MASONRY WALLS SUBMITTED TO OUT-OF-PLANE LOADING: EXPERIMENTAL AND NUMERICAL STUDY

Tan Trung Bui, Ali Limam, Emmanuel Ferrier, Michael Brun, David Bertrand

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BUI, T. T[1]; LIMAM, A[2]; BERTRAND, D[3]; FERRIER, E[4]; BRUN M[5]

ABSTRACT:

This paper presents an experimental investigation conducted on masonry walls subjected to an out-of-plane loading or normal pressure. This study has multiple objectives. It allows, first to quantify the bearing capacity in the case of uniform pressure in quasi-static loading case, and thus to highlight the associated modes of rupture, and secondly to estimate the improvements in terms of global behaviour when the structure is reinforced by Carbon Fiber Reinforced Polymer (CFRP) layers. A gain higher than 140% is observed for the bearing capacity. Finally, numerical simulation with the discrete element method is conducted and compared to the experiments. Good agreement between the numerical and the experimental results is obtained.

Keywords: *Masonry wall, CFRP, reinforcement, discrete element method, lateral pressure*

1 INTRODUCTION

Masonry infill panels can be generally regarded as non-structural secondary elements or partially structural, when they are subjected to out-of-plane loading. The out-of-plane loading can be due to an overpressure due to blast effect induced by an explosion or the overpressure induced by impacts from a snow-avalanche if we consider the habitation in a mountain area, or more generally the effect of extreme wind. The objective of our study is to examine the rupture mechanisms and the behaviour of masonry walls subjected to a normal pressure. This preliminary stage allows us to determine adequate reinforcement in order to increase the bearing capacity substantially. Many studies have shown the relevance of composite materials for the reinforcement of concrete structures [1]. This concept was then extended to the reinforcement of glued-laminated wooden beams [2] or metallic shells subjected to seismic loading [4][4]. Lately, several studies have been conducted on evaluating the use of CFRP for strengthening both the unreinforced and reinforced masonry walls subjected to in-plane loading or out-of-plane bending. This study was conducted to demonstrate the pertinence of such a reinforcing system for structures subjected to lateral pressure load.

2 TEST PROGRAM AND TEST SETUP

2.1. Test program

1) PhD student, INSA Lyon, Civil Engineering, tan-trung.bui@insa-lyon.fr
2) Professor, INSA Lyon, Civil Engineering, ali.limam@insa-lyon.fr
3) Associate Professor, INSA Lyon, Civil Engineering, david.bertrand@insa-lyon.fr
4) Professor, INSA Lyon, Civil Engineering, emmanuel.ferrier@univ-lyon1.fr
5) Associate Professor, INSA Lyon, Civil Engineering, michael.brun@insa-lyon.fr

To reproduce in-situ conditions as closely as possible, a mason and his assistant were hired for constructing the specimens. The test specimens intend to represent a portion of a typical load-bearing wall including corners, like in a low-rise building, or an individual house. As can be seen in Figure 1, for the three walls tested, the top remains free, while the bottom and two sides are supported. The geometrical dimensions of the wall, the nature of the units (standard hollow concrete blocks), as well as the presence of two return walls and of a reinforced concrete slab at base, respect the standards of construction. The most common construction materials are used, from the standard hollow concrete block to industrial ready-mix and bag-conditioned mortar for the joints. The representative structure is a main wall 2.9x2x0.2m, subjected to an out-of-plane loading, and two internal or return walls on both edges 1.0x2x0.2m for specimen 1 and 1.5x2x0.5m for specimens 2 and 3 (Figure 1). The lower bottom edges of the masonry walls were mortar bonded to the concrete slab. The mock-ups include horizontal steel reinforcement (3 bars with a 6mm diameter) at top end into the top bond beam, and vertical steel reinforcement (2 bars with a 12mm diameter) at each corner, in accordance with the constructive process and standards. Only the units containing steel reinforcement were filled with concrete. The sizing of the steel rebar was calculated by considering the construction recommendations for the habitation in "blue zone" (mountain area), that is to say the ability to withstand a pressure induced by a snow-avalanche, of 300mbars.

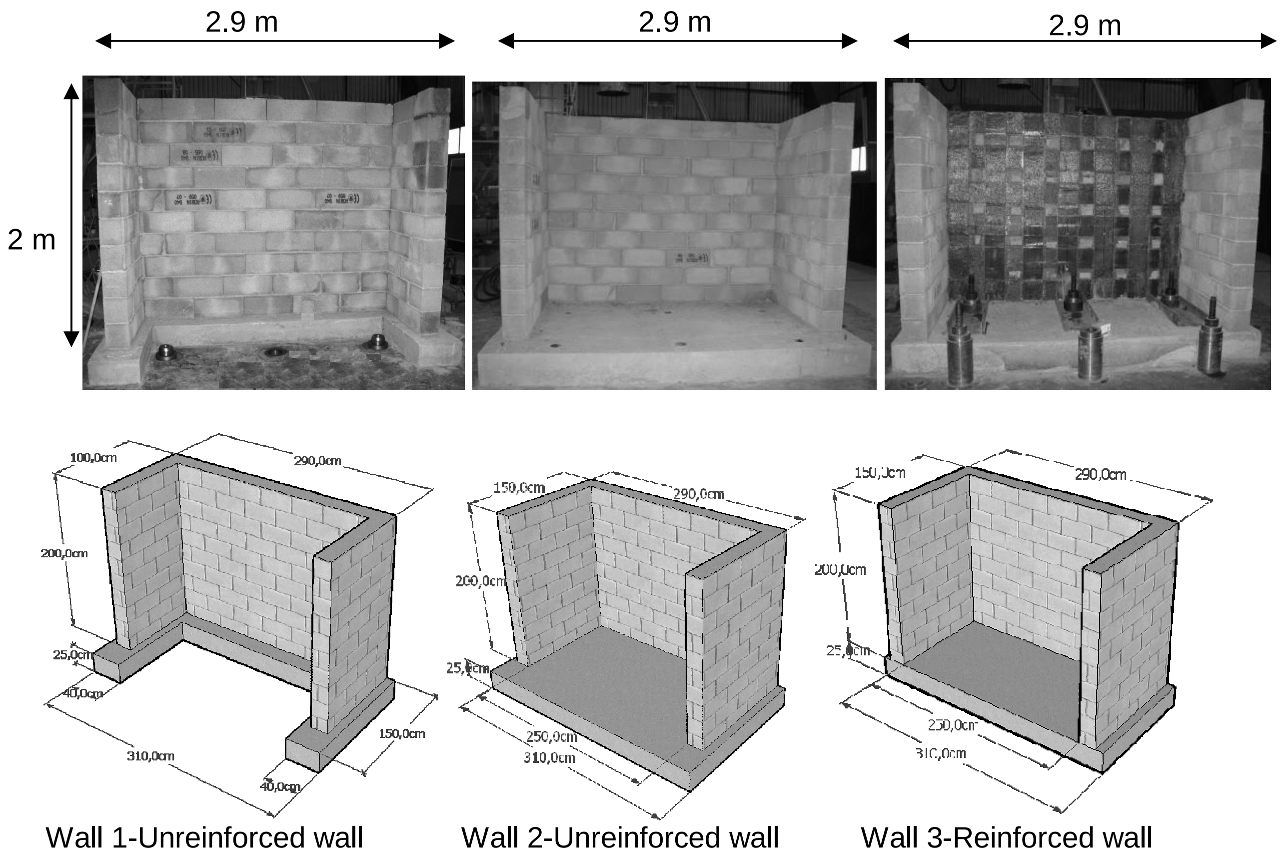

Figure 1. Unreinforced (wall 1 & 2) and reinforced masonry wall (wall 3). Specimens 1, 2 and 3

2.2. Test setup

The main wall is subjected to a quasi-static loading of uniform pressure applied to the outside face, using six inflatable cushions or water-bags. The reaction wall (a reaction frame) consists of a set of metal beams HEB, anchored on the test slab of the laboratory by pre-stressed steel bars (Figure 2).

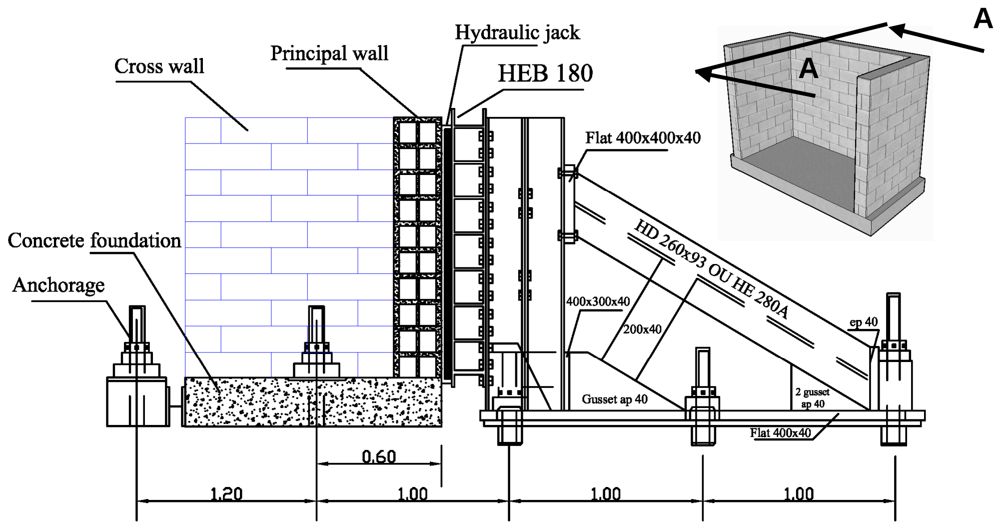

Figure 2. Laboratory test arrangement according to section AA.

The arrangement of the water-bags is shown in Figure 3:

Figure 3. Water-bags on the north side and transducers positioned on the south side of the wall.

The out-of-plane displacement of the structure was measured using linear variable differential transducers (LVDT). Nine transducers were placed on the main unreinforced wall; three transducers were used to measure the displacement of the foundation. A calibrated pressure transducer was used to control and measure the applied water pressure. The positions of the transducers for each wall are indicated in Figure 3.

3 MATERIALS

A standard mortar (ready-mix mortar) was used in the construction of the masonry panels. Prismatic mortar specimens (dimensions 4x4x16 cm) were made to determine mechanical characteristics such as the compressive and the flexural strength. Each of these mortar specimens was tested right before the test was conducted on the structure. The average compressive strength and the mean flexural tensile strength obtained by testing 24 specimens were 16.5MPa (coefficient of variation CV=0.15) and 3.6MPa (CV=0.16), respectively. The masonry units used were hollow concrete blocks of classification B40 in accordance to NF P 14-402. The nominal dimensions were 20cm height, 50cm length and 20cm width. The uniaxial compression tests and masonry prism tests resulted in mean compressive strength, of 12.27MPa (CV=0.07) and of 14.61Mpa (CV=0.01) respectively, based on the net gross cross-sectional area. The concrete used for the reinforcement (the grout that was filled where the horizontal and vertical reinforcements were placed) and the slab support were C40/45 type.

Compressive strength tests were carried out on 11cm diameter x 22cm high cylinders, the average stress obtained 43.18MPa is in adequacy with that awaited by the formulation. For the CFRP strips, tensile tests were conducted according to ASTM D638. As certified by the technical notes of the manufacturer, the average tensile strength and elastic modulus of this carbon laminate were respectively 1700MPa and 105GPa. The application was executed with expert workmanship, following the recommended application procedure.

4 TEST RESULTS

4.1. Load-deflection curves

A reference test of the unreinforced configuration was first conducted to determine the bearing capacity and the modes of rupture. The pressure/deflection curve (Figure 4) shows quasi-linear behaviour up to a pressure intensity of about 270mbars, which corresponds to the onset of crack formation. A redistribution of the stresses however allows the load to increase until failure. The maximum pressure reached was 440mbars, and the maximum associated displacement, obtained in the central zone of the wall, was 17mm. The post-peak behaviour was characterized by a drop in the bearing capacity, but after this drop an increase of the load was observed until a plateau was reached. To avoid wall collapse, the loading process was stopped when the displacement reached 50mm (obtained by transducer 4 located at the middle of wall).

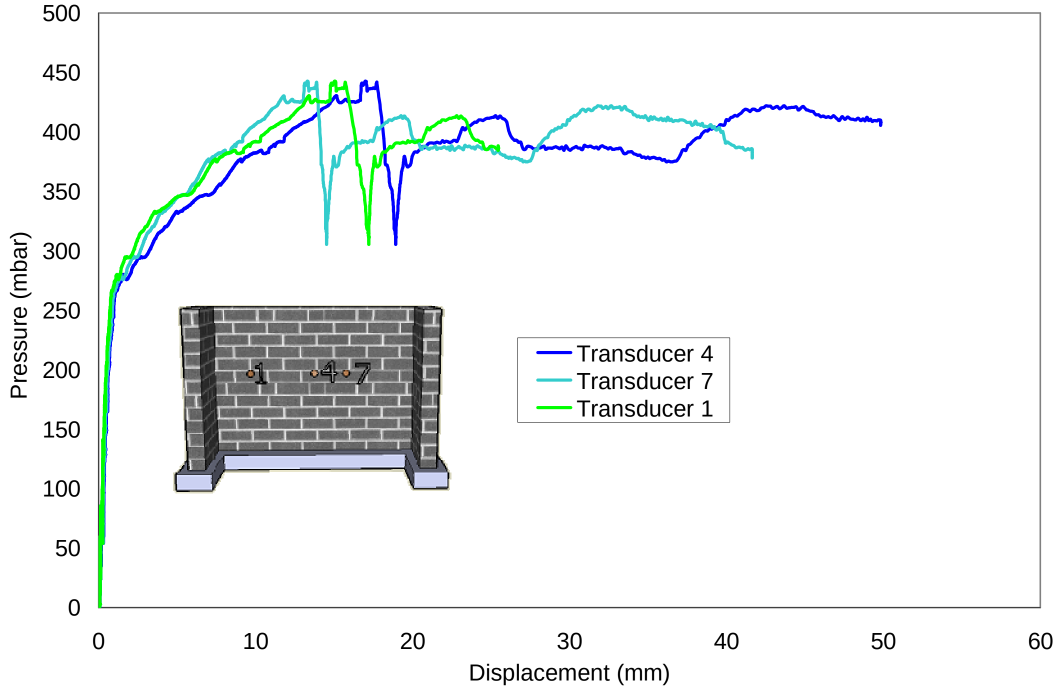

Figure 4. Experimental pressure/deflection curve obtained for reference wall

With the aim of ensuring the boundary condition throughout the test, a second unreinforced wall was tested with a rectangular slab support measuring 310×185×25cm^3 (length×width×height) instead of the slab with the shape in “U” 310×120×20 cm^3, under similar conditions of loading. In this case, the pressure associated with crack appearance, visible to the naked eye, is higher, 340mbars instead of 270mbars (Figure 5). Furthermore, a redistribution of the stresses also allowed an increase in loading to a maximum value of 580mbars, associated with a displacement of 26.7mm in the central zone of the wall.

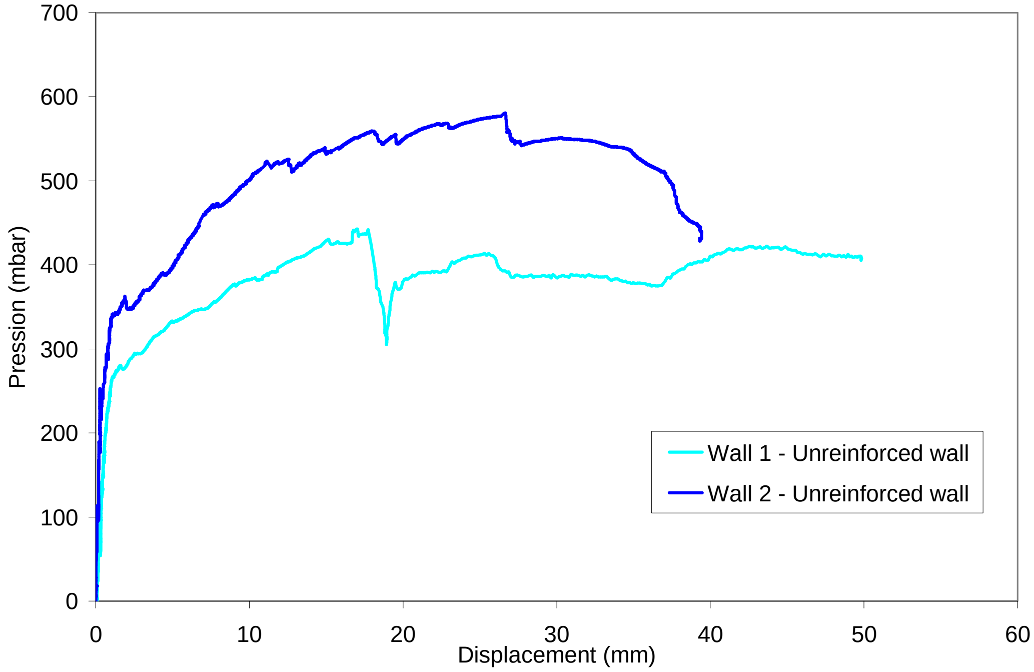

Figure 5. Comparison between two unreinforced walls

The third wall was identical to wall 2 but reinforced with vertical and horizontal CFRP strips (Figure 6). It was tested under similar conditions of boundary condition and loading as the second unreinforced wall. The carbon layers were bonded to the wall surface with an epoxy resin using the wet lay-up procedure.

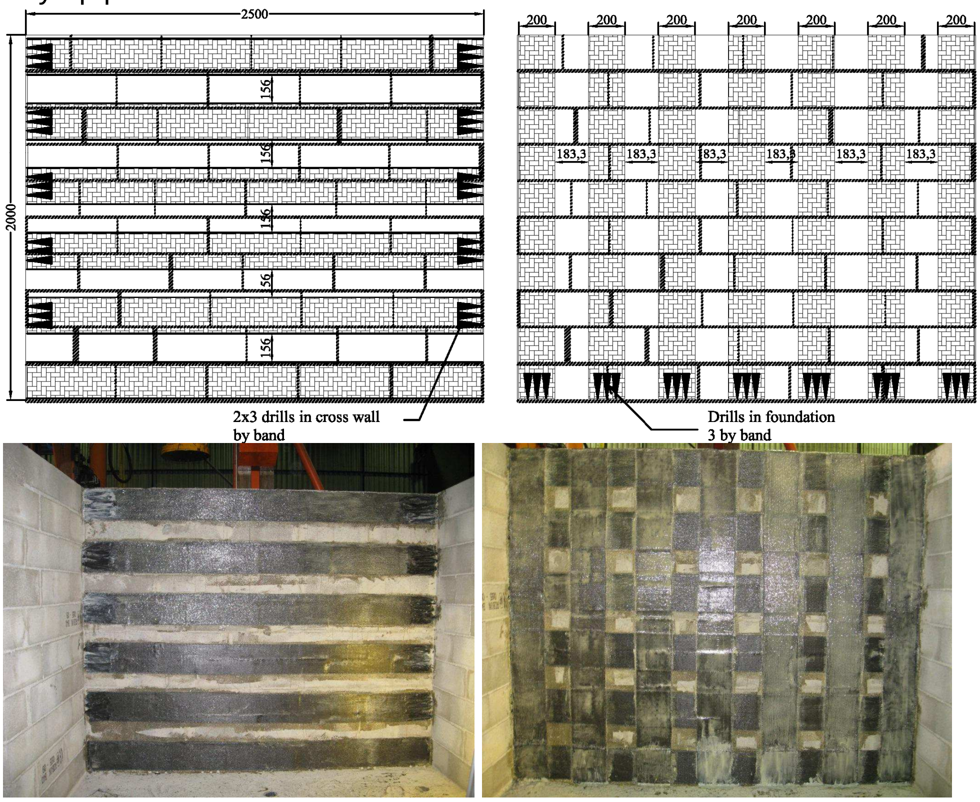

Figure 6. Horizontal and vertical reinforcement of the wall

The reinforcement consisted of seven vertical CFRP strips, 20cm width and 2m length, and six horizontal CFRP strips, with the same size as for the vertical ones. In order to avoid debonding of the CFRP strips and to ensure the effectiveness of the reinforcement to reach the post-peak behaviour (collapse), anchoring drills were installed on the horizontal and the vertical strips (Figure 6). The reinforcement considered, allowed a substantial gain of the bearing capacity, 1410mbars compared to 580mbars obtained for the second unreinforced wall. The gain is 143%. In addition to this enhancement of the bearing capacity, a clear increase of the stiffness was obtained in the cracked phase (Figure 7). The pressure associated with crack appearance was about 800mbars instead of the 340mbars obtained for the unreinforced configuration (wall 2). The gain is 135%, and even more, knowing that these cracks appeared mainly in the interior walls. Thus, CFRP is able to bridge the cracks, and completely inhibits their opening on the main wall.

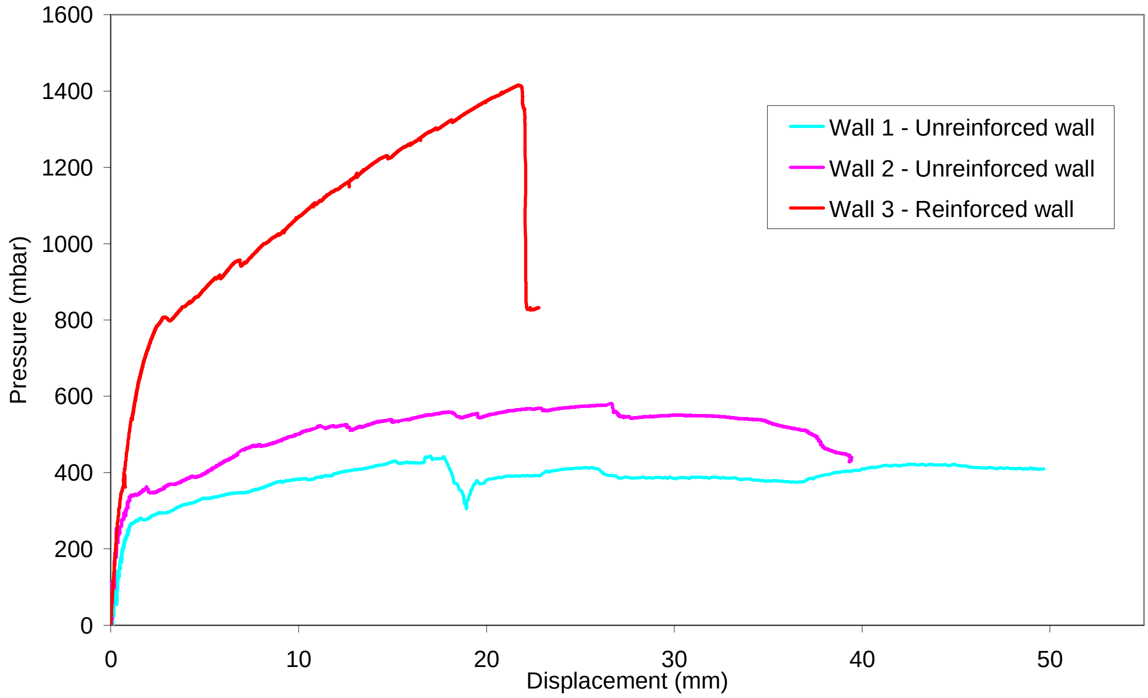

Figure 7. Comparison between unreinforced and reinforced wall

4.2. Failure patterns

In the case of the two unreinforced walls, two types of cracks were observed on the main wall (Figure 8 and Figure 9): first of all the vertical cracks which are characteristic of the bending loading. These cracks appeared near the centre of the wall, more or less at the midline in the upper part of the wall, which is unsupported. The second type of diagonal cracks formed in the lower part running from each corner. These crack patterns are similar to those predicted by the yield line theory adapted from that for reinforced concrete slabs.

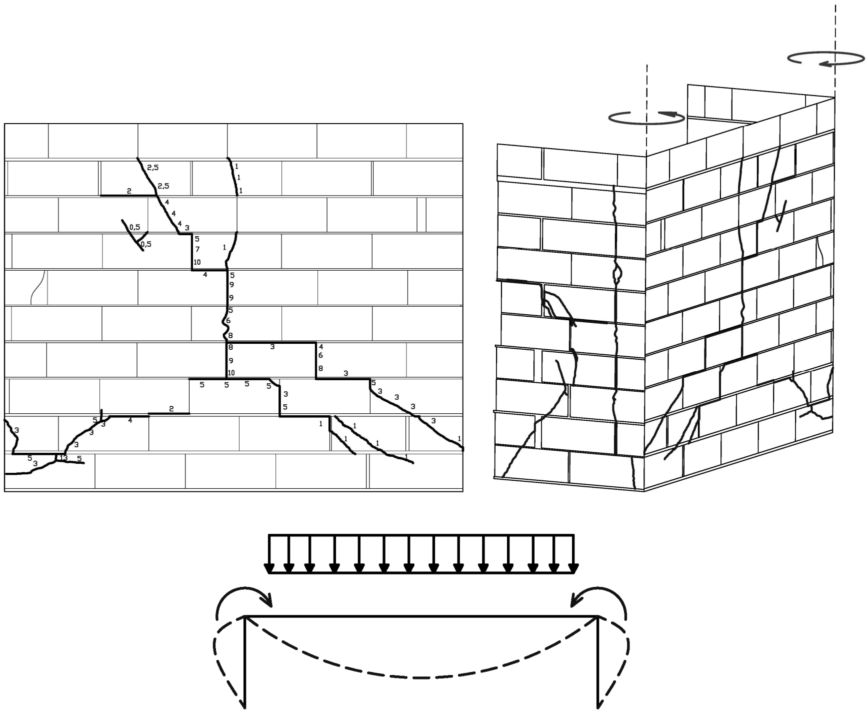

Figure 8. Wall 1- Failure modes of the principal wall

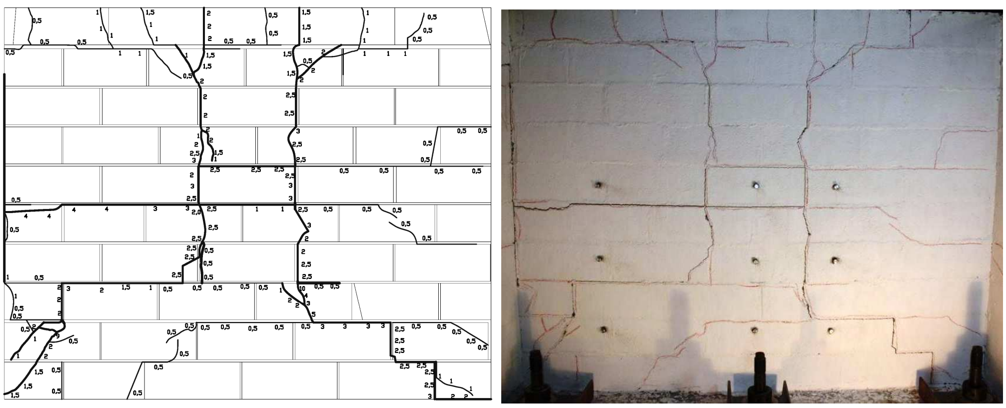

Figure 9. Wall 2- Failure modes of the principal wall

At the final stage, important cracks were also observed on the return walls. First, vertical cracks appeared and propagated near the corner (intersection with the main wall). These cracks are due to the loading of the main wall which induces rotations at the level of corner posts (Figure 10). The return walls were also subjected to in-plane shear which explains the appearance of diagonal cracks (Figure 10).

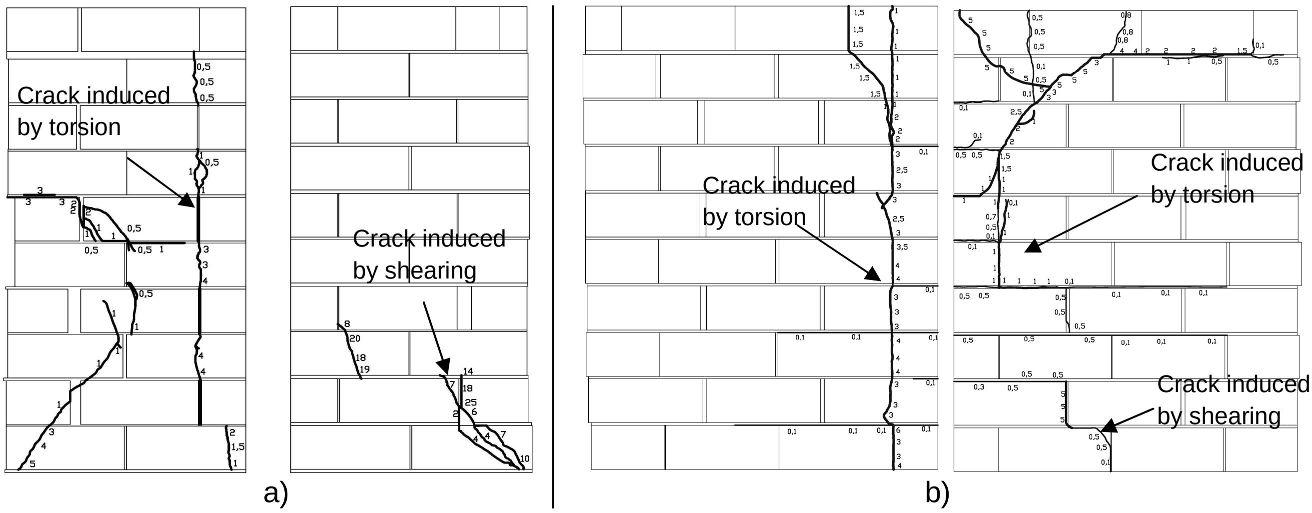

Figure 10. Failure modes of the two return walls: a)Wall 1; b)Wall 2

In the case of the reinforced wall only small cracks were observed on the principal wall (Figure 11), the most important cracks were observed on the return walls and more precisely at the corners or zones of connection between the principal wall and return walls (Figure 12). The flexural bending of the principal wall induced torsion at the connections, which caused the vertical cracks on the return walls. The diagonal cracks located at the base of the return walls correspond to the shear effect.

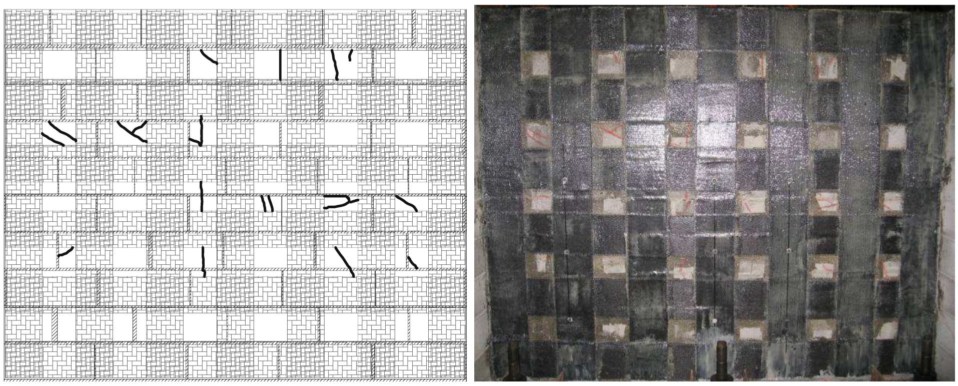

Figure 11. Wall 3- Failure mode of the principal wall (0.01mm average thickness of the cracks)

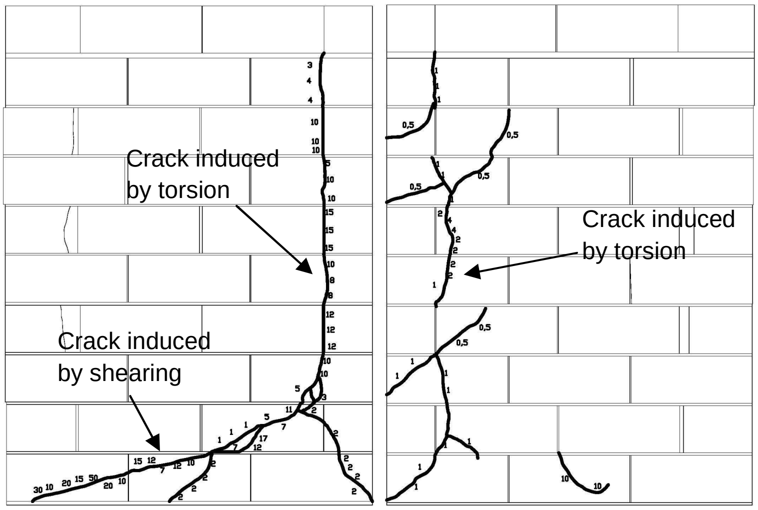

Figure 12. Wall 3 – Failure modes of the two return walls

5 NUMERICAL SIMULATION

The 3DEC (3-Dimensional Distinct Element Codes) code, based on the distinct element method is used for the simulation. The main and the two return walls are meshed by assembling blocks of size 0.5x0.2x0.2m exactly in the same way as for the test (running bond) (Figure 13a).

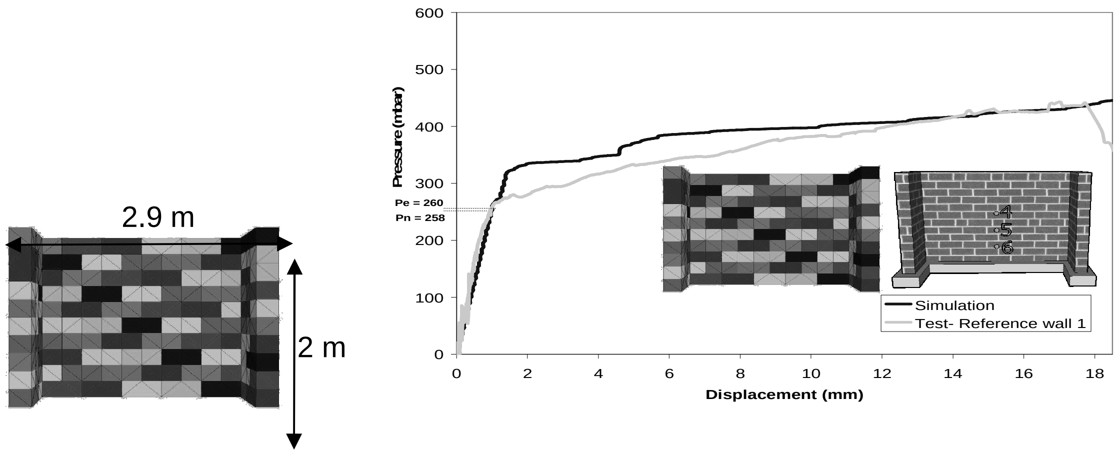

Figure 13. a) Complete mesh ; b) Pressure/deflection curve: comparison test/calculation

For the boundary conditions, the embedment applied to the base of the first line of block represents the embedment between the bottom of the wall and the concrete slab. The mortar joints are not modelled directly as elements, but indirectly by an interface law between the blocks. The interface obeys the Mohr-Coulomb joint model. For the blocks, the behaviour is considered elastic. Horizontal and vertical reinforcement are modelled using 1D elements and their behaviour is considered elastic perfectly plastic.

The load deflection curve obtained numerically (Figure 13b) is relatively close to the experimental one, and good agreement of the cracking pattern is also observed (Figure 14).

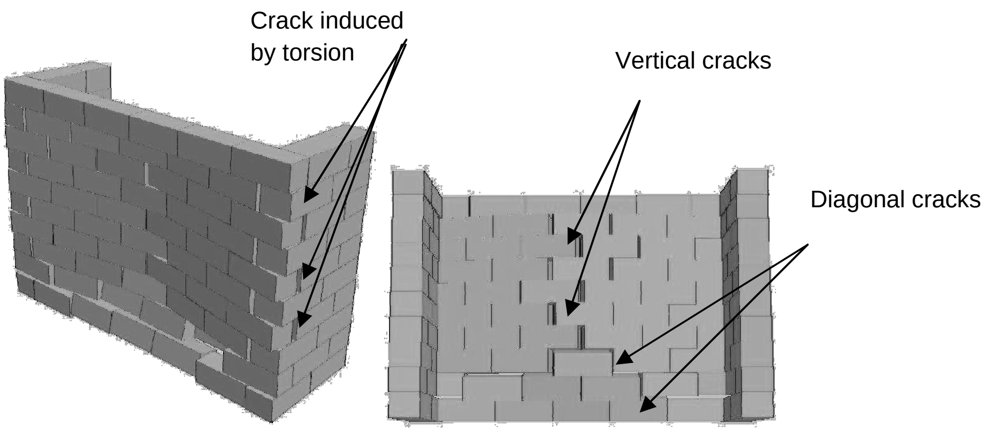

Figure 14. The cracking pattern obtained numerically

6 CONCLUSION

The most important conclusion made from this investigation is that CFRP can significantly increase the bearing capacity of concrete masonry when subjected to out-of-plane loading. An increase of 135% in the load associated with crack appearance was observed, and larger bearing resistance is provided. The gain is about 143%. Numerical simulations using discrete element approach provide good agreement with experimental results.

ACKNOWLEDGEMENTS

The authors would like to gratefully acknowledge FREYSSINET France Company, for providing the CFRP, and the technical staff for the application of the reinforcement following the recommended application procedure. Many thanks go to Mr. Bruno Hugueny and Mr Mario Masapollo (who helped also as experienced mason) for their technical help during testing of the walls.

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