Experimental Tests on Typical Masonry of Messina Area (Italy) Retrofitted with CAM: a Full Scale Arch

CILIA, MAURO[1]; COLAJANNI, PIERO[2]; MARNETTO, ROBERTO[3]; RECUPERO, ANTONINO[4]; SPINELLA, NINO[5]

ABSTRACT: The present paper focuses on an in-situ testing programme recently developed in the Messina area, Italy, aiming at the assessment of the in-plane shear behaviour of traditional masonry structures retrofitted with the innovative CAM system (a system of 3D pre-tensioned stainless steel ties). The typical masonry of Messina area, is characterised by low mechanical properties, for its texture (bond) and for the bad quality of mortar, as well as lack of transverse connections. The high seismic risk characterizing the zone, amplifies the need for in-plane strengthening and transverse connection improvement. The CAM system, which uses masonry ties and active confining of masonry allows one to realise a 3D pre-tensioned tying system which is able to compact stones and mortar and improve the masonry mechanical characteristics. A full scale arch was cut from an ancient building and in-situ tests were carried out. First, the arch was horizontally in-plane loaded until first cracking appeared, then it was reinforced with CAM and re-loaded until failure. The results of the experiments carried out have highlighted the advantages in using CAM, specially its ability to provide a large increase in terms of strength and ductility.

Keywords: *Masonry, In-Plane Shear, CAM, Arch*

NOTATION

f_{yk}, f_{tk}	*Stainless steel yield and failure strength*
K_a, K_m	*Corrective factors between pressure in the flat-jack and the masonry stress*
p	*Pressure applied by the flat-jack*
P	*Load*
ε_h, ε_v	*Horizontal and vertical average strain*
ε_s	*Stainless steel strain*
Φ	*Diameter of the circular saw*
σ_m	*Average compressive stress in the masonry*
σ_s	*Stainless steel stress*
σ_v	*Vertical average stress*

1) Technical Director, Chimetec s.a.s., Ragusa (ITALY), info@chimetec.com
2) Associate Professor, Università di Palermo, Dipartimento di Ingegneria Civile, Ambientale Aerospaziale e dei Materiali; piero.colajanni@unipa.it
3) Research and Development Manager, EdilCAM Sistemi S.r.l., Roma (ITALY), r.marnetto@edilcamsistemi.com
4) Assistant Professor, Università di Messina, Dipartimento di Ingegneria Civile, Informatica, Edile, Ambientale e Matematica Applicata (DICIEAMA), antonino.recupero@unime.it
5) Ph.D. P.E., Università di Messina, Dipartimento di Ingegneria Civile, Informatica, Edile, Ambientale e Matematica Applicata (DICIEAMA), nino.spinella@unime.it – corresponding author

1 INTRODUCTION

Historical Italian Buildings are characterised by low mechanical properties of masonry, both for its texture (bond) and for the bad mechanical characteristics of mortar. Walls are often made of a double masonry layer, without any transverse connections. In addition, masonry is not always homogeneous, parts of the same wall being made of materials with different mechanical characteristics. The low strength of masonry structures is further reduced by the slenderness of the single wall layers, subjected to in-plane vertical compression and shear, as well as to out-of-plane bending. Therefore, these combined actions can induce collapses of masonry elements, even for low-medium intensity earthquakes. In old masonry buildings retrofitting required new connections between structural elements (walls, beams, kerbs), but also dealing with masonry weakness. An effective reinforcement restores structural performance, increases load capacity and prevents brittle collapse. Moreover, reinforcement is also necessary in the cases where the structures must satisfy the requirements of current codes and standards. Coupled with concerns over the maintenance of our architectural heritage, there is wide interest nowadays in studying new strengthening techniques in greater detail. In the past few years, new reinforcement materials used in other applications, such as aeronautics, have been introduced to the world of restoration. These materials have, on the whole, been presented in the form of Fibre Reinforced Polymers (FRP), which are made up of synthetic fibres embedded in resins and largely used for the retrofitting of masonry [1,2] and reinforced concrete [3-5] elements.

However, the need for the compaction of a masonry mass suggested the idea of using a three-dimensional system of tying, capable to confine the masonry structure, giving a beneficial triaxial compression stress state. One such concept, the Active Confinement of Manufacts (or Masonry) CAM system is studied.

CAM system utilises stainless steel, to avoid any durability problem and get good ductility characteristics. Ties are formed of stainless steel ribbons and are prestressed, to apply a light precompression state to the masonry, which is particularly useful in the transverse direction. Special connection elements allow one to realise a continuous tying system, running all along masonry walls, both horizontally and vertically, to improve not only the shear resistance but also the flexural resistance of masonry walls part and as a whole.

The aim of this research is to improve knowledge on the effectiveness of CAM system applied to masonry arches having low mechanical properties due to both irregular texture and low strength mortar, hence a test on a full scale element is performed. More precisely, as a first step, material characterisation was carried out, with the aim to characterise the constitutive behaviour of stainless steel and to analyze the efficiency of joints. The ties were subjected to direct tensile tests then, a double flat-jack test was carried out in-situ on a wall of the same building as the arch to evaluate the mechanical characteristics in compression of the ancient masonry typical of Messina area (Italy). The flat-jack technique can over or underestimate actual stress by around 20% [6].

As a second step, in-situ experimental work on a full scale masonry arch (cut from an ancient building) was designed in order to fulfil the following objectives: to characterise the structural behaviour of non-strengthened arches, and study the influence of strengthening using the CAM system on the seismic behaviour of the arch as it relates to the failure mode, resistive capacity and deformation.

This study is part of a larger research project focused on the increasing performance of masonry both in terms of strength and ductility due to CAM retrofit system [7].

2 CAM SYSTEM

The CAM system is mainly based on the use of stainless steel ribbons, to tie masonry with loops passing through transverse holes, as shown in Figure 1(c). The loops are closed with a special tool, which is able to apply a calibrated prestress to the ribbon. The system includes also drawpieces as connection elements and angles as terminal elements, as shown in Figure 1(b).
In current applications, the ribbon is 0.9-1 mm thick and 19 mm wide, with yielding and failure strengths equal to 250-300 and 600-700 MPa respectively, and more than 40% elongation at failure. The drawpieces, which play the role of connection and force transmission elements between adjacent ribbon loops as well as stress distribution elements on masonry, are usually 125×125 mm, 4 mm thick [Figure 1(a)]. Similar sizes are used for angles in current applications. The distance between holes is typically between 400 and 800 mm.

Figure 1. Stainless steel a) plates; b) angular elements; and c) ribbons.

The ribbon system can be arranged in a square, rectangular, rhombic, triangular or even irregular mesh, so that a horizontal and vertical continuous mesh is realised. Figure 2 shows a typical application on a double layer wall, with an alternate arrangement of holes, to minimise their number. The holes can be eventually injected with any kind of mortar provided there will be no corrosion problems, to improve masonry characteristics around holes. further, diagonal arrangements of ties can be more effective for regular brick masonry walls, as well as to connect floor lintels to masonry walls to limit possible lintel-masonry slipping.

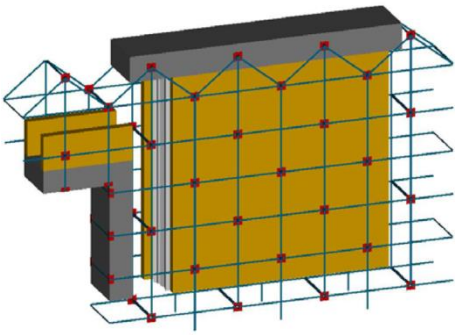

Figure 2. CAM arrangement in a wall with a door and a R/C upper lintel.

3 MASONRY DOUBLE FLAT-JACK TEST

A test with double flat-jacks was performed on the ancient masonry of the building at a wall orthogonal to the considered arch. The objective was to estimate the compression strength in the masonry walls with irregular texture. The flat-jack test technique is a relatively well known semi-destructive one. After the creation of two horizontal slots in the masonry, the compressive stresses present at that point cause the closure of the masonry above and below each slots. Before cutting the

wall, original dimensions are taken between gage points. Once the cuts are made, the flat-jacks are loaded in the cuts and readings between the gage points are taken at various pressures. From this data, one can back calculate the stress present in the wall before the cuts were made as well as the modulus of elasticity of the masonry. After data for both tests are obtained, the cuts are restored to their original conditions with mortar.
Taking into account some corrective factor, the measure of the pressure p applied by the flat-jack approximately corresponds to the local pressure in masonry. Particularly the average compressive stress in the masonry, σ_m, could be calculated as follows:

$$\sigma_m = K_m \cdot K_a \cdot p \qquad (1)$$

where K_a is a factor that accounts for the ratio between the bearing area of the jack in contact with the masonry and the bearing area of the slot; K_m is the factor accounting for the physical characteristic of the jack, and p is the pressure required to restore the gauge points to their original distance. Both the single and double flat-jack tests were conducted with respect to the recommendations issued by the American Society for Testing and Materials [8,9], due to the lack of references on Italian rules. For the execution of the in-situ tests the following equipment was used: semi-oval flat-jack (dimensions: 350×260×4 mm with $K_m = 0.84$ and $K_a = 0.90$); hydraulic circular saw (Φ350 mm and thickness 3.5 mm); hydraulic hand pump (manometer with ranges of 20 and 60 bar); mechanical strain gauge (length 200 and 400 mm, with resolution equal to 0.001 mm). Figure 3 reports the scheme of the test (acquired geometry and position of the bases measurement of the removable strain-gauge), and Figure 4 reports the outcome of the double flat-jack test where vertical strains are plotted as functions of the stress in the wall (mean stress–strain curves).
The results show a weak masonry with a compression strength equal to 1.90 MPa, while the first crack was appeared at a level of compression stress equal to 1.30 MPa.

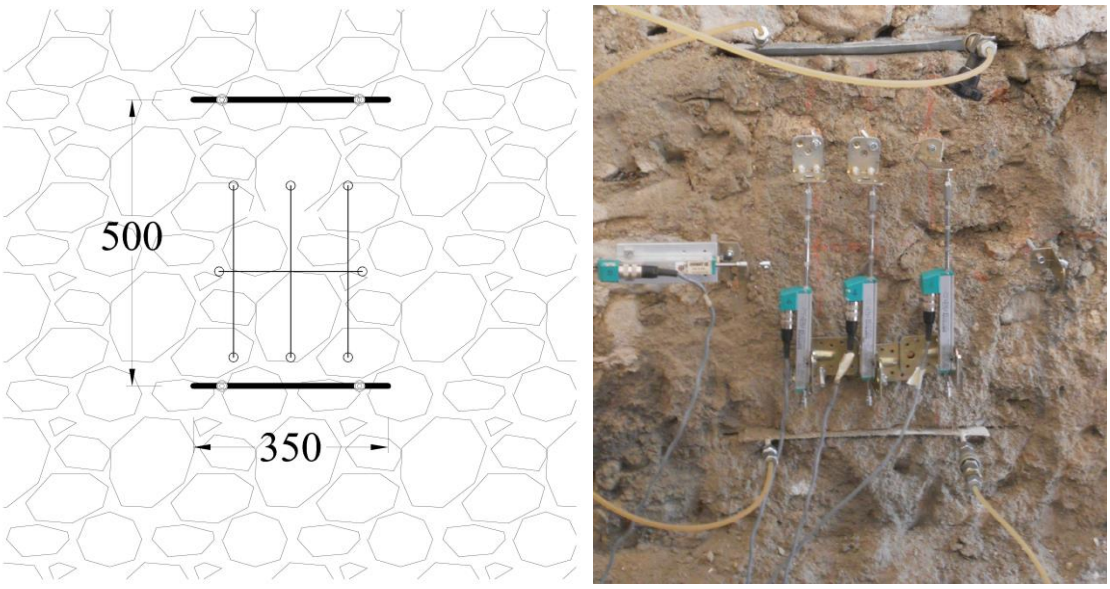

Figure 3. Double flat-jack test: a) setup and b) preparation (dimensions in mm).

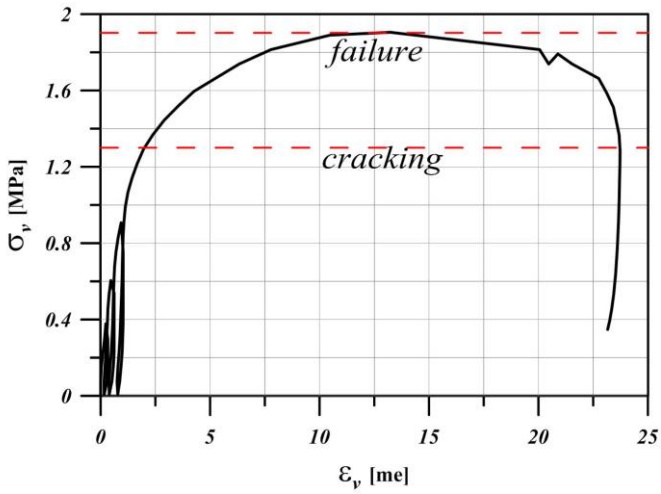

Figure 4. Double flat-jack test: average stress–strain curves.

4 STAINLESS STEEL TENSILE TESTS

The mechanical property and the junction of the stainless steel ribbon affects the efficiency of the CAM system. With the purpose of investigating the efficiency of the reinforcement system, the stainless steel used for the ribbons was subjected to uniaxial tensile test. The stainless steel was chosen to give a minimum characteristic yield (f_{yk}) and a minimum failure (f_{tk}) characteristic strength equal to 220 MPa and 540 MPa, respectively. In detail, six specimens of stainless steel were tested: three with and three without junctions. A servo-hydraulic machine, with a 4000 kN load-carrying capacity, was used for the tensile tests of stainless ribbons, adopting a gauge length of 100 mm as shown in Figure 5(a,b). The specimens without junctions exhibited a classical failure mode characterized by considerable elongation and subsequent necking of the cross-section [Figure 5(c)]; the specimen with the junction at the middle height showed a failure mode characterized by successive losses of load corresponding to slips of the junction [Figure 5(d)]. In Figure 6 the tensile stress-strain curves for each specimens are plotted. The ribbons show a good capacity of axial deformation, with the maximum strain value about of 50 me. The constitutive behaviour for ribbons without junction is typical of stainless steel and the experimental yield strength is higher than expected value of 220 MPa (Figure 6). The specimens prepared with the junctions in the middle are able to reach values of strength less than the corresponding specimens without junction, and the deformation capacity is reduced, as showed in Figure 6. Moreover, for the sealed specimens brittle failure of the junction was observed and the corresponding ultimate strain recorded is about of 5 me (Figure 6).

5 IN-SITU ARCH TESTS

The arch chosen for the investigative campaign was part of an ancient masonry building destined for demolition [Figure 7(a)]. The preparation phase of testing was characterized by the following steps: (1) the external plaster was removed; (2) the railing on the top was demolished; and (3) the masonry wall was isolated from the rest of building with a diamond circular saw. With the aim to obtain a fixed reference for measurements, a light steel frame was built close to the front face of the arch; then eight Linear Variable Differential Transformer (LVTDs) were positioned in different points of wall [Figure 7(b)].

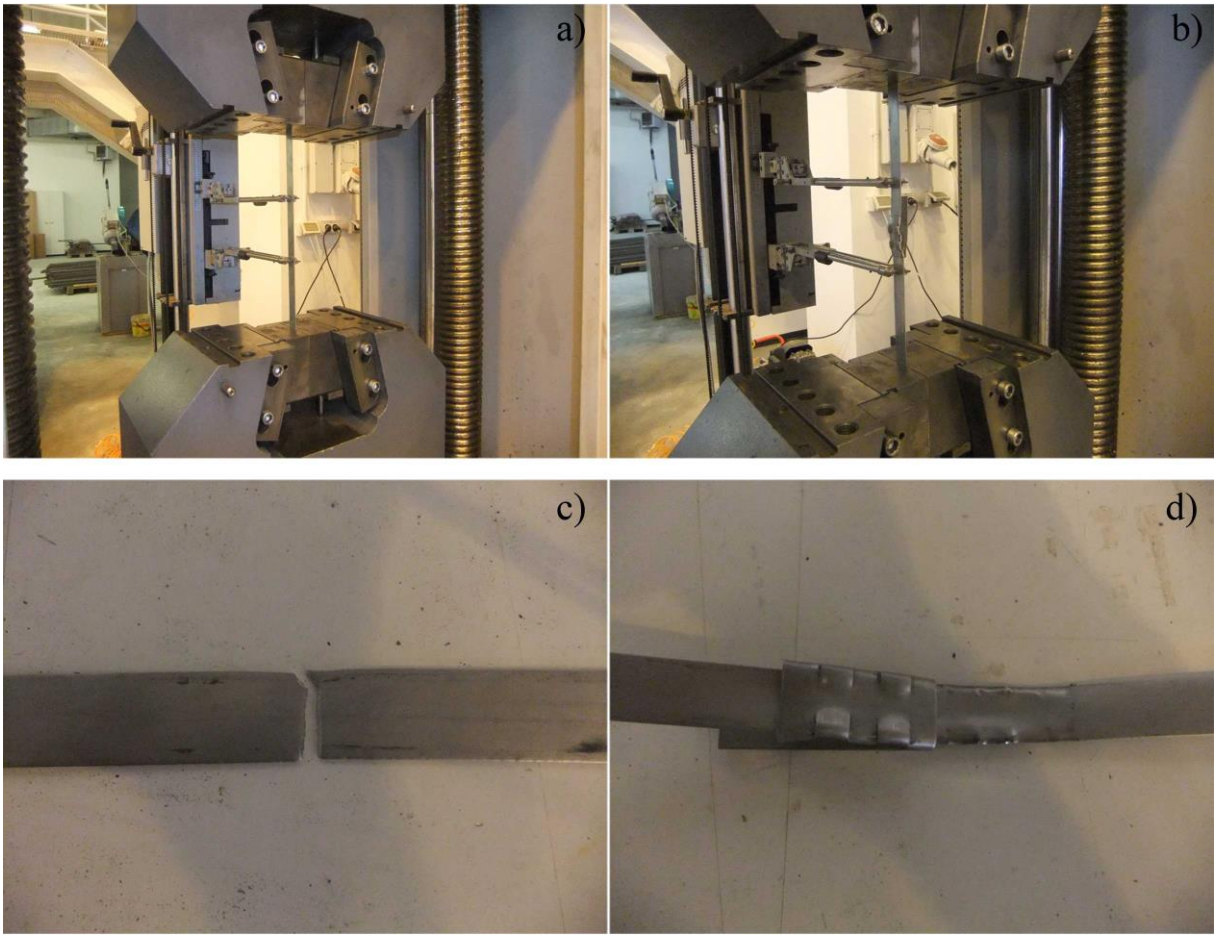

Figure 5. Stainless steel tests: preparation a) without and b) with joint; failure mode c) without and d) with joint.

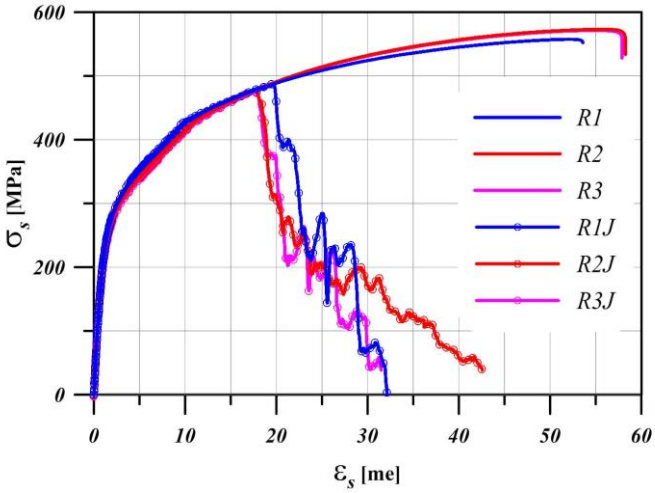

Figure 6. Stainless steel tensile tests: average stress–strain curves for ribbons with (R#) and without (R#J) joint.

The LVDTs S1 and S4 measured the displacements on the left abutment along the vertical and horizontal direction, respectively; on the right abutment, the corresponding LVDTs were S2, vertical, and S5, horizontal; along the horizontal load line, the LVDT S3 was positioned at the middle of the wall, above the keystone, while the LVDT S7 was positioned on the opposite side with respect to the hydraulic jack to measure the global horizontal displacement; the LVDTs S8 and S6 measured the horizontal displacement and were positioned at the top-left and top-right of the wall, respectively. In addition, to check eventual dangerous out-of-plane displacements, two LVDTs orthogonal to the internal wall face and on the same level of LVDTs S8 and S6 were used.
The pseudo-static horizontal load (*P*) was applied by a hydraulic jack governed by a hand pump (manometer with ranges of 20 and 60 bar) linked to a pressure transducer. With the aim to uniformly distribute the horizontal force throughout the wall thickness, a metal element was positioned between the hydraulic jack and the wall.
For the first test on the unreinforced arch, a hydraulic jack with a capacity of 200 kN and a maximum displacement of 50 mm was used. For the second test on the arch reinforced with the CAM system, a hydraulic jack with a capacity of 1000 kN and a maximum displacement of 100 mm was used. Both load and displacements data were acquired by an electronic control unit.
Moreover, a constant vertical compression load equal to 7 kN/m was applied for the entire duration of both tests by positioning cement sacks along the top of the wall.

5.1. Unreinforced arch test

The first test was carried out on the unreinforced arch. The load was applied by several cycles and incremented until some cracks were appeared. The cracks were formed respspectively: (a) in correspondence of the left haunch of the arch; (b) a long horizontal crack close to the top of the wall; and (c) close to the point load (Figure 8). In Figure 11, the curves load-displacement, show the capacity of unreinforced arch both in terms of crack strength and ductility. The maximum load was equal to 50.6 kN, which was reached at a corresponding horizontal displacement (S7) of 5.4 mm [Figure 11(h)]. The left abutment was subjected to a tensile stress, while the right abutment was compressed as expected [Figure 11(c,d)]. In the part of arch over the keystone the measured horizontal displacement was about the same for the LVDTs S8, S6 and S3 [Figure 11(e-g)], because just few parts of masonry were cracked.
During the unload cycles, the first cracks were all closed with small residual plastic deformation. However, the load increment was stopped to avoid wide plastic deformations and a collapse of the wall.

5.2. Reinforced arch test

In the second phase of tests, the constant vertical compression load was removed to allow the retrofitting of the arch with the CAM system. Two overlapped ribbons of stainless steel was used to confine the masonry along the face of the masonry and in the orthogonal direction to the arch plane, with ribbons passing through the holes drilled in the masonry. The ribbons scheme adopted is showed in Figure 9, with a squared arrangement of stainless steel ribbons along the abutments, while a radial arrangement was adopted for the masonry over the keystone. The radial stainless steel ribbons were positioned orthogonal to the haunch of arch. Moreover, four steel angular were used to reinforce the inner edges of each abutments increasing their flexural strength (Figure 9). The adopted arrangement is representative of a realistic situation, which could be optimized in future applications avoiding some holes in the wall.
After the completion of the operations of the arch reinforcement with the CAM system, the constant vertical compression load equal to 7 kN/m was re-applied on the top of the wall [Figure 9(a)]. The removal and re-positioning of the constant vertical load has temporarily modified the stress condition, but no new cracks were observed before the test on the reinforced arch.
Then, horizontal load cycles having increasing displacement amplitude of 4 mm each cycle were applied. The maximum load of 111.1 kN was reached at the end of the 6th cycle, when the horizontal displacement reached about 70 mm. The observed displacement capacity of the arch reinforced with the CAM system was unexpected and higher than the elongation capacity of the hydraulic jack, as shown in Figure 10(a); thus the collapse of the structure was not reached.

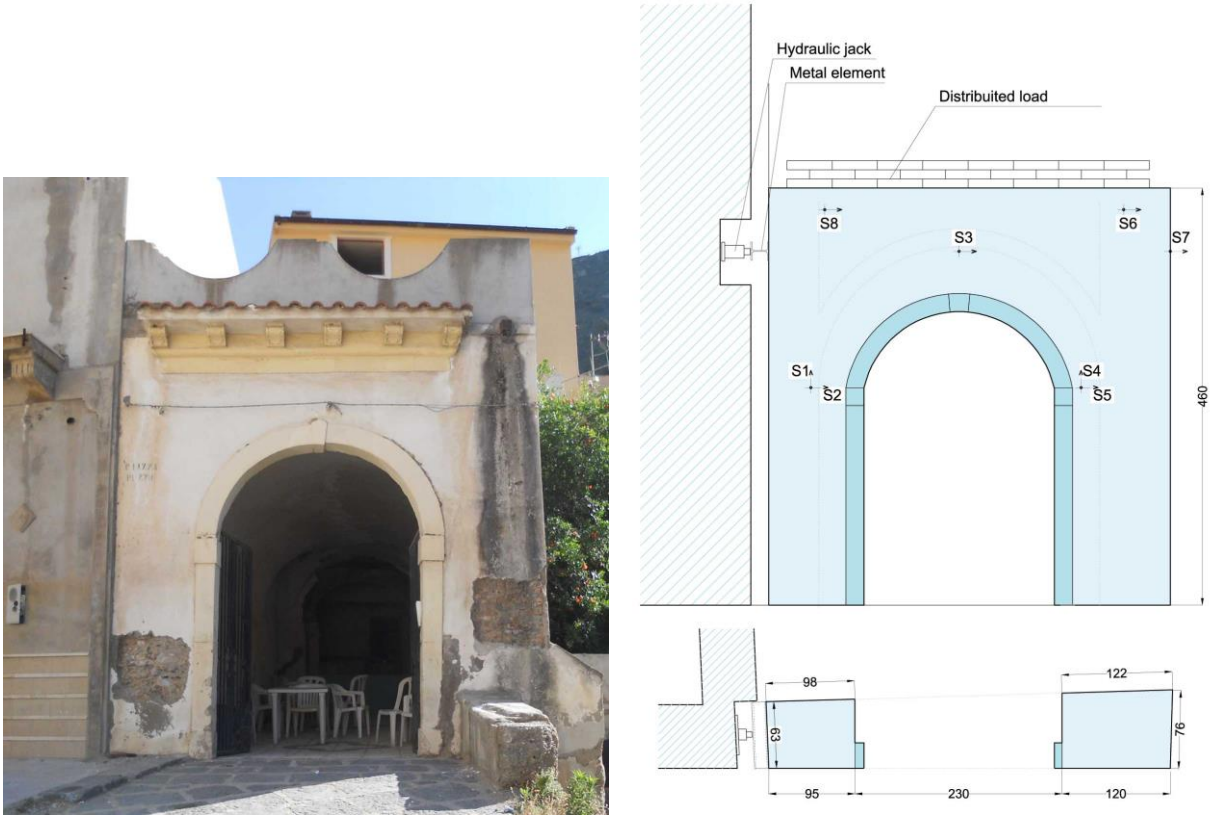

Figure 7. Unreinforced arch: a) preparation and b) setup (dimensions in cm).

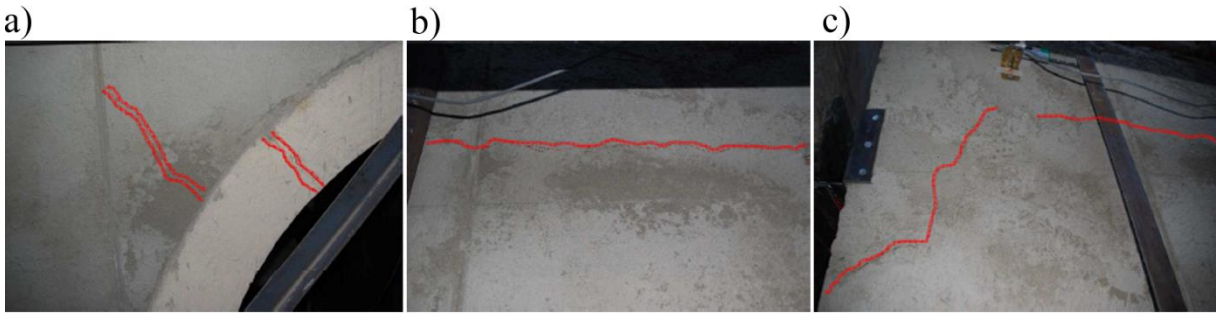

Figure 8. Unreinforced arch: first cracks.

Aiming to evaluate the displacement capacity of the retrofitted masonry arch, the structure was unloaded, and an hydraulic jack with a larger stroke was used to impose the displacement. A new branch of the load curve was realized by imposing displacement up to the value of 150 mm, corresponding to the capacity of the new hydraulic jack; this time also the collapse displacement was not reached and the arch must be unloaded again due to the attainment of the full displacement capacity of the testing jack.

At the maximum level load, both the feet of left and right abutments were subjected to tensile stress and wide cracks were formed [Figure 10(b,c)]. Other cracks were appeared across the top part of the wall. However the CAM system was able to confine the masonry, allowing the stress transfer across cracks. In addition, during the unload cycles, the wide cracks were always closed also for high load levels and limiting the plastic deformations.

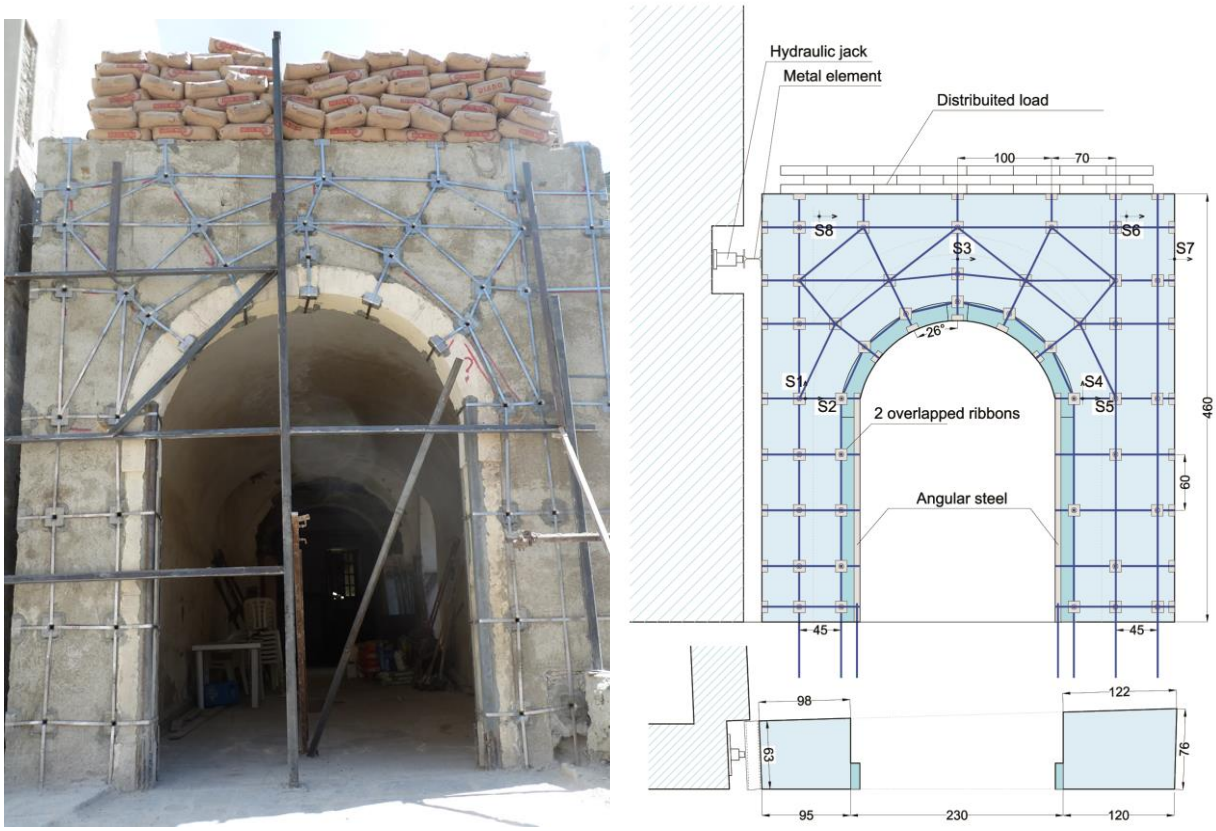

Figure 9. Reinforced arch: a) preparation and b) setup (dimensions in cm).

Figure 10. Reinforced arch: crack patterns before failure of a) arch; b) left abutment; and c) right abutment.

The load-displacement curves obtained in the first (unreinforced arch) and second phase (reinforced arch) of tests are plotted together in the Figures 11 for a comparison of results. The maximum load was equal to 111.1 kN, which was reached at a corresponding horizontal displacement (S7) of 70 mm [Figure 11(h)] with a percentage increment of 120% and 1196% of load and horizontal displacement, respectively.

6 CONCLUSIONS

In this work, the results of an experimental in-situ campaign about the CAM strengthening system are presented. A full scale arch was cut from an ancient masonry building in the Messina area (Italy) and in-situ tests were carried out.

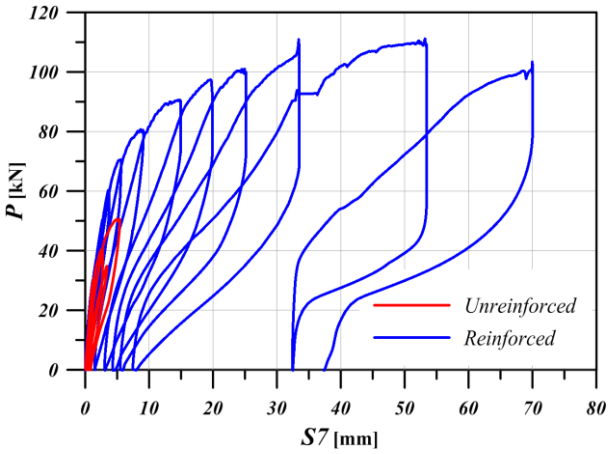

Figure 11. Horizontal load-displacement curve for unreinforced and reinforced arch.

The results of the experiments showed the advantages in using CAM, observing an increasing of 120% and 1196% in terms of strength and ductility of masonry arch, respectively.
The CAM system contributes to improve the transverse link between masonry layers, the increase of the in-plane strength and ductility, and the connections between intersecting walls.
Although the results appear satisfactory, the mechanical features of the CAM system can be better exploited on the typical masonry of existing buildings. Further double layer stone masonry with low quality mortar can benefit from transverse linkage given by CAM and better functioning when an orthogonal CAM arrangement is used with irregular masonry results. Overall there is a general improvement of properties with low strength masonry. Moreover, the experimental results show the CAM reinforcement system performs best when the design is optimised resulting in improved ductility and strength, avoiding waste of material and additional costs.
The analytical modelling of the unreinforced and reinforced masonry arch behaviour is the natural topic of future research. This objective will be pursued applying the theory of plasticity, which were yet largely used in many fields of structural engineering [10-14] and for masonry elements also [15]. Moreover, procedures based on advanced finite element models and continuous stress fields [16,17], will be used and suitably adapted to reproduce the experimental behaviour of masonry arches.

ACKNOWLEDGEMENTS

This research work has been made possible thanks to financing from the Sicilian Government within the research project *PO. FESR 2007-2013 – Sicilia – Linea di Intervento 4.1.1.2* involving ABI Srl (Italy), Chimetec Sas (Italy) and University of Messina (Italy).

REFERENCES

[1] Garmendia, L.; San-José, J.T.; García, D., Larrinaga, P.: Rehabilitation of masonry arches with compatible advanced composite material. *Construction and Building Materials*, 25 (2011), 4374-4385.

[2] Borri, A.; Castori, G.; Corradi, M.; Speranzini, E.: Shear behavior of unreinforced and reinforced masonry panels subjected to in situ diagonal compression tests. *Construction and Building Materials*, 25 (2011), 4403-4414.

[3] Colajanni, P.; De Domenico, F.; Recupero, A.; Spinella, N.: Concrete Columns Confined with Fibre Reinforced Cementitious Mortars: Experimentation and Modelling. *Construction and Building Materials*, 52 (2014), 375-384.

[4] Colajanni, P.; De Domenico, F.; Mantegazza, G.; Maugeri, N.; Recupero, A.; Spinella, N.: Experimental results of RC columns strengthened with Fibre Reinforced Cementitious Mortars. In: *3rd International Conference on Concrete Repair, Rehabilitation and Retrofitting, ICCRRR-3*, 3-5 September 2012, Cape Town, South Africa Edited by Pilate Moyo CRC Press, 1137–1143.

[5] Campione, G.; Colajanni, P.; La Mendola, L.; Spinella, N.: Ductility of reinforced concrete members externally wrapped with fiber-reinforced polymer sheets. *ASCE Journal of Composites for Construction*, 11 (2007) 3, 279–290.

[6] Gregorczyk, P.; Lourenço, P.B.; A review on flat jack testing – *Engenharia Civil*, 9 (2000), 39-50, Università do Minho.

[7] Colajanni, P.; Recupero, A.; Spinella, N.; *Progetto INCAMMINO – Indagini sperimentali su elementi strutturali in muratura rinforzati con il CAM* part of book *Conservare l'edilizia in muratura: il sistema CAM-Cuciture Attive dei Manufatti* edited by Marnetto, R.; Vari, A.; Leonori, M.; Preprogetti S.r.l. Roma, 2014, pp. 173-180, ISBN: 978-88-95027-04-3.

[8] ASTM C1196-91: *Standard test method for in situ compressive stress within solid unit masonry estimated using flat-jack measurements*. Philadelphia: ASTM 1991.

[9] ASTM C1197-91: *Standard test method for in situ measurement of masonry deformability properties the using flat-jack method*. Philadelphia: ASTM 1991.

[10] Colajanni, P.; Recupero, A.; Spinella, N.: Shear strength prediction by modified plasticity theory for sfrc beams. In: *Proceedings of Seismic Engineering Conference: Commemorating the 1908 Messina and Reggio Calabria Earthquake*, Reggio Calabria (Italy), 8–11 July 2008, 888-895.

[11] Spinella, N.; Colajanni, P.; Recupero, A.: Simple plastic model for shear critical sfrc beams. *ASCE Journal of Structural Engineering*, 136 (2010) 4, 390-400.

[12] Colajanni, P.; Recupero, A.; Spinella, N.; Generalization of shear truss model to the case of SFRC beams with stirrups. *Computers & Concrete*, 9 (2012) 3, 227-244.

[13] Spinella N.: Shear strength of full-scale steel fibre-reinforced concrete beams without stirrups. *Computers & Concrete*, 11 (2013) 5, 365-382.

[14] Colajanni, P.; Recupero, A.; Spinella, N.; Design procedure for prestressed concrete beams. *Computers & Concrete*, 13 (2014) 2, 235-253.

[15] Mojsilovic, N.: Strength of masonry subjected to in-plane loading: A contribution. *International Journal of Solids and Structures,* 48 (2011), 865-873.

[16] Mojsilovic, N.; Kostic, N.; Schwartz, J.: Modelling of the behaviour of seismically strengthened masonry walls subjected to cyclic in-plane shear. *Engineering Structures*, 56 (2013) 1117–1129.

[17] Spinella, N.; Colajanni, P.; La Mendola, L.: Nonlinear analysis of beams reinforced in shear with stirrups and steel fibers. *ACI Structural Journal*, 109 (2012) 1, 53-64.