CILAMCE
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RIO DE JANEIRO 2021

The static structural analysis of a historical masonry building at Curitiba, PR.

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Abstract. The structural analysis of historical structures is an important requirement to accomplish the conservation of historical heritage efficiently. Frequently the structural analysis of historical buildings is a hard task. These difficulties are due to the empirical way in which most of these structures were designed and built, making it difficult to obtain the structural material mechanic parameters, and to obtain the knowledge of the way they were built. Although Brazil presents several historical buildings there are few studies concerning the structural analysis of such structures. In this scenario, this work has as aim to contribute to the structural analysis of Brazilian historical building in order to be useful to future researchers of the subject. This work consists of a case study which uses a numerical finite element method using the Abaqus/Cae program to perform the static analysis of a colonial Building, built in structural brick masonry with a wooden truss system roof dated from the 18TH at Curitiba, PR. The obtained static analysis consists of the stresses and displacements of the masonry walls under the action of self-weight and wind loads. The obtained results allow to assess the current structural safety condition of the building.

Keywords: FEM, historical constructions, brick masonry.

1. Introduction

Around the world the exploitation of the historical built heritage is a key issue, as it represents an economic concern especially in contexts where tourism has become a major source of wealth. Normally, the conservation or rehabilitation of a historic buildings, such as those constructed using masonry, requires a deeper knowledge of the understanding of the historical processes of both design and construction (Marques, [1]) While the structural behavior of a new masonry construction is a relatively simple task to the modern engineering professionals (due to the presence of standard codes, inherent literature and accurate knowledge of material properties), the prediction of the structural response of historical buildings is still a challenging task (Bartoli and Orlando, [2]). The structural analysis of historical constructions is interesting not only as a contribution to the enhancement and preservation of the memory of a given society, but also allows the development of rehabilitation techniques for these structures that, when effectively applied, allow their replacement as an active part of their communities, in addition to contribute to reducing the need for new construction (Mesquita et al, [3]).

Historical or traditional materials such as earth, brick or stone masonry, and wood are characterized by very complex mechanical and strength phenomena still challenging the civil engineering modern modeling abilities according to Roca et co-workers [4]. In particular, masonry is characterized by its composite character (it includes stone or brick in combination with mortar or day joints), a brittle response in tension (with almost null tensile strength), a frictional response in shear (once the limited bond between units and mortar is lost) and anisotropy (for the response is highly sensitive to the orientation of loads) (Roca et al, [4]). Besides the material complexities there are additional challenges on the structural analysis due to complex geometries frequently observed on the historic buildings' architecture. Other source of complexity in the structural analysis of ancient buildings relies on the peculiar nature of the construction methods used in old buildings. This fact leads to limitations regarding the application of modern standards and codes. Consequently, the structural analysis and, the diagnosis is a difficult task, being therefore necessary specific analyzes for each building, through more complex methods such as computational methods (Mesquita et al, [3]). The use of finite elements method (FEM) is a very common tool to perform structural analysis of historical masonry constructions. The modern numerical methods (such as FEM) afford a realistic and accurate description of the structural behavior of historic buildings (Roca et al, [4]). In this scenario this work uses a FEM model to obtain the static structural analysis of a colonial brick masonry building situated at Curitiba, Brazil. The FEM model is obtained using the ABAQUS/CAE commercial software. The case study building is described in section 2, including the used mechanical parameters and the loading assessment. The FEM model is described on section 3 and the results for the two main external walls are presented in section 4, which includes the stress and strain fields for the two main walls of the considered building.

2. Case-study.

The building used as case-study is known as Casa Romário Martins, located at Rua Largo Coronel Enéas, 48 - São Francisco – Curitiba – PR (see figure 1a). Casa Romário Martins is the last example of Portuguese colonial architecture in the center of Curitiba. It was built in the 18th century and since 1973 it works as a cultural space (see Figure 1b) [5]. Casa Romário Martins is a single-story rectangular plan building (13.642 m x 13.144 m, see Fig. 2), with a height of 4.20 m built in structural brick masonry and have a hip roof with a height 3.77 m (see Fig. 3), made of colonial ceramic tile supported by a wooden truss system. The northern and western walls have a width equal to 0.74 m and 0.714 m respectively. The analysis was done considering the two main external walls, named according to their orientation, the northern and western wall (see Fig. 3). The northern wall has two doors (1.14 m x 2.63 m and 1.16 x 2.63 m) and one window (1.16 m x 1.71 m), while the western wall has two doors (1.145 m x 2.63 m and 1.31 m x 2.63 m) and two windows (1.13 m x 1.71 m). All windows and doors have an arched lintel.

Figure 1. a) Casa Romário Martins satellite view – Google maps 2021; b) Western façade view (- Fundação Cultural de Curitiba, 2021)

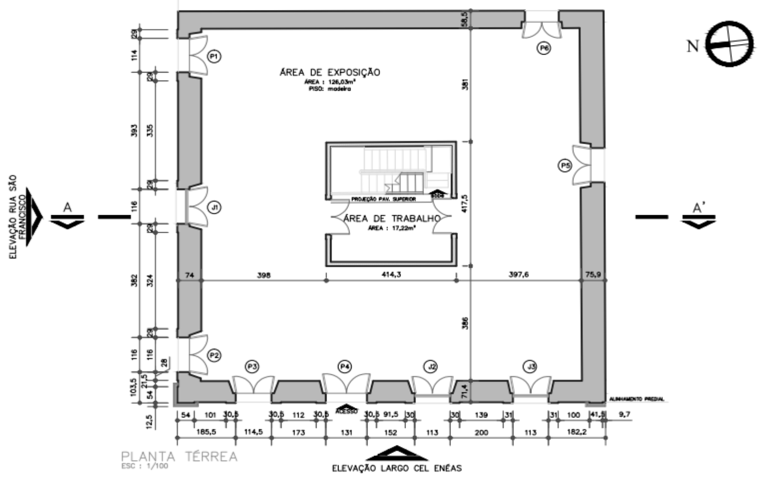

Figure 2. Floor Plan (1:100) [6].

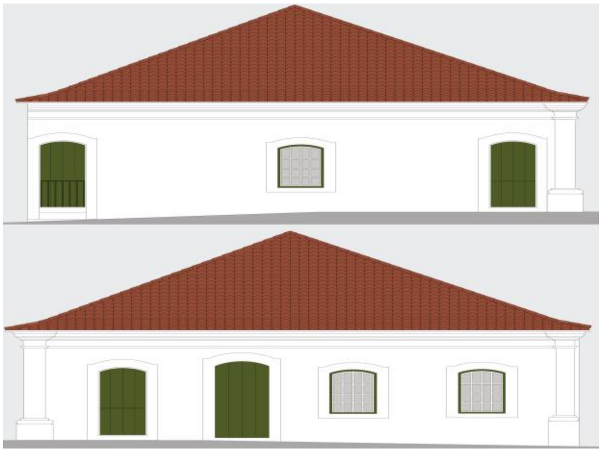

Figure 3. Northern (up) and western (down) walls elevations (Arquivo Arquitetura, [6]).

2.1 Material Parameters

Due to COVID-19 emergence, the laboratory and field testing was not performed and the mechanical parameters used were obtained from the correlated bibliography. The masonry adopted parameters found in [7-15] are showed in Table 1. The roof weight, γ_r, is equal to 0.60 KN/m^2 (ABNT [16]).

Table 1 – Brick masonry wall properties [7-15].

Brick Masonry Wall	Nomenclature	Value
Poisson's Ratio	ν	0.2
Young's Modulus	E	5000 MPa
Specific weigh	γ	1800 kg/m3
Compressive strength	f_c	3.5 MPa
Tensile strength	f_t	0.2 MPa

2.2 Loading

Casa Romário Martins has a hip roof, the weight of each part of the roof is supported by the walls directly below. The estimation of the distributed load on the wall's top was done by the concept of the influence area. The roof loads upon the northern and western walls are equal to 2.34 kN/m and 2.27 kN/m respectively. The wind loads were evaluated using the Brazilian standard [17], while the drag coefficient was evaluated using the European standard [18], since the case of a hip roof is not included on the Brazilian standard [17]. These drag wind force, F_a, is equal to:

$$F_a = C_a q \times A_e \tag{1}$$

where C_a is the drag coefficient, q is the dynamic wind pressure and A_e the orthogonal area under the action of the wind pressure. The wind pressure (under normal conditions of pressure 1 atm and temperature 15°C) is evaluated as follow:

$$q = 0.613Vk^2 \tag{2}$$

where V_k is the site characteristic wind velocity, which is, using the procedures of the Brazilian standard [17], equal to 35 m/s. Using eq. (2) one obtains q equals to 757 N/m^2, which are considerate the same for roof and walls surface. The drag coefficient can be evaluated as:

$$C_a = C_{pe} - C_{pi} \tag{3}$$

where C_{pe} and C_{pi} are the external and internal wind pressure coefficients respectively. The C_{pe} and C_{pi} values for the wall and the Cpi value for the roof was taken from [17] while the Ce for the roof was taken from [18]. The drag coefficients are showed on Table 3, including the wind directions of α=90º and α=0º. As one can observe from the Table 3 results, the worst scenario for both walls was a C_a equal to 1. While the worst wind scenario for the roof was a Ca equal to 0.8. By substituting the q value obtained and the drag coefficients showed on Table 2 in eq. (1) one obtains the drag forces, obtained using eqs (1)-(3) and Table 2 values, are showed in Table 3.

Table 2 – Drag coefficients.

Wind direction	α= 90°			α= 0°		
	C_{pe}	C_{pi}	C_a	C_{pe}	C_{pi}	C_a
Walls	-0.8	-0.3	-0.5	+0.7	0.0	+0.7
	-0.8	0.0	-0.8	+0.7	-0.3	+1.0
Roof	-0.4	-0.3	-0.1	-0.8	-0.3	-0.5
	-0.4	0.0	-0.4	-0.8	0.0	-0.8

Table 3– Effective areas and drag forces.

	Western wall	Western roof	Northern wall	Northern roof
A_e (m²)	46.96	51.68	47.17	51.22
F_a (kN)	35.57	31.01	35.73	30.73

3. FEM model.

Each main wall was modelled as an isolated plate under the action of self-weight and wind loads as evaluated on the previous section. The model was obtained using the student version of ABAQUS software. The mesh was obtained automatically by the problem using shell elements [19] with 959 nodes on northern wall and 984 nodes on western wall as shown in Fig. 4.

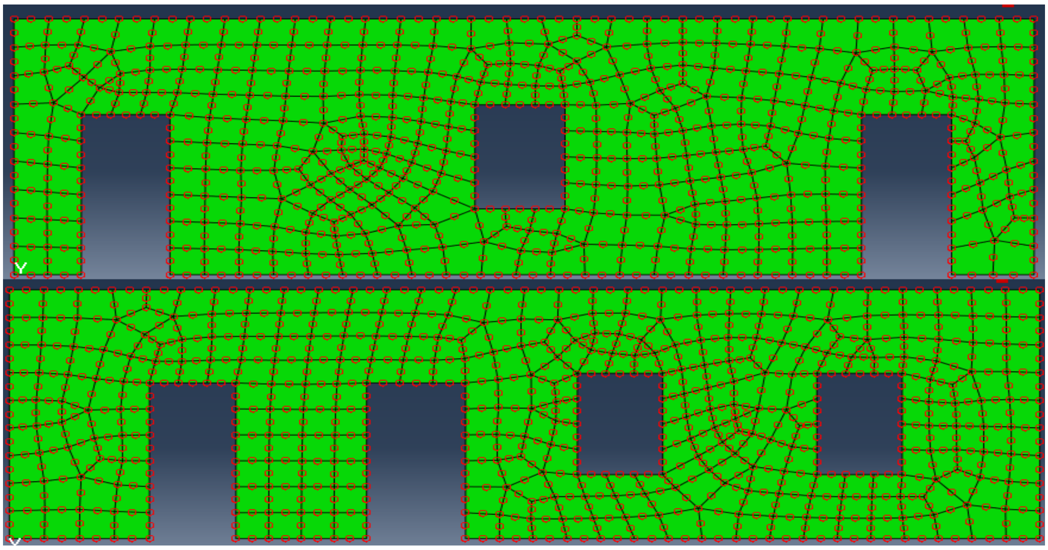

Figure 4 - Northern wall (up) and western wall (down) meshes.

After some convergence study, limited by the nodes number allowed by the student version of ABAQUS, the model for each wall has the geometric features listed on Table 4.

Table 4 – Walls numerical model geometrical features.

wall	Height (m)	Length (m)	Thickness (m)	Element number	Nodes number
Northern wall	4.20	13.144	0.740	283	959
Western wall	4.20	13.642	0.714	287	984

The study considers valid the hypothesis of linear-elastic analysis and isotropy. These simplifications were a common strategy adopted on initial analysis of historical structures (Ramirez, [7]). Windows and doors frames were excluded, since they do not have structural role on the building, their openings, however, were considered as showed in Fig. 4 for the models of both walls. The support conditions considered for each wall were: the lower boundary with no horizontal and no vertical displacements, the lateral boundaries with no horizontal displacements and the upper boundary with no restricted displacements. The parameters values used in the FEM model were taken from Tables 1 and 4. The wind load was considered as constant surface static pressure distributed over the entire area of the walls. This simplification is justified by the smaller height of the walls. Roof weight was modelled as linear distributed loads over the upper boundary of the walls, the same was done for the wind pressure load acting upon the roof.

4. Numerical results.

The normal stress and lateral displacements fields due the simultaneous action of wind and self-weight loads are presented for the northern and western walls in Fig. 5 and Figure 6 respectively.

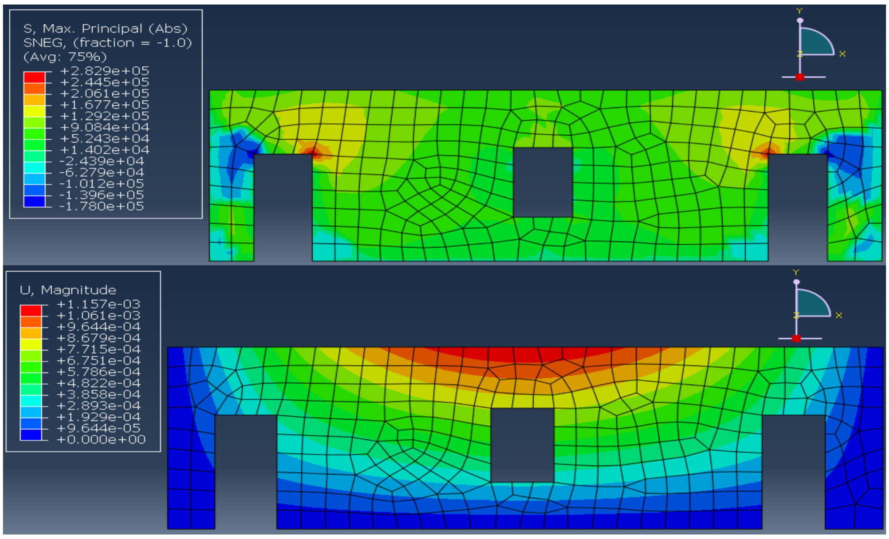

Figure 5 – Northern wall results: Stress in Pa (up) and displacement in m (down).

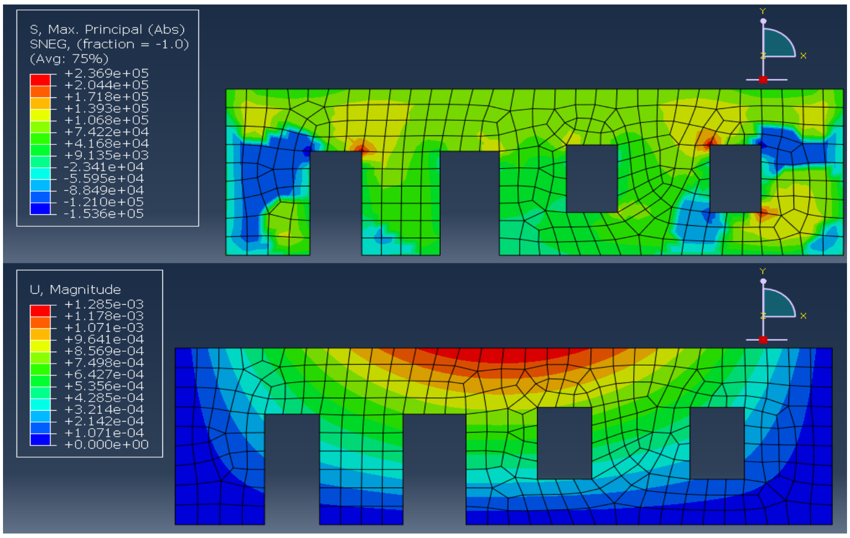

Figure 6 – Western wall results: Stress in Pa (up) and displacement in m (down).

One can observe that the larger stresses areas are located on the edges of the windows and doors due to the stress concentration in those areas. The larger displacements regions are located on the upper central part of the walls, since the wall behaviors ass a plate with 3 supported edge and one free. The maximum displacements and stresses points for each wall were shown in Tables 5 and 6, their coordinates (X,Y) reefer to the lower-left corner of each wall. Although the northern wall has fewer windows and doors opening than the western wall, it experiments larger stresses due to this opening configuration. While the western wall experiments larger lateral displacements once it is thinner than the northern wall.

Table 5 – stress and displacement values.

	Maximum stresses (Pa)		Maximum
	Tensile	Compressive	Displacement (m)
Western wall	+2.369E+05	-1.536E+05	+1.285E-03
Northern wall	+2.829E+05	-1.780E+05	+1.157E-03

Table 6 – Node coordinates in m of maximum and minimum stress and displacement (X,Y).

	Points of maximum stresses		Maximum
	Tensile	Compressive	Displacement (m)
Western wall	(10.69; 2.79)	(1.855; 2.63)	(6.821; 4.20)
Northern wall	(2.01; 2.63)	(0.87; 2.63)	(6.572; 4.20)

From the results of Table 5 one can observe that the structure stress state is very low for the considered loads. This fact confirms the observations made by Heyman [15] that masonry historical buildings have a low background of compressive stress. When the working maximum compression stress, 0.178 MPa (obtained for northern wall) is compared to the crushing stress varying from 1 to 7 MPa [7-15] the results are larger factors of safety between 5.6 and 39.3. This lower compressive state is essential to the stability of the masonry structure, since larger compressive forces could lead to the sliding of bricks (Heyman, [15]). The tensile stress factors of safety are much smaller, between 0.4 and 1.1 (for tensile allowed stresses varying from 0.1 MPa to 0.3 MPa [7-15]). Such fact is also in agreement with the behavior of historical masonry structures. Although bricks could support some tensile stress, the mortar is indeed a weak material and the acting tensile stress could lead to the cracking of the masonry fabric of the wall. However, the building overall stability can be kept by the accommodation of the displacement of the wall cracked parts due its larger width. The so originated displacements cannot be obtained by the elastic analysis, and, for the same reason, they do not correspond to the values observed in Fig. 5, Fig. 6 and Table 5.

However, the resulted elastic displacements are very small when compared to the wall width and do not compromise the building stability.

5. Conclusion.

Casa Romário Martins is an important historical building for the city of Curitiba. This importance justifies the derivation of structural analysis tools to help and guide conservational principles and actions. The obtained results indicate that is possible to use the FEM to obtain numerical models to perform structural analysis of this historical building. The results indicate that the building masonry structure is under a lower compressive stress field for the self-weight and wind loadings. The tensile stress results indicates that some cracks could be occur at the masonry fabric, which is a common structural response in historical masonry buildings. That cracking is perfectly accommodated by some differential lateral displacement which does not affect the building stability, due to the large wall width. Those results, even though relying on simplifications, helps to explain how since its construction in the 18th century, Casa Romário Martins has remained structurally stable. This study is important to help the city administration to maintain its historical heritage preserved in the most original way possible. This is a work in progress. Some suggestions of further research include:1) A more precise modelling of wind and self-weight considering their geometric complexities; 2) Testing the materials used at of Casa Romário Martins to obtain their mechanical properties; 3) Derive an overall 3D model for the building structure; 4) Perform a plastic analysis; 5) Consider the masonry fabric anisotropy; 6) Include the dynamical analysis of the building; 7) Include in the walls model the debris fill used in the construction.

Acknowledgements. The authors would like to thank National Council for Scientific and Technological Development (CNPq), the Programa de Pós-Graduação em Engenharia de Construção Civil (PPGECC/UFPR) and the Centro de Estudos em Engenharia Civil (CESEC/ UFPR) for their support

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References

[1] Marques, R. F. P. (2012). Metodologias Inovadoras no Cálculo Sísmico de Estruturas em Alvenaria Simples e Confinada. Tese de Doutoramento em Engenharia Civil/Estruturas, Universidade do Minho, Guimarães, p 248.
[2] Michele Betti, Gianni Bartoli, Maurizio Orlando. (2010). Evaluation study on structural fault of a Renaissance Italian palace. Department of Civil and Environmental Engineering (DICeA), University of Florence, Via di Santa Marta, 3, I-50139 Firenze, Italy.
[3] Mesquita, E.; Brandão, F.; Diógenes, A.; Antunes, P.; Varum, H. (2017). Ambient vibrational characterization of the Nossa Senhora das Dores Church. Engineering Structures and Technologies, v. 9, n. 4, p. 170-182.
[4] Roca, Pere Cervera, Miguel Gariup, Giuseppe Pela', Luca. (2010). Structural analysis of masonry historical constructions. Classical and advanced approaches. Archives of Computational Methods in Engineering, V.17, p 299-325.
[5] turismo.curitiba.pr.gov.br, 2021.
[6] Arquivoarquitetura.com, 2021.
[7] Ramirez, K. N. (2010). Analysis of the structural behavior of the Cathedral of Sé in São Paulo.
[8] Lourenço, P. B. (2002). Computations on historic masonry structures. Progress in Structural.
[9] Kujawa, M., & Lubowiecka, I. (2020). Finite element modelling of a historic church structure in the context of a masonry damage analysis. 107(June 2019), 1–18.
[10] Lombillo, I., Thomas, C., Villegas, L., Fernández-álvarez, J. P., & Norambuena-contreras, J. (2013). Construction and Building Materials Mechanical characterization of rubble stone masonry walls using non and minor destructive tests. Construction and Building Materials, 43, 266–277.
[11] Peña, F., Lourenço, P. B., Mendes, N., & Oliveira, D. V. (2010). Numerical models for the seismic assessment of an old masonry tower. Engineering Structures, 32(5), 1466–1478.
[12] Dome, N. (2009). Mechanical behaviour of ancient masonry. 123–133.
[13] Heyman, J. (1967). On shell solutions for masonry domes. International Journal of Solids and Structures, 3(2), 227–241.
[14] Lourenço, P. B. (2000). Anisotropic Softening Model for Masonry Plates and Shells.
[15] Heyman, J. (1995). The Stone Skeleton.
[16] ABNT NBR6120:2019 AÇÕES PARA O CÁLCULO DE ESTRUTURAS DE EDIFICAÇÕES.
[17] ABNT NBR6123:1988 FORÇAS DEVIDAS AO VENTO EM EDIFICAÇÕES.
[18] EN 1991-1-4 (2005): Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions.
[19] abaqus-doc.mit.edu, 2021.