Paper 114

Proceedings of the Twelfth International Conference
on Computational Structures Technology,
B.H.V. Topping and P. Iványi, (Editors),
Civil-Comp Press, Stirlingshire, Scotland.

In-Plane Shear Behaviour of Stone Masonry Piers: A Numerical Study

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Abstract

Post-earthquake investigations have shown that if out-of-plane mechanisms are prevented, the seismic performance of a masonry building depends mainly on the in-plane capacity of spandrels beams and especially piers. For this reason, several investigations were done in the past to characterize the in-plane behaviour of masonry walls. A large majority of these studies consist of experimental programmes, testing the lateral response of piers. Nevertheless, very few studies focused on carrying out numerical studies and their potential was disregarded.

A numerical investigation to study the in-plane behaviour of masonry piers was carried out, based on an experimental campaign performed on stone masonry piers. The experimental programme included masonry piers with two slenderness ratios subjected to two distinct levels of axial compression. Finite element models were built on the advanced software, DIANA, and according to the experimental setup test of each wall, with the aim of simulating the experimental tests. Afterwards, the non-linear numerical simulations were compared against the in-plane cyclic test results. The calibration and validation of the numerical models according to the experimental results was conducted. The results of the non-linear analyses carried out on the validated models are presented and discussed. Good agreement between experimental and numerical results was achieved both considering the force-displacement behaviour and failure mechanisms. The numerical strategy can be seen as a complementary way to study masonry piers, particularly useful for further parametrical studies.

Keywords: in-plane behaviour, masonry piers, finite element method, numerical modelling, non-linear analysis.

1 Introduction

As previous earthquakes have shown, masonry buildings are vulnerable to seismic horizontal actions that usually produce widespread cracking and local collapses. When a masonry building is subjected to seismic loads, the resisting elements are the in-plane walls (walls parallel to the direction of ground motion), out-of-plane walls (walls perpendicular to the direction of ground motion), and floor diaphragms. An effective connection between these elements can result in an overall response of the structure since it is capable of transferring forces from the out-of-plane walls to the in-plane walls, with an important role played by the floor and roof diaphragms in transferring loads and coupling the in-plane displacements of the different walls. During an earthquake both out-of-plane and in-plane responses are simultaneously mobilized. The characteristic type of damage in structural walls identified by the analysis of post-earthquake surveys is schematically presented in Figure 1.

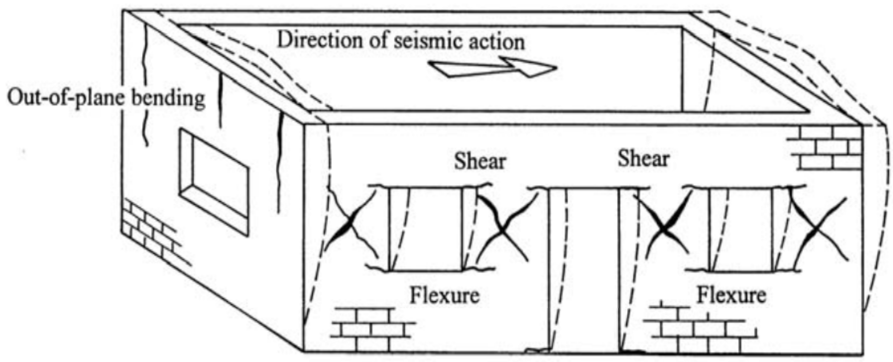


Figure 1: Typical damage in structural walls [1].

A satisfactory seismic behaviour is attained if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited [2]. Walls subjected to compressive and shear loading experiment both shear and flexural behaviour, to which different failure modes are associated: rocking and toe crushing (associated to flexural) and sliding and diagonal cracking (associated to shear). The mobilization of the different modes of failure due to seismic action depends on several parameters: geometry of the wall (aspect ratio); boundary conditions; acting axial load; mechanical properties of masonry, including relative strength between mortar and units and masonry pattern (both in-plane and cross-section) [3], [4]. In the last years several experimental tests have been performed with the aim of analysing the influence of these parameters on the failure mode of masonry walls.

This study intends to provide contributions to the characterization of the in-plane behaviour of masonry walls using the numerical approach (properly validated) as a complementary option to experiments. The basis for this numerical study was the experimental campaign performed at the EUCENTRE and University of Pavia on stone masonry piers [5], [6].

2 Experimental Tests

A comprehensive experimental campaign on undressed double-leaf stone masonry walls was carried out at EUCENTRE and University of Pavia laboratories by Magenes et al. [7] and Galasco et al. [6]. This type of masonry is widely found in Italian ancient buildings. In-plane cyclic shear tests were carried out on four walls in order to study their in-plane behaviour. Two distinct geometries for two compressive stress levels were studied. Within the experimental framework, simple axial compression and diagonal compression tests were performed in order to characterize the mechanical properties of the undressed double leaf stone masonry. Table 1 summarizes the results of the characterization tests. Further information regarding the materials and construction characteristics is reported by Magenes et al. [5].

	f_m [MPa]	E [MPa]	f_t [MPa]	G [MPa]
Range	3.07 – 3.48	2274 - 2826	0.11 – 0.16	740 - 940
Mean	3.28	2550	0.14	840

Table 1: Mechanical properties of masonry (tests on 6 specimens).

The cyclic shear tests aim at representing the behaviour of masonry piers when subjected to in-plane horizontal load cycles, such as those induced by earthquakes. The in-plane shear behaviour of two geometric configurations for the compressive stress levels of 0.2 MPa and 0.5 MPa was experimentally evaluated. All specimens have a total nominal thickness of 0.32 m and 2.5 m high. Two of them are 2.5 m long, which are named “squat” piers (CT01 and CT02) and other two are 1.25 m long, named "slender" piers (CS01 and CS02). Figure 2 presents the geometry and applied vertical load of the piers. The walls were subjected to a cyclic horizontal displacement imposed at the top, keeping constant the compression load. The test setup imposes a fixed-fixed condition to the walls. The experimental force-displacement curves and corresponding envelopes are presented in Figure 3 for each pier type.

Experiments revealed that piers CT01, CT02 and CS01 fail in shear with the formation of diagonal cracks. Specimen CS02, a slender pier characterized by low compression level, showed a combined (hybrid) flexural and shear failure. As expected, the piers subjected to higher compressive stress states proved to have more capacity in terms of horizontal load, when compared with the ones with the same geometry but lower level of compression. It was also observed that the geometry (more specifically the h/l slenderness ratio) influences as well the in-plane behaviour of the masonry piers, reaching levels of horizontal force considerably lower.

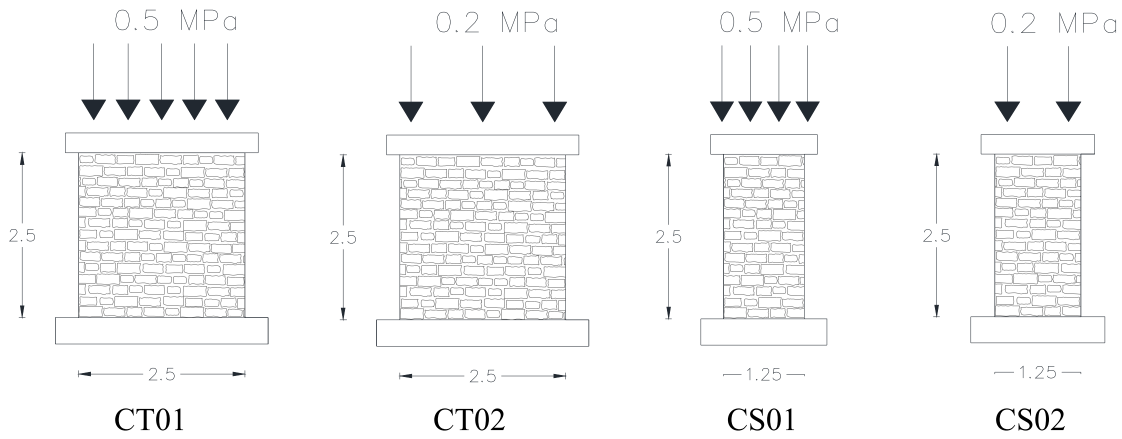


Figure 2: Panels for the in-plane cyclic shear tests.

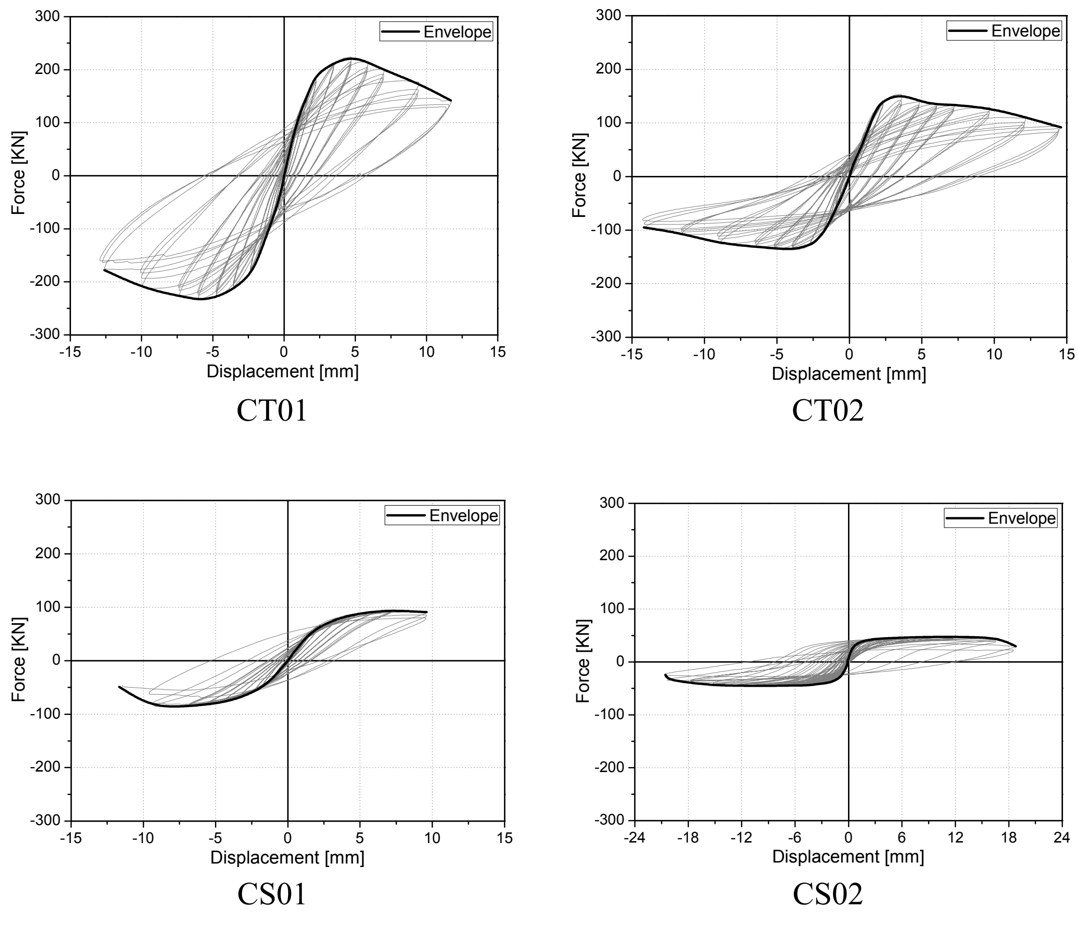
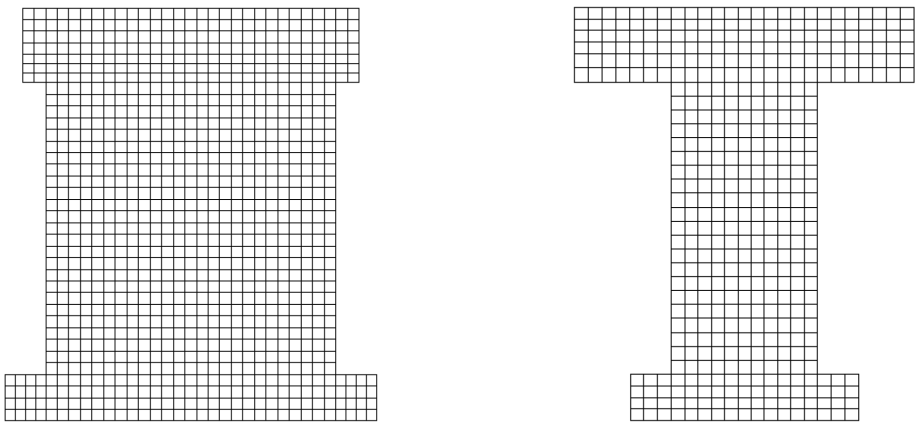


Figure 3: Force-displacement curves.

3 Numerical Study

3.1 Models

Aiming at reproducing the in-plane behaviour of the four piers experimentally tested, two numerical models were constructed with different geometrical configurations. The numerical models were constructed in DIANA [8] and include the bottom concrete beam, the masonry wall, the top concrete beam and the top metallic beam, trying to accurately reproduce the experimental test setup. Both are 2D models with eight-node quadrilateral plane stress elements (CQ16M) based on quadratic interpolation and Gauss integration. The numerical model for the CT walls is presented in Figure 4a and the generated mesh has 1686 nodes and 534 elements. The model for CS walls is shown in Figure 4b and has 1472 nodes and 449 elements. The numerical model assumes that the different components (wall and beams) are fully connected, since no significant sliding occurred during the experimental tests for any wall. Regarding the boundary conditions, a numerical fixed-fixed condition was assured in both models.



(a) (b)

Figure 4: Numerical models (mesh): (a) CT walls and (b) CS walls.

The total strain rotating crack model approach was used for the nonlinear analysis with a parabolic behaviour in compression and softening behaviour in tension for masonry (see Figure 5). The parameters for the definition of the masonry behaviour were defined in accordance with experimental characterization results and typical relations used for masonry [9]. Table 2 presents the initial adopted values for the masonry mechanical parameters. Typical values for the elastic modulus, poison coefficient and density were taken for the concrete and metallic beams. A Newton–Raphson iteration procedure was used in all the analyses with an energetic convergence criterion with a tolerance of 1E-3. The procedure to perform the non-linear analyses began with the application of the total vertical pre-compression load on the top of the metallic beam, followed by the monotonic application of the horizontal load, in terms of incremental displacements, at the left side of the metallic beam.

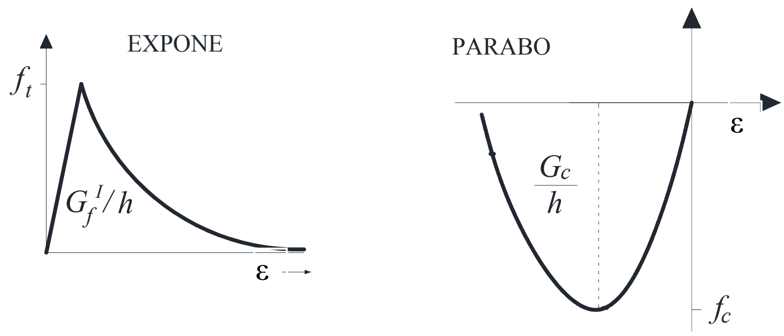


Figure 5: Constitutive models for masonry (tension and compression).

E [MPa]	f_c [MPa]	G_c [N/mm]	f_t [MPa]	G_f^I [N/mm]
2550	3.28	5.25	0.14	0.02

Table 2: Mechanical properties for masonry.

3.2 Validation

The validation of the numerical models is carried out based on the experimental results from the in-plane walls. For this purpose, preliminary non-linear analysis using the material properties defined in Table 2 were carried out. The obtained numerical force-displacement curves were compared against the respective experimental envelope (attained by overlapping the envelope curve in each "positive" and "negative" directions). Figure 6 presents the results for the four walls (CT01, CT02, CS01 and CS02). As can be verified by the analysis of the capacity curves, the stiffness of the numerical models seem to be very high when compared to the experimental results.

The differences between the numerical and experimental stiffness raised some questions regarding the parameters that influence the linear behaviour of these walls. It was verified that the numerical linear stiffness does not depend on the installed compression stress. For instance, varying the pre-compression level and maintaining the same material properties, walls CT01 and CT02 present the same numerical linear stiffness (around 215 kN/mm; the same for walls CS01 and CS02 with a stiffness around 59 kN/mm). Conversely, experimental tests show different linear stiffness depending on the compression stress level. For the same geometric configuration, walls CT01 and CT02 with distinct levels of compression show different linear stiffness values, which also occurs for walls CS01 and CS02 (see Figure 3). This demonstrated numerical stiffness independence from the compression level is a feature of almost advanced numerical tools and thus requires the definition of an equivalent elastic modulus.

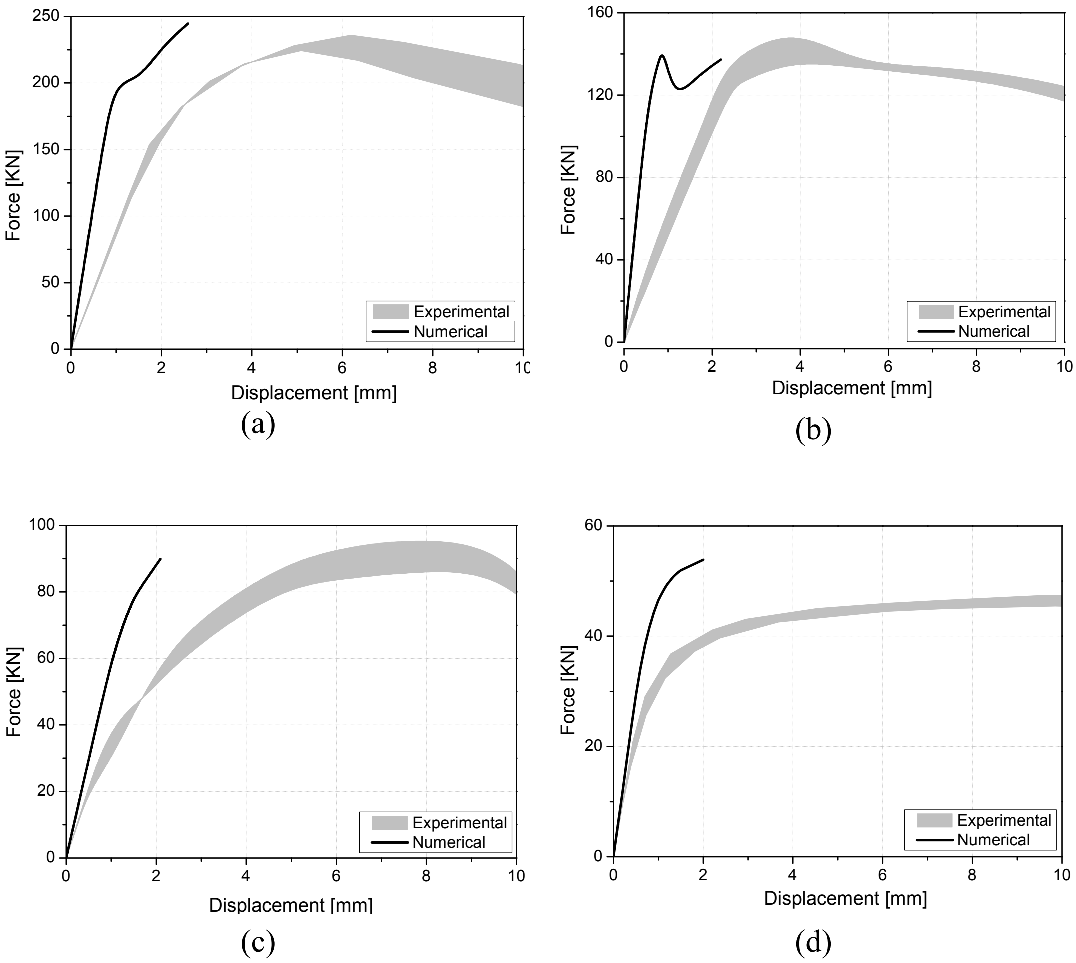
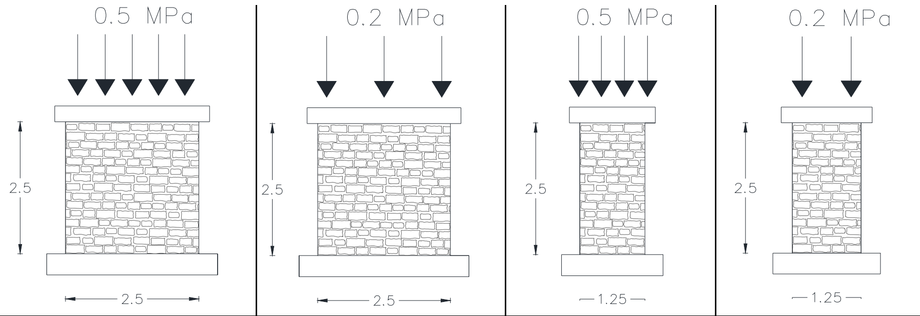


Figure 6: Force-displacement curves: (a) CT01 wall; (b) CT02 wall; (c) CS01 wall; (d) CS02 wall.

Numerically, the wall linear stiffness depends on the masonry elastic modulus, the pier geometric configuration and boundary conditions. Therefore the equivalent masonry elastic modulus needs to be adjusted in order to fit the experimental linear stiffness. By the analysis of other experimental studies on the in-plane behaviour of masonry walls available in literature [10]–[17], it was verified that the linear wall stiffness depends, indeed, on the pre-compression level. This can be justified by the fact that the rearrangement of the masonry units, as a consequence of a higher imposed compression on the wall, result in a more stiff material.

Thus, numerical analyses were carried out in order to calibrate the linear stiffness for each wall and the results are summarized in Table 3. The differences in the elastic modulus in what concerns the two different geometric configurations (CT and CS) can be explained by the in-situ wall construction conditions. As CS walls are smaller maybe a higher care has been put in the arrangement of the units during the construction.



	CT01	CT02	CS01	CS02
E [MPa]	**1000**	**800**	**1500**	**1100**

Table 3: Calibrated linear stiffness for each pier.

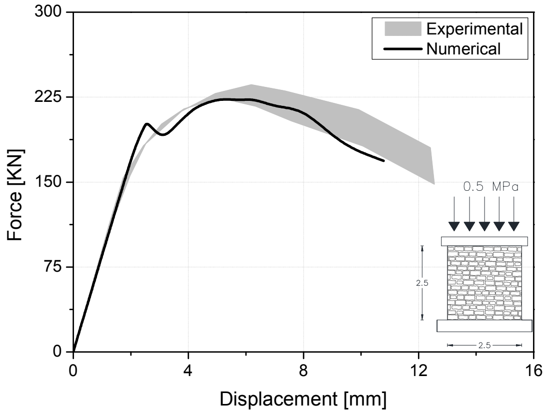
3.2 Non-linear Numerical Analysis

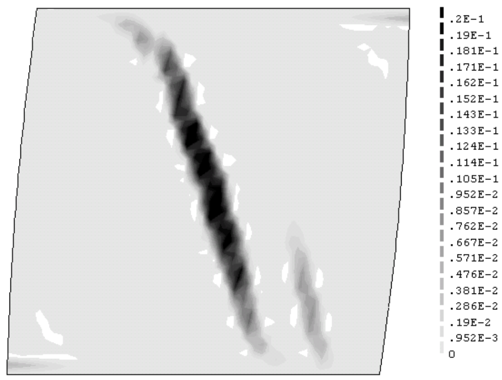
Non-linear analyses were carried out considering the calibrated elastic modulus for each wall, maintaining the non-linear properties equal for all masonry walls. The results are presented both in terms of capacity curve and principal tensile strains distribution as an indicator of damage (see Figure 7 for CT01 wall; Figure 8 for CT02 wall; Figure 9 for CS01 wall and Figure 10 for CS02 wall).

The comparison between the numerical and experimental failure modes and the monotonic envelops of the force–displacement diagrams shows very good agreement with the experimental results. It is observed that for CT01 wall the force–displacement numerical curve fits the experimental monotonic envelope very well, both in terms of initial stiffness, resistance and post peak behaviour (Figure 7). The sudden load variation visible in the capacity curve can be explained by the opening of the first crack in the model, followed by the subsequent stress redistribution to adjacent elements. Failure in this wall is governed by diagonal cracking that develops between the top corner and the wall toe.

The wall with lower compression level (CT02) presents also a good agreement with the experimental results in terms of force-displacement curve. In particular, the maximum horizontal load achieved numerically simulates very well the experimental wall capacity. The shear failure described in the experiments is confirmed by the numerical damage distribution, which shows a clear formation of diagonal cracks at the centre of the pier (see Figure 8b).

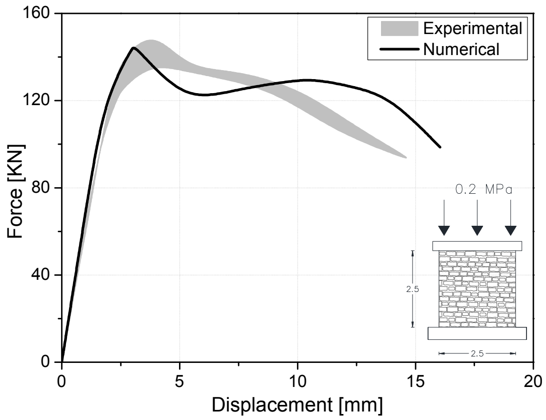
The results for CS01 wall show very good agreement against experiments in terms of force-displacement, both in what concerns the maximum capacity and post-peak behaviour. Here, the deformed shape and damage distribution are also in accordance with the failure mode verified in the tests. The shear behaviour is predominant with diagonal cracks in the centre of the pier.

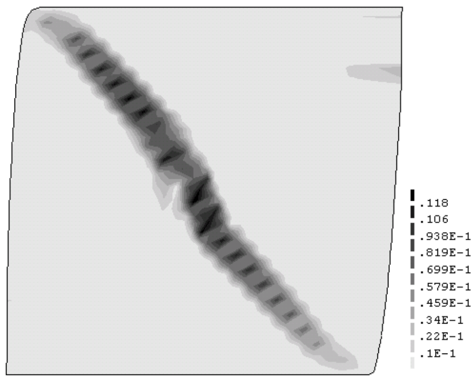




(a) (b)

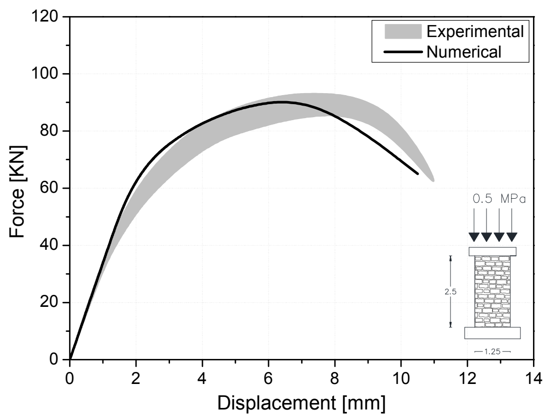
Figure 7: Numerical results for CT01 wall: (a) Force-displacement curve; (b) Principal tensile strains distribution in the post peak.

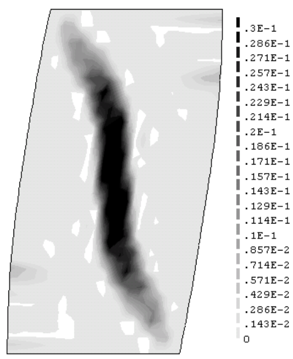




(a) (b)

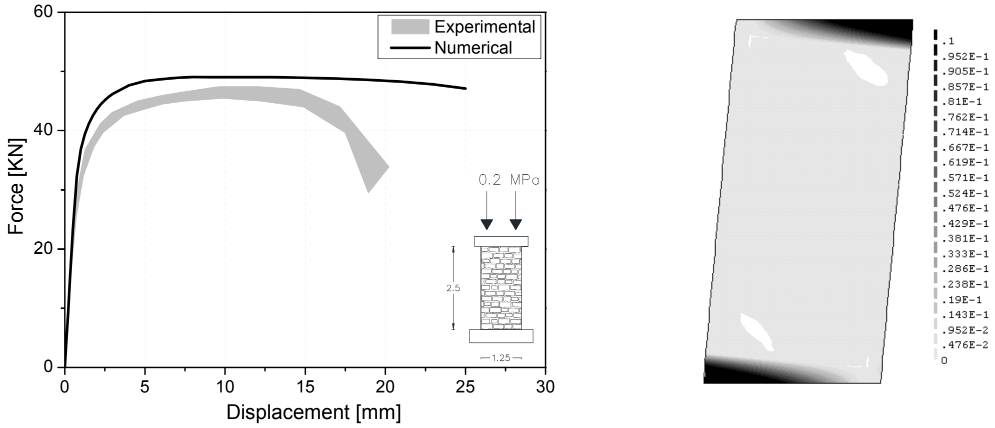
Figure 8: Numerical results for CT02 wall: (a) Force-displacement curve; (b) Principal tensile strains distribution in the post peak.





(a) (b)

Figure 9: Numerical results for CS01 wall: (a) Force-displacement curve; (b) Principal tensile strains distribution in the post peak.



(a) (b)

Figure 10: Numerical results for CS02 wall: (a) Force-displacement curve; (b) Principal tensile strains distribution in the post peak.

Finally, the numerical curve for wall CS02 is in agreement with the experimental envelope, although the peak force is slightly overestimated, and the failure mode is captured. The final branch of the numerical curve is typical from rocking behaviour and convergence is not attained after 25 mm. The analysis of the compressive stress distribution of the pier (not presented here) shows a high concentration in the diagonal direction until the toe.

Hence, the analysis of the numerical results compared against experimental data proved that the proposed numerical approach is capable of simulating correctly the behaviour of these walls in terms of stiffness, load capacity and failure modes.

4 Final Considerations

Aiming at simulating the in-plane behaviour of masonry walls experimentally tested, finite element models were constructed. The numerical models were studied and compared against the experimental in-plane cyclic test results. An initial difference in terms of linear stiffness was verified for all the walls. This is due to the fact that even advanced numerical models are not able to account for the compression stress state influence on the lateral stiffness. In order to properly simulate the influence of the pre-compression level in the walls behaviour, the masonry elastic modulus was calibrated to fit the experiments.

The results of the non-linear analysis regarding the validated model were presented and discussed. A good correspondence between numerical and experimental responses has been found for all the walls, proving the potential of the presented numerical strategy to simulate the in-plane behaviour of masonry walls with accuracy. This numerical strategy can be seen as a complementary way to study masonry piers, particularly useful for parametrical studies after model validation. For future work the aim is to extend the study of masonry piers to other configurations and stress levels.

Acknowledgments

The first author acknowledges the financial support from the Portuguese Science Foundation (Fundação de Ciência e Tecnologia, FCT) through grant SFRH/BD/71599/2010.

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