INFLUENCE OF MODELLING METHODS IN THE ASSESSMENT OF THE SEISMIC BEHAVIOUR OF MASONRY BRICK CHIMNEYS

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Abstract: *Industrial chimneys are common in many parts of the world and are an important asset of the industrial heritage. The methods used to analyse this type of constructions should be in agreement to their importance, namely should follow the best practices concerning the intervention on built heritage, which requires, as starting point, a detailed geometric, material and damage survey, as well as the assessment of dynamic characteristics, to properly calibrate numerical models.*

In this context, a masonry chimney located in the city of Porto was, first, assessed, and after simulated numerically using two different modelling approaches: i) linear elastic analysis with shell elements in SAP2000, with further non-linear analysis at the cross-section level; ii) material and geometrical nonlinear simulation with fibre elements in SeismoStruct.

The paper compares the influence of the two modelling assumptions in the results and concludes with a reflection about the importance of non-linear simulations, when defining the seismic behaviour of this type of structures..

1 Introduction

Industrial heritage holds a fundamental importance in the history of a society, being a repository of the history and development of industrialization processes. This significance is underscored in "The Principles of Dublin" (ICOMOS-TICCIH, 2011) and by the existence of organizations dedicated to the protection and preservation of industrial heritage, such as the International Scientific Committee on Industrial Heritage (ISC Industrial Heritage) within ICOMOS.

Within the scope of its activities, the design office NCREP - Consultancy in Building Rehabilitation and Heritage, Ltd., has conducted inspections and interventions on industrial chimneys, the majority of which are constructed from solid ceramic brick masonry (Guedes *et al*, 2019). However, different approaches have been, and can be followed to perform the structural analysis of these structural typologies to assess structural reinforcement needs under seismic conditions. This paper evaluates the most suitable methods, applying advanced and alternative tools for dynamic behaviour identification and seismic behaviour analysis, namely employing nonlinear material behaviour models, to a case study. It is worth noting that wind can be a conditioning factor in assessing the safety of these structures (including phenomena like resonance and vortex shedding), but the wind effect falls outside the scope of this investigation.

2 PRESENTATION OF THE CASE STUDY

2.1 Description of the Chimney

A chimney located in Porto was selected as case study. It has a total height of approximately 17.95m above ground level, divided into three distinct height zones: i) a square base with a height of around 1.85 m and dimensions ranging from 1.96m to 2.30m on each side; ii) the main body (shaft) with a truncated conical geometry, a height of about 14.34m and an outer diameter that linearly varies between 1.58 m and 0.84 m; iii) the upper finishing structure (crown) with a height of approximately 1.75 m and an outer diameter at the top of about 0.84 m (according to the provided geometrical survey). The base features a small opening on the east face, with an approximate section of 0.40 x 0.60 m2, providing access to the interior of the chimney. The chimney does not have any structural metal bands, but it does include a lightning rod at the top.

The total height of the chimney, 17.95 m, and the base's side length of 2.37 m give this structure a slenderness ratio (height/base dimension) of approximately 7.6, a value that is slightly below, but close to the lower limit typically found in similar chimneys (slenderness ratios ranging from 8 to 11) (Guedes *et al*, 2019).

Figure 2.1 depicts images of the chimney, as well as its cross-section geometry, with the thickness of the side walls estimated from measurements taken during the inspection phase and general information collected from archival elements related to similar geometry chimneys (Costa *et al.*, 2014).

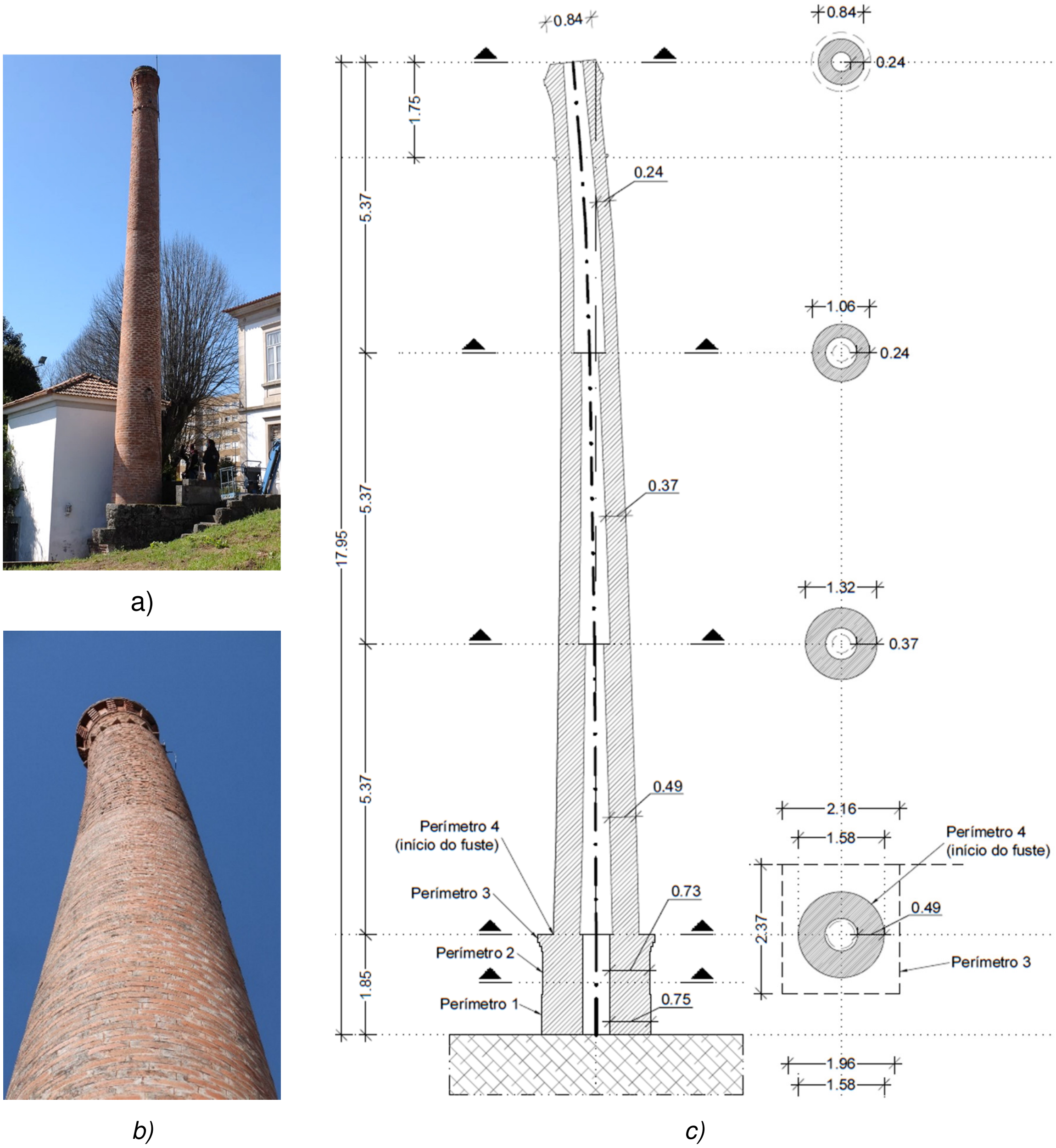


Figure 2.1. Chimney under study: a) overall view; b) close-up of the top 1/3; c) geometry

2.2 Identified Anomalies and Conservation Status

The primary anomaly observed in the chimney is the pronounced deformation it exhibits in the southwest direction, particularly in the upper 1/3 (Figure 2.1). The deformation, obtained through topographic survey, reached a maximum of 0.44 m recorded at the top (0.43 m in the X direction and 0.10 m in the Y direction, following the principal axes defined for the analyses - Figure 2.2). Additionally, material degradation of some ceramic blocks and joints, as well as minor cracks and localized joint openings, were identified, resulting in a material conservation state that can be broadly categorized as good.

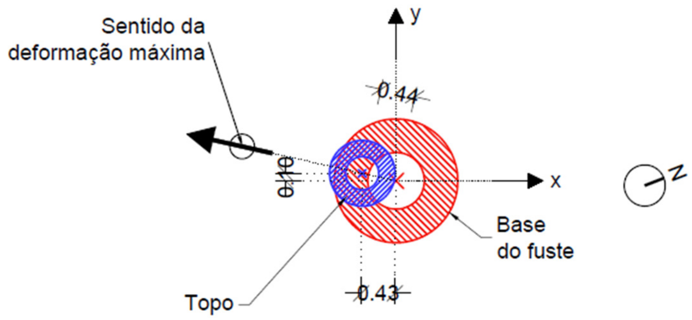


Figure 2.2. Plan view representation of the chimney's maximum deformation and the considered axes.

3 DYNAMIC IDENTIFICATION TESTS

3.1 Tests Description

The dynamic identification tests aimed to obtain the dynamic characteristics of the chimney, specifically its natural vibration frequencies and modal deformations. The results were compared using accelerometers with different characteristics and two distinct techniques: ambient vibration tests (output-only modal identification) and forced vibration tests (input-output modal identification).

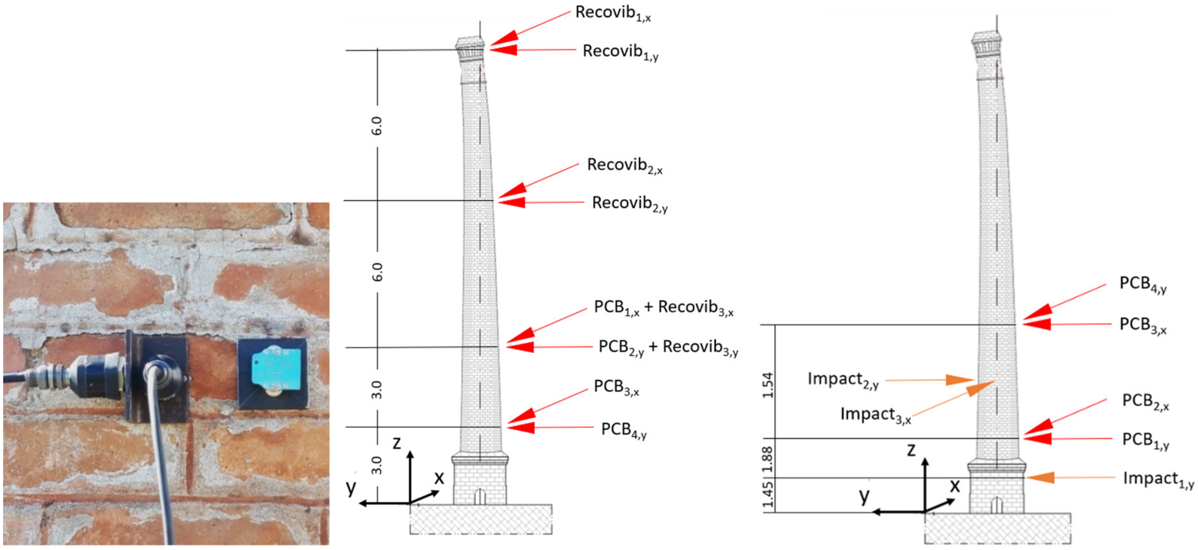
3.2 Ambient Vibration Tests

For the ambient vibration tests, 4 traditional high-sensitivity uniaxial piezoelectric accelerometers (PCB 393B12) with a sensitivity of 10,000 mV/g were used, along with 3 triaxial MEMS accelerometers featuring built-in dataloggers, and automatic wireless synchronization between sensors (Recovib Tiny). The use of Recovib sensors offers advantages such as USB configuration, a single sensor measuring in all three directions, cable-free data transmission, automatic synchronization, and the ability to attach them to metallic elements (e.g., metal plate bolted to the chimney) using magnets. However, they exhibit lower sensitivity (30 µg/√Hz) compared to PCB 393B12 sensors (1.30 µg/√Hz), which can be a limitation for low-amplitude vibrations. Figure 3.1 a) displays the two sensors installed side by side, illustrating the ease of installation and using the Recovib sensor.

To test the applicability of Recovib sensors, they were placed in different positions on the chimney. After the initial tests, it was concluded that when placed in the lower 1/4 of the chimney, they did not allow for the identification of the main vibration frequencies due to their low sensitivity, unlike the PCBs that identified the first 2 frequencies in each direction. Taking advantage of the 3 available Recovib sensors, they were placed at 3 different heights on the chimney, as shown in Figure 3.1 b), to obtain frequencies and modal deformations, comparing the results with those obtained from the PCBs placed at the lower level (simulating a potential difficulty or impossibility of accessing the chimney at a greater height). Regarding data acquisition, in ambient vibration tests, records were made for 20 minutes, with a data acquisition rate of 2048Hz for the PCBs and 1024Hz for the Recovib sensors.

3.3 Forced Vibration Tests

In the search for alternative test solutions that are faster and do not require access to the upper part of the chimney, forced vibration tests were also conducted using a PCB 086D50 impact hammer to assess its applicability in this type of flexible structure. However, the maximum installation height of the accelerometer was limited to the height accessible through a small ladder: 4.8 m = 2.5 m (ladder) + 1.8 m (technician) + 0.5 m (arm amplitude). The impact was considered at the base and at half the height between reading points, as shown in Figure 3.1c).



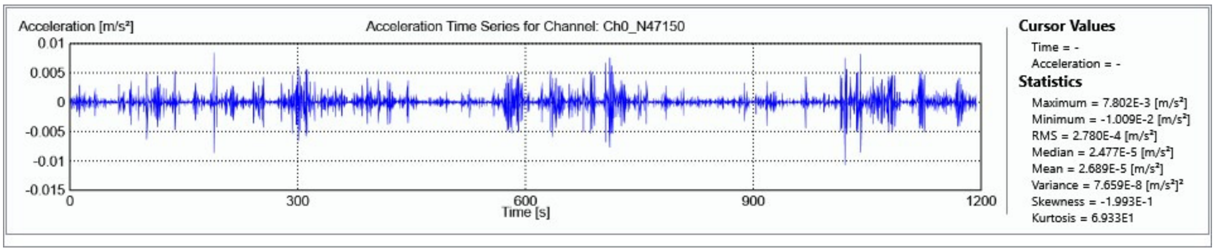
a) b) c)

Figure 3.1. Experimental Setup for Dynamic Identification Tests: a) installation with 2 PCB 393B12 sensors with a metal bracket on the left and a Recovib Tiny sensor magnetically attached to the metal plate on the right; b) setup for ambient vibration tests; c) setup for forced vibration tests.

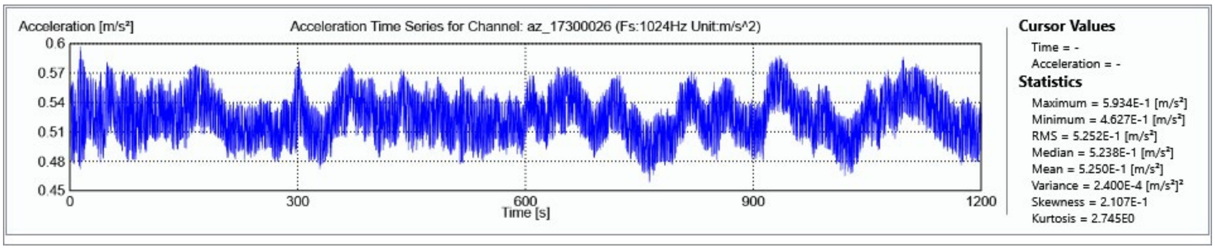
3.4 Comparison of Results

3.4.1. Result Quality

Analysing the results obtained in the time domain (Figure 3.2), one concludes that when installed in the same low position (6.0 m in height), the Recovib sensors do not have sufficient sensitivity to identify acceleration peaks due to ambient excitation, unlike the PCBs. On the other hand, when placed at the top, the quality of the Recovib results significantly improves.



a)



b)

Figure 3.2. Comparison of records obtained in the time domain during ambient vibration tests: a) PCB; b) Recovib Tiny

As for the tests using the impact hammer, it was not possible to obtain good-quality results (Figure 3.3). Possibly, the excitation was not sufficient to identify the chimney's low frequency vibration frequencies due to its mass and stiffness, leading to the conclusion that further studies should be conducted to determine the applicability of this method to this type of structures.

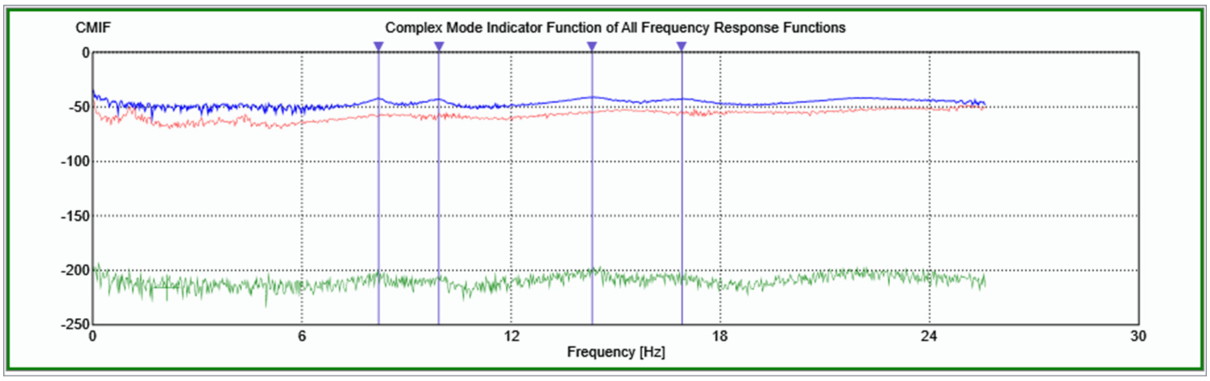


Figure 3.3. Attempt to identify vibration modes through forced excitation

3.4.2 Main Results Obtained

By combining the results from various sensors (PCB + Recovib) and using the common point at a height of 6.0 m to relate the data from all sensors, it was possible to identify the main frequencies and vibration modes up to the 3rd mode, and even higher-order modes (4th and 5th translation mode in X) were found. Figure 3.4 marks the peaks identified in the frequency domain, while Figure 3.5 presents the frequencies and modal configurations.

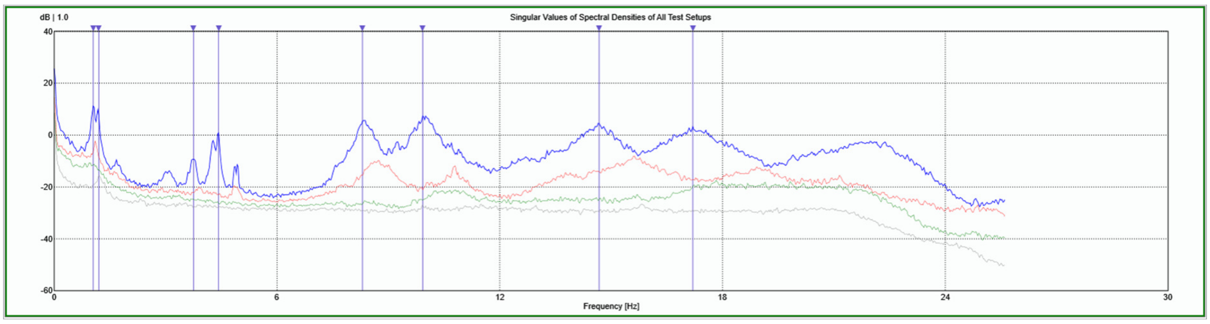
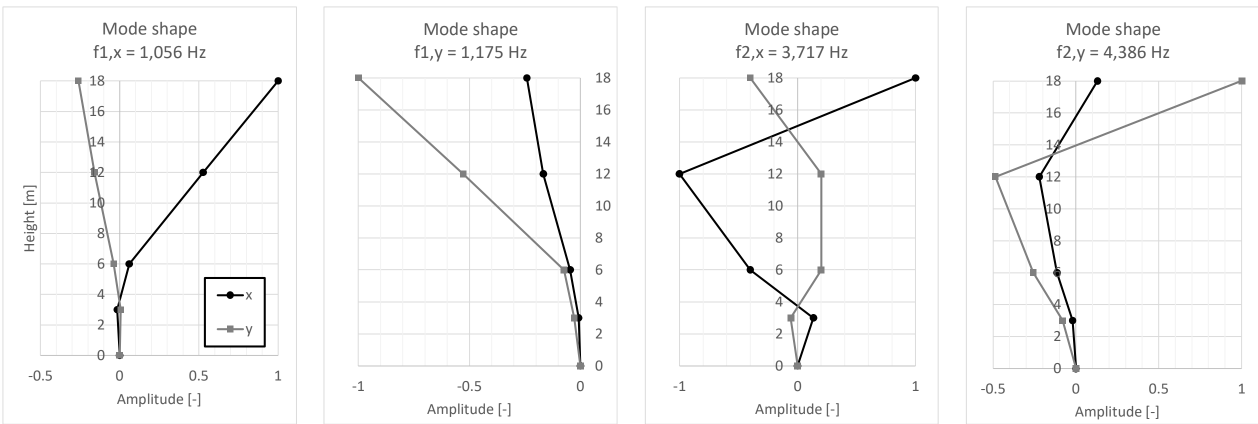


Figure 3.4. Main frequencies identified from combined sensor data (singular value decomposition, SVD)



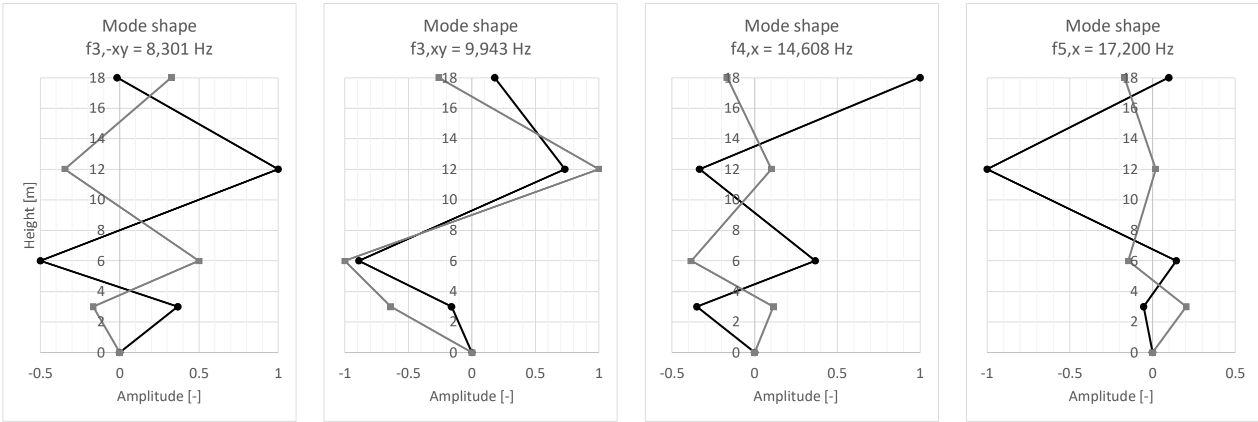


Figure 3.5. Identified frequencies and vibration modes

It is worth noting that after the first 2 translation modes in each direction, a 3rd mode with contributions in both the X and Y directions was identified, which may represent a twisting mode of the chimney.

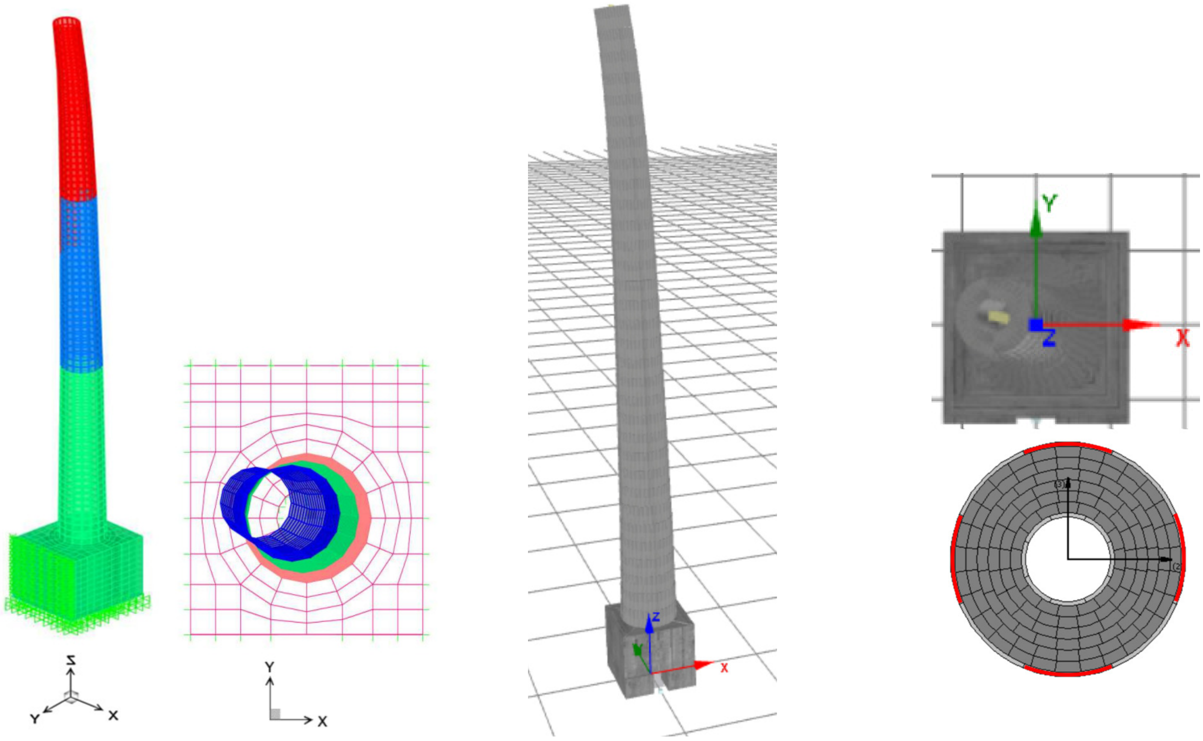
4 SEISMIC BEHAVIOR ANALYSIS

Seismic evaluation was carried out considering seismic zones 1.6 and 2.5, respectively, for seismic action type 1 and type 2, in accordance with the Portuguese National Annex of Eurocode 8-1 (IPQ, 2010). Based on the analysis of response spectra, seismic action type 2 was found to be more critical and was used for the comparisons. However, and despite the existence of a reduction of seismic action depicted in the national annex of Eurocode 8-3 (IPQ, 2017), it was not considered to ensure a higher safety factor. Three main reasons contributed to this decision: i) the importance of the industrial heritage at stake and the need to preserve it; ii) the low seismicity of the site; iii) the high potential impact that damage or chimney collapse could have on its surroundings, considering that it is a structure located in a "populated" area and lacks the capacity to redistribute forces.

4.1 Presentation of Models and Modelling Criteria

Regarding numerical simulations, two different models were used: model A, which uses shell elements with linear behaviour (SAP2000), and model B, which uses beam elements with distributed plasticity and sectional discretization in fibres (inelastic frame force-based elements with 5 integration points), considering material and geometric non-linearity (SeismoStruct). The same cross-section was considered for both models in modelling the chimney's geometry, and the recorded initial deformation was also simulated, as shown in Figure 4.1. It should be noted that for the purpose of comparing with model A, model B was also used for the hypothesis of material and geometric non-linear behaviour, i.e., using elastic beam elements.

Regarding material properties, it was found that identical mechanical properties in different models led to slightly different results in terms of vibration frequencies and modal deformations. Therefore, adapted values were adopted for each model. These values were calibrated considering visual observation, the conservation state, the modal configurations, and the mechanical properties estimated by the design office NCREP in previous works on brick masonry chimneys. Regarding nonlinear properties for model B, the concrete constitutive model by Mander *et al.* (1988) implemented in SeismoStruct was used, considering a tensile strength f_t = 40 kPa, to facilitate solution convergence, compared to the assumption of zero tensile strength. Table 1 provides a summary of the adopted properties.



a) *b)*

Figure 4.1. Numeric models used: a) model A, shell elements (SAP2000), with 3 zones with different types of materials; b) model B, fibre elements (SeismoStruct) with the discretization of the number of fibres in the section and the areas of results control during nonlinear analyses (highlighted in red)

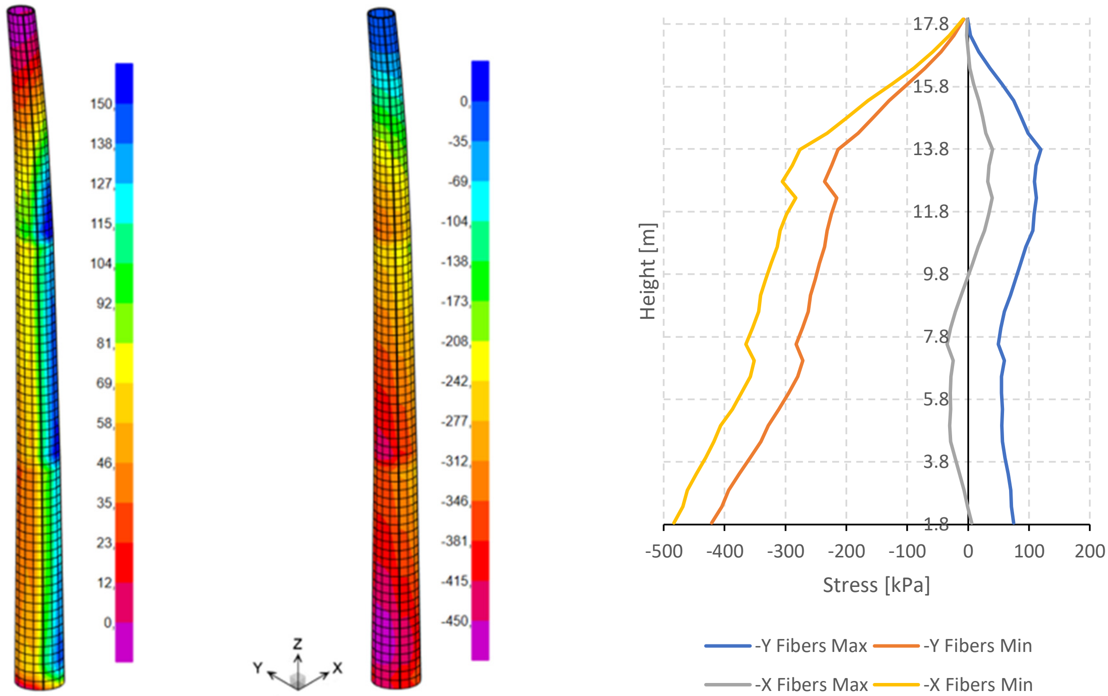
Table 1. Mechanical properties used for masonry

	Model A			Model B			
Material	***E*** **[GPa]**	**γ [kN/m³]**	f_c **[MPa]**	***E*** **[GPa]**	**γ [kN/m³]**	f_c **[MPa]**	f_t **[MPa]**
A (base and lower part)	2,4	17	2,4	2,0	19	2,0	0,04
B (middle part)	1,6	17	1,6	1,0	19	1,0	0,04
C (top)	0,8	17	0,8	0,8	19	0,8	0,04

4.2 Comparison of Results: Linear Analysis

The comparison of results in linear analysis was carried out using Response Spectrum Analysis (RSA), considering a behaviour coefficient of 1.5, which corresponds to the minimum prescribed in EC8. This choice was made due to the fact that the structure lacks the capacity to redistribute forces (it has isostatic boundary conditions). The analyses were conducted considering the 8 combinations of actions provided in EC8-1 (E = ±100%Ex ± 30%EY; E = ±30%Ex ± 100%EY). The combination of results from modal analyses was considered using the square root of the sum of the squares of partial responses (SRSS). Figure 4.2 shows the envelope of results obtained for the 8 combinations, for seismic zone 1.6. As can be seen, the compressive stress values are similar in both models (close to -500 kPa and located at the base), while the maximum stresses assume different values, reaching 230 kPa in model A at 2/3 of the height (at a section transition) and 120 kPa in model B, but 1.2 m above (13.8 m).

As a preliminary analysis, it can be concluded that the order of magnitude of the values is similar, although the distribution in height of tensile stresses is different in the two models. Both reach positive tensile stress values, indicating that non-linear behaviour occurs in the chimney. This is an indicator of the usefulness of performing non-linear analyses, which will be shown in the next section. Finally, it is noted that at 1/3 and 2/3 of the height, a sharp change in the stresses installed in the two models is observed, explained by the change in the thickness of the chimney walls. It is also observed that the Y-direction in model B, despite showing smaller (absolute) compressive stress values, presents higher tensile stress values.



a) *b)*

Figure 4.2. Envelope of vertical stresses in the chimney shaft for seismic zone 2.5: a) model A - maximum (230 kPa) and minimum (-485 kPa); b) model B - maximum (120 kPa) and minimum (-500 kPa), where the four curves refer to the four fibres indicated in red in Figure 4.1b).

4.3 Comparison of Results Obtained: Nonlinear Analysis

For the seismic behaviour analysis of the chimney, considering geometric and material nonlinearity, nonlinear dynamic analyses were conducted using the SeismoStruct software, and eight artificially generated accelerograms compatible with elastic response spectrum type 2 and a duration of 15 s. The Rayleigh damping model with a damping coefficient of 1.5% was used for the first two natural frequencies ($f_{1,x}$ = 1.10 Hz and $f_{2,x}$ = 4.56 Hz), with terms proportional to mass and tangent stiffness, following suggestions from the literature (Correia *et al.*, 2013). All eight analyses converged, which excludes structural instability loss in the chimney, and exhibited negligible residual displacements.

Figure 4.3 presents the results for the most critical load combination (identified through the maximum displacement at the top of the chimney): E = 100%Ex + 30%Ey. To compare the results with the linear models, the evolution of stresses in the chimney throughout the calculation and along its entire height was analysed, for the control sections shown in Figure 4.1b). Maximum compression values of -651 kPa and -291 kPa were identified, occurring in the -X and -Y directions, respectively. The maximum tensile stress values reached +40 kPa, corresponding to the tensile strength considered for the material, which indicates the development of cracks (transient, i.e., followed by closure).

For a more detailed analysis of the impact of nonlinear modelling on the results, Figure 4.3 presents the stress distribution along the height: dark red - minimum values envelope; yellow - maximum values envelope; gray - maximum stress variation in the negative direction in relation to gravity loads (Eq. 1); black - maximum stress variation in the positive direction (Eq. 2); red dashed - tensile strength of the masonry (40 kPa). Finally, the red and yellow dot lines represent, respectively, the minimum and maximum values obtained through the response spectrum analysis for the same combination.

$$delta^{-} = \min(\sigma) - \sigma_{(t=0)} \quad (1)$$

$$delta^{+} = \max(\sigma) - \sigma_{(t=0)} \quad (2)$$

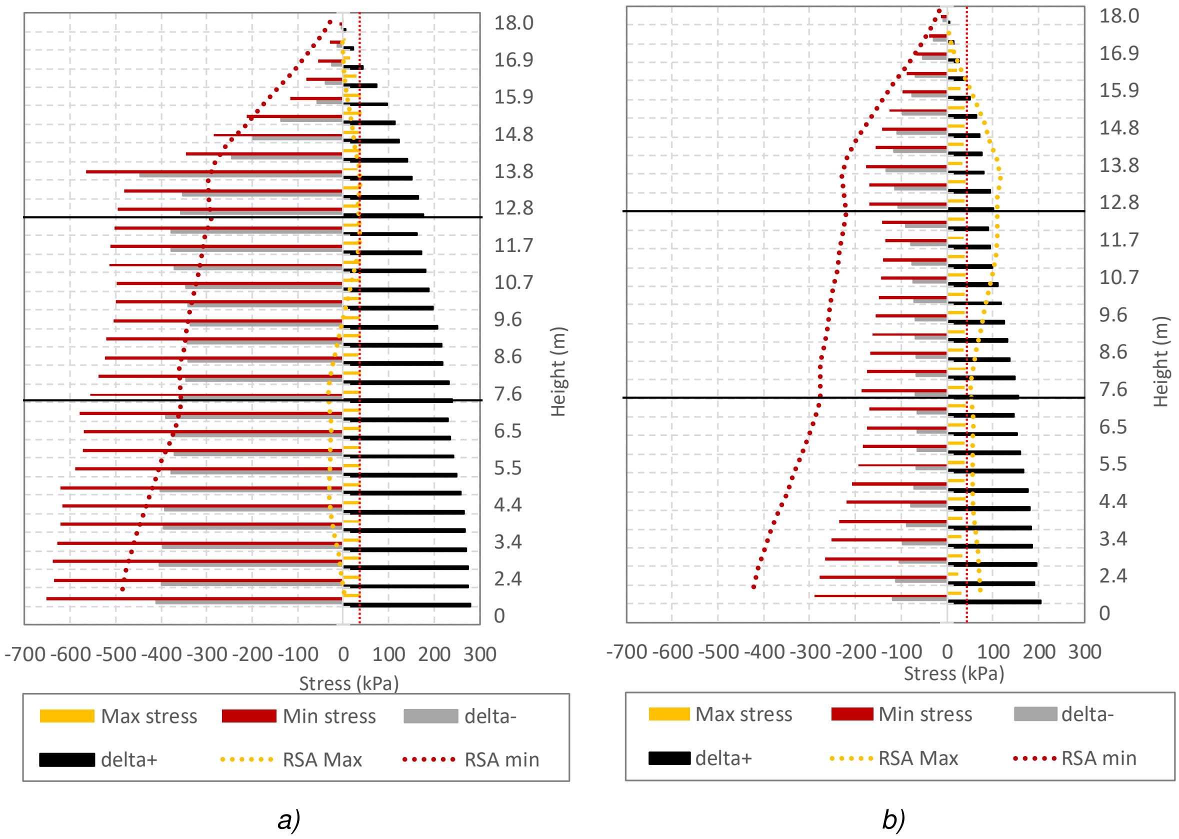


Figure 4.3. Stress distribution in the chimney during dynamic nonlinear analysis 100%X + 30%Y: a) control section -X; b) control section -Y

From the analysis of the Figure 4.3, the following conclusions can be drawn: 1) the masonry's tensile strength is reached in practically all sections, with greater significance in the X direction; 2) the results obtained in both directions are quite different: while in the Y direction, the response spectrum analysis presents conservative results, in the X direction, compressive stresses significantly exceeded the results obtained through the response spectrum analysis; 3) the results obtained by nonlinear dynamic analysis again identify the critical height (h = 13.8 m) in the area located 1.0 m above the thickness transition zone. This identification is visible in the stress variation, coinciding with the largest absolute value of Δ^{-} in both directions.

Finally, in the X direction, the increase of the stresses in the masonry is very high (reaching -450 kPa at 13.8 m and -400 kPa at the base) compared to the stresses installed due to self-weight (-238 kPa). This significant compressive demand on the masonry is justified by the masonry's lack of tensile strength capacity, which is compensated by increasing the compressive stresses. Therefore, to carry out a structural safety assessment of a masonry chimney, even in a low seismicity site, it is necessary to use nonlinear analysis methodologies.

5 FINAL CONSIDERATIONS

This study aimed, on the one hand, to analyse the possibility of applying different methods to identify the frequencies and vibration modes of industrial chimneys, using sensors and testing methodologies alternative to those commonly used in ambient vibration tests (sensors with high sensitivity and chimney height access). On the other hand, it aimed to evaluate the use of alternative tools to linear finite element shell modelling, by introducing geometric and material nonlinearity in the analysis through fibre element modelling to reduce computational time.

The results show that the use of ambient vibration tests for dynamic identification remains the correct option for this type of structure, when compared to forced vibration tests using impact hammers, at least under the conditions described for this research. Also, the use of highly sensitive accelerometers (such as PCB) proves to be the most suitable choice. However, if there is access to the top of the chimney, the use of more recent sensors (such as Recovib Tiny) allows for faster work execution without neglecting the final quality of the obtained results (at least concerning the lower frequencies). When both types of sensors are combined, the quality of the results is very high.

Regarding fibre element modelling, compared to finite element shell models, the results obtained from linear response spectrum analysis are relatively close. However, the results also emphasized the importance of using nonlinear dynamic analysis in structural safety verifications.

6 Acknowledgments

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