Methodology for nonlinear three-dimensional analysis of Pombalino buildings

Bruno Filipe de Paiva Candeias
Instituto Superior Técnico, Universidade de Lisboa, Portugal
April 2017

Abstract

This paper describes the development stages of a methodology to evaluate the seismic performance of existing Pombalino buildings. The proposed methodology allows the simulation of non-linear behaviour of the timber-framed and masonry walls by using a three-dimensional pushover analysis considering the in-plane shear strength of the timber floors. The method was applied to an old building located in Lisbon, which includes a three-dimensional timber structure enclosed in masonry walls. The results obtained highlight the advantages of the in-plane strengthening and numerical modelling of timber floors, resulting in an improvement of the in-plane resistance of the structural walls. The methodology limitations are also discussed.
Keywords: Pombalino buildings, nonlinear analysis, seismic reinforcement,“frontal” walls, masonry walls, timber floors.

1. Introduction

The seismic vulnerability assessment of the buildings constructed in Lisbon after the 1755 earthquake (later called Pombalino buildings) is a priority, due to their significant historical value and their location in an earthquake-prone area. A proper evaluation of the seismic risk of these buildings is a necessary step in order to identify the most critical areas and assess the priorities for the retrofit work. To achieve this, it is necessary to define analysis methods, capable of assessing the safety of the structures in a simple and effective way.

These buildings are characterised by the existence of an interior timber frame structure filled with masonry (called *Gaiola*), surrounded by thick masonry walls. As a consequence, experimental analysis has shown that this kind of structures displays a non-linear behaviour, even for small deformations, forcing the use of non-linear analyses. While the regulations present the pushover analysis as a non-linear analysis method, it has some limitations regarding the analysis of Pombalino buildings, such as being limited to the bi-dimensional in-plane analysis of walls and not accounting for the floor rigidity in the model.

Therefore, the aim of this paper is to explore the possibilities offered by SAP2000, a software with a user-friendly interface, widely used by practicing engineers, in order to develop a non-linear three-dimensional method for the seismic analysis of Pombalino buildings. Each section of the paper shows the application of each described methodology by means of a case study, allowing to observe particular aspects of the modelling and to draw some conclusions. To the effect, a late Pombalino building, built in the late 1800s and located in Belém, Lisbon, was chosen. Even though it is not identical to the buildings located in the Lisbon downtown (Baixa), it has the main characteristics of buildings from this period - three floors, interior walls with timber-framed masonry, and thick outside masonry walls. It was also the subject of studies in the past, consisting of an ambient modal identification, performed by Cismasiu *et al.* [4] and a seismic analysis, consisting of modal response spectrum analysis, pushover analysis of the front masonry wall, and the analysis of local mechanisms, carried out by Giordano [6].

Since this study considers that the resisting mechanism is governed by the in-plane response of the walls, it is assumed that local collapse modes are not considered in the model and should be evaluated separately.

2. Available methods for seismic assessment of existing buildings

In order to evaluate the global seismic behaviour of buildings, Eurocode 8 [1] includes four methods of analysis:

- Lateral force method
- Modal response spectrum
- Non-linear static (pushover)
- Non-linear time history (dynamic)

The first two methods are generally used for the design of new structures in steel or reinforced concrete, adopting a behaviour factor according to the structure type, and performing a safety verification according to limit stresses or deformations. However, due to the inelastic behaviour and anisotropy of masonry and timber structures, this type of analysis does not accurately represent the ultimate limit state of the structure, thus requiring the use of non-linear analysis methods, which are able to trace the complete response of a structure, from the elastic range, through yielding, up to complete failure.

Due to its modelling and interpretation complexity, the non-linear dynamic analysis is not suitable for small scale buildings. Non-linear static analysis provides a good balance, by allowing the consideration of non-linear behaviour while reducing the analysis time.

Eurocode 8 presents the pushover analysis methodology, consisting in the application of a monotonic displacement-controlled lateral load pattern until an ultimate condition is reached, allowing to plot the base reaction/displacement diagram of the structure, also called the capacity curve. Method N2, described in Eurocode 8 Annex B, allows the calculation of the target displacement from the displacement demand of an equivalent SDOF system. Since this method was developed for the analysis of 2D structures, it is not suitable for the global three-dimensional analysis of buildings and does not consider the role of the floors on the global behaviour. The 3Muri software [11] allows for three-dimensional pushover analysis, but does not provide general guidelines and it is not adaptable to more generally used programs, such as SAP2000 [2].

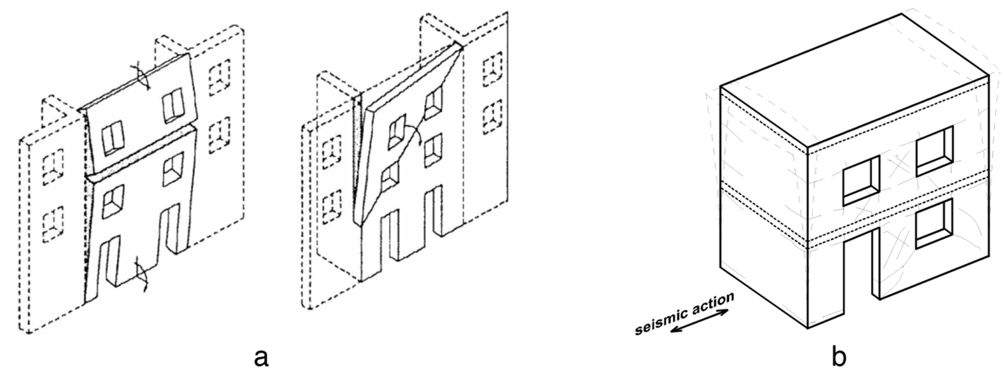


Figure 1: Examples of possible collapse mechanisms: a) local; b) global [9]

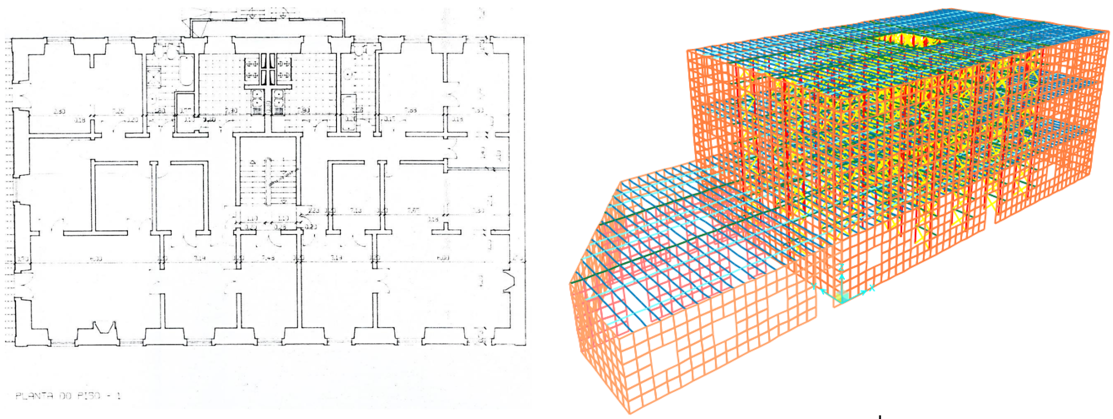
According to previous investigations and the observation of past earthquakes, masonry structures are vulnerable to out-of-plane mechanisms, which are not directly considered in a global analysis. As a consequence, global analyses are only reliable if the safety verification of the building concerning local mechanisms is carried out (Figure 1). This is not the purpose of this work and could be analysed following the Italian *Norme Tecniche per le Costruzioni* [10], which contains a method for the analysis of local mechanisms of existing masonry buildings, by using a cinematic approach.

3. Three-dimensional elastic analysis

Even though the main goal of this research is to study the non-linear behaviour of Pombalino buildings, the first step of the analysis consisted in the elaboration of a three-dimensional elastic model. This model was used to obtain a general information about the structure's behaviour, as well as to calibrate the model, by comparing the results of the modal analysis to the results of the experimental ambient modal identification previously done by Cismasiu [4].

Figure 2.a shows the floor plan of the building, followed by the final structure model in SAP2000 (Figure 2.b). The exterior masonry walls were modelled as thick shell elements, while the timber-framed walls and the floors were modelled as frame elements. To prevent local modes of vibration, masses were considered as concentrated and applied on the structural walls. Material self weight and elasticity properties were taken from the available bibliography regarding this kind of buildings, since it was not possible to perform experimental analysis on the structural elements.

The results of the ambient modal identification campaign showed a first mode of vibration in the Y direction (perpendicular to the façade), with a frequency equal to 6,44 Hz. Since the vibration amplitude was very low during the experimental campaign, the obtained frequency can be compared to the first mode of vibration of the model considering rigid floors. From the results of the numerical analysis, the obtained first mode frequency was 6,76 Hz in the same direction as the experimental results, allowing to validate the model used in the following sections.



a b

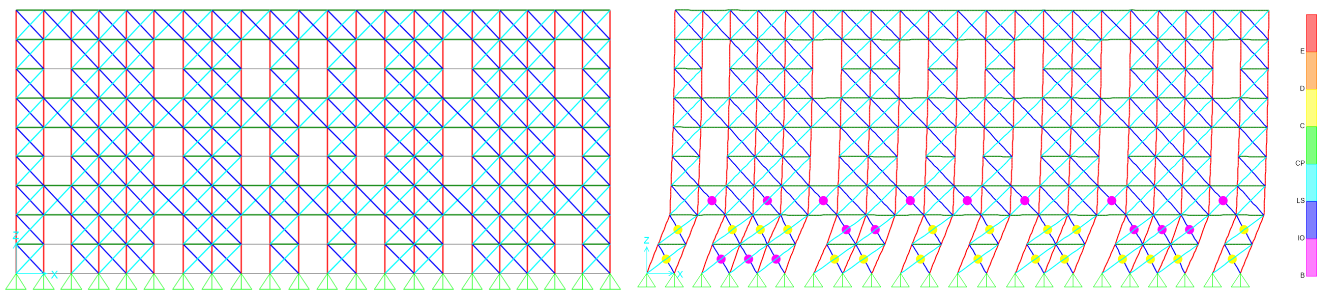
Figure 2: a) Plan view of the first floor of the original building; b) Three-dimensional model

4. Pushover analysis of "frontal" walls

Kouris and Kappos [7] presented a method suitable for the nonlinear static analysis of timber-framed masonry walls ("frontal" walls), considering nonlinear hinges in the struts. The constitutive law for the hinges was derived from a parametric analysis of the main factors that affect the response of timber framed panels subjected to horizontal loading, mostly related to the geometric features of the panel and the timber strength. The model proposed by the authors is implemented through the following steps:

1. Discretisation of the building into individual timber-framed panels;
2. Computation of the equivalent vertical load in each panel;
3. Application of the empirical formulas to define the constitutive law of each panel (horizontal shear/displacement);
4. Correcting the elastic stiffness of the diagonals;
5. Definition of the axial load/deformation plastic hinges on the struts;
6. Performing the pushover analysis of the structure consisting of the braced panels defined in the previous steps.

To illustrate this method, a "frontal" wall parallel to the façade of the case study building (y=4,5 m) was analysed. Figure 3.a shows the wall modelled as a braced structure. As a simplified approach, every strut hinge was modelled with the same mechanical properties, assuming that all panels have the same dimensions and timber resistance. It is possible to observe in Figure 3.b that the collapse mechanism of the wall is the failure of the ground floor, given that all the elements in the wall have the same resistance and the shear stress is larger on the lower floors. On SAP2000, yielded hinges are represented in purple, while collapsed hinges are displayed in yellow.



a b

Figure 3: a) Model of the frontal wall (Y=4,5m); b) Collapse mechanism of the pushover analysis

Figure 5.b displays the obtained capacity curve of the wall, considering the control node in the last floor and a uniform vertical load pattern. A maximum shear stress of almost 300kN is reached, for 11 cm of displacement.

The capacity curve is used by applying the N2 method, in order to calculate the target-displacement of the structure. As illustrated in Figure 4, this process involves the conversion of the structure into a single degree of freedom (SDOF) equivalent structure. The complete expressions for the SDOF conversion and target-displacement calculation can be found in Annex B of Eurocode 8 [1].

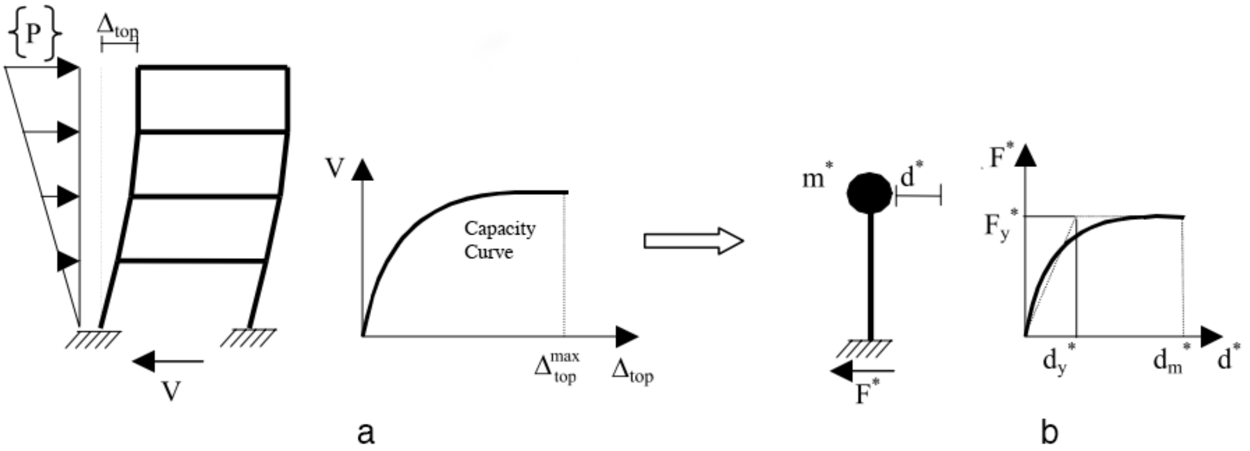


Figure 4: Conversion of the capacity curve of the structure (a) to a SDOF equivalent system (b) (Adapted from [3])

The calculated target-displacement for the “frontal” wall was 5,9 cm. Figure 5.a shows the condition of the hinges when the wall is subjected to the target-displacement. At this point, the majority of the struts of the ground floor and eight of the first floor are on their yield phase. Figure 5.b displays the obtained capacity curve of the wall, considering the control node in the last floor and a uniform vertical load pattern. It shows the target-displacement in the capacity curve, allowing to observe that the displacement is lower than what it is required for the occurrence of the collapse mechanism, therefore validating the safety of the wall with respect to the in-plane response. A maximum shear stress of almost 300kN is reached, for 11 cm of displacement.

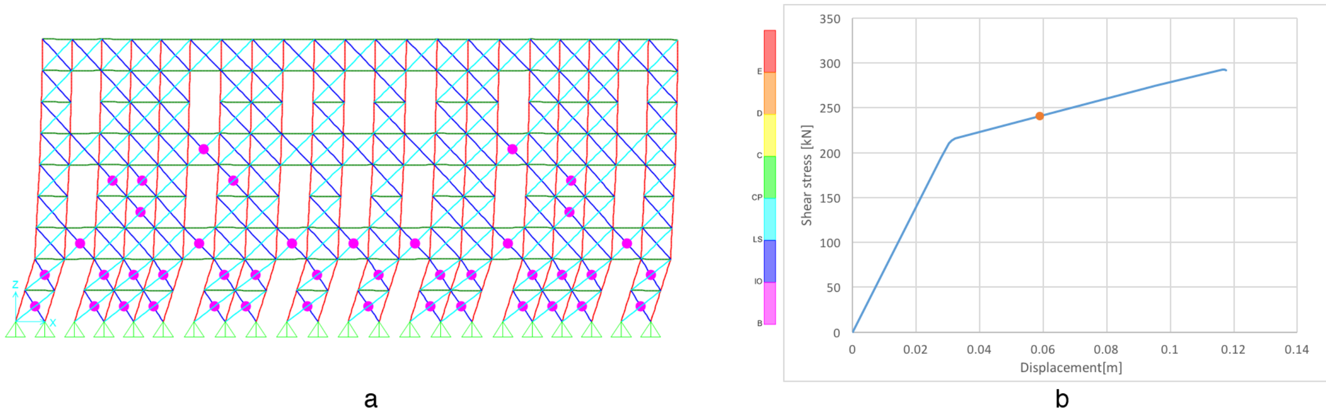


Figure 5: a) Deformed state of the frontal wall at the target-displacement; b) Capacity curve of the frontal wall (y=4,5m) considering a uniform vertical load distribution and the target-displacement point

5. Pushover analysis of masonry walls

Non-linear pushover analysis of the outside masonry walls of Pombalino buildings can be performed using the SAM (Simplified Analysis of Masonry buildings) methodology, developed by Magenes [8] at the University of Pavia, and already applied in the case study building by Giordano [6]. This methodology is based on an equivalent frame model of the structure, where plastic hinges are modelled at the elements’ ends to simulate their non-linear behaviour.

Piers and spandrels are modelled as frame elements, defined by their effective length and connected by rigid links. Regarding the piers, three failure mechanisms are considered: flexural failure, diagonal shear cracking, and shear sliding. As for the spandrel beam elements, only shear failure mechanisms are foreseen. Hinge location on both elements can be seen in Figure 6.

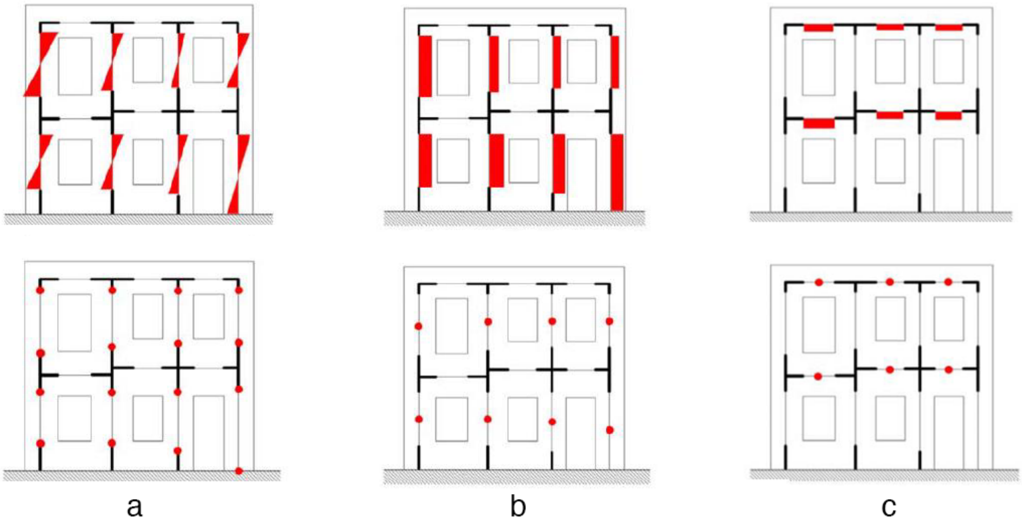


Figure 6: NL hinge location: a) Moment hinges on the piers; b) Shear hinges on the piers; c) Shear hinges on the spandrels (adapted from [6])

The wall model of the main façade (figure 7.a), presented in Figure 7.b is based on the model made by Giordano [6], with some dimension modifications to assure geometrical compatibility with the nonlinear “frontal” wall model previously made. The complete tables with the material and hinge properties can thus be seen in [6].

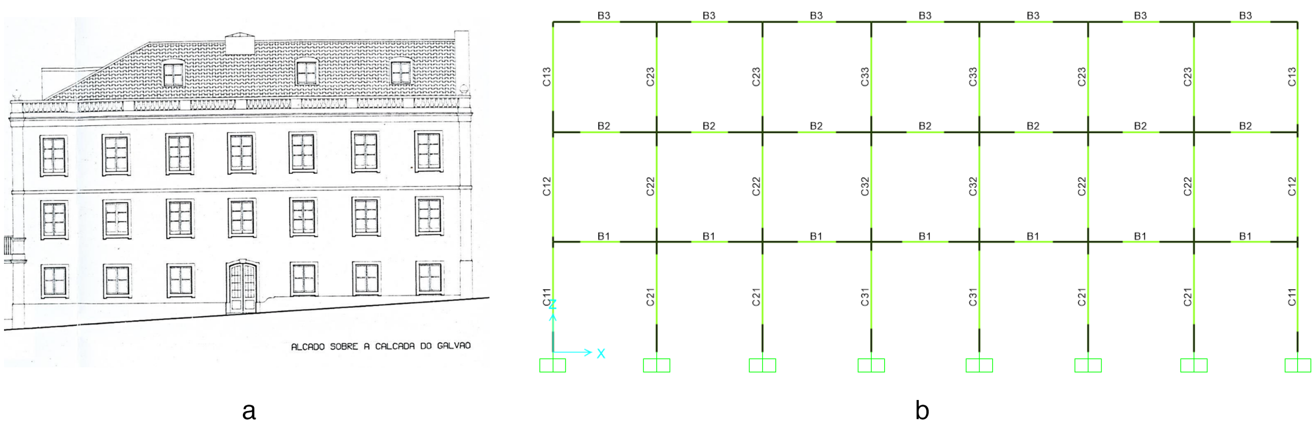


Figure 7: a) Drawing of the main façade of the building (y=0m); b) equivalent frame model

The collapse mechanism for the façade wall occurs by shear failure of the walls at the ground floor level due to the low tensile and shear strength of the masonry. For the calculated target-displacement of 0,2 cm, collapse occurs in the shear hinges at the piers and spandrels of the first two floors (Figure 8.a). However, the target-displacement point is still located in the elastic section of the capacity curve (Figure 8.b), satisfying the requirements imposed by the regulations.

A comparison between the pushover curves of the “frontal” wall (Figure 5.b) and the masonry façade wall (Figure 8.b) reveals their different behaviours: the façade exhibits a high maximum base shear (2200 kN) compared to the “frontal” wall (292 kN). It also has a higher rigidity, collapsing at a top displacement equal to 1,5 cm. On the other hand, the “frontal” wall has a higher ductility, with the collapse mechanism occurring for a 14,5 cm top displacement. These differences can lead to significant relative displacements between the walls when resisting in parallel to a horizontal action. On a three-dimensional model, the floors play an essential role in assuring the compatibility of these displacements and it is necessary to define how to model them.

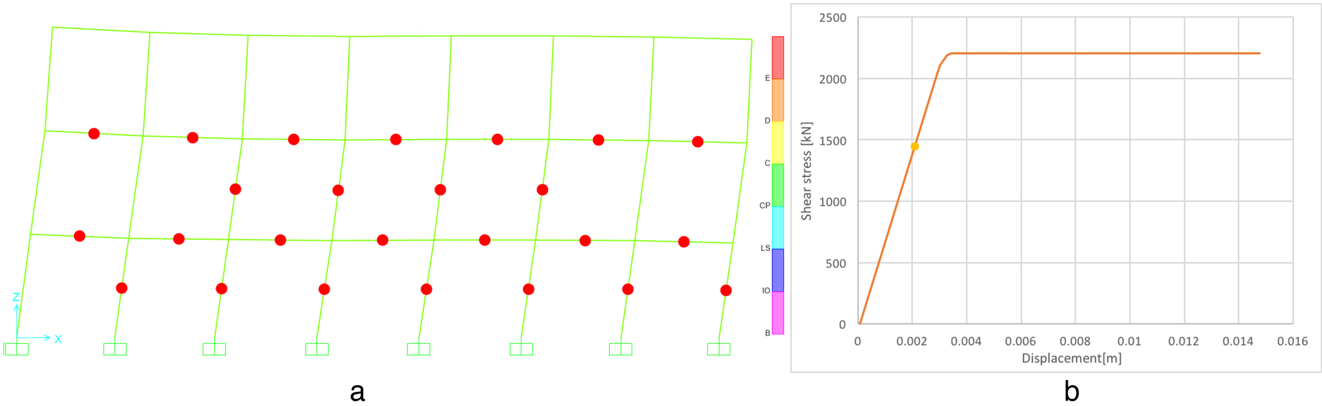


Figure 8: a) Deformed state of the façade wall at the target-displacement; b) Capacity curve of the façade wall (y=0m) considering a uniform vertical load distribution and target-displacement point

6. In-plane strength of timber floors

Valluzzi *et al.* [12] performed tests in real scale floor specimens to assess their in-plane shear stiffness. For more information about the experimental procedure and the configuration of the analysed floor specimens, consulting the original article is recommended [12]. The results of those tests were used in this paper to define the rigidity of the equivalent membrane element to model the floors in the nonlinear three-dimensional analysis performed in Section 7. For that purpose, three floor specimens were considered: FMSB, an unstrengthened timber floor composed by simple supported timber beams, and a transversal boarding; FMSD, a strengthened configuration with diagonal punched metal strips; and FM+45SP(A), with an additional diagonal boarding positioned at 45 degrees with respect to the original boarding direction.

The equivalent thickness t_{eq} of the membrane element for each floor specimen (Table 1) was calculated using the shear modulus equation (1). For this calculation, the value of the shear modulus was fixed ($G = 5000\ MPa$), the secant stiffness (F_t/Δ) was taken from the experimental results, and the dimensions L and B correspond to the tested floor configuration (2,12 x 2,12 m). The difference between the effective and equivalent thickness of the three configurations is considerable, due to the possibility of sliding between the timber elements of the floor.

$$t_{eq} = \frac{F_t}{\Delta} \frac{L}{B \cdot G} \tag{1}$$

Table 1: Equivalent thickness for the three floor specimens considered

Floor specimen	**Secant stiffness** F_t/Δ [kN/mm]	**Equivalent thickness** t_{eq}[mm]
FMSB	0,037	0,0075
FMSD	0,236	0,047
FM + 45SP	0,566	0,113

7. Three-dimensional pushover analysis

The first step in the nonlinear three-dimensional analysis is the modelling of the structure in the finite element program. In order to simplify the demonstration of the concepts introduced on this chapter, only two walls of the building were considered: the “frontal” wall, modelled in Section 4 and the façade masonry wall, analysed in Section 5. Both walls were considered to resist in parallel on a three-dimensional model, withe the membrane elements defined in Section 6 assuring the connection between the them. The final model used for the analysis considering different flexibility conditions for the floor diaphragms, can be seen in Figure 9.

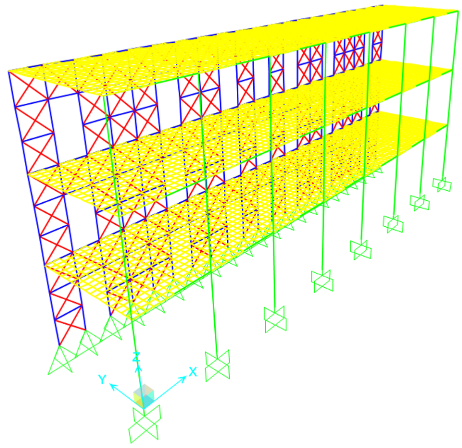


Figure 9: Three-dimensional modal of the frontal and façade walls, and membrane floors

7.1. Rigid diaphragm

To set a benchmark for possible reinforcement solutions, the model was analysed considering rigid floors (using the Assign > Joint Constraints > Diaphragm command on SAP2000). The resulting capacity curve is shown in Figure 10, along with the capacity curves of both walls from the individual 2D analysis. It is possible to observe that the obtained capacity curve consists on the sum of the individual capacity curves, leading to conclude that considering rigid floors allows for the in-plane strength and deformation capacity of the walls to be fully exploited.

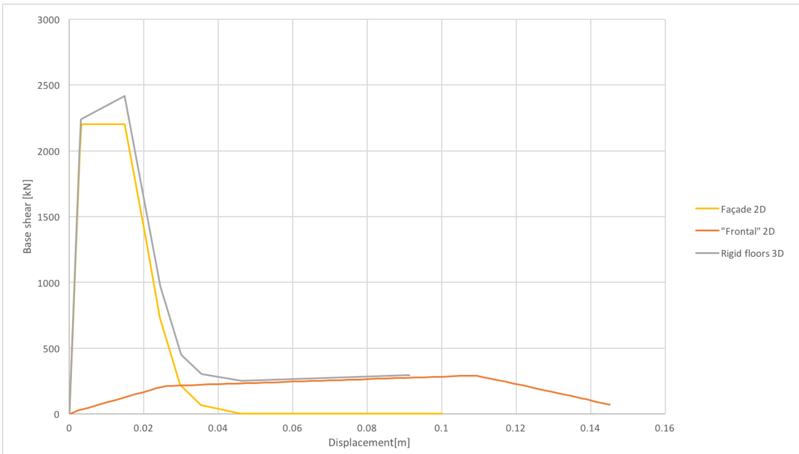


Figure 10: Capacity curve of the three-dimensional model considering rigid floors

The resulting target-displacement for both walls was 0,23 cm (since the consideration of rigid floors implies that both walls have the same displacement at each floor), a minor increase over the 0,2 cm obtained previously for the masonry wall, but a considerable reduction from the 5,9 cm of the “frontal” wall. This is due to the difference in both rigidity and resistance of the walls - the horizontal load is transferred to the masonry wall due to its higher rigidity, leading to a reduction of the target-displacement of the “frontal” wall.

7.2. Original flexible floors

Given the low rigidity of the original timber floors, it was necessary to evaluate whether, for this situation, the proposed methodology presented advantages compared to the isolated wall analysis for this situation. The most flexible floor from the chosen specimens (FMSB) was considered, with an equivalent thickness of 0,0075 mm. Since the capacity curve of the “frontal” wall obtained from the three-dimensional model is nearly identical to the one obtained from the isolated analysis (Figure 11), for this floor specimen there is no real advantage in considering the influence of the floors in the three-dimensional model.

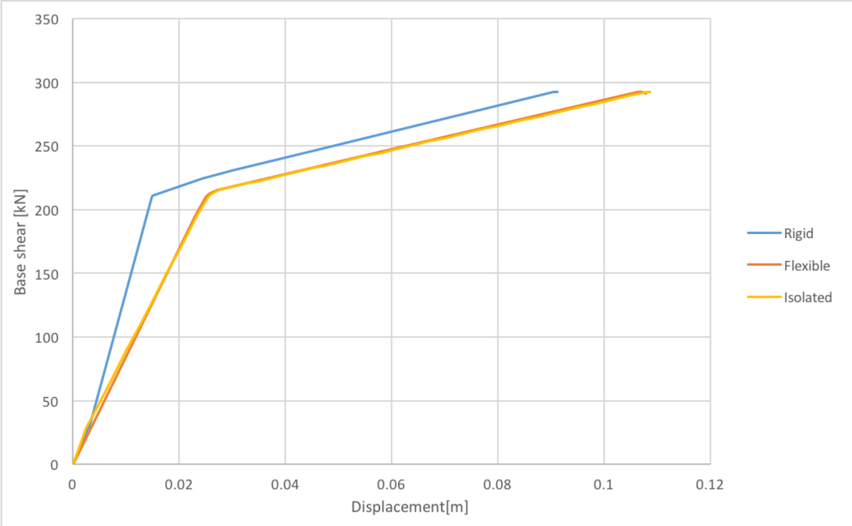


Figure 11: Capacity curves of the "frontal" wall

7.3. Reinforced flexible floors

Following the model with the original timber floors, models with reinforced specimens were analysed. The resulting capacity curves of the building with the control node on the "frontal" wall (Figure 12) show that both models with reinforced specimens (FMSD and FM+45SP) reach the same maximum base shear obtained with the rigid floors model, only with differences regarding the rigidity. These results are consistent with the ones from Giongo *et al.* [5], in which every reinforced floor configuration has the same resistance. It is thus possible to assume that in-plane floor reinforcement is an essential step in improving the behaviour of the building, since it allows to explore the total in-plane strength of both walls.

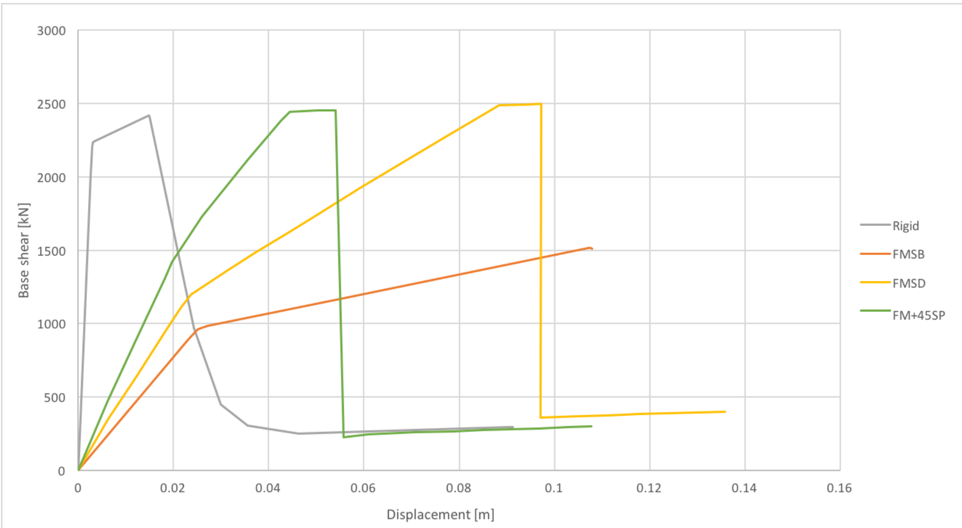


Figure 12: Capacity curves for different floor configurations

As a consequence, an analysis methodology that takes into account the in-plane floor rigidity in the nonlinear model is needed. For that effect, a methodology is proposed: an iterative method that evaluates the maximum displacement of each wall.

Proposed methodology:

- From the three-dimensional model, obtain the capacity curve of each wall, $F_i - \delta_i$ [1];
- For each wall, apply the N2 method to determine the target-displacement, d_{ii};
- In the three-dimensional model, apply a horizontal load to reach the displacement d_{ii}, and obtain the corresponding displacements on the surrounding walls, d_{ij};
- Assess the safety of each wall for the maximum value of d.

[1] Due to the contribution of the adjacent walls, the capacity curves differ from those obtained from a 2D analysis

The results of the methodology applied to the two analysed walls are presented in Figure 13. It is possible to observe the displacement correspondence between the two walls, where the analysis of the "frontal" wall (d_{11}) leads to a higher displacement than the corresponding displacement obtained from the analysis of the façade wall (d_{21}).

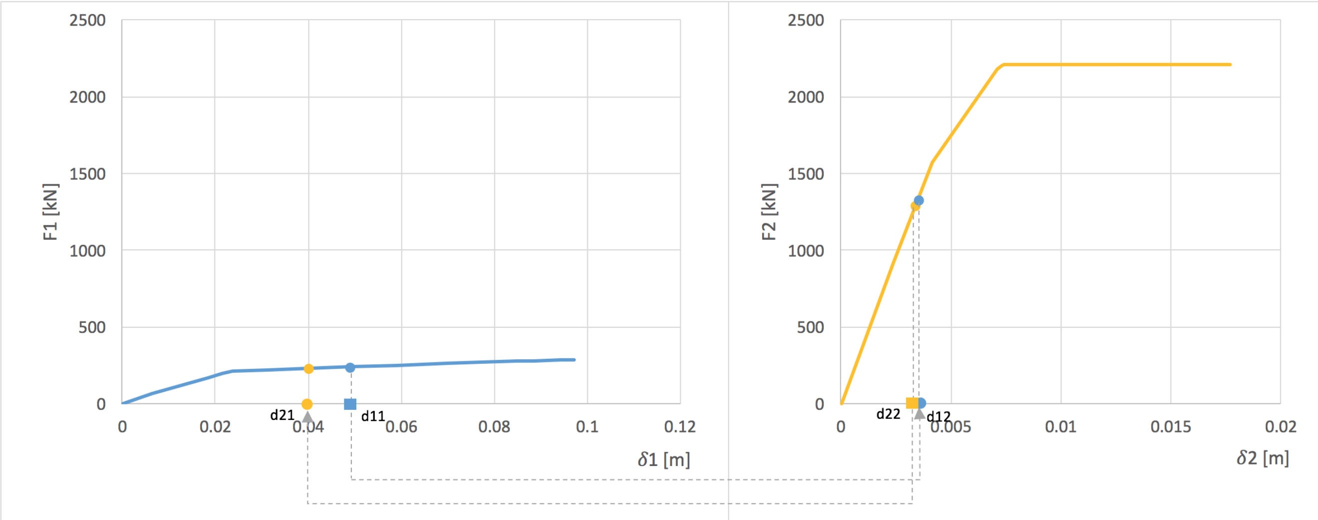


Figure 13: Target-displacement correspondence: a) "frontal" wall; b) façade masonry wall

Table 2 shows a comparison of the displacements calculated throughout the different analyses carried out in this study. It can be seen that both the two-dimensional target displacement (d_{isol}) and the maximum target displacement from the three-dimensional analysis (d_{max}) are lower than the displacement corresponding to the occurrence of the collapse mechanism (d_m). From these results, it can be assumed that both walls verify the safety requirements regarding the in-plane seismic action.

Table 2: Comparison of target-displacements [cm]

1 - "Frontal" wall (y=4,5m)	
d_{11}	4,89
d_{12}	3,99
d_{max}	4,89
d_{isol}	5,89
d_m	9,72
$d_{max} < d_m$ - safe	

2 - Façade wall (y=0m)	
d_{21}	0,35
d_{22}	0,34
d_{max}	0,35
d_{isol}	0,21
d_m	1,77
$d_{max} < d_m$ - safe	

8. Conclusions

The main purpose of this work was the search for a non-linear analysis methodology for Pombalino buildings, combining façade masonry walls and interior "frontal" walls, considering the influence of the shear rigidity of the floors on the behaviour of the building.

The application of the methodology proposed by Kouris and Kappos [7] for the nonlinear analysis of the "frontal" wall was based on empirical expressions to calculate the nonlinear properties of the panels, while the "SAM" methodology was based on a equivalent frame idealisation of the structure and simplified constitutive laws for the structural elements. Both analysis methods were simple to model and it allowed the observation of the progressive collapse of the structure without requiring changes in the model.

It was also possible to observe the difference in behaviour between the two kinds of wall - the façade masonry wall achieved much higher maximum resistance associated to small displacements, while the "frontal" wall showed a high ductility but a low shear resistance. This difference in behaviour required special consideration, hence the study of the in-plane rigidity of floors and the definition of an equivalent membrane element to be used on the three-dimensional nonlinear model.

The consideration of rigid floors in the three-dimensional model presents advantages, such as the reduction of the target-displacement on the "frontal" wall by the transfer of the horizontal load to the more rigid masonry walls, and for fully exploiting the in-plane strength of both walls. However, due to the low rigidity of the timber floors, this type of model provides inaccurate results. On the other hand, the model considering the original unstrengthened floors presented no advantages when compared to the isolated pushover analysis of the walls, due to it's very low rigidity.

Based on the results obtained throughout the research, a methodology was proposed, that allows to perform a three-dimensional analysis of the building, considering the influence of the shear resistance of floors, while using the same principles and models from the two-dimensional pushover analysis of the walls.

The application to the case study building highlighted the advantages of the methodology - the consideration of the reinforced floors on the model improved the seismic response of the "frontal" and the global behaviour becomes similar to the model considering rigid floors as the floor rigidity increases.

References

[1] *Eurocódigo 8: Projecto de estruturas para resistência aos sismos*, 2009.

[2] *CSI Analysis Reference Manual*, volume SAP2000 v16. CSI, 2015.

[3] R. Bento, S. Falcão, and F. Rodrigues. Avaliação sísmica de estruturas de edificios com base em análises estáticas não lineares. *Sísmica 2004 - 6º Congresso Nacional de Sismologia e Engenharia Sísmica, Lisboa*, 2004.

[4] C. Cismasiu, F. Amarante dos Santos, and J. Silva Ferreira. Ensaios experimentais de identificação dinâmica em edifício habitacional. Não publicado, 2016.

[5] I. Giongo, M. Piazza, and R. Tomasi. Pushover analysis of traditional masonry buildings : influence of refurbished timber-floors stiffness. *SHATIS'11 International Conference on Structural Health Assessment of Timber Structures*, 2011.

[6] P. F. Giordano. Seismic assessment of a pombalino building. Master's thesis, University of Bologna, 2015.

[7] L. A. S. Kouris and A. J. Kappos. A practice-oriented model for pushover analysis of a class of timber-framed masonry buildings. *Engineering Structures*, 2014.

[8] G. Magenes and A. della Fontana. Simplified non-linear seismic analysis of masonry buildings. *International Masonry Conference - London, U.K.*, 1998.

[9] G. Magenes and A. Penna. Existing masonry buildings: General code issues and methods of analysis and assessment. *Eurocode 8 - Perspectives from the Italian Standpoint Workshop*, 2009.

[10] Ministero delle Infrastrutture. *Nuove Norme Tecniche per le Costruzioni*, 2008.

[11] S.T.A. Data. *3Muri Program - User Manual*.

[12] M. R. Valluzzi, E. Garbin, M. D. Benetta, and C. Modena. In-plane strengthening of timber floors for the seismic improvement of masonry buildings. *World Conference on Timber Engineering*, 2010.