



## COMPARISON OF DIFFERENT SEISMIC NON-LINEAR ANALYSES OF AN ANCIENT MASONRY BUILDING

N. Mendes<sup>1</sup> and P.B. Lourenço<sup>2</sup>

<sup>1</sup> PhD student, ISISE, Department of Civil Engineering, University of Minho, Azurem, Portugal, nunomendes@civil.uminho.pt

<sup>2</sup> Professor, ISISE, Department of Civil Engineering, University of Minho, Azurem, Portugal, pbl@civil.uminho.pt

### ABSTRACT

This paper presents a study about the possibilities of non-linear static analyses for evaluating the seismic performance of ancient masonry buildings. The adopted case study is a Portuguese typology of the 19<sup>th</sup> century. Non-linear dynamic and static analyses were carried out using a numerical model calibrated with experimental results obtained in 1:3 reduced scale tests made in a 3D shaking table.

In the non-linear dynamic analysis with time integration, the earthquakes are composed by two uncorrelated artificial accelerograms compatible with the elastic response spectrum defined by Eurocode 8. Using this analysis it was verified that the adopted building, with appropriate floor-wall connections, is in the limit of its loading capacity.

In the non-linear static analyses (pushover), lateral load distributions proportional to the mass and 1<sup>st</sup> mode shape in the applicable direction were considered. In an attempt to explore the pushover analyses proportional to the 1<sup>st</sup> mode shape, adaptive pushover analyses were carried out, in which the load distribution was updated as a function of the existing damage. It is concluded that the pushover analysis does not simulate correctly the failure mode of the structure, namely the out-of-plane mechanism, and should be used with caution.

**KEYWORDS:** ancient masonry buildings, non-linear analysis, earthquake, pushover analysis.

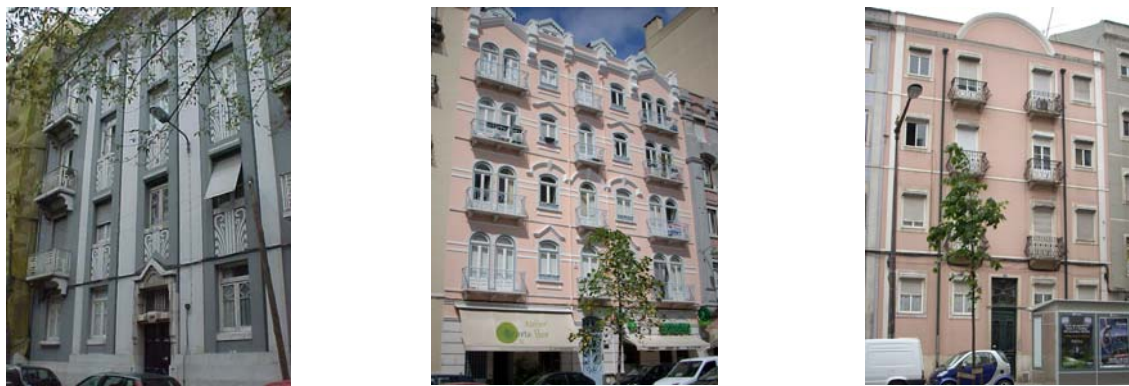
### INTRODUCTION

The study of ancient buildings is receiving increasing attention in the last few decades, due to the importance of conservation of the built heritage and the awareness that life and property must be preserved. With respect to the built heritage, masonry buildings represent a major part of the stock and they were often non-engineered or not designed with reference to any particular code [1].

In general, non-linear dynamic analysis of ancient masonry structures by means of complex models is not possible, because it requires a great amount of computational resources. Nowadays, pushover analysis is often used for evaluating the seismic performance of structures, a procedure that is supported by codes. However, its application to ancient masonry buildings is still a challenge.

The aim of this paper is to study the application of pushover analysis for evaluating the seismic performance of an ancient masonry building. The lateral load distributions were considered proportional to the mass and to the 1<sup>st</sup> modal shape. Two adaptive pushover analyses were also carried out, in which the load distribution was updated as a function of the existing damage. These analyses were compared with non-linear dynamic analysis and experimental results obtained in 1:3 reduced scale tests carried out in the 3D shaking table from the National Laboratory of Civil Engineering, Lisbon (LNEC).

The ancient masonry building corresponds to a Portuguese building typology - “gaioleiro”. This building typology was developed between the mid 19<sup>th</sup> century and early 20<sup>th</sup> century, mainly in the city of Lisbon, and remains still much in use nowadays. The “gaioleiro” buildings (Figure 1) are, usually, four or five storeys high, with masonry walls and timber floors and roof. The external walls are, usually, in rubble masonry with lime mortar [2].



**Figure 1: Examples of “Gaioleiro” buildings, Lisbon, Portugal**

### **SHAKING TABLE TESTS**

LNEC carried out a set of shaking table tests with the purpose of evaluating the seismic performance of “gaioleiro” buildings, before and after strengthening [3]. In the test program, a prototype of an isolated building was defined, with four storeys and an interstorey height of 3.60 m, two opposite facades with a percentage of openings equal to 28.6% of the facade area, two opposite gable walls (with no openings), timber floors and a gable roof.

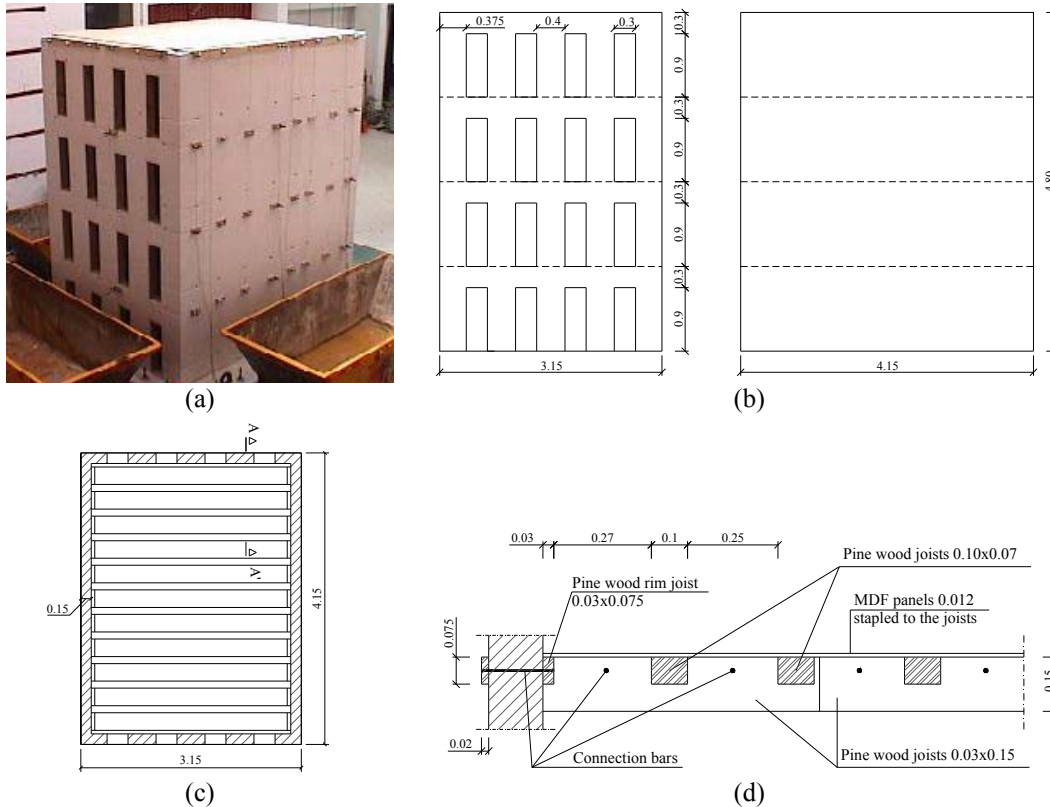
Due to the size and payload of the shaking table, the experimental model was built using a 1:3 reduced scale, taking in account Cauchy’s law of similitude (Table 1). The geometric properties of the experimental model result directly from the application of the scale factor to the prototype, resulting in a model with 3.15 m wide, 4.8 m depth and 0.15 m of wall thickness (Figure 2).

The external walls, originally built in poor quality rubble masonry with lime mortar, were replaced by a self compacting bentonite-lime mix, studied to reproduce the mechanical characteristics of the original masonry walls. In the construction of the timber floors, medium-density fiberboard (MDF) panels connected to a set of timber joists oriented in the direction of the shortest span were used. The panels were cut in rectangles of 0.57 m x 0.105 m and stapled to the joists, keeping a joint of about 1 mm for separating the panels. The purpose was to simulate flexible floors with limited diaphragmatic action (Figure 2d).

**Table 1: Scale factors of the Cauchy similitude [4]**

( $p$  and  $m$  designate prototype and experimental model, respectively)

Parameter	Symbol	Scale factor
Length	L	$L_p/L_m=\lambda=3$
Young's Modulus	E	$E_p/E_m=\lambda=1$
Specific mass	$\rho$	$\rho_p/\rho_m=\lambda=1$
Area	A	$A_p/A_m=\lambda^2=9$
Volume	V	$V_p/V_m=\lambda^3=27$
Mass	m	$m_p/m_m=\lambda^3=27$
Displacement	d	$d_p/d_m=\lambda=3$
Velocity	v	$v_p/v_m=\lambda=1$
Acceleration	a	$a_p/a_m=\lambda^{-1}=1/3$
Weight	W	$W_p/W_m=\lambda^3=27$
Force	F	$F_p/F_m=\lambda^2=9$
Moment	M	$M_p/M_m=\lambda^3=27$
Stress	$\sigma$	$\sigma_p/\sigma_m=\lambda=1$
Strain	$\varepsilon$	$\varepsilon_p/\varepsilon_m=\lambda=1$
Time	t	$t_p/t_m=\lambda=3$
Frequency	f	$f_p/f_m=\lambda^{-1}=1/3$



**Figure 2: Experimental model (dimensions in meters): a) general view; b) geometrical properties; c) plant; d) floors (section AA')**

## DEFINITION AND CALIBRATION OF THE NUMERICAL MODEL

The numerical model of the building was prepared using the Finite Element (FE) software DIANA [5], by using shell elements for the simulation of the walls and three dimensional beam elements for the timber joists, all based on the theory of Mindlin-Reissner. In the modeling of the floors, shell elements were also used with the purpose of simulating the in plane deformability. In the connection between the floors and the masonry walls only the translation degrees of freedom were tied. In the supports, only the translation degrees of freedom in the base were restrained. The full model involves 5816 elements (1080 beam elements and 4736 shell elements) with 15176 nodes, resulting in 75880 degrees of freedom (DOF).

Calibration of the numerical model was accomplished with the methodology proposed by Douglas-Reid [6]. Here, the first two natural frequencies obtained in the modal identification (1<sup>st</sup> translational mode and 1<sup>st</sup> torsional mode) were used. After calibration the optimal values of the Young's modulus were obtained (Table 2). It is noted that the low value found for the Young's modulus of the MDF panels is due to the floors arrangement (open joints).

**Table 2: Linear elastic properties**

	Young's modulus [N/mm <sup>2</sup> ]	Poisson's ratio	Specific mass [Kg/m <sup>3</sup> ]
Walls	779	0.2	1910
MDF panels	240	0.3	760
Wood joist	12000	0.3	580

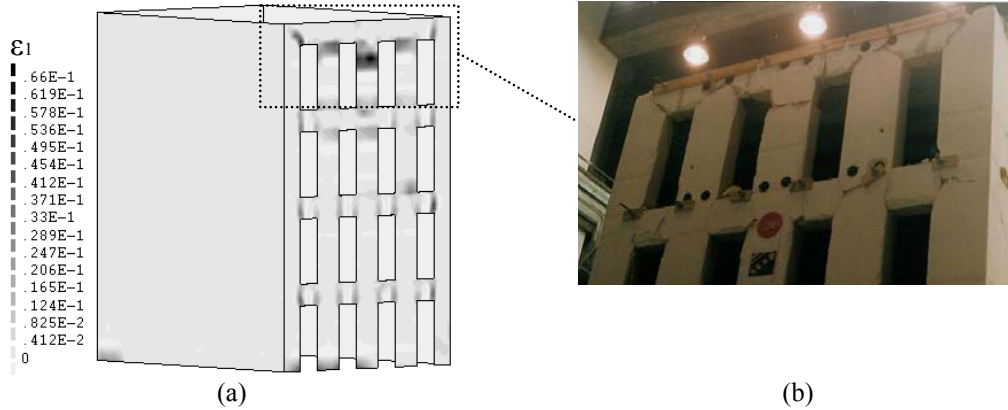
Physical non-linear behaviour of the masonry walls was simulated using the Total Strain Crack Model detailed in [5]. This includes a parabolic stress-strain relation for compression, where the compressive strength,  $f_c$ , is equal to 0.8 N/mm<sup>2</sup> and the respectively fracture energy,  $G_c$ , is equal to 1.25 N/mm. In tension, an exponential tension-softening diagram was adopted, where the tensile strength,  $f_t$ , is equal to 0.125 N/mm<sup>2</sup> and the fracture energy,  $G_t$ , is equal to 0.125 N/mm. The crack bandwidth,  $h$ , was determined as a function of the finite element area,  $A$  (Equation 1). In terms of shear behaviour, a constant shear retention factor equal to 0.01 was adopted.

$$h = \sqrt{A} \quad (1)$$

Damping,  $\underline{C}$ , was simulated according to Rayleigh viscous damping [7], which is a linear combination of the mass,  $\underline{M}$ , and of stiffness,  $\underline{K}$ , matrices (Equation 2). Constants  $\alpha$  (2.18) and  $\beta$  (0.00044) were determined from the results obtained in the dynamic identification tests. In the damping identification, a curve fitting of the Frequency Response Function (FRF) for a Single Degree of Freedom was used.

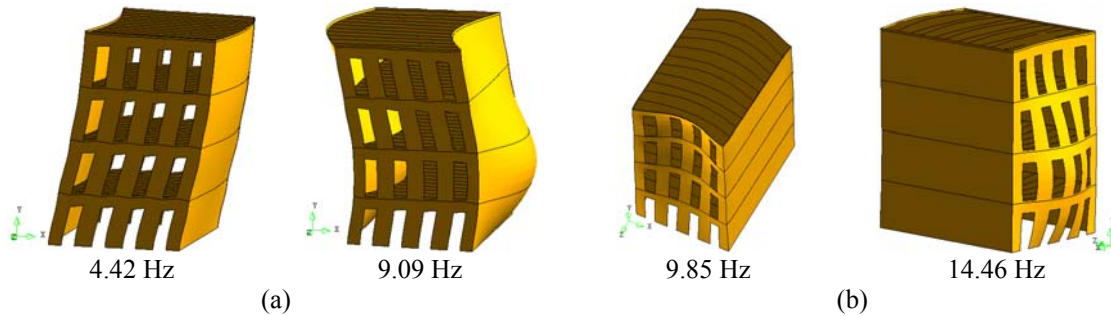
$$\underline{C} = \alpha \underline{M} + \beta \underline{K} \quad (2)$$

Figure 3 shows that, globally, the numerical model simulates correctly the damage present in the experimental model after testing.



**Figure 3: Damage after testing in the model: a) numerical; b) experimental model.**  
 ( $\epsilon_1$  is the principal tensile strain, which is an indicator of crack width)

The experimental model only considered the self-weight of the walls and of the floors. In the calibrated numerical model, all code loads are now considered, including the self weight of the partition walls, cladding and roof, and the quasi-permanent part of the live load. Figure 4 presents four mode shapes in the transversal and longitudinal directions of the calibrated model with additional mass. Using the calibrated model with the code masses, a safety analysis is now made using non-linear time history analysis and pushover analysis.



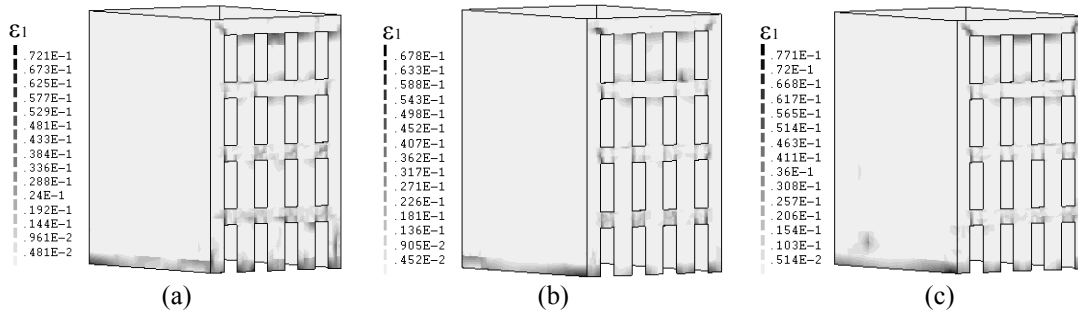
**Figure 4: Mode shapes in the: a) transversal direction; b) longitudinal direction**

### NON-LINEAR DYNAMIC ANALYSIS

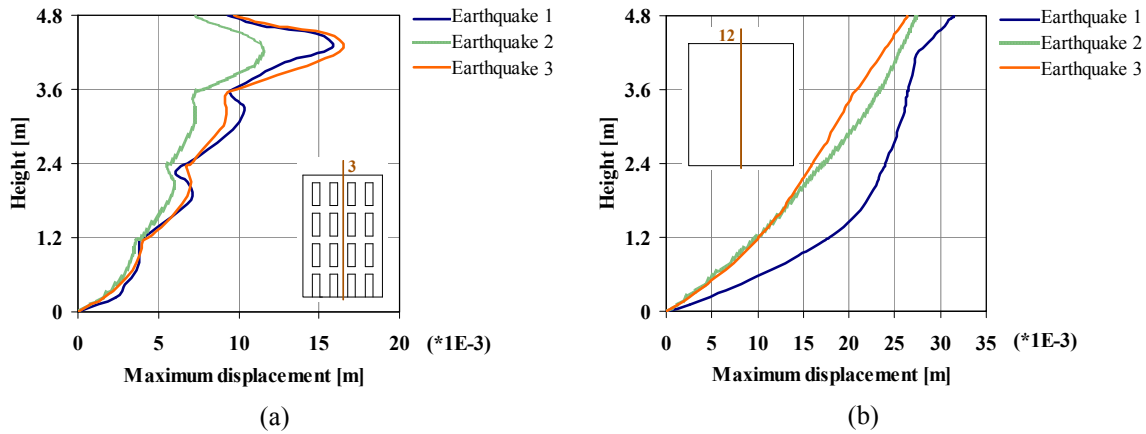
Nonlinear dynamic analysis with time integration was made using three earthquakes, composed, each one, of two uncorrelated artificial accelerograms. The artificial accelerograms are compatible with the elastic response spectrum (type 1) defined by Eurocode 8 [8], for the zone of Lisbon, with a damping ratio  $\zeta$  equal to 5% and a type A soil (rock). Due to the fact that non-linear dynamic analyses are very time consuming and the response spectrum of type 1 (interplate earthquake) is usually more stringent for Lisbon and the type of structures being considered, only one type of earthquake was considered. On 1:3 reduced scale, the accelerograms have a total duration of 6 s, from which 3.33 s correspond to the intense phase, and a *PGA*, on average, equal to 4.51 m/s<sup>2</sup>.

Figure 5 presents the maximum values of the tensile principal strains  $\epsilon_1$  for the three earthquake records. The results indicate that the facades at the 4<sup>th</sup> floor and the base of the structure are the

zones of larger damage concentration, being the high level of damage in the 4<sup>th</sup> floor's piers highlighted. Figure 6 presents the maximum displacement in the middle of the walls, in which the out-of-plane mechanism of the piers is observed.



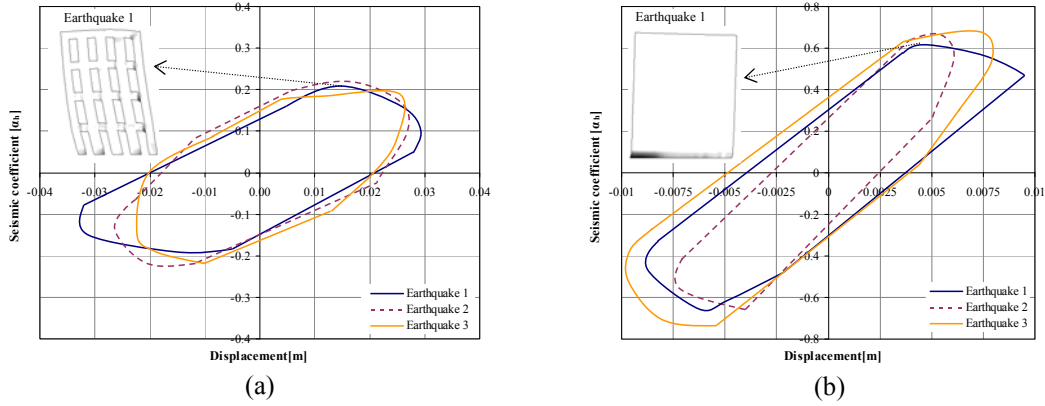
**Figure 5: Tensile principal stains (outside surface): a) earthquake 1; b) earthquake 2; c) earthquake 3**



**Figure 6: Maximum out-of-plane displacement in the middle of the: a) facades; b) gable walls**

The relation between the “seismic coefficient”  $\alpha_h$  defined by Equation 3 and the horizontal displacement at the top of the structure was plotted. The envelopes of these relations, for the different earthquakes and directions, are presented in Figure 7. It is observed that the maximum values of  $\alpha_h$  are about 0.2 and 0.65 in the transversal and longitudinal directions, respectively (approximately a relation 100% longitudinal “+“ 31% transversal).

$$\alpha_h = \frac{\sum \text{Horizontal forces}}{\text{Self weight of the structure}} \quad (3)$$



**Figure 7: Envelope of the relation between the horizontal displacement (4<sup>th</sup> floor) and the seismic coefficient in the: a) transversal direction; b) longitudinal direction**

### PUSHOVER ANALYSES

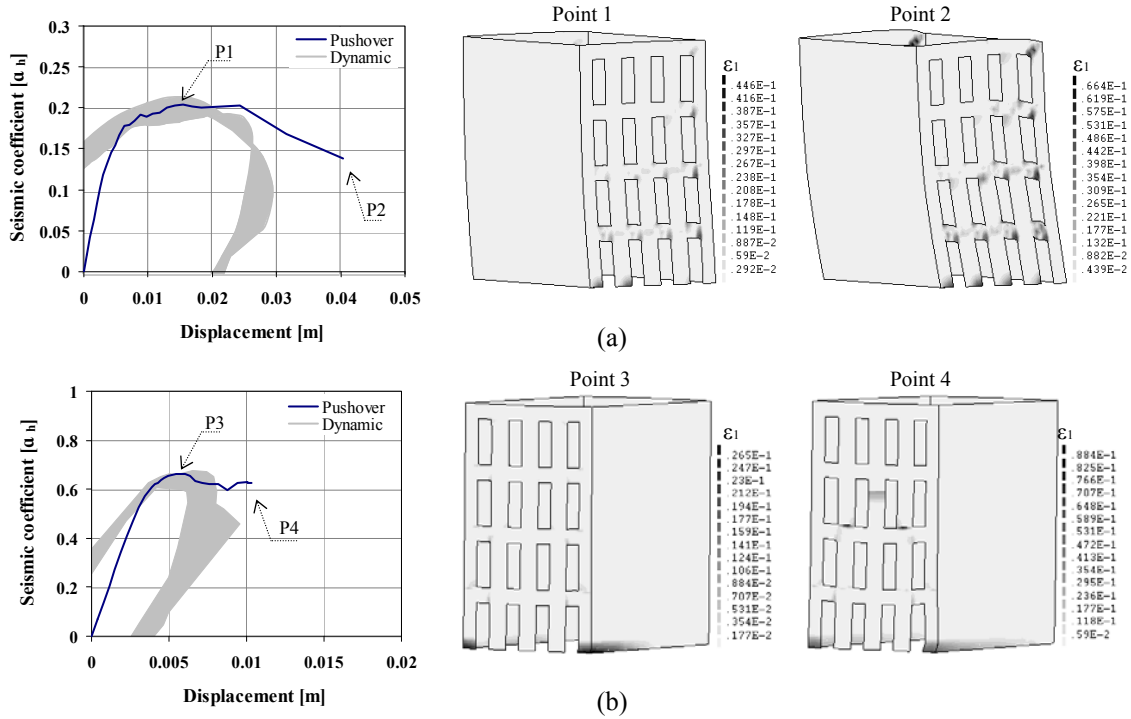
As an alternative to the non-linear dynamic analysis, a pushover analysis was carried out. Here, physical and geometrical nonlinear behaviour was considered, even if it is expected that the geometrical non-linear effects have minor influence in the maximum load and only limited effect in the displacement associated with the maximum load. Two lateral load distributions were used: (a) uniform pattern, based on lateral forces proportional to mass regardless of elevation – uniform response acceleration; (b) modal pattern, proportional to forces consistent with the 1<sup>st</sup> mode shape in the applied direction. In an attempt to explore the pushover analyses proportional to the 1<sup>st</sup> mode shape, two additional adaptive pushover analyses were also carried out.

In the capacity curves of the pushover analyses proportional to the mass, the maximum seismic coefficients are higher than the dynamic analysis (about 24%) and the damage concentration only appears at the lower zone of the structure [9]. It is noted that in the dynamic analysis the damage concentrates at the 4<sup>th</sup> floor (facades) and at the base (Figure 5). Thus, pushover analysis does not simulate correctly the performance of structure under seismic load.

The capacity curves of the pushover analysis proportional to the 1<sup>st</sup> mode (in the applied direction) show that the maximum seismic coefficients approach the dynamic analysis. The crack patterns only provide in plane damage, which is not in agreement with the out-of-plane mechanism found in the time integration analysis and shaking table test (Figure 8). Hence, the damage at the upper storeys must have a significant contribution from the higher modes, including the local modes of the piers.

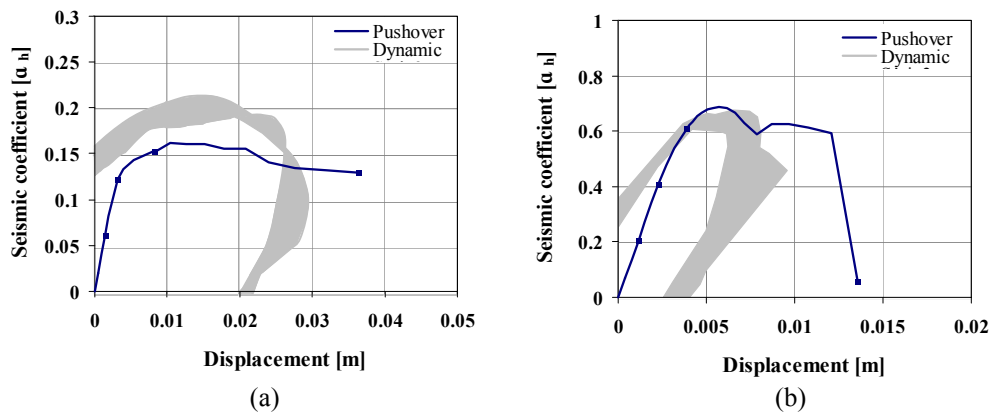
The adaptive pushover analyses were done in four phases and after each phase the new modal shape was calculated by using the tangential stiffness matrix  $k_t$  (Equation 4), allowing the update of the load distribution as function of the damage. Here, the aim is to understand how the update of the external load vector can influence the structural response.

$$\left( \underline{k}_t - w_i^2 \underline{M} \right) \underline{\phi}_i = 0 \quad (4)$$



**Figure 8: Capacity curves and tensile principal strains of the pushover analysis proportional to the 1<sup>st</sup> mode in the: a) transversal direction; b) longitudinal direction**

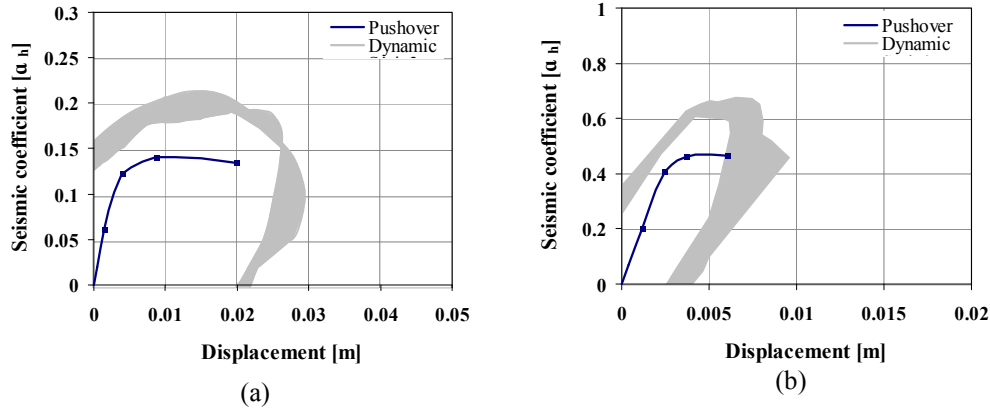
In the first adaptive pushover analysis the lateral loads, proportional to the 1<sup>st</sup> mode shape in the direction considered, were applied independently in the transversal and longitudinal direction (Figure 9). The maximum seismic coefficient decreases in the transversal direction (24% of the maximum seismic coefficient of the pushover analysis proportional to the 1<sup>st</sup> mode in the corresponding direction). In longitudinal direction the analysis provides the same behaviour of the pushover analysis proportional to the 1<sup>st</sup> mode in the corresponding direction. So, in this direction the update of the load distribution does not have influence in the response. Furthermore, the cracks pattern and failure mechanism do not improve.



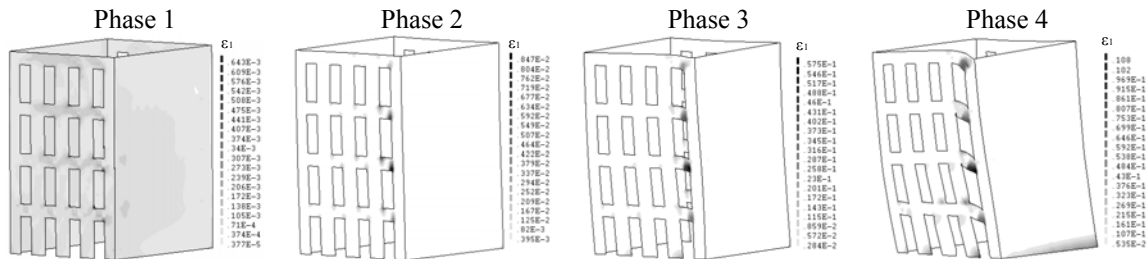
**Figure 9: Capacity curves of the adaptive pushover analysis, in which the lateral loads were applied independently, in the: a) transversal direction; b) longitudinal direction**



In the second adaptive pushover analysis, the lateral loads, proportional to the 1<sup>st</sup> mode shape in the applied direction, were applied simultaneously in the transversal and longitudinal direction (Figure 10). Here, the aim was to obtain the in-plane and the out-of-plane damage together in the same analysis. However, the combined effect of the loads applied simultaneously in the two directions cause damage to concentrate on lintels (Figure 11), not simulating correctly the performance of structure under seismic load.



**Figure 10: Capacity curves of the adaptive pushover analysis, in which the lateral loads were applied simultaneously, in the: a) transversal direction; b) longitudinal direction**



**Figure 11: Tensile principal strains of the adaptive pushover analysis, in which the lateral loads were applied simultaneously**

## CONCLUSIONS

Through the non-linear dynamic analysis and the shaking table tests, it was observed that the buildings of “gaioleiro” type with appropriate floor-wall connection, under seismic action (Lisbon zone and soil of the type A) are in the limit of their loading capacity. Therefore, it seems that a strong floor-wall connection is not enough to guarantee the good performance of the building under seismic load, due to the floors flexibility. The “gaioleiro” buildings present the typical collapse of masonry buildings including: (a) cracking around the corners of the openings; (b) out-of-plane collapse (4<sup>th</sup> floor’s piers).

With respect to pushover analysis (proportional to the mass or to the 1<sup>st</sup> mode), it was concluded that it does not simulate correctly the failure mode of the structure, meaning that vibration modes with higher frequencies have a significant contribution to the behavior, including the local shapes modes of the piers. The pushover analysis proportional to the 1<sup>st</sup> mode shape performed better in

terms of load-displacement diagram than the pushover analysis proportional to the mass, simulating correctly the in-plane behaviour.

In an attempt to explore the pushover analyses proportional to the 1<sup>st</sup> mode shape, two adaptive pushover analyses were carried out, in which the load distribution was updated as a function of the existing damage. This analysis did not provide any improvement in terms of load-displacement diagrams or failure mechanisms.

Even if the structure analyzed presents regularity in plan and in elevation, the flexible floors are most likely the reason for the deficient performance of the pushover analysis. So, it is concluded that pushover analysis should be used with caution for evaluating the seismic performance of ancient masonry buildings.

### **ACKNOWLEDGEMENTS**

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