EXPERIMENTAL INVESTIGATION ON THE BEHAVIOR OF SPANDRELS IN ANCIENT MASONRY BUILDINGS

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ABSTRACT :

The paper presents the main outcomes of experimental tests carried out to analyze the structural behaviour of masonry spandrels. They play a significant role in masonry walls of ancient buildings, as they guarantee an effective coupling between adjacent masonry piers so as to assure high in-plane strength and stiffness for the walls. The main aim of the research is to point out the actual response of such structural components under seismic loads. Two spandrels with different lintels are considered in the study, they have been subjected to a controlled vertical displacement history so as to simulate their response under seismic actions. Moreover the effect of a strengthening intervention for spandrels that consider the coupling between the masonry beams and steel ties has been pointed out. The results achieved make up for the lack of experimental data on the behaviour of these structural elements and constitute a valid reference to calibrate nonlinear numerical models for the assessment of the ancient masonry buildings seismic performance.

KEYWORDS: ancient buildings, masonry spandrel, experimental tests.

1. INTRODUCTION

Ancient masonry buildings (AMB) constitute a significant portion of existing buildings around the world. Many of these structures have a historical and cultural value and are considered as architectural heritage. Such buildings are composed of brick or stone masonry load bearing walls and of wooden floors or masonry vaults that usually form the horizontal structural system.

A large number of these traditionally-built structures are located in earthquake prone regions and have shown poor seismic performance under past earthquakes, which resulted in heavy damage and collapse causing many casualties (Tomaževič 2000). An accurate structural assessment as well as the development of effective strengthening techniques is therefore of paramount importance for AMB. In fact, a rough analysis may lead to either underestimate or overestimate the safety of these structures. In the former case serious risks for human beings can be met, while in the latter case excessive strengthening measures, that cause high intervention costs and important changes in the original structures, could be employed.

In any case, after strengthening the floor systems in order to prevent the out-of-plane displacements of the masonry walls so as to avoid the early collapse of some masonry piers affected by dangerous out-of-plane bending moments, the seismic performance of AMB is governed by the in-plane structural behaviour of the shear walls. These are bi-dimensional elements with openings of different dimension so as to allow for windows and doors. In general, a masonry wall may be considered as an assembly of vertical cantilevers of different dimensions connected together, at floor level, through spandrels (masonry beams). The spandrels in AMB, that normally have the depth comparable with the length, considerably increase both in-plane strength and stiffness of the walls, so that they play a very important role in the AMB earthquake resistance.

In the last years a big effort has been devoted to defining effective analytical and numerical models tailored for AMB seismic assessment and for the strengthening design, as their seismic response dramatically differs from that of the modern structural systems (e.g. steel or concrete structures). So several models according to different theoretical approaches have been developed so far. Many of such models are used for research and, being based on complex finite element formulations (Lourenço 1996), are high computationally demanding so that they cannot be employed

for realistic analyses of whole buildings. Different models have been developed for this last purpose; most of them use one-dimensional macro-elements to schematize the masonry wall so the seismic performance of whole buildings can be assessed with an acceptable computational effort. Among these models the POR method (Tomaževič 1978) is to be mentioned. It was extensively employed for the structural rehabilitation of AMB that had been damaged by past earthquakes. According to the POR method the structural collapse occurs because of a storey mechanism due to piers shear failure, while no allowance for possible damage of the spandrel beams is made, as they are assumed to be infinitely stiff and resistant. Such an approach sometimes can be unsafe, as in the case of weak-spandrel buildings. Therefore, recently, improved numerical models based on the equivalent frame approach have been defined (Magenes and Della Fontana 1998, Magenes 2000). In these models, suggested by the current design codes of practice, i.e. FEMA 356 (FEMA 2000), Eurocode 8 (CEN 2005), different failure mechanisms (Magenes and Calvi 1997), i.e. shear with diagonal cracking, shear with sliding and rocking, are provided for each macro-element. Such numerical models have been developed on the basis of both theoretical and experimental results. But, while many experimental outcomes (shear-compression test, diagonal compression test, etc.) on numerous types of masonry even under cyclic loads are available for vertical elements (FEMA 1998), to date no specific tests have been carried out to study the actual behavior of spandrels. These experimental achievements are of paramount importance because the spandrel structural response is considerably different from that of the vertical elements. In fact, under seismic loads, the masonry beams are subjected to shear and bending with negligible axial force.
Therefore there is a need of experimental outcomes on masonry spandrels so to determine their real behavior in terms of both resistance and deformability. To make up for the lack of experimental data, the authors carried out some tests on full scale specimens, designed to correctly represent the seismic behavior of spandrels in masonry walls, at the Laboratory for Testing Materials and Structures of the University of Trieste, Italy.

2. TEST SET-UP

Three experimental tests were carried out on full scale spandrels subjected to a loading condition that simulates the actual state of stress occurring in a wall with openings, subjected to horizontal forces.

2.1. Specimens

To represent such a situation as best as possible, the specimens comprised a spandrel connected to the mid-height of two piers at its ends so to form an H-shape. The piers length was equal to the distance between the mid-height of two consecutive floors of a representative building (Fig. 1). In order to subject the spandrel to bending and shear, the left pier was fixed at both ends whereas the right pier was translated vertically maintaining both ends horizontal (Fig. 2).

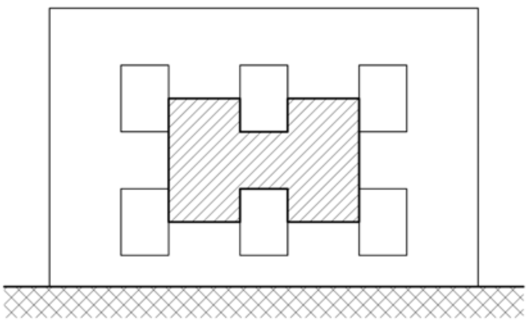


Figure 1. Wall with openings sample.

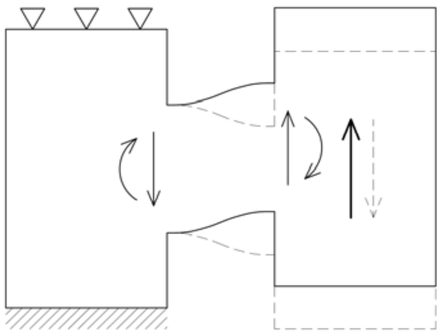
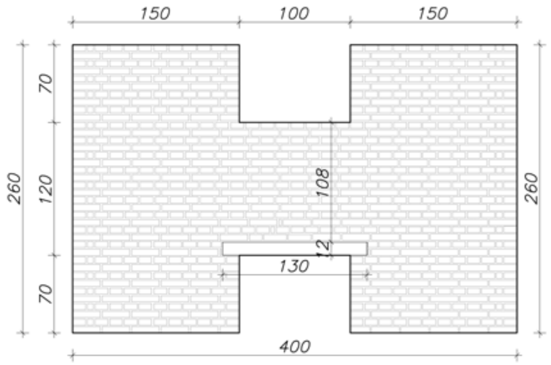


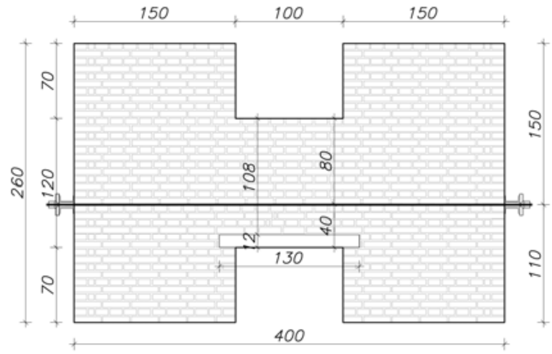
Figure 2. Schematic spandrel loading.

The specimens are samples of clay brick walls with a thickness equal to 38 cm. Some bricks 5.5x12x25cm large are placed with their stretchers (long faces) parallel to the face of the wall, while others are placed at right angles so that their headers (ends) appear in the surface of the wall and they lap over and bond with bricks behind the surface. Courses of stretchers alternate with courses of headers so there are no continuous vertical joints within the thickness of the brickwork. This type of masonry is very common in low rise ancient brick masonry buildings (two/three storeys).

Two different arrangements for the spandrels, that represent the most frequently used in historic buildings, were considered in the experimental investigation. In the former case (specimen MS1) a wooden lintel, having a width equal to the wall thickness, was used to support the masonry over the opening (Fig. 3a). Conversely in the latter (specimen MS2) a flat arch (25 cm thick) and a wooden lintel (12 cm wide) were set over the opening (Fig. 5b). Finally, a third tested wall (MS1r) was the first specimen (MS1) reinforced after the first test with a couple of horizontal steel ties arranged as shown in Fig. 3b. The geometric characteristics of the specimens MS1 and MS1r are reported in Figs. 3a,b.



(a)

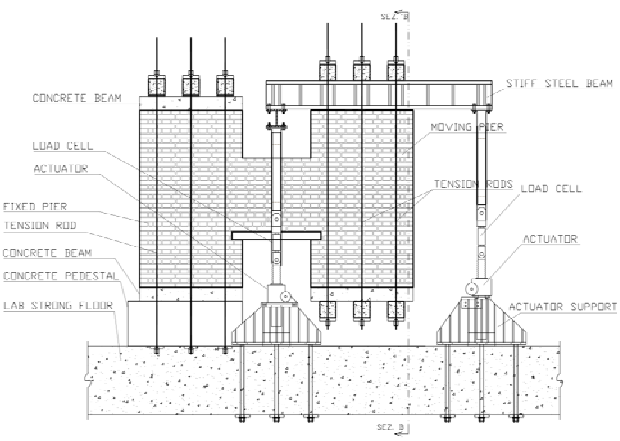


(b)

Figure 3. (a) Specimen with wooden lintel (MS1), (b) specimen reinforced with horizontal ties (MS1r).

2.2. Test arrangement devices

The specimens had been built in the Laboratory for Testing Materials and Structures of the University of Trieste, Italy, in the exact position where they were to be tested. After about 60 days of air curing, the specimens were prepared for testing. The left side pier was fixed to the stiff concrete basement through six vertical Dywidag steel bars. During construction and curing, the right pier was early laid on provisional supports. A stiff steel element was placed on top of the pier and was connected to the bottom of the pier through six vertical Dywidag steel bars. The steel element was linked to three electro-mechanic actuators adequately fixed to the stiff concrete floor. Two actuators were arranged at both sides of the wall with their axis belonging to a plane perpendicular to the wall and containing the symmetry axis of the specimen, whereas the third actuator was set at the right side of the specimen. In Fig. 4 the arrangement of the loading system is schematically given. Two couples of steel braces were positioned on both sides of the specimen so as to contrast out-of-plane displacements of the wall (Fig. 4b). Adequate sliding devices were provided at brace-wall interface; a thin layer of PTFE was used to reduce possible friction.



(a)

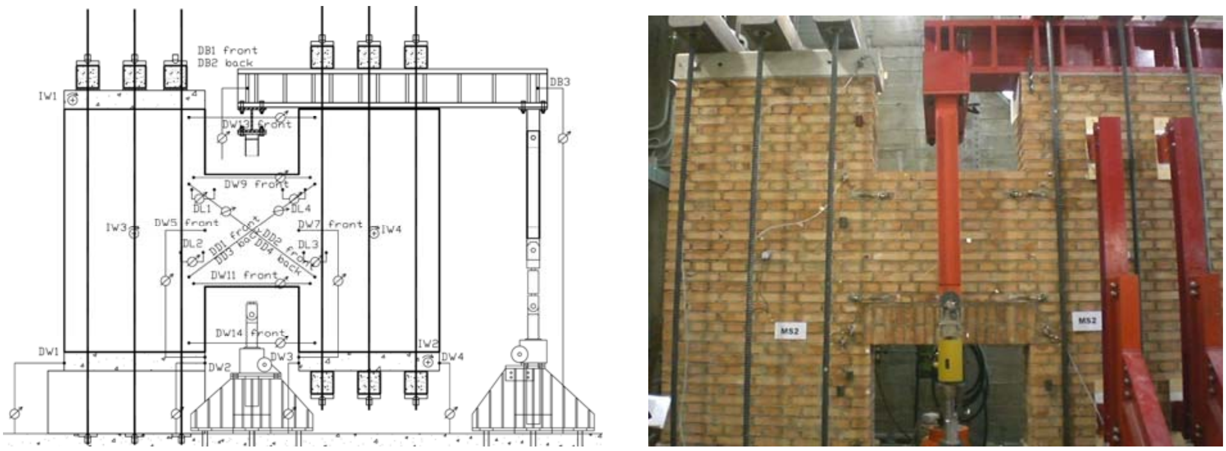
(b)

Figure 4. Experimental test arrangement: (a) schematic drawing, (b) axonometric view.

The steel bars were also used to apply a compressive force to the piers so as to simulate the axial load that normally acts in the wall due to gravity (self weight, floor reaction, etc.). In particular, a distortion was applied to the bars in order to produce an average compressive stress equal to 0.5 MPa. Such a value corresponds to a common stress in a three-storey residential masonry building.

2.3. Loading system and gage location

Many potentiometer linear variable displacement transducers (LVDT) were used to survey the displacements of several points of the specimen. In particular three transducers DB1, DB2, DB3 measured the vertical displacement of the steel element on the top of the right pier in correspondence of the point where the actuators were joined. Two couples of transducers DW1-DW2 and DW3-DW4 were arranged to survey the vertical displacements of the bottom parts of the left and right piers, respectively. Two transducers DW5 and DW7 measured the relative vertical displacements between the left and right mid points of the spandrel and the corresponding points at the bottom of the piers. Two transducers DW9 and DW11 measured the horizontal displacements of two points at the ends of the spandrels, at top and bottom respectively. Two couples of transducers DD1-DD2 and DD3-DD4 were used to measure the diagonal displacements on the spandrel's front and back side. Two transducers DW13 and DW14 were employed to survey the relative horizontal displacements between two points at top and bottom of the piers. Finally four transducers were arranged at the ends of the spandrel, both at top and bottom, in correspondence of where cracks were likely to appear, with the purpose to measure the possible crack opening.



(a) (b)

Figure 5. Experimental test arrangement: (a) scheme, (b) axonometric view.

Four inclinometers IW1, IW2, IW3, IW4 were also set to measure the rotation of some points of the specimen. In particular IW1 and IW2 measured the rotation of the top of left pier and of the bottom of the right pier, respectively. Instead the inclinometers IW3 and IW4 allowed to survey the rotation of both intersecting points between piers and the spandrel axis. The transducers and inclinometers arrangement is illustrated in Figs. 5a,b.
As above stated, three electromechanic actuators (500 kN, ±150 mm stroke) were used to apply the loads to the specimens while three loading cells, connected in series with the actuators, were used to survey the load. The electrical engines that moved the actuators were governed through a computer aided electronic unit.

3. EXPERIMENTAL INVESTIGATION

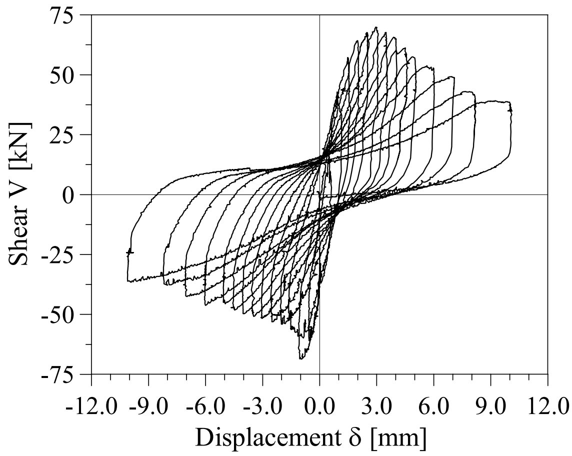
In the experimental investigation carried out to assess the effectiveness of spandrel beams in the global shear behavior of masonry walls with openings, the three specimens described in the previous section were tested. In addition to these tests on the masonry walls, some other tests were also carried out on samples of the mortar used in the masonry wall specimens and on brick triplets. So two average values equal to 2.11 MPa and 3.28 MPa for the mortar compressive strength and two average values for the shear strength at zero compressive stress for the masonry equal to 0.19 MPa and 0.22 MPa were determined for specimen MS1 and MS2 respectively.

3.1. Test procedure

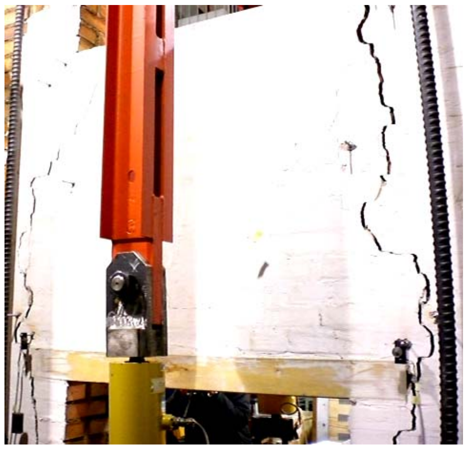
The actuators were forced to move together in order have the same vertical displacements measured through the transducers DB1, DB2, DB3. In fact the tests were carried out by controlling the vertical displacements of the right pier so as to have the stiff steel element on the top of the pier remain horizontal during the test (Figs. 5a,b).
The vertical displacement of all the actuators was varied cyclically between two opposite values gradually increasing them. The displacement history for specimens MS1 and MS2 is shown in Figs. 7 and 9 respectively.
The tests on specimens MS1 and MS2 were stopped when the maximum vertical displacement (transversal drift in the spandrel) reached ±10 mm (1/100 of the spandrel length) because the cracks were very large, up to 7-8 mm in the first case, up to 4-5 mm in the second. Whereas in the test on specimen MS1r vertical displacements up to ±90 mm were reached, due to the considerably larger ductility of the spandrel that was a masonry element reinforced with horizontal steel ties.

3.2. Experimental results

The main results that summarize the global response of the spandrels in a masonry wall with openings concern the relationship between the shear load and the relative transversal end displacement of the spandrel. The shear load is the algebraic sum of the loads measured by the three loading cells arranged on the actuators. The displacement is obtained by difference between the vertical displacement of the end points of the spandrel axis: (DW7+DW3)-(DW5+DW2). Such a value needs to be corrected in order to consider the rotation of the piers at the spandrel connection, which means on the basis of the rotations measured by the inclinometers IW3 and IW4.
Fig. 6a shows the relationship between the shear load and the relative transversal displacement of the spandrel in the specimen MS1. The whole sequence of loading and unloading is plotted. A maximum value of about 70 kN was reached in both loading directions. After the peak load a progressive reduction in shear capacity was observed. In particular, at the end of the test a residual strength equal to 40% of the maximum shear capacity was reached, when a displacement five times greater than that corresponding to the peak load occurred. The decrease in resistance was due to a progressive damage that resulted in terms of crack propagation as can be noticed in Fig. 6b, where the crack pattern in the spandrel at the end of the test is illustrated. The main cracks are sub-vertical at the ends of the spandrel and they cross the entire thickness, which cause the spandrel to rock between the piers. The tie effect that avoids the piers to separate is provided by the wooden lintel encased in the piers.



(a)



(b)

Figure 6. Specimen MS1: (a) shear load against displacement, (b) crack pattern in the spandrel.

The sequence of cracking on the back face of the spandrel is illustrated in Fig. 7. Before the test such wall face was plastered with a thin layer of gypsum so as to better capture the cracks occurrence during the test. As it can be seen, the first cracks are mainly vertical and occur in the four corners of the spandrel (flexure cracks). Increasing the load some diagonal cracks occur (shear cracks) and the vertical ones propagate. For values of the displacement greater than 5 mm the top and bottom cracks tend to meet causing the rocking of the spandrel.
Similarly, the main results concerning the specimen MS2 are shown in Figs. 8a,b and 9. As mentioned before

such specimen differs from MS1 because it has a flat brick arch in parallel with a wooden element over the opening instead of a wooden lintel. The relationship between the shear load and the relative transversal displacement of the spandrel is shown in Fig. 8a considering the whole sequence of loading and unloading. A maximum value of about 50 kN was reached in one direction and 42 kN in the opposite direction. After the peak load a quite rapid reduction in shear capacity was observed and a residual strength value of about 40% of the maximum shear was reached at the end. Such a residual resistance was maintained up to the maximum displacement, which is about 5 times greater than that corresponding to the maximum capacity. In Fig. 8b the crack pattern in the spandrel's back face at the end of the test is shown. As it can be seen the main cracks are diagonal (shear cracks). The sequence of cracking is illustrated in Fig. 9. The diagonal cracks start from the four corners of the spandrel and then, increasing the load, the cracks propagate and for a value of the vertical displacement greater than 3 mm they meet in the middle of the spandrel. After that the shear capacity remains almost the same up to the maximum displacement (±10 mm).

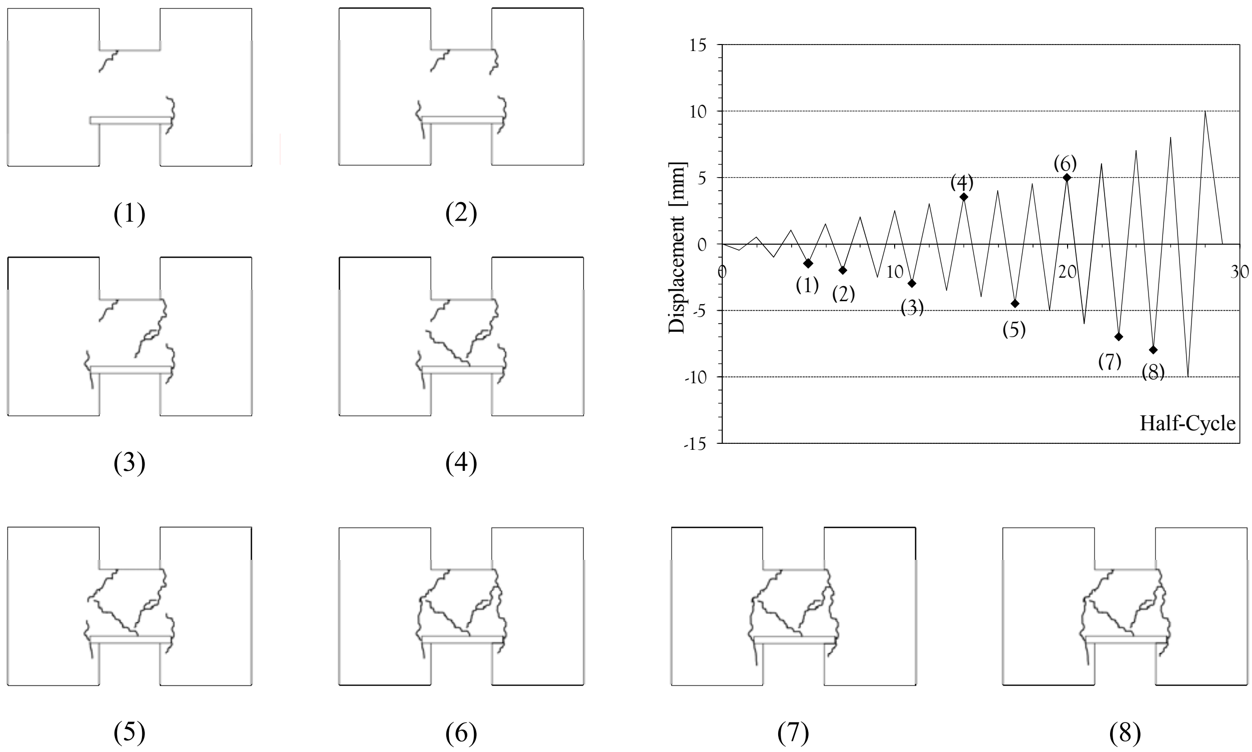
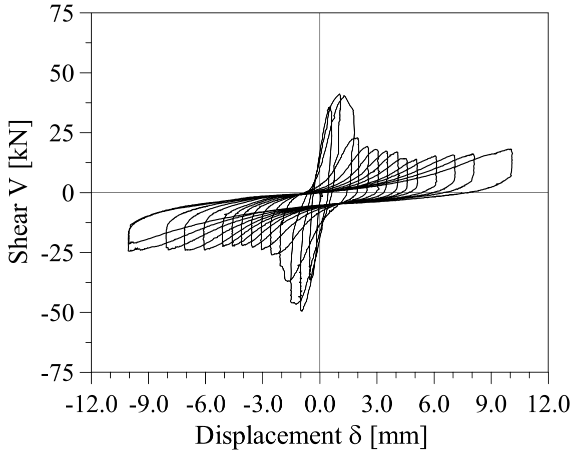


Figure. 7. Sequence of cracks in the spandrel's back face of the specimen MS1.



(a)

(b)

Figure 8. Specimen MS2: (a) shear load against displacement, (b) crack pattern in the spandrel.

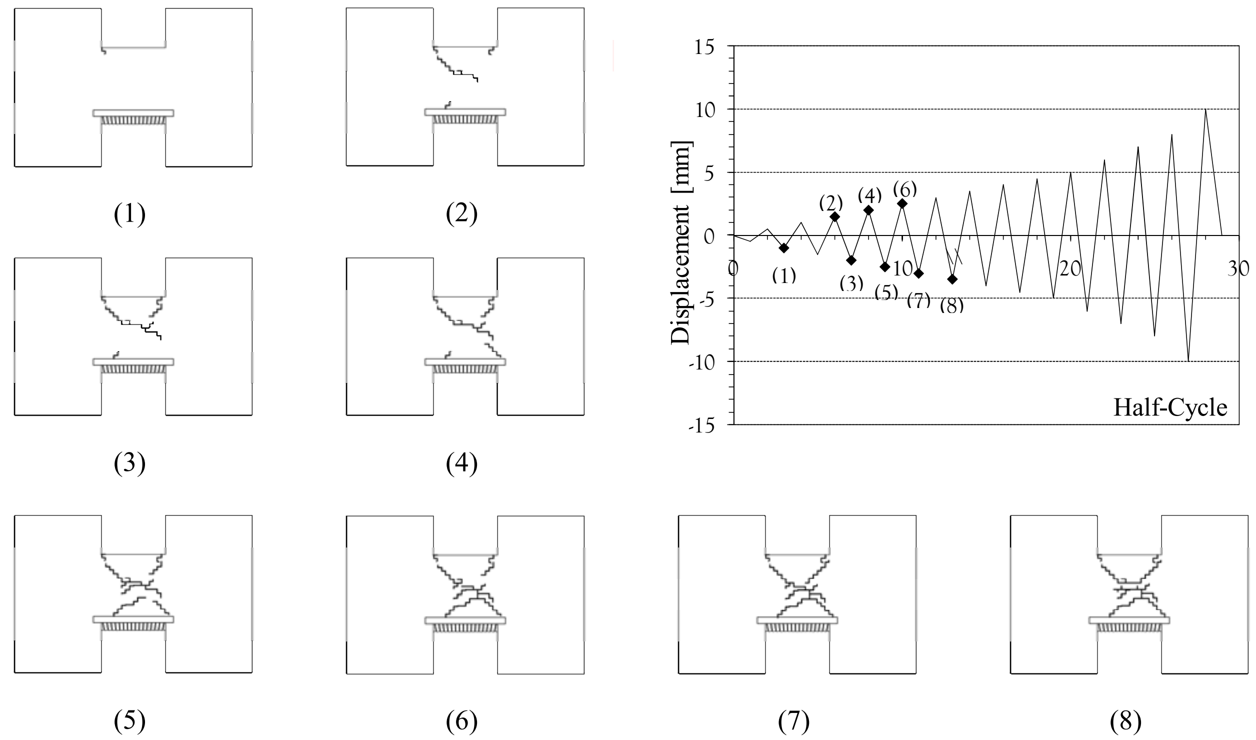
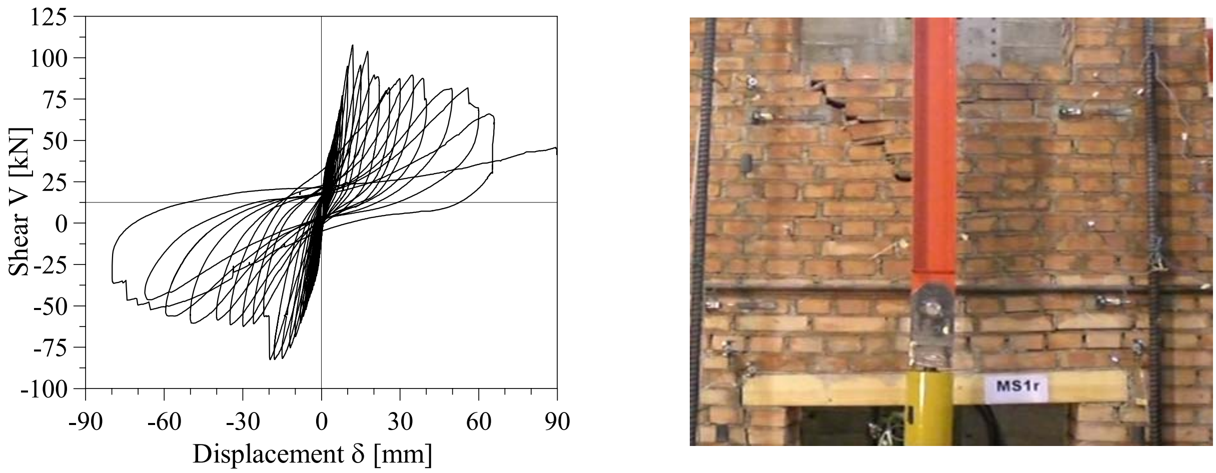


Figure 9. Sequence of cracks in the spandrel's back face of the specimen MS2.

As mentioned above, the specimen MS1, after testing, was reinforced adding a couple of horizontal steel ties that allowed closing the cracks. Actually, by means of the steel ties (two Dywidag bars 27 mm diameter) a horizontal compressive force equal to 85 kN was applied to the spandrel.



(a) (b)

Figure 10. Specimen MS1r: (a) shear load against displacement, (b) crack pattern in the spandrel.

The relationship between the shear load and the relative transversal displacement of the reinforced specimen's spandrel is shown in Fig. 10a. A maximum value of about 100 kN was reached in one direction and 80 kN in the opposite one. As in the previous tests after the peak load a progressive reduction in shear capacity was observed. But in this case a residual strength value of about 60-70% of the maximum strength was measured. Such resistance values were obtained at displacements equal to 60-70 mm, which means about 5 times greater that that corresponding to the maximum shear capacity. Improvement in terms of shear capacity (40-50%) and even more in terms of ductility (ultimate drift equal to 1/15-1/20 of the spandrel length) was significant with respect to the unreinforced spandrel (MS1). The reinforced spandrel collapse occurred when the wooden lintel beaks because of flexure at approximately 1/3 of its length. Sub-vertical cracks did not occur, but only diagonal cracks formed as visible in Fig. 10b, which refers to a loading step just before the collapsing of the spandrel.

5. CONCLUDING REMARKS

The results of the experimental investigation carried out to study the behavior of spandrel beams in brick masonry walls subjected to seismic action allow to draw the following concluding remarks.
Unreinforced masonry spandrels, as commonly found in ancient buildings, evidence a limited shear capacity but they offer a significant dissipative capacity associated to large damage (extensive cracks). Two spandrels with different lintels were considered in the study: a first sample included a wooden lintel on the entire wall thickness and the second features a brick flat arch on partial wall thickness (external face) with a thin wooden lintel (internal face). The spandrels in the two cases performed appreciably different behavior both in terms of resistance and dissipative capacity. The spandrel with wooden lintel showed better performance even though large vertical crossing-through cracks occurred quite early (5 mm displacement) at its ends. On the contrary the other sample showed only diagonal cracks with limited opening. In both cases the test was suspended when the drift was equal to 1/100 of the spandrel length. The test carried out on the first specimen (wooden lintel), after reinforcing the spandrel with two horizontal lightly prestressed steel bars, evidenced significant increase in shear capacity (40-50%) and a considerable improvement in ductility (drift up to 1/15-1/20 of the spandrel length). Such results constitute a valid reference to calibrate numerical models for the nonlinear assessment of masonry buildings subjected to seismic actions. This research is still ongoing and other different types of masonry walls will be studied in future tests.

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REFERENCES

CEN (2005). Eurocode 8: Design of Structures for Earthquake Resistance - Part 3: Assessment and Retrofitting of Buildings. Brussels, Belgium.
Federal Emergency Management Agency FEMA (1998). FEMA 307: Evaluation Of Earthquake Damaged Concrete And Masonry Wall Buildings. Washington DC, USA.
Federal Emergency Management Agency FEMA (2000). FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Washington DC, USA.
Lourenço, P. B. (1996). Computational strategy for masonry structures. PhD Thesis, Delft University of Technology. Delft, the Netherlands.
Magenes, G. and Calvi, G.M. (1997). In-plane seismic response of brick masonry walls. *Earthquake Engng Struct. Dyn.* **26**, 1091-1112.
Magenes, G. and Della Fontana, A. (1998). Simplified non-linear seismic analysis of masonry buildings. *Proceedings of the British Masonry Society* **8**, 190–195.
Magenes, G. (2000). Method for pushover analysis in seismic assessment of masonry buildings. Twelfth World Conference on Earthquake Engineering, Auckland, New Zealand.
Tomaževič M. (2000). Earthquake-resistant design of masonry structures. Series on Innovation in Structures and Construction – Vol.1. Series Editors: A.S. Elnashai and P.J. Dowling, Imperial College Press, London, UK.
Tomaževič, M. (1978). The computer program POR. Report ZRMK, Ljubljana, Slovenia.