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Structural Behaviour of a Masonry Wall Under Horizontal Cyclic Load; Experimental and Numerical Study

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ABSTRACT: This paper presents a numerical and experimental study on the structural behaviour of a stone masonry wall recovered from a house in Faial island, Archipelago of Azores, that collapsed during the 9th July 1998 earthquake. The wall was identified and tested under cyclic horizontal loads at the Laboratory of Seismic and Structural Engineering (LESE) of the Faculty of Engineering of Porto University, to simulate the effects of a horizontal seismic action. In particular, the experimental response allowed accessing the cyclic behaviour and estimation of energy dissipation and ductility capacity of the structure, as well as its strength and stiffness. Afterwards the wall was simulated numerically using a finite element method. The stones and the infill were simulated separately using different behaviour models and the link between the two materials was simulated with joint elements (Pegon, 1999). Finally, the numerical results were compared to the experimental response.

1 INTRODUCTION

This work presents the experimental behaviour and the numerical simulation of a wall recovered from a two storey house located at the parish of Pedro Miguel, Horta council, Faial Island of the Archipelago of Azores, Fig. 1. The wall was located at the ground floor between two main doors of the house façade.

Figure 1 : Original location of the wall

It is a traditional two-sleeve stone masonry wall with a poor cohesionless infill material as found in most of the old constructions in the Archipelago, with a mortar cover. The wall was transported from its original location to the Laboratory for Seismic and Structural Engineering of the Faculty of Engineering of Porto University (LESE - Laboratório de Engenharia Sísmica e Estrutural da FEUP) by sea, where it was set on a reinforced concrete block. The connection between the wall and this block (foundation) was meant to be weak; a sand pillow was set between the two structures. A rigid concrete plate and a steel profile were set at the top of the wall to allow a uniform distribution of the horizontal and vertical loads applied to the wall during the test.

2 LABORATORY TEST

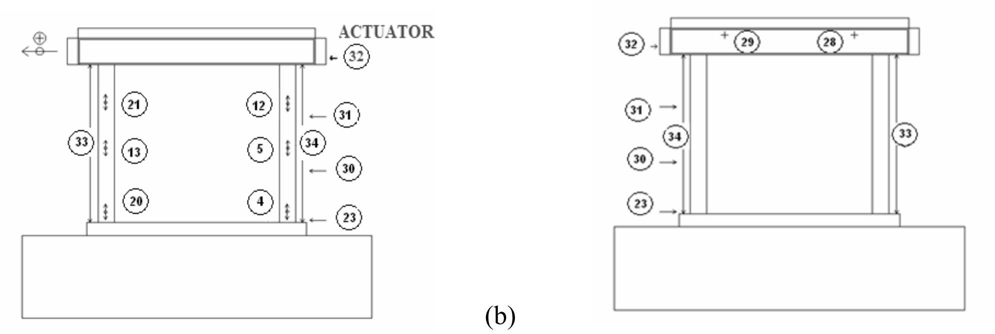
2.1 Test methodology

The test was performed in order to study the behaviour of the masonry wall in terms of strength, ductility and energy dissipation capacity under imposed cyclical horizontal displacements in-plane, to simulate the effect of a horizontal seismic action.

Two hydraulic jacks were set on the top of the wall against a reaction structure linked to the foundation through steel rods, to reproduce the existing vertical load. This system allowed a good distribution of the vertical load applied to the wall. The horizontal displacements were imposed using a hydraulic jack linked on one side to a reaction structure and, on the other, to the wall cap structure through a hinged connection. The wall foundation was connected to the lab floor through high strength pre-stressed steel rods (Fig. 2).

Figure 2 : Wall view and testing scheme.

The test was performed using a displacement control system that simultaneously collected data from all monitored spots: five load cells (Glória, 2004) (one at each steel tie between the top reaction structure and the foundation, and one from the actuator) and thirteen LVDT's, placed as shown in Fig. 3. This figure also indicates the positive direction of the horizontal displacements. Spot 32 refers to the LVDT controlling the actuator. In the text, the vertical elements referred to as door columns correspond to the regular stones at both sides of the wall that were part of the structure of the house doors. In order to simplify, the LVDT's will be referred to by their spot number according to Fig. 3.



(a) (b)

Figure 3 : LVDT's positioning: (a) front view, (b) back view.

3 ANALYSIS AND INTERPRETATION OF THE TEST OUTCOME

3.1 Imposed displacements curve

A set of horizontal displacements was imposed in the longitudinal direction (positive/negative) of the wall with peaks ranging from 0 to 10 mm (Fig. 4). However, due to the different capacity of the hydraulic jack to move forward and backwards, the displacements imposed in both directions were different.

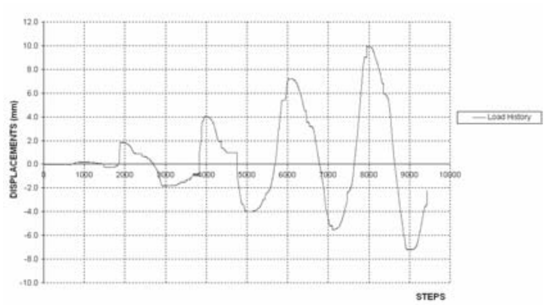
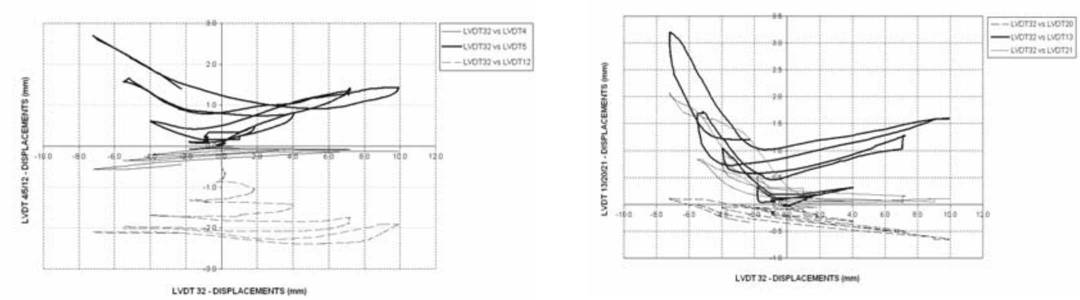


Figure 4 : Horizontal displacements series imposed on the top of the wall.

3.2 Analysis of the joints behaviour

The analysis of the behaviour of the door columns joints cannot be performed separately, since the behaviour of one joint affects the others. Fig. 5 shows the relationship between the wall overall horizontal displacement at the application force (spot 32) and the openings of the door columns joints at the right (4, 5, 12) and left (13, 20, 21) side.



(a) (b)

Figure 5 : (a) Joint openings at the right and (b) left side vs the horizontal top displacement (32).

According to the results, damage is concentrated on the joints immediately above the basement i.e., spots 5 and 13. This situation expresses a formation of an inclined strut from the top till that position, confirmed by an important crack towards this direction as it can be seen in Fig. 6. It should be noticed that the concrete foundation may have contributed to this result avoiding a higher concentration of damage near the basement.

Figure 6: Wall façade after the experiment.

3.3 LVDTs 4 and 20

The outcome of spots 4 and 20, set near the basement, was the expected. During the imposed horizontal displacements, the joints showed a consistent behaviour: when one of the joints closed the other opened. In particular, when the displacement was imposed in the negative direction, the joint at spot 4 showed a closing tendency. However, when the wall moved in the positive direction, the observed decompression was not enough to open the joint. In general terms, this joint showed an overall displacement towards its closure. This behaviour might be the result of an inside rearrangement of the joint particles, due to vertical and tangential forces generated at the joint due to the applied forces. It is also noticed that the joint global displacement at this spot is quite small (~0,3 mm towards closure), when compared to the other joints displacements at the same door column. In fact, damage is concentrated on the joints immediately above the basement joints i.e., spots 5 and 13.

3.4 LVDT 5 and 13

These joints showed a quite unusual behaviour, since their performance is alike. Instead of showing an alternate behaviour like the previous ones, the joints opened and closed at the same time. This might have happened due to the stones configuration and cut surfaces that appeared during the test. These cut surfaces result from the fact that this is a short column type structure, quite heterogeneous, with low cohesion between its constituent elements (blocks and infill) and no tensile strength. In fact, when activated by horizontal forces the wall follows a Strut and Tie type behaviour model. According to this model, the wall resists the applied horizontal forces through its most rigid elements along a diagonal strut. However, since the wall showed a strong resistance at the foundation connection, damage was then concentrated at the immediately above door column joints (5, 13). Damage concentration in this area could be detected through a simple analysis of the results concerning the displacements at the door column intermediate joints (Fig. 7). In particular, the horizontal displacements of the door columns in Fig. 7 (23, 30, 31, 32) for the maximum applied top displacement in the negative direction (7.2 mm for step N = 9000) shows a larger displacement at both central LVDT's than at the top LVDT's (which, theoretically, should show the higher displacements). This larger absolute and relative displacement implies a larger damage concentration at the central area.

Finally, the compression forces draw cracks along the wall, which tend to follow the stones joints. These cracks delimit the above referred sliding/cut surfaces.

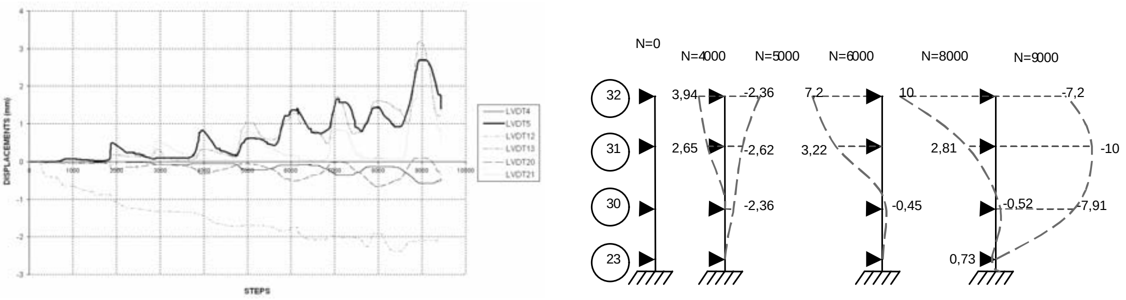


Figure 7 : Joints opening at columns and maxima horizontal displacements profiles.

3.5 LVDT 12 and 21

The behaviour of the spot 12 can be considered as unusual, comparing with the corresponding spot on the other side of the wall. The behaviour of this joint was in the opposite sense to the expected one. As the cyclical horizontal displacements were imposed, instead of opening (for displacements in the positive direction) and closing (for displacements in the negative direction), the joint displacement trend was always towards closure (almost 2,5 mm). Probably this was due to an accidentally damage before the test at the spot where the LVDT 12 was positioned (Fig. 8). Although the stones were reset into their original position, this measure was inefficient to reproduce the expected behaviour of the wall on that area; the existing link between the materials, as well as the contact between stones was lost. However, the joint at the spot 21 acted as expected, with a closing trend upon horizontal displacements applied in the positive sense and an opening trend upon horizontal displacements applied in the negative sense. This joint showed an opening of ~0,6 mm.

Figure 8 : Damaged area.

3.6 *Analysis of the Force - Displacement graphic*

As the force vs displacement curve in Fig. 9 shows, the masonry wall posses a good energy dissipation capacity and, furthermore, an envelope quite similar to a reinforced concrete element one. Notice that for displacements imposed in the positive direction, the wall loses resistance in successive loading cycles for the same displacement figure. In particular, from the 4 mm loading cycle to the 6 mm loading cycle, there was a local resistance loss at 4 mm of ~20%, which was recovered at the cycle end, i.e., for the 6 mm displacement. Furthermore, the last loading cycle shows that a non-recoverable lost of strength occurred, since the curve seems incapable of recovering the previous cycle strength capacity.

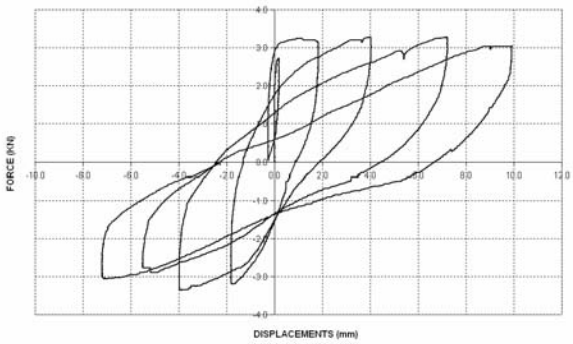


Figure 9 : Hysteretic loop.

Simultaneously, for the displacements on the other direction the curve shows lower energy dissipation. In fact, the wall was not equally loaded in both directions. However, if a symmetrical load had been performed, it would be expected similar dissipated energy in both directions, as well as similar strength at each displacement cycle in both directions.

The structure stiffness also decreased during cyclic loading, as it can be noticed by the progressive decreasing of the angle of the reload curves.

4 METHODOLOGY FOLLOWED TO BUILD THE NUMERICAL MODEL

4.1 *Methodology*

The wall was then simulated using finite elements method. In order to define the geometry underlying the numerical modelling it was necessary to make a geometric survey of the wall. The wall external outline was measured and the cover was removed to get a clear view of the main stones geometry. This survey was followed by the definition of the finite elements mesh. In this procedure a sequence of several auxiliary programs was used (AutoCad 2004, Solidworks 2004, GiD 7.5.0b and a program called BLOCO) (Costa, 2002). All these programs were necessary to convert the Autocad and Solidworks defined geometries into CAST3M Gibiane language finite elements program. Basically, CAST3M is a computer code using finite elements methodologies for structural analysis that was developed by the French Commission for Atomic Energy (CEA). The CAST3M is high-level tool for civil engineering investigation purposes and it integrates pre and post-processing functions (CEA, 2000).

In a first stage, wall geometry was defined using Autocad 2004 and it was saved in a DXF format file. However, the information contained in DXF format is not easily transferable to the CAST3M input file, which is based on a unique language called Gibiane (a set of commands, operators and objects that are internally interpreted by the CAST3M base code). To do so, two programd were used: SolidWorks 2004 and GiD. SolidWorks 2004 enabled the interpretation of the blocks geometry from Autocad and the definition of each block as a volume (3D solid) that later was saved in PARASOLID format. These files containing each block were later introduced in the same SolidWorks object, so that the whole wall geometry could be observed. From this initial geometry, it was necessary to carry out an iterative process, in order to get the most accurate approach to the wall geometric form comparing the geometry observed in Solidworks and the real geometry, as illustrated on Fig. 10. The GiD is a pre and post-processing program for numerical analysis based on the finite elements method. This program allowed the interpretation of PARASOLID format files from Solidworks as volume elements and save them in a file type used by program BLOCO. Finally, the auxiliary program Bloco was used to recover the information resulting from GiD and to transfer it into Gibiane format, in order to be interpreted by CAST3M (CEA, 2000).

In an initial stage, the stiff structure at the top of the wall was not simulated, in order to calibrate the material properties: the elastic modulus, the Poisson coefficient and the volume weight, by achieving the natural frequencies and comparing them to those obtained in the physical model in identical conditions (Pegon et al., 1996). The initial values were taken from tests perfomed in walls with similar characteristics (with stone blocks and infill), (Costa, 1999), and then adjusted in order to match the first two natural frequencies. In the link between elements (block to block or block to infill) joints with a Coulomb non-linear friction model without dilation (Pegon, 1999) were used.

Figure 10 : The 3D image of the wall in SolidWorks and the original wall.

After calibrating the model parameters, the stiff structure positioned at the top of the wall was introduced in the mesh. Regarding the non-linear models, it was decided to use a reinforced concrete type model for the infill (since no soil type model exists in CAST3M with the requested characteristics) and a trilinear shear/sliding law for the joints. The non-linear analysis was performed in two steps: first a monotonic load was applied with a maximum displacement of 10 mm at the top of the wall in order to reach the envelope of the experimental test; as a second step, a cyclic load similar to the experimental test one was applied with the variation of vertical load.

4.2 Numerical results

The hysterisis loop obtained in the numerical simulation is presented in Fig. 11. This figure also shows the stresses chart for the maximum displacement of the monotonic law. Fig. 12 represents the deformed shapes for maximum displacements in both directions for the cyclic loading. The results presented below are the most relevant in order to understand and further comparison with the experimental ones.

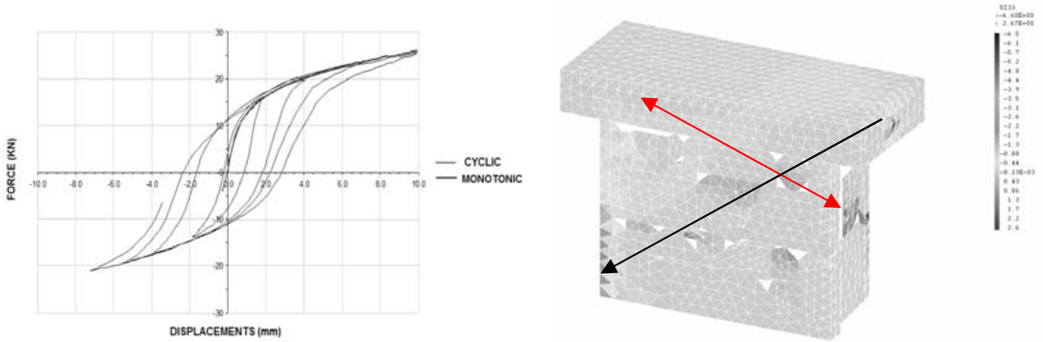


Figure 11 : Numerical horizontal top force vs top displacement loop for monotonic and cyclic load. Chart of the main compression stresses σ33 at the wall.

4.3 Analysis of the results

The analysis of Fig. 11 shows that the selected model provides good energy dissipation and a maximum resisting force of 26kN. On the other hand, from the stresses chart it is possible to assume that the load is transmitted from the top to the foundation through a strut and a tie.

Reviewing Fig. 12, the wall shows a different pattern depending on the direction in which the displacement is applied to the wall (positive\negative). For positive displacements, it is possible to observe a sliding behaviour due to the existence of preferential sliding surfaces characterized by low friction joints. These sliding surfaces follow the horizontal joints at two levels, represented in the two figures below. Moreover, most of the displacements occur at this two sliding levels. For positive displacements the transverse bending of the wall is almost zero. However, there is a slight vertical rotation of the wall, which means the centre of stiffness is not located along the line of the applied displacements.

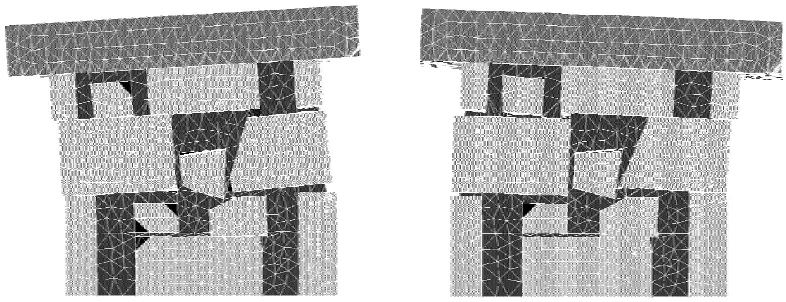


Figure 12 : Deformed shape at +10,0 mm and -7,2 mm during cyclic loading.

For negative displacements due to a lack of stiffness in the upper right corner (in bird view), the wall bends in the transverse direction, weakening the links between the elements. In this case, the majority of the displacements occur along the upper sliding surface.

4.4 Comparison with the experimental results

By comparing the experimental results with the numerical ones, it is possible to conclude that the model used to characterize the infill was inappropriate, showing lower stiffness than the one observed at the physical model (Fig. 13). Besides, some properties typical of such kind of walls, such as the link between the materials, could not be reproduced in the numerical model. However, values such as the peak force or the horizontal displacement at two thirds of the wall height are quite close to those achieved during the experimental test. These values show that more accurate results could have been obtained if another type of model had been used to characterize the infill. On the other hand, the numerical model managed to reproduce the rotations and the horizontal displacements in height of the wall during the test. Besides, the energy dissipation and the forces transmission mechanisms, the sliding surfaces and the blocks movements were sufficiently well reproduced and located through this model too.

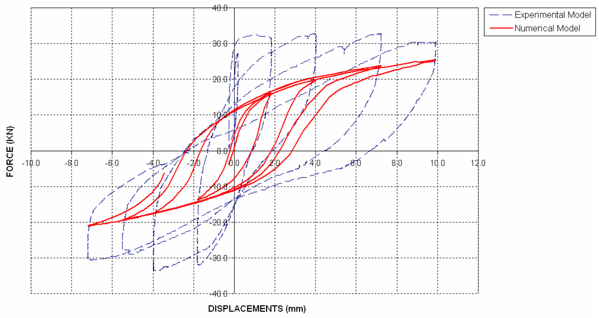


Figure 13 : Numerical and experimental horizontal top force vs top displacement curves.

5 CONCLUSIONS

The phenomena's detected during the experimental test, due to the imposed action were also detected on the adopted numerical model. So it can be concluded that the outcome of the numerical model is reasonable, considering the complexity of the adopted model. However, further work should be carried out to better simulate the infill behaviour, by exploring other type of models or even creating a new mode. Furthermore, new solutions should be pursued to introduce other phenomena in the models, which were not considered in this work but have a strong influence in the wall final behaviour; for instance, the link between normal stress force in the horizontally or vertically positioned joints between blocks and infill material.

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REFERENCES

Almeida, C. (2000) – "Análise do Comportamento da Igreja do Mosteiro da Serra do pilar sob a Acção dos Sismos", Tese de Mestrado em Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal.
CEA(1990) – CASTEM 2000, Guide d'utilisation, CEA, Saclay, França.
Costa, C. (2002) – "Análise do Comportamento da Ponte da Lagoncinha sob a Acção do Tráfego Rodoviário", Tese de Mestrado em Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal
Costa, A. G. (1999) – "Ensaios de Caracterização de Alvenarias Tradicionais", Secretaria Regional de Habitação e Equipamentos, Açores, Portugal.
Glória, D., (2004) – "Relatório das características técnicas das Células de Carga 09,10,11,12", LESE, Porto, Portugal.
Faria, R. (1994) – "Avaliação do Comportamento sísmico de Barragens de Betão através de um Modelo de Dano Contínuo", Tese de Douturamento em Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal.
Pegon, P. (1999) – "Automatic Generation of Blocks Connected with Joints in CASTEM 2000", Special Publication No. I.99.101, ISIS, SMU, JRC, Ispra (VA), Italia.
Pegon, P. and Pinto, A.V. (1996) – *Seismic Study of Monumental Structures – Structural Analysis, Modelling and Definition of Experimental Model*, Report EUR 16387 EN, ISIS, SMU, JRC, Ispra (VA), Italia.