Seismic Pushover Analysis of 'Gaioleiro' Buildings in Lisbon

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ABSTRACT:
It is well known within the Portuguese scientific community that the masonry 'Gaioleiro' buildings, built in Lisbon between the late nineteenth and the early twentieth century, represent the masonry buildings with highest structural weaknesses. In this paper, a brief description of the 'Gaioleiro' buildings is presented and the results of the seismic assessment of an existing building are discussed. The structure was modelled based on the nonlinear equivalent frame model approach considering Tremuri Program (Lagomarsino et al., 2012). The in-plane capacity curves were determined with incremental static (pushover) analysis, supported on the assumption that the out-of-plane failure of the walls is prevented by adequate retrofitting measures. The main goal is to point out the seismic response of 'Gaioleiro' buildings in Lisbon, being the definition of the fragility curves for this typology of buildings the ultimate aim of the study.

Keywords: Masonry Buildings, Nonlinear Static Analysis, Seismic Assessment, Equivalent Frame Model

1. INTRODUCTION

Half of the existing buildings in Lisbon are composed by unreinforced masonry structures with relatively flexible wooden floors. Built before the introduction of proper seismic code provisions, the vulnerability assessment of this class of buildings is a crucial issue. The work herein presented is addressed to the nonlinear 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). Hence, suitable models and effective numerical tools are needed.

The study was extended to the analysis of different building structures representative of the 'Gaioleiro' typology (the variation mainly related with the plane dimension and configuration of the buildings). The structures were modelled based on the nonlinear equivalent frame model approach considering Tremuri Program (Lagomarsino et al., 2012). Each masonry wall was discretized by a set of panels (piers and spandrels) in which the nonlinear response is concentrated, connected by rigid areas, whereas the wooden floors were modelled as orthotropic membrane finite elements. The structures capacity curves were determined by incremental nonlinear static (pushover) analysis, supported on the assumption that the out-of-plane failure of the walls is prevented by adequate retrofitting measures (the code assumes that the global building response is governed by the in-plane behaviour of walls).

In this paper, a brief description of the 'Gaioleiro' buildings is presented and the results of the seismic assessment of an existing building type are discussed. This work represents a first contribution for the reduction of the seismic vulnerability of 'Gaioleiro' buildings in Lisbon, being the definition of the fragility curves for this typology of buildings the ultimate aim of the study. These data is essential for the definition of loss estimation studies, guaranteeing that the reduced structural safety of the masonry buildings is explicitly accounted in future intervention works.

2. UNREINFORCED MASONRY (URM) BUILDINGS

URM buildings are composed by plane structural elements: exterior and interior masonry walls and horizontal floor diaphragms. The internal structure (walls and floors) varies with the region and construction period. While, the walls are the resistant elements for both vertical and horizontal loads, floors have the role of distributing the loads to the walls mainly in function of the area of influence (due to the floor's flexible behaviour). The assessment of existing buildings aims to verify their structural seismic behaviour and to identify the need of retrofitting interventions. Therefore, accurate numerical models to predict the response of the structure are need.

The 'Gaioleiro' of buildings follows the 'Pombalino' buildings built after the 1755 Lisbon earthquake and characterized by the design of seismic resistant structures such as the 'gaiola pombalina', a detailed timber truss used to reinforce the masonry walls above the first floor (Meireles, 2012). A century after, the timber reinforcements were no longer a structural requirement. The fast city expansion determined the evolution of the construction technologies and the design of the 'Gaioleiro' typology of buildings.

The 'Gaioleiro' buildings have four to seven storeys height. The exterior walls are made of rubble stone masonry and air lime mortar. The interior walls are made of brick masonry, usually solid bricks on the ground storeys and hollow on the top storeys. The partition walls have a light timber structure. Floors are made of wooden beams placed perpendicular to the front and back façade walls. The buildings are placed in quarters with central courtyard, though the construction on the aggregate was random. There are cases of side by side buildings with perfectly symmetric structures, buildings with distinct plan and height configuration, buildings sharing the side masonry walls (yet, not necessarily built in simultaneous) and buildings detached in one of the sides.

While the corner buildings have a plan square shape, the buildings positioned on the band have a rectangular shape, leading 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 different types of buildings (Appleton, 2005): Type 1 – strait size façade walls and one side shaft, Type 2 – medium size façade walls and one shaft, Type 3 – large façade walls and more than one shaft and Type 4 – buildings on the corner of the aggregate. In this work the seismic analysis of a building Type 1 is performed, in its original conception and based on the main geometric and mechanical properties of the typology. The main purposes are the identification of the main structural weaknesses and the definition of the necessary retrofitting measures. The geometry of the building is represented in Fig. 2.1.
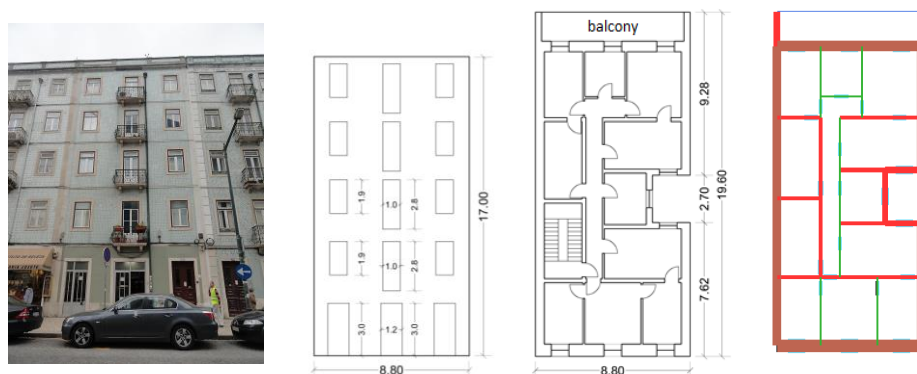

Figure 2.1. Building Type 1: front façade and plan geometry of the building and distribution of main walls (red) and secondary walls (green).

The building is five storeys high with variable interstorey height; 1st storey height is 3.6 m, 2nd storey height 3.5 m and 3rd, 4th and 5th storeys' height 3.3 m. The exterior walls are made of rubble stone masonry and air lime mortar with a decreasing thickness with the height of the building. The front façade wall is 0.80 m thick on the 1st storey, decreasing 0.10 m in each storey till a minimum of 0.50

m. The back façade wall is 0.60 m thick on the 1st storey, decreasing to 0.50 m on the 3rd storey. The side walls have a constant thickness of 0.50 m. The interior walls are made of clay brick masonry with different thicknesses depending on the arrangement of the bricks (brick size: 0.23 m x 0.11 m x 0.07 m). The main walls are 0.23 m thick and are made of solid bricks on the 1st and 2nd storey and hollow bricks on the 3rd and 4th storeys. On the 5th storey the thickness of the wall is reduced to 0.11 m. The secondary walls are made of 0.11 m hollow brick until the last storey, where is reduced to 0.07 m. The walls from the shafts and from the side of the back balconies are made of solid brick masonry with 0.35 m thickness, reduced on the last storey to 0.23 m. The plan distribution of the walls is shown in Fig. 2.1. Floors are made of wood beams placed perpendicular to the main façade walls and covered by wood boards. The balconies on the back façade wall have a composite structure made of steel profiles placed perpendicular to the façade wall and connected by masonry arches. These balconies are supported on a steel frame and on the side brick masonry walls.

The number of experimental campaigns addressed to the assessment of URM buildings in Lisbon is limited (Pompeu dos Santos, 1997; Lopes et al., 1997). Most of the studies are related to the test of masonry specimens built to reproduce existing walls, as the case of Milošević et al. (2012). Despite the efforts taken, the mechanical characterization of such an anisotropic material conducts to a great dispersion of results. In this study, the mechanical masonry parameters were defined based on the referred experimental tests and on the average values proposed in the Italian code (NTC, 2008) for rubble stone masonry, solid brick masonry and hollow brick masonry (in this case defined has the lower of the proposed values for solid brick masonry). Table 2.1 summarizes the mechanical parameters and gravity loads adopted in the numerical model. In case of the rubble stone masonry, the Young's and shear modulus corresponds to the cracked state. As to the brick masonry a 25% reduction of the elastic stiffness was assumed. This reduction was based on the results of parametric nonlinear finite element method analyses described in Cattari (2007) and Calderini et al. (2009).

Table 2.1. Mechanical parameters and loads adopted in the numerical model.

Mechanical Parameters	Young Modulus E (GPa)	Shear Modulus G (GPa)	Compressive Strength f_m (MPa)	Shear Strength τ_o [(1)] (MPa)	Specific Weight Υ (kN/m^3)	Gravity loads G (Variable loads Q) (kN/m^2)
Rubble Stone Masonry	0.9	0.29	1.3	0.026	19.0	Floors: 0.7 (2)
Solid Clay Brick Masonry	1.5 (1.25)	0.5 (0.375)	3.2	0.076	18.0	Staircase: 0.7 (4)
Hollow Clay Brick Masonry	1.2 (0.9)	0.4 (0.3)	2.4	0.06	12.0	Roof: 1.4 (2)
Steel (IPE180)	206	78.4	Steel Strength Class: S235		79.0	Balcony: 2.0 (2.5)

[(1)] Diagonal cracking failure mode by Turnšek and Čačovič criterion.

3. THE EQUIVALENT FRAME MODEL

Past earthquake damage surveys usually classifies the behaviour of URM buildings in: (i) global response, mainly governed by the in-plane capacity of the walls and the connection and load transfer between floors and walls (box behaviour); and (ii) local response, associated to the out-of-plane mechanism of single parts of the structure, usually negligible with respect to the overall behaviour of the building. The global model of the building is in this way defined by assembling the in-plane contribution of the masonry walls and floors.

Theoretical and experimental research also confirmed that the in-plane behaviour of masonry walls with openings may be discretized by a set of masonry panels where cracks and failure modes are concentrated (Magenes et al., 1998; Galasco et al., 2004). In this study, the 'Gaioleiro' buildings were defined based on the nonlinear equivalent frame model approach considering Tremuri Program originally developed and gradually improved at the University of Genoa, starting from 2001 (Galasco et al. 2004, Lagomarsino et al. 2012), and subsequently also implemented in the commercial software 3Muri (S.T.A.DATA s.r.l., release 5.0.4). The masonry walls were discretized into: (i) piers, vertical

elements, supporting both dead and seismic loads; (ii) spandrels, horizontal elements (between two vertically-aligned openings) coupling piers in the case of seismic loads; and (iii) rigid nodes, undamaged masonry portions confined between piers and spandrels. In this section, attention will be taken to the modelling solutions adopted (Fig. 3.1.a).
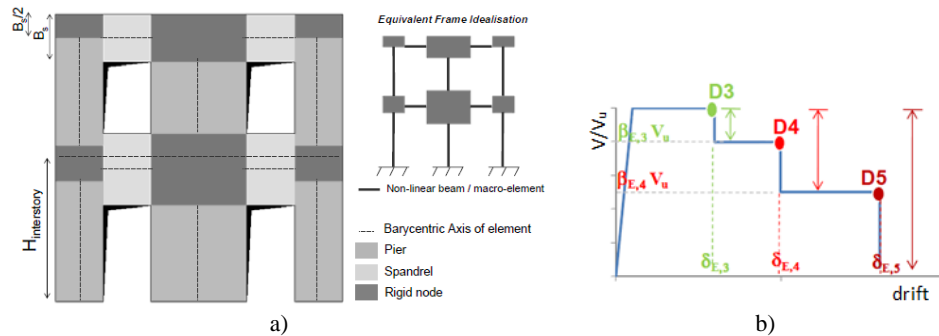

Figure 3.1. Tremuri Program: a) equivalent frame model idealization (Lagomarsino and Cattari, 2009); and b) masonry panels response according to the multi-linear constitutive law (Cattari and Lagomarsino, 2012).

The in-plane behaviour of masonry piers and spandrels was modelled by nonlinear beams with multi-linear constitutive law, a recent formulation developed within Perpetuate Project (http://www.perpetuate.eu, Deliverable 26, 2012) and implemented in the Tremuri Program. This formulation is in line with the multi-criteria approach proposed by the Perpetuate Project (Cattari and Lagomarsino, 2012) which combines the assessment of the damage level at different scales (structural element, macroelement and global).

The multi-linear constitutive law aims to describe the response of masonry panels until very severe damage levels with progressing strength decay and corresponding drift limits (Fig.3.1.b). In the multi-linear constitutive law, damage is described in five levels, ranging from 1 – no structural damage to 5 – total collapse. The reaching of progressing damage levels is defined in terms of drift limits (δ_E) (and corresponding strength percentage decay β_E) separately defined for piers and spandrels in function of the prevailing failure mode (if flexural or shear ones). Further details on these multi-linear constitutive laws are illustrated in Deliverable 26 of Perpetuate Project (2012).

The ultimate strength of the masonry panel is defined according to the simplified criteria proposed in the literature and codes, and implemented in Tremuri Program, in function of the possible failure modes, like as Rocking, Diagonal Cracking and Bed Joint Sliding (Lagomarsino et al. 2012, Cattari and Lagomarsino 2013). Mixed failure modes were also considered in the case of new multi-linear constitutive laws adopted through the combination of the limits defined for the flexural and shear failure modes. Once damage level 5 (DL5) is reached, the element contribution to the overall strength is only related to its capacity to bear vertical loads.

The wooden floors were modelled as anisotropic membrane finite elements. The local flexural behaviour of the floors and the out-of-plane response of the walls are not computed in the Tremuri Program because they are considered negligible with respect to the global seismic response of the building governed by the in-plane behaviour. At this stage of the study, it was assumed that local out-of-plane mechanisms of the walls were inhibited through specific retrofitting interventions (e.g. tie rods insertions, activation of the box behaviour).

4. NONLINEAR SEISMIC ANALYSES OF 'GAIOLEIRO' BUILDINGS

The seismic performance based assessment has led to an increasing use of nonlinear static procedures. These concepts are based on the comparison between the displacement capacity of the structure and the displacement demand of a given seismic action. The structure capacity is defined through a force-displacement curve, which describes the overall inelastic response of the structure and provides

essential information to idealize its behaviour in terms of stiffness, overall strength and ultimate displacement capacity. These curves can be obtained by a nonlinear incremental static (pushover) analysis, i.e. by subjecting the structure to a static lateral load pattern of increasing magnitude (representing the seismic forces).

Nonlinear static analyses were performed by the Tremuri Program for each main direction of the building (X and Y, both positive and negative direction) considering two load patterns: (i) uniform, which is proportional to the mass; and (ii) pseudo-triangular, which is proportional to the product between mass and height. Due to the flexible behaviour of the floors, the pseudo-triangular distribution is more accurate than the modal distribution proposed in codes (proportional to the first modal shape) as the fundamental mode shapes have a low mass participation. The 'Gaioleiro' building pushover curves are plotted in Fig.4.1 in function of the base shear force (V_b) and node control displacement defined at the roof level (d_{roof}). The analyses were stopped for 20% decay of the maximum base shear force (V_b) in agreement with the criterion proposed in EC8 (CEN, 2004) and the Italian code (NTC, 2008). This condition was assumed as reference to define the ultimate displacement capacity of the structure.
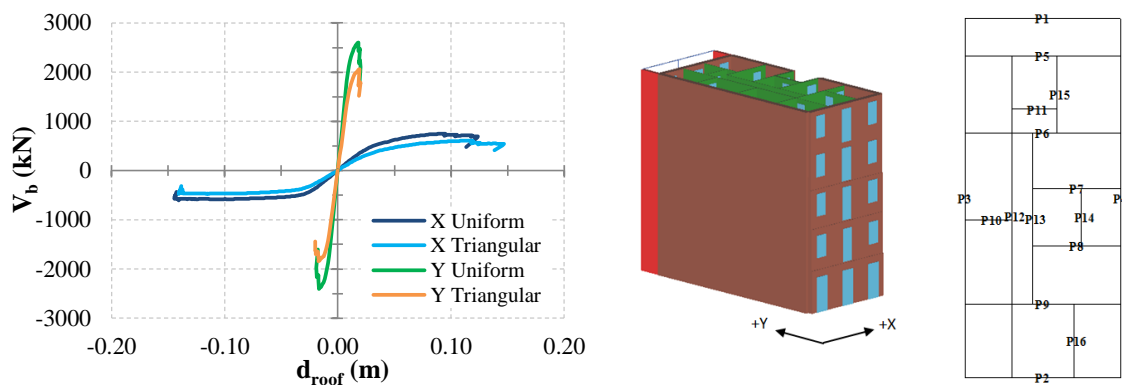

Figure 4.1. 'Gaioleiro' building pushover curves and plan identification of the walls.

It is clear that the building has a higher stiffness and strength on the Y direction than in the X direction. These differences are in part related with the rectangular configuration of the structure, the number of openings on the façade walls in contrast with the side walls of the building and the floor orientation. In what concerns the structure response to the load patterns, a lower base shear force (i.e. lower capacity) is determined with the pseudo-triangular load pattern. However, at a global scale the response of the building to both load distribution is analogous.

For this reason, the pseudo-triangular distribution was adopted as reference for the following remarks. Fig. 4.2 and Fig. 4.3 display the nonlinear behaviour of the building on the X and Y direction, in a) with the contribution of the walls to the total base shear force, while b) e and c) plots the interstorey drift and damage pattern of walls P9 and P4 (see Fig. 4.1), where the worse results were obtained.

The evolution of the damage pattern is characterized by the failure of spandrels followed only in the final phase by the collapse of piers. This performance is related with the moderate axial load acting on spandrel elements and the absence of other tensile resistant elements coupled to them, resulting in much reduced strength of the element and weak coupling for piers. Damage on piers is concentrated at the 2nd storey consequence of the structural variations between the top storeys (reserved for residential purposes) and the ground storey (shops and ware house).

The global failure of the structure on the X direction is triggered by the behaviour of wall P9 (parallel to the main façade wall P2 – Fig. 4.1) leaving the façade walls P2 and P5 with a secondary role, as shown in Fig. 4.2 a). On the positive direction (+X), wall P9 has an important contribution to the structure base shear, and at the end, the mixed shear-flexure failure of the piers from the 2nd storey defines the reduction of the total base shear. On the negative direction (-X) the walls have a uniform

contribution. In this case, damage is extended to the flexure collapse of piers on walls P9, P8 and P10. Damage on the façade walls is due to the failure of spandrels. These differences between the pushover curves on the positive and negative direction are in part due to the asymmetric distribution of the structure on the X direction. On the Y direction (Fig. 4.3 a), the side walls P3 and P4 have the major contribution to the total base shear. The building side shaft creates a structural discontinuity, causing higher displacements on the side wall P4 and the shear failure of the piers from the 2nd storey. Damage on the remaining walls is between 2<DL<3 for the ultimate displacement (d_u).
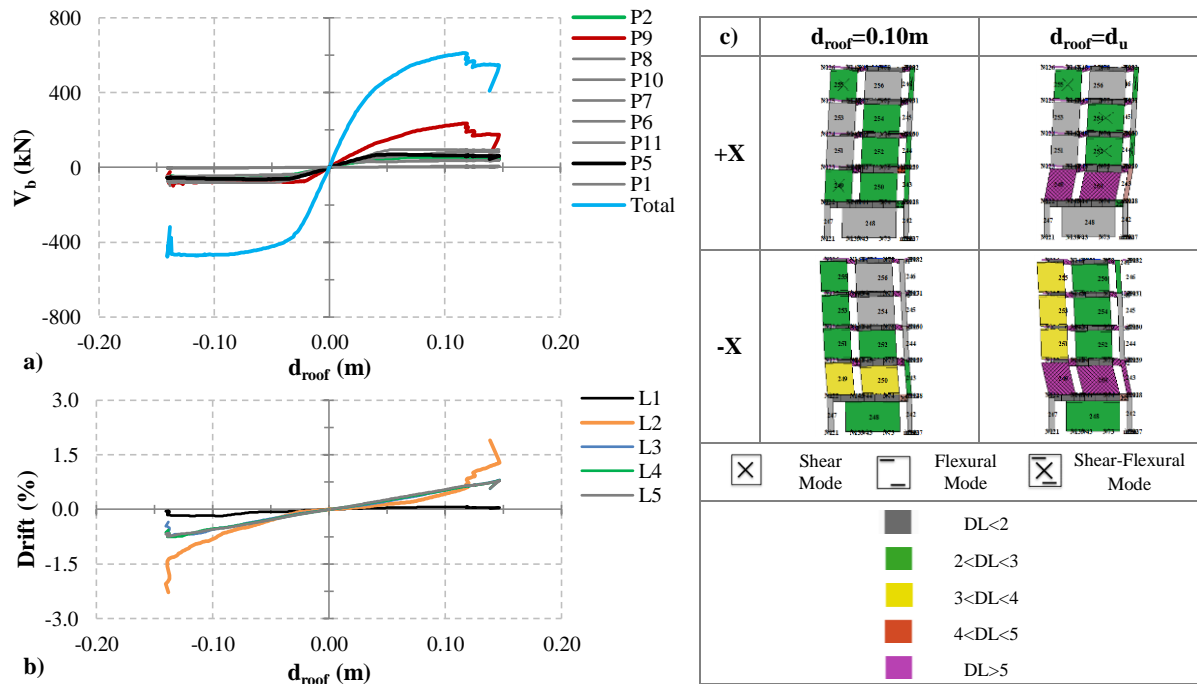

Figure 4.2. Results on the X direction for the pseudo-triangular load pattern: a) contribution of the walls to the total base shear; and wall P9 b) interstorey drift and c) damage pattern.

Figure 4.3. Results on the Y direction for the pseudo-triangular load pattern: a) contribution of the walls to the total base shear; and wall P4 b) interstorey drift and c) damage pattern.
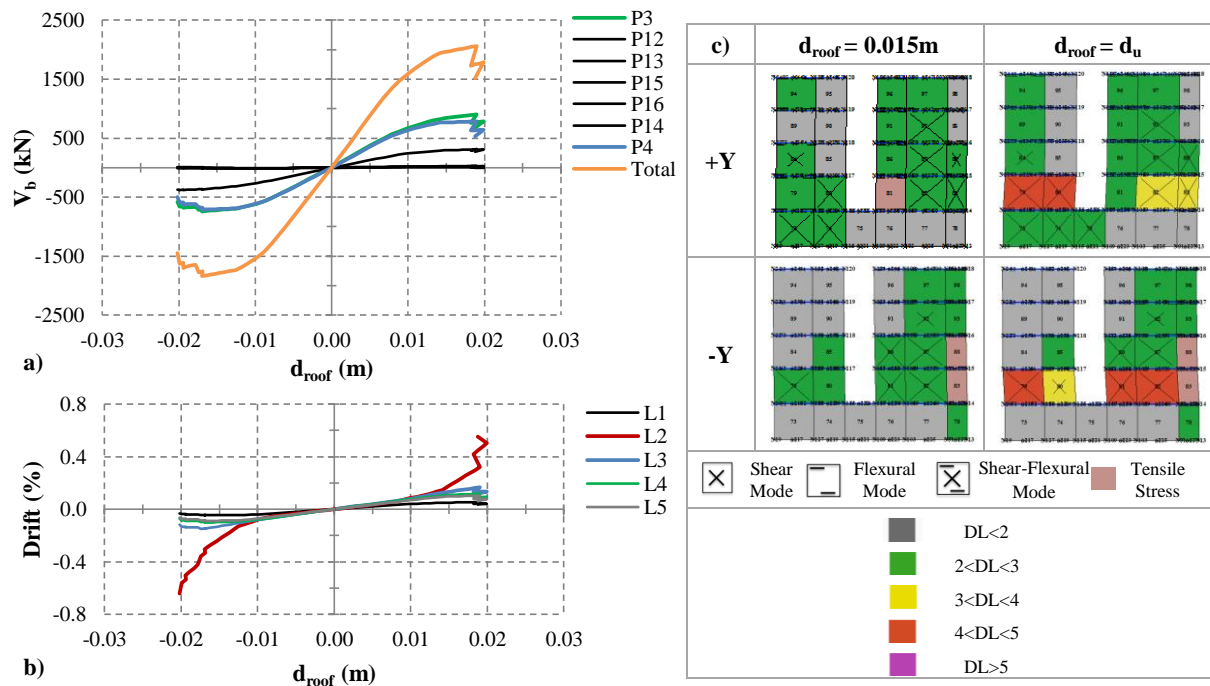
The focus of the nonlinear procedures is the identification of the structure performance point (or target displacement – d_t), computed from the intersection between the capacity curve and the elastic response spectrum that represents the seismic action. The safety verification consists of checking that the target displacement (d_t) is lower than the structure ultimate displacement capacity (d_u), which is assumed to be correspondent to 80% of the maximum base shear force (collapse condition). The N2 Method, adopted in both EC8 (CEN, 2004) and the Italian code (NTC, 2008), was considered to assess the seismic performance of this 'Gaioleiro' building. The seismic demand was defined according to the EC8 (CEN, 2004) for Lisbon with 475 years return period and 5% critical damping coefficient. The far-field (SA 1.3) and near-field action (SA 2.3) have reference peak ground acceleration respectively equal to 1.5 m/s^2 and 1.7 m/s^2. The foundation soil is Type C. The nonlinear static procedure is illustrated in Fig. 4.4 including the idealized SDOF capacity curves, the response spectrum for the far-field action (SA 1.3) and near-field action (SA 2.3) and the corresponding performance points (d_t).
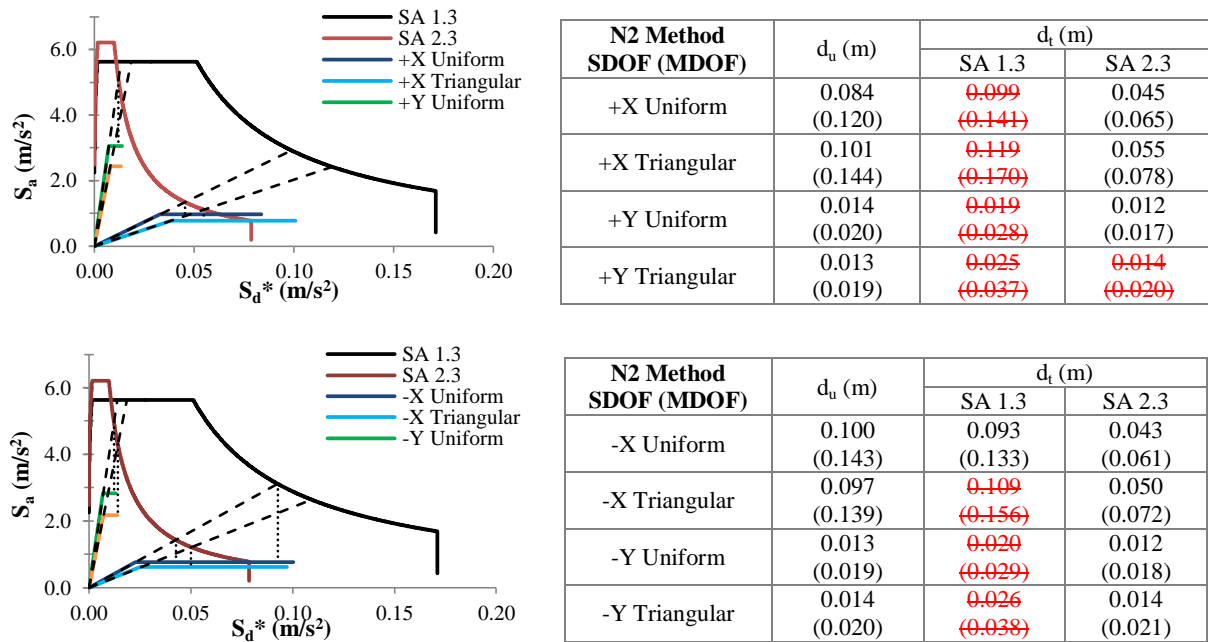

N2 Method SDOF (MDOF)	d_u (m)	d_t (m)	
		SA 1.3	SA 2.3
+X Uniform	0.084 (0.120)	0.099 (0.141)	0.045 (0.065)
+X Triangular	0.101 (0.144)	0.119 (0.170)	0.055 (0.078)
+Y Uniform	0.014 (0.020)	0.019 (0.028)	0.012 (0.017)
+Y Triangular	0.013 (0.019)	0.025 (0.037)	0.014 (0.020)

N2 Method SDOF (MDOF)	d_u (m)	d_t (m)	
		SA 1.3	SA 2.3
-X Uniform	0.100 (0.143)	0.093 (0.133)	0.043 (0.061)
-X Triangular	0.097 (0.139)	0.109 (0.156)	0.050 (0.072)
-Y Uniform	0.013 (0.019)	0.020 (0.029)	0.012 (0.018)
-Y Triangular	0.014 (0.020)	0.026 (0.038)	0.014 (0.021)

Figure 4.4. Identification of the performance point by the N2 Method.

The far-field action (SA 1.3) is the most demanding case. The analysis –X Uniform is the only case where the structure ultimate displacement is higher than the target displacement determined for both actions. Since the requirements must be fulfilled for all cases, the building does not verify the no-collapse requirement and for that reason needs to be retrofitted. Nevertheless, it is important to state that the studied building, representative of 'Gaioleiro' buildings Type 1, was analysed as an isolated structure. This type of building was originally positioned inside an aggregate of buildings with similar structural configuration and, occasionally, sharing the side masonry walls. On the other hand, several 'Gaioleiro' buildings have been demolished and replaced by reinforced concrete buildings, changing, even if temporarily, the boundary conditions of the adjacent buildings. Therefore, the seismic performance of building Type 1 herein presented represents a first approach to the subject. Further studies have to be carried to compare the behaviour of the building inside the aggregate, with different boundary conditions, and the influence of the group effect to its seismic performance.

5. FINAL REMARKS

The results presented in this paper are a starting point to the reduction of the seismic vulnerability of the 'Gaioleiro' masonry buildings in Lisbon. In this paper, a brief description of the typology is presented and the results of the seismic assessment of an existing building based on nonlinear static procedures are discussed. The structure was modelled with the equivalent frame model approach

considering Tremuri Program (Lagomarsino et al., 2012). The building structures capacity curves were determined by incremental nonlinear static (pushover) analysis, supported on the assumption that the out-of-plane failure of the walls is prevented by adequate retrofitting measures (the code assumes that the global building response is governed by the in-plane behaviour of walls).

The analysis procedure herein presented approaches the seismic performance of Type 1 'Gaioleiro' building, characterized by a strait rectangular shape in plan, in its original conception and based on the main geometric and mechanical properties of the typology. It was concluded that the structure does not verify the no-collapse requirement and for that reason the needs to be retrofitted. However, further studies have to be carried to compare the behaviour of the building inside the aggregate, with different boundary conditions, and the influence of the group effect to its seismic performance. The final aim of the study is to extend the seismic assessment to the four types of 'Gaioleiro' buildings in their original conception, in order to define of the fragility curves for this typology of buildings. These data is essential for the definition of loss estimation studies, guaranteeing that the reduced structural safety of the masonry buildings is explicitly accounted in future intervention works.

REFERENCES

Appleton, J. (2005). Rehabilitation of 'Gaioleiro' Buildings. Orion Edition, 1st Edition (in Portuguese).

Calderini, C.; Cattari, S.; Lagomarsino, S. (2009). In-plane strength of unreinforced masonry piers. *Earthquake Engineering and Structural Dynamics,* Volume 38, Issue 2, pp. 243-267.

Cattari S. (2007). Modelling of existing masonry and mixed masonry-reinforced concrete buildings by the equivalent frame approach: formulation of synthetic models. *Ph.D. Thesis*, University of Genoa, Italy.

Cattari S., Lagomarsino S. (2012). Performance-based approach to earthquake protection of masonry cultural heritage. *Proceedings of SAHC Conference*, Jerzy Jasieńko (ed) DWE, Wrocław, Poland, pp. 2914-2922.

Cattari S., Lagomarsino, S. (2013). Seismic assessment of mixed masonry-reinforced concrete buildings by non-linear static analyses. *Earthquake and Structures*, 4 (3).

Deliverable D26 (2012). Modelling strategies for seismic global response of building and local mechanisms. *Perpetuate Project* (downloadable from http://www.perpetuate.eu).

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).

Galasco, A.; Lagomarsino, S.; Penna, A.; Resemini, S. (2004). Nonlinear analysis of masonry structures. *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada.

Italian Code for Structural Design (Norme Tecniche per le Costruzioni – NTC) (2008). D.M. 14/1/2008, Official Bullettin no. 29 of February 4 2008 (In Italian).

Lagomarsino S, Penna A, Galasco A, Cattari S. (2012). TREMURI program: Seismic Analyses of 3D Masonry Buildings, Release 2.0. University of Genoa, Italy (mailto: tremuri@gmail.com).

Lagomarsino, S.; Cattari, S. (2009). Nonlinear seismic analysis of masonry buildings by the equivalent frame model. *Proceedings of the 11th D-A-CH Conference: Masonry and Earthquakes,* Zurich, Switzerland.

Lopes, M.; Azevedo, J. (1997). Avaliação do Comportamento Sísmico de um edifício de alvenaria de Lisboa. *Proceedings of the 3rd Seismology and Seismic Engineering Meeting*, IST, UTL, Portugal (in Portuguese).

Magenes, G.; Della Fontana, A. (1998). Simplified nonlinear seismic analysis of masonry buildings. *Proceedings of the British Masonry Society*, Volume 8, pp. 190-195.

Meireles, H. (2012). Seismic Vulnerability of Pombalino Buildings. *PhD Thesis in Civil Engineering*, IST, UTL, Lisbon, Portugal.

Milošević, J.; Sousa Gago, A.; Lopes, M.; Bento, R. (2012). Experimental Tests on Rubble Masonry Specimens. *Proceedings of the 15th World Earthquake Conference*, Lisbon, Portugal, pp. 1191.

Pompeu dos Santos, S. (1997) Ensaios de Paredes Pombalinas. *Nota Técnica Nº15/97, NCE/DE*, LNEC Lisboa (in Portuguese).

S.T.A.DATA. 3Muri Program, Release 5.0.4; 2012 (http://www.3muri.com)

Turnšek, V.; Čačovič, F. (1970). Some experimental results on the strength of brick masonry walls. *Proceedings of the 2nd International Brick Masonry Conference*, Stoke-on-Trent, pp. 149-156.