

Dynamic analysis of buildings considering the effect of masonry infills in the global structural stiffness

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(Received October 11, 2018, Revised December 11, 2018, Accepted December 13, 2018)

Abstract. This research work presents a study that aims to assess the dynamic structural behaviour and also investigate the human comfort levels of a reinforced concrete building, when subjected to nondeterministic wind dynamic loadings, considering the effect of masonry infills on the global stiffness of the structural model. In general, the masonry fills most of the empty areas within the structural frames of the buildings. Although these masonry infills present structural stiffness, the common practice of engineers is to adopt them as static loads, disregarding the effect of the masonry infills on the global stiffness of the structural system. This way, in this study a numerical model based on sixteen-storey reinforced concrete building with 48 m high and dimensions of 14.20 m×15 m was analysed. This way, static, modal and dynamic analyses were carried out in order to simulate the structural model based on two different strategies: no masonry infills and masonry infills simulated by shell finite elements. In this investigation, the wind action is considered as a nondeterministic process with unstable properties and also random characteristics. The fluctuating parcel of the wind is decomposed into a finite number of harmonic functions proportional to the structure resonant frequency with phase angles randomly determined. The nondeterministic dynamic analysis clearly demonstrates the relevance of a more realistic numerical modelling of the masonry infills, due to the modifications on the global structural stiffness of the building. The maximum displacements and peak accelerations values were reduced when the effect of the masonry infills (structural stiffness) were considered in the dynamic analysis. Finally, it can be concluded that the human comfort evaluation of the sixteen-storey reinforced concrete building can be altered in a favourable way to design.

Keywords: non-deterministic dynamic analysis; concrete buildings; masonry infill; human comfort

1. Introduction

In general, the masonry fills most of the empty areas within the structural frames of the buildings. Although these masonry infills present structural stiffness, the common practice of most design offices is to consider its contribution as static loadings, disregarding the masonry infills effect on the global stiffness of the investigated structural system. This way, the non-consideration of the masonry infills effect for a service limit state verification can lead to unrealistic results that

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can bring unnecessary modifications in the structural project (De Sá 2018).

Over the past few years, several research works have been developed aiming to show the importance of masonry infills in the overall behavior of the buildings. Based on dynamic monitoring investigations, it is possible to verify that the consideration of masonry infill has a considerable influence on buildings dynamic behavior as its mass and stiffness change characteristics as natural frequencies, mode of vibrations and damping ratios of the building. The non-consideration of masonry infills in numeric models may result in incorrect analyzes due to poor fidelity with the real model (Kim 2009, Nascimento 2015). Another important aspect to consider is associated to the increase in lateral stiffness of the building structural frame with the masonry infill (Timuragaoglu 2015).

Masonry infills can be considered in a finite element model according two different approaches, commonly identified as micro modelling and macro modelling. Micro modelling is utilised to study the panel strength and crack propagation according to the description of each masonry component in the numeric model. This way, blocks and mortar are described individually with specific finite elements and materials properties (Lourenço 1997). On the other hand, when the study is related to the influence of masonry infills on the global behavior of the structure, macro modelling strategies are considered and the masonry panel can be modelled by an equivalent diagonal, representing its lateral stiffness, or based on the use of shell finite elements. In both cases, mathematical and physical simplifications are made in order to consider masonry as a single and homogeneous material (Souza 2014, Hu 2015, Albayrak 2017, Steeves 2017, Zahir 2017).

Regarding tall buildings, the wind represents one of the main dynamic loading. Generally, design standards consider the wind as static load calculated according to its average velocity that varies with the height of the building. However, the turbulent part of the wind, specifically the low frequencies of its spectrum, can induce vibrations in the building, resulting in human discomfort. Thus, a more realistic way, is to obtain a wind dynamic load by considering the non-deterministic characteristics in order to represent it according to studies carried out by the natural wind. The turbulent part of the wind velocity can be calculated by the sum of a finite number of overlapping harmonics with randomly generated phase angles (Franco 2011, Bastos 2017, Fernández 2017).

Therefore, aiming to evaluate the structural behaviour of buildings, when submitted to the wind dynamic actions, considering the representation of the masonry infills and adopting the macro modelling approach, this research work seeks to perform static, modal, dynamic analyses and evaluation of the human comfort of a building finite element model developed with the use of software ANSYS (2012). The numerical model was based on a 16 stories residential building made of reinforced concrete, considering two structural models: the first one without any masonry infill in the frames (only girders, columns and slabs), and the second model incorporates the masonry infill panels, based on the use of shell finite elements, adopting the physical characteristics of the masonry commonly used in Brazilian buildings. Based on the results obtained in this investigation, related to the modification of the global stiffness, displacements and accelerations, the relevance of the masonry effect to evaluate the dynamic behaviour of buildings was confirmed.

2. Nondeterministic wind mathematical modelling

Wind properties are unstable, have random variation, and thus, their deterministic consideration

becomes inadequate. However, it can be considered that for generation of nondeterministic dynamic loading 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 during the period in which the simulation is performed (Obata 2009). This way, in this investigation, the wind loadings are calculated by the sum of two parcels: the first one is related to the static parcel (mean wind force) and the second is associated to the turbulent parcel (nondeterministic dynamic load). The turbulent part of the wind is decomposed based on a finite number of harmonic functions, with randomly determined phase angles. The amplitude of each harmonic is obtained based on the use of a wind power spectrum density function (PSD function) (Blessmann 1995), as illustrated in Fig. 1.

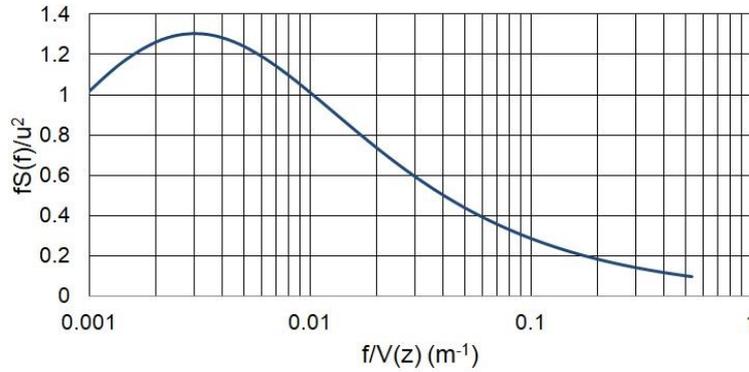


Fig. 1 Kaimal Power Spectral Density (PSD) (Blessmann 1995)

According to the PSD functions existing in the technical literature, the Kaimal spectrum was adopted, see Fig. 1, due to the consideration of the height of the building. This way, the PSD is calculated based on the Eqs. (1)-(3) (Blessmann 1995). In Eqs. (1)-(3), f is the frequency in Hz, S^V is the spectral density of the wind turbulent longitudinal part in m^2/s , x is a dimensionless frequency, 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 calculated based on the Karmán constant k using Eq. (3).

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

$$x(f, z) = \frac{fz}{V_z} \quad (2)$$

$$u_* = \frac{k \bar{V}_z}{\ln(z/z_0)} \quad (3)$$

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

The turbulent part of the wind velocity is simulated based on a zero-stationary random process, obtained from a sum of a finite number of harmonics superposed, Eq. (4), where N corresponds to the number of divisions of the power spectrum, f represents the frequency in Hz, Δf is the frequency increment and θ is the random phase angle uniformly distributed in the range of $[0-2\pi]$.

Finally, using the normative considerations recommended by the Brazilian design code NBR 6123 (1988), the applied nondeterministic dynamic wind forces acting on the investigated structure are calculated based on Eq. (5), where the first part within the brackets represents the wind mean velocity calculated using Eq. (6), and the second represents the wind turbulent part.

In Eqs. (5)-(6), C_a corresponds to the drag coefficients of the structure in the direction of the considered wind, A_{ef} represents the effective area of the building corresponding to the point where the force will be applied, V_0 is the characteristic wind velocity of the region and S_1 , S_2 and S_3 are parameters defined by Brazilian standard NBR 6123 (1988).

$$F(t) = 0.613 C_a A_{ef} \left[\bar{V}(z) + v(t) \right]^2 \quad (5)$$

$$\bar{V}(z) = S_1 S_2 S_3 V_0 \quad (6)$$

3. Investigated reinforced concrete building

The investigated building in this research work presents rectangular dimensions of 15.00 m by 14.20 m, is composed by 16 floors, with a height of 3.0 m, having a total height of 48 m, as shown in Fig. 2(a)-(b). The reinforced concrete structure of the building consists of massive slabs with a thickness equal to 10 cm, girders with sections 12×50 cm and columns with two sections dimensions: 20×80 cm, and 30×150 cm. The building is residential with two apartments per floor, four elevators, located at the city of Rio de Janeiro/RJ, Brazil.

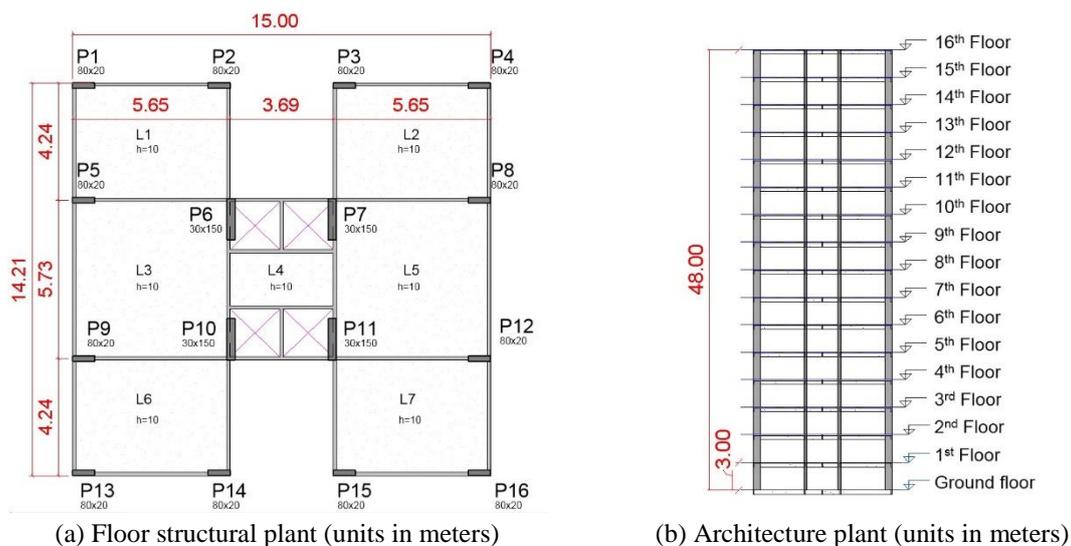


Fig. 2 Investigated sixteen-storey reinforced concrete building

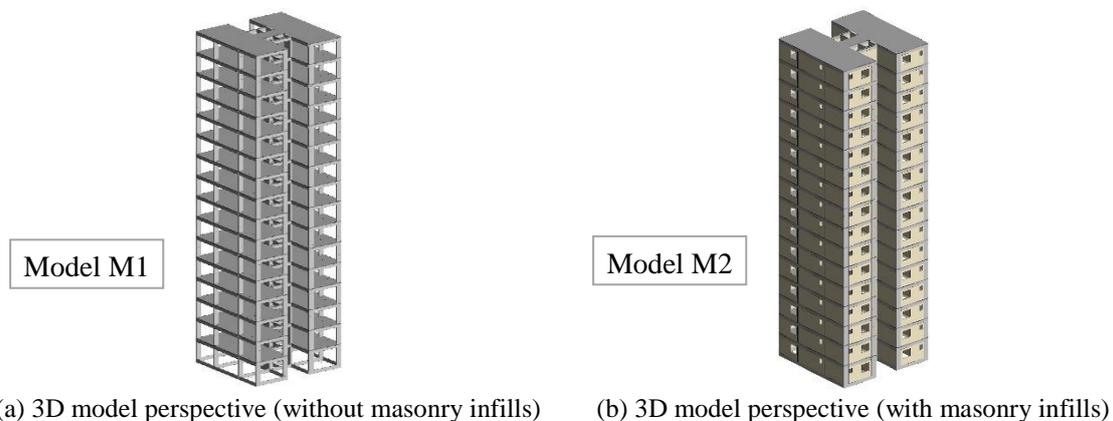
The architectural design of the investigated building is presented in Fig. 3. The concrete presents compressive strength equal to 25 MPa ($f_{ck} = 25$ MPa), Young's modulus equal to 23.8 GPa ($E = 23.8$ GPa), Poisson's ratio of 0.2 ($\nu = 0.2$) and density of 25 kN/m³ ($\gamma_c = 25$ kN/m³). In this research work, the masonry infills were made using an 8-hole ceramic brick and mortar with compressive strength equal to 3.28 MPa strength ($f_{pk} = 3.28$ MPa) (Hendry *et al.* 1997, Moreira 2002), obtained based on experimental tests associated to the compressive strength of the brick, with dimensions equal to 15 cm of width and 2.5 m of height. The masonry infill panels were considered as a homogeneous and isotropic material with longitudinal Young's modulus equal to 2.3 GPa ($E_p = 2.3$ GPa), see Eq. (7) (Hendry *et al.* 1997, Moreira 2002).

$$E_p = 700 f_{pk} \tag{7}$$



Fig. 3 Building floor architecture with location of the masonry infills

Two structural models were developed in this research work, see Fig. 4(a)-(b). The so-called Model 1 (M1) that consists of the reinforced concrete structure only (girders, slabs and columns) and the Model 2 (M2) which presents the same structural system of the first model, however the masonry infills were considered in the modelling, aiming to evaluate the global stiffness of the building.



(a) 3D model perspective (without masonry infills) (b) 3D model perspective (with masonry infills)

Fig. 4 Building structural model: Three-dimensional view of the models M1 and M2

4. Finite element modelling of the building

The proposed computational model, developed for the reinforced concrete building dynamic analysis, adopted the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS program (ANSYS 2009). In this numerical model, the reinforced concrete girders and columns were represented by three-dimensional BEAM44 finite elements, where the effects of bending and torsion are considered. The uniaxial finite element BEAM44 is composed of two nodes and each node with six degrees of freedom (translations and rotations in X, Y and Z directions). The advantage of this element is the possibility of allowing its nodes to be spaced from the centroid axis of the beams, since the slab and the beam are not positioned on the same axis. This eccentricity is considered in the modelling, because affects the values of the building natural frequencies. On the other hand, the concrete slabs and masonry infill were represented based the SHELL63 finite elements. The shell finite element SHELL63 is defined by four nodes and each node with six degrees of freedom (translations and rotations in X, Y and Z directions), see Fig. 5(a)-(b).

The developed numerical model presents an appropriate degree of refinement, aiming to a good representation of the dynamic structural behaviour of the investigated building. Table 1 presents the characteristics of the building numerical model (nodes, elements and degrees of freedom). The support conditions were assumed as hinged, considering that the building supports were restricted to horizontal, translational and vertical displacements and free for rotational displacements.

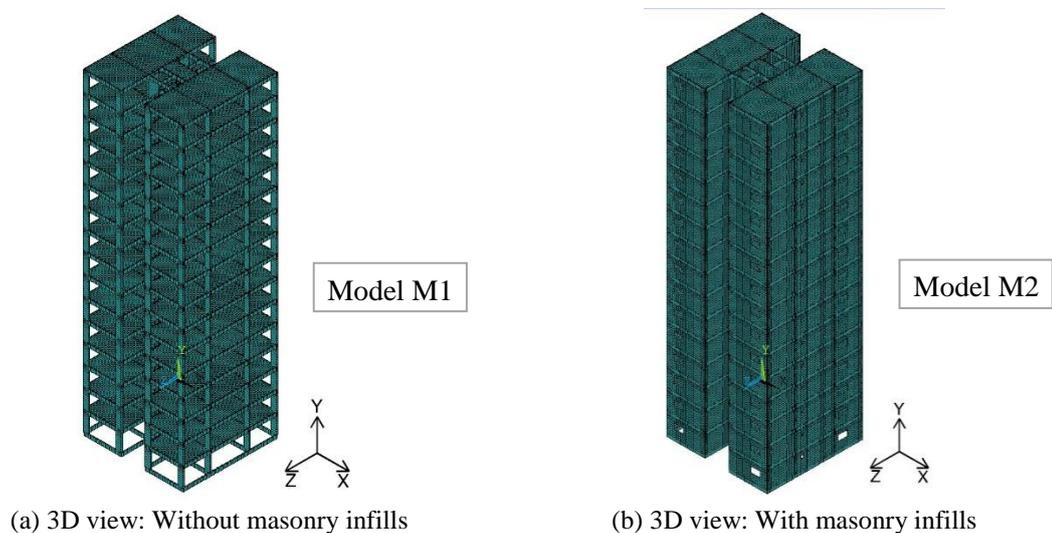


Fig. 5 Finite element model of the investigated reinforced concrete building

Table 1 Description of the FEM M1 and M2

Parameter	Model 1 (M1)	Model 2 (M2)
Nodes	122012	126186
Elements	58138	757068
Degrees of Freedom (DOFs)	732024	134010

In sequence, Fig. 6(a)-(b) presents the numerical discretization of the girders, columns and masonry of the Model (M2). As already mentioned, it is important to remember that the numerical model M2 considers the decoupling of the upper face of the masonry. At the bottom and on the sides of the panel the masonry-structure connections are considered as rigid, as presented in Fig. 6(a)-(b).

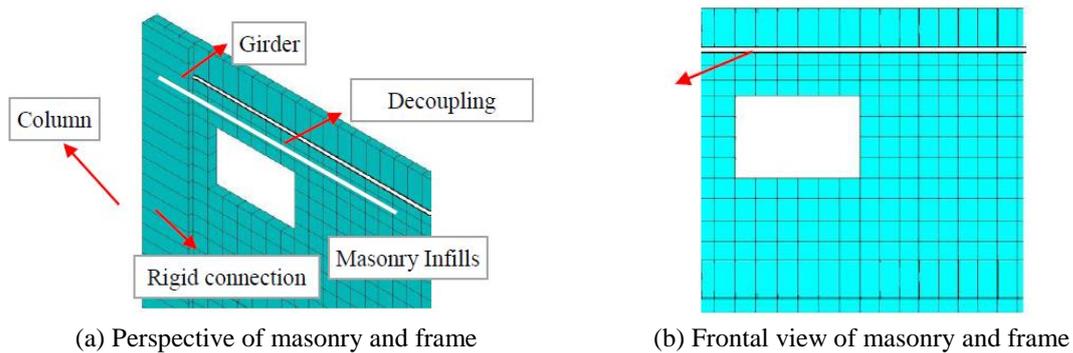


Fig. 6 Detail of the masonry infills in FEM M2

5. Global stiffness of the investigated building

Based on the research work developed by Borges *et al.* (2009), the global effective stiffness coefficient $K_{x,y,z}$ associated at each direction X, Y, Z, can be defined by Eq. (8), where $\Delta_{x,y,z}$ represents the generalized absolute displacements at the top of the building induced by unitary loads, as illustrated in Fig. 7. It must be emphasized that the global effective stiffness can be used as a parameter to compare the structure global stiffness designed to resist different loads, as well as for comparison of the overall stiffness in the elastic phase.

$$K_{x,y,z} = \frac{1}{\Delta_{x,y,z}} \tag{8}$$

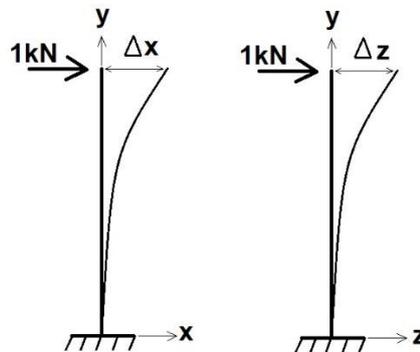


Fig. 7 Global stiffness coefficients

Table 2 Effective global stiffness obtained for each FEM

Model	FEM M1		FEM M2	
	K_X (kN/m)	K_Z (kN/m)	K_X (kN/m)	K_Z (kN/m)
Elastic Phase	6456	6060	35714	43442

According to Table 2 results, it is possible to conclude that the consideration of masonry infills in the numerical models M1 and M2 directly influences the results regarding the global stiffness and the horizontal translational displacements values, due to the application of the unit load. In this situation, Model 2 presents a global stiffness 453% higher in the X direction and 617% higher in the Z direction when compared to Model 1, as illustrated in Fig. 8.

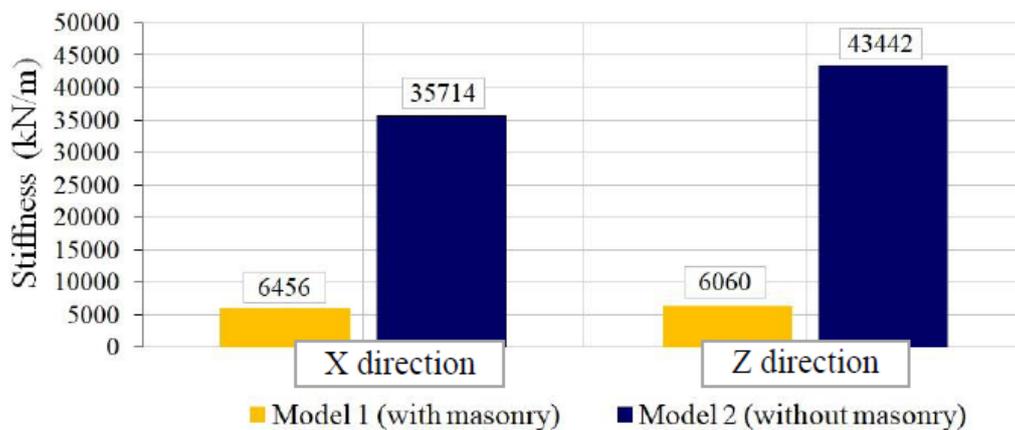


Fig. 8 Comparison between the global stiffness in X and Z directions of the FEM M1 and M2

Table 3 Natural frequencies (f) and periods (T) of the FEM M1 and M2

Vibration Mode	M1			M2		
	Frequency (Hz)	Period (s)	Structural Behaviour	Frequency (Hz)	Period (s)	Structural Behaviour
1	0.53	1.89	Bending around X	1.14	0.88	Bending around Z
2	0.56	1.82	Bending around Z	1.28	0.78	Bending around X
3	0.62	1.61	Torsion	1.88	0.53	Torsion

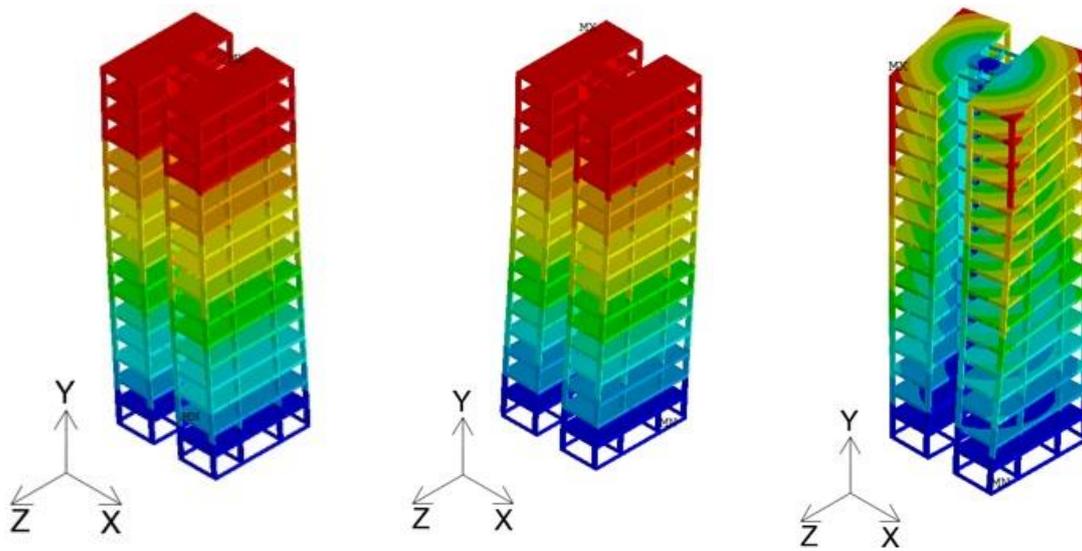
6. Natural frequencies and vibration modes of the building

The natural frequencies (eigenvalues) and the vibration modes (eigenvectors) of the investigated structure were obtained based on numerical methods of extraction (modal analysis), through a free vibration analysis and using the ANSYS (2012) computational program. The first six vibration modes of the Model 1 and Model 2 are shown in Figs. 9-10, respectively.

The results obtained in the modal analysis are described in Table 3 and it is possible to verify that the fundamental frequencies values increase as there are increases in the global stiffness of the

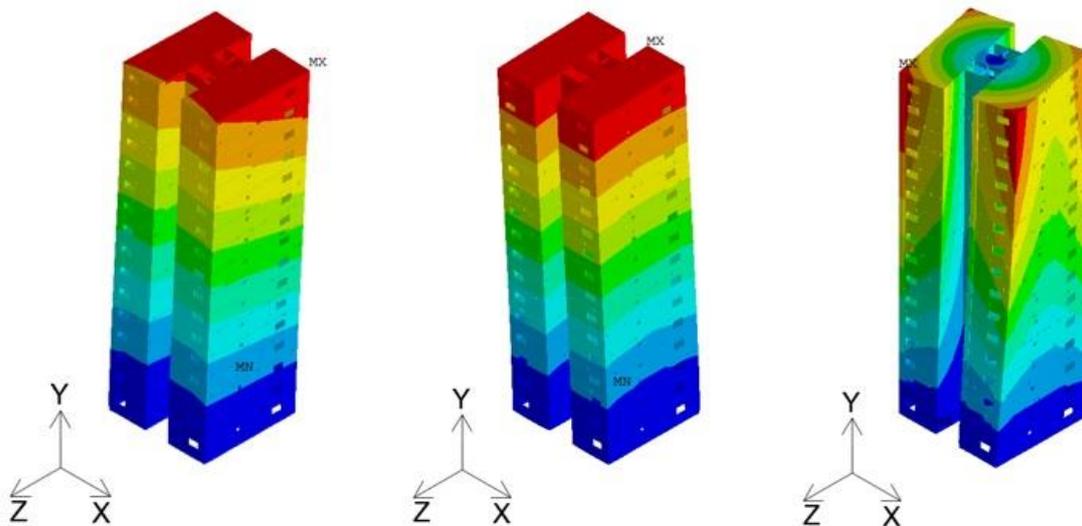
building, based on the consideration of masonry infills. In this situation, the Model 2 (masonry infills modelled based on the use of shell elements), presented a fundamental frequency value 115% higher than the frequency of Model 1, as presented in Table 3.

It is import to emphasize that Model 1 presents its first vibration mode around the X axis, showing greater flexibility in this direction, when compared to Model 2 which presents its fundamental mode shape around the Z axis. This structural behaviour is related to the position of the masonry infills (see Figs. 3 and 5, Fig. 9(a)-(c), and Fig. 10(a)-(c)).



(a) 1st vibration mode ($f_{01}=0.53\text{Hz}$) (b) 2nd vibration mode ($f_{02}=0.56\text{Hz}$) (c) 3rd vibration mode ($f_{03}=0.62\text{Hz}$)

Fig. 9 Vibration modes of the building: M1 (without masonry infills)



(a) 1st vibration mode ($f_{01}=1.14\text{Hz}$) (b) 2nd vibration mode ($f_{02}=1.28\text{Hz}$) (c) 3rd vibration mode ($f_{03}=1.88\text{Hz}$)

Fig. 10 Vibration models of the building: M2 (with masonry infills)

7. Nondeterministic dynamic analysis

Based on the use of the finite element program ANSYS (2012), forced vibration analysis were carried out on the investigated structural model. In addition to the usual vertical design loads, the wind action (nondeterministic wind dynamic loads) was applied on the two facades of the building, referring to the X and Z directions of the numerical model. The basic wind speed was determined considering a time of recurrence equal to 10 years. The results of the dynamic analysis for the maximum horizontal translational displacements values are obtained at the top structural sections of the building (nodes in the FEM: $h=48$ m), and for the maximum accelerations these values are calculated at the floor of the last building storey ($h=45$ m). In this research work 30 nondeterministic wind series were generated, considering the two horizontal displacements in the X and Z axes, for each FEM M1 and M2, respectively. The parameters used to generate the nondeterministic wind series are shown in Table 4.

Table 4 Parameters used to generate the nondeterministic wind series

Wind Basic Velocity (V_0)	35 m/s
Terrain Category	II
Topographic Factor (S1)	1
Parameters for Roughness Factor (S2)	$b=1$ and $p=0.15$
Probability Factor (S3)	0.51
Time Duration	600 seconds
Time Increment	0.03 seconds

According to NBR 6123 (1988), the basic wind velocity represents the wind for the city of Rio de Janeiro that occurs at least one time in 50 years with duration of 3 seconds, the category II is related to open lands in level with few isolated obstacles. The category is used to determine the parameters for the calculation of the factor S2 that indicates the variation of the velocity according to the height and roughness of the terrain. The topographic factor S1 equal to 1 corresponds to a situation of flat terrain that is weakly uneven and the probabilistic factor S3 adopted corresponds to a wind with period of recurrence of 10 years with probability of occurrence of 63%. The time duration of 10 minutes (600 seconds) is usually adopted to study wind effects in structures.

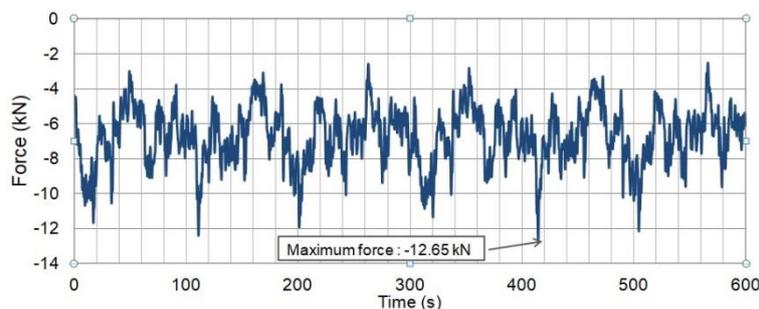


Fig. 11 Example of nondeterministic wind force generated at height of 45 meters in time domain

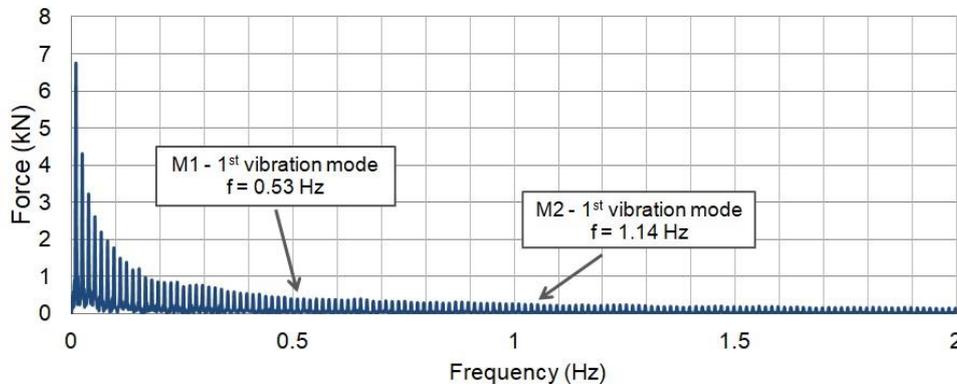


Fig. 12 Example of wind force spectrum generated at height of 45 meters in frequency domain

Fig. 11 presents a typical example of a nondeterministic dynamic wind force generated using Eqs. (1)-(7), where it is possible to observe the nondeterministic characteristic of the dynamic load. The range of frequencies used to generate the wind series can be visualized in Fig. 12 and was determined considering the first natural frequency of each FEM M1 and M2, respectively.

Since the wind dynamic actions considered in this investigation presents nondeterministic characteristics, it is not possible to predict the response of the building at a certain instant of time. Thus, a reliable response can be achieved through an appropriate statistical treatment. This way, considering that the dynamic structural response presents a normal distribution (Gauss distribution), and based on the calculation of the mean (m) and also the standard deviation (σ), it is possible to obtain the characteristic value ($U_{z95\%}$) that corresponds to a reliability of 95%, meaning that only 5% of the sampled values will exceed this value, as presented in Eq. (9) (Chaves 2006).

$$U_{z95\%} = 1,65 \sigma + m \quad (9)$$

As far as the convergence of the numerical results is concerned, Figs. 13-14 illustrates the characteristic displacements ($U_{z95\%}$) at the top of the building (FEM M1 and M2) for the rigid support models (see Fig. 5), calculated gradually based on the results of each nondeterministic series, pointing out to the importance of applying an adequate number of wind load series in order to obtain a consistent result. This way, the dynamic structural analysis was performed for each individual nondeterministic wind series, aiming to achieve a numerical convergence of the results, as presented in Figs. 13-14.

In sequence, based on the Table 5 results, it can be seen that the maximum translational horizontal displacements values calculated at the top of the building ($h=48$ m), associated to the X and Z directions, are equal to 6.1 cm and 5.4 cm, respectively (Model 1 (M1)), and equal to 0.9 cm and 0.6 cm, respectively (Model 2 (M2)). It can be concluded that Model 2 presents displacements values about 6.0 times higher in X direction and 10 times higher in Z direction when compared with the Model 1 structural response, as shown in Table 5.

On the other hand, the maximum acceleration values calculated at the floor of the last building storey ($h=45$ m), related to the X and Z directions, are equal to 0.32 m/s^2 and 0.31 m/s^2 , respectively (Model 1 (M1)), and equal to 0.16 m/s^2 and 0.18 m/s^2 , respectively [Model 2 (M2)]. It is possible to notice that the accelerations obtained with Model 2 are almost 2 times higher in the X and Z directions than in Model 1, see Table 5.

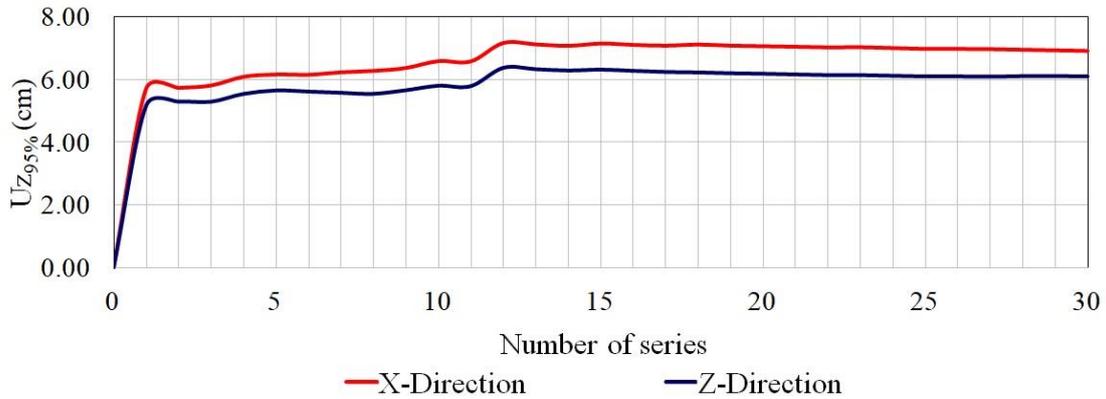


Fig. 13 Convergence of the translational horizontal displacements values: Model 1 (without masonry)

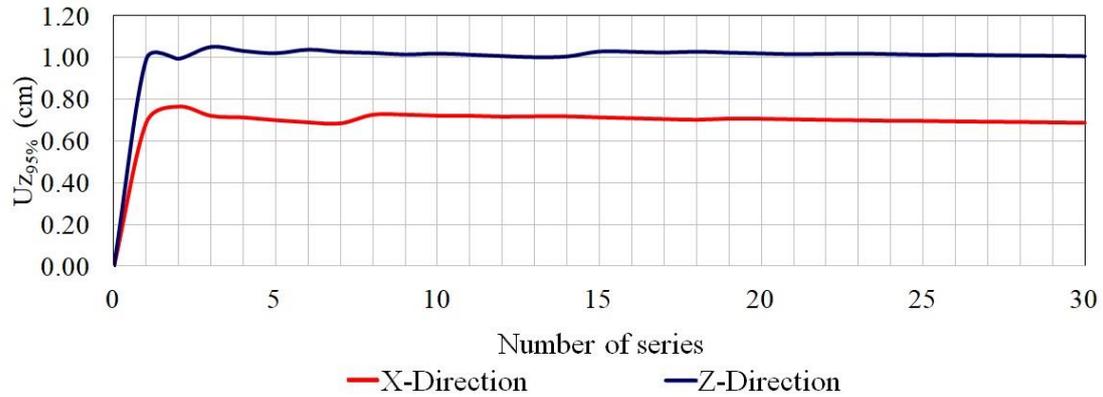


Fig. 14 Convergence of the translational horizontal displacements values: Model 2 (with masonry infills)

Table 5 Mean maximum peak accelerations values associated to the X and Z directions: FEM M1 and M2

Direction	Model	Displacement (cm)	Accelerations (m/s ²)
X	M1	0.90	0.18
	M2	5.04	0.32
	M2/M1	5.60	1.80
Z	M1	0.61	0.16
	M2	6.07	0.31
	M2/M1	9.95	1.94

The maximum translational horizontal displacements calculated at the top of the investigated building ($h=48$ m) for each nondeterministic wind series are presented in Figs. 15-16. It must be emphasized that the maximum displacements values, associated to the X and Z directions, surpassed the Brazilian standard NBR 6118 (2014) recommended limit of $H/1700$ (2.8 cm), when the Model 1 displacements are considered. This fact clearly shows that the horizontal displacements values at the top of the structural model are directly influenced by the structural stiffness provided by the masonry in Model 2, as illustrated in Figs. 15-16.

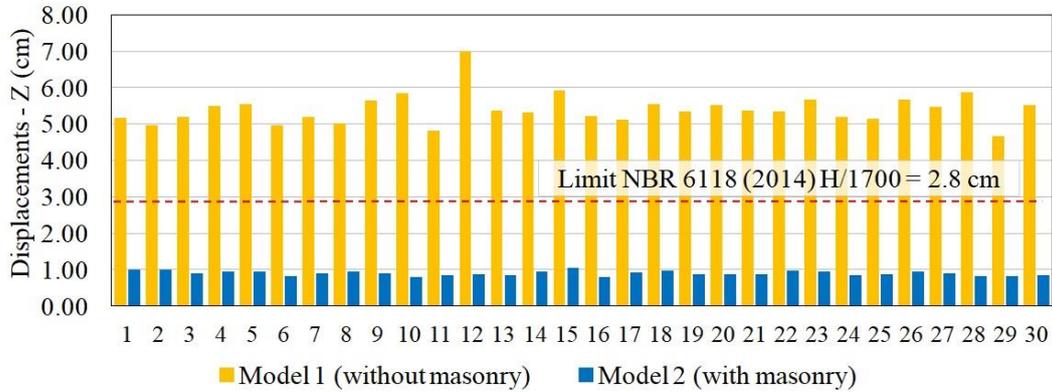


Fig. 15 Maximum translational horizontal displacements in Z direction: Model 1 and Model 2

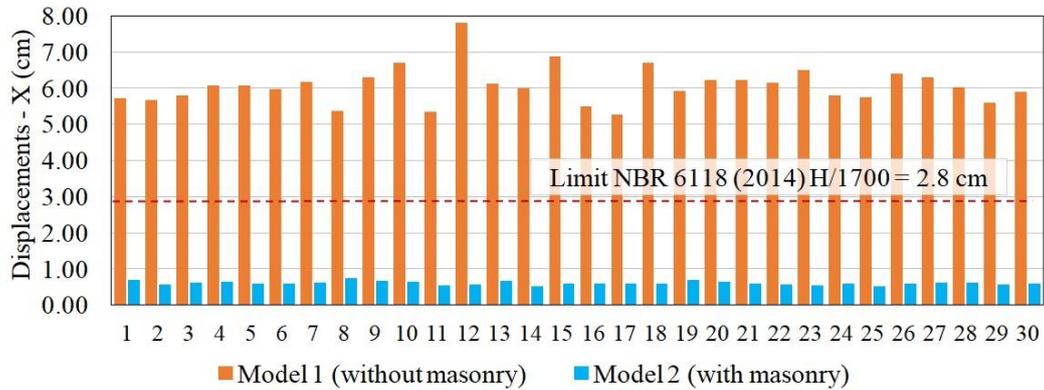


Fig. 16 Maximum translational horizontal displacements in X direction: Model 1 and Model 2

When the human comfort assessment of the building is considered, Figs. 17-18 illustrate the maximum accelerations calculated based on the use of each nondeterministic wind loading series. It must be emphasized that the obtained results, considering a time of recurrence equal to 10 years, exceeded the acceleration limit of 0.10 m/s^2 proposed by Brazilian standard NBR 6123 (1988).

Regarding the analysis of the peak accelerations, in time domain, having in mind a human comfort assessment of the investigated building, the influence of the masonry infills can be observed in Figs. 17-18, where is clearly possible to verify that the consideration of the masonry infills in the numerical models FEM M1 and M2 have produced a reduction in the accelerations values, evidencing the importance of the masonry effect for a reliable human comfort assessment.

It must be emphasized, that the results presented in this research work were obtained using nondeterministic wind series, and considering that the basic wind velocity is calculated for a recurrence time of 10 years. The analyses were carried out aiming the assessment of the building according to the limit of 0.1 m/s^2 proposed by NBR 6123 (1988) that considers admissible if this limit is exceeded, on mean, once every ten years. On the other hand, another international design standards indicates that the human comfort assessment also can be performed considering a time of recurrence of 1 year (AIJ-GEH 2004, ISO-10137 2007), obtained based on investigations with occupants of high buildings.

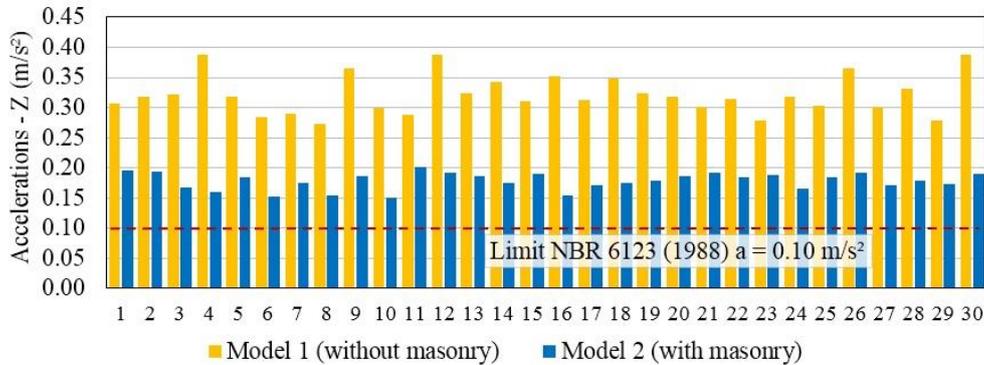


Fig. 17 Peak accelerations values in Z direction: Model 1 and Model 2

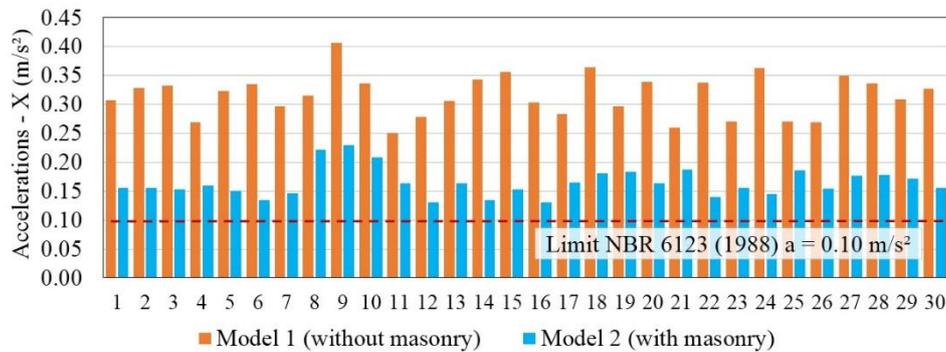


Fig. 18 Peak accelerations values in X direction: Model 1 and Model 2

8. Conclusions

The objective of this investigation was to study the influence of the masonry infills on the global structural behaviour, aiming to assess the human comfort of a residential building, constructed at the city of Rio de Janeiro/RJ, Brazil, when subjected to nondeterministic wind loads. The investigated building presents rectangular dimensions of 15.00 m by 14.20 m, is composed by 16 floors, with a height of 3.0 m, having a total height of 48 m. Static, modal and dynamic analysis (forced vibration analyses) were carried out based on the development of two finite element models Model 1 (M1) that consists of the reinforced concrete structure only (girders, slabs and columns) and the Model 2 (M2) which includes the masonry infills in the modelling, aiming to evaluate the global stiffness of the building. Thus, the main conclusions of the present investigation are:

- Global stiffness: Static analyses have shown that the presence of the masonry infills in the numerical modelling have increased the building global lateral stiffness around 453% in the X direction and 617% in the Z direction.

- Modal analysis: The presence of the masonry caused the increase of the building natural frequencies values. Considering the first and second natural frequencies, there was an increase of 107% in the X direction and 141.5% in the Z direction. It was also possible to conclude that the location of the masonry changed the first two vibration modes of the structure. It was verified that

the structural model without masonry presented the first and second mode shape based on bending around Z and X, respectively, however, the presence of the masonry infills have changed this situation to bending around X and Z.

- **Forced vibration:** Based on the nondeterministic dynamic analysis, it was possible to verify the significant decrease of 6 times in the X direction and 10 times in the Z direction of the displacements values at the top of the building when the masonry infills were considered. The accelerations values also have presented a decrease around 2 times when the masonry was considered in the analysis.

- **Human comfort:** When the human comfort assessment was considered in this investigation, it was concluded that even having in mind that the presence of the masonry infills cause the decrease of the peak accelerations, the user of the building will sense the wind vibrations, based on the Brazilian standard NBR 6123 (1988) criterion.

Thus, it was possible to verify that the consideration of the masonry infills in the analysis have produced relevant modifications in the global structural stiffness of the building and, consequently, changing the natural frequencies values, causing the decrease in horizontal translational displacements and peak accelerations of the building.

This conclusion is important because when the resonance effects related to the wind actions in buildings are considered, these differences can be significant and deserve the attention of the structural designer. It should be noted that for values of natural frequencies below 1 Hz, relevant to define the need for a dynamic analysis of the wind actions (forced vibration analysis), the consideration of the stiffness of the masonry infills in the numerical modelling increased the natural frequency of the bending vibration mode around 200% which is desirable for human comfort assessments.

Finally, the authors would like to emphasize the understanding of the necessity of the following of the research work, based on the evaluation of the soil-structure interaction effect and also the influence of the interface modelling between the masonry infills and the structure when the dynamic response of buildings is investigated, aiming to contribute with a more realistic assessment of the dynamic structural behaviour and human comfort evaluation of tall buildings, due to the fact that in the connections between the masonry infills and the columns, for instance, usually in the construction process these connections are not executed properly and the behaviour can be different to projected.

Acknowledgments

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

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