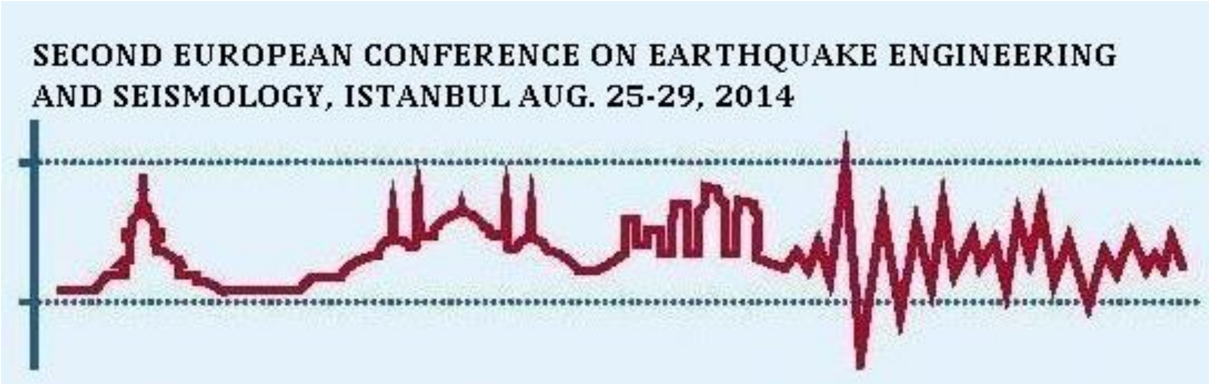

NON-LINEAR STATIC ANALYSES OF UNREINFORCED MASONRY "GAIOLEIRO" BUILDINGS

Ana SIMÕES[1], Rita BENTO[2], Serena CATTARI[3] and Sergio LAGOMARSINO[4]

ABSTRACT

The work herein presented is addressed to the non-linear static analyses of "Gaioleiro" buildings, an unreinforced masonry typology of buildings built in Lisbon in the late nineteenth century and the early twentieth century. The global seismic behaviour of four buildings, representative of different types of "Gaioleiro" buildings, was determined based on the assumption that proper connections prevented the activation of local failure modes mainly associated with the out-of-plane response of the walls. Incremental non-linear static (pushover) analyses were performed on the two main structural directions taking into account the in-plane capacity of the walls and the connection and load transfer between floors and walls. The seismic performance of the buildings was determined by comparing the expected performance of the structures with the seismic demand defined in Eurocode 8 for Lisbon. The results confirm the vulnerability of these buildings and the need of defining appropriate strengthening solutions.

INTRODUCTION

In the recent decades the seismic vulnerability of existing buildings is receiving more attention due to the increasing interest in the conservation of built heritage. It is estimated that unreinforced masonry buildings, built before the introduction of proper seismic code provisions, compose half of the existing building stock in Lisbon County. "Gaioleiro" buildings are characteristic of the construction in Lisbon in the end of the nineteenth century and the early twentieth century. This period was followed by a time of real estate speculation intended to sustain the housing needs of an increasing population, which, in some cases, ended up affecting the construction quality of the buildings.

Motivated by the strong uncertainties on the seismic behaviour of this typology of buildings, several research works have recently been carried out. Candeias (2008) perform a set of shaking table tests on a reduced scale model of a "Gaioleiro" building in order to evaluate its seismic performance before and after strengthening. Three strengthening solutions were tested aiming to prevent the out-of-plane collapse of the façade walls and the generalized cracking of masonry walls. Mendes and Lourenço (2009) present a numerical study based on the mentioned experimental results. The study included non-linear dynamic analysis with time integration and several types of pushover analyses.

Mendes (2012) also performed shaking table tests on a reduced scale model of a "Gaioleiro" building. After the first series of seismic tests, the building was repaired, aiming to re-establishing the

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initial conditions, strengthened and tested again. A numerical model of the building was defined and calibrated based on the test results and used to compare different types of non-linear analyses for the seismic assessment of “Gaioleiro” buildings.

Branco and Guerreiro (2011) developed a numerical study to compare the effects of different strengthening solutions on the seismic behaviour of a “Gaioleiro” building. The solutions included the seismic protection of the building through the insertion of concrete walls, base isolation technique and viscous dampers and different techniques to strength the wood floors.

The above mentioned research works demonstrated, on one hand, the structural vulnerability of “Gaioleiro” buildings and the importance of using suitable models and effective numerical tools for the seismic assessment of unreinforced masonry structures.

The work herein presented is addressed to the non-linear seismic response of four different types of “Gaioleiro” buildings. The building structures were modelled based on the non-linear equivalent frame approach in the Tremuri Program (Lagomarsino et al., 2013). 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 masonry panels follow the recommendations specified on structural codes Eurocode 8 (CEN, 2004) and Italian Code NTC (2008). The wooden floors were modelled as orthotropic membrane finite elements considering theirs effective stiffness.

The global seismic behaviour of the buildings, mainly governed by the in-plane capacity of the walls and to the connection and load transfer through the floor and roof diaphragms, was determined by incremental non-linear static (pushover) analyses. In order to assess the seismic performance of the buildings, the structure capacity was compared with the seismic demand defined in Eurocode 8 for Lisbon. The purpose of this work is to give an overview of the seismic vulnerability of “Gaioleiro” buildings and to highlight the need of considering anti-seismic provisions to preserve the historical and cultural value of these old unreinforced masonry buildings from Lisbon.

CASE STUDIES

Ancient masonry buildings were constructed for many centuries based on the available materials and empirical provisions justifying the strong uncertainties about their structural behaviour. The “Gaioleiro” buildings are mid-high rise structures with unreinforced masonry walls and flexible wooden floors and roof (Appleton, 2005). The exterior walls are made of rubble stone masonry and air lime mortar. The interior structure is composed by clay brick masonry walls: solid bricks on the lower storeys and hollow bricks on the top storeys, and lighter partition timber walls.

These buildings characterize a transition period between the “Pombalino” buildings, built after the 1755 earthquake, and the modern reinforced concrete (RC) buildings. “Gaioleiro” buildings are placed in blocks of buildings mainly in the neighbourhood of “Avenidas Novas”, between Marquês de Pombal and Campo Pequeno Squares (Fig. 1).

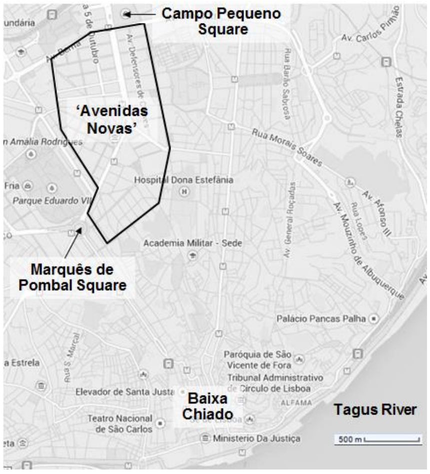


Figure 1. Identification of “Gaioleiro” buildings in Lisbon

The buildings have long rectangular plan shapes with side or interior shafts to provide natural light to the interior of the buildings. In general, “Gaioleiro” buildings can be divided in four types: type I – buildings with small size façade walls and one side shaft, type II – buildings with medium size façade walls and one shaft, type III – buildings with large façade walls and more than one shaft and type IV – buildings on the corner of the aggregate (Fig.2 shows examples of the buildings). To examine the seismic performance of “Gaioleiro” buildings, a building representative of each type was considered in this study: the plan geometry and a three-dimensional view of the case studies are plotted in Fig.3 and Fig.4, respectively.

Figure 2. Front façade wall of buildings type I, II, III and IV

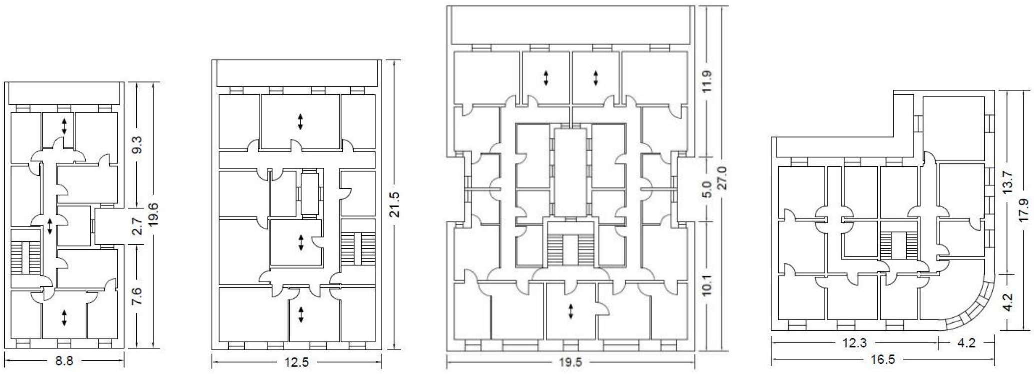


Figure 3. Plan geometry (dimensions in meters [m]) of buildings type I, II, III and IV

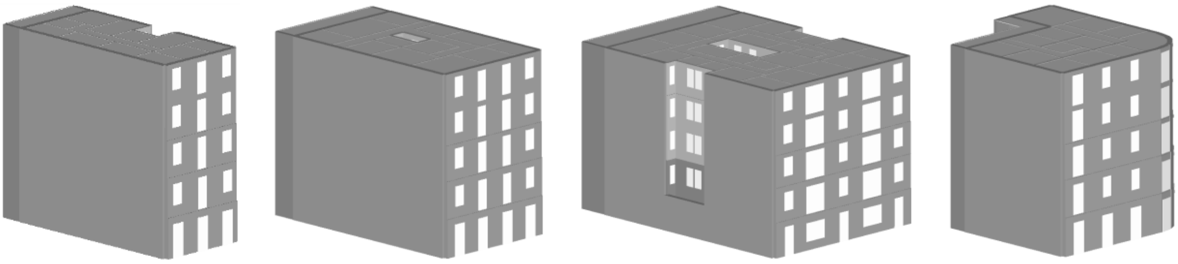


Figure 4. Three-dimensional view of buildings type I, II, III and IV

The main goal is to compare the global seismic performance of the different types of buildings that characterize the “Gaioleiro” typology. To this end, equivalent geometrical and mechanical properties were attributed to the four case studies. The buildings were considered with five storeys high and variable interstorey height: the ground floor has 3.6 m, the 1st floor has 3.5 m and the 2nd, 3rd and 4th floors have 3.3 m storey height. The thickness of the front façade wall is equal to 0.80 m and decreases 0.10 m in each floor till a minimum of 0.50 m. The thickness of the back façade wall is 0.60 m on ground and 1st floors and decreases 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.

The masonry mechanical properties were defined based on the values proposed in the Italian Code NTC (2008) for rubble stone masonry, solid and hollow clay brick masonry and calibrated by some experimental tests performed on existing “Gaioleiro” buildings (Lopes and Azevedo, 1997; Silva and Soares, 1997). Table 1 summarizes the mechanical properties and the gravity and live loads adopted. 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 τ [(1)] [MPa]	Specific Weight γ [KN/m^3]
Rubble Stone	0.90	0.29	1.30	0.026	19.0
Solid Brick	1.13	0.38	3.20	0.076	18.0
Hollow Brick	0.90	0.30	2.40	0.060	12.0

[(1)] Diagonal cracking failure mode according to the criterion proposed in Turnšek and Sheppard (1980)

Gravity Loads (Live Loads) [kN/m^2]
Floors: 0.7 (2.0)
Staircase: 0.7 (4.0)
Roof: 1.4 (2.0)
Balcony: 2.0 (2.5)

NON-LINEAR STATIC ANALYSES

Damage surveys carried out after earthquakes have shown that unreinforced masonry buildings are particularly vulnerable to local failure modes, mainly related to the out-of-plane response of walls. Improving the connections between orthogonal walls and between walls and floors prevents this type of mechanisms. Within this context, and assuming in this study that proper connections prevented the out-of-plane response of the walls, the global seismic response of the buildings is strictly related to the in-plane capacity of walls and the load transfer effects between floor and roof diaphragms and walls.

Thus, a three-dimensional model of the buildings was defined in Tremuri Program research version (Lagomarsino et al., 2012) following the equivalent frame model approach. Each masonry wall was subdivided into a set of deformable masonry panels (piers and spandrels), in which the non-linear response is concentrated, connected by rigid portions. Piers are the main vertical resisting elements supporting both the vertical and lateral loads. Spandrels are considered to be secondary elements in case of vertical loads but, in case of lateral loads, work as connecting elements between adjacent piers.

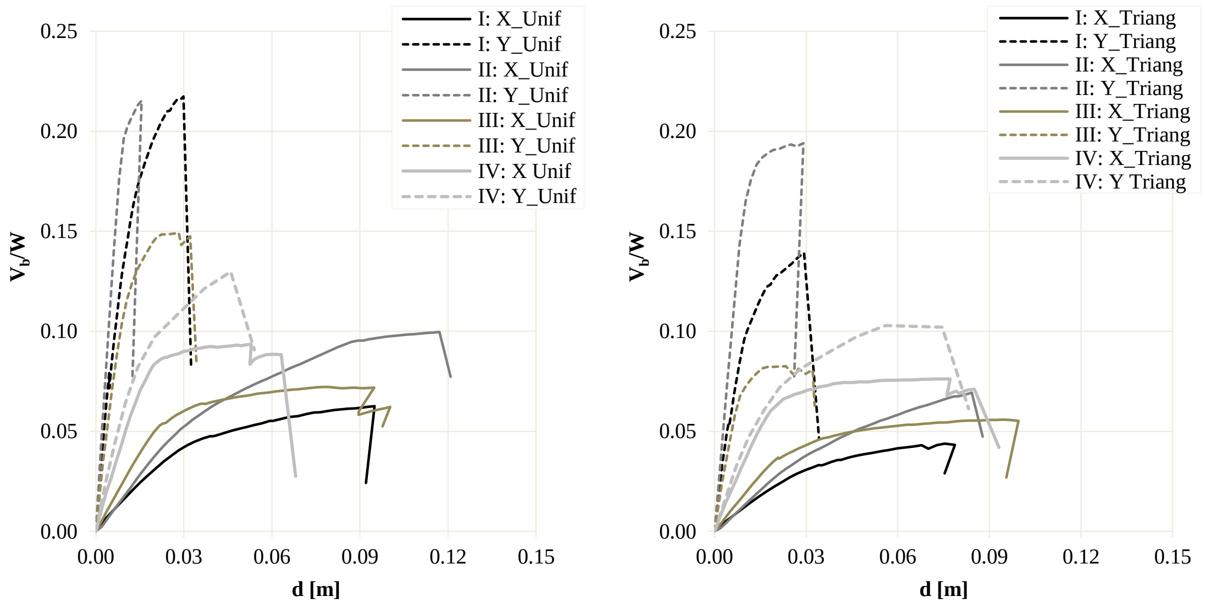
The global model is obtained by assembling the in-plane strength and stiffness of all walls and floors. The behaviour of the masonry piers and spandrels was modelled by non-linear beams with lumped inelasticity idealization (bilinear elastic perfectly plastic behaviour). The strength criteria for both bending and shear failure modes are consistent with the simplified recommendations in codes (Eurocode 8 (CEN, 2004) and the Italian Code NTC (2008)). 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) was considered, following the recommendation of the Italian Code NTC (2008) for existing masonry buildings.

The timber floors were defined through equivalent finite element membranes with 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 balconies of the back façade wall were also modelled. These balconies are made of I steel profiles connected, on the perpendicular direction, by clay brick arches. The I steel profiles are supported on the exterior masonry wall and on a steel frame structure composed by I steel profile beam and circular steel columns. 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.

The global seismic capacity of the buildings was defined through non-linear static (pushover) analysis performed on each main direction of the building considering two load patterns: (1) uniform, proportional to the mass; and (2) pseudo-triangular, proportional to the mass and height. The adoption of the pseudo-triangular load instead of a distribution strictly proportional to the first modal shape, as recommended by codes, is justified as follows.

Results from the modal analysis of the buildings shown a fraction of participant mass associated to the first mode, in average, equal to 60% and 65% in X and Y direction, respectively. This relatively low participation is justified by the flexible behaviour of the wood floors and the limited load transfer between walls. In the limit case of “infinitely” flexible floors, there would be no load transfer and each wall would be independent from the others. Thus, the adoption of the pseudo-triangular load distribution allows involving all the structural masses on the response of the structure.

The pushover curves are plotted in Fig.5 in function of the average displacement at the roof level against the base shear divided by the total weight of the building (V_b/W). The X direction corresponds to the direction of the façade walls and the Y direction is parallel to the side walls. The pushover analyses were stopped for 20% decay of the maximum base shear force specified in on the Eurocode 8 (CEN, 2004) and on the Italian Code NTC (2008) as the collapse condition.



(a) (b)

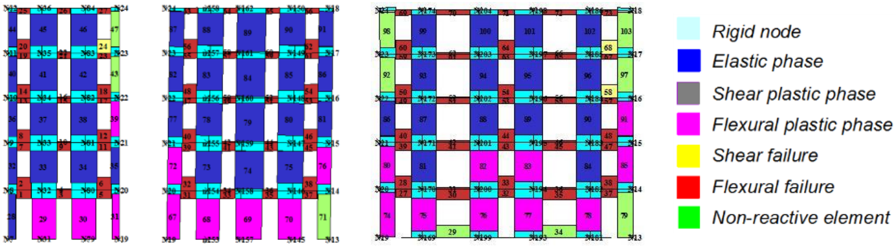
Figure 5. Normalized pushover curves for all types and for both directions adopting: (a) uniform load distribution and (b) pseudo-triangular load distribution

The results obtained with both load distributions define an envelope of the expected behaviour of the building. In general, with the pseudo-triangular load, lower stiffness and strength is determined. It is also evident from Fig.5 differences on the buildings’ capacity on the X and on the Y direction. 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. These differences are in part related with: (1) the rectangular configuration of the building; (2) the presence of blind walls on the Y direction that mainly governs the response of the building in this direction; in contrast with (3) the presence of several openings on the walls from the X direction. This variation is not so important in case of building type IV due to structural configuration of the corner building and the equal distribution on both directions of façade walls with openings and side blind walls.

Comparing the behaviour of buildings type I, II and III (Fig.5), 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.

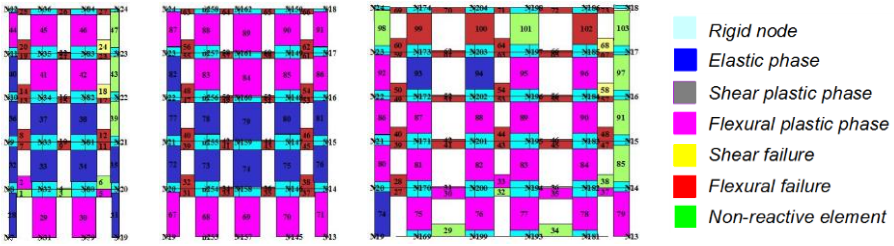
Building type IV is, from all cases, the one with higher stiffness and strength on the X direction and the one with the lowest on the orthogonal direction. The differences between structural directions are not so relevant, due to the L shape of the building that is slightly longer on the Y direction. One feature common on both structural directions is that the capacity curve obtained with the pseudo-triangular load has lower maximum base shear force, but higher ultimate displacement.

The damage pattern corresponding to the last step of the pushover analysis is depicted from Fig.6 to Fig.9 for the main walls of the buildings type I, II and III and in Fig.10 and Fig.11 for building type IV. 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 in each structural element.



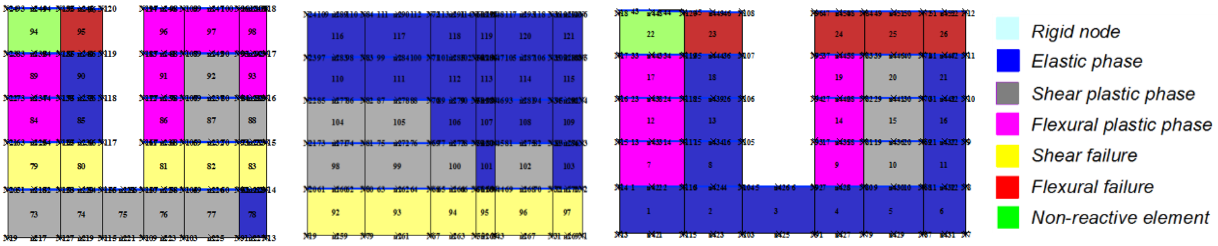
(a) (b) (c)

Figure 6. Damage pattern on the X direction façade walls from building (a) type I, (b) type II and (c) type III for the ultimate displacement of the analysis with uniform load



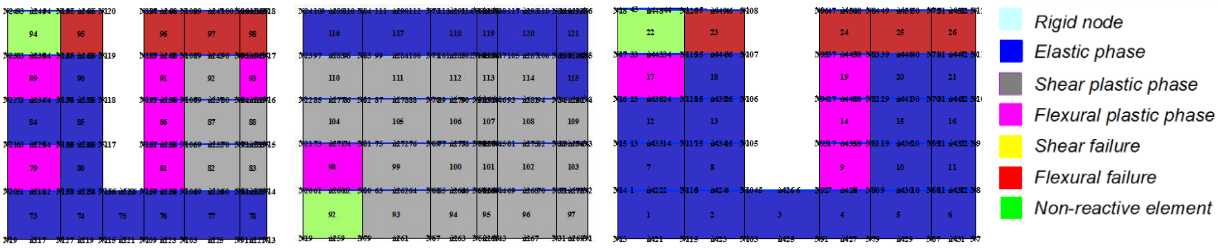
(a) (b) (c)

Figure 7. Damage pattern on the X direction façade walls from building (a) type I, (b) type II and (c) type III for the ultimate displacement of the analysis with pseudo-triangular load



(a) (b) (c)

Figure 8. Damage pattern on the Y direction side walls from building (a) type I, (b) type II and (c) type III for the ultimate displacement of the analysis with uniform load



(a) (b) (c)

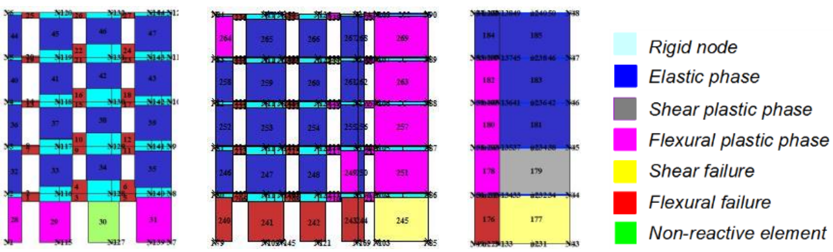
Figure 9. Damage pattern on the Y direction side walls from building (a) type I, (b) type II and (c) type III for the ultimate displacement of the analysis with pseudo-triangular load

In case of buildings type I, II and III, 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. 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 placed on the X direction also present disperse damage, in general due to flexural behaviour. These walls have an important contribution to the overall strength of the building, which also puts in evidence some capacity of the wood floors to transfer part of the loads from much damaged walls to still efficient ones. If a uniform load is considered, damage is mainly concentrated on piers from the ground floors, as can be seen in Fig.6. If a pseudo-triangular load is adopted, damage affects piers from both the ground and top floors, as is depicted in Fig.7.

On the Y direction, damage is mostly concentrated on the side 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 failure for the ultimate displacement (Fig.8 and Fig.9 (a) and (c)). Building type I exhibits, in addition, shear failure at the ground floor, which can be related to the asymmetric configuration of the building on the Y direction (shaft is only in one side of the building – see Fig.3) and the occurrence of torsional deformations. In what concerns building type II, damage is due to shear behaviour (Fig.8 and Fig.9 (b)). Notice that this building also has an asymmetric configuration on the Y direction due to the eccentric position of the interior shaft. The most demanding would certainly occur with the uniform load distribution case, which led to the shear failure of the piers from the ground floor for buildings type I and II.

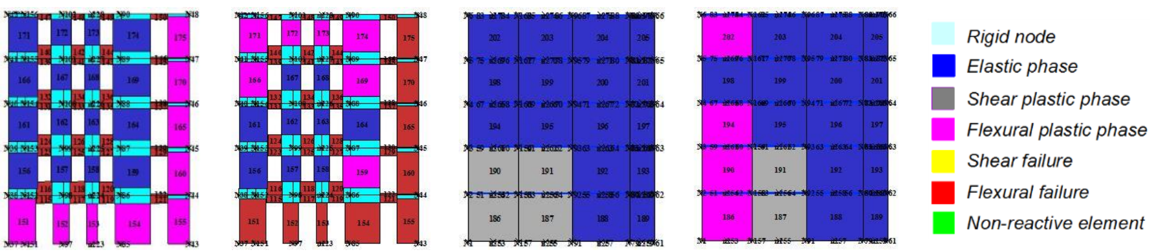
Damage on building type IV is a combination of the damage pattern described for the previous cases. In this case, X direction presents a similar pattern with both load patterns (Fig.10, corresponds to the damage with the uniform load) characterized by the flexural failure of the spandrels, the flexural damage of piers from the front façade wall (Fig.10 (a)) and the flexural failure of piers on the back façade wall (Fig.10 (b)), which is, in part, consequence of the L plan shape of the building. In addition, in Fig. 10 (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 damage 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 Fig.11 (a)), while with the pseudo-triangular these piers reached the flexural failure (Fig. 11 (b)). As to the side blind wall, damage is due to shear in the first case (Fig.11 (c)), and due to shear and flexure in the second (Fig.11 (d)).



(a) (b) (c)

Figure 10. Damage pattern for building type IV on the X direction with uniform load: (a) front façade wall; (b) back façade wall; and (c) side wall



(a) (b) (c) (d)

Figure 11. 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

SEISMIC PERFORMANCE

The seismic performance-based assessment comprehends the determination of the performance point or target displacement of a structure. This point is computed from the intersection between the capacity curve and the response spectrum that represents the seismic action. The safety condition consists of verifying that structure's ultimate displacement (d_u) is higher than the performance point (d_{max}).

The structure 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 defined on the Eurocode 8 (CEN, 2004), was adopted.

An elasto-perfectly plastic force-displacement relationship was assumed to define the SDOF capacity curve: the initial stiffness was determined based on the intersection with the point corresponding to 70% of the maximum base shear reached (0.70 $V_{b,max}$) 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 2 and Table 3 contain the properties of the capacity curves in the idealized elasto-perfectly plastic relationship, respectively for the X and Y direction, namely: period (T*), ductility (μ*) computed by the ratio between the ultimate displacement (d_u*) and the yielding displacement (d_y*) and strength, resulting from the ratio between the yielding force (F_y*) and the mass (m*).

Table 2. Properties of capacity curves on X direction

Building	Type I		Type II		Type III		Type IV	
X direction	Unif.	Triang.	Unif.	Triang.	Unif.	Triang.	Unif.	Triang.
T* [s]	1.17	1.35	1.13	1.27	0.85	1.06	0.64	0.76
μ*	2.20	1.90	1.78	1.48	3.71	2.96	3.40	4.13
F_y*/m* [m/s^2]	0.85	0.61	1.49	1.02	0.99	0.80	1.28	1.04

Table 3. Properties of capacity curves on Y direction

Building	Type I		Type II		Type III		Type IV	
Y direction	Unif.	Triang.	Unif.	Triang.	Unif.	Triang.	Unif.	Triang.
T* [s]	0.39	0.45	0.31	0.35	0.41	0.47	0.61	0.75
μ*	1.97	2.35	1.60	2.61	2.70	3.64	2.20	2.70
F_y*/m* [m/s^2]	2.92	1.81	2.77	2.51	2.04	1.12	1.75	1.42

Referring to the equivalent period of the buildings, type I and II have the highest values on the X direction and the lowest on the Y direction. This can be explained by the fact that types I and II have squat rectangular shapes with (1) small to medium size façade walls, which defines a more flexible behaviour of the buildings on X direction, but (2) relevant side blind walls which have higher stiffness and major contribution to the behaviour of the buildings on the Y direction. Moreover, it can also be noticed that, in general, X direction presents higher equivalent period, which is in agreement with a more deformable structural system (higher number of openings on the façade walls). In case of building type IV, the equivalent period is similar on both directions related to the L configuration of the building.

In terms of ductility, significant variations occur within the load distributions considered. This may derived from the type and distribution of damage on the structure. Focusing on the behaviour of buildings type I, II and III on the X direction, it was determined in the previous section that, for the uniform load distribution, damage on the structure is mainly concentrated on piers from the ground floors. However, if a pseudo-triangular load is considered damage is distributed on the structure, affecting piers from both the ground and top floors, supporting therefore the lower ductility obtained in this case. In general, buildings type III and IV present higher ductility on both directions. In all cases, the Y direction presents higher structural strength due to the contribution of the side walls, and as expected buildings I and II present higher value.

In order to assess the seismic performance of these building structures, the seismic demand defined in the Eurocode 8 (CEN, 2004) for Lisbon was considered with a return period (T_R) of 475 years (Ultimate Limit State requirement). The far-field (type 1.3) seismic action was adopted with 5% equivalent viscous damping (ξ) for a foundation soil type C. The results are presented in Fig.12 in terms of: (a) ratio between the ultimate displacement and the performance point of the equivalent structure (d_u*/d_{max}*); and (b) ratio between the maximum admissible ground acceleration ($a_{g,max}$) and the reference ground acceleration (a_{gR}), in this case equal to 1.5 m/s^2. The results in Fig.12 (b) take into account the ductility limitation recommended on the structural codes (q*<3). The structural safety is verified if in both cases the ratio is higher than the unit.

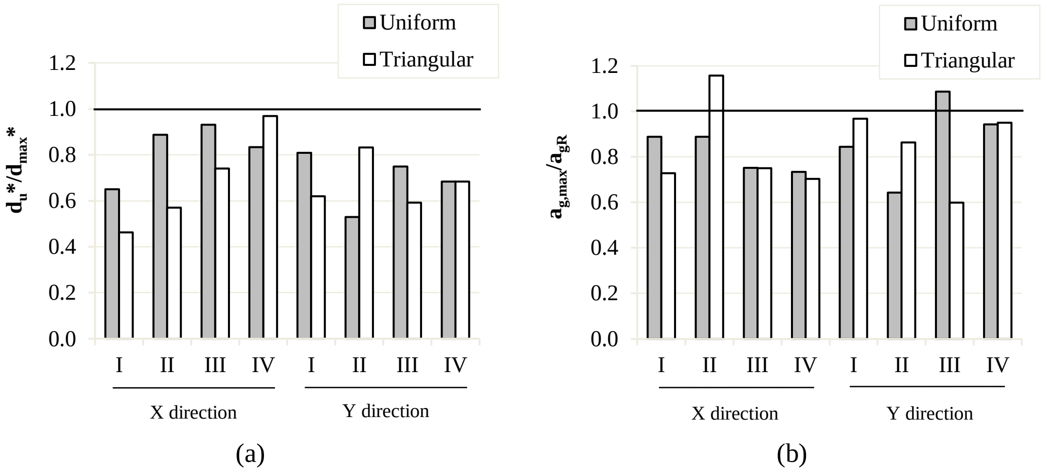


Figure 12. Seismic performance: (a) ratio between the ultimate and the maximum displacement and (b) ratio between the maximum admissible acceleration and the reference ground acceleration (taking q*<3)

It is clear that none of the building types fulfil the safety requirements for the Ultimate Limit State. In all cases, the ultimate displacement (d_u*) is lower than the performance point (d_{max}*), being the worst cases building type I on the X direction and building types II and III on the Y direction. It can be said from Fig.12 (b) that, in general, the maximum admissible ground acceleration is lower than the reference seismic demand acceleration. The exceptions are the results obtained on building type II for the pseudo-triangular load on the X direction and building type III with the uniform load on the Y direction, yet the verification as to be fulfilled on both structural directions.

It can also be concluded that, the limitation of the overall ductility of the building (q*<3) mainly affects the results on the X direction, as in this case, buildings type III and IV are clearly the worst cases in terms of safety verification. As far as the Y direction concerns, the results are more inconsistent. 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.

In overall, it can be stated, from the results obtained, that the case studies considered have very high seismic vulnerability as they do not fulfil the requirements for the Ultimate Limit State defined in codes considering the seismic demand for Lisbon. Nevertheless, it is important to refer that, in this study the "Gaioleiro" buildings were analysed as isolated structures, but in reality they exist in aggregates being therefore restrained by the side buildings. Thus, it will be relevant to assess the influence of the boundary conditions on the global seismic behaviour of these buildings. After all things considered, results confirmed the need of improving the seismic capacity of "Gaioleiro" buildings by appropriate retrofitting solutions.

CONCLUSION

The work addresses the seismic 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. 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.

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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