Nonlinear dynamic analysis of tall buildings considering nondeterministic wind loads

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Abstract. In recent years, the technological advances of the civil construction associated to a favourable economic scenario have promoted the project and construction of tall buildings in several countries, such as the United States, and more recently, some of the Asian countries, like China, Malaysia and the United Arab Emirates. However, the architectural dareness and also the increasing in the structural project's slenderness have been crucial to reducing the natural frequencies values of these buildings, and in some situations, these facts could induce excessive vibration problems and human discomfort. On the other hand, other aspect generally disregarded in the current design practice is related to influence of the effect of geometric nonlinearity on the building's structural response. This way, this research work aims to develop an analysis methodology to evaluate the dynamic structural behaviour and assess the human comfort of tall buildings, when subjected to the wind nondeterministic actions, including in the analysis the effect of geometric nonlinearity. This way, the dynamic structural behaviour of a 40-storey reinforced concrete building, 140 m high and dimensions of 29.05 m by 9.00 m was investigated. Several numerical models were developed to obtain a more realistic representation of the structural system, based on the Finite Element Method (FEM), using the ANSYS program. The results obtained throughout this investigation have indicated important quantitative differences when the dynamic structural response of the studied building was analysed, having in mind the effect of geometric nonlinearity.

Keywords: tall buildings; human comfort assessment; geometric nonlinearity.

1 Introduction

Technological advances linked to a favourable economic scenario in recent years have promoted the construction of tall buildings in several countries, such as the United States, and more recently, some of the Asian countries, like China, Malaysia and the United Arab Emirates. This trend has also gained prominence in Brazil, where the implementation of tall buildings began in Balneário Camboriú (SC) and continued in Goiânia (GO) and João Pessoa (PB), with the construction of a reasonable amount of buildings exceeding 200 meters tall, according to Silva [1], Bastos [2] and Silva [3].

Currently, tall building projects increasingly use simple structural systems, which promote agility in their assembly, cost reduction and greater flexibility in the use of built spaces as reported by Silva [1], Bastos [2], Silva [3] and Barboza [4]. On the other hand, there is 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 for structural design, according to Ferreira [5]. In this context, the one factor that exert an influence in conjunction with a random wind action on tall buildings is the effect of geometric nonlinearity.

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 like observed by Corelhano [6]. Effects due to geometric nonlinearity are those arising from the change of position of the structure in space. These effects are determined through an analysis in which the structure is considered in its final equilibrium condition. In the design of tall buildings, one must be aware of the problem of geometric nonlinearity when the structure is simultaneously loaded by vertical loading and by horizontal actions (effects of wind action) like verified in Silva [7]. This is because the vertical loading acting on the displaced structure can cause the appearance of increases in efforts capable of leading it to collapse. In rigid structures, these effects are small and can be neglected, however, in flexible structures, such effects become significant and must be considered, according Pinto [8].

This work approaches the study of the dynamic response of buildings when subjected to nondeterministic wind action, considering the effect of geometric nonlinearity. Thereby, throughout the dynamic analysis, the design of a reinforced concrete building will be used as a base, with a height of 140 m, consisting of 40 floors and plant dimensions of 29.05 m by 9.00 m. The numerical modelling of the building will be performed using the Finite Element Method (FEM), and linear and nonlinear geometric analysis will be carried out in the ANSYS program [9]. The dynamic response (natural frequencies, displacements and accelerations) has indicated important quantitative differences when the dynamic structural response of the studied building was analysed, having in mind the effects of geometric nonlinearity.

2 Nondeterministic wind mathematical modelling

Wind properties are unstable, have 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 [10].

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. The amplitude of each harmonic is obtained based on the use of a Kaimal Power Spectrum function (Fig. 1).

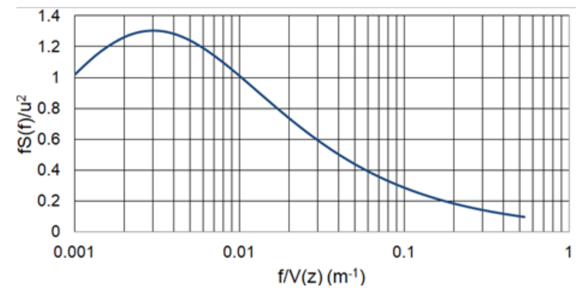


Figure 1. Kaimal Power Spectral Density

This work adopts the Kaimal Power Spectrum because it considers the influence of the building height in the formulation according to Bastos [2]. The energy spectrum is calculated using eq. (1) and eq. (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. The friction velocity u^*, in m/s, is obtained 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 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.

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

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

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

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

In this work, it is assumed that the wind pressure acting on the system structural is directly a function of velocity using the model Davenport classic. In this way the air 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. With the value of the dynamic pressure of the wind acting on the structure, it is possible to calculate the dynamic wind load over time F(t), in N, at each node of the building through the eq. (6), where C_{ai} is the drag coefficient in the "i" direction and A_i is the frontal area of the surface in the "i" region, limited by the outline of the lattice in m². The drag coefficient Ca depends on the relationships between the dimension of the structure and can be determined through the NBR 6123 [11]. Developing the eq. (6), the eq. (7) is obtained, where $\overline{V}_0$ is the basic wind velocity and p is exponent of the potential law of variation of factor S_2 according NBR 6123 [11].

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

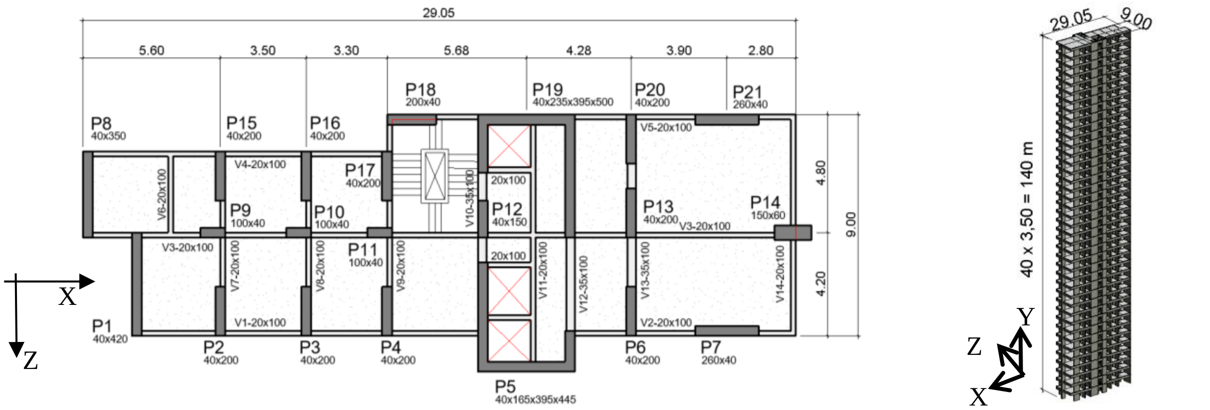
$$F(t) = C_{ai}\, q(t)\, A_i \tag{6}$$

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

3 Investigated reinforced concrete building

The investigated building in this research work presents rectangular dimensions of 29.05 m by 9.00 m, 40 floors, with a height of 3.5 m, and total height of 140 m, as shown in Fig. 2. The reinforced concrete structure of the building consists of massive slabs whose thickness is equal to 17 cm, beams with 20 by 100 cm sections and columns with section of varied dimensions. The building is residential, with one apartment per floor, tree elevators, and was based on a real building in the city of Balneario Camboriu/SC, Brazil, as approached by Silva [1].

The structural model idealized for the building, shown in Fig. 2, is composed of reinforced concrete columns, beams and slabs. The concrete has a compressive strength (*fck*) and is equal to 45 MPa, a Young's modulus (E) is equal to 34.0 GPa and Poisson's ratio (ν) is equal to 0.2 like observed in Silva [1].



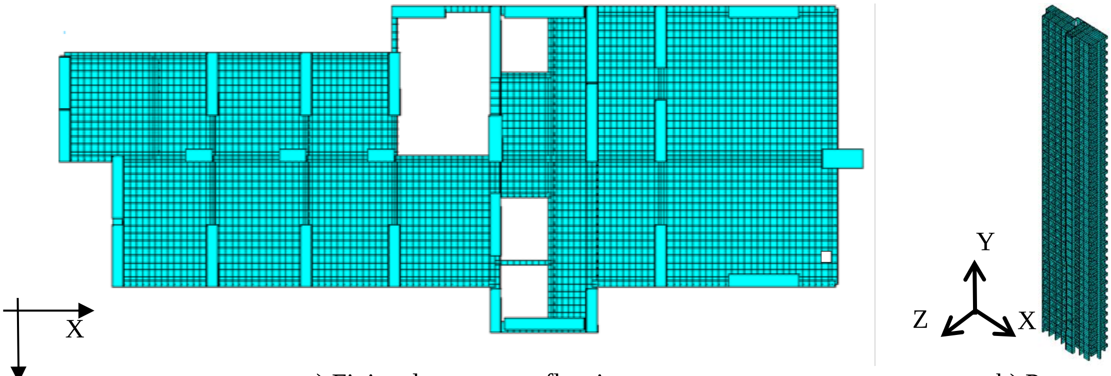
a) Building floor architecture b) Floor structural plan

Figure 2. Investigated reinforced concrete building (units in meters)

4 Finite element numerical modelling

The proposed numerical model developed for the dynamic analysis of buildings, adopted the usual mesh refinement techniques present in the Finite Element Method (FEM) simulations, implemented in the ANSYS program [9], see Fig. 3. This finite element model satisfies the mesh convergence study carried out previously by Silva [1]. In the numerical model the beams and columns were simulated based on the use of BEAM44 three-dimensional finite elements, in which the bending and torsion effects are considered. Concrete slabs were represented based on the use of the SHELL63 finite element. Regarding the boundary conditions, the horizontal translational displacements of the columns were restricted in the X, Y and Z axis.

The building finite element model adopted Newmark's time integration method for the solution of the equilibrium equations of structural dynamics, and for the nonlinear solutions, the Newton-Raphson method was employed along with Newmark's formulation. This strategy for solving the nonlinear equations is based on the implicit time integration method, which despite being more complicated in terms of calculation, is the most appropriate, given the problem high nonlinearity. The geometric nonlinearity of the building model was based on the total Lagrangian formulation, which allows large displacements and rotations.

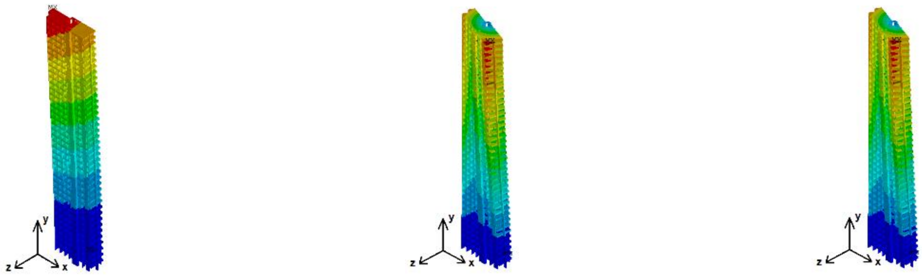


a) Finite element type flooring b) Perspective

Figure 3. Finite element model of the investigated reinforced concrete building

5 Modal analysis: eigenvalues and eigenvectors

The natural frequencies (eigenvalues) and vibration modes (eigenvectors) of the building were obtained based on numerical methods of extraction (modal analysis) through a free vibration analysis using the ANSYS program [9]. The natural frequencies and the vibration modes are the illustrated in Fig. 4. The vibration modes indicate the tendency of the structure's vibration. The colour red indicates the maximum modal amplitude and blue the minimum.



1st vibration mode: bending around X (f_{01}= 0.29Hz) 2nd vibration mode: torsion around Y (f_{02}= 0.46 Hz) 3rd vibration mode: bending around Z and torsion (f_{03}= 0.47 Hz)

Figure 4. Vibration modes of the building

6 Nondeterministic dynamic analysis

It is noteworthy that for the analysis of the dynamic structural response of the building, besides the usual vertical design loads, the nondeterministic dynamic wind action was applied over the building facade (Z direction of the numerical model: see Fig. 3). The results of the analysis dynamic for the maximum horizontal translational displacement values are obtained at the top structural sections of the building (H = 140 m), and for the maximum accelerations these values are calculated at the floor of the last building storey (H = 136.5 m).

It is important to emphasize that 20 different situations were simulated considering variations in the basic wind velocity and changes in the type of analysis (linear and geometric nonlinear). For velocity variations, intervals of 5 m/s, starting at 0 and ending at 45 m/s were simulated, which would cover a good part of the range of the basic wind velocities present in NBR 6123 [11]. In this work 10 nondeterministic wind series was generated (one for each velocity), and the parameters used to generate the nondeterministic wind series are shown in Tab. 1.

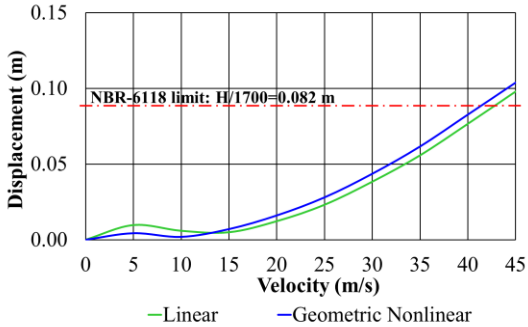
Table 1. Parameters used to generate the nondeterministic wind series

NBR 6123 design parameters [11]	Parameters used in the analysis
Basic Wind Velocity (V_0)	0 to 45 m/s
Terrain Category	IV
Recurrence Time	10 years
Topographic Factor (S1)	1
Parameters for Roughness Factor (S2)	b = 0.84 e p = 0.135
Probability Factor (S3)	0.78
Time Duration	600 seconds
Time increment	0.1 seconds

The results of the nondeterministic linear and geometric nonlinear dynamic analysis considering the basic wind velocity ranges covered in this research work for displacements and horizontal accelerations are observed in the Tab. 2 and Figs. 5 to 8. In order to assess the maximum differences of the effect of geometric nonlinearity, the mean of the results, the mean squared values (RMS), the peak value and the mean of the ten highest peak values is evaluated.

Table 2. Dynamic structural response of the building: mean, RMS, peaks and mean ten peaks

Wind velocity (m/s)		Type of analysis	5	10	15	20	25	30	35	40	45
Mean	Displacement (m)	Nonlinear	0.004	0.002	0.007	0.016	0.028	0.044	0.062	0.083	0.104
		Linear	0.010	0.006	0.005	0.012	0.023	0.038	0.056	0.077	0.098
		%	-56%	-68%	42%	31%	21%	13%	10%	8%	6%
	Acceleration (m/s²)	Nonlinear	0.001	0.005	0.014	0.027	0.042	0.065	0.097	0.132	0.179
		Linear	0.001	0.005	0.015	0.029	0.044	0.067	0.099	0.140	0.177
		%	-4%	-10%	-6%	-6%	-4%	-3%	-2%	-5%	1%
RMS	Displacement (m)	Nonlinear	0.004	0.002	0.009	0.019	0.033	0.051	0.073	0.098	0.124
		Linear	0.010	0.006	0.006	0.015	0.028	0.046	0.067	0.092	0.118
		%	-55%	-63%	39%	27%	17%	11%	9%	6%	6%
	Acceleration (m/s²)	Nonlinear	0.001	0.006	0.016	0.032	0.055	0.083	0.120	0.164	0.218
		Linear	0.001	0.007	0.017	0.034	0.056	0.085	0.121	0.170	0.220
		%	-4%	-7%	-5%	-4%	-1%	-2%	-1%	-3%	-1%
Peaks	Displacement (m)	Nonlinear	0.006	0.007	0.025	0.049	0.084	0.150	0.211	0.243	0.354
		Linear	0.011	0.013	0.020	0.041	0.076	0.147	0.185	0.237	0.337
		%	-49%	-45%	21%	21%	11%	2%	14%	2%	5%
	Acceleration (m/s²)	Nonlinear	0.004	0.023	0.049	0.084	0.151	0.303	0.354	0.489	0.775
		Linear	0.004	0.026	0.054	0.077	0.153	0.318	0.359	0.490	0.769
		%	0%	-13%	-10%	9%	-2%	-5%	-1%	0%	1%
Mean ten peaks	Displacement (m)	Nonlinear	0.006	0.007	0.024	0.048	0.083	0.145	0.206	0.240	0.347
		Linear	0.011	0.012	0.019	0.040	0.074	0.140	0.181	0.231	0.332
		%	-50%	-45%	25%	21%	10%	3%	12%	4%	4%
	Acceleration (m/s²)	Nonlinear	0.003	0.022	0.045	0.079	0.149	0.278	0.345	0.478	0.722
		Linear	0.003	0.024	0.050	0.075	0.147	0.292	0.343	0.470	0.707
		%	0%	-8%	-10%	5%	1%	-5%	1%	2%	2%



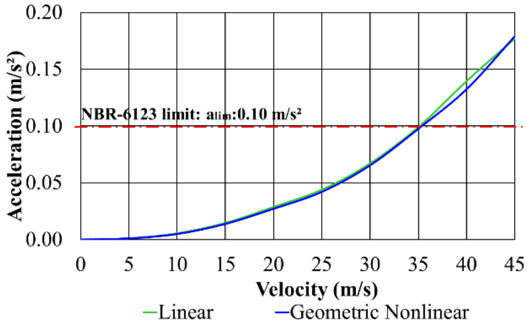


Figure 5. Mean displacements and accelerations versus basic wind velocity

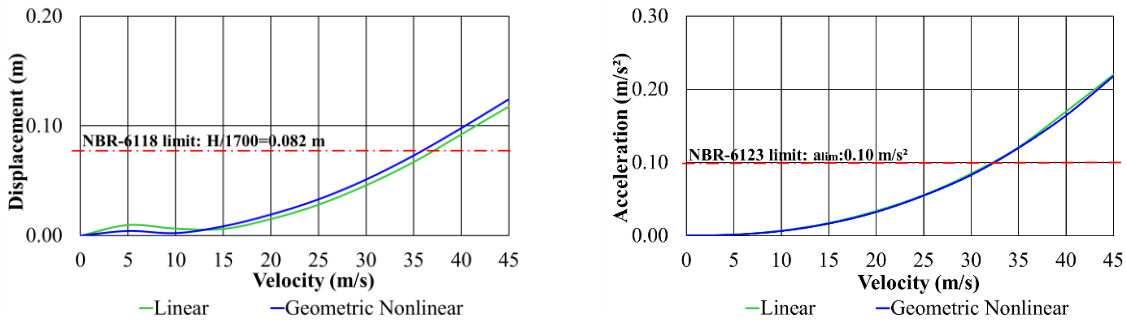


Figure 6. RMS displacements and accelerations versus basic wind velocity

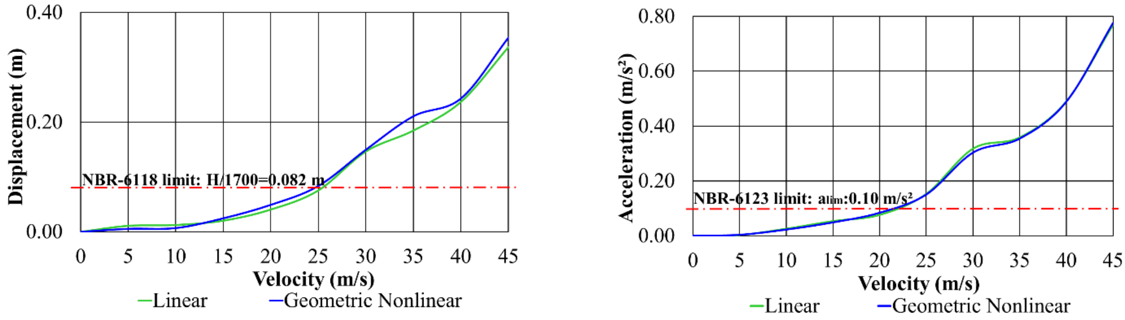


Figure 7. Peaks of displacements and accelerations versus basic wind velocity

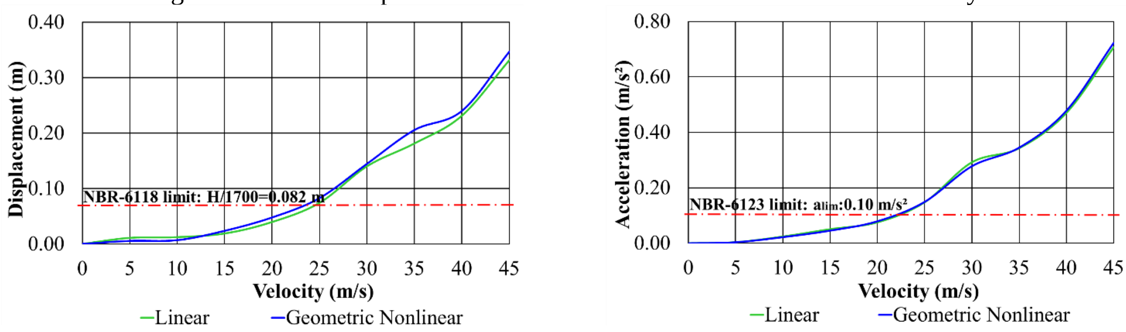


Figure. 8. Mean of ten peaks of displacements and accelerations versus basic wind velocity

Based to the results presented Tab. 2 and Figs. 5 to 8, it appears that, taking into account the issues associated with numerical accuracy for the determination of the nondeterministic response in the permanent phase, in general, it is observed that important changes occur in the values of the displacements and accelerations considering the studied building, when the effect of geometric nonlinearity is considered in the dynamic structural analysis (forced vibration), with maximum differences in the range of 50% to 70%.

Figure 9 presents the geometric linear and nonlinear dynamic structural response of the building [V_0= 35 m/s] in the frequency domain, where the difference between the values of the natural frequencies of the building is clearly verified. model. Furthermore, the effect of geometric nonlinearity produces changes on displacements and accelerations, regarding the energy transfer levels of the structure response.

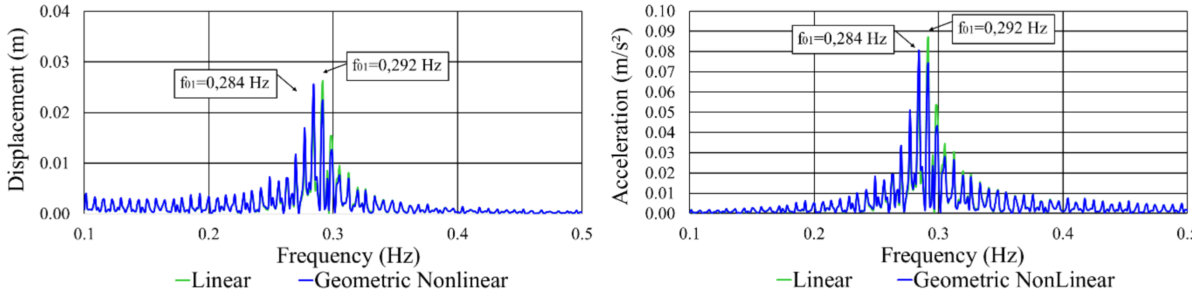


Figure 9. Displacements and accelerations versus basic wind velocity

Finally, the evaluation of the maximum displacement limit considering NBR 6118 [12]: 30% of the wind action (H/1700=8.23 cm) and the evaluation of human comfort according to NBR 6123 [11] for a recurrence time of 10 years considered the limit of 0.1 m/s² for the studied building can also be evaluated in Figs. 5 to 8.

7 Conclusions

The main conclusions of this investigation focused on alerting structural engineers to the possible variations, associated to the reinforced concrete building dynamic response, subjected to dynamic wind actions, due to the effect of the geometric nonlinearity in the analysis. Thus, the following conclusions can be drawn from the results presented in this study for the building with mass equal to 2.29×10^7 kg and stiffness equal to 1850 kN/m:

1. Based on the results achieved in this study, it is concluded that the dynamic structural response of the building is modified from the inclusion of the effect of geometric nonlinearity, with important changes in the final values of displacements and accelerations.

2. The consideration of the effects of geometric nonlinearity produced relevant changes in the dynamic response of the building (displacements and accelerations), with maximum differences in the order of 50% to 70%, for variations in the values of the basic wind velocity in the range of 5 m/s to 45 m/s, mainly for basic wind velocities of 5 and 10 m/s.

3. The effect of geometric nonlinearity produces changes on displacements and accelerations, regarding the energy transfer levels of the structure response.

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