Pombalino Constructions: Description and Seismic Assessment

Mário Lopes, Helena Meireles, Serena Cattari, Rita Bento and Sergio Lagomarsino

Abstract This chapter describes the *Pombalino* building structures built in Lisbon downtown and other parts of Portugal during the reconstruction after the 1755 earthquake, as well as their earthquake resistant features. In particular the importance of the *Gaiola Pombalina*, a tridimensional wood truss characteristic of those constructions, in the potential seismic resistance of these buildings is discussed. The effects in their seismic resistance of the architectural and structural changes to which these buildings have been submitted since the original construction, usually with negative consequences,, is also discussed. Some strengthening and advanced analytical modelling strategies for these buildings are also mentioned. Finally, the socio-economic feasibility of strengthening this construction is briefly discussed, as well as the importance of their preservation. To be noticed that the reconstruction of Lisbon is the first time in the history of mankind that a large town was built providing widespread seismic resistance to its buildings aiming at avoiding future tragedies of the same type.

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1 Introduction

Pombalino is the designation of the structural typology of the buildings built in Lisbon and other parts of Portugal during the reconstruction after the great Lisbon earthquake of 1755. The purpose of this chapter is to give an overview of the main characteristics of these buildings, their potential seismic performance and present some of the studies already performed on these buildings, including some advanced modelling techniques. In Sect. 1 a general description of this type of buildings is done, with emphasis on the earthquake resistant characteristics. In Sect. 2 the main experimental and analytical studies performed to evaluate the seismic resistance of these buildings are described. Section 3 presents some of the main changes that these buildings suffered during their already long existence and that also affect their potential seismic performance. Section 4 refers some of the main strategies and strengthening techniques to improve the seismic resistance of these buildings. Section 5 presents some advanced modelling techniques and examples of their application to *Pombalino* buildings, as well as the main results of a study at the level of an entire quarter. Section 6 presents a brief discussion on the social and economic issues that condition the feasibility of rehabilitation and strengthen of these buildings, necessary to deliver them in safe conditions to future generations but preserving their authenticity. Finally Sect. 7 presents some short notes on the historical and cultural value of these buildings.

1.1 Description of Pombalino Buildings

Lisbon and the south of Portugal were devastated by a magnitude Mw = 8.5–8.75 earthquake [1] with epicentre at southwest of the Algarve in 1755. It is estimated that more than 5 % of the population of the Lisbon region died. The reconstruction that followed was done with the concern of avoiding future tragedies by means of providing seismic resistance to the new buildings.

Of the several features of *Pombalino* buildings related with the purpose of providing seismic resistance, the most relevant and notorious is the *Gaiola Pombalina*. *Gaiola* is the Portuguese word for cage, as the *Gaiola* consists on a tridimensional wood truss that looks like a cage. The *Gaiola* is constituted by a set of plane trusses, called *frontal* walls, connected at the corners by vertical bars that belong to orthogonal *frontal* walls. Each *frontal* wall is constituted by a set of triangles, a geometry similar to the steel trusses of nowadays. Since the triangle is a geometric figure that cannot deform without variation of the length of the sides, in fact it is the only one, each *frontal* wall only needs to mobilize the axial force of its bars to resist to forces in any direction in its own plan. Therefore the

Fig. 1 *Gaiola* after removal of cover and masonry

connection between orthogonal *frontal* walls by means of common vertical wood bars yields a tridimensional truss capable of resisting forces applied in any direction. In general the space between the wood bars of the *frontal* walls is filled with weak masonry, and the surfaces are covered with a finishing material, therefore the *Gaiola* in general is not visible. Figure 1 shows photos of the *Gaiola* after removal of the masonry in a building recently demolished in downtown Lisbon, and Fig. 2 a *Gaiola* wall with the masonry filling.

Usually the *Gaiola* only develops above the top of the ground floor level in the interior walls. The façades and gable walls (between adjacent buildings) are usually built with ordinary rubble stone masonry, with some exceptions of better quality masonry, mainly at corners and some ground floor columns and walls. Their thickness may vary along the height, being between 0.60 and 0.90 m in most cases. The spandrel beams, which connect the masonry columns of the façades are of the same thickness of the columns below floor levels. When there are windows and not doors the spandrel beams extend above floor level but with a much smaller thickness, in order to allow people′s access to the windows. Very often the two parts of these spandrel beams were not built simultaneously, yielding a weak horizontal surface between them. The result is that those beams have a cross-section as shown in Fig. 3.

Some interior walls, with partition purposes only, called *tabiques*, are made of one or two sets of boards or small wood bars, are thinner than *frontal* walls and have much less resistance to horizontal loads than the *frontal* walls. Figure 4 shows a photo of the interior of one of those walls, after removing the cover.

The floors are made of wood planks (typically 2 cm thick) supported on perpendicular wood joists (typically 10×20 cm^2), which are supported on the exterior walls (more often on the façades) and *frontal* walls. In buildings of better quality the wood joists are continuous from façade to façade, in others they have discontinuities on the intersection with *frontal* walls. According to the original practice the horizontal wood joists of the floors should be properly anchored inside the façades, by means of iron anchors embedded in the masonry of the façades at floor level. The *frontal* walls should also be anchored inside the façades. However, there are doubts about the quality of those connections, as well as of their widespread execution.

Fig. 2 *Gaiola* wall with masonry filling [2]

The connections between the different wood bars of the *Gaiola* are done by means of iron nails and cuts on the wood bars in order that they fit in each other, as shown in Fig. 5.

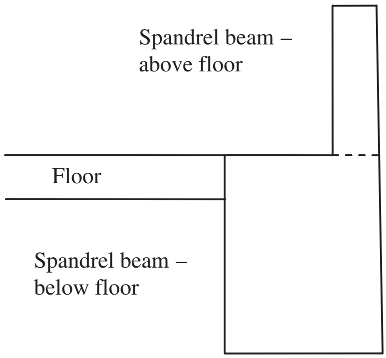

Fig. 3 Location and cross-section of spandrel beams

Fig. 4 Partition wall [3]

The pavements of the 1st floor (ceiling of ground floor) are usually constituted by masonry arches and vaults, as shown in Fig. 6, for two reasons: (i) create a barrier to fire, in order that possible fires on the ground floors do not spread to the upper floors, and (ii) to avoid that the soil humidity reaches the *Gaiola* wood structure above the 1st floor. At ground floor level, where the *Gaiola* structure does not exist, the arches and vaults are supported in the interior by masonry piers and walls and on the exterior by façades and gable walls.

Another characteristic of *Pombalino* construction was the standardization of the construction process, aiming at its widespread application to an entire city. For instances first the carpenters would built the wood truss, the *Gaiola*. After the bricklayers would come in and would add the masonry and the finishing to

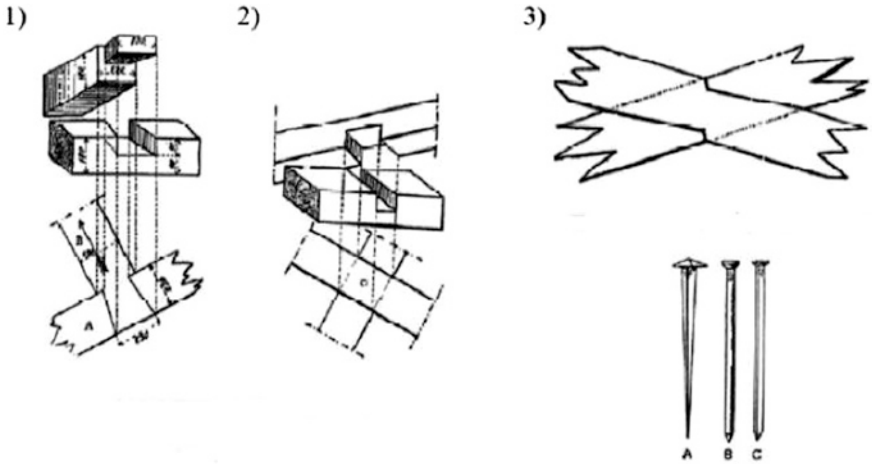

Fig. 5 Connections between wood bars of the *Gaiola* [4]

Fig. 6 Masonry arches and vaults at the ground floor ceiling [2, 5]

the *frontal* walls. The gable walls are common to both adjacent buildings (there were no expansion joints, as the flexibility of the wood pavements was enough to accommodate the effects of variations of temperature) and extend above the buildings to offer a barrier to fire propagation between buildings. Since the construction of the different buildings of each quarter was not simultaneous, there are vertical surfaces of separation of façades and gable walls, built at different periods. The standardization also extended well beyond individual buildings. The quarters of downtown Lisbon have a rectangular shape as they developed between parallel and orthogonal streets, and the buildings were all of the same height, comprising the ground floor, three upper floors and the attic. This way each quarter was constituted by a set of buildings of similar dynamic characteristics, yielding a better overall performance under seismic actions. A situation very similar to this, but with smaller buildings, can be found at the centre of Vila Real de Stº António, a southern town in Algarve also devastated by the earthquake of 1755.

In downtown Lisbon the water table is almost near the surface, as this zone is adjacent to the Tagus river. The soil is an alluvium of variable thickness, of approximately

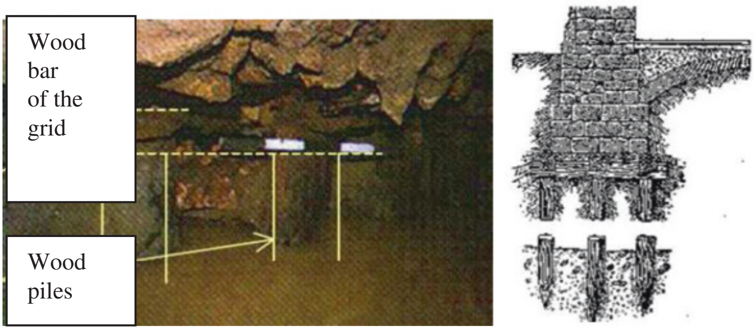

Fig. 7 Foundation scheme and piles [5]

20–30 m near the river and progressively reducing as the distance to the river increases, with a very weak load bearing capacity. Due to this the foundation system of *Pombalino* buildings is based on a tridimensional grid of wood bars on top of short length (around 1.5 m [6]) and small diameter (around 15 cm) wood piles, embedded on a large embankment made with the debris of the buildings destroyed by the 1755 earthquake and compacted by the piles. The embankment receives the loads from the structure through the wood grid and piles and distributes them by a larger area of the underlying alluvium, reducing the stresses at this level.

Figure 7 shows the constructive scheme and photos of the top of the piles at one building in downtown Lisbon, the BCP Museum. Figure 8 shows a schematic representation of a *Pombalino* building

2 Seismic Resistance

The seismic resistance of existing constructions received less attention than new constructions at the early days of modern seismic engineering, at the first half and middle of the twentieth century. In Portugal more attention was given to old buildings mainly from the decade of 1990 onwards. One of the first studies to assess the

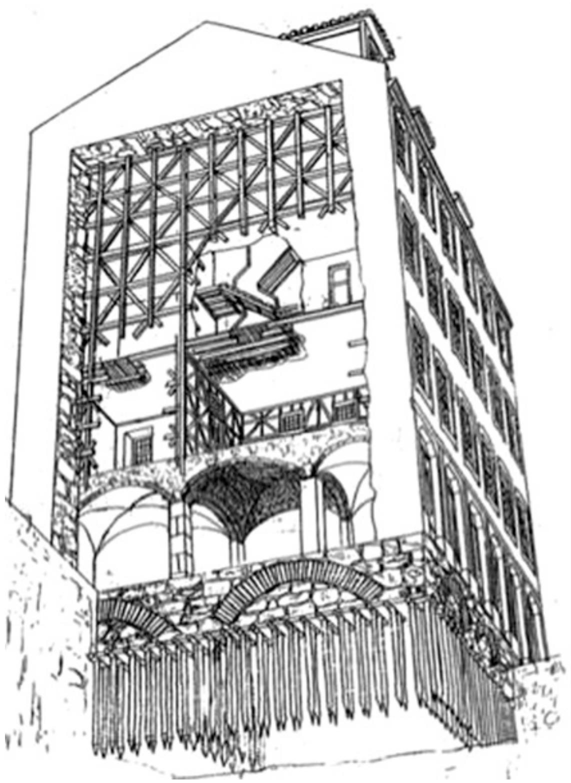

Fig. 8 Example of a *Pombalino* building [7]

Fig. 9 In-situ tests to rupture [8]

seismic performance of an old building in Lisbon focused on a *Gaioleiro* building [8], the type of buildings that were built after the *Pombalino* buildings during the late nineteenth and early twentieth centuries. This building type resulted of the progressive adulteration of their main characteristics of the *Pombalino* buildings, such as the absence of the diagonal wood bars of the *Gaiola*, weaker connections between elements and adding more floors. The studied arises from the opportunity to do tests to rupture, on site, on a building being demolished. In these tests part of the façade was used as reaction wall, in order to apply monotonic increasing horizontal forces in the plan of the façade strong enough to take the rest of the façade at that level to rupture. Figure 9 shows part of the façade (including instrumentation details) that was divided into two unequal parts: the smallest one, that was tested, and the largest one, that is stronger and was used as reaction wall. These tests allowed to find out the stiffness for small and large displacements, as well as the respective failure loads, for the element tested to rupture.

Several tests of the same type were also performed in interior walls. This allowed calibrating a tridimensional model of the structure, that, together with the knowledge of the elements failure loads, allowed to evaluate the seismic capacity of the building. The result indicated that the building would collapse for a seismic action of approximately 43 % of the respective code (RSA, [9]) prescribed seismic action, showing the tremendous weaknesses of this type of buildings. However, this conclusion cannot be extrapolated to *Pombalino* buildings.

In the early years of the 2000 decade Rafaela Cardoso [5] analysed with detail the model of a *Pombalino* building with ground floor, 4 upper floors and attic, with the numbers 210–220 of *Rua da Prata*, whose front façade is shown in Fig. 10 and whose project was available for consultation. It is thought that this building is a late *Pombalino*, probably of the early nineteenth century period, due to the fact that it has one more floor than usual. At this time the memory of the 1755 earthquake was starting to fade away and the strict construction rules imposed after the earthquake were being relaxed due to pressure of urban developers.

Fig. 10 Front façade of the building of *Rua da Prata* [5]

It was not possible to do an experimental characterization of the materials, therefore the structural model was based on the assumption that the materials would have average properties, estimated and calibrated from in situ and laboratory tests on specimens removed from other buildings or built in laboratory. Particular attention was devoted to the stiffness properties of the *frontal* walls, due to their relevance for the potential seismic performance of *Pombalino* buildings. Several models for the simulation of *frontal* walls were analysed [5]. In all cases each individual wood bar was represented by a linear bar hinged at both extremities, with the masonry between the wood bars represented by finite elements, as shown in Fig. 11.

The comparison between the models and the calibration of their stiffness properties was based on experimental results. Several experiments on *frontal* walls were performed at Laboratório Nacional de Engenharia Civil (LNEC) in Lisbon: (i) a *frontal* wall removed from a *Pombalino* building in downtown Lisbon, and carefully transported to LNEC, was tested under constant vertical loads and horizontal cyclic loads applied on top [10], and (ii) a set of pre-fabricated *frontal* panels tested also at LNEC [11]. Figure 12a shows schematically the dimensions of the full scale panel removed from a building and the applied loads, and Fig. 12b shows the panel after the test.

The comparison of analytical results with the results of these tests showed that the analytical stiffness always overestimated the respective experimental value. Three possible reasons for this observation were identified: (1) the connections on the extremity of the diagonal bars of the frontal wall, considered in the analytical

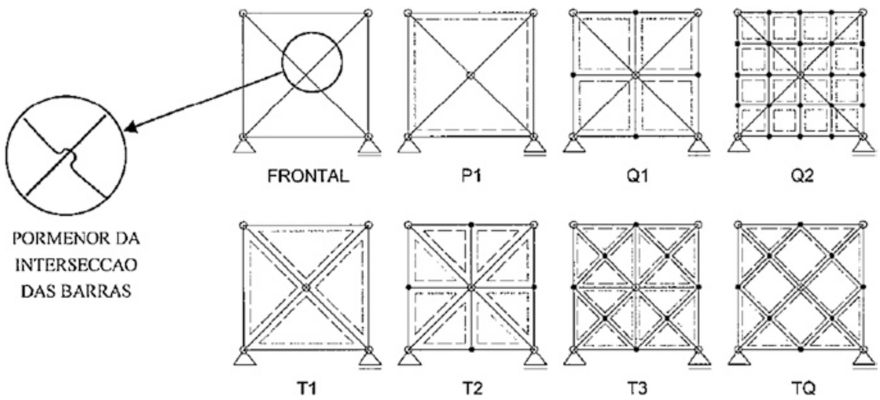

Fig. 11 Models for *frontal* walls [5]

models, could not transfer tensile forces; (2) the masonry filling contributed very little to the stiffness of the panels as it tend to detach from the rest of the panel when deformations increase; this conclusion was recently strengthened during a set of tests of panels similar to *Pombalinofrontal* walls, built and tested in the Laboratory of Structures and Strength of Materials of IST [12]. Even though, the detachment of masonry from the wood structure was clearly noticeable by eye sight at large displacements, as can be clearly seen in Fig. 13, this confirms the little importance of the characteristics of masonry for the stiffness under strong seismic actions; (3) the gaps between wood bars allow initial deformations before mobilizing the compressive strength of the diagonal of the *Gaiola*.

Figure 14a and b shows the existence of the gaps between wood bars in real buildings and Fig. 14c shows their influence on the force–displacement diagrams

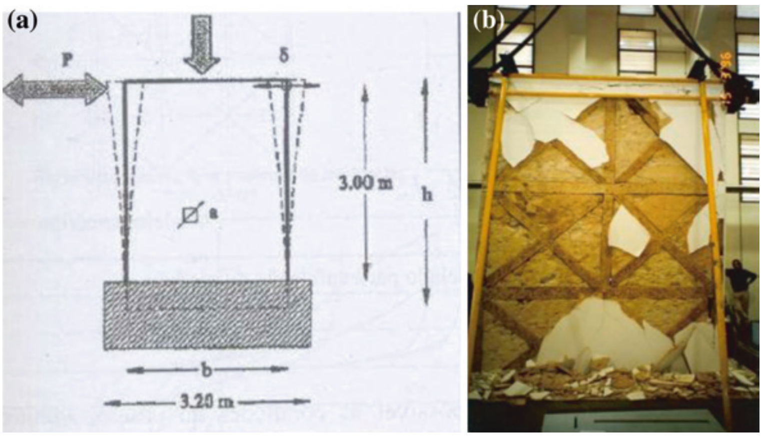

Fig. 12 Scheme of test and panel after the test [10]

Fig. 13 Detachment between the masonry filling and the *Gaiola* structure [12]

of the panels [5]. The comparison of the stiffness of the analytical models with the experimental stiffness, disregarding the low stiffness branch before closing of the gaps, led to the conclusion that the masonry and the tensile diagonal of the *frontal wall*should be disregarded (except at small displacement, of little relevance under strong seismic actions). From the range of values for the wood Young′s modulus given at EC7 [13] (8000–12000 MPa), the one that yielded the best match with the experimental results was the lowest value that was chosen for the analysis.

The tests on pre-fabricated panels at LNEC [11], referred above, also confirmed the conclusions about the influence of the gaps. These were diagonal compression tests on 1:3 scale models of *frontal* wall panels. Figure 15a and b shows one of the specimens and the loading shoes at top and bottom of the specimen. These tests also confirmed the detachment of the masonry filling from the surrounding wood bars of the *frontal* walls.

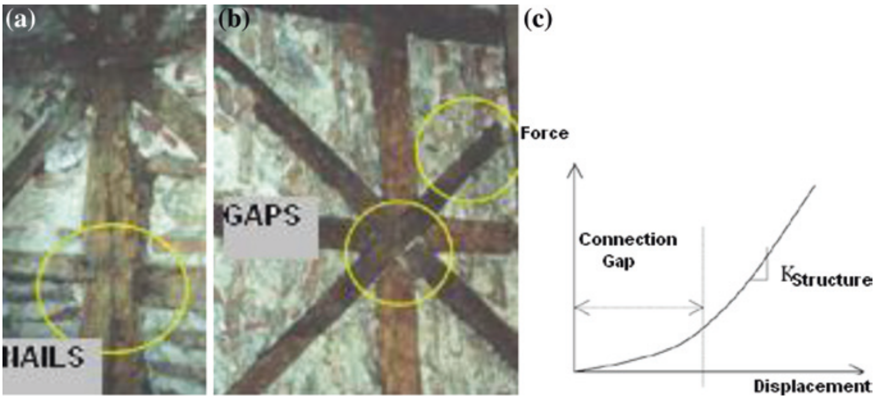

Fig. 14 **a**, **b** Connections with gaps and **c** influence on force–displacement relationship [5]

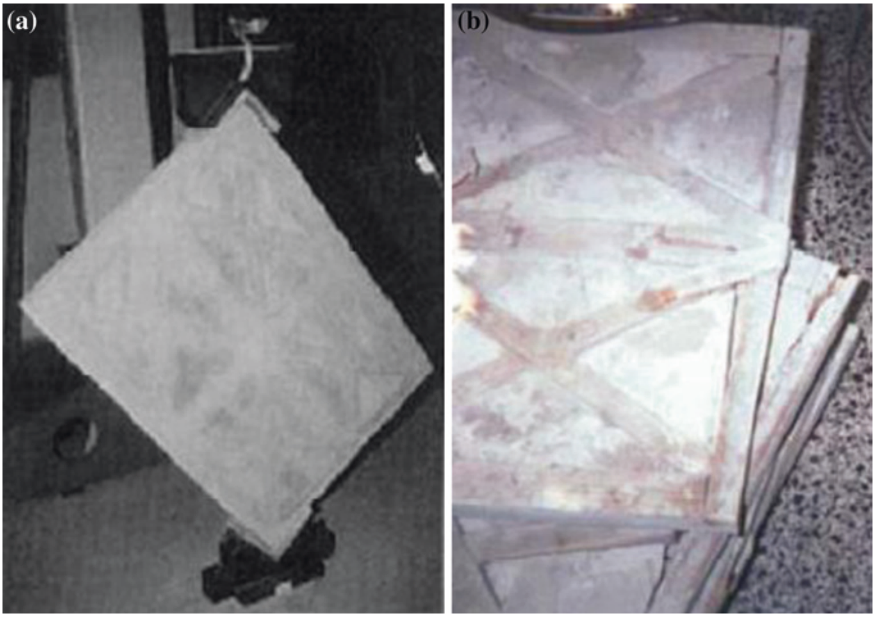

Fig. 15 **a** Panel built and **b** Panel after testing in laboratory [11]

This detachment was also observed in recent experimental studies where the influence of the masonry filling on the seismic behaviour of *frontal* walls was studied by means of the comparison of behaviour of *frontal* walls with and without masonry filling, built in laboratory [14]. The results showed that the masonry contributes to prevent buckling of the compressive diagonals, which starts at the middle section where the cross-section is reduced to half due to the crossing with the other diagonal, as shown in the details of Fig. 5. Figure 16a and b shows the specimens after testing. These tests have shown that the masonry increases the global stiffness, even though

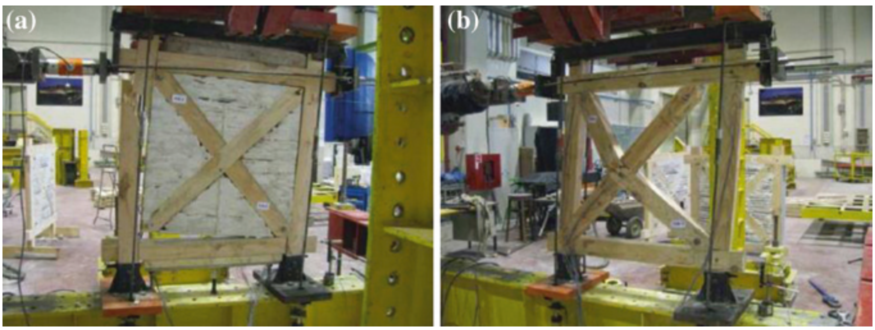

Fig. 16 *Frontal* walls: **a** with masonry and **b** without masonry [14]

for large displacements it detaches from the wood structure. However the good quality of execution and of the materials cast some doubts on the fact that this stiffness increase would also take place on real *Pombalino* buildings or if it would be more reduced.

The global nonlinear behaviour of the building analysed by Rafaela Cardoso [5] was considered by means of an iterative procedure based on a sequence of tridimensional linear analysis by response spectra. This option allowed to use current commercial software available for structural analysis (SAP2000 [15]), as it was intended to use a methodology and software that could be used in current design practice. More details about material properties and element dimension can be found in Sect. 5.2. Figure 17 shows the plan of the structure at the ground floor.

The façades and gable walls were modelled by shell finite elements. The contribution of the *tabique* partition walls was disregarded. The foundations were modelled as built-in supports.

At each iteration the connections that failed at the previous iteration, mainly between the *frontal* walls and the façades, were removed from the model of the next iteration, as it was assumed that after a wood bar was pulled-out from the masonry the tensile strength of the connection could not be recovered. As the building was a late *Pombalino* it was conservatively assumed that the connection of the *frontal* walls with the façades were weak ($f_{tensile} \leq 5kN$). It was assumed that a façade would fail in its own plan if all the columns would fail, which corresponds to admit a limited redistribution capacity. It was concluded that failure would take place by out-of-plane collapse of the façades due to sequential rupture

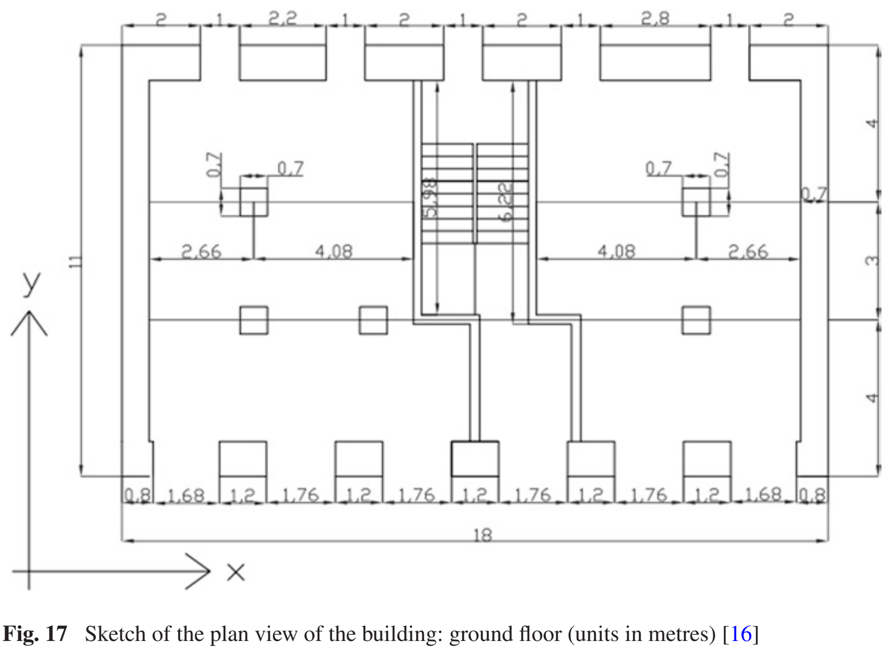

Fig. 17 Sketch of the plan view of the building: ground floor (units in metres) [16]

Fig. 18 Out-of-plane failure of façade walls [17]

of the connections, starting at the upper floor and propagating to the lower floors at almost the same seismic intensity, as illustrated in Fig. 18.

Rupture would take place for a seismic action of 40 % of the one prescribed for the Portuguese code RSA [9] for far field events of high magnitude, to which corresponds a value of PGA = 0.18 g (it was assumed a relative damping factor of $\xi = 10$ % and a behaviour factor $q = 1.5$) and a response spectra rich in high periods. However if this collapse mode was prevented because the connections were better than assumed, or had been strengthened, collapse would be triggered by failure of the ground floor columns and walls (base shear mode) but at a seismic action slightly above 100 % of the code prescribed seismic action. It is likely that many original *Pombalino* buildings would have a seismic resistance above this level, as they had one floor less and better connections. Even though there may be doubts about the strength of the connections, which should be verified for each building in strengthening projects, this result is remarkable for buildings built more than 200 years ago.

It should also be noted that the methodology is conservative, since the analysis is not done in time domain and in each iteration the spectra is the initial one, as if the earthquake would start again. However, opposed to what happens in most of Europe, where near-field events are relevant, in Portugal for many buildings as most Pombalino buildings, far-field events, such as the 1755 earthquake, condition seismic design as the zone of higher spectral accelerations extends more to the lower frequencies. These events are associated to much higher durations than near-field events, reducing the conservatism associated to the methodology used for the analysis.

The conclusion regarding the seismic resistance can be extrapolated to the *Pombalino* buildings of the present, as long as they have not been altered after the original construction, as it is thought that the *Gaiola* still continues intact in

Fig. 19 Evidence of nowadays excellent conservation state of *frontal* walls

the original buildings. It is known, from works in those buildings, that the conservation state of the *Gaiola* is good in general. For instances Fig. 19 shows parts of *frontal* walls removed from a *Pombalino* building in 2010 in excellent conditions.

The interest on *Pombalino* buildings largely exceeds their city of origin. Relevant studies were also performed the University of Minho, where the behaviour of the quarter *Martinho da Arcada*, in the corner of *Rua da Prata* and *Rua do Comércio* in downtown Lisbon, was studied [18]. Even though the buildings of this quarter had been the subject of several interventions with little concern for seismic safety, with removal of part of the interior structure and addition of steel and reinforced concrete elements, the conclusion was that the quarter would resist to a seismic action of 70 % of the one prescribed in the Portuguese code of actions RSA [9].

One of the technical issues related to *Pombalino*a building that has deserved attention from the public opinion is the possible deterioration of their foundations. During the last decade several large holes were found in the subsoil of downtown Lisbon, that fortunately caused almost little or no damage to the buildings so far. It is assumed that these holes were created by changes in the underground water flow due to the numerous underground works (basements, car parks, tube lines) that were done during the last decades. Relevant variations of the level of the water table in downtown Lisbon have also been observed in the past decade, for instances at the BCP Museum, where the photos of Fig. 7 were taken.

As it is well known the wood under strong variations of humidity tends to get rotten, which has been observed in several piles. However no relevant consequences, for instance damage in buildings due to differential settlements, has been observed so far. Together with the fact that the length of the piles is short (about 1.5 m) and do not reach the competent soil that is deeper, this shows that *Pombalino* buildings are not fully supported on the piles. It is therefore thought that the main purpose of the piles was the compaction of the superficial embankment where the buildings are supported, and that distributes the vertical loads by larger areas, transmitting much lower stresses to the weak soil below. Even though the vertical load bearing capacity seems not to have been affected, at least in the short term, to analyse better the possible consequences of deterioration of the piles, the sensitivity of *Pombalino* buildings to differential settlements that could be induced by filling the void on the rotten part of the piles, was studied. Therefore the effect of differential settlements with different profiles at the base of the building was analysed by Rafaela Cardoso [19]. The results indicate that 20 cm differential settlements from the centre to the periphery of the building would be necessary to induce collapse of connections or significant cracking, this is, only considerable settlements would have significant and noticeable effects. However, this does not mean that the deterioration of the piles is not relevant, as the voids created, even though without strong visible consequences in the short term, may induced much larger differential settlements during an earthquake due to changes in soil conditions, weakening the buildings and increasing the potential for much more damage.

3 Structural Changes

In the previous section the potential seismic resistance of original *Pombalino* buildings (without relevant structural changes after the initial construction) was discussed. However, the real seismic resistance of *Pombalino* buildings nowadays depends not only on their original characteristics but is also strongly affected by the alterations to which they were subjected during their lifetime. These were usually associated to the introduction of new facilities (for instances water, sewage or gas pipes), increasing of areas by adding more floors or changes of use and removal of columns and walls in particular at the ground floor to open large spaces for shop windows and in the interior to create larger spaces. Most of those changes were done without any concern for the seismic strength of the buildings, facilitated by a legislative gap and inexistence of technical standards applicable to works on old buildings. Several examples are shown next.

Figure 20a shows on the left hand side photo a case of water pipes introduced inside a *frontal* wall, probably during the twentieth century, cutting the wood bars of the *Gaiola* and strongly weakening the resistance of the frontal wall, specially under horizontal loads, and b) on the right hand side a photo of a case in which the pipes cross the wall in the perpendicular direction causing much less impact.

Fig. 20 **a** Cut in frontal wall to introduce pipes and **b** pipes crossing wall in the perpendicular direction [20]

Figure 21 shows a street in downtown Lisbon in which it is clear that there are buildings of different heights, being known that the original buildings were all of the same height, according to the reconstruction plans. The differences are due essentially to more floors added after the original construction, which strongly increase seismic effects.

Figure 22 shows one of the many buildings in which apparently façade ground floor columns were cut to create a wider shop window, weakening the building where seismic effects are stronger. Usually this is done introducing a beam on top of the columns that are cut to transfer the vertical loads to adjacent columns. However, there are cases in which entire *Gaiola* panels (above ground floor) are removed without addition of strengthening beams and without collapse, showing the excellent performance of the *Gaiola* (in the upper floors) that allows the redistribution of vertical loads from the zone that is removed to the adjacent ones.

This type of interventions took place in most of the buildings in downtown Lisbon, strongly reducing their seismic strengthening, but as the resistance to vertical loads is much less affected, the consequences will only become visible when the next strong earthquake hits Lisbon.

It can be concluded from the above that original *Pombalino* buildings possessed good characteristics of seismic resistance, considering the materials and scientific knowledge available at the time of the reconstruction. However, those

Fig. 21 Street in downtown Lisbon with buildings of different heights [21]

characteristics were progressively adulterated during their lifetime, leading to buildings that nowadays have high seismic vulnerability.

4 Strengthening

In many cases it may not be economically worth or feasible, and without excessive adulteration of the main characteristics, to strengthen old buildings to the same safety levels prescribed for new constructions. Therefore the objective of strengthening old buildings may be the improvement of their potential seismic performance up to minimum standards, subjected to economic restrictions and on the level of adulteration of the original building. In this framework it may be necessary to be more selective in the interventions on these buildings, by means of identifying the potential collapse mechanisms and acting only upon the weakest ones. This philosophy can be illustrated graphically as shown in Fig. 23. If a parallel between the links of the chain and the collapse mechanisms of a building is made, it is clear that strengthening the weakest link

Fig. 22 Cut of columns at ground floor level [21]

up to the strength level of another link is enough to improve the potential performance of the system (building or chain).

This issue can be exemplified with the study of the building of *Rua da Prata* [5], previously mentioned, where different strengthening strategies are discussed. In order to quantify the increase in seismic performance associated to a given strengthening solution, the parameter γ_{sis} was established; this parameter is the value that multiplied by the code prescribed seismic action (in this case the far field event prescribed by the Portuguese code of actions RSA [9]) yields the seismic action that leads to rupture. The analysis of the original structure was

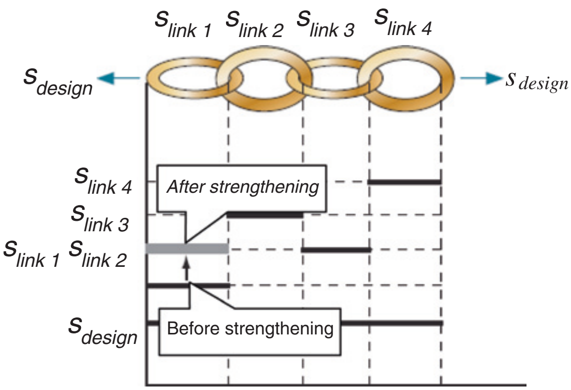

Fig. 23 Strengthening strategies: analogy with chain under tensile force (adapted from [15])

Fig. 24 Schematic representation of strengthening with a ring RC beam [22]

performed assuming weak connections, as already referred to in Sect. 2, yielding a value of $\gamma_{sis} = 0.4$, associated with out-of-plane failure of the façades. If this mechanism was prevented, for instances by strong façade-frontal connections, the collapse would take place by the base shear mechanism, which in this situation correspond to a value of $\gamma_{sis} = 1.05$.

One possible strengthening solution for the original building would be to build a reinforced concrete beam at the perimeter of the top floor, connecting the inner structure to the façades and gable walls and restricting the out-of-plane movement of the façades and gable walls at the top. Figure 24 shows a schematic representation of a transversal cut of such a solution. However it should be noted that the use of reinforced concrete elements in the rehabilitation of historical buildings has been criticised by UNESCO and ICOMOS.

It should be noticed that, in this strengthening solution, the ring beams should be properly connected to the rest of the structure so that there are no differential movements between the masonry walls, the ring beams and the floors/roof.

The analysis of the structure after introducing the ring beams revealed that the connections façade-frontal collapse at intermediate floors and the façade moves out-of-plane but not as if it was a cantilever as in the original structure but as if it was a simply supported beam, with supports on top and bottom. Due to this, collapse occurs at a higher intensity $\gamma_{sis} = 0.7$. However, the structure would become stiffer due to the new beam, increasing the frequencies, the spectral accelerations and therefore the inertia forces. As the resistance to horizontal forces at the base level did not increase, the seismic intensity associated to collapse in the base shear mechanism decreases to $\gamma_{sis} = 0.9$. The strengthening by means of ring beams can be extended to all floors. Despite the increase in stiffness and inertia forces, the resistance to the out-of-plane collapse of the façades increases to $\gamma_{sis} = 0.75$. The seismic intensity at which base shear failure takes place decreases to $\gamma_{sis} = 0.8$, since the resistance to horizontal forces at the ground floor did not increase. From this stage onwards any strengthening strategy that would increase the stiffness of the façades would be counterproductive, as the increase in the inertia forces would reduce the seismic intensity associated to base shear failure. This means that from this stage onwards the increase in the seismic resistance of the building would require strengthening both mechanisms simultaneously. This inconvenient could be eventually solved by means of an alternative strengthening strategy, for instances strengthening only the connections façades-frontals. As this consists of localized interventions that increase the strength without increasing the stiffness, it allows increasing the resistance to one collapse mechanisms without reducing the resistance to the others.

Another strategy that may be efficient is the strengthening of the pavements in their own plan by means of a set of steel angles in two orthogonal directions at 45º to the façades and side walls, in order to create an effect similar to a rigid floor that allows to transfer inertia forces to the stiffer elements of the structure, in particular the gable walls. The efficiency of this solution was studied by means of its application in a particular case study [23]. The results indicated that despite the fact that the inertia forces increased by 17 % due to the global stiffness increase, this effect was largely compensated by the redistribution of forces to the stiffer and stronger elements, reducing the action-effects in the more vulnerable elements, namely ground floor columns, façades (in the out-of-plane direction) and façades-frontal connections. Figure 25 shows (a) a scheme of the distribution of the steel angles and the deformation of one of the pavement, (b) the deformation of the same pavement without the angles, and (c) the out-of-plane deformations of the façades in both cases (the full line corresponds to the pavement strengthened with 2L200 × 200 × 20 or 2L100 × 100 × 10 angles and the dashed line to the unstrengthened pavement).

In what regards practical cases of strengthening of Portuguese old buildings, there is already considerable experience and capacity. In order to illustrate this capacity, developed essentially during the past two decades, examples of the strengthening of two *Pombalino* buildings are referred next.

The first example regards the rehabilitation and strengthening of a Pombalino building in *Rua do Comércio*, executed by the companies MONUMENTA and STAP [24], that included the following works, some of which are documented in Fig. 26:

1. Repair original wood bars and selective replacement of the bars in more advanced stage of degradation
2. Strengthening or reconstruction of interior *frontal* and *tabique* walls, filling the panels following the original techniques of the wood truss (rebuilding the *Gaiola*).

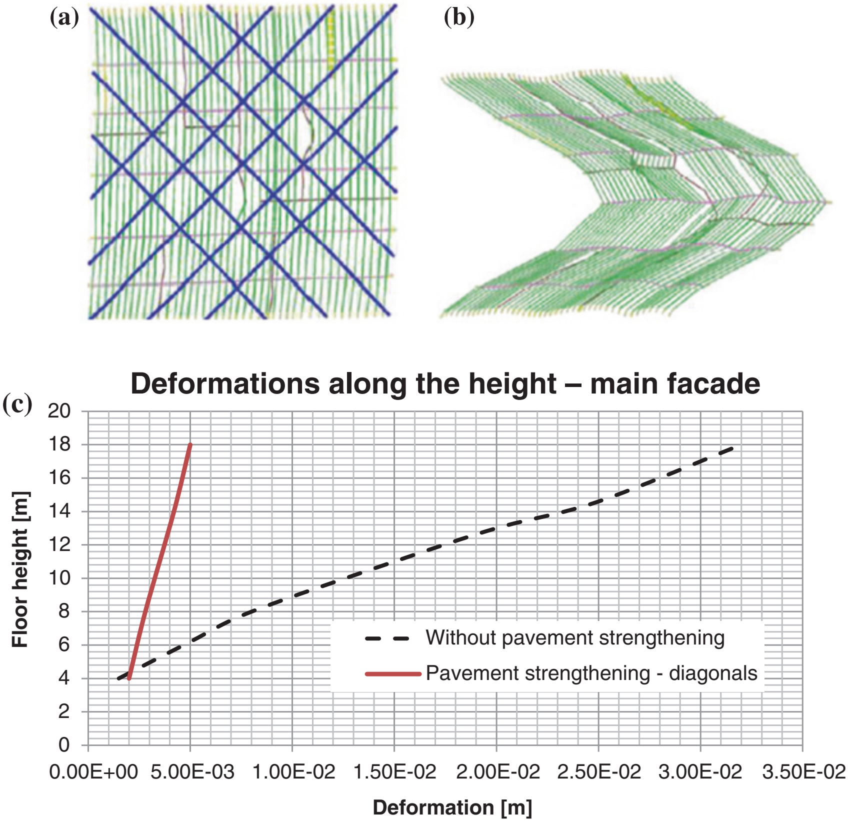

Fig. 25 Floor deformation: **a** with diagonals; **b** without diagonals; and **c** out-of-plan deformation of the façades [23]

3. Execution of a strengthening solution to increase the global resistance to horizontal forces, which consisted of:

- System of cables anchored with ductile anchor plates at the extremities, connecting the façades and gable walls to prevent independent out-of-plane movements of those walls;
- Steel plates to connect beams on the same alignment to ensure continuity;
- Devices to improve the wall–wall and wall-pavements connections.

It should be noted that the *Gaiola* continues to be part of the structural system, this is, it is not part of the problem, it is part of the solution, allowing solutions much less extensive than would be the case if the *Gaiola* was not there.

The second example is the rehabilitation of a building at *Rua Nova do Carvalho*. The structural project, by the design office A2P [25], explicitly

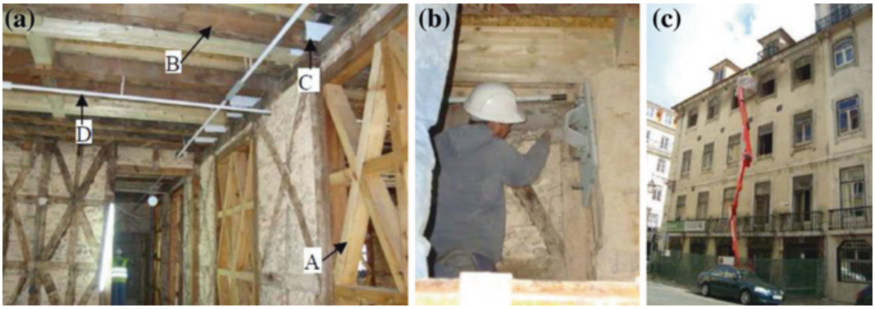

Fig. 26 **a** *A* selective replacement of deteriorated wood bars of the *Gaiola*; *B* selective replacement of deteriorated wood bars of the pavements; *C* steel reinforcements to ensure continuity of the wood bars of the pavements over the supports; *D* cables connecting to the façades; **b** installation of an anchorage device of a cable; **c** preparation of a façade to place anchorage plates [24]

comprised the objective of seismic strengthening. In the works on this building it was possible to preserve most of the primary elements of the building, namely:

- Foundations
- Masonry walls, strengthened with reinforced concrete layers of small thickness
- Frontal walls, repaired with new wood bars and new masonry filling at selected locations
- Ground floor columns and vaults
- Stairs and staircase
- Main bars of the wood floors
- Stones of the ground floor pavement
- Steps, horizontal platforms and ceiling of the stairs
- Part of doors and window frames

Figure 27b shows new filling of *frontal* walls made with hollow bricks and hydraulic and cement mortar. As it was already mentioned the masonry filling of frontal walls is

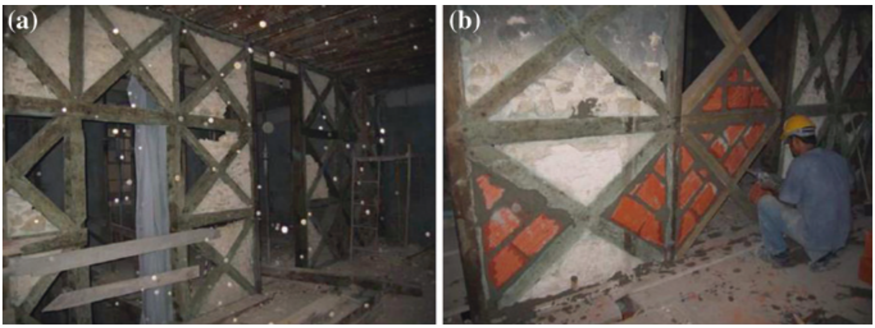

Fig. 27 **a** *Frontal* walls with the original filling and **b** damaged panels filled with hollow bricks with hydraulic and cement mortar

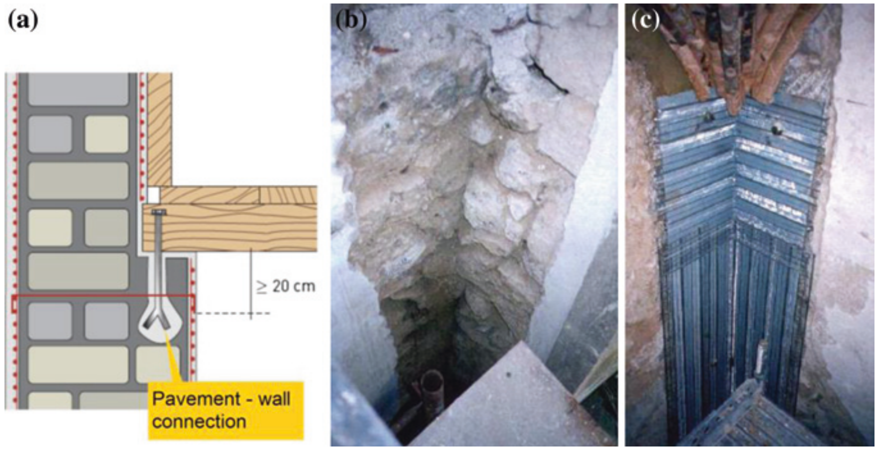

Fig. 28 **a** Connections wood pavements-masonry wall; and **b** and **c** between orthogonal masonry walls (adapted from [22, 26])

not as important as other parts of the structure, since what is clearly important for the seismic performance of the frontal walls is the wood structure.

In Portugal there is considerable experience in strengthening other types of masonry buildings for earthquake resistance, for example the repair and strengthening of the constructions damaged by the 1998 Faial earthquake, in Azores. Several strengthening techniques and details used in those cases can also be used in *Pombalino* buildings. Figure 28 shows two examples, the first ((a) referring to the connection between wood floors to masonry walls [22] and the second (b) and (c)) to the connection between two orthogonal masonry walls [26]. Figure 29a shows a scheme for the strengthening of a masonry wall by adding thin layers of cement mortar reinforced with steel meshes on either side [27] and, Fig. 29b, a photo of a wall strengthened in that way before adding the mortar [17].

5 Advanced Modelling

The models previously presented were intended for practical applications, therefore were based on models able to be analysed with currently available commercial software for structural analysis. In this section some advanced modelling techniques, which allow the explicit consideration of nonlinear material behaviour, are presented. These techniques make use of sophisticated computer programs, and allow performing pushover and nonlinear dynamic analysis. Actually, according to the widespread of performance-based earthquake engineering concepts, which research trends and various international and national codes [28, 29] now refer to, the possibility to perform nonlinear analyses becomes common, especially in research works: this is relevant in case of masonry buildings, that are also affected by the nonlinear behaviour. Of course,

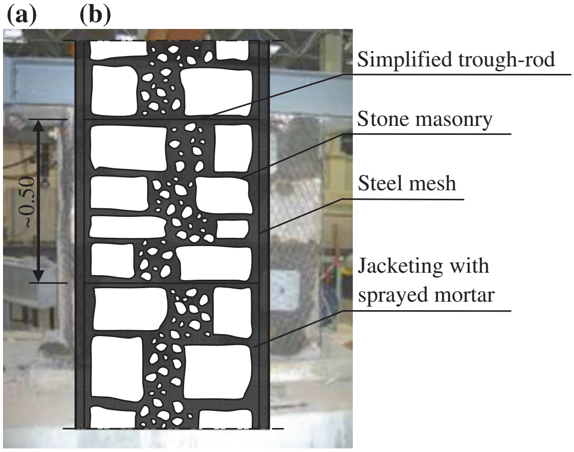

Fig. 29 **a** and **b** Strengthening of masonry wall with reinforced cement mortar on both sides (adapted from [17, 27])

it implies the need of reliable models able to simulate the nonlinear response of various element types: for example, in case of *Pombalino* buildings, not only that of URM panels but also of *frontal* walls.

As it is known, a complete seismic assessment should include the analysis and verification of two types of response: the global one (type a), mainly related to the activation of the in-plane response of walls, and that (type b) associated to the activation of local mechanisms, which mainly involve the out-of-plane response of walls. Common assumption is to verify these two types of response separately by neglecting their mutual interaction. Type a) is usually analysed by referring to a 3D model of the structure: to this aim, among the different modelling strategies proposed in the literature, due to the regular pattern of openings in *Pombalino* building, the equivalent frame approach seems particularly suitable. One common way to analyse type b) response may be by using discrete macro-block models. In both cases, according to performance based assessment and, in particular, the use of nonlinear static analyses, the seismic verification may be: in case a), by adopting nonlinear static procedures, such as the Capacity Spectrum Method [30] or the N2 Method [31]; in case b), by referring to the nonlinear kinematic approach based on the limit analysis (e.g. as proposed in the Italian Code for Structural Design [32] and described in Lagomarsino and Resemini [33]). In the following, the attention is focused only on the global response by assuming local mechanisms are inhibited through proper constructive details: thus, for example, it is assumed that *frontal* walls are properly attached to masonry façades at reasonable distances, preventing their out-of-plane failure.

5.1 Equivalent Frame Modelling Approach

The equivalent frame approach starts from the main idea (supported by the earthquake damage survey) that, referring to the in-plane response of complex masonry walls with openings, it is possible to recognize two main structural components: piers and spandrels. Piers are the principal vertical resistant elements for both dead and seismic loads; spandrels, which are intended to be those parts of walls between two vertically aligned openings, are the secondary horizontal elements, coupling piers in the case of seismic loads. Thus, according to the equivalent frame idealisation, each wall is discretized by a set of masonry panels (piers and spandrels), in which the nonlinear response is concentrated, connected by a rigid area (nodes). Thus, by assembling 2D walls (considering only their in-plane contribution) and including the floor modelling, this approach allows one to analyse complex 3D models by performing nonlinear analyses with a reasonable computational effort; moreover, it agrees with recommendations of both national and international codes. This strategy seems particularly suitable in case of *Pombalino* buildings with good connections, as the façades are characterized by a quite regular opening pattern for which the idealisation in equivalent frame does not pose strong difficulties. Among the different models and software that work according to this approach, in the following particular attention is paid to Tremuri program which has been originally developed at the University of Genoa, starting from 2002 [34], and subsequently implemented in the software package Tremuri [35]. In fact, recently, in Tremuri program a specific element intended to simulate the response of *frontal* walls has been implemented [12, 36].

Once having idealised the masonry wall into an assemblage of structural elements, the reliable prediction of its overall behaviour mainly depends on the proper interpretation of the single element response. Different formulations, characterized by different degrees of accuracy, may be adopted. In the following, the attention is focused on a formulation based on a nonlinear beam idealization (Fig. 30): thus, the response in terms of global stiffness, strength and ultimate displacement capacity may be obtained by assuming a proper shear-drift relationship.

In case of URM panels, the formulation is based on a phenomenological representation of the main in-plane failure modes, which may occur (such as rocking, crushing, bed joint sliding and diagonal cracking); in particular, a bi-linear relation with cut-off in strength (without hardening) and stiffness decay in the nonlinear phase (for non-monotonic action) is adopted. The initial elastic branch is directly determined by the shear and flexural stiffness, computed on the basis of the geometric and mechanical properties (Young modulus E and shear modulus G) of the panel. Since the progressive degradation of the stiffness is not actually modelled, a calibration of the initial mechanical properties is necessary: in fact, they should be more properly representative of "cracked" conditions. The ultimate strength is computed according to some simplified criteria, which are consistent with the most common ones proposed in the literature and codes (e.g. in Eurocode 8—Part 3[29] and in the Italian Code for Structural Design [32]). Table 1 summarizes the strength criteria implemented in Tremuri program. Then, the failure of the panel is checked in terms of drift limit values differentiated as a function of the prevailing failure mode occurred (if shear or flexural one). This formulation is particularly suitable for nonlinear static analyses since it requires a reasonable computational effort, suitable also in engineering practice, and it is based on a few

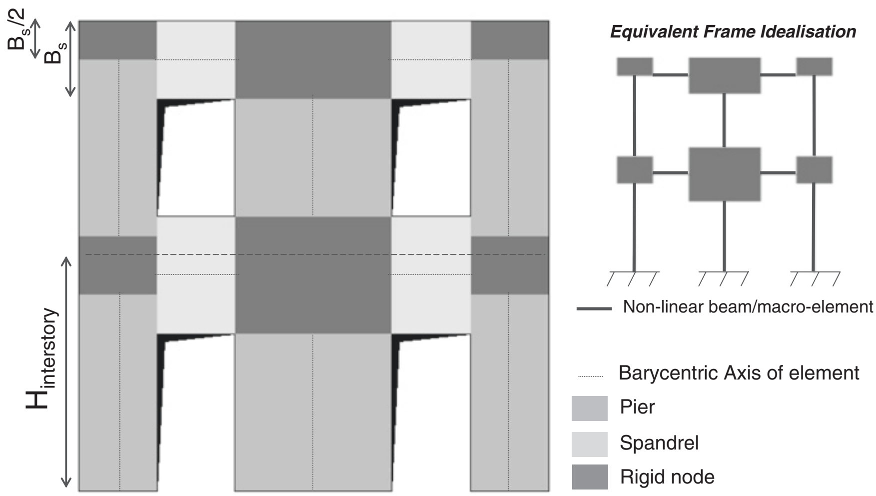

Fig. 30 Equivalent frame idealisation of a masonry wall [38]

mechanical parameters, which may be quite simply defined and related to results of standard tests. Further details on URM nonlinear beam and also on more accurate formulations implemented in Tremuri program may be found in Galasco et al. [37], Lagomarsino and Resemini [38] and Lagomarsino and Cattari [39].

To provide a reliable modelling also of *Pombalino* buildings, it is necessary to be able to describe the nonlinear response of typical *frontal* walls. In this context, the formulation proposed in Meireles et al. [36] and Meireles et al. [12] has been implemented in a nonlinear beam in Tremuri program. It aims to reproduce the hysteretic shear response of *frontal* walls and it has been formulated and calibrated on basis of the work of Meireles and Bento [42]. This work was the first experimental work to test the *frontal* walls built in laboratory under static cyclic shear testing with imposed horizontal displacements, where a specific loading protocol was used. Vertical loading was also applied to the specimen by four hydraulic jacks and rods. The objective of the experimental work was to obtain the hysteretic behaviour of *frontal* walls, by means of static cyclic shear testing with imposed displacements. Then, the hysteresis model was developed based on a minimum number of path-following rules that can reproduce the response of the wall tested under general monotonic, cyclic or earthquake loading. It was constructed using a series of exponential functions and linear functions. The hysteresis rule incorporates stiffness and strength degradations and pinching effect. It was then developed based on the experimental tests carried out [42] and the parameters are calibrated by such results. This model uses 9 parameters to capture the nonlinear hysteretic response of the wall: a first set of parameters aimed to define the envelope curve (F_0, K_0, r_1, r_2, F_u, δ_{ult}); two parameters to define the unloading curve; a last one to define the reloading curve. Figure 31 shows the assumed hysteresis model of the wall.

Table 1 Strength criteria for masonry panels implemented in Tremuri program

	Failure mechanism	Ultimate strength	Notes
Piers	Rocking/Crushing	$M_u = \frac{Nl}{0.425\ f_m}\left(1 - \frac{N}{lt}\right)$	f_m masonry compressive strength of masonry, l length of section, t thickness
	Bed joint sliding	$V_{u,bjs} = l^{'}tc + \mu N \leq V_{u,blocks}$	Mohr–Coulomb criterion with: l' length of compressed part of cross section; μ and c friction coefficient and cohesion of mortar joint, respectively. A limit value is imposed to take into account in approximate way the failure modes of blocks
	Diagonal cracking	$V_{u,dc\_1} = lt\frac{1.5\tau_o}{b}\sqrt{1 + \frac{N}{1.5\tau_o lt}}$	τ_0 masonry shear strength, b reduction factor as function of slenderness [40]
		$V_{u,dc\_2} = \frac{1}{b}\left(lt\tilde{c} + \hat{\mu}N\right) \leq V_{u,blocks}$	Mohr–Coulomb type criterion with: and equivalent cohesion and friction parameters, related to the interlocking due to mortar head and bed joints (such as proposed in [41])
Spandrel[a]	Rocking/Crushing	$M_u = \frac{dH_p}{2}\left[1 - \frac{H_p}{0.85 f_{hu} dt}\right]$	H_p: minimum value between the tensile strength of elements coupled to the spandrel (such as RC beam or tie-rod) and 0.4 f_{hu} dt, where f_{hu} is the compression strength of masonry in the horizontal direction
	Shear	$V_u = htc$	h height of spandrel transversal section

[a]The Italian Code for structural design [32]. Differently from Eurocode 8, makes a distinction in the strength criteria to be adopted for spandrels as a function of the acting axial load: if known from the analysis, the same criteria as piers are assumed; if unknown, a response as equivalent strut is assumed. In Tremuri program, since the axial force computed for spandrels usually represents an underestimation of the actual one, the maximum value provided by these two cases is assumed as reference

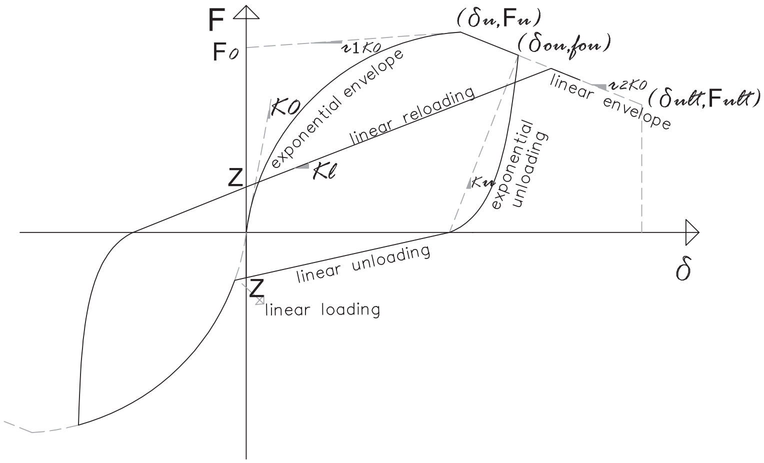

Fig. 31 Hysteresis model of frontal walls [37]

As an example, here a comparison is made between a *frontal* wall and a URM wall of equivalent dimensions (height 2.48 m; width 2.56 m; thickness 0.15 m). The masonry wall is composed of rubble masonry. The strength of the masonry panel, associated to shear failure, when subjected to a vertical stress of 20 % of the compressive capacity, is 73 kN. The ultimate drift of the masonry panel is 0.4 % (as proposed in Eurocode 8 [29] in case of a prevailing shear response). The stiffness relative to the transverse displacement between extremities of a masonry panel is calculated, according to the beam theory, considering that the panels are built-in in one (cantilever) or both extremities. By observing Fig. 32 one can see how the *frontal* walls have lower stiffness when compared to a masonry wall of approximately the same size.

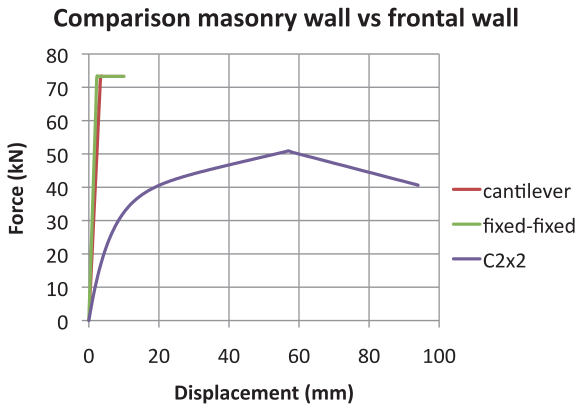

Fig. 32 Comparison between a masonry wall (cantilever and fixed–fixed) and a *frontal* wall (C2x2) of the same dimensions [37]

In addition to masonry and *frontal* elements, RC elements, steel and wooden nonlinear beam or tie-rods may be modelled as well.

Finally, the complete 3D model is obtained by introducing also floor elements. In particular, they are modelled as orthotropic membrane finite elements where normal stiffness provides a link between piers of a wall, influencing the axial force on spandrels; shear stiffness influences the horizontal force transferred among the walls, both in linear and non-linear phases.

In the following, a building aimed to replicate a typical *Pombalino* building was modelled and analysed by using Tremuri program. It was necessary to choose an example building. It was decided to use a modified version of the building of *Rua da Prata* previously mentioned but with some alterations in order to yield a building more similar to the original *Pombalino* buildings built after the 1755 earthquake.

5.2 Example of Equivalent Frame Modelling for a Case Study of a Pombalino Building

The building that was chosen to be analysed tries to replicate a typical *Pombalino* building. It had been the subject of research in the study by Cardoso [5] and later on in Meireles et al. [16, 43]. Its historical background and architectural drawings are also referred to and shown in the book *Baixa Pombalina: Passado e Futuro* (*Pombalino* Downtown: Past and Future) [44]. This building is recognized by the existence of a pharmacy in the ground floor, which is covered by a well-decorated panel of blue tiles, dating from 1860. Nevertheless, as it is usual in the *Pombalino* buildings of Lisbon downtown, this building has been subjected to some alterations with respect to the original layout. In this particular case one floor has been added to the original layout of 4 floors plus roof, making a total number of 5 floors plus attic. In the current study, given that it was intended to study a typical *Pombalino* building, only 4 floors plus roof were considered in the layout, so the last floor below the roof was eliminated in the drawings and modelling.

The building has six entries on the main façade and a height of approximately 15 m until the last floor (without the height of the roof). The openings have a width of 1.66 m or 1.76 m, the door on the ground floor a height of 3.5 m, the balcony on the first floor a height of 3 m and the windows on the second and third floors a height of 2 m.

At the back the openings are smaller and have a width of 1 m. At ground floor level the height of the door is 3 m and on the first, second, and third floors there are windows 1.5 m high. There are only 5 entries. The plan drawings of the building are shown in Figs. 17 and 33 for the ground floor and upper floors, respectively.

The plan of the building has dimensions of $18 \times 11\ m^2$ referring to the façade and gable walls, respectively. The ground floor has 5 internal piers with dimensions of $0.7 \times 0.7\ m^2$. There are stairs in the middle of the building facing towards the back façade. The staircase is made with brick masonry only on the ground

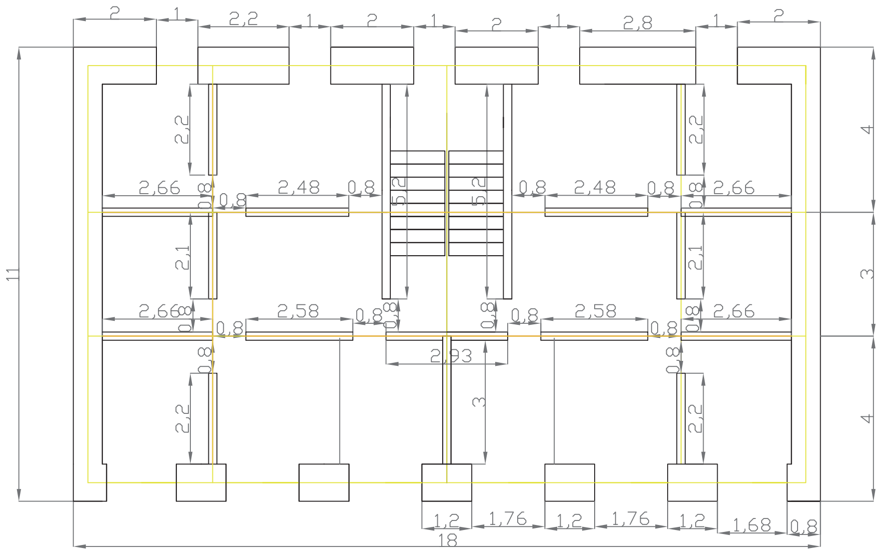

Fig. 33 Sketch of the plan view of building: upper floors (dimensions in metres) [45]

floor (on the upper floors the staircases are *frontal* walls) with a thickness of 0.24 m. On the ground floor, brick masonry walls extend up to the front of the building with a small misalignment towards the right. On the ground floor, the front and back façade piers as well as the internal piers are made of stone masonry. The gable walls as well as the front and back façades of the upper floors are made of rubble masonry.

On the upper floors (from the first to the third floor) one can find the *frontal* walls. There are two alignments of *frontal* walls parallel to the façades and five alignments (including the staircase) of *frontal* walls parallel to the gable walls. Connecting the *frontal* walls there are doors 0.8 m wide. The structural elements with their respective type of material and thickness/area can be found on Table 2. As it can be observed in Table 2, the façades (front and back) reduce in thickness the higher they are, being of 0.90 m on the ground floor and 0.75 m on the third floor.

The actions considered on the structure are the self-weight, given by the weights of the roof, the floors, the ceilings, the partition walls and the *frontal* walls, combined with the respective live loads given by Eurocode 1 [46]. The vertical loading (Table 3) to be imposed on the structure was determined based on Eurocode 1 [46] (design load = dead load + 0.3 × live load).

Table 4 summarizes the mechanical properties adopted for URM and *frontal* walls, respectively. For URM panels a drift limit value of 0.4 % and 0.8 % (as suggested also in the Italian code for structural Design [32]) has been adopted in case of prevailing shear and flexural failure modes, respectively. For *frontal* walls the value of F_{ult} (denotes failure) is taken as 80 % of the value of F_u.

Table 2 Thickness/area and material of building components [43]

Geometrical data and masonry types		
Element	Material[a]	Thickness/area
Piers (ground floor)	SM	0.7×0.7 m^2
Façades (front and back):		
Ground floor	SM	0.90 m
First floor	RM	0.85 m
Second floor	RM	0.80 m
Third floor	RM	0.75 m
Spandrels	RM	0.20 m
Gable walls	RM	0.70 m
Staircase (ground floor)	BM	0.24 m
Internal walls (ground floor)	BM	0.24 m
Frontal walls	Wood, RM	0.15 m

[a]SM, RM and BM mean stone masonry, rubble masonry and brick masonry, respectively

Regarding the floors, the joists of the floors have a section of 10×20 cm^2 and the wood pavement a thickness of 2 cm. The floors are supported by the front and back façades and by the *frontal* walls; the stairs are supported by the staircase. The floors have been modelled as orthotropic membrane elements.

The structure is modelled according to the equivalent frame model (by adopting Tremuri program) using nonlinear beams for the ordinary masonry panels and *frontal* walls according to the formulation described in 5.1. The final model of the building is presented in Fig. 34a. Here, represented in grey are the parts of the structure that are composed of rubble masonry; in purple are the parts of the structure that are composed of stone masonry; in green (dark and light depending on the size) are the *frontal* walls and in light brown are the timber beams connecting the *frontal* walls. Figure 34b identifies the alignments of the different structural elements in the plan view of the building.

Table 3 Vertical loads considered in the case study [45]

Actions considered		
Element	Location	Value[a]
–	Floors	2.0 kN/m(*ll*)
–	Stair floor	4.0 kN/m(*ll*)
Stairs	Stair floor	0.7 kN/m(*dl*)
Compartment walls	Floors	0.1 kN/m(*dl*)
Wooden floors	Floors	0.7 kN/m(*dl*)
Ceilings	Floors	0.6 kN/m(*dl*)
Frontal wall	*Frontal* walls	3.0 kN/m (*dl*)
Vaults	Masonry walls ground floor	3.5 kN/m (*dl*)
Gable walls roof	Masonry walls 4th floor	17.3 kN/m (*dl*)
Roof	Masonry walls 4th floor	4.4 kN/m (*dl*)

[a]The load type is summarized in brackets: if live load (*ll*) or dead load (*dl*), respectively

Table 4 Mechanical characteristics of masonry types and parameters of frontal walls

Masonry type	Average young modulus E (GPa)	Average shear modulus G (GPa)	Weight W (kN/m^3)	Average compressive strength f_m (MPa)	Average shear strength τ_0 (MPa)
Stone Masonry	2.8[a]	0.86[a]	22	7	0.105
Rubble Masonry	1.23	0.41	20	2.5	0.043
Brick Masonry	1.5[a]	0.5[a]	18	3.2	0.076
Frontal wall	F_u (kN)	K_0 (kN/mm)	r_1K_0	r_2K_0	F_0/F_u
2 × 2[b]	50.8	6.1	0.244	−0.2745	0.728
3 × 2[c]	49.9	2.9	0.244	−0.2745	0.728

[a]Cracked stiffness assumed, 50 % of the value in the table was used
[b]Parameters have been calibrated on basis of experimental results obtained in Meireles and Bento [42]
[c]F_u and K_0 (as defined in Fig. 2) have been obtained for different configurations (2 × 3, 2 × 4, 3 × 2, 3 × 3 and 3 × 4) based on analytical models, see Meireles [45]

The mesh, that is the equivalent frame idealization, has been created by using the software package Tremuri [35] in which Tremuri has been implemented. The software creates a mesh of macro-elements for each alignment and this can be viewed for front and back façades, in Fig. 35, respectively: in red are the piers; in green are the spandrels and in blue are the parts of the façade where no damage is foreseen (rigid nodes). Furthermore, in this modelling, the foundations are modelled as built-in (no displacements or rotations allowed).

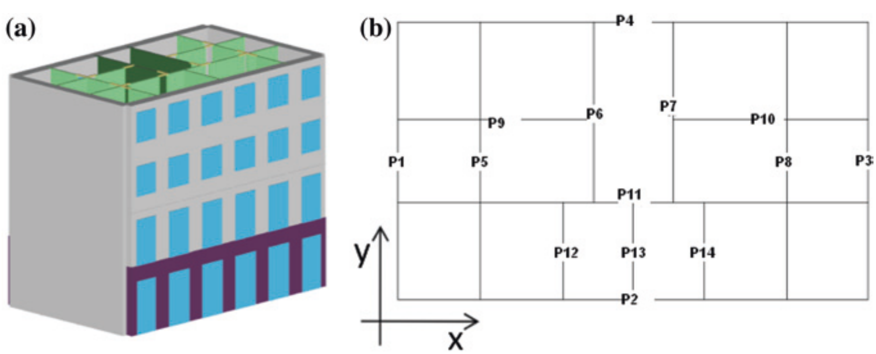

Fig. 34 **a** 3D view and **b** numbering of the alignments of the elements of the model [43]

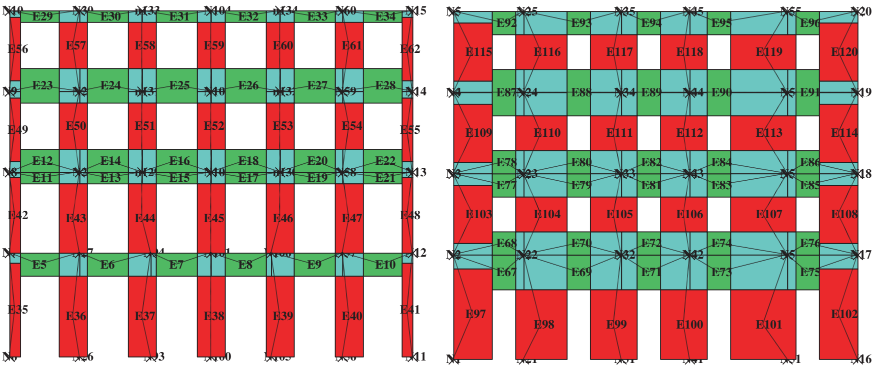

Fig. 35 Equivalent frame idealisation of front façade (*up*) and back façade (*bottom*) [45]

5.3 Example of Nonlinear Static Analysis for the Case Study Examined and Discussion of Results

The case study described in 5.2 refers to an original configuration of a *Pombalino* building. However, it should be noted, that, in reality, a considerable part of the building stock of Lisbon downtown probably is not in its original state but has been subjected to changes in its structural system,such as the ones referred to in Sect. 3. It is foreseen that these changed buildings will have a behaviour that is worse than the original building.

In the following, firstly the results of nonlinear static analyses performed on the original configuration are examined; then, the effects of some strengthening solutions on the overall response are discussed by comparing results in terms of probabilistic seismic assessment through the introduction of fragility curve concept. Pushover analyses were carried out for both *xx* and *yy* directions (see Fig. 34) and for two lateral load patterns along the height: load pattern proportional to the mass (uniform) and load pattern proportional to the mass and height (triangular). Pushover analyses enable us to have an idea of the lateral resistance of a building; the output in terms of overall base shear versus top displacement is presented in Fig. 36. At each floor the applied forces were distributed according to the distribution of mass and the top displacement refers to the average of the displacements of the nodes on the top floor. Actually, while in case of rigid floors the result of the pushover analysis is almost insensitive to the control node (usually assumed at the centre of mass), much critical is the case of the flexible ones. Actually, in this latter case, the results may be significantly affected by the control node adopted and points in the same floor may exhibit very different displacements, in particular in case of shear masonry walls characterized by very different stiffness. Thus, a reasonable compromise is to assume, for the analysis, a generic node at the level of the last floor, but to refer for the pushover curve to the average displacement of all nodes located at this level (eventually weighted with the respective pushover nodal

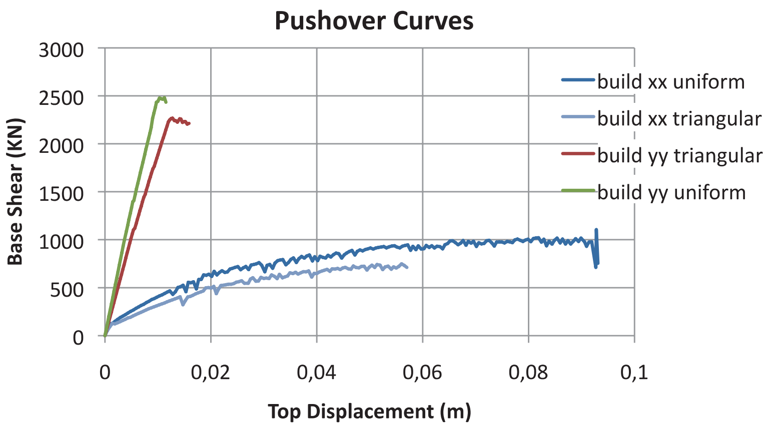

Fig. 36 Pushover curves for original building in the two directions for both uniform and triangular load patterns [45]

force) in order to consider a result representative of the whole structure and not only of local portions.

Pushover analyses performed on this basic configuration showed a significant difference between the seismic capacity of the building in *xx* and *yy* directions, in particular the stiffness and strength is much higher in the *yy* direction than in the *xx* direction; but on the other hand, the ductility of the system is much higher on the *xx* direction and is practically non-existing in the *yy* direction. In fact, in *xx* direction the piers are very slender (due to the opening's configuration) and with a very moderate coupling provided by spandrels (which show a "weak" behaviour): thus, a prevailing flexural response occurs associated to higher drift than in case of the shear failure. In general the structure exhibits a soft storey failure mode; moreover, since floors are quite flexible, a very moderate redistribution of seismic loads may occur among masonry walls. Indeed, in neither of the two directions the building seems to provide a reliable system against the earthquake.

Starting from the study of the response of the basic configuration of the *Pombalino* structure, the following strengthening solutions have been analysed mainly based on engineering judgement.

Due to this, the following retrofitting schemes have been proposed and analysed:

a. Increase the in-plane stiffness of floors (transforming flexible floors into rigid floors);
b. Increase the in-plane stiffness of floors plus inclusion of four shear walls on the ground floor;
c. Increase the in-plane stiffness of floors plus inclusion of eight steel frames on the ground floor;
d. Increase the in-plane stiffness of floors plus inclusion of tie-rods at front and back façades.

The first one is the one that will be seen to be the most effective and crucial improvement to the structure. The last three are seen to be added improvements to the structure if one wishes to increase the earthquake resistance of the building even more. The first intervention may be reversible or not, depending however on the type of intervention. All the other interventions are reversible.

As regards to the *increase of the in-plane stiffness of floors* (case a), traditional timber floors are typically flexible. The increase of the in-plane stiffness of floors is an evident and most effective method of improving the seismic behaviour of old masonry structures. This is mainly because the increase of in-plane stiffness of floors enables the horizontal forces to be redistributed between the failing walls to the adjacent remaining walls and the structure behaves like a box. A significant role in the stability of the entire building is assigned to the floors. These structures are required, in addition to an adequate performance level, an adequate rigidity and an efficient connection to the supporting walls, especially in what concerns seismic actions. For this reason, the restoration of a floor is an opportunity to improve the behaviour and efficiency of the entire structure.

Starting from the original configuration, mechanical parameters of orthotropic membranes aimed to simulate floors have been increased to simulate such type of intervention (e.g. obtained by the insertion of plywood or horizontal bracing composed of steel ties and arranged in crosses). Figures 37 and 38 show the resultant pushover curves, in *xx* and *yy* directions, respectively. The contribution that each alignment (walls) has to the base shear of the building was also evaluated in both directions. For this purpose, and taking the *xx* direction as an example (Fig. 37), a graph was plotted with, firstly, the total base shear as a function of the top displacement ("Building" legend), secondly, the base shear corresponding to the façade masonry walls (P2 and P4 alignments) as a function of the respective top displacement of that alignment ("P2" and "P4" legend) and, thirdly, the base shear

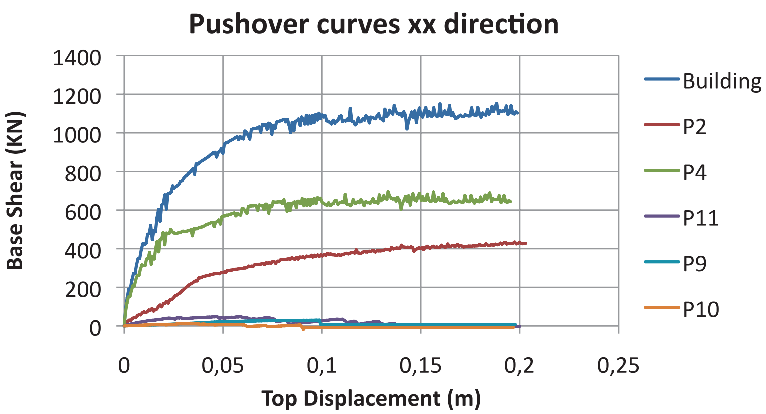

Fig. 37 Pushover curves, contribution of each wall to the base shear, *xx* direction [45]

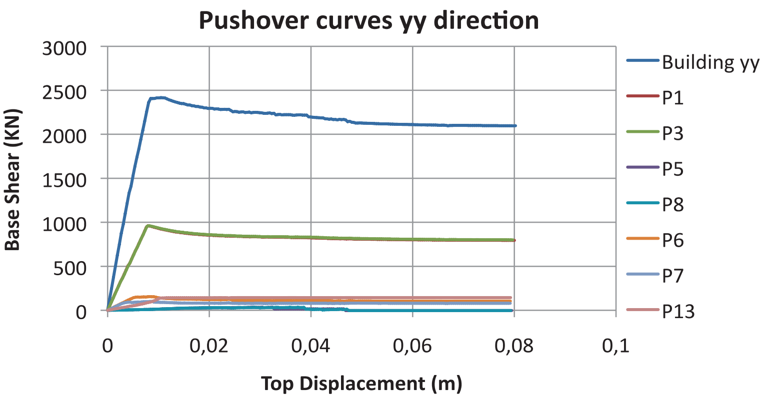

Fig. 38 Pushover curves, contribution of each wall to the base shear, *yy* direction [45]

corresponding to the alignments of the"frontal" walls as a function of the respective top displacement of that alignment ("P11", "P9" and "P10" legend).

Based on the previous graphs, the highest contribution to the base shear comes from the outside masonry walls. The contribution to the base shear given by the internal walls/columns is not negligible but is small. In other words, the *frontal* walls/internal walls alignments contribute very little to the total base shear of the respective alignments, the majority of base shear being a contribution of the surrounding masonry walls. This is because the *frontal* walls do not have continuity in height, they are interrupted at ground floor (above the first floor the contribution of the *frontal* walls to the horizontal shear may be not so low), and because they have a lower stiffness when compared to the masonry walls. Indeed, from the comparison between a single URM panel and a *frontal* wall illustrated in 5.2, one can conclude that the stiffness of the *frontal* wall is lower than the stiffness of the thick (see Table 2) surrounding masonry walls of the *Pombalino* buildings.

As regards to the *increase the in-plane stiffness of floors plus inclusion of four shear walls on the ground floor* (case b),the inclusion of shear walls is a typical procedure to improve the seismic resistance of a building. The modelled shear walls are 48 cm thick and are composed of brick masonry. It was decided that the shear walls should only be placed in the *xx* direction since this direction is the most vulnerable one (after the strengthening of the diaphragms and given the presence of the gable walls with no openings on the *yy* direction).

As regards the *increase the in-plane stiffness of floors plus inclusion of eight steel frames on the ground* (case c), the inclusion of eight steel frames on the ground floor arises from the idea that including shear walls with no openings on the ground floor is not a very much welcoming idea from the architectural and functional perspective. The ground floors of these buildings are often used as restaurants, cafés or stores facilities and the inclusion of shear walls here is not very convenient from the point of view of the owners. The eight steel frames (pillars and beams) are each one composed of four HEA140 cross sections. Again, it was decided that the steel frames should be placed only in the *xx* direction for the reasons previously described.

Finally, as regards to the *increase the in-plane stiffness of floors plus inclusion of tie-rods at front and back façades* (case d), the model was prepared for the case of tie-rods at the front and back façades. In the model bar elements with prestressing were introduced. The tie-rods are placed at the top of the piers (placed along the spandrels), connecting the piers between each other. They are prestressed, prestressing the spandrels. The idea is to couple the piers with the prestressed spandrels. The modelled tie-rods are 2.4 cm in diameter and made of steel. An initial strain of 20 % the yielding strain of the steel was used. The tie-rods were only placed in the *xx* direction, where we have spandrels.

Figure 39 shows the comparison among the pushover curves obtained for all the different configurations examined. Different configurations vary in terms of strength, stiffness and ductility. Since all these three aspects play a fundamental role in the seismic assessment, a more effective comparison is discussed in the following in terms of probabilistic assessment through the introduction of fragility curves.

To this aim, firstly pushover curves have been converted in the equivalent SDOF oscillator (according to criteria proposed in Eurocode 8—Part 3 [29]); then proper damage states (from—slight damage—to 4—collapse) have been defined on the resultant capacity curves by adopting the criteria proposed in Lagomarsino and Giovinazzi [47]. Figure 40 shows the fragility curves obtained by assuming a β value (that is the standard deviation of the natural logarithm of spectral displacement associated to different damage states) equal to: 0.53, 0.54, 0.51 and 0.49 from damage state 1–4, respectively. These values summarize the uncertainties associated to errors in the model, input parameters, definition of limit states and variability of the seismic input; they have been computed according to the proposal of Pagnini et al. [48]. Moreover further details may be found in Meireles et al. [43]. The seismic input has been assumed as the earthquake type 1 (far-field

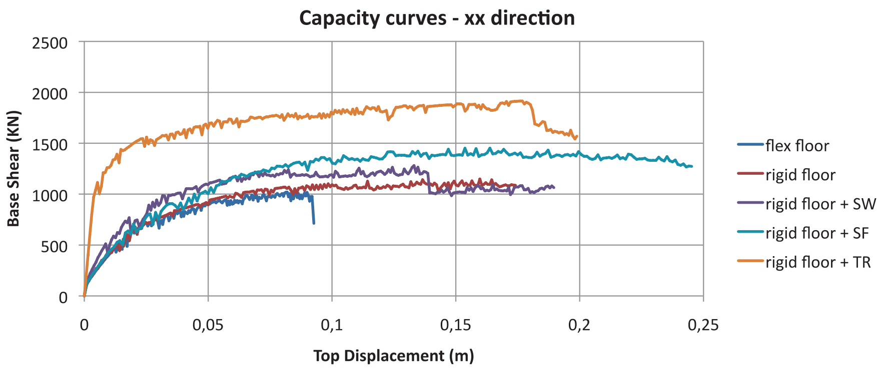

Fig. 39 Pushover curves for all the examined configurations

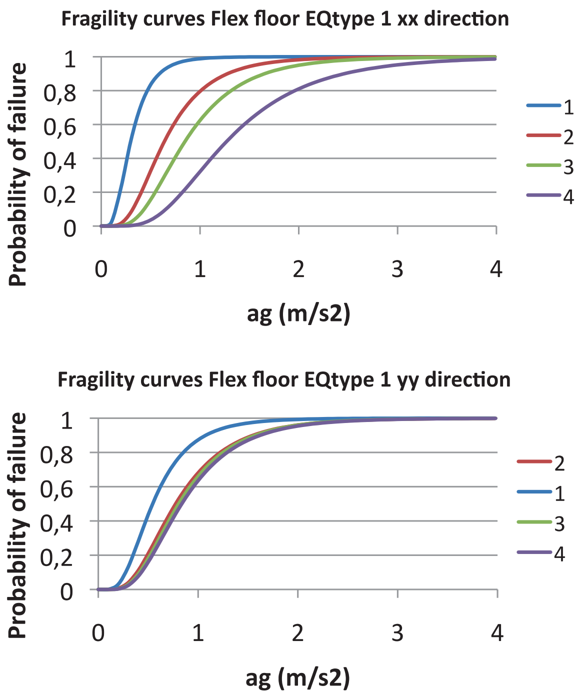

Fig. 40 Fragility curves for earthquake type 1: *xx* direction (*top*) and *yy* direction (*bottom*) [43]

event of high magnitude, richer in larger periods) recommended in the Portuguese national annex of Eurocode 8 [29], for Lisbon.

Finally, Fig. 41 illustrates the damage probability for earthquake type 1(high magnitude, far field event, with low frequency contents) in the *xx* direction for all the studied cases. Discrete damage-state probabilities can be calculated as the difference of the cumulative probabilities of reaching, or exceeding, successive damage states (as computed from the fragility curves):

$$P_0 = 1 - P\,[L_1|\,S_d]\,; \quad P_k = P\,[L_k|\,S_d] - P\left[L_{k+1}|\,S_d\right]\ for$$
$$k = 1{,}2{,}3;\ \ P_4 = P\,[L_4|\,S_d]$$

In Fig. 41 Pr0 represents the probability of having “no damage”, Pr1 the probability of having “slight damage”, Pr2 the probability of having “moderate damage”, Pr3 the probability of having “heavy damage” while Pr4 the probability of reaching “collapse”.

Based on the results obtained, it is clear that building without retrofitting presents the highest value of probability of damage Pr4 (“collapse”). Retrofitting the building by stiffening the floors enables reducing this value significantly. Retrofitting the building by stiffening the floors and including shear walls or steel frames does improve slightly the situation, reducing the value of Pr4 and spreading it more through Pr3 to Pr1. The retrofitting scheme that mostly improves the seismic performance of the building, with respect to the previous cases, is the case

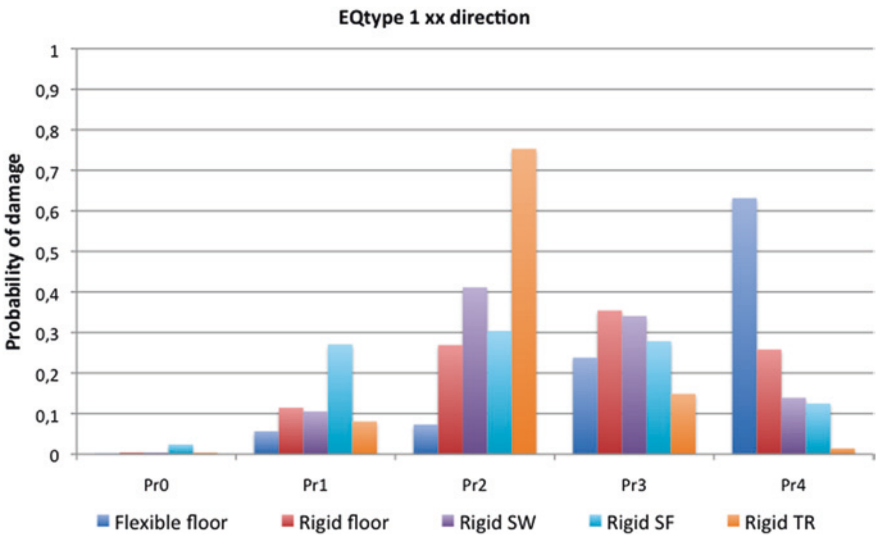

Fig. 41 Probability of damage for earthquake type 1 in the *xx* direction

of the inclusion of tie-rods in the front and back façades. This reduces significantly the damage probability Pr4. Nevertheless, this retrofitting possibility seems to increase very much the damage probability Pr2 when compared to the other retrofitting strategies.

5.4 Analysis of a Pombalino Quarter

Besides modelling buildings and structural elements, it is also worth mentioning a study of an idealized *Pombalino* quarter [49]. The purpose was to get some insight on how the buildings interact with each other, since the gable walls are common to adjacent buildings. For this purpose the model of an idealized quarter was developed based on the design of three real *Pombalino* buildings that constitute one quarter of the entire quarter of buildings, that was then replicated twice, giving rise to a model of a quarter with double symmetry. The wood floors were simulated by a set of bars with axial stiffness in both directions, since it is not expected that the floors exhibit any relevant distortion stiffness, specially under strong seismic actions. The main conclusion is that, even though the floors have no distortional stiffness, they have enough axial stiffness to force the buildings in one band (alignment of buildings) of the quarter to move together, with similar horizontal displacements at each floor level, for the modes with lower frequencies. Figure 42 shows the deformed shapes of the 1st and 2nd modes, that illustrated the band effect.

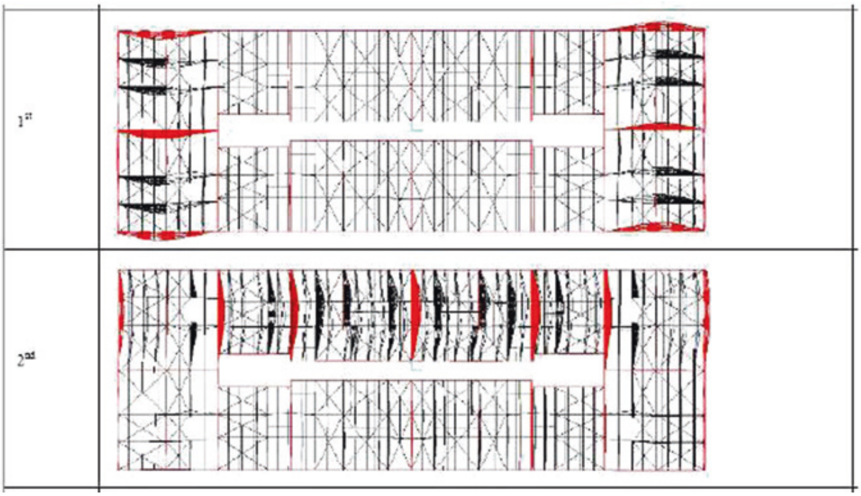

Fig. 42 First and second mode shapes of a *Pombalino* quarter [47]

However, for higher frequency modes, buildings in the same band exhibit deformed configurations with different horizontal displacements at each floor in the direction of the alignment, thus with relevant axial deformation of the floors. These effects show that strengthening one building in the direction of a band may lead to shared improvements on seismic behaviour with other buildings in the same band, at the expenses of less improvements of that building.

As it should be expected, the analysis of the quarter confirms that the buildings cannot rotate freely, as this is restricted due to the fact that they share the gable walls. In this context it does not make sense to consider accidental eccentricities in strengthening design of these buildings, such as the ones prescribed by several codes for the design of new buildings (assumed isolated).

6 Economic Feasibility of Strengthening

The economic feasibility of maintain and strengthen *Pombalino* buildings depends on the capacity to adapt the buildings to new functions or to the same functions but with different demands. For instances *Pombalino* buildings had very small rooms with 5 or 6 m^2, no lifts, etc. Some of these characteristics are not compatible with nowadays activities and architectural requirements. For instances office buildings require larger areas, and are not a solution for the whole downtown, as it would become desert at night and weekends, which is not desirable. Local authorities also want downtown Lisbon to have a life of its own, therefore part of the *Pombalino* buildings should be used for housing purposes. It is therefore important to adapt these buildings to modern uses and functions according to modern standards. Besides, this adaption is critical for the buildings to provide some income and stimulate the private sector to contribute to rehabilitation and strengthening works.

The adaptation to new uses and functions requires larger rooms, pointing to solutions that may imply to remove some interior walls. Even though this is a debatable issue, it is the opinion of the authors that economic of preservation fundamentalisms are not the best option, but one must be very demanding and assertive and try to compatibilize both criteria. In this framework it may be arguable that the secondary may have to be sacrificed to preserve the essential. The issue is what is essential and what is secondary. The *Gaiola* can be considered essential, in general. This is due not only to its symbolism and associated cultural value, that will be discussed further in the next section, but also due to the fact that due to its good state of conservation and its structural capacity it can still contribute significantly to the buildings seismic capacity. This may be relevant to reduce or avoid the need for widespread works throughout the buildings, as it could be the case without the *Gaiola*, for instances in some or many *Gaioleiro* buildings. On the other hand the partition walls called *tabiques*, as the one shown in Fig. 4, can be considered as secondary, both from the structural as well as historic and cultural heritage points of view: they have no special characteristics that distinguish them from partition walls in other types of masonry buildings, and have much less strength and stiffness than the frontal walls. Therefore the removal of interior walls to create larger spaces should follow strict criteria for the preservation of the most relevant characteristics of the buildings (from the point of view of cultural heritage) and should not be done only according to architectural criteria related to future uses of the buildings. The removal of some partition walls may weaken slightly a building, what can be compensated, and allow an architecture more compatible with the uses and functions of nowadays without reducing the historic and cultural value of the buildings [50]. However this issue deserves a deeper analysis and debate, not only by architects, engineers and promoters, but by the whole society, as urban rehabilitation and the preservation of the cultural heritage is an issue that interests the whole society, and not only the main economic and technical agents involved in design and construction.

A common problem that arises from the adaptation of old buildings to new functions is the cut of façade columns at ground floor levels aiming at creating larger spaces for shop windows. This is not acceptable, given the potential consequences. Therefore the actual and future owners of shops in zones and/or buildings of relevant cultural value should assume that if they want to keep or set-up a business in such zones they must accept some restrictions to the changes that can be done in those buildings, namely that cut ground floor columns is not acceptable. This does not seem to be a problem difficult to solve or that creates incompatibilities with most modern uses. There are in downtown Lisbon several good examples of integration of shop windows with ground floor columns, for instances by using the columns as supports for shelves or just leaving the columns between exterior accesses. Figure 43 shows two examples of compatibility between original structure and modern uses and functions.

Fig. 43 External accesses and window shops with integration of original columns [21]

7 Cultural Heritage

The reconstruction of downtown Lisbon after the destruction caused by the great Lisbon earthquake of 1755 was the first time in the History of mankind that a large town, a European capital, was built with techniques aiming at the explicit purpose of providing seismic resistance. These include for instance the *Gaiola*; the fact that the buildings were built in blocks and had similar characteristics such as the same number of floors and were generally symmetrical; the fact that they had regular openings in the front and back façades, were robust and had good quality of construction and besides they had thick masonry exterior walls surrounding the *Gaiola*. The *Pombalino* architecture was also austere with no useless decorative features, especially on the façades.

Therefore Lisbon downtown is a part of mankind′s cultural heritage, a landmark that must be preserved and transferred to future generations in safe conditions and preserving the authenticity of its buildings. This is of the interest of the Portuguese people and authorities that should also promote the international recognition of downtown Lisbon historical and cultural value.

The need to preserve/improve safety standards in what regards the earthquake resistance of nowadays *Pombalino* buildings is not incompatible with the interests on the preservation of the original structure, as it continues to offer a relevant contribution to the buildings earthquake resistant capacity, as it was previously referred to: the seismic resistance of original *Pombalino* can be above the actual Portuguese code prescribed value of the seismic action, a noticeable fact for 250 years old buildings.

However, the compatibility of the preservation of the most important characteristics of the original structure with the requirements of modern uses and functions requires the art and skill of architects and engineers, and the acceptance some restrictions to architectural changes by financial institutions, public authorities, owners, tenants and all other agents involved in urban rehabilitation.

8 Synopsis

The reconstruction of Lisbon and other Portuguese southern towns after the Great Lisbon Earthquake of 1755 was performed with the major concern of avoiding future catastrophes of the same kind. The buildings where designed with a tridimensional wood truss embodied in the interior masonry walls, that provides resistance to horizontal forces in any direction. The wood truss, called *Gaiola* (cage) *Pombalina*, is a characteristic of these buildings, and its widespread application during the reconstruction of Lisbon was the first case in history of an entire town built with the purpose of providing seismic resistance to its buildings.

Other characteristics of these buildings are also described: their regular distribution in quarters within a rectangular mesh of streets, the same number of floors for all buildings, the foundation system that includes short wood piles, the lack of the *Gaiola* in the ground floor to isolate it from the water in the soil as the water table is very near the surface, the wood pavements in all floors except on the first one, that is made of masonry for fire protection, the gable walls belonging simultaneously to two adjacent buildings and the industrialization and systematization of the construction process for mass production, etc. However after the generation that lived the earthquake was gone, progressive adulteration of these characteristics took place, such as the addition of more floors, addition of heavy decorative elements of the façades, removal of the diagonals of the *Gaiola* and poorer workmanship and poorer connections between elements. This process continued and was aggravated more recently, during the twentieth century, with the removal of ground floor columns for window shops, insertion of water and gas pipes inside the *Gaiola* walls, removal of entire *Gaiola* panels to create larger spaces, etc.

Recently the scientific interest by this type of buildings has increased and several studies to evaluate their seismic resistance were performed. It was concluded that the original buildings would probably possess the capacity to resist the seismic action prescribed for new buildings by the current Portuguese code for actions in structures, without considering the 1.5 seismic factor prescribed by the code. Even though this is less than the resistance of many modern buildings, it is a remarkable result considering the materials and knowledge available 250 years ago. However the reality is that most *Pombalino* buildings don′t meet these standards due to the negative alterations during the nineteenth and twentieth centuries. Some strengthening techniques used to increase the seismic resistance of these buildings, taking advantage of the *Gaiola Pombalina* are described. Some advanced modelling techniques, able to simulate the nonlinear behaviour of this type of buildings, are described and some results shown. Since this buildings share the gable walls, there are no expansion joints between buildings and they interact within each quarter. Results of the analysis of these effects on the dynamic behaviour of an entire quarter are presented. The economic feasibility of strengthening these buildings, as well as the interest in their preservation due to their historical value are briefly discussed.

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