6123 [5]).

3. Based on the analyses conducted, it is concluded that the assessment of human comfort of tall buildings through the recommendations of design standards and guidelines must be approached with caution, as substantial discrepancies were identified in the results obtained. Therefore, the adoption of more advanced analytical procedures is essential to provide an accurate evaluation of dynamic responses, ensuring occupant comfort while simultaneously enabling an optimised structural model.

Acknowledgements. The authors gratefully acknowledge the financial support for this investigation provided by Brazilian Science Foundation's CAPES, CNPq and FAPERJ.

Authorship statement. The authors hereby confirm that they are the sole liable persons responsible for the authorship of this work, and that all material that has been herein included as part of the present paper is either the property (and authorship) of the authors, or has the permission of the owners to be included here.

References

[1] Quintanilha, G. L., Study of nondeterministic dynamic structural behaviour and assessment of human comfort in tall buildings. Master's Dissertation. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2024.
[2] Silva, J. C. M., Assessment of the nondeterministic dynamic structural behaviour of tall buildings considering the effect of geometric nonlinearity and aerodynamic damping. PhD Thesis. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2025.
[3] Bastos, L. S., Study of structural behavior and human comfort analysis of reinforced concrete buildings. Master's Dissertation. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2015.
[4] ANSYS Swanson Analysis Systems Inc. Theory Reference (2022 R1), 2022.
[5] NBR 6123. *Forças devidas ao vento em edificações* (In Portuguese). Associação Brasileira de Normas Técnicas. Rio de Janeiro/RJ, Brazil, 2023.
[6] Bachmann, H. et al., *Vibration problems in structures: Practical guidelines*, Birkhauser, Basel, 1997.
[7] EN 1991-1-4:2005+A1. Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions, 2010.
[8] International Organization for Standardization. ISO 2631-1: Evaluation of human exposure to whole-body vibration – Part 1: General requirements, Switzerland, 1997.