Influence of the geometric nonlinearity and the aerodynamic damping on the dynamic response of tall buildings

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Abstract. In recent decades, the increasing of slenderness in buildings structural design has been relevant leading to the reduction of the natural frequencies values and the damping levels, resulting in excessive vibrations and human discomfort, in some situations. Having in mind the current day-to-day project practice aiming to determine the dynamic structural response of tall buildings, two aspects are generally disregarded, associated to the effect of the geometric nonlinearity and the aerodynamic damping. Therefore, this research work aims to assess 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 effects of the geometric nonlinearity and the aerodynamic damping. The building numerical model was developed to obtain a realistic representation of the analysed structural system, based on the Finite Element Method (FEM), through the use of the ANSYS software. The conclusions of this study, obtained based on the displacements and accelerations values, pointed out to the fact that the effect of the geometric nonlinearity led to relevant differences on the dynamic structural response of the investigated building. On the other hand, the contribution of the aerodynamic damping was not significant.

Keywords: tall buildings, geometric nonlinearity, buildings dynamic analysis, aerodynamic damping.

1 Introduction

The growing urbanization and the search for efficient use of space have led to the proliferation of tall buildings in urban landscapes. The evolution of architecture and civil engineering made it possible to build increasingly impressive structures in terms of height and complexity. However, increasing the building's height brings with it significant challenges related to the structural dynamics, as previously observed [1,2,3].

As a result of the construction of tall buildings, there is the use of slender structural systems, with very low natural frequency values and, therefore, more susceptible excessive vibration problems, human discomfort and the fissures opening. The correct characterization of the structural model and wind loading used in the project is mandatory [1,4]. In this context, the effects of geometric nonlinearity and the aerodynamic damping can become important having in mind the effect of the nondeterministic wind dynamic actions on tall buildings.

Considering the design of tall buildings, the relevance of geometric nonlinearity can be significant when the structure is subjected to simultaneous vertical and horizontal forces (wind loads) [5,6]. This is because the load acting on the deformed structural system can induce higher force values compared to those computed 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.

The aerodynamic damping can be defined as a retarding force derived from the relative motion between the structure and the air [7]. Depending on the structure velocity, the dynamic response can be reduced due to the aerodynamic damping effect. In most cases, the structure velocity developed when excited by wind is low, which does not change the dynamic pressure values. However, considering flexible structural systems, these velocities can be relevant and may have a considerable impact on the dynamic pressure values.

This way, this work aims to assess the dynamic structural behaviour of a steel-concrete composite building with 48 floors and 172.8m height, when subjected to wind nondeterministic actions, including in the analysis the effects of the geometric nonlinearity and the aerodynamic damping. The building numerical modelling is performed using the Finite Element Method (FEM), based on the ANSYS software [8]. The dynamic response (displacements and acceleration) is evaluated and compared to the design criteria limits for human comfort.

2 Mathematical formulation of the aerodynamic damping

In this investigation, the aerodynamic damping mathematical formulation is directly considered in the wind pressure calculations, having in mind the relative velocity between the wind and the structure, both in the same direction. This way, the wind pressure and the relative velocity can be calculated based on the eqs. (1) to (4).

$$q_{wind}=\frac{1}{2}\rho\, V_R^2 = 0.613V_R^2 \tag{1}$$

$$V_R= V(t) - V_{srt} \tag{2}$$

$$V(t) = \overline{V}(z) + v(t) \tag{3}$$

$$\overline{V}(z) = \overline{V}_{10}\left(\frac{z}{10}\right)^p \tag{4}$$

The parameter q_{wind} is the wind dynamic pressure; ρ is the specific mass of the air under normal pressure conditions (101,320Pa) and temperature (15ºC); V_R is the relative velocity between the wind and the structure at the considered structural section; $V(t)$ is the wind velocity; V_{srt} is the structure velocity in wind direction, at the considered section; v(t) is the fluctuant component of velocity; $\overline{V}(z)$ is the average longitudinal velocity component, calculated in 10 minutes; $\overline{V}_{10}$ is the project average velocity at 10 meters from the ground, calculated in 10 minutes; z is a height from the ground, in meters; p is exponential ground roughness coefficient.

3 Mathematical modelling of the nondeterministic wind loads

The wind properties are unstable, present a random variation and therefore the deterministic consideration can become inadequate. However, to generate the nondeterministic wind dynamic 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 wind statistical characteristics do not change when the simulation period is performed [1,3].

In this investigation, the wind dynamic 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. The amplitude of each harmonic is obtained based on the use of a Kaimal Power Spectrum function [1,3].

This research work adopted the Kaimal Power Spectrum by considering the influence of the building height on the dynamic response [1,3]. The energy spectrum is calculated based on the use of eqs. (5) and (6), 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.

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

$$x(f,z)=\frac{f\, z}{\overline{V}_Z} \tag{6}$$

The friction velocity u^* is calculated using eq. (7), in m/s, with Karmán k constant equal to 0.4 and z_0 corresponding to the roughness length in meters. 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, as presented in eq. (8), 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 seconds.

$$u^*=\frac{k\,\overline{V}_Z}{\ln(z/z_0)} \tag{7}$$

$$v(t)=\sum_{i=1}^{N}\sqrt{2S^V(f_i)\Delta f}\cos(2\pi f_i t + \theta_i) \tag{8}$$

In this study, it is assumed that the wind pressure acting on the building façades is a direct function of the wind velocity, as in the Davenport classic model adopted in the Brazilian design standard NBR 6123 [9]. This way, the wind pressure can be calculated according to eq. (9), 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. (10), where C_{ai} is the drag coefficient in the "i" direction and A_i is the influence area in m². The drag coefficient C_a depends on the relationships between the structure dimension and can be determined through the Brazilian design standard NBR 6123 [9]. This way, the eq. (11) can be written based on the expansion of the eq. (10), 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 to the NBR 6123 [9].

$$q(t)=0.613\ [\overline{V}+v(t)]^2 \tag{9}$$

$$F(t)=C_{ai}q(t)A_i \tag{10}$$

$$F(t)=0.613c_DA_i\left[\overline{V}_0\left(\frac{z}{z_0}\right)^p+\sum_{i=1}^{N}\sqrt{2S^V(f_i)\Delta f}\cos\left(2\pi f_i t+\theta_i\right)\right]^2 \tag{11}$$

4 Investigated steel-concrete composite building

The studied steel-concrete building presents 48 floors, each floor with height of 3.6m and the structural system presents height of 172.8m (see Fig. 1). The building has dimensions of 45m long and 32m wide and central core with dimensions of 27m x 9m. The main beams are made of W460x106 steel profiles and the secondary beams by W410x60 profiles (see Fig. 1).

The used steel is the traditional ASTM A572. The concrete slab is 15cm thick and the steel columns are made of HD profiles (steel ASTM A913), with all geometric characteristics presented in Tab. 1 [1]. The concrete used in the model presents compressive strength (fck) equal to 30MPa, modulus of elasticity (E_{cs}) of 26GPa, Poisson's ratio (ν) of 0.2 and specific weight (γ_c) equal to 25kN/m³. The used steel presents characteristic strength (f_y) of 345MPa, modulus of elasticity (E_s) of 205GPa, Poisson's ratio (ν) of 0.3 and specific weight (γ_s) of 78.5kN/m³. Figure 1 presents a floor plan of the investigated steel-concrete building (dimensions in meters).

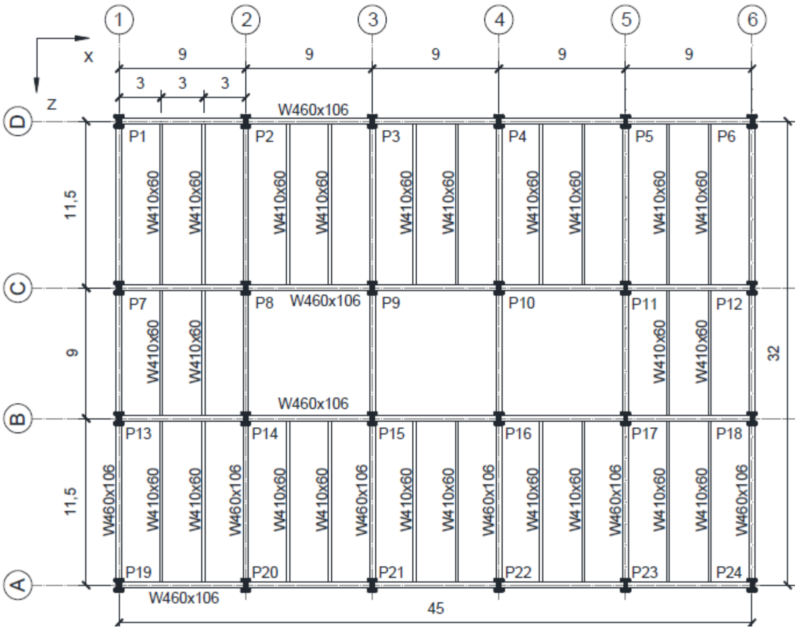


Figure 1. Structural project of the steel-concrete composite multi-storey building: H = 172.8m.

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

5 Finite element modelling

The steel-concrete composite building was investigated using the ANSYS program [8], based on usual discretization techniques associated with the Finite Element Method (FEM) (see Fig. 2). The building finite element model satisfies the mesh convergence study previously performed [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 the bending and torsional effects are considered. The building concrete slabs are simulated considering shell finite elements SHELL63. The foundation block was discretized based on the use of the SOLID45 element. The soil spring coefficients are modelled based on the use of the COMINB14 element.

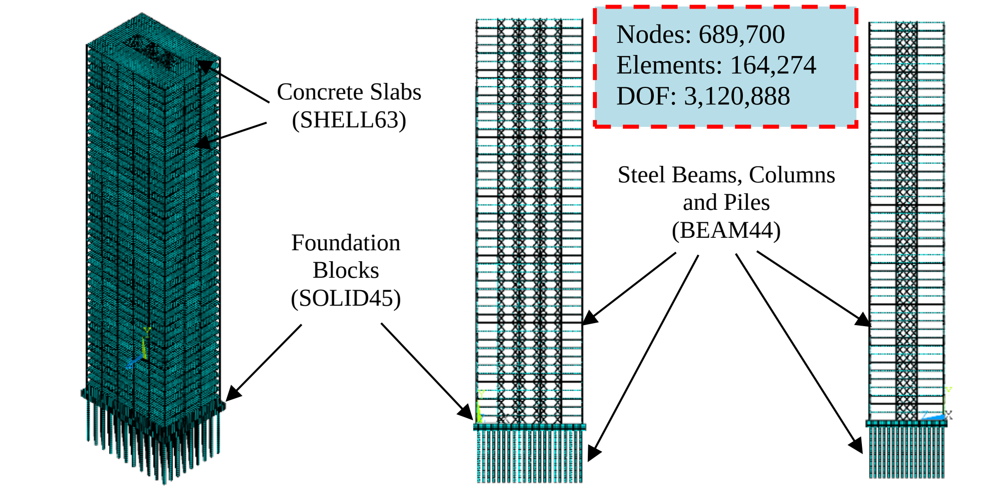


Figure 2. Steel-concrete composite building finite element model: H = 172.8m.

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. In structural mechanics, a problem is said to be nonlinear if the stiffness matrix or the force vector depend on displacements. The effective stiffness matrix is calculated by eq. (12), where $[K]^{eff}$; [M]; [C]; [K] represents the effective stiffness matrix; mass matrix; damping matrix; stiffness matrix, respectively; and a_0 and a_1 are the numerical parameters of the equilibrium equation integration [1].

$$[K]^{eff} = a_0[M] + a_1[C] + [K] \tag{12}$$

Considering linear models, the effective stiffness matrix remains constant in all analysis steps, unless the time interval is changed. On the other hand, for nonlinear analysis, the effective stiffness is modified for each time interval and depends on the displacements. In the nonlinear analysis, the effective stiffness matrix can be written as shown in eq. (13), where $[K_T]$ is the tangent matrix [1].

$$[K]^{eff} = a_0[M] + a_1[C_T] + [K_T] \tag{13}$$

The parameters α and δ, determine the stability and accuracy characteristics of the Newmark method. The Newmark response is unconditionally stable for conditions observed in eq. (14). When $\delta = 1/2$ and $\alpha = 1/4$, the Newmark method correspond to the constant average acceleration method. The numerical method is implicit, unconditionally stable, second-order accurate and one of the most effective for resolution of structural dynamic problems.

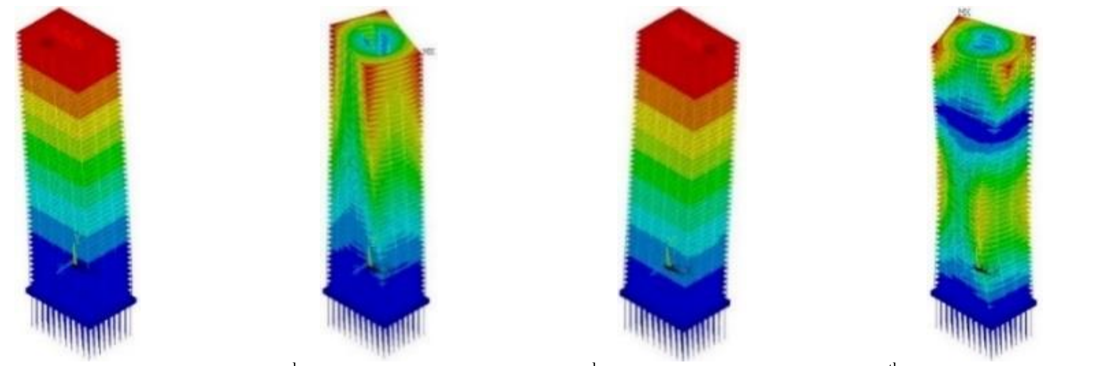
$$\alpha \geq \frac{1}{4}\left(\frac{1}{2}+\delta\right)^2,\ \delta \geq \frac{1}{2},\ \frac{1}{2}+\delta+\alpha>0 \tag{14}$$

6 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 [8]. In this investigation, the linear modal analysis was performed, in which there is no load application on the structure. In addition, the nonlinear modal analysis was also performed, 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, the structure is considered in its deformed position.

The loads utilised to provoke the deformed position of the building are associated to the usual design loads (vertical loads: self-weight, permanent loads, overloads; and horizontal loads: static wind loads). This way, for the calculation of static wind loads, intervals of 18 km/h were considered, starting at 18 km/h up to 162 km/h, covering most of the of basic wind velocities present in NBR 6123 [9].

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).



1st: Bending around X 2nd: Torsion around Y 3rd: Bending around Z 4th: Torsion around Y

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

Table 2. Natural frequencies of the building.

Frequency (Hz)	Linear model	Geometric nonlinear model Velocity - V_0 (km/h)								
		18	36	54	72	90	108	126	144	162
f_{01}	0.161	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

It is verified that the fundamental frequency value of the investigated building is equal to 0.161Hz (f_{01} = 0.161Hz), 10% higher than the value calculated in the nonlinear modal analysis (f_{01} = 0.146Hz). This fact is 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 [9], buildings presenting natural frequencies values lower than 1Hz, particularly those that have low structural damping, may present relevant floating dynamic response along-wind, indicative of excessive vibrations.

7 Nondeterministic dynamic structural analysis

Initially, besides the usual vertical design loads, the nondeterministic wind dynamic actions were applied over the façade, considering the direction perpendicular to the 45m of the building side (see Fig. 1), aiming to assess the nonlinear dynamic response. This way, the maximum horizontal displacements values were calculated at the building top (H = 172.8m) and the maximum accelerations values were determined at last building floor storey (H = 169.2m). In this work, the following analyses were developed: linear and geometric nonlinear with and without the aerodynamic damping. In addition, twenty series of nondeterministic wind dynamic loads were generated and used for the response statistical analysis. The parameters utilised to determine the wind series are the wind basic velocity (V_0) 18km/h to 162km/h; terrain category IV; recurrence time of 10 years; topographic factor (S_1) 1; probability factor (S_3) 0.78; roughness Factor (S_2) b = 0.84, p = 0.135 and F_r=0.69.

Based on the response statistical analysis and considering only the effect of geometric nonlinearity, it must be emphasized that important quantitative changes occur on the building displacements and accelerations mean maximum values (see Table 3), calculated in the steady state response, with maximum differences in the range of 5% to 30% for horizontal translational displacements and 5% to 45% for the accelerations.

This way, Fig. 4 presents the geometric linear and nonlinear building dynamic response (V_0 = 126km/h) [9] in frequency domain, where the difference between the building fundamental frequency value is clearly verified. The results presented in Fig. 4 have considered the wind load series that produced the values closest to the characteristic values of the system response. The effect of geometric nonlinearity clearly produced modifications on the displacements and accelerations, considering the structure response energy transfer levels (see Fig. 4).

Table 3. Dynamic structural response of the building: RMS and peak values.

Wind velocity (km/h)		Structural analysis	18	36	54	72	90	108	126	144	162
RMS	Displacement (m)	Nonlinear	0.002	0.007	0.016	0.029	0.048	0.070	0.095	0.127	0.162
		Linear	0.001	0.006	0.013	0.024	0.038	0.058	0.080	0.105	0.131
		%	24%	19%	25%	21%	26%	21%	20%	21%	24%
	Acceleration (m/s²)	Nonlinear	0.001	0.004	0.010	0.020	0.034	0.051	0.071	0.092	0.124
		Linear	0.001	0.003	0.008	0.016	0.025	0.041	0.057	0.077	0.094
		%	26%	23%	32%	24%	34%	24%	25%	20%	33%
Peak	Displacement (m)	Nonlinear	0.004	0.018	0.047	0.084	0.146	0.211	0.288	0.373	0.510
		Linear	0.003	0.015	0.038	0.080	0.122	0.182	0.262	0.347	0.408
		%	13%	27%	25%	5%	19%	16%	10%	7%	25%
	Acceleration (m/s²)	Nonlinear	0.003	0.013	0.036	0.067	0.121	0.175	0.231	0.321	0.472
		Linear	0.002	0.010	0.028	0.053	0.093	0.132	0.199	0.253	0.330
		%	20%	34%	31%	26%	30%	32%	16%	27%	43%

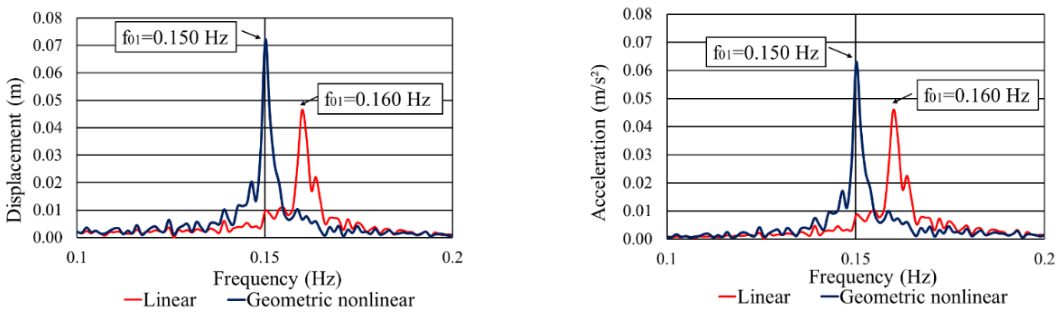


Figure 4. Dynamic response (frequency domain): displacements (H=172.8 m) and accelerations (H=169.2 m).

In sequence, in order to study the aerodynamic damping effect on the building structural response, the basic wind velocity of V_0 = 126km/h [9] was utilised, to determine the displacements and accelerations considering the statistical treatment associated to the twenty wind load series. This way, Tab. 4 shows the building dynamic response with comparisons between the responses associated to the linear and the geometric nonlinear model.

Table 4. Displacements and accelerations: effect of the aerodynamic damping.

Structural response		Linear model			Geometric nonlinear model		
		No aerodynamic damping	Aerodynamic damping	%	No aerodynamic damping	Aerodynamic damping	%
Displacements (m)	RMS	0.080	0.077	4	0.095	0.090	5
	Peak	0.262	0.251	4	0.288	0.272	5
Accelerations (m/s²)	RMS	0.057	0.052	9	0.071	0.064	10
	Peak	0.199	0.188	5	0.231	0.213	8

Based on Tab. 4 results, it was concluded that relevant quantitative changes occur on the mean maximum values of the building displacements and accelerations, calculated in the steady state response, when the effect of geometric nonlinearity and the aerodynamic damping are considered. On the other hand, when the effect of the aerodynamic damping is investigated, there is a reduction in the building mean maximum displacements and accelerations. It is possible to verify the changes occurred in the building dynamic response, when the effect of the aerodynamic damping is considered, with maximum differences up to 5% for horizontal translational displacements and up to 10% for the accelerations (see Tab. 4).

8 Conclusions

The main conclusions obtained in this research work are related to assessments of the tall buildings dynamic response, when subjected to nondeterministic wind dynamic actions, considering the effects of the geometric nonlinearity and aerodynamic damping. 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 effects of the geometric nonlinearity and the aerodynamic damping were considered, with modifications in the displacements and accelerations values.

2. Considering a parametric study related to the wind velocities [18km/h to 162km/h] and the statistical treatment of twenty nondeterministic wind series, it was concluded that the geometric nonlinearity effects have produced relevant changes on the building dynamic response, with maximum differences in the range of 5% to 30% for displacements and 5% to 45% for accelerations.

3. Based on the investigated building dynamic response in the frequency domains, 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.

4. Considering the basic wind velocity of 126km/h and the statistical analysis of twenty nondeterministic wind series, it was verified that the aerodynamic damping effects have produced changes in the building dynamic response, with maximum differences up to 5% for displacements up to 10% for accelerations.

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

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