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Retrofit of shear deficient reinforced concrete beam-column joints

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ABSTRACT: Many older reinforced concrete structures were built using inadequate construction practices with no consideration for lateral forces leading to their collapse in the case of a strong earthquake. Seismic vulnerability of these structures can be avoided if proper measures are taken in upgrading them. Fiber reinforced polymer (FRP) materials have proven to be effective for retrofit of substructure components. While many investigations have been conducted on retrofit of beams and columns, limited studies address the FRP retrofit of beam-column joints with insufficient and nonseismic internal reinforcement details. Furthermore, most of these studies concentrate on FRP strengthening of undamaged joints. The present study involves large-scale experimentation on shear deficient reinforced concrete (RC) beam-column joints under simulated earthquake loading. Carbon fiber-reinforced polymer (CFRP) sheets were used to strengthen the joints for pre- and post-damaged scenarios. The focus of this paper is to investigate the effect of repair materials on the response of the damaged, repaired and then strengthened specimen and compare it to the specimen with no damage but strengthened with identical FRP wrapping configuration. The test results of both specimens are also compared to control specimen to understand the effectiveness of repair materials in the case of pre-damaged joint and to see if the FRP retrofit scheme could resolve the unconfinement issues associated with pre-1970 beam-column joints design. The failure mode, yielding of steel bars, along with the rotation of the joint, and stiffness and energy dissipation levels before and after retrofit are discussed in this paper. The CFRP retrofit was able to control the joint rotation and increased the lateral load capacity significantly for both pre and post damaged CFRP-retrofitted beam-column joints.

1 INTRODUCTION

Vast majority of older RC buildings have nonseismic design flaws and can experience severe damage during an earthquake causing loss of life and properties, and extensive expenses. Instead of demolishing and reconstructing, strengthening of these buildings would be cost efficient and would prevent large damage and collapse when subjected to earthquake loads. Beam-column joints play a critical role in the stability of these pre-1970 deficient building frames. There are two scenarios for retrofitting. One is preventive measure such that the deficient beam-column joints are strengthened with FRP sheets prior to an extreme event. The other scenario is that the beam-column joints have experienced some damage after an earthquake. Therefore they are repaired and strengthened with FRP sheets.

Some studies have been performed on FRP-strengthening of beam-column joints. However, majority of these studies relate to FRP retrofit of undamaged joints as a preventive measure to protect them against future earthquakes (Alsayed et al. (2010), Del Vecchio et al. (2014), Garcia et al. (2014), Ghobarah and Said (2002), Granata and Parvin (2001), Pampanin and Akguzel (2011), Parvin et al. (2010), Sezen (2012)). The present study looks at deficient beam-column joints that have been damaged under an earthquake load. Repair materials were used to fill the cracks. After the repair, the beam-column joint was retrofitted with FRP sheets. The results of the joint were also compared with counterpart undamaged FRP-retrofitted and control RC joints to understand the differences in their behavior due to repair materials and FRP application under simulated seismic loading. The specimens were constructed according to pre-1970 codes with some deficiencies such as widely spaced column ties, no transverse reinforcement within the joint, and construction joints below and above the beam column joint (Beres et al. (1992)).

In the following sections, test set-up and material properties for three beam-column joints namely US-1 control, US-1 FRP (undamaged joint retrofitted with FRP sheets), and US-1 RFRP (damaged joint repaired and retrofitted with FRP) are presented.

2 EXPERIMENTAL PROCEDURE

The beam-column joint specimens were designed according to 1971 American Earthquake Code (ACI-71) where the stirrup spacing was 14” (350 mm) in the column and 6” (150 mm) in the beam regions. No stirrups were installed in the joint panel. Two construction joints were done below and above the beam. The specimens were tested with the beam in horizontal position and hinged at the free end. The column was in the vertical position and supported by universal pin at the bottom end. Constant axial load was applied by a load control actuator, while lateral displacement control load was applied using a horizontal actuator. The amount of the axial force applied was 30% f''_c A_g where f''_c is the compressive strength of concrete and A_g is gross cross-sectional area of column. A cyclic lateral load was applied at the top of the column to simulate an earthquake load.

2.1 Material Properties

Concrete strength for the beams was 24 MPa and for the columns was 26 MPa. Deformed bars were used for transverse and longitudinal reinforcement with the yield strength of 438 MPa and 449 MPa, respectively. The CFRP material, *C Fibre 160*, produced by Degussa was used (Table 1).

Table 1. Properties of CFRP

Nominal Thickness	0.33 mm/ply
Ultimate Tensile Strength (0°)	3800 MPa
Tensile Modulus (0°)	227 GPa
Ultimate Rupture Strain (0°)	1.67 %

Concresive 1495 epoxy based repair material was used for filling the wide cracks at the damaged region. The bond strength of *Concresive 1495* is very high. Therefore, it assures the closure of each crack opening in the specimen after it was injected into the core through small pipes. The damaged region were reconstructed using *Emaco S88C*, a cement based repair material. After curing process, *Concresive 1302* was injected to fill smaller cracks up to 1 mm.

Due to its low viscosity, it can penetrate the hairline cracks easily even with a low pressure. Next, another type of epoxy based repair material, *Concresive 1406*, was used for replacing lost concrete at the joint region. Prior to repair material injection, joint core was covered with *Concresive 1406*. Mechanical properties of *Concresive 1406*, *1302* and, *Emaco S88C* are given in Tables 2-4, respectively.

Table 2. Properties of Concresive 1406

Application thickness	2 mm – 30 mm
Bending strength	25 MPa (7 days)
Compressive strength (20°)	75 MPa (7 days)
Bond strength	3.0 MPa (concrete) 3.5 MPa (steel)

Table 3. Properties of Concresive 1302

Viscosity	100-350 MPa.s
Tensile strength	45 MPa (7 day)
Compressive strength (20°)	110 MPa (7 day)
Bond strength (steel and concrete)	3.5 MPa
Application thickness	0.2 – 1.0 mm
Modulus of elasticity (0°)	3.1 - 3.3 GPa

Table 4. Properties of Emaco S88C

Application thickness	Min. 10 mm
Tensile strength (20°)	3.6 MPa (28 day)
Compressive strength (20°)	70 MPa
Bond strength (steel)	14 MPa (smooth bar) 30 MPa (ribbed bar)
Bond strength (concrete)	6.5 MPa
Modulus of elasticity (0°)	>28 GPa

2.2 Reinforcement Details

The main deficiencies which were studied for beam-column joints were widely spaced column ties that provide little confinement to the concrete, no transverse reinforcement in the joint region and construction joints below and above the beam column joint. All three specimens had 3 ϕ20 longitudinal bars at the top of the beam and 2 ϕ20 at the bottom. The column was reinforced with 6 ϕ20 bars in longitudinal direction. ϕ12 bars were used as ties for both column and beam with different spacing according to the previous code requirements. The support regions for beam and column were reinforced very dense in order to prevent possible failure in these regions. Figure 1 shows the internal reinforcement deficiencies for all three beam-column joint specimens identified as US-1 (control specimen), US-1 FRP (undamaged and FRP-strengthened specimen), and US1-R-FRP (damaged, repaired, and strengthened with identical FRP configuration as undamaged strengthened specimen).

Figure 1. Beam-column joint reinforcement arrangement.

2.3 FRP Wrapping Orientation for US-1 FRP and US1 R-FRP Joints

At first, the FRP sheets were wrapped around the joint region diagonally to provide shear reinforcement for the joint core. As a second step, L-shaped overlays were applied to the top and bottom face of the beam and column face. Third, the columns were wrapped with FRP sheets to overcome the lack of confinement associated with widely spaced column ties, and in order to increase the ductility. Forth, a U-shaped FRP wrap was applied to the joint and was extended 30 cm on the beam face. The purpose of this application was to provide additional confinement and to decrease the shear damage at the joint. The fifth step involved the application of the anchorage for the L-shaped overlays and U-shaped wraps. A 30 cm width of the FRP sheet was wrapped around the beam. This application provided also some shear reinforcement for the beam in addition to its main purpose to serve as anchorage.

2.4 Repairing Technique for US1 R-FRP Joint

Repair procedure was performed after the test of the control specimen. Loose concrete pieces were removed and the surface and the cracks were cleaned. The big visible cracks were filled with epoxy based high strength repair material, *Concresive 1495*. The damaged region was repaired with cement based repair material, *Emaco S88*. After curing process, low viscosity epoxy *(Concresive 1302)* was injected. When the injection was completed the concrete surface was prepared for FRP wrapping. The repaired joint was strengthened with identical FRP configuration as undamaged strengthened specimen.

3 COMPARISON OF AS-BUILT, UNDAMAGED FRP-RETROFITTED, AND DAMAGED REPAIRED AND FRP-RETROFITTED JOINTS RESULTS

Hysteresis curves of all joint specimens are compared in Figure 2. When the behavior of the control specimen was examined, the location of the failure was within the joint core due to shear cracks prior to the yielding of the steel bar reinforcements. The ductility and energy dissipation levels were not satisfactory while the maximum load of 75.8 kN in pull and 60.40 kN in push directions were below the calculated loads.

For US1-FRP retrofitted joint, due to the stroke limitations, the test was stopped at the drift level of 3.5% where the maximum loads of 100 kN in push and 141 kN in pull were recorded.

No strength degradation was observed. The CFRP wrapping configuration was able to deter the occurrence of shear cracks at the joint core as witnessed by large hysteretic loops and no pinching effect. As a result, the reinforcement bars reached the yield point. As compared to US1 control specimen, the maximum load levels were increased by 86% and 66% for US-1 FRP specimen in pull and push directions, respectively.

For US1-RFRP joint specimen, diagonal FRP sheets were debonded at 1.40% drift level. The FRP was ruptured at the bottom corner and a 5 mm wide crack was developed at the top corner of the beam column intersection at 2.20% drift level, when the maximum load was 95 kN. Additionally, the FRP sheets were debonded from the beam surface.

The load increase for the damaged and repaired specimen (US1-RFRP) was 69% in pull and 57% percent in push as compared to the control specimen. These results show that the maximum load carrying capacity was increased to a very satisfactory level even for the damaged and repaired specimen. However after 1.5 drift level, strength degradation was observed. The steel bars yielded much earlier than the previous case of US1-FRP joint in the push direction when the displacement was about 10 mm as oppose to 40 mm. Although considerable deformation was observed in the joint region of US-1 RFRP, there was no bond strength loss between concrete and reinforcing bars after the repair. That is another evidence of the satisfactory repair methodology.

When examining the hysteretic behavior, as compared to the FRP-retrofitted, and repaired and FRP-retrofitted specimens, the control specimen showed a severe pinching and strength degradation after 1% drift level followed by spalling of concrete and subsequent formation of plastic hinge in the joint region leading to failure of the joint. No considerable pinching and strength deterioration was observed in the last cycles of US-1 FRP which was due to the formation of the plastic hinge in the beam rather than the joint core. Although the strength was recovered in the repaired specimen without any significant pinching, there was a degradation of strength at 1.5% drift level followed by debonding and rupture of the FRP sheets. Additional wrap confinement may be suggested for the case of repaired and FRP retrofitted joint.

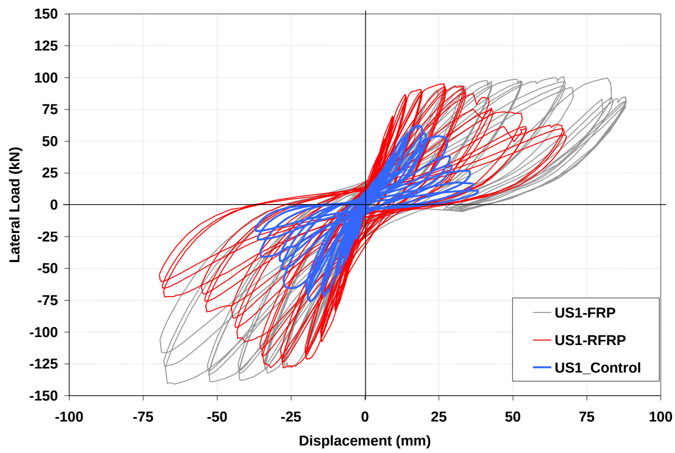

Figure 2. Lateral load versus displacement.

The shear deformation versus lateral load curves of all three specimens are also compared in Figure 3. The FRP retrofit could control the shear deformation and increase the load capacity, more so for the undamaged and FRP retrofitted than the damaged repaired FRP retrofitted joints. Furthermore, the stiffness degradation of control specimen occurred at much lower drift level as compared to the other two FRP retrofitted specimens. The FRP retrofit was able to delay the degradation of the beam-column joints, more so for the undamaged specimen than the repaired one.

The beam-column joint stiffness for all specimens was approximated by calculating the slope of the peak to peak line in each hysteresis cycle. Since there were three cycles in each drift level, the averages of these values versus drift curves were plotted in Figure 4. Stiffness degradation for these specimens were due to nonlinear deformation, flexural and shear cracking, distortion of the joint, loss of cover, FRP sheet debonding and rupture. Although US-1 Control specimen initial stiffness was reasonably adequate, but as the load increased, there was a sudden degradation of stiffness, which was caused by the formation of cracks in the joint and loss of force transfer between the column and beam members, showing a brittle behavior.

In Figure 5, the stiffness values at each drift level of all three specimens are compared. In the case of US-1 RFRP which the damaged joint was repaired and then retrofitted with FRP, the initial stiffness values up to 1.5% drift level were on the average about 20% higher than US-1 FRP. This can be due to the properties of repair materials (see Tables 2-4). However, from drift level 1.5 and up, the decrease in stiffness level of US-1 RFRP fell below US1-FRP joint. This was observed in the experiment. After the failure of the repair material, there was large deformation leading to high tensile stresses in the FRP sheets, resulting in rupture of FRP sheets and subsequent specimens' failure.

Table 5 presents the stiffness levels and percent differences in stiffness and degradation of US-1 FRP and US-1 RFRP with respect to the control specimen. At the very beginning of the tests, there was a stiffness increase of 8% and 15% for US-1 FRP and US-1 RFRP over control specimen, respectively. As the displacement increased, this difference increased gradually for both specimens. At higher displacement level, US-1 RFRP specimen degradation rate was higher than US-1 FRP.

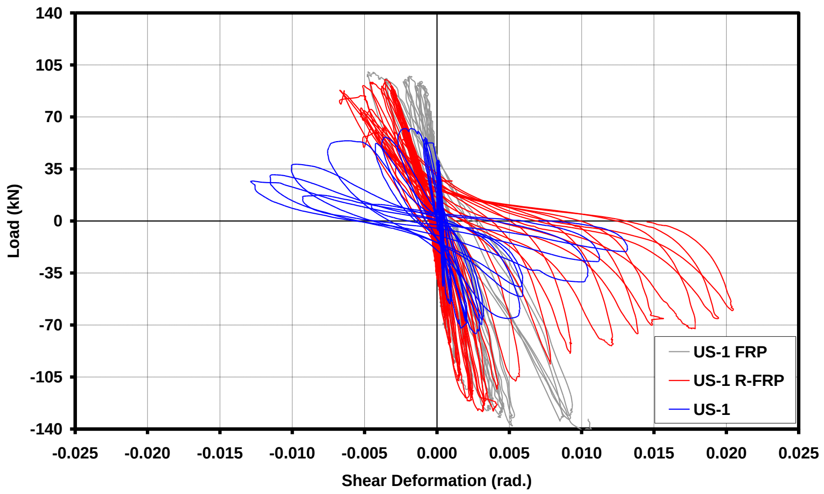

Figure 3. Shear deformation versus lateral load.

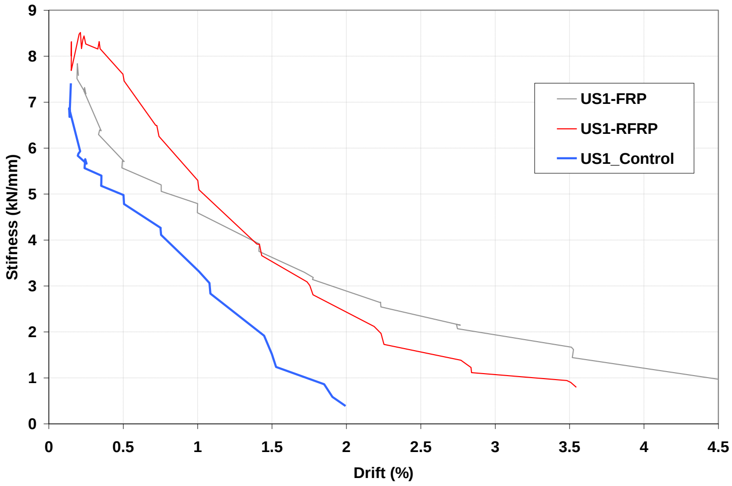

Figure 4. Stiffness degradation.

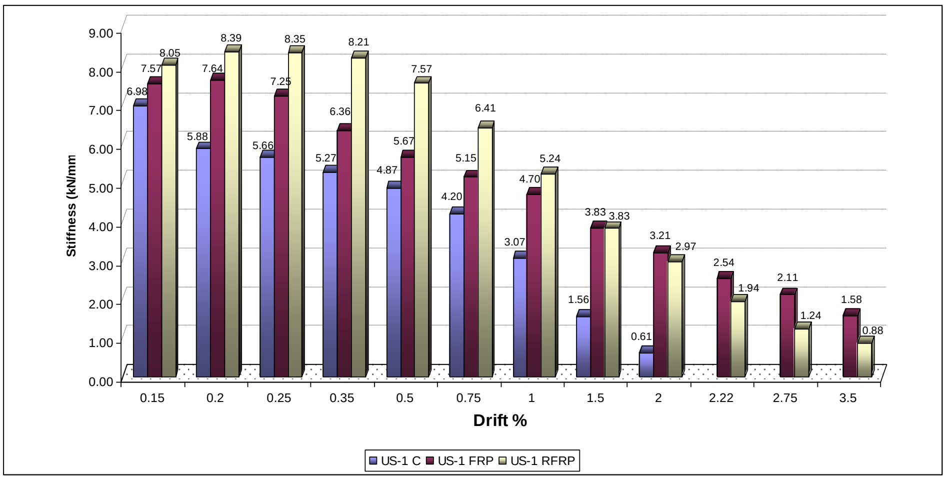

Figure 5. Specimens stiffness value at each drift level.

Table 5. Stiffness and stiffness degradation at various displacement levels

Specimen	2.88 mm	Percent difference	19.25 mm	Percent difference	Percent degradation	42.5 mm	Percent degradation
US-1 C	6.98	-	3.07	-	56.04	NA	NA
US-1 FRP	7.57	8.38	4.70	53.10	37.90	2.54	66.38
US-1 RFRP	8.05	15.22	5.24	70.57	34.92	1.94	75.92

The area enclosed by a hysteresis loop at a given cycle represents the energy dissipated by the specimen during that cycle. The ability of a structure to dissipate energy plays a big role in its behavior under earthquake load. There are three main factors that compose the total energy dissipation. These are a) friction along existing cracks in concrete b) yielding of reinforcing bars and c) occurrence of new cracks. Figure 6 compares the total energy dissipated by US-1 Control, US-1 FRP, and US-1 RFRP specimens. Comparing the total energy dissipated, US-1 FRP had the highest energy dissipation which was 6.52 times more than the control specimen, while US-1 RFRP was 5.73 times higher. Although US-1 RFRP had lower level of energy dissipation as compared to US1-FRP, but still it was satisfactory and it revealed that the FRP strengthening and repair technique is effective for pre-damaged as well as undamaged buildings.

Figure 7 gives the average energy dissipation at each drift level for all joint specimens. Up to 1% drift level, US-1 Control has higher energy dissipation than US1-FRP retrofitted specimens. As it was mentioned earlier, energy dissipation correlates to formation of new cracks and yielding of steel. Therefore, the presence of FRP in US1-FRP joint prevented the steel bars deformation and crack formation. However, for the control specimen these phenomena occurred in early stages of experiment. Furthermore, US-1 RFRP had significantly high amount of energy dissipation even in the early stages of experiment due to friction between preexisting cracks, and the deformation of steel reinforcing bars. At the higher drift level (2.75 and above), the amount of energy dissipated by US1-FRP surpassed US1-RFRP specimen, drastically.

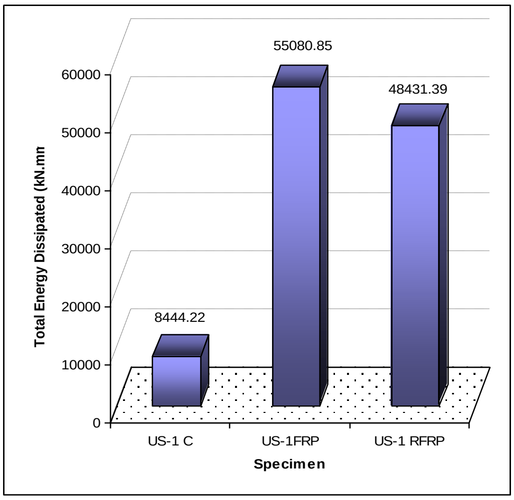

Figure 6. Total amount of energy dissipated by each specimen.

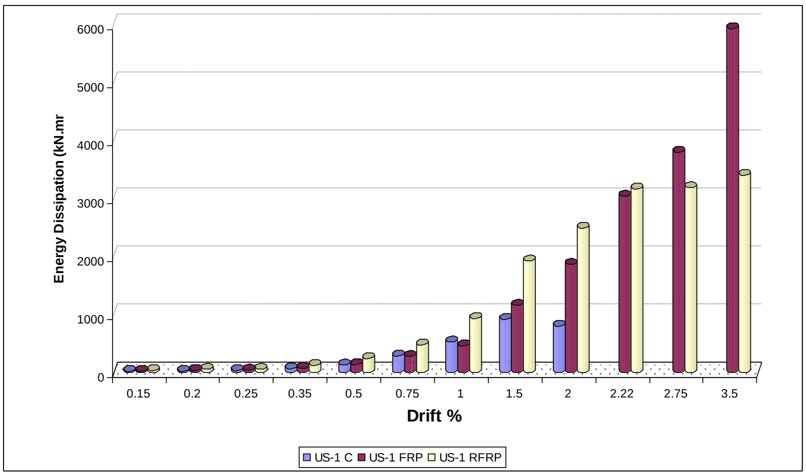

Figure 7. Dissipated energy at each drift level for all specimens.

5. CONCLUSIONS

The results showed that FRP retrofitting system did not only increase the maximum load carrying capacity, but also kept the stiffness integrity of the beam-column subassemblies. The shift in the plastic hinge location also showed that with proper FRP retrofit configuration, it is very possible to shift the location of the plastic hinge away from the joint core and to create a ductile system component which will help buildings to survive under earthquake loadings. At low drift level CFRP retrofitted specimens dissipated almost equal amount energy as that of control specimen. However, after ultimate drift level of the control specimen, CFRP retrofitted specimens dissipated a significant amount of energy. Although a very ductile behavior could not be obtained for the damaged repaired and CFRP retrofitted specimen, the load carrying capacity level was still satisfactory. From the experiment observations, if the integrity of wrapping remains intact, it may be possible to obtain a ductile behavior for repaired and retrofitted beam-column joints. This indicates that even a heavily damaged joint could be repaired and then retrofitted with FRP sheets to resist extreme seismic loadings.

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