News TECHNOLOGY

HFE-tec® masonries High Fracture Energy Technologies

TECNOCHEM ITALIANA SpA

INNOVATION

ECC – ***E****ngineered* ***C****ementitious* ***C****omposites*

- **Rational Systems for Repair, Structural Strengthening and Seismic Retrofitting of Masonries**
- **Fire resistance**
- **Anticorrosion**

Innovative systems with very high performance

SYSTEMS

HFE-tec® masonries

High Fracture Energy Technologies

High Deformation Energy

REINFORCED RENDERING WITH HIGH FRACTURE ENERGY AND DEFORMATION CAPACITY FOR STRUCTURAL STRENGTHENING AND SEISMIC RETROFITTING OF BEARING MASONRIES, PLUGGING, PARTITIONS

DUCTILITY

ENERGY DISSIPATION

RIGIDITY

DIAGONAL COMPRESSION TEST

TANGENTIAL TENSILE

SHEAR LIMIT DEFORMATION

SYSTEMS

HFE-tec® masonries

REINFORCED RENDERING SYSTEMS WITH HIGH DUCTILITY FOR THE STRUCTURAL REINFORCEMENT AND SEISMIC ADJUSTMENT OF MASONRY WALLS

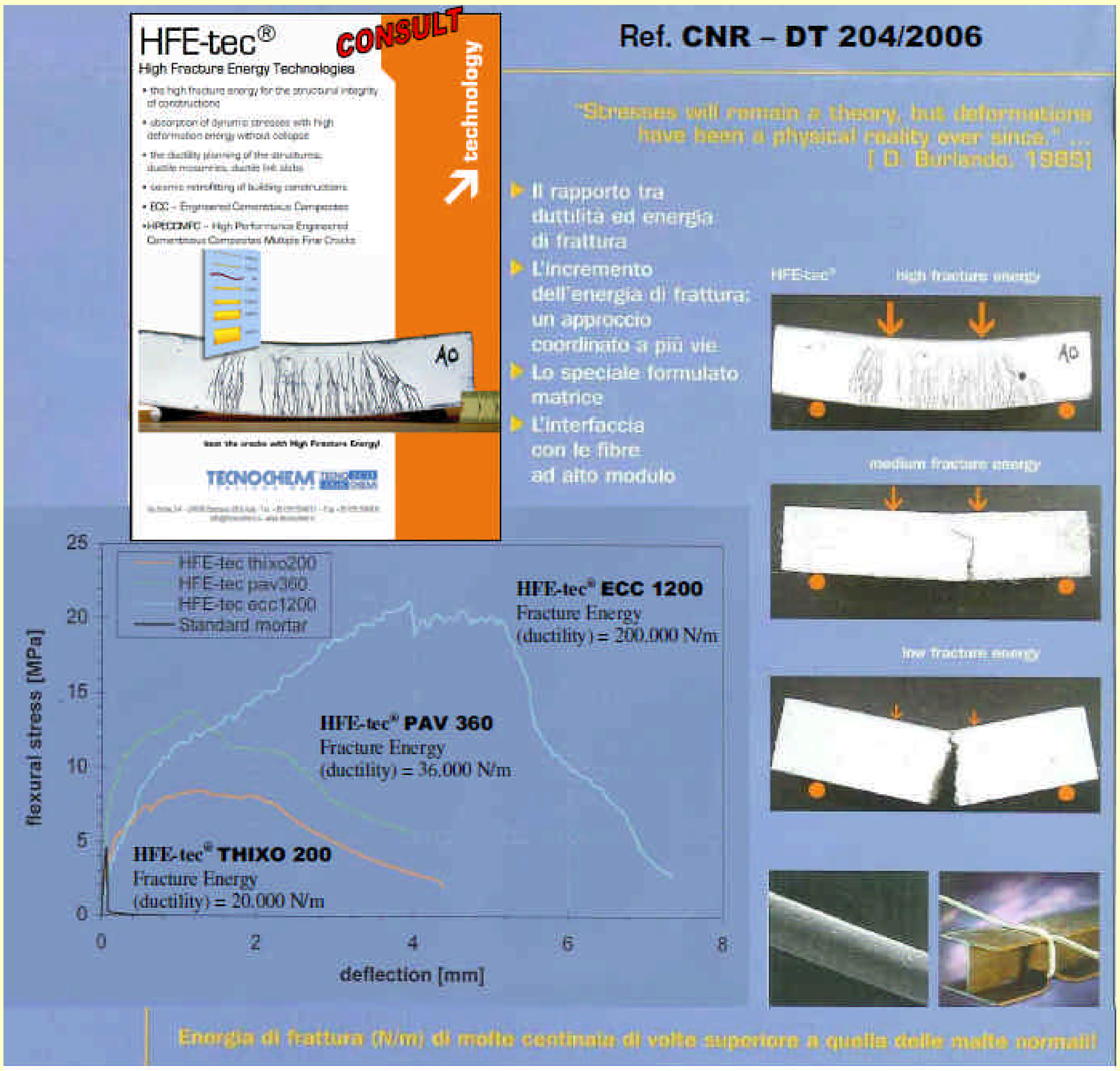
The technique of consolidation using reinforced renders is to achieve, in adherence to the wall to be strengthened, a layer of mortar, reinforced with mesh, fixed to the wall through bulkhead connectors.

The technique allows to improve the characteristics of ***resistance*** of the wall, thanks to the increase in section resistance made from the morar layer and the effect of confinement excercised on the degraded masonry, and, at the same time, to increase the ductility. This technique may be suitable to those walls in a particular advanced state of degradation and not able to withstand excessive handling (in the presence of complex and extended cracking patterns, other techniques, such as injections or the construction of walls, may in fact be difficult to apply).

The technique is also applicable to reinforce a limited section of a damaged wall, or vertical intersections of walls which are not sufficiently connected.
The consolidation with reinforced renders is a simple method and rather rapid, suitable also for seriously poor masonry walls, when adequately and extensively treated, requiring appropriate dimentioning and structural load distribution.

The application of the reinforced render does not change the state of stress of the structure during the execution, but can change the stiffness of the walls and thus their seismic response.

The application of system HFE-tec® masonries ensures maximum reliability of this technique through the following fundamental features : the intrinsic Ductility and Energy of Deformation of used Render, the Adhesive Capacity of the mortar to the wall, some peculiarities in the Mechanical Characteristics of Mortar, the choice Reinforcement net and transversal Connectors, Correct Application.
The Structural reinforcement system HFE-tec® masonries ensures the transfer and stress distribution from the wall to the reinforced render slab with the highest ductility and fracture energy of the SYSTEM, particularly in the case of Seismic Stress.



With our Engineers and Technicians of our ***Project Assistance and Promotion Office***
From Project to Jobsite

➘ SYSTEMS

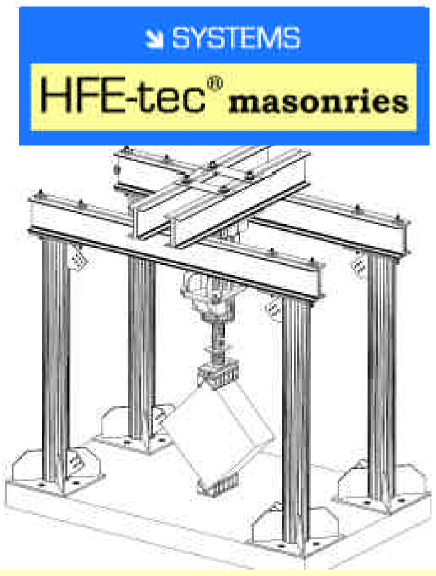
HFE-tec® **masonries**

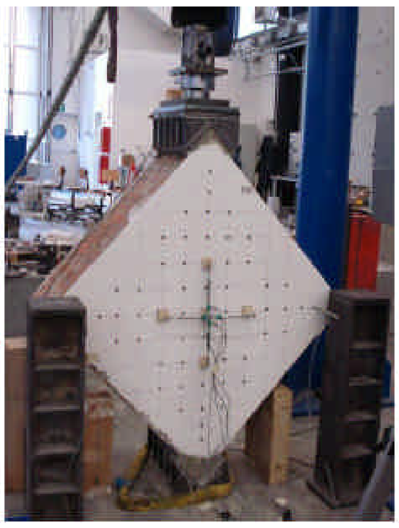
- In the System HFE-tec® **masonries** all the materials which compose the the Design of Structural Reinforcement play a fundamental role and synergy in the transfer of the Forces in the Masonry Reinforcement system.
- **Systems** HFE-tec® **masonries** allow the durable functional recovery in many different grades of degradation and different purposes of structural reinforcement, including seismic adaptation of constructions.
- Fundamental role is played by the quality of the binders and reinforcement used, that must allow:
  - ***Increase of ductility without stiffness or shear differences in the structure.***
    Dissipation of the energy without collapsing in case of seismic event.

Two experimental examples of the System HFE-tec® masonries by measuring the diagonal compressive strength

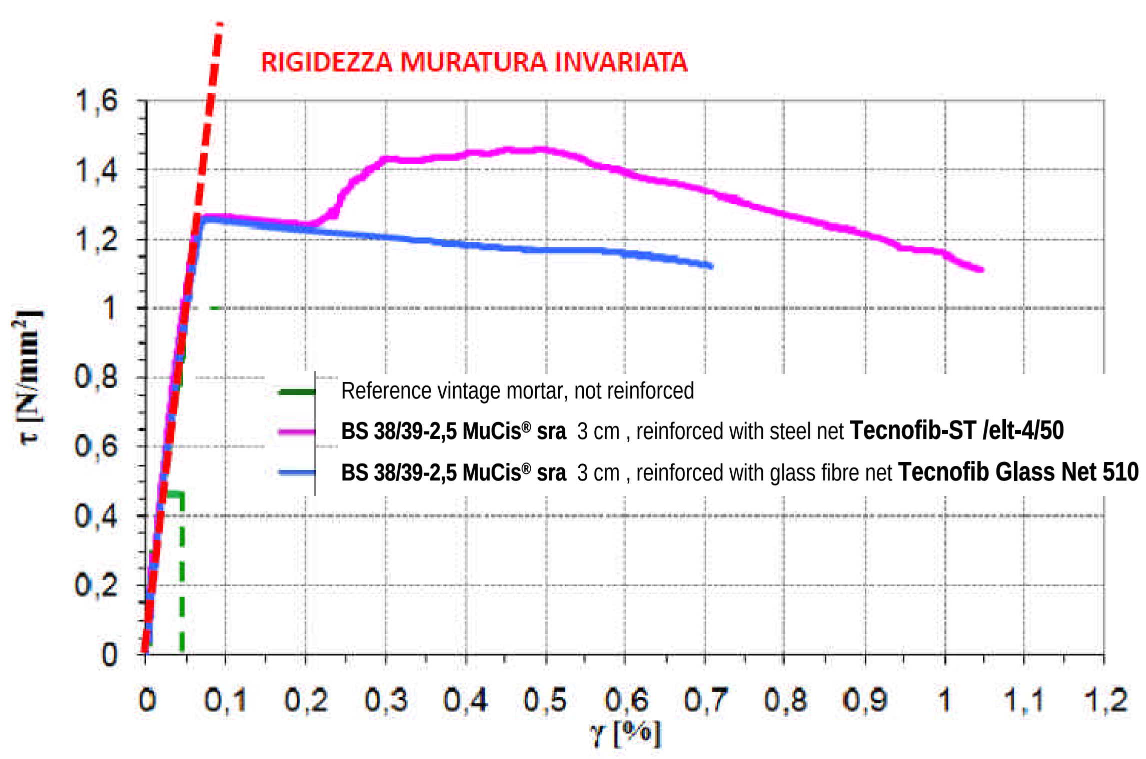
on wall with dimension 100x100x40 cm

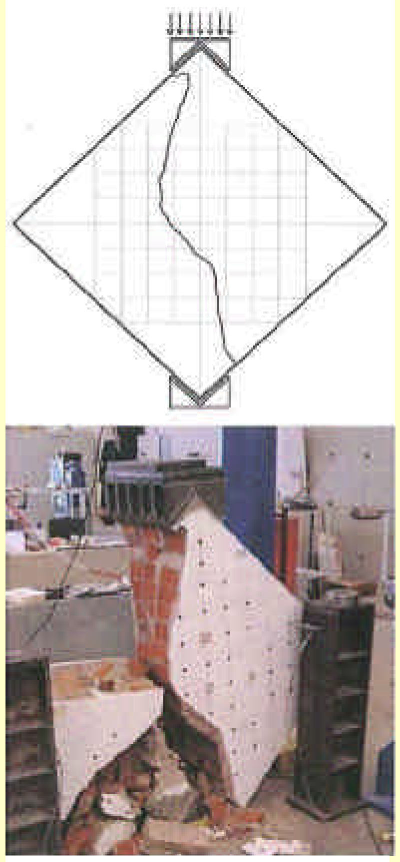
thickness render 3 cm



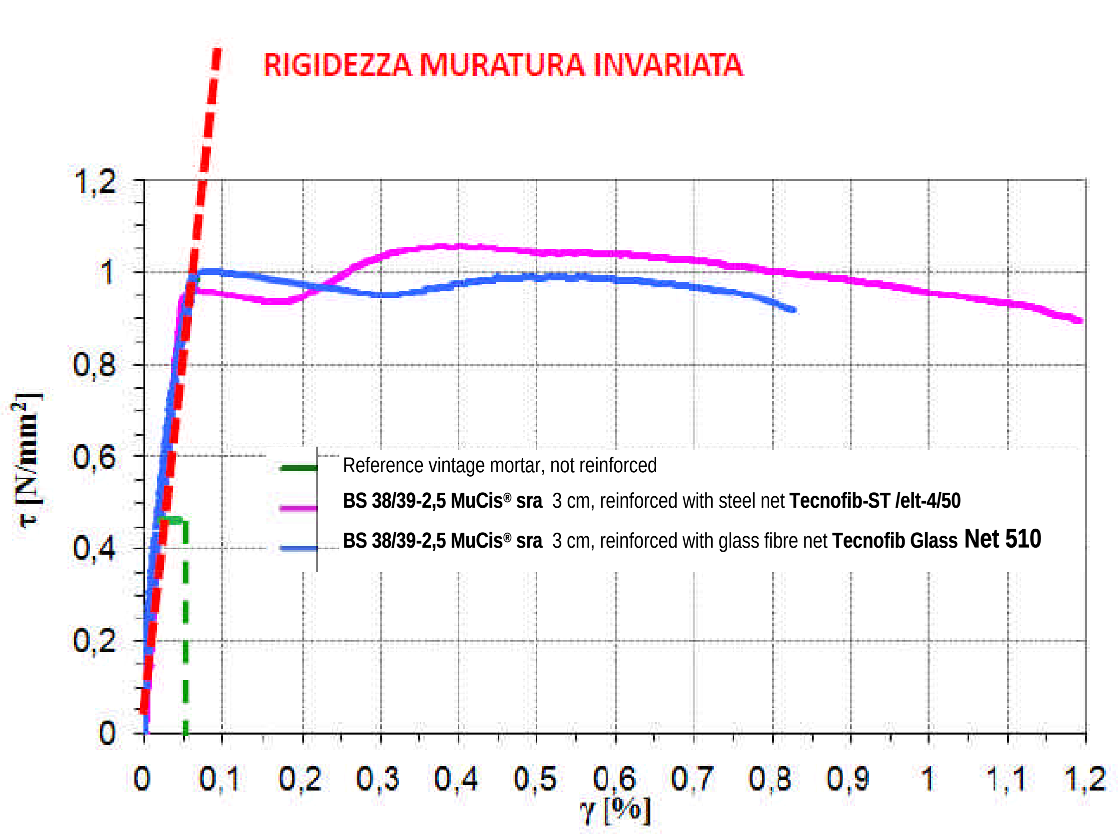


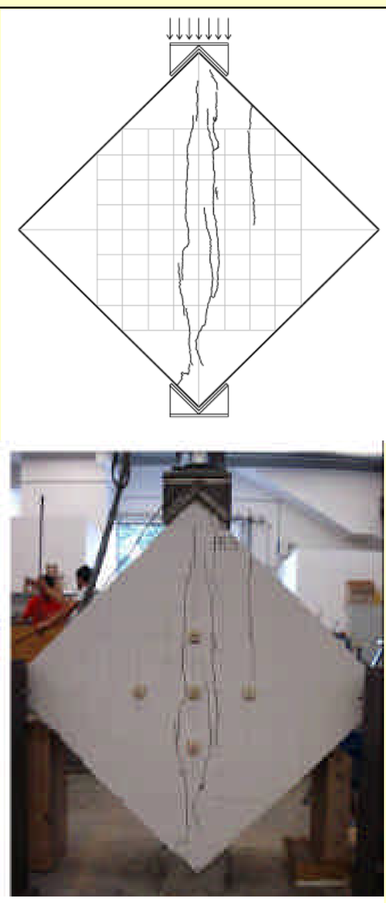
Determination and measuring the diagonal compression strength on masonry walls rendered with a fibre-reinforced mortar with very high Deformation Energy, reinforced with a steel or glass fibre net : **Systems HFE-tec® Masonries**



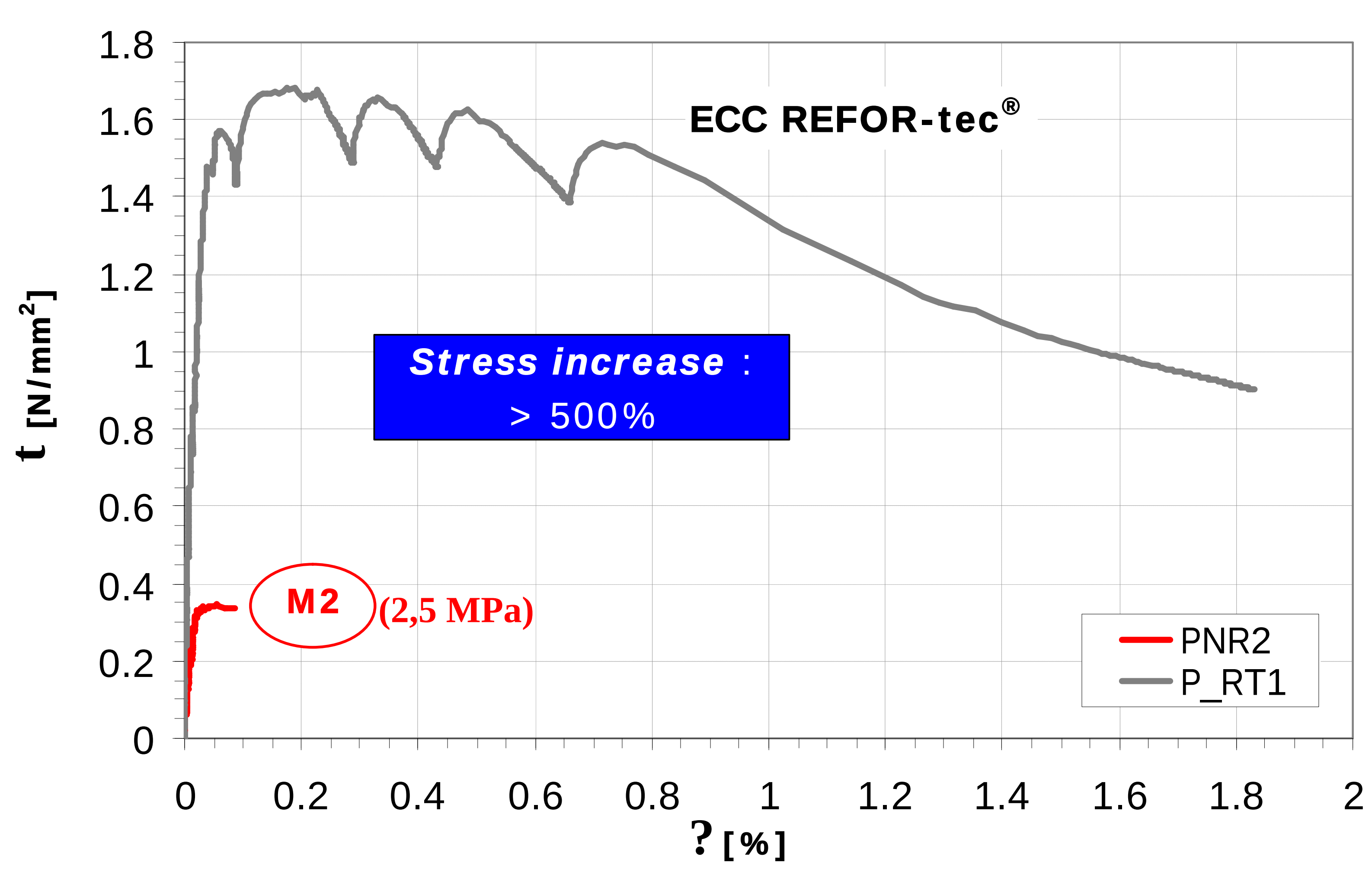


The crack of unreinforced masonry: the wall "bursts"



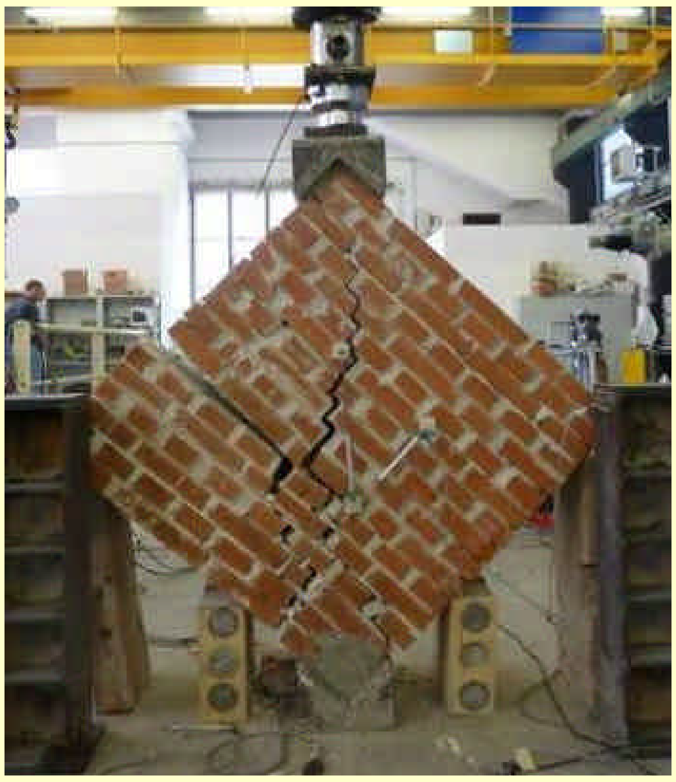


The crack of reinforced masonry **HFE-tec® : not damaged**



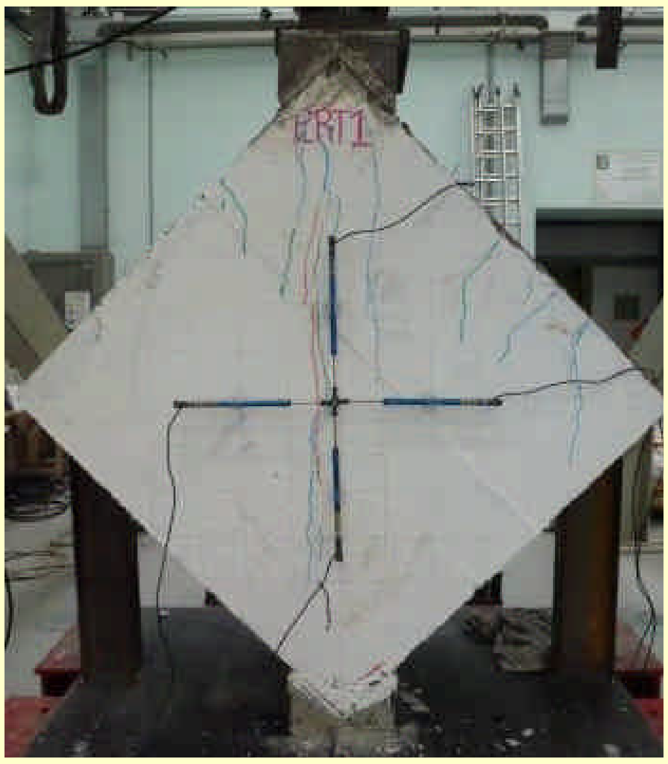
PNR2

Panel built with mortar M2 (2,5 MPa)
not reinforced



P_RT1

Panel built with mortar M2 (2,5 MPa)
reinforced with
3 cm ECC REFOR-tec® - NO MESH -
passing steel connector Æ 8 mm

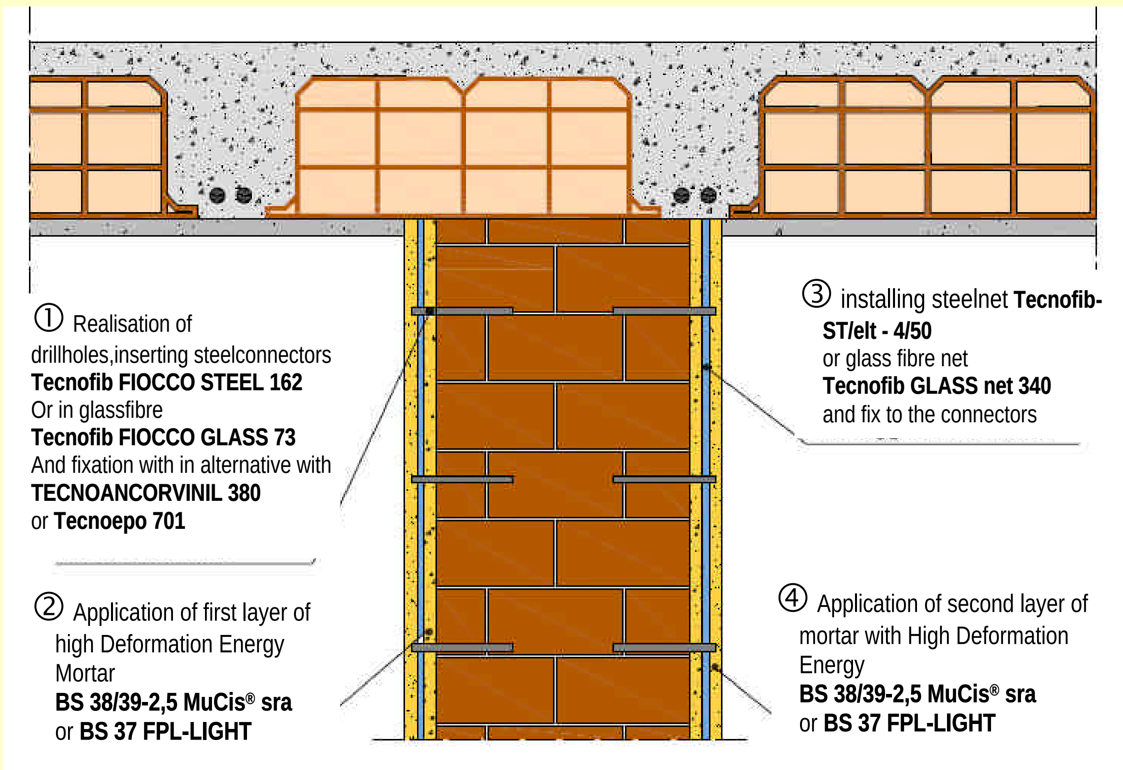


APPLICATIVE TYPOLOGY

SYSTEMS

of reinforced renders with the technology of the "structural sandwich"

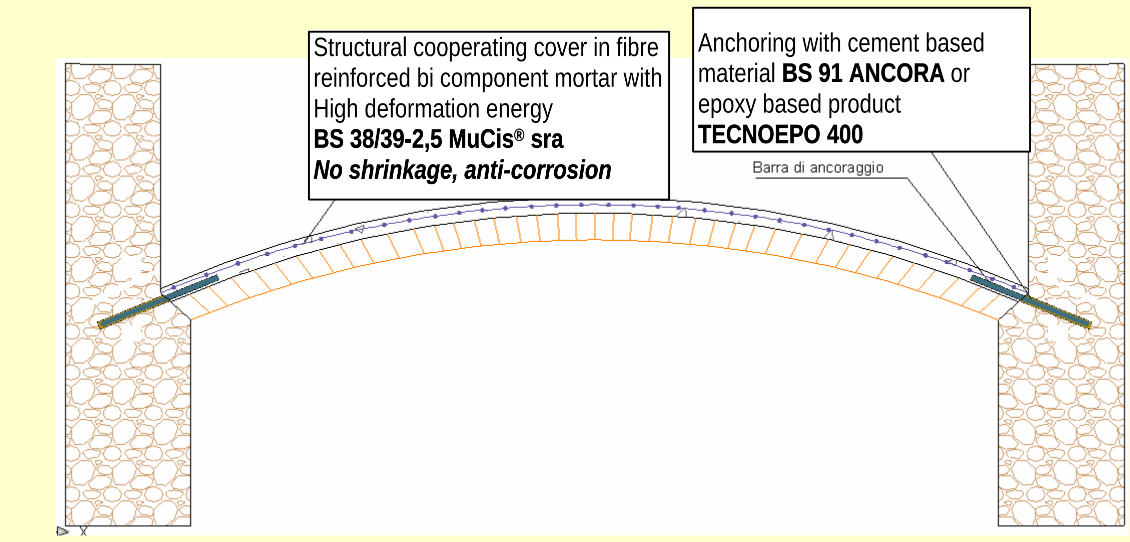
HIGH DUCTILITY



SYSTEMS

with external cover on vaulted structure

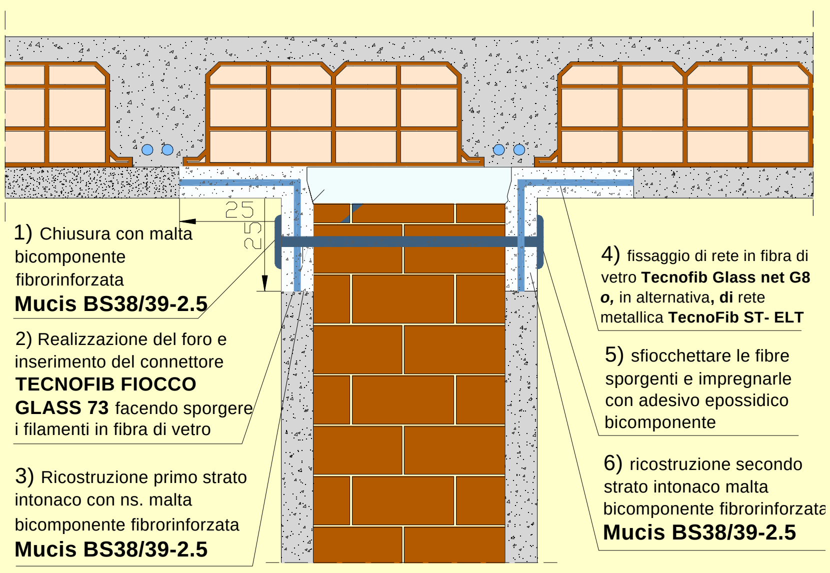
HIGH DUCTILITY



SYSTEMS

Anti-tilt system for walls and partitions (in this case by reducing the thickness to no more than 10 ÷ 20 mm)

HIGH DUCTILITY

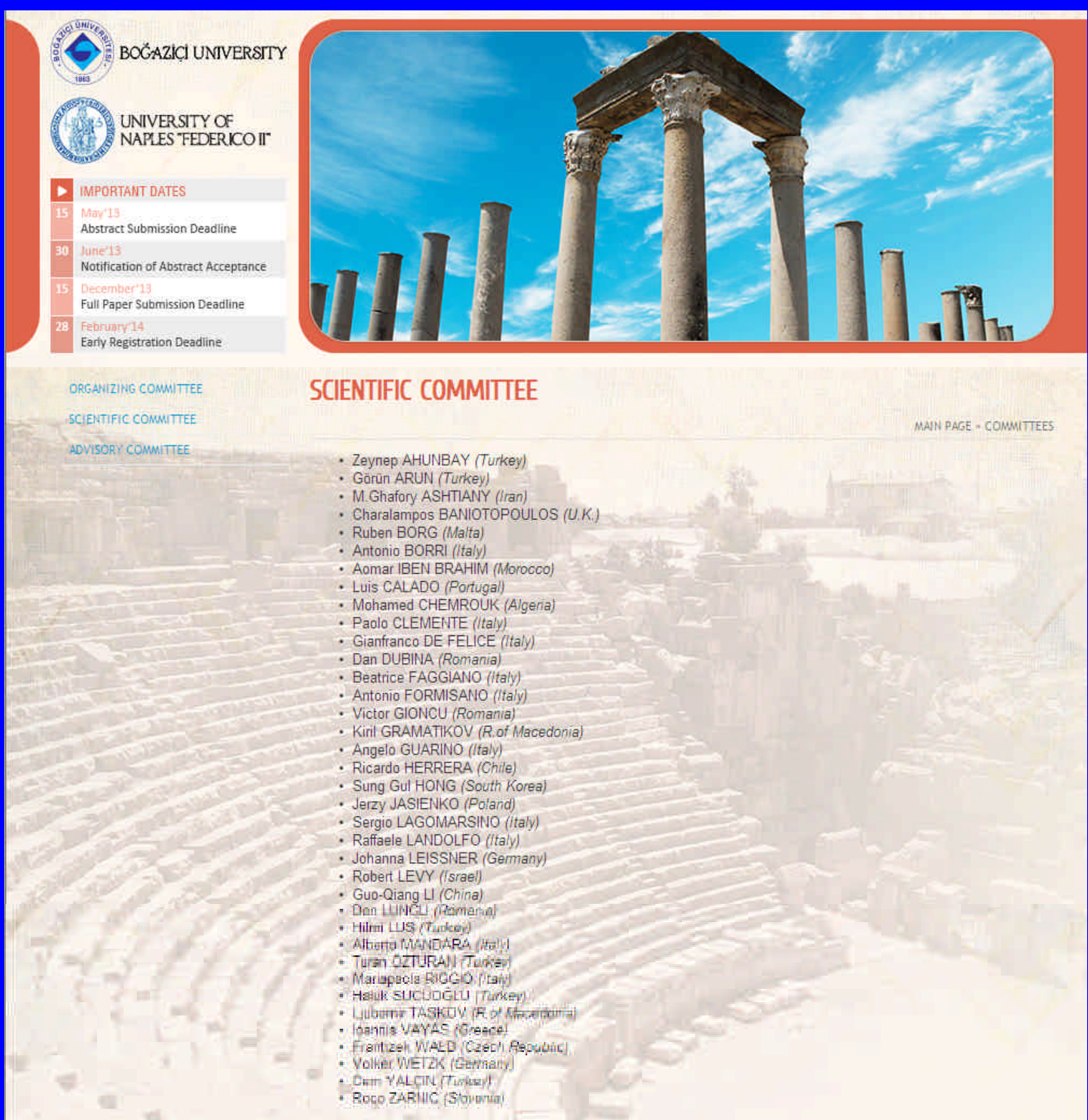


Paper which will be showed to the **PROHITECH '14**

"2nd International Conference on Protection of Historical Constructions"

7th-9th May 2014, Antalya - Turkey

High Fracture Energy Technology and Engineered Cementitious Composites for the Ductile Reinforcement of Historical Structures



PROHITECH'14

2nd INTERNATIONAL CONFERENCE on PROTECTION of HISTORICAL CONSTRUCTIONS
7 - 9 MAY 2014, ANTALYA - TURKEY

High Fracture Energy Technology and Engineered Cementitious Composites for the Ductile Reinforcement of Historical Structures

Dario Rosignoli [1], Francesco Rosignoli[2], Roland Vaes[3]

[1]TECNOCHEM ITALIANA SpA, Barzana Bergamo, Italy, dario.rosignoli@tecnochem.it
[2]TECNOCHEM ITALIANA SpA, Barzana Bergamo, Italy, francesco.rosignoli@tecnochem.it
[3]TECNOCHEM ITALIANA SpA, Houthalen, Belgium, rolandvaes@tecnochem.eu

In order to be properly and better understood putting in evidence REFOR-tec® performances :

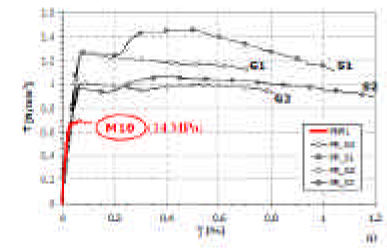
Table 2. Mechanical values of mortars.

Mortar type	Compression strength [N/mm²]	Flexural strength [N/mm²]
M10	14	4
M2	2.5	1
BS38/39 2.5	40	10
BS37 FPL-LIGHT	20	8
HFE/ECC	130	32

Mortar Type used between bricks	Compression strength [N/mm²]	Flexural strength [N/mm²]
M10	14	4
M2	2.5	1

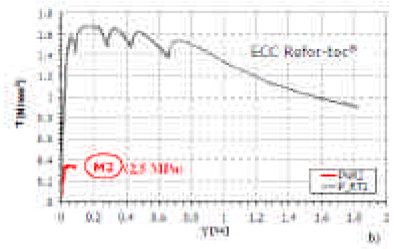
Reinforcing Mortar Type	Compression strength [N/mm²]	Flexural strength [N/mm²]	Mortar Type used between bricks
BS38/39 2.5	40	10	M10
BS37 FPL-LIGHT	20	8	M10
HFE/ECC (REFOR-tec®)	130	32	M2

Figure 3. Shear stress strain relationship for the panels tested: a) Panel built with mortar M10 / UNI-EN 998-2



Stress increase :
from 29% min.
to 115% max.

Figure 3. Shear stress strain relationship for the panels tested: b) Panel built with mortar M2 / UNI-EN 998-2



Stress increase :
> 500%

Table 1. Reinforcement configuration for the seven panels tested.

	Type mortar	Type mortar strengthened layer	Type connector	Type mesh
PNR1	M10/UNI-EN 998-2	No layer	No connector	No mesh
PR_G 1	M10/UNI-EN 998-2	BS 38/39 2.5	Fiber glass connectors - Ø 10 mm	Fiber glass mesh - 10x8cm
PR_S 1	M10/UNI-EN 998-2	BS 38/39 2.5	Steel connectors - Ø 6 mm	Steel mesh – 50x50 mm Wire diameter 4 mm
PR_G 2	M10/UNI-EN 998-2	BS 37 FPL-LIGTH	Steel connectors - Ø 6 mm	Fiber glass mesh - 10x8cm
PR_S 2	M10/UNI-EN 998-2	BS 37 FPL-LIGTH	Steel connectors - Ø 6 mm	Steel mesh – 50x50 mm Wire diameter Ø 4 mm
PNR2	M2/UNI-EN 998-2	No layer	No connector	No mesh
P_RT1	M2/UNI-EN 998-2	High Fracture Energy/ ECC Refor-tec	Steel connectors - Ø 8 mm	No mesh

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ABSTRACT: Cracks are an inevitable phenomena of cement based materials such as mortars and concretes. Even though they will never be fully preventable there are possibilities to control their formation and characteristics such as their total number as well as their depth and width in order to exclude negative effects on a material's or structure's durability and integrity.
Among others cracks can be the reason for serious deterioration processes within concrete structures. Crack width control becomes important. In other cases the more important point is to find a way how the load bearing capacity of a concrete element within a building structure can be maintained even though cracks cannot be prevented. Earthquakes, for example, lead to deformations that inevitably cause cracks within stiff concrete structures. Only if there are elements that consume a certain amount of this deformation energy, for example by the formation of cracks without collapsing, the structural integrity of buildings can be assured.
The High Fracture Energy Technology allows to control micro cracks formation in cement based materials by increasing the materials overall ductility which is referred to as fracture energy in materials engineering.
Formulations based on ECC – Engineered Cementitious Composites can take tensile loading, are able to control cracks in a narrow range, exhibit high fracture energy and large tensile deformation and ductility, open a wide range of applications.
A targeted adjustment for ECC of cement based material requires a three way approach : the cement & binders based matrix and the high elasticity modulus polymer fibers as well as their interfaced need to feature coordinated properties whose interaction produce the intended performances. High Fracture Energy Technology and Engineered Cementitious Composites enable Engineers & Architects to wide range of applications in historical structures also and particularly in seismic areas for ductile structural reinforcement. Wide range of HFE / ECC formulations have been formulated and applied. Case studies are briefly described.

1 INTRODUCTION

Traditionally a large percentage of the Italian building stock is made of masonry, with walls often made of hollow core clay bricks. These buildings are usually designed only for gravity loads, with no or little concern for seismic actions. Accordingly, they are extremely vulnerable to seismic actions, as shown by the recent earthquakes of L'Aquila (2009) and Emilia (2012).
After the new seismic classification of the Italian territory, a large number of these buildings will need seismic retrofit works in order to be able to meet the new code requirements. Hence, seismic strengthening techniques for masonry buildings are rapidly gaining interest.
Seismic performance of existing masonry buildings is affected by various failure dealing with either out of plane (bending) and in plane (shear) behavior of walls. The present paper will focus on the shear failure mechanisms.
Seismic in plane behavior of masonry walls can be experimentally simulated by two kinds of tests. On one hand it can be reproduced by the so called "diagonal compression test", ruled by ASTM 519, and, on the other hand, it can be simulated by "shear compression test". Besides other findings gained by the above research, it can be observed that strength values obtained by diagonal compression tests are generally more conservative than those given by shear compression tests. Both tests methods pointed out the general lack in shear strength of those masonry walls. Consequently, masonry structures are generally in need for strengthening in shear and various technique can be adopted with that aim.
Several strengthening techniques have been used for this purpose such as the use of grout injection, deep re-sealing of mortar joint and the use of composite materials based on carbon or glass fibers. One of the latest technique for shear strengthening of the masonry walls consist of using composite material fiber reinforced polymers (FRP). This reinforcement technique provides a series of advantages, such as the negligible influence of the self weight of the reinforcement on the total mass of the structure and the ease of installation. However, this type of reinforcement has several limitations as the relatively high costs and low fire resistance due to the use of epoxy resins for glueing the fibers to the surface of the walls.
The present paper reports the main results obtained by an experimental campaign carried out at the laboratory of the university of Bergamo on brick masonry panels. In particular, two unreinforced masonry panel and five strengthened panels have been tested under diagonal compression with the main aim of quantifying their shear strength.
Four specimen were reinforced by using an innovative strengthening system based on the combined use of a steel or glass fiber grid embedded in a base mortar. Such system is composed by two layers applied on both sides of the panels and connected by through joints made of steel bars or glass fiber wires.
A specimen has been reinforced by the application, on both faces of the masonry panel, of a 30 mm thickness layer of high fracture energy fiber reinforced ECC microconcrete.
The tests results show that the strengthening system present significant benefits in terms of increasing the shear strength and ductility with considerable advantages in the case of a seismic event.

2 EXPERIMENTAL PROGRAM

The experimental program consists of a diagonal compression tests on a total of seven brick masonry panels with dimensions of 100x100 cm and thickness equal to 40 cm. Each panel was made of sixteen parallel rows of solid 22.5x10x5 cm bricks.
Two panel (PNR1 and PNR2), used as a reference specimens, have not been strengthened.
Four specimens were reinforced by using an innovative strengthening system based on the combined use of a steel or glass fiber grid embedded in a base mortar. Such system is composed by two layers applied on both sides of the panels and connected by through joints made of steel bars or glass fiber wires. Two strengthening panels were reinforced with a cement mortar (BS38/39 2.5) and the other two panels with a cement mortar with a lower compression strength (BS37 FPL-LIGHT). The different reinforcement configurations, with different combination of grid type, mortar type and connectors type, are shown in table 1.
The procedure for the application of the strengthening technique can be summarized in the following phases: [1] Execution of five through hole with a diameter of 30 mm for the insertion of the connectors. [2] Insertion of the connectors (steel bars or glass fiber wires) and subsequently injection of epoxy resin (Tecnoepo 400) into the holes to ensure the anchoring of the connectors. [3] Application of a layer of cement rough coat. [4] Application of the first hand of mortar with a thickness of 15 mm. [5] Positioning of the mesh on both faces of the panels and anchoring to the connectors. [6] Application of the second hand of mortar with a thickness of 15 mm. Five connectors for square meter of panel were placed. The thickness of the strengthening layer for all four reinforced panels is equal to 30 mm for each side for a total thickness of the specimen of 46 cm. Figure 1 shows the different phases for the application of the strengthening layer of the panel PR_G1. The mechanical properties of the mortars, which were used for the construction of the panels and of the strengthening layers, were derived from

Table 1. Reinforcement configuration for the seven panel tested.

	Type mortar	Type mortar strengthened layer	Type connector	Type mesh
PNR1	M10/UNI-EN 998-2	No layer	No connector	No mesh
PR_G 1	M10/UNI-EN 998-2	BS 38/39 2.5	Fiber glass connectors - Ø 10 mm	Fiber glass mesh - 10x8cm
PR_S 1	M10/UNI-EN 998-2	BS 38/39 2.5	Steel connectors - Ø 6 mm	Steel mesh – 50x50 mm Wire diameter 4 mm
PR_G 2	M10/UNI-EN 998-2	BS 37 FPL-LIGTH	Steel connectors - Ø 6 mm	Fiber glass mesh - 10x8cm
PR_S 2	M10/UNI-EN 998-2	BS 37 FPL-LIGTH	Steel connectors - Ø 6 mm	Steel mesh – 50x50 mm Wire diameter Ø 4 mm
PNR2	M2/UNI-EN 998-2	No layer	No connector	No mesh
P_RT1	M2/UNI-EN 998-2	High Fracture Energy/ ECC Refor-tec	Steel connectors - Ø 8 mm	No mesh

bending and compression tests (according to UNI EN 998-2; 2004): 40mm x 40 mm x 160 mm mortar prisms were tested in flexure with three point bending tests and 8 cubes, obtained from failed mortar specimen in flexure, were subjected to the compression test. The results of the tests for the three types of mortar used are reported in table 2.

Table 2. Mechanicals values of mortars.

Mortar type	Compression strength [N/mm²]	Flexural strength [N/mm²]
M10	14	4
M2	2.5	1
BS38/39 2.5	40	10
BS37 FPL LIGHT	20	8
HFE/ECC	130	32

The mechanical properties of steel and fiber glass grids were provided by the manufacturer. For the glass grid the tensile strength was 6600 N/cm and the ultimate tensile strain is 3.5%, for the steel mesh the tensile strength of a single wire is 550 N/mm² and the ultimate tensile strain about 10%.

The last panel (P_RT1) has been reinforced by the application of a layer, 30 mm thickness, of high fracture energy fiber reinforced ECC microconcrete. Five through connectors, made with steel bars with diameter of 8 mm, were placed. The procedure for the application of the strengthening technique can be summarized in the following phases: [1] Execution of five through hole with a diameter of 40 mm for the insertion of the connectors; [2] Hydro cleaning of the surface of the masonry to ensure the maximum bond between the substrate and HFE/ECC; [3] Insertion of the connectors; [4] Realization of the moulds; [5] Saturation of the surface of the masonry to allow the maximum adhesion of the high performance HFE/ECC microconcrete; [6] Casting of the self levelling high performance HFE/ECC microconcrete. The free flowing property of the microconcrete is penetrating with complete filling up of the 40 mm hole containing the steel bar. Figure 2 shows the different phases for the application of the strengthening layer of the panel P_RT1.

The HFE/ECC, used for the reinforcement, presents an almost self levelling rheology, that should be cast in moulds, a compressive strength of 130 MPa and a tensile strength of 6 MPa. Direct tensile test on dog-bone specimens and four point bending tests on small beams were performed in order to characterize the material in tension and the results show the strain hardening behavior in tension of the material. The strengthening material is reinforced with straight steel and polymer fibers.

Figure 1. Application of the strengthening layer on the panel PR_G1: a) Insertion of the glass fiber wires; b) Positioning of the mesh; c) Anchoring of the connectors

Figure 2. Application of the strengthening layer on the panel P_RT1: a) Execution of five through hole; b) Hydro cleaning of the surface; c) Realization of the moulds e casting

3 TEST SET UP

The diagonal compression load is applied to the corners of the panels by adopting a steel reacting frame. The load was applied by means of an electromechanical jack having a loading capacity of 1000 kN with a close loop control system. The tests were conducted under displacement control, in order to record the panels post-peak response, with a constant speed equal to 0.01 mm/sec. The compression load is applied to the masonry through two steel shoes placed in correspondence of two opposite corners of the panels. The test layout follows the requirements of ASTM E 519-81, although some change has been introduced, as the different size of the panels to be tested and of the loading shoes, in order to properly account for the size of the type of masonry to be tested. Between the steel shoes and the specimen has been realized a fast setting shrinkage free mortar bed for a better distribution of the load and in order to avoid a brittle failure of the panels edges.

Potentiometric and LVDT transducer were used for monitoring the in-plane and out-of-plane displacement. Two potentiometric transducers were placed on each side of the panels along the two diagonals to record the vertical and horizontal displacement and therefore strains. These transducers had a measurement length of 400 mm. This was based on experimental observations from similar experiments, where it was found that shear cracks developed in the central area of the panels. Two LVDT were installed perpendicularly to the panel surface to measure out-of-plane displacements. Before setting the instruments, the panels were whitewashed in order to record the crack pattern by means of a high-resolution camera.

4 EXPERIMENTAL RESULTS

Figure 3 show the shear stress – shear strain curves for the seven panels tested.

Shear strength τ, reported in figure 3, for the various panel tested, can be obtained on the basis of the current experimental load P, according to ASTM E 519-81, with the following conventional formula:

$$\tau = 0.707 \cdot \frac{P}{A_n} \tag{1}$$

where A_n is the net section area of the un-cracked section of the panels.

The average strains, ε_c and ε_t, can be calculated on the basis of the average displacements on the two sides of the panels:

$$\varepsilon_c = \frac{\Delta V}{g} \qquad \varepsilon_t = \frac{\Delta H}{g} \tag{2}$$

where ΔV and ΔH are the vertical shortening and horizontal extension along the compressive and tensile diagonal, respectively, and g is the vertical gage length (400mm).

The shear strain value, γ, which is reported in figure 3, is computed as:

$$\gamma = \varepsilon_c + \varepsilon_t \tag{3}$$

The two unreinforced specimens (PNR1 and PNR2) presented a brittle failure due to the rupture of the mortar beds along the loaded diagonal. The average failure loads, used as reference value for the comparison with the strengthened specimens results, are equal to 389 kN (panel PNR1) and 197 kN (panel PNR2). The ductility factor is about 1.6% for both panels. The PNR2 panel after the diagonal compression test is shown in figure 4. All the five strengthened panels show a considerable increase of the maximum load compared to unreinforced panel.

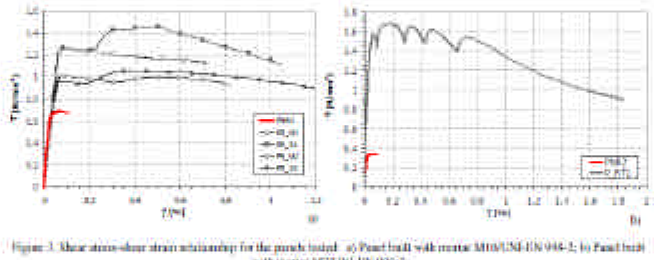

Figure 3. Shear stress-shear strain relationship for the panels tested: a) Panel built with mortar M10/UNI-EN 998-2; b) Panel built with mortar M2/UNI-EN 998-2

Figure 4. Specimens at the end of the tests

The panel P_RT1, strengthened with a HFE/ECC layers, shows the maximum increase of resistance: the peak load exhibits an increment of 4.82 times compared to that of the panel without reinforcement (PNR2). Also the two panels strengthened with a BS38/39 layers show a good increase of the maximum load with respect to the unreinforced panels (PNR1). For the specimen with a fiber glass grid (PR_G1), the maximum load exhibits an increase of 2 times compared to the peak load of the un-strengthened panels, while for the specimens with a steel grid (PR_S1) the increase is equal to 2.4 times. For the two panels strengthened with a BS37 mortar, that shows a lower compression strength, the increase of strength is smaller: the increment of maximum load is about 1.7 times for both panels compared to the panel PNR1. The presence of the strengthened layers on the both sides of the specimens has considerably increased the ductility for both types of reinforced panels. The wall with the HFE/ECC layers shows the greater ductility: the ductility factor is 26.14%, value sixteen times higher compared to the unreinforced specimen. The two walls strengthened with a steel grid (PR_S1 and PR_S2) showed high ductility too. After the onset of a first vertical crack, the load begun to increase again and several vertical cracks appeared along the compression diagonal. The tests were stopped when the load dropped below 80% of the maximum load, to avoid the collapse of the panels and damage to the instrumentation. The ductility factor is 11.60 for the wall with a BS38/39 layers and 16.61 for the panel with a BS37 mortar, values more than ten times higher than those of the non-reinforced panel. Even panels strengthened with glass fiber mesh (PR_G1 and PR_G2) showed a moderate increase in ductility. After reaching the maximum load the two panels have achieved shear strains equal to about 0.8%. The collapse occurred due to the opening of a single vertical crack which ran through bricks and mortar beds by ripping the fiber glass mesh. The ductility factor for both panels is equal to 9.6%, a value 7.5 times grater than the one shown by the panel without strengthening layers. The panels at the end of the tests are shown in Figure 4.

The two strengthened systems studied in this research display considerable increase in ductility without, however, producing significant changes in the shear stiffness of the structure. Therefore, this type of strengthening intervention does not change the static scheme of the structure neither cause redistribution of stiffness in the building.

The main results for the tests are summarized in table 3. The values of τ_{max}, γ_{max}, $\varepsilon_{c,max}$, $\varepsilon_{t,max}$ are the stress and strain values evaluated at the maximum load and the τ_u, γ_u are the stress and strain values evaluated when the load drops at the 80% of the maximum load.

Table 3. Experimental results

*For the panels showing a brittle failure, the values of τ_u and γ_u have been evaluated at the end of the tests

Evaluated using the first cracking load (γ_{80}/γ_{fc})

Specimen	P_{max} [kN]	τ_{max} [N/mm²]	$\varepsilon_{c,max}$ [%]	$\varepsilon_{t,max}$ [%]	γ_{max} [%]	τ_u [N/mm²]	γ_u [%]	G [N/mm²]	μ [%]
PNR1*	389.3	0.668	0.032	0.038	0.070	0.662	0.115	2410	1.52
PR_G1	814.8	1.252	0.050	0.024	0.074	1.122	0.707	2569	9.59
PR_S1	818.9	1.258	0.095	0.397	0.092	1.167	0.957	2893	11.60**
PR_G2	652.3	1.003	0.050	0.036	0.086	0.918	0.827	2644	9.65
PR_S2	687.7	1.057	0.111	0.266	0.377	0.894	1.194	2912	16.61**
PNR2*	197.4	0.349	0.026	0.029	0.054	0.335	0.087	2351	1.59
P_RT1	1005.2	1.663	0.065	0.124	0.189	1.317	1.023	8774	26.14**

The modulus of rigidity, G, is calculated as the secant modulus between the origin and the stress equal to 30% of the peak stress. The local panel ductility, μ, has been computed by the following equation:

$$\mu = \frac{\gamma_u}{\gamma_{max}} \tag{4}$$

where γ_{max} is the shear strain corresponding to the maximum load and γ_u is the shear strain at 80% of the maximum load (or at the end of the test for the panels that show a brittle failure).

5 CONCLUSION

Diagonal compression tests were conducted on six masonry panels to confirm the effectiveness of this seismic strengthening technique. On the basis of the experimental results the following conclusions can be drawn:

- All the strengthened panels shows a significant increase in strength due to the use of the high performance mortar. The specimen strengthened with the HFE/ECC layers exhibits an increment of 4.82 times compared to that of the unreinforced panel (PNR2).
- The strengthened panels show a significant increase of ductility. The specimen strengthened with the HFE/ECC layers exhibit the highest ductility: its ductility factor is 26.14% value sixteen times higher than the un-reinforced panel. The walls strengthened with a glass fiber mesh show the smaller increase of ductility with ductility factor of about 9.6%.
- The strengthened system studied in this research does not modify the shear stiffness of the structure; therefore it does not change the static scheme of the structure neither cause redistribution of stiffness in the buildings.
- The crack pattern demonstrates a very good adhesion between bricks masonry and reinforced mortar or HFE/ECC microconcrete.

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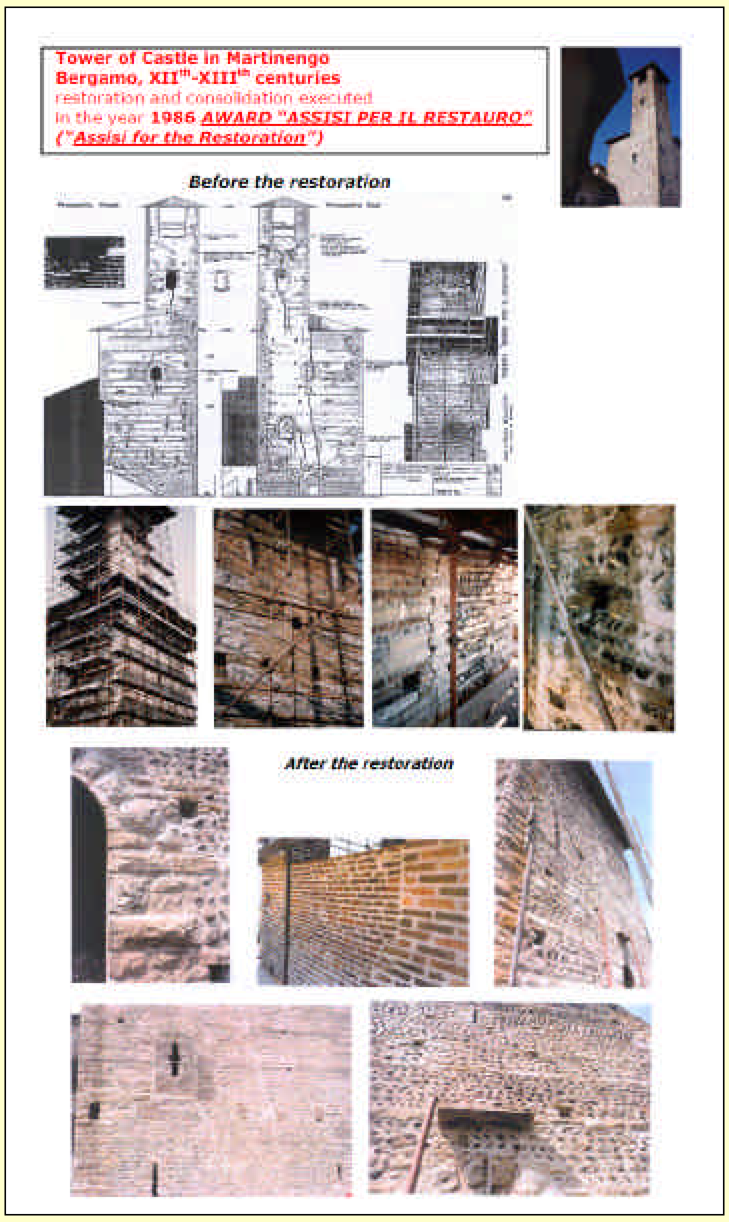

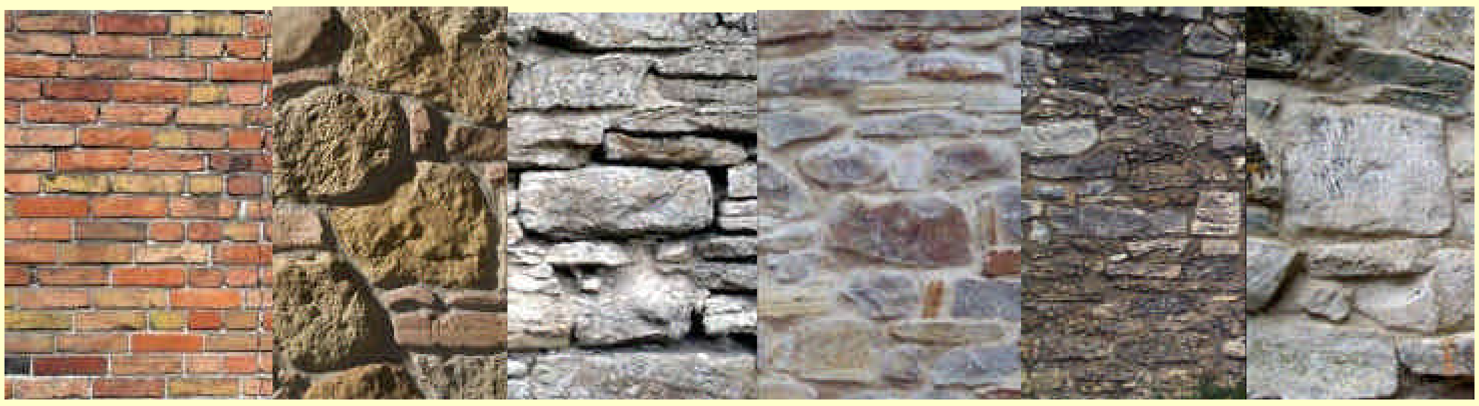

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