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The use of textile-reinforced mortar as a strengthening technique for the infill walls out-of-plane behaviour

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Abstract

The primary objective of this work is to present a experimental analysis of textile-reinforced mortar based strengthening solutions to prevent the out-of-plane (OOP) collapse of masonry infills in RC buildings. The experimental campaign comprises the OOP testing of five full-scale infill walls made up of horizontal hollow clay bricks, two of them unstrengthened and the remaining three strengthened, with and without prior damage. All the tests consisted in the application of a semi-cyclic (loading-unloading-reloading) history of imposed displacements in the OOP direction by means of a uniformly distributed load. The mechanical properties of the adopted materials are characterized and presented. The results will be presented in terms of OOP force-displacement responses, deformed shapes, damage evolution, energy dissipation capacity and damping. In the end, the tests' results are compared to each other to assess the effectiveness of the selected strengthening techniques and to provide a support towards the choice of the best strategies for future further investigations and applications

Keywords: *masonry infill walls, strengthening, textile-reinforced mortar, out-of-plane behaviour, experimental behaviour*

1. INTRODUCTION

One of the major challenges concerning the earthquake risk mitigation is the seismic vulnerability assessment of existing buildings that were not designed according to modern codes and the development of effective retrofitting techniques. Recent post-earthquake damage assessment reports recognized that the masonry infill walls played an important role in the seismic response of reinforced concrete (RC) buildings. Likewise, the infill walls seismic behaviour is being characterized by extensive damages and collapses due to combined in-plane (IP) and out-of-plane (OOP) loading demands which resulted in significant casualties, injuries and economic losses [1-4]. The retrofit and strengthening of infill panels must be a priority nowadays both in new constructions and in existing constructions (e.g. rehabilitations). The OOP collapse vulnerability shown by the infill panels during the last few earthquakes, often mentioned in the previous chapters, reinforces the needs of strengthening techniques which can improve the infill walls performance in future events. Nowadays the major concern during the design of a new structure is related to the structural members to ensure, proper design of those ones with the adequate capacity with the expected seismic demand. Few concerns are related with the “design” of the infill panels, considered by the codes as non-structural elements. In fact, the next generation of codes should demand the proper and adequate design of strengthening solutions to minimize their risk of collapse [5]. Regarding the rehabilitation of structures, the strengthening and retrofit of the infill panels distributed along the buildings’ façades should be a priority. According to the literature, the use of textile-reinforced mortars in civil engineering started in 1980, The TRM consists in the using of reinforcing meshes such as textile meshes with different continuous fibres, embedded in mortar which can have normal properties or high strength and/or ductility characteristics.

The most basic application is the fibre reinforced mortar, which consists in a mixture of mortar with a percentage of fibres randomly distributed within its composition. It is generally used as shotcrete, which became widespread for tunnel reinforcements. Some of the factors that can affect the efficiency of this strengthening solution is the fibre slenderness and length, as well as the size of aggregates that compose the mortar matrix, since they define the bonding properties and consequently the capacity to behave as a composite [6]. More complex solutions using the same kind of material imply the definition of a principal direction for the reinforcement (according to the interest of the designer) along which the fibres can develop their maximum strength capacity. The constructive solutions can be divided into two solutions, namely unidirectional and bidirectional reinforcement meshes. In the literature, few applications of TRM in infilled RC frames are focused on improving their IP behaviour [7-9]. Regarding the study of the using TRM to improve the OOP behaviour of infill panels in RC frames even more few. Angel, et al. [10] repaired three specimens that were previously subjected to OOP monotonic tests. Hollow clay

horizontal bricks were used to build the infill walls. The repair method consisted of spraying a half-inch thick ferrocement plaster to one or both faces of the infill panel. A single sheet wire mesh was applied on each face. The wire mesh was embedded in the plaster. Multiple wire sheets were lap spliced to cover the entire infill surface area. The wire mesh was anchored to the damaged infill panel by using steel bolts. A flexible PVC sleeve was placed over the bolts to provide relaxation to the system and to prevent premature cracking of the plaster. From the tests, the authors concluded that the OOP maximum peak load increased about 5 times; and the OOP strength of the repaired specimens was not affected by the level of the previous damage. The authors also stated that the repair methods were very efficient due to the panels' lower slenderness ratio and that the compressive strength of the repairing material was high. Calvi and Bolognini [11] tested the combination of using bed joints reinforcement with external reinforced plaster based solution. The adopted mesh was a welded wire metallic one. The strengthening was applied in both panel faces, being the meshes connected by steel plates in the bed joints, located at the distance equal to 1m width and 0.6m high, and bent around the wires of the meshes. From the testing campaign, the authors observed that the strengthening of the panel improved significantly the panel response, namely by increasing the deformation capacity and by modifying the damage limit states for higher drift levels. It was observed that the OOP acceleration demand to trigger the collapse of the panel was around 2.5g without prior IP damage and between 0.5-0.7g with prior IP damage. With the strengthening material, the acceleration increased to 3.5g without prior IP damage and to 1.6g with prior IP drift. De Risi, et al. [12] carried out a testing campaign comprising a total of four full-scale specimens' subjected quasi-static OOP tests using pneumatic actuators. One of them was tested in its as-built condition and the remaining three using different anchorages to connect the textile mesh to the surrounding frame (plastic connectors and L-shape GFRP connectors) and using normal or ductile mortar for the plaster. The authors used a GFRP textile mesh with a tensile strength equal to 40.0kN/m, and a mesh grid equal to 16.7x16.7mm, supplied by Fassa Bortolo. The authors reported a poor performance of the plastic connector as anchorage element to the RC frame-textile mesh even using a ductile mortar. The same problem was observed when using L-Shape GFRP connectors. The authors also pointed out that the shear cut of the textile mesh was observed in many cases due to the stress concentrations, and also by sliding of the mesh. This cutting and sliding phenomena contributed to the poor inefficiency of the adopted TRM solution. Globally, the authors found a good performance of the TRM solutions since they increased both cracking and maximum strength around 1.77 to 2.22 times, respectively. The energy dissipation of each strengthened panel was significantly higher than the as-built specimens. The increment provided by the strengthening solutions, for the same level of OOP displacement, was around 1.93 times.

Finally, some other authors such as Ismail, et al. [13], Kariou, et al. [14] and Shabdin, et al. [8] carried out flexural strength tests to characterize the effect of the TRM technique in the improvement of the infill panels OOP capacity.

This manuscript aims presenting a experimental study to assess the efficiency of using TRM-based strengthening solutions to prevent the out-of-plane collapse of masonry infills in RC buildings. The testing campaign comprises the OOP testing of five full-scale infill walls made up of horizontal hollow clay bricks, 150mm thick, two of them unstrengthened and the remaining three strengthened, with and without prior damage. In this testing campaign, the reference specimens are the panels Inf_08 and Inf_11, and the strengthened panels are Inf_10, Inf_12 and Inf_13. The following considerations (by each specimen) can be added:

- Inf_08: One-leaf panel; aligned with the external face of the supporting beam; OOP half-cyclic load, no columns axial load; no previous damage; total panel width supported in the bottom beam; 10mm plaster;
- Inf_11: One-leaf panel; aligned with the external face of the supporting beam; OOP half-cyclic load, no columns axial load; with previous damage due to IP test (0.3% inter-storey drift); total panel width supported in the bottom beam; 10mm plaster;
- The remaining three specimens (Inf_10, Inf_12 and Inf_13) were strengthened to prevent the out-of-plane collapse with two strengthening strategies, all based on the application of systems made up of common mortar plaster and glass fibre-reinforced polymer meshes. The influence of the mesh properties (tensile strength and ductility) and the efficiency of the strengthening under combined IP-OOP loadings are investigated in this experimental testing campaign. Details regarding the strengthened specimens in section 2.

All the tests consisted in the application of a semi-cyclic (loading-unloading-reloading) history of imposed displacements in the OOP direction by means of a uniformly distributed load through pneumatic actuators. The mechanical properties of the adopted materials are characterized and presented. The results will be presented in terms of OOP force-displacement responses, deformed shapes, damage evolution, energy dissipation capacity and damping. In the end, the tests' results are compared to each other to assess the effectiveness of the selected strengthening techniques and to provide a support towards the choice of the best strategies for future further investigations and applications.

2. Experimental campaign

This section aims to provide complete information concerning the experimental campaign, starting from the specimens details and characteristics in Section 2.1. After that, the material properties of each infill panels are presented in Section 2.2. In Section 2.3 are provided the details

of the strengthening strategies. Finally, the test setup is presented and deeply explained in 2.3 and the loading protocol and instrumentation are presented in 2.4.

2.1. Specimens' details

The infill wall specimens' geometric dimensions were defined as 4.20x2.30m (length and width respectively), which are representative of those existing in the Portuguese building stock according to the study developed by Furtado, et al. [15], and shown in Figure 1a and Figure 1b. The columns' and beams' cross sections were considered as 0.30x0.30m and 0.30x0.50m, respectively. Figure 1 shows the schematic layout of the RC frame geometry with the corresponding columns' and beams' dimensions and reinforcement detailing (Figure 1c, Figure 1d, and Figure 1e, respectively).

Figure 1 – Infilled RC frame specimen dimensions (units in meters): a) general dimensions; b) front view of the specimen; c) RC frame reinforcement detailing; d) column and e) beam dimensions and reinforcement detailing

All the infill panels have equal geometry with the above mentioned in-elevation dimensions, made of hollow clay horizontal brick units with 150mm thickness, as usually found in the most common infill masonry in Portugal. No reinforcement was used to connect the infill panel and the surrounding RC frame, and no gaps were adopted between the panel and the frame. All the panels were built aligned with the external side of the RC beam. The testing campaign is comprised by

5 specimens, namely 2 reference specimens without strengthening (one without prior damage imposed by a previous IP test – Inf_08; one with prior damage due to previous IP test – Inf_11) and 3 strengthened panels. These specimens were strengthened by using a fibreglass (GFRP) reinforcing mesh coupled to non-ductile (hereinafter referred to as “common”) cement-mortar. Table 1 reports a summary of the five specimens tested and analysed in the following sections. The characteristics of each specimens are the following:

- Inf_10: strong mesh, plastic connector to anchor the mesh-infill panel; steel connector to anchor the mesh to the frame, common cement-mortar M5 class;
- Inf_12: medium mesh; plastic connector to anchor the mesh-infill panel; steel connector to anchor the mesh to the frame plus a thin steel plate distributed along the contour where the connectors were applied; common cement-mortar M5 class;
- Inf_13: Same solution of the specimen Inf_12.

Table 1 - Experimental tests matrix.

Specimens	Strengthening	Type of mesh	Previous IP damage	Note
Inf_08	No	N/A	No	Reference specimen
Inf_10	Yes	Strong	No	Comparison with Inf_08
Inf_11	No	N/A	Yes (0.3%)	Reference specimen
Inf_12	Yes	Medium	Yes (0.3%)	Comparison with Inf_11
Inf_13	Yes	Medium	No	Comparison with Inf_08

2.2. Material Properties

The material ordered for the RC frame specimen construction consisted of regular C20/25-class concrete [16], namely with a mean cubic compressive strength equal to $f_{cm,cyl}=21.4\text{MPa}$ with a standard deviation equal to 1.35MPa and a coefficient of variation equal to 6.1% . The average elastic modulus obtained was 24.3GPa with a standard deviation equal to 0.21GPa and a coefficient of variation equal to 0.9% . These concrete material results were obtained from cylindrical specimens collected during the casting of the frame. Concerning the steel reinforcement, three different bar diameters were used, from the same lot, namely 6mm, 10mm and 16mm. Three samples of each were taken from each diameter bar and tested according to [17]. From the test results, the yield strength and the young modulus of the steel bars are 444MPa and 204.2GPa regarding the $\phi 6\text{mm}$ bars, 598.9MPa and 209.7GPa regarding the $\phi 10\text{mm}$ bars and 494.4MPa and 209.4GPa regarding the $\phi 16\text{mm}$ bars.

Concerning to the material properties of the masonry unit, Table 2 provides the main characteristics that can be found in the datasheet provided by the manufacturer. The compressive strength of the masonry units was determined by performing experimental tests according to NP EN 771-1 [18]. From the tests it was found an average compression strength equal to 1.04MPa, a coefficient of variation (CoV) of 23.6% and standard deviation of 0.24MPa.

Table 2 - Summary of the masonry units' material properties (information provided by the product datasheet).

Material Properties	
Compressive strength $f_{c,unit}$ (MPa)	1.04
Content of active soluble salts	S0 category
Fire reaction	A1 Euroclass
Masonry unit mass (Kg/unit)	5.2
Thermal transmission coefficient U (m^2K/W)	0,42
Acoustic Insulation R_w (dB)	43
% voids	80.5

Additionally, flexural and compression strength mortar tests were carried out in mortar specimens according to EN 196-2006 [19]. Six specimens, 40x40x160mm were tested for each infill panel build (both mortar used to build the panel and mortar used for strengthening). The major results are summarized in Table 3. From the results it is possible to observe that the compressive strength of the mortar used for the infill bed joints are around 5MPa as expected, being the highest value achieved by the specimen Inf_11 with 6.49MPa and the lowest by Inf_13 with 3.52MPa. Concerning the flexural strength, the values are around 1.8MPa, being the largest value obtained again by Inf_11 with 2.22MPa and the lowest by Inf_13 with 1.53MPa. The results of the mortar used for the strengthening/plaster present similar trends, being the highest values achieved for the specimen Inf_11 and the lowest for Inf_08.

Table 3 – Results from flexure and compressive strength tests on mortar specimens.

Specimen	Mortar used for the construction of the infill panel		Mortar used for plaster	
	Flexure strength (MPa)	Compressive strength (MPa)	Flexure strength (MPa)	Compressive strength (MPa)
Inf_08	1.90 (C.o.V.=6.6%)	5.24 (C.o.V.=7.8%)	1.81* (C.o.V.=7.3%)	4.01* (C.o.V.=8.2%)
Inf_10	1.82 (C.o.V.=11.6%)	4.74 (C.o.V.=10.9%)	2.28 (C.o.V.=5.8%)	4.94 (C.o.V.=7.2%)
Inf_11	2.22 (C.o.V.=10.7%)	6.49 (C.o.V.=11.3%)	2.32* (C.o.V.=4.9%)	8.20* (C.o.V.=5.5%)
Inf_12	1.84 (C.o.V.=7.6%)	6.25 (C.o.V.=8.9%)	1.65 (C.o.V.=7.8%)	5.71 (C.o.V.=5.8%)
Inf_13	1.53 (C.o.V.=4.9%)	3.52 (C.o.V.=6.6%)	2.56 (C.o.V.=11.9%)	6.52 (C.o.V.=8.6%)

2.3. Description of the strengthening strategies

2.3.1. Inf_10

The specimen Inf_10 was strengthened using a TRM solution (herein designated strengthening solution A) with the glass fibre mesh “FASSANET ARG 40”, supplied by Fassa Bortolo, with a matrix 4x4cm and a tensile strength equal to 70kN/m. This mesh was the same used in the wallets subjected to flexural strength tests and described in section 7.2.2. Steel connectors ($\phi 6$ mm and 8cm length) ensured the connection of the mesh to the panel and that two of the brick septa were crossed by (Figure 2a and Figure 2b). In order to ensure a better connection of the mesh to the wall, plastic discs with 6cm diameter were used with the metallic connectors. Regarding the anchorage of the mesh to the RC frame it was adopted a solution made with M8 steel connectors also with a plastic disk with 6cm diameter, as shown in Figure 2c and Figure 2d.

Figure 2: Strengthening materials used for the panel Inf_10: a) detail of the steel connectors and plastic disk used in the panel; b) general view of the mesh fixed to the panel; c) detail of the M8 steel connectors and plastic disk used to fix the mesh to the frame; and d) detail view of the application.

The strengthening application followed the steps described hereafter:

- Application of a first layer of plaster (thickness around 0.5cm), shown in Figure 3a;
- Placement and positioning of the mesh, shown in Figure 3b;
- Fixation of the mesh with the connectors, shown in Figure 3c;
- Application of a second layer of plaster 1-1.5cm thick, shown in Figure 3d.

The mortar used for the plaster application was a current one, M5 class, as previously mentioned. Five vertical strips with 1 meter width each, were applied with an overlapping width equal to 10cm, according to the supplier recommendation. The transition between the panel and the frame elements was reinforced with two layers of mesh with a total width equal to 30cm (15cm in the

panel plus 15cm in the frame elements). The schematic layout of the strengthening strategy adopted is plotted in Figure 4.

Figure 3 - Strengthening process of the panel Inf_10: a) Application of the first layer of plaster; b) positioning of the mesh; c) anchorage of the mesh to the frame; and d) application of the second layer of mortar.

Figure 4: Schematic layout of the Inf_10 strengthening strategy.

2.3.2. Inf_12 and Inf_13

It was adopted the same strengthening solution for the panels Inf_12 and Inf_13, again a TRM solution and herein designated solution B. The used reinforcing mesh was a bidirectional

fibreglass (GFRP) mesh with a $16.7 \times 16.7 \text{ mm}^2$ matrix (see Figure 5a), a weight of 185 g/m^2 , a nominal tensile strength equal to 40.0 kN/m , and a maximum ultimate strain equal to 3.4% .

The application procedure of this strengthening strategy started with the application of 1 cm plaster. Then plastic connectors, see Figure 5b, were applied throughout the infill panel to position and fix the reinforcing mesh. For the tested specimens, the plastic connectors have been applied aiming to position and fix the mesh. The roll of mesh was provided with 1 m width and 50 meters length. Five vertical strips (1 m width) were used to strengthen the wall overlapped to each other. The application of vertical strips resulted easier with respect to the application of horizontal strips (whose length can be also very different depending on the bay-length). The overlap length used between each vertical strip was 10 cm . The mesh was extended for 15 cm both on the beams and columns, Then, in the overlapping regions for the transition RC frame-infill panel it was assumed a duplicated mesh with an overlap equal to 30 cm (15 cm for the RC frame and 15 cm for the infill panel).

A different strategy was adopted for the mesh-frame anchorage, namely it was adopted a combined solution using a steel plate (3 mm thick and 30 mm width) along the alignment defined for the mesh-frame anchors. $\varnothing 10 \text{ mm}$ holes were drilled in each location defined for the anchor to insert M8 steel connectors. Thus, the mesh was placed between the frame and the steel plate, which was anchored with the steel connectors to the frame. The main goal was to minimize the local sliding/shear failure of the mesh, a common problem reported by De Risi, et al. [12]. Details of the steel connectors and the steel plate are shown in Figure 5c and Figure 5d.

a)

$L_a = 75 \text{ mm}$
 $h_{ef} = 25 \text{ mm}$

b)

Figure 5: Strengthening materials used for the panel Inf_12 and Inf_13: a) general view of the mesh fixed to the panel; b) detail of the plastic connector for the mesh-panel anchorage; c) detail of the M8 steel connectors for the frame-mesh anchorage and general view of the steel plate; and d) detail view of the steel connector and plate.

The strengthening procedures for the panels Inf_12 and Inf_13 were according to the following steps:

- Application of a first layer of plaster (thickness around 0.5cm-1cm), shown in Figure 6b;
- Placement and positioning of the mesh, shown in Figure 6c;
- Fixation of the mesh to the top RC beam, with the application of the steel plate and connectors, shown in Figure 6c. Details of the holes' drilling in the frame for the connector are shown in Figure 6d. The tightening of the steel connector is shown in Figure 6e. The detail of the mesh-frame anchorage is shown in Figure 6f;
- Application of the plastic connectors for the mesh-panel anchorage. The general view of the specimen with the plastic connectors is shown in Figure 6g;
- Application of a second layer of plaster 2cm thick, shown in Figure 3h.

a)

b)

c)

d)

e)

f)

g)

h)

Figure 6 - Strengthening process of the panels Inf_12 and Inf_13: a) general view of the panel as-built condition; b) application of the first layer of plaster; c) positioning of the mesh; d) holes drilling in the column for connector; e) tightening of the steel connector; f) detail view of the mesh-frame anchorage; g) general view of the strengthened panel with mesh and all the connectors applied; and h) application of the second layer of mortar.

The final layout of the strengthening solution herein designated “B” is shown in Figure 7.

Figure 7 - Strengthening process of the panel Inf_12 and Inf_13: a) Schematic layout.

2.4. Test Setup

The OOP tests consisted in the application of a distributed OOP loading through several pneumatic actuators that mobilize the entire infill panel surface resorting to wood plates (one per actuator) placed between the actuators and the panel. The pneumatic actuators are linked to four horizontal alignments performed by HEB140 steel shapes, which react against five vertical alignments also made of HEB200 steel shapes. The horizontal alignments are coupled with hinged devices that allow lateral sliding. The steel structure is attached to the RC frame in twelve points (5 in the bottom beam, 5 in the top beam, and 2 in middle-height columns) with steel bars that are coupled with load cells to allow monitoring the OOP loadings. This self-equilibrated system balances the transmission of the OOP loadings to the reaction frame. This innovative testing platform is able to test infill panels with openings (with different configurations and dimensions). The instrumentation was also placed in the back of the reaction structure, allowing performing OOP tests until the panel collapse without damaging any equipment. Figure 8a and Figure 8b show the schematic layout and the general view of the test setup.

Figure 8 – Layout of the OOP test set-up using pneumatic actuators: a) front and b) near view.

The panel loading was performed by the pneumatic actuators, the control is performed by monitoring their interior pressure and the imposed OOP displacements in a selected control point. General views of the testing setup are shown in Figure 9.

Figure 9 – View of the OOP test set-up using pneumatic actuators: a) overall view; and b) lateral view.

2.5. Instrumentation and loading protocol

Twenty-five displacement transducers were used to measure the OOP displacements of the panel along five horizontal and vertical alignments; the detachment of the panel from the surrounding corners was measured by Linear variable displacement transducers (LVDT's) placed in the transition infill plane-frame; the OOP displacements of columns and beams were monitored by 6 displacement transducers. Additionally, the vertical displacements of the top beam were measured by one LVDT placed at mid-span to monitor the evolution of possible arching mechanism (Figure 10).

Figure 10 – Instrumentation: schematic front layout.

The loading protocol consisted on the application half-cyclic OOP displacements (loading-unloading-reloading) that were imposed with steadily increasing displacement levels, targeting the following nominal peak displacements at the control node located in the centre of the panel: 0.5, 1, 1.5, 2, 2.5, 3.5; 5; 7.5; 10 mm; and then 5 by 5 mm up to a maximum OOP displacement of 120 mm. Two half-cycles were repeated for each lateral deformation demand level.

3. Experimental results

In this Section, the results of the experimental testing procedure described above are presented and analysed test by test responses in terms of OOP load (F_{OOP}) versus OOP displacement in the centre of the infill panel (d_{OOP}) are shown, together with the damage evolution from the first observed cracks on the panel until the end of the test. The OOP drift was also computed related to the panel control node. Some considerations will be drawn regarding the first cracking (instant in which occur the first crack, maximum peak load and ultimate (residual) load).

3.1. Reference specimens

3.1.1. Specimen Inf_08 (Pure OOP test w/o prior damage)

Error! Reference source not found.a shows the first cracking of the panel in the central horizontal between the panel half-height and quarter half-height. This first cracking occurred for an OOP drift equal to 0.11% and a corresponding load ($F_{OOP,cr}$) equal to 22.82kN. A trilinear cracking was visible with slight detachment of the plaster in the right and left sides of the panel and the beginning of detachment of the panel from the top beam. The peak load, equal to 44.22kN (1.93 times higher than $F_{OOP,cr}$) occurred for an OOP drift equal to 2.21%. The collapse of the panel occurred for an OOP drift equal to 3.20%, very close to the OOP drift at maximum peak load, and for a corresponding strength equal to 37.94kN (15% lower than the peak load), having occurred the detachment of the panel from the top and bottom beams, which led to a sudden loss of OOP stability and to the progression of the vertical crack until the bottom beam. The final damage visible at panel collapse is presented in **Error! Reference source not found.**b. Note that the panel did not collapse to the laboratory floor due to the existence of safety steel cables surrounding the frame to prevent that occurrence. **Error! Reference source not found.**c presents the OOP force-displacement response, where the 3 major stages are indicated (red circle – cracking point; blue circle – peak load; and green circle – panel collapse).

a)

b)

Figure 11 – Specimen Inf_08: a) First cracking; b) Final damage observe; and c) force-displacement curve.

3.1.2. Specimen Inf_11 (OOP test w/ prior damage – 0.3% IP drift)

Figure 12 shows the damages obtained after the IP test (with a maximum 0.3% drift) where it was observed the detachment of the panel from the top beam, the detachment of some plaster and a horizontal slide cracking in the left bottom of the panel.

Figure 12 – Specimen Inf_11: Damages after IP test - Cracking pattern.

Concerning the OOP test, the first cracking occurred at 0.67% drift and it was found vertical from the top of the panel down to mid-height, as shown in Figure 13a, with a corresponding force equal to 23.5kN. The maximum peak load equal to 30.7kN was reached for OOP drift equal to 4.47% where it was visible a trilinear cracking pattern with some slight cracks in the centre of the panel (Figure 13b). The test was carried out until the maximum stroke of the pneumatic actuators, with an OOP displacement equal to 102.10mm (drift equal to 8.87%) and a corresponding force of 13.3kN (Figure 13c).

Figure 13 – Specimen Inf_11: a) First cracking; b) Final observed damages; and c) force-displacement curve.

3.2. Strengthened specimens

3.2.1. Specimen Inf_10 (Strengthening solution A; w/o prior damage)

The strengthened panel Inf_10 was only subjected to a pure OOP test, similarly to the reference specimen Inf_08. The force-displacement curve is shown in Figure 14a. The initial (secant) stiffness of the F_{OOP} - d_{OOP} response, calculated as the ratio between F_{OOP} and d_{OOP} at the first peak relative to the first applied displacement level, is equal to $k_{OOP,sec,ini} = 18.90$ kN/mm. Afterwards, the first macro-cracking occurred for an OOP drift equal to 0.33% and a corresponding force equal to 47.23kN. The first cracking was a horizontal crack located at the panel mid-height with a total length corresponding to 2/3 of the panel length, as shown in Figure 14b. Diagonal cracks and a parallel horizontal crack developed with the increase of the OOP displacements. The maximum peak load equal to 77.10kN was reached for an OOP drift equal to 2.35%. After this point, it was visible the beginning of strength degradation. The displacement transducers were not capable to capture the maximum value of that displacement (which, however and based on the remaining transducers, was possible to estimate around 50mm), because they exceeded their measuring range. However, it was possible to measure the value of the residual displacement of the wall (about 41.9mm) and to restart the test from that point (Figure 14a). The force-displacement response includes a hidden red line at the moment from which it was not possible to record the OOP displacements. After that moment, the stiffness degradation was more evident,

but still ten more cycles were possible to be performed until reaching the ultimate drift equal to 6.31% without any strength degradation. The ultimate strength was 39.40kN. The final damage of the panel is shown in Figure 14c, and was characterized by a distributed cracking, particularly from the panel mid-height to the bottom.

a)

b)

c)

Figure 14 – Experimental results: Specimen Inf_10: a) force-displacement curve; b) first cracking; and c) cracking pattern.

Figure 15a shows the lateral view of the bottom part of the panel where no panel detachment or sliding is visible. Some concentration of horizontal cracks are noticed which are related to the failure of the bricks from the first row of the panel, as evidenced in the Figure 15b and Figure 15c that showing the nearside of the panel. Some crushing of bricks was also visible as well as some sliding of the panel from bottom beam. It was observed that the failure of those bricks occurred at the maximum peak load and caused the large decrease of OOP strength and the larger OOP displacements.

Figure 15 – Experimental results: Specimen Inf_10: a) Lateral view of the damage in the front of the panel; b) right view of the panel near side; and c) left view of the panel back side.

3.2.2. Specimen Inf_12 (Strengthening solution B; w/ prior damage – 0.3% IP drift)

The specimen Inf_12 was subjected to an IP-OOP test sequence, i.e. the panel was first subjected to a quasi-static IP test with a maximum drift equal to 0.3% and afterwards to the OOP test, similarly to the reference specimen Inf_11. During the IP, slight detachment occurred between the panel and top beam, as proved by the horizontal crack in the top, evidenced in Figure 16. Some plaster detachment occurred, however without any shear sliding failure of the mesh. A diagonal crack from the right top corner of the panel to the left bottom corner developed for the maximum IP drift (0.3%). Horizontal and vertical cracks over the panel contour were noticed with some concentration in the panel corners, due to the increase of the panel stresses during the distortion of the frame.

Figure 16 – Experimental results: Specimen Inf_12: a) general view of the final damage after IP test; and d) cracking pattern after IP test.

The force-displacement curve is plotted in Figure 17a, where it is possible to observe that the initial secant OOP stiffness of the panel was equal to 7.44kN/mm and the first cracking occurred for an OOP drift equal to 0.71% and a corresponding force equal to 44.90kN. The observed crack was a diagonal one, parallel to that developed during the IP test, as shown in Figure 17b. After

the first cracking, some diagonal cracks and horizontal ones occurred from all over the panel until the maximum peak load equal to 75kN corresponding to an OOP drift equal to 2.75%.

At the peak load, the panel detached from the bottom beam, but with some sliding of the mesh from the steel plate anchored to the beam. The panel OOP strength reduced 20% until 60kN, and continued to reduce the strength until the maximum drift equal to 8.91% with a residual force equal to 39.16kN. The final damages and cracking pattern of the specimen Inf_10 are shown in Figure 17c.

Figure 17 – Experimental results: Specimen Inf_12: a) force-displacement curve; b) first cracking; and c) cracking pattern.

From the test, the strengthening solution was found effective since it prevented OOP collapse of the panel. During the test, large OOP displacements occurred in the bottom of the panel, which again were due to the failure of bricks distributed in the panel first row. Figure 18a presents the lateral view of the bottom part of the wall where it is visible the steel plate still anchored to the beam, but exhibiting sliding failure of the anchored mesh at mid-length. An OOP displacement of the panel equal to 120mm was measured, i.e. 70% of the panel thickness, which revealed the impressive capacity of the mesh to sustain the panel, as shown in Figure 18b.

The near side of the panel shows crushing of some bricks located in the top and bottom rows which can be justified by the fact that they were subjected to larger flexural than their flexural strengths (Figure 18c and Figure 18d). Figure 18e and Figure 18f shows the nearside of the panel, where it is possible to observe the panel deformation. In fact, the panel was almost totally detached from the bottom support beam without collapsing. It was not visible any detachment or sliding failure of the mesh in the other regions of the panel.

Figure 18 – Experimental results: Specimen Inf_12: a) detail of the mesh sliding; b) detail of the detachment of the panel and the mesh sliding; c) left view of the panel back side; d) right view of the panel back view; e) detail of the deformed shape of the panel; and f) detail of the measurement of the OOP displacement.

3.2.3. Specimen Inf_13 (Strengthening solution B; w/o prior damage)

The strengthened panel Inf_13 was only subjected to a pure OOP test, similarly to the reference specimen Inf_08, and to assess the efficiency of the strengthening solution B first implemented and tested in specimen Inf_12. The force-displacement curve is shown in Figure 19a. The initial secant stiffness was equal to 80.92kN/mm, as visible in the quick increase of the OOP strength for low displacements.

The first macro-cracking occurred for an OOP drift equal to 0.17% and was an horizontal crack located above the panel mid-height, as shown in Figure 19a. Again, diagonal and horizontal cracks appear until the maximum peak load (92.31kN) corresponding to a drift of 2.55%. After that, it occurred the fracture/crushing of the bricks located at the firs and, consequently, the OOP strength reduced about 44kN and the OOP drift suddenly increased until approximately 7%.

Until the last stage of the test, it was not observed any significant reduction of the panel strength and the residual capacity of the panel was found equal to 42.61kN for an OOP drift equal to 8.70%. The test has stopped due to the limit of the pneumatic actuators stroke that was reached. The panel cracking pattern is shown in Figure 19c.

a)

b)

c)

Figure 19 – Experimental results: Specimen Inf_13: a) force-displacement curve; b) first cracking; and c) cracking pattern.

After the test, a detailed damage assessment was carried out. The main focus was to assess the anchorage of the mesh, the performance of the frame-mesh connectors, and the importance of the steel plate. This test evidenced the importance of the adopted frame-mesh anchorage since it allowed to reach high OOP displacements avoiding the shear sliding of the mesh. Figure 20a shows the lateral view of the bottom part of the panel where it is possible to observe the mesh-frame anchorage. The superficial plaster layer was removed to verify the state of the mesh, shown in Figure 20b. The mesh was found in good conditions, with one example of mesh sliding illustrated in Figure 20c. The same procedure was performed in the top part of the wall, where it was possible to observe that the mesh was intact with perfect anchorage to the frame (Figure 20d). As previously described, the back side of the panel (Figure 20e and Figure 20f) shows the fracture and crushing of the bricks located in the first and last row of the panel, in particular in the mid-length zone.

a)

b)

c)

d)

e)

f)

Figure 20 – Experimental results: Specimen Inf_13: a) lateral view of the bottom part of the panel front; b) detail of the mesh-frame anchorage after detached the plaster; c) detail of few examples of sliding of the mesh; d) detail of the top mesh-frame anchorage; e) left view of the panel back side; and f) right view of the panel back side.

4. Comparison of the results and critical analysis

In this section, the experimental results above described are compared to each other in terms of OOP force-displacement envelopes, failure modes, and dissipated energy. Final remarks concerning the comparison between the efficiency of each strengthening solution are discussed.

4.1. Out-of-plane force-displacement envelopes

Figure 21a and Figure 21b shows a comparison among the test results in terms of F_{OOP} - d_{OOP} envelopes of the specimens without prior damage (Inf_08, Inf_10 and Inf_13) and with previous IP damage (Inf_11 and Inf_12). In the same way, the relative secant stiffness evolution is shown in Figure 21c and Figure 21d. Additionally, Table 4 and Figure 22 provide a summary of the main results, as commented in the following remarks.

Figure 21 – Assessment of the efficiency of the strengthening techniques: a) force displacement envelope curves (specimens without prior damage); b) force displacement envelope curves (specimens with previous IP damage); c) relative stiffness (specimens without prior damage); and d) relative stiffness (specimens without prior damage).

Initial secant stiffness

The initial secant stiffness, $k_{\text{OOP,sec,ini}}$, was calculated for each specimen, and from the results summarized in Table 4 it follows that the maximum value was reached by the strengthened panel Inf_13 with 80.92kN/mm and the lowest value was achieved by the strengthened specimen Inf_12 with 7.44kN/mm. The no expected result obtained by the panel Inf_12 can be justified by the fact that it was firstly damaged during the IP test which reduced the initial stiffness of the panel. Actually, comparing with the reference specimen, Inf_11, the result is quite similar, which allows to conclude that the strengthening solution does not contribute to the panel initial stiffness.

Regarding the strengthened panels subjected only to pure OOP load (Inf_10 and Inf_13), they achieved initial secant stiffness values that were 0.93 and 3.98 times the value reached by the reference specimen.

First cracking

From the analysis of the results obtained by each specimen, it is possible to observe that, regarding the force corresponding to the development of the first cracking, the panel Inf_13 reached the maximum value of 59.11kN, 62% higher than the result obtained by Inf_08 that achieved the lowest cracking strength. From the comparison between the specimen Inf_10 and the reference specimen Inf_08, it is possible to conclude that the strengthening solution contributed to the increase of the cracking strength about 2.08 times. By the comparison, of the specimen Inf_10 with the other strengthened specimen subjected to a pure OOP test, specimen Inf_13, it was observed that the strengthening solution B reached a cracking strength about 1.31 times higher.

Regarding the comparison the specimen Inf_12 and the reference one Inf_11, it is possible to observe that the strengthening solution improved the cracking strength about 1.95 times. Finally, concerning the impact of the damage caused by the IP test in the efficiency of the strengthening solution B, the cracking strength of the specimen Inf_12 reduced about 24% when compared with the result of the specimen Inf_13.

Interesting results were obtained regarding the OOP drift for which occurred the first macro-cracking, namely the maximum value reached by Inf_12 with 0.71%, which is justified by the fact that the Inf_12 was first subjected to the IP test. The lowest value was achieved by 0.11% by specimen Inf_08. From the comparison between the strengthened specimens and the respective reference specimens, it can be noticed that the strengthening solutions contributed to the increase of the $d_{\text{OOP,crack}}$ in all the examples between 1.06 and 3 times.

Maximum peak stage

Concerning maximum peak load, very satisfactory results were obtained by all the strengthened panels as proved by the increase between 1.7 and 2.44 times. The largest $F_{\text{OOP,max}}$ was achieved

by Inf_13 with a value equal to 92.31kN, 2.44 times higher than the as-built reference specimen (Inf_08). The specimen Inf_10 maximum OOP strength was 77.10kN, 1.74 times higher than Inf_08 and 16% lower than the strengthened panel Inf_13. Regarding the last strengthened specimen Inf_12, the maximum strength equal to 75kN was 2.44 times higher than Inf_11 and only 18% lower than the result obtained by Inf_13 which is a proof that the efficiency of the strengthening solution was not significantly affected by the prior damage.

From the analysis of Figure 22a, it is possible to observe that the strengthening was efficient, since all the strengthened panels reached higher maximum peak loads. Regarding the OOP drift corresponding to each stage (Figure 22b), the same trends can be noticed. The specimens without prior damage, reached the maximum peak load for $d_{OOP,max}$ values between 1.06-1.15 times higher. On the other hand, concerning the specimens with previous damage, the maximum peak load of the strengthened specimen occurred for a $d_{OOP,max}$ 39% lower.

Table 4 - Assessment of the efficiency of the strengthening techniques: Comparisons of the results.

Parameters	Reference	Strengthening solution A	Reference	Strengthening solution B	Strengthening solution B
	Inf_08 (OOP)	Inf_10 (OOP)	Inf_11 (IP-OOP)	Inf_12 (IP-OOP)	Inf_13 (OOP)
$F_{OOP,max}$ (kN)	44.15	77.10	30.72	75	92.31
$F_{OOP,crack}$ (kN)	22.60	47.23	23.09	44.90	59.11
$F_{OOP,ult}$ (kN)	37.96	39.40	13.33	39.16	42.61
$k_{OOP,sec,ini}$ (kN/mm)	20.32	18.90	7.97	7.44	80.92
$d_{OOP,max}$ (%)	2.21	2.35	4.47	2.75	2.55
$d_{OOP,crack}$ (%)	0.11	0.33	0.67	0.71	0.17
$d_{OOP,ult}$ (%)	3.20	6.31	8.87	8.91	8.70

Ultimate stage

The ultimate stage of all the specimens was studied and it correspond to the last instant of the tests ($F_{OOP,ult}$ and $d_{OOP,ult}$), which in one case (Inf_08) was due to the panel collapse, in another case (Inf_10) was due to the risk of collapse and consequent protection of the instrumentation and in the remaining specimens correspond to the limit of the pneumatic actuators. From the point of view of the collapse prevention, it becomes clear that both strengthening solutions were effective. The test of panel Inf_10 did not allow to take solid conclusions concerning this aspect, since the test has stopped for earlier OOP displacements. However, the results obtained by the panels Inf_12 and Inf_13 show that the use of a proper TRM strengthening solution can prevent the panel OOP collapse and its consequences.

The results are particularly good since the residual strength of the panels Inf_12 and Inf_13 were 39.16kN and 42.61kN, respectively which allow to highlight the following conclusions: i) the negligible effect of the IP damage in the efficiency of the mesh for the OOP displacement; ii) the increase of the OOP deformation capacity for larger displacements without collapse; iii) increase of the ultimate OOP strength. In the case of the comparison among the panel Inf_13 and Inf_08 the major difference is the OOP drift level achieved by the strengthened panel, which is about 2.72 times higher without collapse.

Figure 22 – Assessment of the efficiency of the strengthening techniques: a) OOP strength F_{OOP} ; and b) OOP drift d_{OOP} .

4.2. Failure mode and collapse prevention

An additional comparison among the presented test results can be carried out in terms of observed collapse prevention. Regarding the as-built specimens, the panel Inf_08 presented at the end of the test an OOP drift of 3.20%, characterized by the OOP collapse of the panel. The cracking pattern of the panel immediately prior to the collapse is shown in Figure 23a. The cracking pattern was characterized by a trilinear shape, with the detachment of the panel from the top beam. The other as-built specimen, and first subjected to prior IP damage, the panel Inf_11 reached a maximum OOP drift equal to 8.87%. Similarly to Inf_08, a trilinear crack pattern developed throughout the test (Figure 23c), with detachment from the top beam. The collapse of the panel did not occur.

Regarding the strengthened specimens subjected to pure OOP tests (Inf_10 and Inf_13), both behaved similarly with distributed cracking throughout the panel without sliding failure of the mesh or detachment of the mesh-frame anchorage. However, some differences can be noticed namely the fact that the cracking pattern of the specimen Inf_10 was more concentrated from the mid-height to the bottom of the panel, by contrast with the specimen Inf_13, where it is visible a uniform distribution of cracking over the panel, as shown in Figure 23b and Figure 23e respectively. Finally, it is important to remark the differences between the strengthened panel Inf_12 and the as-built panel Inf_11. The use of the TRM solution, Inf_12, allowed to reach a

similar OOP drift, through ensuring that the panel is protected by the mesh which increased the OOP strength. In the strengthened specimen the spread cracking over the entire panel was visible, in contrast to the trilinear crack reached by the specimen Inf_08. The cracking pattern of the specimens Inf_11 and Inf_12 are shown in Figure 23c and Figure 23d, respectively.

Figure 23 – Assessment of the efficiency of the strengthening techniques: Cracking pattern a) Inf_08; b) Inf_10; c) Inf_11; d) Inf_12 and e) Inf_13.

4.3. Cumulative energy dissipation

The energy dissipated in each individual loading half-cycle and the cumulative energy dissipation throughout the whole test history were calculated and plotted in Figure 24. From the plots of the energy dissipated in each cycle, some differences can be pointed: i) some peaks can be identified in the strengthened panels that corresponded to sudden large displacements reached by the panels when the crushing occurred in the bricks located in the top and bottom rows (visible in the force-

displacement response); and ii) the cycle with largest energy dissipation in the specimen Inf_08 was precisely the last one, when the panel reached sudden increase of the OOP displacement.

Figure 24 – Assessment of the efficiency of the strengthening techniques: Dissipated energy per cycle: a) Inf_08; b) Inf_10; c) Inf_11; d) Inf_12 and e) Inf_13.

The cumulative energy dissipation was computed not considering the peak of energy dissipation which is visible in the response of the specimens Inf_10, Inf_12 and Inf_13. From that, the total cumulative energy dissipation reached by the specimen Inf_08, Inf_10, Inf_11, Inf_12 and Inf_13 were, respectively, 1.9, 6.2, 3.9, 8.0 and 9.2 kNm.

The cumulative energy dissipation evolution is plotted in Figure 25, from which it is possible to observe that, for the same OOP displacement demand, the panels Inf_10 and Inf_13 dissipated similar levels of energy. From the comparison between the panels without previous IP damage, it can be observed that for an OOP displacement around 30mm, the specimens Inf_10 and Inf_13 reached a cumulative energy dissipation 45% and 48% higher than that of Inf_08, respectively.

Regarding the total cumulative energy dissipation, it is clear the efficiency of the strengthening, since it was reached by the strengthened specimens' values 3.26 and 4.84 times higher, respectively Inf_10 and Inf_12. It is also visible that the strengthening solution B was more efficient, since the cumulative energy dissipation reached by Inf_13 reached 1.48 times higher than that of Inf_10. From the comparison of the results of the panels with previous IP damage, it is visible that the strengthening was efficient and contributed to the increase of the energy dissipation about 2.05 times.

Figure 25 – Assessment of the efficiency of the strengthening techniques: Cumulative energy dissipation.

4.4. Critical analysis of the strengthening solutions efficiency

To conclude the analysis of the strengthening solutions efficiency it is important to explore and discuss the force-displacement curves, where it is possible to verify the effect of the reinforcement in the response parameters, such as initial stiffness, cracking point ($d_{OOP,crack}$ and $F_{OOP,crack}$), maximum peak load ($d_{OOP,max}$ and $F_{OOP,max}$) and ultimate point ($d_{OOP,ult}$ and $F_{OOP,ult}$).

Table 5 presents a summary of strengthening factors (ratio between the as-built specimens and the strengthened ones or ratio between the strengthened specimens).

Table 5 - Assessment of the efficiency of the strengthening techniques.

Parameters	Ratio Inf_10/Inf_08	Ratio Inf_13/Inf_08	Ratio Inf_10/Inf_13	Ratio Inf_12/Inf_11	Ratio Inf_12/Inf_13
$F_{OOP,max}$	1.75	2.09	0.84	2.44	0.81
$F_{OOP,crack}$	2.10	2.62	0.80	1.94	0.76
$F_{OOP,ult}$	1.04	1.12	0.92	2.94	0.92
$k_{OOP,sec,ini}$	0.93	3.98	0.23	0.94	0.09
$d_{OOP,max}$	1.06	1.15	0.92	0.62	1.08
$d_{OOP,crack}$	3.00	1.55	1.94	1.06	4.18

$d_{\text{OOP,ult}}$	1.97	2.72	0.73	1.01	1.02
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Starting from the analysis of the results obtained by the panels without previous IP damage (As-built: Inf_08; strengthened panels: Inf_10 and Inf_13) that are plotted in Figure 26a, Figure 26b and Figure 26c, the following observations can be drawn:

- From the comparison between the specimen Inf_08 and Inf_10 force-displacement curves, shown in Figure 26a, it is possible to observe the strengthening solutions did not contributed to the increase of the initial stiffness. In fact, the initial stiffness of the panel Inf_10 was 7% lower. The first crack occurred for in the strengthened panel for an OOP displacement ($d_{\text{OOP,crack}}$) 3 times higher and for a corresponding load ($F_{\text{OOP,crack}}$) 2.10 times higher. The large efficiency of the strengthening is visible by the improvement of the maximum peak load ($F_{\text{OOP,max}}$) about 1.75 times that occurred for a similar OOP displacement. It is possible to observe that, despite the significant strength reduction after the peak load, the residual OOP strength is quite significant (39.4kN) and the strengthening allowed to reach OOP displacement 1.97 ($d_{\text{OOP,ult}}$) times higher than those reached by Inf_08 without collapsing;
- Concerning the comparison between the strengthened panel Inf_13 and the as-built specimen Inf_08, shown Figure 26b, it can be observed that the initial stiffness of the strengthened one was 3.98 times higher. The first crack occurred in the panel Inf_13 for an OOP displacement ($d_{\text{OOP,crack}}$) 1.55 times higher and for a corresponding load ($F_{\text{OOP,crack}}$) 2.62 times higher than that of Inf_08. The strengthening solutions increased the panel maximum peak load in 2.09 times, which occurred for a $d_{\text{OOP,max}}$ 15% higher. Again, the strengthening allowed to reach a $d_{\text{OOP,ult}}$ 2.72 times higher with a residual load 1.12 times higher. In the response of the strengthened specimen, after the peak load it occurred a sudden increase of the OOP displacement with a reduction of the OOP load, however the presence strengthening prevented the possibility of collapse;
- Regarding the assessment of the most efficient strengthening solution, the responses of the panels Inf_10 (Strengthening solution A) and Inf_13 (Strengthening solution B), are shown in Figure 26c. In all the response parameters (except $d_{\text{OOP,crack}}$), the specimen Inf_13 is more efficient by achieving higher results than that of Inf_10. For example, the initial stiffness of the panel Inf_13 was 75% higher. Concerning the first crack occurrence, it occurred for and OOP displacement 8% higher and for a corresponding load 20% higher. The maximum peak load reached by the panel Inf_13 was 16% higher and occurred for an OOP displacement 8%. Smoother after-peak strength degradation was visible in the panel Inf_13 response. The ultimate load of the panel Inf_13 was 8% higher and occurred for a $d_{\text{OOP,ult}}$ 27% larger.

From the analysis of the force-displacement results of the panels subjected to previous IP damage (Inf_11 and Inf_12), shown in Figure 26d, the strengthening increased all the strengthening parameters, namely $F_{OOP,crack}$, $F_{OOP,max}$, and $F_{OOP,ult}$ about 1.94, 2.44 and 2.94 times higher, respectively. From this comparison, it is visible that the strengthening is more effective for larger OOP displacements. Through the analysis of the initial stiffness, the strengthened panel reached a value 6% lower. It is also important to say that, concerning the discussion of the OOP displacement parameters, the $d_{OOP,crack}$ and $d_{OOP,ult}$ were similar but the $d_{OOP,max}$ achieved by the specimen Inf_11 was 38% higher. To conclude, Figure 26e shows the force-displacement curves of the specimens Inf_12 and Inf_13 curves, both strengthened with solution B. From this comparison it is possible to assess the impact of prior damage in the strengthening performance. Through the observation of the force-displacement curves, it is possible to observe that the strength degradation due to the IP damage occurs for similar OOP displacement demands, which means that it corresponds to stiffness degradation. The cracking, maximum and ultimate strength reduced about 24%, 19% and 8%, respectively.

To analyse the efficiency of the strengthening solution A for larger displacements, it is mandatory the realization of an additional test until reached the maximum OOP displacement as possible (only limited by the pneumatic actuators capacity).

Figure 26 – Analysis of the strengthening techniques efficiency: force-displacement curves: a) Inf_08 vs Inf_10; b) Inf_08 vs Inf_13; c) Inf_10 vs Inf_13; d) Inf_11 vs Inf_12 and e) Inf_12 vs Inf_13.

5. FINAL REMARKS AND CONCLUSIONS

The major goal of this manuscript was the execution of a testing campaign to assess the efficiency of TRM strengthening solutions to prevent the collapse of infills OOP behaviour. An experimental campaign was carried out comprising by five nominally identical full-scale specimens were built. Two as-built specimens, were used as references, one with and one without prior damage caused by IP test. The remaining three specimens were strengthened using TRM technique. Two different solutions were tested by varying the type of mesh (strong and medium) and the anchorage (using and not using steel plates to distribute the mesh-frame anchorage. All the tests have been performed by imposing a half-cyclic (loading-unloading-reloading) history of displacements in the out of plane (OOP) direction by means of a uniform distributed load provided by small pneumatic jacks. From the testing campaign the following conclusions can be drawn:

- Both strengthening solutions revealed to be very efficient since they prevented the collapse of the infill panels. It was observed that the use of steel connectors for the mesh-frame anchorage was very effective since it was not observed any detachment or failure. The use of the steel plate improved the mesh performance by reducing the sliding failure of the mesh in the regions surrounding the steel connector;
- The use of the strong mesh did not presented better results in terms of strength than the observed with the medium strength mesh. Although, this analysis cannot be directly extracted because different anchorage solutions were adopted, still due to the large strength characteristics and the lower ductility of the strong mesh A, it was not totally explored as proved by the cracking pattern of the specimen Inf_10;
- The results of the strengthened specimens revealed that the TRM solutions improved about 1.75-2.44 times the maximum strength, 1.04-2.94 times the ultimate load, until 4 times higher initial stiffness, until 2.72 times the deformation capacity of the panel and

improved the energy dissipation capacity at least 2 times, when compared with the as-built specimens tested in similar conditions.

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