5.3.4 NUMERICAL MODEL

The numerical analysis was carried out using commercial software and a linear elastic analysis intended to obtain the forces acting on the adopted façade retention system, taking into consideration the mechanical properties of the materials, different boundary conditions and different states of conservation of the building. The geometry of the building has been defined through the analysis of the original architecture project and from the inspection campaigns carried out in situ. The structure was modelled using the following elements: (1) four node shell elements for masonry panels and inner tabique walls and (2) bar elements for wooden beams and wooden trusses of the roof. The walls were modelled with different thicknesses, taking into consideration walls with thicknesses of 25, 50, 65 and 70 cm. Adjacent to window openings, thick panels made of clay bricks (with a thickness of 19 cm) were used with the objective of assessing the influence that these elements might have on the overall behaviour. Floor and roof structures were modelled with articulated bar elements in the connection to the walls as well as with decorative elements. Pinned supports were considered for the foundations.

According to European norms as well as Portuguese National Annexes (EN-1990 2002, EN-1991-1-1 2002, EN-1991-1-4 2005, EN-1998-1 2004, EN-1998-1-3 2005) and Portuguese Standard (RSA 1983), both static and dynamic actions were considered: (1) permanent actions; (2) variable actions; and (3) seismic actions. The permanent actions considered were the self-weight and other permanent weights such as coatings in kitchens and bathrooms.

![Diagram](image_url)
As variable actions, the wind and imposed loads for floors and roofs, according to the category of use, were considered. In accordance with Eurocode 8-part 1 (EN-1998-1 2004), the ground type considered was Type D.

According to the provisions of Eurocode 8-part 3 (EN-1998-1-3 2005), a behaviour coefficient equal to 1 was considered. This is a common situation in modelling masonry buildings with low ductility and low energy dissipation capacity (Vicente 2008).

For the mechanical characteristics of structural elements, the authors adopted values from the experimental laboratory, from field tests performed by other research reference studies, and from the literature (Vicente 2008, Farinha and Reis 1993, Rodrigues 2015): for masonry walls, a self-weight equal to 19.60 kN/m^3 and a Young elastic modulus equal to 1.75 GPa; for wooden elements (beams, boards, trusses and tabique walls), the corresponding values for class C18 (EN-338 2008).

According to the pathologies identified in Section 5.3.2, the authors have considered, in a numerical model, joint elements of negligible width and stiffness to simulate the cracks and the weaknesses detected in the corner connections between masonry walls. These elements also enable the mechanisms of collapse of the façades, specifically those associated with the out-of-plan deformations, to be simulated (see Section 5.2.2). These joint elements are shell elements of four nodes with a width of 5 cm each. The same elements were used in the connections between the masonry walls and the wooden beams of the floor.

Six different numerical models (Cases 1–6) were studied to assess the out-of-plane displacement for each of them and also to assess the load transfer in the two cases in which façade retention structures were added. In Figure 5.5a through 5.5f, the six different cases are presented.

All of the analysis was performed considering the XX direction and the combinations with the seismic action. For the façade retention structure, wooden elements of 10 and 24 cm in diameter, of class C18 (EN-338 2008), were adopted, considering the fact that these materials are easy to obtain and easy to work with and have adequate strength.

In Case 1 (Figure 5.5a), the original structure of the building was considered, including joint elements in the corners and also a decrease in the stiffness of these elements in the connections between wooden beams and masonry walls, to simulate the material deterioration and the weak tension resistance.

Concerning Case 2, only the façade walls were studied, without any horizontal restraint, simulating the constructive phases of a current method of rehabilitation that involves dismantling all the internal structural elements, leaving only the envelope walls, which are then replaced by reinforced concrete or steel structures, as illustrated in Figure 5.5b. This case also reproduces many real cases where the retaining structure is neglected due to cost management considerations.

Figure 5.5c shows Case 3, which considers the attachment of an isolated retaining structure as an autonomous structure with the function of transferring the loads to the ground. For the façade retention system, a 3D structure with vertical, horizontal and diagonal elements 10 and 24 cm in length was adopted. An internal portal frame had to be added to limit the lateral deflection at the top of the façade wall, as recommended by CIRIA 519 (Bussell et al. 2003), which limits this deflection to height/750. This consideration is aimed at increasing the stiffness of the retaining structure, avoiding displacements at the top of the wall, which could lead to collapse.
The model presented in Case 4 is directly related to Case 2, but including the façade retention structure adopted in Case 3 (Figure 5.5d).

Based on Case 1, in Case 5, a faced retention structure with a structural scheme (illustrated in Figure 5.5e) was added to the model. To simulate the real supporting conditions of the retention structure, part of the surrounding buildings (across the street) were modelled. The retention structure is directly supported on the façades of.
the building at the floor levels. Such a structure requires an extensive survey on those buildings and their structural condition to avoid local or global damage. This type of structural solution also requires the permission of the owners of these buildings.

As presented in Case 5, the model shown in Case 6 compares the displacements of the façade wall when the support conditions of the retention structures are changed from an autonomous retention structure to one supported by surrounding buildings (Figure 5.5f).

5.4 ANALYSES AND DISCUSSION OF RESULTS

In Figure 5.6a, the first three modes of vibration, obtained from the numerical model used in Case 1, are presented. As can be observed, the frequency value obtained for the first mode (Figure 5.6a) is directly comparable to the values obtained from the environmental vibration tests (see Table 5.3), with an average relative error of 8.20%. The values obtained for the masonry walls of the building are within a range typically observed in structures of this nature (Vicente 2008).

In Figure 5.6b, the first shape modes associated with the out-of-plan deformation of the façades are presented.

The numerical model was calibrated based on the results obtained in environmental vibration testing as well as in the aforementioned pathologies.

The analysis of the previously mentioned maps reinforces the importance of the orthogonal walls in the structural behaviour of the masonry structures, thus demonstrating the need for shoring up these elements with façade retention structures, which control the out-of-plan deformation.

In Case 1, the importance of an efficient connection between the orthogonal walls and the façade to the local behaviour of the façade is clear; a large amplitude of

FIGURE 5.6 Results from numerical model.
TABLE 5.3
Summary of Results Obtained for Different Cases

Case 1 and Case 2

Max: 39.98 mm; Min: −0.03 mm
Max: 46.76 mm; Min: 0.00 mm

Case 3

Max: 8.41 mm; Min: −0.02 mm

Case 4

Max: 9.28 mm; Min: −0.03 mm

Case 5

Max: 8.89 mm; Min: −0.04 mm

Case 6

Axial forces on the retaining structure

Axial forces on the retaining structure

Axial forces on the retaining structure

Axial forces on the retaining structure

Axial forces on the retaining structure
displacements is shown when these connections are not efficient. In the same way, displacements of internal walls parallel to the façade analysed have been observed. These displacements have been revealed in the form of effects on the façade through the diaphragm effect of the structure of the floor.

The results of Case 2 show the instability of the façade walls in an earthquake event when the inner structural elements are removed. Compared with Case 1, there is an increase in the amplitude of displacements, which may be explained by the resistance, however limited, of the elements of the joint.

The addition of the inner portal frame (Case 3) shows the importance that orthogonal walls can have for the behaviour of the structural scheme as a whole. In this case, wooden elements were anchored in the orthogonal walls, and displacements crossing these were transduced into small displacements at the top of the façade.

As expected, the results of Case 4 show a larger amplitude of displacements when compared with the previous case. This is supported by the fact that an autonomous retaining structure requires more inertia within the vertical elements of the portal frame than one that is attached to other elements, either surrounding buildings or inner walls, as in Case 3.

The results obtained in Case 5 show the influence of the retaining structure on the amplitude of displacements associated with the scenario suggested in Case 1, manifesting itself in a reduction of 22.4% in relation to the displacement of the initial case. It is also clear that the energy produced in the out-of-plane movement of the façade under study is not entirely dissipated within the elements that constitute the retention structure, thereby affecting its neighbouring buildings.

In Case 6, as observed in Case 5, the addition of the retaining structure controls the out-of-plane displacements, manifesting itself in forces transferred to the buildings that support it. In that case, the out-of-plane displacements have a reduction of 19.85%.

5.5 FINAL REMARKS

The previous sections represent some guidelines for the design of façade retention structures usually used in the rehabilitation and conservation of buildings and in prevention or emergency scenarios, showing the importance of numerical analysis, mainly modal analysis. They also show state-of-practice tools and proposals to solve current engineering problems related to the study of old masonry buildings.

The importance of a rigorous survey of the state of conservation of the structural elements of the building was emphasised, namely, the cracking of load-bearing walls and the degree of conservation of the wooden beams supporting the masonry. This allowed a more realistic calculation model that included the weaknesses of the façades and other structural elements, and hence, the design of a more optimised containment structure calculated for more realistic actions and conditions.

It was also verified that, for buildings with characteristics and sizes similar to the building studied, the state of conservation tends to define the conditioning actions for the design of the containment structures. In buildings in a reasonable state of conservation, such as the building studied, the seismic action is the conditioning. In buildings in an advanced state of degradation, the structural elements have a lower
resistance capacity, and actions of lower intensity, such as gravitational and wind actions, can be determinant.

It was concluded that for the façade retention structure, wooden elements can be used, in the form of commercial sections and/or tree trunks, a natural and renewable material that is ecologically better than steel, which is most commonly used.

The current study also shows that, when allowed, the consideration of surrounding buildings is clearly a reliable option to support a façade retention structure. This façade support system may require an extensive structural survey of both the building supported and the building used as support, justified by the eventual damage that may occur in the supporting building, which may cause a chain reaction in the façade retention, leading to its collapse.

When an isolated retention structure is considered, stiffer structural elements may be needed. In extreme cases, this can lead to the use of mechanical elevation cranes and consequently, higher costs.

The corridor passage at the lowest level of the retaining structure is also a feature that has a direct influence on the façade displacements and stresses in the retaining structure. This problem also requires the adoption of larger wooden beam sections and stiffer connections between bars.

Further studies should be carried out to obtain optimal bar connections, with the purpose of achieving more secure and sustainable bars and connectors and an easier assemblage system, using preferably nonindustrialised solutions.

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6 3D Structural Analysis to Support the Rehabilitation of Old Masonry Buildings

Nelson Silva, João Veludo and Pedro Santos

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6.1 INTRODUCTION

The rehabilitation of old buildings is fundamental to ensure the preservation of existing cities and to promote their economic development, creating new opportunities for residential housing and commercial activities. This activity is critical when the existing construction was built using mainly materials such as masonry or timber instead of concrete and steel.

Old masonry buildings were built with structural stone exterior walls and wooden interiors with mortar walls, wooden floors and roofs covered with ceramic tiles. Steel members, usually disposed at floor levels, were implemented to increase the stiffness of the buildings. The foundations were frequently built using stone masonry footings with the same width as that of the base wall, or even wider. Wooden piles were frequently also adopted in areas with weak soils. In Portugal, a traditional wooden wall, commonly known as *tabique*, was developed for the interior walls.
The *tabique* walls, which were made up of vertical wooden boards and laths (horizontal wooden slats), were connected to the boards with steel nails. After this, the structure was covered by clay or mortar with very fine aggregates. *Tabiques* were intended to be used as structural and nonstructural members.

One of the main issues in the rehabilitation of old masonry buildings is the structural rehabilitation and the verification of the structural safety. The structural rehabilitation is driven by various circumstances, such as visible failure in structural members, change in use, or increased use, and even for seismic assessment and retrofit (Santos 2003).

In structural rehabilitation, certain aspects should be taken into account: namely, the architecture, the state of conservation, the existent structure and the historical values of the building.

Nowadays, the structural analysis of an old building is based on the modelling of its structural behaviour, and it is necessary to take into account the actual behaviour of the building when faced with various actions that are imposed on it, which should include static and seismic actions.

### 6.2 STRUCTURAL ANALYSIS AND MODELLING

The modelling of old structures presents a different challenge from the current (existing or new) structures of concrete or steel and becomes more complex due to the heterogeneous materials: in particular, the masonry and wooden walls that are composed of more than one material, the nonlinearity of the materials, and the anisotropy, which means different structural behaviour at different orientations of the loads (Roca et al. 2010, Asteris et al. 2015, Lourenço 2002).

Nowadays, there are several methods available for structural analysis, resulting in different structural approaches. Each approach is carefully selected and then adopted, all depending on the specific characteristics of each case study.

The adopted numerical models should always describe the original structure. When properly calibrated, these models will permit the identification of the theoretical damage produced by different types of action and the comparison of this with the actual damage observed in the structure (ICOMOS 2003). Therefore, a properly calibrated numerical model is a fundamental tool to ensure an adequate intervention. A numerical model of the existing structure, probably damaged, and of the strengthened structure will help the structural engineer to assess the current safety levels and to achieve the most economical intervention.

According to these statements, two different approaches can be used for the structural analysis of masonry structures: micro-modelling and macro-modelling. Macro-modelling can be used for the structural analysis of masonry structures built with thick walls, which will result in uniform stresses in those elements. This type of modelling is also faster and requires less memory and less complex finite element meshes, thereby presenting a lower cost. On the other hand, micro-modelling is used to understand the behaviour of the masonry structures, considering the heterogeneity and the nonlinear behaviour of the materials, and, when related to the seismic analysis, the various forms of mechanism failure (Lourenço et al. 1995).