BEHAVIOUR OF SMALL MASONRY ASSEMBLAGES REINFORCED WITH A STEEL CORD REINFORCED JOINT REPOINTING

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ABSTRACT: Unreinforced masonry (URM) walls are one of the oldest and most common types of construction elements in the world. However they are susceptible to failure when exposed to overstresses, caused by out-of-plane and in-plane loads. In this paper a new method for reinforcing brickwork masonry using steel cords embedded into the mortar joints is proposed. The average cord diameter of 3 mm, enables reinforcement to be retrofitted to brick and irregular stone-masonry walls. Joints can then be repointed to hide the cords, so that no evidence of strengthening work is visible on the final finished façade. The bending behaviour of 20 brick-masonry small assemblages has been evaluated. Specimens have been prepared using two courses of bricks with nominal dimensions of 215 x 102 x 65 mm and two different types of lime-based mortar. After the mechanical characterization of the materials used for construction and reinforcement, an experimental was carried out to assess the potential of the technique comparing the results of bending test achieved for unreinforced specimens. Test results show an increase of bending capacity and of flexural stiffness up to 49.8 and 475%, respectively.

INTRODUCTION

Masonry is a construction material manufactured as combination of components (stones, bricks or blocks) laid in a cementitious or lime-based mortar and usually arranged in a regular way. Despite of its simplicity of construction, the study and analysis of the mechanical behaviour of masonry structures remains a challenge for researchers and engineers. Masonry is a heterogeneous and anisotropic material. The presence of mortar joints creates resistance to tensile stress, although it is noticeably less than the resistance to compression.

Historic masonry buildings were designed to resist mainly gravity loads with no attention to the actions caused by a seismic event. In general earthquakes introduce in-plane and out-of-plane loads to unreinforced walls. Typical damage suffered by these buildings varies from minor cracking to catastrophic failure. In the last decades numerous seismic events in southern Europe have emphasized the vulnerability of the masonry buildings (Bayraktar et al; 2007), (Corradi et al; 2011), (D'Ayala and Paganoni; 2011), (Brandonisio et al.; 2013). Because these buildings constitute a large part of the European building heritage, a process of structural strengthening is often necessary in order to increase the service life and improve their seismic capability.

Some of the most known traditional strengthening methods used to upgrade the mechanical behaviour of unreinforced masonry buildings are:grout injections used to fill voids and cracks in the damaged masonry assemblages (Schuller et al.; 1994), (Vintzileou and Tassios; 1995), (Binda et al.; 1997); steel-mesh jackting which constitutes of a thin coatings of cement plaster over a steel mesh (Jabarov et al.; 1980), (Sheppard and Tercely; 1980) and ferrocement which is realized applying onto one or both external surfaces of the wall reinforced cement or concrete layers (Prawel et al. 1988).

However these traditional strengthening techniques sometimes have not led to the expected results in term of efficiency (Valluzzi, 2007), (Corradi et al.; 2008) because sometimes they increased the mass of the structure which increase the earthquake induced inertia forces (Gilstrap and Dolan; 1998). These problems

have been overcome by using Fibre Reinforced Polymer (FRP) materials as a replacement for the conventional methods. In the last three decades, FRPs have been used extensively as reinforcement materials for unreinforced masonry panels (Bakis et al.; 2002), because of their characteristics:high-strength, light-weight and low costs availability. FRPs strips or laminates have been externally applied directly on the unreinforced elements in order to increase their shear strength (Triantafillou; 1998), (Valluzzi et al.; 2002), (Roca and Araiza; 2010). However, heritage conservation authorities have prepared guidance which restrictions in the use of these techniques in historic-monumental building, due to the irreversibility of the intervention and the alteration of the external aesthetic facades of the structures (Papanicoulau et al.; 2008). Other studies addressed the strengthening of the mortar bed joints, the technique consists by removing the superficial mortar and then by repointing the joints with reinforcement elements (steel plate, FRP laminates or bars) and new good quality mortar. D'Ayala (1998) proposed a strengthening intervention on an existing masonry building by inserting steel thin plates into the bed joints in order to reduce the cracking associated with bending failures. Valluzzi et al. (2005) carried out and experimental campaign on masonry walls by inserting two steel bars, 6 mm in diameter, and repointing them with hydraulic lime mortar or polymeric resin. The previous technique was applied also in two existing historical masonry buildings (St. Giustina's bell tower and St. Sofia's church). Borri et al. (2013) investigated the behaviour of small masonry columns with four different cross sections reinforced with pre-tensioned steel cords embedded into the horizontal bed-joints.
In earlier studies some of the Authors proposed a system known as 'Reticolatus', which involves inserting into the mortar joints of masonry a continuous mesh of thin high strength stainless steel cords (Corradi et al.; 2010), (Borri et al., 2014). With the aim to further investigate the mechanical behavior of this reinforcing technique, this paper presents the result of an experimental investigation on the use of high strength steel cords used to improve the bending behaviour of small brick masonry panels. Specimens have been prepared using five solid bricks with only one horizontal mortar bed joint. Masonry units have been overlapped on alternate courses and two different hydraulic lime-based mortars have been used. The reinforcing technique is based on the insertion of steel cords in the mortar bed joints previously partially cleared out and then refilled with new mortar.

STRENGTHENING TECHNIQUE DESCRIPTION

The bed joint strengthening technique consists of inserting a high strength steel cord in the mortar bed joint previously raked out by a few (approx. 10 mm) millimeters and repointing with new mortar. It is mainly suitable for masonry characterized by regular courses and is more effective if realized on both sides of the wall. However it can be also applied on stonemasonry wall panels (Fig. 1). In order to apply the reinforcement on existing masonry walls the following stages are necessary:

- Remove any render finishing or plaster from the wall's surface;

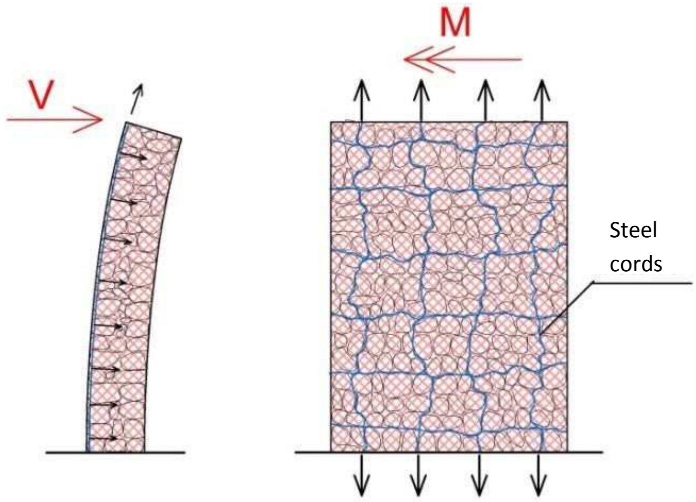

Fig. 1: Mechanical behavior of the reinforcement.

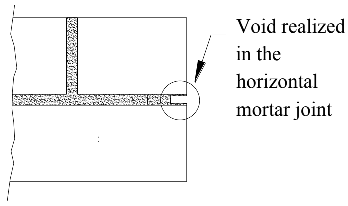

Fig.2: Rake out the horizontal mortar joints.

- Rake out the horizontal mortar joints using appropriate tools (Fig. 2), the excavation should be 10-15 mm deep, with the width dependent on the material used to reinforce the masonry wall and the original dimensions of the joints;
- Air-jet clean the bed joints to remove powder and small debrises;
- Place a first layer of repointing material (mortar);
- Apply the reinforcing materials (steel cord) in the raked residual voids (Figs. 3-4);
- Place a final layer of repointing material to cover the reinforcing material and restore the original aspect of the wall.

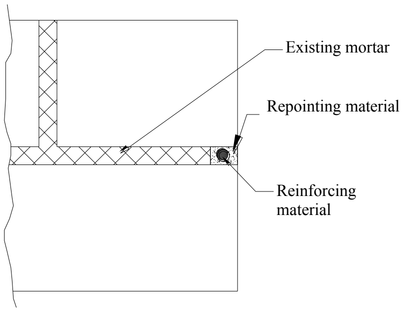

Fig. 3: Detail of the reinforcing technique.

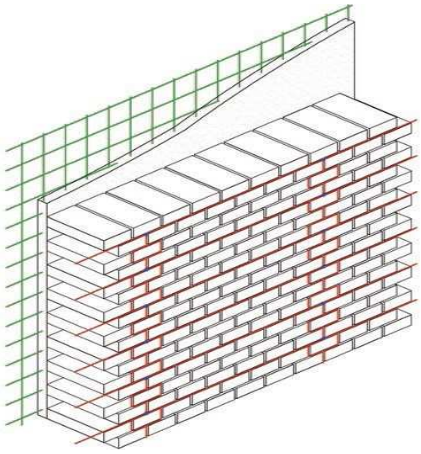

Fig. 4: Example of the application of the reinforcement technique: the indoor façade could be reinforced with a more traditional steel-mesh jacketing or the proposed technique.

MATERIALS CHARACTERIZATION

(1) Bricks

Solid clay units, with nominal dimensions 215 x 102 x 65 mm, have been used for the construction of the masonry specimens (Fig. 5). The compressive strength of the masonry units has been evaluated in according with EN 772-1:2011 standard. Uniaxial compression tests have been carried out on six specimens. Test results showed a mean compressive strength of 121.01 N/mm^2 (Standard deviation (SD) = 9.36 N/mm^2).

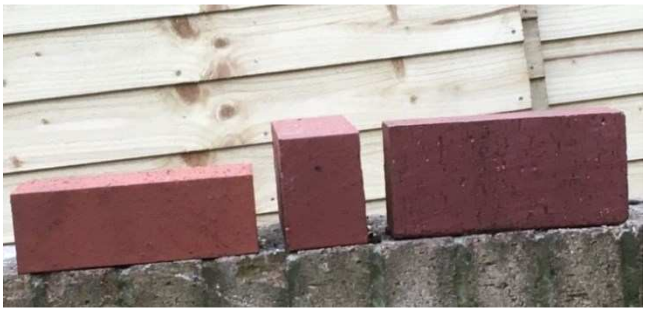

Fig. 5: Bricks used for construction of specimens.

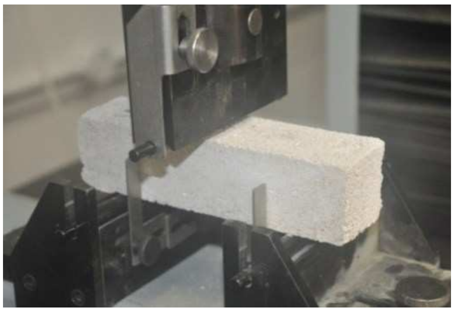

Fig. 6: Bending test on mortar specimen.

(2) Mortars

The masonry assemblages have been prepared using two different lime-based mortars. The mortar type A has a ratio sand/binder = 2/1 in volume while type B is made with a ratio sand/binder = 3/2 in volume. In order to characterize the two mortars, bending and compression tests (Fig. 6) have been performed in according with EN 1015-11:1999 standard. Three 160 x 40 x 40 mm rectangular prisms have been tested in three-point bending and in compression. Specimens have been cured for 28 days at room temperature and test results are reported in Table 1.

Table 1: Mechanical properties of the mortars

Type	Sand/binder volume ratio	Compressive strength [N/mm^2]	Bending strength [N/mm^2]
A	2/1	0.55 (SD= 0.11)	0.39 (SD= 0.10)
B	3/2	0.44 (SD= 0.10)	0.33 (SD= 0.09)

(3)Steel cord

High strength cords are manufactured with four steel filaments each covered with a layer of brass to increase bonding with the mortar and avoid oxidation. Three filaments are wound together by a single external filament characterized by a small diameter compared with the others (Fig. 7). Seven specimens have been tested in tension and the mechanical and geometrical characteristics are described in Table 2.

Fig. 7: Detail of the steel cord.

Table 2: Mechanical properties of high strength steel cord.

Diameter [mm]	1.016
Cross section area [mm^2]	0.810
Failure tensile load [N]	1343
Failure stress [N/mm^2]	1658
Young's modulus [N/mm^2]	206842
Strain at failure [%]	2.3

TEST SPECIMENS

Twenty small masonry specimens (Fig. 8) have been constructed using 5 clay bricks to be tested in three-point bending. They were 440 mm wide and 140 mm high. The minimum width of the masonry assemblages was 102 mm (Fig. 9). Ten specimens have been prepared for each type of mortar. For each batch, five rectangular prisms have been subjected to manual excavation along the horizontal mortar joint, realizing a notch 10-15 mm deep (Fig. 10) where the steel cords have been allocated. After a curing period of 28 days at room temperature notches have been air-cleaned in order to remove powder and possible residual elements, finally two steel cords have been applied for each specimen.

Fig. 8: Unreinforced specimens.

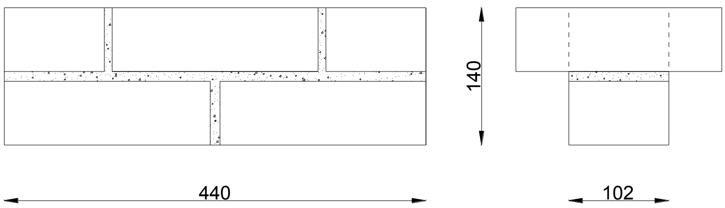

Fig. 9: Geometry of the unreinforced masonry assemblages (dimensions in mm).

Fig. 10: Excavation manually realized in the horizontal mortar joint.

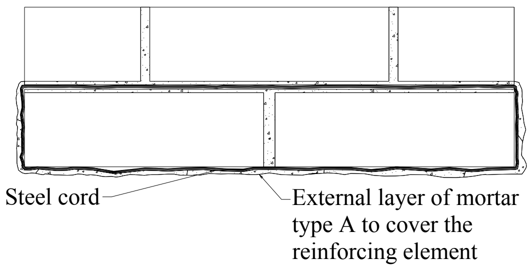

Fig. 11: Arrangement of the reinforcement.

Each cords length was approximately 980 mm to allow the tightening and overlap cord ends. Finally, a layer of mortar type A has been applied to cover the reinforcing cord and restoring the original pointed finish of the horizontal joints of the masonry assemblages (Fig. 11). Reinforcement was applied on both sides of the walls. Table 3 shows the test matrix. Each specimen is identified by an alphanumeric index: the first letter indicates if the sample is unreinforced (U) or reinforced (R), the second the mortar type (A or B) according with the material characterization and the third a progressive number (from 1 to 5).

Table 3: Test matrix.

Index	Mortar used for specimen construction (Sand/binder volume ratio)	Reinforced
UA_series	2/1	No
UB_series	3/2	No
RA_series	2/1	Yes
RB_series	3/2	Yes

The bending stiffness *k* has been evaluated using:

$$k = \frac{(0.4F_{\max} - 0.1F_{\max})}{(d_{0.4F_{\max}} - d_{0.1F_{\max}})} \tag{1}$$

Where: F_{max} is the maximum load and $d_{0.4Fmax}$ and $d_{0.1Fmax}$ are the corresponding mid-span deflections at 40% and 10% of F_{max}.

TEST SET-UP

In order to study the effectiveness of the reinforcement technique all specimens have been subjected to the three-point bending test to evaluate their flexural behavior. The span was 400 mm and the supports were made of two steel semi-cylinder (diameter 40 mm). Load was applied monotonically up to failure using a hydraulic jack with capacity of 250 kN. Deflections at mid-span were recorded using an inductive transducer (LVDT). To avoid the local damage of the sample, the load was applied through a square steel plate with side of 80 mm (thickness 10 mm). A data acquisition system (Geodatalog series 6000) connected with a software (Datacomm) was used to record load, deflection and time readings. The test set-up is shown in Figures 12 and 13.

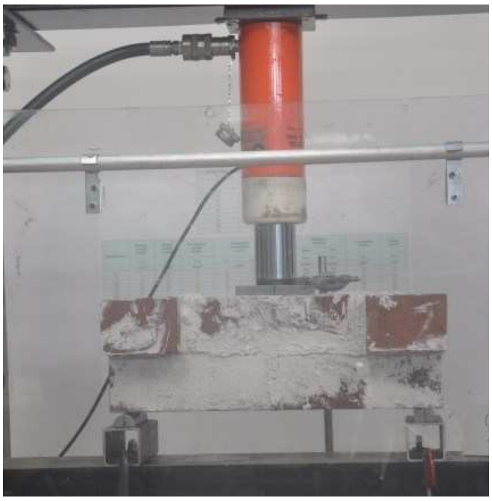

Fig. 12: Three-point bending test.

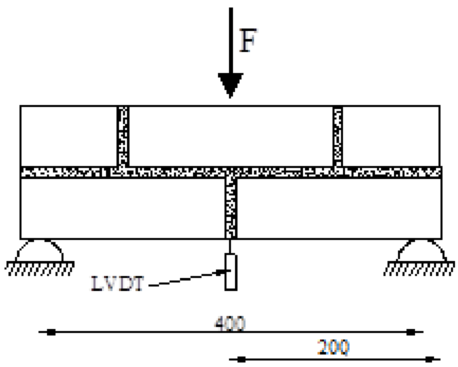

Fig. 13: Test layout (dim. in mm).

TEST RESULTS

(1) Unreinforced masonry specimens

Ten unreinforced masonry specimens (five constructed with mortar type A and the remaining five with type B) have been tested to evaluate the bending behavior for the purpose to study the effectiveness of the strengthening technique through a comparison with the same tests carried out on reinforced samples.

Table 4 shows the results of the tests in terms of failure load and mid-span deflection for each unreinforced specimens. The average failure load is 2.098 kN (standard deviation (SD) 0.312 kN) and 1.035 kN (SD 0.128 kN) for samples built using mortar type A and type B respectively. The difference between the results is due to the different mortar type. As can be seen from Figure 14, the failure, for all the specimens, was due to the mortar cracking and the resulting de-bonding between the lower bricks of the assemblages which produced the failure. Load-displacement curves (Fig. 15) are linear up to the failure for both the specimen typologies and the brittle failure occurred suddenly, without warning for small displacement values.

Table 4: Test results of unreinforced specimens.

Index	Failure Load [kN]	Mid-span deflection [mm]	Stiffness [kN/mm]
UA_1	2.219	2.132	0.940
UA_2	1.879	1.916	0.889
UA_3	1.882	2.066	0.787
UA_4	2.657	2.289	1.069
UA_5	1.913	1.887	0.898
Average	**2.098** (SD=0.312)	**2.058** (SD=0.165)	**0.917** (SD=0.102)
UB_1	0.835	1.181	0.586
UB_2	1.114	1.966	0.575
UB_3	1.122	1.456	0.669
UB_4	0.965	1.658	0.518
UB_5	0.841	1.303	0.707
Average	**1.035** (SD=0.128)	**1.513** (SD=0.310)	**0.611** (SD=0.080)

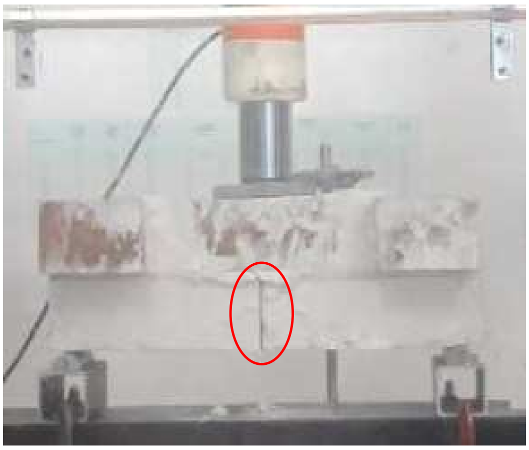

Fig. 14: Mortar failure and de-bonding between the lower bricks of the specimen.

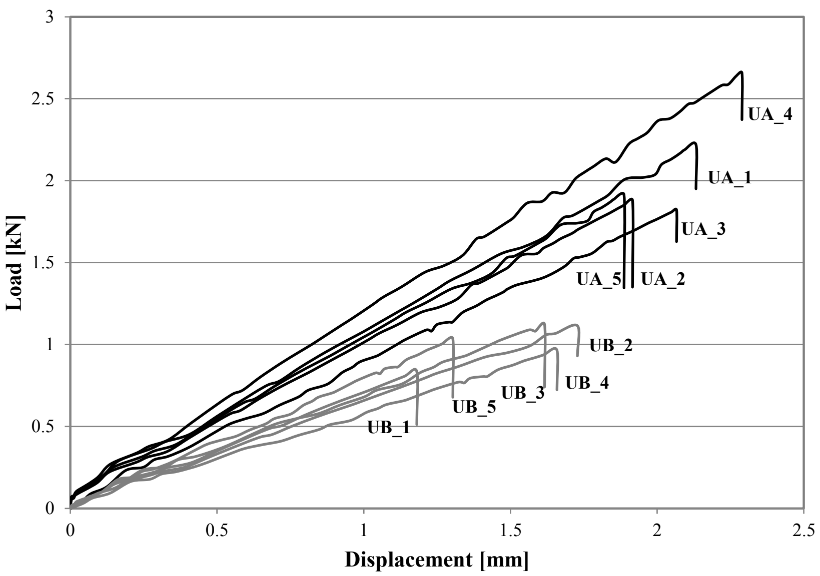

Fig. 15: Load-Displacement curves of unreinforced specimens.

(2) Reinforced masonry specimens

In order to evaluate the effectiveness of the technique described above, ten reinforced specimens (five from the RA and five of the RB series) have been tested in bending (Fig. 16). Test results are reported in Table 5. The average failure load increased to 3.142 kN (SD. 0.195 kN) and 1.718 kN (SD 0.075 kN) for the specimens built with mortar type A and B, respectively. Fig. 16 shows the load – displacement curves for all reinforced specimens: after an initial linear phase, reinforced specimens exhibited a plastic phase in which the load increased less compared with the elastic phase. The linear phase ended when the mortar started cracking and the resulting detachment between the lower bricks. However the value of the load at the end of the elastic phase increased by 34% and 43% for mortar type A and B, respectively. It is evident that the application of the steel cords on the bottom side of the masonry element provided the needed tensile strength. After the mortar cracked, bending loads kept increasing due to the presence of the steel cords until the failure of reinforcement occurred (Fig. 17). Figure 18 shows the load – displacement curves for reinforced specimens (RA and RB). It can be seen that after an initial linear behavior, all samples exhibited a plastic phase in which the bending load increases with large deformations.

Fig. 16: Application of the steel cord in the horizontal mortar joint.

Table 5: Test results of reinforced specimens.

Index	Failure Load [kN]	Mid-span deflection [mm]	Stiffness [kN/mm]
RA_1	3.121	2.773	5.464
RA_2	3.418	2.181	7.321
RA_3	2.905	1.913	3.391
RA_4	3.231	2.031	5.422
RA_5	3.033	2.197	4.766
Average	**3.142** (SD= 0.195)	**2.215** (SD= 0.331)	**5.273** (SD= 1.419)
RB_1	1.768	1.609	2.894
RB_2	1.791	1.672	1.396
RB_3	1.599	2.956	1.855
RB_4	1.706	3.316	1.023
RB_5	1.728	1.621	2.884
Average	**1.718** (SD=0.075)	**2.235** (SD=0.833)	**2.010** (SD=0.854)

Fig. 17: Failure mode of an unreinforced specimen

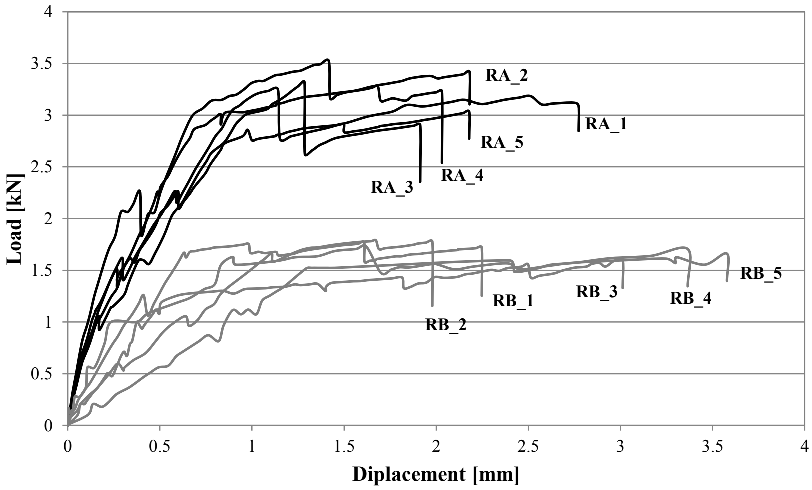

Fig. 18: Load – displacement curves for reinforced samples.

The application of reinforcement produced an increase of the bending capacity (in terms of failure load) and stiffness. For A-series specimens the use of the steel cords caused an increment of 49.8 and 475% (Fig. 19), respectively. The application of reinforcement on B-series samples produced an increase of 65.9 and 229% (Fig. 20) for the failure load and stiffness, compared to unreinforced specimens. It is evident that the application of the described bed joint reinforcement is more effective on elements prepared with B-type mortar, probably because of the poor mechanical properties of the mortar. However, considering the small dimension of the tested specimens, results should be verified with new tests on full-scale wall panels.

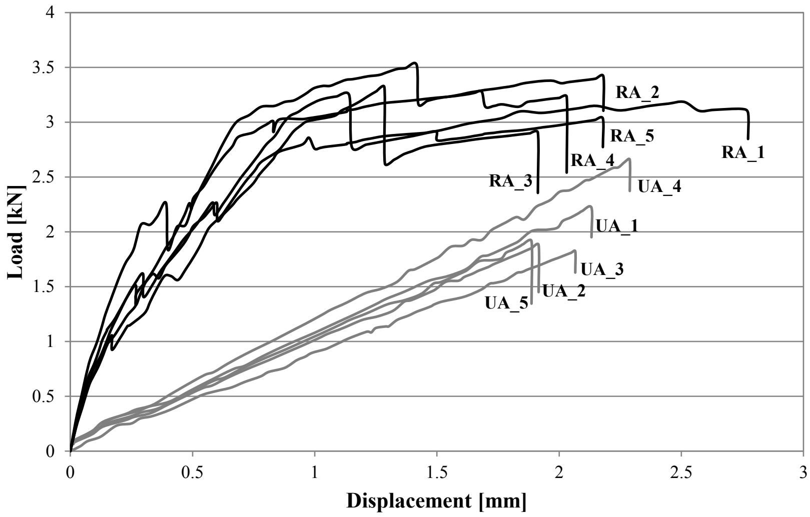

Fig. 19: Load – Displacement curves for unreinforced and reinforced specimens (mortar type A).

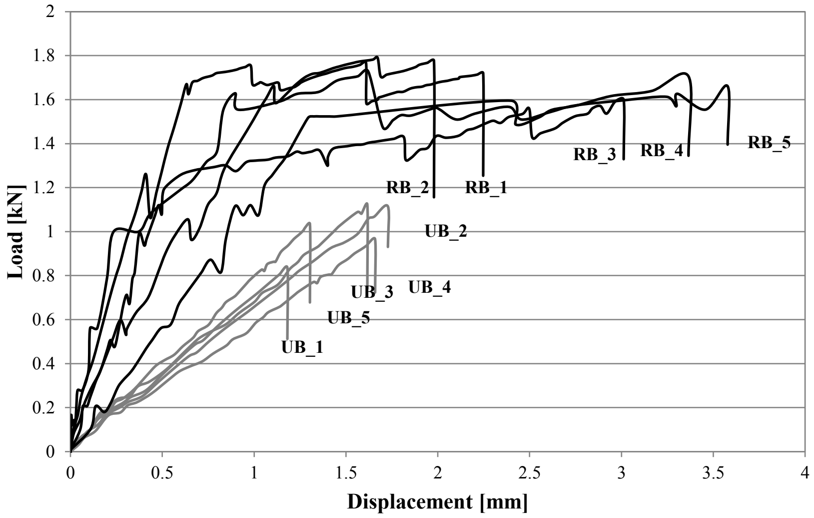

Fig. 20: Load – Displacement curves for unreinforced and reinforced specimens (mortar type B).

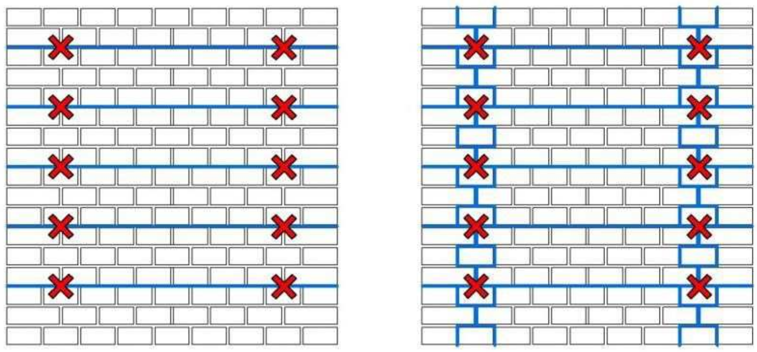

Fig. 21: The reinforcing technique for a masonry wall panel (blue lines: steel cords, red crosses: through connectors).

Fig. 21 shows the application of the described reinforcing technique for a real-scale masonry panel: the steel chords are applied on both faces of the element. However, the reinforcing elements are applied only in the horizontal mortar joints on one side of the wall and in the other in both the horizontal and the vertical joints. The steel cords applied are connected to the other face of the wall by using transverse stainless steel connectors.

CONCLUSIONS

This research focused on the bending behavior of small brickwork assemblages reinforced by the insertion of two high strength steel cords, applied into manually prepared voids and repointed with new mortar. Two different lime based mortars have been used: one (type A) characterized by ratio sand/binder = 2/1 in volume while the other (type B) by ratio sand/binder= 3/2 in volume. Twenty small specimens have been built and tested in three-point bending.

The following conclusion can be drawn from the investigation:

- The unreinforced specimens exhibited a linear-elastic behavior. The average bending capacity of the samples realized with the mortar type A is approximately two times bigger than the specimens realized with mortar type B.
- Both unreinforced specimens are characterized by brittle collapse due to the mortar failure and the resulting de-bonding between the lower bricks of the assemblages.
- Application of the steel cords on the specimens manufactured with both mortar's type produced an increasing of the bending capacity on RA and RB samples. In particular, for A-series, the capacity has increased by 49.8% while for the B-series the same parameter was 65.9% compared with the respective unreinforced masonry assemblages. The reinforcement technique is more effective for elements realized with the weakest mortar type.
- For the reinforced assemblages, the elastic trend finish with the mortar failure and the de-bonding between the lower bricks of the specimens, however, the maximum load at the end of the elastic phase is higher compared with the unreinforced of 34% and 43% respectively for mortar type A and B.
- After the initial linear phase, the reinforced specimens exhibited a plastic phase in which the bending load keep increasing up to failure of the steel cord which lost effectiveness for the excessive bricks' deformation but without breaking and the test finished with the failure of the specimen.
- Reinforced specimens exhibited an increment in flexural stiffness. In particular the highest increase was for the specimens realized with mortar type A (475%) compared with the unreinforced ones. For the specimens made with mortar type B stiffness increased of 229%.
- Considering the small dimension of the tested specimens, the above conclusions should be verified on specimens realized with larger dimensions.

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