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Analysis of the in-plane shear behaviour of FRP reinforced hollow brick masonry walls

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Abstract. This paper presents an experimental as well as a numerical analysis of the in-plane shear behaviour of hollow, 870 × 840 × 100 mm masonry walls, externally strengthened with FRP composites. The experimental approach is devoted to the evaluation of the effectiveness of different composite strengthening configurations and the methodology consists in the diagonal compression of masonry walls. The numerical study assesses the stress and strain state distribution in the unreinforced and strengthened panels using a commercial finite element code. The effect of FRP reinforcement on the masonry behaviour and the capability of modelling to forecast a representative failure mode of the unreinforced and reinforced masonry walls is investigated.

Key words: hollow brick masonry; FRP reinforcement; diagonal compression test; finite element modeling.

1. Introduction

Since most of the masonry buildings situated in seismic zones have not been designed and built for seismic loading, the vulnerability of large populations of these structures must be assessed and many structures must be seismically upgraded. This task is technically difficult and has tremendous socio-economic implications. In order to answer this challenge, assessment methods and retrofitting techniques are continuously improved. Nowadays, research is centred upon innovative strengthening techniques, involving composite materials, especially externally bonded FRPs. A state-of-the-art of the FRP strengthening of civil engineering structures are presented in (Priestley and Seible 1995, Hamelin 1998, Triantafillou 2001). The main objective is to enhance the strength and the deformation capability of these structures, in order to avoid failure modes that occur in brittle and unforeseen manner. The behaviour and damage pathology of masonry walls submitted to predominant shear load is classified in this category (FEMA 306 1999).

Generally, the behaviour of masonry structures or masonry structural elements is approached considering the out-of-plane and the in-plane behaviour. A wide amount of scientific work is dedicated to the assessment of the out-of-plane behaviour of unreinforced and reinforced masonry

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walls, since the preservation of the structural integrity of a masonry building during earthquakes is primordial.

Experimental studies concerning the out-of-plane flexural strength and deformation capability were conducted by Ehsani *et al.* on half scale brick walls strengthened with FRP composite strips (Ehsani *et al.* 1999). The masonry panels were tested under cyclic out-of-plane loading. Different reinforcement ratios and glass fiber types were investigated. The tested specimens showed an increasing of about 30 times in bearing capacity. The type of the composite influences the failure mode: a wider and lighter composite favorises a tensile failure, whereas a stronger composite produces a failure by delamination. Certain energy dissipation due to delamination phenomenon is noticed, although masonry and composite strips behave in an elastic manner.

In a similar study (Albert *et al.* 2001), the influences of several experimental parameters on the load carrying capacity of carbon-FRP reinforced panels are investigated: the type, amount, layout of the reinforcement and the effects of a moderate compressive axial load. The reinforced walls present a non-linear global behaviour, characterized by an increased load carrying capacity. Two phases of the behaviour are distinguished: the first one determined by the stiffness and the strength of the wall and the second one directly related to the equivalent stiffness of the applied reinforcement. The observed prevailing failure mode occurs in the core of the masonry panels and is the consequence of a combination of shear and flexion stresses. The applied composite strips show an increased strain state in the zones of the brick/joint interfaces. In fact, the adherence between the masonry wall and the reinforcement is weaker at the mortar joint than at the bricks, leading to an increased stress and strain distribution in the composite after the joint fissuring. This fact emphasizes the role of the reinforcement layout on the global out-of-plane behaviour.

If the out-of-plane failure is avoided, then the structural resistance is mainly influenced by the in-plane behaviour of the masonry structural wall. Because of the small height/width ratio of masonry walls, relatively large shear stresses could develop, being favourable to a non-ductile behaviour (Paulay and Priestley 1992). In addition, the brittle behaviour of masonry units and mortar reduce the energy dissipation capabilities of the masonry elements. Therefore, in order to predict the shear behaviour, it is necessary to properly evaluate the mechanical properties of the masonry. These properties depend on the characteristics of constituents (bricks and mortar) and their interaction (Hendry *et al.* 1997). Besides, the mechanical properties of the constituents vary according to their nature (plain or hollow bricks, concrete blocks, composition of the mortar, etc.). Consequently, for any type of masonry an extensive testing program is necessary, ranging from constituents to the assemblage scale.

Concerning the in-plane behaviour, Corradi *et al.* (2002) pursued a comparative in-situ study on the effectiveness of different strengthening procedures applied to ancient stone masonries. Particularly, the influence on the in-plane shear behaviour of grouting and of surface strengthening with glass or carbon FRP composites have been studied. The obtained results show an increase of the shear strength up to 200% for both strengthening techniques. However, the deformation capability of walls reinforced with FRPs is superior than those strengthened by grouting. The global stiffness of walls is characterized by an important increase for the grouted walls (up to 20 times), while the change in stiffness for the FRP strengthened walls is negligible.

In the same context, Valluzzi *et al.* (2002) performed an experimental study in order to investigate the efficiency of an FRP shear reinforcement technique. Different reinforcement configurations were evaluated on small masonry panels submitted to diagonal compression tests. Experimental results emphasized that FRP reinforcement, applied only at one side of the panels did not significantly modify the shear behaviour of the unreinforced masonry, while double-side configurations provided a modified failure and an important ultimate capacity increase. Experimental results are also used to calibrate the analytical formulation given in Triantafillou (1998): the difference between analytical prediction for ultimate shear strength and experimental results has to be improved by considering a supplementary parameter, which account for the layout of the composite reinforcement.

On the basis of the studies presented, it appears that the use of FRP composites to retrofit unreinforced masonry walls might be an efficient alternative to enhance the wall's global out-of-plane and in-plane behaviour. However, some problems need further analysis, especially the choice of an optimized strengthening reinforcement in terms of strength, elastic moduli and layout. Additionally, the local acting mechanisms of the reinforcement strips or sheets need to be analyzed.

In this work we choose to analyse masonry assemblies built with hollow brick units and a ready-to-use mortar. Combined experimental and numerical studies are performed in order to describe the shear behaviour of fiber reinforced polymer (FRP) strengthened masonry panels. The experimental part is dedicated to the assessment of the in-plane shear behaviour using the diagonal compression test method. The numerical study is devoted to the set-up of a finite element simulation, based on the mechanical properties of the constituents in order to describe the overall behaviour of the unreinforced and strengthened masonry panels until failure. In addition we also study the effectiveness and reliability of different types of FRP composites.

2. Mechanical properties of the masonry constituents

Masonry panels employed for the diagonal compression test were constructed using 210 × 100 × 50 mm hollow bricks and a ready-to-use mortar with a 0 - 5 mm sand and Portland cement composition. The water quantity added to the dry mixture was determined assuring a good workability of the fresh mortar.

The experimental procedures, setups and detailed results employed to evaluate the mechanical parameters of masonry constituents are referred in previous works (Gabor 2002, Gabor *et al*. 2004). The description of the experimental procedures and results is given below.

We recall that the scope of the experimental evaluation of the mechanical parameters of the masonry constituents is to obtain parameters that can be implemented directly in a finite element model of a commercial code, in order to simulate the behaviour of masonry panels submitted to diagonal compression loading. The diagonal compression generates a combined state of shear and compression stress along the direction of the horizontal and vertical joints. Thus, for the considered approach, we evaluate the mechanical parameters of the masonry in compression and shear.

Consequently, the experimental set-up is realized in a manner that it allows the measurement of the apparent mechanical parameters of bricks and mortar. Moreover, we have chosen to determine the mechanical parameters of bricks and mortar from tests performed on masonry prisms rather than on individual specimens, since tests on individual specimens can induce some errors, as reported in the literature. First, given the relatively small vertical size of bricks, the confining effect induced by the loading platens of the experimental apparatus artificially increases the strength of the tested specimens, so stress-strain diagrams could be false (Paulay and Priestley 1992). Second, the mortar characteristics in joints could be different from those measured on cylindrical prisms. This difference is caused by the differentiated drying and water absorption at the mortar block interface (Delmotte *et al*. 1992).

2.1 Mechanical parameters of the compression behaviour

We are principally interested in the measurement of the elastic moduli and the Poisson's ratio of the bricks and the mortar, since we implement these parameters in the finite element modelling. In order to determine these parameters, three masonry prisms were realized and tested on the basis of RILEM recommendations (RILEM 1994a). A masonry prism can be considered as being extracted from a real masonry wall. Joint thickness and brick dimensions are the same as for the masonry panels employed later for the diagonal compression test. Vertical and horizontal strain in masonry units was measured directly by strain gauges, while in the whole prism they were measured by LVDT extensometers (Fig. 1). The precision of the gauges and of the LVDT extensometers is 1 μm. The load is displacement controlled (0.5 mm/min) and measured by a 500 kN load cell.

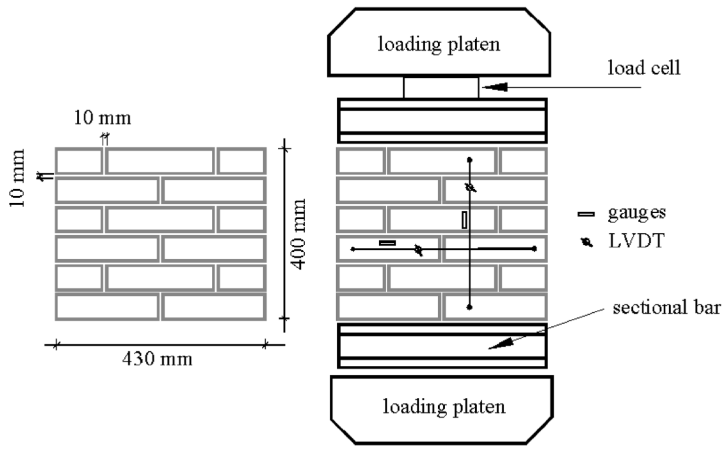

Fig. 1 Geometry of masonry prism and distribution of measurement devices for the uniaxial compression test

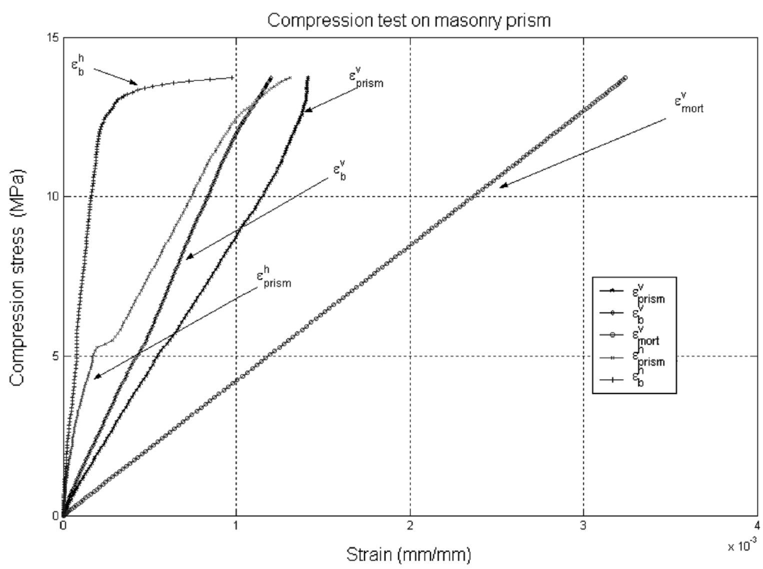

Fig. 2 Stress-strain diagrams for uniaxial compression test

The compression stress is calculated as the ratio of the applied force and the rough surfaces of bricks. The elastic moduli of the bricks and of the masonry prism in compression are evaluated on the basis of the compression stress vs. vertical strain curves for 30% of the ultimate stress ($\sigma - \varepsilon_b^v$ for the bricks and respectively $\sigma - \varepsilon_{prism}^v$ the prism, Fig. 2). We notice that masonry and brick have linear elastic behaviour; the elastic modulus of bricks is greater than that of masonry. This agrees with the results given in Hendry *et al*. (1997).

The elastic modulus of the mortar joint (E_{mort}) is computed on the base of the moduli of the brick (E_b) and of the masonry prism (E_{prism}), considering that the total vertical displacement of the prism is the sum of the displacement undergone by the joints and by the bricks:

$$E_{mort} = \frac{E_{prism}E_b}{\alpha(E_b - E_{prism}) + E_b} \tag{1}$$

where α represents the ratio between the brick height and mortar joint height.

Given the fact that strain gauges were placed horizontally on the brick surface and the horizontal dilation of the masonry prism is measured by an LVDT device, we can dress the compression stress vs. horizontal strain diagrams for the bricks and for the masonry prism ($\sigma - \varepsilon_b^h$ and $\sigma - \varepsilon_{prism}^h$, Fig. 2), which allow determining Poisson's ratios. The Poisson's ratio of the mortar is considered equal to that of the bricks. Table 1 summarizes the average results of the three tests.

Considering the cracking mode, we noted the apparition of vertical cracks along the head joints. The cracks crossed also the joined bricks (Fig. 3). This type of failure is similar to that described in the literature (Paulay and Priestley 1992), and it is explained by the lateral "confinement" effect of the bricks on the mortar joints. Thus, horizontal tensile stresses develop in the bricks, which lead to the fissuring. Besides, we notice that until cracking, the brick behaviour is quasi-elastic. At cracking, horizontal deformation develops instantaneously, without relevant increase of the compression stress.

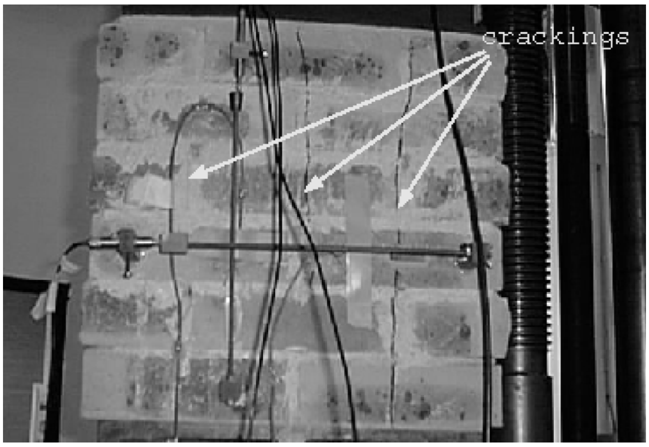

Fig. 3 Cracking pattern of the masonry prism for the uniaxial compression test

2.2 Mechanical parameters of the shear behaviour

Shear characteristics of the masonry and brick/mortar joint interaction parameters at the interface are determined on masonry prisms (triplets) using the double shear method. This test method is

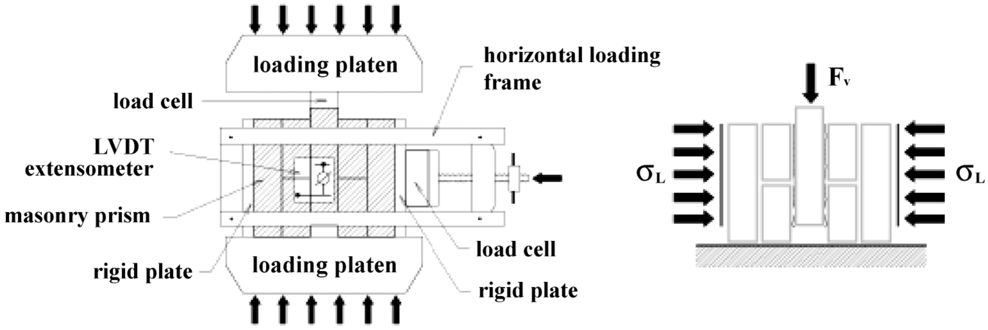

Fig. 4 Experimental set-up for shear test on masonry prisms

Table 1 Mechanical properties of masonry and constituents

	Bricks	Mortar	Masonry
Elastic modulus (MPa)	12800	4000	9400
Poisson's ratio	0.2	0.2	0.3
Shear strength (MPa)	-	1.63	-
Residual friction angle	-	43^o	-

adopted from RILEM recommendation (RILEM 1994b). The experimental device is conceived in such a way that it can apply a static horizontal confinement load and a steadily increasing vertical displacement simultaneously to the specimen (see Fig. 4). Loading forces are measured by force cells while the axial load is maintained constant during tests using a special device. Relative displacement between two adjacent bricks is measured by LVDT devices. The precision of the measurement devices is the same as that used for the uniaxial compression test.

The test results for 18 triplets using confining stress varying from 0 to 1.8 MPa allowed us to find by linear regression the parameters of a Drucker-Prager constitutive equation : shear strength and residual friction coefficient. Given the fact that parameters that are assessed will be used in a two-dimensional modelling, they are referred to the rough geometrical dimensions. Therefore, shear and confinement stresses are calculated considering apparent contact surfaces between bricks. We present the values found for the above mentioned parameters in Table 1.

For the same triplet test, the values obtained for the shear strength show variations up to 25%. This value is relatively high, but it can be explained by the internal structure of the triplets. During the preparation of specimens, the fresh mortar fills the cavities of the bricks not in a uniform and compact way, even if a special attention is accorded to the spreading of mortar in order to realize joints with uniform thicknesses. Therefore, the shear behaviour is determined by the interaction of the mortar cores with the internal walls of the bricks. Thus, the size and the regularity of the distribution of mortar cores strongly influence the shear strength level.

2.3 Mechanical properties of the FRP reinforcement

Three types of FRP composites are employed for reinforcement: unidirectional glass fiber (noted RFV), unidirectional carbon fiber (noted RFC) and bi-directional glass fiber (noted RFW). We have chosen the composites that are commercially available in the market and are generally used for

Table 2 Mechanical properties of composite reinforcements

	Elastic modulus (MPa)		Strength (MPa)	Ultimate strain (%)	Standard deviation (%)
	E_{xx}	E_{yy}			
RFV	23000	2500	460	2	11
RFC	80000	3000	720	0.9	2
RFW	10000	10000	100	1	1

strengthening of concrete and masonry structural elements.

Mechanical properties of composites have been determined by tensile test on coupons. The composite coupons were manufactured by embedding the fibers in epoxy resin, in the same conditions as the reinforcement was applied on the walls. All composites reveal a linear elastic behaviour, but the mechanical properties are completely different from one composite to another (see Table 2). The composites RFV and RFC can be considered as high strength and high modulus, while RFW can be considered as having low mechanical properties. Besides, the ultimate strain of RFV composite is two times higher than the ultimate strain of the other composites.

3. Description of the masonry panels

Masonry panels employed for diagonal compression tests were built according to RILEM recommendations (RILEM 1994c). The size of a masonry panel is established as function of the brick units size, in order to assure that panels have a representative number of joints and units. Thus, five masonry panels were built, having nominal dimensions of 870 × 840 × 100 mm, (Fig. 5).

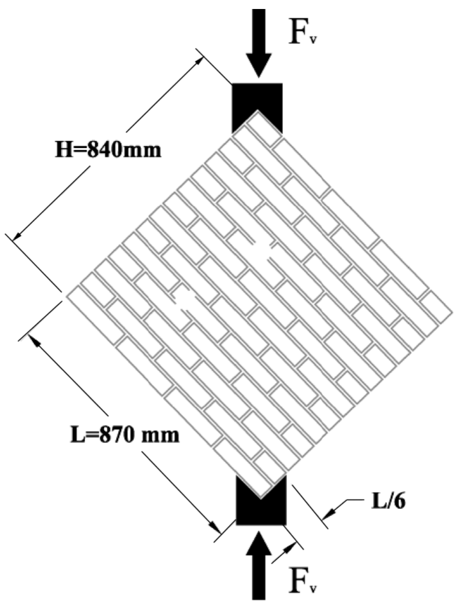

Fig. 5 Geometrical configuration and boundary conditions for masonry panels

They were made of hollow bricks and have a 10 mm thick mortar joint. Two of them were kept without reinforcement; the three other panels were strengthened with the three types of FRP composites. In order to apply the diagonal compression load, the panel corners are embedded in stiff loading shoes, filled with concrete. The length of the embedding is approximately equal to the 1/6th of the wall length.

In order to study the influence of the effectiveness of different FRP composites two configurations of the retrofit system were investigated. For the masonry panels reinforced with RFV and RFC unidirectional composites, four strips were bonded orthogonally to the loaded diagonal. The dimensions of strips were 400 × 150 mm. The vertical spacing between strips is approximately 100 mm. The choice of this configuration was guided by several factors:

- Composites work efficiently only when charged in traction; this justifies the orthogonal disposition to the compressed diagonal, the direction of the principal tensile stresses;
- The unidirectional composites are the most often manufactured in strips; the fabrication width of the RFC and RFV composites is 150 mm.
- The length of the strips is chosen considering that the principally solicited zone is localized along the compressed diagonal, where fissures are susceptible to occur during the diagonal compression test (Marzahn 1998). Besides, as described below, the tests of the unreinforced panels (section 4.1) and the finite element modelling (section 5.1) confirmed the failure along the compressed diagonal. Thus, we considered a 400 mm wide zone to be reinforced along the compressed diagonal.
- The choice to keep the zones unstrengthened along the compressed diagonal is guided by the saving reasons.

For the third, bi-directional composite (denoted RFW) the entire surface of the panel was reinforced. The reason of the employment of this type of reinforcement is to compare the effectiveness of high strength and high modulus unidirectional composites (RFV and RFC) with that of a bi-directional composite having weaker mechanical properties (RFW).

The orientation of the RFW composite fibers follows the directions of compressed and stretched diagonal. Panels reinforcement scheme is presented on Fig. 6. For the identification of masonry panels we use the same denominations as for the composites.

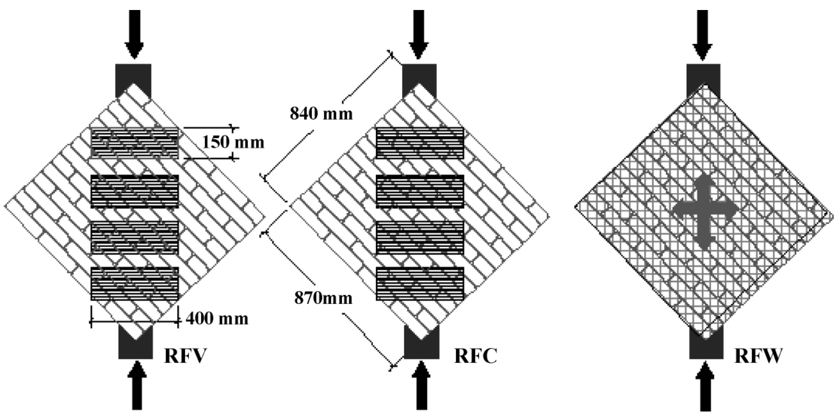

Fig. 6 Configuration for strengthening of masonry panels

4. Experimental results

The experimental setup for the diagonal compression is presented in Fig. 7. The load is gradually applied by a 500 kN hydraulic jack and controlled by a load cell. LVDT transducers measure the displacements of compressed and stretched diagonals of masonry panels. Moreover, the strain evolution in RFV and RFC composite strips during loading is measured by strain gauges (Fig. 8). For the RFW composite sheet the strains were measured at heights corresponding to those of RFV and RFC strips.

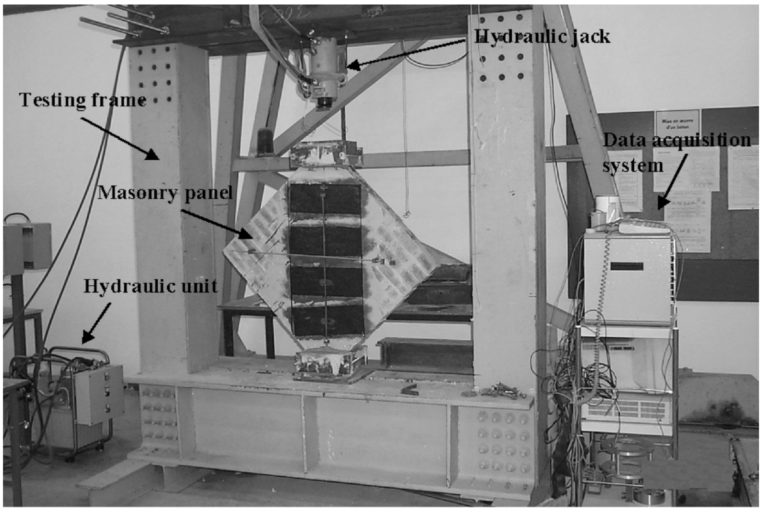

Fig. 7 Experimental set-up for diagonal compression test on masonry walls

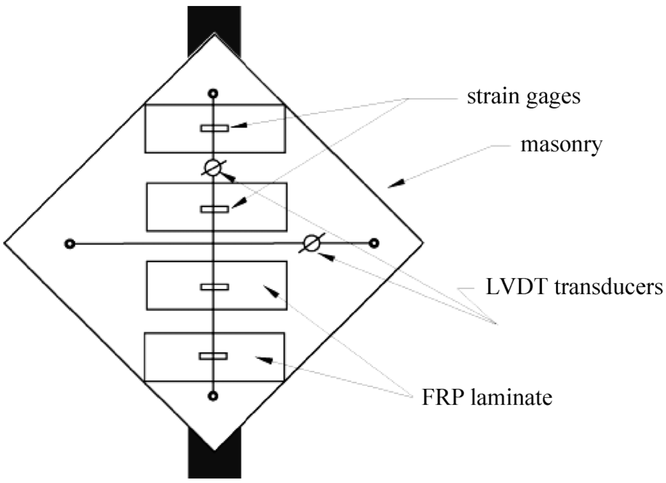

Fig. 8 Instrumentation of masonry panels

4.1 Unreinforced panels

The two unreinforced panels (denoted NR1 and NR2) presented a brittle failure along the compressed diagonal (Fig. 9). Cracking appeared suddenly in the mortar joints and in the bricks, producing the instantaneous failure of the masonry walls. In order to analyze the global behaviour of the unreinforced panels we represent the force-strain curves along the two diagonals (Fig. 10 and

Fig. 9 Failure pattern of unreinforced masonry panels

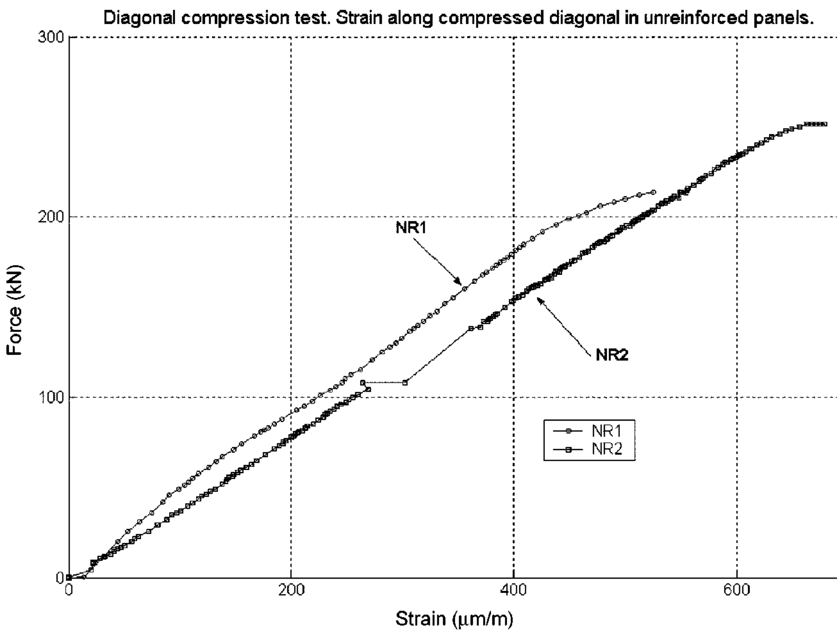

Fig. 10 Strain evolution along the compressed diagonal of the unreinforced masonries NR1 and NR2

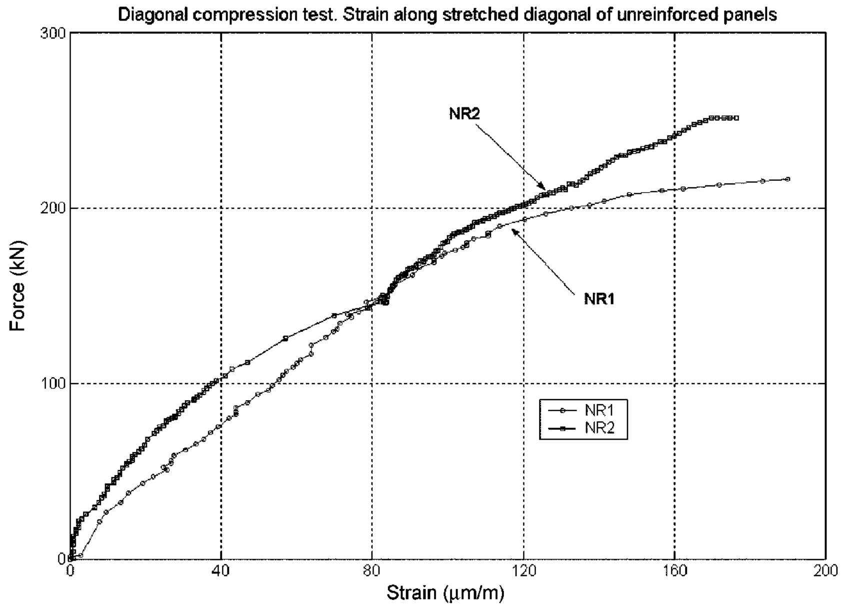

Fig. 11 Strain evolution along the stretched diagonal of the unreinforced masonries NR1 and NR2

Table 3 Experimental tests results

	Reinforcement type	Failure load (kN)	Failure mode	Ultimate strain (μm/m)
NR1	-	215.3	Diagonal splitting	470
NR2	-	251.8	Diagonal splitting	700
RFV	1D glass fiber	332.0	Splitting	1500
RFC	1D carbon fiber	361.0	Crushing	1500
RFW	2D glass fiber	384.0	Crushing	1800

Fig. 11). The strain is computed as the ratio of the displacement measured by the LVDT devices and the initial distance between the two points of measurement. These curves show a quasi-elastic behaviour with a weak yield plateau along the compressed diagonal and a more pronounced non-linear behaviour along the stretched diagonal. This is a consequence of the shear phenomenon in joints. The measured failure loads are 215.3 kN and respectively 251.8 kN (see Table 3).

The difference between the ultimate loads is relatively important (17%) but it is explained by the internal structure of the walls. In fact, the failure strength is conditioned by the shear strength induced by the interaction of mortar notches at the brick/joint interface. The random distribution and size of mortar notches in the hollows affect the shear strength, as it was observed on the triplets during material characterization. These tests showed dispersions of the same order.

4.2 Reinforced panels

Main features of the global behaviour of reinforced masonry panels can be highlighted on force-strain diagrams.

First, consider the force - strain diagrams of the tested masonry walls (Fig. 12 and Fig. 13). NR1,

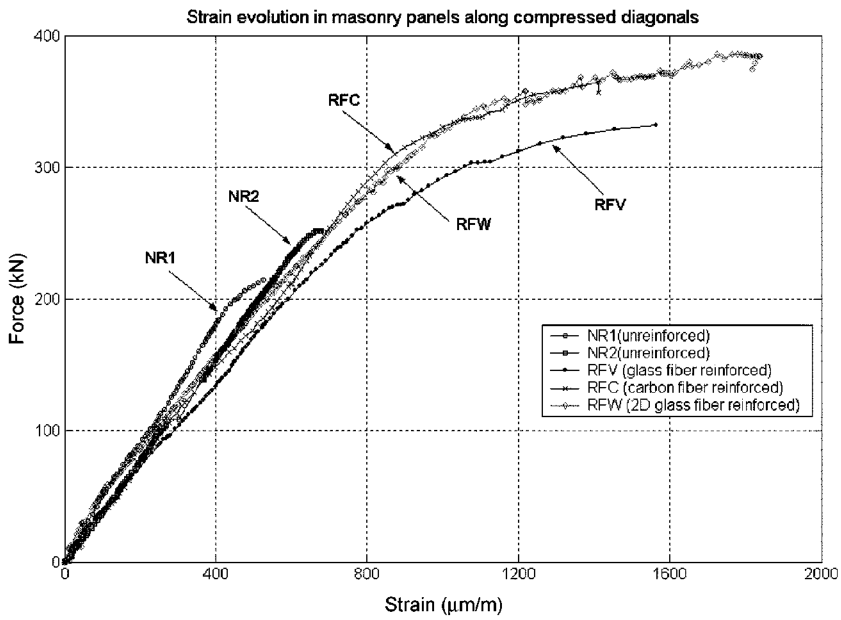

Fig. 12 Comparison of the global behaviour of unreinforced and strengthened panels. Strain evolution along compressed diagonals

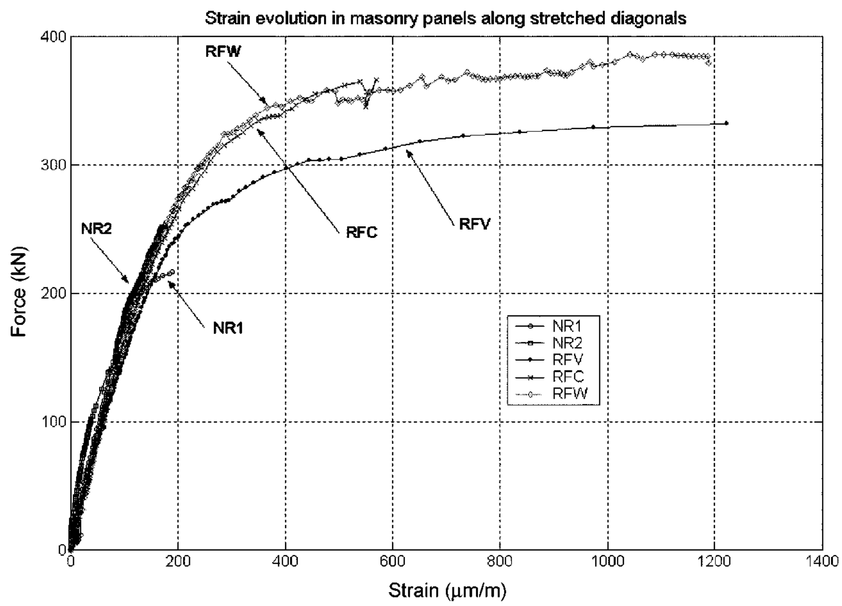

Fig. 13 Comparison of the global behaviour of unreinforced and strengthened panels. Strain evolution along stretched diagonals

NR2 denote the two unreinforced masonry walls while RFV, RFC, and RFW denote the three reinforced walls. The diagrams underline two stages of the global behaviour: a first elastic and a

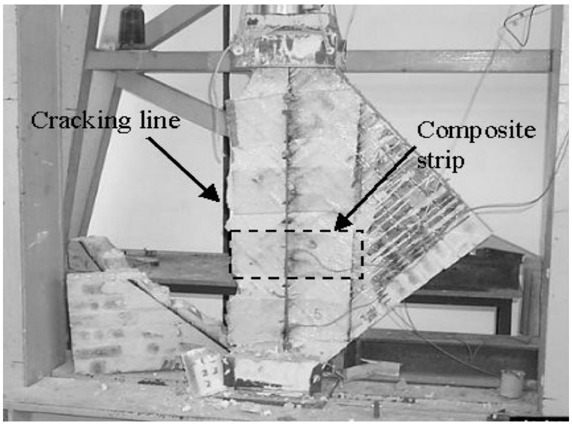

Fig. 14 Failure pattern for the masonry wall strengthened with RFV composite

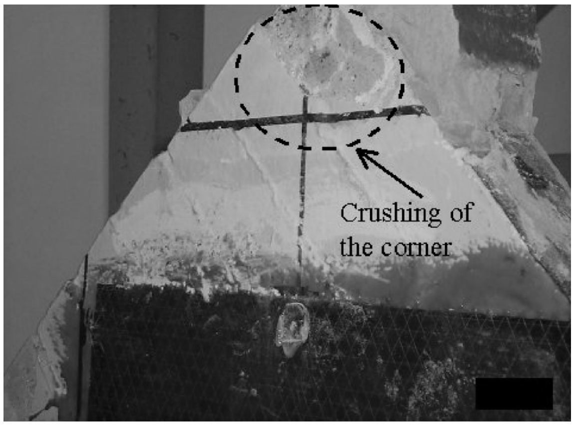

Fig. 15 Failure pattern for the masonry walls strengthened with RFC and RFW composites

second plastic. On the one hand, the elastic phase of the curves of reinforced panels are characterized by the same slope as those of unreinforced, regardless to the type of the composite. On the other hand, the load corresponding to the elastic limit and the ultimate load of the reinforced panels are much higher than that of the unreinforced panels. The gain in strength is quite remarquable: 42% for the RFV reinforcement and over 65% for the RFW. Thus, a first consequence of the reinforcement is the growth of the strength of the wall while its initial in-plane stiffness is kept unmodified.

Moreover, we remark an important deformation capability of the reinforced walls, emphasized by the presence of a relevant post-elastic plateau. The deformations corresponding to the maximum loads of the reinforced walls are three times higher than those of the unreinforced. Results are summarized in Table 3.

The failure modes observed for the three walls are as follows. The panel reinforced with the RFV composite failed suddenly due to a cracking along the compressed diagonal at the ends of the composite strips (Fig. 14). The two other walls, strengthened with RFC and RFW strips, have failed locally at the compressed corners, in the loading shoes (Fig. 15). In this latter case, the tests were stopped when debris detached from the loading shoes.

We must mention that the load-controlled test is deficient in the evaluation of the post-peak resources of the structural elements. However, in the case of the masonry walls submitted to a predominant shear load the post-peak deformation is quite low: the ductility coefficient is often considered equal to 1 (FEMA306 1999), (Paulay and Priestley 1992). Thus, in a first approach, the comparison of the behaviours of the unreinforced and reinforced panels considering only the behaviour until the maximum supported load can be useful. As results show, the deformation capability is increased in a significant manner: for two panels the ruine is avoided.

In order to evaluate the effectiveness of the composite reinforcements, we compare for every reinforced panel the measured strain in the masonry along the diagonal (denoted “horizontal strain”) and the strain in the reinforcement strips (denoted “gauge1”, ..., “gauge4”), in conformity with Fig. 16, Fig. 17 and Fig. 18.

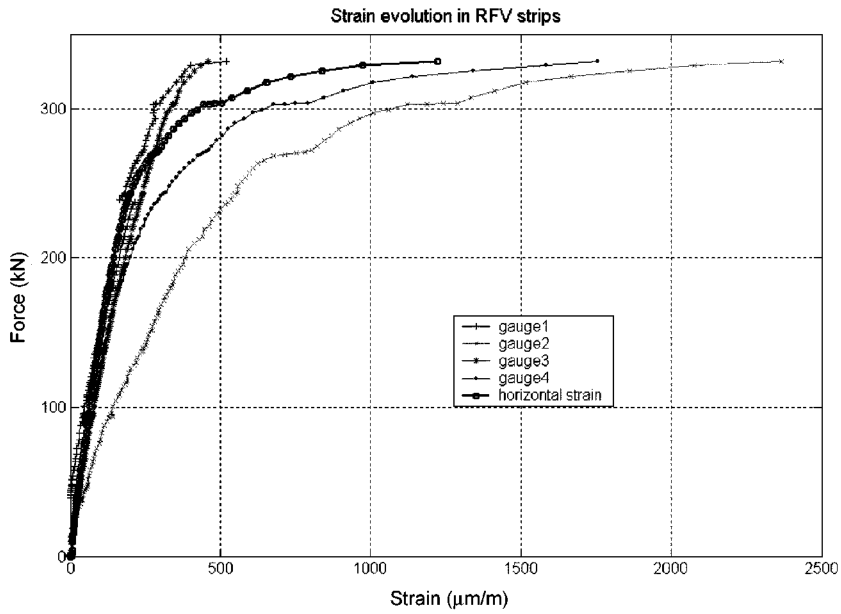

Fig. 16 Strain evolution in the RFV composite strips

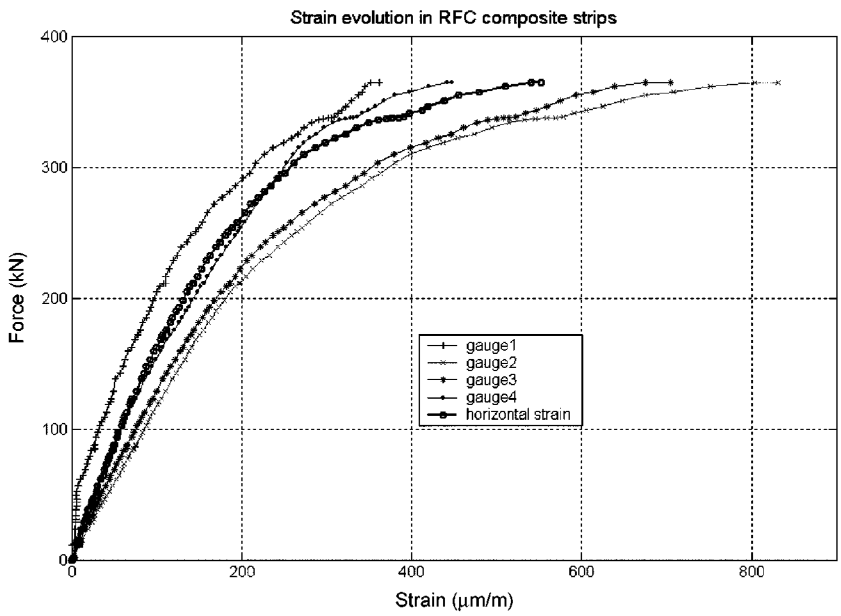

Fig. 17 Strain evolution in the RFC composite strips

We remark that the strain in RFV and RFC strips is non-homogenous, the strips are less or more solicited in function of their position and the local weakness of the masonry. Thus, a strain state can occur in the masonry, between two strips, which is susceptible to produce the failure of the masonry. This is not the case of the masonry strengthened with the RFW sheet, where the strain state is homogenous on the height of the wall. In terms of ultimate load and strain this type of

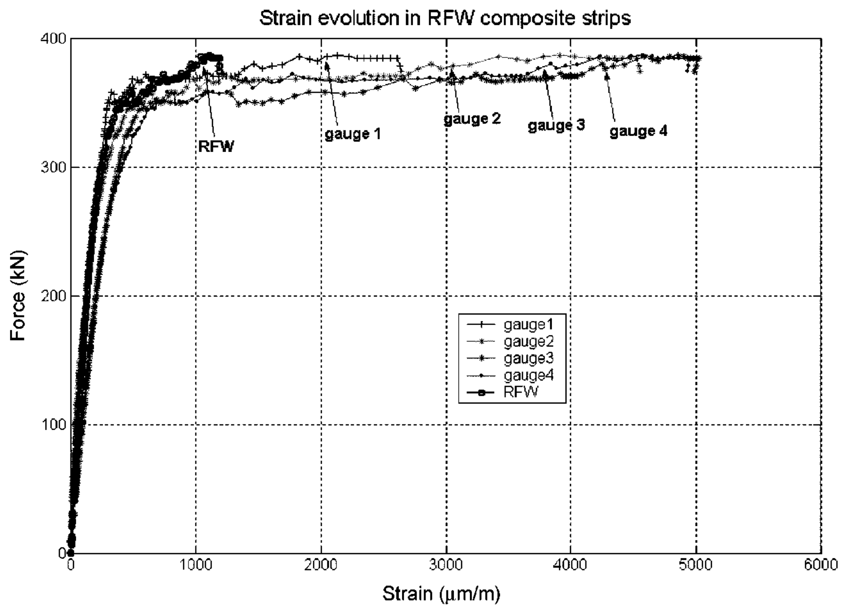

Fig. 18 Strain evolution in the RFW composite sheet

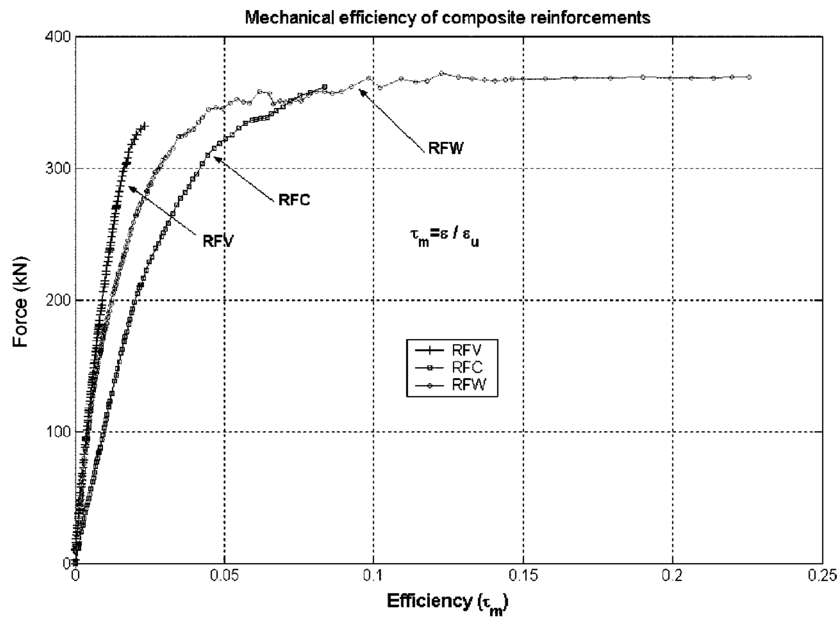

Fig. 19 Mechanical efficiency factor of RFV, RFC and RFW composites

composite is more efficient then the high strength and high resistance composites (RFC or RFV). These facts highlight the importance of reinforcing the entire solicited zone of the masonry panel. Moreover, bi-directional composites are better adapted than the unidirectional ones because of the multidirectional strain state induced by the local geometrical arrangement of the masonry.

Considering the mechanical efficiency factor (τ_m), defined as the ratio between the actual strain in the composite (ε) and its ultimate strain (ε_u), we can remark that it is very small (Fig. 19): about 2.5% for the RFV, 9% for the RFC and respectively 25% for the RFW composite. Observe, that again, the use of RFW composite is more efficient than the use of RFV or RFC. This efficiency factor is in the same range as that described by other authors (Mingo *et al.* 2001).

Even if the number of tested panels is not exhaustive, we can conclude that the ultimate strength of the panels increases significantly due to reinforcement. The failure is caused by the excess of the compression strength of the masonry in regions where this type of load is predominant.

5. Finite element modelling of the behaviour of masonry panels

Nowadays, finite element analysis of unreinforced masonry panels is principally devoted to the development of reliable interface models via adapted constitutive laws or incorporating fracture mechanics and plasticity concepts. The approach developed by Lourenço (1996) considers the mortar joint as the weakest element of the brickwork, where all type of plastic deformation take place. The mortar joint is modelled by an interface element, using multisurface plasticity in order to describe compression, shear and tensile behaviour. A similar approach is developed in Gambarotta and Lagomarsino (1997) using a composite model considering a mortar joint interface model and an elasto-plastic behaviour of the units. The constitutive equation of the mortar joint interface is given in terms of two internal variables representing the frictional sliding and the mortar joint damage. The bricks are modelled as elasto-plastic solids having brittle interfaces located at the head joints of the neighbouring layer. This model has a brittle response under tensile stresses and presents frictional dissipation possibilities together with stiffness degrading under compressive stresses. These characteristics are used to describe the hysteretic in-plane behaviour of masonry walls.

In order to reduce the complexity of the above-mentioned techniques homogenized models are built. From this point of view, we can mention the model based on homogenized anisotropic elasto-plasticity (Lopez *et al.* 1999). The model is built considering the equilibrium and compatibility equations of a basic cell under elementary load. These equations are introduced in the constitutive equations of each material component, leading to the establishement of the homogenized stress-strain relations. The plastic behaviour of the homogenous material is described using modified Mohr-Coulomb plasticity. The effect of anisotropy is introduced by means of fictitious isotropic stress and strain spaces, which are determined using the theory of mapped spaces.

One of the main messages of these works is that significant information on the real behaviour of masonry walls can be obtained only by using a micro-mechanical approach. The global behaviour has to be evaluated on the basis of local mechanical parameters of the masonry. The developed models are quite complex and deal with most of the mechanical phenomena that occur in masonry, but they are not implemented on widely distributed commercial codes.

We implement a two-dimensional finite element simulation by means of a commercial code (ANSYS) that fits the specific frame of the shear soliciting of masonry walls. The final goal is to evaluate the strain and damage distribution in the unreinforced and FRP strengthened panels when they are submitted to a predominant shear load generated by the diagonal compression test.

This approach considers fully elastic bricks with an appropriate elasto-plastic model for the mortar joint. In fact, the non-linear shear behaviour of the masonry panels is mainly governed by the phenomena that occur at the brick/mortar interface (the interaction of mortar cores with the internal walettes of the bricks). However, the introduction in the model of elasto-plastic interface elements increases its complexity: the construction of the model and calculation become lengthy. Thus, we transposed the non-linearity of the brick/mortar interface to the behaviour of the mortar joint supposing that this artifice has no effect on the global behaviour, given the low volume of mortar compared to the volume of bricks. The comparison of numerical and experimental results presented below shows that this choice is valid.

The masonry model is built as a material realized by a regular inclusion of bricks into a matrix of mortar. The modelling lies on the previously determined mechanical parameters of constituants in compression and shear. Therefore, we can consider the apparent dimensions of bricks (210 × 50 × 100 mm) and the apparent thickness of 10 mm of the mortar joints. The mortar is considered as a net that perfectly bonds to bricks. The geometrical configuration and boundary conditions are identical to the real ones (Fig. 20). A plane stress modelling is pursued using four node standard elements having two degrees of freedom per nodes (plane42), four Gauss integration points and Lagrangian polynomials as shape functions. The relative dimensions of units and mortar impose mesh size: the size of the elements of the mortar is uniform and equals to the thickness of the joint, whereas element size of bricks becomes coarse in their interior (Fig. 21).

The chosen constitutive law for the modelling of the mortar joint is elastic-perfectly plastic in a Drucker-Prager formulation. This implementation lays on the Mohr-Coulomb mechanical parameters that have been experimentally determined and summarized in Table 1: the shear strength (cohesion), the residual friction coefficient and the dilatancy angle (considered equal to zero in the model).

The reinforced masonry panel is obtained by "bonding" the sheets of composite onto the masonry. The perfect bonding is performed by coupling the coincident nodes of the mesh of the masonry with the nodes of the composites strips. The behaviour of composite sheets is considered as elastic and characterized by the experimentally determined moduli. The composite strips are modelled with elements admitting membrane stiffness and tension-only option (shell41). This is a 3D element, characterized by four nodes and three degrees of freedom per each node, four Gauss integration points and Lagrangian polynomials as shape functions. The degrees of freedom of the nodes of composite strips in the third direction, perpendicular to the masonry panel, are blocked. The thickness of the element is considered equal to 1 mm for the three types of reinforcement.

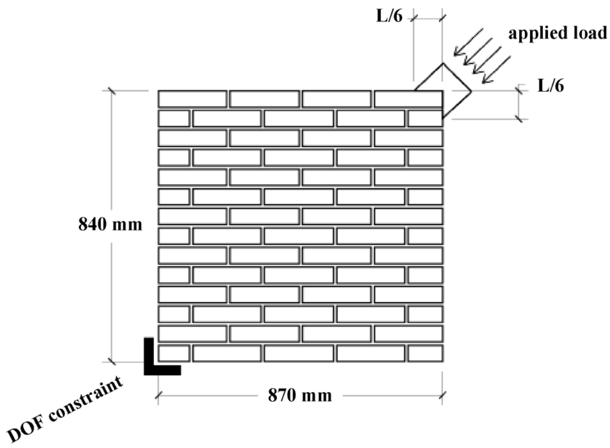

Fig. 20 Geometrical configuration of a masonry panel

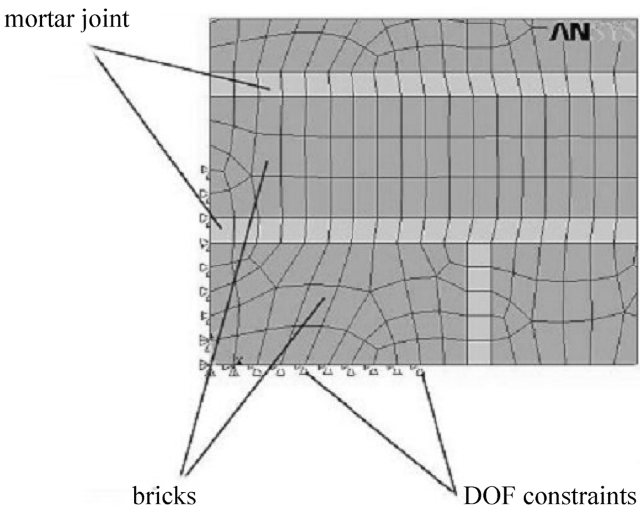

Fig. 21 Meshing detail of the masonry panel

5.1 Unreinforced masonry panels

The analysis of strain and stress distribution reveals that plastic strain appears and grows in the center of panels (Fig. 22). We can consider that failure occurs in this zone because of excess of the strain in the mortar joint. In order to evaluate the global response of a masonry wall, we represented in Fig. 23 and Fig. 24 the experimental and numerical force-strain diagrams. We remark that the modelling approaches quite well the global behaviour of the masonry in terms of stiffness and maximal force. When elastic limit is reached, we remark a sudden change of the global stiffness

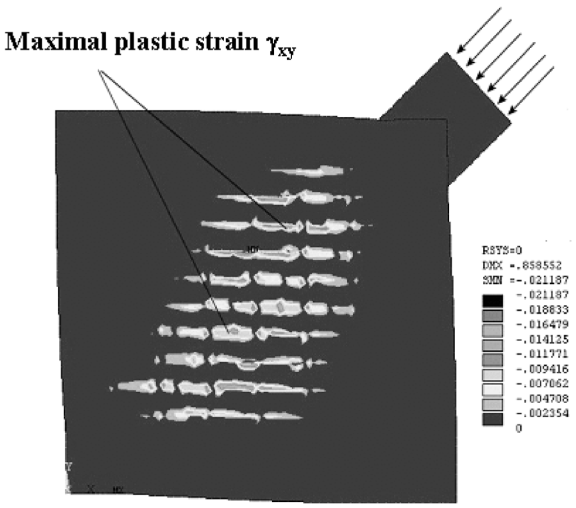

Fig. 22 Plastic shear strain distribution in unreinforced masonry panel

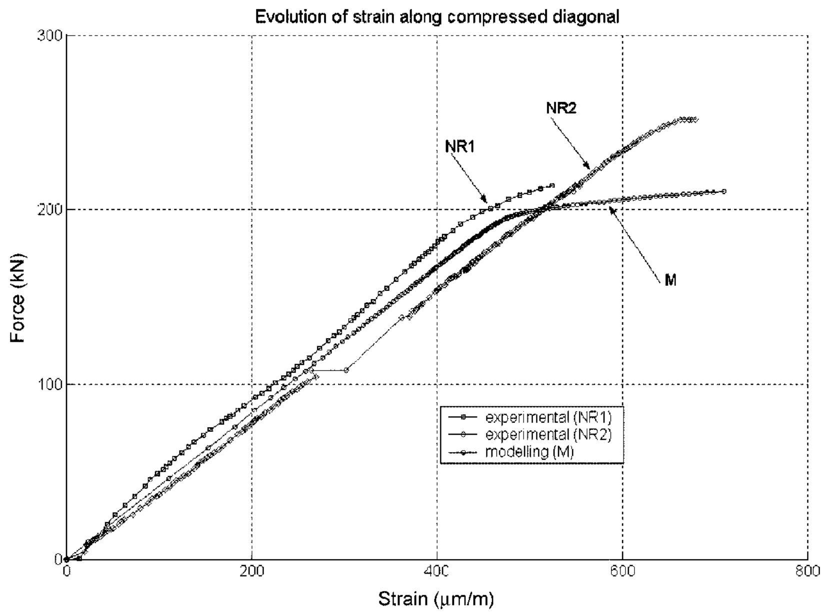

Fig. 23 Comparison of experimental and numerical results for the unreinforced masonry panels. Force-strain diagram for the compressed diagonal

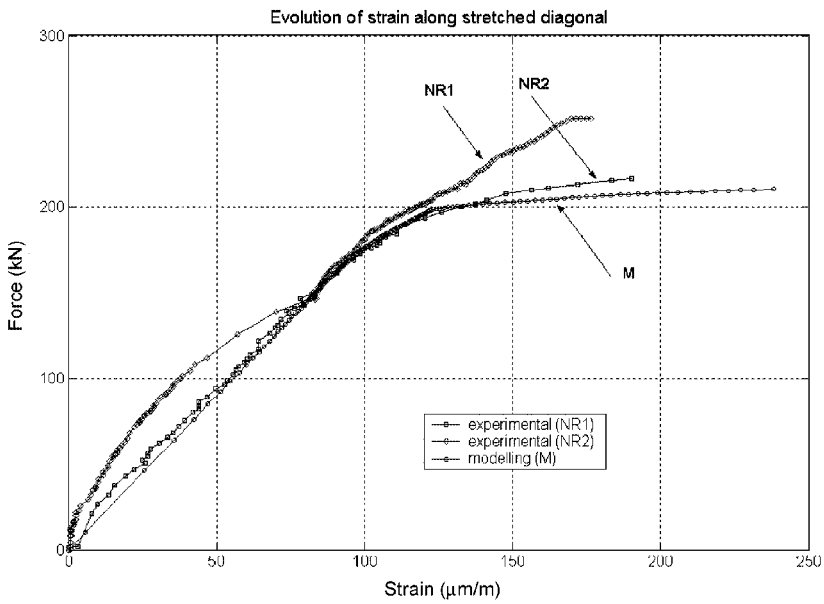

Fig. 24 Comparison of experimental and numerical results for the unreinforced masonry panels. Force-strain diagram for the stretched diagonal

that predicts the degradation of the mechanical properties and the failure. The differences between experimental and numerical results are in the range of spreadings of the experimental values obtained for the mechanical parameters.

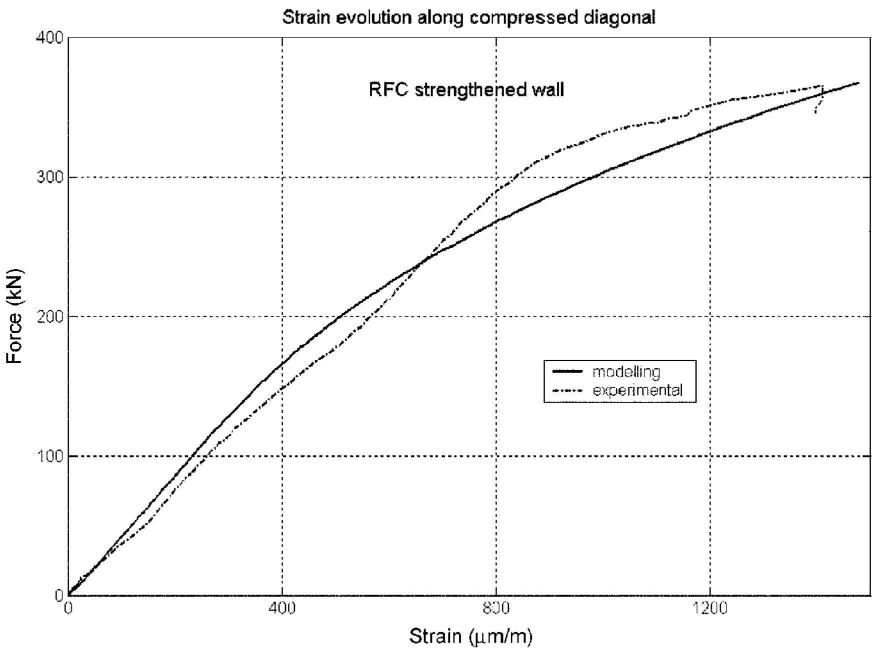

Fig. 25 Comparison of numerical and experimental results for the masonry panel strengthened with RFV composite strips. Force-strain diagram of the compressed diagonal

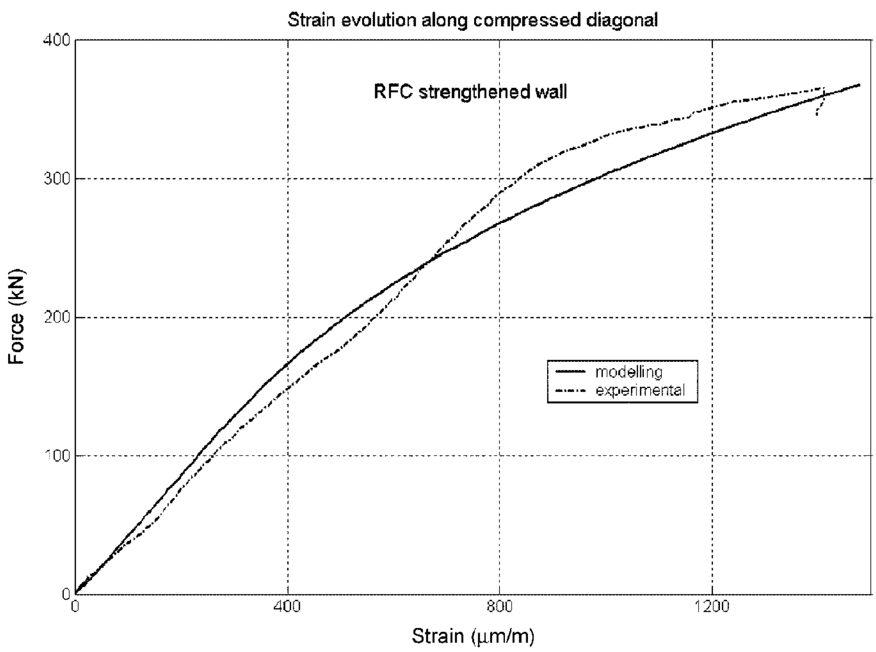

Fig. 26 Comparison of numerical and experimental results for the masonry panel strengthened with RFC composite strips. Force-strain diagram of the compressed diagonal

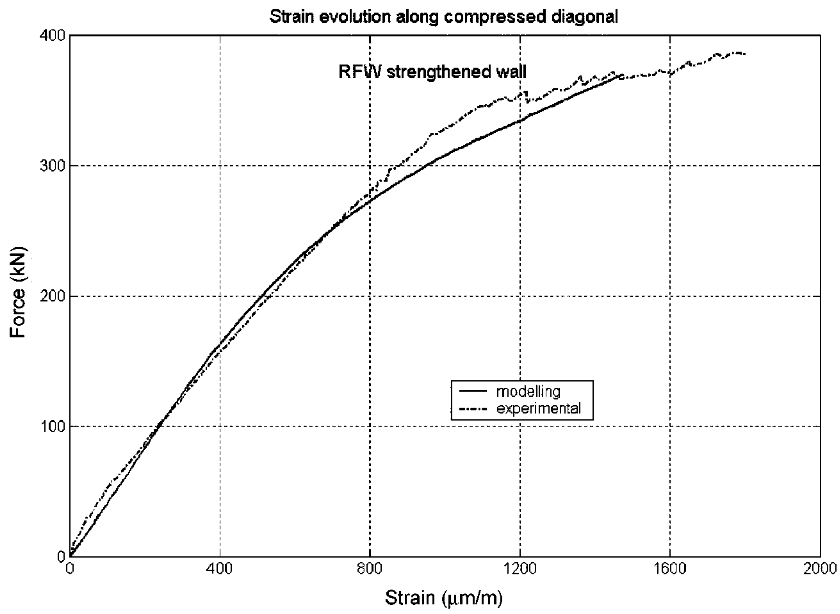

Fig. 27 Comparison of numerical and experimental results for the masonry panel strengthened with RFW composite sheet. Force-strain diagram of the compressed diagonal

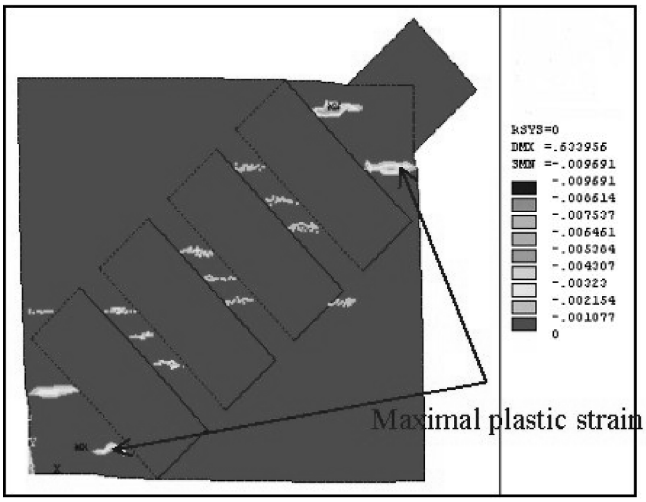

Fig. 28 Distribution of plastic shear strain in the masonry walls strengthened with RFV and RFC composites

5.2 Reinforced masonry panels

The comparison of experimental and numerical curves for the three types of composite shows a quite good capacity of the modelling in the evaluation of the global behaviour of the strengthened walls (Fig. 25, Fig. 26 and Fig. 27). The evaluation of elastic domain is delicate for the RFV and RFC reinforced walls because numerical curves are rather parabolic while the experimental ones are

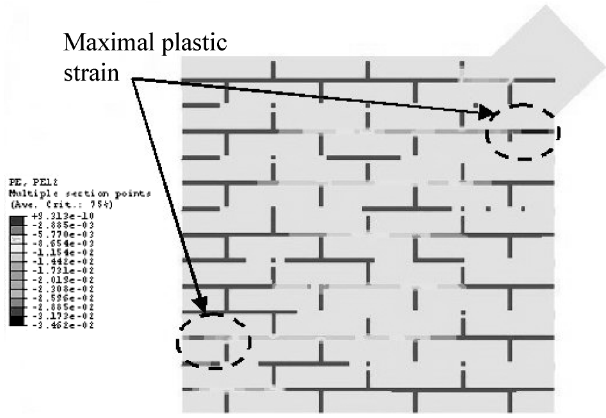

Fig. 29 Distribution of plastic shear strain in the masonry wall strengthened with RFW composite

linear. However, the slope of the curves is in good correlation. For the RFW reinforced wall the two curves coincide.

Zones with maximal strain for the walls reinforced with RFV and RFC composites appear in the neighborhood of the corners (Fig. 28). Besides, we observe the appearance of a less intense plastic strain in the mortar joints situated between the composite strips. Thus, we can consider that failure occurs in the compressed zones near the supports. In the case of the RFW reinforced wall (Fig. 29) we note the same distribution tendency of plastic deformation near the corners.

These facts agree with the experimental observations both at the level of global response and at the level of failure modes. Moreover, the modelling confirms the same behaviour of unreinforced and strengthened walls for the elastic domain: the reinforcement contributes to the increase of the strength of the masonry, but it does not modify the in-plane stiffness of the wall.

6. Conclusions

In the first part of the paper we presented an experimental study of the in-plane shear behaviour of FRP composite strengthened masonries. The obtained results revealed the capacity of the FRP reinforcement to ameliorate the behaviour of the masonry walls. The FRP provided not only a better resistance (increase up to 60%) but also a significant deformation capability (up to 3 times). Moreover, for the elastic phase, the reinforcement does not modify the in-plane stiffness of the wall, which is beneficial to the masonry structure when submitted to earthquake loadings. These results agree with those found by Corradi *et al*. (2002) who studied the influence of composite reinforcement on the in-plane behaviour of stone masonry panels.

When the behaviour of the masonry panel is dominated by shear phenomena then the prevailing failure mode is that of diagonal fissuring. The reinforcement must be placed in directions normal to the cracking line. In this context, the unidirectional composite strips allow the increase of the strength of the masonry panel. However, their effectiveness is limited: the composite strips have a bridging effect, ensuring only the local reinforcement of the masonry. This explains the failure at the end of the RFV composite strips. Therefore, the reinforcement employing bi-directional composites and applied on the entire surface becomes very interesting: the problem of the local effectiveness is eliminated. Moreover, the fact to apply bi-directional composites is necessary since loads generated by earthquakes produce alternatively compression efforts along the two diagonals of a masonry panel.

In the second part of the paper we presented a finite element modelling approach for the study of the behaviour of unreinforced and strengthened masonry under predominant shear stress. The accomplishment of the modelling in the prevision of the behaviour of walls depends on the right choice of implemented mechanical parameters. The principal parameters of the chosen formulation have been the elasto-plastic properties of the mortar joint: cohesion and residual friction. The obtained numerical results have been validated experimentally in the case of diagonal compression test of masonry panels. Comparing the numerical and experimental results, we remark that finite element modelling gives a realistic image of the behaviour of masonry panels: ultimate loads, plastic strain evolution and failure modes are rendered with good approximation. The finite element modelling can be a helpful tool when the reinforcement of these structures with FRP composite is proposed. The knowledge of failure and fissuring zones permits the optimization of the placement of composite strips in order to have maximal efficiency, if such reinforcement is used.

For the reinforced panels with unidirectional composites, the numerical simulations show that plastic deformations can develop in the masonry joints of the central zone of the panels, between the composite strips. As the strips are close one to another the failure along the diagonal is avoided since the strips block the development of plastic strains. In the case of the panel reinforced with bidirectional composite, the failure zone is located at the compressed corners. These results agree with those experimental and the results of the modelling pursued in Valluzzi *et al.* (2002).

To these observations we can add that the employment of composite reinforcements having important deformation capability is useful. On the other hand, the use of composites with high elastic modulus and high strength is not needed. In this way the reinforcement with FRP composites of masonry structures could be an economic option.

The shear behaviour of masonry walls is important when the structure is submitted to important horizontal loads having their sources in earthquakes or winds. Further numerical and experimental research will be conducted in order to evaluate the retrofit capabilities of FRP reinforcements in the case of masonry walls submitted to cyclic shear loads.

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