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Seismic assessment and retrofitting of Pombalino buildings by pushover analyses

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Abstract. The heritage value of the mixed wood-masonry 18th century Pombalino buildings of downtown Lisbon is recognized both nationally and internationally. The present paper focuses on the seismic assessment of global response and retrofitting of a typical Pombalino building by nonlinear static analyses, performed by the research software Tremuri, which is able to model 3D configurations. The structure is modelled using nonlinear beams for masonry panels, while in case of the internal walls (frontal walls) an original formulation has been developed in order to take into account their specific seismic behaviour. Floors are modelled as orthotropic membrane finite elements: this feature allows to simulate the presence of both flexible and rigid diaphragms, being the first ones more representative of the original state while the second ones of retrofitted configurations. Seismic assessment has been evaluated by applying nonlinear static procedure and comparing the performance of different configurations (by considering various retrofitting strategies). Finally, assuming a lognormal cumulative distribution, fragility curves are obtained to be representative of Pombalino buildings: the most important application of such curves is for seismic risk and loss estimation analyses.

Keywords: pombalino buildings; equivalent frame model; retrofitting; nonlinear static analysis; fragility curves

1. Introduction

The heritage value of the mixed wood-masonry 18th century *Pombalino* buildings in downtown Lisbon is recognized both nationally and internationally. In 1755 a catastrophic earthquake followed by a major tsunami struck the capital of Portugal causing severe damage to the city. The Prime Minister at the time, Marquis of *Pombal,* was set in charge of rebuilding the city and he delegated to a group of engineers the development of a structural solution that would guarantee the required seismic resistance of the buildings. Based on the know-how of that time and on the empirical knowledge gathered from the buildings that survived the earthquake, a new type of

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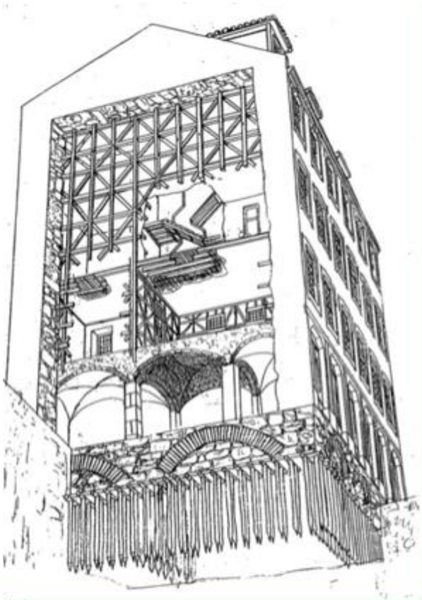


Fig. 1 Example of a Pombalino building (Mascarenhas 2005)

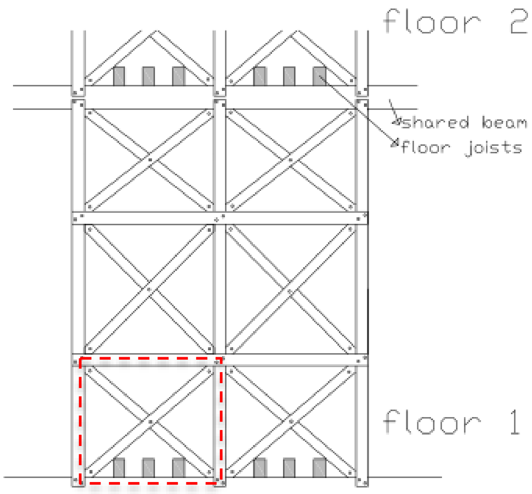


Fig. 2 Drawing of a frontal wall and its connection to the above floors (by dotted lines the modular structural scheme is marked, that is composed by the horizontal and vertical timber elements and the double diagonal bracing)

construction was created, which is now generally referred to as *Pombalino* construction. An example of the construction elements that compose a *Pombalino* building can be seen in Fig. 1. A more detailed description of this building typology can be found in Cardoso *et al*. (2005).

Based on Mascarenhas (2005), the following can be said. The foundation system was ingenious; it is based on a system of wooden piles over the alluvium layers. The piles are similar and repetitive, on average 15 cm in diameter and 1.5 m in length. These form two parallel rows in the direction of the main walls, which were linked at the top by horizontal wood cross-members attached by thick iron nails. The construction at ground floor consisted of solid walls and piers linked by a system of arches. In more elaborate cases, thick-groined vaults spanned between the arches, which protected the upper floors from the spread of any fire that might start at ground floor level. From the first floor up the basis of this building system is a three-dimensional timber structure called *gaiola* (cage), thought to be an improved system based on prior traditional wooden houses. The *gaiola* is composed of traditional timber floors and improved mixed timber-framed masonry infilled walls/panels (called *frontal* walls for *Pombalino* buildings) that would support not only the vertical loads but also act as a restrain for the seismic horizontal loading. *Frontal* walls consist of a wooden truss system filled with a weak mortar in the empty spaces (Fig. 2): they are one of the main speciousness of these buildings. Finally, the buildings are encompassed by façade and gable walls made of stone and rubble masonry. These walls decrease thickness in height. The gable walls are shared between adjacent buildings. Also, the buildings were built in quarters comprising each quarter an average of 10 buildings.

Due to the high heritage value of this constructive system, reliable tools to assess its seismic vulnerability in such prone seismic area are fundamental. According to the widespread of performance-based earthquake engineering concepts, which research trends and various international and national codes (Eurocode 8 2005, Italian Code for Structural Design – NTC 2008, ASCE/SEI 41/06 2006) now refer to, the possibility to perform nonlinear analyses becomes common, especially in research works and this is also relevant in case of masonry buildings. Of course, 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. Despite this, as regard the modelling of *frontal* walls, only some analytical models (e.g. Doudomis 2010, Kouris and Kappos 2012) have focused on these timber-framed masonry infilled walls to simulate and understand their behaviour. Nevertheless, the actual role of the *frontal* walls in supporting horizontal seismic loads and affecting the global behaviour of *Pombalino* buildings still requires additional studies.

Moreover, 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. As regard to these issues, several studies stressed the potential vulnerability of *Pombalino* building to local mechanisms. For example, Ramos and Lourenço (2004) analysed a Pombalino quarter through the finite element method and nonlinear analyses highlighted the out-of-plane failure of the perimeter walls; moreover, also in Cardoso *et al*. (2005) this issue has been pointed out. Despite the high vulnerability to type b) of seismic response, in general, it has to be stressed how in most of cases these mechanisms can be efficiently inhibited through specific seismic retrofitting interventions (like the tie rod insertions or the improvement of connections between floors and masonry walls). In Cardoso *et al*. (2005) a discussion about the effectiveness of different retrofitting strategies is proposed.

In the following, the attention is focused only on the global response in order to examine the role of different constructive systems present in supporting the seismic actions and to quantify the actual vulnerability of such building type. Thus it is assumed that local mechanisms are inhibited through proper constructive details or interventions: thus, for example, it is assumed that *frontal* walls are properly attached to masonry façades preventing their out-of-plane failure or tie-rods are present.

In the paper, the modelling and the seismic assessment of a typical *Pombalino* building has been performed with the structural software Tremuri (Lagomarsino *et al*. 2013). In particular, it works according to the equivalent frame approach and focuses only on the global building response (which is assumed to be governed only by the in-plane behaviour of walls – type a). Indeed, among the different modelling strategies proposed in literature, due to the regular pattern of openings in *Pombalino* building, the equivalent frame approach seems particularly suitable; moreover, it agrees with recommendations of both national and international codes (e.g. Eurocode 8 2004). The program enables nonlinear static and dynamic analyses to be performed. The structure is modelled by using nonlinear beams for the ordinary masonry panels and a specific formulation for the *frontal* walls (Meireles *et al*. 2012a and 2012c). By using Tremuri, pushover analyses were performed on different configurations of the examined building aimed to simulate the effect of various strengthening techniques. On basis of the capacity curves obtained, the seismic assessment has been evaluated by applying the nonlinear static procedures proposed in Eurocode 8 (2004). Finally, assuming a lognormal distribution probability function, fragility curves were obtained as well as damage probability plots in order to compare the effectiveness of different interventions analysed. Main aim of such probabilistic assessment is to provide useful information for risk scenario at territorial scale and seismic loss estimation studies.

2. Equivalent frame modelling approach

The equivalent frame approach (Fig. 3) 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). This strategy seems particularly suitable in the case of *Pombalino* buildings characterized by a quite regular opening pattern on walls 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 2001 (Galasco *et al*. 2004, Lagomarsino *et al*. 2013, Lagomarsino *et al*. 2012), and subsequently implemented in the software package 3Muri (distributed by S.T.A.DATA s.r.l.). The reliability of this program has been tested on several applications. For instance, in Pujades *et al*. (2012) and Gonzales-Drigo *et al*. (2013) the seismic risk assessment of buildings of the example district in Barcelona is presented: indeed, from the architectural system point of view, they present several analogies with *Pombalino* buildings. Moreover, recently, in Tremuri program a specific element intended to simulate the response of *frontal* walls has been implemented (Meireles *et al*. 2012a). According to this program, starting from the equivalent frame modelling of single walls, complete 3D models may be assembled on basis of following basic hypotheses: a) the construction bearing structure, is identified with walls (considering only their in-plane contribution) and horizontal diaphragms (roofs, floors or vaults,

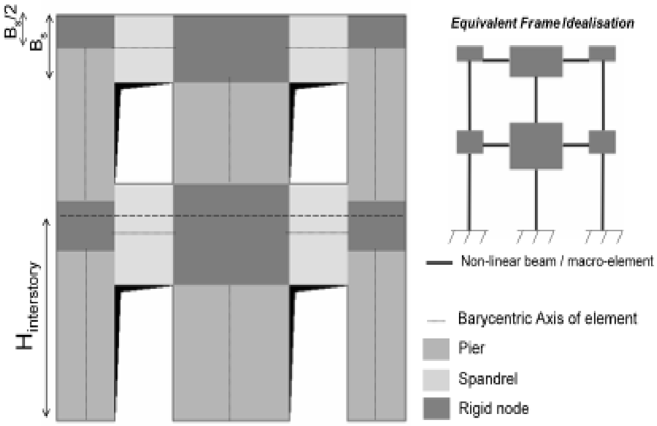


Fig. 3 Equivalent frame idealisation of a masonry wall (Lagomarsino *et al*. 2013)

modelled as orthotropic membrane); b)the walls are the bearing elements, while diaphragms are the elements governing the sharing of horizontal actions among the walls. The assembly is obtained condensing the degrees of freedom of two 2-dimensional nodes by assuming the full coupling among the connected walls (Lagomarsino *et al*. 2013). This solution is particularly efficient to reduce the total number of DOF and perform nonlinear analyses with a reasonable computational effort also in case of large and complex building models.

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 (as suggested also in Eurocode 8 2005 and NTC 2008): 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 (Lagomarsino *et al*. 2013). The ultimate strength is computed according to some simplified criteria, which are consistent with the most common ones proposed in the literature (Turnsek and Cacovic 1971, Mann and Muller 1980) and codes (Eurocode 8 2004, Italian Code for Structural Design 2008). As a function of the masonry type that characterize the examined building the most proper criteria have to be chosen by the user; the reliability of these criteria and their use has recently been assessed (Calderini *et al*. 2009). 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 mechanical parameters, which may be quite simply defined and related to results of standard tests (e.g. the compressive strength of the masonry f_{cu}, the diagonal tensile strength of masonry f_t, the tensile strength of block f_{bt}, the parameters characterizing the mortar joints). Further details on URM nonlinear beam and also on more accurate formulations implemented in Tremuri program may be found in Lagomarsino *et al*. (2012 and 2013). In addition to masonry and *frontal* elements, r.c. elements (Cattari and Lagomarsino 2013), steel and wooden nonlinear beam or tie-rods (non-compressive spar elements) may be modelled as well: such elements are particularly useful in this application to simulate the retrofitted configurations (as illustrated in section 3.1).

Finally, the complete 3D model is obtained by introducing also floor elements. In particular, they are modelled as orthotropic membrane finite elements where in particular: normal stiffness ($E_{1,eq}$) provides a link between piers of a wall, influencing the axial force on spandrels; shear stiffness (G_{eq}) influences the horizontal force transferred among the walls, both in linear and nonlinear phases. The parameters which orthotropic membrane are based on maybe calibrated in order to simulate the effects related to different types of diaphragms (like as wooden floor, reinforced concrete slabs or vaults): this constitutes one of the main capabilities of the adopted software in reproducing the actual behaviour of existing buildings for which the basic hypothesis of rigid floor is not appropriate.

2.1 Formulation proposed for frontal walls

To provide a reliable modelling of *Pombalino* building, it is necessary to be able to describe also the nonlinear response of typical *frontal* walls. To this aim, the formulation proposed recently in Meireles *et al*. (2012a) 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 the basis of some available experimental work carried out (Meireles and Bento 2010, Meireles *et al*. 2012a). This was the first to test *frontal* walls built in laboratory under static cyclic shear testing with imposed displacements, where a specific loading protocol was used and vertical loading applied to the specimen by four hydraulic jacks and rods. The objective of this experimental work was to obtain the hysteretic behaviour of *frontal* walls, by means of static cyclic shear testing with imposed displacements. Thus, 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. 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. Fig. 4 shows the assumed hysteresis model of the wall.

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 (2004) 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 (fixed-fixed). By observing Fig. 5 one can see how the *frontal* walls have lower stiffness when compared to a masonry wall of approximately the same size. The masonry wall also fails first.

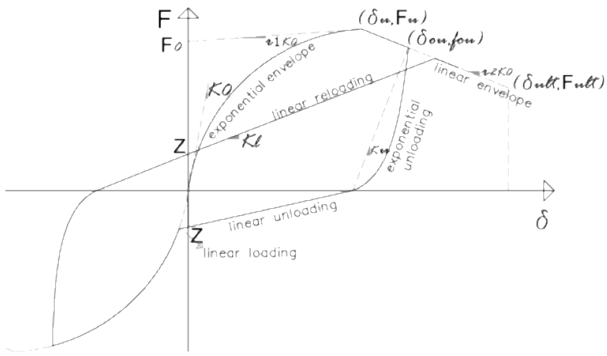


Fig. 4 Hysteresis model of frontal walls (Meireles *et al*. 2012a)

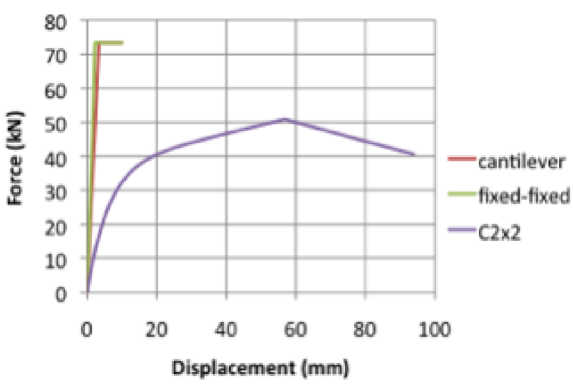


Fig. 5 Comparison between a masonry wall (cantilever and fixed-fixed) and a *frontal* wall (C2 × 2) of the same dimensions

3. Example of application

3.1 Typical Pombalino building and retrofitting solutions

The building that was chosen to be analysed tries to replicate a typical *Pombalino* building. A building was found that had been the subject of research previously (Cardoso 2003, Meireles 2012). This existing building is located at 210 to 220 on the street *Rua da Prata* and 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, Santos 2000). This building is recognized by the existence of a pharmacy on the ground floor, which is covered by a well-decorated panel of blue tiles, dating from 1860. Nevertheless, as usual in the *Pombalino* buildings of downtown, this building has been subjected to some structural 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 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

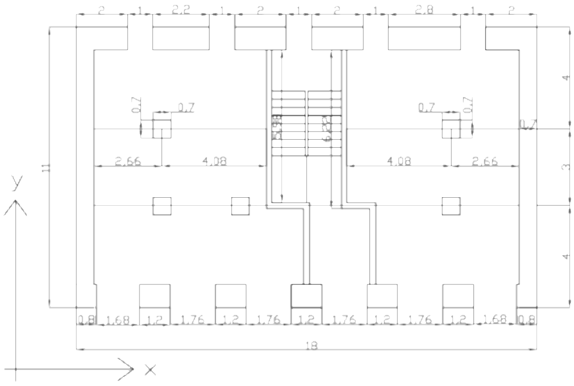


Fig. 6 Sketch of the plan view of building: ground floor – units in metres

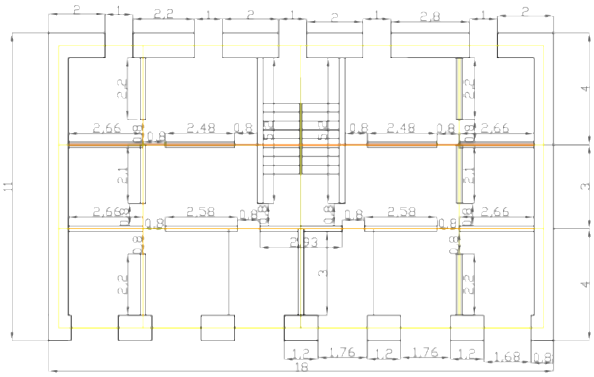


Fig. 7 Sketch of the plan view of building: upper floors – units in metres

the height of the roof). The openings have a width of 1.66 m, the door at the ground floor a height of 3.5 m, the balcony at the first floor a height of 3 m and the windows at 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 the height of the door is 3 m and at first, second, and third floors there are windows of 1.5 m high. There are 5 entries. The plan drawings of the building are shown in Figs. 6 and 7 for the ground floor and upper floors, respectively. The plan of the building has dimensions 18x11 m^2 referred to the façade and gable walls, respectively. The ground floor has 5 internal piers of dimensions 0.7×0.7 m^2. There are stairs in the middle of the building facing towards the back façade. These have brick masonry staircases only at ground floor (at the upper floors the staircases are frontal walls) of thickness 0.24 m. On the ground floor, the staircase brick masonry walls go further until 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 constituted of rubble masonry.

On the upper floors (from the first until 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 openings (doors) of 0.8 m. The actions considered on the structure are the self-weight loads given by the weights of

Table 1 Thickness and material of building components and actions considered

Geometrical data and masonry types		
Element	Material*	Thickness /area
Pillars (ground floor)	SM	0.7x0.7 m^2
External walls (façade and backwards):		
Ground floor	SM	0.90 m
1º floor	RM	0.85 m
2º floor	RM	0.80 m
3º 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

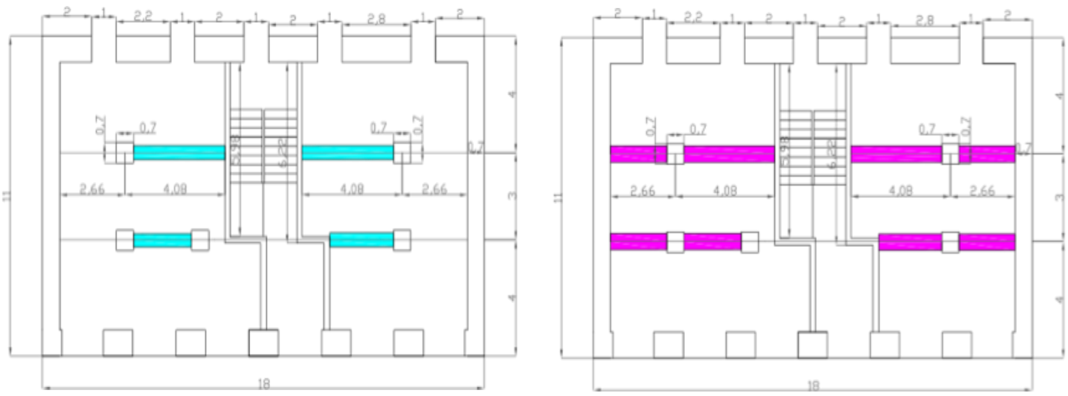
*SM, RM and BM mean stone masonry, rubble masonry and brick masonry, respectively

Actions considered	
Element (Location)	Value**
Floors	2 kN/m^2 (*ll*)
Stairs (stair floor)	4 kN/m^2 (*ll*)
Stairs (stair floor)	0.7 kN/m^2 (*dl*)
Compartment walls (floors)	0.1 kN/m^2 (*dl*)
Wooden floors (floors)	0.7 kN/m^2 (*dl*)
Ceilings (floors)	0.6 kN/m^2 (*dl*)
"Frontal" wall	3.0 kN/m (*dl*)
Roof (Masonry walls 4th floor)	4.4 kN/m (*dl*)
Gable walls roof (Masonry walls 4th floor)	17.3 kN/m (*dl*)

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

(a) (b)

Fig. 8 In-plane stiffening with metallic diagonals and reinforcement of connection floor-wall (a, picture from Edifer) and eccentric bracing core (b, picture from Dunning Thornton Consultants)



(a) (b)

Fig. 9 a) Positioning of the four shear walls (case b) and b) the eight steel frames (case c) on the ground floor – units in metres the roof, the floors, the ceilings, the partition walls and the frontal walls themselves combined with the live loads respectively given by the Eurocode 1 (2001). Table 1 summarizes the geometrical data and masonry types assigned to structural elements and actions considered in the model. From this table, one can see the external walls (façade and backwards) are reducing their thickness towards height, being of 0.90 m on the ground floor and 0.75 m on the third floor.

The joists of the floors have a section of 10x20 cm^2 and the wood boards a thickness of 2 cm. In this basic configuration, the stiffness contribution of floor is mainly related to the boards contribution: thus, they result as quite flexible orthotropic membrane finite elements. The stairs have been modelled as floors having the following cross sections: 10x10 cm^2 for the joists and 2 cm for the pavement. The joists run every 30 cm for both stairs and floors. Depending on the quality of the construction, on the ground floor level there may exist quadripartite vaults, normal vaults or no vaults at all or only timber beams making the ground floor structure. By way of example, only timber floors have been assumed; the cross section of the timber beams (useful to compute the equivalent parameters to be assumed for the orthotropic membrane) considered has a width of 20 cm and a height of 30 cm. In case of vaults, equivalent parameters for the orthotropic membrane could be properly calibrated (as proposed in Cattari *et al*. 2008); indeed, the stiffness properties of vaults are usually representative of quite flexible floors, thus, no significant differences than those discussed in the following are expected in terms of the seismic global behaviour.

The basic configuration previously illustrated refers to an original configuration of a *Pombalino* building. However, it should be noted, that, in reality, a considerable part of the building stock probably exists that is not in its original state but has been subjected to changes that may even worsen its structural system, like removing parts of frontal walls or base piers.

Starting from this basic configuration, to analyse the effect of different retrofitting solutions, the following interventions have been proposed and analysed:

a. “Rigid floor”: increase the in-plane stiffness of floors (transforming flexible floors into more rigid floors). According to solutions aimed to respect the original configuration of timber floors, in Fig. 8a) one can see one possible solution of strengthening floors with steel crossed braces; an other possibility is through the insertion of plywood panels (e.g. as discussed in Brignola *et al*. 2012, in which also analytical expressions to compute the equivalent shear stiffness of wooden floors are proposed).

b. “Rigid SW”: increase the in-plane stiffness of floors plus inclusion of four shear walls on the ground floor. The inclusion of shear walls is a typical procedure for improving the seismic resistance of a building. It was decided that the inclusion of four shear walls on the ground floor be modelled according to the scheme presented in Fig. 9a). The 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 is the most vulnerable and is the weakest direction (after the strengthening of the diaphragms, as discussed at 3.3.3 and 3.3.4).

c. “Rigid SF”: increase the in-plane stiffness of floors plus inclusion of eight steel frames on the ground floor (Fig. 9b). The inclusion of eight steel frames on the ground floor comes 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), modelled as nonlinear elements, 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 same reasons as previously. In Fig. 8b) one can see an example of a retrofitting with steel elements (in this case eccentric) in a masonry building.

d. “Rigid TR”: increase the in-plane stiffness of floors plus inclusion of tie-rods at front and back façades. The input file of the software 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 of 2.4 cm in diameter and made of steel. They are placed along the spandrels, connecting the piers between each other. Being the thickness of spandrels significantly lower than that of piers (0.2 m against to 0.75-0.9 m as illustrated in Table 1), it is hypothesized to place the tie-rods at level of pavement of each floor below the finishing and along spandrels, drilling only piers and avoiding a continuous coring of masonry wall: in this way, such intervention results less invasive and complicated in terms of implementation. An initial strain of 20% the yielding strain of the steel was used. The tie-rods were only placed in the xx direction, the most vulnerable one and that characterized by façade walls with a significant number of openings (thus most affected by such intervention).

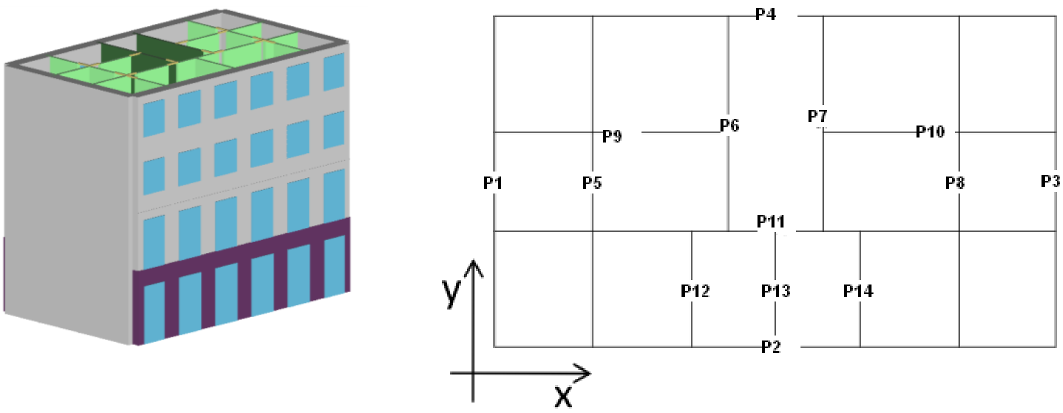
Fig. 9 a) shows the positioning of the four shear walls and b) the positioning of the eight steel frames in the plan view of the building.

3.2 Equivalent frame model of the examined building

The structure is modelled according to the equivalent frame model (by adopting the Tremuri program) using nonlinear beams for the ordinary masonry panels and for *frontal* walls according to the formulation described in 2.1. The final model of the building is presented in Fig. 10 a). 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. The *frontal* walls are composed by a 3x2 modular scheme (see Fig. 2), apart the case of walls P6 and P7 (Fig. 10b) composed by a 3 × 4 scheme (the legend adopted in Fig. 11 is only indicative and not corresponds to the actual scheme of each frontal wall). Fig. 10b) identifies the alignments of the different structural elements in the plan view of the building.

The mesh of panels, that is the equivalent frame idealization, has been created by using the software package 3Muri (release 4.0.5) 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 and for two frontal walls in Fig. 11 a/b) and c/d), respectively: in red are piers; in green are the spandrels and in blue are the parts of the façade where no damage is foreseen (rigid nodes). In the case of back façade, dimensions of portions assumed as rigid are quite large; however, despite the possible consequent overestimation of overall stiffness of the wall, it has to be observed that, as illustrated more in detail at 3.3.1, the seismic response of the façade is far from the shear type idealisation being characterized by weak spandrels and a prevailing contribution of rotations instead of shear strain. Moreover, the use of cracked conditions for defining the stiffness contribution of panels (as specified in the following) contributes also to balance the assumption of some portions as rigid.

Table 2 summarizes the mechanical properties adopted for URM and *frontal* walls. Those related to URM walls have been defined on the basis of ranges proposed in the Italian Code for Structural Design (NTC 2008, MIT 2009) in case of masonry types similar to those of the



(a) (b)

Fig. 10 a) 3D view and b) numbering of the alignments of the elements of the model (Meireles *et al*. 2012c)

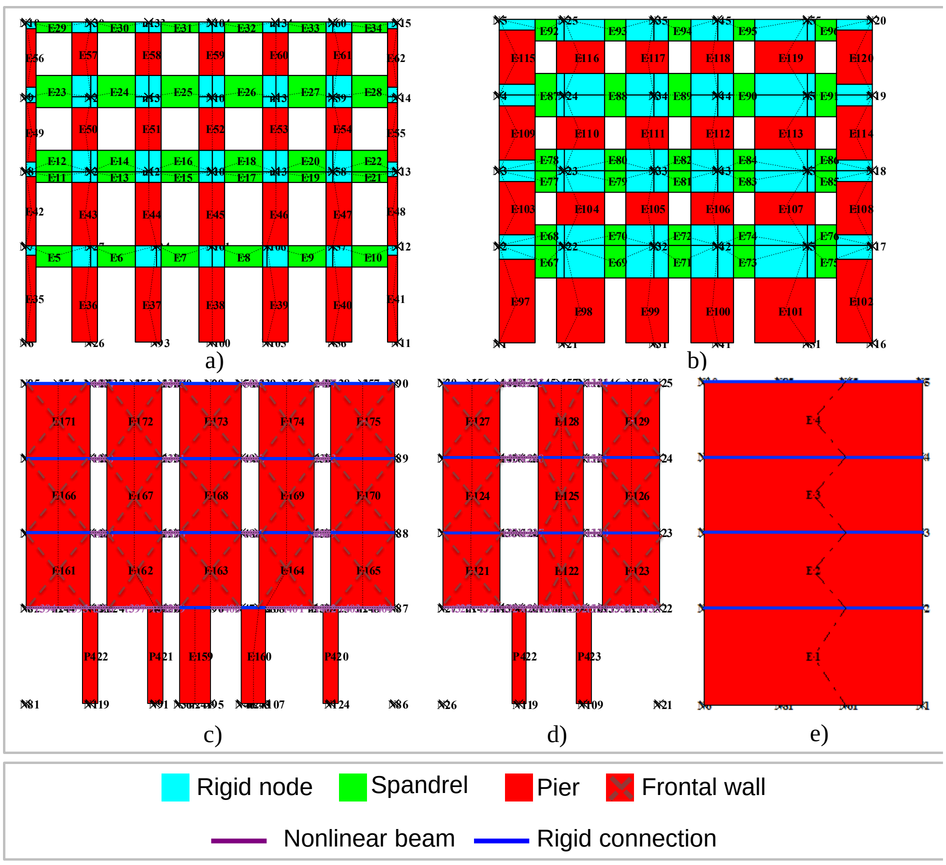


Fig. 11 Equivalent frame idealisation of: a) front façade and b) back façade (Meireles *et al*. 2012c); c) frontal walls (composed by 3x2 modular structural schemes) oriented in X –P11 and d) Y- P5 directions; e) external masonry y-y wall (P1)

Table 2 Mechanical characteristics of masonry types and parameters of *frontal* walls

Masonry type	Young Modulus E [GPa]	Shear Modulus G[GPa]	Weight W [KN/m3]	Compressive Strength f_m [MPa]	ShearStrength $\tau_0^{(+)}$[MPa]
Stone Masonry	2.8*	0.86*	22	7	0.105
RubbleMasonry	1.23	0.41	20	2,5	0.043
Brick Masonry	1.5*	0.5*	18	3.2	0.076
Frontal wall	F_u [KN]	K_0[KN/mm]	r_1K_0	r_2K_0	F_0/F_u
2x2**	50.8	6.1	0.244	-0.2745	0.728
3x2***	49.9	2.9	0.244	-0.2745	0.728
3x3***	68.6	6.8	0.244	-0.2745	0.728
3x4***	90.0	13.6	0.244	-0.2745	0.728

(+) the criterion proposed in Turnsek and Cacovic (1971) has been assumed as reference for the computation of the shear strength

* starting from these values, a factor equal to 0.5 has been applied in order to simulate cracked stiffness conditions

** calibrated on basis of experimental results done previously (Meireles *et al*. 2012a,Meireles and Bento 2010)

*** F_u and K_0 have been obtained for different configurations (2x3, 2x4, 3x2, 3x3 and 3x4) based on analytical models (Meireles 2012)

examined building (that is stone, brick and rubble masonry); in particular, values adopted correspond to the average value proposed in such code in the case of “basic” condition of masonry without the application of any corrective factors a bracket for example associated to the presence of a good mortar quality a bracket: in fact, it seems more representative of the actual condition of the examined masonry. As regard the strength domain assumed for piers and spandrels, those proposed in the Italian Code for Structural Design (NTC 2008, MIT 2009) in case of existing masonry buildings have been adopted; they are based on the criterion proposed by Turnsek and Cacovic (1971) in the case of the shear response and on the beam theory (neglecting the tensile strength of the material and assuming a stress block stress distribution at the compressed toe) in the case of flexural response. In the case of spandrels, if it is coupled to another tensile resistant element (e.g. steel tie rod or RC beam), a strut-and-tie mechanism is assumed to be developed in case of a prevailing flexural response, with a maximum compression force in the spandrel equal to the tensile strength in the coupled element (otherwise the same flexural criterion adopted for piers is assumed). For URM panels a drift limit value of 0.4% and 0.8% (as suggested also in) has been adopted in case of prevailing shear and flexural failure modes, respectively; in case of masonry pillars (that characterize in particular the ground floor) the limit value of 0.8% has been increased until 1.2%.For *frontal* walls the value of F_{ult} (that denotes failure) is taken as 80% of the value of F_u.

In the work of Meireles (2012), an analytical model of the *frontal* walls tested before (Meireles *et al*. 2012a, Meireles and Bento 2010) was performed in order to obtain a structural model that could predict the initial stiffness, K_0, of the walls tested and of walls of other dimensions/configurations (3 × 2 crosses, 3 × 4 crosses, ...) since in reality other walls with different dimensions can co-exist in the same building. To this aim, parametrical analyses have been performed in SAP 2000 (1998) under such assumptions (corroborated by the results of the experimental tests aforementioned): the diagonal of the wall are not able to work under tension; shell elements were used to model the masonry; pinned connections were used at the nodes; rigid links were used to connect the shell elements of the masonry to the diagonal in order to simulate the thickness of diagonals; at the support, springs were used to simulate the effect of rigid body movement (noticed from experimental tests). The calculation of the strength F_u of the tested walls and the prediction of F_u for other configurations were only related to the collapse of the most loaded diagonal (considering the most punishing between the possible failure of connections and the buckling effects). Indeed, the contribution of masonry is mainly related to the definition of the initial stiffness, while it is neglected at collapse since at this stage it is extensively detached from the truss elements and considerably cracked at some locations. Results of calibration done for configurations present in the examined building are summarized in Table 2. For further information please refer to the work of Meireles (2012).

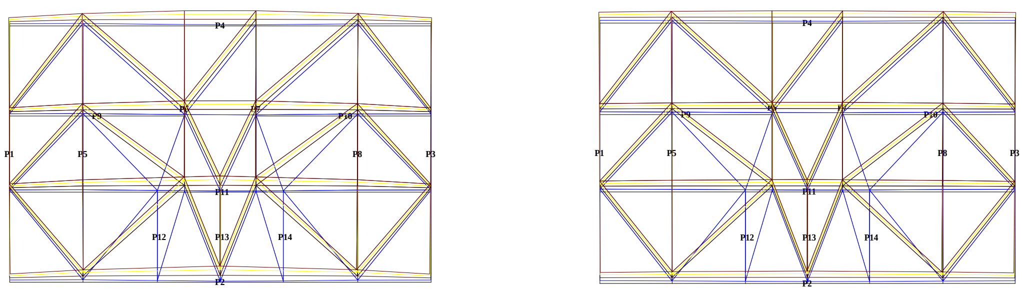
As regard the diaphragms, in the case of basic configuration, the main stiffness contribution has been associated to the boards by assuming equivalent membrane finite elements of thickness equal to 0.02 m and characterized by $E_{1,eq}$ and G_{eq} equal to 52 and 0.75 GPa , respectively. The value of G_{eq} has been increased to 7.5 GPa in the case of rigid floor. The resulting stiffness roughly corresponds to 3% and 30% of that of an equivalent reinforced concrete slab (of same thickness) in the case of the flexible and rigid configurations, respectively.

Furthermore, in this modelling, the foundations are modelled as built-in (no displacement or rotations allowed).

3.3 Seismic assessment by nonlinear static analyses

For each main direction X and Y, nonlinear static analyses were parametrically performed as a function of: the different configurations examined as defined in 3.1; two load patterns (uniform that is proportional to mass, pseudo-triangular that is proportional to the mass and height product).

Before proceeding to the nonlinear static analyses, a modal analysis has been performed in order to check the fraction of participating mass activated by first modes and verify the reliability of adopting a verification approach based on a 3D model for the global response, even in the case of quite flexible floors. Indeed, results showed that the Y direction is the most affected by the floor



(a) (b)

Fig. 12 Plan view of the first modal shape in Y direction in the case of a) flexible and b) rigid diaphragms, respectively and corresponding period values (in seconds) and partecipating mass (%)

stiffening, due to the most pronounced difference in stiffness between external and internal walls. Fig. 12 shows the plan view of the first modal shape in Y direction in the case of both flexible and rigid diaphragms; moreover, the corresponding participating masses (M_Y) are summarized. Values obtained (about 70% in the worst case) confirm as, for the building examined, a 3D model is justified to perform nonlinear static analyses also in the case of flexible floors.

In the following, firstly the results of nonlinear static analyses are examined in terms of comparison of resulting pushover curves and seismic assessment through the application of a non linear static procedure based on the use of inelastic spectra (see 3.3.3); then, the effects of strengthening solutions are discussed by comparing the results in terms of probabilistic seismic assessment through the introduction of fragility curve concept (see 3.3.4).

3.3.1 Comparison of results in terms of pushover curves

Pushover analysis enables us to have an idea of the lateral resistance of a building: the output in terms of overall base shear versus the average of the displacements of the nodes at the top floor is presented in Fig. 13. 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. In fact, in this latter case, 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 pushover nodal force) in order to consider a result representative of the whole structure and not only of some local portions (Lagomarsino and Cattari 2009).

Pushover analyses performed on this basic configuration showed a significant difference between the seismic capacity of the building in X and Y directions, in particular: the stiffness and strength is much higher in the Y direction than in the X direction; but on the other hand, the ductility of the system is much higher on the X direction and is practically non-existing in the Y direction. In fact, in X direction piers are very slender (due to the opening's configuration) and with a very moderate coupling provided by spandrels (which show a "weak" behaviour due to the lack of other tensile resistant element coupled to them): thus, a prevailing flexural response occurs associated to higher drift than in case of the shear failure. In general, at the final stage, 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, neither of the two directions seems to provide an effective system against the earthquake, as stressed at paragraph 3.3.3.

Starting from the original configuration, mechanical parameters of orthotropic membranes aimed to simulate floors have been increased to simulate such type of intervention (retrofitting solution a- Rigid Floors). Figs. 14 and 15 show the resultant pushover curves (the uniform load pattern was used in the following graphs), in X and Y 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 X direction as an example (Fig. 14), 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 afunction of the respective top displacement of that alignment ("P2" and "P4" legend) and; thirdly, the base shear corresponding to the alignments of the *frontal* walls as a function of the respective

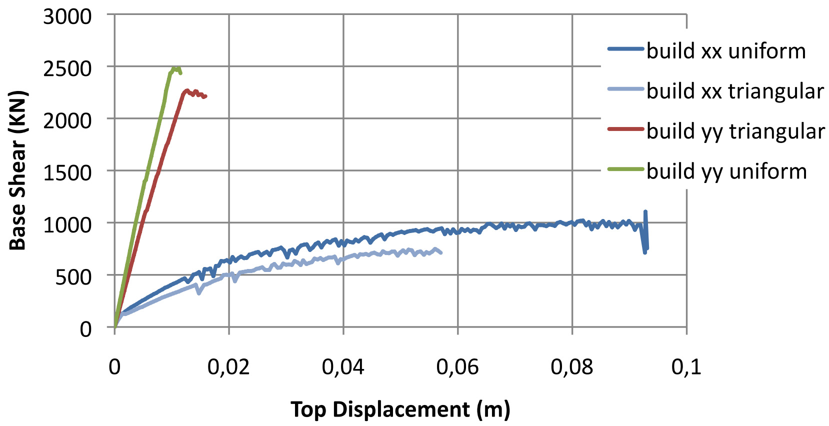


Fig. 13 Pushover curves in the two directions for both uniform and pseudo-triangular load patterns (flexible diaphragms)

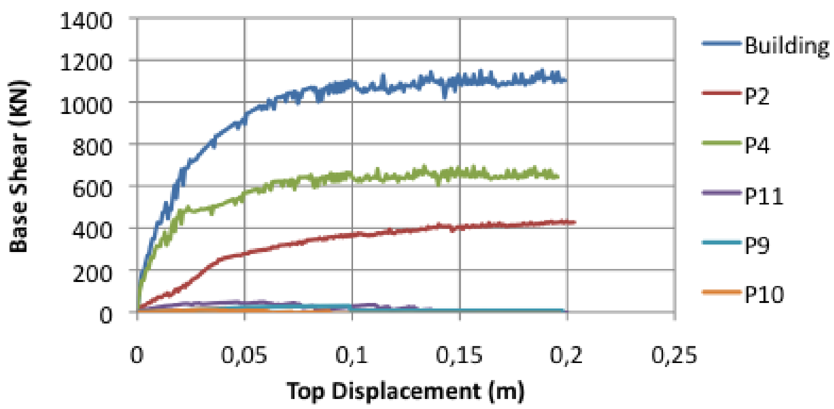


Fig. 14 Pushover curves, contribution of each wall to the base shear, X direction (rigid diaphragms)

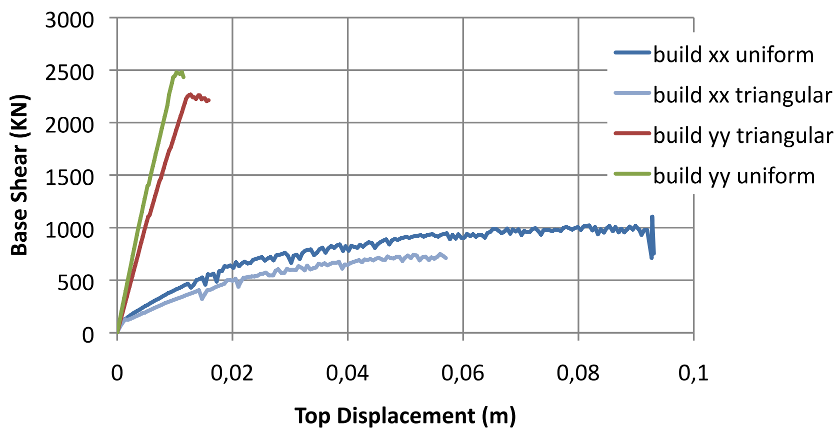


Fig. 15 Pushover curves, contribution of each wall to the base shear, Y direction (rigid diaphragms)

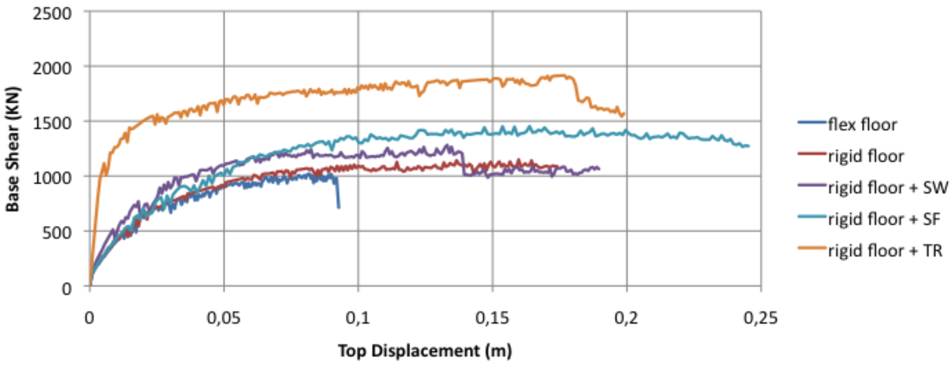


Fig. 16 Pushover curves for all the examined configurations

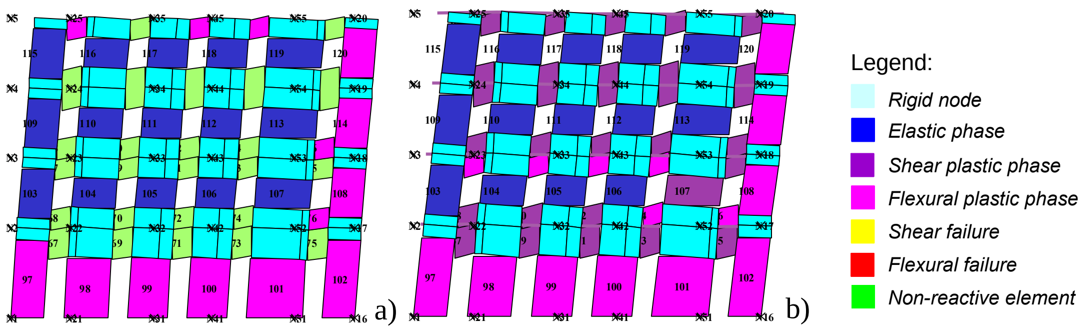


Fig. 17 Damage pattern occurred in the back façade in case of flexible diaphragms (a) and Rigid TR configuration (b), X direction (for a value of top displacement equal to 0.08 and 0.15 cm in cases a) and b) respectively)

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 is not negligible but is very small. In other words, the *frontal* wall alignments contribute very little to the total base shear of the building, the majority of this force 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 and also because of their lower stiffness when compared to the masonry walls. Indeed, from the comparison between a single URM panel and a *frontal* wall illustrated in Fig. 5, one can conclude that the stiffness of the *frontal* wall is much lower when compared to the thick (see Table 1) surrounding masonry walls of the *Pombalino* buildings.

Fig. 16 shows the comparison among the pushover curves obtained for all the different building configurations examined.

In the case of Rigid TR configuration a significant increase of both initial stiffness and strength may be noticed. It may be mainly attributed to the modification in the response of spandrels (due to the coupling with tensile strength elements constituted by tie rods) and the consequent improved coupling effectiveness provided to piers. Fig. 17 clarifies the variation occurred in the global response by examining the damage pattern of the back façade for two different values of top displacements. In fact, while in the basic configuration, spandrels are very weak and starting from the pseudo-elastic phase of building response have collapsed according to a flexural mechanism (then being non-reactive and coupling only the horizontal displacement of piers), in the case of Rigid TR configuration they exhibited a prevailing shear failure that occurred only with the progressing of global non linear response.

Indeed, in the other configurations, the adoption for spandrels of the same flexural criterion of piers may lead to a slight underestimation of the actual strength of spandrels related to the flexural response; in fact, as testified by recent experimental tests and numerical studies (e.g. Beyer 2012 and Cattari and Lagomarsino 2008), the interlocking phenomena that may develop at end section of spandrels and the interaction with architrave elements may affect their response. Indeed, the modelling of spandrels is quite debated in literature (e.g. in Calderoni *et al*. 2011) and still represents an open issue which the increasing number of experimental tests carried out in last decade offers an essential support (Gattesco *et al*. 2008, Graziotti 2011, Beyer and Dazio 2012). Despite this, in the paper, the criteria coherent with those proposed in the current codes have been conventionally assumed as reference.

In general, the different configurations examined vary in terms of strength, stiffness and ductility. Since all these three aspects play a fundamental role in the seismic assessment, a more effective discussion is illustrated in the following at 3.3.3 and 3.3.4 by introducing also the comparison with the seismic input demand (3.3.2).

3.3.2 Seismic demand

The seismic action for the downtown area of Lisbon is here presented. In Portugal, for the design and assessment of structures one must consider two types of seismic actions:

- Seismic action type 1 corresponding to a scenario of faraway earthquake;
- Seismic action type 2 corresponding to a scenario of nearby earthquake.

For each seismic action it should be selected a seismic zone depending on where our structure is located. For Lisbon city, and for normal residential buildings, the seismic zone 1.3 is defined for seismic action type 1 with design ground acceleration on soil type A (ag) of 1.5 m/s^2 and for seismic action type 2, seismic zone 2.3 is chosen with a_g=1.7 m/s^2. The parameters for defining the configurations of the response spectra for the two types of seismic action, the referred seismic zones and for ground type C are presented on Table 3, based on Eurocode 8 with national annex (2004). The defined ground types for Lisbon downtown were A and C (Meireles and Bento 2008). The ground type C was chosen since it is the most demanding situation corresponding to Lisbon downtown soil type.

The corresponding elastic response spectra for Lisbon downtown can be seen in Fig. 18 for the two types of seismic action. In general, for most of the range of periods of the structure, the seismic action type 1 (EQtype1) is the most demanding but, for the range of low values of the period (high frequencies), the seismic action type 2 (EQtype2) is seen to be more demanding.

Table3 Elastic response spectrum parameters for the seismic action (type 1 and seismic zone 1.3 and type 2 and seismic zone 2.3, ground type C)

Seismic action type and eismic zones	Ground Type	S	T_B (s)	T_C (s)	T_D (s)
Type 1 - zone 1.3 (a_g=1.5 m/s^2)	C	1.5	0.1	0.6	2
Type 2- zone 2.3 (a_g=1.7 m/s^2)	C	1.5	0.1	0.25	2

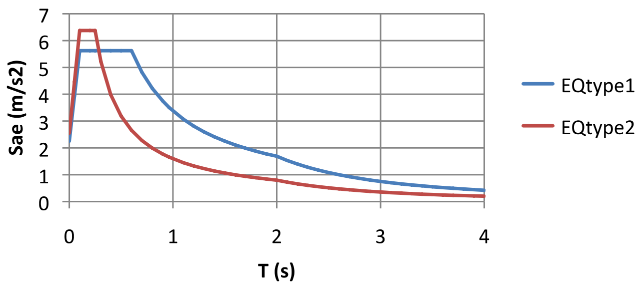
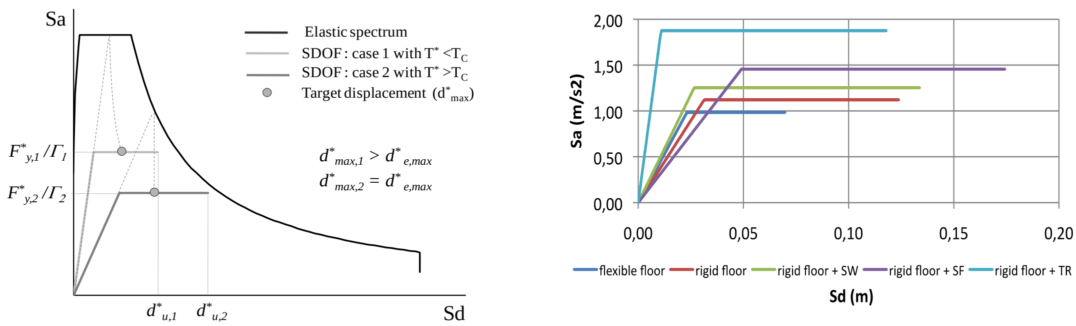
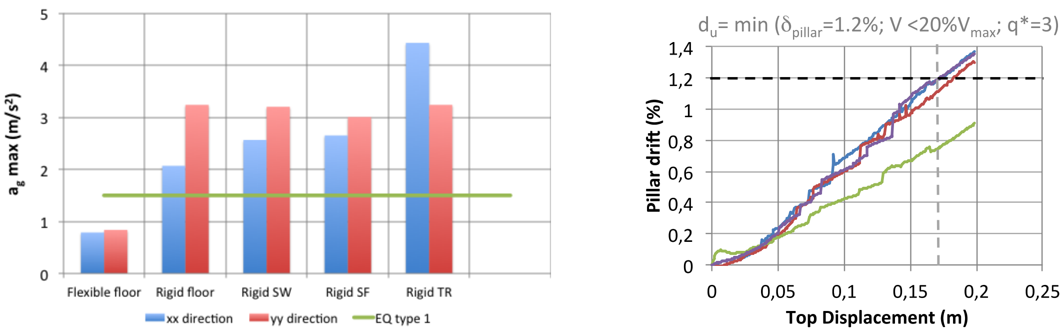


Fig. 18 Elastic response spectra for Lisbon downtown (Legend: EQtype1: Seismic action type 1; EQtype2: Seismic action type 2)



a) b)

Fig. 19 a) Identification of performance point by nonlinear static procedure (Cattari and Lagomarsino 2013);b) Resulting idealized SDOF curves for the examined cases (X direction – Uniform load pattern)



a) b)

Fig. 20 a) Comparison of the seismic assessment resultant for the different configuration examined in terms of a gmax; b) evolution of drift on pillars at ground floor (X direction – Rigid floor)

3.3.3 Definition of capacity curves and seismic assessment

Starting from pushover curves representative of the original multi degree of freedom, they have been properly converted into equivalent SDOF systems; to this aim, the criteria originally proposed by Fajfar (2000) and also adopted in Eurocode 8 (2004) have been assumed as reference.

Fig. 19 a) illustrates the procedure for the identification of the perfomance point according to the N2 Method (Fajfar 2000, Eurocode 8 2004) as known, based on the use of inelastic spectra. Fig. 19 b) shows the resultant idealized elasto-perfectly plastic capacity curves for the examined configurations.

In order to compare the outcome of the seismic assessment for the different configurations examined, a comparison of the maximum acceleration compatible with the fulfillment of the ultimate limit state (a_{gmax}) was done, as illustrated in Fig. 20. It represents a synthetic parameter that allows including at the same time the response of the structure - in terms of strength, stiffness and ductility- and the comparison with the seismic demand. The value of a_{gmax} has been estimated according to the Eurocode 8 expression for target displacement by equating the latter with the ultimate displacement capacity of the structure (d_u). The value of d_u has been assumed as the worst among the displacements on the pushover curve corresponding: i) to 20% decay of the maximum base shear reached (V_{max}); ii) to the attainment of a drift value in pillars (δ_{pillar}) at ground floor equal to 1.2%; iii) to the limitation of the q* (ratio between the acceleration in the structure with unlimited elastic behavior and in the structure with limited strength) factor equal to 3. Condition ii) has been introduced since the base shear contribution carried out by internal walls is quite limited (as evident also from Figs. 14 and 15) and significant damage here occurred could not correspond to a significant overall base shear decay. Condition iii) aims to limit the overall acceptable ductility of building as recommended also in Italian Structural Code (NTC 2008).

Then, resultant values of a_{gmax} are compared with the design ground acceleration corresponding to the Type 1- seismic demand as introduced in 3.3.2. The very high vulnerability of the original configuration is evident: all the other configurations examined seems provide a satysfing security level. Thus, in order to analyze more in detail the effectiveness of these possible solutions, a comparison in terms of damage probability distribution is discussed in 3.3.4 by introducing the fragility curve concept.

3.3.4 Fragility curves of Pombalino buildings

In order to compare the seismic performances of the different analysed configurations in a more effective way in probabilistic terms, results are discussed by computing the damage probability distributions resultant from the seismic input fixed as reference. To this aim the fragility curve concept and conventional criteria to define damage states on the capacity curve have been introduced. Aim of this section is also to provide useful information for assessment on building stocks characterized by homogeneous behaviour for a seismic loss estimation. In this case, results assume a statistical meaning in order to perform a risk analysis, that is to evaluate the probability by having certain consequences on the examined area (country, region, town …). Usually, fragility curves are defined by lognormal functions that describe the probability of reaching, or exceeding, a defined damage state, given deterministic (median) estimates of spectral response (for example the spectral displacement); the variability and uncertainties associated with capacity curve properties, damage states, model errors and ground shaking should be properly taken into account. In particular, the conditional probability $P[ds \mid S_d]$ of being in, or exceeding, a particular damage state (ds), given the spectral displacement S_d, is defined by the following expression (Eqn 1):

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}}\right]\ln\left(\frac{S_d}{\overline{S}_{d,ds}}\right) \tag{1}$$

where: Φ is the standard normal cumulative distribution function; β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state ds; $\overline{S}_{d,ds}$ is the median value of spectral displacement at which a building reaches the threshold of damage state ds. Once known the $\overline{S}_{d,ds}$ values, then it is immediate to express fragility curves also in terms of other variables, such as for example the corresponding peak ground acceleration.

Then, the definition of damage states on the capacity curves is discussed. The damage scale used in this work includes four levels of damage (plus the case of no damage): slight damage (1), moderate damage (2), heavy damage (3) and collapse (4). Damage limit states $S_{d,k}$ (k=1 to 4) are directly identified on the capacity diagrams in AD format as a function of the yielding displacement, S_{dy}, and the ultimate displacement, S_{du}(Eqn 2). These are based on the proposal present in Lagomarsino and Giovinazzi (2006):

$$\begin{aligned} S_{d,1} &= 0.7 S_{dy} \\ S_{d,2} &= 1.5 S_{dy} \\ S_{d,3} &= 0.5\left(S_{dy} + S_{du}\right) \\ S_{d,4} &= S_{du} \end{aligned} \tag{2}$$

Slight damage (1) indicates a condition still far from the reaching of the maximum strength and corresponds to local damage in few structural elements. Moderate damage (2) corresponds to the maximum value of the restoring force in the pushover curve, and is located, in terms of spectral displacement, after the yielding condition of the equivalent bilinear. Collapse (4) is defined on the basis of the ultimate displacement conditions for structural walls. Finally, heavy damage (3) lies in an intermediate position between moderate damage and collapse.

Regarding the definition of β_{ds}, as previously introduced, this parameter summarizes the variability and uncertainties associated with different factors; in particular, it may be expressed as (Eq. 3):

$$\beta_{ds} = \sqrt{\beta_C^2 + \beta_D^2 + |\, \beta_{ls}^2 + \beta_\varepsilon^2} \tag{3}$$

where respectively: β_C represents the variance related to the variability of the capacity curve (i.e. related to that of the mechanical or geometrical parameters which affect the global response); β_D represents the variance related to the variability associated to the seismic demand; β_{ls} describes the variability of the threshold of damage state and β_ε summarizes the model error.

In particular, the following assumptions have been adopted: β_C was assumed equal to 0.35; β_ε and β_D were assumed equal to 0.2 and 0.25, respectively on a heuristic basis; β_{ls} was computed by considering a discrete damage state distribution like that proposed in Pagnini *et al*. (2011). As regard β_C, it is worthy stressing that it has been assumed to be representative of a whole class of buildings with homogeneous behaviour (in fact, at scale of a single asset a lower variability is expected); this value has been adopted from

Table 4 Resultant values of β_{ds} as a function of the different building configurations and directions (X and Y) examined

Building Configuration	Damage Limit State						
	1	2		3		4	
	X/Y	X	Y	X	Y	X	Y
Flexible floor	0.541	0.504	0.497	0.486	0.474	0.484	0.474
Rigid floor	0.541	0.512	0.586	0.493	0.523	0.486	0.492
Rigid SW	0.541	0.529	0.607	0.503	0.523	0.489	0.492
Rigid SF	0.541	0.508	0.586	0.491	0.519	0.486	0.491
Rigid TR	0.541	0.607	0.607	0.525	0.523	0.493	0.492

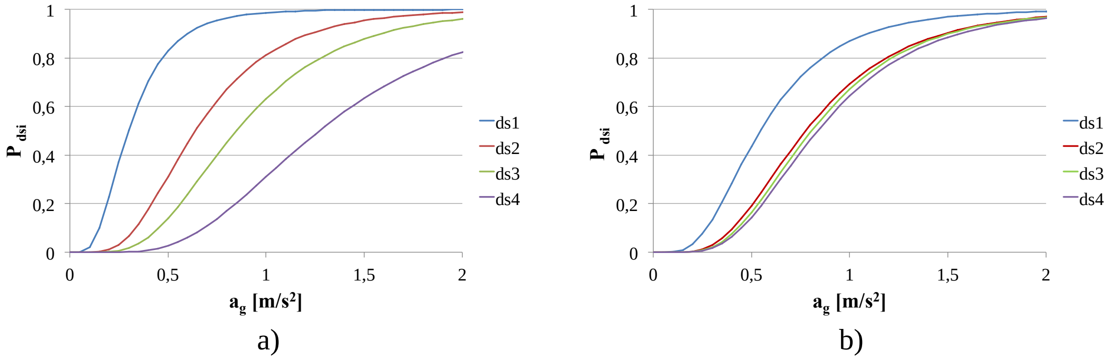


Fig. 21 Fragility curves for earthquake type 1: a) X direction and b) Y direction

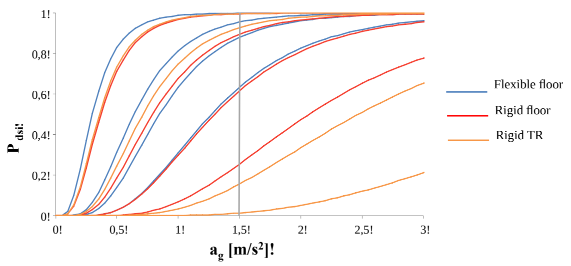


Fig. 22 Fragility curves for X direction in case of Flexible floor, Rigid floor and rigid TR cases

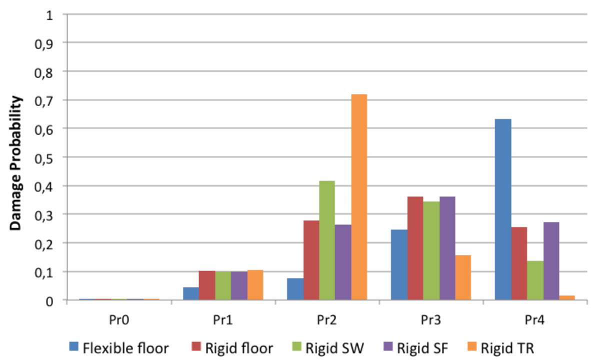


Fig. 23 Probability of damage for earthquake type 1 in the X direction

Pagnini *et al*. (2011) for a case of vulnerability assessment performed at territorial scale. The β_D was conventionally assumed given not enough information was available on Eurocode 8 (2004) for a more precise estimation (e.g. related to the input definition for different percentile values). In general, this value may be considered as representative of a quite accurate hazard evaluation; moreover, it has been stressed that *Pombalino* buildings are concentrated in a well defined area of Lisbon for which a not so wide scatter of the seismic input is expected for. Finally, as regard β_{ls}, it is assumed that the limit state displacement thresholds ($\overline{S}_{d,ds}$) correspond to the conditional probability of 50% of being in or exceeding the corresponding limit state; thus, by assuming a uniform probability density function (in an interval around $\overline{S}_{d,ds}$), as proposed in Pagnini *et al*. (2011), the resulting value of β_{ls} varies, for each configuration, as a function of the ductility of the capacity curve. Other uncertainties on the definition of damage states (e.g. related to the conventional criteria proposed in Eq. 2) are included in β_{ε}. Table 4 summarizes the resultant values of β_{ds} for each configuration examined (as a function of two directions X and Y).

Figs. 21a) and b) show the resulting fragility curves for the case of the original building for earthquake type 1 in both directions. Fig. 22 illustrates – for X direction - the comparison among the resulting fragility curves in the case of basic configuration (Flexible floor) and the “Rigid floor” and “Rigid TR” ones, respectively (for increasing damage states from 1 to 4); the grey line corresponds to the value design ground acceleration in the case of Type 1 seismic demand.

Finally, Fig. 23 illustrates the damage probability for earthquake type 1 in the X direction for all the studied cases.

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; to add the steel frames does not change significantly this behaviour. Retrofitting the building by stiffening the floors and including shear walls does improve slightly the situation, reducing the value of Pr4 and spreading it to Pr2. The retrofitting scheme that mostly improves the seismic performance of the building, with respect to the previous cases, is the case of the inclusion of tie-rods in the front and back façades. This reduces significantly the damage probability Pr4; despite this, the damage probabilities related to lower damage states present a significant peak for ds2 (without leading to a spreading also in ds1 with respect the other configurations, as evident also from the fragility curves illustrated in Fig. 22). This result highlights how this solution probably guarantees to fulfil the ultimate limit state but not the damage limitation one.

4. Conclusions

In the present paper a *Pombalino* building, made by a mixed timber and masonry structure, was modelled with both external masonry walls and internal *frontal* walls. The element formulated for *frontal* walls has been implemented in the Tremuri software, which enables the nonlinear modelling of the masonry buildings. Thus nonlinear static analyses were carried out by focusing the attention on the seismic assessment of global response. Analyses highlighted that the building in its original state is vulnerable, because the floors are flexible, with a consequent quite limited redistribution of seismic actions among walls. Simply by stiffening the horizontal diaphragms one is able to have an improved structure: this could be achieved by still adopting wooden floors or as showed in Fig. 8 of Section 3.1. Additional retrofitting of the structure is possible and advisable if one wants to increase its resistance towards earthquakes. The most profitable solution is the inclusion of tie-rods in the lintels at front and back façades. Being almost comparable the benefit achievable, the solution of the inclusion of steel frames in the ground floor is advisable and present minor architectural drawbacks than the inclusion of shear walls.

Analyses performed showed that the frontal walls provide a quite limited contribution to the overall base shear: this is mainly due to the fact they are not continuous until the ground level (where they are replaced by isolated pillars and arches). Indeed, *frontal* walls mainly play the role of connecting the masonry façades and preventing their out-of-plane failure (if they are properly attached to these walls). The lower stiffness of frontal walls with respect to masonry walls (as showed in Fig. 5) appear a favorable feature to this aim, supporting even larger displacements of façade walls without collapsing. Indeed, in the analyses addressed to the assessment of global response it has been assumed the out-of-plane failure is prevented both by the proper connections of the frontal walls and the floors to the masonry façades or by proper retrofitting strategies (like as the insertion of tie-rods). However, in reality this may not be the case: as a consequence, actual buildings might be even more vulnerable to seismic actions then the considered original building in this study. On the other hand, in the existing building stock in downtown Lisbon, there are many buildings that have been subjected to structural changes. These changes are, for example, removing façade piers to have a larger entrance or removing frontal walls in the above floors. In this way, it is possible to understand that these altered buildings are even more vulnerable than the building evaluated in this study.

It is possible to further ahead in the research conduct loss estimation studies with the results obtained. The current trend in seismic risk analysis and loss estimation involves the use of fragility curves derived from nonlinear static and dynamic analyses of representative structures.

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