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Masonry columns behaviour analyses due to a different mode of confinement with GFRP straps

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Abstract

The paper describes the experimental research of masonry columns behavior under the load of vertical compression. A total of thirty-four specimens were tested: three unconfined specimens and thirty-one specimens confined with GFRP straps. In addition to the load-bearing efficiency analysis of confined columns in relation to the number of the confinement layers, the intention of these tests was to determine the efficiency of spiral confinement in relation to conventional confinement. The impact of the existing compressive stress in a column during confinement to the final increased load-bearing capacity of the confined column was also studied.. The test results have shown that all of the confined specimens have a greater load-bearing capacity and ductility than the unconfined specimens. The results of spiral confinement were almost identical to the results of conventional confinement, which is vital considering that spiral confinement is easier to perform. The results of the test lead to the conclusion that the presence of compressive stress in a column during confinement does not significantly reduce confinement efficiency. This makes it possible to effectively increase the bearing capacity of masonry columns without the need to previously unload the structure, while the structure is in service. The paper also provides expressions for the estimated increase in the compressive strength of confined columns that well correspond to the testing results.

Keywords: *tests, masonry columns, strengthening, glass fibre reinforced polymer straps, GFRP, results*

Notation

- E_f modulus of elasticity of GFRP straps;
- E_M modulus of elasticity of wall;
- $E_{M1\text{-}3}$ secant modulus of elasticity of wall determined for the compressive stress of 1,0 to 3,0 MPa;
- F compressive force in masonry column during testing;
- f_f tensile strength of glass fibre fabric (GFRP);
- f_M compressive strength of unconfined masonry column;
- f_{Mc} compressive strength of confined masonry column;
- f_l' effective lateral stress in column during confinement;
- ε_{fu} ultimate strain of glass fibre strap;
- ε_M longitudinal strain of column;
- ε_{Mu} longitudinal strain of masonry column at failure;
- ε_{Mc} longitudinal strain of masonry column for $\sigma_M = f_{Mc}$;
- ε_l lateral strain of column;
- σ_M compressive stress of column;

1. Introduction

Due to the growing need for reconstruction and rehabilitation of the existing masonry, new methods and technology of strengthening the existing load-bearing elements of masonry structures are being investigated today. The greatest number of investigations is focused on increasing the load-bearing capacity of masonry structural elements by applying fibre reinforced polymer (FRP) [1]. This type of strengthening has more advantages than the traditional methods. The most important benefits of such a type of strengthening are in the fact that the existing bearing structure of the building is not undermined; that it is fast and simple to perform, and that it does not violate the aesthetic requirements of the building and its functionality during the strengthening process. Although FRP is not a ductile material, it could be used for strengthening masonry structures as the collapse is mainly achieved through masonry. Over the past few years extensive experimental, analytical and numerical investigations of masonry wall [2], as well as masonry columns being strengthened with glass fibre reinforced polymer straps (GFRP) have been conducted. The purpose of these investigations was to research and describe in the most helpful way the behaviour of masonry walls subjected to horizontal in-plane load strengthened with glass fibre straps as well as the behaviour of masonry columns under compressive load that are confined with glass fibre straps.

The paper shows a part of the investigation relevant to the behaviour of masonry columns under variable axial compressive load until failure. Experimental testing of confined and unconfined masonry columns were conducted, which is only a continuation of similar investigations in the world [3], [4], [5]. The experimental research was aimed at determining the impact of the method and volume of confinement with glass fibre straps on increasing the compressive strength of masonry columns. Considering that in practice it is difficult to obtain full load release of a column during confinement, the compressive strength of a column during confinement on the increase n load-bearing capacity was analysed within the conducted investigations. The aim was also to obtain good working diagrams of the confined columns behaviour, as well as the relations between longitudinal and lateral strains with the load change for further numerical and analytical analysis. The paper shows the results of the completed experimental investigations and proposes analytical expressions for the estimate of the total bearing capacity of confined columns.

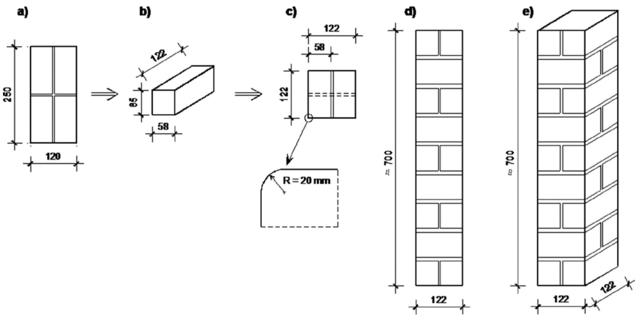

Fig. 1. Tested sample of masonry column

2. Experimental program

a. Test specimens and material properties

Testing was conducted on small masonry column specimens with dimensions $a/b/h$ = 122/122/700 mm (Figure 1). All specimens were made with masonry elements of solid brick of width/height/length = 58/65/122 mm, made by cutting brick of standard dimensions w/h/l = 120/65/250 mm into 4 identical parts. The mean values of tensile strength and compressive strength of bricks were 3.98 MPa and 23.33 MPa, respectively. The mortar used in making specimens was cement-lime mortar of the same volume content (cement/lime/sand = 1/3/9). The mean value of compressive strength of mortar was 5.70 MPa, tensile strength of mortar was 1.74 MPa. Straps and glass fibre fabrics were used for confinement of masonry columns, along with the epoxy adherent and levelling mortar. The description of the testing procedure was given in Soric et.al. [6]. The tensile strength of FRP composite (glass strap embedded in epoxy resin) was 890.1 MPa; strain at failure: $\varepsilon_{f,max}$ = 19.3 ‰; modulus of elasticity was 46169 MPa. The properties of levelling paste declared by the manufacturer were: compressive strength > 80 MPa; tensile strength > 30 MPa; strain at failure 12 ‰; modulus of elasticity 3000 MPa. The specimens had curves R = 20 mm on vertical edges to avoid sharp edges and possible damaging of strengthening straps at the bending location of vertical edges.

Twenty-six masonry columns were tested to determine the impact of the method and volume of confinement with glass fibre straps on the increase in compressive strength of masonry columns: three unconfined column specimens (type A) and twenty-three confined specimens. Confined masonry columns were strengthened in eight different ways (Figure 2). Seven series had three specimens, and one had two specimens. All confined specimens were confined with the same GFRP straps (Mapewrap G UNI-AX 900/60, Mapei); with the only difference being the method of confinement (horizontal, spiral), the number of strap layers and their width.

Specimens of type B, C and D were confined with horizontal confinement (conventionally confined specimens) over the entire surface. Confinement was made with two 300 mm wide straps, and one 100 mm wide strap (2×300+100=700 mm = height of specimen). The strap length depends on the number of confinement layers (1-3 layers). The strap overlap of these specimen types was 100 mm. The neighbouring straps were set to each other without overlapping.

Specimens of type E, F, G have spiral confinement with 100 mm wide straps. The confinement was made with a vertical overlap. The value of overlap varies from the type of specimen. E type specimens have vertical overlapping of 5 mm and represent specimens with one layer of spiral straps. F and G type specimens have an overlap of 50 and 70 mm respectively and represent specimens with two or three layers of spiral straps. The percentage of confinement which corresponds to one, two or three layers of glass fibre strap is made by a smaller or larger spiral pitch. Specimens of type H and I have a spiral confinement with 50 mm wide straps, but the unconfined space between two straps is also 50 mm. H type specimens have one layer of strap while I type specimens have two layers. This type of confinement is used to determine strengthening efficiency without covering the entire specimen surface.

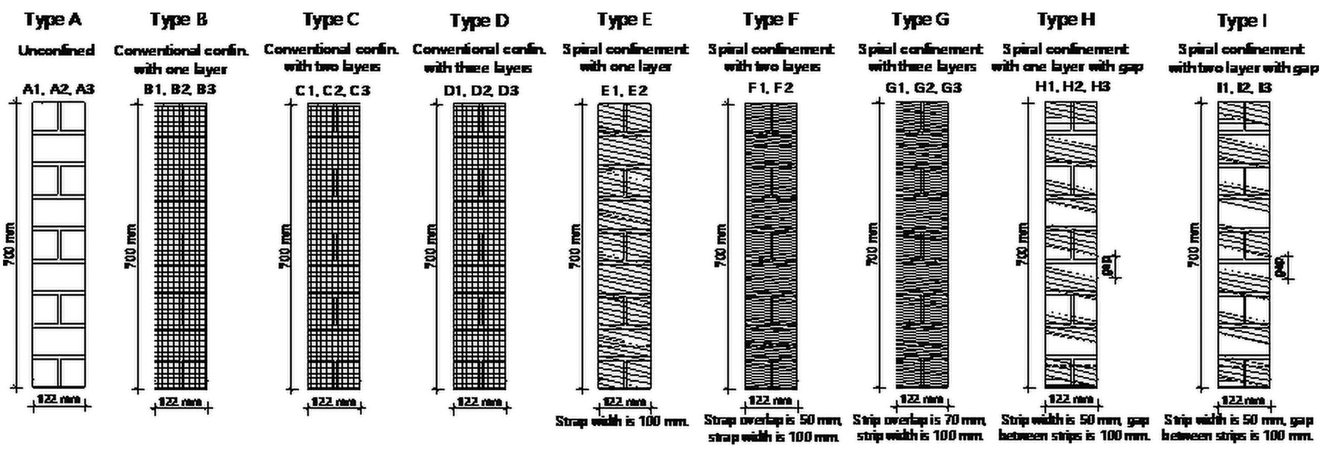

Fig. 2. Specimen type of confined masonry columns

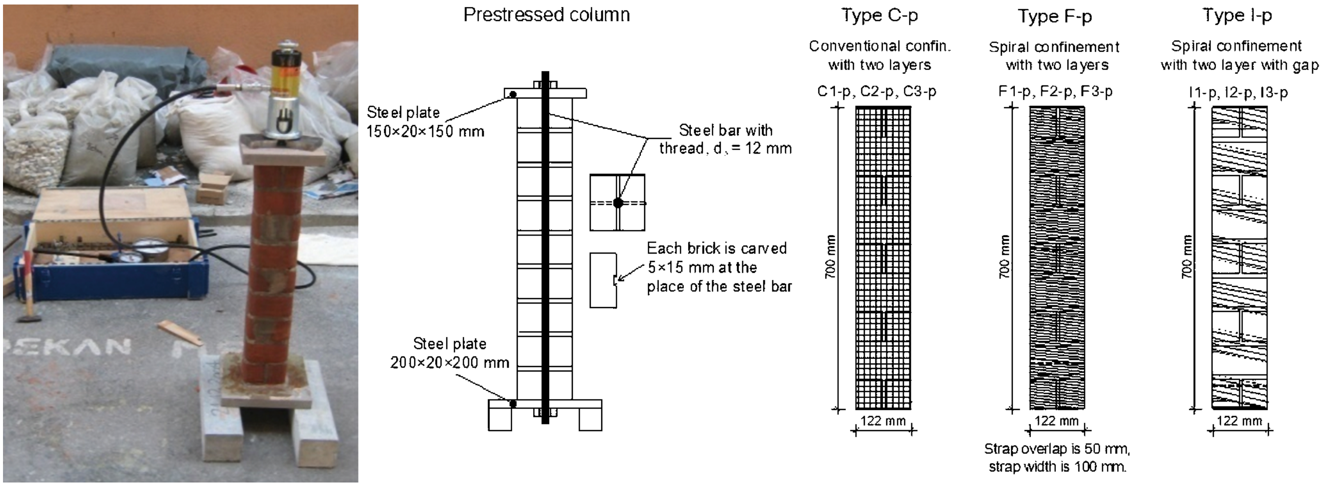

Fig. 3. Specimens which were exposed to compressive load (prestressing) during confinement

All specimens were made on 15 mm thick steel plates. Before confinement with GFRP straps, masonry surfaces of the specimens were well brushed, coated with primer, and then levelled with levelling paste. After the column surfaces had been levelled, glass fibre straps were glued on surfaces using epoxy resin.

Additionally nine masonry columns were tested to determine the impact of compressive stress in column during confinement on the increase in bearing capacity of the confined masonry column.

All nine specimens were confined and their method of confinement matched the strengthening types C, F and I shown in Figure 2 except for their exposure to compressive load during confinement. Compressive load was achieved by prestressing with a steel bar d_b = 12 mm placed at the centre of the cross section. Compressive stress during confinement was σ_M = 2,0 MPa which corresponds to 21.5% of bearing strength of unconfined masonry column. The compressive stress value was determined based on the fact that compressive stress in masonry columns under the dead load of the structure (after load release during removal of floor layers, facade and useful load) was 20-25% of the wall strength. That way the real situation was simulated in practice of applying FRP strengthening. Prestressed specimens are shown in Figure 3.

b. Testing procedure

The main objective of testing was to record the axial stress-strain curve and the failure mode of all masonry specimens that were subjected to axial load applied monotonically under the displacement control mode in a compression testing machine. The testing machine was Zwick Z600E with a 600 kN capacity. Before the beginning of the main testing every specimen had been "trained" , i.e. loaded twice up to force F = 30.0 kN (corresponding to stress σ = 2.0 MPa) and unloaded to 0 kN. In the specimens that were prestressed immediate-

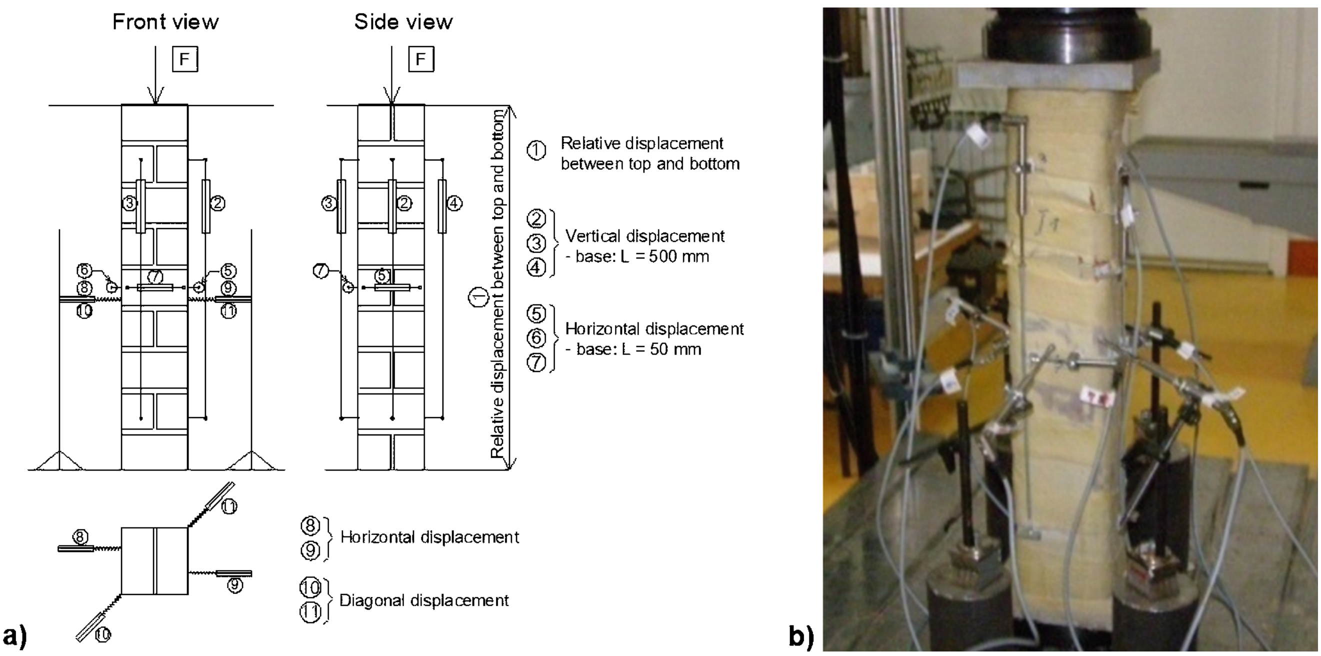

Fig. 4. a) Scheme of loading and LVDT setup and b) Specimen during testing

ly before testing, the specimens were unloaded by releasing prestressing force and removing the bar in the middle of the section. Specimens were tested using displacement control, at a displacement speed of 0.2 mm/s. The loading scheme and LVDT setup is shown in Figure 4. During testing the value of vertical compression force (F) was measured as well as the vertical displacement of the hydraulic jack (1). Also, vertical deformations were measured in three places (2, 3, 4) as well as horizontal deformations in the middle of the specimen height (5, 6, 7). Horizontal displacements (8, 9, 10, 11) in the middle of the specimen were also measured.

3. Experimental results and discussion

Table 1 shows the mean values of test results for each series of column specimens. The following values are shown: F_{max} = maximum compression force; f_{Mc} = compressive strength of confined masonry specimen; ε_{Mu} = sample failure (ultimate) strain; $E_{M1\text{-}3}$ = starting modulus of elasticity (secant modulus of elasticity for the stress level from σ_M = 1.00 to 3.00 MPa); E_M = secant modulus of elasticity for the stress level $\sigma_M = 0.3 \times f_{Mc}$.

a. Description of the behaviour of particular specimen types

Unconfined columns (Type A) – For a comparison with other specimens, three unconfined (reference) specimens were tested. The first visible (vertical) cracks appeared at approximately 90% of compressive strength. Just before the brittle failure of the specimen, they spread and developed at the full height of the specimen. The results of testing and the appearance of the specimen are shown in Figure 5.

Conventionally confined columns (Types B, C, D) – Until the compressive force F = 230 to 250 kN (σ_M = 14.68 to 15.83 MPa) for the specimens type B, the force F = 275 kN (σ_M = 17.36 MPa) for the specimens type C, and the force F = 320 kN (σ_M = 19.65 MPa) for the specimens type D, there was no visible damage of specimens. Specimens acted as a single monolithic structure which is confirmed by the diagram "σ_M-ε_M" up to these stress

Table 1. Test results for column specimens (mean values)

Type of specimens	F_{max} (kN)	f_{Mc} (MPa)	ε_{Mu} (‰)	$E_{M1\text{-}3}$ (MPa)	E_M (MPa)
A	140.1	9.45	2.45	7078.3	7078.3
B	287.2	18.20	20.46	7283.7	6370.8
C	349.1	22.04	24.12	7257.7	5648.6
D	456.7	28.04	40.92	7348.0	5049.5
E	294.1	18.78	17.15	7320.0	6619.3
F	361.6	23.73	25.76	7269.8	6082.2
G	429.5	26.57	31.24	7252.1	5646.6
H	252.4	15.74	7.46	7125.5	6379.2
I	252.4	16.27	7.36	7160.3	6283.9
C-p	327.0	20.67	24.49	7074.0	5859.9
F-p	323.0	20.39	20.03	7177.5	6283.6
I-p	254.6	16.28	7.66	7074.5	6359.1

values. Up to the previously stated values of compressive load the serviceability of specimens was not impaired. After that the first visible damage followed by a cracking sound begins to appear. There are no visible vertical or horizontal cracks, but the fabric confinement at the place of horizontal joints begins to fold (Figure 6). This folding increased with an increase in compression and deformation. The reason for that is crushing of mortar in joints between two bricks, which leads to greater longitudinal masonry deformation at these places.

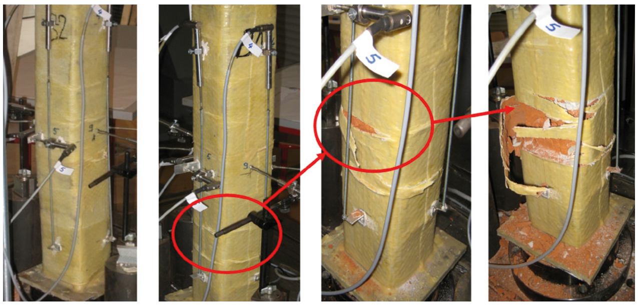

Fig. 6. Development of specimen damaging during testing until failure (B2 specimen)

After the first visible damage had appeared, the increase in the deformation increment at the same load increment was significantly higher than before the damage. This could be seen in the diagram "σ_M-ε_M", which has an approximately horizontal branch (Figure 7a). That "yielding" pattern is the result of mortar and bricks crushing where the confinement straps do not allow its decompo-

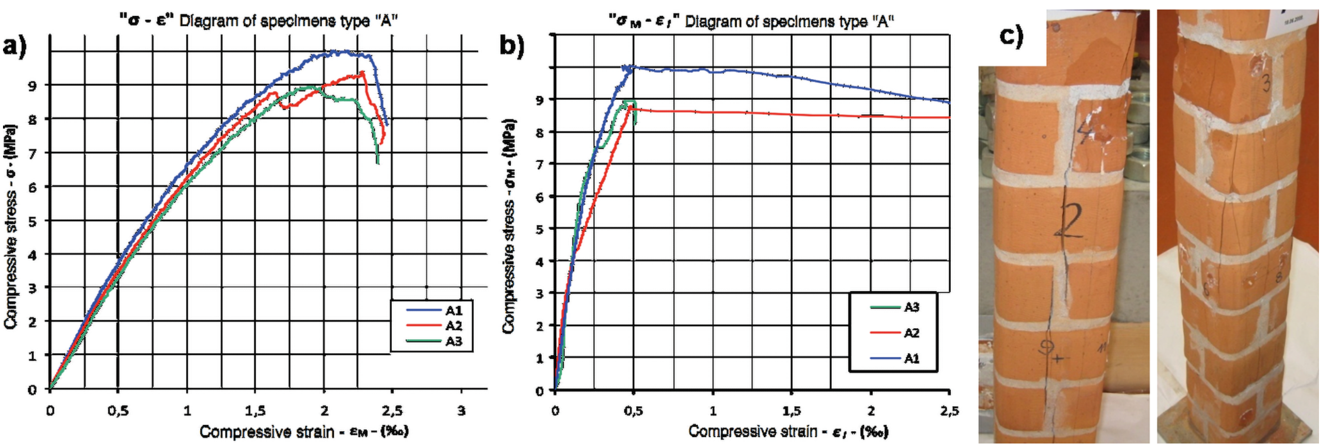

Fig. 5. Test results of unconfined specimens: a) "$\sigma_M - \varepsilon_M$" diagrams; b) "$\sigma_M - \varepsilon_l$" diagrams; c) appearance of specimen failure

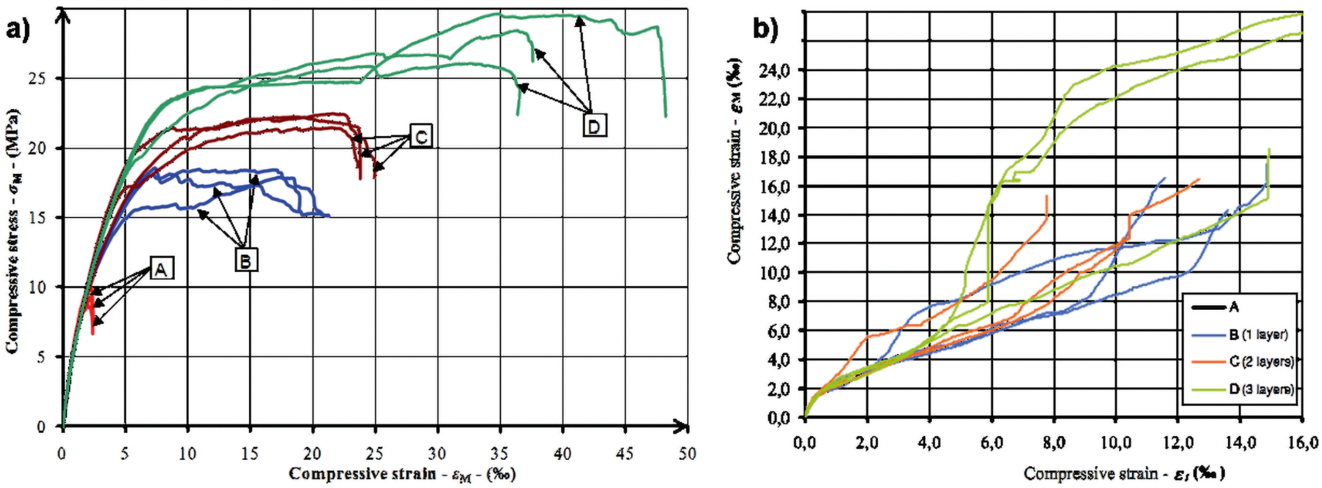

Fig. 7. Comparison diagrams of specimens A, B, C and D: a) "$\sigma_M - \varepsilon_M$" diagrams; b) "$\varepsilon_M - \varepsilon_l$" diagrams

sition (the effect of sand in a bag). Furthermore, "yielding" is the result of specimen bending where bending is not solely the result of physical stability loss. Bending is also the result of material homogeneity lack where the damage is localized in one part of the specimen, resulting in larger deformations in the area which causes buckling. This occurrence should be taken into consideration in the case of slender columns since it affects the stability and bearing capacity of columns. With the number of fabric layers increasing, the strength and deformability of specimen increased too. The specimen failure occurred when local deformations caused confinement failure at one specimen edge. The straps were damaged due to folding and inclining little brick pieces into the strap. The tearing of straps occurred in one of the places where confinement was significantly folded and where masonry was completely squashed and almost turned into dust. Conventionally confined specimens had significantly greater load bearing capacity than unconfined specimens. They also had much greater ductility, i.e. there was no brittle failure at the point where maximum stress was reached. The failure of the specimen was ductile with large longitudinal and transversal deformations (Figure 7a). Specimens of type D with three layers of confinement had the greatest ductility, while the specimens of type B with one layer, the lowest. The confinement increased the specimen compressive strength up to three times in case of confinement with three layers of straps, while the increase in the ultimate strain was as much as seventeen times. An increase in the load bearing capacity and the ultimate strain depended on the number of confinement layers. The greatest step of load bearing capacity and ductility growth was achieved with one layer of confinement in comparison with the unconfined specimen. An additional strap layer increased the load bearing capacity and specimen deformation, but for a smaller step than in case of one confinement layer.

The modulus of elasticity of the unconfined specimen was increased by approximately 5% by specimen confinement. This increase was more a result of primer and epoxy usage than of the confinement itself. The first part of "σ_M-ε_M" diagram is similar for both the unconfined and confined specimens.

After failure stress, the confined specimens showed further resistance followed by large deformation and stiffness reduction due to cracking. It is important to notice that modulus of elasticity of the confined specimens

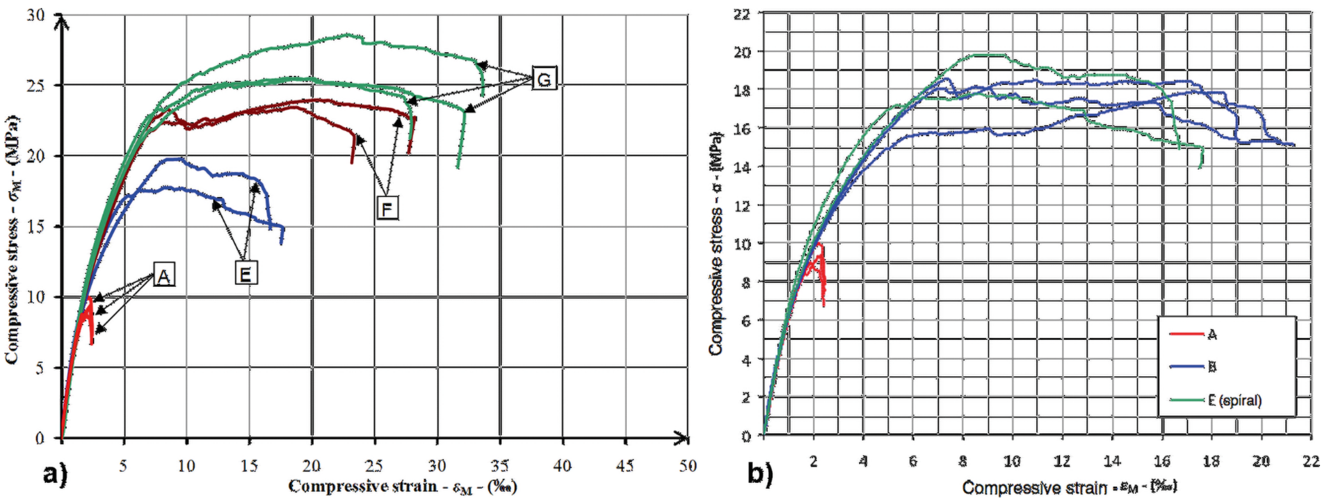

Fig. 8. a) "$\sigma_M - \varepsilon_M$" diagram of spiral confined columns; b) Comparison "$\sigma_M - \varepsilon_M$" diagram for conventional and spiral confinement with one layer of GFRP straps

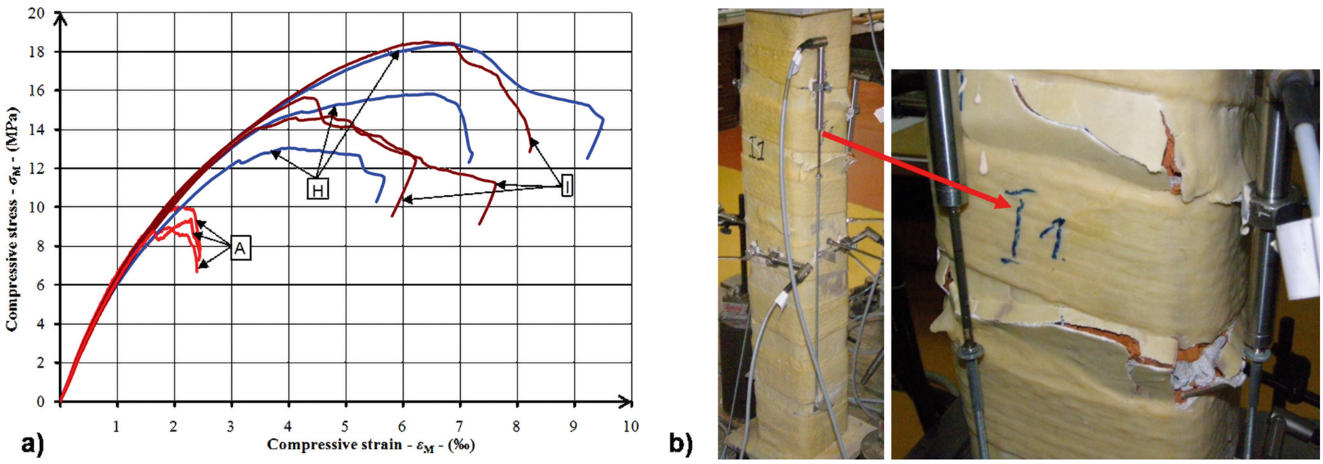

Fig. 9. a) Comparison "$\sigma_M - \varepsilon_M$" diagram of specimens A, H i I; b) failure of specimen I

wasn't proportional to their compressive strength (as it is proposed by codes, for instance $E_M \approx 1000 f_M$). This should be considered in some numerical models.

<u>Spirally confined columns (Types E, F, G)</u> – The only difference between specimen types E, F and G from types B, C and D was the spiral type of confinement. The straps were wrapped spirally because of easier confinement process, especially if there were two or three confinement layers. The behaviour of spirally confined specimens is identical to the one in conventional confinement. Compressive strength of spirally confined specimens was almost identical as in the case of conventional confinement while ductility was somewhat reduced. The specimens still had high ductility and strength, and the reason for reduced ductility in relation to the conventionally confined specimen was that the spirally confined strap could easily be damaged because of the specimen longitudinal deformation which could cause tearing of straps. Figure 8a shows "σ_M-ε_M" diagrams for specimen type A, E, G and F, and Figure 8b gives a comparison of spiral and conventional confinement in case of confinement with one layer of fibre glass strap.

<u>Spirally confined columns with gap between straps (Types H and I)</u> – The failure of both H and I specimen types occurred due to a damage at the area of specimen without straps i.e. between straps. The specimens did not show substantial ductility and considerable deformations. However, the specimens had an increase in bearing capacity compared to unconfined specimens because the straps prevented the development of vertical cracks and splitting of specimens. Figure 9a shows comparative diagrams for specimens A, H and I. These diagrams show an increase in bearing capacity of 80%. However, the increase in bearing capacity was much smaller than in fully confined specimens because the specimen failed in the area between spiral straps (Figure 9b). Increasing the number of confinement layers did not increase the bearing capacity. The reason for that was that the failure occurred in the area without confinement between spiral straps.

<u>Specimens subjected to compressive load during confinement (Types C-p, F-p and I-p)</u> – Behaviour of masonry columns subjected to compressive load during confinement was identical to the behaviour of the same specimens that were not subjected to compressive load during confinement (specimens types C, F and I). In columns with full confinement an increase in compressive strength was smaller by 6.2% compared to the same col-

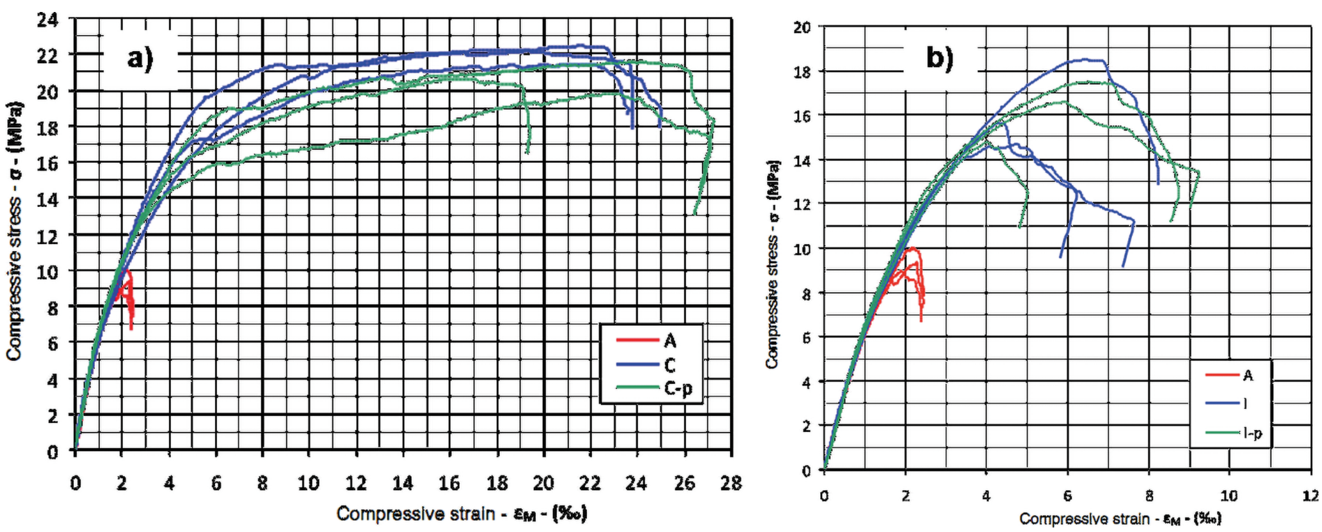

Fig. 10. a) Comparison "$\sigma_M - \varepsilon_M$" diagram of A, C and C-p; b) Comparison "$\sigma_M - \varepsilon_M$" diagram of A, I and I-p

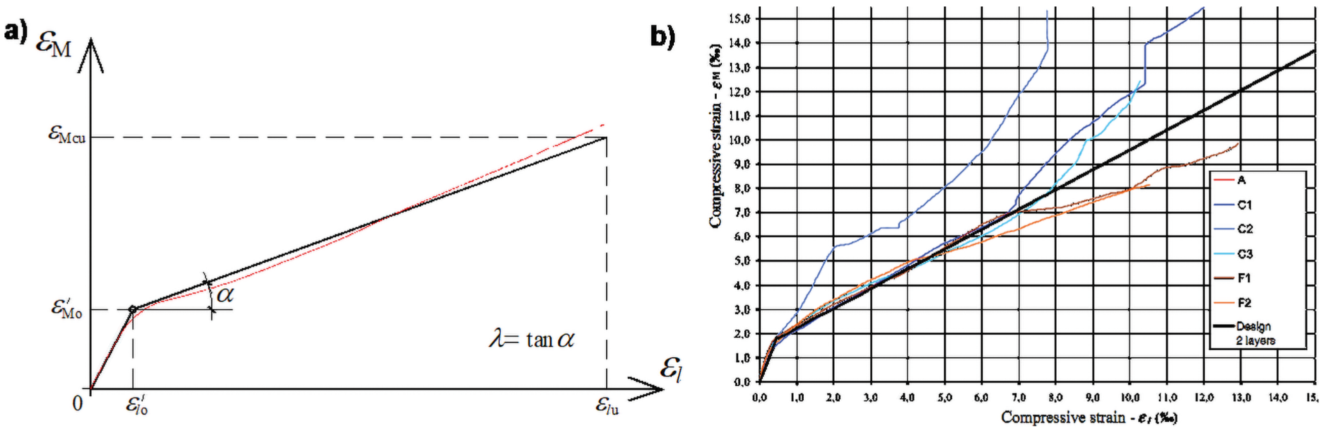

Fig. 11. a) Bilinear “$\varepsilon_M - \varepsilon_l$” diagram; b) Bilinear “$\varepsilon_M - \varepsilon_l$” diagram of confined columns with two layers

umns that were not subjected to compressive load during confinement (Figure 10), while the modulus of elasticity was somewhat higher (3.7%). Although the efficiency of confinement in these specimens is somewhat lower, a further increase in bearing capacity due to confinement is significant compared to unconfined columns (an increase by 118% for confinement in two layers). A slightly higher modulus of elasticity that was achieved is the result of compressive stress during curing of specimen which ensured a better consolidation of the column than in the case of the specimens that were ”trained” during testing. In spirally confined specimens with a gap between straps almost identical results were obtained as in specimens that were not subjected to compressive stress. The results of testing are shown in Figure 10.

b. The relation between longitudinal and lateral strain

In addition to diagram “$\sigma_M - \varepsilon_M$” during testing the diagrams of the relation between longitudinal and lateral strain were analysed i.e. “$\varepsilon_M - \varepsilon_l$”. Although the working diagram “$\varepsilon_M - \varepsilon_l$” has three areas described by the design model for confined concrete by researchers Saenz and Pantelides [8], for practical application the relation “$\varepsilon_M - \varepsilon_l$” can be approximated by a bilinear diagram (Figure 11a) and expressions (1).

$$\text{For } 0 \le \varepsilon_l(f'_l) \le \varepsilon'_{lo} \rightarrow \varepsilon_M(\varepsilon_l, f'_l) = \frac{\varepsilon'_{Mo}}{\varepsilon'_{lo}} \cdot \varepsilon_l(f'_l)$$

$$\text{for } \varepsilon'_{lo} \le \varepsilon_l(f'_l) \le \varepsilon_{lu} \rightarrow \tag{1}$$

$$\varepsilon_M(\varepsilon_l, f'_l) = \varepsilon'_{Mo} + \lambda \cdot (\varepsilon_l(f'_l) - \varepsilon'_{lo})$$

Where: $\varepsilon'_{Mo}, \varepsilon'_{lo}$ = longitudinal and lateral strain that determine the point of failure of diagram “$\varepsilon_M - \varepsilon_l$”; λ = line gradient of the other area of diagram in Figure 11a ($\lambda = \tan\alpha$). A good congruence with the test results was obtained for values: $\varepsilon'_{Mo} = 1{,}74$ ‰, $\varepsilon'_{lo} = 0{,}44$ ‰, λ = 0,71; 0,83; 0,96 (for confinement with one, two and three layers of strap) which can be seen in Figure 11b. For strain values that determine the point of failure of diagram “$\varepsilon_M - \varepsilon_l$” it can be assumed that it is $\varepsilon'_{Mo} \cong 0{,}82 \cdot \varepsilon_{Mo} = 0{,}82 \cdot 2{,}13 = 1{,}74$ ‰ and $\varepsilon'_{lo} \cong 0{,}25 \cdot \varepsilon'_{Mo} = 0{,}44$ ‰. Bilinear diagrams for individual cases of confinement

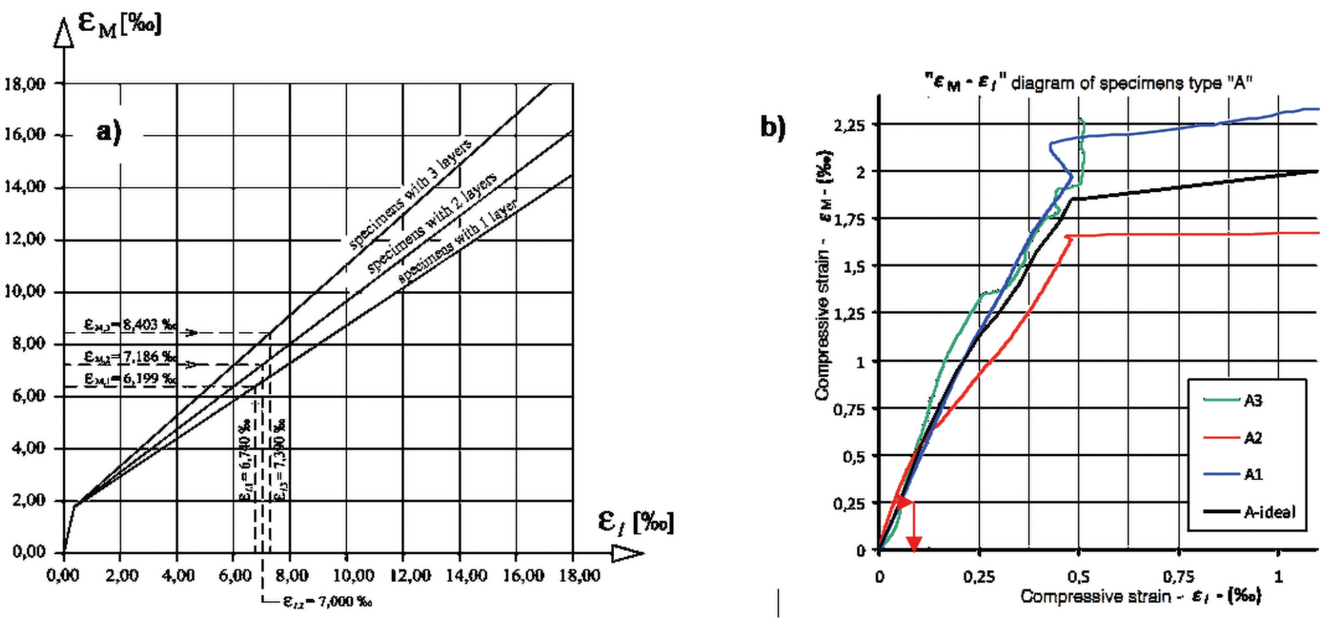

Fig. 12. a) Bilinear “$\varepsilon_M - \varepsilon_l$“ diagram for different number of confinement layers; b) “$\varepsilon_M - \varepsilon_l$” diagram of unconfined columns

with strain that is congruent with the compressive strength of columns are shown in Figure 12a. In line with the testing and graph shown in Figure 12a lateral strain and effective strain in FRP that is congruent with the compressive strength of the confined column is $\varepsilon_l = \varepsilon_{fe} = 7{,}04$ ‰.

Accordingly, stressing in FRP that is congruent with the compressive strength of the confined column is $f_{fe} = E_f \cdot \varepsilon_{fe} = 46169 \cdot 0.00704 = 325.16$ MPa. With that stress value an analysis was conducted in Chapter 4.

4. Estimate of compressive strength of confined masonry column

Based on the testing results the calibration of general expression (2) was made to determine the increase indec compressive strength of confined column as well as the modification of the expression (3) proposed by Mander [9] in 1988, for the estimate of an increase in compressive strength of concrete columns due to the lateral stress.

$$\frac{f_{Mc}}{f_M} = 1 + k_1 \left(\frac{f_l'}{f_M} \right)^{\alpha} \tag{2}$$

Where: α, k_1 = constants determined by calibration of the testing results; f_l' = effective lateral stress in column during confinement taking into account geometric characteristics of cross section in line with [4].

$$\frac{f_{Mc}}{f_M} = 2{,}254\sqrt{1 + 7{,}94 \cdot k \cdot \left(\frac{f_l'}{f_M} \right)} - 2 \cdot k \cdot \left(\frac{f_l'}{f_M} \right) - 1{,}254 \tag{3}$$

Where: k = correction coefficient with which the existing expression for confined concrete proposed by Mander [8] is adjusted for the design of masonry columns that are subjected to triaxial load.

Expression (2) taking into account that lateral strain of the confined column at maximum bearing capacity $\varepsilon_l = \varepsilon_{fe} = 7.04$ ‰ best matches the testing results for values $\alpha = 0{,}68$ and $k_1 = 2{,}3$ i.e. as presented in expression (4).

$$\frac{f_{Mc}}{f_M} = 1{,}0 + 2{,}3 \left(\frac{f_l'}{f_M} \right)^{0{,}68} \tag{4}$$

Columns that were subjected to compressive load during confinement had compressive prestressing $\sigma_M = 2.0$ MPa in which longitudinal strain was $\varepsilon_M = 0.28$ ‰ and lateral strain $\varepsilon_l = 0.05$ ‰ (see Figures 5a and 12b). In the columns that were subjected to compressive load during confinement, the effective strain of FRP that matches the compressive strength of the confined column and amounts to $\varepsilon_{fe} = 7.04 - 0.05 = 6.99$ ‰ was decreased by that exact value of lateral strain.

Taking into consideration a decrease in effective strain of FRP due to the already incurred lateral deformation of the column in prestressing according to expression (5) the obtained value of design compressive strength in expression was $f_{Mc} = 20.45$ MPa. Comparing the design

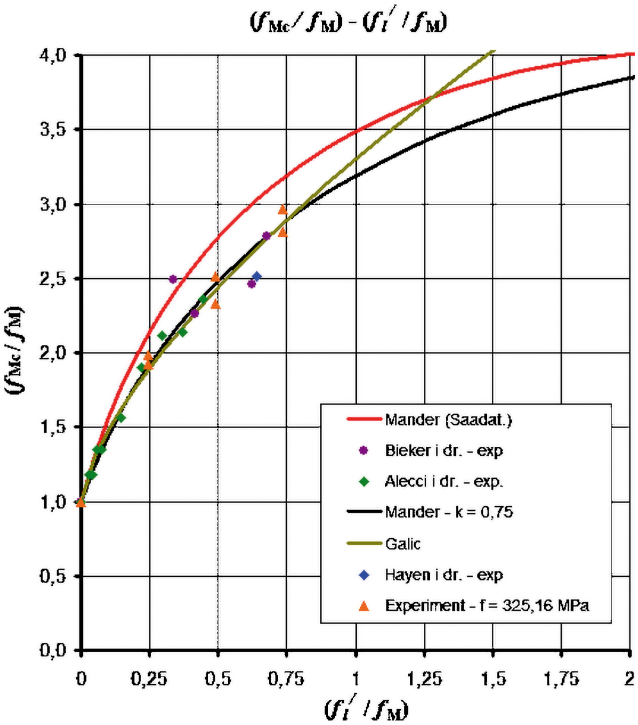

Fig. 13. Comparison of experimental results with theoretical expressions (4) and (5)

compressive strength determined for specimens that were not prestressed before confinement, $f_{Mc} = 21.58$ MPa, and the design compressive strength determined for specimens that were subjected to compressive strength before confinement, $f_{Mc} = 20.45$ MPa, the obtained decrease in comp. strength $\eta = (21{,}58 - 20{,}45)/21{,}58 = 0{,}053$ i.e. 5,3%. A decrease in bearing capacity of prestressed specimens from the previous analysis (5.3%) approximately matches the bearing capacity decrease from experimental testing (6,2% for C-p type specimens). This also explains why the presence of the usual level of compressive stress in masonry columns in practice during the time of confinement has no significant impact on an increase in compressive strength and bearing capacity. In practice it allows the application of confinement of masonry column with compressive load without a need to implement full unloading of columns.

5. Conclusion

Full confinement of masonry columns significantly added to increased bearing capacity. An increase in bearing capacity approximately linearly depends on the number of layers in which the greatest increment of increase in bearing capacity is achieved with one layer of confinement, whereas for several layers the increment of growth is somewhat lower. Full spiral confinement yields results almost identical to full conventional confinement which is important considering that spiral confinement is easier to perform. The use of spiral confinement with a gap between straps increases the bearing capacity, though it is lesser than in the case of full confinement since the failure of column takes place in the area between straps. An increase in bearing capacity in columns with a gap among straps does not depend on the number of layers in confinement; it is almost equal for one, two or three layers. The presence of moderate compressive stress (≈ 0.21 f_M) during column confinement insignificantly decreases the bearing capacity of the confined column compared to columns without compressive prestressing. In practice that permits implementation of confinement in masonry columns with compressive load without necessity for full unloading.

For the practical application, the relation of longitudinal and transversal strain can be described with a bilinear relation diagram. The paper also provides expressions for the estimate of compressive strength of confined columns that correspond with the testing results presented in this paper as well as with the results published by certain researchers [3], [5] and [9].

References

[1] Motavalli, M. & Czaderski, C., FRP Composites for Retrofitting of Existing Civil Structures in Europe: State-of-the-Art Review, In: *Proc. of the Composites & Polycon, ACMA 2007 Conference,* Tampa, USA, October 17-19, 2007. p.10.

[2] Galic, J., Kisicek, T. & Galic, M., Strengthening Masonry Walls with GFRP Straps – Testing and Application. *Advanced Materials Research*, 688, 230-236 (2013).

[3] Bieker, C., Seim, W. & Sturz, J., Post-strengthening of Masonry Columns by Use of Fiber-reinforced Polymers, Presented at *3rd International Conference of Composites in Infrastructure*, San Francisco, CA, USA, 2002.

[4] Krevaikas, D. T. & Triantafillou, C. T., Masonry Confinement with Fiber-Reinforced Polymers. *Journal of Composites for Construction*, ASCE, 9(2), 128-135 (2005).

[5] Alecci, V., Bati, S. B., Ranocchiai, G. Study of Brick Masonry Columns Confined with CFRP Composite, *Journal of Composites for Construction*, ASCE, 13(3), 179-187 (2009).

[6] Soric, Z., Galic, J. & Rukavina T., Determination of Tensile Strength of Glass Fiber Straps, *Journal Materials and Structures*, 41, 879-890 (2008).

[7] Saenz, N. & Pantelides, C. P. Strain-Based Design Model for FRP-Confined Concrete Columns, ACI Special Publication, SP230: 7th International Symposium on Fiber-Reinforced Polymer Reinforcement for Concrete Structures (2005), Paper SP230-57, 1011-1028.

[8] Mander, J.B., Priestley, M.J.N. & Park, R. Theoretical stress-strain model for confined concrete, *Journal of Structural Engineering*, ASCE, 114(8), 1804-1826 (1988).

[9] Hayen, R., Schueremans, L., Van Balen, K. & Van Gemert, D. Triaxial Testing of Historic Masonry, Test Set-up and First Results, *Proceedings of Historical buildings VII*, WIT Press, Southampton, pp. 151-160, Bologna (2001).