Seismic Vulnerability of Old Masonry 'Gaioleiro' Buildings in Lisbon

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SUMMARY:

Current strategies for the seismic assessment of existing buildings emphasize the use of nonlinear static (pushover) analysis. The present paper aims to make a contribution to the assessment of the seismic vulnerability of unreinforced masonry based on the nonlinear equivalent frame modelling analysis. The façade wall of a 'Gaioleiro' building was modelled making use of the software SAP2000® (CSI) for the evaluation of its in-plane seismic performance. The masonry nonlinear behaviour was considered through lumped plastic hinges defined according to the masonry macro-elements failure mechanisms. The wall capacity curve was determined with an incremental static (pushover) analysis. The limit states compliance criteria defined for existing masonry structures were verified for the seismic performance point of the structure. This in-plane analysis is supported on the assumption that the out-of-plane failure of the wall was prevented by adequate retrofitting measures.

Keywords: Masonry Buildings, 'Gaioleiro' Buildings, Nonlinear Static Analysis, Performance Based Assessment

1. INTRODUCTION

A significant part of the building stock in Portugal was built before the introduction of proper seismic code provisions. It is estimated that old masonry buildings composed of several load-bearing masonry walls arranged in perpendicular plans and relatively flexible wooden floor diaphragms constitute half of the existing building stock in Lisbon County. The characteristics of these buildings coupled with the functions that they still maintain nowadays, justify the concern about their structural safety and seismic vulnerability.

The modern seismic performance based procedures comprehend the assessment of the expected performance of a building structure under a defined seismic demand. The structural capacity is based on a force-displacement (pushover) curve determined with a nonlinear static analysis in which the structure is subjected to a static lateral load or displacement pattern of increasing magnitude (describing the seismic forces). Although the performance based seismic design/assessment of masonry buildings is proposed on Eurocode 8 (CEN), on the Italian Standard OPCM 3431 (2005) and on the FEMA 356 (2000) by means of the nonlinear static analyses, the common practice in Portugal is still based on elastic linear analyses.

This paper aims to make a contribution for the seismic assessment of a particular typology of masonry buildings known as 'Gaioleiro', which is believed to present the highest seismic vulnerability of the old buildings in Lisbon. The façade wall of an existing building was modelled making use of the software SAP2000® (CSI) based on the equivalent frame modelling strategy. The masonry nonlinear behaviour was considered through lumped plastic hinges defined according to the masonry macro-elements failure mechanisms. The in-plane capacity of the masonry façade wall was determined through a nonlinear static (pushover) analysis. The limit states compliance criteria defined for existing masonry structures were verified for the seismic performance point of the structure. This in-plane analysis is supported on the assumption the out-of-plane failure of the wall was prevented by adequate retrofitting measures. The out-of-plane response of the wall should be evaluated with separate modelling.

2. 'GAIOLEIRO' BUILDINGS

The masonry 'Gaioleiro' buildings are characteristic of the urban expansion of Lisbon to the north upland at the end of the nineteenth century and the beginning of the twentieth century. The exterior walls were made of rubble stone masonry with a decreasing thickness along the height of the buildings. The side walls, frequently shared by adjacent buildings, are interrupted by light-shafts providing natural light and ventilation to the interior rooms.

The three-dimensional timber structure enclosed on the interior walls of the building above the first story, characteristic of the preceding 'Pombalino' buildings, was progressively abandoned. Conversely, the masonry walls are not laterally supported by the interior structure and, are therefore prone to out-of-plane failure. The connections between walls and between walls and floors are probably one of the main weaknesses of these buildings when subjected to seismic actions. Other structural limitations are related with the increasing number of floors and high ceiling heights. The age of the buildings combined with the lack of proper maintenance work and the posterior structural interventions performed on the buildings, affects the strength and durability of the structural materials.

The building under analysis was built in 1911 in Duque de Loulé Avenue, Lisbon. The left side of the building is next to a new reinforced concrete structure, while the right side is next to a pedestrian access (Figure 1). The building comprises a basement and five storeys with variable ceiling height. The exterior walls were built in rubble limestone masonry bounded by air lime mortar. The interior walls parallel to the façade walls are made of hollow brick masonry, while the remaining partition walls have a light timber structure. The floors are made by wooden beams perpendicular to the façade walls.

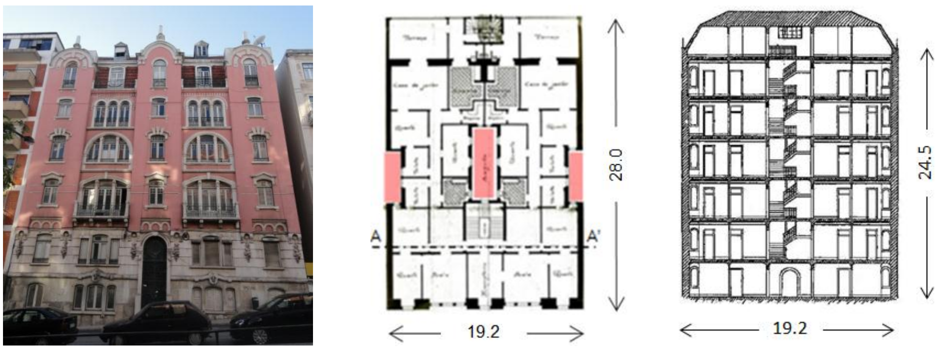


Figure 1. Façade wall of the building, plan drawing of a current floor and cut AA' (dimensions in meters).

3. THE PROPOSED NUMERICAL MODEL

The equivalent frame modelling strategy is based on the discretization of the bearing walls of the building by a set of panels (piers and spandrel beams) function of the pattern of openings or of pre-existing damage state. The panels are modelled with elasto-plastic behaviour and limited deformation, while the joint elements are modelled with rigid behaviour to model the coupling effect between masonry piers and spandrel beams.

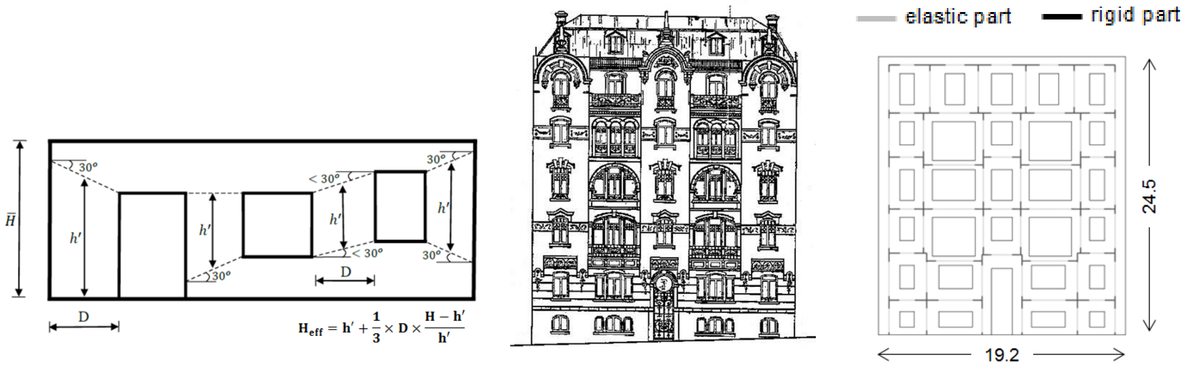
This modelling approach was first proposed by Tomaževic and after reviewed by Magenes and Della Fontana (Magenes and Della Fontana, 1998). The method has proved to be very attractive in comparison to more complex finite element models (Calderini *et al*., 2009; Penelis, 2006; Kappos *et al*., 2002); nonetheless its reliability depends on the consistency between simplified hypotheses on the geometry and the mechanical behaviour of materials and the actual building.

Pasticier *et al*. (Pasticier *et al*., 2008) verified the possibility of performing nonlinear static analysis of masonry structures making use of the equivalent frame method on the software SAP2000® (CSI). The

reliability and limitations of the code were first investigated by performing the static pushover analysis of two multi-storey walls already analysed by other authors using advance softwares (Liberatore, 2000). The main limitation of SAP2000® (CSI) is the impossibility of updating the axial load values during the analysis, which can change with the increment of the lateral loads during the analysis. However, comparing the analysis results it was concluded that this variation was not relevant.
A similar procedure will be used in this work to analyse the in-plane performance of the masonry façade wall of the 'Gaioleiro' building. The out-of-plane response of the wall should be evaluated with separate modelling. Nevertheless, adequate retrofitting measures should be taken to provide a good connection between orthogonal masonry walls and floors that prevent the out-of-plane failure mechanism of the building.

3.1. Geometric Properties

The façade wall of the 'Gaioleiro' building has a constant thickness of 0.90 m from the basement up until the fourth floor, where the wall thickness is reduced to 0.80 m till the rooftop. In the equivalent frame modelling, each pier and spandrel beam is defined by frame elements with the equivalent cross section dimension and height. To take the coupling effect between piers and spandrels in account, rigid offsets are assigned at the ends of the frame elements. The rigid length offset of the piers was determined based on an empirical approach proposed by Dolce (1989) as depict in Figure 2 a), whereas full rigid offset is used for spandrel beams. The façade wall of the building and the equivalent frame model are represented in Figure 2 b) and c). Due to the irregularity of the openings, additional rigid elements were introduced to make the connection between the elements that are not aligned.



a) b) c)

Figure 2. a) Effective height determination proposed by Dolce (Dolce, 1989); b) Façade wall of the building and c) Equivalent Frame Model (dimensions in m).

3.2. Mechanical Properties

Masonry structures are heterogeneous, anisotropic and largely influenced by the constitutive materials and constructive system. With the macro-scale of the equivalent frame model the masonry behaviour is idealized as a homogenous and isotropic material, characterized by the Young's modulus and the Poisson ratio. The masonry piers and spandrel beams are defined with elastic behaviour, while the nonlinear behaviour is assigned with concentrated plasticity (the only available approach in the software SAP2000®) making use of plastic hinges. The connection between the frame elements is modelled by fully rigid offsets.
The nonlinear behaviour of the masonry, mainly dependent on cracking and crushing of masonry, is modelled as defined in specific sections according to the masonry macro-elements failure mechanisms represented in Figure 3. The rocking failure occurs by the overturning of the wall and simultaneous crushing of the compressed corner. Sliding shear derives from the formation of tensile horizontal crack in the bedjoints related with low levels of vertical load and/or low friction coefficients. On the diagonal shear cracking, the peak resistance is governed by the formation and development of inclined diagonal cracks (Magenes *et al*., 1997).

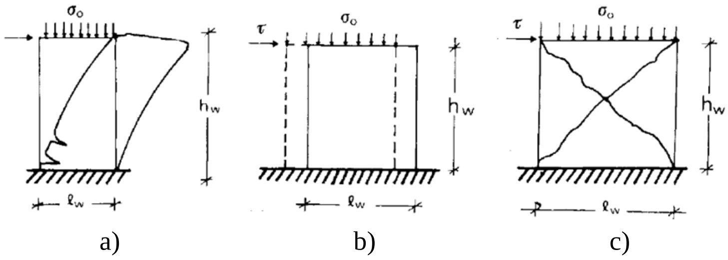


Figure 3. Failure mechanisms of a masonry pier (Pasticier *et al.*, 2008): a) rocking, b) sliding shear, c) diagonal shear cracking.

In equivalent frame model, the masonry piers were modelled with elasto-plastic behaviour followed by failure (sudden loss of load carrying capacity) (Figure 4 a) considering rocking hinges at both ends of the flexible part of the piers and shear hinges at middle height. It is assumed that the plastic hinges of masonry piers have a rigid-perfectly plastic behaviour (Figure 4 b).

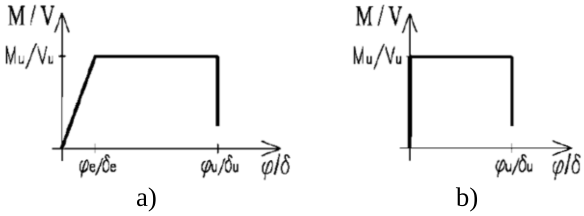


Figure 4. Masonry Piers (Pasticier *et al.*, 2008): a) Elasto-plastic behaviour of the element, b) Plastic hinge behaviour.

Rocking failure occurs when the moment (M) at any of the end sections of the effective pier length attains the ultimate moment (M_u – Eqn. 3.1). According to Eurocode 6 Part 1 (EC6-1, CEN, 2001), the ultimate moment (M_u) may be approximated to a proper stress distribution for the masonry in compression (tensile strength is neglected).

$$M_u = \frac{\sigma_0 \times D^2 \times t}{2}\left(1 - \frac{\sigma_0}{k \times f_d}\right) \tag{3.1}$$

In Eqn. 3.1 σ_0 is the mean vertical stress, D the pier width, t the pier thickness, k accounts for the location of the resultant of vertical compressive stresses (equal to 0.85) and f_d is the design compressive strength of masonry.
The shear strength was modelled according to two strength criteria. The first is defined in OPCM 3431 (2005) for the diagonal shear cracking strength for in-plane actions and is given by Eqn. 3.2:

$$V_u^{\,f} = \frac{1.5 \times f_{v0d} \times D \times t}{\xi}\sqrt{1 + \frac{\sigma_0}{1.5 \times f_{v0d}}} \tag{3.2}$$

where f_{v0d} is the design shear stress and ξ depends on the geometry of the pier and accounts for the distribution of shear stress at the centre of the pier (1.5 if H/D>1.5; H/D if 1≤H/D≤1.5; 1 if H/D<1).
The second criteria refers to sliding shear failure based on a Mohr-Coulomb formulation (Magenes *et al.*, 1997) and considers that the average ultimate shear stress is related to the uncracked section assuming a simplified distribution of compression stress. The shear strength is given by Eqn. 3.3:

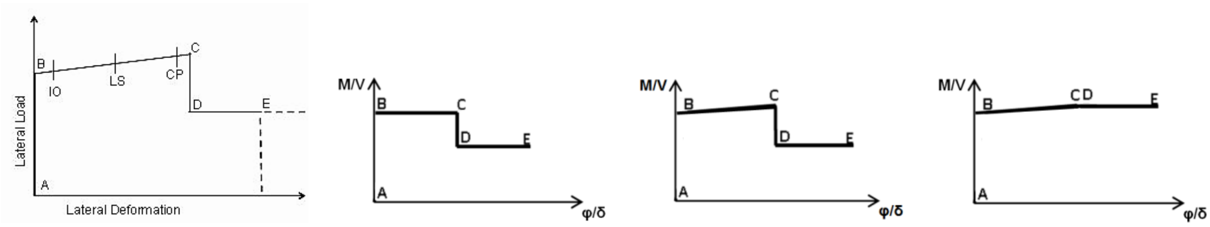
$$V_u^{\,s} = \frac{1.5 \times f_{v0d} + \mu \times \sigma_0/\gamma_m}{1 + \frac{3 \times H_0}{D \times \sigma_0} \times f_{v0d}} \times D \times t \tag{3.3}$$

where H_0 is the effective height of the pier (up to the inflection point), γ_m the safety factor (equal to 2) and μ the coefficient of friction.

The shear hinge strength is given by the minimum value from Eqn. 3.2 and 3.3. According to OPCM 3431 (2005), the ultimate displacement (δ_u) for in-plane actions of every pier is assumed to be 0.4% of the deformable height (H) of the pier for shear failure and 0.6% for flexure failure.
Pasticier *et al*. (Pasticier *et al*., 2008) modelled the masonry spandrel beams with elasto-brittle behaviour with residual strength after cracking by assigning one shear hinge at mid-span. The shear strength criterion was defined according to OPCM 3431 (OPCM 3431, 2005 – Eqn. 3.4) assuming the presence of reinforced concrete ring beams or a lintel beam strong in flexure, which is not the case on the ‘Gaioleiro’ building under analysis.

$$V_u = e \times l \times f_{v0d} \tag{3.4}$$

In Eqn. 3.4 e is the spandrel thickness, l the spandrel width and f_{v0d} the shear stress.
Even though spandrel beams are prone to early cracking due to low level of axial load, it is assumed that the ultimate resistance of a masonry building is dictated by the failure of masonry piers (Magenes and Della Fontana, 1998), (Calvi *et al*., 1996). Therefore, in the present case the spandrel beams were modelled with linear elastic behaviour; however the shear strength criterion considered by Pasticier *et al*. (Pasticier *et al*., 2008) and defined by Eqn. 3.4 will be assigned to identify in each step which elements would have exceed the maximum shear force.
On the software SAP2000® (CSI) the constitutive relationship of plastic hinges are defined by five points (A – origin, B – yielding, C – ultimate capacity, D – residual strength, E – failure) as shown in Figure 5 a. Even foreseen by the code, the sudden strength loss (C to D) is often unrealistic and may introduce numerical problems in the analysis (hinges violate their predefined path). To overcome this problem Demírel (Demirél, 2010) suggests the introduction of a 10% post elastic stiffness increment to the hinge model (Figure 5 c – Case 2), and after the elimination of the sudden strength loss after the ultimate capacity (Point C) (Figure 5 d – Case 3). With this last configuration (Case 3) no convergence problems or unreliable solutions arose throughout the nonlinear analysis. These results were subsequently compared with a nonlinear analysis finite element model generated using ANSYS code and a good agreement was found with Case 3 hinge configuration. These plastic hinge modelling options (Case 1, 2 and 3) will also be tested in the present study.



a) b) c) d)

Figure 5. Plastic Hinge (Demirél, 2010): a) SAP2000 hinge definition; b) hinge with perfectly plastic behaviour – Case 1; c) hinge with post elastic stiffness – Case 2; d) hinge with post elastic stiffness without final strength loss – Case 3.

It must be emphasized that the strength criteria herein adopted were developed for a particular type of brick masonry structures. The façade wall of the ‘Gaioleiro’ building under analysis was built in rubble limestone masonry. Nevertheless, the lack of experimental tests for the mechanical characterization of this typology of masonry walls leads to the need of using the referred formulation. The properties considered were based on other research works developed on old rubble stone masonry structures as no experimental in situ tests were performed. Table 1 summarizes the adopted values and the data source. The masonry behaviour was described by a bi-linear relation with zero tensile strength according to EC6-1 (CEN, 2001). The weight loads were concentrated on the top section of the façade piers considering the masonry wall self-weight and the weight of the roof, timber floors and interior partition walls estimated trough the influence area of the elements on the façade wall. As the building is uninhabited, acting live loads were not considered.

Table 1. Masonry's mechanical properties.

Density (ρ)	2.24 ton/m^3	The Young's Modulus was calibrated based on in situ dynamic characterization of the building (Branco *et al*., 2007). The compressive strength value was adopted from Branco *et al*. (2007).
Young's Modulus (E)	1000 MPa	
Distortion Modulus (G)	416.7 MPa	
Poisson's ratio (ν)	0.20	
Compressive Strength (f_d)	4.0 MPa	
Cohesion (τ_0)	0.065 MPa	The shear parameters were calibrated with experimental triplet tests developed on limestone rubble masonry with air lime mortar prototypes representative of old masonry walls (Milošević *et al*., 2012).
Coefficient of friction (μ)	0.447	

The dynamic characteristics of the façade wall were determined by means of a modal analysis. The fundamental period of the structure (T_1=0.40seconds (s)) is characterized by a significant mass on the direction of the façade wall (ΣM_x=0.75), whereas the second mode deformed shape is vertical (T_2=0.15s and ΣM_z=0.82). If 12th modes are considered, an accumulated participation of mass greater than 90% in both in-plane directions is found (ΣM_x=0.97 and ΣM_z=0.93).

4. PERFORMANCE BASED ASSESSMENT

4.1. Capacity Curve

Performance based assessment of unreinforced masonry structures requires the estimative of the system damage under a given earthquake demand. The structure overall capacity is defined by a force-displacement curve that provides the description of the inelastic response of the structure in terms of stiffness, strength and ultimate displacement (Calderini *et al*., 2009).
The nonlinear behaviour was restricted to masonry piers as defined in section 3.2, assuming that the ultimate failure of the masonry buildings is controlled by piers. To obtain the structural capacity curve the equivalent frame model was subjected to a static lateral load pattern of increasing magnitude representing the inertia forces. Two lateral load patterns are recommended in Eurocode 8 Part 1 (EC8-1, CEN, 2004): (i) Uniform Pattern – proportional to mass (uniform response acceleration) and (ii) Modal Pattern – proportional to mass and modal acceleration. The load patterns were applied at each storey level in proportion to the weight load concentrated on top of the façade piers.
The corresponding capacity curves in terms of roof displacement (right corner of the wall) and the base shear force are plotted in Figure 6 considering the plastic hinge modelling options as follows: Case 1 – perfectly plastic behaviour (Figure 5 b), Case 2 – 10% post elastic stiffness (Figure 5 c) and Case 3 – 10% post elastic stiffness without final strength loss (Figure 5 d).

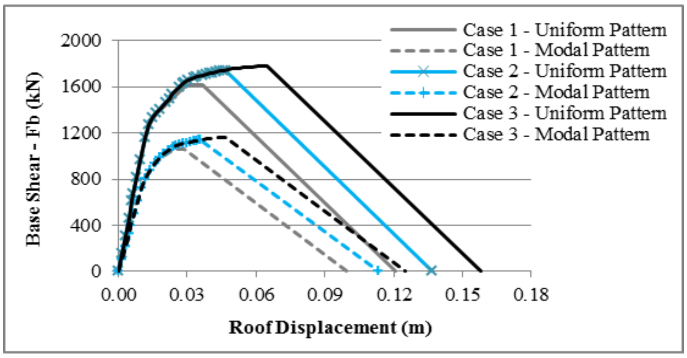


Figure 6. Façade wall capacity curves.

Table 2. Capacity curve's ultimate results.

Hinges Model	$F_{b,max}$ (kN)	d_{max} (m)	Load Pattern
Case 1	1619.0	0.121	Uniform
	1054.7	0.100	Modal
Case 2	1741.7	0.137	Uniform
	1136.0	0.113	Modal
Case 3	1780.1	0.158	Uniform
	1160.2	0.125	Modal

Within each load pattern, a good agreement between the capacity curves from the three hinges modelling options is obtained; however, with Case 1 and 2 several plastic hinges were not following the defined force-displacement or moment-rotation path. In case 1, the structure's capacity curves stopped before the effective exploration of the plastic behaviour assumed for the masonry elements.

With the 10% increment post elastic stiffness (Case 2 and 3) the analysis proceeds in the nonlinear range originating capacity curves with higher base shear strength. With Case 3, no convergence problems or unreliable solutions arose throughout the analysis, in accordance with the conclusions advanced by Demírel (Demirél, 2010). Although this 10% increment corresponds to a strain hardening ratio of 2.6% at maximum, which does not noticeably change the hinge behaviour, additional research is needed to support these hypotheses. With no further considerations, the distribution of the plastic hinges at the final stage of Case 3 capacity curve is provided in Figure 7. As referred in section 3.2, spandrel beams were modelled with linear elastic behaviour; however the elements, which have exceeded the shear strength defined by Eqn. 3.4, were identified in Figure 7. Unfilled circles represent the hinge yielding stage, while filled circles represent the hinge failure.

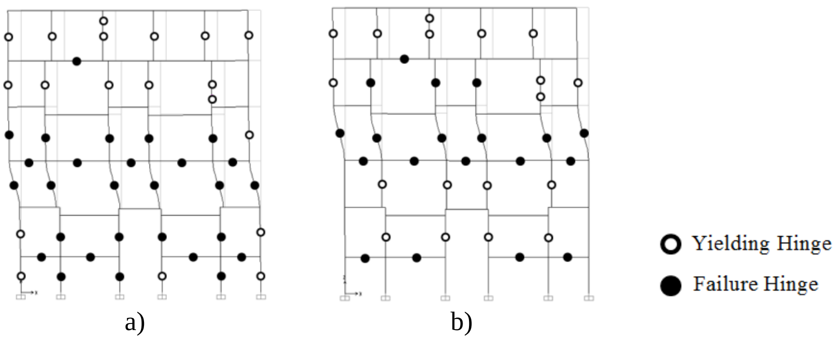


Figure 7. Plastic hinges distribution at the final stage of Case 3: a) Uniform Load; b) Modal Load.

For the uniform load pattern (Figure 7 a), the piers from the 2nd floor collapsed with a shear failure mechanism for a final roof displacement of 0.158 m. For the modal load pattern (Figure 7 b), the collapse is also caused by shear failure of the piers but concentrated at the 3rd floor, for a final roof displacement of 0.125 m. As mentioned before, although the predicted shear failure in the beams may not cause a global collapse of the building, according to the results it seems also necessary to provide a proper retrofitting scheme to these elements. Different collapse mechanisms were generated with both load pattern analysis. Nevertheless, the more reliable (numerical convergence) and more conservative (lower base shear force) load pattern 'Case 3 – Modal Pattern' was adopted for this study.

4.2. Structural Performance

The seismic performance based analysis comprehends the determination of the expected behaviour of a structure when subjected to an earthquake. The state of the structure is defined based on the intersection between the structure's capacity curve and the seismic demand (elastic response spectrum) both plotted in spectral coordinates (acceleration-displacement response spectra – ADRS). In the N2 Method (N stands for nonlinear analysis and 2 stands for two mathematical models), recommended in the EC8-1 (Annex B, CEN, 2004), the transformation of the capacity curve (multiple-degrees-of-freedom – MDOF) into an equivalent single-degree-of-freedom (SDOF) capacity curve is performed. This conversion enables the relation of the structure's capacity curves with the seismic demand already defined for a SDOF system. Knowing the intersection point, called as the target displacement, and converting it to the MDOF structure, the maximum level of deformation and stresses expected in the structure for that seismic demand are defined.
The seismic demand was defined in accordance with the EC8-1 Portuguese Annex (NP EN 1998-1, 2009) for Lisbon, seismic zone 1.3 and 2.3, with reference peak ground acceleration (a_{gR}) respectively equal to 1.5 m/s^2 and 1.7 m/s^2. The action was defined for a return period (T_R) of 475 years and critical damping of 5%. The foundation soil was considered as ground type C and the building was considered to belong to importance class II. Attending to the local hazard and the dynamic properties of the model (T_1= 0.40s), the seismic action type 1 has a higher spectral acceleration and is therefore more important for the seismic assessment of the structure. A 65% reduction of the elastic seismic action was also considered in accordance with OPCM 3431 (OPCM 3431, 2005) for the assessment of existing buildings. Figure 8 depicts the capacity curve (idealized equivalent SDOF system) obtained

with the modal load pattern and the seismic demand (including the 65% reduction of the action) in spectral coordinates (Acceleration-Displacement Response Spectra – ADRS) including the graphic intersection between the curves according to the N2 Method procedure for short period range ($T^*<T_C$). Table 3 presents the values of the target displacement for the equivalent SDOF system (d_t^*) and the correspondent target displacement (d_t) for the MDOF system.

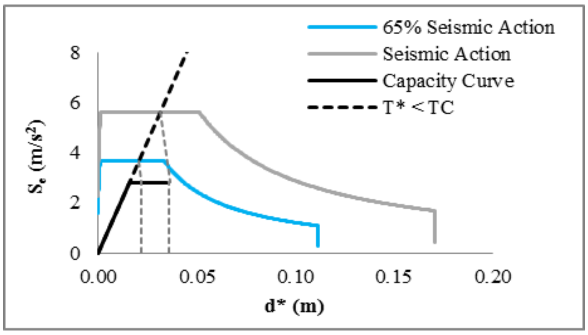


Figure 8. Acceleration-Displacement Response Spectra (Short Period Range).

Table 3. Target Displacement.

Target Displacement	d_t^*	d_t (m)
65% Seismic Action	0.022	0.028
Seismic Action	0.036	0.047

In order to assess the structural performance for both seismic demands, the Eurocode 8 Part 3 (EC8-3, CEN, 2005) defines three Limit States (LS) related to the expected state of damage in the structure: (i) LS of Limited Damage (LD) correspondent to the yield point of the idealized Elasto-perfectly plastic force – displacement relationship of the equivalent SDOF system; (ii) LS of Significant Damage (SD) defined for ¾ of the ultimate top displacement capacity; and (iii) LS of Near Collapse (NC) displacement corresponding to at least 20% reduction in the peak strength. Figure 9 plots the capacity curve along with the limit states criteria and the target displacements determined.

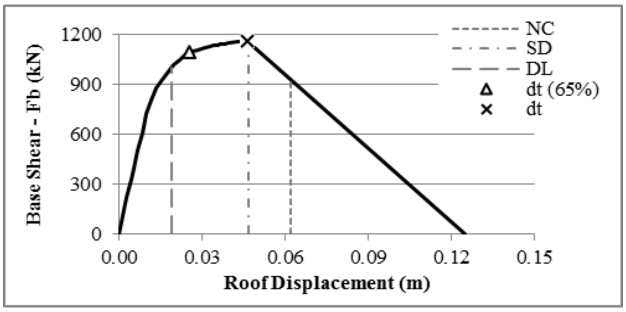


Figure 9. Limit States Criteria.

The seismic fragility of a structure is defined as the probability of reaching a defined limit state for a chosen seismic intensity. The EC8-3 (Annex C, CEN, 2005) states that for the design seismic action (T_R=475years) the structural performance should remain with the LS of Significant Damage (SD). For the reference seismic action, the target displacement of 0.047 m matches the structure point of maximum base shear force (F_b=1160.2 kN - Table 2) and is coincident with this limit state placing the wall structure in a vulnerable position. For the reduced seismic action, proposed in OPCM 3431 (OPCM 3431, 2005) for the assessment of unreinforced masonry buildings, the roof target displacement is of 0.028 m, being the global performance requirement verified. The performance of the wall structure is also assessed at the element scale by comparing the storey displacement demands with the capacity of the masonry piers defined by the hinge behaviour and with interstorey drift limits defined in the EC8-3 (CEN, 2005). Figure 10 shows the structure deformed shape correspondent to both target displacements. Step 15 and 16 (of the capacity curve) correspond to the steps before and after the target displacement defined for the reduced seismic action ($d_{t,65\%}$=0.028m). From Figure 10 a) it is possible to realise that for this seismic demand only the spandrel beams from the 1st and 3rd floor exceed their shear strength. The piers elements that proceed for the nonlinear range do not reach collapse. On the other hand, for the reference seismic action (d_t=0.047m Figure 10 b) the shear hinges from the 3rd floor exceed their ultimate displacement pushing the structure for a storey collapse mechanism (expected to occur for a roof displacement of 0.125m – section 4.1).

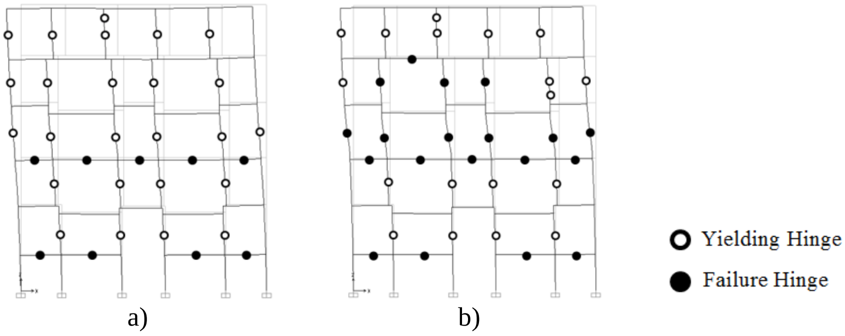


Figure 10. Structure deformed shape: a) Step 15 ($d_{t,65\%}$=0.028m); b) Step 34 (d_t=0.047m).

EC8-3 (Annex C, CEN, 2005) states that if the element capacity is controlled by shear force (depending on the relation between Eqn. C.1 and C.2 from Annex C) the drift limit for primary seismic elements shall be taken equal to 0.4% for the LS of Significant Damage (SD) and Damage Limitation (DL) and equal to 0.53% for the LS of Near Collapse (NC). Figure 11 plots the wall interstorey drift along with the referred drift limits. For the reduced seismic action, the limits defined for the LS of Significant Damage (SD) are verified. However, and as expected, for the reference seismic action the elements from the 3rd floor exceed the compliance criteria and are close to the Near Collapse (NC) drift limit.

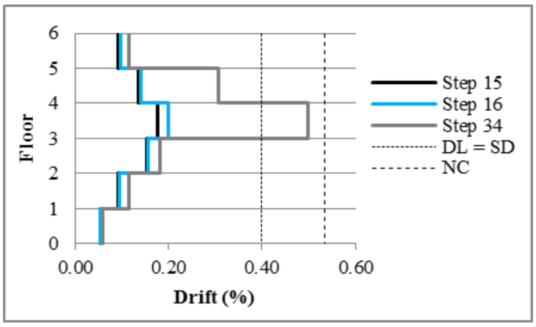


Figure 11. Structure’s interstorey drifts and compliance limits.

5. CONCLUSION

This paper aims to make a contribution for the reduction of the seismic vulnerability of unreinforced masonry structures based on the nonlinear equivalent frame modelling analysis. The method was applied for the seismic assessment of an existing building representative of particular typology of masonry buildings built in Lisbon at the end of the nineteenth and beginning of the twentieth century.
The façade wall of the building was modelled making use of the software SAP2000® (CSI) based on the equivalent frame modelling strategy. The masonry piers and spandrel beams were defined with elastic behaviour, while the nonlinear behaviour was assigned with plastic hinges. The structure capacity curve was determined with an incremental nonlinear static (pushover) analysis and the limit states compliance criteria defined in EC8-3 (CEN, 2005) for existing masonry structures were verified for the seismic performance point of the structure. It was concluded that the wall structure is controlled by the piers shear force capacity. For the seismic action defined in the EC8-1 (CEN, 2004) for Lisbon, the shear hinges from the 3rd floor exceed their ultimate displacement pushing the structure for a storey collapse mechanism. Moreover, according to the results spandrel beams from the 1st and 3rd floor also exceed their shear capacity. Although the predicted brittle shear failure in the beams may

not cause a global collapse of the building, strengthening measures on these elements seem to be necessary.

The analysis procedure herein presented provides a first approach to expected seismic performance of the structure and the identification of the weakest elements and correspondent failure mechanisms. The studies up-to-date developed have shown the reliability of the method for the assessment of existing structures. Although the present study was limited to the in-plane analysis of a façade wall, the procedure may be extrapolated for the assessment of the overall building structure through the directional performance combination of the building bearing masonry walls. Additional retrofitting schemes should be applied to provide a good connection between orthogonal masonry walls and floors in order to prevent the out-of-plane failure mechanism of the building, which probably are the main weaknesses of the masonry buildings when subjected to seismic actions.

AKCNOWLEDGEMENT

The authors would like to acknowledge the financial support of the Portuguese Foundation for Science and Technology (Ministry of Science and Technology of the Republic of Portugal) through the research project PTDC/ECM/100872/2008 named 'Seismic Vulnerability of Old Masonry Buildings'.

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