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Effectiveness of some conventional seismic retrofitting techniques for bare and infilled R/C frames

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Abstract. The effectiveness of a technique for the repair of reinforced concrete members in combination with a technique for the repair of masonry walls of infilled frames, damaged due to cyclic loading, is experimentally investigated. Three single - story, one - bay, 1/3 - scale frame specimens are tested under cyclic horizontal loading, up to a drift level of 4%. One bare frame and two infilled frames with weak and strong infills, respectively, have been tasted. Specimens have spirals as shear reinforcement. The applied repair technique is mainly based on the use of thin epoxy resin infused under pressure into the crack system of the damaged RC joint bodies, the use of a polymer modified cement mortar with or without a fiberglass reinforcing mesh for the damaged infill masonry walls and the use of CFRP plates to the surfaces of the damaged structural RC members, as external reinforcement. Specimens after repair, were retested in the same way. Conclusions concerning the effectiveness of the applied repair technique, based on maximum cycles load, loading stiffness, and hysteretic energy absorption capabilities of the tested specimens, are drawn and commented upon.

Keywords: infilled RC frames; masonry strength; repair technique; experimental results

1. Introduction

In trying to address the issue of the effectiveness of any repair technique of masonry infilled RC frames, it is essential to bear in mind that the behavior of masonry infilled RC frames involves the influence of complex interactive phenomena such as the interaction of infills with the bounding frame, the shear, bond, confinement, fatigue of reinforced concrete connections which, though independently are well understood, there is not universal consensus about models (Kakaletsis 2007). On the other hand, although nowadays modern retrofitting techniques have been developed using bracing-friction damper systems or using other added viscous damping (Lee *et al.* 2008, Lavan and Levy 2010), it has been proved that retrofitting infill walls, damaged during a credible earthquake, could further improve the favorable contribution of infills to the overall seismic behavior of the vulnerable RC buildings when walls are placed regularly throughout the structure and/or they do not cause shear failure of columns (Dolsek and Faifar 2008, Mosallam *et al.* 1998). However, since

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unanswered questions exist even for the behavior of undamaged infilled RC frames designed to withstand seismic excitations, it is reasonable that there are many uncertainties for the behavior of the repaired ones. Additional questions concerning the effectiveness of generally applied reinforced concrete joints and masonry repair techniques also arise from the existing variety of masonry strength and shear reinforcement design practices. In spite of all the uncertainties and based on the justified idea that a sub assemblage can be adequately designed or repaired to withstand a loading even without full knowledge of all influencing parameters, some of the repair procedures covering most usually applied practices are experimentally investigated.

Epoxy injections can be applied for the repair of damaged joints with slight to moderate cracks without damaged concrete or bent, or failed reinforcement. However, the restoration of the bond between the reinforcement and the concrete by injections is inadequate and unreliable (UNIDO 1983, Eurocode 8 2005). Some results of reverse cyclic loading tests performed on exterior beam-column joints indicate that the epoxy injection technique is an efficient repair procedure for moderate earthquake damage (Karayannis *et al.* 1998, Engindeniz *et al.* 2005). By this technique, the increases in peak load and dissipated energy were 8 to 40% and 53 to 139%, respectively. The change in stiffness varied between a 27% decrease and a 10% increase. Despite the fact that so far only a few research works have been published of the effectiveness of this technique, the technique has been extensively applied after past earthquakes in Greece, for the repair of reinforced concrete elements and beam-column joints with low or moderate level damage. In this study, thin resin solution was infused under pressure in the damaged joint body.

Common techniques for retrofitting of masonry so far (ElGawady *et al*. 2004), include the construction of jackets made of steel reinforced shot concrete, external post tensioning with steel ties and strips made of carbon respectably E-glass fiber reinforced polymer or steel glued and potentially mechanically anchored on the wall's surface. Especially, the technique of surface treatment was able to improve the in plane resistance of masonry walls by a factor of 1.25-3. Even though the results are to some extent quite promising, each of these techniques suffers from diverse disadvantages. Heavy jackets add considerable mass to the structure resulting in increased dynamic loads, may cause shear failure of the frame members, often a large amount of work input is required and the long term and environmental behavior of some materials are debatable, e.g., steel corrosion (Perera *et al*. 2004). Furthermore, due to its low tensile strength, masonry is very sensitive regarding stress concentrations caused by surface-glued strips yielding to brittle failure (Antoniades *et al.* 2005). Therefore reliable, cost effective and applicable repairing methods for having cheaper rehabilitation alternatives to retrofit masonry have to be experimentally evaluated (Erdem *et al*. 2006). In this study, alternatively, a polymer modified cement mortar for the damaged infill masonry walls was used.

The strengthening of existing RC flexural members by external FRP laminates is one of the most widely adopted solution for supplying additional external tensile reinforcement. Applications regard shear and flexural reinforcement, confinement of columns and joints (Karayannis and Sirkelis 2008, Antonopoulos and Triantafillou 2003). Relative low FRP area fractions increased both strength and cumulative dissipated energy up to about 70-80%. The increase in stiffness varied with the imposed displacement level and reached values in the orders of 100%. Some experimental investigations have been conducted, also, on full scale structure on the use of composite materials for controlling the type of plastic collapse mechanism of existing reinforced concrete structures (Della Corte *et al.* 2006). National and international code indications are now available for the design of elements externally bonded with FRP (ACI Committee 440.2R 2002, fib Bulletin 14 2001, Japan Society of

Civil Engineers 1997). However, there is still a lack of information and indications about performances of strengthened members in terms of ductility and strength behavior for cyclic load histories, that are both topic aspects for seismic retrofit. In this study, RC joints were externally retrofitted with carbon FRP sheets.

There are, also, a number of papers that cover the specific subject of bare and infilled concrete frames and retrofitting techniques. They presents PsD tests with analytical evaluations that have been conducted in the last years on full size or 1/3 scaled RC structures by using innovative and traditional retrofitting techniques (Di Ludovico *et al*. 2008a, Di Ludovico *et al*. 2008b, Ozkaynak *et al*. 2011). A significant seismic performance enhancement in examined frames was determined in terms of inter story drift, lateral load capacity, energy dissipation capacity, stiffness and the observed damages. Furthermore, a few researchers carried out experimental investigations on RC infilled frames developing seismic retrofit strategies involving the use of different FRP orientations on the masonry wall elements that were integrated to the boundary frame members by means of FRP anchors (Almusallam and Al-Salloum 2007, Binici *et al*. 2007, Yuksel *et al*. 2009).

The purpose of this paper is to investigate experimentally the effectiveness of the application some of the above more conventional techniques for the repair of the masonry infilled RC frames, damaged under cycling loading. Special attention is given to the examination of the repair efficiency with reference to the strength of the infill walls. Three RC frames, one bare and two constructed with different strength of the infills that were tested in cyclic loading, have been repaired and retested in the same way. Both qualitative and quantitative conclusions, based on the observed maximum loads, loading and reloading stiffness and hysteretic energy absorption are presented. Apparently, not only the repair process and materials, but also the type and degree of the original damage, introduce significant additional uncertainty in any attempt to compare the findings of the present work with the results of similar work from other researchers. Taking into account all the involved uncertainties, the scale effects and the inadequate number of samples for each specimen, it has to be emphasized that the experimental results of the presented work and the yielded conclusions are mainly limited to the study cases and must be used and extrapolated carefully and cautiously.

2. Significance and objectives of the research

The repair and upgrading of reinforced concrete structures damaged by seismic actions are relatively new and challenging fields of study in earthquake engineering. Since the philosophy of modern seismic design codes (Eurocode 8 2005) is based on a specific acceptable degree of structural damage, even in the event of the design earthquake, redesign-repairing structures designed to these codes and subsequently damaged, constitute a requisite part of the conceptual aim of the entire design process for seismic safety. Research in this area is essential, since engineers in seismic regions often face the dilemma of either analyzing and designing repair, or strengthening works of damaged building without quantitative guidance. The applied nature and financial importance of this research field is therefore apparent and its application immediate. To help in this direction, the experimental investigation reported herein is aimed to evaluate the effectiveness of some practical retrofitting techniques.

3. Experimental program for the original specimens

3.1 Test specimens

The experimental program as shown in Table 1 consisted of testing three single-story, one-bay, 1/3-scale specimens of reinforced concrete frames. Specimen BS was bare frame. Specimen SS was infilled frame with a weak infill, without openings, of clay bricks. Specimen ISS was infilled frame with a strong infill, without openings, of vitrified ceramic bricks. Specimens had transverse steel in the form of continuous rectangular spiral reinforcement. The geometric characteristics of the RC frames were the same for all specimens. The elevation, the corresponding cross-sections of the members and the design details for the RC frame specimens are shown in Figs. 1(a), (b). Specimen dimensions correspond to one third (1/3) scale of the prototype frame sections, 300 × 600 mm for the beam and 450 × 450 mm for the column. The column had closer spirals throughout the length, because the entire length of columns that are in contact with infills on one side per vertical plane are subjected to the special detailing and confinement requirements applying to critical regions. The beam had more shear reinforcement in the critical regions. Each beam-to-column joint had five spirals to prohibit brittle shear failure. The longitudinal reinforcement diameter Φ5.60 mm and stirrups diameter Φ3 mm of the frame members corresponds to one third (1/3) scale of Φ18 mm and Φ8 mm reinforcement diameters respectively of the prototype frame. The transverse reinforcement in the critical regions of specimens satisfied the requirements of the Greek standards, $s_h = 34$ mm $\approx s_{h(\mathrm{required})} = 33.3$ mm for the columns and $s_h = 40$ mm $< s_{h(\mathrm{required})} = 56$ mm for the beams and corresponds to one third (1/3) scale of $s_h = 100$ mm $\approx s_{h(\mathrm{required})}$ for the columns and $s_h = 120$ mm $< s_{h(\mathrm{required})} = 180$ cm for the beams of the prototype frame. The average sum of the flexural capacity of column to that of beam was also confirmed by the currently used codes and standards in Greece, which are very similar to EC2 and EC8. Thus, for the beam-column connections examined in this investigation, the ratio was 1.50, greater than 1.40. The purpose was to push the formation of plastic hinge in the beams. At the specimens, low strength plain bars were used, although the rule for the construction practice is to use high strength deformed bars. The frames were cast in one piece, on their side, all at the same time and from the same batch of concrete. Masonry infills had a height/length ratio $H/l = 1/1.5$ and were constructed with two selected brick types cut into two halves for complete simulation to the test scale. Their configuration is shown in Fig. 1(c). The former "weak" common clay brick usually used in Greece had a thickness 60 mm, while the latter "strong" vitrified ceramic brick that felt to be important for the specimen behavior had a thickness 52 mm. A representative mortar mix was used for the two types of infills contained the portions 1:1:6 (cement: lime: sand) and produced mechanical properties similar to type M1 mortar according to EN 998-2 (2001) standard. The experimental

Table 1 Test specimens

Specimen	Masonry type		
	Bare frame	Weak	Strong
BS	■		
SS		■	
ISS			■

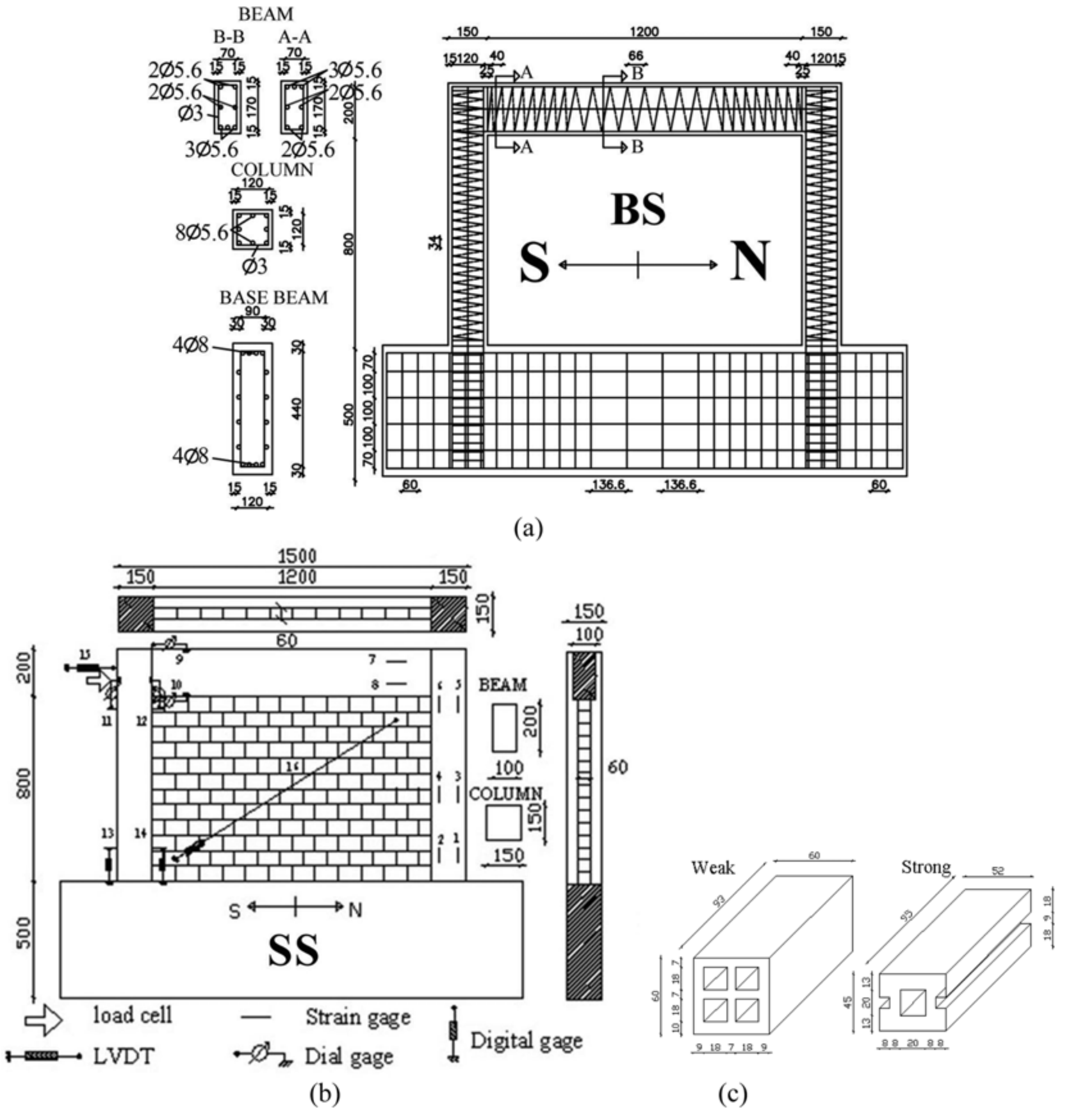


Fig. 1 Description of infilled frame specimens (a) Reinforcement detailing of the RC frame models (mm), (b) infilled frame and instrumentation (mm), (c) weak and strong brick units (mm)

brick units corresponds to one third (1/3) scale of the prototype brick unit with dimensions 180 × 180 × 300 mm, which is used in exterior partition walls. The mortar joints were not scaled. It must be mentioned that scaling problems did not permit a complete simulation of the materials usually used, although it is well known that the materials which are commonly used to fill RC frames (bricks and blocks) are very sensitive to scale effects. Masonry properties were chosen in such a way to produce the desired lateral strength of the two types in a magnitude $V_{w,u}$ = 27.36 or 25.58 kN lower than that of the lateral strength of the frame F_f = 42.48 kN as presented in the following paragraph. This closely represent actual construction in Greece and shows a rather successful scaling attempt.

Table 2 Mechanical properties of the materials used (MPa)

Material properties		Masonry type	
		Weak t = 6 cm	Strong t = 5.2 cm
MORTAR			
Compressive Strength	f_m	1.53	1.75
BRICK UNITS			
Compressive Strength	f_{bc}	3.1	26.4
MASONRY			
Compressive Strength ⊥ to hollows	f_{wc}	2.63	15.18
Elastic Modulus ⊥ to hollows	E	660.66	2837.14
Compressive strength // to hollows	f_{wc90}	5.11	17.68
Elastic Modulus // to hollows	E_{90}	670.3	540.19
Shear Modulus	G	259.39	351.37
Shear Strength without normal stress	f_{vo}	0.08	0.12
Shear Strength with normal stress	f_v / f_n†	0.38/0.25*	0.41/0.27*
		0.33/0.22	0.26/0.17
		0.39/0.30	0.60/0.61
		0.21/0.37	0.39/0.72
		0.20/0.73	0.41/1.55

†On masonry panels of length l and height H: $f_v / f_n = l / H$ = 1.50*, 1.50, 1.00, 0.58, 0.27 respectively
*On full size infills, l/H = 120 cm/80 cm = 1.50

3.2 Material properties

Material tests were conducted on concrete, reinforcing steel and masonry samples. The mean cubic compressive strength of the frame concrete was 28.51 MPa. The yield stress of longitudinal and transverse steel was 390.47 and 212.2 MPa respectively. The main results of mortar, bricks and infill masonry tests are presented in Table 2. The relationship of the shear strength of the bed joints f_v at a particular precompressive stress, f_n, versus the precompressive stress, f_n, derived from the cohesion tests and the diagonal compression tests of masonry panels with various length, l, to height, H, ratios and full size panels as well ($f_v/f_n = l/H$) is presented in Table 2. It can be noted from the table that the compressive strength of the "weak" masonry prisms was considerably lower than those of the "strong" while the shear strength of the bed joints in the "weak" and "strong" specimens with the same to the full size infills length/height ratio ($l/H = f_v/f_n$ = 1.5/1) was almost identical.

3.3 Test setup and instrumentation

The test setup is shown in Fig. 2. The lateral load was applied by means of a double action hydraulic actuator. The vertical loads were exerted by manually controlled hydraulic jacks that were tensioning four strands at the top of the column whose forces were maintained constant during each test. The top and bottom connections used during the tests for the application of the vertical load to the columns is presented in Fig. 2(b). The bottom connection for the strands consisted of a steel

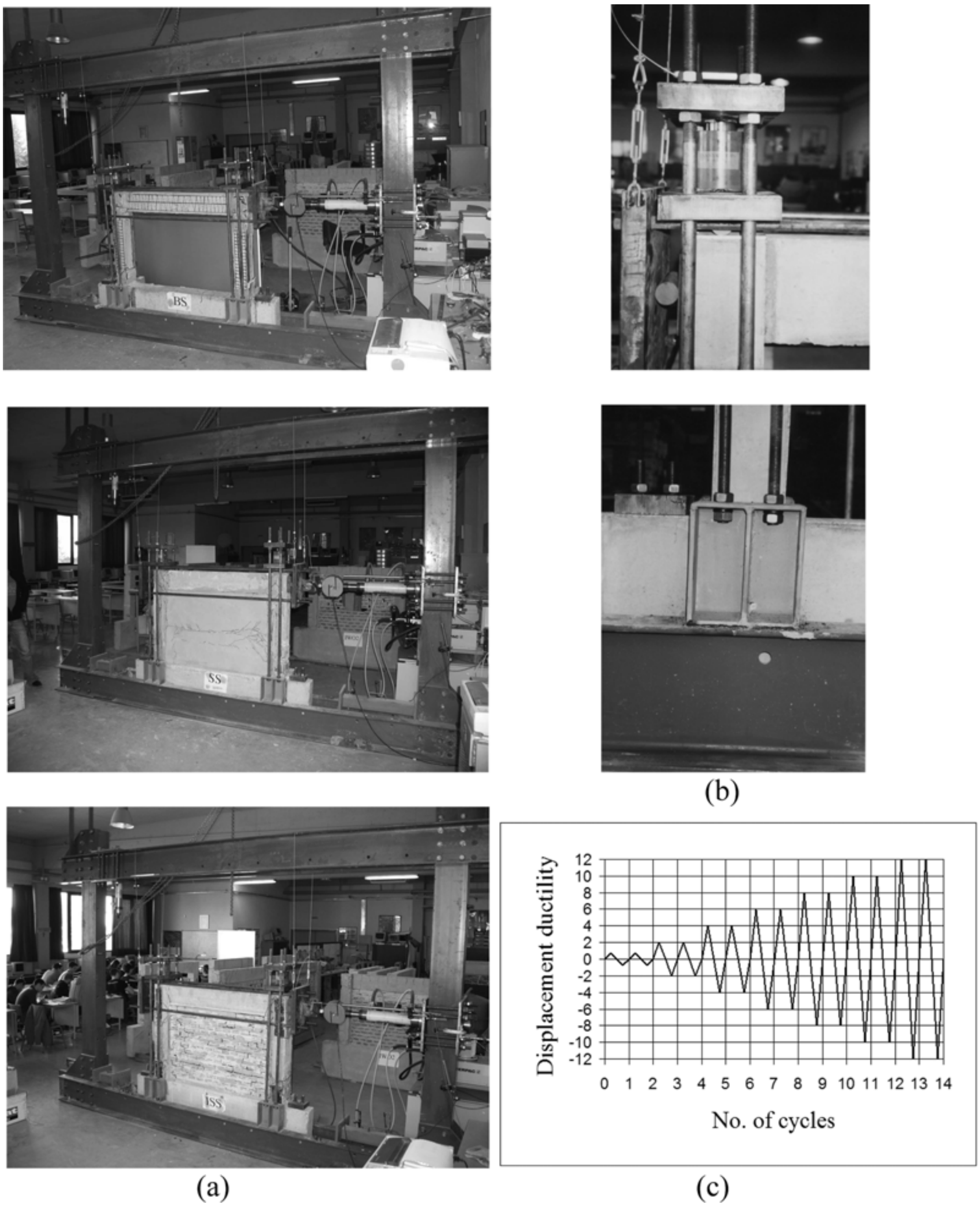


Fig. 2 (a) Test setup of specimens. Specimens "BS", "SS" and "ISS" are actually repaired specimens "RBS", "RSS" and "RISS" respectively, (b) top and bottom connections of vertical loading system, (c) loading program

plate, per column face, attached to the base steel beam of the reaction frame and a pair of Φ24 re useable high strength chucks that restrain the strand to generate the desirable loads. For the top connection a steel plate was mounted on top of the column, after placing a cylinder to roll against a thinner plate to ensure a smooth bearing area, and a hydraulic jack was placed on the top of the steel plate. Four adjustable re useable chucks, per column, were placed above a small steel beam that distributed the load, applied by the jack, equal to the four strands. The level of this axial compressive load per column was set 50 kN (0.1 of the ultimate). It must be also mentioned that the axial loads on the columns were imposed after the infills were constructed. This is a violation of the actual construction technique where a great portion of the axial loads on the columns is due to self weight of the RC structural system and is imposed before the construction of the infills. One LVDT measured the lateral drift of the frame and a load cell measured the lateral force of the hydraulic actuator. Strain gauges, 1 to 8, were placed on the center steel bars of the members at their critical sections to directly monitor the behavior of the reinforcement steel during tests. Dial gauges (9 to 12) and digital gauges (13 and 14) were placed at critical sections of the frame to estimate the relative rotation of the members. To ensure that the measured displacements and loads were applied to the test specimens alone, an additional dial gauge was placed at reaction frame column behind the actuator, as can be seen almost clearly in Fig. 2(a) of specimen RBS, against a stiff and strong concrete wall. The displacement measurements of the reaction frame gave zero values. The loading program included full reversals of gradually increasing displacements. Two reversals were applied for each displacement level. The cycles started from a ductility level 0.8 corresponding to an amplitude of about ±2 mm (the displacement of yield initiation to the system is considered as ductility level $\mu = 1$) and were followed gradually by ductility levels 2, 4, 6, 8, 10, 12 corresponding about to amplitudes 6, 12, 18, 24, 30, 36 mm (Fig. 2).

3.4 Experimental results

The main output of the experimental investigation was a load-displacement curve for each frame (Fig. 3). It must be pointed out that the hysteretic characteristic values of the weak masonry infill were in some cases higher than the corresponding ones of the strong masonry infill. It may be attributed to the larger units of the weak masonry infill. The appearance and propagation of cracking was also recorded for both infill and frame throughout each test (Figs. 4(a), 5(a), 6(a)).

Specimen "BS" was bare reference frame. Flexural cracks and corresponding plastic hinges occurred at predicted critical locations at the bottom and the top of the columns and the ends of the beam – at a drift (0.4-0.6)% – (Fig. 4(a)).

Specimens "SS" and "ISS" had solid weak and solid strong infill respectively. The nonlinear behavior was initiated by the cracking of the infill. Then plastic hinges developed at the top and the bottom of the columns – at a drift (0.4-1.1)% –. However, as shown by the damage patterns of specimens, failure of specimen "SS" with the weak solid infill (Fig. 5(a)) was dominated by internal crushing in the infill – at a drift 1.9% corresponding to the maximum lateral resistance – while failure of specimen "ISS" with the strong solid infill (Fig. 6(a)) was dominated by sliding of the infill along its bed joints – at a drift 1.4% corresponding to the maximum lateral resistance –.

In all infilled specimens cracking of the beam occurred far from the column face towards the mid – span vicinity of the beam. Plastic hinges were developed at drifts higher than 1.1% in bare frame or they did not developed at all in infilled frames. Generally the infills restrained the beams from bending and, there by, postponed the development of plastic hinges in the beams. In the case of the present project shear failure of columns was not observed, as was expected in design, because the infill was weak in relation to the reinforced concrete frames.

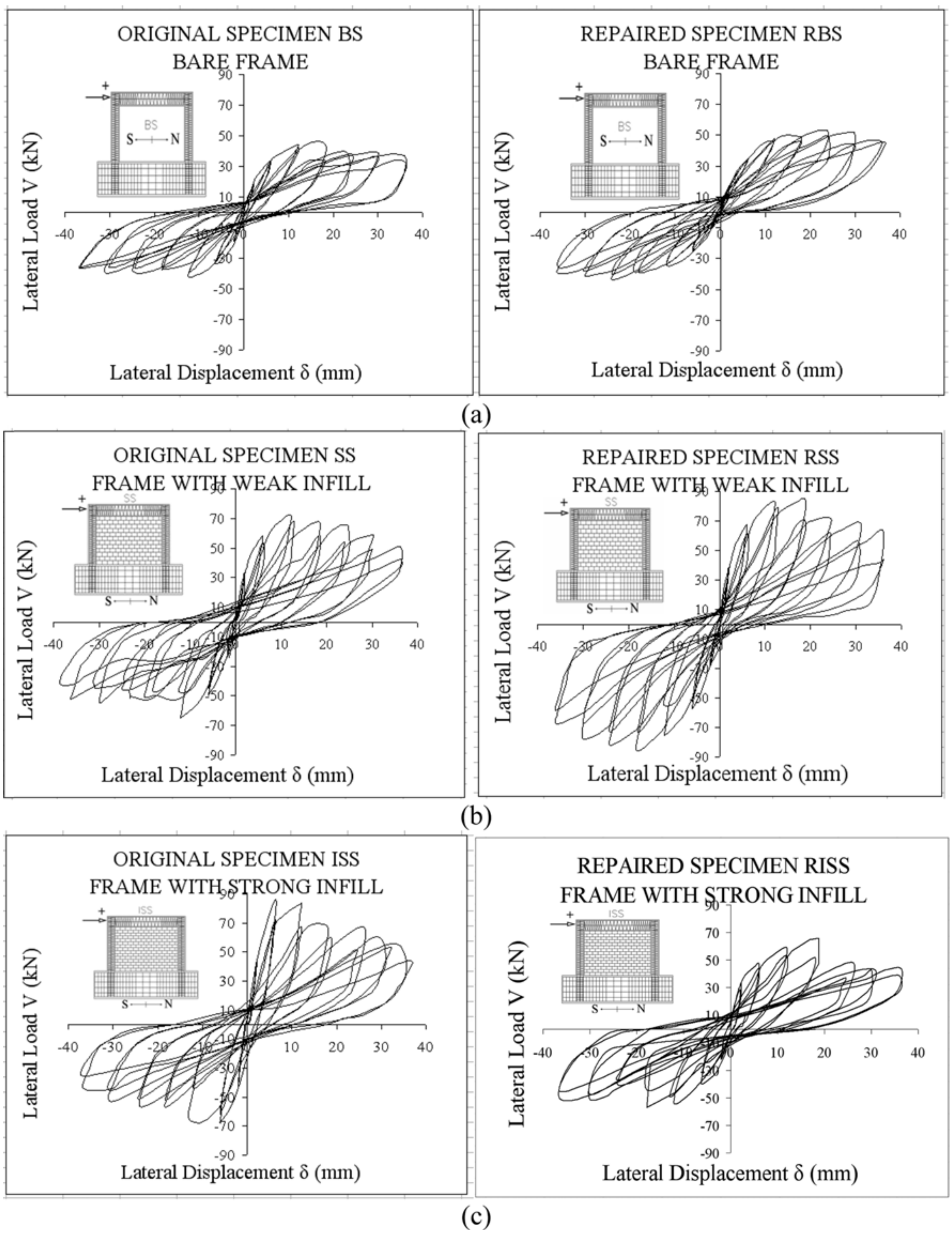


Fig. 3 Load – Displacement hysteresis curves of original and repaired specimens (a) BS and RBS, (b) SS and RSS, (c) ISS and RISS

4. Experimental program for the effectiveness of the repair technique

To achieve the purpose of the designed program tested specimens were repaired, as shown in Table 3 where R denotes repaired specimen, using: thin epoxy resin infused under pressure into the crack system of the damaged RC joint bodies; a polymer modified cement mortar for the damaged

Table 3 Used repair procedures

Repaired specimen	Beam-column connections	Masonry infills
RBS	Epoxy resin injections	-
RSS	Epoxy resin injections	• Polymer modified cement mortar overlays • Fiber glass reinforcing mesh
RISS	• Epoxy resin injections • CFRP plates	Polymer modified cement mortar overlay only along the bed-joints

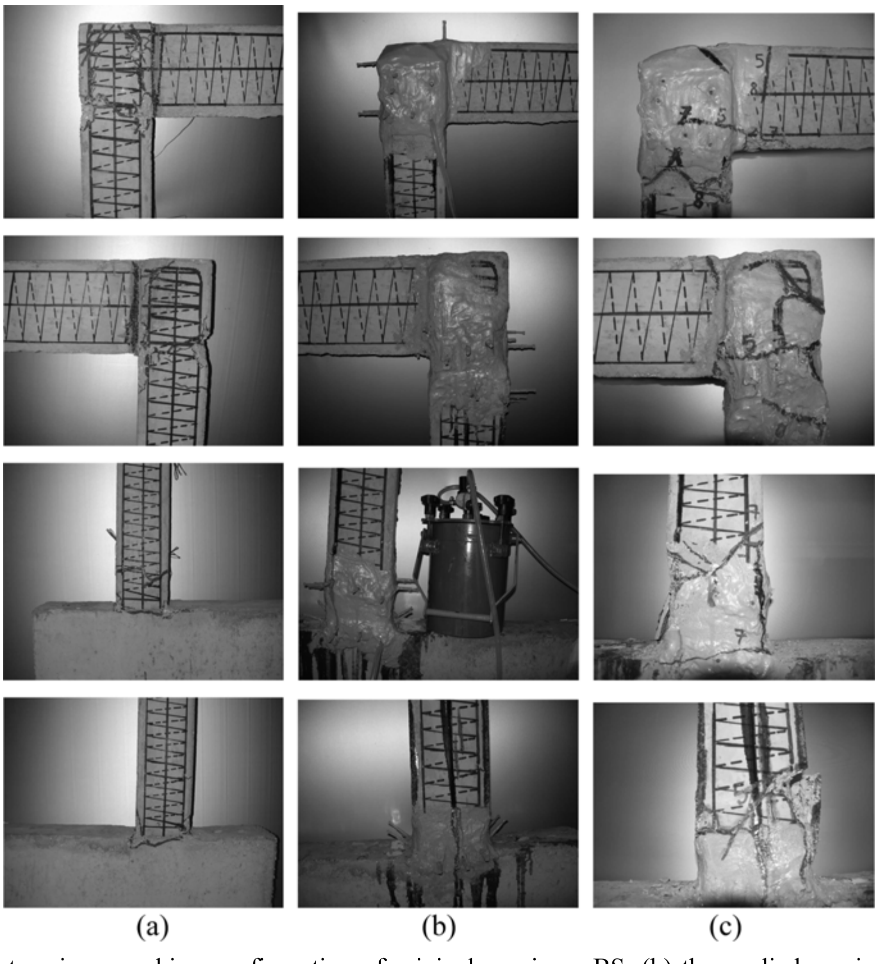
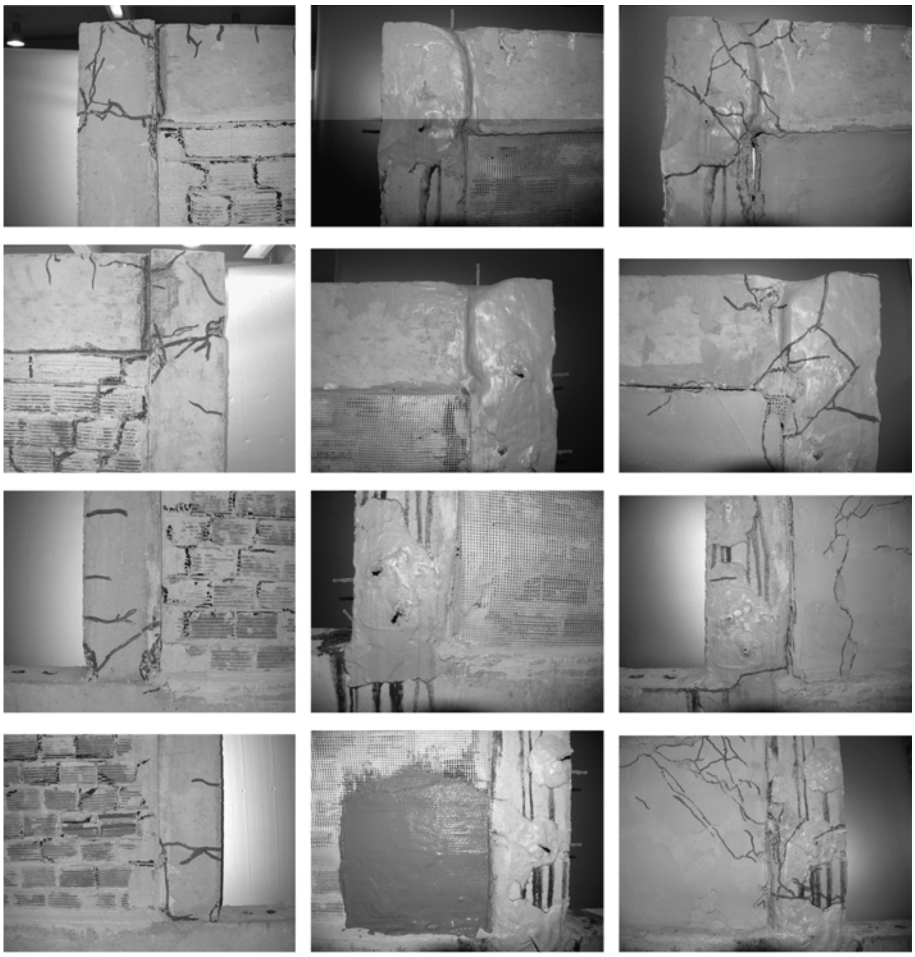


Fig. 4 (a) Joint regions cracking configuration of original specimen BS, (b) the applied repair procedure, (c) joint regions cracking configuration of repaired specimen RBS infill masonry walls; and CFRP factory pultruded plates to the surfaces of the damaged structural members (columns) as external reinforcement. Then, all repaired specimens were retested in the same way to the initial loading (Fig. 2).

4.1 Repair techniques

Concrete structures have been long repaired using pressure injection of epoxy resin. It is a popular strengthening technique, as it does not alter the aesthetic and architectural features of the existing structures and the member strength hierarchy. The main purpose of injections is to restore the original integrity of the retrofitted R/C member and to fill the small voids and small cracks (less than 3 mm wide), which are present in concrete due to seismic actions. This was the case of specimen BS. So, the procedure applied for the repair of specimen BS (repaired bare frame specimen RBS), after its initial loading, includes the following operations: Superficial sealing of all



(a) (b) (c)

Fig. 5 (a) Joint regions cracking configuration of original specimen SS, (b) the applied repair procedure, (c) joint regions cracking configuration of repaired specimen RSS visible cracks with a thick layer of epoxy resin paste except for ports located along the cracks, which allow inlet access for thin epoxy resin to be injected into the system and outlet access for air to escape from the voids, as shown in Fig. 4(b). Injections under pressure of thin epoxy resin into the crack system of the damaged area of the joints until total filling up. The whole infusion procedure requires special care in order to avoid local air trapping. The repaired specimen was retested after the period of resin hardening (that was at least six days).

The procedure applied for the repair of specimen SS (repaired infilled frame specimen RSS with a weak infill without openings, of clay bricks), after its initial loading, includes the following operations: The same procedure of epoxy resin injections was applied for the repair of beam-column connections. Especially for this specimen it has been noted that after initial loading, the masonry bed-joint areas and many bricks were seriously fragmented and practically destroyed. In these cases, it becomes necessary for the lost brick fragments to be removed prior to the resin infusion, and the missing part of the wall to be reconstructed after to the resin infusion. As the retrofit of masonry infill was intended to consist of a "light" intervention, to avoid adverse effects of the wall, a surface treatment was chosen in order to obtain a level of infill strength significant but not so high as to be

(a) (b) (c) (d)

Fig. 6 (a) Joint regions cracking configuration of original specimen ISS; (b), (c) the applied repair procedure; (d) joint regions cracking configuration of repaired specimen RISS beyond initial infill strength. Surface treatment is a common method, which has largely developed through experience. The selected treatment incorporates the technique that consists of a 5 mm × 5 mm fiberglass reinforcing mesh with volumetric ratio of 0.25% completely embedded, without any anchorage system, in a 10 mm thick polymer modified cement mortar overlay of a shear strength value similar to the shear strength value of the masonry wall, troweled on through the mesh with covering thickness of 5 mm and added to both sides of the repaired infill, as shown in Fig. 5(b). By nature this treatment covers the masonry exterior and affects the architectural or historical appearance of the structure. The technique is ideal for low cost repair since it is cheap and can be done with unskilled workers.

The procedure applied for the repair of specimen ISS (repaired infilled frame specimen RISS with a strong infill without openings, of vitrified ceramic bricks), after its initial loading, includes the following operations: The same procedure of epoxy resin injections was applied for the repair of beam-column connections. The lost brick fragments were removed prior to the resin infusion, and the missing parts of the wall were reconstructed after to the resin infusion, as shown from a comparison between Figs. 6(b) and 6(c). Since the degree of masonry damage was not high, with a view to a "light" strengthening intervention, it was decided also to apply the polymer modified cement mortar, but only along the bed-joints, to restore the crack system of the damaged area of the infill wall, because mortar bed-joints were not seriously fragmented after initial loading. Such bed joints treatment affects significantly the cracking or ultimate load of the retrofitted walls. Especially for this specimen, it has been noted that after initial loading, the RC beam-column joint areas were seriously cracked. In these cases, it becomes necessary to enhance the structural member capacity by the application of additional composite overlays acting as external reinforcement on the surfaces. The selective upgrade of the joint by applying composites as continuous reinforcement through critical sections could allow the preferred failure mode to occur. So, increasing the bending strength of the column could allow the development of a ductile flexural plastic hinge in the beam, thus satisfying modern criteria of member strength hierarchy. However the main problem faced in this approach is that peeling off at the corners of longitudinally applied FRP reinforcement usually does not allow the development of the full composite action. In addition, as one of the main qualities of FRP systems, i.e., the simplicity of application, must not be lost, no special anchoring devices have been used. On the other hand the FRP contribution to the increase of column flexural strength reduces as far as the axial force increases, because of the usually small compressive strength of externally bonded fiber composites. However, specimen design implies that the ratio between column and beam bending strengths for the initial structure is 1.50, relative higher, due to the smaller plastic strength of beams, thus reducing the required flexural strength increase in columns. Finally, one layer of CFRP has been applied over the column top and column bottom on both sides in vertical direction, as shown in Fig. 6(c). CFRP plates that have 100 mm width and 400 mm length, have been extended for 200 mm from the beam-column interface and have been bonded to the column with epoxy resin. The CFRP system had to bridge the cracks in the basic material adequately, so that the load of the column passes through the matrix and induces tension forces in the carbon fabric. This transfer of the forces is achieved via shear forces in the matrix and bond forces between overlay and column, as well as bond forces between matrix and fibers.

4.2 Repair material properties

Basic information about the performance of the materials used in the repair are given in Table 4.

Table 4 Properties of the retrofitting materials used

Properties	Resin infusion	Resin paste	CFRP	Polymer mortar
Compression strength (MPa)	71	70	-	13
Bending tension strength (MPa)	35	41	-	3.95
Tension strength (MPa)	-	20.6	2800	-
Adhesive tensile strength (MPa)	3	4	-	-
Young's modulus (GPa)	2	12.6	163	-
Failure strain (%)	-	-	1.60	-
Density (g/cm^3)	1.1	1.71	1.61	1.78

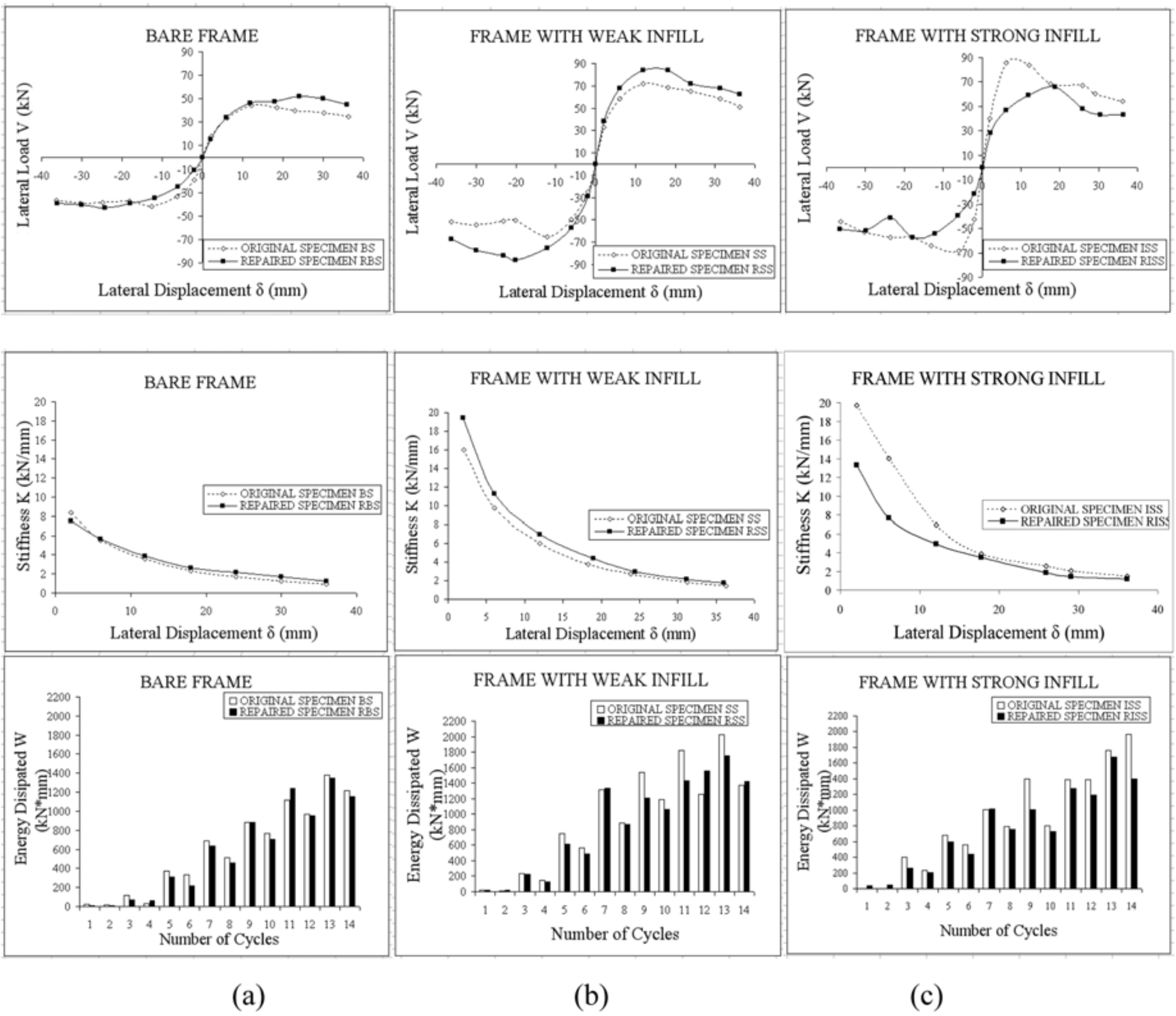


Fig. 7 Comparison of load-displacement envelops, stiffness, energy dissipation between original and repaired specimens: (a) BS and RBS, (b) SS and RSS, (c) ISS and RISS

The strength characteristics of the thin infusion resin and the resin paste, provided by the manufactures for the employed materials used in the repair, were based mainly on ASTM D 695-08 (2008) and ASTM D 790-07e1 (2007) tests. The injection epoxy resin used in all specimens of the test series has a low viscosity of 0.2-0.3 Pa·s for application temperatures of 0^{o}-25^{o}C and a 40 min pot life, which allowed sufficient working time for the injection technique. The strength characteristics of the applied carbon plates of 1.20 mm thickness, provided by the manufactures for the employed materials used in the repair, were based mainly on EN 2561 (1995) tensile strength tests. For the determination of the strength characteristics of the cement mortar used for the repair of the wall, supplementary bending tension and compression tests were included in the experimental program, based mainly on EN 1015-11 (1999). The measured yield stress, from a tensile strength test, of the fiberglass reinforcing mesh was 100 MPa. It must be pointed out that because repairing materials available on market were used, scaling laws are not entirely complied with. So, an engineer applying the results of the paper to a real full scale structure must do so after some engineering judgment.

4.3 Hysteretic responses

To assess the effectiveness of the applied repair techniques, the hysteretic responses of tested specimens under the initial loading were examined and compared to the hysteretic responses of the same specimens under the same loading after the repair, as shown in Fig. 3.

To enable better understanding of the behavioral characteristics of the repaired specimens, data concerning maximum cycle loads, loading stiffness and hysteretic energy absorption capabilities of all tested specimens, were acquired and examined in comparison with the ones of the same specimens under the initial loading, as shown in Fig. 7.

The main objects of the present article does not include the study of the effect of masonry strength of infilled frames under cycling loading. Full details of this study have been published in Kakaletsis and Karayannis (2008). Nevertheless, the influence of weak and strong infills, in a seismic situation, can be discussed with reference to Fig. 7. It appears, from the present case study that frames with strong infills have high initial stiffness and strength which quickly degrades to values lower than a corresponding frame with a weak infill. Alternatively, the strength of frames with weak infills does not appear to quickly degrade.

4.3.1 Response cycles and peak loads

Comparisons of the maximum loads of each loading cycle of the test after repair to the ones of the same specimen in the initial loading, for all specimens, are presented in Fig. 7. Based on these figures it can be deduced that the repaired specimens RBS and RSS, compared with the original ones, resisted the same full loading cycles of constantly increasing displacement, without any loss of their strength. Furthermore, it is also observed that the repaired specimens RBS and RSS exhibit higher maximum loading cycle response values than the same specimens in the initial loading. Indeed, the lateral load capacity ratio (repaired/initial) for bare frame specimen RBS and infilled frame specimen RSS repaired by reinforced mortar overlay, are 1.17 and 1.16, respectively. For specimen RSS a sudden load drop at 9th loading cycle caused by a sliding crack in the middle of the infill height. However, for specimen RISS repaired with mortar in bed-joints and CFRP it was observed a lateral load capacity ratio (repaired/initial) equal to 0.77 and a significant loss of strength at all loading cycles, as well as a sudden load drop in the load-displacement envelope at 9th loading

Table 5 Comparison of response ratios (repaired/initial)

Specimen	Loads ratio cycles							Mean value
	1st	3rd	5th	7th	9th	11th	13th	
RBS	0.85	1.02	1.0	1.11	1.30	1.32	1.30	1.13
RSS	1.18	1.16	1.16	1.23	1.10	1.16	1.22	1.24
RISS	0.70	0.55	0.70	0.96	0.72	0.72	0.80	0.75
Specimen	Stiffness ratio cycles							Mean value
	1st	3rd	5th	7th	9th	11th	13th	
RBS	0.90	1.04	1.08	1.03	1.25	1.33	1.30	1.13
RSS	1.12	1.16	1.16	1.18	1.08	1.16	1.22	1.17
RISS	0.68	0.54	0.70	0.90	0.72	0.69	0.80	0.72
Specimen	Energy ratio cycles							Mean value
	1st	3rd	5th	7th	9th	11th	13th	
RBS	0.45	0.61	0.83	0.93	0.96	1.11	0.98	0.84
RSS	1.00	0.95	0.81	0.97	0.77	0.79	0.87	0.88
RISS	19.56	0.65	0.89	0.95	0.72	0.88	0.95	0.85

cycle, caused by CFRP rupture. Table 5 shows the ratio of the response positive loads of the repaired specimens over the response positive loads of the same specimens in the initial loading for the first loading cycles of all tested specimens. As can be seen from this Table, especially for repaired with plates specimen RISS, maximum loads at all loading cycles were significantly lower than those of the specimen RSS repaired without plates. This may be attributed to the strengthening solution adopted herein by the application of the CFRP technique itself. Indeed, during the test, it seems that the destroyed bond around the reinforcing bars was not restored by epoxy injection, due to delamination of CFRP that caused the concrete peeling. Nevertheless, if the above motivations reported and both the higher level of initial damage of the specimen ISS and the variations in being able to inject epoxy successfully into the joint cracks are not a problem, this result could be strongly related to the fact that a frame with a strong infill cannot be repaired to its original strength but the converse is true for a frame with a weak infill.

4.3.2 Stiffness

The effectiveness of the examined repair techniques in restoring the stiffness of damaged specimens is evaluated by studying the loading stiffness of all loading cycles. The stiffness of each loading cycle of the bare and infilled frames was defined as the slope of the line joining the origin of the load against displacement positive envelope curve, presented in Fig. 3, and the point at which the maximum load of each loading cycle was appeared. Thus, values of the loading tangent stiffness were measured in the tests both prior and after the repair for all loading cycles, as presented in Fig. 7. The ratios of the measured loading stiffness of the repaired specimens over the stiffness of the same specimens in the initial loading for the first loading cycles of all tested specimens are presented in Table 5. Thus, it can be seen that for the repaired specimens RBS and RSS the loading stiffness was restored satisfactorily achieving similar or higher stiffness levels to the virgin specimens. Indeed, the mean response stiffness ratio (repaired/initial) for bare frame specimen RBS and infilled frame specimen RSS repaired by reinforced mortar overlay, are 1.13 and 1.24, respectively. However, as can be seen from Table 5, during the first cycle, epoxy-repaired bare frame had (as anticipated from the existing literature) 15% lower strength and 10% lower stiffness than the virgin specimen. A cracking of the layer of epoxy resin paste occurred at early loading stages causing the initial degradation. After test the layer of epoxy resin paste was removed and it was shown that the RC core was neither cracked nor damaged. However, for specimen RISS repaired with mortar in bed-joints and CFRP, a significant loss of stiffness has been noted at all loading cycles and it was observed a mean response stiffness ratio (repaired/initial) equal to 0.72. These results show that the reliability of the repair technique of specimen RISS in recovery of stiffness is questionable.

4.3.3 Hysteretic energy absorption

In order to ascertain the hysteretic energy absorption capabilities of the repaired specimens, the energy dissipated per cycle is used, both for the repaired and virgin specimens. The energy dissipated at a given cycle of Fig. 3 (in terms of the area bounded by the hysteretic curve for that cycle) is presented in Fig. 7. The ratios of the measured energy absorption of the repaired specimens to the initial energy absorption of the same specimens are presented in Table 5. Based on these values, it can be deduced that the energy absorption capability of all repaired specimens appears to have decreased during almost all loading cycles, in comparison with the ones of the same specimens in the initial loading. The mean response energy ratio (repaired/initial) for bare frame specimen RBS, infilled frame specimen RSS repaired by reinforced mortar overlay and specimen RISS repaired with mortar in bed-joints and CFRP are 0.84, 0.88 and 0.85 respectively.

For repaired specimen RBS the apparent improvement of the energy absorption capability of the loading cycles after a drift of about $\gamma = 2\%$ (9th cycle), compared to the same virgin specimen, can be attributed mainly to the higher rate of deterioration that the virgin specimen exhibited during the loading sequence. However, for repaired specimens RSS and RISS, the energy dissipation was greatest before a drift of about $\gamma = 2\%$ (7th cycle). After this, dissipation dropped with a steep branch. In specimen RSS, the drop was attributed to a simultaneous, abrupt formation of a sliding crack in the middle height of the infill. This crack prevented the dispersion of cracks in a uniform way throughout the wall. In specimen RISS, the CFRP plates became very brittle when debonding occured and they buckled under compressive stresses after displacement level of 7^{th} cycle. So, weakened cross sections caused sudden peeling off CFRP at the following cycle, which caused tension on CFRP. Furthermore, the abrupt dissipation dropping in specimen RISS may be attributed to the degree that the crack system of the damaged bed-joints of the infill was filled by the polymer modified cement mortar during the repair works.

4.3.4 Failure modes

In order to investigate whether the applied repair technique modifies the type of the infilled frame failure, observed failure modes of both the initial and the repaired infilled frames are examined and compared to each other.

Failure modes of specimens in the initial loading have been presented in Section 3.4 and Figs. 4(a), 5(a), 6(a).

Failure of repaired specimen “RBS”, as shown in Fig. 4(c), is characterized by cracks in the beam region, next to the repaired parts which developed from the beginning of the loading sequence.

Subsequent loading cycles resulted in a gradual increase of the width of these cracks and finally in the formation of clear flexural hinges. Slight crack were observed during the first loading cycles (2 or 3) both at the joint regions and at the end of the columns. In subsequent loading cycles, the main damage was localized at the end of the columns rather than at the joint regions. Spalling at the joint regions was not observed. Finally, a distinct flexural hinge was developed in the column region, next to the repaired parts. It can be seen that the safety of the frame was not jeopardised by changing the members strength hierarchy after repair.

Failure of repaired specimen "RSS", as shown in Figs. 2, 5(c), is characterized by the formation of a plane of weakness near the mid-height level of the infill panel. This behavior mode commonly occurs if the mortar beds are relatively weak compared to the adjacent masonry units. Finally, the units of the infill wall were not fragmented or removed. In this specimen, from the first cycles of loading, the cracks of RC frame were concentrated at the regions of the joints. Subsequent loading cycles resulted in a gradual increase of the width of the cracks across the joint regions, but the observed serious cracking of the joint body did not result in destruction and removal of the concrete or the epoxy resin. Although, by the end of the initial loading, bricks had been loosened within the frame, such that this might leave the infill vulnerable to fall-out from out-of-plane loads, masonry has been kept in place during the same loading after the repair (Fig. 2). It is likely that the out-of-plane integrity of infill, with repaired by reinforced mortar overlay pasted on the wall, has been improved due to tensile membrane action.

Failure of repaired specimens "RISS", as shown in Figs. 2, 6(d), is characterized by the bed-joint sliding of the infill. In this case cracks appeared from the beginning of the loading in the column and beam region next to the repaired part of the frame. In subsequent loading cycles, these cracks were increased and became wider. Finally, the concrete from the column region next to the repaired part was seriously crushed and fragmented. The delamination mechanism caused the concrete peeling. This occurred because the stresses exceeded the tensile strength of concrete. However, one of the main objectives of the CFRP strengthening system was to force plastic hinges forming in the beam. Experimental results previously discussed demonstrate that this objective has not been achieved. This is due to the fact that in specimens RSS and RISS, the infill walls, as in the initial tests, restrained the beams from bending and, thereby, postponed the development of plastic hinges in the beams. Nevertheless, CFRP was not observed to offer a clear total improvement of the response of specimen RISS, due to lack of threaded-rod anchorage and the resulting early delamination that has destroyed the bond around the reinforcing bars.

The failure modes of all specimens in both the initial loading and the loading after the repair are presented in Table 6.

Table 6 Failure modes of initial and repaired specimens

Specimen	R/C members	Masonry infills
BS	Hinges in beam and columns	-
RBS	The same but next to the repaired parts	-
SS	Hinges at column ends	Interior crushing
RSS	The same plus cracking of joints	Mid-height level cracking
ISS	Hinges at column ends	Shear– sliding
RISS	The same but next to the repaired parts	Bed-joint sliding

In this Table it can be seen that the examined repair techniques either improve the failure behavior of the bare and infilled frames or, in the worst case of specimen RSS, the repair strengthens the infill leading to a cracking of insignificant severity of the joint body. The observed changes in the failure behavior of the specimens after the repair can be reasonably attributed to the changes brought about by the repair technique to the material of the infilled frame to a composite material which consists of wall parts, concrete parts and parts or layers of the retrofitting materials used.

5. Analytical justifications

In the mechanism for repaired bare frame RBS, assuming that plastic hinges occurred at the bottom and the top of the columns, flexural resistance is

$$F_f = 4 \cdot M_{pc}/h \tag{1}$$

where M_{pc} is the plastic moment of the column considering the effect of the axial force, taking into account full restoration of the destroyed bond around the reinforcing bars, $h = H\text{-}l_p$, H is the height of masonry infill, l_p is the plastic hinge length, normally equal to 0.5 times the column depth (Paulay and Priestley 1992) and after repair, equal to the observed increased value of 0.15 m.

In the mechanism for repaired infilled frames RSS and RISS the frame and the infill are considered as two parallel systems with displacement compatibility at the compression corners. Hence, the lateral resistance of this mechanism is considered to be the sum of the flexural resistance of the frame, F_f, and the shear-sliding resistance of the wall, $V_{w,u}$, (and the shear carried by the mortar overlay, $V_{m,u}$, with or without the shear carried by the mesh in the mortar, $V_{m,s}$), as can be obtained from FEMA 306 (1999)

$$V_u = F_f + V_{w,u} + (V_{m,u} + V_{m,s}) = 4 \cdot M_{pc}/h + f_v\, t\, l + (f_{vm}\, t_m\, l + \rho_m f_{ym} t_m\, l) \tag{2}$$

where f_v is the masonry shear strength of the bed joints subjected to normal stress f_n, from diagonal tests on full size bare infills ($f_v/f_n = l/H$ = 120 cm/80 cm, as shown in Table 2), l is the length of masonry infill, t is the thickness of masonry infill, f_{vm} is the mortar shear strength equal to 0.167 $f_{cm}^{1/2}$ where f_{cm} is the mortar compression strength, t_m is the mortar thickness, ρ_m is the volumetric ratio of the mesh, f_{ym} is the yield strength of the mesh.

For repaired infilled frame RISS, $M_{pc} = (M_{pc}^A + M_{pc}^B)/2$ are plastic moments at the bottom, A, and the top, B, of the columns, due only to CFRP reinforcement, obtained taking into account that the contribution of existing steel rebars has been neglected in such a way to assign the whole tension to the new FRP reinforcement, assuming a concrete tensile strength equal to 2.50 MPa, and estimating an anchorage length after last plastic hinge crack equal to 100 mm that leads to a portion of CFRP tensile strength equal to 312.50 MPa.

For repaired infilled frame RSS, M_{pc} is the plastic moment of the column considering the effect of the axial force, taking into account full restoration of the destroyed bond around the reinforcing bars, $h = H\text{-}l_p$, H is the height of masonry infill, l_p is the plastic hinge length, equal to 0.5 times the column depth.

The shear strength capacities computed using these equations are summarized for the repaired bare and infilled frame specimens in Table 7. Analytical predictions of shear strength were found to be in good agreement with the experimental results.

Table 7 Comparison of experimental and analytical result

Repaired Specimen:			RBS	RSS	RISS
Frame strength:	$F_f = 4M_{pc} / h$	kN	47.38	42.48	41.54
Wall strength:					
Mason. shear failure	$V_{w,u} = f_v\, t\, l$	kN	-	27.36	25.58
Mortar shear failure	$V_{m,u} = f_{vm}\, t_m\, l$	kN	-	14.40	-
Shear by mesh	$V_{m,s} = \rho_m f_{ym} t_m\, l$	kN	-	6.00	-
Infilled frame strength:	$V_u = F_f + V_{w,u} + (V_{m,u} + V_{m,s})$	kN	47.38	90.24	67.12
Observed strength:	$V_{u,\exp}$	kN	51.79	84.10	65.85
Strength ratios	$V_{u,\exp} / V_u$		1.09	0.93	0.98

6. Conclusions

The authors have carried out investigations, under cyclic loads, on both bare and masonry infilled frames with both weak and strong infills, providing data for evaluation of two different infill compressive strengths.

The damaged specimens were repaired using repair procedures covering most usually-applied practices: beam-column connections were repaired using resin injections and external FRP laminates and infill walls were repaired using polymer modified cement mortar overlays. The repaired sub assemblages were retested under the same loading regime. The following conclusions concerning the applied repair technique are drawn from the test reported herein:

a) The examined repair techniques can be considered to be satisfactory, since most of the repaired specimens exhibited equal or higher response load values and loading stiffness, compared to the virgin ones, and tended to withstand the same loading cycles without a significant loss of energy absorption.

b) The examined repair technique of resin infusion for the joint region, in most of the cases, does not alter significantly the type of beam-column joint failure and damage is mainly developed and concentrated in the beam or column region next to the repaired part of the joint. In any case, the examined repair technique of the infill walls either improves the failure behavior of the beam-column joint, or it does not change the failure of the virgin joint.

c) Using an enhanced polymer modified cement mortar overlay on the infill panel, which also contains fiber glass reinforcing mesh as reinforcement, provides an improved seismic capacity for the infill panel. An enhanced overlay should improve the general seismic performance of such an infilled wall system. The reinforcing mesh also help to prevent out-of-plane buckling of the mortar at the center of the panel. Such rehabilitated infills could be used in the lower story of a multi story frame where plastic hinging would normally be expected to occur in structural elements under earthquake loading.

d) Further, in the loading after the repair the type of failure of specimen RISS tends to exhibit a brittle behavior being in compliance with the brittle behavior of CFRP after the application of epoxy adhesive. So, although CFRP externally bonded laminates are an effective technique for retrofitting of flexural elements, interfacial issues between concrete and CFRP can control the behavior under cycling loading.

e) In addition, it seems that a frame with a strong infill cannot be repaired to its original strength but the converse is true for a frame with a weak infill. Considering this, and despite the inherent weakness, that results from the small number of specimens tested, that is particularly true for specimens with infills, due to the variability of masonry and the contact between infill and frame, the authors could recommend that it is better to use a weak than a strong infill in a seismic situation.

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