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SHAKING TABLE TESTS OF 1:3 REDUCED SCALE MODELS OF FOUR STORY UNREINFORCED MASONRY BUILDINGS

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SUMMARY

This paper presents an ongoing experimental programme that aims at the quantification of vulnerability curves of typical Portuguese "gaioleiro" buildings, before and after reinforcement, adequate for global seismic risk assessment. The tests are performed in the LNEC 3D shaking table on 1:3 reduced scale models of 4 story unreinforced masonry buildings with masonry shear walls and wood-framed floors. The experimental tests are described and the reinforcement schemes, as well as the reasons behind their selection, are summarized. The preliminary results obtained so far show that the models replicate the typical collapse modes of these buildings.

INTRODUCTION

Scope of the study

The preservation of the building stock is an issue of utmost importance in modern societies, especially for those in seismic regions. The loss of lives or the disruption of normal function in plants, or even the interruption of lifelines, due to earthquake damage to buildings presents a high toll. When the attention is focused on global seismic risk, one of the main concerns go to the old residential buildings, originally built with insufficient earthquake resistance or none at all, whose seismic vulnerability is, therefore, high.

In an attempt to mitigate the seismic risk associated to these buildings, large-scale rehabilitation solutions are sought in order to improve their seismic performance. These solutions are, naturally, constrained by the minimization of the global cost of the retrofitting program. The determination of reliable vulnerability curves able to represent the expected level of damage before and after rehabilitation is, therefore, necessary to evaluate the global seismic risk.

In view of these aspects, an ongoing experimental programme was set to assess the vulnerability of a particular type of buildings and to evaluate the efficiency of different reinforcement solutions. To meet the purpose of the research programme, the selection of the prototypes is done amongst the building typologies according to their percentage in the housing stock and their vulnerability. Previous studies

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made on the Portuguese housing stock [1] have already identified and characterized several building typologies and their level of seismic vulnerability.

The selected prototypes [2] correspond to one of the Portuguese housing stock typologies that show the highest seismic vulnerability usually known as “gaioleiros”. This building typology developed between the mid XIXth century and the beginning of the XXth century, mainly in the city of Lisbon where many residential neighbourhoods were built and are still in use nowadays. The typical buildings are 4 storeys high, have unreinforced masonry walls and gable roofs.

The exterior walls of these buildings are, predominantly, in coursed rubble masonry with bonding mortar [3], with thicknesses ranging from 0.30m to 0.60m and even more depending on the height of the building. The partitions are mainly stud walls sheathed with thin wood boards and plaster, although there can be also some unreinforced masonry walls. The floors in the older buildings are wood-framed floors, whereas in the more recent ones they are made of hollow core slabs. These latter buildings represent only a small percentage of this type of buildings and so they are considered not representative and, therefore, have been discarded from the study.

Description of the prototypes

The prototypes considered in the study are 4 stories high, with an interstory height of 3.60m, and have wood-framed floors composed of joists and decking. The decking is usually made of wood boards nailed to the joists and, in some cases, there are also rim joists connecting the floors to the walls. The façades have openings that assume several sizes and shapes, ranging from typical windows to over-sized doors, and occupy an area between 25% and 35% of the total area of the façade. When the buildings are inserted in a block the side walls are broken to include vertical shafts for natural ventilation and light because the depth of buildings is usually high.

The buildings can have many different sizes in plan and assume different geometries according to their location in urban areas and within blocks. Moreover, they are usually roughly rectangular, less wide than deep. Considering all this information, two prototypes were idealized:

Prototype S – isolated building, with 9.45m×12.45m in plan, a constant wall thickness of 0.45m, two opposing façades with a percentage of openings of 28.6% of the area of the façade, two opposing side walls plane with no openings, and a gable roof;

Prototype B – building inserted in a block, with 9.45m×12.45m in plan, a constant wall thickness of 0.45m, two opposing façades with a percentage of openings of 28.6% of the area of the façade, two opposing side walls with vertical shafts and an opening per story, and a gable roof.

The seismic performance of these buildings is known to be deficient even though the seismicity of the Portuguese Mainland, characterized by seismic events rather spaced in time, does not allow observing it. However, the observed behaviour in buildings similar to the selected prototypes in other countries like Italy is characterized, among other features, by generalized cracking of the exterior walls and out of plane collapse. It is therefore of great interest to evaluate their behaviour experimentally since seismic tests present a unique way to evaluate directly the evolution of global structural damage, and provide quite realistic information on the expected structural response [4] [5].

MODEL CONSTRUCTION

Preparation of the physical models

The preparation of the physical models involves the study of several aspects related to the reproduction of the main geometrical, physical and behavioural characteristics of the prototypes, in view of the dynamic tests in the LNEC shaking platform. Therefore, the dimensions of the different elements and their contribution to the global performance of the prototype, the characteristics of structural and non-structural materials and their interconnections, the building processes used in the prototype were taken into account in the construction of the physical models.

In the first place, due to the shaking platform size and payload capacity, the models had to be geometrically reduced from the original prototypes. For those reasons it was adopted a geometrical scale factor of 1:3, which verified the referred constraints. As a consequence of this reduction factor, all phenomena involved in the dynamic tests that were being set are scaled according to a similitude law. The Cauchy similitude law was chosen in the present case, which implies the relations for the different parameters in terms of the geometrical scale factor presented in Table 1:

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

Parameter	Symbol	Scale factor
Length	L	$L_p/L_m=\lambda=3$
Elasticity modulus	E	$E_p/E_m=e=1$
Specific mass	ρ	$\rho_p/\rho_m=\rho=1$
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	$v_p/v_m=1$
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	$\sigma_p/\sigma_m=1$
Strain	ε	$\varepsilon_p/\varepsilon_m=1$
Time	t	$t_p/t_m=\lambda=3$
Frequency	f	$f_p/f_m=\lambda^{-1}=1/3$

Another aspect studied was the building process to use in the construction of the models. Although the construction technique used in the prototypes is known, its reproduction in the laboratory environment is very much time consuming and would require specific workmanship that is no longer available. As such, some simplifying hypothesis were adopted that, nevertheless, preserve the key aspects necessary for the characterization of the seismic performance of the prototypes such as their geometry and the interconnections wall-to-wall and wall-to-floor.

A simplified model was finally adopted that contemplates only the exterior walls and the floors. The dimensions of the model result directly from the application of the geometric scale factor to the prototype, resulting in a model with 3.15m wide, 4.8m high, 4.15m deep and 0.15m in wall thickness. In Figure 1 is presented the geometry of the Model type S that corresponds to the 1:3 reduction of the Prototype S.

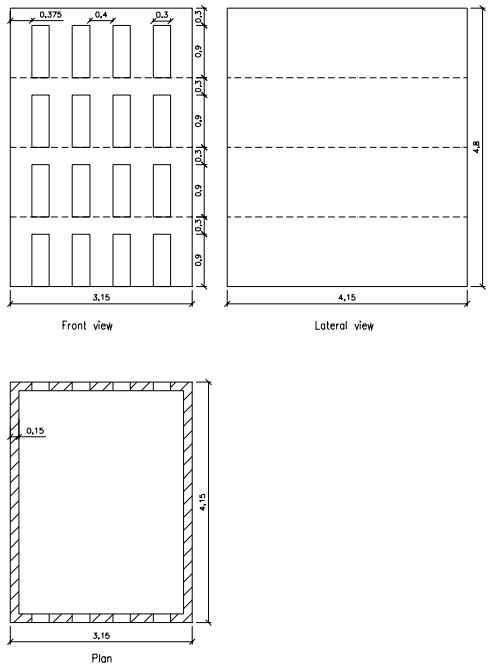
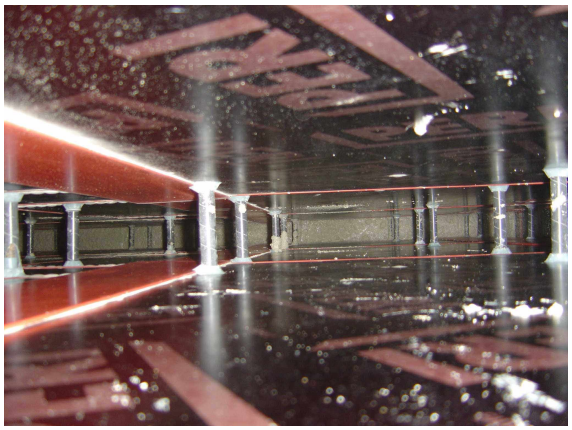


Figure 1: Model type S geometry

The exterior walls, originally built in coursed rubble masonry, were replaced by a self compacting concrete with a composition studied to reproduce the brittle behaviour of the masonry walls, with a very low compressive resistance and an almost null tension resistance. This way, values of around 1MPa were achieved for the compressive resistance at 28 days, which were considered adequate to the simulation of the prototypes. In Figure 2 are presented two aspects related with the construction of the models, where can be seen the formwork used for the construction of the walls and the pouring of the self compacting concrete.

a) general view



b) view from inside the formwork

Figure 2: Some views of the construction of the models

On the other hand, the floors, originally made of wood boards, have been replaced by MDF panels set on top of wood joists oriented in the direction of the smaller span. The panels were cut in rectangles of 0.57m×0.68m and 0.55m×1.55m and stapled to the joists with broken joints of approximately 2mm. This way, the discontinuities and the joints existent in the floors of the prototypes are simulated and thus the floor flexibility. The wood joists are set on top of discontinuous rim joists placed along the side walls. In Figures 3 and 4 are presented some detailed aspects of the wood-framed floor of Model type B, corresponding to the 1:3 reduction of the Prototype B, and in Figure 5 a front view of one of the Model 0.

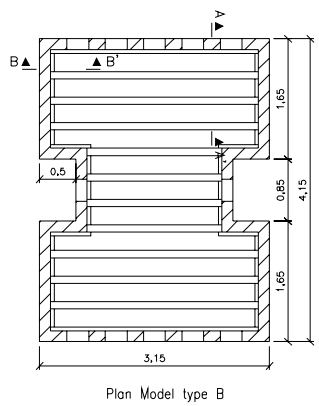


Figure 3: Model type B plan geometry and floor joists

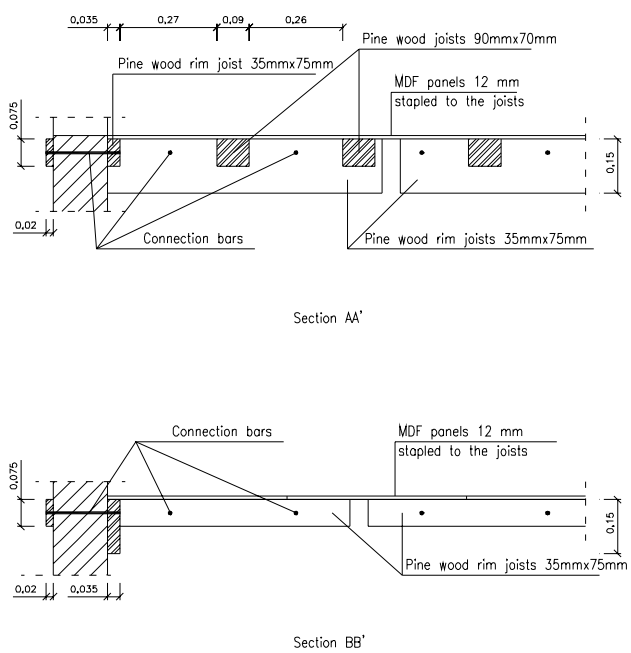


Figure 4: Structural details of the wood-framed floors

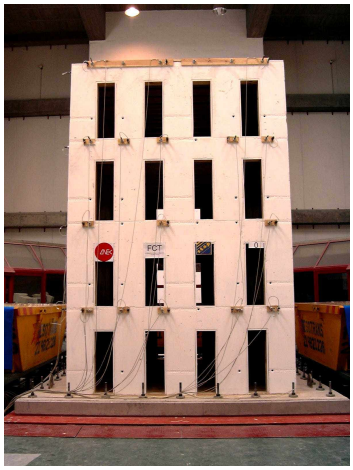


Figure 5: Front view of Model 0

Reinforcement solutions

The search for retrofitting schemes adequate for large scale risk mitigation is based both on simplicity and effectiveness, without overlooking economic aspects related to applicability and modularity. In view of the typical collapse modes revealed by the prototypes, reinforcement solutions were studied that allowed to reduce the out of plane collapse of the walls, to improve the in plane behaviour of the walls and to control the generalized cracking. Three different reinforcement solutions were devised:

1) Reinforcement of the wall-to-floor connection, using steel connectors and fibre glass strips glued with epoxy resins: the reinforcement of the connections is done only in the 3rd and 4th storeys near the piers (façades) and on the supports of the joists (side walls), as shown in Figure 6;

2) Connection of opposing walls by means of steel ties at the storey levels: the ties are placed at the level of the 3rd and 4th storeys, connecting opposing façades with 3 ties per story and opposing side walls with 4 ties per story, as shown in Figure 7;

3) Reinforcement of existing piers in the façades by means of fibre glass strips glued with epoxy resins and steel connectors [6]: the reinforcements are placed on the outside and in the full height of the piers, forming a net in two directions as shown in Figure 8. Additionally steel ties have been placed connecting opposing façades at the level of the 3rd and 4th storeys and connecting opposing side walls at the level of the 4th story.

a) connection of the piers to the floors b) connection of the side walls to the floors

Figure 6: Reinforcement of the wall-to-floor connection in the 3rd and 4th storeys

Figure 7: Connection of opposing walls with steel ties in the 3rd and 4th storeys

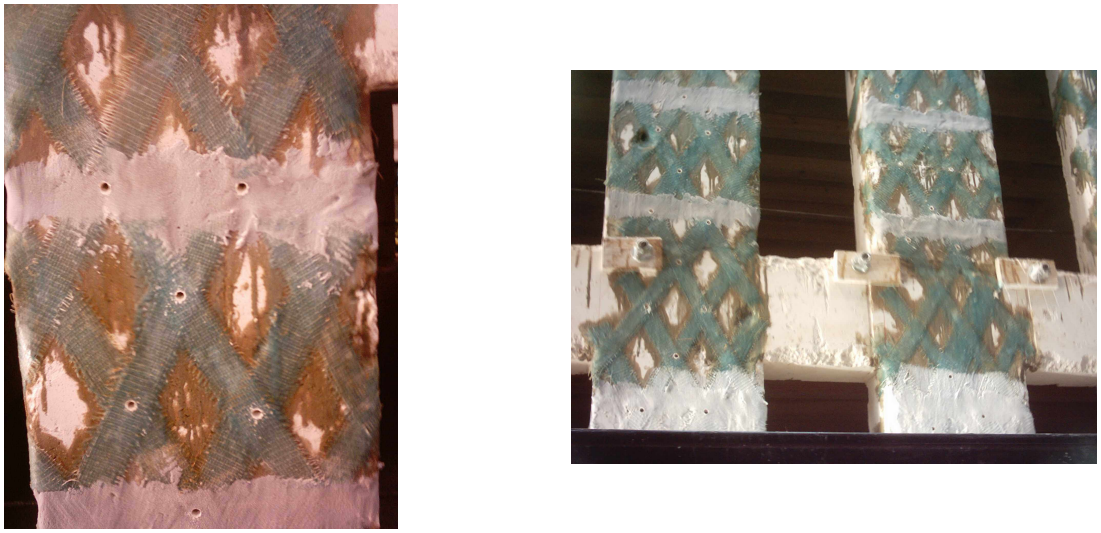


Figure 8: Reinforcement of piers with fibre glass strips glued with epoxy resins and steel connectors

According to the two prototype geometries and the three reinforcement solutions, there are a total of 8 tests to be performed, as presented in Table 2:

Table 2: Models to test

Model	Prototype	Reinforcement solution
0	S	Without reinforcement
1	S	Reinforcement of the wall-to-floor connection
2	S	Connection of opposing walls with steel ties
3	S	Reinforcement of piers and steel ties
4	B	Without reinforcement
5	B	Reinforcement of the wall-to-floor connection
6	B	Connection of opposing walls with steel ties
7	B	Reinforcement of piers and steel ties

EXPERIMENTAL TESTS

Tests description

The dynamic tests are performed on the LNEC shaking table by imposing time series of artificial accelerations compatible with the design response spectrum defined by the codes for zone A and stiff soil conditions. The time series are imposed with increasing amplitude and in two uncorrelated orthogonal directions. Before the beginning of the test and after each time series is used a signal to characterize the dynamic properties of the model, in order to quantify the damage based on the degradation of the vibration frequencies as a function of the intensity of the time series.

In complement to the dynamic tests, other tests are performed to characterize the mechanical properties of the concrete used to simulate the masonry, namely its resistance to compression and tension. For this purpose cubic specimens have been prepared for compression tests, as well as small beams for bending tests as shown in Figure 9:

Figure 9: Concrete beams for bending tests

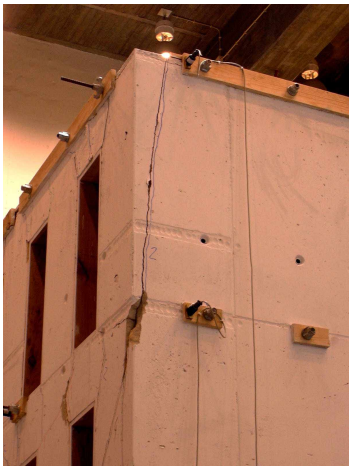
Instrumentation plan

The instrumentation of the models involves the measurement of several quantities necessary to the quantification of the models behaviour. Along with the instrumentation necessary for the control of the shaking platform itself, which includes accelerometers and displacement transducers, were used accelerometers in several positions along the walls and at several levels so as to characterize the acceleration field in the walls. In the models reinforced with ties additional force cells were used to measure the forces in the ties.

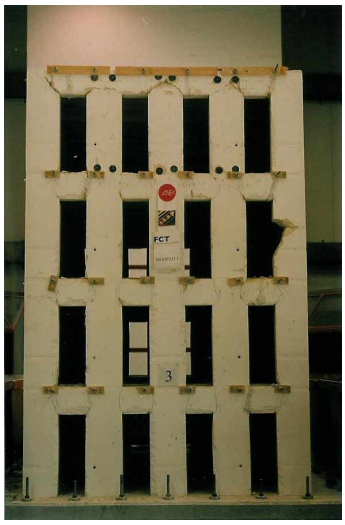
Preliminary results

The assessment of the vulnerability of the models will be based on the numerical results obtained during the tests as well as on qualitative results registered in video and photography. The experimental programme is on the way, having been performed so far some of the tests foreseen, reason why the available results are so scarce. Nevertheless, the damage evolution in the models can be recognised visually, with the progressive opening of large cracks and the fall of some blocks that come apart from the walls.

The results already available show the formation of the typical collapse modes observed on the prototypes, with generalized cracking of the walls and out of plane collapses associated to global vibration modes of the side walls in the case of the unreinforced model. On the other hand, in the reinforced models were observed local vibration modes on the piers and a different cracking pattern on the side walls. In Figures 10 and 11 are presented some aspects of Models 0, 1, 2 and 3 at the end of the tests, where it can be seen the opening of the top corners in Model 0, a clear concentration of damage to the piers between the 3rd and 4th storeys in Models 1 and 2, and the total collapse of the 4th storey masonry spandrel in Model 4.



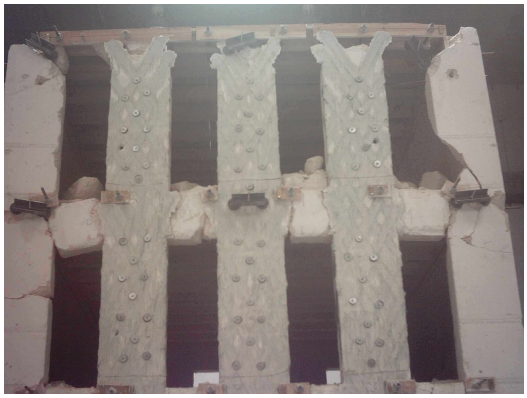
a) Model 0 (Without reinforcement) – corner cracking



b) Model 1 (Reinforcement of the wall-to-floor connection)

Figure 10: Detail of the damage to the façades on Models 0 and 1

a) Model 2 (Connection of opposing walls with steel ties)



b) Model 3 (Reinforcement of piers and steel ties) – collapse of the 4th story spandrel

Figure 11: Detail of the damage to the façades on Models 2 and 3

CONCLUSIONS

The preliminary qualitative results show that the typical collapse modes of the prototypes are replicated in the tests performed on the models, with different damage patterns being observed depending on whether there is, or is not, a reinforcement scheme. In view of the results presented, the numerical results not yet available should be able to confirm these conclusions and to determine the seismic vulnerability curves of the “gaioleiro” buildings.

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