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STRUCTURAL ANALYSIS OF TRANSVERSAL STEEL CONNECTORS APPLIED ON MULTI-LEAF WALLS

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ABSTRACT

Over the last fifteen years, new techniques and materials have been used to retrofit masonry structures for improved seismic performance as well as a variety of other strengthening applications. The global behavior of a stone masonry wall is often governed by the level of connection between masonry leaves and the overall quality of the masonry material (mortar, block and arrangement). This paper presents the results of an investigation carried out on multi-leaf stone masonry panels retrofitted using stainless steel rod inserted in a grouted fabric sleeve. The paper also reports the results of a non-linear numerical investigation calibrated using laboratory tests. Several wall panels were assembled in the laboratory using solid calcareous stones and weak mortar and the effectiveness of the connectors was tested in shear and compression on both virgin and damaged wall panels. Experimental results show that a substantial improvement of the panels' mechanical behavior can be achieved by applying transverse connectors. The feasibility of using the 3-Dimensional (3D) finite element model to analyze multi-leaf walls reinforced with transverse connectors is examined by comparing the model to experimental data.

KEYWORDS: *connections, historic masonry, multi-leaf walls, numerical analysis*

INTRODUCTION

As early as the mid 1990s, civil engineers have been researching and utilizing the many unique material advantages of *Fiber Reinforced Polymer* (FRP) composite materials in a variety of applications on historic masonry structures. The majority of extant experimental work investigating the retrofit of walls using these new materials has been conducted on wall panels

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having an FRP material bonded to the panel's surface using inorganic (lime- or cement-based mortars) or organic (epoxy, polyester resins) matrices [1-3]. This method of FRP application has been shown to be a feasible method for upgrading the lateral (shear) strength and stiffness of cracked or uncracked (virgin) walls [4-7]. The in-plane behaviour of wall panels received significant attention and several studies addressed the problem of the behavior of historic masonry when in-plane loaded. All investigations conducted using specimens reinforced with the FRPs having no additional anchorage other than the adhesive resulted in specimens failing through some form of debonding of the FRP or, more frequently for multi-leaf walls, by separation of the wall leaves. For a single-leaf wall, the most commonly reported debonding mode is the peel mode which initiates where the bonding stresses are maximum (at the panel's center) and progresses through the mortar/plaster toward the edges of the panel. If the FRP is directly bonded to the bricks/stones constituting the wall panel using high-strength resins (i.e. epoxy), the peel mode is still frequent and failure occurs for cracking in the bricks/stones with the expulsion of small thin layers of brick/stone material bonded to the FRP.

Despite the prevalence of the end peel mode in the literature, it is believed that the leaf-separation mode is just as relevant in "real life" applications due to often common presence of multi-leaf walls in historic constructions [8-12].

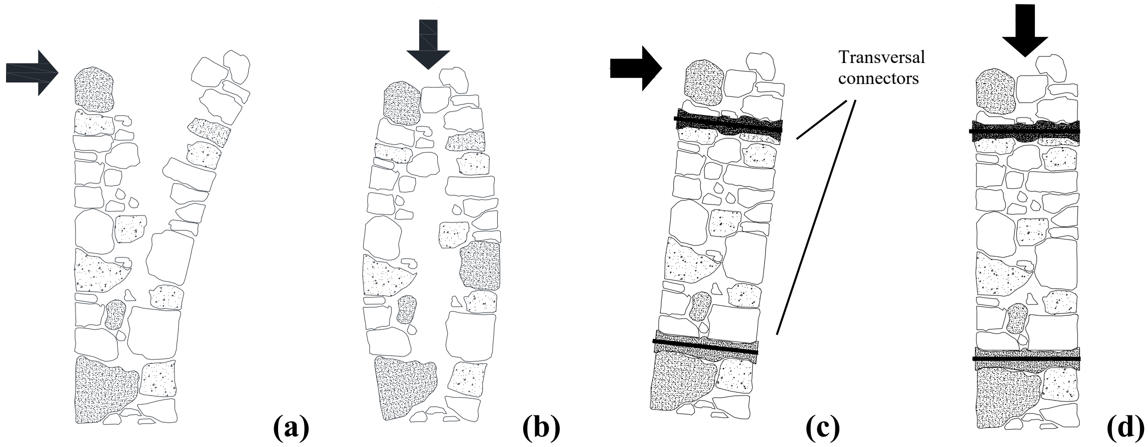
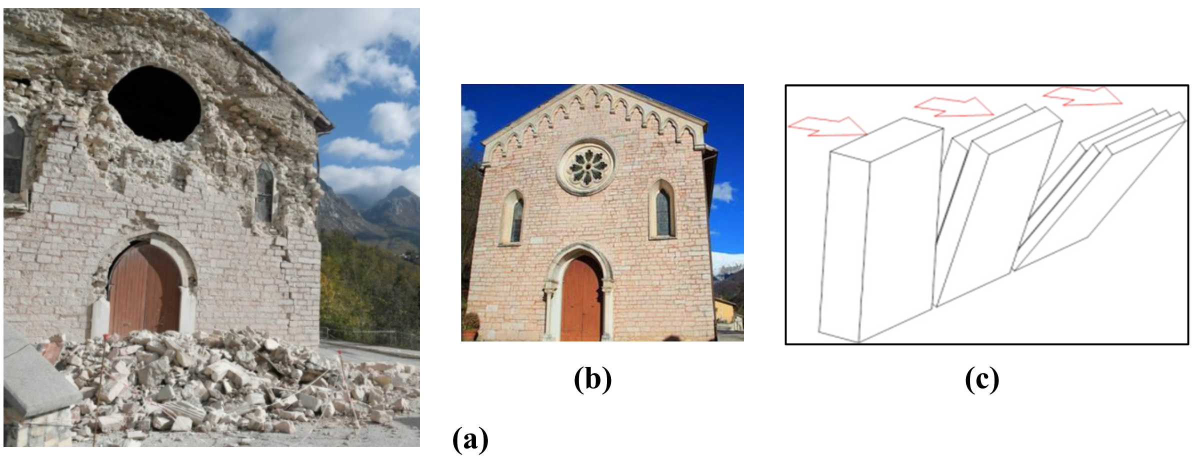


Figure 1: Failure Mechanism of a Double-Leaf Wall Panel: (a) Unreinforced: Subjected to a Horizontal (Out-of-Plane) Load; (b) Unreinforced: Subjected to a Compressive Vertical Load; (c) Reinforced: Out-of-Plane; (d): Reinforced: in-Compression

A relatively small number of studies investigating the behavior of multi-leaf walls (mainly made of stone masonry) have been conducted. The consensus is that the behavior of such wall specimens is governed by the mechanical interlocking between the wall leaves and by the presence of large connecting stones (*headers, diatoni* in Italian) placed transversally to the surface of the wall panel during construction [13-14]. However, the presence of these connecting stones in ordinary historic constructions is rare and the load's sharing between the wall leaves is only controlled by the quality of the mechanical interlocking and bond between the leaves. The ability to transfer stress from the one wall leaf to another depends only on the mechanical characteristics of the old mortar and on the mechanical interlocking between the stones of the adjacent leaves.

The out-of-plane behaviour of historic wall panels received less attention with few studies present in the literature. The analysis of the level of connection between adjacent wall leaves is critical as these walls tend to separate when loaded out of their plane. Multi-leaf stone masonry walls are very common in historic constructions and their behavior under in-plane and out-of-plane actions is usually unsatisfactory (Figs. 1-2). The presence of headers may significantly increase the mechanical performance of such multi-leaf walls. Several studies have addressed the importance of transversal connections and their considerable effect on the out-of-plane behavior [15-19].

Multi-leaf walls poorly behave when also loaded in compression. Compressive loads may produce buckling problems and again result in the separation of the adjacent wall leaves (Fig. 1b). However high compressive loads are rare on historic constructions and this problem is limited to tall structures (bell and civic towers, lighthouses, etc.).



(b) (c)

(a)

Figure 2: Examples of Out-of-Plane Collapse Mechanisms: (a) The Parish Church of Ussita (Italy) after the 2016 Umbria Earthquake (Italy), (b) Before the Seismic Event, (c) Schematic Representation of the Behavior for a Single and a Multi-leaf Wall Panel

A possible solution is to insert new headers (Figs. 1c, 1d and 3). This could improve the global behavior of multi-leaf walls. This insertion could be made using new stones or bricks, or with steel or RC (Reinforced Concrete) elements. However limited research has been done in this area, mainly for the difficulty in reproducing historic masonry and in performing tests able to measure the effect of the insertion of new headers [20-22]. This insertion is a difficult task since hollow steel tubes (core drills) are often used to drill holes in the masonry material: this causes a stress re-distribution in the area around the hole leaving this area uncompressed. Without an adequate interlocking, the new connector cannot act efficiently and its application could result ineffective.


(a)


(b)

Figure 3: The connector: (a) Before Mortar Injection; (b) After Insertion and Injection

THE REINFORCEMENT SYSTEM

The artificial connector consists in a stainless treaded rod inserted in a fabric sleeve (Fig. 3a) and injected with a high-strength cement-based grout into holes previously drilled in the wall (Fig. 3b). Rod and a fabric sleeve are then inserted transversally from one side of the wall only. In order to activate a mechanical interlocking between the fabric sleeve and the pre-existing wall, the hole can be countersunk at both ends. Fabric sleeve is typically injected with grout using a pressure of 2.5-5 atm. Steel rods can be also post-tensioned up to a 10-20% of the steel yielding strength in order to confine the wall leaves and increase collaboration.

The diameter of the connectors and of the holes, the mutual position among tie-rods are crucial to the effectiveness of the reinforcement method. This depends on many factors, like the masonry arrangement and typology, the thickness of the wall panels and the mechanical properties of masonry and it is difficult to provide values that can be applied for design purposes given the masonry's variability. The main aim is to avoid stress concentrations in the area around the connectors and prevent all slippage phenomena at interface rod-grout and grout-masonry. This has been partially studied and calibrated by performing pull-out tests: a tensile force has been applied on-site on the connectors with different diameters and lengths. The results of these tests demonstrated that, for historic hard-stone masonry (barely cut stones and pebbles assembled with lime-based mortars), the following parameters may be applied: hole's diameter / connector's diameter = 3-4, mutual connectors distance/hole diameter = 9-11, hole depth / hole diameter = 7-8. Typical multi-leaf stone masonry walls with thicknesses varying between 400 and 700 mm were bonded using 16 or 20 mm diameter connectors, by doing 40 or 64 mm diameter holes and placing connectors at a mutual distance of 450-700 mm. The results of a large number of pull-out tests showed that the typical failure mode was the bar yielding. No interaction has been noted between adjacent connectors, when tested in tension.

LABORATORY TESTS

The effect of the application of transversal connectors has been studied by performing in laboratory three compression tests on two double-leaf wall panels, assembled in the structures laboratory of the University of Perugia (Fig. 4). The two full-scale panels had identical dimensions of 0.22 x 0.71 x 1.45 m (thickness x width x height). A H-shaped steel profile (flange's width = 250 mm) was placed over the panel to achieve a uniform distribution of the vertical load along the panel's width. This was applied using two 50-tons hydraulic jacks. The panels were loaded at a rate of approximately 4 kN/min up to failure. The vertical deflections of both wall leaves were measured using contact instrumentations (LVDTs: Linear Variable Differential Transformers).


(a)


(b)

Figure 4: Wall Panels under Construction: (a) Detail of the Horizontal Cross Section; (b) Lateral View

Both oil pressure at the jacks (compressive load) and vertical deformations were digitally recorded using a four channel data acquisition system operating at a frequency of 0.5 Hz. A low-strength cement-based mortar was used for panel construction (mortar's compressive and bending strengths are 1.92 and 0.292 MPa, respectively). Test results are presented in detail in [19].

Table 1: Results of Compression Tests

	No. of Connectors	**Rod and Hole Diameter** (mm)	**Maximum Load P** (kN)	**Max Compressive Stress σ** (MPa)	**Young's Modulus E** (MPa)
Panel 1Un	-	12-30	48.6	0.312	359.3
Panel 1Rp	6	12-30	91.1	0.583	356.2
Panel 2Rf	6	12-30	101.5	0.650	285.8

Three wall panels were tested in uniaxial compression. For the first test (Panel 1Un) a maximum compression load P of 48.6 kN was recorded, corresponding to a compressive stress of 0.312 MPa (σ=P/A) (A=220x710 mm). By analysing the data recorded by the LVDTs it is possible to note that the two adjacent masonry leaves deformed differently: the maximum (vertical) axial strain ($\varepsilon=\Delta L/L$ where the gage length was L=1450 mm) was 0.0011 for side B, while it was only 0.0006 for side A (Fig. 5a). This demonstrated that the weak connection, assured by the mortar, between the leaves was not able to guarantee a uniform distribution of the vertical load and axial strains. This panel was also repaired using 6 traversal connectors applied according to the scheme in Figure 4.

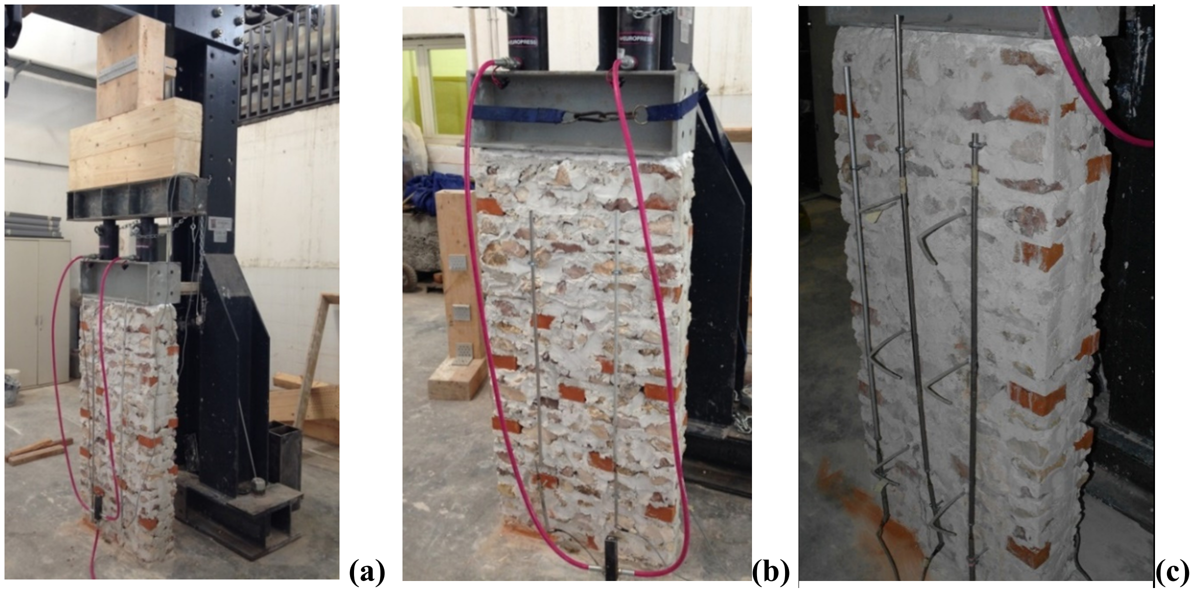


Figure 5: Lab Tests: (a) Unreinforced Panel (Panel 1Un); (b) Detail of the Position of LVDTs; (c) Arrangement of the Connectors

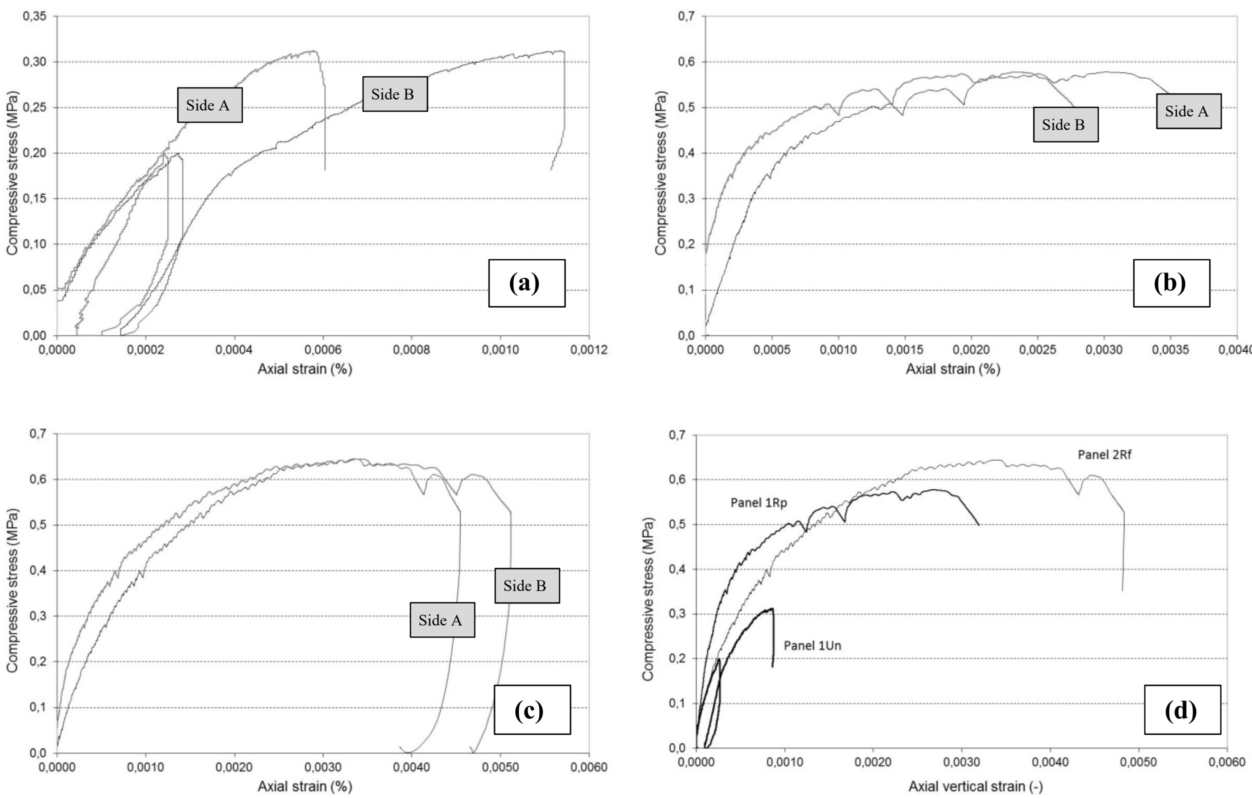


Figure 6: Compressive Stress σ vs. Axial Strain: (a) Panel 1Un; (b) Panel 1Rp; (c) Panel 2Rf; (d) Comparison (Average Strain Values)

The test on the repaired panel (Panel 1Rp) showed a capacity of 91.1 kN, corresponding to a compressive stress of 0.583 MPa. The application of the connectors not only restored the original (uncracked) capacity of the wall panel, but increased it of approx. 87%. Several parallel vertical cracks opened at failure on both panel's sides. The load started decreasing slowly and a clear non-linear behavior occurred: this is the typical masonry's compressive failure (Tab. 1, Fig. 6b).

The second panel (Panel 2Rf), similar to the previous one for dimensions and stone arrangement, was reinforced before the application of the load. The reinforcement layout was also similar to the one used to repair the previous panel. Table 1 shows the test results: the panel could resist a compressive load of 101.5 kN and failure occurred when vertical through cracks started to open. Large vertical deformations were measured. Both wall leaves deformed similarly (Fig. 6c and 6d).

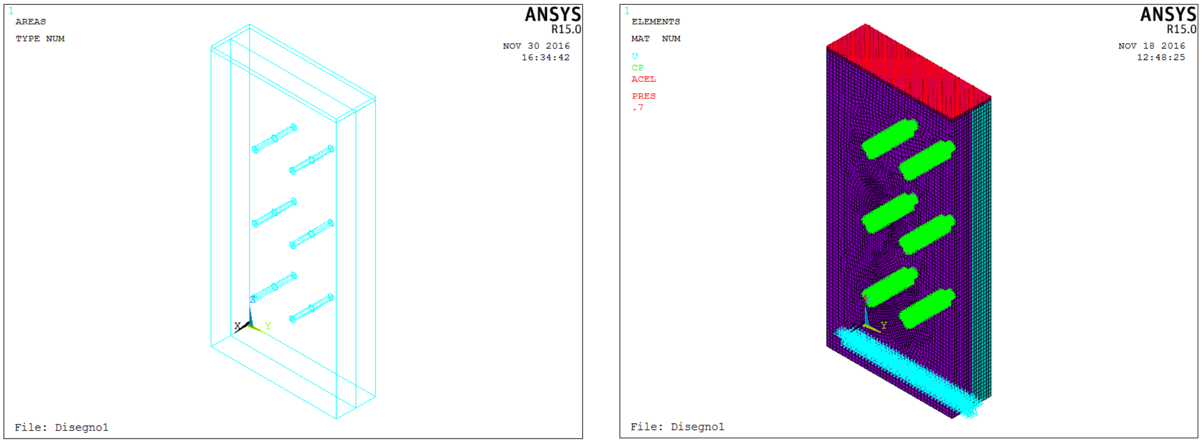
In the experimental campaign the stress state and, in detail, the elastic behaviour of steel connectors were not recorded, but an accurate visual analysis, after the disposal and demolition of the wall panels, did not highlight any residual axial deformation and no yielding. Also, the fabric sleeves, filled with high-strength mortar, resulted un-cracked and intact, confirming the fact that panel's failure has been mainly caused by the attainment of the masonry compressive and tensile strengths.

FINITE ELEMENT MODEL

A commercial finite-element analysis package (Ansys ver. 15 [23]) was used to set up a 3-dimensional finite element model to simulate the mechanical behavior of multi-leaf wall panels retrofitted with transversal connectors when loaded in compression. The numerical model was built to accurately reproduce the geometry of the specimens tested in the laboratory. To this end, the geometry of the panels was firstly reconstructed by means of CAD tools, next the volumes were imported and modeled using Solid65 elements, which are defined by eight nodes with three degrees of freedom at each node and isotropic material properties. These 8-node brick elements are the ideal solution to represent non-linear isotropic materials like historic rubble stone masonry.

The average size of the brick elements was chosen so as to have 16 elements across the specimen thickness: this allows the more critical details to be captured avoiding shear lock effects. The same mesh size was used to model the transversal connectors in order to provide full overlap at the joints. Figure 7 shows the finite element model consisting of 105,060 nodes and 92,046 elements, with 315,180 degrees of freedom.

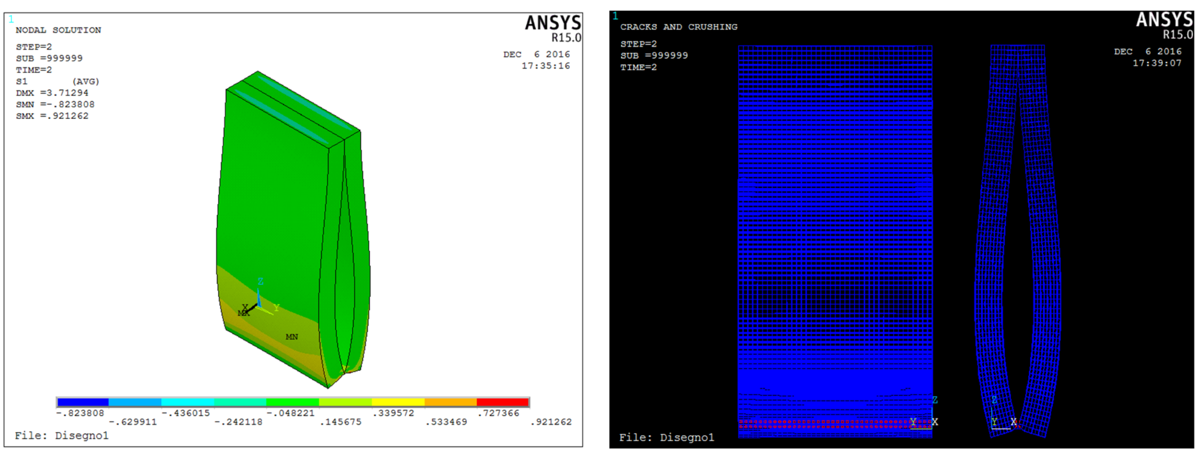
A tension cut-off type material model (with a Poisson's ratio of 0.20 and Young's modulus of 360 MPa) was assumed for masonry. This material model initially used for concrete, accounts for both crushing and cracking failure modes by means of a smeared model. More specifically, the brittle behavior of masonry was defined here by means of only two material parameters: uniaxial compressive strength (f_c = 0.312 MPa) and uniaxial tensile strength (f_t = 0.081 MPa). Conversely, the transversal connectors were modelled with a linear elastic material law without damage (data on the material properties were taken from manufacturer's technical information, the final inputs had the following values: E = 206000 MPa, υ = 0.3).



(a) **(b)**

Figure 7: Autocad and Ansys Numerical Model Representation of Experimental Specimen

Lastly, to increase the reliability of the proposed FE model, unilateral contact interfaces were used to simulate of the contacts between the masonry panel and the bearing supports and load plates, respectively. The joint modelling requires the use of flexible-flexible contact elements (contact pair type Target 170/Contact 174 in ANSYS). In detail, in this application, a unilateral contact law was applied in the tangential direction, assuming that sliding may – or may not – occur by using the Coulomb friction law.

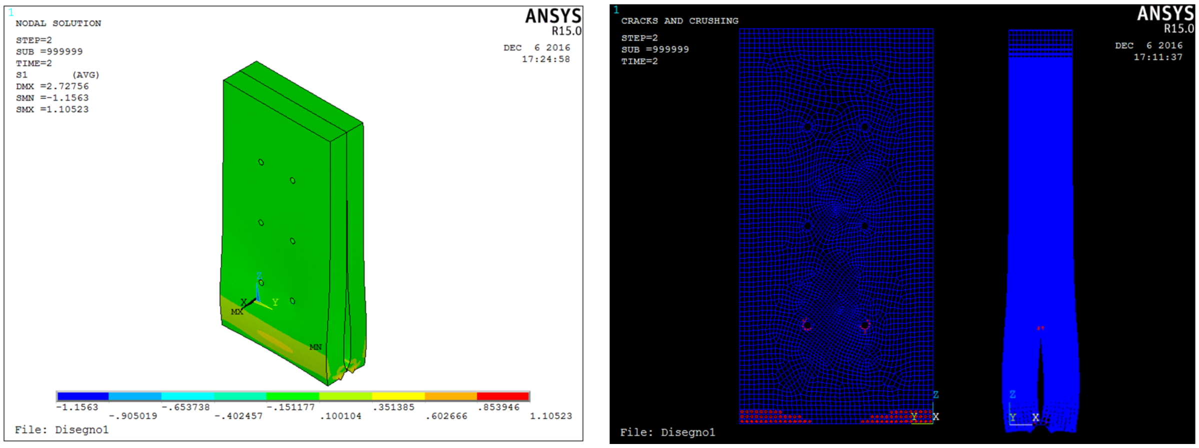


(a) **(b)**

Figure 8: Unreinforced Panel: (a) Contour Plots of the Maximum Tensile Stresses; (b) Cracking Pattern

In order to find the actual stress field at maximum load, a non-linear analysis was conducted, in which the masonry panels were subjected to both self-weight and a uniform load pressure under different load stages. The results of the analysis are summarized in Figure 8 and 9, showing the contour plots of the maximum principal stress (maximum tensile stress) on the tests specimens.

To show the efficiency and accuracy of the proposed FEM model, the predictions of the ultimate load capacity are compared with experimental results in Table 2. Dimensions of results reported in Figures 8 and 9 are in (MPa).



(a) **(b)**

Figure 9: Reinforced Panel: (a) Contour Plots of the Maximum Tensile Stresses; (b) Cracking Pattern

This comparison reveals good agreement between the theoretical predictions and experimental data for the collapse mode and for the corresponding load-carrying capacity. The unreinforced panel was predicted to fail due to buckling problems (separation of the adjacent wall leaves), as was also determined experimentally, and the deviations between the calculated and measured values was found to be no more than 4 %.

A good agreement between theoretical and experimental results was also detected for the reinforced panel. The error of the model was in fact 18 %. As for the failure mode, the FEM model properly simulated the experimental behavior of the panel, which failed because of masonry.

Table 2: Experimental vs Predicted Ultimate Compressive Load Capacities

	Experimental load capacity (kN)	**Predicted load capacity** (kN)	**Error of the model** (%)
Panel 1Un	48.6	50.8	4.0
Panel 2Rf	101.5	83.1	18.1

CONCLUSIONS

Natural stone has been used in construction for centuries and, traditionally, barely cut stones were employed to build multi-leaf masonry walls with the emphasis on creating a structure able to resist to vertical static loads, but with little attention to their behavior under seismic actions.

Recent earthquakes in southern Europe have highlighted the high vulnerability of multi-leaf stone masonry walls. The cause of collapse of many historic buildings is under investigation by researchers, but the low level of connection between adjacent wall leaves has been proved to be a critical aspect.

The aim of this study was to measure the effect of the application of artificial transversal connectors on multi-leaf stone masonry panels. An experimental investigation has been conducted in the laboratory on two slender stone masonry wall panels. The results of the study have revealed that the application of artificial connectors can produce a significant increase of the panels' compressive strength.

The paper also described a 3-dimensional finite element model to simulate multi-leaf wall panels retrofitted with transversal connectors. In this process, connecting elements can be solved indirectly, which is very convenient for nonlinear analysis of masonry structures, since the structural nonlinear properties are not only governed by masonry material but also by the connector. The connecting element currently incorporated in the program only applies for headers that are small enough so that their bending strength is negligible. This 3D modelling approach was applied to model double-leaf walls reinforced with transversal connectors loaded in compression. Very good agreement was found between the results of the FEA model and a set of compression tests on reinforced wall panels.

Future work will involve investigating the influence of the connectors' arrangement and reinforcement scheme and the effect on different masonry typologies in order to contribute further to the issue of the structural behavior of historic masonry constructions with regard to out-of-plane collapse mechanisms.

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