

# Strengthening of Solid Brick Masonry Columns with Joints Collared by Steel Wires

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**Abstract-**This work presents the results of an experimental investigation on the behavior of columns made of solid clay bricks strengthened by stainless steel wires placed in the joints. The characterization of each constituent property (bricks, mortar, and steel wires) and the compression tests on columns show the effectiveness of the proposed strengthening technique with the result of increased ductility of brick columns. From the analytical point of view a model is proposed which gives the response in terms of the stress-strain curve, by taking into account the presence of steel wires placed in the joints, and the consequences on the effective confinement pressure with the following resulting enhancement of column strength and ductility. The analytical model is validated through the results obtained by the experimental campaign and those given in the literature.

**Keywords-** Masonry; Confinement; Steel Wires; Compression; Ductility; Strength

## I. INTRODUCTION

In strengthening interventions on masonry buildings, one of the recurrent problems is the need to enhance the load-carrying capacity and pseudo-ductility of columns, walls and vaults [1-17]. For masonry columns, different retrofit techniques have been applied in order to obtain increased strength values; among them concrete jackets or steel strip wrapping have been used and more recently, the external wrapping of elements through Fiber Reinforced Polymer (FRP) jackets was introduced. These lightweight materials, though currently used for the confinement of reinforced concrete columns, have good mechanical properties and they are simple to apply on site. Their application to masonry structures is less common but quite effective [1-5] and not well established for different reasons. Firstly, the heterogeneity of masonry typologies which make generalization of these techniques difficult. Secondly, aesthetic and architectural instances problems in masonry - buildings, where FRP jackets are intrusive, with the result of hiding the building elements and preventing transpiration of masonry. Thirdly, effects of stress concentration in columns corners especially for square or polygonal cross-section shapes, which reduce the effectiveness of the strengthening intervention. Recently, a different option with respect to the application of FRP jackets was introduced, that is, inserting steel fibers into cement mortar joints. Iurina [6] used high-strength steel strands placed in the cement mortar joints, arranged in parallel filaments, gathered together by a polyester chainmail. The most interesting characteristic of this technique is the reduced dimension of the steel strands (average diameter about 1 mm) as well as their shape and arrangement, obtained by rolling up steel filaments on each other in a helicoidal shape (typically from 3 to 5 filaments). The result is high-strength strands with high bonding capacity and very good compatibility between the steel elements and the mortar in which they are inserted, ensuring good behavior of the system composed of mortar and steel strands. Moreover, the reduced dimensions make it possible, within certain limits, to give the required curvature to strands, in order to make them adherent to the surfaces of the masonry columns to be confined, adapting them to the column edges. From the experimental point of view, the present study carried out with a technique that can be framed within the latter methodology of recent masonry column strengthening, but that is different from it because ordinary steel with low yielding stress is used instead of high-strength steel. This choice related to the reduced cost of this steel compared with high-strength steel. The proposed technique consists of steel wires inserted in joints simply by tying them manually with the global effect of a passive confinement action. By contrast, Iurina [6] and Borri and Corradi [7] used pre-tensioned high-strength steel strands, stressed before load application, giving active confinement. From the theoretical point of view, several models are available in the literature. In the following section, to describe the compressive behavior of the confined columns, the model reference of Campione and Miraglia [8] is provided. The analytical model of Campione and Miraglia [8] originally introduced for reinforced concrete elements wrapped by FRP and also used in Corradi et al. [7] for masonry elements wrapped by FRP was adapted to the case of masonry elements externally strengthened with steel strands. The principles and mechanics involved in deriving the relation proposed in Campione and Miraglia [8] are based as follows:

The definition of the effective confined area of transverse cross-section of the confined columns;

The arch actions occurring at the edges of square cross-sections externally wrapped;

The effective stress acting on the confining device (the latter depending on the radius of the corners at the edges);

The stress concentration at the edges depends on the number of layers of reinforcements and on the type of reinforcing material.

The analytical law proposed in [8] in term of stress-strain ( $\sigma$ - $\varepsilon$ ) curve is the following one:

$$\frac{f_{cm}}{f_{m0}} = \beta \cdot \frac{\varepsilon}{\varepsilon_{m0}} + \left\{ \left[ (1-\beta) \cdot \frac{\varepsilon}{\varepsilon_{m0}} \right] / \left[ 1 + \left( \frac{\varepsilon}{\varepsilon_{m0}} \right)^R \right]^{1/R} \right\} \quad (1)$$

$\beta$  being a shape factor defined as the following modulus ratio:

$$\beta = E_h/E_0 \quad (2)$$

For which the hardening modulus is defined as:

$$E_h = \frac{f_{cm} - f_{m0}}{\varepsilon_{cm} - \varepsilon_{m0}} \quad (3)$$

In Eq. (1)  $f_{cm}$  and  $\varepsilon_{cm}$  are the peak stress and strain values of the confined masonry, while  $f_{m0}$  and  $\varepsilon_{m0}$  are the peak stress and strain values of the unconfined masonry;  $R$  is another shape factor that was assumed as a fixed value of 3 in [8] and  $E_0$  the initial modulus of unreinforced masonry.

The confinement pressure was evaluated according to the expression given by Campione and Miraglia [8] taking into account the fact that in this case a discontinuous wrapping along the masonry element has to be considered (steel wire collaring).

The effective confinement pressure is:

$$f_l = \frac{2 \cdot A_{tot,f}}{b_d \cdot s} \cdot f_r \cdot k_e \cdot k_v \quad (4)$$

with  $A_{tot,f}$  the cross-section area of the steel wire, considered as the area of the three steel wires and the value of  $f_r$ , which is the working stress in the strand, is calculated as suggested in [8], in the following form:

$$f_r = f_y \cdot \left[ k + (1-k) \cdot \frac{2 \cdot r}{b_d} \right] \quad (5)$$

with  $k = 0.2121$  as in [8].

The coefficient proposed by Campione [9] was introduced:

$$k_v = e^{\left( -1.5 \cdot \frac{s}{b_d} \right)} \quad (6)$$

$b_d$  is the side dimension of the square masonry column section; and  $s$  is the distance between strand groups or collars that in the current case is assumed the distance between two cement mortar joints.

The effectiveness coefficient in plane  $k_e$ , adopted in Eq. (5) is derived as in [8] in the form:

$$k_e \cong 1 - \frac{2}{3} \cdot \left( 1 - \frac{2 \cdot r}{b_d} \right)^2 \quad (7)$$

With  $r$  the radius of filled at the corners here assumed of few millimetres.

In order to evaluate the peak values of the confined masonry,  $f_{mc}$  and  $\varepsilon_{muc}$ , the following relations were used:

$$f_{cm} = f_{m0} \cdot \left( 1 + 2 \cdot \frac{f_l}{f_{m0}} \right) \quad (8)$$

$$\varepsilon_{cm} = \varepsilon_{m0} + \frac{2 \cdot A_{tot,f}}{b_d \cdot s} \cdot \frac{f_r^2}{\varepsilon_{m0} \cdot E_s \cdot f_{cm}} \quad (9)$$

## II. EXPERIMENTAL INVESTIGATION

The effectiveness of the proposed strengthening technique to improve strength and ductility of unreinforced masonry members, eight uniaxial compression tests were carried out on specimens of masonry columns made of solid clay bricks, unconfined or confined by stainless steel wires inserted into the horizontal joints. Four of the eight specimens were made up of bricks and mortar of type 1 (the mechanical characterization is illustrated in the following section) and four of them were made up of the same bricks with mortar of type 2, similar to the first one, but with the addition of hydraulic lime. For each kind of mortar two columns were confined and the two others were not confined. As is shown in Fig. 1, each column was made up of five rows of bricks that were not sandblasted but assembled with vertical and horizontal cement mortar joints with an average thickness of 8-10 mm. This thickness was chosen in agreement with the dimensions of the 10 mm prisms, although this value is small if compared with the prevalent thickness of cement mortar joints in masonry constructions, which ranges from 10 mm to 25 mm.

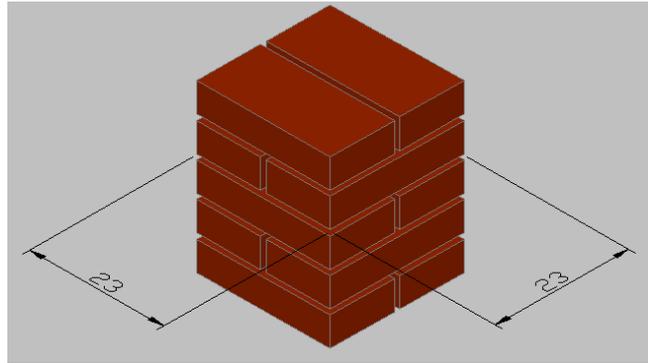


Fig. 1 Solid brick columns for specimens

The cross-section dimensions of the columns were 230x230 mm while the total height was 350 mm. The specimens were capped with high-strength mortar with a thickness of a few millimeters to distribute the compression load on the masonry. The collaring wires on the lateral surface of the compressed element were applied within each joint by using stainless steel wires with a diameter of 1.6 mm, collaring the masonry three times (i.e. three coils), as shown in Fig. 2.



Fig. 2 Solid brick masonry columns: a) unconfined; b) confined with steel wires

### A. Mechanical Characterization of Materials

In order to characterize the materials used for the specimens, the following two tests were carried out: uniaxial compression tests on bricks and mortar and uniaxial tensile tests on steel wires. Cubic specimens with sides of 55 mm were prepared with regard to the characterization of the bricks' compressive behavior. The weight per unit volume of clay bricks was  $18.60 \text{ kN/m}^3$ .

Fig. 3 shows the results of compression tests on the bricks; the average strength was about 40 MPa.

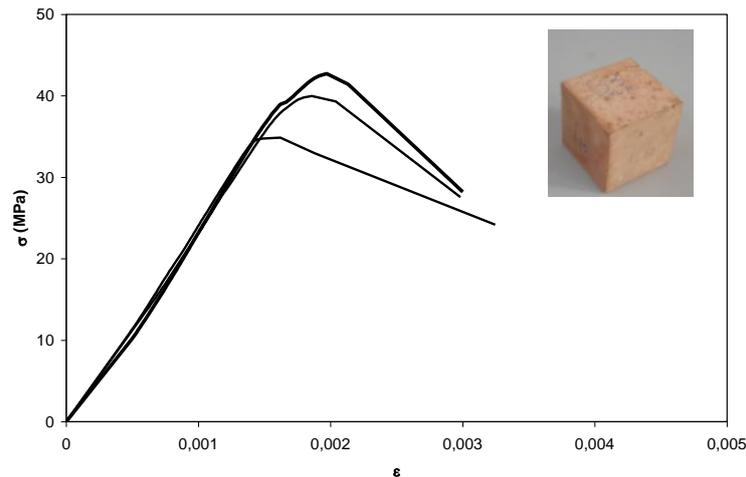


Fig. 3 Stress-strain curves of bricks

In order to characterize the behavior of the mortar, three-point bending tests were carried out on specimens made up of small prismatic beams measuring 40 x 40 x 160 mm. From these specimens, small cubes were obtained to be tested in uniaxial compression. The mortar, as mentioned, was distinguished into two types: type 1 was only made of cement and sand, while type 2 was made of cement, sand and hydraulic lime. Mortar 1 was obtained with a cement/sand ratio of 3:1 and a water/cement ratio of 0.5, ASTM type I. The cement paste of type 2 was the same as that of type 1 with one-third of the cement quantity replaced by lime. Although the composition of this mortar is not that of a mortar of masonry structures, its use was considered significant in this study in order to underline the effect of collaring. In this connection, the confinement acts on the joint of a material characterized by a reduced value of dilatancy, i.e. of a material with a higher stiffness value, due to the presence of a high Portland cement content, so the effects of confinement are reduced. Further investigations will be carried out on mortar typologies for masonry. The values of modulus of rupture of mortar were 6.32, 7.06 and 4.60 for M1-1, M1-2 and M1-3 (mortar type 1), and 3.52, 3.31, 3.89 for M2-1, M2-2 and M2-3 (mortar type 2), respectively.

Fig. 4a) shows the stress-strain curves of cubic specimens of mortars subjected to compression tests. The test results show that the peak strength and deformation values of the different mortars were very similar. The strength value was between 32 and 40 MPa. Paste 2 showed more marked ductile behavior. For characterization of the steel wires, tensile tests were carried out. The steel used had a yielding stress of 400 MPa. Fig. 4b) shows the stress-strain curves of tests on strands made of one wires. Yielding stress was approximately 400 MPa and at rupture 500 MPa. Strain hardening behavior was observed.

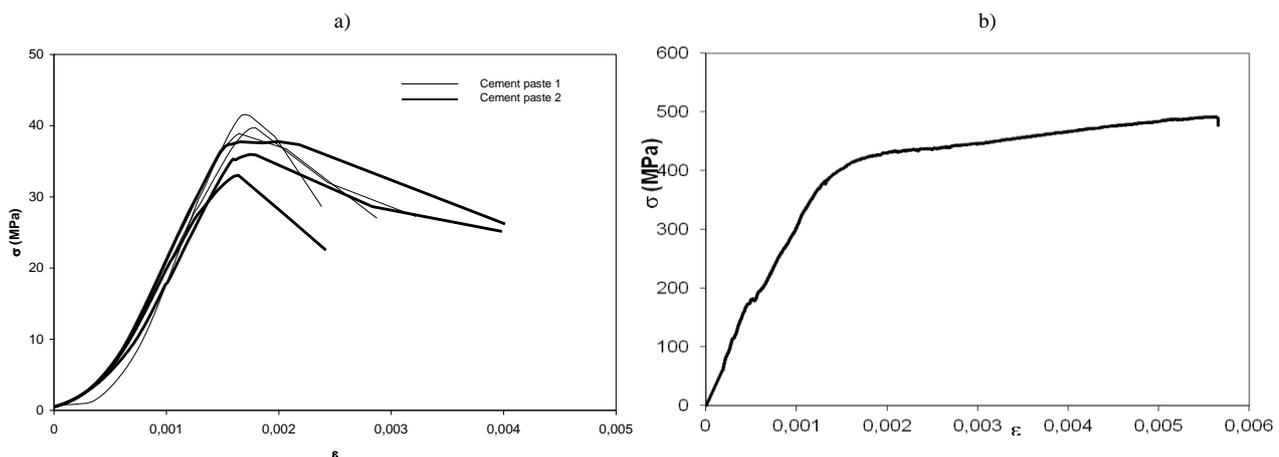


Fig. 4 a) Compression tests on mortar, types 1 and 2; b) Tensile tests on steel wires

### B. Compression Tests on Masonry Columns

All tests were performed in displacement control and the upward load was applied through a cylindrical jack, placed at the bottom of the machine, with a load cell of 4000 kN. The jack displacement was monotonic with a constant speed of 0.5 mm/min. The top of the column was placed in contrast with the upper crossbeam of the machine, which had a spherical joint in order to adapt the crossbeam to the specimen surface with its roughness (rough surface of the crossbeam).

Vertical loads were read directly from the machine through a specific internal transducer linked to the load cell; the loads values were sent to the control software (Testxpert v. 7.11). Moreover, two opposing transducers LVDT were placed between

the bottom load loading plate and the upper contrast one, in order to evaluate the axial shortening of the tested column (Fig. 5). Figs. 6a and 6b show the results of compression tests on masonry with cement mortar joints of type 1 and type 2, in the case of confined and unconfined columns, respectively. During tests a first phase with a linear elastic behaviour can be seen, for which the strains value are small, but when the applied load increases, the stress-strain curve becomes non-linear. This behavior is due to the cracks that appear both in joints and bricks, gradually involving the core of the column, when the failure load is approaching. At the end of the tests the failure of steel wires had never occurred.



Fig. 5 Compression tests on masonry columns

The results of the compressive tests showed the utility of this strengthening technique: while the compressive strength of unconfined columns was 15 and 20 MPa (for mortar of type 1 and 2), the case of confined masonry columns showed a compressive strength of 17 and 23 MPa, with an improvement of 13% and 15% respectively for the two kinds of mortar.

As shown in Fig. 6 the initial elasticity modulus values were 5000 and 10000 MPa for type 1 and type 2 unconfined and confined masonry columns. No differences were observed between unconfined and confined material. The secant elasticity moduli (measured at first peak load) were 3809 and 4750 MPa for unconfined type 1 and 2 materials, respectively, and 5000 and 6571 MPa for confined type 1 and 2 materials, respectively. This highlights the fact that external wrapping does not produce variation in initial stiffness but less marked degradation of secant stiffness when specimens are confined. The strength and pseudo-ductility (measured by the area sustained under the stress-strain curves up to failure) of confined specimens increase with respect to that of unconfined specimens.

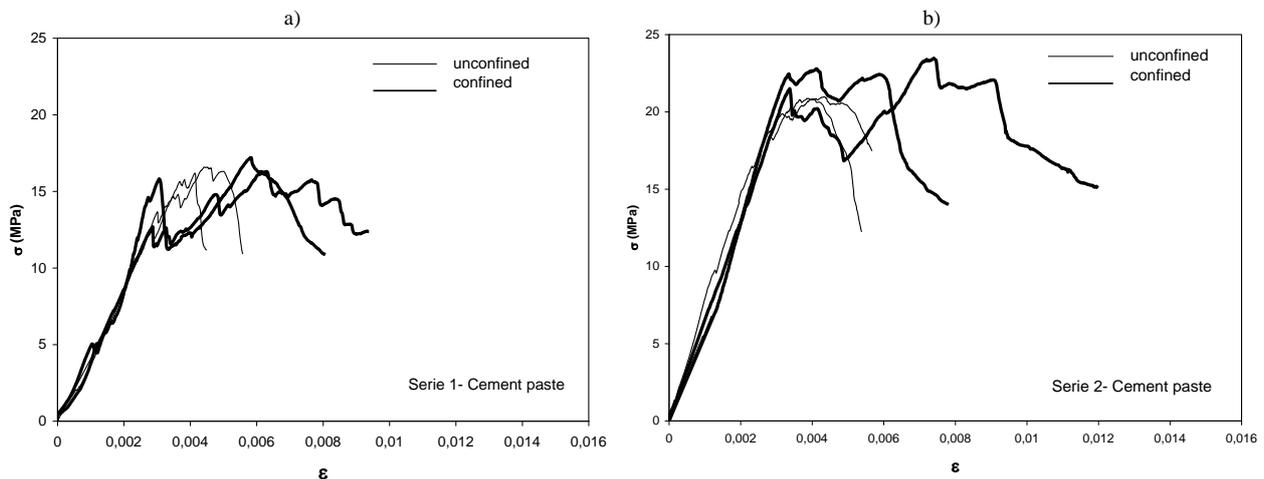


Fig. 6 Compression tests on masonry columns series: a) joints of type 1; b) joints of type 2

In this context pseudo-ductility was introduced instead of ductility because for unconfined specimens it is very limited and is only a representation of the energy dissipated rather than the ability of the structure to deform laterally. In the case of unconfined masonry of type 1 (with stiff mortar, without lime) a reduced number of cracks could be observed: the bricks cracked but only a few joints were affected by cracking. In the case of unconfined masonry of type 2 (mortar with lime), several cracks occurred, often affecting the joints (as shown in Fig. 7a). In the case of collared columns (Fig. 7b) the masonry was broken up mainly in the areas of the edges, showing that the confinement was more effective. Masonry elements next to the steel wires were less cracked than the central part between two joints and the cracked elements remained attached to the

steel reinforcement. It has to be stressed that using one, two or three strands no strand rupture at the conclusion of testing was observed.



Fig. 7 Failure modes of masonry columns. a) Unconfined; b) confined

This technique has three main advantages: - it is very simple to use on site, because the steel wires can be manually rolled around the column and can be inserted in joints, disappearing into the mortar; for the same reason it is cheap, which is an important characteristic for actions on monuments and buildings in the architectural heritage; it is effective, especially for the increment of ductility under axial loads. By contrast it has to be stressed that when the steel reinforcement is embedded into existing cement mortar joints and the joints are re-pointed, the technique is not completely reversible and particular care has to be taken in selecting compatible mortar so as not to create stress points in the masonry.

### III. ANALYTICAL INTERPRETATION OF RESULTS

In this section the compressive response of masonry elements wrapped by steel wires is considered. The model of Campione and Miraglia [8] discussed in the previous sections was here adopted. The results obtained with analytical models are compared with the experimental data obtained by the authors and with others found in the literature calculating the effective confinement pressures using the Campione and Miraglia [8] model. Comparisons are made with test data on solid clay brick masonry with steel wires in the joints.

The value of the masonry elastic modulus  $E_0$  is the one directly determined through compression tests on unconfined masonry. The previous expressions were applied to the specimens in this study; the analytical stress-strain curves were drawn and then compared with the experimental ones. Fig. 8 shows a comparison between the proposed analytical model and the experimental curves in this study. It can be noted that the analytical model reproduces with good approximation the elastic modulus at the origin and the maximum value of compressive strength, up to the peak value.

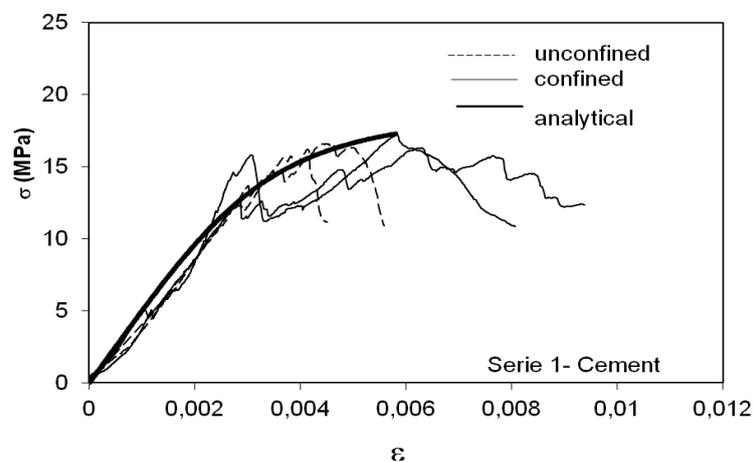


Fig. 8 Comparison of results for confined masonry columns with the proposed analytical model

For a clearer comparison, the strength enhancements due to the steel collaring of masonry columns were also calculated referring to the analytical expressions given in Table 1 of Di Ludovico et al. [2] and CNR DT 204 [9], Corradi et al. [5], Kreviakas and Trintafillou [4]. Fig. 9 shows the comparison between above mentioned studies and experimental results here derived. A comparison with the results of the experimental investigations presented by Borri and Corradi [7] and Iurina (2009) regarding clay bricks confined by steel wires is made. Both these references report researches on masonry confined by pre-tensioned steel wires or strands, inserted in the joints. In the investigation by Borri and Corradi [7] columns made of clay bricks with cross-section dimensions of 250x250 mm were used, with eight rows of bricks for a total height of 500 mm, obtained by assembling two bricks that were not sandblasted, with dimensions of 245 x 120 x 55 mm and lime joints with a thickness of 8-10 mm. The mortar compressive strength was 10.75 MPa, while the bending strength was 3.56 MPa. The clay bricks had a value of compressive strength of 20.78 MPa and a tensile strength value of 2.95 MPa evaluated through the bending test. The strengthening technique used strands made of 3SX fiber, composed of brass-coated steel filaments with high carbon content. Each strand was obtained by rolling up 4 filaments, three of them gathered by a thinner one. The mechanical properties of the 3SX strands were verified through tensile tests, measuring an average failure load of 1407 N and a failure strain of 2.5%. The square specimens (the strengthened one named as number 4 and the unconfined one named as number 17) had the ultimate strength values of 17 MPa and 14.8 MPa, respectively. The increase in strength was about 14.8 %, agreeing with the results found in the present study. In the experimental investigation presented by Iurina [6], clay brick masonry columns with octagonal cross-section were tested with an octagon side of 200 mm.

TABLE 1 ANALYTICAL EXPRESSIONS FOR ULTIMATE STRENGTH OF FRP CONFINED MASONRY MEMBERS

Author	Analytical expressions
Di Ludovico et al. (2007)	$f_{mc} = f_{m0} \cdot \left( 1 + 1.53 \cdot \left( \frac{f_l}{f_{m0}} \right)^{0.90} \right)$
CNR DT (2004)	$f_{mc} = f_{m0} \cdot \left( 1 + \frac{\gamma_m}{1000} \cdot \frac{f_l}{f_{m0}} \right)$ with $\gamma_m$ masonry mass-density expressed as kg/m <sup>3</sup>
Corradi et al. (2007)	$f_{mc} = f_{m0} \cdot \left( 1 + 2.4 \cdot \left( \frac{f_l}{f_{m0}} \right)^{0.83} \right)$
Kreviakas and Trintafillou (2005)	$f_{mc} = f_{m0} \quad \text{if} \quad \frac{f_{mc}}{f_{m0}} \leq 0.24$ $f_{mc} = f_{m0} \cdot \left( 0.65 + 1.65 \cdot \frac{f_l}{f_{m0}} \right) \quad \text{if} \quad \frac{f_{mc}}{f_{m0}} > 0.24$

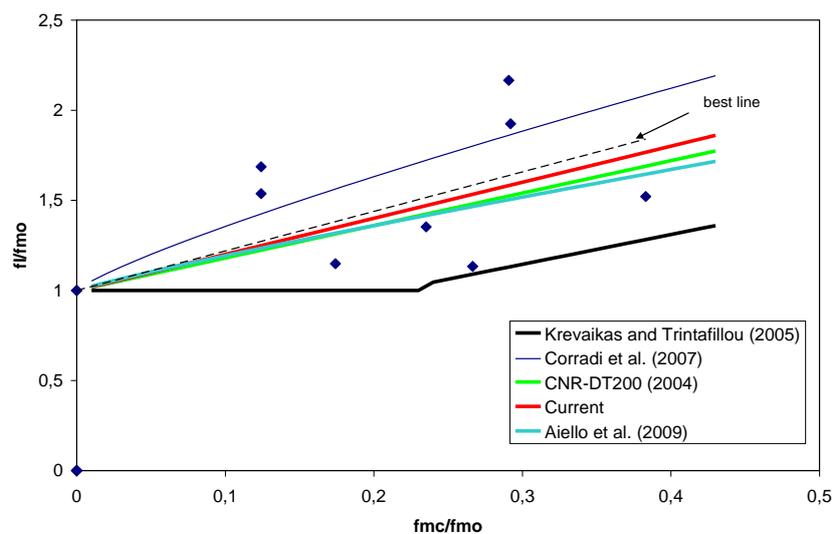


Fig. 9 Variation in dimensionless compressive strength with dimensionless confinement pressures

Bricks were not sandblasted and their dimensions were 245 x 120 x 55 mm, with lime joints with a thickness of about 8-10 mm. The columns had a diameter of 520 mm and a total height of 1200 mm. Confinement was achieved through a steel strand

made up of 10 filaments, the steel having high carbon content, and the diameter of each filament was about 1 mm. To each filament a pretension of 250 N was applied. The confinement technique was applied in all horizontal joints for specimens 5 and 6 and in alternate joints for specimens 3 and 4, while specimens 1 and 2 were not confined. The latter had a failure load of 757 kN and 825 kN. Specimens 3 and 4 had failure loads of 1275 kN and 1162 kN, while specimens 5 and 6 had failure loads of 1455 and 1637 kN. The maximum displacements in the six specimens were the following: 14.66, 29.60, 48.97, 46.20, 60.79 and 61.28 mm. In this case the increase in strength was more evident, due to the pretension of the strands.

In all cases examined it was observed that the effects of external wrapping were increased in strength and pseudo-ductility of masonry columns. As a consequence, a higher area was sustained by the stress-strain curve in compression. If stress-block parameters are utilized (with the stress block approach for sectional analysis of masonry walls in plane and out of plane) higher values are expected with respect to those of unconfined masonry columns. Because of the limited number of specimens tested, no particular values are here suggested.

#### IV. CONCLUSIONS

In the present work the effect of confinement on masonry columns made of solid clay bricks was investigated, using a technique in which the horizontal joints were collared by steel wires. A comparison of the experimental results with the stress-strain curves derived using an analytical model is shown. The analytical model was initially proposed for elements wrapped by FRP and it was adapted here to the case of discontinuous strengthening through steel wires.

The following conclusions can be made on the basis of the experimental and analytical results:

- The external strengthening obtained by steel wires inserted in the joints is a passive confinement that is capable of slightly enhancing the load-carrying capacity, with a significant increase in ductility.
- The behavior of confined columns is sensitive to mortar properties; particularly with a lime mortar (type 2) the dilatancy makes it possible to better activate the effects of collaring, showing greater improvements in ductility.
- From the analytical point of view, the model originally proposed by Campione and Miraglia (2003) for reinforced concrete elements wrapped by FRP can also fit the stress-strain curves in the case of masonry columns confined by steel wires, even though some coefficients of the model have to be adapted to the new methodology.

The proposed technique is very simple to adopt on site for masonry column strengthening because steel wires can be manually rolled and fixed around the columns and can be hidden in the joints. These peculiarities make it a technique that is particularly suitable for architectural heritage masonry, being reversible and not invasive. Moreover, ordinary steel wires for passive confinement can be more convenient than pre-tensioned ones because they are simpler to apply. Further studies and developments will regard other types of mortar, a different number of steel coils around the column and other typologies of loads (axially centered or eccentric loads).

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