

Fragility curves for old masonry building types in Lisbon

Ana Simões¹ · Jelena Milošević¹ · Helena Meireles¹ ·
Rita Bento¹ · Serena Cattari² · Sergio Lagomarsino²

Received: 2 September 2014 / Accepted: 11 March 2015 / Published online: 19 March 2015
© Springer Science+Business Media Dordrecht 2015

Abstract Improving the seismic resistance of traditional buildings is essential for preserving cultural heritage and increasing their safety. This is especially important for old masonry buildings in Lisbon (“Pombalino”, “Gaioleiro” and “Placa”), which are still used for housing and services. Taking this into account, this paper is focused on the seismic assessment of these three types of buildings. The buildings were modelled based on the equivalent frame model approach, their dynamic characteristics were determined and non-linear static (pushover) analyses were performed. Furthermore, for the seismic demand of Lisbon, the seismic performance of such building classes was defined and compared. Finally, with the aim of supporting seismic risk and loss estimation studies, a probabilistic assessment was carried out and the fragility curves for each building type derived.

Keywords Masonry buildings · Performance-based assessment · Non-linear static (pushover) analysis · Fragility curves

1 Introduction

An important part of the building stock in Lisbon, a city characterized by moderate to high seismic risk, is composed by old masonry buildings built before proper seismic code provisions entered in force, highlighting their potential high vulnerability.

The main goal of this work is to compare the seismic vulnerability of three types of old masonry buildings from Lisbon—“Pombalino”, “Gaioleiro” and “Placa”—following the most recent strategies for the seismic performance-based assessment of existing buildings

✉ Ana Simões
ana.g.simoese@ist.utl.pt

¹ CERIS, ICIST, DECivil, Instituto Superior Técnico, Universidade de Lisboa, Lisbon, Portugal

² Department of Civil, Environmental and Chemical Engineering (DICCA), University of Genoa, Genoa, Italy

and adopting effective numerical tools for a quantitative assessment of their seismic behaviour. A complete seismic assessment should include the analysis of two types of response: (1) the global one, mainly related to the activation of the in-plane response of the walls and the connection and load transfer between floors and walls (box behaviour); (2) and the local one, associated with the out-of-plane mechanism of single parts of the structure. In the paper, only the global in-plane response was addressed. In Mendes et al. (2014) and Simões et al. (2014a) additional results on the out-of-plane vulnerability of “Gaioleiro” buildings are illustrated.

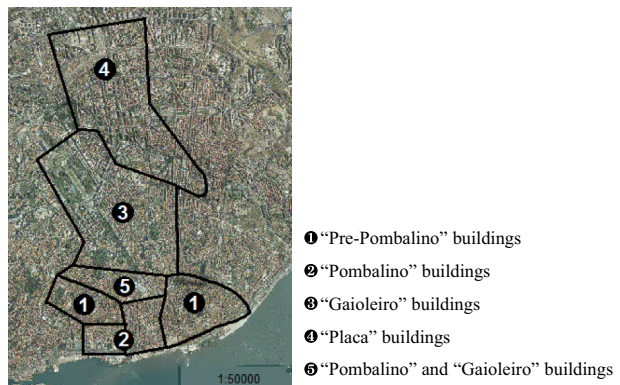
A building representative of each type was selected and a three-dimensional model of the buildings was developed in Tremuri Program based on the equivalent frame model approach (Lagomarsino et al. 2013). Each masonry wall was discretized by a set of panels (piers and spandrels), in which the non-linear response was concentrated, connected by rigid areas (nodes). The masonry panels, as well as the reinforced concrete (RC) beams and columns, present in “Placa” buildings, were modelled as non-linear beams. Floor elements were modelled as orthotropic membrane finite elements taking into account the effective stiffness of the floor system.

The in-plane capacity of the building structures was determined through non-linear static (pushover) analyses. The structures performance points were after determined by comparing the structural capacity with the seismic demand for Lisbon. Finally, fragility analyses were carried out for each building type, being the corresponding fragility curves and damage probability obtained for the ultimate limit state. Such probabilistic assessment provides useful information for risk scenario at territorial scale and seismic loss estimation studies.

2 Old masonry buildings in Lisbon

Masonry buildings were constructed for many centuries based on the available materials and empirical provisions justifying the strong uncertainties about their structural behaviour. Four types of masonry buildings are usually recognized in the Lisbon County: “Pre-Pombalino” buildings, “Pombalino” buildings built after the 1755 earthquake, “Gaioleiro” buildings built between 1870 and 1930 and “Placa” buildings built between 1940 and 1960 (preceding the modern RC buildings). Figure 1 shows the area where these buildings are located in the city, starting from the Tagus River, with the “Pre-Pombalino”

Fig. 1 Map of Lisbon city center



and “Pombalino” buildings, followed by the development of the city and the construction of “Gaioleiro” and “Placa” buildings. Examples of each type of building are shown in Fig. 2.

These buildings are generally mid-rise structures with three to six storeys (Fig. 2). The façade walls are made of rubble stone masonry with decreased thickness up to the height. In case of “Pre-Pombalino”, “Pombalino” and “Gaioleiro” buildings, stones were linked by air lime mortar, while in “Placa” buildings it was more common to use hydraulic lime or cement mortars. The side walls, first made of rubble stone masonry, started to be replaced by brick masonry with the “Gaioleiro” and by concrete blocks in case of “Placa” buildings. The “Pre-Pombalino” buildings represent a disperse group of buildings that result from centuries of unplanned construction; thus, the structural assessment of these type of buildings should be done case by case. Nonetheless, these structures survived the 1755 earthquake and tsunami which caused severe damage in Lisbon downtown. The affected area was after rebuilt based on rigorous planning and construction techniques, generally referred to as “Pombalino” construction, after the Minister Marquês de Pombal.

The “Pombalino” buildings are characterized by the design of a mixed timber-masonry structure, known as the “gaiola pombalina” (cage) intended to withstand the horizontal seismic loads above the first storey (Lopes et al. 2014; Meireles et al. 2014). The cage is composed of timber floors and improved mixed timber-masonry shear walls (“frontal” walls). The buildings were placed in rectangular quarters with similar dimensions (plan and height) and structure. The foundation system is composed of timber piles linked at the top by horizontal timber cross-members placed on the base of the walls to stiffen the alluvium layers of the downtown area. The ground floor of the building consists of solid stone walls and piers linked by a system of masonry arches. In more elaborate cases, groined vaults spanned between the arches are used to protect the upper floors from the spread of any fire that might start at ground floor level.

At the end of the nineteenth century, the fast city growth to the north upland (planned by engineer Ressano Garcia in 1888) originated the construction of “Gaioleiro” buildings.

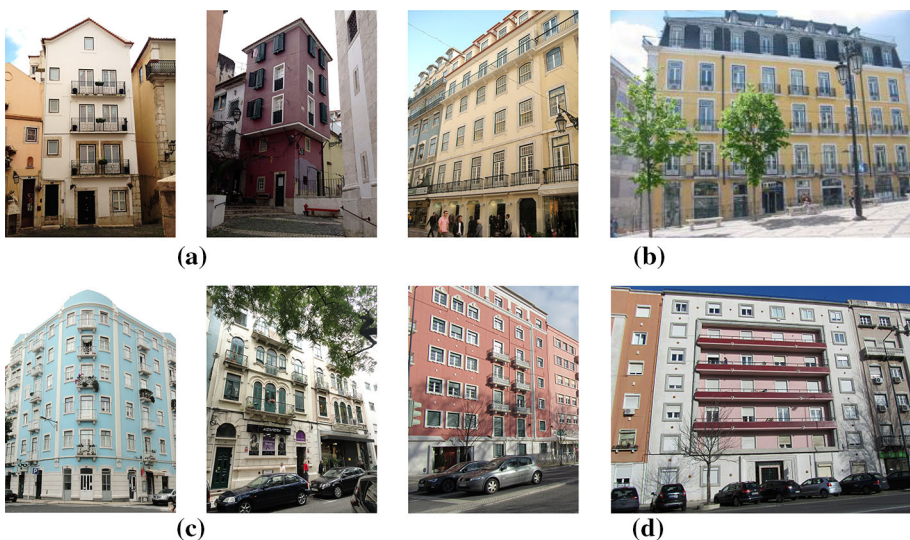


Fig. 2 Old buildings in Lisbon: **a** “Pre-Pombalino”, **b** “Pombalino”, **c** “Gaioleiro” and **d** “Placa”

These buildings, mainly located in “Avenidas Novas” (Fig. 1), were built during a period of real estate speculation which ended up affecting the structural reliability of the buildings (in terms of materials and constructive details). The timber structure from the “frontal” walls was replaced by clay brick masonry walls, usually solid bricks on the bottom storeys and hollow on the top storeys. The thickness of the walls decreases by changing the orientation of the bricks and, on the top floors, these were often replaced by light timber partition walls.

The construction inside the aggregates of “Gaioleiro” buildings was random. The corner buildings have a square plan shape, while the interior buildings have long rectangular shapes which lead to the introduction of side or interior shafts to provide natural light to the interior rooms. Based on these features it is possible to define four building types (Appleton 2005): type I—strait size façade walls and one side shaft, type II—medium size façade walls and one shaft, type III—large façade walls and more than one shaft and type IV—buildings on the corner of the quarter. “Gaioleiro” buildings are also recognized by the balconies on the back façade wall with a composite structure made of steel profiles with ‘I’ or ‘T’ shape connected by clay brick arches.

Later on, a new expansion plan was developed by the engineer Duarte Pacheco in 1938, for the construction of “Bairro dos Actores” and “Bairro de Alvalade” (Fig. 1). During this time, the first RC elements started to appear. First, by means of RC peripheral lintels used to strengthen the timber floors and RC frames at the ground floor when larger spans and open spaces were needed. The timber floors were after replaced by RC slabs, supporting the name “Placa” (meaning RC slab) given to these buildings. Within, there is a particular “Placa” building characteristic from “Bairro dos Actores” area with a salient shape on the back of the building, denominated “Rabo de Bacalhau”, which is made of RC frames and slabs (Milošević et al. 2014). “Rabo de Bacalhau” buildings can be divided in four sub-types (A–D) according to the dimension of the salient shape, use, number of rooms and position of staircases.

It is important to highlight that these mixed masonry–reinforced concrete buildings represent a transition period between old unreinforced masonry buildings and modern RC buildings, resulting on the evident absence of specific design features to ensure the ductility of the structure. The concrete slabs present on “Placa” buildings are slender and lightly reinforced and occasionally without continuity between spans. In addition, concrete has low to moderate resistance class. Thus, in most cases the insertion of RC elements is not conceived with a well-defined role with respect to the seismic action (Cattari and Lagomarsino 2013).

As regards the foundation system, the exterior walls of both “Gaioleiro” and “Placa” buildings are thickened on the base defining continuous and direct foundation systems. In case of “Rabo de Bacalhau”, the RC frame structure on the salient back shape has RC footings.

Based on statistical data collected by Sequeira (1999) and considering construction period of the buildings (Table 1) it was estimated that only 3.0 % of buildings were built before 1755 (“Pre-Pombalino”), 27.2 % of the buildings correspond to the “Pombalino” type, 24.4 % to the “Gaioleiro” buildings, 21.4 % were built between 1940 and 1960 belonging to “Placa” building type and the remaining 24 % correspond to the buildings built after 1960 with RC frame structure.

From these results it can be inferred that the majority of the buildings from Lisbon are composed of old masonry buildings, which already exceeded their expected service life time. The National Census 2011 (Instituto Nacional de Estatísticas (INE – Portugal Statistics) 2011) estimated, in a qualitative way, that 50 % of the buildings built before

Table 1 Distribution of buildings in Lisbon (Sequeira 1999)

Year	Type	Number of buildings	Percentage
<1755	“Pre-Pombalino”	1742	3.0
1755–1880	“Pombalino”	15,711	27.2
1880–1940	“Gaioleiro”	14,067	24.4
1940–1960	“Placa”	12,328	21.4
>1960	RC	13,876	24.0
Total	Lisbon	57,724	100

1960 are in need of repairing works. In addition to the lack of maintenance works, the material degradation, the modification in the use (from housing to services) and the structural changes that result from it, exposes the need to assess their structural vulnerability and the definition of retrofiting solutions.

Finally, Fig. 3 shows the distribution of buildings for storey number, type and age as from the data carried out by the National Census 2011 (INE – Portugal Statistics 2011). Neglecting the structures characterized by one or two storeys, which are expected to be less vulnerable to seismic action than the taller ones, the majority of buildings varies from three to five stories with an almost equal percentage.

3 Global seismic behaviour

3.1 Old masonry buildings case studies

To examine the vulnerability of the different constructive systems and to compare the seismic performance of “Pombalino”, “Gaioleiro” and “Placa” buildings, a building representative of each type was selected as case study. The selection of the buildings was based on previous works carried out. Meireles et al. (2014) studied the seismic behaviour of a “Pombalino” building based on pushover analyses before and after the application retrofiting schemes. Simões et al. (2014a) addressed both the global and local response of a “Gaioleiro” building type I, including some sensitivity analysis related with the modelling process. In Simões et al. (2014b) the seismic behaviour of four different types of “Gaioleiro” buildings is compared. As to “Placa” buildings, Milošević et al. (2014) analysed the seismic behaviour of a “Rabo de Bacalhau” building type D.

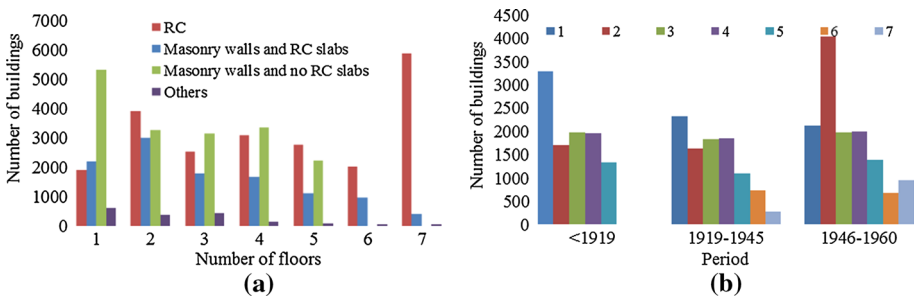


Fig. 3 Distribution of the buildings in Lisbon for storey number, type (a) and age (b) from data of National Census 2011

This paper represents a starting point on the comparison of the seismic performance of these three old masonry typologies of buildings. In future, more configurations per typology will be studied, the comparisons of the seismic performance of different case studies will be done and some sensitivity analyses developed, aiming to identify the factors that most affect the structural seismic response of the old masonry buildings in Lisbon.

In what concerns the case studies selected, the “Pombalino” has a rectangular plan shape with $18\text{ m} \times 11\text{ m}$ and a total height of 15 m (Fig. 4; Meireles et al. 2014), while the “Placa” building is a type D “Rabo de Bacalhau” with $14.5\text{ m} \times 20.5\text{ m}$ plan shape and a total height of 12.3 m (Fig. 5; Milošević et al. 2014). Both are characterized by four storeys, as particularly representative of the entire building stock (Fig. 3). The “Gaioleiro” building has a rectangular plan shape with $8.8\text{ m} \times 19.6\text{ m}$, a side shaft and five storeys high with a total height of 17 m (Fig. 6), being representative of a “Gaioleiro” building type I (Simões et al. 2014a). The choice of focusing the attention on type I follows the preliminary results on the vulnerability of four types of “Gaioleiro” buildings discussed in Simões et al. (2014b), where type I was found as the most vulnerable (although slightly) from the comparison with the other ones in terms of fragility curves (being characterized in general by a lower redundancy and ductility).

The exterior walls are made of rubble stone masonry with decreased thickness up the height, with exception to: the ground floor walls of the “Pombalino” building which are made of cut stone masonry and the side walls from the “Placa” building which are made of concrete blocks. The salient shape on the back of the “Placa” building is composed of RC beams and columns filled with brick masonry walls and RC slabs. There are, in addition, RC lintel beams on top of the windows from the front façade wall. As to the interior structure, the ground floor of the “Pombalino” building is composed of masonry arches and columns and by “frontal” walls on the other storeys. In case of the “Gaioleiro” and “Placa” buildings, the interior walls are made of brick masonry.

Regarding the material properties, the number of experimental campaigns addressed to the masonry buildings in Lisbon is limited. Santos (1997) carried out tests on the bearing masonry walls from a “Pombalino” obtaining a Young’s modulus (E) of 1.0 GPa . Silva and Soares (1997) conducted several tests on “Gaioleiro” buildings, determining for the compressive strength (f_m) values from 0.8 to 1.5 MPa and for E values from 0.7 to 1.0 GPa . Lopes and Azevedo (1997) also conducted a test on a “Gaioleiro” building obtaining E equal to 0.66 GPa . Moreira et al. (2012) performed experimental tests on rubble stone masonry specimens with air lime mortar, where E resulted equal to 1 GPa and f_m equal to 1.6 MPa . Proença and Gago (2011) carried out tests on a “Placa” building where E equal to 2.0 GPa was estimated. Milošević et al. (2013) performed different tests on rubble stone masonry specimens with hydraulic mortar (related to “Placa” buildings) obtaining E equal

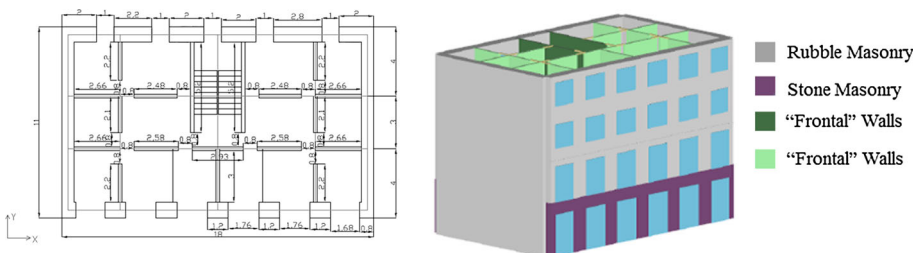


Fig. 4 Plan geometry (dimensions in meters) and three-dimensional view of the “Pombalino” building

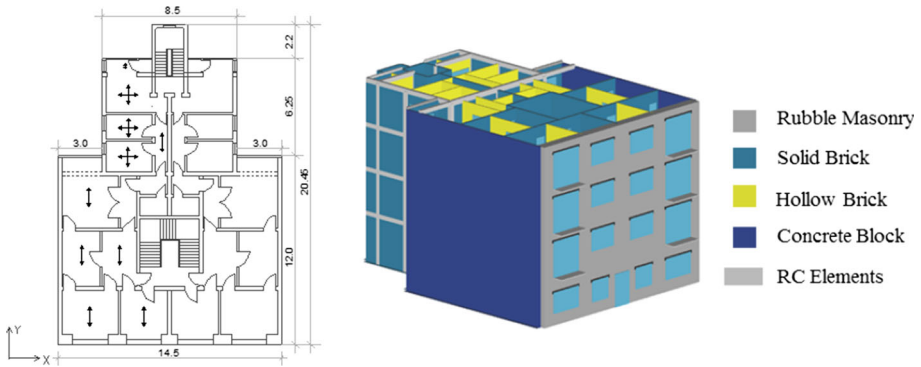


Fig. 5 Plan geometry (dimensions in meters) and three-dimensional view of “Placa” building

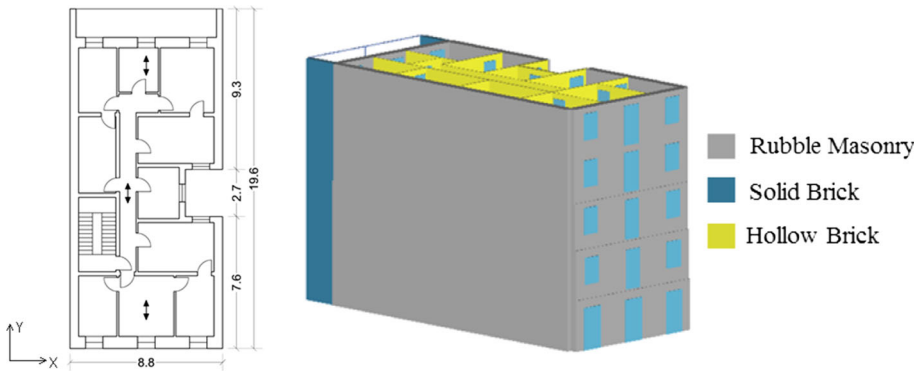


Fig. 6 Plan geometry (dimensions in meters) and three-dimensional view of the “Gaioleiro” building

to 1.64 GPa. Despite some unavoidable differences on the experimental results, the mechanical behaviour of masonry has generally a common feature: high specific mass, low tensile and shear strength and low ductile behaviour (Lourenço et al. 2011).

In addition, from the tests and in situ inspections performed to existing buildings in Lisbon, it is to expect that the construction materials used on “Gaioleiro” buildings have weaker mechanical properties than on “Pombalino” and “Placa” buildings. This can be related to the fact that, “Gaioleiro” buildings were built during a period of fast development of the city and, in general, the construction details and materials used were with lower quality. On the other hand, “Placa” buildings are supposed to be more resistant due to the composition of the mortar from the masonry walls.

Table 2 presents the geometrical properties and expected range for the mechanical properties of the masonry and reinforced concrete structures according to the above mentioned experimental data and also taking into account the values proposed in the Italian code (Norme Tecniche per le Costruzioni – NTC 2008) for rubble stone masonry, solid and hollow clay brick masonry. In this study, the average values were considered, except in case of the “Placa” building as the values of G for rubble stone masonry and concrete block were directly adopted from the experimental tests from Proença and

Table 2 Mechanical and geometrical properties

Mechanical and geometrical properties	Thickness (m)	Young modulus E (GPa)	Shear modulus G (GPa)	Comp. strength f_m (MPa)	Shear strength τ_0^a (MPa)	Specific weight γ (kN/m ³)
Regular stone masonry						
“Pombalino”	0.90	2.40	0.78	6.0	0.090	22
		3.20	0.94	8.0	0.120	
Rubble stone masonry						
“Pombalino”	0.85–0.75	1.02	0.34	2.0	0.035	20
		1.44	0.48	3.0	0.051	
“Gaioleiro”	0.80–0.50	0.69	0.23	1.0	0.020	19
		1.05	0.35	1.8	0.032	
“Placa”	0.70	1.50	0.50	2.6	0.056	21
		2.50	0.66	3.8	0.074	
Solid clay brick masonry						
“Pombalino”	0.24	1.20	0.40	2.4	0.060	18
“Gaioleiro”	0.23–0.11	1.80	0.60	4.0	0.092	
“Placa”	0.20					
Hollow clay brick masonry						
“Gaioleiro”	0.23–0.11	0.90	0.30	1.6	0.044	12
“Placa”	0.10	1.50	0.50	3.2	0.076	
Concrete block						
“Placa”	0.20	2.00	0.60	3.0	0.180	14
		3.52	0.88	4.4	0.240	
Reinforced concrete						
“Placa”	Concrete class: C16/20 Steel class: A235					

^a Diagonal cracking failure mode according to the criterion proposed in Turnšek and Sheppard (1980) and adopted in the Italian code (NTC 2008)

Gago (2011). In what concerns “frontal” walls present on “Pombalino” building, Meireles et al. (2012) performed a comprehensive experimental study for the characterization of the hysteretic behaviour of these walls based on static cyclic shear tests with imposed displacements. These experimental results were after used to calibrate a macro-element model developed and implemented in Tremuri Program, and considered in this work (additional comments on this are illustrated in Sect. 3.2).

Values related to stiffness properties (E and G) of masonry panels are representative of a cracked condition; they have been defined by applying a reduction factor to elastic values equal to 0.50 in the case of cut stone and rubble stone masonry and 0.75 for clay brick masonry. These factors were defined basis on the recommendations from the EC8-1 (European Committee for Standardization (CEN) 2004), Italian code (NTC 2008) and Lagomarsino et al. (2013). The 0.75 reduction factor adopted for clay brick masonry is justified by the prevalence of a flexural response of such panels that tend to concentrate cracks in end sections. Finally, Table 3 depicts the gravity and live loads considered in each case study.

3.2 Equivalent frame modelling

To assess the seismic global response of the old masonry buildings from Lisbon, a three-dimensional model of each case study was developed in Tremuri Program: the commercial version (3Muri release 5.0.4) to generate the mesh of masonry walls and the research version (Lagomarsino et al. 2012) to perform the non-linear analyses. The Tremuri Program is based on the equivalent frame model approach, starting from the main idea (supported by earthquake damage surveys) that the in-plane response of each masonry walls with openings may be discretized by a set of panels (Fig. 7): (1) piers, vertical elements, supporting both dead and seismic loads; (2) spandrels, horizontal elements between two vertically-aligned openings, which couple piers in the case of seismic loads; and (3) rigid nodes, undamaged masonry portions confined between piers and spandrels. The corresponding numerical models are represented in Figs. 4, 5 and 6.

The in-plane behaviour of masonry piers and spandrels, as well as, the RC elements from the “Placa” building, were modelled by non-linear beams (Lagomarsino et al. 2013), being the strength criteria defined according to the EC8-1 (European Committee for Standardization (CEN) 2004) and Italian code (NTC 2008) recommendations. The flexural response (combining both compressive and bending failure) is based on the beam theory, neglecting the tensile strength of the material, and assuming a rectangular normal stress distribution at the compressed toe. In case of shear response, only the diagonal cracking type of failure according to Turnšek and Sheppard (1980) criterion was considered, following the proposal of the Italian code (NTC 2008) for existing masonry buildings. The “frontal” walls from the “Pombalino” building were modelled by a macro-element model calibrated based on experimental campaign from Meireles et al. (2012); it aims to reproduce the non-linear hysteretic behaviour of “frontal” walls with different dimensions and configurations, being a very powerful tool for the seismic assessment of this typology of buildings.

Table 3 Gravity and live loads considered

Gravity loads (live loads) [kN/m ²]	
Floors	
“Pombalino”	0.7 (2.0)
“Gaioleiro”	
“Placa”	1.3 (2.0) ^a
	3.0 (2.0) ^b
Staircase	
“Pombalino”	0.7 (4.0)
“Gaioleiro”	
“Placa”	1.3 (4.0) ^a
	3.0 (4.5) ^b
Roof	
“Pombalino”	1.4 (2.0)
“Gaioleiro”	
“Placa”	1.9 (0.4)
Balcony	
“Gaioleiro”	2.0 (2.5)
“Placa”	3.0 (2.0)

^a Timber floor

^b RC floor

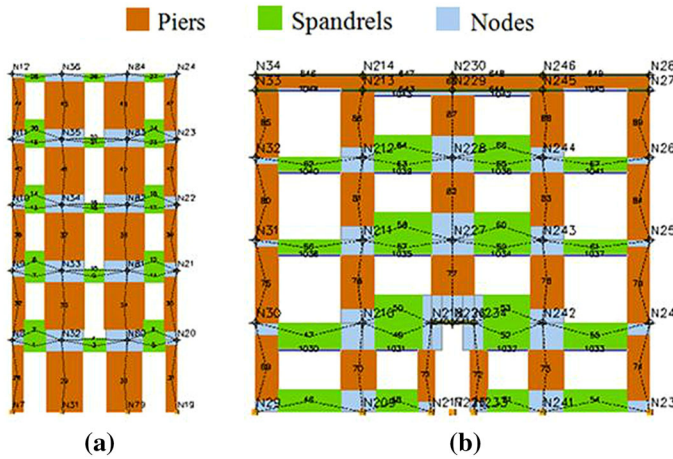


Fig. 7 Macro-Element mesh of the front façade: **a** “Gaioleiro” building; **b** “Placa” building

Floors were defined as orthotropic membrane finite elements (Lagomarsino et al. 2013), characterized by an equivalent thickness and by a Young modulus $E_{1,eq}$ in the floor warping direction (perpendicular to the façade walls), $E_{2,eq}$ (in the orthogonal direction) and equivalent shear modulus G_{eq} . The acting loads were distributed only in the warping direction of the floors. The adopted values for the floors are presented in Table 4.

The variation of the Young modulus and, in particular, of the shear modulus between building types, is justified by the expected behaviour of the floor system and their contribution to the load transfer between the walls and floors. Therefore, a lower value of shear modulus was considered in case of the “Gaioleiro” building in order to simulate a very flexible behaviour of the timber floor. This value was defined based on the recommendations of FEMA 365 (2000) for single straight sheeting. For the “Placa” and “Pombalino” buildings, higher values were considered based on the assumption that the timber floors and the connections to the masonry walls are of improved quality. In particular, in case of “Placa” building a sensitivity analysis was carried out in Milošević et al. (2014) to better understand the influence of this parameter in terms of the modal properties of the building; moreover, some experimental results from ambient vibration tests was used for supporting the choice of the G_{eq} summarized in Table 4.

Table 4 Parameters for the floor equivalent membrane

Type of building/floor	Thickness (m)	$E_{1,eq}$ (GPa)	$E_{2,eq}$ (GPa)	G_{eq} (GPa)
“Pombalino”				
Timber	0.02	52.0	12.0	0.75
“Gaioleiro”				
Timber	0.02	20.6	8.0	0.04
Composite Balcony (back façade)	0.04	30.8	0	13.40
“Placa”				
Timber	0.02	22.4	8.0	0.40
RC	0.10	29.0	29.0	12.00

Modal analyses were performed in order to get the dynamic properties of the buildings and to check the percentage of participating mass activated by the first modes. The period (T) and participation mass (M) are summarized in Table 5 for all cases considered (being the X direction parallel to the façade walls). The “Pombalino” and “Gaioleiro” buildings have similar fundamental periods in both directions. The lower period obtained with the “Placa” building may derive from the contribution of the exterior walls (made of rubble stone masonry in the X direction and concrete blocks in the Y direction) which present higher strength and stiffness than in case of the “Pombalino” and “Gaioleiro” buildings (Table 2) and due to the irregular plan shape configuration of the structure (back salient shape). These modal properties obtained numerically are also in agreement with the values proposed by Oliveira (2004) based on in situ ambient vibration tests carried out in several buildings in Lisbon.

Table 5 also shows that the participation mass of the fundamental modes is in all cases higher than 60 %, supporting the option to carry out the seismic assessment based on three-dimensional models of the buildings proposed in this work. Notwithstanding, it is important to refer that at the scale of the city these buildings are placed in aggregates being restrained by the adjacent buildings on both or only in one of the sides. The analysis of the aggregate of buildings (group effect) provides, in general, higher strength to the building, as shown in Monteiro et al. (2005) for case the of “Pombalino” buildings. These boundary conditions mainly affect the behaviour of the buildings in the longitudinal (X) direction in terms of deformation, whereas the results in the transversal (Y) direction are expected to be more reliable and comparable to the ones obtained with the isolated structures (at least in case of symmetric configurations). For asymmetric configuration, as in case of “Gaioleiro” building analysed, the expected torsional effects may be reduced if the structure is analysed considering the adjacent buildings. In future work it will be important to compare the influence of the group effect to the seismic performance of these buildings; however, the results for the isolated structures presented in this work are on the safety side as they represent, in most of the cases, the worst case scenario.

3.3 Non-linear seismic analyses

The in-plane capacity of the buildings was obtained by non-linear static (pushover) analysis, i.e. by subjecting the structure to a static lateral load pattern of increasing magnitude. The analyses were performed for each main direction of the building considering two load patterns: (1) uniform, proportional to the mass; and (2) pseudo-triangular, proportional to the product between the mass and height. Figure 8 plots the pushover curves for all cases studies in function of the average displacement of the nodes located at the roof level (d) and the base shear force (V_b).

The pushover analyses were stopped for 20 % decay of the maximum base shear force in agreement with the EC8-1 (European Committee for Standardization (CEN) 2004),

Table 5 Modal properties

Building	“Pombalino”	“Gaioleiro”	“Placa”
T_X (s)	0.69	0.71	0.34
M_X (%)	60	61	68
T_Y (s)	0.36	0.35	0.25
M_Y (%)	70	62	86

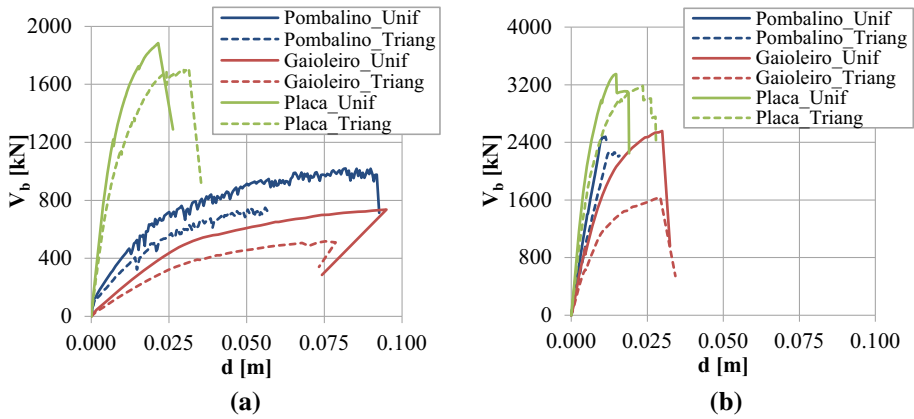


Fig. 8 Pushover curves for all cases considered: **a** X direction and **b** Y direction

EC8-3 (European Committee for Standardization (CEN) 2005) and Italian code (NTC 2008) that define this condition as that associated with the ultimate displacement capacity of the structure. Nonetheless, the adoption of such criteria in case of very flexible floors may lead, in some cases, to non-conservative results, as shown in Simões et al. (2014a). In fact, the limited load transfers between walls results on the almost independent response of the walls. In these cases, the reaching of serious damage in a wall may not appear evident on the structure capacity curve (representative of the global behaviour). Therefore, the attainment of a certain limit state should consider the lack of homogeneity on the distribution of damage and its possible premature concentration in some walls.

From Fig. 8 it is evident that for all building types the stiffness and strength is much higher in the Y direction than in the X direction, where a greater area of openings can be found. It can also be stated that the “Placa” building has the higher strength in both directions. The “Pombalino” and “Gaioleiro” buildings present a similar type of behaviour, however lower strength was obtained with the “Gaioleiro” building. The ductility is much higher in the X direction and, in case of “Pombalino” building, is practically non-existent in the Y direction. For the “Placa” building these differences are not so evident. Comparing the results obtained with the two lateral load patterns, in general, a lower base shear force is determined with the pseudo-triangular load.



Fig. 9 Damage pattern on the back façade of the “Pombalino” building for the uniform load

Figures 9, 10 and 11 plot the damage pattern, respectively, on the “Pombalino”, “Gaioleiro” and “Placa” building for the last steps of the pushover analysis (here the value of ultimate displacement varies with the load pattern considered). The legend of the figures displays the type of behaviour and damage failure in each structural element.

In the X direction, damage is characterized, in case of the “Pombalino” and “Gaioleiro” buildings, by the flexural failure of spandrels followed by the flexural damage of piers from the ground floor (Figs. 9, 10a, b). This type of behaviour is due to the very slender piers (consequence of the opening’s configuration) and the very moderate coupling provided by spandrels (which show a “weak” behaviour due to the lack of other tensile resistant element coupled to them). The presence of the RC lintel beams on the “Placa” building front façade wall influences the behaviour of spandrels, which have a shear failure type (Fig. 11a).

The interior brick walls from the “Gaioleiro” and “Placa” buildings present disperse damage due to flexural behaviour—Fig. 10b, c—resulting in less damage façade walls. On the other hand, the effect of the interior “frontal” walls from the “Pombalino” building is relatively low, being the major contribution to the total base shear force provided by the exterior masonry walls. This is due to the fact that the “frontal” walls have no continuity up the height of the building (as they are interrupted at ground floor) and also because of their lower stiffness when compared to the masonry walls (Meireles et al. 2014).

In the Y direction, damage is mostly concentrated on the side blind walls. For instance, in case of the “Gaioleiro” building, if a uniform load is considered, these walls present shear damage on the base, whereas with the pseudo-triangular load, damage is due to flexural behaviour (Fig. 10d, e). On the contrary, with the “Placa” building (Fig. 11c, d), piers present, in general, flexural behaviour for both load distributions. Furthermore, in case of the RC frame on the back of the building, damage is concentrated on the ground

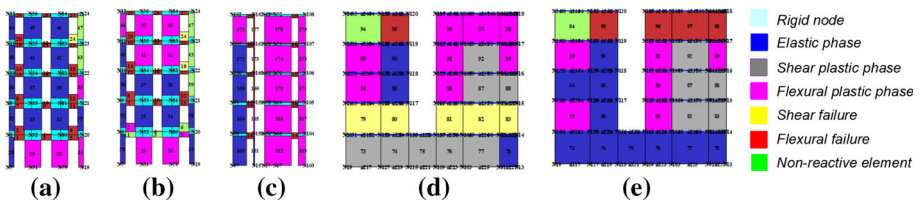


Fig. 10 Damage pattern on the “Gaioleiro” building: **a** and **b** front façade wall, respectively, for uniform and pseudo-triangular load; **c** interior wall in the X direction for pseudo-triangular load; **d** and **e** side wall, respectively, with uniform and pseudo-triangular load

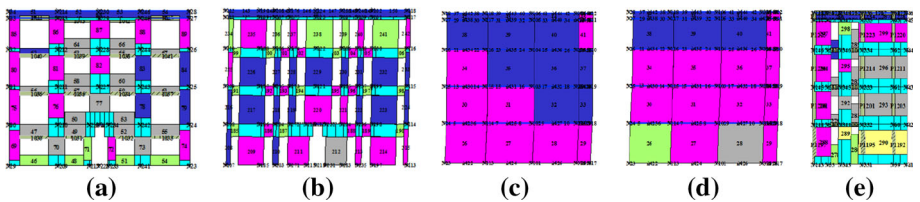


Fig. 11 Damage pattern on the “Placa” building: **a** front façade wall and **b** interior wall in the X direction for uniform load; **c** and **d** side wall, respectively, for uniform and pseudo-triangular load; and **e** wall on the back salient shape in the Y direction for uniform load

floor RC columns and on the masonry piers, in both cases due to flexural behaviour (Fig. 11e).

It can also be noticed that with the uniform load, damage is mainly located on piers from the ground floor levels, while with the pseudo-triangular load damage is spread up the height of the building, as exemplified on Fig. 10d, e, in case of the “Gaioleiro” building and in Fig. 11c, d for the “Placa” building. This is because the ultimate displacement obtained with the pseudo-triangular load is higher than the one with the uniform load; therefore, the buildings were able to explore more the redistribution of the loads and the non-linear behaviour of its structural elements.

3.4 Performance-based assessment

The seismic performance-based assessment comprehends the determination of the performance point or target displacement (d_{max}^*) of the building, computed from the intersection between the capacity curve of the structure and the seismic demand (in terms of response spectrum). The capacity curve is obtained by converting the pushover curve from the original Multi Degree Of Freedom (MDOF) to an equivalent Single Degree Of Freedom (SDOF) system. Among the different approaches proposed in the literature, the N2 Method, originally proposed by Fajfar (1999) and adopted on structural codes (EC8-1 (European Committee for Standardization (CEN) 2004) and Italian code (NTC 2008)), was assumed as reference.

An elasto-perfectly plastic force–displacement relationship was assumed to define the SDOF capacity curve (EC8 European Committee for Standardization (CEN) 2004; NTC 2008): the initial stiffness was determined based on the intersection with the point corresponding to 70 % of the maximum base shear reached ($0.70V_{b,max}$) on the initial branch of the pushover curve; the yield force (F_y) was determined in such way that the areas under the SDOF pushover curve and the elasto-perfectly plastic capacity curve are equal. Table 6 summarizes the properties of the capacity curves for both load patterns, namely the period (T^*), the ductility (μ^*), computed by the ratio between the ultimate displacement (d_u^*) and the yielding displacement (d_y^*), and the strength, obtained from the ratio between the yielding force (F_y^*) and the mass (m^*). Figure 12 depicts the capacity curves in the X and Y direction in the idealized elasto-perfectly plastic relationship for the uniform load.

It is possible to observe that: (1) the X direction presents higher equivalent period, which is in agreement with a more deformable structural system (higher number of openings on the façade walls); (2) the “Gaioleiro” building has the highest equivalent period in both directions; (3) the Y direction presents higher structural strength due to the contribution of the blind side walls. In terms of ductility, “Pombalino” and “Gaioleiro” buildings have significant differences between structural directions, more evident in the

Table 6 Properties of the capacity curves

Capacity curves	Uniform load						Pseudo-triangular load					
	X direction			Y direction			X direction			Y direction		
	T^*	μ^*	F_y^*/m^*	T^*	μ^*	F_y^*/m^*	T^*	μ^*	F_y^*/m^*	T^*	μ^*	F_y^*/m^*
“Pombalino”	0.96	3.01	0.98	0.31	1.18	2.88	1.07	2.06	0.72	0.35	1.56	2.27
“Gaioleiro”	1.17	2.06	0.85	0.39	1.97	2.92	1.35	1.87	0.61	0.45	2.35	1.81
“Placa”	0.39	2.37	2.21	0.26	2.25	3.86	0.46	2.24	2.11	0.30	2.55	3.71

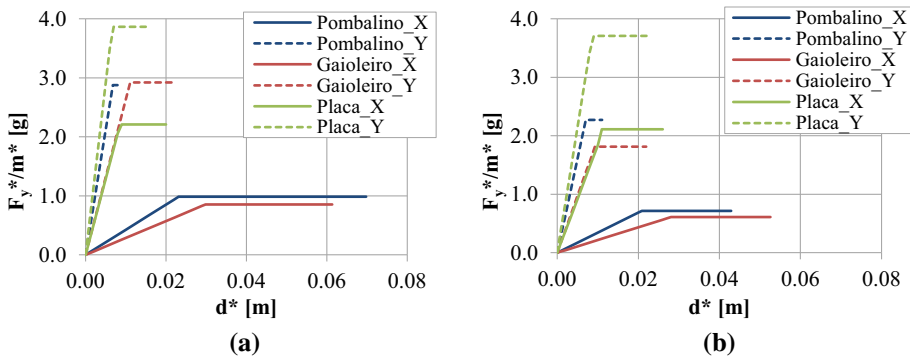


Fig. 12 Capacity curves: **a** uniform load and **b** pseudo-triangular load

first case where a very fragile behaviour was determined in the Y direction. As to the “Placa” building, higher strength and ductility was obtained in both directions.

The verification of the ultimate limit state consists of checking if the structure withstands the seismic demand defined on the EC8-1 (European Committee for Standardization (CEN) 2004) for Lisbon with a return period (T_R) of 475 years. The far-field (type 1.3) seismic action was adopted with 5 % equivalent viscous damping (ζ) for a foundation soil type C. The buildings’ importance factors (γ_I) are equal to 1. Figure 13 plots the results of the seismic performance-based assessment by means of: (a) the ratio q^* between the acceleration in the structure with unlimited elastic behaviour $S_e(T^*)$ and in the structure with limited strength F_y^*/m^* and (b) the ratio between the ultimate and the performance displacements (d_u^*/d_{max}^*).

Firstly, it is worth noting that the Italian code (NTC 2008) recommend $q^* < 3$ aiming to limit the overall acceptable ductility of the building. From Fig. 13a it can be verified that only the “Placa” building verifies this condition in the X and Y directions. Both “Pombalino” and “Gaioleiro” buildings show an insufficient capacity in terms of strength, in particular in the X direction. According to the seismic performance-based assessment, safety is verified when $d_u^*/d_{max}^* > 1$, thus, it can be inferred from Fig. 13b that none of buildings (exception made to the “Placa” building in Y direction) fulfil the safety

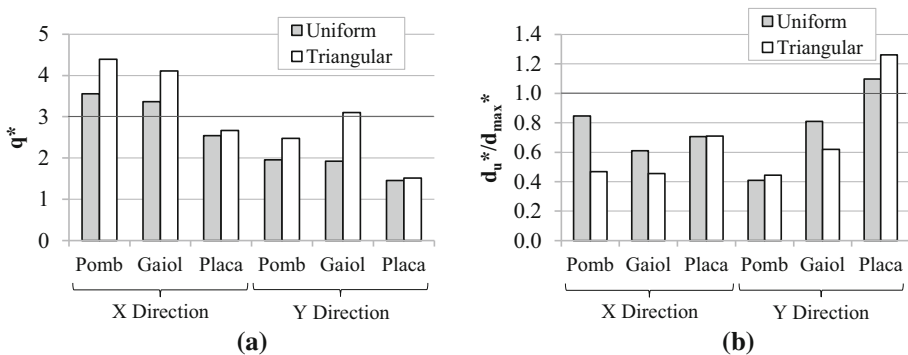
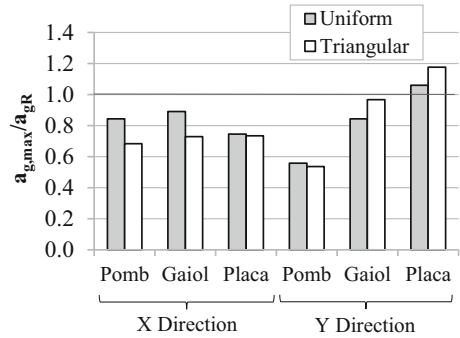


Fig. 13 Results from the seismic performance-based assessment: **a** factor q^* and **b** ratio between the ultimate and the performance displacements

Fig. 14 Ratio between the maximum admissible and reference ground acceleration (taking $q^* < 3$)



requirements for the ultimate limit state as defined on EC8-1 (European Committee for Standardization (CEN) 2004) and Italian code (NTC 2008).

Finally, Fig. 14 plots the ratio between the maximum admissible ground acceleration ($a_{g,max}$) compatible with the fulfilment of the ultimate limit state taking into account $q^* < 3$, and the reference ground acceleration ($a_{g,R}$), which in case of seismic action 1.3 is equal to 1.5 m/s^2 . If $a_{g,max}/a_{g,R} > 1$, safety is verified (Fig. 14): indeed, only “Placa” building in the Y direction fulfils this requested requirement. The results confirm that the maximum admissible ground acceleration is much lower than the reference values for the seismic demand. Consequently, it can be concluded that the case studies considered, representative of “Pombalino”, “Gaioleiro” and “Placa” buildings in their original configuration, have a high seismic vulnerability and do not fulfil the requirements for the ultimate limit state as defined in codes. Due to this reason, a comparison in terms of damage probability distribution is discussed in the following section by introducing the fragility curve concept.

4 Fragility analyses

The seismic performance of the three buildings was compared based on the fragility curve concept considering the probability of exceeding a specific damage limit state (ds). Four levels of damage were adopted and directly defined on the capacity curve based on conventional displacement limits ($S_{d,ds}$). Taking into consideration the suggestions in Lagomarsino and Giovinazzi (2006), the following limits were taken as reference: (1) slight damage $S_{d,1} = 0.7S_{dy}$; (2) moderate damage $S_{d,2} = 0.8S_{dy} + 0.2S_{du} \leq 1.5S_{dy}$; (3) extensive damage $S_{d,3} = 0.5(S_{dy} + S_{du})$; and (4) near collapse $S_{d,4} = S_{du}$.

The fragility curves were defined by lognormal functions that describe the probability of reaching, or exceeding, a defined damage state (ds) given the spectral acceleration ($a_{g,k}$) or associated displacement (S_d), according to the following Eq. (1):

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

where, Φ is the standard normal cumulative distribution function; β_{ds} is the standard deviation of the natural logarithm of spectral displacement for ds ; $\bar{S}_{d,ds}$ is the median value of the spectral displacement at which a building reaches the threshold of damage state ds . Equation (1) can also be defined in terms of acceleration coordinates ($P[a_{g,k}|a_g]$),

considering $a_{g,k}$ associated with the previous displacement limit stated ($S_{d,k}$) and a_g the ground acceleration.

The parameter β_{ds} considers different sources of variability and uncertainty related to the model used (β_e), the seismic demand (β_D), the capacity curve (β_C), i.e. related to the input parameters—geometrical and mechanical—which affect the global response, and the definition of the damage limit states (β_{LS}). The value of β_e was assumed equal to 0.20, as in Meireles et al. (2014). As regards β_D , it was conventionally assumed equal to 0.25 given that not enough information was available on the EC8-1 (European Committee for Standardization (CEN) 2004) for a more precise estimation (e.g. related to the input definition for different percentile values).

The value of β_C is representative of the expected variability of the capacity within a group/typology of masonry buildings with homogeneous behaviour; thus, the adoption of values greater than the case of the assessment on a single building are usually justifiable. Different values have been assumed for the three building types under study taking into account: (1) the type of construction, properties of the materials and constructive details discussed in Sects. 2 and 3.1; and (2) the results obtained in terms of seismic capacity presented in Sects. 3.3 and 3.4. Starting with the “Pombalino” building, β_C was considered equal to 0.35 as in the work from Meireles et al. (2014) and, already adopted in other studies at territorial scale in Portugal (e.g. Pagnini et al. 2011). In case of “Placa” buildings, with a mixed masonry–reinforced concrete structure, it is suggested to define a lower value of β_C and equal to 0.30 due to the fact that these buildings were built during a short period of time (from 1940 to 1960, see Table 1) with a structural system based on a material and rules quite standardized. On the contrary, in the case of “Gaioleiro” building, the variability is expected to be higher than the other two types supporting the adoption of a value of β_C equal to 0.40: indeed, with respect to the case of “Pombalino” buildings, that followed specific and homogeneous rules defined after the 1755 earthquake event, and “Placa” buildings, “Gaioleiro” buildings present a wider variation in material and geometrical configurations (i.e. the thickness, see Table 2).

Finally, β_{LS} was determined assuming the displacement limit state thresholds ($\bar{S}_{d,ds}$) correspond to the conditional probability of 50 % of being or exceeding the corresponding limit state. Thus, by assuming a uniform probability density function (in an interval around $\bar{S}_{d,ds}$), as proposed in Lagomarsino and Cattari (2014) and Pagnini et al. (2011), the resulting value of β_{LS} varies, for each building and type of analyses, as a function of the ductility of the capacity curve. The final values for $\beta_{ds,i}$ defined for the “Pombalino”, “Gaioleiro” and “Placa” case studies are summarized in Tables 7 and 8, respectively for the uniform and pseudo-triangular load.

Figures 15, 16, 17, 18, 19 and 20 plot the fragility curves in function of the ground acceleration (a_g). Analysing the X direction, the curves obtained with the pseudo-triangular load lead, in all cases, to a higher probability of damage for low values of ground

Table 7 Resultant values of $\beta_{ds,i}$ considering the uniform load

Building type	$\beta_{ds,1}$		$\beta_{ds,2}$		$\beta_{ds,3}$		$\beta_{ds,4}$	
	X/Y	X	Y	X	Y	X	Y	
“Pombalino”	0.541	0.504	0.497	0.486	0.474	0.484	0.474	
“Gaioleiro”	0.575	0.520	0.519	0.518	0.518	0.518	0.517	
“Placa”	0.511	0.453	0.451	0.449	0.448	0.447	0.446	

Table 8 Resultant values of $\beta_{ds,i}$ considering the pseudo-triangular load

Building type	$\beta_{ds,1}$		$\beta_{ds,2}$		$\beta_{ds,3}$		$\beta_{ds,4}$	
	X/Y	X	Y	X	Y	X	Y	
“Pombalino”	0.541	0.483	0.477	0.481	0.477	0.480	0.477	
“Gaioleiro”	0.575	0.518	0.525	0.517	0.521	0.517	0.519	
“Placa”	0.511	0.451	0.457	0.447	0.450	0.446	0.448	

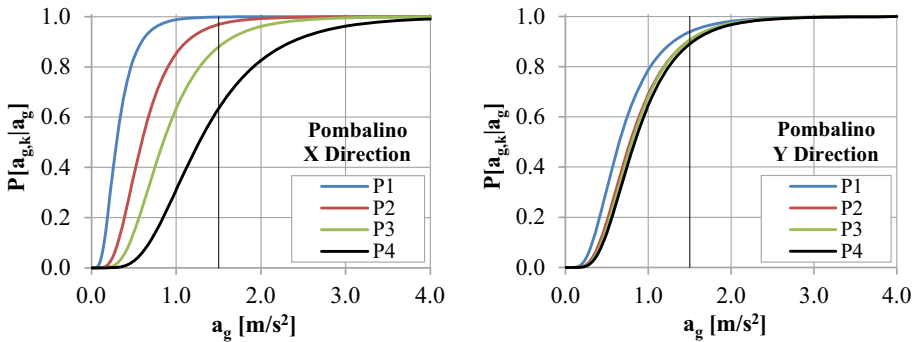


Fig. 15 Fragility curves for the “Pombalino” building: uniform load

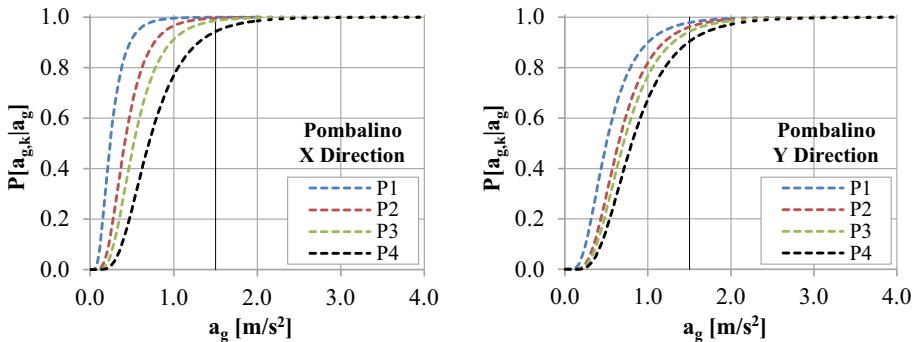


Fig. 16 Fragility curves for the “Pombalino” building: pseudo-triangular load

acceleration (a_g). In the Y direction, the “Pombalino” building is the most vulnerable structure, which is also in agreement with the pushover results obtained in Sect. 3.3, concerning the low ductility of the building on this direction.

Figures 21 and 22 depict the probability of damage associated to the different limit states in case of the seismic demand type 1.3 ($a_g = 1.5 \text{ m/s}^2$) for all studied cases, respectively for the uniform and pseudo-triangular load. As expected, these old masonry buildings have a high probability of reaching collapse ($ds4$) considering the code seismic action.

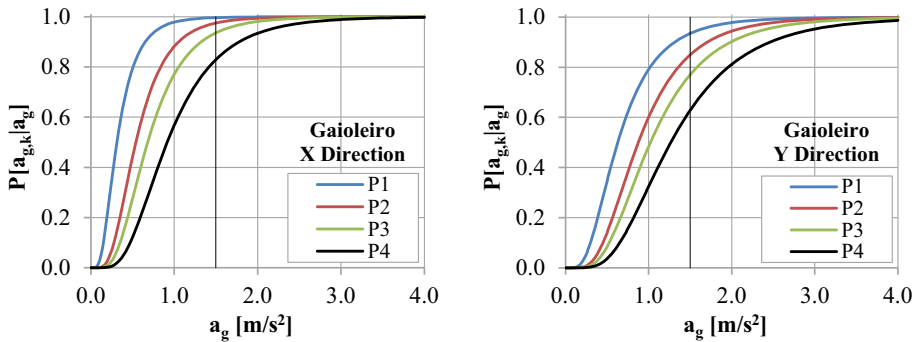


Fig. 17 Fragility curves for the “Gaioleiro” building: uniform load

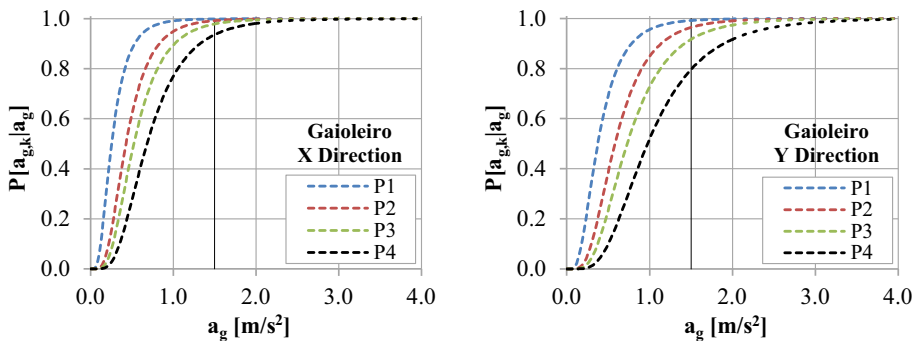


Fig. 18 Fragility curves for the “Gaioleiro” building: pseudo-triangular load

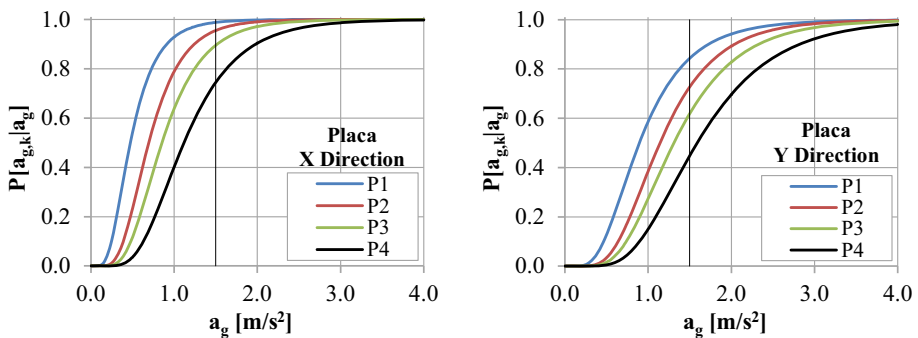


Fig. 19 Fragility curves for the “Placa” building: uniform load

In the X direction this probability is, in all cases, higher than 60 %. The results obtained with the pseudo-triangular load tend to be more penalizing for these structures, with a probability of collapse for both “Pombalino” and “Gaioleiro” building around 90 %. However, in some cases, the uniform load distribution could be more reliable in general terms for the seismic assessment. For example, for the “Pombalino” building, the

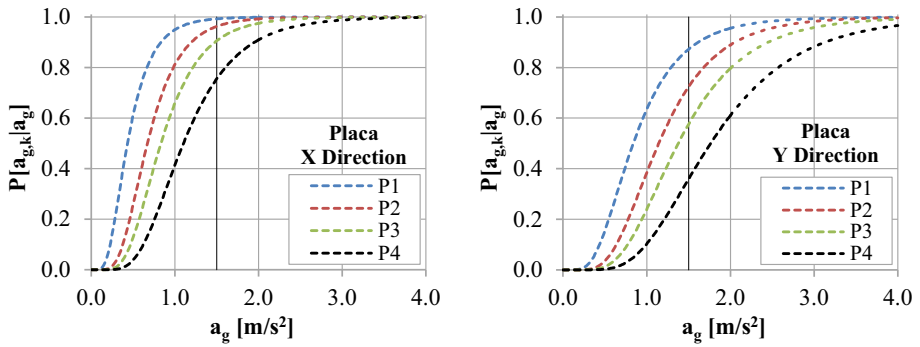


Fig. 20 Fragility curves for the “Placa” building: pseudo-triangular load

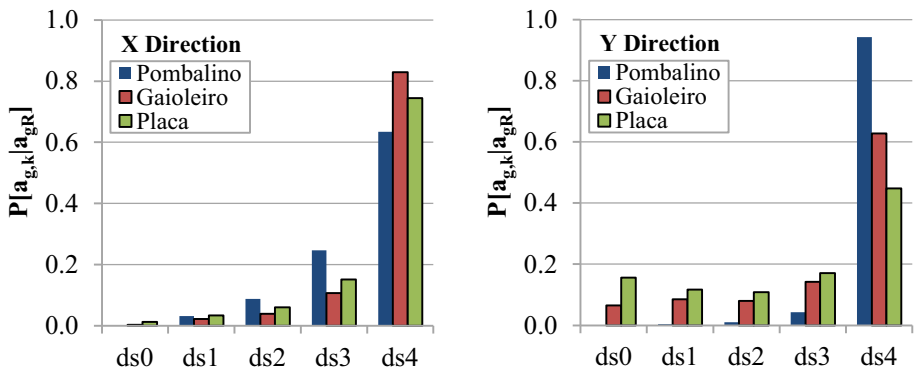


Fig. 21 Damage probability for all case studies: uniform load

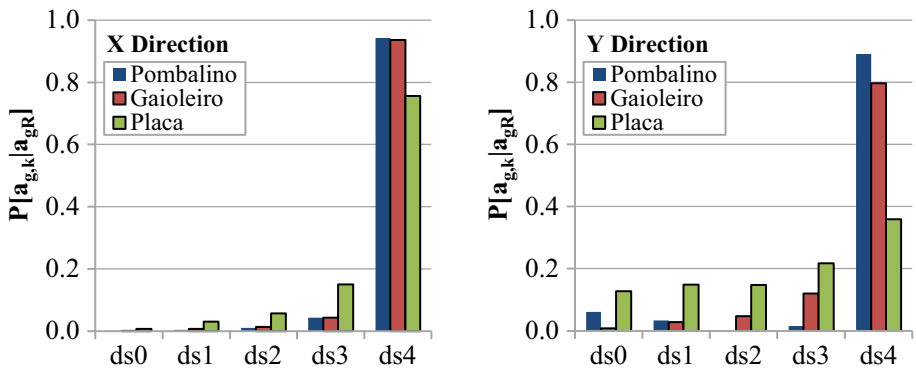


Fig. 22 Damage probability for all case studies: pseudo-triangular load

occurrence of a soft storey mechanism at the ground floor level is more likely to occur due to the presence of masonry pillars at this level (Meireles et al. 2014). Notwithstanding the results obtained, it has to be observed that the X direction could be affected by the

beneficial effect of the aggregate configuration (not included in this study). Thus, the results herein achieved tend to be conservative in this sense.

In the Y direction, “Placa” building is the only case where the probability of collapse is under 50 %, which is also coherent with the results obtained in Sect. 3.4. The “Pombalino” building is from all the most vulnerable structure. It is important to remember, that the behaviour of both “Gaioleiro” and “Placa” buildings is influenced by the contribution (in both directions) of the interior brick masonry walls, in terms of stiffness and strength, load redistribution and structural redundancy, while the “frontal” walls from the “Pombalino” building have a reduced influence to the global behaviour (Meireles et al. 2014). Finally, it is worth mentioning that the results confirmed, as expected, the need of improving the seismic capacity of these buildings.

5 Conclusion

The work addresses the seismic performance-based assessment of building case studies representative of three types of old masonry buildings from Lisbon. The main goal was to examine the vulnerability of the different constructive systems and to compare the seismic performance of “Pombalino”, “Gaioleiro” and “Placa” buildings considering the original configuration and average material properties and architectural features for each building type. The “Pombalino” building adopted as case study can be classified as representative of these building types due to this specific period of construction in Lisbon after the 1755 earthquake. However, for the “Gaioleiro” and “Placa” buildings, the examples herein considered, are only representative of buildings type I and “Rabo de Bacalhau” type D, respectively.

From the seismic performance-based assessment analyses it was concluded that all buildings have very high seismic vulnerability and do not fulfil the requirements for the ultimate limit state as defined in EC8-1 (European Committee for Standardization (CEN) 2004) and Italian code (NTC 2008). A comparison in terms of damage probability distribution was, in addition, carried out supporting similar conclusions. The results obtained confirmed, as expected, the need of improving the seismic capacity of these buildings.

In this study the seismic performance-based assessment has been carried out following the criteria proposed in codes; of course, future developments could be oriented to evaluate the sensitivity of the risk scenario results through other non-linear static procedures (for example, by adopting overdamped spectra instead of the inelastic ones) and by improving the definition of limit states to the specific case of historical buildings (as recently proposed in Lagomarsino and Cattari 2015 and applied in Simões et al. 2014a).

Acknowledgments The authors would like to acknowledge the financial support of the Portuguese Foundation for Science and Technology (Ministry of Science and Technology of the Republic of Portugal) through the Research Project PTDC/ECM/100872.

References

- 3Muri Program, S.T.A.DATA s.r.l., release 5.0.4
- American Society of Civil Engineers (ASCE) (2000) Pre-standard and commentary for the seismic rehabilitation of buildings. FEMA Publication 365, Federal Emergency Management Society, USA
- Appleton J (2005) Rehabilitation of Gaioleiro buildings, 1st edn. Orion Editors, Lisbon (in Portuguese)
- Cattari S, Lagomarsino S (2013) Seismic assessment of mixed masonry–reinforced concrete buildings by non-linear static analysis. *Earthq Struct* 4(3):241–264

- European Committee for Standardization (CEN) (2004) Eurocode 8: design of structures for earthquake resistance—part 1: general rules, seismic actions and rules for buildings (EC8-1)
- European Committee for Standardization (CEN) (2005) Eurocode 8: design of structures for earthquake resistance—part 3: assessment and retrofitting of buildings (EC8-3)
- Fajfar P (1999) Capacity spectrum method based on inelastic demand spectra. *Earthq Eng Struct Dyn* 28(9):979–993. doi:[10.1002/\(SICI\)1096-9845\(199909\)28:9<979::AID-EQE850>3.0.CO;2-1](https://doi.org/10.1002/(SICI)1096-9845(199909)28:9<979::AID-EQE850>3.0.CO;2-1)
- Instituto Nacional de Estatísticas (INE – Portugal Statistics) (2011) Census 2011. Instituto Nacional de Estatísticas (INE—Portugal Statistics). <http://www.ine.pt>
- Italian Code for Structural Design (Norme Tecniche per le Costruzioni – NTC) (2008) D.M. 14/1/2008, Official Bulletin N° 29 of February 4, 2008 (in Italian)
- Lagomarsino S, Cattari S (2014) Fragility functions of masonry buildings. In Ptilakis K, Crowley H, Kaynia A (eds) SYNER-G: typology definition and fragility functions for physical elements at seismic risk, buildings, lifelines, transportation networks and critical facilities series, vol 27. Geotechnical, Geological and Earthquake Engineering, pp 111–156. doi:[10.1007/978-94-007-7872-6_5](https://doi.org/10.1007/978-94-007-7872-6_5)
- Lagomarsino S, Cattari S (2015) PERPETUATE guidelines for seismic performance-based assessment of cultural heritage masonry structures. *Bull Earthq Eng* 13(1):13–47. doi:[10.1007/s10518-014-9674-1](https://doi.org/10.1007/s10518-014-9674-1)
- Lagomarsino S, Giovinazzi S (2006) Macroseismic and mechanical models for the vulnerability assessment of current buildings. *Bull Earthq Eng* 4:415–443. doi:[10.1007/s10518-006-9024-z](https://doi.org/10.1007/s10518-006-9024-z)
- Lagomarsino S, Penna A, Galasco A, Cattari S (2012) TREMURI program: seismic analyses of 3D masonry buildings, Release 2.0. University of Genoa, Italy (mail to: tremuri@gmail.com)
- Lagomarsino S, Penna A, Galasco A, Cattari S (2013) TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Eng Struct* 56:1787–1799. doi:[10.1016/j.engstruct.2013.08.002](https://doi.org/10.1016/j.engstruct.2013.08.002)
- Lopes M, Azevedo J (1997) Assessment of the seismic behaviour of a Lisbon masonry building. In: Proceedings of the 3rd seismology and seismic engineering meeting, Lisbon, Portugal (in Portuguese)
- Lopes M, Meireles H, Cattari S, Bento R, Lagomarsino S (2014) Pombalino constructions: description and seismic assessment. In: Costa A, Guedes J, Varum H (eds) Structural rehabilitation of old buildings, building pathology and rehabilitation, vol 2. Springer, Berlin. doi:[10.1007/978-3-642-39686-1_7](https://doi.org/10.1007/978-3-642-39686-1_7)
- Lourenço P, Mendes N, Ramos L, Oliveira D (2011) Analysis of masonry structures without box behaviour. *Int J Archit Herit* 5(4-5):369–382. doi:[10.1080/15583058.2010.528824](https://doi.org/10.1080/15583058.2010.528824)
- Meireles H, Bento R, Cattari S, Lagomarsino S (2012) A hysteretic model for “frontal” walls in Pombalino buildings. *Bull Earthq Eng* 10(5):1481–1502. doi:[10.1007/s10518-012-9360-0](https://doi.org/10.1007/s10518-012-9360-0)
- Meireles H, Bento R, Cattari S, Lagomarsino S (2014) Seismic assessment and retrofitting of Pombalino buildings by pushover analyses. *Earthq Struct* 7(1):57–82. doi:[10.12989/eas.2014.6.7.000](https://doi.org/10.12989/eas.2014.6.7.000)
- Mendes N, Lourenço P, Costa A (2014) Shaking table testing of an existing masonry building: assessment and improvement of the seismic performance. *Earthq Eng Struct Dyn* 43(2):247–266. doi:[10.1002/eqe.2342](https://doi.org/10.1002/eqe.2342)
- Milošević J, Gago A, Lopes M, Bento R (2013) Experimental assessment of shear strength parameters on rubble stone masonry specimens. *Constr Build Mater* 47:1372–1380. doi:[10.1016/j.conbuildmat.2013.06.036](https://doi.org/10.1016/j.conbuildmat.2013.06.036)
- Milošević J, Bento R, Cattari S (2014) Seismic assessment of a “Placa” building in Lisbon. In: Proceedings of the 2nd European conference on earthquake engineering and seismology, Istanbul, Turkey
- Monteiro M, Lopes M, Bento R (2005) Dynamic behaviour of Pombalino Quarter. In: Proceedings of the 250th Anniversary of the 1755 Lisbon Earthquake, Lisbon, Portugal
- Moreira S, Oliveira D, Ramos L, Lourenço P, Fernandes R, Guerreiro J (2012) Experimental study on the seismic behaviour of masonry wall-to-floor connections. In: Proceedings of the 15th World conference on earthquake engineering, pp 32–92
- Oliveira C (2004) Update of the data-base on fundamental frequencies of building structures, bridges, viaducts, pedestrian crossings based on in situ measuring. In: Proceedings of the 6th national meeting on seismology and earthquake engineering (Sísmica 2004), Aveiro, Portugal (in Portuguese)
- Pagnini L, Vicente R, Lagomarsino S, Varum H (2011) A mechanical model for the seismic vulnerability assessment of old masonry buildings. *Earthq Struct* 2(1):25–42
- Proença J, Gago A (2011) Seismic strengthening of school buildings, 1st edn. Parque Escolar, Lisbon
- Santos S (1997) Tests on Pombalino Walls. Report N° 15/97, Núcleo de Comportamento de Estruturas (NCE), Laboratório Nacional de Engenharia Civil (LNEC), Lisbon, Portugal (in Portuguese)
- Sequeira A (1999) Characterization and assessment of the market related to maintenance and conservation of the architectural heritage. Report GEORPA—Grémio do Património, Lisbon, Portugal (in Portuguese)
- Silva V, Soares I (1997) Seismic vulnerability of the Gaioleiro buildings of Lisbon and possible measures to reduce it. In: Proceedings of the 3rd seismology and earthquake engineering meeting, Lisbon, Portugal (in Portuguese)

- Simões A, Bento R, Cattari S, Lagomarsino S (2014a) Seismic performance-based assessment of “Gaioleiro” buildings. *Eng Struct* 80:486–500. doi:[10.1016/j.engstruct.2014.09.025](https://doi.org/10.1016/j.engstruct.2014.09.025)
- Simões A, Bento R, Cattari S, Lagomarsino S (2014b) Seismic assessment of “Gaioleiro” buildings in Lisbon. In: Proceedings of the 9th international masonry conference, Guimarães, Portugal
- Turnšek V, Sheppard P (1980) The shear and flexural resistance of masonry walls. In: Proceedings of the international research conference on earthquake engineering, Skopje, Macedonia, pp 517–573