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SEISMIC ASSESSMENT OF 'POMBALINO' BUILDINGS

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SUMMARY

'Pombalino' Buildings are old masonry buildings from Lisbon Downtown. These buildings were built after the 1755 Lisbon Earthquake and they include a three-dimensional timber structure enclosed in masonry walls aiming at providing seismic resistance to these structures. Seismic vulnerability of a 'Pombalino' building from Lisbon Downtown will be evaluated and its collapse mechanism will be identified. According to the collapse mechanism obtained, a strengthening solution will also be analysed defining the increment of building seismic capacity. Discussion will be made in terms of (i) the differences observed in seismic behavior of the building before and after strengthening, (ii) how the knowledge of the expected structural collapse mechanism allows identifying the structural elements that should be strengthened, leading to a better seismic performance of the building.

INTRODUCTION

Old masonry buildings are an important part of the building stock of most European cities. These buildings are still being used and their main functions at present days are mostly housing and the installation of services (offices of companies and banks). The importance of the preservation of the cultural heritage and the functions that old masonry structures still maintain in our days justify the concern about their structural safety, including under earthquake actions. Recent earthquakes in Europe showed a bad performance of masonry buildings under seismic actions and Portuguese buildings are not expected to be an exception.

'Pombalino' Buildings are old masonry buildings from Lisbon Downtown built after 1755 Earthquake. These constructions include a three-dimensional timber structure enclosed in interior masonry walls above the first floor. This timber structure is named 'gaiola pombalina'

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and was included in the buildings to provide seismic resistance to these constructions. The influence of 'gaiola' in 'Pombalino' Buildings global behavior is described in Cardoso *et al.* [1].

Seismic assessment of a 'Pombalino' Building from Lisbon Downtown was performed, allowing identifying its expected collapse mechanism. The strengthening solution was defined according to the results obtained and the increase of the building seismic capacity after strengthening was evaluated.

DESCRIPTION OF THE BUILDING

Figure 1 presents the structural details of 'Pombalino' buildings. The three-dimensional timber structure of 'gaiola' in the interior walls above the first floor can be identified in this figure, as well as other interior walls (partition walls), which are wooden panels that do not have structural functions. Façades are made of masonry without the 'gaiola'. The openings (doors and windows) in these walls define the geometry of masonry structural elements. Roofs are made with timber truss and ceramic tiles and may include window openings. Floors are timber slabs and should be considered as flexible diaphragms. Ground floor interior walls are masonry walls supporting a system of vaults made of blocks of ceramic masonry and stone arches. Foundations include short and small diameter woodpiles connected by a timber grid.

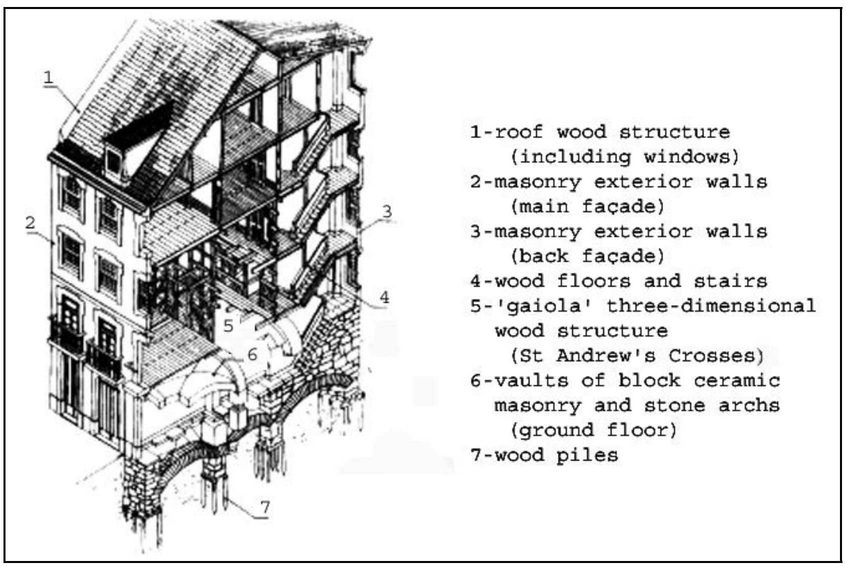


Fig. 1 – Structural details of a 'Pombalino' Building (*adapted from* Mascarenhas [2])

The timber structure of 'gaiola' is like a birdcage made of vertical and horizontal elements braced with diagonals. The masonry of the exterior walls is made of irregular blocks of calcareous stone and lime mortar with very poor strength capacity. Masonry infill of the 'gaiola' can be stone (rubble) or clay bricks. It is usual to find both type of masonry at interior walls. Most of the times, timber elements of 'gaiola' are notched together or connected by nails or iron ties, according to historical information regarding construction techniques. A more detailed description of 'Pombalino' Buildings can be found in Cardoso *et al.* [1].

Figure 2 presents the Building analysed from Lisbon Downtown (Prata Street, 210). It is a five-floor 'Pombalino' building and its plan, with the 'gaiola' walls identified, is presented in Figure 3.

According to the original conception of Lisbon Downtown, rebuilt after the 1755's Earthquake, 'Pombalino' buildings should have similar characteristics, such as number of floors, spans, materials, structural conception, in order that, in each block, all buildings would perform in similar manner. Therefore, the chosen building was analyzed as a single building, assuming that the error by disregarding interaction between adjacent buildings is small, as discussed in Ramos *et al.* [4].

Masonry exterior walls were simulated by thin bi-dimensional elements (shell elements) considering only bending deformation in and out of plane. Trussed bars, transmitting only axial forces (rotations free at the connections), were introduced to simulate the interior walls of 'gaiola'.

Figure 2 – Analysed Building (Prata Street, 210 to 220) (Santos [3])

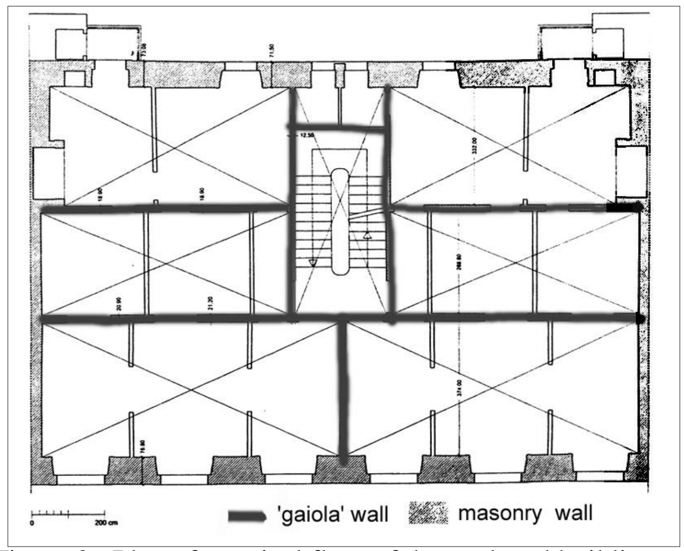


Figure 3 –Plan of a typical floor of the analyzed building (*adapted from* Santos [3])

Only timber elements were considered in the model of 'gaiola' walls, excluding the masonry where this timber structure is wrapped in. This provision was adopted to simulate the stiffness of 'gaiola' panels for horizontal forces acting parallel to their plane, according to the results of a previous study (Cardoso [5]). In this study, a numerical model of a 'gaiola' wall panel, removed from a 'Pombalino' building, was defined (Figure 4) and the results obtained were compared with experimental data (Figure 5). These data were the results of cyclic tests where a horizontal force, parallel to the plane of the wall and applied on top, was applied.

The stiffness obtained in both situations was compared. The experimental stiffness considered was the slope of the beginning of the first load cycle. The most relevant conclusion of the study was that the connections of diagonal timber elements could not transmit tensile forces. This conclusion is compatible with historical data regarding construction practices, the observed gaps between the different timber elements of 'gaiola', and the absence of iron elements at the connections. Therefore the contribution of the diagonal bars under tension was not considered in the numerical model.

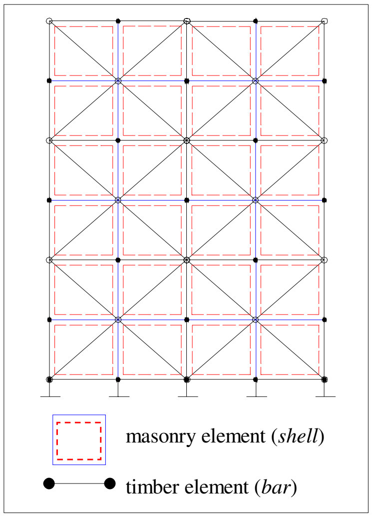


Figure 4 - Numerical model of a ‘gaiola’ wall panel (Cardoso [5])

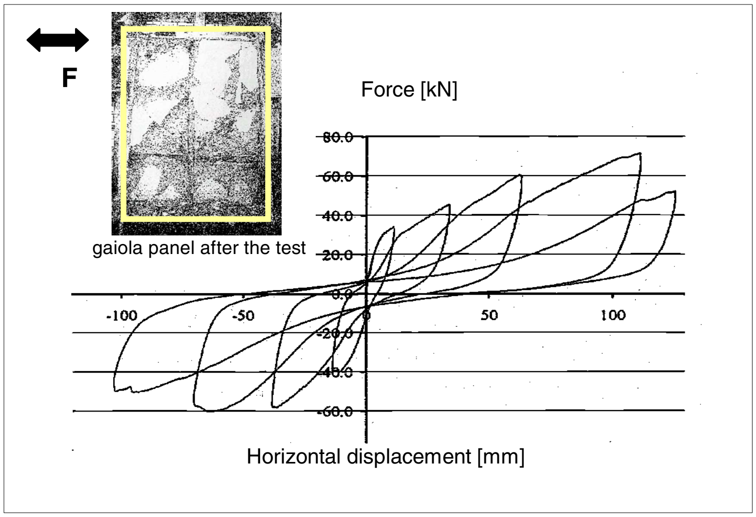
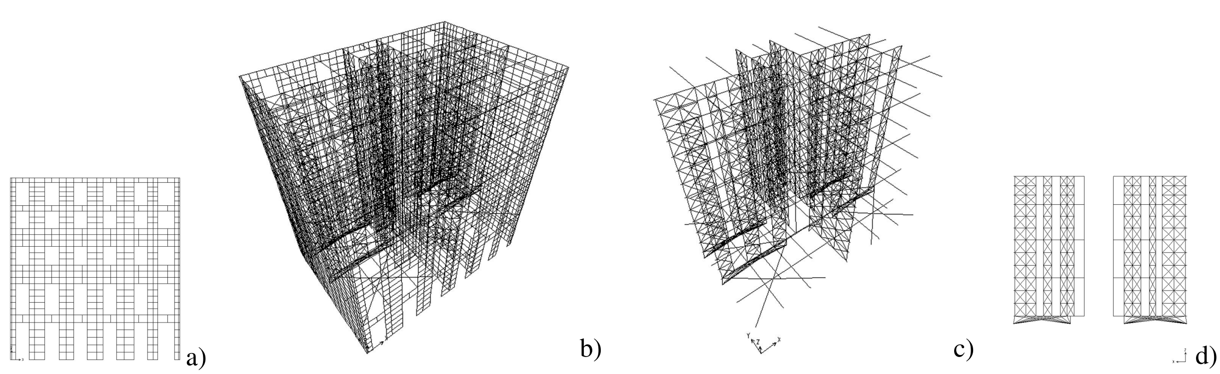


Figure 5 - Results of cyclic tests of a ‘gaiola’ wall panel removed from a ‘Pombalino’ building (Silva *et al.* [6])

NUMERICAL MODEL

A numerical model of the building was defined (Figure 6) and a commercial program (SAP2000® [6]) was chosen to perform the structural analysis.



a) Shell elements of masonry walls of front façade;
b) Complete three-dimensional structure with masonry shell elements and bars (detailed in c));
c) 3D ‘gaiola’ interior walls, floors bars, vaults (crosses) and stone arches at ground floor;
d) Interior wall of ‘gaiola’ and stone arches, parallel to front façade.

Figure 6 - Numerical model of the building (Cardoso [5])

The floors were modelled as truss bars with free rotations at the connections to the walls, simulating flexible diaphragms. The floor bars were orientated perpendicular to the façades, restraining the relative out-of-plan displacements of parallel masonry walls. The connections between timber elements of the ‘gaiola’ and perpendicular masonry walls were simulated considering timber bars that only resist to axial forces. In spite of the information about old construction techniques, that indicate connections between timber elements of the floors and ‘gaiola’ and masonry walls are done with iron elements, no iron elements were considered at these connections due to the uncertainties about their real existence in the buildings.

The masonry vaults of the ground floor were simulated by two crossed rigid diagonal bars with fixed displacements and free rotations in their connections to masonry walls. A triangular truss of rigid bars modeled the stone arches of ground floor. The connections between these bars allow relative rotations transmitting compression forces simulating the arch effect. These arches were connected to the interior 'gaiola' walls of the first floor parallels to the main façade and were supported by ground floor masonry walls. The self-weight of the roof structure was included in the nodes of the *shell* elements at the top of the building. The foundations were simulated by built-in connections.

Table 1 presents the Young's modulus, E, of the structural materials adopted in the numerical model. For the Poisson coefficient of all materials was assigned the value 0.2. According to the Portuguese Code RSA [8], a uniform service load (1.2kN/m^2) acting at all the floors was considered. The seismic action was defined by the acceleration response spectrum presented in the mentioned code, acting along the three orthogonal directions. Since the floors cannot be considered rigid in their own plan the mass was distributed by all the nodes of the model.

Table 1 – Properties of structural materials considered at numerical model (Cardoso [5])

Materials and structural elements		*Young's modulus E (MPa)*
Masonry	Façades and walls between buildings	600
	Damaged/ cracked masonry [(1)]	150
Timber	Floors and 'gaiola'	8000
Stone	Arches and vaults (ground floor) [(2)]	3000

(1) Masonry between perpendicular masonry walls (2) Calcareous stone

COLLAPSE MECHANISM OF THE BUILDING

Methodology of Analysis

It was intended to develop a methodology that could be useful in current Strengthening Design of old masonry buildings, by using a commercial program (SAP2000® [7]). An iterative procedure was adopted, which allows simulating in an approximately manner the main sources of non-linear behavior of masonry buildings. Linear dynamic analyses by response spectrum were performed. The acceleration response spectrum used is presented in Figure 7.

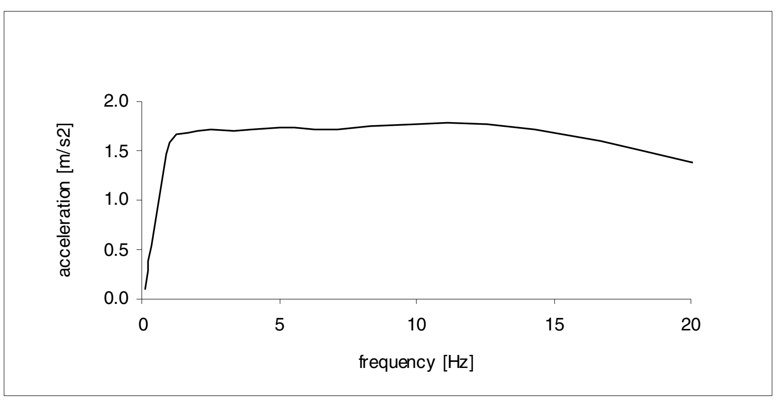


Figure 7 – Response spectrum of accelerations (RSA [8]) (Seismic Action n. 2, Soil type III (soft soil))

For the analysis, it was useful to define a scale factor of the response spectrum, γ_{sis}, in order to simulate different levels of seismic action. The value of γ_{sis} was increased intending to obtain the value that quantifies the intensity of the seismic action corresponding to the collapse of the structure. The final value of γ_{sis} obtained in the calculation was named $\gamma_{sis}^{máx}$ and allows to define the intensity of seismic action corresponding to the collapse of the structure quantifying, in this manner, the building seismic resistance.

The main sources of non-linear behavior that influence the behavior of 'Pombalino' Buildings are (i) crack openings in masonry walls, (ii) the rupture of connections between structural timber elements of 'gaiola' and (iii) the rupture of connections between structural timber elements from 'gaiola' walls and from the floors and masonry walls. The analyzed connections are identified in Figure 8. The masonry damages considered were tension and compression due to bending and shear in and out of plane of the wall. The main sources of non-linear behavior were simulated by a number of changes in structural configuration, corresponding to the rupture of structural elements that were named damages. The structure analyzed in each step is defined according to the damages obtained in the structure analyzed in the previous step. The process was repeated until the accumulated damages allow identifying the collapse mechanism of the structure.

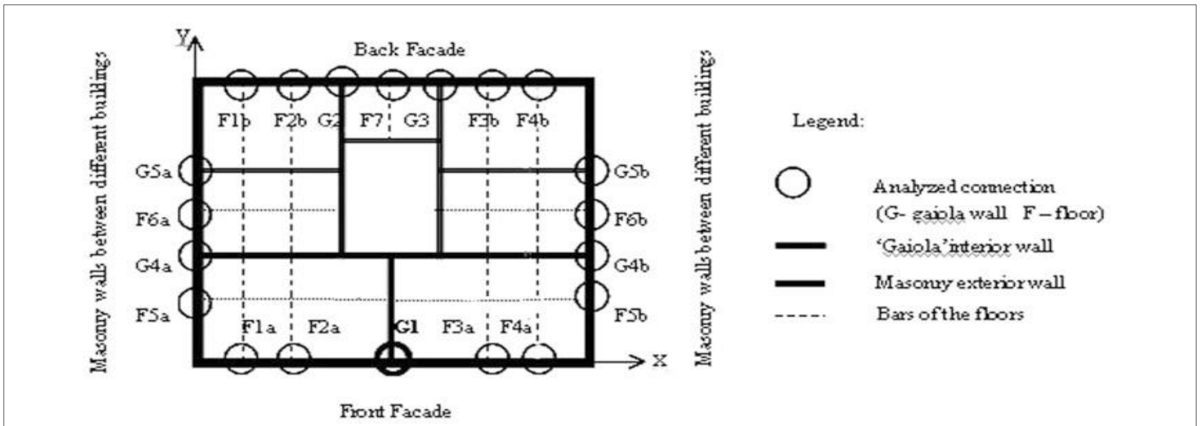


Figure 8 – Plan of the numerical model of the building identifying the different walls, the bars simulating the floors and the connections between the timber elements and the masonry walls

The design action effects (internal forces) in structural elements, F_{Sd}, are defined according to Equation 1, where F_{Perm} are the effects of vertical permanent loads and F_E are the effects of the seismic action, multiplied by the scale factor γ_{sis} to simulate the intensity of the seismic action. Different seismic intensities can be considered in seismic structural design by changing the value of γ_{sis}.

$$F_{Sd} = F_{Perm} \pm \gamma_{sis} F_E \qquad \text{(Equation 1)}$$

For each structural element or connection, the design actions effects, F_{Sd}, will be compared with the respective resistance, F_{Rd}, identifying rupture if F_{Sd} are larger than F_{Rd}. Table 2 presents the strength values (F_{Rd}) adopted for the structural elements considered in the analyses. These values are average values and cannot be considered equal to all masonry buildings, mainly due to the large variability of the properties of structural materials and to the variety of structural solutions found in old masonry buildings.

It was considered that the connections achieve rupture only in tension, corresponding to the pullout of timber elements, from the rest of the structure. Since the structural elements for which the rupture has occurred are removed from the model, it is assumed that, once the element is pulled out the wall, it will not resist to tension again. This procedure is equivalent to assume brittle behavior of the connections.

Table 2 – Strength values adopted for damage calculation in relevant structural elements (Cardoso [5])

Structural element		*Strength values - F_{Rd}*
Connections	Braced timber bars in 'gaiola' (only tension)	0 kN
	Timber bars – masonry walls (only tension)	5 kN
Masonry	Compression	1.3 MPa
	Tension	0.1 MPa
	Shear	0.1 MPa

Collapse Mechanism obtained

The collapse mechanism obtained was the bending of the front façade out of its plane. Three iterations were necessary to identify this mechanism. The corresponding seismic intensities were $\gamma_{sis}(1)=\gamma_{sis}(2)=\gamma_{sis}(3)=0.25$, therefore $\gamma_{sis}^{máx}=0.25$. Figure 9 presents the evolution of the tension-damaged masonry from the front façade due to bending. These damages allow predicting masonry cracks similar to those presented in Figure 10.

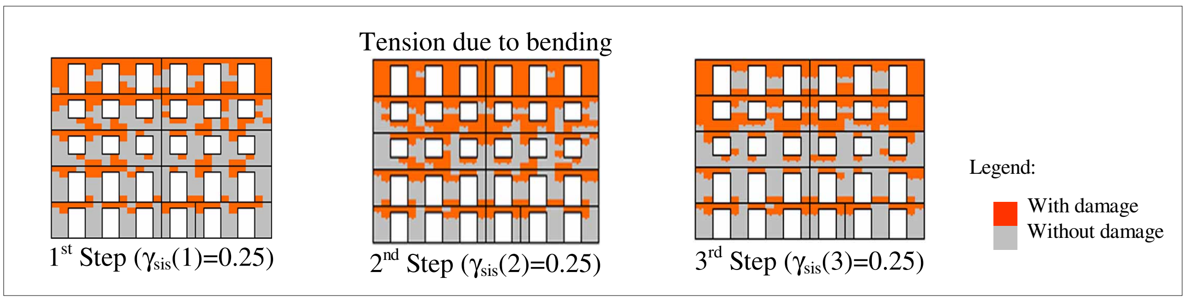


Figure 9 - Damage levels in masonry elements from main façade in iterative calculation

Sequential structural collapse will be expected because the seismic intensity obtained for all the intermediate steps of calculation was the same. The sequential collapse means that the rupture of some structural elements leads to the rupture of more elements for the same level/intensity of the seismic action.

Figure 10 – Damages due to 1998 Azores Earthquake, similar to those obtained in the analysis

Figure 11 presents the damages at the end of the iterative process, which allow to identify the out-of-plane bending of the front façade as the building collapse mechanism. Since masonry walls can fall only after the rupture of their connections to the perpendicular 'gaiola' walls that support them, the described collapse mechanism was identified by considering simultaneously the rupture of the connections G1

(Figure 8), between the front façade and the perpendicular 'gaiola' wall, and the damages in the masonry wall.

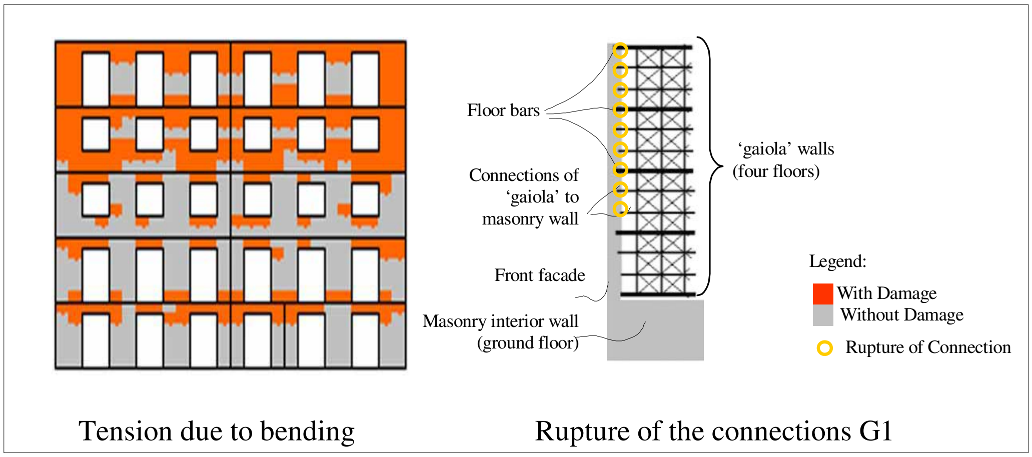


Figure 11 - Masonry damages and the rupture of connections at the end of the iterative process that allowed identifying the bending of the front façade collapse mechanism

The collapse mechanism obtained eventually corresponds to the original designers conception presented in Figure 12 (Mascarenhas [2]) that intended to preserve the inner wood structure of the buildings aiming at the safekeeping of human live. This conception may be efficient for one or two floor buildings as it shown in Figure 13 that represents a building whose exterior walls have fallen out-of-plane in the 1998 Azores earthquake. However, there are some uncertainties regarding its efficiency in buildings with more than two floors because the out-of-plane fall of the façades may bring down other parts of the building.

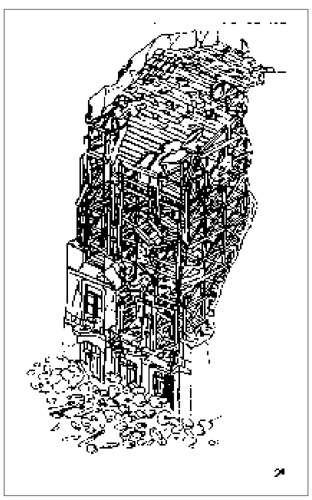


Figure 12 - Collapse mechanism according to original conception

Figure 13 - Fall out the plane of exterior walls without complete collapse (1998 Azores earthquake)

The collapse mechanism can also be identified by the out-of-plane horizontal displacements of the main façade presented in Figure 14. For better understanding the real out-of-plane deformed configuration of the wall, the displacements presented are the difference between the horizontal displacements observed in the connection G1 (Figure 8) and those observed in the left corner of the front façade of the building. The maximum values were observed at the top of the building and are presented in Table 3 for each step of the iterative calculation, considering the intensity of the seismic action associated to the corresponding values of γ_{sis} ($\gamma_{sis}(1)=\gamma_{sis}(2)=\gamma_{sis}(3)=0.25$).

The displacements observed (Table 3) at the end of the iterative calculation (2.50cm - step 3) were increased 217% above the value observed in the first iteration. The displacements obtained in the first step are the displacements that would be obtained in linear analyses, indicating that the iterative calculation can lead to a more realistic evaluation of the displacements.

Figure 14 and Table 3 also include the displacements observed in the numeric model of the same building without the 'gaiola'. It can be observed that displacements measured at the end of the previous iterative calculation were 0.2cm (5%) smaller than the displacements measured in the same building without the 'gaiola' (Table 3). The difference in these values allows understanding the contribution of the remaining active connections after the collapse of the building (step 3) to its stiffness, as the displacements obtained are smaller than those measured in the same building without the 'gaiola' structure.

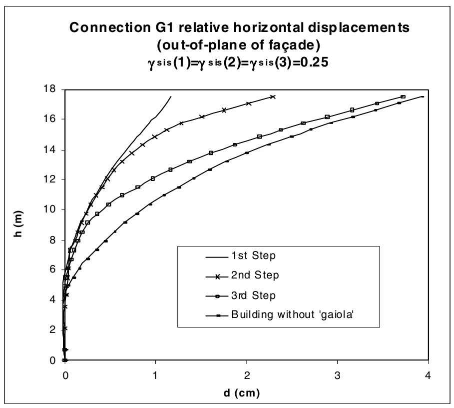


Figure 14- Horizontal displacements out-of-plane of the façade along the connection G1, relative to the corner of the building.

Table 3 – Horizontal relative displacements out-of-plane at the top of connection G1 ($\gamma_{sis}(1)=\gamma_{sis}(2)=\gamma_{sis}(3)=0.25$)

	Step 1	*Step 2*	*Step 3*	*Building without 'gaiola'* $\gamma_{sis}^{max}=0.25$
Displacement[(1)] (cm)	1.20	2.30	3.70	3.90
Increasing displacement (cm) step by step	-	1.10	1.40	-
Increment	2.50cm (217%)			0.2cm (5%)[(2)]

(1)Relative to the left corner of the front façade (2) Relative to the last step of iterative calculation

STRENGTHENED BUILDING ANALYSES

The collapse mechanism obtained provided useful information for the definition of a strengthening solution. The solution adopted was the inclusion of a concrete beam around the exterior perimeter of the building (h=0.25m b=0.60m), on the top, as presented in Figure 15. The expected collapse mechanism was analyzed by the iterative method previously described. The deformed shapes of the front façade of the buildings for the seismic action ($\gamma_{sis}=1$), before and after strengthening, are presented in Figure 16. It can be observed (Figure 16) that the deformed shape of the front façade after strengthening is smoother than before, showing the stiffness increment of the façade to out-of-plane displacements due to the presence of the concrete beam.

The global stiffness of the structure is also affected by the strengthening solution, which can be confirmed by analyzing the frequencies and configuration of the first modes of both buildings - Figure 17.

According to Figure 17, the 2nd mode after strengthening corresponds to the 1st mode before strengthening. The increment of the frequencies is higher than the increment observed for the 2nd mode before strengthening, which corresponds to the 1st mode after strengthening. The sequence of the dynamic mode shapes indicates that for the 1st mode the direction of horizontal displacements changes from the out-of-plane of the façade, before strengthening, to the in-plane direction, after strengthening. This result indicates global stiffness increment in out-of-plane direction, which was expected after the strengthening solution adopted.

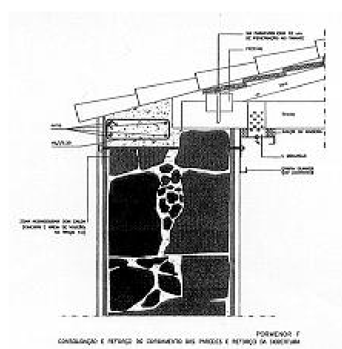


Figure 15 – Strengthening solution adopted (Costa *et al.* [9])

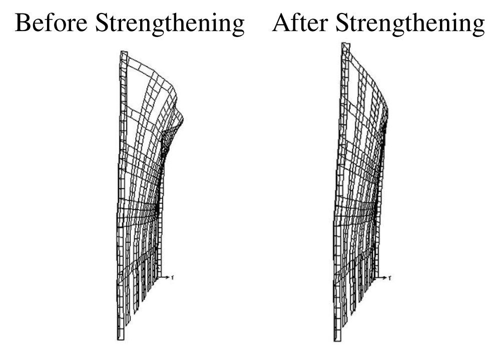


Figure 16 - Deformed shape of the front façade of the building ($\gamma_{sis}(1)=1$ for both buildings)

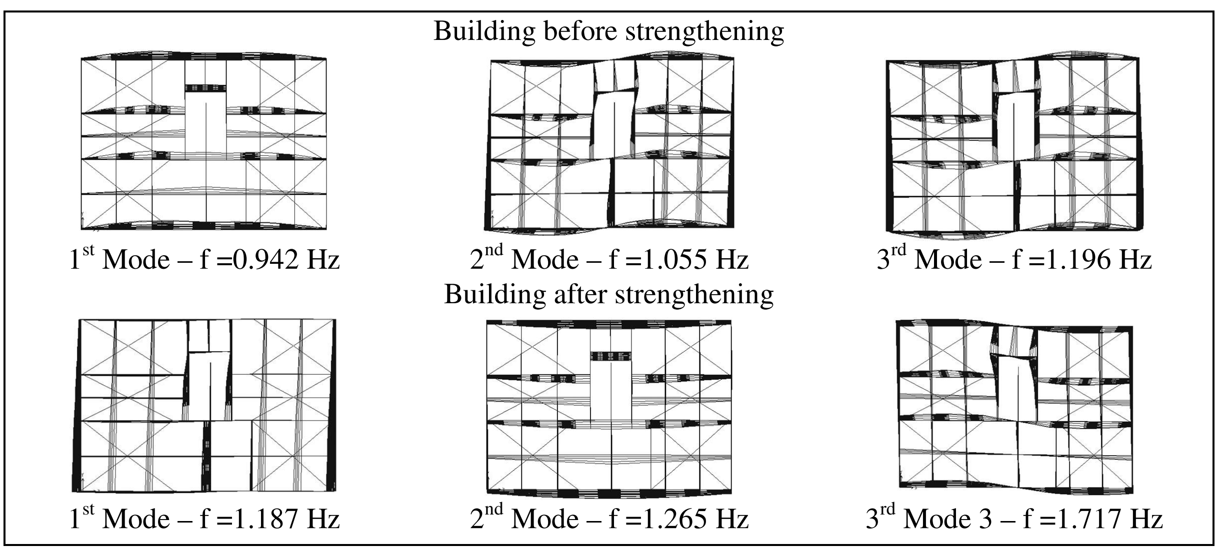


Figure 17 – Dynamic shapes of the building before and after strengthening, for the first step

The collapse mechanism of the building after strengthening was identified after three steps of calculation. The seismic intensities obtained were $\gamma_{sis}(1)=\gamma_{sis}(2)=\gamma_{sis}(3)=0.45$, equal to γ_{sis}^{max}. As obtained before strengthening, the collapse of the structure will be sequential because γ_{sis} is equal for all steps. Figure 18 presents the evolution of the horizontal displacements of the front façade due to seismic action, obtained in connection G1 (Figure 8). The displacements presented are the difference of the displacements at the connection G1 and the displacements in the left corner of the front façade.

Figure 18 indicates that the collapse mechanism after strengthening is still the out-of-plane fall of the façade, triggered by the collapse of the connections with the 'gaiola' wall. However the sequence of collapse starts by the connections at third floor level and not at the top. Figure 18 shows the difference in the displacement pattern along the high as compared to the structure before strengthening (Figure 13). The obtained collapse mechanism may indicate that the introduction of a concrete beam, or of a metallic tie, in all the floors of the building connecting the façade to the floors and to the 'gaiola' walls at these levels, as presented in Figure 19, will be an efficient solution to increase the seismic capacity even further.

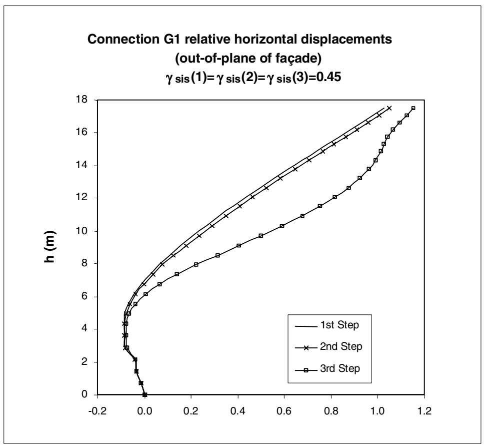


Figure 18 – Horizontal displacements out-of-plane of the façade after strengthening

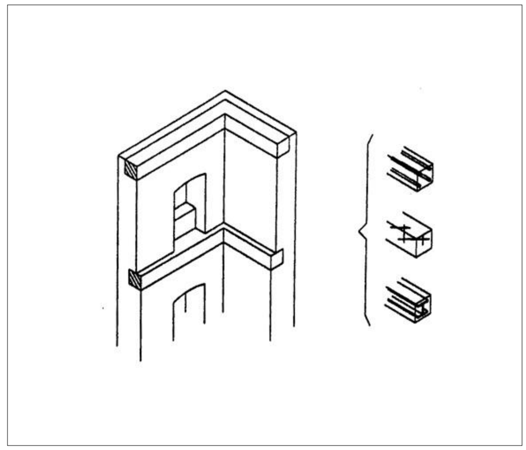


Figure 19 – More adequate strengthening solution (Croci [10])

According to the obtained results, the expected collapse mechanism of the building before and after strengthening will be the overturning of the front façade. Nevertheless, the strengthening solution increases global seismic resistance of the building because γ_{sis}^{max} was increased from 0.25, before strengthening, to 0.45, after strengthening.

GLOBAL SHEAR MECHANISM

The global base shear mechanism was also analyzed. Damage state corresponding to this mechanism is considered to occur when the damaged masonry elements define a continuous horizontal slip surface. For the building before strengthening, this mechanism was reached for the value γ_{sis}=0.70. The first masonry elements showing the complete slip surface due to shear collapse were the vertical elements of the façades (masonry between the doors of ground floor), as presented in Figure 20. This result was expected because the shear resistance of the masonry walls of the façades, due to the doors and windows openings, is smaller than the resistance of masonry walls without openings, like the walls between adjoining buildings.

The shear base collapse mechanism was also analyzed for the strengthened building and a similar value γ_{sis}=0.70, only slightly inferior, was obtained. The values of the global shear base reactions presented in Table 4 allow understanding the small difference observed in the γ_{sis} for shear base collapse mechanism for the building before and after strengthening. According to the values presented in Table 4, the increment of the global shear base reaction parallel to the front façade due to strengthening is 9%, which is a reduced difference, justifying the similarity of the values of results obtained for both situations.

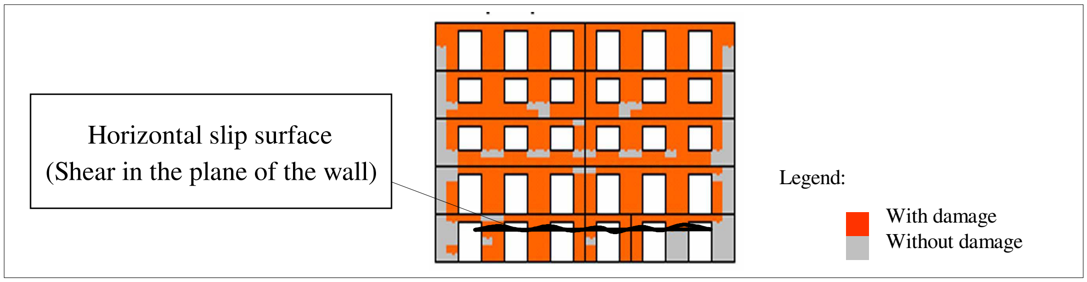


Figure 20 – Masonry shear damages identifying the shear base collapse mechanism (γ_{sis}=0.70)

Table 4 – Global Shear-Base reactions for seismic actions for the building before and after strengthening (γ_{sis}=0.70 for each case)

Global Shear Base Force	*Before strengthening [kN]*	*After strengthening [kN]*
Parallel to the front façade	1733	1888 (+9%)
Out-of-plane of the front façade	1474	1756 (+19%)

The value of γ_{sis}=0.70 (for shear collapse) is higher than the values γ_{sis}=0.25 or γ_{sis}=0.45 (for collapse due to out-of-plane displacements) indicating that global shear mechanism will not be the expected collapse mechanism of these type of structures (before strengthening). According to the results, it is expected that the overturning of façades will take place before the occurrence of the shear mechanism, because the expected collapse mechanism will be the one corresponding to the lowest value of γ_{sis}.

CONCLUSIONS

The iterative procedure considering the damage evolution in structural elements in each step allows understanding structural behavior until the collapse. Since it provides information about the sequence of structural damages, the first damaged elements are the weakest links detected in the analysis. These are the structural elements whose resistance must be checked and improved in real structures, providing this way an important contribution to identify strengthening solutions. The iterative process also allows modeling non-linear behavior of masonry buildings, because the structure is changed step by step, allowing the simulation of sequential collapse.

The values of γ_{sis}^{max} obtained in this study give information about seismic vulnerability of old buildings and seismic resistance improvement after strengthening. The results obtained from seismic assessment of the building before and after strengthening are presented in Figure 21.

The main conclusions regarding the results of the analyses performed (Figure 21) are: (i) the expected collapse mechanism of the 'Pombalino' building is the bending out-of-plane of the front façade, corresponding to γ_{sis}=0.25; (ii) the strengthening solution adopted increases the seismic resistance of the building to γ_{sis}=0.45; (iii) after strengthening, the collapse mechanism is still the bending out-of-plane of the front façade but at an inferior level (third floor instead of fourth floor, which is the top floor of the building), therefore the most efficient strengthening solution would be the inclusion of a concrete beam or metallic tie in all the floor of the building; (iv) global base shear mechanism occurs for γ_{sis}=0.70 but the bending out-of-plane of the front façade mechanism occurs first in both analyzed buildings because the expected collapse mechanism for the building is the one corresponding to the lowest γ_{sis}.

The low values obtained to factor γ_{sis}^{max} (γ_{sis}^{max} =0.25, before strengthening the building, and γ_{sis}^{max}=0.45, after strengthening) show a low strength of these structures for seismic actions. These values, corresponding to the collapse of the analyzed structures, are related to the strength parameters adopted in this work and presented in Table 2. In real design of strengthening solutions, it is fundamental to define carefully these parameters. 'In situ' techniques and non-destructive testing are advisable because the results would allow a better definition of the strength parameters adopted for each particular building. The most relevant information regarding realistic seismic assessment of 'Pombalino' Buildings and old masonry buildings in general are (i) the definition of masonry shear strength, which is fundamental to quantify the resistance to the global shear mechanism, and (ii) the definition of the resistance of the connections between structural elements, due to their relevance to the building global behavior.

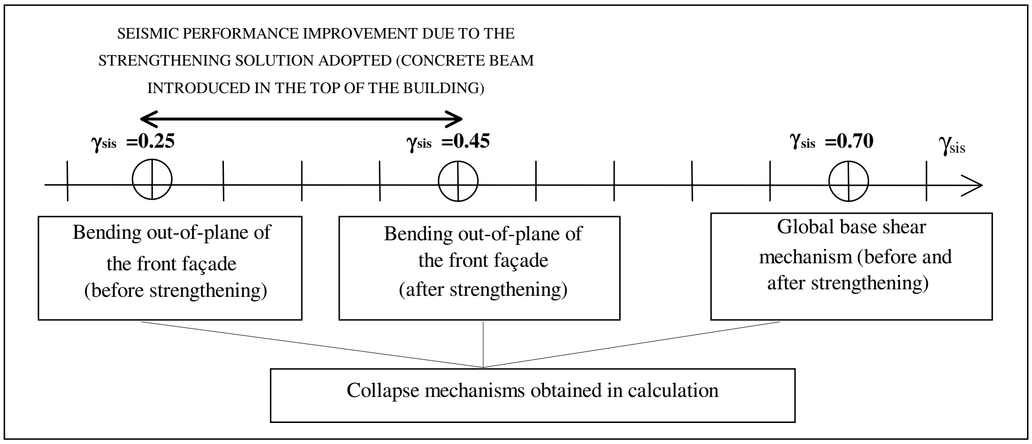


Figure 21 - Results from seismic assessment of the 'Pombalino' building analyzed

The value of factor γ_{sis}^{max} also depends of the hypothesis adopted to define the model: (i) assuming that the seismic action starts in all the steps of calculation by adopting the same response spectrum in all the iterations and (ii) accounting the hysteretic capacity of 'gaiola' walls to dissipate energy by adopting a viscous damping coefficient equal to 10%. These hypotheses are conservative and may lead to the underestimation of the value of γ_{sis}^{max}.

Since the strength parameters adopted correspond to mean values and, according to the hypothesis adopted to define the model, the final value of factor γ_{sis}^{max} cannot be evaluated with the precision that would be for a reinforced concrete or steel building. Even so, the calculated value can be considered a parameter of seismic vulnerability of the building. The low values of this factor obtained in the analyses justify the concern about seismic performance of these buildings.

The results of the seismic assessment of the 'Pombalino' building studied cannot be extrapolated to all the other masonry buildings. In fact, they can be different due to (i) different construction practices; (ii) different structural materials, age and consequent degradation; (iii) the high number and variety of structural changes introduced in these structures, adapting them to new functions. Moreover, the presence of 'gaiola' in the building analyzed indicates a better seismic behavior for the constructions where this timber structure exists. Since the results obtained for the analyzed building justify the concern about its safety, seismic strengthening of old masonry buildings in general should be a priority.

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