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AN EXPERIMENTAL STUDY ON FRP STRENGTHENED REINFORCED CONCRETE FRAMES UNDER SIMULATED SEISMIC LOADING[*]

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Abstract– The current study focuses on the experimental investigation of FRP strengthening of RC portal frame subjected to combined vertical and horizontal loads. The main target of this research is to enhance the proposed structural properties that the RC portal frame had before the seismic occurrences by providing retrofitting of both columns and joints with more deformation capacity. The effect of various parameters on the effectiveness of FRP is examined through 1/3 scaled testing of 6 single bay RC portal frames. Comparisons between intact and retrofitted specimens are discussed in detail in terms of local and global performance modes. The information about the crack development, the damage characteristics, the hysteretic curves, the skeleton curves of frame and energy dissipation curves were presented. In addition, the strength and stiffness of frames were measured. Test results indicate that the FRP rehabilitated frame shows a good hysteretic energy capacity which indicates that this frame has a better seismic behaviour. The results provide an important insight to the role of FRP in improving the earthquake resistance of frame buildings.

Keywords– Reinforced concrete, portal frame, FRP strengthened frames, experimental, cyclic loading test

1. INTRODUCTION

Seismic performance of existing buildings and bridges is becoming a growing matter of concern, after the devastating earthquakes worldwide experienced in recent years, such as the 1995 Kobe (Japan), the 1999 Kocaeli (Turkey), the 2003 Boumerdes (Algeria) and 2003 Bam (Iran) earthquakes. These events repeatedly demonstrated the vulnerability of those structures that were designed and built based on the old design codes, many of which would be considered inadequate according to today's design codes. In addition, some recently built structures may also have deficiencies as a result of design or construction errors.

In the past decade, many researchers focus on the behavior of FRP retrofitted RC beam-column joints, e.g. Gergely et al. [1], Mosallam [2], Antonopoulos and Triantafillou [3], Prota et al. [4], Mukherjee and Joshi [5], Le-Trung et al. [6] and Parvin et al. [7]. These researches show considerable increase in strength and ductility of the joints. Also, failure modes are transformed from brittle to ductile by shifting plastic hinge region from joint core to beam ends which are more appropriate. Bousselham [8] reviewed and summarized most of the published experimental studies on the seismic rehabilitation of RC joints with FRP and concluded that there are some gaps such as lack of rational explanation of resistance mechanism in FRP retrofitted joints which need to be addressed.

Experimental study of RC frame retrofitted with FRP is so limited because of its difficulty and financial limitations. Balsamo et al. [9] tests a full scale RC dual-system under unidirectional pseudo-

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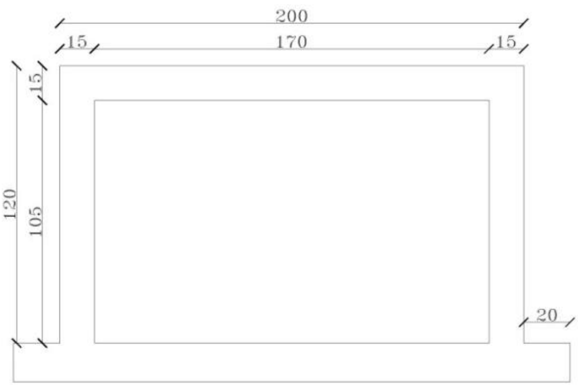
dynamic loading in the ELSA laboratory of the Join Research Center (JRC) in Ispra (Italy).Their experimental program aimed to compare the response of frames designed according to different approaches and to assess the opportunity of using composite materials as an effective technique for the seismic repair of RC frames. Their test shows increase in displacement demand, energy absorption and strength of RC structure due to FRP retrofit. Duong et al. [10] experimentally investigate the behavior of a shear-critical reinforced concrete frame under seismic loading. For this purpose they examine a single-span, two-story, reinforced concrete frame with shear-critical beams under a lateral reverse cyclic manner until severe shear damage took place in the beams. Then beams were repaired with carbon fiber reinforced polymer (CFRP), and the frame was tested again.

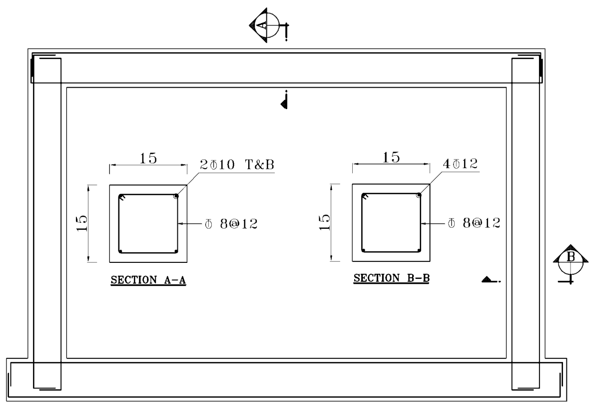
In the current study, the results of a comprehensive experimental program on seismic behavior of retrofitted RC frames are presented. The experimental program was aimed at achieving a fundamental understanding of the behavior of RC portal frames strengthened with composite materials in joint regions under simulated seismic actions, through the investigation of a number of retrofit design parameters. For this purpose 6 RC portal frames with 1:3 scales were constructed, two specimens were selected as control specimen and the other ones were rehabilitated by CFRP and GFRP sheets with different fiber configurations. Control specimens were loaded to failure, then retrofitted with CFRP and tested again. The results are compared in strength and ductility. Moreover, the progressive damage of critical area was monitored and explained for both intact and retrofitted specimens.

2. EXPERIMENTAL PROGRAM

A total of 6, 1:3-scale, reinforced concrete portal frames were constructed and tested. Frame designed was carried out based on ACI318 code [11]. These specimens were typical as-built frames of existing middle-rise residential buildings. Dimensions of scaled specimen are shown in Fig. 1a. Scaling down of the properties of specimens is based on recommendations given by Harris and Sabnis [12].All the specimens were reinforced such that they would represent a poorly detailed exterior joint of a RC frame. To provide a rigid base for columns a RC foundation is constructed and casted at the same time with beam and columns.



(a)



(b)

Fig. 1. Dimension and reinforcement of tested frame

Frames were divided in three groups: 1) two frames as control specimens; 2) four frames as rehabilitated specimens which were retrofitted from the beginning and 3) two repaired specimens (repair of control specimens). All frames were tested under cyclic horizontal loading and vertical gravity load.

a) Description of test specimen

The frame height and span length are 1200 mm and 2000 mm, respectively. The cross sections of the beam and column are 150 mm×150 mm and cross section of foundation is 250 mm × 400 mm. Reinforcement consisted of four, 12-mm diameter rebars in the column, two 10-mm diameter rebars in each side (top, bottom) of the beam, 8-mm stirrups at a spacing of 12 mm in the column. Also, all of the specimens had no stirrups in the joint. Details of the reinforcement are shown in Fig. 1b.

b) Material properties

The compressive strength of concrete is taken 30 MPa. This strength for each specimen was controlled by three 150-mm cubes taken during casting of each frame. The type of steel used for longitudinal reinforcement of column was S400 with average yield stress of 445 MPa. Longitudinal reinforcement of beam and stirrups were made of steel type S300 with average yield stress equal to 330 MPa.

The properties of FRP layers are as follows: **Carbon**: elastic modulus and failure strain of carbon strips 3400 GPa and 0.015, respectively; also density and design thickness of these fibers are 300 gr/m^2 and 0.167 mm, respectively. **Glass**: elastic modulus, failure strain, density and design thickness of glass fibers are 2250 GPa, 0.028, 430 gr/m^2 and 0.17 mm, respectively. These properties were provided by the manufactures for the composite materials

The concrete was prepared using type II Portland cement and crushed aggregates with a maximum size of 12 mm in a water-cement-aggregate ratio of approximately 0.55:1:4 by weight (using Harris and Sabnis [12] diagrams for scaled down concrete). Casting of the specimens took place in steel molds, which were placed horizontally. Because of limitations of workplace space and mold, frames were cast in three dates and at each time two frames were cast. The specimens were cured using wet sacks for 1 week and then left in site conditions. Bonding of the composite materials took place at a concrete age of about 9 months. To ensure a high quality bond between the concrete and the FRP reinforcement, surfaces of RC frames were prepared prior to FRP installation based on ASTM D4258-83 [13] and other related references such as ACI 440.2R-02 [14] and ICRI manual [15]. For this reason the specimens were thoroughly wire brushed until any loose material was removed and vacuumed. Bonding of the sheets took place in several steps, which included: application of a two-part epoxy adhesive on the concrete; bonding of the first FRP layer; application of epoxy and impregnation of the sheet; application of the next layer of sheet, etc. For preventing any inaccuracy in results proceeded by FRP bonding, installation of FRPs was done by an expert from the FRP supplier company.

c) Experimental setup and test fixture

The specimens were tested in a 3D test frame in structure laboratory of civil and environmental engineering department in AmirKabir University of Technology, Tehran, Iran. The testing assembly consisted of vertical and lateral loading systems. Vertical load was applied through a 1000 kN hydraulic jack mounted to the loading frame’s upper beam. Horizontal loading was applied using a displacement-controlled actuator positioned at the beam centerline (Fig. 2a). This actuator was anchored to a horizontal beam of loading frame and had a load capacity of 100 kN and a stroke capacity of approximately ±10 mm.

To provide fixity at the bottom, a RC base post-tensioned to the strong floor in four points by means of jacket and two bolts prior to testing. Also, unexpected slip of frame was prevented by horizontally constraining the basement.

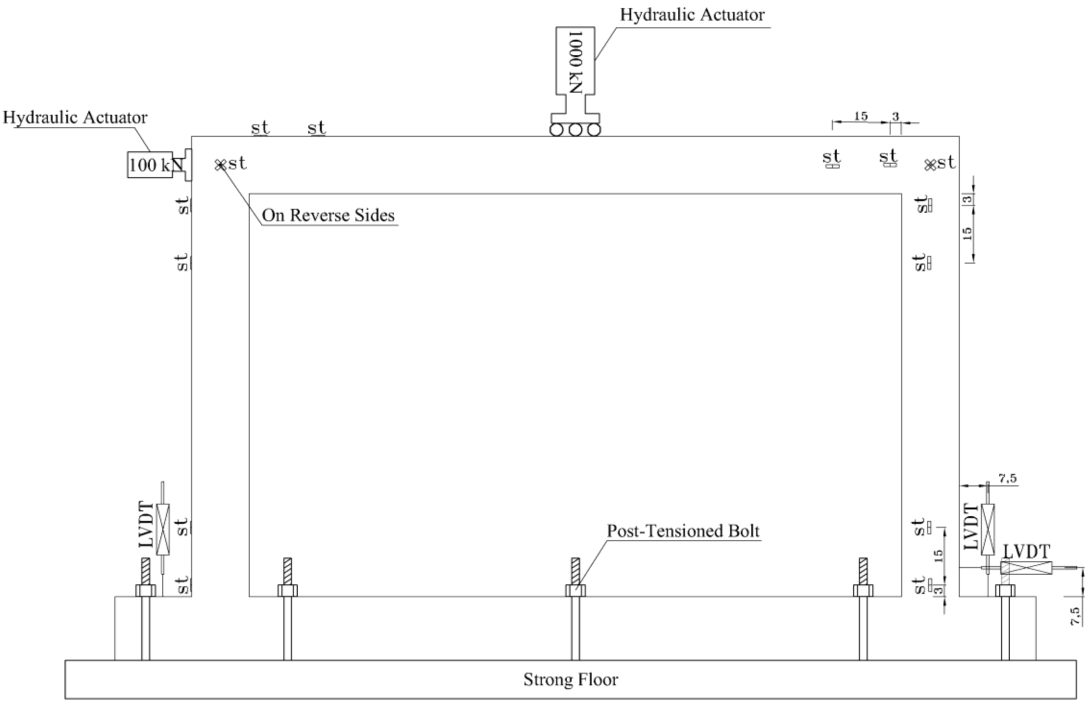


Fig. 2. Frame instrumentation

d) Loading protocol and instrumentation

Testing of each model began by slowly applying loads to simulate vertical loading of the frame. This was accomplished by means of a 1000 kN hydraulic jack. During testing, application of the axial load was controlled manually and kept constant at a level of 10 kN. Once the full axial load was applied, seismic lateral loads were simulated by applying an alternating force to the beam through a loading beam. This force was applied in a quasi-static cyclic pattern using a horizontally positioned 100 kN hydraulic jack. Data from the actuator’s load and displacement transducer were recorded using a computer controlled data acquisition system. The displacement-controlled loading sequence for each specimen consisted of three cycles at a series of progressively increasing displacement amplitudes in each direction (push and pull). The loading history is illustrated in Fig. 3. The loading frequency was selected at 0.05 Hz. Loading history and frequency were selected based on ACI374.1-05 [16] recommendations.

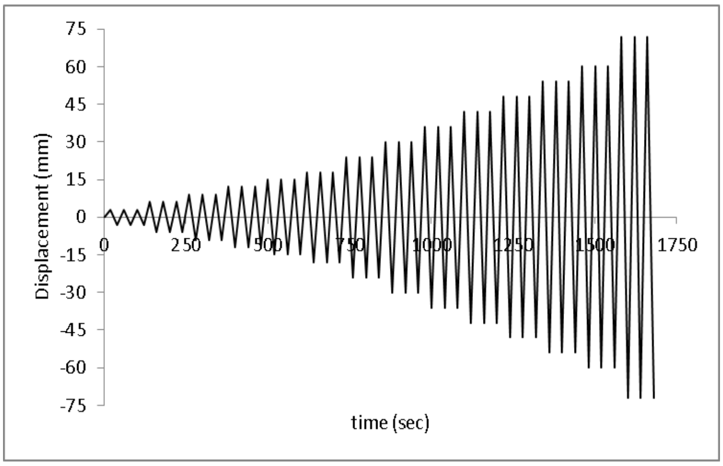


Fig. 3. Loading protocol

Load-displacement behavior of frame was automatically recorded using a computerized data acquisition system. Two types of strain gauges were used in this experiment: 5 mm long gauges for reinforcing concrete (TML PFL-10-11) and 60 mm long gauges for FRP (TML BFLA-5-5). A total of 45 strain gauges were mounted on the specimens, almost 7 strain gauges for each frame, at various locations for measuring strains caused by lateral cyclic loading and performance of retrofit plan. Also, two 25 mm linear variable differential transducers (LVDTs) were placed at frame basement, to investigate foundations unexpected uplift or slip (Fig. 2).

e) Retrofit scheme

The retrofitted specimens were built with different ways of FRP wrapping configuration at the frame's joints and both ends of beams and columns. This was aimed to investigate the effective ways of strengthening and repairing of the frame by applying different layout of CFRP and GFRP fibers. In Table 1 specimens descriptions are presented.

Table 1. Specimen description

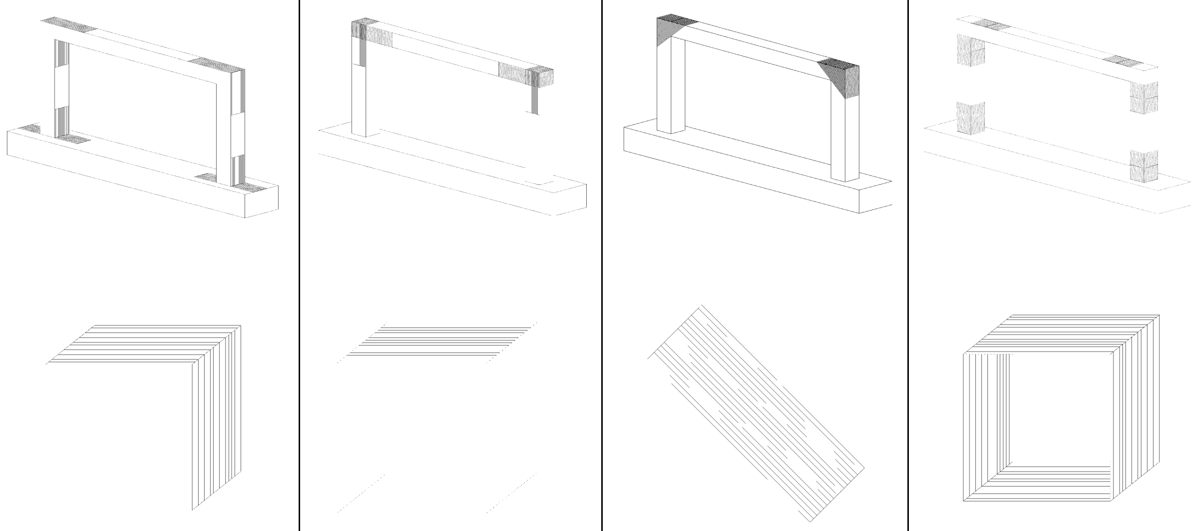
	Specimen	loading	FRP Type	FRP Configuration
1	FC1	H	-	-
2	FC2	H, V	-	-
3	FCLW	H	CFRP	L, W
4	FCUW	H, V	CFRP	U, W
5	FGLW	H, V	GFRP	L, W
6	FGUW	H, V	GFRP	U,W
7	FCWIUS	H, V	CFRP	W, I, US*
8	FCLWI	H, V	CFRP	L, W, I

* US = **U**n-**S**ymmetric

FRP "L" shaped, "U" shaped, 45'-degree inclined and wrapping fibers were used in strengthening of frame joints as shown in Fig. 4. "L"shaped segments were applied to the inner and outer surfaces of beam-column joint and inner and outer surfaces of column base. "U" shaped segments were applied to joint faces in beam and column directions. FRP wrapping is used in both ends of beams and columns for confining and preventing debonding. Also, unidirectional FRP laminates with fiber direction of 45 degree to beam direction were used in joint areas in repaired specimens.

All frames were retrofitted in base level by L-shape and wrap laminates. Also, all retrofit schemes were symmetric in both frame sides, only in specimen FCWIUS were joints and column bases un-symmetric. In this specimen one joint is repaired by 45 degree CFRP fibers and the other end has the same plus CFRP wrap in column and beam end which confine sections and prevent undesirable debonding. Also, column bases are repaired by two L-shaped CFRP fibers and one layer of CFRP wrap is added to one end. It should be mentioned that both wrapped ends were at the same side of the frame.

Bond between concrete and FRP layers in beam and column ends was prevented by FRP wrapping in retrofitted and repaired specimens, and all FRP layers in panel zone were extended as far as bond length (based on ACI440-02 recommendations) in beam and column direction. Furthermore, debonding is a possible mode of failure which could affect the behavior of FRP strengthened frames and study of debonding preventive methods is not included in the current tests because of limitations in the number of specimens.



(a) **L** shaped fibers | (b) **U** shaped fibers | (c) **I**nclined fibers | (d) fiber **W**rapping

Fig. 4. Retrofit Schemes

3. EXPERIMENTAL RESULTS

a) Discussion of test results

Test results illustrate that beam-column joints and column bases are the critical regions in behavior of portal frames under transverse loading. Joints lose their strength and stiffness due to forces imparted to them during earthquakes which can cause complete collapse of the frame. So by considering the shear capacity of the joint as the major parameter controling the frame behavior, the overall safety of frame would be expected to be controlled by this parameter. In the present section, the effectiveness of FRP retrofit in improving frame behavior by emphasizing on joint regions retrofit has been studied. The results are presented and discussed in the following:

The damage occurred in specimens is briefly given in Fig. 5. This failure mechanism was mostly developed in the form of diagonal cracking in the joint and was observed in almost all tests.

b) General behavior

Control Specimen (FC1 & FC2): Figure 5a shows the crack pattern in joint area in specimen FC1. During the loading stages, significant X-shape shear cracks appeared in both joints and both sides. These shear cracks initiated in diagonal directions and propagated toward the ends of joint. Also, horizontal cracks were observed in column base. This specimen was loaded horizontally until 3% drift then repaired by CFRP and retested as specimen FCWIUS.

Specimen FC2 was accidentally loaded (because of a problem in horizontal hydraulic actuator) in push direction until large cracks occurred in joint regions (Fig. 5b) and column bases. This test was canceled and specimen repaired by CFRP and retested as specimen FCLWI.

Strengthened specimen with Carbon FRP (FCLW & FCUW): Specimen FCLW was flexural, strengthened mainly by L-shaped. This specimen has no fiber reinforcement in joint faces, so damage was concentrated in joint region, as expected. X-shaped cracks occurred in joint area as shown in Fig. 5c, also some horizontal cracks were generated in column base. By increasing lateral displacement, crack widths increased gradually and plastic hinges were concentrated in joints and column bases.

(a) FC1

(b) FC2

(c) FCLW

(d) FCUW

(e) FGLW

(f) FGUW

(g) FCLWI

(h) FCWIUS

Fig. 5. Damage patterns of specimens

Specimen FCUW was shear strengthened mainly with U-shaped fibers oriented in both beam and column directions. Cracks in this specimen are generated in joint region (Fig. 5d) and column base. The crack in joint region was nearly horizontal. This crack starts from joint inner corner and growth toward joint outer face till reaching the middle of joint vertical face.

Strengthened specimen with Glass FRP (FCLW & FCUW): Specimen FGLW was strengthened in a scheme similar to specimen FCLW. In this specimen damage was concentrated in joint region, like FCLW and X-shaped cracks occurred in joint area as shown in Fig. 5e, also some horizontal cracks were generated in column base. By increasing lateral displacement, crack widths increased gradually and joint region concrete fractured completely. Plastic hinge locations were concentrated in joints and column bases, but as mentioned before damage in joint regions is much more severe.

Specimen FGUW was strengthened similar to FCUW. Cracks in this specimen are generated in joint horizontal boundaries (Fig. 5f) and column base. The crack in joint region was completely horizontal and in higher levels of loading a vertical crack occurred in column corner and growth downward and at last a strip of column's wrap was debonded. Crack in column's edge could be caused by sharpness of the edge which could be prevented mostly by smoothing the edge. Horizontal cracks start from joint inner corner and growth on joints bottom boundary. Also, like other specimens horizontal cracks occur in column base.

Repaired Specimen (FCLWI & FCWIUS): Specimen FC1 was repaired and retested as specimen FCWIUS. For repair reason, the first cracks were filled by primer, then CFRP fibers were installed. This specimen was the only one in which joint and column base regions were retrofitted unsymmetrically. Failure in this specimen was concentrated in joint region and column base like other specimens, but cracks were different from others. In this specimen a diagonal crack from joint inner corner to its outer corner was formed (perpendicular to fibers direction), also column base, in base which had no wraps, was cracked and swelled. In other specimens swelling and decay of column base was prevented by FRP wrap, which shows its effectiveness.

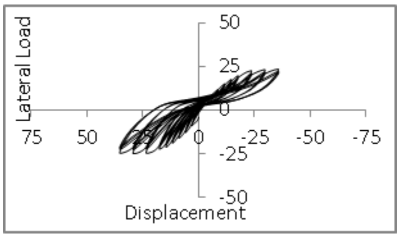
Specimen FCLWI is repaired specimen FC2 which cracked unexpectedly as mentioned before. This specimen was repaired by L-shaped and unidirectional laminates with 45' fibers to beam direction in joint regions, confined by CFRP wrap in column and beam ends. Crack pattern in joint area was similar to FCWIUS and consisted of a diagonal crack beginning from joint inner corner and growth through its outer corner as shown in Fig 5h. Base of this frame also experienced some horizontal cracks.

Table 2. Experimental results conclusion

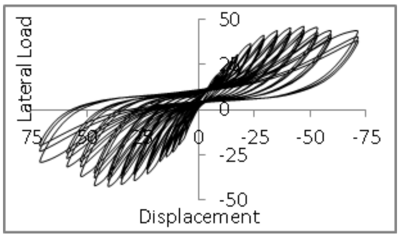
specimen		Max. H. Load	Max. Load increase	Stiffness increase	Energy dissipation			
					0.5% drift	1% drift	3% drift	6% drift
FC1	push	22.88	1	1	35.51	116.63	608.19	-
	pull	25.53			31.27	113.45	674.17	-
FCLW	push	43.3	1.777	1.189	42.5	155.99	955.62	1933.82
	pull	45.3			36.82	143.43	963.15	2022.77
FCUW	push	31.78	1.245	1.235	42.5	137.37	770.56	1477.21
	pull	28.81			38.3	131.80	744.39	1335.56
FGLW	push	48.58	1.903	1.363	36.71	155.38	1076.49	2206.04
	pull	43.7			50.12	160.58	928.40	1939.45
FGUW	push	46.37	1.816	1.757	52.05	161.17	1032.80	2126.68
	pull	44.73			57.59	184.89	1012.29	2045.32
FCLWI	push	39.21	1.535	0.797	25.24	85.47	316.75	762.55
	pull	29.33			27.85	89.58	277.64	542.56
FCWIUS	push	22.65	0.895	0.556	21.59	69.76	220.28	429.09
	pull	22.83			15.68	56.79	204.10	431.38

c) Lateral load-displacement relationship

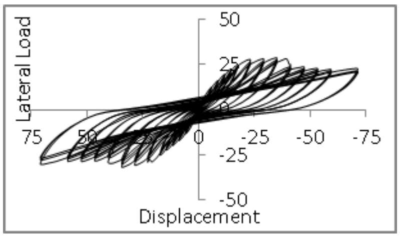
Each specimen was controlled by the displacement at the top of the column provided by the actuator until the specimen reached the required drift level. The behavior of each specimen was monitored during the test. Lateral loads-displacements hysteresis curves for the test specimens are shown in Fig. 6. It is clear from the figure that the responses of the retrofitted specimens were considerably improved compared to that observed from the control specimen.



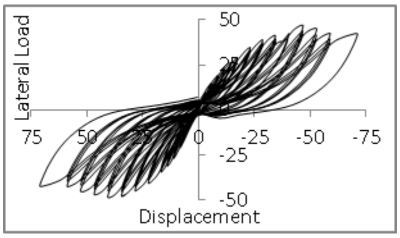
(a) FC1



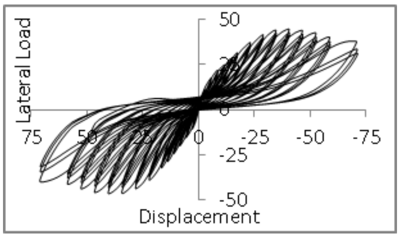
(b) FCLW



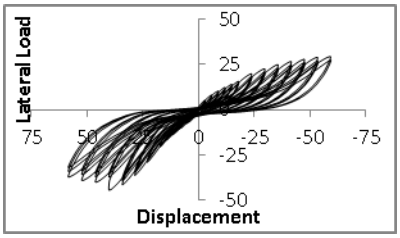
(c) FCUW



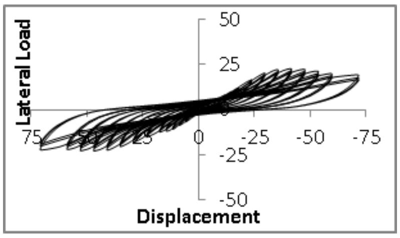
(d) FGLW



(e) FGUW



(f) FCLWI

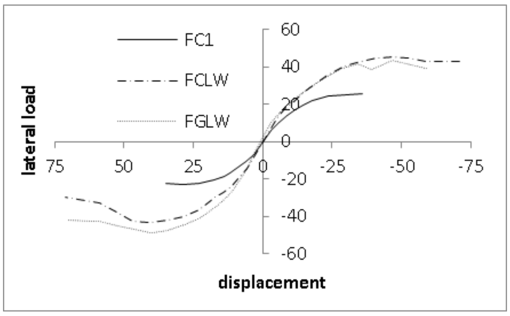


(g) FCWIUS

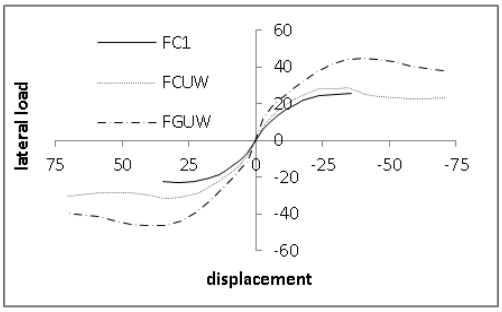
Fig. 6. Load versus displacement curves

In earlier stages of loading and before the cracking of specimen, the hysteretic curve followed a straight line and the deformation is recovered in the elastic deformation stage. After cracking the hysteretic loop gradually tilts toward the horizontal axis (i.e., a fast increase in the displacement rate corresponding to a slow increase in the load rate). With the increase of the lateral load, the area of the hysteretic loop gradually increased, showing some degree of energy dissipation capacity. In the early stages of loading, the area of the hysteretic loop was stable under both push and pull loads. However, the area of the hysteretic loop gradually increased because of the degradation of stiffness in the specimen. A comparison between the hysteretic curves of specimens showed that hysteretic loops of FRP-retrofitted specimens were much wider than the control specimen which demonstrated that these specimens had a better capacity of energy dissipation. The hysteretic loops of control specimen, FC1, showed considerable pinching and severe stiffness degradation. The loss of stiffness was primarily attributed to concrete deterioration in the beam-column joint region.

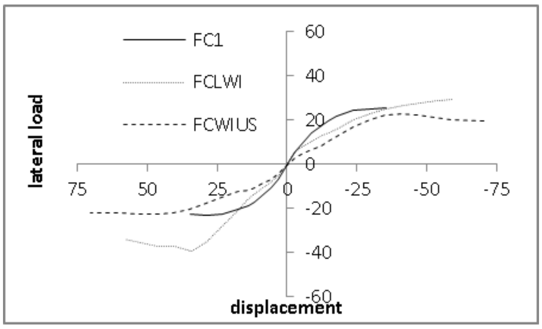
By connecting the vertexes of the first loop of three repetitive cycles at each displacement level, in hysteretic curve, the skeleton curve is formed. This curve reflects the performance of the monotonic loading curve. Lateral load-displacement skeleton curves of test specimens are shown in Fig. 7. As this figure shows, the skeleton curve comprises three distinct phases, which correspond to the three working stats of the specimens, namely the cracking, yielding and ultimate state. As shown in Fig. 7 FRP-retrofitted specimen shows greater initial stiffness in comparison to control specimen. Also, repaired specimen shows lower stiffness in comparison to control specimen which could be because of initial damage in joint region that reduces its initial stiffness.



(a) L-shaped FRP strengthened specimens



(b) U-shaped FRP strengthened specimens

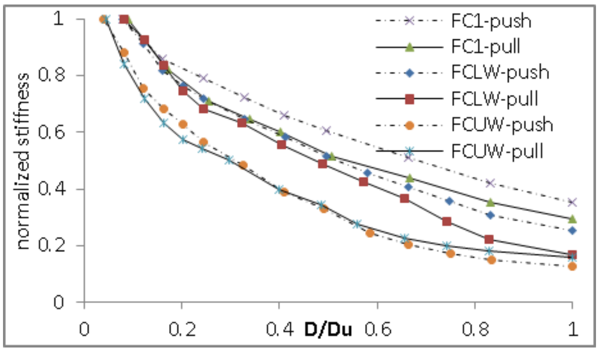


(c) Carbon-FRP repaired specimen

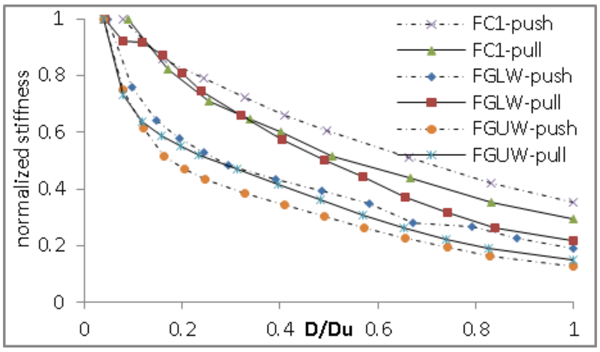
Fig. 7. Lateral load-displacement skeleton curves

With the increase in displacement and the number of cycles, the hysteresis loops tend to be inclined. Figure 8 shows the normalized-stiffness versus normalized lateral displacement relationship for control, retrofitted and repaired specimens. In all retrofitted specimens, it was observed that there was a tremendous increase in stiffness during the first few cycles. After which, there was a gradual decrease in stiffness for the subsequent cycles. This explains the phenomenon of concrete cracking, steel yielding and failure of steel-concrete adherence taking place. The loss of stiffness may be primarily attributed to concrete deterioration in the beam-column joint and column base regions. All the FRP retrofitted specimens had a total loss of stiffness at a higher displacement level than the control specimen. This is a highly desirable phenomenon because the joint collapse can be deferred through FRP strengthening. The GFRP retrofitted specimens have higher initial stiffness and slower rate of degradation.

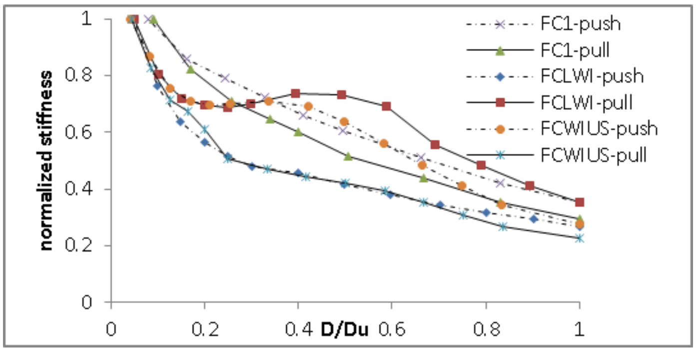
The capability of a structure to survive an earthquake depends on its ability to dissipate the energy input by the ground motion. Forms of energy dissipation include: kinematic energy, viscous damping energy, recoverable elastic energy and irrecoverable inelastic (hysteretic) energy. The cumulative energy dissipated was calculated by summing up the energy dissipated in consecutive loops throughout the test. Energy dissipation of all the specimens under cyclic loading has been calculated from enclosed area under load–displacement hysteresis at different drift ratio as shown in Fig. 9. From Fig. 9 it may be noted that the energy dissipation capacity at high drift ratio is significantly improved in the case of retrofitted specimens compared with the control specimen. Energy dissipation of retrofitted specimens at each loading cycle was found to be quite superior compared to the control specimen. Control specimen was restricted to a drift ratio of 3% and was not imposed to a very large displacement demand at final stage but the retrofitted and repaired specimens were subjected to a large displacement of 72 mm. At the same drift ratio before the final large displacement, cumulative energy dissipation obtained from the retrofitted specimens were almost 10% to 70% more than that produced by the control specimen. But in repaired specimens at the same drift ratio the cumulative energy dissipation was almost 10% to 40% lower than the control specimen. This is because the repaired specimens were already cracked from the earlier test.



(a) Carbon-FRP retrofitted Specimens

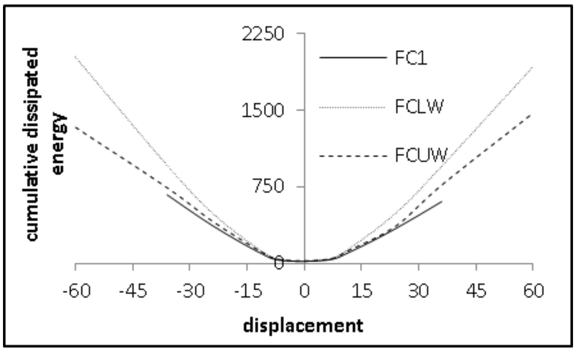


(b) Glass-FRP retrofitted Specimens

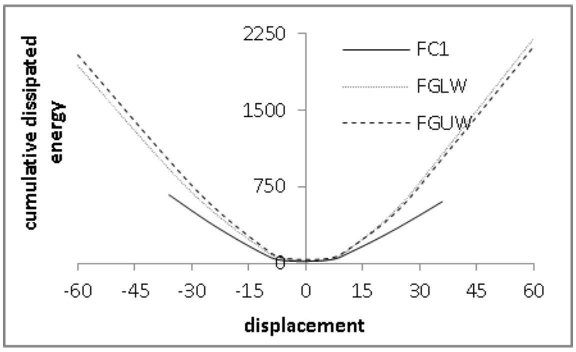


(C) Carbon-FRP repaired specimens

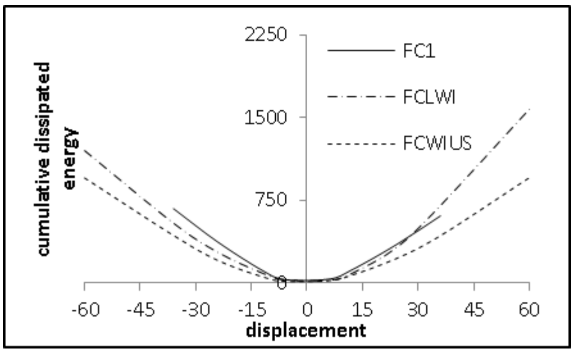
Fig. 8. Curves of normalized-stiffness versus normalized-displacement



(a) Carbon-FRP retrofitted Specimens



(b) Glass-FRP retrofitted Specimens



(C) Carbon-FRP repaired specimens

Fig. 9. Curves of energy dissipation versus displacement

To evaluate the effectiveness of the various FRP reinforcement configurations, the maximum load in push and pull directions, the stiffness (corresponding to the peak-to-peak slope of each first out of three cycles of equal displacement), and the energy dissipation capacity corresponding to the 0.5%, 1%, 3% & 6% drift of frame for every load cycle were recorded.

4. CONCLUSION

A total of six RC Portal frame specimens were tested to study the behavior of FRP-retrofitted and FRP-repaired frames subjected to cyclic loadings. The research focused on the effect of using FRP laminates for enhancing strength and increasing ductility of portal frames. Based on the results of this experimental program, the following general conclusions were reached:

- The control un-retrofitted specimen (FC1) exhibited rapid degradation in both stiffness and strength.
- The use of FRP laminates increases both the stiffness and the ultimate strength of the reinforced concrete moment frame. The strength growth is about 77%, 25%, 90% and 81% for FCLW, FCUW, FGLW and FGUW, respectively, as compared to FC1.
- The use of CFRP laminates to retrofit undamaged frames in specimen FCLW and FCUW has contributed in an increase of about 20%-25% of the initial stiffness, as compared to the initial stiffness of the control specimen FC1. For GFRP-strengthened specimens FGLW and FGUW, an increase of about 40%-75% of the initial stiffness, as compared to the initial stiffness of the control specimen FC1 was observed.
- The initial damage affected the response of the repaired frames. In these frames, the strength was increased about 54% for FCLWI and decreased about 10% for FCLIUS. For both specimens stiffness was decreased. This reduction is about 20% for FCLWI and 44% for FCLIUS. These results indicate that in repaired frame, FRP materials are less effective in terms of frame stiffness as opposed to strength increase.
- In spite of differences in employed Carbon and Glass fibers thickness, which caused a stronger GFRP laminate compared to CFRP one, test results have shown that Glass fibers are more effective than Carbon fibers in terms of energy dissipation, but Carbon fibers are slightly less effective than Glass fibers in strength point of view.
- Shear strengthening of joint region by more layers or thicker laminates could increase frame strength notably and could change frame failure mode.
- In all tests, joints and column bases are the most vulnerable regions for failure and damage are concentrated in these regions. In specimens which had no FRP laminates in joint region (FC1, FCLW, FGLW) X-shaped cracks occurred in joint region. In specimens in which joint region is strengthened with U-shaped laminates (FCUW and FGUW) cracks moved out of joint region and generated nearly horizontal. Finally, in joints retrofitted with unidirectional laminates with 45 degree to beam direction inclined laminate crack pattern was diagonal and its direction was perpendicular to fibers direction.
- Repair of frames with FRP not only can retrieve damaged frames initial condition, but also can enhance its strength and ductility, which depends on repair scheme.

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