Assessment of the tall buildings dynamic response considering the geometric nonlinearity and the aerodynamic damping

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Abstract. This research work aims to evaluate the dynamic structural behaviour of tall buildings when subjected to wind loads considering the effect of the geometric nonlinearity and also the aerodynamic damping. This way, the project associated to a steel-concrete composite building with 48 floors and 172.8 m height is investigated, when subjected to wind nondeterministic dynamic actions. The composite building finite element model was developed based on the use of the Finite Element Method (FEM), utilising the ANSYS computational program, and considering the soil-structure interaction effect, aiming to obtain a realistic representation of the dynamic behaviour. The building dynamic response was obtained based on the displacements and accelerations values, determined having in mind a wind velocity range between 5 m/s [18 km/h] and 45 m/s [162 km/h]. The conclusions of this investigation pointed out to the fact that when the geometric nonlinearity effect was considered in the analysis the investigated building dynamic response presented relevant differences, with maximum differences up to 30% to horizontal translational displacements and up to 45% to accelerations. On the other hand, when the aerodynamic damping was considered, the contribution was not significant to the dynamic response, with maximum differences up to 5% for the displacements and up to 10% for the accelerations.

Keywords: tall buildings, geometric nonlinearity, aerodynamic damping.

1 Introduction

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. 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 [1,2].

In fact, most of the structures cannot be considered linear, particularly under severe loading conditions. It is precisely under these conditions that a linear structural analysis is found to be inadequate and a more elaborate nonlinear analysis must be performed. 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 loading acting on the deformed structural system can induce higher efforts values when compared to those calculated based on a linear analysis. In rigid structures, these effects are small and generally can be neglected. When flexible structures are assessed such effects become significant and must be investigated [3,4].

In this context, the effect of aerodynamic damping must be evaluated, which is defined as a force associated to the relative movement between the structure and the air. Depending on the structure velocity, the dynamic response can be reduced due to the aerodynamic damping effect. In most cases, when excited by wind, the developed structure velocity is low, which does not change the dynamic pressure values, but with flexible systems, these velocities can be relevant and may have a considerable impact on the dynamic pressure [5,6].

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 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 [7]. Based on the displacement and acceleration values, this study concluded that the effect of geometric nonlinearity led to relevant differences in the dynamic structural response of the investigated building, with maximum differences up to 30% to displacements and up to 45% to accelerations. On the other hand, the contribution of aerodynamic damping was not significant, with maximum differences up to 5% for horizontal translational displacements and up to 10% for the accelerations.

2 Nondeterministic dynamic wind force

Wind properties are unstable and present a random variation, and therefore the deterministic consideration can become inadequate. However, to generate a nondeterministic dynamic wind series, in this study, the wind flow was assumed to be 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].

This work adopted the Kaimal power spectrum by considering the influence of building height on dynamic response [1]. The energy spectrum was calculated based on 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, obtained using eq. (3), z is the height in meters and V_{10} is the project average velocity at 10 meters from the ground, calculated in 10 minutes, by eq. (4). V_0 is wind basic velocity, calculated in a 3-second interval, S_1 is the topographic factor, and S_3 is the statistical factor associated with the destruction probability, according to NBR 6123 [8]. The friction velocity u* was calculated using eq. (5), in m/s, with a Kármán k constant equal to 0.4 and z_0 corresponding to the roughness length in meters.

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

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

$$\overline{V}(z) = \overline{V}_{10}\left(\frac{z}{10}\right)^p \tag{3}$$

$$\overline{V}_{10} = 0.69V_0S_1S_3 \tag{4}$$

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

The turbulent part of wind velocity v(t) is simulated based on a random process obtained from the sum of a finite number of harmonics, as presented in eq. (6), where N corresponds to the number of power spectrum divisions, f is the frequency in Hz, Δf is the frequency increment, θ is the random phase angle uniformly distributed in the range of [0-2π], and t is the time in s.

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

In this study, it was assumed that the wind pressure acting on the building's façades was a direct function of the wind velocity, as in the Davenport classic model adopted in the Brazilian design standard NBR 6123 [8]. This means that the wind pressure can be calculated according to eq. (7), 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 dynamic wind pressure acting on the structure, it was possible to calculate the dynamic wind load along the time F(t), in N, at each investigated building structural section through Eq. (8), where Cai 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 dimensions and can be determined through NBR 6123 [8]. This way, eq. (8) can be written based on the expansion of eq. (9), where c_D is the drag coefficient corresponding to the angle of attack, V_0 is the wind basic velocity, and p is the exponent of the potential law of variation of the S_2 factor according to NBR 6123 [8].

$$q(t) = 0.613[\overline{V} + v(t)]^2 \tag{7}$$

$$F(t) = C_{ai} q(t) A_i \tag{8}$$

$$F(t) = 0.613 C_D A_i \left[V_0 \left(\frac{z}{z_0} \right)^p + \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f} \cos(2\pi f_i t + \theta_i) \right]^2 \tag{9}$$

The aerodynamic damping mathematical formulation was directly considered in the wind pressure calculations, keeping in mind the relative velocity between the wind and the structure, both in the same direction. Therefore, the wind pressure and relative velocity can be calculated based on eqs. (10) to (12).

$$q_{wind} = \frac{1}{2}\rho V_R^2 = 0.613 V_R^2 \tag{10}$$

$$V_R = [V(t) - V_{str}] \tag{11}$$

$$V(t) = \overline{V}(z) + v(t) \tag{12}$$

Equation (10) presents the classical formulation for the dynamic wind pressure calculation present in NBR 6123 [8] with the modification of the adopted reference velocity. In the conventional formulation, wind velocity is adopted, while this version uses the relative velocity between wind and structure. The new nondeterministic dynamic force that considers the effect of aerodynamic damping, eq. (13), can be calculated from the wind pressure expression, obtained through eq. (10), substituting it in eq. (8).

$$F_R(t) = 0.613 C_D A_i \left[V_0 \left(\frac{z}{z_0} \right)^p + \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f} \cos(2\pi f_i t + \theta_i) - V_{str} \right]^2 \tag{13}$$

3 Investigated steel–concrete composite building

The steel–concrete building contains 48 floors, each of which is 3.6 m high, and the structural system has an overall height of 172.8 m. The building is 45 m long and 32 m wide (floor plan), and the central core is 27 m x 9 m. The main beams are made of W460x106 steel profiles and the secondary beams have W410x60 profiles [1]. Figure 1 shows a floor plan of the building (dimensions in meters).

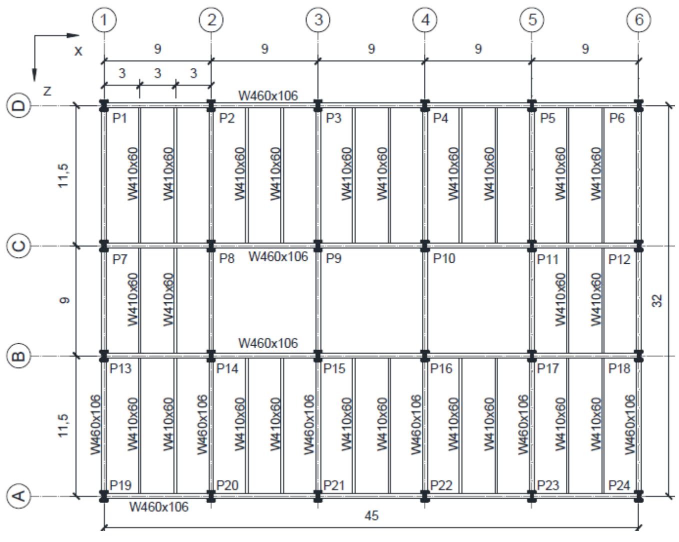


Figure 1. Floor plan of the steel–concrete composite multi-storey building: H = 172.8 m

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.

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 using the ANSYS program [8], based on the usual discretisation techniques associated with the finite element method. The finite element model of the building satisfied the mesh convergence study previously performed [1,2]. For the numerical modelling, the steel beams, columns and piles were represented based on the use of the three-dimensional finite elements BEAM44, where bending and torsional effects were considered. The concrete slabs of the building were simulated considering finite shell elements, using SHELL63. The foundation block was discretised based on the use of the SOLID45 element. Soil spring coefficients were modelled using the COMINB14 element. Figure 2 shows that the foundation (piled raft) of the building was modelled to consider the effect of the soil–structure interaction.

The full interaction between the concrete slabs and the steel beams was considered in the study, so the nodes of the finite element model were coupled to prevent the occurrence of slips. Steel and concrete are considered to have elastic linear behaviour, and all structural sections of the model remained plane in the deformed state. The final computational model adopted used 689,700 nodes and 164,274 elements, which resulted in a numeric model with 3,120,888 degrees of freedom (see Fig. 2).

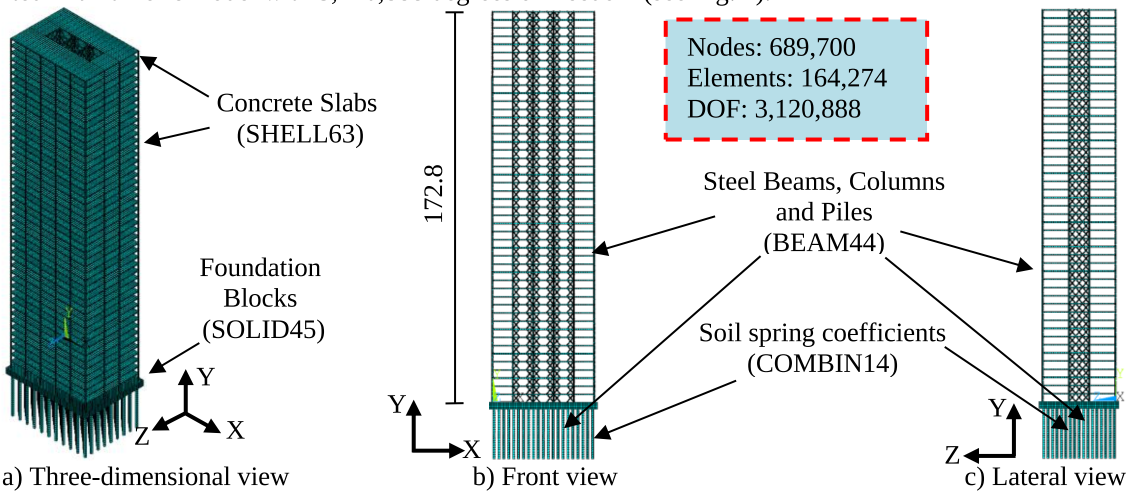


Figure 2. Steel-concrete composite building finite element model: H = 172.8 m

Geometric nonlinearity appears in the theory of elasticity both in the equilibrium equations, which are written using the deformed configurations, and in the deformation-displacement relations, which include nonlinear terms in the displacements and their derivatives. An incremental-iterative procedure is used to trace the equilibrium path of the structure over time. The principle of virtual displacements for deformable bodies is given by $\delta W_{int} = \delta W_{ext}$ [1]. The discretization of the structure leads to the dynamic equilibrium equation and can be express in eq. (14). [M]; [C]; [K]; $\{F^a\}$; $\{\ddot{u}\}$; $\{\dot{u}\}$; $\{u\}$ represent the mass matrix; damping matrix; stiffness matrix; applied load vector; acceleration vector; velocity vector and displacement vector, respectively.

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F^a\} \tag{14}$$

The finite element software ANSYS utilises Newmark's time integration method to solve transient problems, which despite being more complicated in terms of calculation was adequate given the nonlinearity effect. For nonlinear dynamic solutions, it combines the Newton-Raphson method with Newmark's method. The implicit method uses eq. (15) to obtain the solution. The geometric nonlinearity was included using the total Lagrangian formulation, which allows large displacements and rotations [1].

$$\{u_{n+1}\} = [K]^{-1}\{F_{n+1}{}^{a}\} \tag{15}$$

5 Modal analysis: eigenvalues and eigenvectors

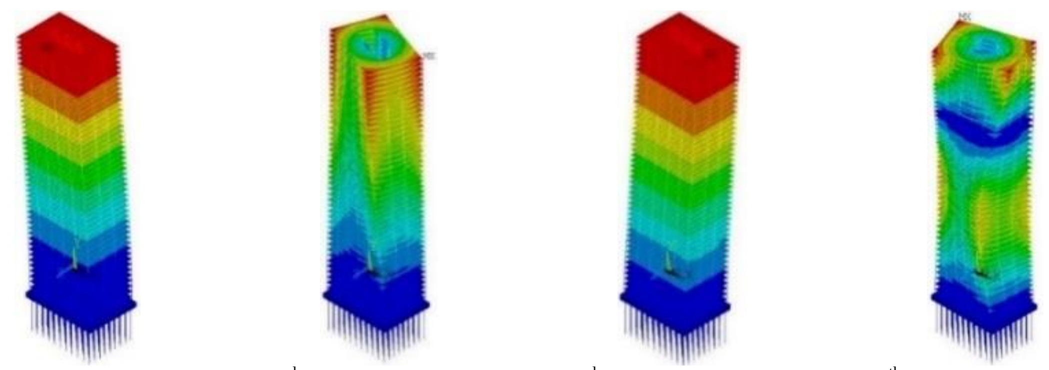
The building natural frequencies (eigenvalues) and the vibration modes (eigenvectors) were calculated using numerical extraction methods (modal analysis), through a free vibration analysis, utilizing the ANSYS program [7]. In this investigation, the linear modal analysis was performed, in which there 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 [8].

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 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



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

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 [8], 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.

6 Nondeterministic dynamic structural 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, see Fig. 2. 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 work, four analyses were developed: linear and geometric nonlinear with and without aerodynamic damping. In addition, twenty series of nondeterministic dynamic wind loading were generated, used for the statistical treatment of the response. The parameters used to determine the wind series are 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 results presented in Tab. 3, and taking into account the numerical accuracy for the assessment of the nondeterministic steady state response, significant changes occur in the values of the displacements and accelerations of the studied building when the effect of geometric nonlinearity is considered in the dynamic analysis (forced vibration), with maximum differences up to 30% for horizontal translational displacements and 15% to 45% for the accelerations. In terms of mean maximum horizontal displacements, when comparing the peak values with the limit established in NBR 8800 [9] [H/400: 172.8/400 = 0.43 m], for velocities from 18 to 144 km/h, the displacement limit is attended. However, for a velocity of 162 km/h, the limit is violated.

Table 3. Dynamic structural response of the building: peaks

Wind velocity (km/h)	Type of analysis	18	36	54	72	90	108	126	144	162
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 presents the linear and geometric nonlinear dynamic structural response of the building [V_0 = 35 m/s (126 km/h)] in frequency domain, with and without the effects of aerodynamic damping, where the difference between the values of the natural frequencies of the building is clearly verified. The results considered the wind load series that produced the values closest to the characteristic values of the system response.

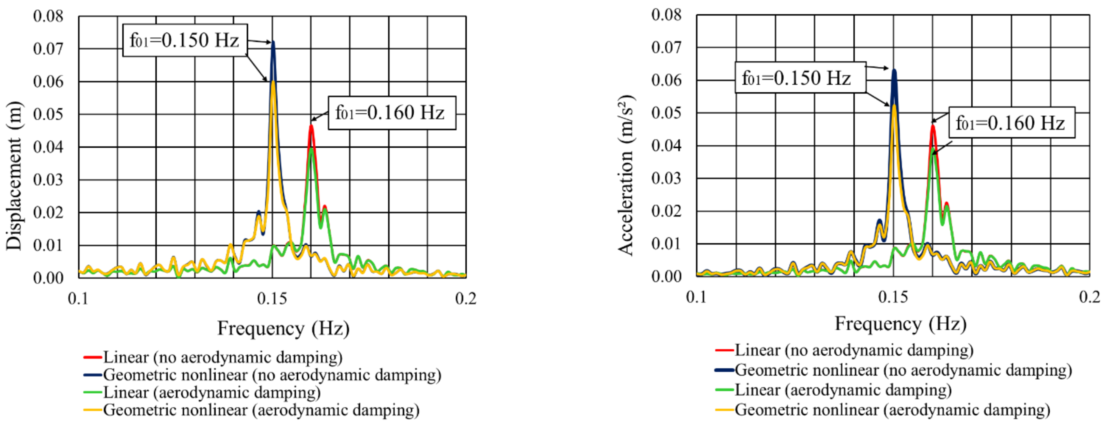


Figure 4. Dynamic response (frequency domain): displacements and accelerations [V_0 = 126 km/h]

Aiming to study the aerodynamic damping effect on the building's structural response, the basic wind velocity of V_0 = 35 m/s [126 km/h] [8] was used to determine the displacements and accelerations taking into account the statistical treatment associated with the twenty wind load series. Table 4 shows the building's dynamic response with comparisons between the linear and the geometric nonlinear models.

Table 4. Displacements and accelerations: effect of aerodynamic damping [V_0 = 126 km/h]

Structural response	Linear model			Geometric nonlinear model		
	No aerodynamic damping	Aerodynamic damping	%	No aerodynamic damping	Aerodynamic damping	%
Displacements (m)	0.262	0.251	4	0.288	0.272	5
Accelerations (m/s²)	0.199	0.188	5	0.231	0.213	8

Based on the results shown in Tab. 4, it was concluded that significant quantitative changes occur to the mean maximum values of the building's displacements and accelerations, calculated in the steady state response, when the effects of geometric nonlinearity and aerodynamic damping are considered. On the other hand, when the effect of aerodynamic damping is available, there is a reduction in the mean maximum displacements and accelerations. It is possible to verify the changes that occurred in the building's dynamic response when the effect of aerodynamic damping is considered, with maximum differences of up to 5% for horizontal translational displacements and up to 10% for the accelerations. Although the consideration of aerodynamic damping reduces the maximum values obtained, it is not relevant to the structure behaviour. Considering the effect of aerodynamic damping when comparing peak values, the mean maximum values of accelerations for a velocity of 35 m/s [126 km/h] exceed the limit value established by NBR 6123 [8] (a_{lim} = 0.10 m/s²), violating the human comfort. The conclusion is the same for the linear and nonlinear geometric model.

7 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: 4.56×107 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 in the building dynamic response, with maximum differences up to 30% for displacements and up to 45% for accelerations.

3. Considering the basic wind velocity of 126 km/h and the statistical treatment 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.

4. 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.

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