The use of FRPs in seismic repair and retrofit: Experimental verification

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ABSTRACT: The application of FRPs in the seismic repair and retrofit of structures is addressed. The results from a few tests on full-scale structures, repaired and/or retrofitted with composites, performed at the ELSA laboratory are presented and discussed.

1 INTRODUCTION

Developed some years ago for the aerospace industry, Fiber Reinforced Polymers (FRPs) have since then been used in a variety of other applications and are now emerging in civil engineering applications.

The use of fiber reinforced polymers in seismic repair and retrofit seems to be an economical alternative to traditional materials and technologies allowing addressing most of the deficiencies commonly found in non-seismic resisting structures. However, there is a lack of experimental evidence on the performance of structures and elements repaired and/or retrofitted with FRPs. Moreover, there is a lack of specific codified rules for design (re-design), which represents a major drawback and delays a much wider use and application of FRPs in seismic retrofit.

The ELSA laboratory has recently started repairing and retrofitting a few test models with carbon fiber polymers testing them subsequently to assess their performance under earthquake induced and quasi-static cyclic deformations.

In addition to reinforced concrete elements/structures, also other elements, such as load bearing masonry and masonry infill panels, can improve their performance by the application of new polymeric materials.

This paper addresses the issue of seismic repair and retrofit of existing structures and presents a few typical examples of full-scale models of buildings repaired and retrofitted with fiber composite materials. These models were subjected to earthquake input motions and their earthquake performance was compared to the performance of the original structures. It is anticipated that, in general, improved behaviours were achieved. However, it is recognized that design of the retrofitting schemes requires a sounder scientific and technical basis.

2 ISSUES AND CHALLENGES IN SEISMIC RETROFIT OF EXISTING STRUCTURES

World-wide experience from past earthquakes shows that non-ductile RC frame buildings and unreinforced masonry structures are the most vulnerable and represent overall the largest threat to human life and property in future earthquakes. Concrete and masonry are by far the most common structural materials in the European seismic areas. However, transport infrastructures, such as bridges, should also be taken into account, as well as the priceless European architectural heritage, which may suffer very heavy and/or irreversible damages from earthquakes.

2.1 *RC Buildings*

The vast majority of buildings in earthquake prone areas of Europe constructed before the 80’s are seismically deficient in light of our current knowledge. In fact, until the 60’s no specific seismic design provisions were included in the codes and, from that period on, only seismic equivalent lateral loading has been considered in their design. Provisions for design and detailing of members and structures resembling those of modern codes only appeared in Europe in the 80’s in the national codes (Fardis 1998).

Most of the existing buildings constitute a major source of risk to human life and property loss (see Figure 1) therefore, their retrofit or replacement should be made in order to reduce vulnerability and consequently risk to currently acceptable levels. The issue involves political, social and economic considerations but there are also scientific and technical aspects to be addressed. Providing new buildings with seismic resisting characteristics to is presently easy and inexpensive whereas the seismic rehabilitation of existing buildings involves considerable costs, including those of disruption of use, and poses several specific problems.

Figure 1. Failure of a building during the 1999 Turkish earthquakes (Elnashai 2000)

Concerning scientific aspects: first - there is a need for more rational assessment methods able to predict seismic response and to identify local deficiencies leading to failure; second - effective and economic retrofitting solutions and techniques should be found. As far as technical issues are concerned, there is still a lack of codified criteria for redesign as well as a specific code for assessment and redesign of seismic vulnerable buildings.

Eurocode 8 (EC8 2000) is deemed to be an advanced design code for new structures but its part 1.4, strengthening and repair, is cast into a form similar to that of new buildings and therefore redesign is performed as for a new structure. This approach is nowadays considered inappropriate (nor rational and nor practical). In fact, the requirements in terms of ductility (local and/or global) are not cost-effective making total replacement a more economical alternative.

The challenge to the scientific earthquake engineering community is to define appropriate criteria for retrofit of existing structures, to develop rational redesign methods and to investigate and assess innovative cost-effective strengthening solutions and techniques. Particular attention should be devoted to RC buildings because most of public critical facilities (schools, hospitals and local or state administrative services), as well as buildings with high rates and duration of occupancy (commercial and office buildings, hotels, etc.) belong to this class of buildings.

2.2 *Bridges*

The response of bridge structures to seismic events is of great importance since it is usually required that they remain fully operational after an earthquake.

Throughout Europe the bulk of existing bridges is not designed for seismic resistance. They are poorly detailed (no capacity design provisions, insufficient overlapping length of longitudinal reinforcement, lap-splices in the potential plastic hinge zones, ineffective confinement, transversal reinforcement not sufficiently anchored and lack of stirrups or crossties) and consequently, they have limited deformation capacity (Priestley et al. 1996).

There are no codified requirements in Europe (e.g. EC8) for existing bridge structures and components, particularly for hollow box cross sections commonly used for bridge piers (see Figure 2). Also, the seismic zonation maps of many European countries have been revised recently, prescribing now higher ground accelerations in several regions.

Finally, local soil conditions and the possibility of asynchronous motion at the base of the piers of long bridges are factors which can cause additional difficulties in properly designing bridges.

Regarding retrofitting, performance and cost relations are yet to be identified for conventional and advanced techniques, since few experimental results are currently available. There is therefore a need for reliable methods for assessing the seismic vulnerability of existing bridges, in particular large and irregular motorway bridges having lifeline character.

Figure 2. Bridge pier model (scale 1:2.5 - hollow box cross-section) of an existing highway-bridge in Austria tested at ELSA. Failure at 3.5 m from the base (Pinto & Tsionis 2001, Flesch et al. 2000)

2.3 *Masonry and Monumental structures*

Recent seismic events have confirmed that masonry structures are the most earthquake vulnerable constructions. In Europe a significant part of masonry structures are located in earthquake prone areas, namely in rural zones (Figure 3) and in historical parts of our cities. Moreover, the built cultural heritage (monumental structures and other patrimony) has suffered irreversible damages during recent earthquakes (e.g. Figure 4 (Croci 2000)).

Figure 3. Failure (collapse of a corner) of a masonry house during the Umbria-Marche earthquakes, September 1997, Italy

Masonry is a ‘Satanic’ material (Tassios, pers. comm.) in the sense that it’s very brittle with mechanical properties, which may vary significantly with minor construction differences, boundary conditions and aging. Therefore, any intervention leading to increased ductility as well as monolithic structural behavior will improve earthquake performance. In fact, past strengthening guidelines for masonry structures recommend in-plan rigid floors for a monolithic behaviour but this leads to higher stress (see Figure 3), which requires higher strength of the load-bearing masonry walls.

The possibility of an appropriate utilization of FRP materials for strengthening of historical masonry buildings and/or monuments offers several advantages that cannot be neglected in Southern European countries, which have a rich cultural heritage. The ‘competence’ required for acting on valued masonry structures (historic, architectonic and engineering aspects) underlines the importance of research on technological and mechanical aspects of the applications and requires full collaboration between the different actors.

Figure 4. Repair and strengthening works at the Basilica of St Francis of Assisi (Tympanum) after the 1997 earthquakes, Italy

2.4 *The Potential of advanced materials in seismic retrofit*

It was already stated that seismic retrofit of existing constructions is a complex and difficult issue involving political, social, economic, technical and scientific aspects. In particular, it was stated that -the challenge to the scientific earthquake engineering community is to define appropriate criteria for retrofit of existing structures, to develop rational redesign methods and to investigate and assess innovative cost-effective strengthening solutions and techniques.

Developed some years ago for the aerospace industry, fiber-reinforced polymers (FRPs) have been used in a variety of other applications and are now emerging in civil engineering applications.

Concerning retrofitting of structures, the material and its application are very attractive because they are light-weight, no heavy equipment is required, they can be installed with limited preparation in close spaces, and they require minimal access and disruption and occupant relocations are minimized. Furthermore, they are resistant to corrosion and virtually maintenance-free; composites can be covered with decorative coatings or finishes without concern for future access.

In contrast, there is little experimental evidence on the performance of structures and elements repaired and/or retrofitted with FRPs. Apart from a few cases in USA of RC bridge column jackets, which performed well during the recent earthquakes and the research conducted at UBC, San Diego (Seible et al. 1995) as well as in other few US, Japanese and European universities and research institutions (Meier and Deuring 1991, Meier and Kaiser 1991), there is no widespread research activity on the subject. Furthermore, numerical modelling of the combination (‘wedding’) of existing and FRP materials did not yet reach a confident stage.

It is recalled that tradition in civil engineering design and construction is still an important factor (current RC design and detailing practice is cast on a long period of research activity and lessons from earthquakes), which makes difficult the introduction of new materials and technologies.

Moreover, there is a quasi-complete lack of codified rules for design (re-design) of structures using FRPs retrofit materials. This constitutes a major drawback and inhibits a much wider use and application of FRPs in seismic retrofit.

The ELSA laboratory has recently started repairing and retrofitting a few test models of buildings with carbon fiber polymers testing them subsequently to assess their performance under earthquake induced and quasi-static cyclic deformations. A few typical examples are presented.

3 SEISMIC REHABILITATION OF REINFORCED CONCRETE BUILDINGS BY MEANS OF CARBON FIBRE REINFORCED POLYMERS

Two experimental activities have been conducted on two reinforced concrete buildings with the aim of investigating the effectiveness of the adoption of Carbon Fibre Reinforced Polymers (CFRPs) in improving the seismic behaviour of frames. In particular, the first activity was concentrated on the increase of the global ductility of a non-ductile building, whereas with the second experimental campaign CFRPs have been used to repair the damage sustained during previous tests.

3.1 *Seismic rehabilitation of a non-ductile reinforced concrete frame*

An experimental activity was conducted on a nearly full-scale (2:3) reinforced concrete building. The building, a two-storey non-ductile framed structure, was designed as representative of old construction lacking seismic provisions. At the end of the experimental campaign it was decided to take advantage of the availability of a non-ductile frame to study the effectiveness of the adoption of Carbon Fibre Reinforced Polymers as a rehabilitation technique (Castellani et al. 2000). Two objectives have been pursued in performing the intervention: 1) to recover the original stiffness of the frame, 2) to increase the global ductility capacity of the building.

The goal of increasing the global stiffness of the building was pursued by means of the insertion of carbon fibre strips. The role played by the strips was to increase the stiffness of the columns. For this reason, they were glued on the two sides of the columns perpendicular to the direction of the test.

The attempt to increase the global ductility capacity of the frame had to take into account the possibility of the formation of column-sideways mechanism as a consequence of the violation of the capacity design checks in the design. Being the rationale of the intervention to leave the structure with its original dimensions and to avoid the use of pneumatic hammer or other mechanical means, the activity included positioning unidirectionally oriented carbon fibre mats at all member ends. These were expected to increase the local ductility capacity in the member ends –mainly those of the columns- by means of the confinement effect exerted by the carbon fibre mats. In fact, the action accomplished by the membrane was to hold the layers of cover in their position, preventing them from spalling apart. The intervention is schematically shown in Figure 5.

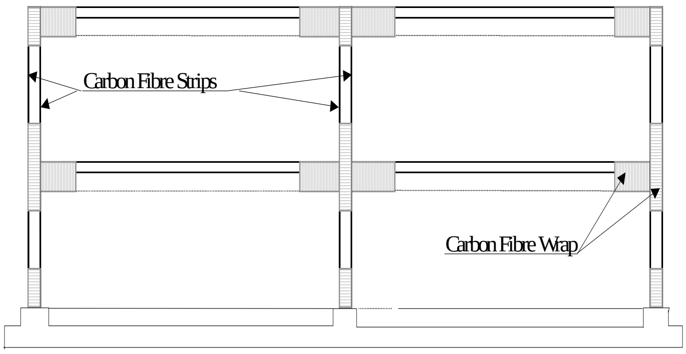


Figure 5. General layout of the REEDS frame location and type of strengthening

3.1.1 *Seismic tests*

After the intervention, the frame was subjected to the same input motion applied during the previous experimental campaign. During and after the repetition of the seismic excitation no further cracks were detected. Besides, the cracks that opened on the bare frame did not reopen during the second earthquake simulation.

The comparison between the natural frequencies of the virgin frame and those of the frame after the rehabilitation demonstrated that more than 60% of the loss in stiffness was recovered with the intervention.

The analysis of the energy dissipation capacity of the frame before and after the intervention showed that the adoption of this rehabilitation technique seems to avoid the concentration of damage. In fact, the effect of the CFRP on the behaviour of the frame was to uniformly distribute the storey energy dissipation contribution.

3.1.2 *Final cyclic test*

The experimental programme was concluded by performing a cyclic test, with the aim of deriving information about the failure mechanism of the frame and its ultimate capacity. The test strategy was to control the displacement at the second floor (roof) of the frame, while holding an inverted-triangular force-distribution pattern in the stronger frame. Additionally, the rotation of the storeys about the vertical axis was constrained. The cyclic history consisted of a set of cycles at prescribed ductility levels (the reference value being the displacement corresponding to the first yielding in the push-over analysis), whose level was increased up to 6.5, corresponding to a maximum displacement of ± 245 mm.

As a frame of low ductility, the building was characterised by a behaviour factor of 2.5. In spite of this value, the structure was able to sustain a maximum top displacement equivalent to a global ductility level of 6.5 without any major strength decay (see shear drift diagram in Figure 6).

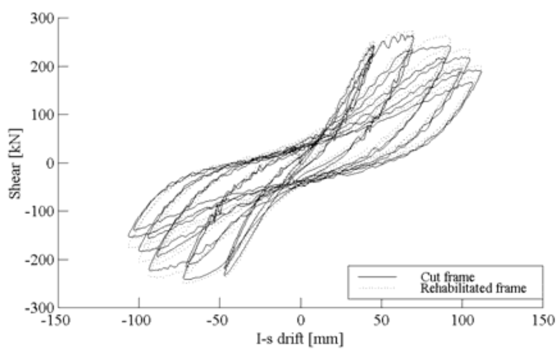


Figure 6. Storey shear vs. interstorey-drift at first storey for the cut (after cutting the fiber strips) frame and the rehabilitated frame with composite material

4 REPAIR OF A REINFORCED CONCRETE STRUCTURE SUBJECTED TO EARTHQUAKE SIMULATIONS

A full-scale reinforced concrete dual structure (Figure 7) was recently subjected to earthquake simulations at the ELSA Laboratory (Tsionis et al. 2001). The building consisted of two frames; one was designed according to Eurocode 8 and the other according to an innovative deformation-based design philosophy.

The structure was pseudodynamically subjected to an earthquake corresponding to the design intensity. The test was then repeated with the input increased by a factor of 1.5. During the second test, the building suffered extensive damage. Horizontal cracks typical of flexural failure were observed in the EC8 side of the building, whereas diagonal shear cracks appeared in the other part.

The structure was then used to investigate the effectiveness of CFRP as a repairing technique. After the injection of all cracks with epoxy resin, monodirectional carbon fibre wraps have been glued around all the member (beams and columns) ends (see Figure 9). The shear walls have been wrapped throughout the height of the building (Figure 8). Being the damage in the two frames of the structure caused by different phenomena, it has been decided to intervene in a different way in the walls of the two frames. In particular, four-directionally-oriented wraps have been applied to the shear walls of the deformation-based designed frame in order to recover the loss in shear resistance. In the other frame, the intervention has been limited to the increase of the confinement effects by using monodirectional oriented fibre wraps.

After the intervention, the frame was subjected to the same input motion applied during the previous experimental campaign.

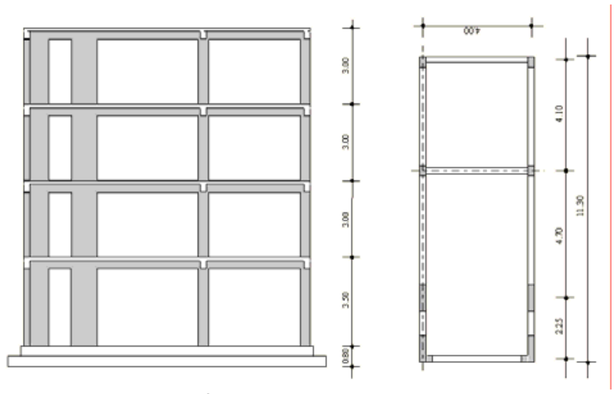
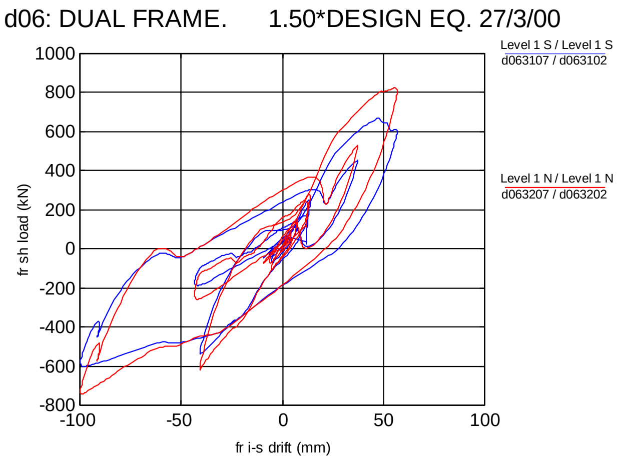


Figure 7. Layout of the dual structure elevation and pan-view)

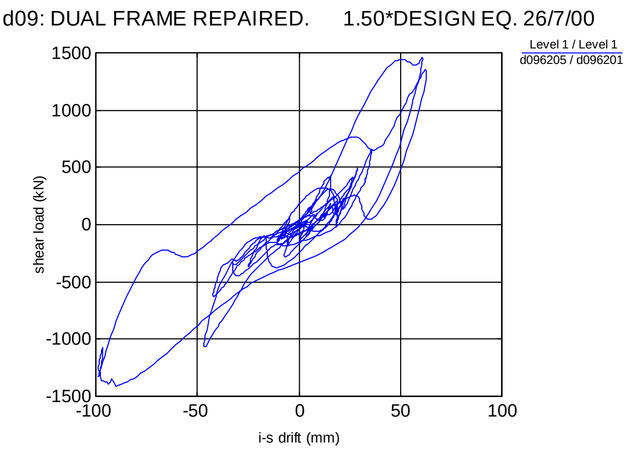
Figure 8. Earthquake tests at ELSA. Experimental set-up

Figure 9. Member ends wrapped with CFRP

A preliminary analysis of the results seems to demonstrate that, thanks to the intervention, the loss in strength caused by the damage suffered during the previous tests was recovered. In addition, the capacity of the frame to behave in a ductile fashion appears improved (see shear-drift diagrams in Figure 10). The application of a quasi-isotropic wrap on the shear walls resulted in a local modification of the cinematic behaviour of the walls. The energy dissipation mechanism passed from the opening/closing of shear cracks to a rocking movement around the base of the walls.



a)



b)

Figure 10. First storey shear-drift diagrams for the high-level earthquake test for: a) the two original frames, b) for the repaired and strengthened frame

5 IMPROVING DUCTILITY AND ENERGY DISSIPATION CAPACITY OF MASONRY PANELS BY MEANS OF POLYMERIC NETS

The importance of the development of strengthening/repair techniques based on the adoption of these materials is highlighted by the fact that in many highly seismic prone countries in southern Europe, a commonly used structural system consists of reinforced concrete frames with masonry infill panels. Many researchers have already demonstrated that, even though infills are regarded as non-structural elements, they can significantly modify the seismic response of the structure (e.g. Mainstone 1971, Bertero & Broken 1983, Zarnic & Tomazevic 1985, Negro & Verzeletti 1996). One of the effects of the presence of the panels is the increased capability of the building to dissipate energy. By increasing the capacity of the infills to dissipate energy, the structure as a whole could become less vulnerable with respect to earthquakes.

A research program, funded by the European Commission, aimed at investigating the possibility to increase the intrinsic ductility and energy dissipation capacity of panels, has been recently concluded at the European Laboratory for Structural Assessment (ELSA) of the European Commission. The problem has been focussed on the possibility of confining the panels by using polymeric grids.

The effectiveness of this solution was investigated by performing tests on differing patterns of infills (Colombo et al. 2000). Two infill layouts with dimensions of 4.6m x 2.6m, one without openings and the other with non-symmetric openings (a door and a window) have been analysed. The reinforcement consisted in the adoption of a polymeric net characterised by a 40x40mm ribbed mesh. The plastic material had strength of 30kN/m, determined in accordance with BS 6906. The scheme adopted for the panel with openings is reported in Figure 11.

Figure 11. Application of the plastic nets for the wall with openings

In the following paragraphs, the behaviour of the panels is discussed in some detail. The differences in the performance of the panels with and without polymeric nets have been identified by analysing the envelope curve of the hysteresis loops expressed in terms of shear force vs. storey drift.

5.1 *Solid panels*

The envelope curves extrapolated from the hysteresis loops of the panels without openings showed that the application of the displacement cycles of small amplitude (corresponding rotation smaller that 0.002 rad) resulted in substantially similar behaviour in the two panels. By increasing the amplitude of the load beyond this level, the maximum shear strength of the conventional panel was very soon reached.

Whereas the rotations corresponding to the ultimate strength for the plain and reinforced panels are quite close, the maximum force of the strengthened panel resulted 12% larger than the one of the conventional panel. Both panels were characterised by a non-symmetrical response in the two loading directions, thus confirming that the behaviour of the masonry panels is strongly influenced by the previously suffered deformations (Zarnic, 1994). The most important effect of the confinement obtained by the insertion of the plastic net resulted to be the change of the slope of the softening branch of the curve. Plain walls are usually characterised by a large drop in the shear strength. By using the polymeric grids, this drop is strongly reduced. This result is of particular importance as for the ability of the panels to dissipate a larger amount of energy.

5.2 *Panels with openings*

The difference in the responses of the plain and the confined panels was much more significant for the case of panels with non-symmetric openings (Figure 12). The unreinforced infill totally collapsed at a displacement corresponding to a storey rotation of about 0.03 rad. At this level of deformation, the panel with the plastic grid, even though dramatically damaged, was still able to provide 65% of its maximum strength. The effects of the confinement accomplished by the insertion of the net resulted in a significant shift of the yielding point up to larger forces (the difference on the shear resistance was about 40%) and larger rotations (from 0.006 to 0.015). This means that by the adoption of the proposed methodology, the panel could be able to dissipate an important amount of energy without major strength decay up to large storey rotations. This level of deformation is close to the limit fixed by the Eurocode 8 for the check of the serviceability Limit State. As a result of the capability of the panels to dissipate such a large amount of energy, these elements could act as dissipation devices, strongly reducing the damage in the structural elements (Colombo et al., 2000).

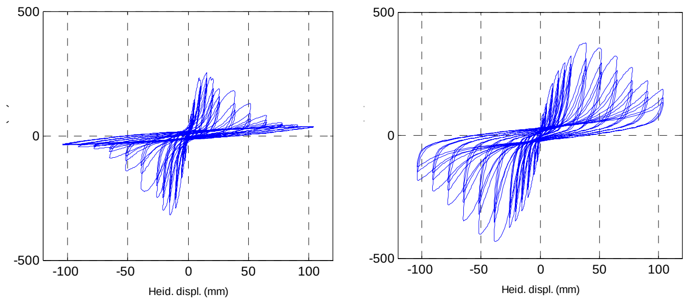


Figure 12. Panels with openings: Shear (kN)-drift (mm) diagrams for plain (left) and reinforced (right) panels

6 REPAIR AND RETROFIT OF DAMAGED COLUMN/JOINTS IN A RC FRAME

The research programme of the network ICONS, Topic 2 - Assessment, Strengthening and Repair, addresses the issues of seismic assessment and retrofit of existing structures covering several aspects of the problem. In particular, are considered the assessment of reinforced concrete buildings with and without infill panels and several strengthening solutions and techniques are investigated through analytical and experimental work.

The experimental work included several studies carried out at European shaking-table laboratories as well as the experimental research performed at the ELSA laboratory, which comprised several earthquake tests on two full-scale models of a 4-storey, three bay, RC frame representative of existing buildings constructed in Southern European countries before the 80’s (see general layout in Figure 13).

Two similar frames, representative of "non-seismic" designs were constructed allowing for a vast testing campaign, which includes: assessment of bare and infilled frames, repair and strengthening.

The testing campaign comprised several pseudo-dynamic tests on the bare, infilled and retrofitted frames for several earthquake intensity levels. In order to assess the ultimate capacity of the Bare Frame (BF) and of the Selective Repaired Frame (SR), a final collapse (capacity) test was foreseen. However, the local damages inflicted on the structures inhibited these final collapse tests, which required repair of the local damages as well as strengthening using carbon reinforced composites.

6.1 *Damage-state after Earthquake Tests and Repair/strengthening of ASR Frames*

The tests performed on the frames with masonry infill walls led to severe local damage (shearing-off by the infill panels) of a few columns (top part) with lateral dislocations of the external columns of 50 *mm* (~25% of the column cross-section characteristic height) (see Figure 14). Relocation, repair and strengthening of the column/joint parts were required to carry out the final collapse tests.

The preparation and execution of the repair/strengthening works for six columns/joints and for one column was carried-out in four working days; the time elapsed being a period of 2 weeks between the application of the first repair products and when the final operation was carried out. They comprised: a) Removal of concrete at the top of the damaged column (20 *cm*); b) Positioning of columns to its original vertical position; c) Concreting of the external heavy damaged joints and column (1st storey, strong column) (see Figure 15); d) Injection of cracks in beams (with epoxy resin); e) Roughening of the external superficies (column and joints) for application of carbon fiber system; f) External carbon fiber repair system of the heavy damaged joints (see Figure 15).

6.1.1 *Crack Injection and FRPs Application*

Most of the large cracks in critical zones were injected with epoxy resin. It should be underlined that many other zones would require light interventions (mainly crack injections), but they were considered not particularly relevant for the tests to be carried out (the final collapse tests).

A single layer of carbon fiber was used in all strengthening interventions. Furthermore, the carbon fiber fabrics are unidirectional, which requires control during their application. In fact, they must be oriented along the direction requiring strengthening. For required bi-directional strengthening two superposed carbon orthogonally oriented fiber fabrics must be used and, for shear-controlled situations, inclined carbon fiber fabrics are required, unless multi-oriented fabrics are used.

Column - the first floor strong column suffered heavy damage at the base and top extremities. Shear failure at the bars termination zone (0.70 *m* from the column base) was particularly evident. Therefore, it was necessary to increase strength and to provide sufficient confinement in the critical zones, including the bars termination zone. According to these requirements, confinement was continuously provided from the base up to 0.90 *m*, and the top column confinement was limited to 0.30 *m*.

Joints - As already explained, a few joints were seriously damaged and dislocated from their original positions due to the cut-off effect caused by the infill panels. They were forced back to their original position and partially reconstructed. It is however known that reconstruction processes do not provide lateral resistance for horizontal or even vertical loads. It was therefore necessary to provide confinement to the joint and anchoring top-columns to the adjacent beams, which was provided by means of the carbon fiber fabrics. Anchoring of the column to the internal beam is somewhat disregarded, but it is too difficult to perform (due to the geometry of the joint).

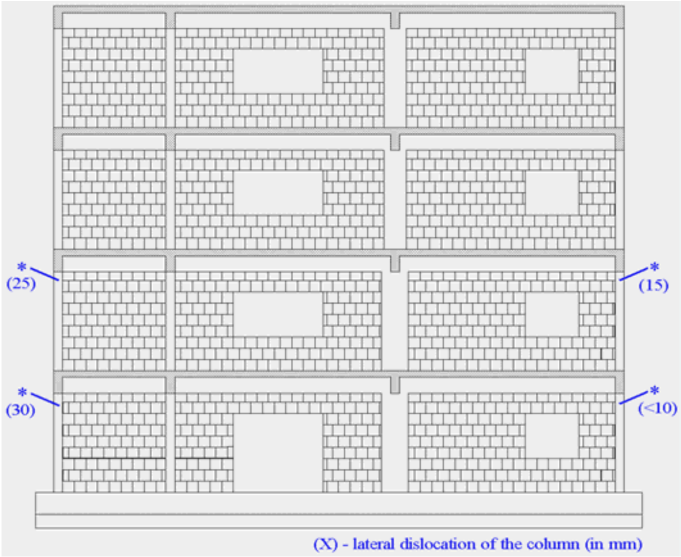


Figure 13. Layout of the RC frame with indication of the damaged columns/joints

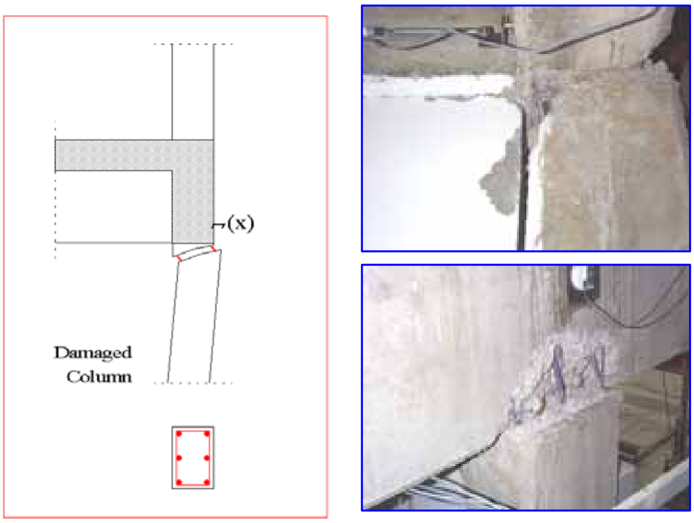


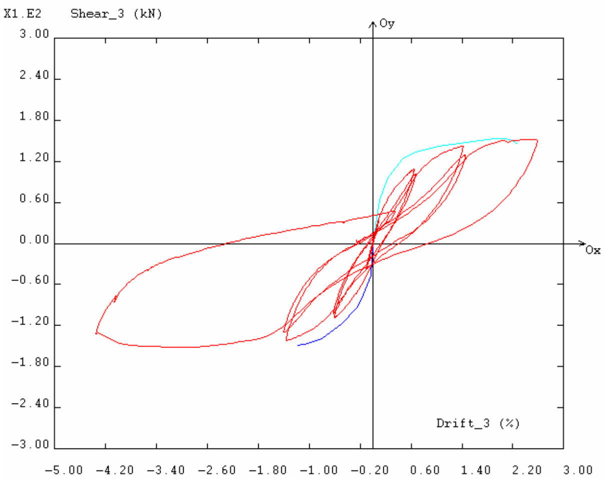
Figure 14. Failure (shear-out) of the column/joint interface: Schematic representation (left), photographic documentation (right)

Figure 15. Reconstruction of the column/joint and strengthening with carbon fiber fabrics with epoxy based impregnation resin

6.2 *Final Collapse Tests*

After repair and strengthening, the frames were subjected to a final collapse test consisting of an imposed top displacement cycles with increasing amplitude and assuming a triangular inverted force distribution. The shear-drift diagrams for the second and third storeys are give in Figure 16 together with the envelope of the earthquake response obtained from the earthquake tests.

It is noted that the repaired/strengthened frame was able to withstand storey deformations higher than the ones reached in the earthquake tests (these tests led to local failure of the column/joints) maintaining its load carrying capacity.



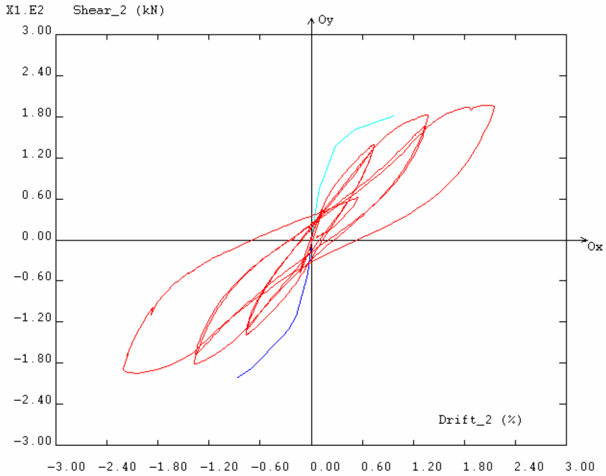


Figure 16. Shear-drift diagrams for the RC frame: 1) repaired/retrofitted frame - cyclic deformations; 2) envelope curves from the earthquake tests, which led to local column/joint failure

7 CONCLUSIONS

The results from the tests performed at ELSA on full-scale structures repaired and strengthened with FRPs demonstrate that by using advanced composites it is possible to significantly improve the seismic behaviour/performance of vulnerable structures.

It has been shown that the application of carbon fibre wraps on the critical regions of damaged structural elements can recover an important part of the loss in stiffness and strength of those members. In addition, improvement in energy dissipation capacity and the global ductility of the structure can be achieved.

The results of the tests presented in the paper seem to demonstrate the effectiveness of the adoption of the plastic nets in increasing the energy dissipation capacity of masonry infill panels. The comparison between the behaviour of the tested panels showed that the adoption of this methodology could significantly modify the strength-decay characteristics of the panels. In particular, it was observed that the severe drop characterising the softening branch of the strength-deformation relationship for traditional infills is strongly reduced by the application of the plastic net. This modification may allow for a larger amount of energy to be dissipated. For the case of panels with openings, the proposed provisions also result in a shift of the ultimate shear strength towards larger forces and deformations. It is however noted that reinforcement of infills panels in frame structures may lead to local premature failures of the surrounding columns if no appropriate dowels are provided to transfer the forces developed in the panels directly to the adjacent girders.

It should be noted that specialized companies, producers of composite materials or their associates carried out most of the repair strengthening works using fiber composite materials. Technical characteristics of the products, including nominal values for "adherence" between fabrics and concrete are not systematically made available, which could be a commercial strategy or a consequence of lacking of fundamental reliable data for civil engineering applications.

This fact creates serious problems for the widespread use of these materials in repair and strengthening of existing structures. Urgent actions should be taken by universities and technical communities to teach and update technical background in the field. Furthermore, appropriate norms and design guidelines for strengthening of exiting structures with FRPs should be developed.

ACKNOWLEDGEMENTS

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