SHAKING TABLE TESTS OF STONE MASONRY BUILDINGS

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ABSTRACT
This paper presents the main results of experimental tests concerning the reduction of the seismic vulnerability of stone masonry buildings with flexible floors. The tests were performed in the LNEC 3D shaking table by imposing artificial accelerograms in two horizontal uncorrelated orthogonal directions, triggering in-plane and out-of-plane response of two tested mock-ups; one in original condition and another repaired. The preliminary results show that adopted measures are efficient, allowing to improve the seismic performance of this building typology.

INTRODUCTION

Ancient masonry buildings 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 performance assessment of ancient masonry buildings is particularly difficult to characterize and depends of several factors. Besides the quality of masonry materials and 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 programme 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) and to evaluate the efficiency of a repairing solution.

The ''gaioleiro'' buildings typology was developed between the mid 19th century and beginning of the 20th century, mainly in the city of Lisbon, and still remains much in use nowadays. This typology characterize a transition period from the anti-seismic practices used in the ''pombalino'' buildings originated after the earthquake of 1755, see e.g. (Ramos & 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 et al., 2004).

In the urban areas the "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; Viviane, 2007). It is noted the ''block'' effect is usually beneficial and provides higher strength of the building, as shown in Ramos & Lourenço (2004).

The experimental program involved shaking table tests by imposing artificial accelerograms in two horizontal uncorrelated orthogonal directions, inducing in-plane and out-of-plane response of tested mock-ups.

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.

The mock-ups are prepared to reproduce the geometrical, physical and dynamical characteristics of the prototypes of buildings typologies (e.g. reinforced concrete structures, unreinforced masonry structures with flexible floors) or individual structures (e. g. monuments, bridges). However, usually the mock-ups are simplified due to difficulties related to its reproduction in laboratory, namely related to the geometrical properties of the prototype or individual structures and the size of the facilities and, consequently, to the preparation of reduced scale mock-ups. In fact, it is difficult to fulfil 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 geometrical reduced. Thus, a 1:3 reduced scale taking in account Cauchy’s law of similitude was adopted. In this law 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. Table 1 presents the factors for the satisfaction of the Cauchy’s law similitude law.

Table 1 Scale factors of the Cauchy similitude (Carvalho, 1998).
(where $_p$ 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	ρ	$\rho_p/\rho_m=\lambda=1$
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	σ	$\sigma_p/\sigma_m=\lambda=1$
Strain	ε	$\varepsilon_p/\varepsilon_m=\lambda=1$
Time	t	$t_p/t_m=\lambda=3$
Frequency	f	$f_p/f_m=\lambda^{-1}=1/3$

The geometric properties of the non-strengthened mock-up (Fig. 1) result 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. 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 specialised workmanship.

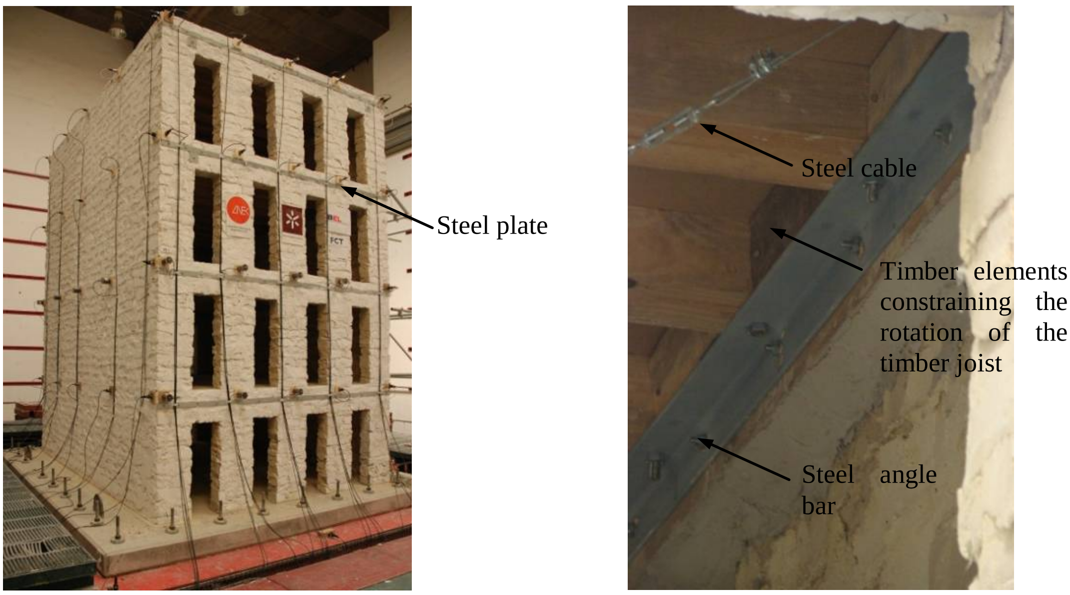
In the construction of the timber floors, medium-density fibreboard (MDF) panels connected to a set of timber joists oriented in the direction of the shortest span were used. The panels were cut in rectangles and stapled to the joists, keeping a joint of about 1 mm for separating the panels. The purpose was to simulate flexible floors with very limited diaphragmatic action (Fig 1b).

After the tests, the piers and the lintels of the façades 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 (Fig. 2) steel angle bars (internal surface) and plates (external surface) at the floor levels were used. These strengthening elements are connected among themselves by bolds, 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 no possible to apply reinforcement elements at 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 steel cables 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)

Fig. 1 Non-strengthened mock-up: (a) general view; (b) details of the floors.



(a) (b)

Fig. 2 Strengthened mock-up: (a) general view; (b) detail of the strengthening.

TEST PLANNING

Description of the tests

The seismic performance assessment 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 (Degée et al., 2007, Bairrão & Falcão Silva, 2009, 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) 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.

Due to costs involved, the mock-up does not have the same initial conditions, i.e. before the application of the seismic input the mock-up presents (cumulative) damage, with exception of the first one. The damage observed in the nominal test “i" is not only caused by the seismic action applied in the particular test, but it is also related with the excitation induced in the previous seismic tests. Thus, the damage indicator of the test “i" must be associated to the energy/intensity accumulated, which is a seismic action parameter obtained through the integration of the acceleration series. The characterization of the input series through the peak values must be adjusted taking into account the test planning. Eq. 1 presents a proposal to determine the equivalent PGA (PGA_{eqi}) through the use of the energy concept, in which E_{ac} is the accumulated energy until the actual test “i"; E_{noi} and PGA_{noi} are the nominal energy and peak ground acceleration in the test “i", respectively. This proposal does not take into account that the response of the mock-up (damage) observed in the test “i" is also a function of its initial conditions. It is noted that mock-ups with different initial conditions have different energy dissipation capacities.

$$PGA_{eq_i} = \left(\frac{E_{ac_i}}{E_{no_i}} \right)^{0.5} PGA_{no_i} \qquad (1)$$

The dynamic properties of the mock-ups were identified through forced vibration tests at the shaking table (Mendes et al., 2010) and its evolution is based on the experimental transfer functions (e.g. Frequency Response Function, FRF) obtained along the tests (Coelho et al., 2000).

The reduction of the natural frequencies is related to the stiffness variation and, consequently, to the evolution of the damage. Eq 2 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 presents different values for each mode shape.

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

In this procedure, the experimental vulnerability curves of the mock-ups are defined relating the seismic excitation parameters (energy/intensity accumulated, PGA_{eqi}) and the damage indicator “d”. Furthermore, the seismic performance of the mock-ups is assessed trough the results of the seismic tests (maximum displacement, drifts, crack patterns, etc).

Complementary to the dynamic tests, other tests were carried out to characterize the material properties of the masonry. In order to determine the Young's modulus, the Poisson ratio, the compression and the tensile strengths, ten specimens were prepared for axial and diagonal compression tests. The specimens are square with 1 m by 1 m and the thickness is equal to 0.17 m (thickness of the walls of the mock-up).

Instrumentation

The instrumentation used in the dynamic tests involves the measurement of several signals necessary for the quantification of the mock-ups behaviour. Besides the shaking table and the instrumentation necessary for its control, accelerometers were used in the masonry walls to characterize the response of the mock-ups.

The 3D LNEC shaking table is composed by a rigid platform, where the mock-ups are fixed, which is moved by four servo-controlled hydraulic actuators (one longitudinal, two transversal and one vertical) This equipment has six degrees of freedom, i.e. three translational and three rotational, which require a very sophisticated control system In plan the rigid platform has 4.6 m by 5.6 and the maximum load capacity is equal to 392 KN (Coelho & Carvalhal, 2005). In the case study only the transversal and longitudinal actuators were used and the vertical component of the earthquake was not considered. The input signals were measured by the accelerometers and displacement transducers installed on the shaking table.

In each façade twenty piezoelectric accelerometers (five per floor) with different sensibilities (10 V/g, 1 V/g and 0.1 V/g) were used. On the whole, the instrumentation of the mock-ups includes eighty accelerometers (Fig. 1 and 2), aiming at obtaining a detailed acceleration field of the walls. The simultaneous recording of 84 signals (4 input and 80 output signals) involved the use of two acquisition systems connected through a trigger.

In the axial and diagonal tests of the specimens a static hydraulic system was used, in which the applied load was measured directly from the system. Furthermore, the test planning included an internal instrumentation to measure the deformation of the specimens. Here, two vertical and two horizontal LVDTs were used in each surface (Fig. 3).

(a) (b)

Fig. 3 Test setups for specimens in (a) axial and (b) diagonal compression tests.

RESULTS OF THE TESTS

Axial and diagonal tests

Table 2 presents the results obtained in the axial compression tests. The compressive strength is, on average, equal to 6 MPa and was determined assuming a uniform stress in the cross-section of the wallets. The Young's modulus and Poisson ratio were calculated from the variation of the strains (average of the vertical and horizontal LVDTs) between 0.05 and 0.2 of the compressive strength. The average of the Young's modulus is equal to 3.37 GPa. The last three specimens presented unexpected values of Poisson ratio. Thus, due to lack of data, this parameter was not statically analysed.

The Young's modulus presents a significant coefficient of variation (19.5%) and the value of the last specimen W5 (2.51 GPa) appears to deviate markedly from other specimens of the sample. Thus, the Grubbs and Dixon criteria for testing outliers (ASTM E178-02, 2002) were used. Both tests indicated that the Young's modulus of the specimen W5 should not be considered as an outlier.

Table 2 Results of the axial compression tests.

Specimen	Specific mass (kg)	Compressive strength (MPa)	Young's modulus (GPa)	Poisson Ratio
W1	2182	5.81	4.07	0.23
W2	2135	5.55	3.32	0.20
W3	2171	6.17	3.97	0.09
W4	2141	5.91	3.00	0.44
W5	2182	6.56	2.51	0.05
Average	2162	6.00	3.37	-
CV (%)	1.1	6.4	19.5	-

In the standard interpretation of the diagonal compression test, the diagonal tensile strength is obtained by assuming that the specimen collapses when the principal stress, σ_I, at its centre achieves its maximum value. According to Frocht theory, as reported by Calderini et al. (2009), the principal stresses at the centre of the specimen are equal to: σ_I (tensile strength) = 0.5 P/A and σ_{II} = -1.62 P/A, in which the P is the load and A is the transversal area of the specimen.

Table 3 presents the principal stresses obtained through the diagonal compression tests. The average of the tensile strength is equal to 100 KPa, leading to the conclusion that, as expected, this value is significantly lower than the compressive strength (6.00 MPa). It is noted that according to Grubbs and Dixon criteria the principal stresses of the specimen W6 are outliers and were not considered in the average of the results.

Table 3 Results of the diagonal compression tests.

Specimen	Specific mass (kg)	Tensile strength (σ_I) (KPa)	Principal stress σ_{II} (KPa)
W6	2118	130†	-422†
W7	2129	104	-338
W8	2153	96	-310
W9	2159	103	-332
W10	2141	98	-318
Average*	2140	100	-325
CV* (%)	0.8	3.9	3.9

† outlier according to Grubbs and Dixon criteria

* discarding outliers

Dynamic tests of the mock-ups

The shaking table tests of the non-strengthened mock-up (NSM) 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, aiming at evaluating the reduction of the frequencies of the mode shapes along the seismic tests. In the preliminary analysis of the results only the 1st mode shape (translation in the transversal direction) and the crack pattern were considered. It is noted that the results are presented to1:3 reduced scale (Table 1).

Due to the seismic action induced, the frequency of the 1st mode shape ranged from 4.93 Hz (before the first seismic test) to 2.2 Hz (after the last seismic test). Fig. 4 presents the vulnerability curves, in which the damage indicator “d” (Eq. 2) is related to the amplitude of the seismic action. Besides the nominal values of the seismic action parameters in the test “i” (PGA_{noi} and IA_{noi}), the vulnerability curves were also plotted for the equivalent peak ground acceleration (PGA_{eqi}) and accumulative Arias Intensity (IA_{aci}) taking into account the cumulative damage along the tests. In the last test 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.

In the last crack pattern of the non-strengthened mock-up (4th test) only the lintels and the piers of the façades presents serious damage (Fig. 5). The concentration of damage at the piers of the last floor is highlighted, in which the horizontal cracks are related to its out-of-plane bending.

After the last test, the mock-up was repaired, strengthened and tested again. In the first dynamic identification the frequency of the 1st mode shape of the strengthened mock-up (SM) was equal to 4.49 Hz (variation equal to 9%).

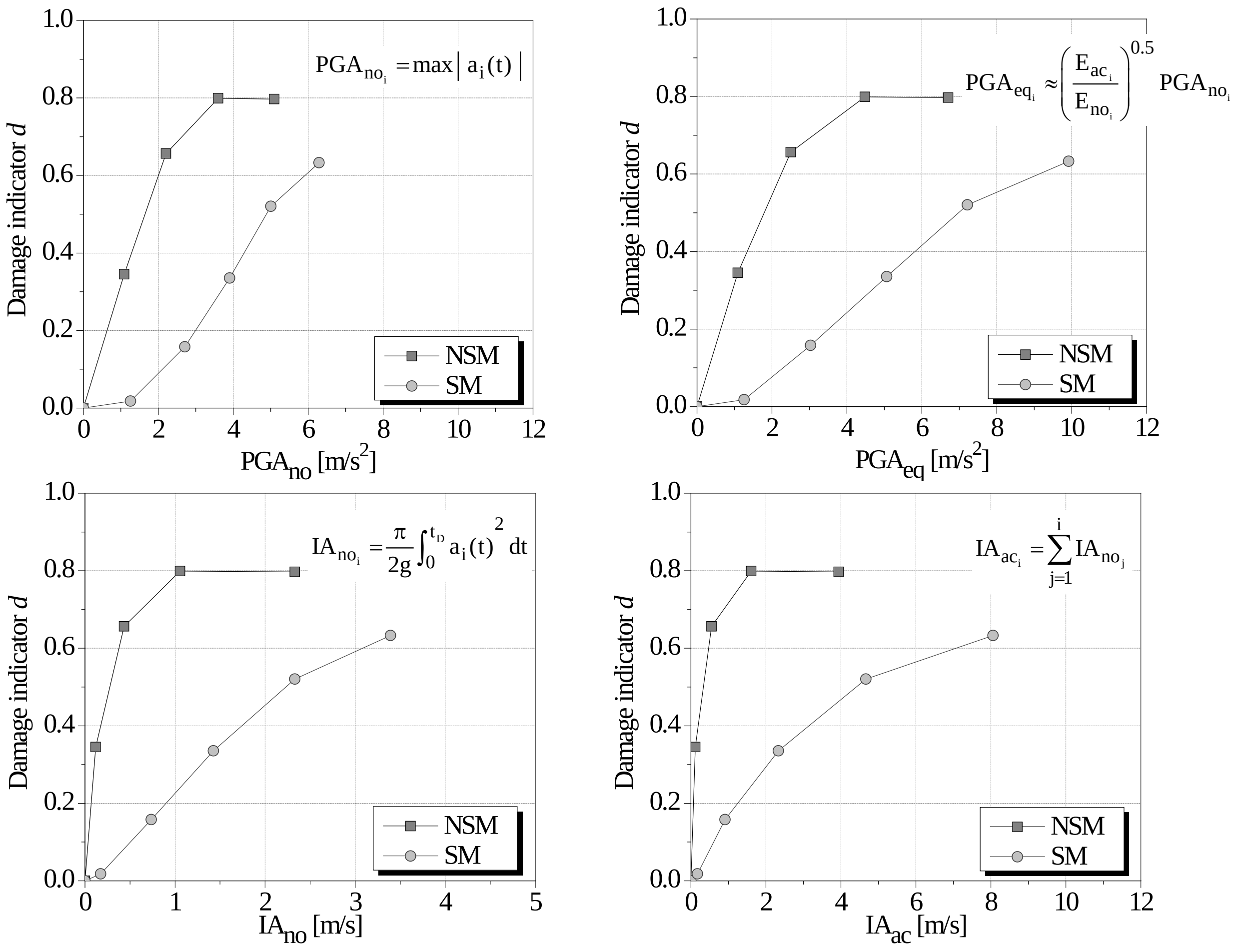


Fig. 4 Vulnerability curves of the mock-ups for different parameters of the seismic action only taking into account the frequency of the 1st mode shape.

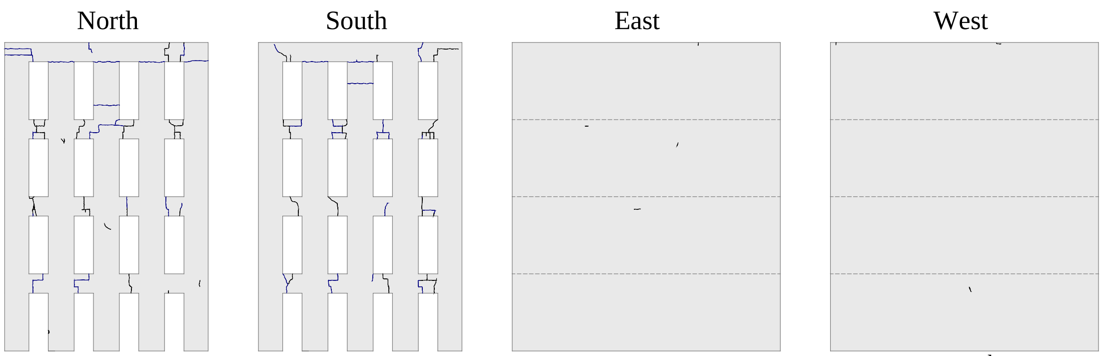


Fig. 5 Crack pattern of the non-strengthened mock-up after final testing (4th test).

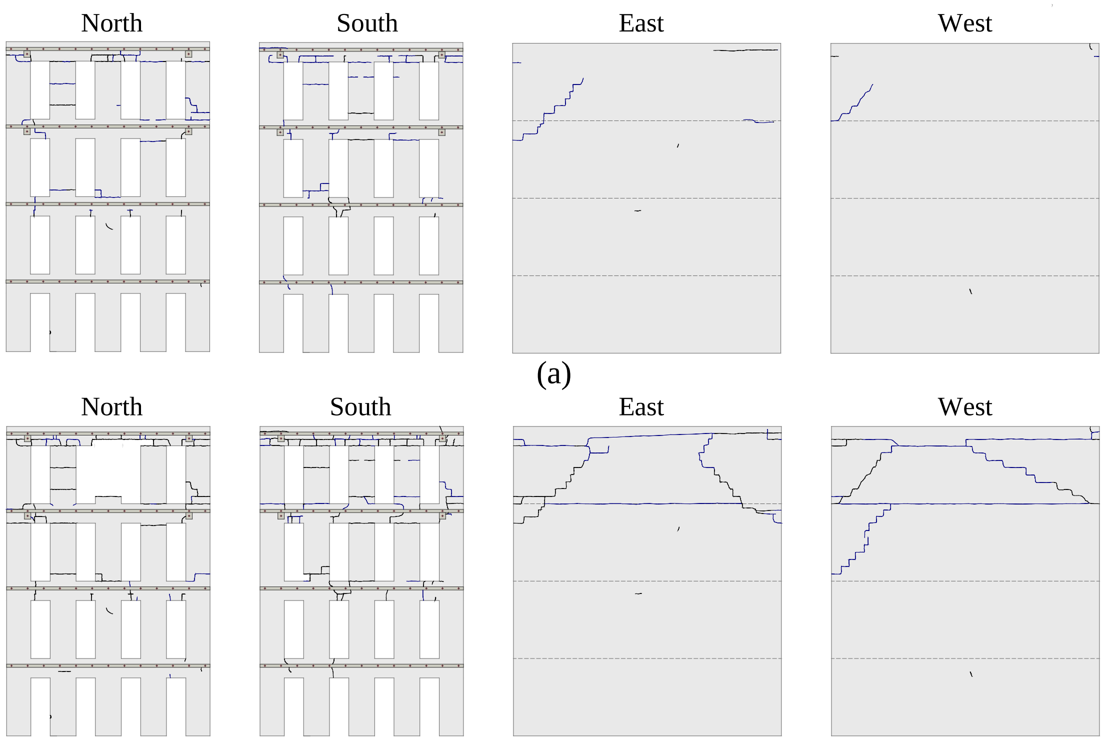


Fig. 6 Crack pattern of the strengthened mock-up after (a) 4th test (b) final testing (6th test).

Additionally to the tests carried out in the non-strengthened mock-up, in the strengthened mock-up two more 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 vulnerability curves of the strengthened mock-up (Fig. 4) show that the reinforcements were efficient to reduce the seismic vulnerability of the mock-up. In the 4th seismic test (100% of the code amplitude) the strengthened mock-up presented a reduction of the damage indicator (0.52) of 35%, with respect to the original building. The crack patterns also presented different characteristics. Contrarily to observe in the non-strengthened mock-up (Fig. 5), in which all lintels presented damage, the crack pattern of the strengthened mock-up (Fig. 6a) shows that the cracking of the lintels concentrates at the top floors. Furthermore, the gable walls present diagonal cracks and it is seems that part of the out-of-plane inertial forces of the façades were transferred to the gable walls.

In the last seismic test (Fig. 6b) in-plane rocking and out-of-plane bending of the piers of the top floor were observed. The crack pattern shows that the damage concentrates at the top floor (façades and gable walls) and the lintels of the 1st and 2nd floors of the façades not present serious cracking. Furthermore, the collapse of pier at the top floor of the North façade is highlighted.

CONCLUSIONS

The main aims of the paper are to assess and reduce the seismic vulnerability of stone masonry buildings with flexible floors. 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 non-strengthened and the strengthened buildings. The non-strengthened is representative of the "gaioleiros" buildings.

The preliminary results of the shaking table tests showed that the façades of the non-strengthened mock-up present serious damage. The reinforcement solution improved the seismic performance of the mock-up and a reduction of 35% of the damage indicator was obtained.

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