

Seismic Assessment of ‘Gaioleiro’ buildings in Lisbon

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ABSTRACT: ‘Gaioleiro’ buildings, built between the mid nineteenth and the beginning of the twentieth century in Lisbon, represent the masonry typology with the highest structural weaknesses from the existing building stock. Thus, four building case studies with different plan dimension and configuration were selected to represent the typology. The building structures were modelled based on the non-linear equivalent frame model approach considering Tremuri Program. The in-plane capacity curves were determined by incremental non-linear static (pushover) analyses in order to compare the seismic performance of the different types of buildings. Finally, assuming a lognormal distribution probability function, fragility curves were obtained. The results confirm the seismic vulnerability of these buildings and the need of defining appropriate strengthening solutions.

Keywords: Unreinforced Masonry Buildings, Performance-Based Assessment, Non-linear Static (Pushover) Analysis, Fragility Analysis

NOTATION

E	Young modulus;
G	Shear modulus;
f_m	Compressive strength;
τ	Shear strength;
γ	Specific weight;
T	Period;
M	Mass;
d	Displacement;
d_{max}	Performance point;
d_y	Yielding displacement;
d_u	Ultimate displacement;
v_b	Base shear force;
F_y	Yielding force;
T_R	Return period;
ξ	Equivalent viscous damping;
γ_I	Importance factor;
S_e	Spectral acceleration.

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1 INTRODUCTION

The assessment of existing buildings is a crucial issue, in particular in areas with a moderate to high seismic risk. The work herein presented is addressed to the non-linear seismic response of the 'Gaioleiro' buildings, a masonry typology of buildings built in Lisbon in the late nineteenth century and the early twentieth century. This period of construction was followed by a time of real estate speculation which ended up affecting the quality of the buildings (design, construction materials and connections between walls and between walls and floors), justifying therefore the strong uncertainties about their structural behaviour.

Four buildings representative of the 'Gaioleiro' typology of buildings selected as case studies. The building structures were modelled in the Tremuri Program [1] based on the non-linear equivalent frame model. Therefore, each masonry wall was discretized into a set of panels (piers and spandrels) in which the non-linear response is concentrated, connected by rigid areas. The strength criteria assigned to the panels follow the recommendations specified on structural codes [2-4]. The wooden floors are modelled as orthotropic membrane finite elements considering their effective stiffness.

To assess the seismic global response of the buildings, mainly governed by the in-plane capacity of the walls and the connection and load transfer between floors and walls, incremental non-linear static (pushover) analyses were performed on the two main structural directions. The performance based assessment was determined by comparing the expected performance of the structures with the seismic demand defined in Eurocode 8 [2] for Lisbon according to the N2 Method. The seismic performance of the four building types was compared based on the fragility curve concept considering the probability of exceeding a specific damage limit state. The aim of such probabilistic assessment is to provide useful information for risk scenario at territorial scale and seismic loss estimation studies.

2 THE 'GAIOLEIRO' BUILDINGS CASE STUDIES

The 'Gaioleiro' buildings have four to seven storeys height. The exterior walls are made of rubble stone masonry and air lime mortar [5]. The interior walls are made of brick masonry: solid bricks on the ground floors and hollow on the top floors. Floors are made of wooden beams placed perpendicular to the façade walls. This typology of buildings follows the 'Pombalino' buildings built after the 1755 Lisbon earthquake and characterized by the design of a three-dimensional timber structure, known as the 'gaiola pombalina' [6]. This interior structure is composed by the wooden floors and mixed timber-masonry shear walls ('frontal' walls). A century after, the fast city growth determined the evolution of the construction technologies and the design of 'Gaioleiro' buildings, leading to the replacement of the 'frontal' walls by clay brick masonry walls.

The aggregates of 'Gaioleiro' buildings are composed by different building configurations: the corner buildings have a square plan shape; the inner buildings have a long rectangular shape, which lead to the introduction of side or interior shafts that provide ventilation/ light to the interior rooms. In general, four types of buildings are identified: type I – small 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. To examine the seismic performance of 'Gaioleiro' buildings, a building representative of each type was considered in this study. Figure 1 and Figure 2 plot the plan geometry and a three-dimensional view of the buildings.

The buildings are five floors with variable interstorey height: ground floor height is 3.6 m, 1st floor height 3.5 m and 2nd, 3rd and 4th floors' height 3.3 m. The front façade wall is 0.80 m thickness on the ground floor, decreasing 0.10 m in each floor till a minimum of 0.50 m. The back façade wall is 0.60 m thickness on the ground floor, decreasing to 0.50 m on the 2nd floor. The side walls have a constant thickness of 0.50 m. The interior walls are made of clay brick masonry with different thicknesses.

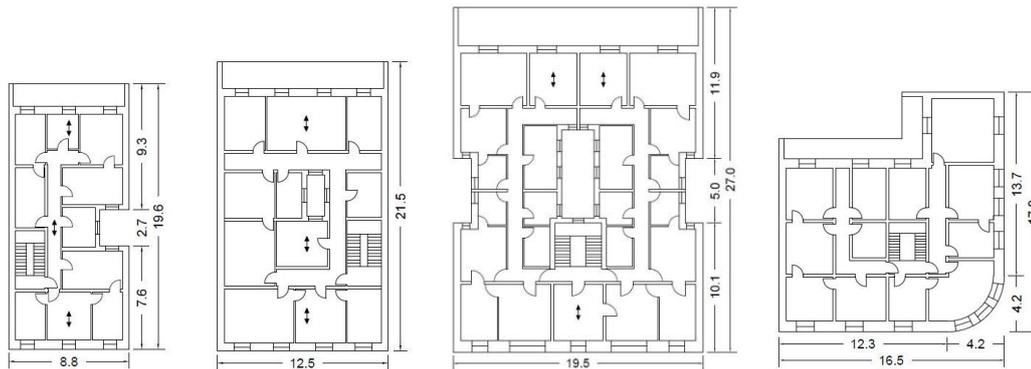


Figure 1. Plan geometry (dimensions in meters) of buildings type I, II, III and IV

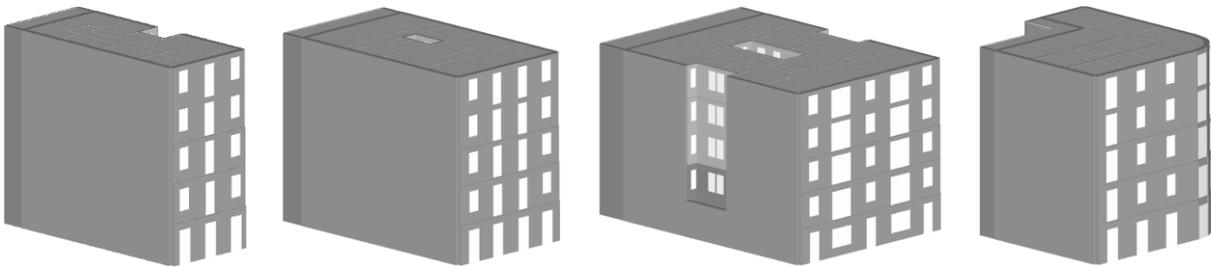


Figure 2. Three-dimensional view of buildings type I, II, III and IV

In what concerns the mechanical characterization of material, the number of experimental campaigns addressed to the assessment of 'Gaioleiro' buildings is limited [7,8] or related to the test of reduced scale models of buildings for shaking table tests [9,10]. The masonry parameters adopted in this study were defined based both on experimental test results and on the average values proposed in [4] for rubble stone masonry, solid and hollow clay brick masonry. Table 1 summarizes the adopted mechanical properties and the gravity and live loads. The stiffness properties (E and G) are representative of the cracked condition of the material.

Table 1. Mechanical properties and loads adopted

Masonry	Young Modulus E [GPa]	Shear Modulus G [GPa]	Compressive Strength f_m [MPa]	Shear Strength $\tau^{(1)}$ [MPa]	Specific Weight γ [kN/m ³]	Gravity Loads (Live Loads) [kN/m ²]
Rubble Stone	0.90	0.29	1.30	0.026	19.0	Floors: 0.7 (2.0)
Solid Brick	1.13	0.38	3.20	0.076	18.0	Staircase: 0.7 (4.0)
Hollow Brick	0.90	0.30	2.40	0.060	12.0	Roof: 1.4 (2.0)
						Balcony: 2.0 (2.5)

⁽¹⁾ Diagonal cracking failure mode according to the criterion proposed in [5].

3 EQUIVALENT FRAME MODELLING

In order to assess the seismic global response of 'Gaioleiro' typology of buildings, a three-dimensional model of the each case study was defined based on the equivalent frame model approach considering Tremuri Program: the commercial version [11] to generate the mesh of elements and the research version [12] to perform the non-linear analyses. This modelling approach is supported on theoretical and experimental research work which confirmed that the global seismic

behaviour of masonry buildings is mainly governed by the in-plane capacity of the walls and the connection and load transfer between floors and walls. Each masonry wall is discretized by a set of panels (piers and spandrels), in which the non-linear response is concentrated, connected by a rigid area (nodes) where no damage occurs. Floors are modelled as anisotropic membrane finite elements.

The in-plane behaviour of masonry piers and spandrels is modelled by non-linear beams [1], being the strength criteria defined according to the simplified recommendations in codes and literature [2-4]. 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 [13] criterion was considered, following the recommendation of [4] for existing masonry buildings.

The timber floors were defined with an equivalent membrane of 0.02 m thickness and characterized by $E_{1,eq}$ (in the floor warping direction, perpendicular to the façade walls), $E_{2,eq}$ (in orthogonal direction) and G_{eq} respectively equal to 20.6, 8.0 and 0.04 GPa. The back balconies made of 'I' steel profiles and connected by clay brick arches, were also modelled. The equivalent floor membrane was defined with the following properties: 0.04 m thickness, 30.8 GPa for $E_{1,eq}$ and 13.4 GPa for G_{eq} . The acting loads were distributed only on the warping direction of the floors.

4 PERFORMANCE-BASED SEISMIC ASSESSMENT

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 (representing the seismic forces). The analyses were performed for each main direction of the building considering two load patterns: (i) uniform, proportional to the mass; and (ii) pseudo-triangular, proportional to the product between the mass and height. Figure 3 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 normalized by the weight of each building (V_b/W) for a better comparison of the performance. The pushover analyses were stopped for 20% decay of the maximum base shear force specified in [2-4] as the collapse condition of the structure.

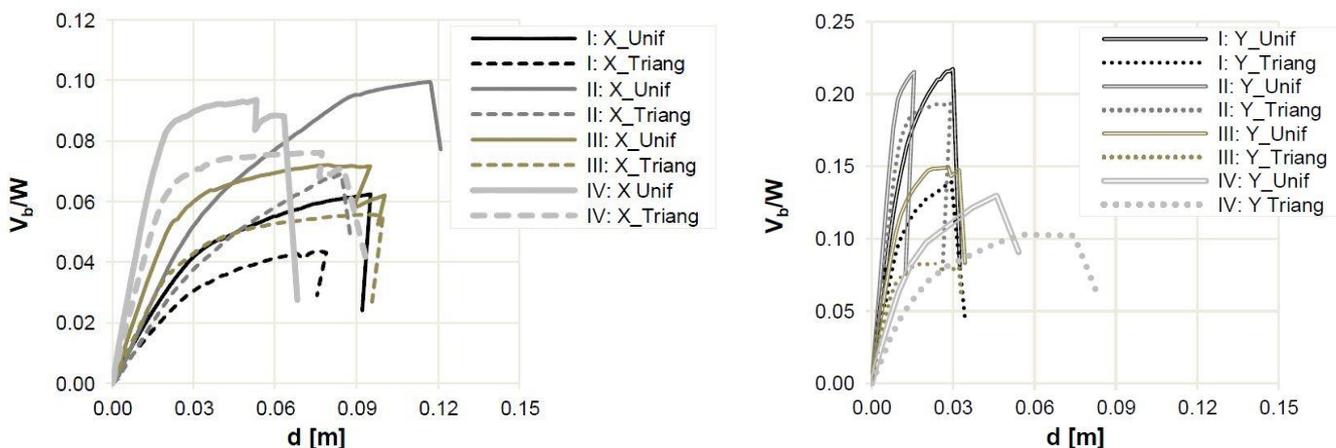


Figure 3. Normalized pushover curves on the X and Y direction

The capacity curves obtained show the differences on the buildings' capacity between the main structural directions. In general, buildings type I, II and III have a similar behaviour, characterized by higher stiffness and strength on the Y direction than in the X direction, which is consequence of the greater area of openings on the direction of the façade walls. These differences are not so important in case of building type IV due to structural configuration of the corner building.

Comparing the behaviour of buildings type I, II and III (Figure 3), it can be said that building type III has highest initial stiffness on the X direction and lowest on the Y direction. Against the expectations, building type III has lower strength than building type II on both directions. This fact may be related to the larger area of openings present on building type III on the X direction and connected to the position of the shafts on the Y direction. For instance, in case of building type II, the shaft is located close to the centre of the building, while in type I and III the shafts interrupt the side walls, decreasing therefore the contribution of these main structural elements on this direction.

Building type IV is, from all cases, the one with higher stiffness and strength on the X direction and the one with the lowest on the orthogonal direction. The differences between structural directions are so relevant, yet still better on the Y direction due to the slightly longer plan shape of the building. One feature from both structural directions is that the capacity curve obtained with the pseudo-triangular load has lower base shear force, but higher ultimate displacement.

Figure 4 to Figure 8 plot the damage pattern on the main walls of the buildings for the last steps of the pushover analysis (here, it is important to notice that 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.

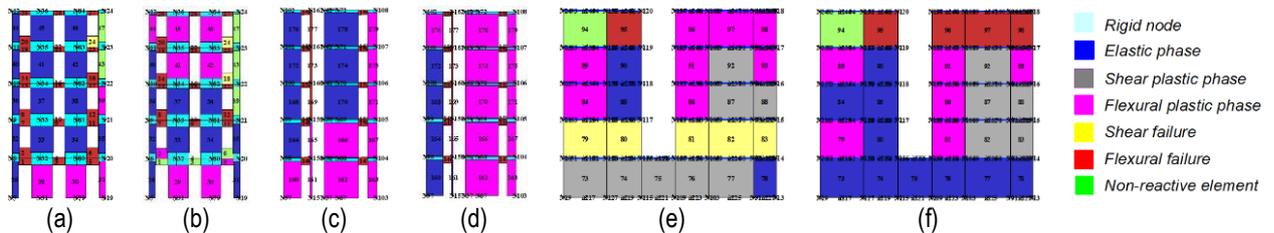


Figure 4. Damage pattern from type I, respectively, with uniform and pseudo-triangular load: (a) and (b) front façade wall; (c) and (d) interior wall on the X direction; (e) and (d) side wall on the Y direction

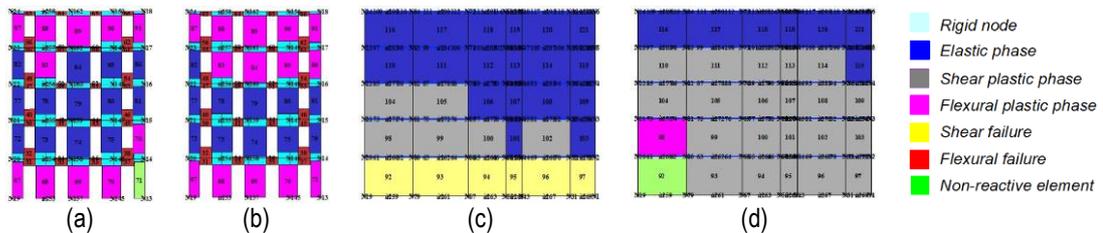


Figure 5. Damage pattern from type II, respectively, with uniform and pseudo-triangular load: (a) and (b) front façade wall; (c) and (d) side wall on the Y direction (opposite to the shaft)

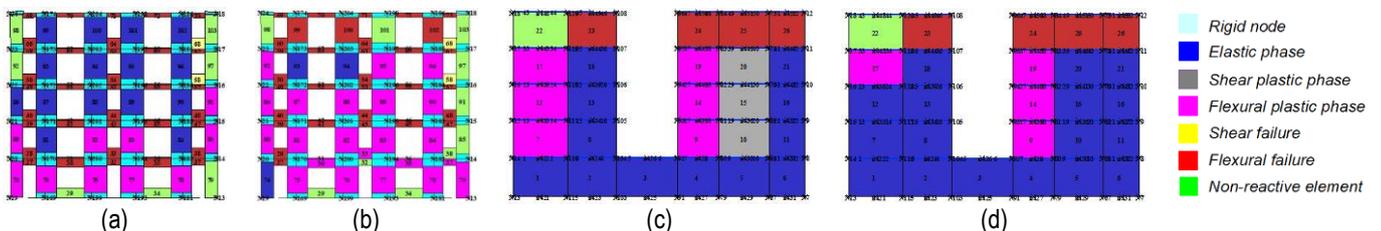


Figure 6. Damage pattern from type III, respectively, with uniform and pseudo-triangular load: (a) and (b) front façade wall; (c) and (d) side wall on the Y direction

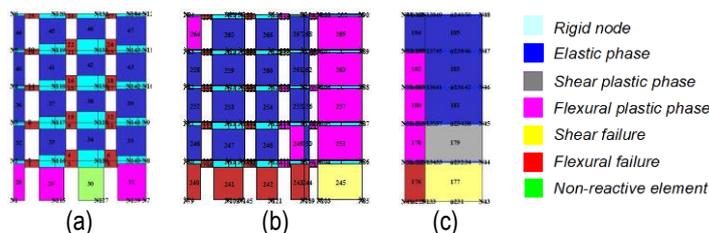


Figure 7. Damage pattern from type IV on the X direction with uniform load: (a) front façade wall; (b) back façade wall; and (c) side wall

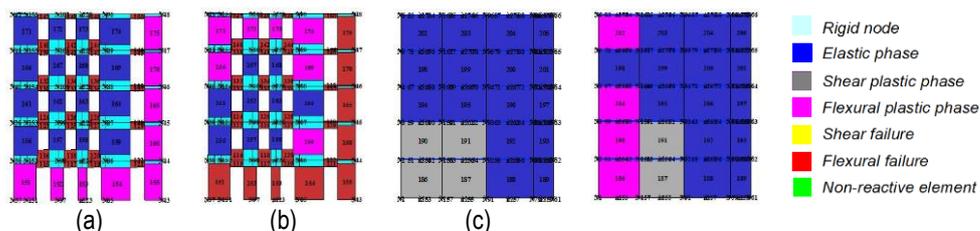


Figure 8. Damage pattern from type IV on the Y direction, respectively, with uniform and pseudo-triangular load: (a) and (b) front façade wall; and (c) and (d) side wall

In case of buildings type I, II and III (Figure 4 to Figure 6), the damage pattern on the X direction is characterized by the flexural failure of spandrel beams, in an early stage of the non-linear analysis, and by the flexural damage of the piers. If a uniform load is considered, damage is mainly concentrated on piers from the ground floors, as can be seen on Figure 4 (a) and (c), in case of building type I, and Figure 6 (a), in case of building type III. For building type II, piers from the top floors also present flexural damage (Figure 5 (a)), which supports the evolution of the pushover curve and the higher ultimate displacement obtained (see Figure 3). If a pseudo-triangular load, in all cases, damage affects piers from both the ground and top floors, as can be seen on Figure 4 (b) and (d), respectively, for the façade and an interior wall from building type I, or on Figure 5 (b) and Figure 6 (b), respectively for type II and III. This type of behaviour is due to the very slender piers (due to 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 interior brick walls also present disperse damage, in general due to flexural behaviour (Figure 4 (c) and (d)), supporting the redistribution of the loads between masonry walls and less damaged façade walls.

On the Y direction, damage is mostly concentrated on the side blind walls. In this case, it is relevant to compare the behaviour of buildings type I and III, as in the two cases the side walls are interrupted by the side shafts. Considering both load patterns, piers from the last floors are damage due to flexure, and in some cases, piers reached the flexural failure for the ultimate displacement (Figure 4 (e) and (f) and Figure 6 (c) and (d)). Therefore, the buildings were able to explore the redistribution of the loads and the non-linear behaviour of its structural elements. Building type I exhibits, in addition, shear failure at the 1st floor, which can be related to the asymmetric configuration of the building on the Y direction (shaft only in one side of the building – see Figure 1) and the occurrence of torsional deformations. In what concerns building type II, damage is due to shear behaviour (Figure 5 (c) and (d)). The most demanding case occurs with the uniform load which led to the shear failure of the piers from the ground floor.

Damage on building type IV is a combination of the damage pattern described for the previous cases. In this case, X direction presents a similar pattern with both load patterns (Figure 7, corresponds to the damage with the uniform load) characterized by the flexural failure of the spandrels, the flexural damage of piers from the front façade wall (Figure 7 (a)) and the flexural failure of piers on the back façade wall (Figure 7 (b)), which is, in part, consequence of the L plan shape of the building. In addition, in Figure 7 (b), the column of piers damage by flexure represents an interior

alignment of brick walls on the salient part of the building. The side blind wall is damaged by shear on the base. On the Y direction, there are more variations in terms of behaviour. For instance, in case of the façade walls, with the uniform load, there is the flexural damage of the piers from the ground and the column of piers on the salient part of the building (right side of Figure 8 (a)), while with the pseudo-triangular these piers reached the flexural failure (Figure 8 (b)). As to the side blind wall damage is due to shear in the first case (Figure 8 (c)), and due to shear and flexure in the second (Figure 8 (d)).

The displacement performance-based assessment comprehends the determination of the performance point or target displacement (d_{max}^*), computed from the intersection between the structure capacity curve and the seismic demand (in terms of response spectrum). The capacity curves obtained for all cases were converted to an equivalent single degree of freedom (SDOF) system following the N2 Method, originally proposed by Fajfar [14] and defined on structural codes [2,4]. Figure 9 depicts the SDOF capacity curves on the X and Y direction in the idealized elasto-perfectly plastic relationship for all cases considered. Table 2 presents the properties of the capacity curves, 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, resulting from the ratio between the yielding force (F_y^*) and the mass (m^*).

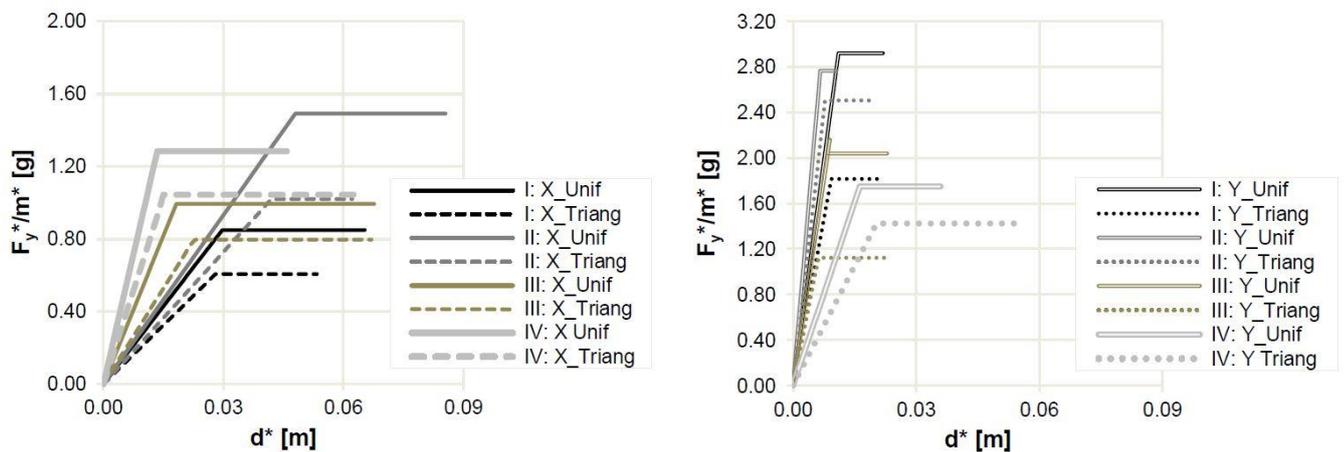


Figure 9. Capacity curves on the X and Y direction

Table 2. Properties of the capacity curves

Type	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^*
I	1.17	2.20	0.85	0.39	1.97	2.92	1.35	1.90	0.61	0.45	2.35	1.81
II	1.13	1.78	1.49	0.31	1.60	2.77	1.27	1.48	1.02	0.35	2.61	2.51
III	0.85	3.71	0.99	0.41	2.70	2.04	1.06	2.96	0.80	0.47	3.64	1.12
IV	0.64	3.40	1.28	0.61	2.20	1.75	0.76	4.13	1.04	0.75	2.70	1.42

It can be stated that, in general, a lower equivalent period is determined with the uniform load pattern and that building type IV has the lowest equivalent period from all cases. On the X direction,

the equivalent period increases from type IV to type I, which is in accordance to the decrease of the plan dimension of the buildings. On the Y direction, the equivalent period increases from building type II, I, III and IV. In case of building type I and II, they have a similar plan dimension on the Y direction, yet the position of the shaft is different. As to building type III, the higher equivalent period obtained on the Y direction is related to the presence of the shafts that interrupt the side masonry walls.

Building type IV has similar equivalent period on both directions, which can be related to the structural configuration of the corner building. In terms of ductility, buildings III and IV present the higher values on both directions, which is consequence of higher number of elements on the structure (redundancy) in comparison with buildings type I and II and the redistribution of the loads. As to strength, there are significant differences between the main structural directions. For instance, if the X direction is considered, building type II and IV have higher strength than buildings type I and III. However, if the Y direction is considered, buildings type I and II have higher strength.

The verification of the Ultimate Limit State consists of checking if the structure withstands the seismic demand defined in [2] for Lisbon with a return period of 475 years. The far-field (type 1.3) seismic action was adopted with 5% equivalent viscous damping (ξ) for a foundation soil type C. The buildings' importance factors are equal to 1. Figure 10 plots the results of the PBA by means of: (a) the ratio between the acceleration in the structure with unlimited elastic behaviour $S_e(T^*)$ and in the structure with limited strength F_y^*/m^* , defined as q^* , and (b) the ratio between the ultimate and the performance displacements (d_u^*/d_{max}^*).

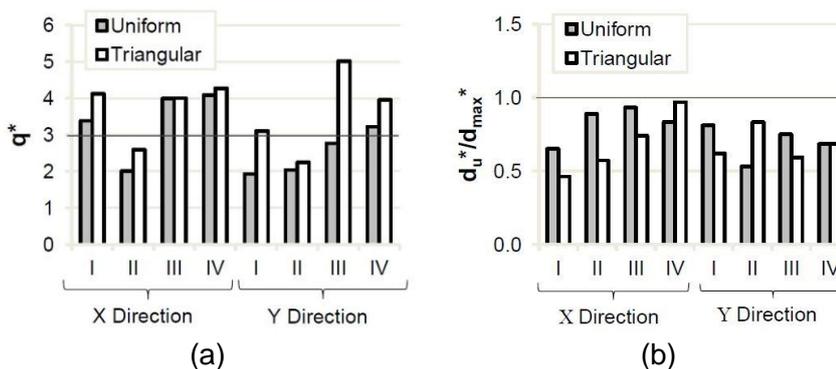


Figure 10. Results for the PBA: (a) factor q^* and (b) ratio between the ultimate and the performance displacements

Firstly, it is worth noting that the structural code [4] recommend $q^* < 3$ aiming to limit the overall acceptable ductility of the building. From Figure 10 (a) it can be concluded that only building type II verifies this condition on both directions, along with building type I on the X direction. All other cases show an insufficient strength. According to the PBA, safety is verified when $d_u^*/d_{max}^* > 1$, therefore, it can be inferred from Figure 10 (b) that none of the buildings fulfil the safety requirements for the Ultimate Limit State defined in [2,4] being the worst cases are building type I (mainly on the X direction) and building type II and III (on the Y direction).

Figure 11 plots the ratio between the maximum admissible ground acceleration ($a_{g,max}$), taking into account $q^* < 3$, and the reference ground acceleration (a_{gR}), which in case of seismic action 1.3 is equal to 1.5 m/s^2 . In this case, if $a_{g,max}/a_{gR} > 1$, the building seismic safety is verified. The results show that all four types of 'Gaioleiro' buildings considered in this study are not on the safe side, as, in general, the maximum admissible ground acceleration is much lower than the reference seismic demand acceleration. The exceptions are the results obtained on building type II for the pseudo-triangular load and building type III with the uniform load, yet the verification as to be fulfilled on both structural directions. It can also be concluded that buildings type III and IV are clearly the worst cases on the X direction. As far as the Y direction concerns, with the uniform load, the lower ratio between

accelerations was determined for building type II, whereas with the pseudo-triangular load is building type III. From all, building type IV is the one closer to verify the seismic safety.

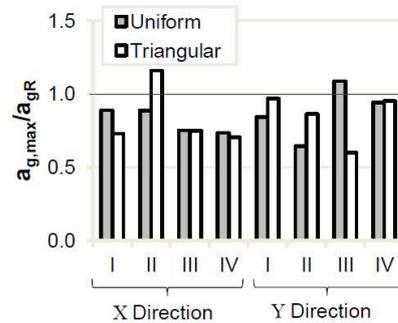


Figure 11. Ratio between the maximum admissible ground acceleration and the reference ground acceleration (taking $q^* < 3$)

In overall, it can be concluded that the cases studies considered, representative of four types of 'Gaioleiro' buildings in their original configuration, have a high seismic vulnerability and do not fulfil the requirements for the Ultimate Limit State defined in codes [2,4]. Due to this reason, a comparison in terms of damage probability distribution is discussed in the following section by introducing the fragility curve concept.

5 FRAGILITY ANALYSES

The seismic performance of the four types of 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 considered and directly defined on the capacity curve based on conventional displacement limits ($S_{d,ds}$). In this work, the limits proposed in [15] were adapted, being in case of (1) slight damage $S_{d,1}=0.7S_{dy}$; (2) moderate damage $S_{d,2}=S_{dy}$; (3) heavy damage $S_{d,3}=0.5(S_{dy}+S_{du})$; and (4) collapse $S_{d,4}=S_{du}$. The fragility curve were defined by lognormal functions that describe the probability of reaching, or exceeding, a defined damage state (ds) given the spectral acceleration ($a_{g,k}$) associated displacement (S_d), according to the following equation (1):

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

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 to the previous displacement limit stated ($S_{d,k}$) and a_g the ground acceleration.

The parameter β_{ds} represents different sources of variability and uncertainty related with the software model used (β_ϵ), the capacity curve (β_C) (i.e., related with the input parameters (geometrical and mechanical) which affect the global response), the seismic demand (β_D) and the definition of the damage limit states (β_{LS}). The value of β_ϵ was assumed equal to 0.25, β_C and β_{LS} were adopted from [16], while β_D was conventionally assumed equal to 0.25 given that not enough information was available on [2] for a more precise estimation (e.g. related to the input definition for different percentile values). The following $\beta_{ds,i}$ values were adopted 0.53, 0.54, 0.51, 0.49, respectively from 1 to 4, and equally adopted for the all building types.

Figure 12 and Figure 13 depict the probability of damage in case of the seismic demand type 1.3 for all the studied cases, respectively for X and Y directions. On this, P0 stands for “no damage”, P1 represents the probability of having “slight damage”, P2 the probability of having “moderate damage”, P3 the probability of having “heavy damage” while P4 the case of reaching “collapse”.

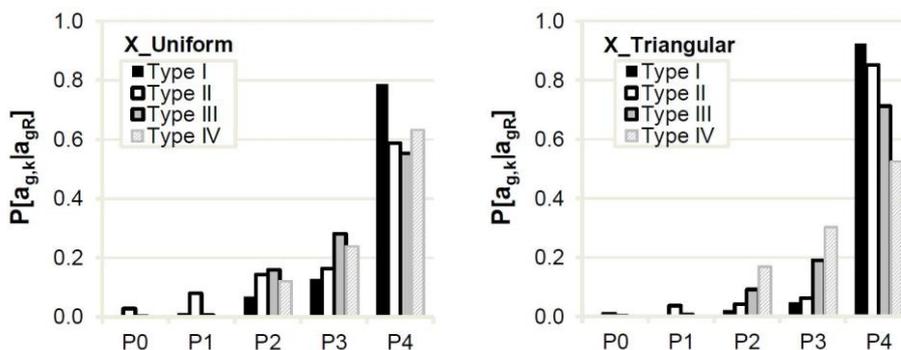


Figure 12. Damage probability on the X direction

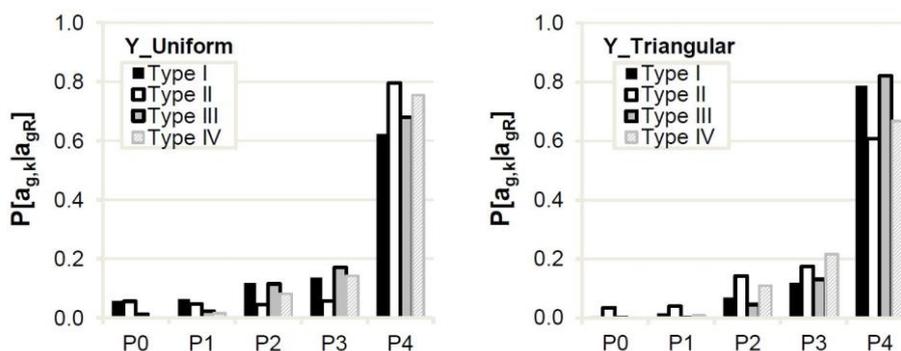


Figure 13. Damage probability on the Y direction

As expected, for the code seismic action, there is a high probability of reaching collapse (P4). On the X direction, the probability of reaching collapse is more than 0.5 in all cases, and considering both load patterns. From all buildings, type I is the most vulnerable case. On the Y direction, the probability of reaching collapse is between 0.6 and 0.8, being in the worst condition building type II for the uniform load and building type III for the pseudo-triangular load. In general, the Y direction presents the most penalizing structural condition. The results obtained are also in agreement with the verification of the Ultimate Limit State in §4. The probability of having moderate damage (P2) and heavy damage (P3) cannot also be ignored, in particular in case of buildings type III and IV on the X direction. After all things considered, it can be stated that the cases studied herein considered have very high seismic vulnerability considering the seismic demand for Lisbon, putting in evidence the need of defining appropriated strengthening solutions to increase their seismic capacity.

6 CONCLUSION

The work addresses the performance-based assessment of four types of ‘Gaioleiro’ buildings, an unreinforced masonry typology of buildings characteristic from Lisbon. The main goal was to compare the seismic performance of these types of buildings considering the original configuration and average properties of this typology. The assessment was addressed to the whole seismic response of the buildings. The in-plane capacity curves of the buildings case studies were derived by non-linear static (pushover) analyses and the structures’ performance points were defined through non-linear

procedures. A probabilistic assessment was, in addition, carried out and the damage probability obtained for each type of building.

The results show, as expected, that the buildings have higher stiffness and strength on the direction perpendicular to the façade walls, where a lower area of openings can be found. The evolution of the damage is, in general, characterized by the failure of spandrel beams by flexure, followed by the collapse of piers mainly by flexure in the lower floors of the building. It was concluded that all case studies considered have very high seismic vulnerability and do not fulfil the requirements for the Ultimate Limit State defined in codes [2,4].

A comparison in terms of damage probability distribution was, in addition, carried out supporting similar conclusions. From the cases considered, it can be stated that building type I, with strait size façade walls and one side shaft, has a very high probability of collapse on the direction parallel to the façade walls. On the perpendicular direction, the worst cases are buildings type III, with large façade walls and more than one shaft, and building type IV, the corner building. It can also be stated that this structural direction presents, in general, higher probability of collapse.

The results obtained confirmed, as expected, the need of improving the seismic capacity of these buildings. Nevertheless, it is important to refer that, in this study the buildings were analysed as isolated structures, but in reality they exist in aggregates being therefore restrained by the side buildings. Thus, in future work it is relevant to assess the influence of the boundary conditions on the global seismic behaviour of these buildings. All in all, results herein presented confirmed the need of improving the seismic capacity of these buildings by appropriate retrofitting solutions.

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REFERENCES

- [1] Lagomarsino, S.; Penna, A.; Galasco, A.; Cattari, S.: *TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings*. Engineering Structures, 56 (2013), 1787-1799, <http://dx.doi.org/10.1016/j.engstruct.2013.08.002>.
- [2] European Committee for Standardization (CEN): *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings (EC8-1)*. CEN 2004.
- [3] European Committee for Standardization (CEN): *Eurocode 8: Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings (EC8-3)*. CEN 2005.
- [4] Italian Code for Structural Design (Norme Tecniche per le Costruzioni – NTC), D.M. 14/1/2008, Official Bulletin Nº 29 of February 4, 2008 (In Italian).
- [5] Appleton, J.: *Rehabilitation of Gaioleiro Buildings*, Book edited by Orion Editors, 1st Edition 2005 (In Portuguese).
- [6] Lopes, M.; Meireles, H.; Cattari, S.; Bento, R.; Lagomarsino, S.: *Pombalino construction: description and seismic assessment*. In Structural Rehabilitation of Old Buildings (Eds.; A.Costa et al.), Building Pathology and Rehabilitation 2, Springer-Verlag Berlin Heidelberg, Chapter 7, 187-233, ISBN:978-3-642-39685-4, DOI 10.1007/978-3-642-39686-1_7.
- [7] Silva, V.; Soares, I.: *Seismic vulnerability of the Gaioleiro buildings of Lisbon and possible measures to reduce it*. Proc. of the 3rd Seismology and Earthquake Engineering Meeting, Lisbon, Portugal 1997 (In Portuguese).

- [8] Lopes, M.; Azevedo, J.: *Assessment of the seismic behaviour of a Lisbon masonry building. Proc. of the 3rd Seismology and Seismic Engineering Meeting*, Lisbon, Portugal 1997 (In Portuguese).
- [9] Candeias, P.: *Assessment of the Seismic Vulnerability of Masonry Buildings*. Ph.D. Thesis, University of Minho, Portugal 2008 (in Portuguese).
- [10] Mendes, N.; Lourenço, P.; Costa, A.: *Shaking table testing of an existing masonry building: assessment and improvement of the seismic performance*. Earthquake Engineering & Structural Dynamics, 2013, <http://dx.doi.org/10.1002/eqe.2342>.
- [11] 3Muri Program, S.T.A.DATA s.r.l., release 5.0.4.
- [12] Lagomarsino, S.; Penna, A.; Galasco, A.; Cattari, S.: *TREMURI program: Seismic Analyses of 3D Masonry Buildings, Release 2.0*. University of Genoa, Italy 2012 (mail to: tremuri@gmail.com).
- [13] Turnšek, V.; Sheppard, P.: *The shear and flexural resistance of masonry walls*. Proceedings of the International Research Conference on Earthquake Engineering, Skopje, Macedonia, 1980, pp. 517-573.
- [14] Fajfar, P.: *Capacity spectrum method based on inelastic spectra*. Earthquake Engineering and Structural Dynamics, Vol. 28 (9), 1999, 979-993, [http://dx.doi.org/10.1002/\(SICI\)1096-9845\(199909\)28:9<979::AID-EQE850>3.0.CO;2-1](http://dx.doi.org/10.1002/(SICI)1096-9845(199909)28:9<979::AID-EQE850>3.0.CO;2-1)
- [15] Lagomarsino, S.; Cattari, S.: *Fragility functions of masonry buildings*. Book from SYNER-G: Typology Definition and Fragility Functions for Physical Elements at Seismic Risk, Buildings, Lifelines, Transportation Networks and Critical Facilities Series, Geotechnical, Geological and Earthquake Engineering, Vol. 27 (2014), Edited by Pitilakis, K.; Crowley, H.; Kaynia, A., ISBN 978-94-007-7871-9.
- [16] Pagnini, L.; Vicente, R.; Lagomarsino, S.; Varum, H.: *A mechanical model for the seismic vulnerability assessment of old masonry buildings*. *Earthquake and Structures*, Vol. 2 (1), 2011, 25-42.