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Nonlinear dynamic structural analysis of tall buildings considering the nondeterministic wind-induced actions

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Abstract. In recent decades, technological advances in civil construction have driven the project and construction of tall buildings in several countries around the world. However, the structural design slenderness increasing has been crucial for reducing the natural frequencies values and the damping structural levels generating, in some situations, excessive vibrations and human discomfort. Aspects generally disregarded in current day-to-day design practice are related to the geometric nonlinearity effect and the soil-structure interaction on the tall buildings structural response. Thus, this investigation aims to evaluate the dynamic structural behaviour of a steel-concrete composite building with 48 floors and 172.8 m height, when subjected to wind nondeterministic actions, including in the dynamic analysis the geometric nonlinearity effects and the soil-structure interaction. The investigated building numerical model was developed to obtain a more realistic representation of the structural system, based on the Finite Element Method (FEM), using the ANSYS program. In this investigation, the found results, based on the calculated displacements and accelerations values, have indicated relevant differences, when the geometric nonlinearity effects and the soil-structure interaction were included in the building dynamic response assessment.

Keywords: tall buildings, geometric nonlinearity, soil-structure interaction, buildings dynamic analysis.

1 Introduction

The action of wind loads on buildings was never a problem for low and heavy constructions, composed of thick masonry walls, but it became, to an increasing extent, when these buildings became taller and slender and the structural systems composed of materials lighter and lighter like observed by Silva; Blessmann; Bastos [1,2,3].

Currently, tall building projects increasingly have used simple structural systems, which promote agility in their assembly, cost reduction and greater flexibility in the use of built spaces as reported by Silva; Barboza [1, 4]. On the other hand, based on the use of this construction methodology it has been observed a reduction in the natural frequencies of these structures, generating more sensitivity to the wind dynamic effects, and this way the human comfort is frequently the prevailing criterion when the serviceability limit states are considered according to Ferreira [5]. In this context, the effects of geometric nonlinearity and the soil-structure interaction become important in conjunction with the nondeterministic wind dynamic actions on tall buildings.

According Corelhano [6], most of the structures cannot be considered linear, particularly under severe loading conditions. It is precisely under these severe loading conditions that a linear structural analysis is found to be inadequate and a more elaborate nonlinear analysis must be performed. As reported by Silva [7], in the design of tall buildings, the geometric nonlinearity effect becomes relevant when the structure is simultaneously loaded by vertical and horizontal actions (wind actions). This is because the load acting on the deformed structural system can induce higher values of efforts when compared to those calculated based on a linear analysis. In rigid structures, these effects are small and can usually be neglected. However, when flexible structures are considered such effects become significant and must be investigated according to Pinto [8].

The soil has great complexity attributable to the varied characteristics such as heterogeneity, anisotropy, nonlinear behaviour between force and displacement and property changes with varying amount of water in its constitution. Consideration of soil-structure interaction over foundations can provide significant differences in building calculations. Hence, due to the dependence between these elements (soil and structure), it is important to obtain by numerical analysis the effect of this interaction like observed by Borges [9].

In this research the study of buildings dynamic structural response when subjected to nondeterministic wind action considering the effect of geometric nonlinearity and the soil-structure interaction is investigated. Thereby, throughout the dynamic analysis, the design of a steel-concrete building is analysed. The building is 172.8 m height, presenting 48 storeys and floor dimensions of 45 m by 32 m. The numerical modelling of the building will be performed using the Finite Element Method (FEM), and linear and nonlinear geometric analyses are carried out based on the use of the ANSYS program [10]. Based on the dynamic analysis of the investigated building, the dynamic structural response (natural frequencies, displacements and acceleration) will be evaluated when considering the effects of geometric nonlinearity and soil-structure interaction.

2 Nondeterministic wind mathematical modelling

Wind properties are unstable, present a random variation and therefore their deterministic consideration becomes inadequate. However, to generate nondeterministic dynamic load time series, it is assumed that the wind flow is unidirectional, stationary and homogeneous. This implies that the direction of the main flow is constant in time and space and that the statistical characteristics of the wind do not change when the simulation period is performed according to Obata [11].

In this investigation, dynamic wind loads are calculated by the sum of two parcels: one turbulent parcel (nondeterministic dynamic load) and the other static parcel (mean wind force). The turbulent part of the wind is decomposed into a finite number of harmonic functions with randomly determined phase angles as reported by Silva; Bastos; Barboza [1,3,4]. The amplitude of each harmonic is obtained based on the use of a Kaimal Power Spectrum function.

This research work adopts the Kaimal Power Spectrum by considering the influence of the building height on the dynamic response as reported by Bastos [3]. The energy spectrum is calculated using eqs. (1) and (2), where f is the frequency in Hz, S^V is the spectral density of the wind turbulent longitudinal part in m²/s, x is a dimensionless frequency, $\overline{V_z}$ is the mean wind velocity relative to the height in m/s and z is the height in meters.

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

$$x(f,z) = \frac{f\,z}{\overline{V}_Z} \tag{2}$$

The friction velocity u^*, in m/s, is calculated using eq. (3), with Karmán k constant equal to 0.4 and z_0 corresponding to the roughness length in m. The turbulent part of wind velocity v(t), is simulated based on a random process obtained from a sum of a finite number of harmonics eq. (4), where N corresponds the number of power spectrum divisions, f is the frequency in Hz, Δf is the frequency increment and, θ is the random phase angle uniformly distributed in the range of [0-2π] and t is the time in s.

$$u^* = \frac{k\overline{V}_Z}{\ln(z/z_0)} \tag{3}$$

$$v(t) = \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f}\cos(2\pi f_i t + \theta_i) \tag{4}$$

In this research work, it is assumed that the wind pressure acting on the structural system is directly a function of the wind velocity (Davenport classic model as observed by Silva; Bastos; Barboza [1,3,4]). This way, the wind pressure can be calculated according to eq. (5), where q(t) is the dynamic wind pressure in N/m² and $\overline{V}$ is the mean part of wind velocity in m/s. After that, with the wind dynamic pressure acting on the structure, it is possible to calculate the dynamic wind load along the time F(t), in N, at each investigated structural section of the building through eq. (6), where C_{ai} is the drag coefficient in the "i" direction and A_i is the influence area in m².

$$q(t) = 0.613\,[\overline{V} + v(t)]^2 \tag{5}$$

$$F(t) = C_{ai}\,q(t)A_i \tag{6}$$

The drag coefficient Ca depends on the relationships between the structure dimension and can be determined through the NBR 6123 [12]. Developing the eq. (6), the eq. (7) is obtained, where c_D is the drag coefficient corresponding to the angle of attack, $\overline{V}_0$ is the basic wind velocity, and p is exponent of the potential law of variation of the S_2 factor according NBR 6123 [12].

$$F(t)=0.613c_DA_i\left[\overline{V}_0\left(\frac{z}{z_0}\right)^p+\sum_{i=1}^{N}\sqrt{2S^V(f_i)\Delta f}\cos\left(2\pi f_it+\theta_i\right)\right]^2 \qquad (7)$$

3 Investigated steel-concrete composite building

The studied steel-concrete building presents 48 floors, each floor with height of 3.6 m and the structural system presents height of 172.8 m. The building has dimensions of 45 m long and 32 m wide (floor plan), and central core with dimensions of 27 m x 9 m. The main beams are made of W460x106 steel profiles and the secondary beams by W410x60 profiles. The steel is the ASTM A572. The concrete slab is 15 cm thick and the steel columns are made of HD profiles (steel ASTM A913), with all geometric characteristics presented in Tab. 1 by Silva; Bastos [1,2].

The concrete used in the model presents compressive strength (*fck*) equal to 30 MPa, modulus of elasticity (E_{cs}) of 26 GPa, Poisson's ratio (ν) of 0.2 and specific weight (γ_c) equal to 25 kN/m³. The used steel presents characteristic strength (*fy*) of 345 MPa, modulus of elasticity (E_s) of 205 GPa, Poisson's ratio (ν) of 0.3 and specific weight (γ_s) of 78.5 kN/m³. In sequence, Fig. 1 presents a floor plan of the studied steel-concrete building.

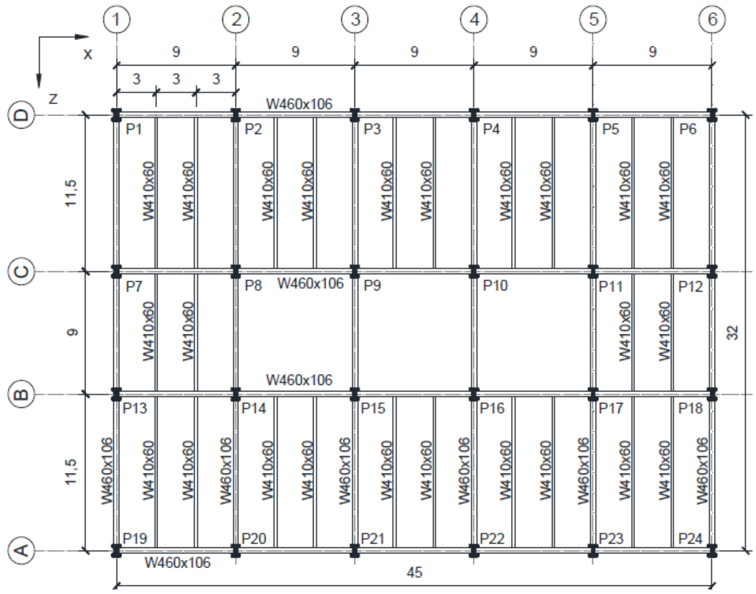


Figure 1. Structural project of the steel-concrete composite multi-storey building: H = 172.8 m

Table 1. Steel profiles of the investigated structural model

Floor	Centre core columns	Facade 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 numerical modelling

The steel-concrete composite building was investigated using the ANSYS program [10], based on usual discretization techniques associated with the Finite Element Method (FEM) (Fig. 2). The building finite element model satisfies the mesh convergence study previously performed by Silva; Bastos [1, 3]. Regarding the numerical modelling, the steel beams, columns and piles are represented based on the use of the three-dimensional finite elements BEAM44, where bending and torsional effects are considered. The concrete slabs of the building are simulated considering finite shell elements, using the SHELL63. The foundation block was discretized based on the use of the SOLID45 element. Soil spring coefficients are modelled using the COMINB14 element.

The complete interaction between the concrete slabs and the steel beams was considered in the study, and this means that the nodes of the finite element model are coupled to prevent the occurrence of slips. The material steel and concrete are considered to have elastic linear behaviour, and all structural sections of the model remain plane in the deformed state. The final computational model adopted used 689,700 nodes, 164,274 elements, which resulted in a numeric model with 3,120,888 degrees of freedom.

The strategy for numerical analysis uses the Newton-Raphson method for solving the equilibrium nonlinear equations system, which despite being more complicated in terms of calculation is adequate given the nonlinearity effect. The geometric nonlinearity was included using the Total Lagrangian Formulation, which allows large displacements and rotations.

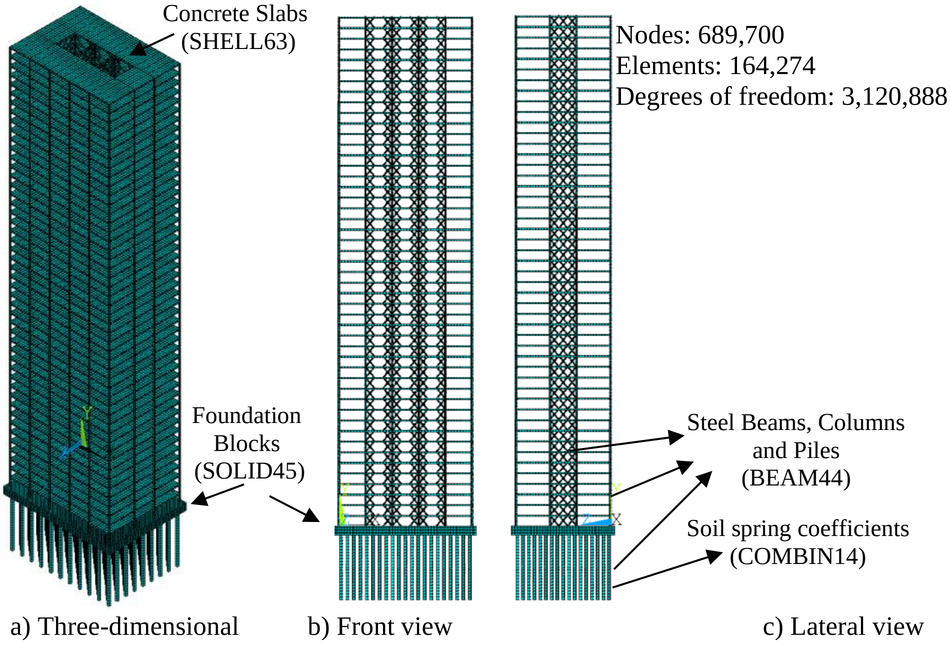


Figure 2. Steel-concrete composite building finite element model: H = 172.8 m.

5 Modal analysis: eigenvalues and eigenvectors

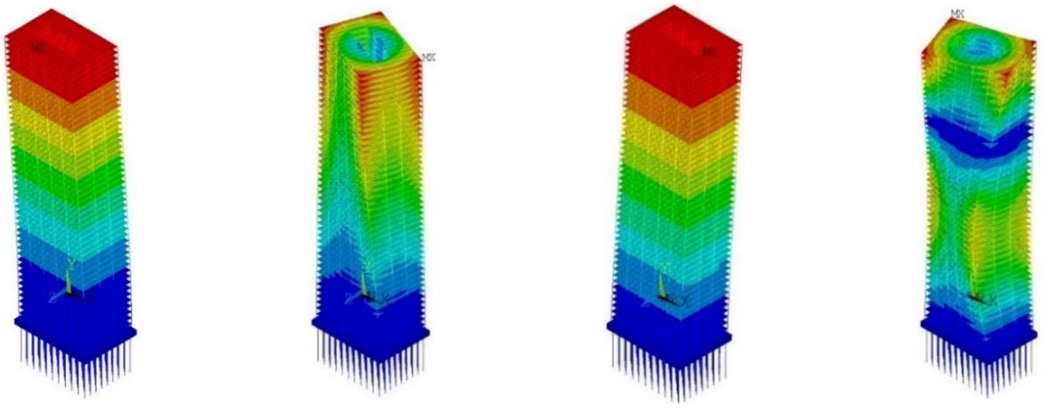
The building natural frequencies (eigenvalues) and the vibration modes (eigenvectors) were calculated using numerical extraction methods (modal analysis), through a free vibration analysis, utilizing the ANSYS program [10]. In this investigation, the conventional modal analysis (linear modal analysis) was performed, in which there is no load application on the structure. In addition, nonlinear modal analysis was also performed, that is, based on the use of prestressing loads. It is noteworthy that for the nonlinear modal analysis (prestressed), which aims to evaluate the effects of geometric nonlinearity on the eigenvalues and eigenvectors of the mixed building (steel-concrete), the structure is considered in its deformed position.

The utilised loads to provoke the building deformed condition are associated to the usual design loads (vertical loads: self-weight, permanent loads, overloads; and horizontal loads: static wind loads). For the calculation of static wind loads, intervals of 5 m/s [18 km/h] were considered, starting at 5 m/s [18 km/h] up to 45 m/s [162 km/h], covering most of the of basic wind velocities present in NBR 6123 [12].

The first four natural frequencies of the building are shown in Tab. 2 and the first four vibration modes are illustrated in Fig. 3. The mode shapes indicate the tendency of the building's vibration, the red colour indicates the maximum modal amplitude, and blue the minimum. It is noteworthy that only the vibration modes of the linear modal analysis were presented, since despite the existing differences on the values of the natural frequencies of the system, the vibration modes remained unchanged (linear and nonlinear modal analysis).

Table 2. Natural frequencies (f) of the FEM.

Frequency	Linear	Geometric nonlinear								
(Hz) V_0(km/h)	-	18	36	54	72	90	108	126	144	162
f_{01}	0.160	0.146	0.146	0.146	0.146	0.146	0.146	0.146	0.146	0.146
f_{02}	0.188	0.172	0.172	0.172	0.172	0.171	0.171	0.170	0.169	0.169
f_{03}	0.194	0.182	0.182	0.182	0.182	0.182	0.182	0.182	0.182	0.182
f_{04}	0.565	0.536	0.536	0.536	0.536	0.536	0.536	0.536	0.536	0.536



1º Vibration mode	2º Vibration mode	3º Vibration mode	4º Vibration mode
Bending around X	Torsion around Y	Bending around Z	Torsion around Y
f_{01} = 0.160 Hz	f_{02} = 0.188 Hz	f_{03} = 0.194 Hz	f_{04} = 0.565 Hz

Figure 3. Vibration modes of the investigated composite building

It is verified that the fundamental frequency value of the investigated steel-concrete composite building is equal to 0.160 Hz (f_{01} = 0.160 Hz), 10% higher than the value calculated in the nonlinear modal analysis (f_{01} = 0.146 Hz). This fact is very relevant because, in addition to the reduction in the value of the natural frequencies of the structure, due to the effects of geometric nonlinearity, according to the Brazilian design standard NBR 6123 [12], buildings presenting natural frequencies values lower than 1 Hz, particularly those that have low structural damping, may present relevant floating dynamic response along-wind, indicative of excessive vibrations.

6 Nondeterministic dynamic analysis

Considering the analysis methodology development for the building nonlinear dynamic structural response, besides the usual vertical design loads, the nondeterministic dynamic wind actions were applied over the building façade, having in mind the perpendicular direction to the 45 m side of the building, see Fig. 1. This way, the maximum horizontal displacements values were calculated at the building top (H = 172.8 m) and the maximum accelerations values were determined at last building floor storey (H = 169.2 m). In this research work, two types of analysis (forced vibration) were developed: linear and geometric nonlinear. In addition, 10 series of nondeterministic dynamic wind loading were generated, used for the statistical treatment of the response. Table 3 presents the parameters used to generate the wind series.

Table 3. Parameters used to generate the nondeterministic wind series

NBR 6123 design parameters [12]	Parameters used in the analysis
Wind Basic Velocity (V_0)	35 m/s [162 km/h]
Terrain Category	IV
Recurrence time	10 years
Topographic Factor (S_1)	1
Parameters for Roughness Factor (S_2)	b = 0.84, p = 0.135 and F_r=0.69
Probability Factor (S_3)	0.78
Time Duration and Time Increment	600 seconds and 0.1 seconds

Since the dynamic wind actions considered in this research work have nondeterministic characteristics, it is not possible to predict the response of the structure at a certain instant of time. A reliable response can be obtained through an appropriate statistical treatment using eq. (8), according to Walpole *et al.* [13]. Thus, considering that the dynamic structural response presents a normal distribution, and based on the calculation of the mean (μ) and also the standard deviation (σ), it is possible to obtain the characteristic value ($U_{Z95\%}$), that corresponds to a reliability of 95%, which means that only 5% of the sampled values will exceed this value as observed by Walpole *et al.* [13].

$$U_{Z95\%} = 1.96\ \sigma + \mu \tag{8}$$

Concerning the convergence of the numerical results of the dynamic structural analysis, based on the statistical treatment of the 10 nondeterministic wind loading series, Tab. 4 presents the mean maximum displacements (calculated at the top of the building) and mean maximum acceleration values (at the floor of the last building storey) of the building structural response. This way, aiming to evaluate the geometric nonlinearity effect on the building structural behaviour the mean, root mean square (RMS), peak and the mean of the ten most representative peak values were calculated considering the steady-state response.

Table 4. Mean maximum horizontal displacements [d (m)] and accelerations [a (m/s²)]: Z direction (see Fig. 1).

		Geometric nonlinear				Linear				% (NLG/L)			
		RMS	Peak	10 peaks	Mean	RMS	Pico	10 peaks	Mean	RMS	Peak	10 peaks	Mean
d (m)	μ	0.09	0.24	0.23	0.07	0.07	0.21	0.21	0.06	-	-	-	-
	σ	0.00	0.03	0.03	0.00	0.00	0.03	0.03	0.00	-	-	-	-
	$U_{z95\%}$	0.10	0.30	0.29	0.08	0.08	0.27	0.26	0.07	18	12	12	17
a (m/s²)	μ	0.06	0.20	0.18	0.05	0.05	0.16	0.15	0.04	-	-	-	-
	σ	0.01	0.02	0.02	0.00	0.00	0.02	0.02	0.00	-	-	-	-
	$U_{z95\%}$	0.07	0.23	0.21	0.06	0.06	0.20	0.19	0.05	21	15	14	24

Considering the results presented in Tab. 4, referring to the statistical treatment of the response (10 nondeterministic wind series), in general, it appears that important quantitative changes occur on the mean maximum values of the displacements and accelerations of the building , determined in the permanent phase of the response, when the effect of geometric nonlinearity is considered in the dynamic analysis of the model (forced vibration), with maximum differences in the range of 10% to 20% for horizontal translational displacements and 10% to 25% for the accelerations. It is also worth noting that the mean maximum values of the accelerations calculated in this study, in general, exceed the limit value recommended by NBR 6123 [12] (a_{lim} = 0.10 m/s²), violating the criterion of human comfort.

Figure 4 presents the geometric linear and nonlinear dynamic structural response of the building [V_0= 35 m/s (162 km/h)] in the frequency domain, where the difference between the values of the natural frequencies of the building is clearly verified. The results obtained in Fig. 4 consider the wind loading series that produced the values closest to the characteristic values of the system response, obtained via statistical treatment (Tab. 4). Furthermore, the effect of geometric nonlinearity produces modifications on the displacements and accelerations, considering the structure response energy transfer levels, when subjected to the wind actions.

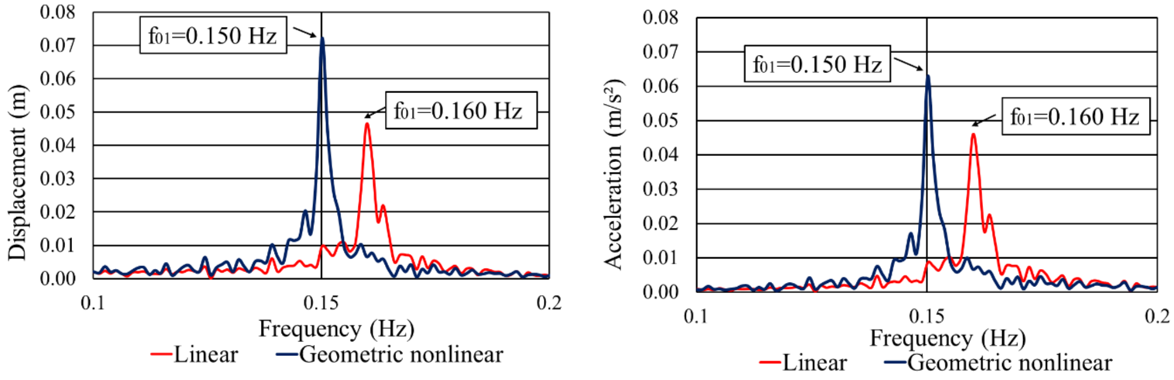


Figure 4. Dynamic response (frequency domain): displacements (H=172.8 m) and accelerations (H=169.2 m).

7 Conclusions

The main conclusions obtained in this research work are related to the tall buildings dynamic response modifications, subjected to nondeterministic wind dynamic actions, when the geometric nonlinearity effects and soil-structure interaction are considered in the analysis. The following conclusions can be stated, based on the results associated to the investigated building (H=172.8 m; total mass: 4.56×10^7 kg; stiffness: 1176 kN/m):

1. It is concluded that the building dynamic structural response was modified when the geometric nonlinearity effect was considered, with important modifications in the displacements and accelerations values. It is noteworthy that the mean maximum values of the accelerations calculated in this study (nonlinear analysis), in several different ways, exceed the limit value established by NBR 6123 (a_{lim} = 0.10 m/s^2), violating the criterion of human comfort [RMS: a_{RMS} = 0.07 m/s^2; Peak: a_p = 0.23 m/s^2; Mean of the 10 peaks: a_{10p} = 0.21 m/s^2; Mean: a_m = 0.06 m/s^2].

2. The consideration of geometric nonlinearity effects have produced relevant changes in the building dynamic response (displacements and accelerations), with maximum differences in the range of 10% to 20% for displacements and 10% to 25% for accelerations.

3. Based on the investigated building frequency domain dynamic analysis, it must be emphasized that the geometric nonlinearity effect has produced modifications on the displacements and accelerations values, considering the structure response energy transfer levels, when subjected to the wind actions.

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References

[1] J. C. M. Silva. *Study of nondeterministic dynamic structural behaviour and assessment of human comfort in tall buildings*. *(In Portuguese. In development).* PhD Thesis. Civil Engineering Postgraduate Programme. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2022.
[2] J. Blessmann. *Introduction to the study of the dynamic actions of the wind*. Publisher of the Federal University of Rio Grande do Sul. 2ª Edition, Porto Alegre/RS, Brazil, 2001.
[3] L. S. Bastos. *Assessment of the nondeterministic dynamic structural behaviour of tall buildings considering the across-wind effect and the modelling of the soil-structure interaction*. PhD Thesis. Civil Engineering Postgraduate Programme. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2020.
[4] R. R. Barboza. *Dynamic behaviour analysis, human comfort assessment and vibration control of tall buildings subjected to nondeterministic wind action based on consideration of the effect of soil-structure interaction*. PhD Thesis. Civil Engineering Postgraduate Programme. State University of Rio de Janeiro. Rio de Janeiro/RJ, Brazil, 2016.
[5] N. A. C. Ferreira. *Wind Effect in Tall Buildings - Application to a specific case*. MSc Dissertation. In: Civil Engineering Department, Faculty of Engineering, University of Porto, Porto, Portugal, 2008.
[6] A. G. B. Corelhano. *Nonlinear geometrical and physical analysis of cores of reinforced concrete tall buildings*. MSc Dissertation. School of Engineering of São Carlos. University of São Paulo, São Carlos, 2010.
[7] W. Q. Silva. *On geometric nonlinear analysis of tall buildings structures considering the warping of the structural cores and the soil-structure interaction*. PhD Thesis. School of São Carlos. University of São Paulo, São Carlos, 2014.
[8] R. da S. Pinto. *Physical and geometrical nonlinearity in design of usual reinforced concrete buildings*. MSc Dissertation. School of Engineering of São Carlos. University of São Paulo, São Carlos, 1997.
[9] A. C. L. Borges. *Methodology for assessing structural behaviour of tall reinforced concrete buildings with consideration of soil-structure interaction*. PhD Thesis. Postgraduate Program in Civil Engineering. Federal University of Pernambuco. Recife/PE, Brazil, 2009.
[10] ANSYS Swanson Analysis Systems Inc. Theory Reference (2022 R1), 2022.
[11] S. H. Obata, "Synthetic wind and Monte Carlo simulation - a way to consider the random characteristic of wind loading in structures". *Exacta*, vol. 7, p. 77-85, 2009.
[12] NBR-6123. *Forças devidas ao vento em edificações (In Portuguese).* Associação Brasileira de Normas Técnicas. Rio de Janeiro/RJ, Brazil, 1998.
[13] R. E. Walpole, R. H. Myers, S. L. Myers, K. Ye. *Probability & statistics for engineers & scientists*. Pearson Prentice Hall, 2009.