



SEISMIC VULNERABILITY OF ANCIENT MASONRY BUILDINGS

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KEYWORDS

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ABSTRACT

This paper presents the main results of experimental tests concerning the assessment and reduction of the seismic vulnerability of stone masonry buildings with flexible floors. The “gaioleiro” building typology, which probability presents the highest seismic vulnerability of the housing stock of Portugal, was used as case study. 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 the adopted measures are efficient, allowing to improve the seismic performance of this building typology.

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. Earthquakes are one of the most devastating natural hazards on Earth.

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 222,570 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.

The cities are areas of concentration of risk elements (people, buildings, bridge, infrastructures, etc.). Teheran (8.5 million), Jakarta (8.5 million) and Mexico City (18.13 million) are examples of large cities that suffered from earthquakes in the past.

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 (Tomažević 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.

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 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; Viviane 2007). It is noted the “block” effect is usually beneficial and provides higher strength of the building, as shown in Ramos and 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 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. 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)

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	$f_p/f_m=\lambda^{-1}=1/3$

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

The geometric properties of the non-strengthened mock-up (NSM) 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 (Figure 1). 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 for separating the panels. The purpose was to simulate flexible floors with very limited diaphragmatic action (Figure 1b).

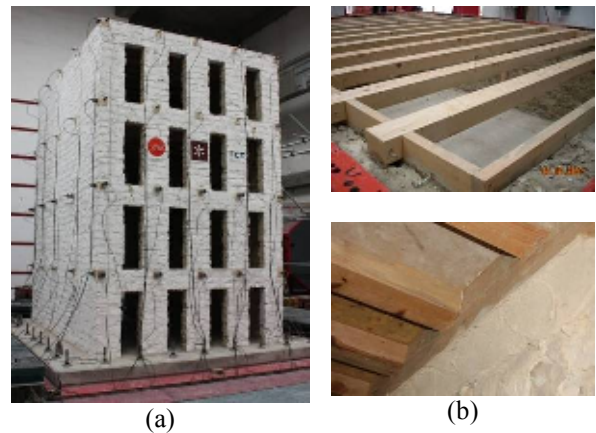


Figure 1: Non-strengthened mock-up: (a) general view; (b) details of the floors

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 (SM) steel angle bars (internal surface) and plates (external surface) at the floor levels were used (Figure 2). 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 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.

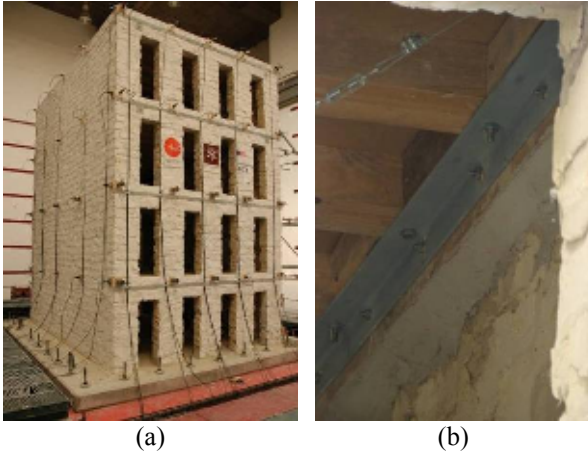


Figure 2: Strengthened mock-up: (a) general view;
(b) detail of the strengthening

TEST PLANNING

Description of the tests

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 (Degée et al. 2007; Bairrão and 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) 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.

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. Equation (1) presents a proposal to determine the equivalent PGA (PGA_{eq_i}) 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.

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

The energy in the tests “i”, E_i , has been defined as a work done by the shaking table when moving the models, idealized as rigid bodies, and can be determined by the following Equation (Tomažević et al. 1996):

$$E_i = m \int_0^{td} |a_i(t) v_i(t)| dt \quad (2)$$

where m is the mass of the model; a and v are the acceleration and the velocity at base of the model, respectively, and td is the duration of the signals.

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 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. Equation (3) 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.

$$d_{k,i} = 1 - \left(\frac{f_{k,i}}{f_{k,0}} \right)^2 \quad (3)$$

In this procedure, the experimental vulnerability curves of the mock-ups are defined relating the seismic excitation parameters (PGA, PGA_{eq_i} , energy/intensity accumulated) and the damage indicator “d”. Furthermore, the seismic performance of the mock-ups is assessed through the results of the seismic tests (maximum displacement, drifts, crack patterns, etc). Additional 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 behavior. 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 and Carvalhal 2005). In the present 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 (Figure 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 specimens subjected to axial (Figure 3) and diagonal (Figure 4) loading 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.



Figure 3: Test setups for specimens in axial compression tests



Figure 4: Test setups for specimens in 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 5% and 20% 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 and these results were discarded.

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 $\sigma_{II} = -1.62 P/A$, 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 identification 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, aiming at evaluating the reduction of the frequencies of the mode shapes along the seismic tests. Additionally, 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.

In the first dynamic characterization of the non-strengthened mock-up (before the first seismic test) 11 mode shapes were estimated (Figure 5). The transversal modes are mainly associated to the global behavior of the mock-up and were clearly identified (1st, 5th and 10th modes). The longitudinal modes are mainly related to the local behavior of the façades and can be distinguished by the type of curvature (single, double and triple). Due to the presence of two peaks very close in the FRF's, the 2nd mode of the North façade was not clearly identified (6th and 7th modes). A distortional mode (2nd mode) and a combined mode (8th mode) were also identified.

Due to the damage that occurred in the intense seismic tests, it was not possible to continue estimating the 11

mode shapes along the tests. In fact, only the three transversal modes and the first two longitudinal modes were adequately estimated in all characterization tests. The average MACs (Allemang 2003) of the first and second transversal modes are equal to 0.95 and 0.77, respectively. The others mode shapes, mainly the longitudinal modes, present very low MACs, due to the damage concentration of the façades that occurred along the seismic tests. As an example, the frequency of the 1st mode shape ranged from 4.93 Hz (before the first seismic test) to 2.22 Hz (after the seismic test with 100% of the code amplitude).

The analysis of the dynamic identification tests of the strengthened mock-up is still under development. However, in a preliminary analysis of the results the 1st mode shape was clearly identified. The frequency of this mode shape ranged from 4.49 Hz (before the first seismic test) to 2.72 Hz (after the seismic test with 125% of the code amplitude).

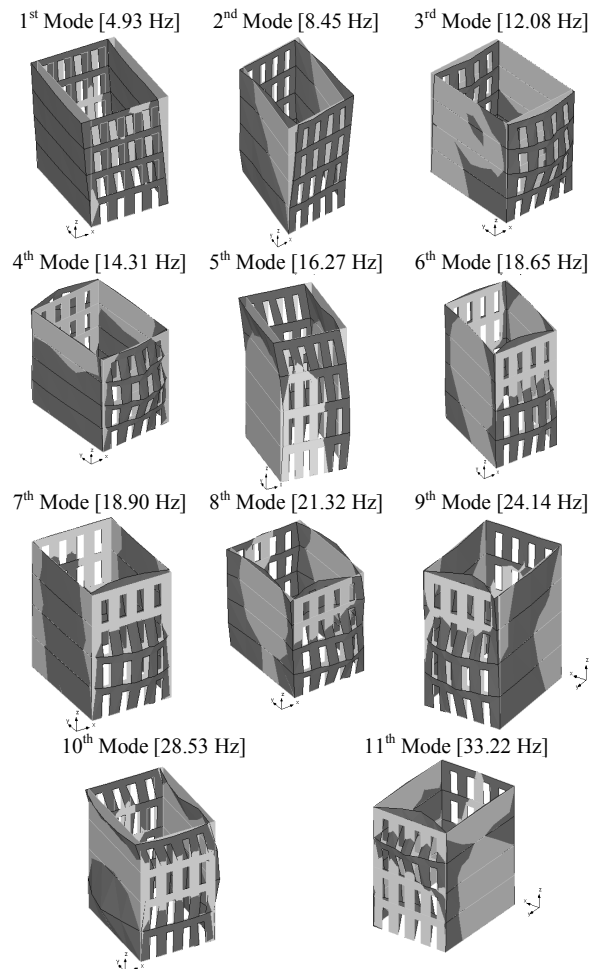


Figure 5: Mode shapes of the first dynamic identification test of the non-strengthened mock-up

Seismic vulnerability assessment

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 the results are presented to 1:3 reduced scale (Table 1).

Figure 6 presents the vulnerability curves, in which the damage indicator “d” (Equation 3) is related to the amplitude of the seismic action. Besides the nominal values of the seismic action parameters in the test “i” (PGA_{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. As an example, the PGA_{eqi} in the last test of the non-strengthened mock-up is about 1.32 of the PGA_{no} .

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 (Figure 6). 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 (4th test) shows that only the lintels and the piers of the façades present serious damage (Figure 7). 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.

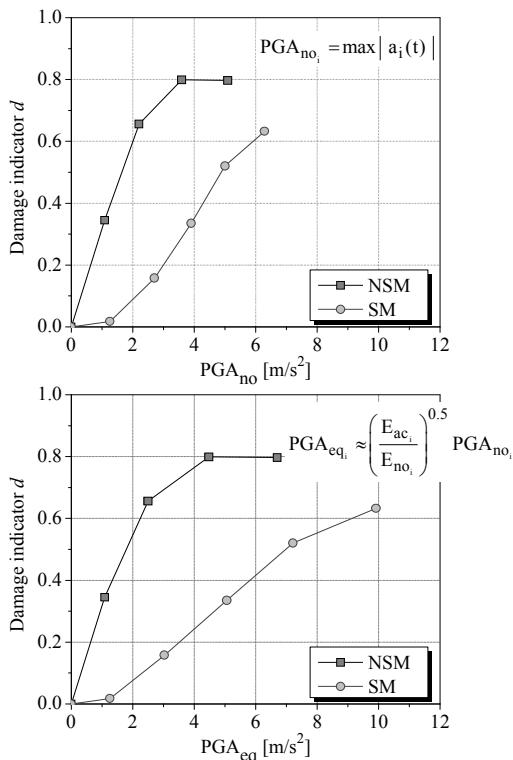


Figure 6: Vulnerability curves of the mock-ups for different parameters of the seismic action only taking into account the frequency of the 1st mode shape. After the last test, the non-strengthened mock-up was repaired, strengthened and tested again. The vulnerability curves of the strengthened mock-up (Figure 6) show that the strengthening was 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 the observation in the non-strengthened mock-up (Figure 7), in which all lintels presented damage, the crack pattern of the strengthened mock-up (Figure 8) shows that the cracking of the lintels concentrates at the top floors. Furthermore, the gable walls (4th floor) present diagonal cracks, indicating that part of the out-of-plane inertial forces of the façades were transferred to the gable walls.

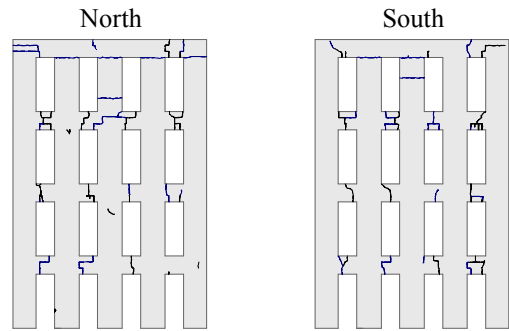


Figure 7: Crack pattern of the façades of the non-strengthened mock-up after final testing (4th test)

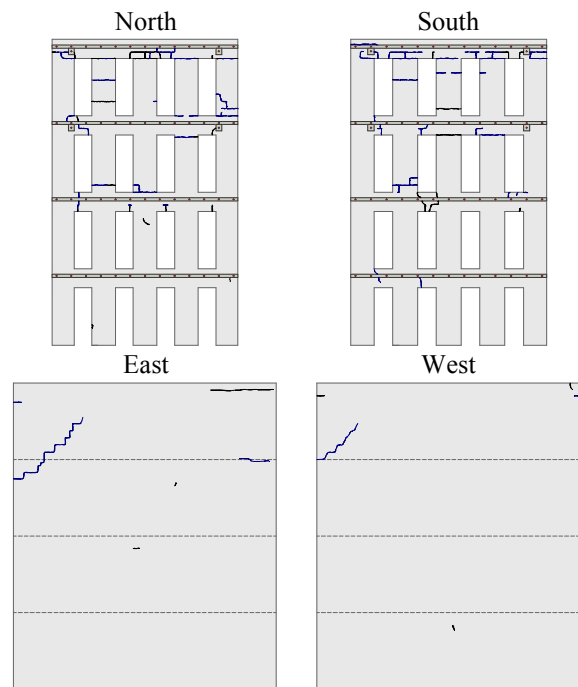


Figure 8: Crack pattern of the strengthened mock-up after 4th test

In the last seismic test of the strengthened mock-up (Figure 9), 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 1st and 2nd 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.

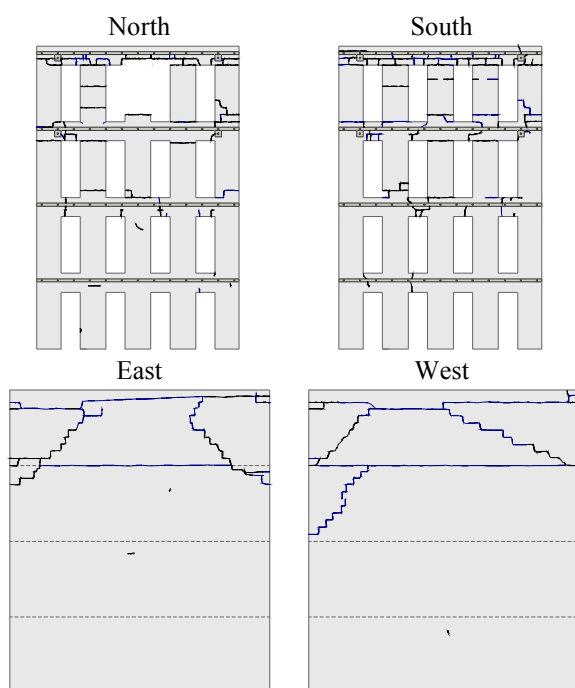


Figure 9: Crack pattern of the strengthened mock-up after final testing (6th test)

PREPARATION AND CALIBRATION OF THE NUMERICAL MODEL

The numerical model of the non-strengthened mock-up was prepared using the Finite Element (FE) 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 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. It is noted that the numerical model was prepared on the 1:3 reduced scale (Table 1).

The first stage of calibration of the numerical model was based on the comparison between experimental and numerical frequencies and MACs of the first six modes shapes. In this stage five numerical models, taking into account different of 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. Table 4 presents the values of the variables after calibration. The Young's modulus of the gables walls (3.17 GPa) approaches the value obtained in the uniaxial compression tests (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 specimens, and to the connection between façades and floors.

Besides the calibration of the modes shapes (linear dynamic behavior), the calibration of the non-linear behavior of numerical model is planned, namely from the collapse mechanism observed in the seismic tests.

Table 4: Variables of the 5th model after calibration

Variable	Young's Modulus (GPa)
Façades	0.58
Gable walls	3.17
MDF panels	0.15
Corners	1.59

CONCLUSIONS AND FUTURE WORKS

This paper presents an experimental method to assessment the seismic vulnerability of masonry buildings with flexible floors. Furthermore, a strengthening solution was also proposed. In the case study the "gaioleiro" building typology, presents in main cities of 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 non-strengthened and the strengthened 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.

The prototype of the tested mock-ups was prepared taking into account the mean characteristics of the "gaioleiro" buildings" and the deviation within typology was not evaluated. Thus, the future works should involve the non-linear calibration of numerical model and a parametric study, assuming as variables the type of soil, the material properties and the number and the stiffness of the floors. Moreover, future works will include the study on different techniques of structural analysis of masonry structures, namely, non-linear dynamic analysis with time integration, pushover

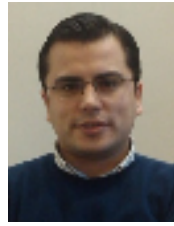


analysis, limit analysis and hybrid frequency time domain analysis.

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