# Seismic Analysis of a Heritage Building Compound in the Old Town of Lisbon

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## ABSTRACT

Architectural heritage is a key aspect to modern societies due to cultural and economical aspects. Besides a part of the history, tourism and leisure will be a major industry in the 3rd millennium and the existence of a monument or a monumental compound is often a key attraction of cities and countries.

This paper presents a set of numeric analyses regarding a compound of buildings, the "Pombaline" downtown buildings, from the 18th century in downtown Lisbon, Portugal, built after the 1755 earthquake. The buildings were made with a composite wood-masonry structure.

The following issues are addressed in the present study: (a) a historical survey, where the type of "Pombaline" construction was investigated; (b) an inspection of the actual condition of the selected compound; (c) a preliminary study of an isolated building where the finite element method was adopted for the analyses, introducing non-linear behaviour of the materials to simulate the structural damage; (d) a study with the complete building compound; and (e) a methodology for an adequate approach towards remedial measures in historical city centres.

## INTRODUCTION

The high cultural value of the buildings in the Pombaline area, completely rebuilt after the earthquake of 1755 and named after the Marquis of Pombal, and their actual state of structural conservation were the main reasons for carrying out the present project. For this purpose, a building compound near the famous square on downtown Lisbon, Praça do Comércio, was selected. The compound is located between the streets of Prata (Silver), Comércio (Commerce), Fanqueiros (Drapers) and Alfândega (Custom-House).

All the Pombaline buildings were built together in a block or compound. The external walls of the buildings, built in stone masonry made of limestone units and lime mortar joints, were linked to an internal stiffening structure of wood made in oak or holm-oak. The typical thickness of the external walls is 0.90 m, in the ground floor, with decreasing thickness with the elevation of the building. Additionally, internal walls, perpendicular to the external walls and with a thickness of about 0.50 m, could be found. These internal walls were built from the ground floor up to and beyond the roofs, without any opening. Such stone masonry walls had the purpose of dividing the space and preventing fire propagation (see Fig. 1).

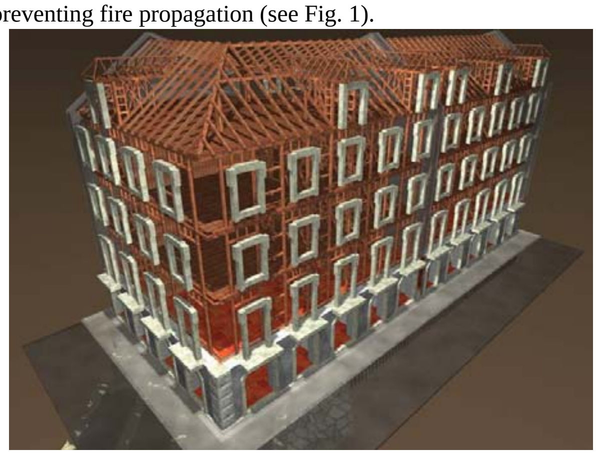


Fig. 1. Virtual reconstruction of the Pombaline structures, (Silva, 1999)

The ground floor structure was built in stone masonry. Besides the external walls, it was possible to find barrel vaults, carefully laid in dry stone masonry, or crossed vaults, made from clay brick masonry. Walls, arches or columns in dry stone masonry support these vaults. This system provides large stiffness to the structure in its base and, again, stops fire propagation from the retail area (ground floor) to the rest of the building. The infill of the vaults was recycled, using the loose material from the ruins of the earthquake of 1755, with the purpose of levelling the floor surface.

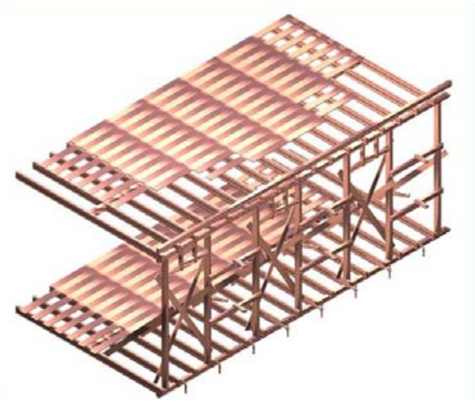
### The "New" Anti-seismic System

The usage of the composite wood-masonry walls, aimed at an efficient anti-seismic system. For this purpose, different wall types have been proposed using regular geometry in plan and constant height in elevation for all the buildings in the same block, namely: (a) the cage walls ("gaiolas"), that added a timber structure to the external masonry walls; (b) the transverse walls ("frontais"), formed by a timber truss filled with clay bricks and mortar; (c) and the non-load bearing walls ("tabiques"), built using a light timber wall with small thickness.

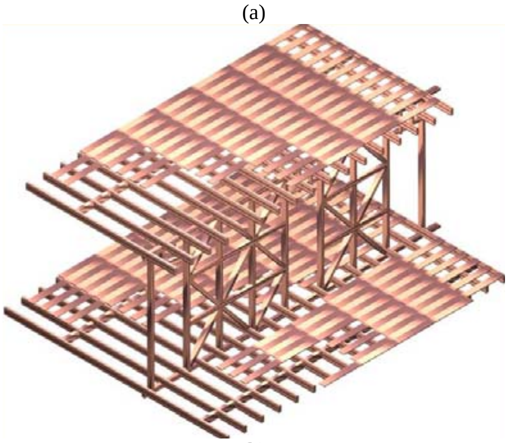
The "gaiolas" were solid structures placed as a backing structure of the external masonry walls, built by a group of timber elements denoted as "frechais". These elements had a rectangular cross section, $0.14 \times 0.10$ m$^2$, separated about 0.05 m from the internal side of the stone masonry walls (see Fig. 2a). The "frontais" walls had a construction similar to the "gaiolas", with differences in the execution of connections and bracing elements (see Fig. 2b). The "frontais" were placed in orthogonal directions, together with the "tabiques" walls, for the division of the interior compartments.

Both "gaiolas" and "frontais" included the bracing crossed timber elements. The empty spaces of the walls were filled with rubble masonry made of small stones and ceramic elements recovered from the ruins of the earthquake, assembled with lime mortar. In the end, the walls were rendered to hide the composite nature of the walls.

It was believed by the original designers that the introduction of these composite walls resulted in buildings with enough strength and energy dissipation capacity needed to resist the seismic actions, without suffering considerable damage or collapse.



(a)



(b)

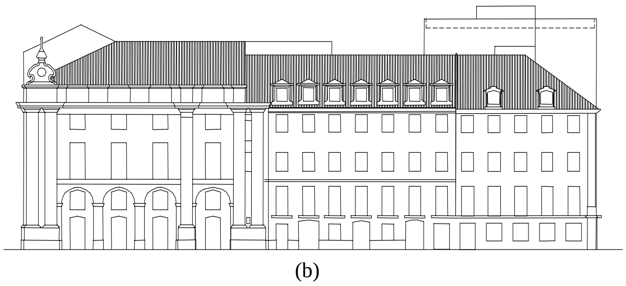
Fig. 2. Load-bearing structure: (a) external cage walls; and (b) internal walls

## DESCRIPTION OF THE SELECTED COMPOUND

Information on the date of construction for the compound studied in the present work is not available. The compound has a rectangular shape in plan, totalling $62.5 \times 43.5$ m$^2$, with one corner turned to the square (Praça do Comércio), in which an arcade is found (see Fig. 3).

Through time, successive and occasional modifications occurred in the compound. The introduction of new materials, with mechanical behaviour considerably different from the original structure, resulted in uncontrolled changes in the original structural system, possibly decreasing its strength and capacity to dissipate the energy associated with seismic actions.

(a)



(b)

Fig. 3. Selected block for study: (a) view of the Square of Comércio; and (b) main façade

A survey of the actual condition of this compound, (Ramos, 2000), indicated that:

More than 80% of the structure of the buildings that compose the block suffered changes. 54% of these are major changes, modifying the original structural system;

New materials (steel and reinforced concrete) and older materials (stone masonry and timber truss) co-exist;

Bracing elements were removed in several parts of the block, meaning that the ability of the structure to resist horizontal forces is compromised;

Openings in the firewalls that separate the buildings were made. No fire-doors have been installed to stop fire propagation between adjacent buildings. The staircases are also not provided with fire-doors.

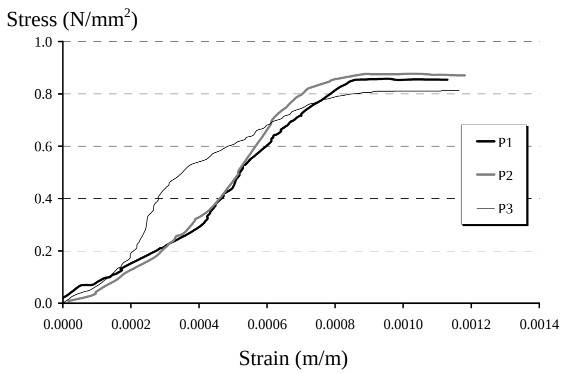
## EVALUATION OF THE CONTRIBUTION OF THE INTERNAL WALLS

Given the complexity of the selected compound in terms of geometry and load-bearing elements and its large size, a simpler model was analyzed first. For the purpose of discussing the composite behaviour of the external and internal masonry walls, it was decided to adopt a model of an isolated typical "Pombaline" building for non-linear analysis. This analysis will allow to define a modelling strategy for the large compound, namely with respect to the inclusion of the low stiffness "frontais" walls.

The elastic properties have been obtained from experimental data, (Ramos, 2002), resulting in a Young's modulus of 1000 and 50 N/mm$^2$, for the external walls and for the internal walls, respectively. The Young's modulus of timber and concrete were assumed equal to 10 000 and 30 000 N/mm$^2$, respectively. The Poisson coefficient for all materials was assumed equal to 0.2.

Three specimens were removed for testing from a gable masonry wall. The specimens had average dimensions of $0.70 \times 0.75 \times 1.40$ m$^3$ and were tested under uniaxial compression, see Fig. 4. The response is very ductile with an average uniaxial compressive strength of 0.85 N/mm$^2$.

(a)

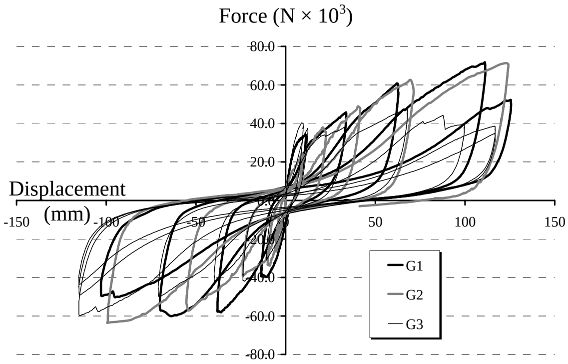



(b)

Fig. 4. Mechanical characterization of the external walls under uniaxial compressive loading (three specimens): (a) typical aspect of the specimens; and (b) obtained stress–strain diagrams.

For the internal walls, three specimens were also removed for testing. The specimens had average dimensions of $2.58 \times 3.46 \times 0.21$ m$^3$, and were tested under combined constant vertical loading and cyclic horizontal loading (as shear walls), see Fig. 5. The response is again very ductile and features enormous energy capacity deformation, even if the stiffness and strength are rather low.

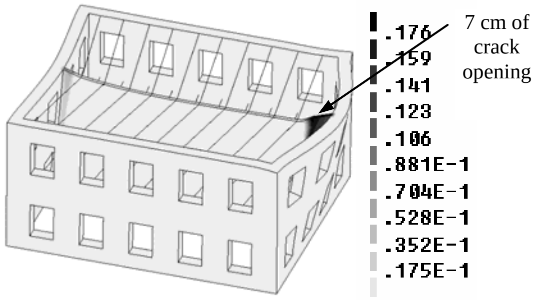
(a)



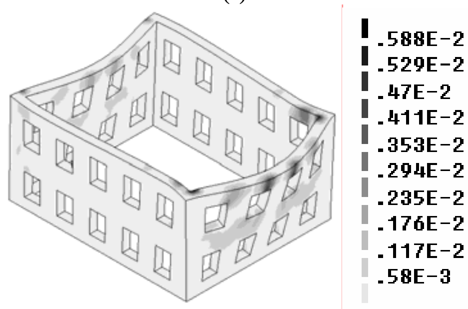

(b)

Fig. 5. Mechanical characterization of the internal walls under combined in-plane vertical and horizontal loading (three specimens): (a) typical aspect of the specimens; and (b) obtained horizontal force–displacement diagrams.

With the previous results, two numeric models of a typical building were prepared, with and without the interior "frontais" walls (see Fig. 6). Given the size of the model and the low strength of the material, the seismic analysis was carried out replacing the seismic action by the application of a set of horizontal loads proportional to the weight of the structure. This static analysis method can be easily coupled with non-linear material behaviour, according to a fixed smeared cracking model, e.g. Rots (1988). For this purpose, a zero tensile strength was adopted. More sophisticated models can be used, e.g. Lourenço (2000), but in situ testing would be required to find the necessary experimental data. All the numeric calculations were carried out with the finite element method, using the program DIANA (DIsplacement method ANAlyser), version 7.2, TNO (1999).




(a)




(b)

Fig. 6. Numerical cracks patterns (maximum principal strains) for a seismic load parallel to the long Pombaline walls: (a) model with internal wall and timber floors; and (b) model without internal wall and timber floors. The results are shown for the load factor required by the Portuguese code

Conclusions from the results obtained in the numeric analysis, Ramos (2002) and Ramos and Lourenço (2004):

Collapse modes in the stone walls were mostly related to out-of-plane failure mechanisms;

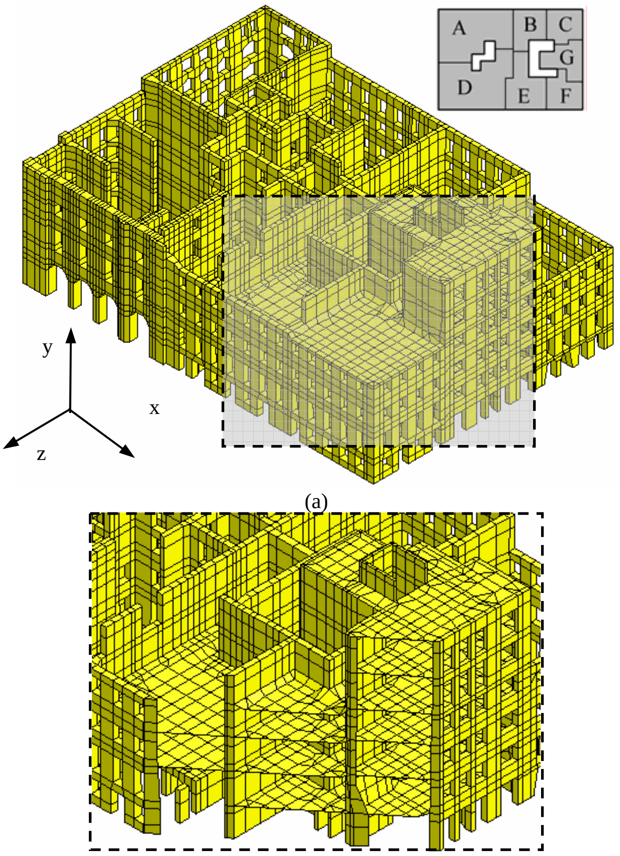
The base reaction of the internal wall is just 10% of the shear forces, when the seismic acts along the direction of the "frontais" walls. When the seismic action acts perpendicular to the internal walls, the contribution of the internal walls to resist the seismic action is negligible;

Modelling the building without the internal walls and timber floors is conservative, in the study of the seismic vulnerabilities of this type of structures.

In addition, the quality of the timber connections after two hundred years must be questioned, particularly in the vicinity of the external walls, due to high moisture contents. For the compound to be studied in the next section, the structural survey indicated that most of the internal walls have been removed due to questionable modifications. Therefore, for the full analysis of the compound, internal walls and timber floors have not been included in the model, being replaced by equivalent static loads.

## ADOPTED NUMERICAL MODEL

The full model of the compound includes different structural elements, see Fig. 7, namely: (a) stone masonry walls with mortar joints; (b) columns and arches in dry stone masonry; crossed vaults on the ground floor; (c) columns of reinforced concrete in the building G – higher body with T form; and (d) floor slabs in reinforced concrete.




(a)

(b)

Fig. 7. Numerical model of the full building compound

It is important to stress that timber floors were not considered in the model and the hypothesis of rigid floor diaphragms in the buildings was not adopted. The reason for this hypothesis is the fact that most floors are not well preserved with respect to the connections of the floor beams with the masonry walls. The roofs were also not included in the model, being its action replaced by equivalent static forces.

All the degrees of freedom of the nodes belonging to the cross section at the base of the buildings were fully restrained. Such model assumption is normal for linear elastic analysis of modern buildings with adequate footings. In the present case, the existence of high bending moments at the foundation is not possible, due to the material non-linear behaviour adopted for the masonry walls.

The full model is rather complex from the geometrical and material points of view, comprising 8 820 elements with 57 267 nodes, totalling approximately 160 000 degrees of freedom. One vertical load combination and four load combinations of horizontal actions were applied, associated with the seismic action acting along the main directions of the block. These load combinations analyses were carried out using non-linear material behaviour.

## Results for Vertical Loading

The minimum principal (compressive) stresses obtained in the analysis for the case of vertical loads are illustrated in Fig. 8. Here, it is noted that compression is negative as usually adopted in finite element analysis. The average value of the compressive stresses at the base of the buildings is around 0.5 N/mm$^2$, but stress peaks of 1.5 N/mm$^2$ can be encountered, namely in masonry columns. Such peaks can be considered acceptable as it is expected that the columns are made of good quality masonry. In the areas where reinforced concrete columns exist, the stress peaks reach 5.7 N/mm$^2$, which is a notably high value in comparison with the average stresses in the masonry walls. On the other hand, cracking is rather limited at this ultimate limit state, being more severe at the vaults in the base of building A, with a maximum crack width of 0.3 mm. Therefore, it seems that structural changes (introduction of structural elements with stiffness and weight very different from the original construction, and removal of the composite internal walls) did not affect significantly the structural performance of the block with respect to vertical loading.

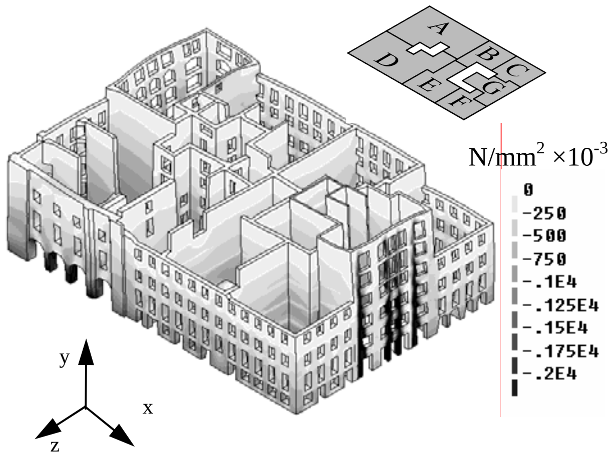
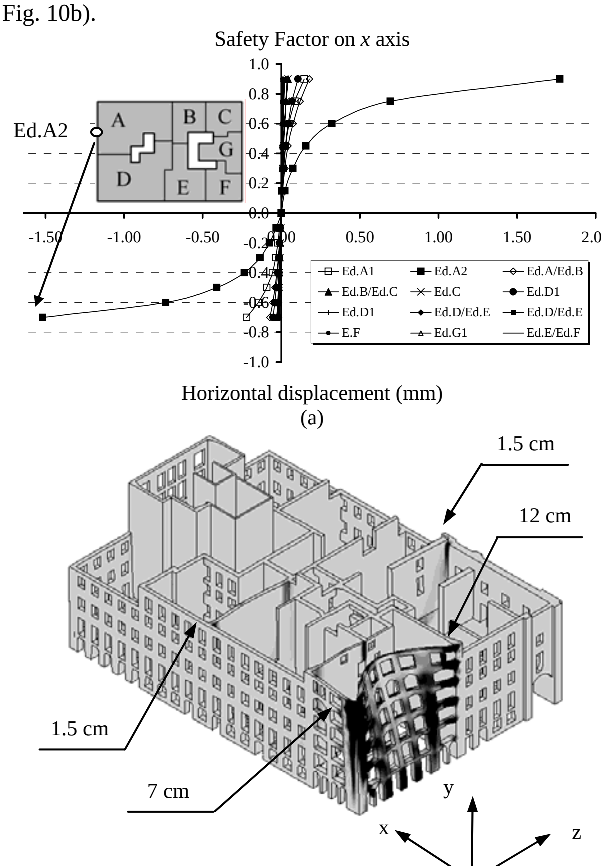



Fig. 8. Compressive stresses for vertical loading, in the complete block and in a detail of the reinforced concrete columns (values in N/mm$^2$ × 10$^{-3}$).

## Global Seismic Analysis of the Compound

Through the non-linear seismic analysis, it was possible to verify that, in the full model, the global safety factor is below the value of 1.5 required by the Portuguese code (see Fig. 9a and Fig. 10a). Collapse of the buildings occurs due to out-of-plane failure of the external masonry walls (see Fig. 9b and Fig. 10b).

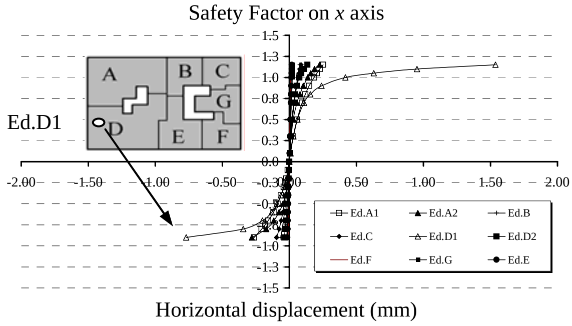



(a)

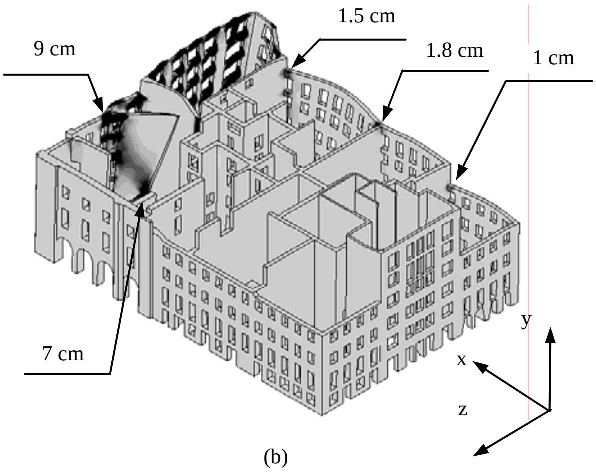
(b)

Fig. 9. Global behavior of the block for seismic action along *x* direction: (a) force-displacement diagrams; and (b) cracking pattern for the seismic action +*x*

The maximum load factor was approximately 0.7 and 0.9 for seismic action along the *x* and *z* directions, respectively. For the analysis in the *x* direction, collapse is governed by building A. For the analysis in the *z* direction, collapse is governed by building D. It is also stressed that the responses are non-symmetric as shown in Fig. 9b and Fig. 9b.




(a)

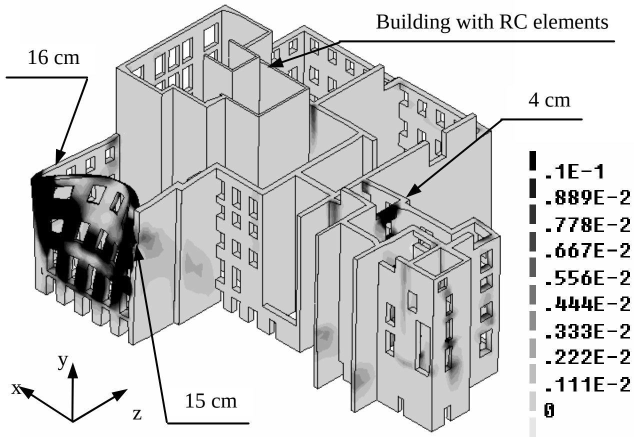



(b)

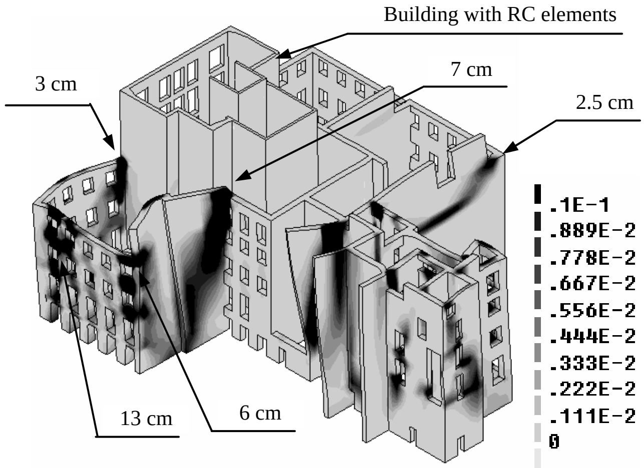
Fig. 10. Global behavior of the block for seismic action along *z* direction: (a) force-displacement diagrams; and (b) cracking pattern for the seismic action –*z*

In order to calculate the safety factors ($S_f$) of the different buildings, a second model was prepared, including solely the group of buildings and walls which exhibited no significant non-linear behavior up to collapse of buildings A and D. This consisted mostly of buildings C, G and F, see Fig. 11. This figure shows the results of the new analysis for the seismic action along axes *x* and *z*, in terms of deformed meshes and cracking patterns.

In this analysis with a reduced model, it was also possible to observe the difference between the stiffness of the buildings with added elements in reinforced concrete (taller building in the compound) and the buildings with stone masonry and timber floors. The difference is rather significant and seems to result in premature detachment between the two types of buildings (see Fig. 11b).




(a)




(b)

Fig. 11. Results for the reduced model: (a) seismic action in the direction +*z*; and (b) seismic action in the direction +*x*

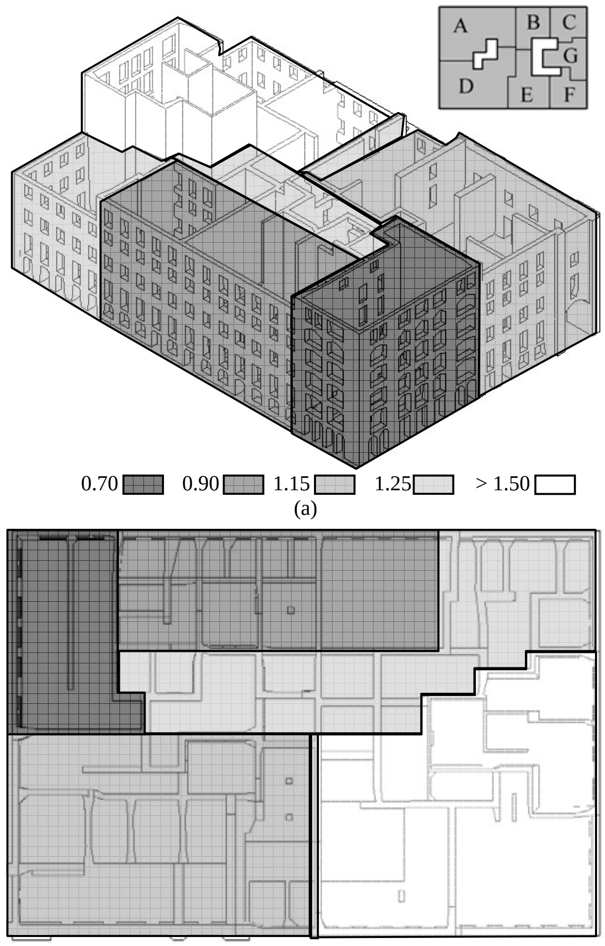
Through the seismic analyses of the full and reduced models, it can be concluded that the zones more sensitive to seismic actions correspond to the buildings without rigid floors. Obviously, the analyses shown were carried out assuming that the connections between the floors and the masonry walls are not damaged during the seismic action, which is debatable. The obvious conclusion seems to be the need for a tie at each floor level, with possible strengthening of the masonry piers between windows, or simply a tie at the roof level, if the resulting safety level is adequate.

Assuming that highly intrusive solutions are not acceptable, in the light of modern intervention principles, it seems advisable to tie the structures using steel elements placed horizontally or using the timber floor logs, adequately connected with the masonry walls, for this purpose.

## Seismic Vulnerability of the Compound

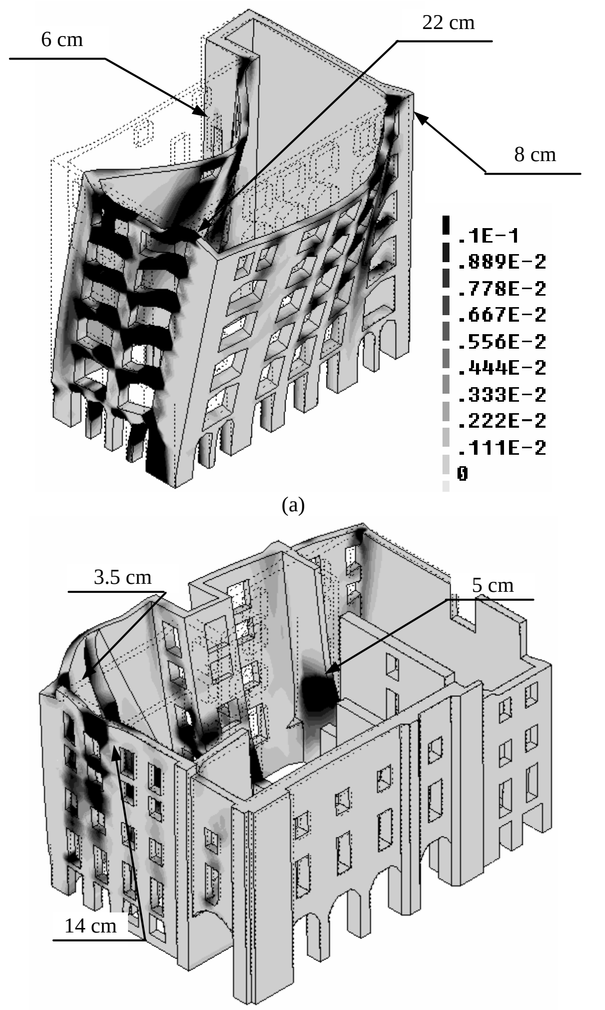
From the successive numerical analyses it was possible to assess the seismic vulnerability of the different zones of the structure. The obtained risk map distinguishes the most vulnerable zones to seismic actions, which is a valuable guide for the public authorities and for the decision of possible interventions for strengthening.

Fig. 12 shows the map of safety factors where it can be observed that the zone of the corner of the building A corresponds to the smallest seismic factor (0.70). Then, buildings A and B exhibit a value of 0.90 and building D exhibits a value of 1.15. Building C, with minor structural interventions with respect to the original conception, obtained a safety factor equal to 1.25. The zones with a higher safety factor are associated with buildings that suffered more intrusive modifications, namely, buildings E, F and G.



(a)
(b)

Fig. 12. Map of safety factors of the block: (a) perspective; and (b) plan



(a)
(d)

Fig. 13. Results for the isolated models: (a) building A; and (b) building D

## The "Block Effect"

To study the "block effect", i.e. the influence of the group of buildings in the seismic behavior of the individual buildings that compose the block, two different analyses were carried out to study the behavior of two buildings separately. These buildings are the building A, corresponding to a zone with larger structural damage and smaller safety factor, and the building D, corresponding to a zone with structural configuration different from the traditional "Pombaline" construction, even if mostly original.

Fig. 13 shows typical results of the analysis carried out with the isolated models.

The seismic safety factors for both isolated models and the load-displacement diagrams, seem to indicate that the "block effect" is beneficial for the buildings, as observed in Table 1. This means that the analysis can be safely carried out with isolated buildings, which reduces the effort and time to great extent. Nevertheless, it must be stressed that the difference in the results are rather large and, if the isolated building analysis indicates a unsafe condition, it may be suitable and economically justifiable to refine the analysis using the full compound.

TABLE 1: Safety factors of the buildings

| Building | Isolated model | | | Full model | | |
|---|---|---|---|---|---|---|
| | $Sf_x$ | $Sf_z$ | Sf | $Sf_x$ | $Sf_z$ | Sf |
| A | 0.50 | 0.75 | 0.50 | 0.70 | ≥0.90 | 0.70 |
| D | 0.65 | 0.95 | 0.65 | >>0.70 | 0.90 | 0.90 |

## PROPOSED METHODOLOGY

Based on the above, a simple methodology is proposed next, with the objective to deal systematically with the issue of substantial structural alterations in the blocks of historical city centers in seismic areas. Such methodology consists of the following steps, to be carried out along a reasonable time span by the local / national authorities:

Carry out a geometrical survey and identification of the urban compound;

For each of the blocks of buildings, perform a survey of the existing structure, a characterization of the structural materials and a classification in categories of existing structural changes. The following criteria to classify the modifications are suggested: (a) Buildings with significant modifications – structural changes in more than 50% of the original volume of the building, with or without extra floors; (b) Buildings with moderate modifications – structural changes between 20% and 50% of the original volume, with a maximum of one extra floor; (c) Buildings with minor modifications – structural changes in less than 20% of the original volume, without extra floors;

For each block, construct a finite element model and perform the analysis, determining the shear forces in the structural elements and the seismic vulnerability of the buildings. With this information, set-up a plan and schedule with the definition of priorities for retrofitting. If the owner does not carry out the retrofit according to the schedule, the municipality takes over this task and claims the costs from the owner;

Finally, the survey and structural model of the complete block should be provided to the owners. Any construction works in a building should only be approved if it is demonstrated by the owner that the works do not have a negative contribution to the safety of the block.

The last years witnessed large developments in survey and characterization tools. A comprehensive set of tests, devices and techniques, mostly non-destructive or low invasive, are available, enabling the gathering of the data required to construct and validate structural models. These methods are of interest not only before the actual works in the construction but also during and after the works, as modeling of the structure is only one specific phase of the overall process.

The preparation, modeling and analysis of historical buildings can be extremely complex and time consuming. The diversity of the structural schemes together with the use of different materials, results in the need to adopt simplified numerical models of the constructions, in order to make the analysis feasible. The obvious difficulty is the trade off between the ability of the model to accurately represent the structural behavior of the construction and the level of simplification (and inherent costs).

It is not realistic to use detailed models of walls and connections in large scale analysis, not only because the geometric and material properties of the constituents are difficult to characterize but also due to the time / economy constraints. For most large-scale analyses, it is acceptable to model both regular masonry and rubble masonry walls assuming a continuum homogeneous material.

From the experience gained in the work described here, the cost estimate of Table 2 can be made for setting up a priority plan in Lisbon. This estimate seems quite reasonable compared to the municipality annual budget and the fact that the estimated costs are to be spread over a number of years. The amount per building, around 27,000 Euro, seems quite reasonable as well, taking into account the high real estate prices in the area, not to mention its importance as architectural heritage. Nevertheless, this value must be considered in the specific economical context of each historical center and local conditions.

TABLE 2: Costs involved in survey and characterization and modelling ($10^3$ €)

| Task | Cost per building | Cost per block | Total |
|---|---|---|---|
| Architectural survey | 10 | 70 | 4200 |
| Structural survey | 9 | 63 | 3780 |
| Materials characterization[a] | - | - | 250 |
| Foundation survey | 2 | 14 | 840 |
| Geotechnical survey[b] | - | - | 400 |
| Modelling | - | 30 | 1800 |
| Total | - | - | 11 270 |

a - Overall survey and sampling.
b - Using sampling and existing data.

## CONCLUSIONS

The analysis shown in the present paper, indicate that the safety of the Pombaline construction does not comply with existing codes. Therefore, owners and regulators must address the issue of retrofitting these structures.

As preventive measures, it seems advisable to tie the buildings with steel rods or to strengthen the timber floors, especially taking into account the connections with the masonry walls.

With relation to the need to model the full building compound for the safety analysis, there is some evidence from the results that the “block effect” is beneficial for the seismic behavior of the “Pombaline” downtown buildings. Therefore, the independent seismic analysis of each individual building, as it is usual in practice, is conservative.

Finally, a methodology to define the approach towards works in historical city centers in seismic areas is defined, including survey, analysis and definition of priorities of action. The cost of the proposed approach is also addressed.

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