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STONE MASONRY BUILDINGS: SHAKING TABLE TESTING AND ADVANCED METHODS OF ANALYSIS

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SUMMARY

The paper presents the main results of experimental tests concerning the reduction of the seismic vulnerability of stone masonry buildings with flexible floors. Two mock-ups (original condition and repaired) were tested in a 3D shaking table. The results show that the adopted measures are efficient. A set of non-linear static analyses (pushover) were considered, together with time integration analysis. The non-linear dynamic analyses reproduced the seismic behavior observed in the seismic tests. However, the pushover analyses did not simulate correctly all the failure mode of the structure and should be used with caution.

INTRODUCTION

Natural disasters are an effect of natural hazards (e. g. tornado, volcanic eruption, landslide, tsunami or earthquakes) that has caused millions of deaths (1975-2007) and serious socio-economic impacts, affecting the development of many countries.

According Hough and Bilham (2006), earthquakes caused 6 million fatalities in 500 years (1500-2000). Recently, the magnitude 7 earthquake in Haiti Region (2010) alone triggered disastrous destruction and over 200,000 deaths. But earthquakes hardly kill people, being the collapse of the buildings the main reason of the deaths. This means that efforts should be conducted to reduce the seismic vulnerability of buildings.

Ancient masonry buildings are one of the most vulnerable elements and were built for many centuries according to the experience of the builder, taking into account simple rules of construction and without reference to any particular seismic code. Still, in seismic areas, unreinforced masonry structures represent an important part of the building stock. Thus, in the recent decades, the study of the vulnerability of ancient buildings is receiving much attention due to the increasing interest in the conservation of the built heritage and the awareness that life and property must be preserved. The seismic assessment of ancient masonry buildings is particularly difficult and depends of several factors. Besides the quality of masonry materials and the distribution of structural walls in plan, also the connection between the walls and floors significantly influences the seismic resistance (Tomaževič et al. 1996).

In view of these aspects, an experimental program was carried out to assess the seismic vulnerability of a building typology that is believed to present the highest seismic vulnerability of the housing stock of Portugal ("gaioleiro" buildings). The program also aims at evaluating the efficiency of repairing solutions. Currently, a numerical study is still being carried out. Thereby, this paper presents the first results.

The ''gaioleiro'' buildings typology was used between the mid 19th century and beginning of the 20th century, mainly in the city of Lisbon, and many buildings remain of this type. This typology characterizes a transition period from the anti-seismic practices used in the ''pombalino'' buildings originated after the earthquake of 1755, see e.g. (Ramos and Lourenço 2004), and the modern reinforced concrete frame buildings. These buildings are, usually, four to six stories high, with masonry walls (thicknesses ranging from 0.30 m to 0.60 m) and timber floors and roof. The external walls are, usually, in rubble masonry with lime mortar (Pinho 2000). The partitions are mainly stud walls sheathed with thin wood boards and plaster, although there can be also some brick masonry walls. The floor is usually made of timber boards nailed to the joists and, in some cases, there are also rim joists connecting the floors to the walls (Candeias 2009).

"Gaioleiro" buildings are usually semi-detached and belong to a block of buildings. Although it is not an objective of this article, pounding can be taken in account when the adjacent buildings present different heights or the separation distance is not large enough to accommodate the displacements (Gulkan et al. 2002). It is noted the ''block'' effect is usually beneficial and provides higher strength of the building, as shown in Ramos and Lourenço (2004).

PROTOTYPE AND MOCK-UPS

In order to study the seismic performance through experimental tests, a prototype of an isolated building representative of the "gaioleiro" buildings was defined. This is constituted by four stories with an interstory height of 3.60 m and 9.45 m x 12.45 m in plan, two opposite façades with a percentage of openings equal to 28.6% of the façade area, two opposite gable walls (with no openings), timber floors, and a gable roof.

Mock-ups replicate the geometrical, physical and dynamical characteristics of buildings typologies (e.g. reinforced concrete framed structures, unreinforced masonry structures with flexible floors) or individual structures (e. g. monuments, bridges). However, mock-ups are usually simplified due to difficulties related to its reproduction in laboratory, namely the geometrical properties of the prototype or individual structures and the size of the facilities. Often reduced scale mock-ups are used even it is difficult to fulfill the similitude laws using very small scales, as e. g. the preparation of masonry units and reinforcement elements.

In the case study, due to size and payload capacity of the shaking table the mock-up had to be geometrically reduced. Thus, a 1:3 reduced scale mock-up taking into account only Cauchy's law of similitude was adopted. In this law of similitude the Cauchy value (ratio between the inertia forces and the elastic restoring forces) is the same in the prototype and in the mock-up. For the realistic modeling of non-linear dynamic behavior of the structures Froude's law of similitude (ratio between inertia forces and gravity forces) must be also respected (Carvalho 1998). However, this procedure involves a mock-up with total mass 3 times higher than that according to the Cauchy's law, and which exceeds the payload capacity of the shaking table. Table 1 presents the factors of the Cauchy and both laws of similitude.

Table 1. Scale factors of the Cauchy similitude (Carvalho 1998)

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

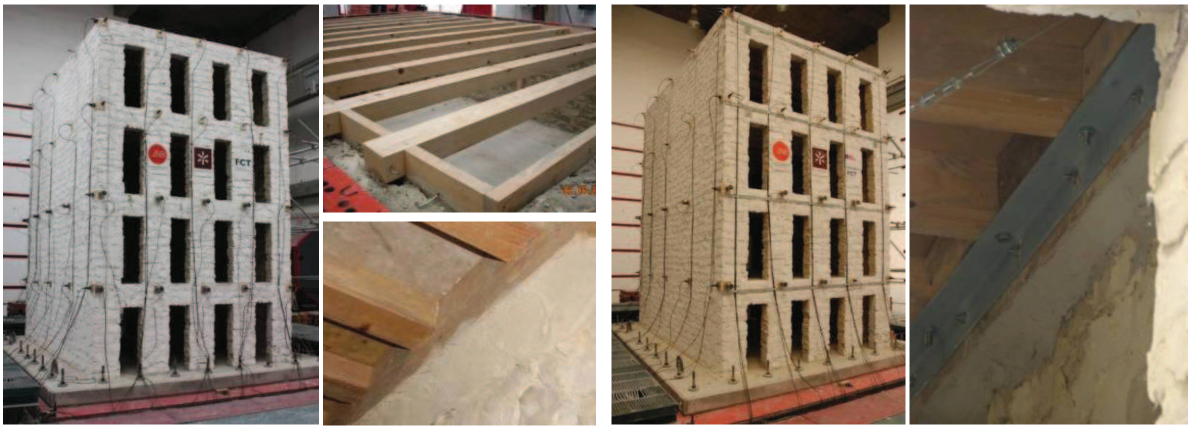
(*p* and *m* designate prototype and mock-up, respectively)

The geometric properties of the non-strengthened mock-up (NSM) are obtained directly from the application of the scale factor to the prototype, resulting in a model 3.15 m wide and 4.8 m deep, with 0.17 m of wall thickness (see Figure 1a). The interstory height is equal to 1.2 m. The mock-up only has the top ceiling, due to difficulties in reproducing the gable roof at reduced scale. The external walls have a single leaf of stone masonry (limestone and lime mortar) and were built by specialized workmanship.

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 and stapled to the joists, keeping a joint of about 1 mm. The purpose was to simulate flexible floors with very limited diaphragmatic action (see Figure 1a).

After the tests, the piers and the lintels were repaired, aiming at re-establishing the initial conditions of the mock-up. Afterwards, the mock-up was strengthened and tested again.

In the strengthened mock-up (SM) steel angle bars (internal surface) and plates (external surface) at the floor levels were used (see Figure 1b). These strengthening elements are connected among themselves by bolts, with exception of the gable walls, in which the steel angle bars are connected to the masonry. It is noted that, usually, in real application it is not possible to apply strengthening elements to the external surface of the gable walls, due to the presence of adjacent buildings. Additionally, timber elements to constrain the rotation of the timber joists were used. In the two top floors, crossed steel ties were also installed. Each floor has two pairs of steel cables connecting the middle of the façades to the corners of the opposite façades, leading the inertial forces in the out-of-plane direction of the façades to the plane of the gable walls. The main goals of the strengthening techniques adopted are to improve the connection between the floors and the masonry walls, mainly to the gable walls, and to prevent the global out-of-plane collapse of the façades.



(a) (b)

Figure 1. Non-strengthened (a) and strengthened (b) mock-up

TEST PLANNING

The assessment of the seismic performance of the "gaioleiros" buildings was based on previous experience from the National Laboratory for Civil Engineering (LNEC). The methodology includes seismic tests on shaking table with increasing input excitations and characterization tests of the dynamic properties of the mock-ups before the first seismic test and after each of the seismic tests (Candeias 2009). The dynamic properties give inherent information of the mock-up and its evolution is related to the damage induced by a given seismic input.

The seismic tests were performed at the LNEC 3D shaking table by imposing accelerograms compatible with the design response spectrum defined by the Eurocode 8 (EN 1998-1 2004) and Portuguese National Annex for Lisbon, with a damping ratio equal to 5% and a type A soil (rock). The accelerograms were imposed with increasing amplitude in two uncorrelated orthogonal directions that should present approximately the same PGA.

The dynamic properties of the mock-ups were identified through forced vibration tests at the shaking table (Mendes and Lourenço 2010) and its evolution is identified through experimental transfer functions (Frequency Response Function) obtained along the tests.

The shaking table tests of the non-strengthened mock-up involved four seismic tests with amplitudes of the seismic action equal to 25%, 50%, 75% and 100% of the code amplitude and five dynamic identification tests. Additionally, in the strengthened mock-up two extra seismic tests, with amplitudes of the seismic action equal to 125% and 150% of the code amplitude, were done. Due to serious damage of the mock-up, it was not possible to carry out the dynamic identification after the final seismic test.

The reduction of the natural frequencies is related to the stiffness variation and, consequently, to the evolution of the damage. Equation (1) presents a simplified damage indicator $d_{k,i}$ based on the variation of the natural frequencies $f_{k,i}$ ($f_{k,0}$ is the natural frequency of the mode shape k before the application of the first seismic test). This damage indicator assumes that the global mass of the mode shape k does not change meaningfully in the different tests, and it presents different values after each seismic test.

$$d_{k,i} = 1 - \left(\frac{f_{k,i}}{f_{k,0}} \right)^2 \tag{1}$$

EXPERIMENTAL RESULTS

In the preliminary study of the seismic vulnerability of the mock-ups only the 1st mode shape (translation in the transversal direction) and the crack pattern were considered. It is noted that results are presented to1:3 reduced scale, according to Cauchy's law of similitude (Table 1).

Figure 2 presents the vulnerability curves, in which the damage indicator d (equation 1) is related to the amplitude of the seismic action. In the last test of the non-strengthened mock-up the damage indicator is equal to 0.80 and remains equal to value of the previous test. Probably, after the third seismic test, the 1st transversal mode is, mainly, related with the stiffness of the gable walls connected by floors. The last crack pattern of the non-strengthened mock-up (1.0×code) shows that only the lintels and the piers of the façades present serious damage (see Figure 3). The concentration of damage at the piers of the top floor is highlighted, where the horizontal cracks are related to its out-of-plane bending. The gable walls did not present any damage.

After the last test, the non-strengthened mock-up was repaired, strengthened and tested again. The vulnerability curves of the strengthened mock-up (see Figure 2) show that the strengthening was efficient and reduced the seismic vulnerability of the mock-up. In the 4th seismic test (1.0×code) the strengthened mock-up presented a reduction of the damage indicator (0.52) of 35%, with respect to the original building.

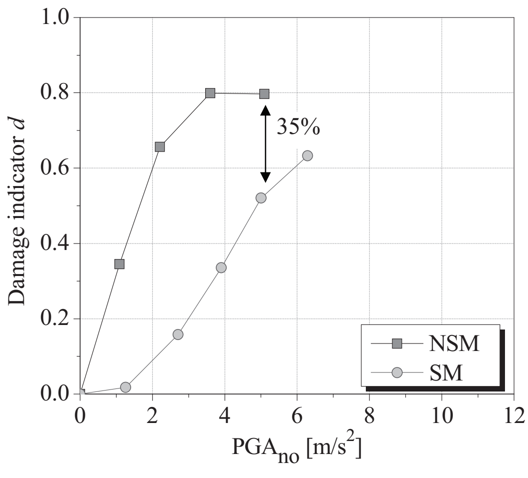


Figure 2. Seismic vulnerability curves.

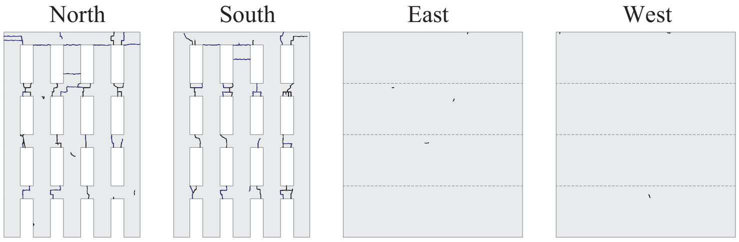


Figure 3. Crack pattern of the non-strengthened mock-up after final testing (1.0×code)

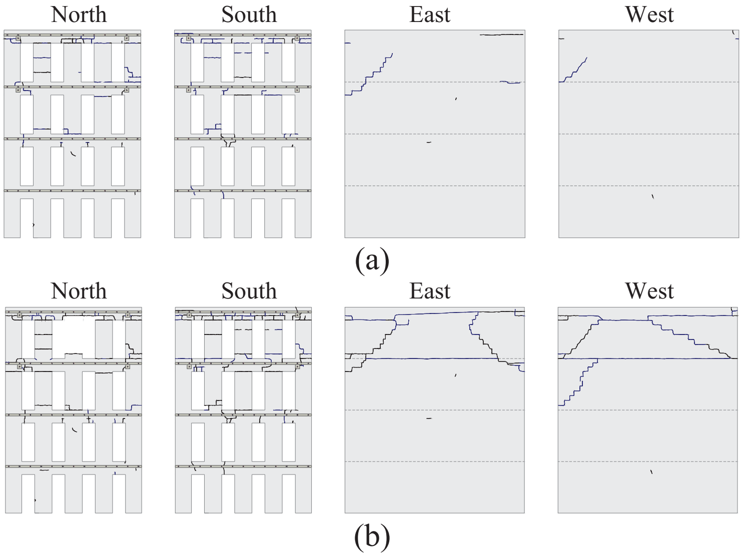


Figure 4. Crack pattern of the strengthened mock-up after: (a) 1.0×code; (b) 1.5×code

The crack patterns also presented different characteristics. Contrarily to the observation in the non-strengthened mock-up (see Figure 3), in which all lintels presented damage, the crack pattern of the strengthened mock-up (see Figure 4a) shows that the cracking of the lintels concentrates at the top floors. Furthermore, the gable walls (4^{th} floor) present diagonal cracks, indicating that part of the out-of-plane inertial forces of the façades were transferred to the gable walls. In the last seismic test of the strengthened mock-up (see Figure 4b), in-plane rocking and out-of-plane bending of the piers of the top floor were observed. The crack pattern shows that damage concentrates at the top floor (façades and gable walls) and the lintels of the 1^{st} and 2^{nd} floors of the façades do not present serious cracking. Furthermore, the collapse of the piers at the top floor of the North façade is highlighted.

PREPARATION AND CALIBRATION OF THE NUMERICAL MODEL

The numerical model of the non-strengthened mock-up was prepared using the Finite Element software DIANA (TNO 2010), 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 supports, only the translation degrees of freedom at the base were restrained.

The first stage of calibration of the numerical model was based on the comparison between experimental and numerical frequencies and MACs (Modal Assurance Criteria) of the first six modes shapes. In this stage five numerical models, taking into account different calibration variables, were used. The sensitivity analysis leads to the conclusion that the Young´s modulus of the timber joists has no significant influence on the frequencies variation and can be considered constant (12 GPa). The models present different alternatives to simulate the connections between floors and masonry walls, and between orthogonal walls. The process presented difficulties in the calibration of the higher mode shapes and only the first four experimental modes were calibrated.

After calibration in the 2nd stage, the 5th numerical model is the one that best fits the experimental data, in which the average of the errors of the frequencies is equal to 2.6% and the average of the MACs is equal to 0.89. In this model four variables were considered, namely the Young´s modulus of the façades, gable wall, MDF panels and corners of orthogonal walls. The Young's modulus of the gables walls (3.17 GPa) approaches the value obtained from uniaxial compression tests on wall-specimens (3.37 GPa). The low value of the Young´s Modulus of the façades (0.58 GPa) can be related to the highest percentage of mortar, with respect to the gable walls and wall-specimens, and to the connection between façades and floors. The Young´s modulus of the corner is equal to 1.59 GPa.

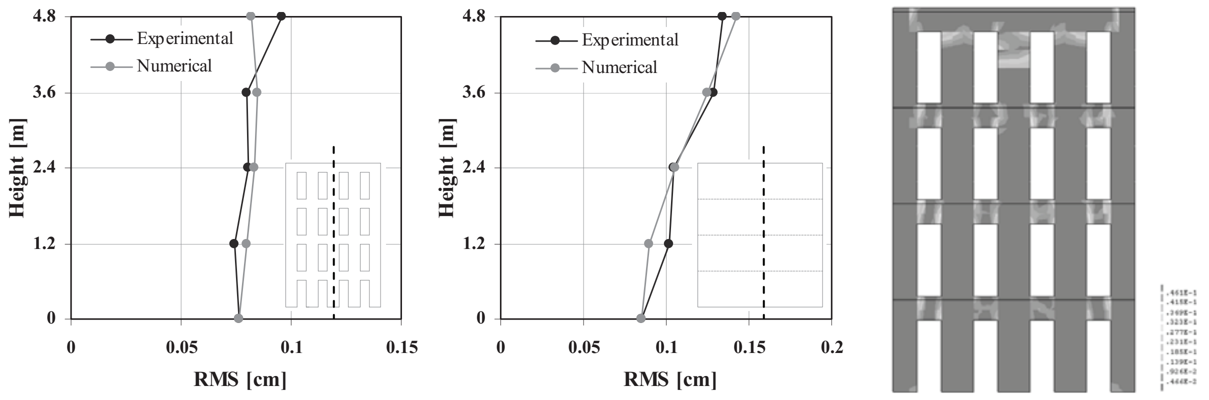
NON-LINEAR DYNAMIC ANALYSES WITH TIME INTEGRATION

After calibration of the frequencies and modes shapes, non-linear dynamic analyses with time integration were performed, aiming at validating the dynamic response of the non-strengthened numerical model (1:3 reduced scale). The physical nonlinear behavior of the masonry walls was simulated using the Total Strain Crack Model detailed in DIANA (TNO 2010). This includes a parabolic stress-strain relation for compression, where the compressive strength, f_c, is equal to 6 N/mm^2 and the respectively fracture energy, G_c, is equal to 9.6 N/mm. In tension, an exponential tension-softening diagram was adopted, where the tensile strength, f_t, is equal to 0.1 N/mm^2 and the fracture energy, G_f, is equal to 0.12 N/mm. The crack bandwidth was determined as a function of the finite element area. In terms of shear behavior, a constant shear retention factor equal to 0.1 was adopted. Regarding the behavior of the MDF panels and timber beams, it was assumed to be linear elastic. The damping C was simulated according Rayleigh viscous damping ($C= \alpha M + \beta K$), in which a 5% damping ratio for the 1st and 3rd transversal mode shapes was assumed.

In the first non-linear dynamic analysis, with seismic action equal to 0.25 of the code amplitude, the peak values and the Root Mean Squares (RMS) of the acceleration, velocities and displacements were used. The response of the numerical model approaches that of experimental one. Figures 5a and 5b show the comparison of RMS of displacements (equation 2) between experimental and numerical results in the middle of the North façade and East gable wall, respectively. As observed in the experimental test, the numerical model did not present serious damage and its behavior is, mainly, linear dynamic. The maximum displacements at the top of the North façade and East gable wall are equal to 3.8 mm and 7.1 mm (numerical model), respectively.

$$RMS_{disp} = \sqrt{\frac{1}{t_d}\int_0^{t_d} d(t)^2\,dt} \tag{2}$$

in which d(t) is the displacement time series and t_d is the duration of the series.



(a) (b) (c)

Figure 5. Numerical results: RMS of displacement in the (a) North façade and (b) East gable wall for earthquake with amplitude equal to 0.25×code; (c) tensile principal strains of the North façade for earthquake with amplitude equal to 1.0×code

A nonlinear dynamic analysis with seismic action equal to 1.0 of the code amplitude was carried out. A qualitative comparison between numerical and experimental was also done. However, the main objective was to obtain a crack pattern similar to the one observed in the experimental test. It is noted that in the experimental testing the time series were imposed with increasing amplitude in the same mock-up. This means that the numerical model and the mock-up did not present, with exception of the first seismic action, the same initial conditions. In the Figure 5c the tensile principal strains of the North façade are presented, in which the damage concentrates at the lintels and at the piers of the 4th floor, and it is in agreement with the crack pattern observed in the last experimental test (see Figure 3).

MODERN NUMERICAL THECNIQUES: PUSHOVER ANALYSIS

The non-linear dynamic analysis is a complex and time consuming tool hardly available for practitioners. An alternative option seems to be non-linear static methods, as recommended in most codes for earthquake safety assessment. However, its application to masonry structures with flexible floors is still a challenge.

Candeias (2009) carried out a set of shaking table tests with the purpose of evaluating the seismic performance of the ‘‘gaioleiro’’ buildings, before and after strengthening. The mock-up is similar to that presented previously. However, the stone masonry walls were replaced by a self compacting bentonite-lime concrete, studied to reproduce the mechanical characteristics of the original masonry walls. The seismic tests were performed by imposing accelerograms with increasing amplitude in two uncorrelated orthogonal directions and compatible with the Portuguese code. Mendes and Lourenço (2010) performed an extensive numerical modeling based on these experimental tests. Besides non-linear dynamic analyses, several types of pushover analyses were carried out.

The damage obtained through the non-linear dynamic analysis is in agreement with crack pattern of the last experimental test (see Figures 6a and 6b). Furthermore, three non-linear dynamic analyses with seismic action compatible with the new European codes (Eurocode 8) were carried out. Figure 7 presents the envelopes of the non-linear dynamic analyses in terms of seismic coefficient (ratio between the sum of the horizontal loads and self-weight) and displacement at the top of the structure (capacity curves).

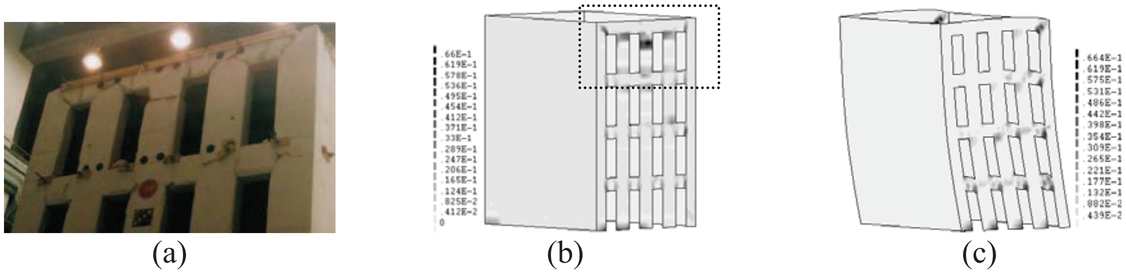


Figure 6. Damage: (a) experimental; (b) non-linear dynamic analysis; (c) pushover analysis proportional to the 1st mode shape in the transversal direction

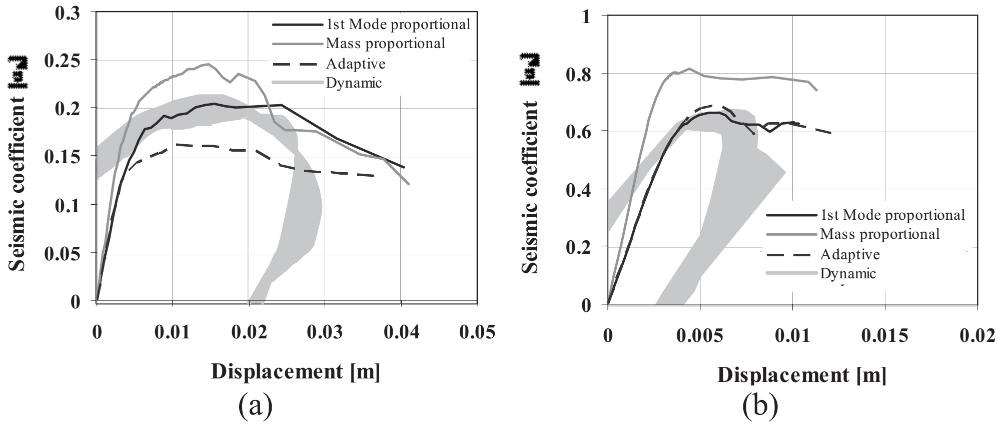


Figure 7. Capacity curves of the pushover analyses in the (a) transversal and (b) longitudinal direction (the silver pattern represents the envelope of the three non-linear dynamic analyses)

In the pushover analyses, the capacity curves were considered by increasing a set of lateral loads applied to the structure in two independent directions. Two vertical distributions of lateral loads were used: (a) proportional to mass regardless of elevation; (b) proportional to forces consistent with the 1^{st} mode shape in the applied direction. The capacity curves of the pushover analyses proportional to the 1^{st} mode shape show that the maximum seismic coefficients approach the dynamic analysis (see Figure 7) and, as expected, the crack patterns only provide in plane damage (see Figure 6c). This type of analyses did not reproduce the out-of-plane mechanism of the piers in the 4^{th} floor (see Figures 6a and 6b). In an attempt to explore the pushover analyses proportional to the 1^{st} mode shape, an additional adaptive pushover analysis was also carried out, in which the lateral load is updated as a function of the existing damage. However, this pushover analysis did not provide any improvement in terms of load-displacement diagrams or failure mechanisms.

CONCLUSIONS

This paper presents an experimental method to assess the seismic vulnerability of masonry buildings with flexible floors. Furthermore, a strengthening solution was also proposed. In the case study, the "gaioleiro" building typology (Portugal) was adopted. The study involved tests in the LNEC 3D shaking table by imposing artificial accelerograms in two horizontal uncorrelated orthogonal directions, inducing in-plane and out-of-plane response of two tested mock-ups. The results showed that the façades of the non-strengthened mock-up present serious damage. The strengthened solution improved the seismic performance of the mock-up and a reduction of 35% of the damage indicator was obtained.

The non-linear dynamic analysis with time integration reproduced the seismic behavior observed in the seismic tests. However, it is not an available option for practitioners. Thus, the application of modern techniques of structural analysis to the masonry structures with flexible floors should be studied, namely, the pushover analyses, the limit analysis and the hybrid frequency time domain analysis.

Through a previous numerical study, it was concluded that the pushover analyses (proportional to the mass, 1st mode shape and adaptive), did not simulate correctly all failure mode of the structure, namely the out-of-plane mechanism, and should be used with caution. Thus, more research about application of these methods to the masonry buildings with flexible floors should be provide, namely the application of the new modal pushover analyses.

ACKNOWLEDGES

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