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Comparative experimental assessment of seismic rehabilitation with CFRP strips and sheets on RC frames

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Abstract. The effectiveness of the use of modern repair schemes for the seismic retrofit of existing RC structures were assessed on a comparative experimental study of carbon fiber-reinforced polymer (CFRP) strips and sheets for the repair of reinforced concrete members of RC frames, damaged because of cyclic loading. Two virgin, single - story, one - bay, 1/3 - scale frame specimens were tested under cyclic horizontal loading, up to a drift level of 4%. Then, virgin specimens, B and F, respectively, were repaired and retested in the same way. One, specimen RB, was repaired with epoxy injections and CFRP strips and one, specimen RF, was repaired with epoxy injections and CFRP sheets. The two specimens are used to examine the differences between the structural behavior of frames repaired using CFRP strips and frames repaired using CFRP sheets. Both qualitative and quantitative conclusions, based on the observed maximum loads, loading and reloading stiffness, hysteretic energy absorption and failure mechanisms are presented and compared. The repaired frames recovered their strength, stiffness and energy dissipated reasonably. The use of CFRP sheets was found more effective than CFRP strips, due to the proper anchorage.

Keywords: RC frames; seismic loading; rehabilitation; CFRP strips; CFRP sheets

1. Introduction

The issue of upgrading the existing civil engineering infrastructure has been one of great importance in recent years. Deterioration of bridge and building structural elements may be attributed to environmentally induced degradation, poor initial design and construction and to accidental events such as earthquakes. The problem of structural deficiency of existing constructions is especially acute in seismic regions, as, even there, seismic design of structures is relatively recent. So, seismic retrofit has become very important in these areas of high seismic risk.

Recent developments related to materials, methods and techniques for structural strengthening have been enormous. So, nowadays, modern retrofitting techniques have been developed using bracing-friction damper systems or other added viscous damping (Lavan and Levy 2010) and several composite materials protecting the beam-column joint region have been presented (Li *et al*. 2015). One of today's state-of-the-art techniques is based on the use of fibre reinforced polymer (FRP) composites, which are currently viewed by structural engineers as "new" and highly

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promising materials in the construction industry.

Composite materials for strengthening of civil engineering structures are available today mainly in the form of: (a) thin unidirectional strips, with thickness in the order of 1 mm, made by pultrusion, best suited for plane and straight surfaces and (b) flexible sheets or fabrics or textiles made of fibres in one, two or more directions that can be used to plane as well as to convex surfaces.

In particular, the use of composites as externally bonded reinforcement (EBR) of reinforced concrete (RC) elements has increased in many seismic retrofit projects in order to replace: (a) the surface-bonded steel plates which, although less costly, are much heavier, more difficult to apply and mechanically less effective (Fardis 2009); (b) the conventional concrete jackets, providing substantial increase in strength and ductility without much affecting the stiffness as described in Bousias *et al*. (2004). Moreover, typical examples of recently developed techniques of shear strengthening in beam-column joints are described in Engindeniz *et al.* (2005). Also, the use of various types of anchors with FRPs in strengthening of RC beams and frames, that is an issue of great importance, have been experimentally investigated by Koutas and Triantafillou (2013) and Koutas *et al.* (2014).

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 have been presented by Yurdakul and Avsar (2015), 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). There are, also, a number of papers that presents PsD tests with analytical evaluations that have been conducted in the last decade 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). 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.

On the other hand, 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 (Eurocode 8 2005). Nevertheless, 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 (Engindeniz *et al.* 2005). By this technique, an increase in peak load was observed between 8% and 40% and an increase in dissipated energy was observed between 53% and 139%. The change in stiffness varied between a 27% decrease and a 10% increase.

National and international code indications are now available for the design of elements externally bonded with FRP (ACI Committee 440.2R 2002, fib Bulletin 35 2006). 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 and about comparisons of different criteria for the seismic strengthening of RC framed buildings in order to find the optimal combinations of these retrofitting techniques, as have been presented in Mazza (2015).

Development and standardization of simple, cost effective structural retrofitting solutions, that can fulfill the requirements for public safety with least disruption of occupancy, will enhance public safety and improve quality of life at costs that both the owners and the national economy can bear. Retrofitting with externally bonded advanced composites, to which the present work is devoted, is cleaner and easier to apply than conventional retrofitting techniques (notably jackets of cast in situ concrete or shotcrete), disrupts less the occupancy and operation of the facility, does not generate debris or waste, reduces health and accident hazards at the construction site and noise or air pollution there and in the surroundings. Therefore, application of externally bonded Fibre Reinforced Polymers (FRPs) are rapidly becoming the technique of choice for structural retrofitting.

This work aims to evaluate experimentally the lateral strength, stiffness and energy dissipation capacity of reinforced concrete structures, damaged under cycling loading and repaired by the application of the above techniques, and to investigate technically appropriate and relative cost effective retrofitting methods using CFRP. Two virgin RC frame specimens, B and F, respectively, that were tested in cyclic loading, have been repaired and retested in the same way. One, specimen RB, repaired with epoxy injections and CFRP strips and one, specimen RF, repaired with epoxy injections and CFRP sheets. The two specimens are used to examine the differences between the structural behavior of frames repaired using CFRP strips and frames repaired using CFRP sheets. Both qualitative and quantitative conclusions, based on the observed maximum loads, loading and reloading stiffness and hysteretic energy absorption are presented and compared.

2. Shape-Mechanical characteristics-Loading of initial specimens

The design characteristics of R/C frames, the mechanical properties of the materials used and the loading program are shown in Table 1 and Fig. 1, Table 2, Fig. 2 respectively. In Table 2 the mechanical proprieties of concrete and reinforcement were obtained by material tests that were conducted on concrete and reinforcing steel samples. The properties for the employed materials in the repair were provided by the manufacturer. 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. Lateral loading was applied by a double acting hydraulic jack at the level of

Table 1 Design characteristics of R/C frames

Design characteristic	Prototype (mm)	1/3 Model (mm)
Length axis to axis of columns	4050	135
Hight from beam axis	2700	90
Cross section of columns	450/450	150/150
Cross section of beams	300/600	100/200
Longitudinal reinforcement of columns	8Φ18	8Φ5.6 (8.75‰)
Tensile reinforcement of beams	3Φ18	3Φ5.6 (3.69‰)
Compression reinforcement of beams	2Φ18	2Φ5.6 (2.46‰)
Stirrups	Φ10	Φ3
Spacing of stirrups-critical regions	100(col.), 120(beam)	34(col.), 40(beam)
Spacing of stirrups- non-critical regions	100(col.), 200(beam)	34(col.), 66(beam)

Table 2 Mechanical properties of the materials used

Material	Property
Concrete	Cubic compressive strength=28.51 MPa.
Longitudinal steel bars Φ5,6	Yield stress=390.47 MPa
Transverse steel bars Φ3	Yield stress=212.2 MPa
Epoxy resin	Elastic modulus and tensile strength=2 GPa and 25.7 MPa, respectively
CFRP strips (carbon plates)	Elastic modulus, tensile strength, ultimate strain and thickness=163 GPa, 2800 MPa, 1.60% and 1.20mm, respectively
CFRP sheets (carbon fabrics)	Elastic modulus, tensile strength, ultimate strain and thickness=235 GPa, 3800 MPa, 1.50% and 0.11 mm, respectively

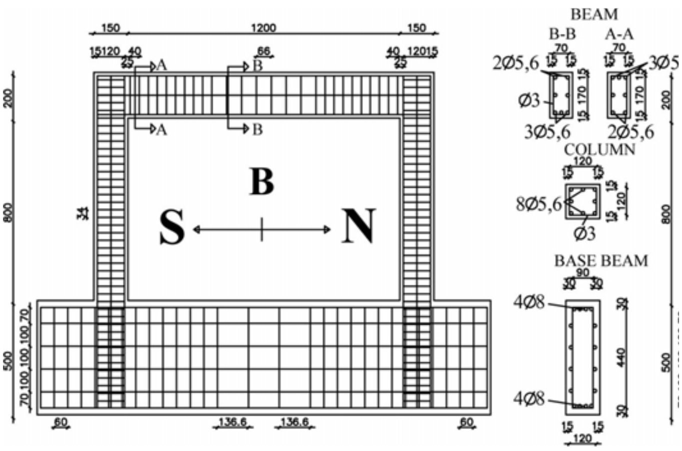


Fig. 1 Reinforcement detailing of the RC frame models (mm)

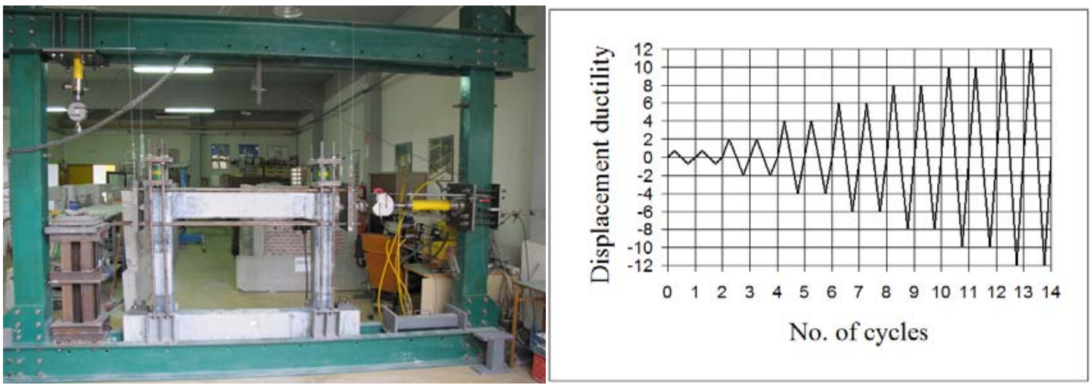


Fig. 2 Test setup of specimen RF and loading program

the axis of the RC beam. An axial constant compressive load per column equal to 50 kN was applied too. The loading program included full reversals of gradually increasing displacements, comprising seven displacement amplitude sets, of two loading cycles per set. The cycles started from a ductility level 0.8 corresponding to an amplitude of about ±2 mm (the displacement of yield initiation of the system is considered as ductility level μ=1) and were followed gradually by ductility levels 2, 4, 6, 8, 10, 12 corresponding to amplitudes 6, 12, 18, 24, 30, 36 mm. Results from a push over analysis were used to estimate the displacement of yield initiation of the system being about ±2.5 mm. Test data used in the present investigation are consistent with the descriptions of anticipated damage at structural performance level of "Structural stability", since the final horizontal offset approaches 4% interstory drift (Fardis 2009).

The appearance and propagation of cracking was also recovered throughout each test. In Figs. 3(a), 4(a), 6(a) and 7(a) the failure modes of the original specimens are shown. Only flexural types of failure concentrated at critical sections of RC members appeared. 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 of 0.4-0.6%. The complete information and detailed results of the aforementioned experimental study have been reported in Kakaletsis and Karayannis (2009).

3. Repair techniques

The retrofit design strategy was focused on the following aspects: Methods and techniques have to be applied which will enable repairing of buildings with minimum disturbance to the occupants. The author thought that owing to their high strength and ease in application, CFRP sheets and strips could be used in developing such techniques. Most of researchers on this field had focused their attention on the strengthening of individual members (Engindeniz *et al.* 2005). The present research was intended to investigate the feasibility of strengthening damaged RC framed structures by using CFRPs. It was thought that by strengthening the RC members using externally applied CFRP layers and integrating these elements into the existing structural system, it might be possible to end up with a new lateral load resisting system having a much higher shear capacity than the damaged one. In order to evaluate the test results of the repaired with CFRPs two frames, the test results of the corresponding virgin frames, were used. To achieve the purpose of the designed program, the following repair strategies have been applied:

Resin injection is a popular strengthening technique, as it does not alter the member strength hierarchy, the aesthetic and architectural features of the existing structures. Epoxy resin is used for relative small cracks (less than 2 mm wide). As this was the case of both specimens B and F, the procedure applied for the repair of specimens (repaired frame specimens RB and RF respectively), after their initial loading, includes the following operations: Superficial sealing of all 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 (Figs. 3(a) and 4(a)). 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 specimens remained unloaded during the period of resin hardening (for at least six days).

It must be pointed out that in the original specimens, the flexural capacity of column to that of beam was 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. So, the purpose of retrofitting mainly the columns was to push the formation of plastic hinge in the beam, so that the safety of the frame is not jeopardized. 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, the above special specimen design reduces the required flexural strength increase in columns.

Especially for specimen B (repaired frame specimen RB), it was observed that after initial loading, the R/C beam-column critical regions were seriously cracked. The lateral load capacity on frame could be increased by using FRP plates by increasing the column flexural strength. This goal could be achieved by using FRP plates (strips) well anchored at columns' ends and applied on proper faces. So, one CFRP plate has been applied over the column head on both side faces (strips 3 and 4) and one on the exterior face, in vertical direction (strip 2), one CFRP plate has been applied over the column bottom on both side faces in vertical direction (strips 5 and 6), and one CFRP plate has been applied only over the upper face of beam edge in horizontal direction (strip 1), as shown in Fig. 3(b). It is obvious that the CFRP plates have to bridge the cracks in the basic material adequately. The proper face selection was primarily dictated by the fact that the achievement of proper anchorages for plates is a much more difficult task than the CFRP technique itself. So, the application of CFRP plates on the interior faces of the beam edge and column head or both the interior and exterior faces of the column bottom has been omitted, because it could not provide adequate anchorages. As a result the strengthening scheme has certain difficulties and may not always be applicable in practice. CFRP plates, that have 100mm width and 400 mm length, have been extended for 200 mm from the beam-column interface and have been bonded on the column and beam with epoxy resin. The CFRP system has been applied to the RC members in this manner that the load of the member passes through the matrix and induces tension forces in the carbon strip. This transfer of the forces is achieved via shear forces in the matrix and bond forces between overlay and member, as well as bond forces between matrix and fibers. The above technique and behavior of specimen RB have been reported in detail in Kakaletsis (2011).

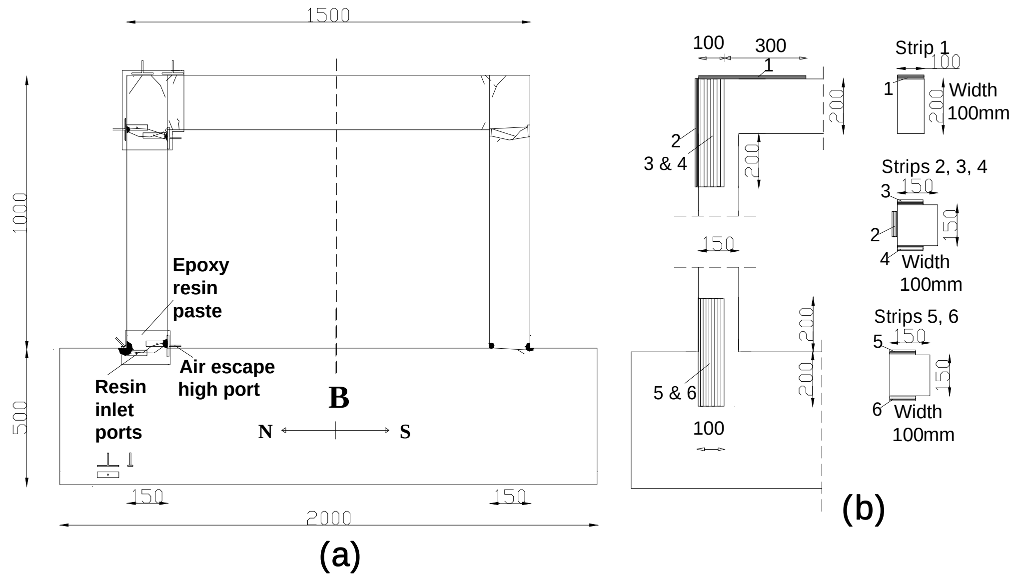


Fig. 3 (a) Application of epoxy injections; (b) arrangement and location of CFRP strips in Specimen B (units in mm)

For specimen F (repaired frame specimen RF), the seriously cracked R/C beam-column critical regions were strengthened in flexure through the application of CFRP fabrics to their tension zones with the direction of fibres parallel to that of high tensile stresses. So, one U-shaped CFRP fabric has been applied over the column head on both side faces in vertical direction (sheet 3), one U-shaped CFRP fabric has been applied over the beam edge on both side faces in horizontal direction (sheet 1), two L-shaped CFRP fabrics have been applied on the exterior and interior faces of column head joint, (sheets 2 and 4, respectively), two CFRP fabrics have been applied over the column bottom on both side faces in vertical direction (sheets 7), and two L-shaped CFRP fabrics have been applied on the exterior and interior faces of column bottom joint (sheets 8 and 9, respectively), as shown in Fig. 4(b). All above sheets had an anchorage length equal to 100 mm, after the transverse confinement. Furthermore, to increase of member capacity through confinement, sheet 5 has been wrapped around critical regions of beam and sheets 6 and 10 have been winded around critical regions of column head and bottom, respectively. These sheets enclose the longitudinal sheets at their ends and prevent peeling-off, extending along the plastic hinge length of at least 100 mm. It must be emphasized that for L-shaped CFRP fabrics 4, 8 and 9 special adhesive CFRP anchors were developed to transfer the applied load to the concrete through the bond surface along the embedment depth, as shown in Fig. 4(b). Anchors, as shown in Fig. 5, were prepared by rolling the CFRP sheet around itself, such that it has a cylindrical form of a 100 mm

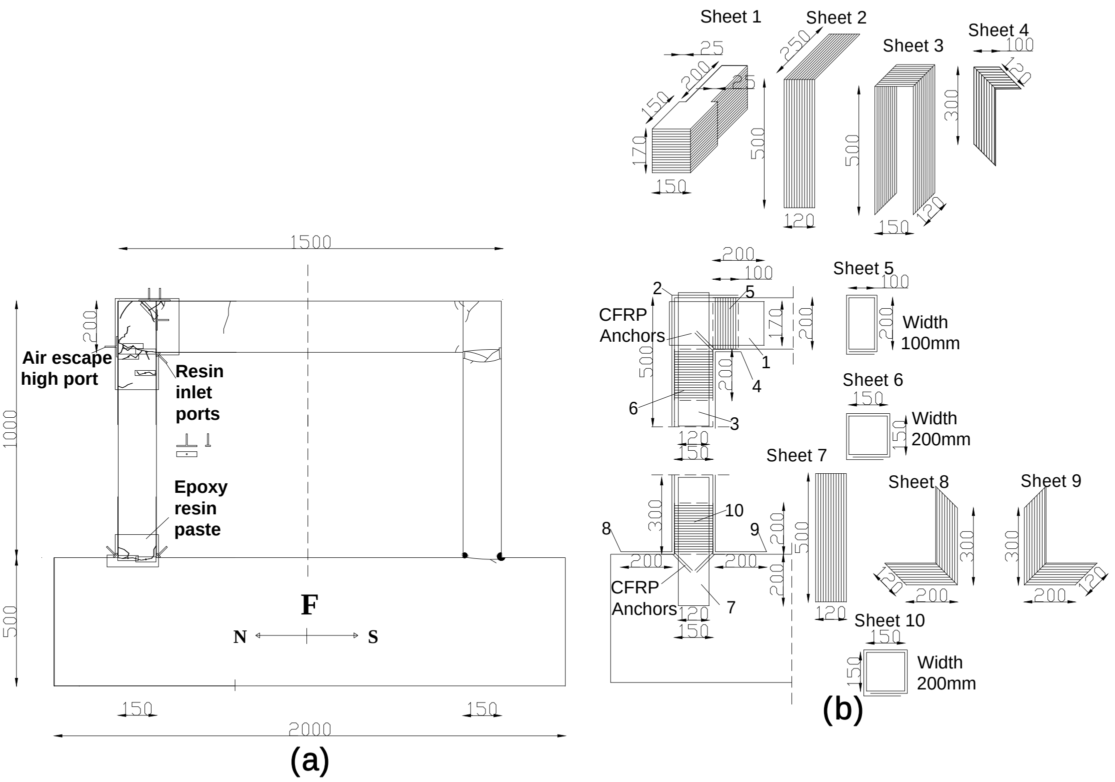


Fig. 4 (a) Application of epoxy injections; (b) arrangement and location of CFRP sheets in Specimen F (units in mm)

(a) (b)

Fig. 5 (a) Construction of CFRP anchors; (b) location of CFRP anchors in Specimen F

length. Then the rolled CFRP sheets were tied in a 50 mm length to a guide wire of a 1 mm diameter and embedded into epoxy resin so that the tied portion is covered with epoxy and inserted into drilled holes of a depth of 50 mm and a diameter of 10 mm, filled with epoxy, pushing it by means of the thin steel wire. The rest 50 mm length CFRP sheet was unrolled, pierced and bonded on L-shaped CFRP fabrics. Two anchors were used for each L-shaped CFRP sheet.

The amount of CFRP reinforcement is determined on the basis of experience and is verified in section of analytical justifications.

4. Behaviour of test specimens

The repaired specimens were tested according to the loading program of Fig. 2 and the failure modes presented in Fig. 6 and Fig. 7 and the hysteretic responses presented in Fig. 8 were obtained and compared with those of the original specimens and to each other.

It can be deduced, from Fig. 9, that the repaired specimens RB and RF, 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 specimen RF with sheets exhibited higher maximum loading cycle response values than the specimen RB with strips. Table 3 shows the ratio of the response loads of the repaired specimens to the response loads of the same specimens in the initial loading for the first reversal of each displacement level of tested specimens. For repaired with fabrics specimen RF, the maximum loads at all loading cycles were higher than those of the specimen RB repaired with plates. This result could be strongly related to the strengthening solution adopted herein. So, experimental results demonstrate that one of the main objectives of the CFRP strengthening system to obtain a new lateral load resisting system having a much higher shear capacity has been achieved.

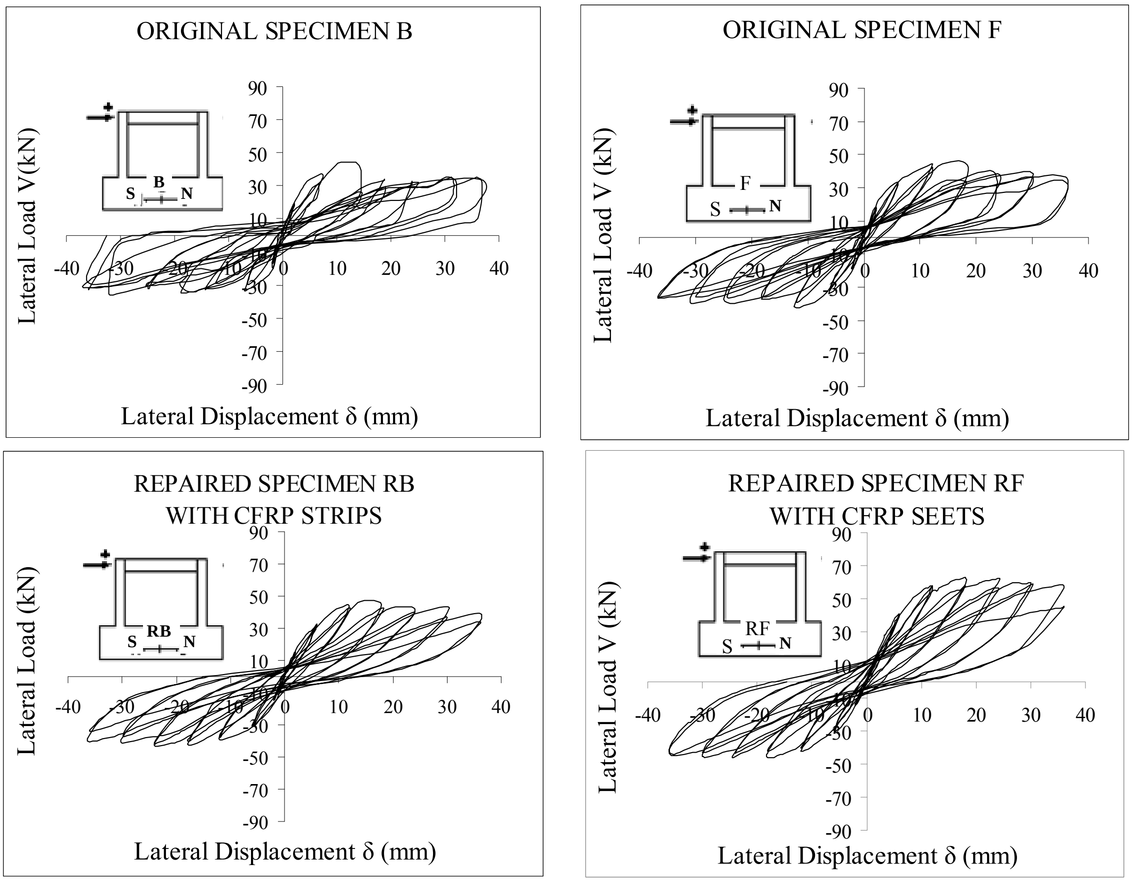
The values of the loading tangent stiffness were measured in the tests both prior and after the repair for all loading cycles, as presented in Fig. 9. The ratios of the measured loading stiffness of the repaired specimens to the stiffness of the same specimens in the initial loading for the first reversal of each displacement level of tested specimens are presented in Table 3. For the repaired specimens RB with strips, the loading stiffness was restored satisfactorily achieving similar stiffness levels to the virgin specimen. However, for repaired specimen RF with sheets it was

(a) (b) (c)

Fig. 6 (a) Joint regions cracking configuration of original specimen B; (b) The applied repair procedure with strips; (c) Joint regions cracking configuration of repaired specimen RB

(a) (b) (c)

Fig. 7 (a) Joint regions cracking configuration of original specimen F; (b) The applied repair procedure with sheets; (c) Joint regions cracking configuration of repaired specimen RF



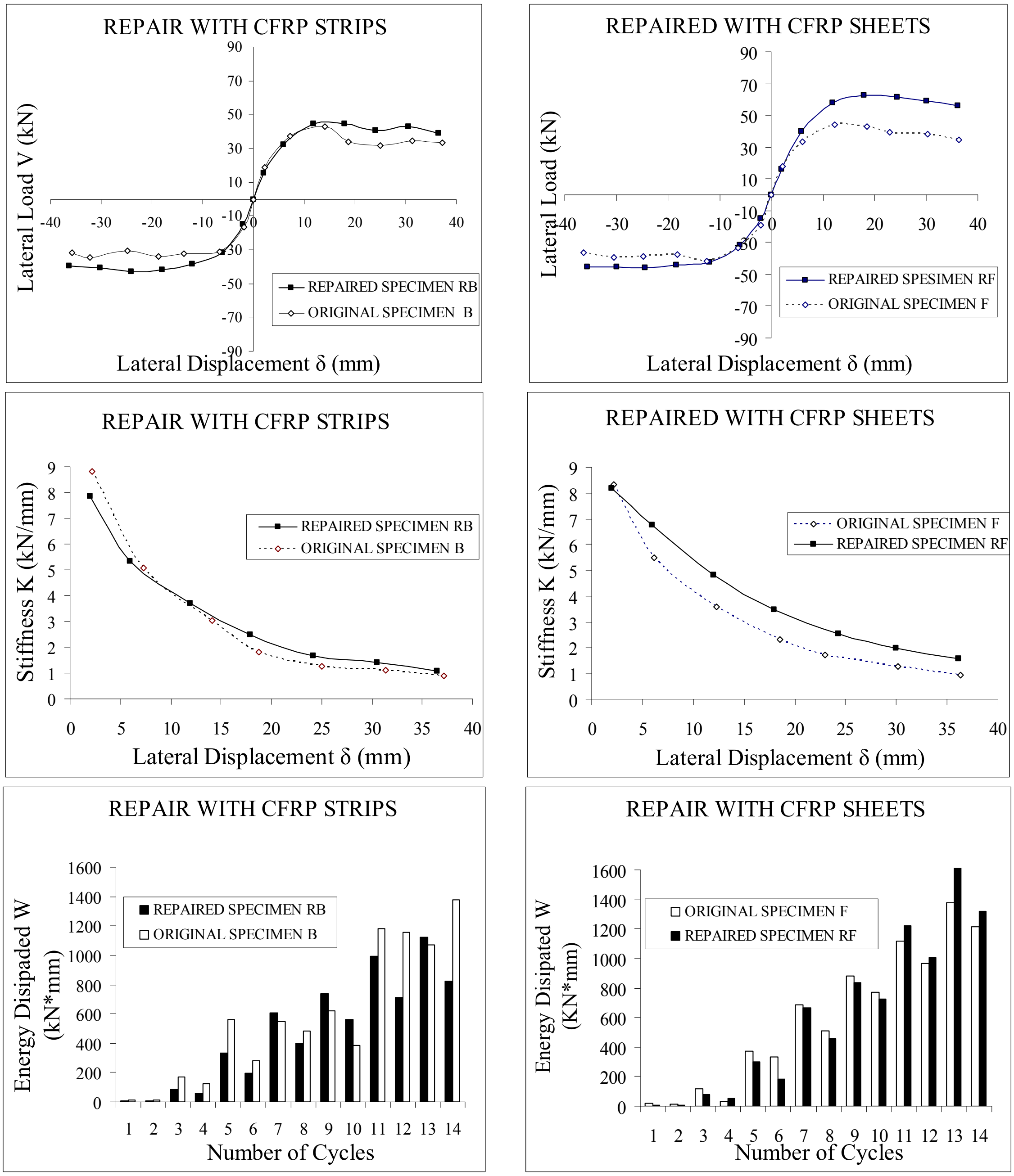
(a) B and RB (b) F and RF

Fig. 8 Load - Displacement hysteresis curves of original and repaired specimens

observed a significant increase of stiffness at all loading cycles. This may be attributed to the strengthening solution adopted herein.

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. 8 (in terms of the area bounded by the hysteretic curve for that cycle) is presented in Fig. 9. The ratios of the measured energy absorption of the repaired specimens to the initial energy absorption of the same specimens are presented in Table 3. Based on these values, it can be deduced that the energy absorption capability of both 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 frame specimen RB, repaired with strips and specimen RF repaired with sheets are 0.82 and 0.85 respectively. For both specimens the apparent improvement of the energy absorption capability of the loading cycles after a drift of about γ=2% (7th-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. In specimen RB, dissipation dropped after a drift of about

γ=2% (9th cycle) because the CFRP plates became very brittle when debonding occured and they buckled under compressive stresses after displacement level of 8th cycle. So, weakened cross sections caused peeling off CFRP at the following cycle, which caused tension on CFRP.



(a) B and RB (b) F and RF

Fig. 9 Comparison of load-displacement envelopes, stiffness, energy dissipation between specimens

Table 3 Comparison of response ratios (repaired / initial)

	Cycles							
Specimen	1st	3rd	5th	7th	9th	11th	13th	Mean
				Load ratio				
RB/B	0.82	0.86	1.04	1.33	1.28	1.24	1.17	1.10
RF/F	0.90	1.19	1.31	1.47	1.55	1.56	1.62	1.37
				Stiffness ratio				
RB/B	0.89	1.05	1.23	1.38	1.32	1.27	1.19	1.19
RF/F	0.98	1.23	1.34	1.52	1.47	1.56	1.64	1.39
				Energy ratio				
RB/B	0.47	0.50	0.59	1.10	1.19	0.84	1.05	0.82
RF/F	0.31	0.63	0.81	0.97	0.95	1.10	1.18	0.85

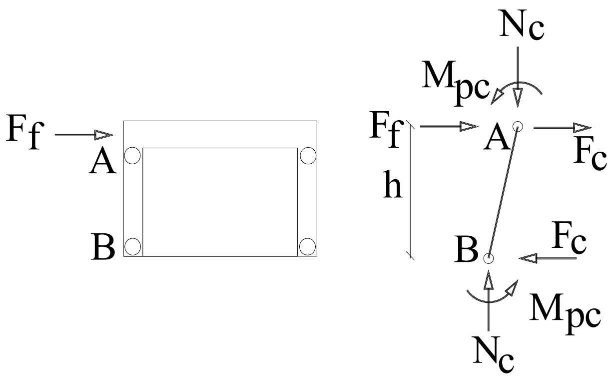


Fig. 10 Failure mechanism

In order to investigate whether the applied repair technique alters the character of the frame failure, that was a ductile flexural one, observed failure modes of both the initial and the repaired frames are examined and compared to each other.

Failure of repaired specimens “RB” is shown in Fig. 6(c). In this specimen, from the first cycles of loading, the cracks of RC frame were concentrated in the column and beam region next to the repaired part of the frame. Subsequent loading cycles resulted in a gradual increase of the width of the cracks across the joint regions, and the observed cracking of the joint body resulted in destruction and removal of the concrete. The delamination mechanism of CFRP plates caused the concrete peeling, because the tensile strength of concrete is much lower than the adhesive one.

Failure of repaired specimens “RF” is shown in Fig. 7(c). 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 beam region next to the repaired part was seriously crushed and fragmented. However, the transverse confinement by CFRP sheets enclosed effectively the longitudinal sheets at their ends and prevent premature concrete peeling-off.

In both specimens, 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. Increasing the bending strength of the column does not allow the development of a ductile flexural plastic hinge in the beam, thus satisfying modern criteria of member strength hierarchy.

Consequently, the main problem faced in this approach is the fact that peeling off at the corners of longitudinally applied CFRP reinforcement does not allow the development of the full composite action. However, the repair with strips as it has been adopted on specimen "RB" appeared to be less effective than the repair of specimen "RF" with sheets due to lack of threaded-rod anchorage and the resulting early delamination that has destroyed the bond around the reinforcing bars.

5. Analytical justifications

By applying commonly used engineering strength assessment techniques, analytical formulas have been derived to evaluate the lateral resistance, F_f, of the identified specimens failure mechanism, as presented in Kakaletsis and Karayannis (2008).

In the mechanism of frame specimens *B*, *RB*, *F*, and *RF*, as shown in Fig. 10, plastic hinges are assumed to develop at both ends of the columns. Assuming moment about A in column AB results in

$$F_c \cdot h = 2 \cdot M_{pc} \tag{1}$$

where F_c=shear force in each column, M_{pc}=plastic moment of the column considering the effect of the axial force, h=H-l_p; H=clear column height; and l_p=plastic hinge length equal to 0.5 times the column depth (Fardis 2009) or the changed one after repair. Hence, considering the equilibrium of the frame in the horizontal direction results in

$$F_f = 4 \cdot M_{pc}/h \tag{2}$$

where F_f=flexural resistance of frame specimen.

For design application, the bond strength of developed CFRP strips or sheets in tension may be calculated using the following expressions

$$U_b = b_j \cdot t_j \cdot \sigma_{jo,\max} = l_b \cdot b \cdot f_{ctm} \tag{3}$$

where $U_b(N)$=the ultimate bond capacity of CFRP, at bond failure, b_j(mm)=CFRP width, t_j(mm)=CFRP thickness, $\sigma_{jo,\max}$(MPa)=maximum tensile stress carried by CFRP, l_b(mm)=development CFRP anchorage length after last plastic hinge crack, b(mm)=member width, f_{ctm}(MPa)=mean tensile concrete strength.

Hence, the additional moment carried by strengthened cross-section is

$$M_{pc,j} = A_j \cdot z \cdot \sigma_{jo,\max} \tag{4}$$

where A_j(mm^2)=area of external reinforcement by CFRP, z(mm)=internal lever arm within the column member and in lieu of a precise analysis, may be set at 90% of the overall member width, $\sigma_{jo,\max}$(MPa)=maximum tensile stress carried by CFRP.

The total plastic moment of the column, M_{pc}, can be expressed by combining plastic moment due to existing steel rebars, $M_{pc,s}$, with plastic moment due to CFRP reinforcement, $M_{pc,j}$, as

$$M_{pc} = M_{pc,s} + M_{pc,j} \tag{5}$$

Table 4 Comparison of experimental and analytical result

Specimen	B	RB	F	RF
Phase	Initial	Repaired	Initial	Repaired
Plastic moment of column, *Mpc* (kNm)	7.70	8.00*	7.70	11.39
Plastic hinge length, l_p (m)	0.075	0.075	0.075	0.07
h=H-l_p (m)	0.725	0.725	0.725	0.73
Frame strength, F_f=4M_{pc}/h (kN)	42.48	44.13	42.48	62,41
Observed strength, $F_{f,exp}$ (kN)	44.27	45.95	44.27	62,77
Strength ratios, $F_{f,exp}$/F_f	1.04	1.04	1.04	1.00

$^*M_{pc}$=$(M_{pc}^A+M_{pc}^B)/2$

For repaired frame RB, M_{pc} are plastic moments at the bottom, A, and the top, B, of the columns, due only to CFRP reinforcement, taking into account that no restoration of the destroyed bond around the reinforcing bars was achieved by epoxy injections, due to concrete peeling, as shown in Fig. 6(c). Assuming a concrete tensile strength equal to 2.50 MPa, and estimating an anchorage length after last plastic hinge crack equal to 100 mm, a portion of CFRP tensile strength equal to 312.50 MPa is developed.

For repaired frame RF, M_{pc} are plastic moments at the bottom, A, and the top, B, of the columns, due to the existing steel rebars and the contribution of CFRP reinforcement, taking into account full restoration of the destroyed bond around the reinforcing bars. Assuming a concrete tensile strength equal to 2.50 MPa, and estimating an anchorage length after last plastic hinge crack equal to 70 mm, a portion of CFRP tensile strength equal to 1988.64 MPa is developed.

The flexural strength capacities computed using these equations are summarized for each of the stages of loading for the frame specimens in Table 4. Analytical predictions of flexural resistance were found to be in good agreement with the experimental results. Also, the lateral resistance of the repaired specimen RF was found about 1.4 times that of the repaired specimen RB, due to the effectiveness of the existing anchorage of longitudinally applied CFRP reinforcement.

6. Conclusions

The author has carried out experimental investigations, under cyclic loads, on two RC frames, providing data for a broad evaluation of two retrofitting techniques. The compared repair techniques of the frames are based on the use of thin epoxy resin infused under pressure into the crack system, and the additional use of CFRP plates or CFRP fabrics, as external reinforcement to the surfaces of the damaged structural RC members (specimens RB and RF, respectively).

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

• Further, in the loading after the repair the character of the failure of repaired with CFRP plates specimen RB tends to exhibit a brittle behavior consistent with the brittle behavior of CFRP due to debonding occurrence. 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.

• On the other hand, a fully wrapped CFRP sheet, as in specimen RF, may offer a proper anchorage to longitudinally applied CFRP reinforcement, allowing the development of a much higher CFRP tensile strength and a significant increase of the lateral resistance.

• The study marks the importance of anchorage of composite sheets and strips in developing the full fiber strength in a small joint area, while, lacking this kind of anchorage could provide a less effective, but a more practical and economical solution.

• Since the strengthen solutions studied seem not allow the formation of the plastic hinges in the beams, the seismic rehabilitation with CFRP strips and sheets seems not to be recommended for RC framed buildings; nevertheless, it seems to be a solution with potential for dual systems.

• Finally, 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 aforementioned yielded conclusions are mainly limited to the study cases and must be used and extrapolated carefully and cautiously.

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