Meccanica delle Strutture in Muratura Rinforzate con FRP-materials: Modellazione, Sperimentazione, Progetto, Controllo.
***Mechanics of Masonry Structures Strengthened with FRP-materials: Modeling, Testing, Design, Control**, 6-8 dicembre 2004, Venezia, Italia*

RINFORZO DI EDIFICI STORICI IN MURATURA UTILIZZANDO MATERIALI COMPOSITI E POLIUREA

PRESERVATION OF HISTORICAL MASONRY BUILDINGS USING GFRP COMBINED WITH SPRAYED POLYUREA

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SOMMARIO

L'articolo presenta i risultati sperimentali di tre prove eseguite su due pannelli murari in scala reale realizzati con blocchi di tufo e sottoposti a carichi fuori dal piano per simulare forze di natura sismica. I pannelli di prova sono stati rinforzati utilizzando una tecnica innovativa che abbina la resistenza delle reti in composito in fibra di vetro al basso tempo di polimerizzazione della poliurea utilizzata come adesivo.

Le prove confermano che il sistema costituito dalle reti in fibra di vetro annegate nella poliurea costituisce un efficace rinforzo dei pannelli murari. La discussione presentata nell'articolo evidenzia come la capacità portante e la duttilità degli elementi rinforzati risultino migliorate rispetto al pannello originale, non rinforzato.

ABSTRACT

This paper presents the results of three experimental tests performed on two real-scale tuff masonry walls to provide insights about the behavior of strengthened and un-strengthened masonry panels subjected to out-of-plane loads occurring during seismic events. The test masonry panels were strengthened using an innovative technique that combines the strength of GFRP grids to the low polymerization time of the polyurea resin as the adhesive.

Tests confirmed that the system consisting of GFRP grids embedded in polyurea resin is effective for the strengthening of tuff masonry walls. The discussion of test results presented in the paper highlights that both ultimate capacity and member ductility have been improved with respect to the original, un-strengthened, wall.

INTRODUCTION

In this paper results of three experimental tests performed on tuff masonry walls belonging to the ex Convento San Lorenzo located in Salerno (Italy) are presented. The entire building of the ex Convento San Lorenzo is facing a series of restoration/remedial works to become the new public library of the city of Salerno. The contractor agreed to donate to the Department of Structural Analysis and Design of the University of Naples Federico II, two panels made out of tuff masonry to perform out-of-plane flexural tests. These tests will help in understanding the behavior of strengthened and un-strengthened masonry panels subjected to out-of-plane loads occurring during seismic events.

The goal of the experimental campaign is broader and it is related to the safety of historic masonry walls. The test masonry panels were strengthened using an innovative technique that combines the strength of GFRP (Glass Fiber Reinforced Polymer) grids to the low polymerization time of the polyurea resin [1] as the adhesive.

TEST DESCRIPTION

The two tested masonry panels were obtained by saw-cutting along vertical lines the wall chosen for the experimental campaign (Figure 1). Their thickness is equal to 0.45 m while the total height is 3.30 m. The wall surface is scarified to show the masonry units and allow a better bond between the masonry and the applied composite material.

Three four-point bending tests were performed as listed: Test #1 on the virgin panel A (Figure 1) without any type of strengthening; Test #2 on the same panel of Test #1 after it was damaged and repaired with one layer of GFRP grid; Test #3 on panel B strengthened with 3 layers of GFRP grid.

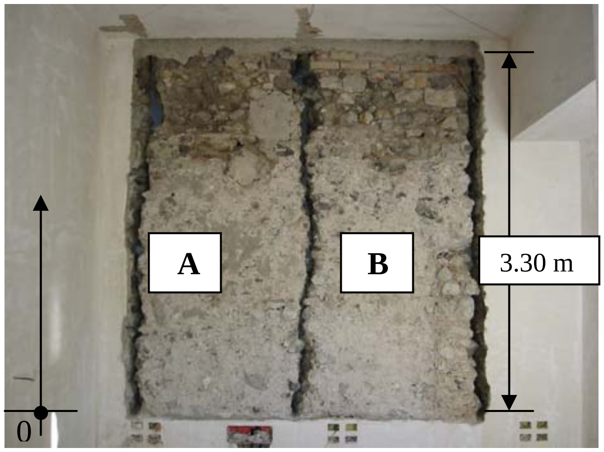

Figure 1 - Tuff Masonry Panels A and B before the Tests

The installation of the external GFRP reinforcement was easy and simplified with respect to the wet lay-up technique normally used to apply traditional FRP. The main advantages of the proposed technique are related to the fact that the primer is not needed and the impregnation of the reinforcement is performed by spraying the Polyurea with no need for touching the surface (Figure 2).

Figure 2 - Pictures of the Strengthening Phase

Before applying the reinforcing material, a series of connections were realized between the panels at the top and bottom regions of the masonry walls by using GFRP reinforcing bars. These bars (Figure 3) were installed to avoid a premature shear failure that would have originated a rigid translation of the panel with respect to the upper and lower boundaries.

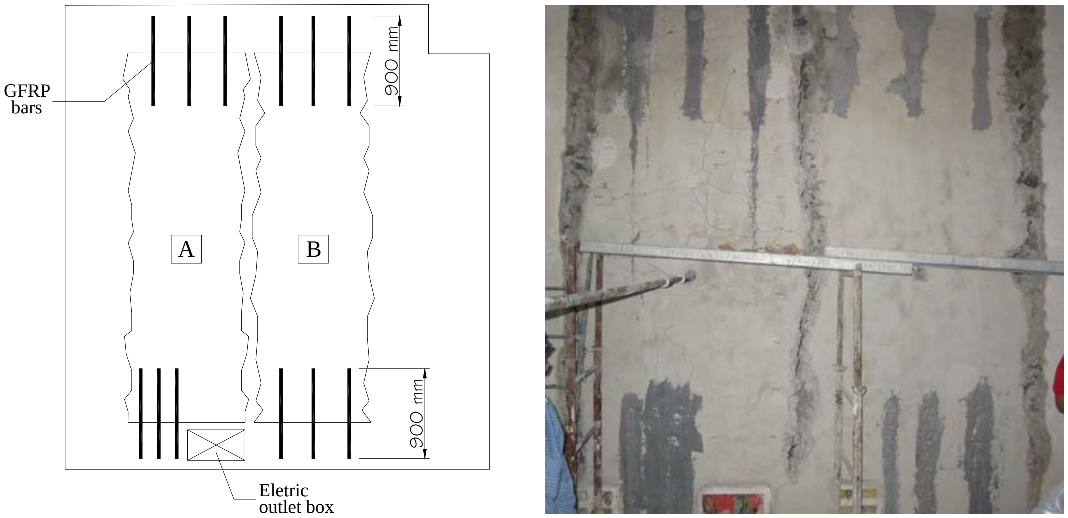

Figure 3 - Front View of the Panels with the GFRP Bars to Avoid Shear Failures

GFRP AND MASONRY MECHANICAL PROPERTIES

Material characterization was conducted at the job site to obtain the properties of the tuff masonry panel, and in the laboratory where tensile tests were performed on the GFRP reinforcing grid.

The tuff masonry specimen to be tested in compression was obtain with two horizontal saw cuts at a distance of 400 mm on a portion of an undisturbed wall where the plaster was previously removed. Upon completion of the saw cuts, two semicircular flat jacks were inserted inside the masonry wall and the test commenced according to [2]. The masonry compressive strength resulted equal to f'_m=4.32 MPa.

The GFRP reinforcing grid to be tested was obtained by cutting pre-dimensioned strips following the main direction of the fibers of the laminate. For each laminate, four specimens were obtained. For the determination of the tensile properties, 12 specimens obtained from three different laminates were tested. The fabrication of the three laminates was done at the job site following the same procedures used in the application of the external GFRP reinforcing grid to the two tuff masonry panels. Mechanical properties of GFRP reinforcement were determined according to the procedures highlighted in [3]. The final results are summarized in Table 1.

Table 1 - Guaranteed Tensile Properties

Guaranteed Tensile Strength f^*_{fu} *(MPa)*	*Guaranteed Tensile Strain* ε^*_{fu} *(-)*	*Tensile Modulus of Elasticity* $E_f=f^*_{fu}/\varepsilon^*_{fu}$ *(MPa)*
305.4	0.0065	46985

More detail related to the material characterization is reported in [4].

TEST SETUP

All three tests were performed using the reaction frame shown in Figure 4 and Figure 5. In the same figures are also reported the directions from which the pictures of Figure 6 were taken. Figure 7 shows a vertical section of the test setup.

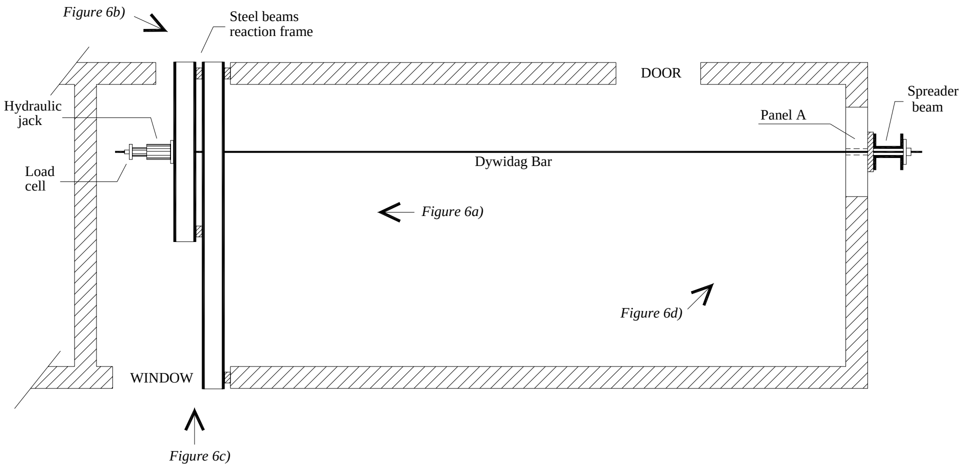

Figure 4 - Top View of the Test Setup for Panel A (Not to Scale)

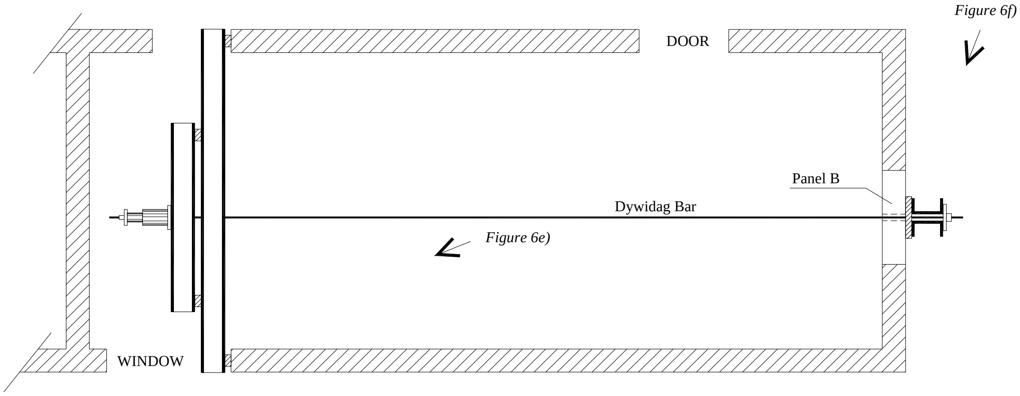

Figure 5 - Top View of the Test Setup for Panel B (Not to Scale)

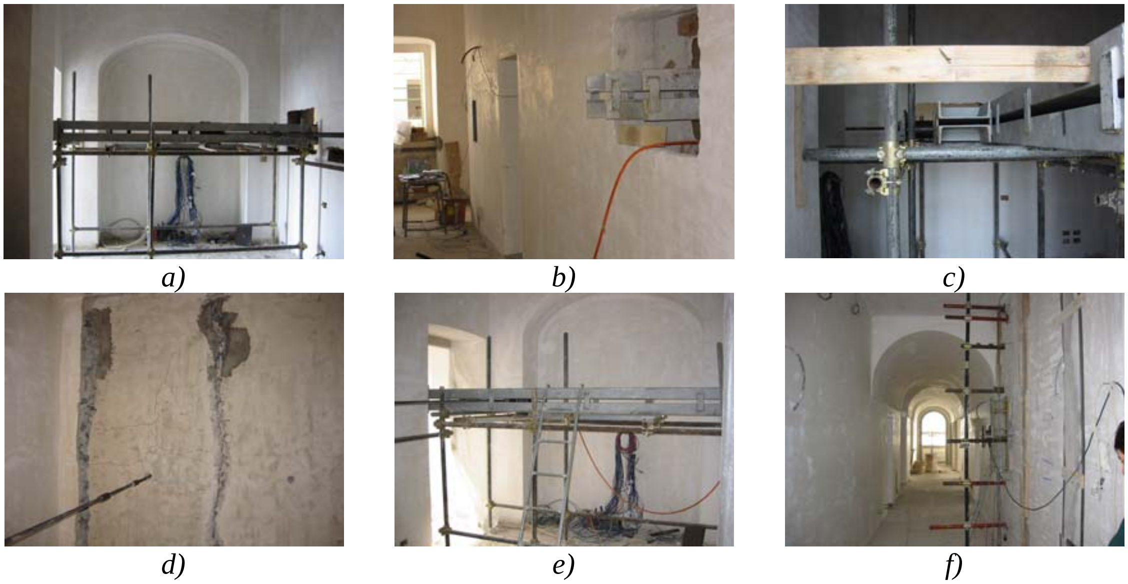

Figure 6 - Pictures of the Test Setup

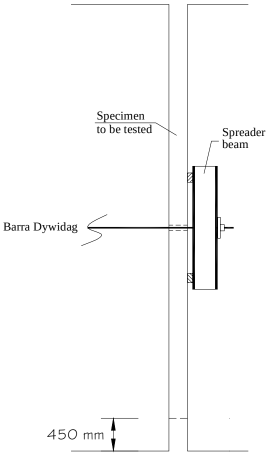

Figure 7 - Test Setup Vertical Section (Not to Scale)

The load was measured using a load cell positioned on the hydraulic jack. Displacement readings were recorded using LVDTs (Figure 8) placed on the compression face of the panel as reported in Table 2.

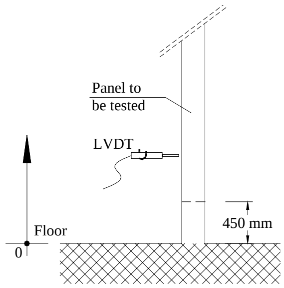

Figure 8 - LVDTs Used to Record Deflections

Table 2 - LVDTs Locations

Height† (mm)	*Test #1*	*Test #2*	*Test #3*
600	•	•	•
900	x	•	x
1200	x	x	•
1750	•	•	x
1800	x	x	•
2400	x	x	•
2700	•	•	x
3000	x	x	•
3250	x	x	•
3550	•	•	•

† Height is measured starting from the floor (Figure 8)
Note: • = LVDT present; x = LVDT not present

TEST RESULTS

Test #1 was performed on Panel A without any reinforcement. Location of the four LVDTs listed in Table 2 is also reported in Figure 9 where the envelope curves obtained by plotting applied load vs. horizontal displacement of the panel are shown. As expected, the two LVDTs closer to mid-span of the panel recorded the maximum displacement approximately equal to 15 mm. The non-symmetrical behavior of the panel is evident as a larger deflection was recorded on the bottom half.

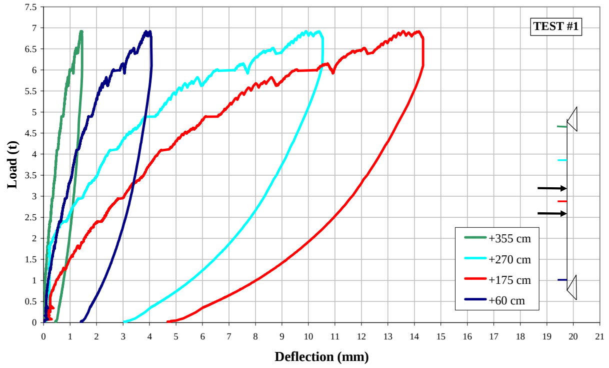

Figure 9 - Load Deflection Diagram (Test #1)

The test was performed by applying the load in cycles up to predetermined levels, in order to avoid any anomalies in the behavior of the tested panel. An example of such loading-unloading procedure is reported in Figure 10 relatively to the displacement at mid-span. The test was terminated when the deflection displayed a flat pattern with no further increase of the applied load. At this point we were approaching the member's ultimate capacity; however, it is unclear whether the rupture was initiated by failure of the masonry or by shear failure of the top and bottom portions of the panel. A theoretical analysis shows that the nominal flexural capacity of the panel when the rupture is initiated by failure of the masonry would have been equal to M_n=61 kNm, assuming a tensile strength of the masonry equal to 1/3 of its compression strength, f'_m. The experimental moment recorded for this test was equal to M_{exp}=64 kNm. Therefore, it is reasonable to assume that the member's rupture was due to failure of the masonry. A masonry failure for this type of structure is the result of the arching effect. At this point, it was clear that the addition of external reinforcement would not necessarily increase the capacity of the panel, but could have important effects on the panel stiffness and stability.

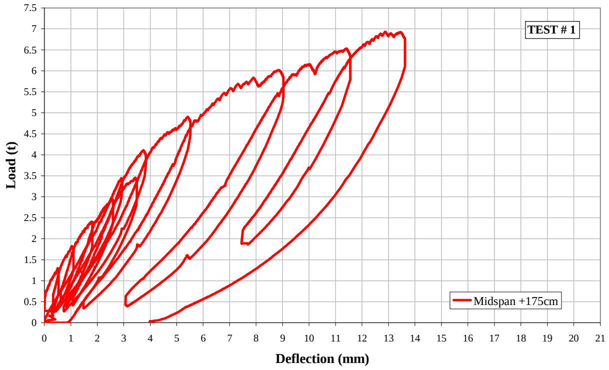

Figure 10 - Loading-Unloading Cycles (Test #1)

Figure 11 reports the deformed shape of the panel, for different load steps, with reference to the room floor level rather than the stating point of the tested panel. As the load increases, the displacement recorded increases proportionally; the maximum value is recorded at mid-span of the panel. This demonstrates that the predominant deformation is caused by flexural stresses, although shear deformations may not be disregarded as it is shown by the horizontal deformation recorded at 0.6 m elevation.

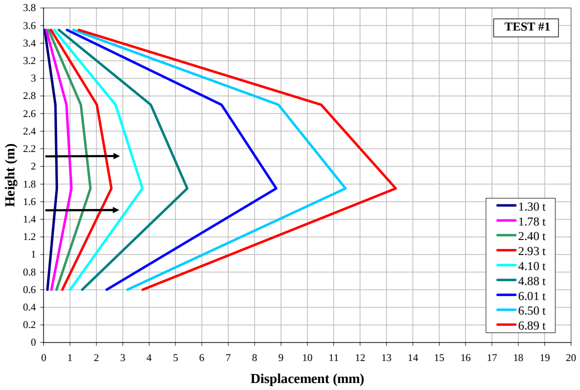

Figure 11 - Horizontal Displacement as a Function of the Wall Height (Test #1)

The second test (Test #2) was conducted on Panel A once the first test was terminated and the panel had been strengthened with one layer of GFRP grid 80 cm wide over the entire height. The same instrumentation was used to monitor also this panel with the addition of another LVDT placed at one fourth of the height of the panel as reported in Table 2 and in Figure 12 showing the load-deflection envelopes. The results obtained from this second test show the attainment of the same maximum capacity of the previous test with a maximum recorded displacement of about 20 mm. Also this test was conducted by applying the load in cycles.

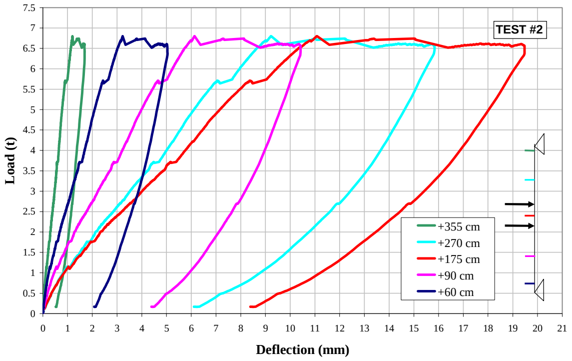

Figure 12 - Load Deflection Diagram (Test #2)

Figure 13 shows the deformed shape of the panel at different heights starting from the floor level, for increasing applied loads. The dashed line represents the final deformation registered after Test #2 considering that the panel presented a residual permanent deflection. The residual displacements measured at the end of Test #1 at heights of 600, 1750, 2700 and 3550 mm are equal to 1.2, 4.0, 3.3 and 0.4 mm, respectively, as indicated in Figure 11.

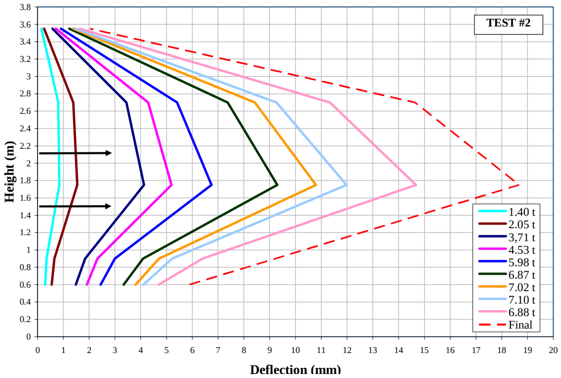

Figure 13 - Horizontal Displacement as a Function of the wall Height (Test #2)

The third and last test (Test #3) was performed on Panel B strengthened with three layers of GFRP grid 80 cm wide and extended for the entire height of the panel. Two more LVDTs were added to monitor the panel deflection at the locations reported in Table 2 and Figure 14 showing the load-deflection envelopes. The maximum recorded displacement was 14 mm, measured at 2.4 m from the floor. Figure 15 shows the deformed shape of the panel as a function of the applied load. Also in this instance, the test was terminated when the deflection displayed a flat pattern with no further increase of the applied load. A theoretical analysis indicates a nominal flexural capacity for FRP rupture failure mode equal to M_n=106.5 kNm [5], much larger than the experimental recorded moment. This behavior could lead to the conclusion that the full flexural capacity is not achievable because shear failure of the panel is initiated in the top and bottom portions of the tested specimen. It is most likely the bottom portion of the wall to show major signs of distress as depicted in Figure 15 where a displacement of almost 4 mm was recorded.

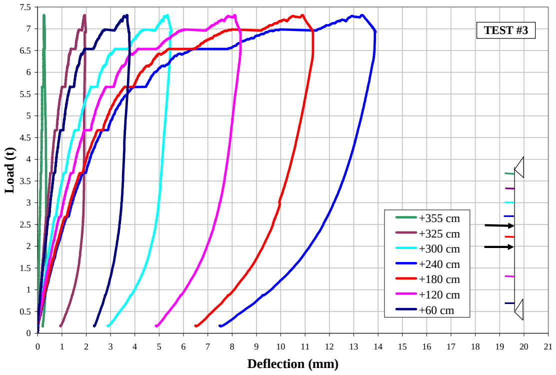

Figure 14 - Load Deflection Diagram (Test #3)

Figure 16 reports the load-displacement plots at mid span for Test #1 and #2. The increase in ductility due to the externally bonded GFRP reinforcement as shown by the second test is very evident. Such response confirms the beneficial effect of FRP strengthening even after the panel had been tested to its maximum capacity and failed. As expected, the presence of the GFRP reinforcement could not change the ultimate failure mode that was caused by either failure of the masonry or shear failure at the two ends of the panel. As a result, the ultimate load reached by the panels is almost identical for the two tests as summarized in Table 3.

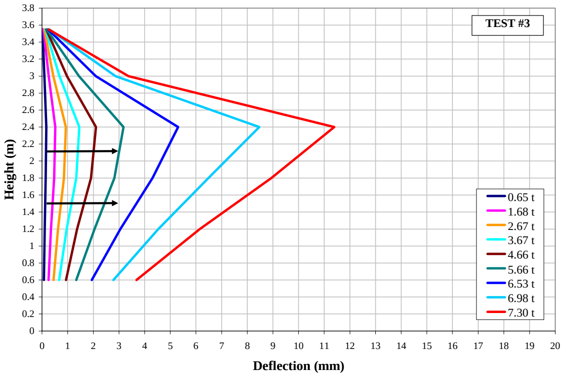

Figure 15 - Horizontal Displacement as a Function of the Wall Height (Test #2)

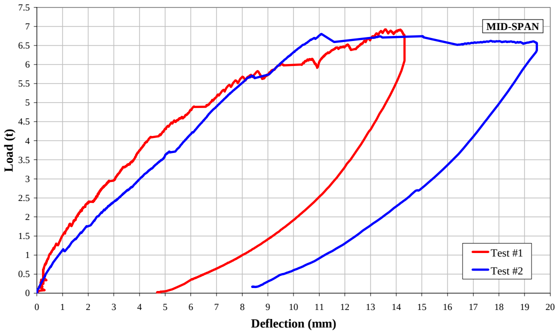

Figure 16 - Mid-span Deflection for Test #1 and #2

Table 3 – Summary of Test Results

Description	*Test #1*	*Test #2*	*Test #3*
Ultimate Load, *(kN)*	6.8	6.7	7.3
Maximum Displacement, *(mm)*	14	19	11
Residual Deflection, *(mm)*	5	9	6

Figure 17 reports the load-displacement plots at mid span for Test #1 and #3. As expected, the application of three layers of GFRP reinforcement (Test #3) results in increased stiffness in the linear-elastic cracked region of the load deflection behavior. Also, the GFRP strengthening allows for an increase of 10% of the wall capacity with respect to the virgin panel even in the presence of arching effect.

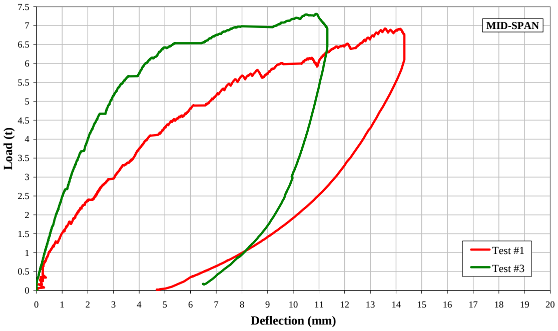

Figure 17 - Mid-span Deflection for Test #1 and #3

The maximum displacement recorded in Test #3, about 11 mm, is smaller than the one recorded for the un-strengthened panel equal to 14 mm. However, rather than the displacement, it is of interest to emphasize the way the tested panel reaches its ultimate capacity. While the un-strengthened panel reached failure with a gradual stiffness deterioration, for the specimen strengthened with FRP there is a clear change in stiffness at about 5 mm of horizontal displacement (Figure 17). It is at this point that the shear failure of the two ends starts to occur.

CONCLUSIONS

The system consisting of GFRP grids embedded in polyurea resin has proven to be effective in strengthening tuff masonry walls. Both ultimate capacity and member ductility were improved with respect to the original, un-strengthened wall, even though the geometry and compressive strength of the masonry were such that GFRP rupture was never the obtained failure mode.

The increase in ultimate capacity, obtained only in the case of three layers of GFRP reinforcement, may not be apparent as it has been in similar applications, but this is due to the controlling failure mode of the wall due mainly to arching.

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