INSTITUTO SUPERIOR TÉCNICO

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EXTENDED ABSTRACT

Rehabilitation of Pombalino buildings
Experimental analysis of frontal walls

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October 2010

1. INTRODUCTION

Nowadays an awareness of cultural, architectural and urban value of cities historical centers is present in citizens. Thereby, the conception and construction quality in this type of interventions, specifically in "pombalino" construction, which is highly appreciated by Lisbon citizens, becomes an increasing concern for both designers and constructors. This type of construction can be seen not only in Lisbon but in other cities in Portugal and even in Brazil.

First of all, this dissertation aimed to provide some guides to project, preparation and intervention execution of housing heritage rehabilitation in the "Pombalino" buildings, in a constructional and structural approach.

A study about the "Pombalino" buildings structure and the main pathologies to which they are subjected was performed, including some rehabilitation suggestions.

The "pombalino" buildings' good seismic behavior is ensured by its complex wooden cage structure. The great motivation of this project was to understand and study the cage's frontal walls behavior in order to comprehend the building's global behavior, in terms of safety assessment as well as in terms of rehabilitation and strengthening design projects execution.

Therefore, an experimental campaign was held to support the realization of this project. Due to the great complexity of the "pombalino" structure and the wide variety of factors involved, and in order to start this study in a simple way, the experiment would focus the simplest "pombalino" cage elements, for example, the "Santo André" Cross.

This element is composed by a close wooden structure with two plumb lines, two battens and two diagonals which are interconnected. The space between these elements is filled with masonry. Starting with such simple element, it is believed that this project can be a starting point to a more global and complex study on this theme.

In this context, the purpose was to analyze the behavior of this structure under the action of an increasing horizontal load up to a cracker point, conjugated with a constant vertical load which intends to simulate the weight above this element. Three modules consisting of a "Santo André" cross, called wooden cage, and three modules composed by the wooden cage filled with masonry, called masonry walls, were tested. This differentiation intended to assess the contribution of each constituent – wood and masonry – to the global action. Then, a numerical model was made, in this case, a linear elastic model, attending to reproduce the experimentally measured structural behavior. This work aimed to define a more extended investigation program, using an experimental evaluation and a numerical modeling of frontal walls, towards seismic reinforcement analysis of "pombalino" buildings.

2. DESCRIPTION OF "POMBALINOS" BUILDINGS

The "pombalino" construction arose from the great effort in the reconstruction of the city of Lisbon due to the earthquake on the 1st. November of 1755. This construction is characterized by its originality, improvement and technological revolution, as it uses methods of construction, such as pre-fabrication and normalization.

The buildings called “pombalino” area have some specific structural characteristics, although there is a wide range of variety in the buildings’ structure. In this part of the dissertation the general characteristics are described. The general foundation system is consisted of caissons and a group of masonry arches which is the basis to all upper structure. The arches, on the other hand, lay on a system of wooden piles (Figure 1). These short, made of green pine with the capacity to absorb the fill’s humidity piles have a 15 cm diameter and 1.5 m length, 30 to 40 cm apart from each other. This system creates a rigid system which improves the resistant capacity of the ground, by its’ consolidation and confinement [RAMOS, 2002].

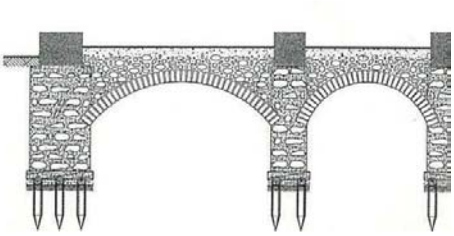


Figure 1 Arches and pile foundation [SILVA, 2007]

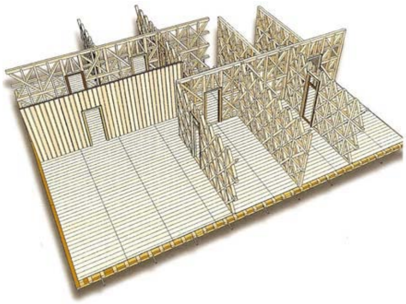


Figure 2 One floor wooden structure [APPLETON, 2003]

Inside each building, there is a three-dimensional bracing system called “gaiola pombalino” or ‘Pombalino’ cage. This system is formed by an orthogonal mesh of walls in two directions called “pombalino” frontals, the wooden structure of the pavement and roofing plus a wooden structure which is connected to the exterior stone walls by wooden connectors. This structure can be observed in Figure 2. The structure complexity is responsible for the bracing which is the key to the good seismic behavior of these buildings [APPLETON, 2008]. The “pombalino” frontals are composed by wooden pieces: vertical (plumb lines), horizontal (battens) and diagonal (shores). This intersection creates elements of “Santo André” Cross, which spaces are filled with masonry (Figure 3 and Figure 4).

Figure 3 Frontal wall, ‘Santo André’ cross

Figure 4 Frontal wall missing some masonry parts [APPLETON, 2008]

The frontals above the first floor, in the two directions, play an important structural role in the cage behavior, not only to absorb part of the vertical load, but also for the general bracing of the structure. These characteristics are ensured, partly, by appropriate connections between walls and pavements, roofing and load-bearing walls. In fact, these connections are very important, as they constitute the restraints that limit the deformations and the different structural elements stresses [APPLETON, 2008].

In a “pombalino” building, in general, there is a great difference between the ground floor ceilings (pavement of first floor) and the other floors. The ground floor ceilings are constituted by arches and stone masonry arch roves. However, in the higher pavements there is a significant predominance of wood as structural material, combined with the cage. An ordinary floor pavement is made of wood beams, both under and upper coated. Generally, the beams are orthogonal to the facade walls, with 0.14 to 0.16 m large and height sections, almost squared. The roofing of “pombalino” buildings are inclined and can have several forms, but a main constitution of a gabled roof is composed by tiles placed over a wooden structure of saddle roof truss, purlins, tie beams, principal rafters and other beams supported in the walls extensions.

3. MAIN PATHOLOGIES AND REHABILITATION TECNIQUES IN “POMBALINO” BUILDINGS

3.1 Pathologies

The main cause of anomalies in “pombalino” buildings is natural, like the inevitable aging of materials, which leads to the alteration of elasticity, mechanical resistance, and others [APPLETON, 2003]. The pathologies found in masonry are mainly cracking, located crush and disintegration. The ones found in wood elements are due to humidity, which causes wood deterioration. This feature can be extended to the joints made of metallic elements, increasing therefore the local deformability [LNEC, 2006]. This problem may not be expanded to a global failure due to the structure high degree of indeterminacy and its ductility.

Some options taken in the project or in the whole constructive process can result in other type of anomalies or inadequacy. Throughout time the “pombalino” buildings have suffered many modifications by their users, sometimes with total disrespect for early constructive characteristics. These modifications may be punctual, such as walls removals, room enlargement, and modifications of flat conditions of use, usually associated with a loads increase that reduces the former endurance of the material. Another type of modification, more critical to the building, is both the height enlargement and basements introduction. Without a previous study of security and stability, those types of modifications are not adequate, as they may cause additional loads in the structure thereby increasing stresses and displacement of the whole structure, in particular during a seismic scenario. A special caution must be considered with the external load-bearing walls, as the added floors may not have been properly connected to them through interior perpendicular walls [LOPES e MONTEIRO, 2008] or proper connectors.

Other severe interventions are the changes performed in the ground floor. These are associated to opening interspaces, suppressing vertical structure elements in order to enlarge retail spaces, which cause an abruptly reducing of stiffness, creating “cast” floors or artificial *soft-storeys*. The observation of earthquakes damage in the past shows that this type of structural irregularity is quite harmful. These suppressed vertical elements are easily detected by the contrast with upper floors, where these elements are visible in the facade [LOPES e MONTEIRO, 2008].

Nevertheless, the need to modify and adapt the commercial areas in the ground floor to new functions does not intend a severe structural alteration, if studied. In these interventions a special care must be considered regarding this issue.

The global behavior of the building is influenced by the plant disposition but also the arrangement of the walls and masses. So, the insertion of new materials, like metallic or reinforced concrete, should be avoided as it accentuates differential stiffness, leading to an increase of the plan eccentricity between mass and stiffness. This fact induces extra torsions that may origin damage in the structure [COSTA, 2008].

3.2 Rehabilitation and strengthening

In rehabilitation and reinforcement actions, a perfect knowledge about the structural situation of the building has to be considered, in order to make the "selective reinforcement". In other words, the intervention should be minimized to strictly necessary spots exploring the efficiency, the ductility and the structure resistant capacity. The technique and solutions chosen should minimize the modification of the original structure, as well as avoid new pathologies. A differentiation of the rehabilitation technique is made according to its constituent material (masonry or wood) and global reinforcement techniques.

The consolidation and reinforcement solutions in masonry walls are generally injections, degraded material substitution, metallic or concrete elements addition, reinforced plastering execution, and transversal confining placing. The technique's main objective consists in a resistance capacity increase, mainly to compressive strength, re-establishing the local or global integrity of the degraded sections.

Regarding the wood components, the rehabilitation technique includes interventions which involve resistance increase by total removal and adding new materials such as splicing, among others, or reconstruction of degraded constituents technique (partial removal).

Regarding frontals, in the reinforcement of wood elements joints, it is also possible to implement a wire-mesh reinforcement of composite material. Furthermore, for pavements which have a major deformability problem, reinforcement beams can be applied. The enforcement of elements by placing pieces of steel is a common solution in rehabilitation of "pombalino" buildings.

The techniques designed to improve the global structure behavior are extremely important to increase the whole resistance in an earthquake scenario. For instance, it is possible to improve perpendicular masonry walls connections using steel ties, the connections between frontal and masonry walls with the use of anchorages or transversal confiners, and also the connections between pavements and walls using nailing process.

4. EXPERIMENTAL CAMPAIGN

4.1 Introduction

The limited knowledge about the whole behavior of the “pombalino” cage system, and, particularly, about frontal walls’ behavior, motivated the experimental campaign presented in this chapter which aims to give a positive contribution and help guiding future projects in this area. Due to the “pombalino” cage structural complexity and the several factors involved, it was decided that these tests would involve the simplest elements. Therefore it was decided to analyze only one part of “pombalino” frontal wall, in other words, the simple “Santo André” cross element.

In this context, the purpose was to analyze this structure behavior in an increasing horizontal load test until fracture, conjugated with a constant vertical load that intends to simulate the existing load above this element. Therefore, three modules named GM1, GM2 and GM3 (Figure 5) were tested, consisting of a “Santo André” cross called wooden cage, and three modules PA1, PA2 e PA3 (Figure 6) filled with masonry called masonry walls. This distinction intends to evaluate the contribution of the constituents – wood and masonry – to global behavior and their interaction. Concerning the connections used in these samples they were made using a half-wood method shown in the Figure 7. These connections were reinforced using nails.

Figure 5 - Wooden cage (GM)

Figure 6 Masonry wall (PA)

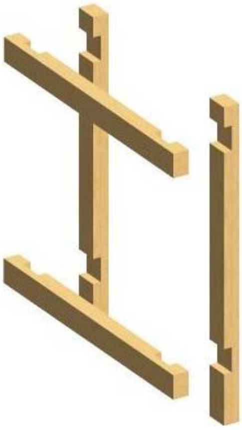


Figure 7 Connections

As for the devices, various deformeters (d1, d2 e d3) and thirty two strain gauges were used, positioned and numbered as shown in Figure 8. A structure was constructed in the laboratory to perform these tests, consisting of a foudation, one grounding metalic beam, screw actuator, reaction wall and a lateral bracing frame (Figure 9). In the reaction wall there is a jack which is responsible for the imposition of the horizontal displacement.

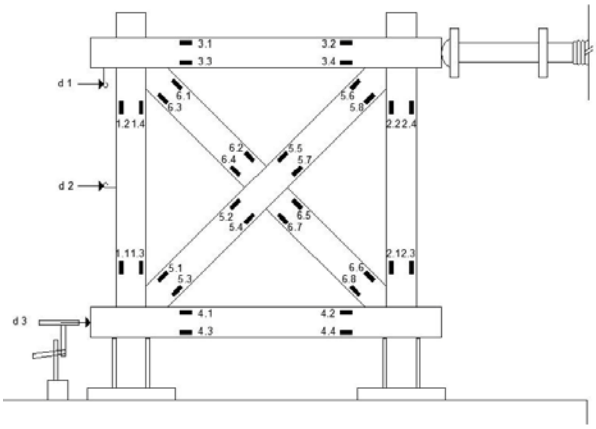


Figure 8 - Plan of the used instruments

Figure 9 - Test structure

4.2 Results

The results concerning the failure mode of each sample are presented in the table below (Table 1) as well as the corresponding failure force and superior displacemente (measured with the d1 deformeter).

Table 1 – Resume of all the samples' failures

Sample	Failure mode	Failure force Superior displacement d1	Image
GM1	Fracture of the central joint of the compressed diagonal (instability) – half wood section	33,1 KN 23,6 mm	
GM2	Fracture of plumb line (tensile) – full section	37,8 KN 38,6 mm	
GM3	Fracture of the joint in the applied load – half wood section	36,2 KN 20,74 mm	

PA1	Crushing failure of the load's farthest application joint – half wood section	60,3 KN 24,26 mm	
PA2	Fracture of the plumb line (tensile) – full section	62,5 KN 36,10 mm	
PA3	Rupture of the plumb line in the support – half wood section	45,1 KN 23,31 mm	

In the next figure (Figure 9) is represented the force/ displacement diagram. This displacement is the intermediate (d2) and its value is relative, meaning deducting the support's displacement (d3). This deduction is necessary in order to have a plainer understanding since the superior deformeter had a problem.

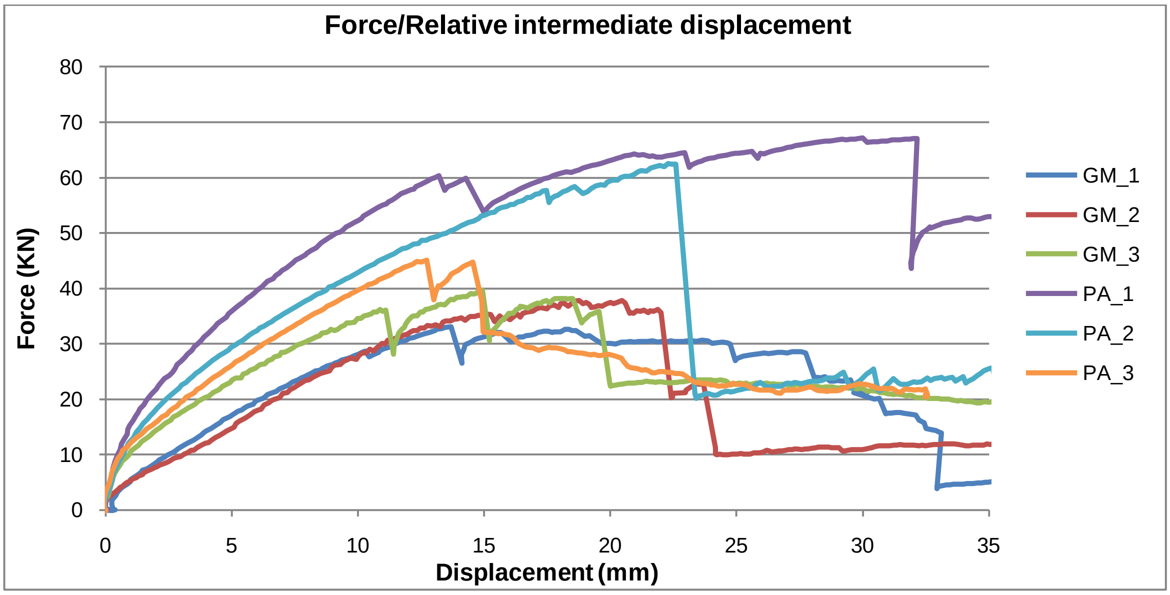


Figure 9 – Force/displacement diagram

There is a very similar behavior between GM1 and GM2 in the wooden cages. GM3 showed a higher stiffness, however, had an earlier starting fracture comparing to the others. In other words, GM3 was the first to experience an abrupt fall of the load value, that is to say, reached a lower displacement. GM2 presented a later failure, in other words, reached higher displacement values. This sample's failure occurred in a full section. Considering the failure modes, GM2 presented a more abrupt failure, while the others even regained a little bit stiffness. Regarding the failure forces, GM1 showed the minimum force value, which is the one whose diagonal had a lateral instability rupture developed by the central joint's failure (half-wood section).

The masonry walls had all similar behavior. PA1 presented a higher stiffness while PA3 had the lowest. PA1 and PA2 reached superior forces values, as PA3 suffered a premature failure caused by any local failure, probably in the support. PA2 reached the highest value of force and displacement.

As expected, masonry walls presented more stiffness than wooden cages. Nevertheless, the behavior of the wooden cage number 3 was very similar to masonry wall number 3. In fact, when comparing to the others wooden cages, GM3 owned a superior stiffness, as referred above, almost comparable to a masonry wall, especially in the initial phase.

This similarity suggests that the masonry does not have the expected influence concerning this type of loading. However, masonry does have a slightly contribution to the resistance of the whole set and the way it fractures, for instance, avoiding the lateral instability. The presence of masonry was probably more important to vertical loads. Nevertheless, the average effect of the masonry was quite substantial in the stiffness' walls and resistance.

The models that achieved superior failure forces were GM2 and PA2, corresponding to fractures occurred in full sections which were not as fragile as the half-wooden ones. Besides, these models registered the latest (within each group) and most abrupt ruptures and correspond to the same failure mode (plumb line in tensile).

In addition, wood beams from the same lot used in the other tests were used to determinate the modulus of elasticity in order to design the wood properly.

5. NUMERICAL MODELING

A linear elastic numerical model was elaborated, seeking to reproduce the structural behavior experimentally measured. The trial models for wooden cages are specified in Table 2.

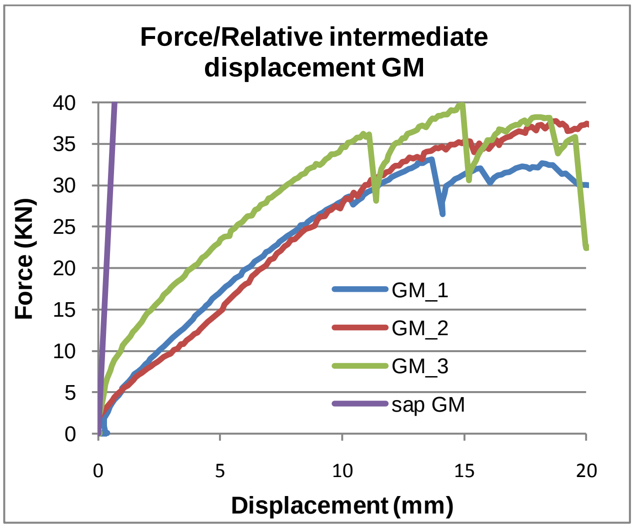
Table 2 Different models tested in the program

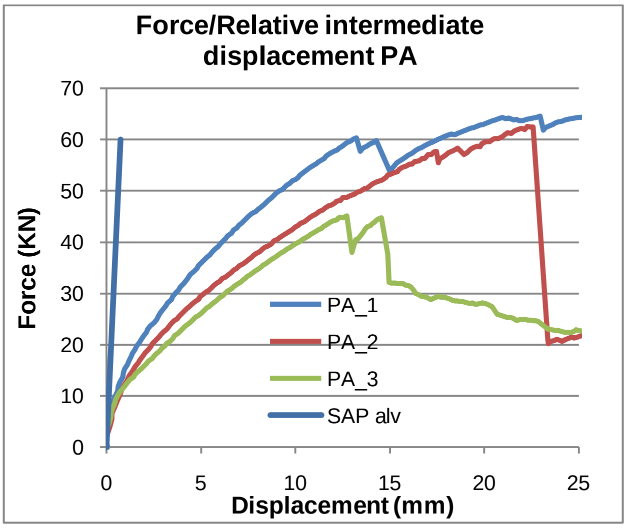
Wooden cage (GM)			
Model	**E (GPa)**	**Connections**	**d (mm)**
1	10,6	All joints encastered	1,1
2	10,6	Hinged diagonals	1,1
3	10,6	All joints hinged	1,2

4	**10,6/5,3**	**Hinged diagonals with E/2**	**2**
5	10,6	Compressed diagonal hinged with E	2,4
6	10,6/5,3	Hinged diagonal with E/2 and Support	2,1

The model 4 was chosen because it appeared to be the most appropriate as it better reproduces the constructed samples concerning the connections. The diagonal connections to the structure were weaker than the others because they only settle on the structure and are reinforced with nails so they are considered as hinges.

The masonry was considered to be a homogeneous material equivalent with the reduced properties. Thus, shell bi-dimensional elements with a 400 MPa elasticity module were used. The masonry walls were studied in order to select which elements to use in the modulation (triangular or quadrangular) and its refinement, where the displacement value measured was the same. Therefore, it was used a model (model 7) to modulate a masonry wall which consisted in a wooden skeleton of model 4 and where the masonry was defined with quadrangular elements.





The numerical models selected to represent the wooden cage and the masonry wall, present a higher stiffness compared to the models tested. They present only a tangency for an initial period, corresponding to the linear elastic stage. This tangency was only achieved for very low load values.

Furthermore, the experimental forces were calculated, using the values of the strain gauges. In order to compare the forces obtained by the model, strength values of 1/3 the failure force were chosen in order to ensure that they were within the elastic interval.

Therefore, the forces obtained by the numerical models described above were compared with the numerical model plus some variations, such as the including restraint connections in all joints and using of hinged supports. In conclusion, concerning strength, the numerical models constituted by hinged better represented the tested samples.

Regarding axial forces, the numerical model was overestimated in the tensile diagonal by the sample tests. In fact, this element does not absorb significant tensile, since their joints easily disconnect from the structure during the tests. Considering bending, the numerical models were generally underestimated by the experimental results, suggesting that these numerical models do not represent the real behavior observed properly.

6. FINAL CONCLUSIONS

Observing the obtained diagrams in the experimental tests, each module behavior was related with its failure mode. Furthermore, it was found that a higher failure force was associated with rupture profiles in filled sections. On the other hand, the half-wood sections were more fragile. A stiffness decrease in the elements endurance was observed since the beginning of the tests. This feature can be explained by the eventual occurrence rotations of the whole structure during testing, which have not been measured.

Nevertheless, the presence of masonry contributes to the increase of the wall's resistance and stiffness, for the type of actions previously studied.

Regarding the numerical models, the most fitted to the cage behavior was the one constituted by a hinged supports and where the diagonal was defined by hinged elements significantly reducing bending strength. Considering strength, the axial force is overestimated by the experimental tests in the tensile diagonal whereas the bending moment is underestimated in almost all elements. These proves that this structure is far too complicated and complex to be represented by a linear elastic analysis.

Finally, this study intends to be part of a rehabilitation context of "pombalino" buildings, allowing the designers to achieve data that can help the numerical modeling in which these structures are analyzed. This project expects to support the development of a more extended investigation program, through an experimental evaluation and a numerical modeling of front walls, approaching the analysis and seismic strengthening of the "pombalino" buildings.

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