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AN EXPERIMENTAL STUDY OF RC COLUMNS WITH CAPTIVE COLUMN SHEAR FAILURE REPAIRED BY CFRP

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Abstract

Seismic response of a RC (reinforced concrete) bare frame, a similar frame with partial infills causing captive column defect and repair of the damaged specimen with CFRPs (Carbon Fiber Reinforced Polymer) are investigated in this study. This study mainly aims to investigate the effectiveness of the repair scheme with CFRPs in terms of response quantities such as strength, ductility, dissipated energy and stiffness degradation. Therefore, two 1/3 scale, one bay, one story RC (Reinforced Concrete) frames, which have certain deficiencies resulting from low strength concrete and plain round bar were designed to represent the existing Turkish building stock. Bare frame, which is also reference specimen of this test series, was constructed without infilled wall. On the other hand, the captive column failure described in Turkish Earthquake Code 2007 (TEC 2007) is expected in the second specimen built with partially infilled wall. Severe damage was observed with the concentration of shear cracks in the columns of the specimen built with partially infilled wall. Then, the damaged members were repaired by CFRPs. Damaged RC columns were wrapped by CFRP sheets both in the longitudinal and transverse direction. Repaired specimen was retested to identify how effective the CFRP wrapping is in repairing the shear damage of captive columns. Reversed cyclic lateral displacement under combined effect of axial load was applied to the top of the column. After testing the specimens, flexural cracks were dominated the overall response of the bare frame. A brittle shear failure in the column top ends was observed in the specimen with partially infilled wall. It is found that the former capacity of the damaged specimen was recovered by the applied repair scheme.

Keywords: RC Frames; Captive-column defect; CFRP; Repair

1. Introduction

Recent earthquakes have exposed that the majority of the existing RC buildings in Turkey have common deficiencies such as inadequate material quality, improper design applications and detailing in RC members in contrast with the earthquake resistant design principles [1]. The above-mentioned one or more deficiencies can cause a non-ductile behavior for the corresponding buildings. In order to prevent the brittle type of behavior for RC buildings, Turkish Earthquake Code (TEC 2007) has imposed several limitations on the formation of structural irregularities in the plan and elevation as well as non-ductile applications such as captive column, short column, strong beam-weak column [2]. During the high intensity ground motions, buildings can be exposed to moderate to severe damage according to the structural deficiencies. Depending on the level of structural damage, damaged building can either be demolished or repaired to satisfy the code specified serviceability as well as the life-safety limit states. By applying an appropriate repairing technique, the seismic capacity of the damaged members can be recovered or even improved compared to its original capacity.

Captive column defect is mostly due to the presence of openings for strip window provided in infill walls between the columns. A brittle type of shear failure can be caused by captive column defect. Infill walls constrain the lateral displacement of the adjacent columns and hence clear height of the corresponding columns becomes shorter, which cause a substantial increase in the column stiffness. Consequently, such columns, namely captive columns, attract excessive amount of shear forces, before attaining the flexural capacity of the columns.

Many researchers have investigated the shear failure caused by short and captive column defects. A study was presented to prevent the short and captive column defects with an interdisciplinary solution from engineering, architecture and construction point of view. The Factors that might cause short-column and captive-column defects were explained in the work and shear damages were examined after various earthquakes. Moreover, the behavior of the frames with short-column and captive-column were explained in detail [3]. During Adana-Ceyhan Earthquake in Turkey (1998) the outer columns of an industrial building were exposed to shear damages due to captive column effect. A finite element model was developed and the effect of the opening ratio on the infill wall was examined analytically [4]. Another study discussed the stiffness behavior and the effect of shear in partially infilled RC frames. An equation was adopted for determining the stiffness behavior and shear effect with partially infilled RC frame within the elastic limits. It was concluded that partially infilled walls improve the stiffness response but columns might be damaged by the high shear forces [5].

In case of column shear damage due to the formation of captive column with the partial infill walls, the corresponding building should be either demolished or repaired with an appropriate technique to meet the serviceability and life-safety criteria. In this study, it was planned to repair the damaged columns by FRP sheets. In the previous studies, although retrofitting RC members with FRPs were experimentally investigated and positive results were observed, the use of FRP for repair purposes is limited compared to studies with retrofit applications [6, 7, 8, 9, 10].

Considering the structural repairing studies in the literature, scaled tests were performed with slabs, beams and columns to examine the performance of RC buildings after repairing them with FRP [11]. In the previous studies, CFRP sheets were mostly used for retrofit and repair purposes for various RC structural members. However seismic repair of captive column damage by CFRPs was not investigated. An experimental study on the repairing of captive column failure by GFRP (Glass Fiber Reinforced Polymer) was conducted. The conclusion of the study was the retrofitted frame with GFRP wrap exhibited an enhanced performance with adequate confinement. This increased the shear strength and improved the seismic performance of the captive-column but it was limited by the bond between concrete and GFRP interface [12].

This research program was conducted to find the behavior of a frame having captive column defect and the effectiveness of the CFRP sheets used for repairing the heavily damaged captive columns. For this purpose, two 1/3 scale, one-story one-bay substandard RC frames were constructed. After testing the RC frame with partial infill wall, the damaged columns were repaired by CFRP sheets and retested with the same loading protocol. In order to examine the effectiveness of the repairing technique, experimental results of the bare frame, the frame with captive column and the repaired frame were compared in terms of various response quantities such as strength, ductility, dissipated energy and stiffness degradation.

2. Experimental program

Two 1/3 scale, one-story one-bay RC frames, all of which were designed with certain deficiencies such as low strength concrete, plain bars with improper detailing, to represent the existing substandard RC buildings in the Turkish RC building stock were tested to investigate their performance. The project consisted of three testing series: The first test specimen (IE01) was the bare frame, which was considered as the reference specimen. The reference specimen was constructed without infill wall. The second test specimen (IE02) was constructed with a partial infill wall between the columns leaving an opening at the top to cause captive column defect. The ratio of the opening height to the wall height is approximately 25%, which is a typical ratio for a strip window in existing RC buildings. The partial infill wall was constructed by a mason, who is actively working in the building constructions for constructing infill walls. Hence the infill wall was constructed in the test specimen IE02 represents the infill walls in the existing RC buildings. The geometric dimensions of the brick units and the overall infill wall dimensions were scaled in accordance with the scale of the RC frame. After the severe damage was observed with the same loading history of the reference specimen, damaged IE02 specimen was repaired with CFRP sheets and the repaired specimen was named as IE03. Before repairing the IE03 specimen, the damaged infill wall was removed from the frame and the damaged columns were wrapped with CFRP sheets. After the repair, no infill wall was constructed between the columns of the IE03 to prevent the recurrence of the captive column defect. In real life applications, in case of any requirement for the construction of infill wall with the opening resulting in a captive column defect, the partial infill wall should be separated from the frame with sufficient amount of gap between the frame. At the same time, the out-of-plane failure of the infill wall should be prevented by taking relevant precautions. This will enable the frame to behave like a bare frame without formation of captive column defect. With this approach, the infill wall was removed after testing IE02 and not reconstructed in the repaired specimen. All the three tested specimens were subjected to similar lateral displacement loading protocol and vertical loading. General view of the tested specimens is shown in Fig. 1.

Fig. 1 - Properties of the test specimens

After the severe damage was observed at the IE02 specimen, the corners of the columns and beam were rounded with a radius of 10 mm to get smooth corners (Fig. 2(a)). To fill the large shear cracks that were observed at the top of the columns, an epoxy acrylate resin called chemical anchorage was injected into the cracks (Fig. 2(b)). The next step of the repairing procedure is the application of repair mortar in the place of spalled concrete (Fig. 2(c)). A primer epoxy coat at around 0.1 mm was used to provide an efficient adhesion between the concrete and epoxy based repair-anchorage mortar (Fig. 2(d)). The last step before the wrapping CFRP is both to cover the repairing parts with an epoxy based repair-anchorage mortar for repairing the wide cracks and to provide certain level of bonding between the repaired specimen and the CFRP sheets (Fig. 2(e)). Finally, the beam and columns were wrapped by CFRP sheets with an epoxy resin before hardening the repair-anchorage mortar. The resin is used for establishing connection between the CFRP sheets and the repair-anchorage mortar. It also increases the flexural and shear capacity of the repaired members (Fig. 2(f)). The number of layers in application of the CFRP sheets were shown in Fig. 3.

(a) Rounding corners (b) Injecting chemical anchorage (c) Applying repair mortar

(d) Applying primer epoxy coater (e) Application of epoxy based repair and anchorage mortar (f) Wrapping CFRP sheets with saturant material

Fig. 2 - Repairing procedure for the damaged captive columns

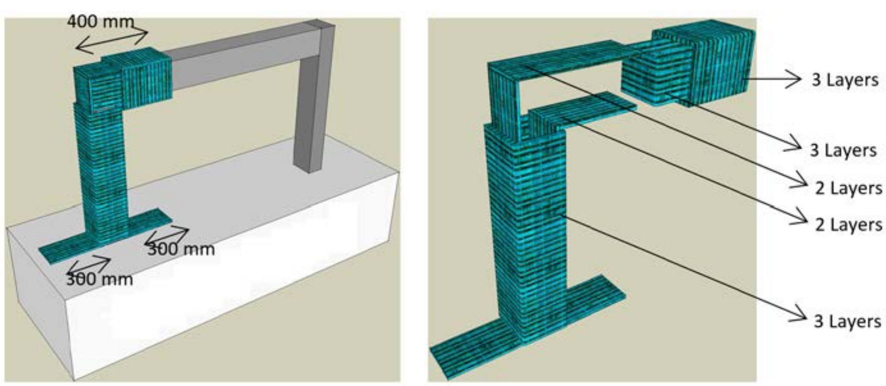


Fig. 3 - Schematic representation of CFRP sheets

3. Test specimen details and material properties

Fig. 4 and Fig. 5 illustrates the reinforcement and geometric details of the tested specimens. All the specimens have the same column and beam dimensions of 100 mm x 150 mm and, 150 mm x 150 mm respectively. The foundations of the test specimens were designed much stiffer than the columns to provide a fully restraint support condition for the column bottom ends. The foundation was connected to the strong floor with six M64 bolts with the purpose of restricting any displacement.

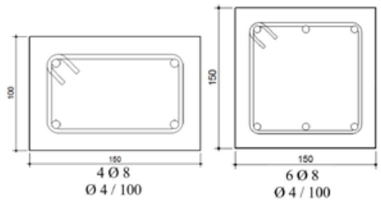


Fig. 4 - Reinforcement details of the column and beam cross sections

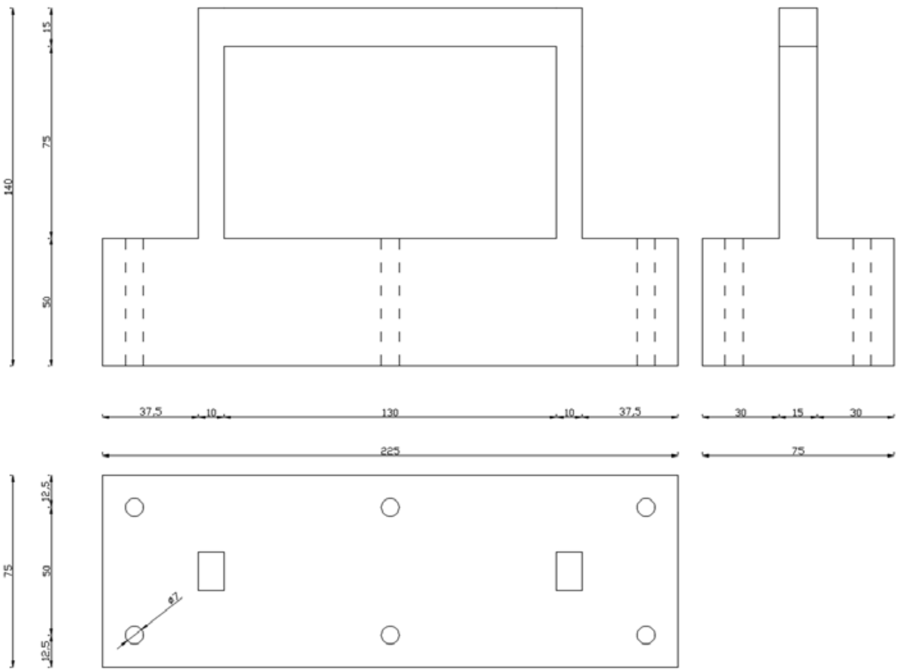


Fig. 5 – Geometric properties of the beam, columns and foundation of the test specimens

In order to represent the substandard RC beam and columns, Ø8 and Ø4 plain bars were used for longitudinal and transverse reinforcement respectively. Several material tests were conducted on the reinforcement samples to obtain the material properties of the reinforcement steel. The average values of yield strength, tensile strength and modulus of elasticity were determined as 311 MPa, 417 MPa and 200 GPa respectively. The target compressive strength of concrete was selected to be 10 MPa in order to represent the low concrete strength. The average concrete compressive strength of the specimens is 10.9 MPa and 9.4 MPa for IE01 and IE02 on the date of experiment. The unidirectional CFRP sheets were used in the repair of IE03. Some of the geometric and material properties of the CFRP sheets were provided by the manufacturer. The thickness is 0.111 mm, the modulus of elasticity is 230 GPa, the ultimate tensile strength is 4900 MPa and the ultimate strain is 2.10%. Before wrapping the columns with CFRP sheets, the shear cracks at the top of the columns were filled by injecting the chemical anchorage based on an epoxy acrylate resin. After injecting the chemical anchorage, a repair mortar with a 28-day compressive strength of 40 MPa was used. A primer epoxy coater, which has a 20 MPa flexural bending capacity, was applied before the application of CFRP sheets. Another epoxy based repair and anchorage mortar which has 75 MPa compressive strength was applied to obtain a smooth surface. Finally, a saturant material with a 60 MPa compressive strength was used which was recommended by the manufacturer.

4. Test results

Quasi-static cyclic tests were conducted to obtain the several response quantities such as strength, ductility, dissipated energy and stiffness degradation. The first frame (IE01) was the reference frame without infill wall. The maximum lateral load was measured as 11.1 kN and the test was ended at the top displacement of 50 mm (6% drift ratio). The first crack was observed at the joint at 0.1% drift ratio. At the same cycle, a crack occurred between the column and the foundation connection. During the test, flexural behavior dominated the overall response of IE01 with the concentration of the flexural cracks mostly in the columns. Under constant axial load and reversed cyclic lateral displacement, a ductile behavior was observed with limited lateral load capacity.

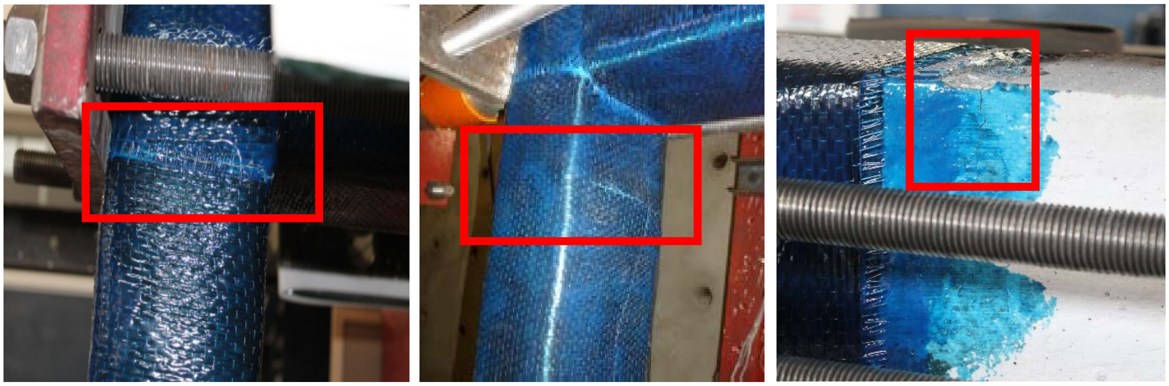
The second frame (IE02) was the test specimen with captive column defects. Due to the increased strength and stiffness with the presence of the partial infill wall, the maximum lateral load was obtained as 28.2 kN, which is 2.5 times more than the lateral load capacity of the reference frame. Flexural cracks started to occur from the beginning of the experiment at both columns and joints. The separation of the infill walls from the columns was observed at 0.1% drift ratio. When the drift ratio reached to 0.5%, shear cracks occurred in the captive columns. A sudden drop in the lateral capacity of the frame was observed at 1.0% drift ratio. Because of the severe damage in the captive columns in the succeeding drift ratios, the test for IE02 was ended at the displacement of 16 mm (2% drift ratio) due to safety concerns. After the expected captive column damage occurred and the excessive shear cracks were monitored, the test was terminated. Although the infill wall with opening in the frame caused a considerable amount of increase in the lateral load capacity of the frame in the initial cycles, partial infill wall constrained the lateral displacement of the captive column and caused a brittle type of column shear failure in the succeeding cycles.

After the damage occurred in IE02, it was repaired with CFRPs and retested as IE03, which was the repaired frame. The maximum lateral load of IE03 was recorded as 21.8 kN, which is almost twice the lateral load capacity of the reference specimen. The test was ended at the top displacement of 50 mm (6% drift ratio). Since the columns, beam and joints of the frame were wrapped with CFRPs, crack formation in the RC members as well as the rupture of the CFRP sheets could not be monitored apparently. After 4% drift ratio, some of the inner CFRP layers were ruptured, which caused strength deterioration specifically in the second cycle for the 4.4% drift ratio in the positive direction and in the first cycle of the 4.3% drift ratio in the negative direction (Fig. 8(c)). Although a ductile behavior was observed up to 4% drift ratio, a sharp decrement in the strength occurred as a result of rupture in the inner layer of CFRP sheets. The damage pattern of all the test specimens at 2% drift ratio are shown in Fig. 6. Nevertheless, only IE01 and IE03 specimens can displace up to 6% drift ratio.

(a) IE01 (b) IE02 (c) IE03

Fig. 6 - Damage pattern of the test specimens at 2% drift ratio

Since the section cracks have remained under the CFRP layers, they could not be apparently seen in the IE03 specimen. The first fracture at the interface of the column and joint CFRP sheets in the column initiated at 1.5% drift ratio as seen in Fig. 7(a). In Fig. 7(b) the swelling of CFRP sheets was observed as the drift ratio increased. After the 5% drift ratio, flexural cracks were observed at the middle part of the beam, which was not wrapped with CFRP (Fig. 7(c)). In Fig. 8(c) two sudden decrement of load values were monitored when the target drift ratio was 4% in the negative direction. At that point a sound similar to the fracture sound was heard. However, the fracture of the CFRP sheets could not be apparently observed from the outer CFRP layers of the repaired specimen.

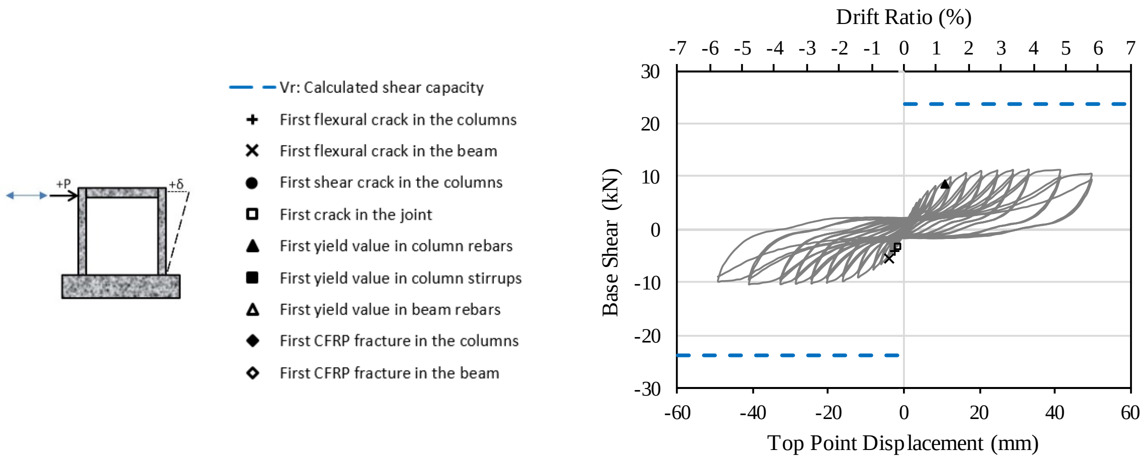


(a) CFRP fracture of column (b) CFRP swelling (c) Flexural crack in the beam

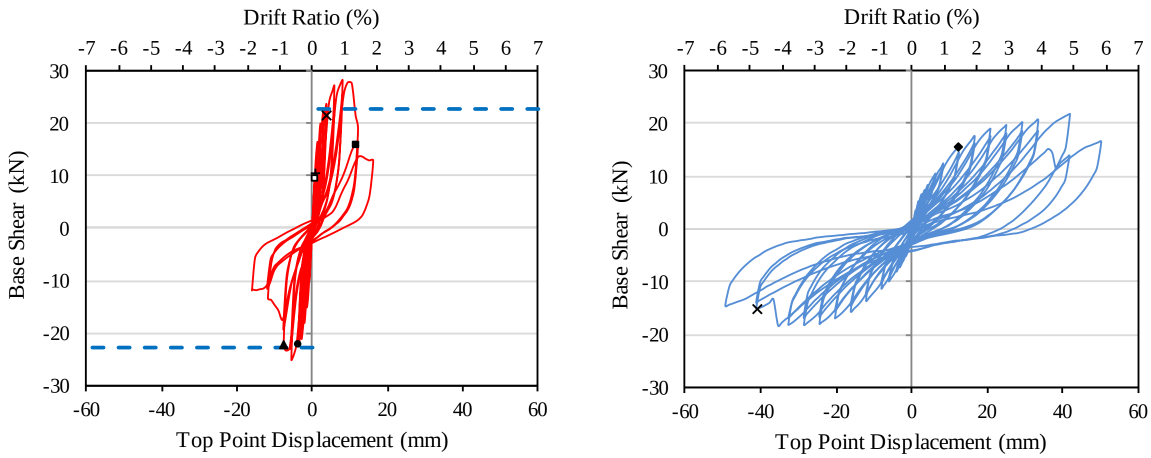
Fig. 7 - Failure pattern of CFRP

In order to determine the lateral load as well as the displacement capacity of the tested specimens, base shear force vs. top displacement graphs were plotted as given in Fig. 8. As shown in Fig. 8(a) columns did not reach their shear capacity and shear crack did not occur. The first flexural crack in the joints, columns and beam was observed at the drift ratio of 0.2%, 0.3% and 0.5% respectively. Only longitudinal rebars of the columns yielded and the first yield point was observed at 1.3% drift ratio through the strain gauge measurements.

According to Fig. 8(b), a sudden drop in the lateral load was observed after attaining the peak lateral load values due to the shear damage at the captive columns. Due to the reduction in the effective length of the captive columns by the partial infill wall, the captive columns attract a shear demand exceeding their shear capacity. This phenomenon caused the shear failure of the columns, which violates the capacity design principles defined in TEC (2007). In the test specimen IE02, the first flexural cracks were observed in the columns and joints at 0.1% drift ratio, whereas it was at the 0.5% drift ratio in the beam. At 0.5% drift ratio first shear crack in the column of IE02 was observed due to the captive column defect. Strain gauge measurements imply that first yield occurred at 0.9% drift ratio for the longitudinal reinforcement bars and 1.4% for the stirrups in the columns.



(a) IE01



(b) IE02 (c) IE03

Fig. 8 - Hysteretic loops of the specimens

Lateral load capacity is one of the important parameters to discuss the effectiveness of the applied repairing technique. Response envelope curves were used to comment on the strength characteristics of the test specimens. Envelope curves were constructed by combining the ultimate lateral load values at the target displacement points at the first peak of each cycle, as seen in Fig. 9. It can be inferred that the strength of the repaired specimen (IE03) is greater than the one for reference specimen (IE01). However, if the results were compared with the second specimen (IE02), the repaired specimen had a 23% lower strength value because of the contribution of partial infill wall in IE02.

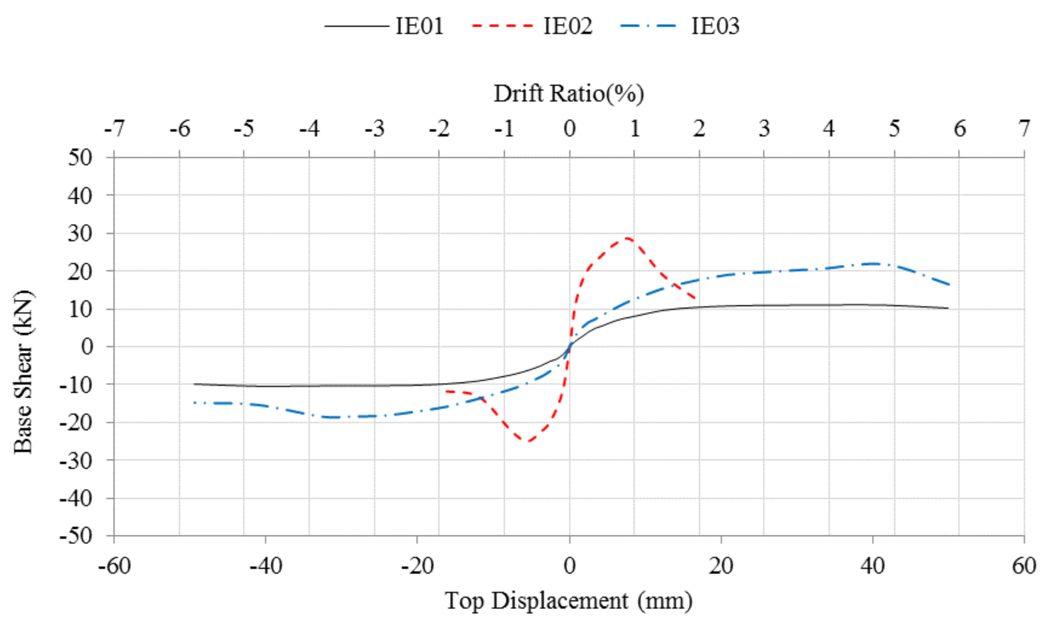


Fig. 9 - Envelope curves for hysteretic loops

As the lateral displacement increased, lateral stiffness of the test specimens decreased very rapidly due to the formation of cracks and imposed plastic deformations (Fig. 10). The first step of specifying stiffness degradation was to determine the peak-to-peak stiffness for each displacement cycle. Peak-to-peak stiffness was defined as the slope of the line between peak lateral load values corresponding to the displacement value for positive and negative cycles. Then the peak-to-peak stiffness for each cycle was normalized with respect to the one for reference specimen for the first cycle. The normalized peak-to-peak stiffness versus drift ratio curves are shown in Fig. 10. As it can be observed from Fig. 10, IE02 specimen has the highest initial stiffness value due to the lateral rigidity of the partial infill wall for the first cycle, and also a rapid reduction in stiffness was observed in this specimen. IE03 specimen has greater stiffness value compared to the reference specimen. However, after 2% drift ratio both have very close stiffness values. Although the initial stiffness of the repaired specimen without infill wall is lower than the frame with partial infill wall, its stiffness is almost twice the initial stiffness of the reference specimen. This implies that CFRP wrapping can improve the stiffness of the damaged RC frames even more than its original stiffness values for the bare frame.

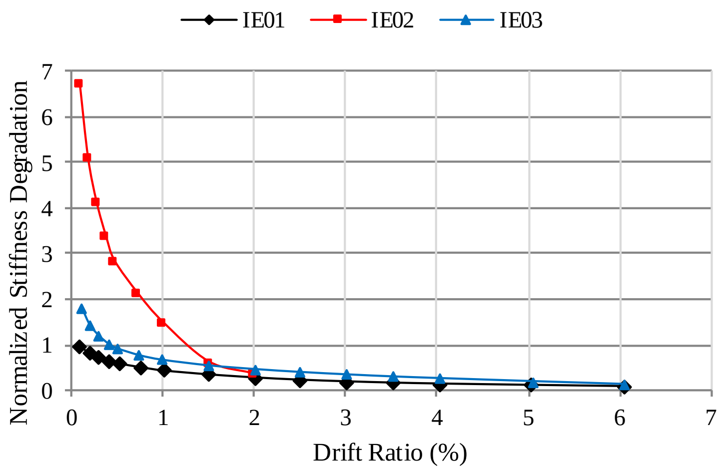


Fig. 10 - Stiffness degradation

It was calculated that the yield displacement and the ductility of a frame with an idealized elasto-plastic load-displacement relation [13]. Fig. 11 represents the idealized bi-linear form of envelope curves for all test specimens. The inclined part of the graphs represents the equivalent initial stiffness of the RC frames. Equivalent initial stiffness is defined as the slope of a line connecting the origin and the point that corresponds to the 60% of the ultimate load on the increasing part of the envelope curve [14]. The post-yield region of the graph was extended up to the displacement value at which the lateral load reduces to 80% of the ultimate load on the descending part of the envelope curve. In order to specify the yield point an iterative procedure was employed such that the areas under the idealized bi-linear and envelope curve are equal to each other.

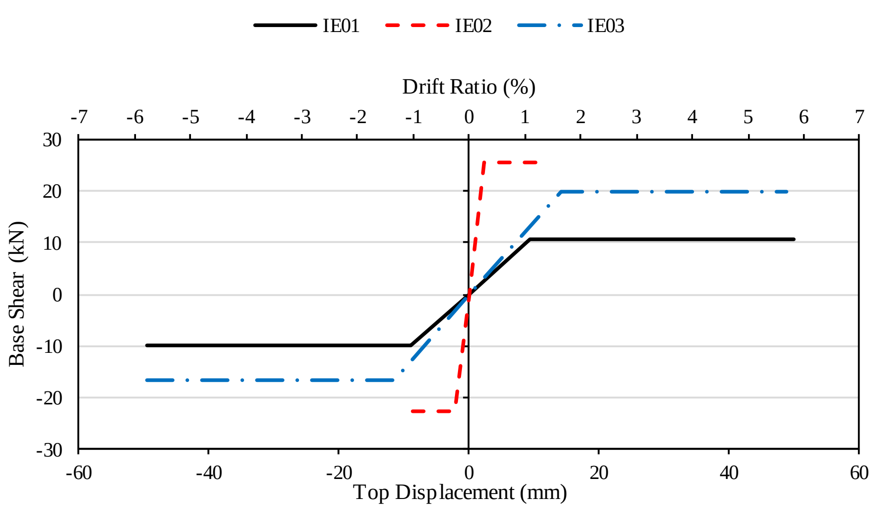


Fig. 11 - Idealized bi-linear representation of the envelope curves

IE02 specimen reached its ultimate lateral load capacity at 0.29% drift ratio, and ultimate drift ratio was observed at 1.30%, which corresponds to the 20% decrement in the ultimate lateral load of the specimens. The ductility value for IE02 was calculated to be higher than the ductility of the repaired specimen (IE03). These ductility values do not represent the actual behavior of the tested specimens. Due to the excessive shear damage on the captive columns of IE02, test had to be terminated at 2% drift ratio. Yield displacement plays a very important role in the calculation of ductility parameter. Due to the high rigidity of the partial infill wall in IE02, the yield displacement for this specimen was calculated to be much smaller compared to the other specimens. Although the repaired specimen has the lowest ductility, its ultimate displacement value is almost five times greater than the one for IE02 specimen. For this reason, both ductility and displacement capacities of the specimens should be taken into account in comparing the seismic response of the tested specimens. If the ultimate displacement capacities of IE02 and IE03 are compared, it can be inferred that the repairing technique applied for the damaged captive columns seems to be quite effective for improving displacement capacity of the damaged frames.

The energy dissipation capacities of the tested specimens in each cycle were calculated as the area enclosed by the first cycle of the corresponding hysteretic loops. The cumulative energy dissipation was calculated by the summation of the areas of the hysteretic loops up to the corresponding drift ratios. Fig. 12 presents the relation between cumulative energy dissipation vs. drift ratio of the specimens. At 2% drift ratio IE02 specimen has the greatest dissipated energy capacity and the remaining specimens have almost similar capacities. When Fig. 12 is examined for 6% drift ratio, it can be concluded that the repaired specimen has more energy dissipation capacity than the reference specimen.

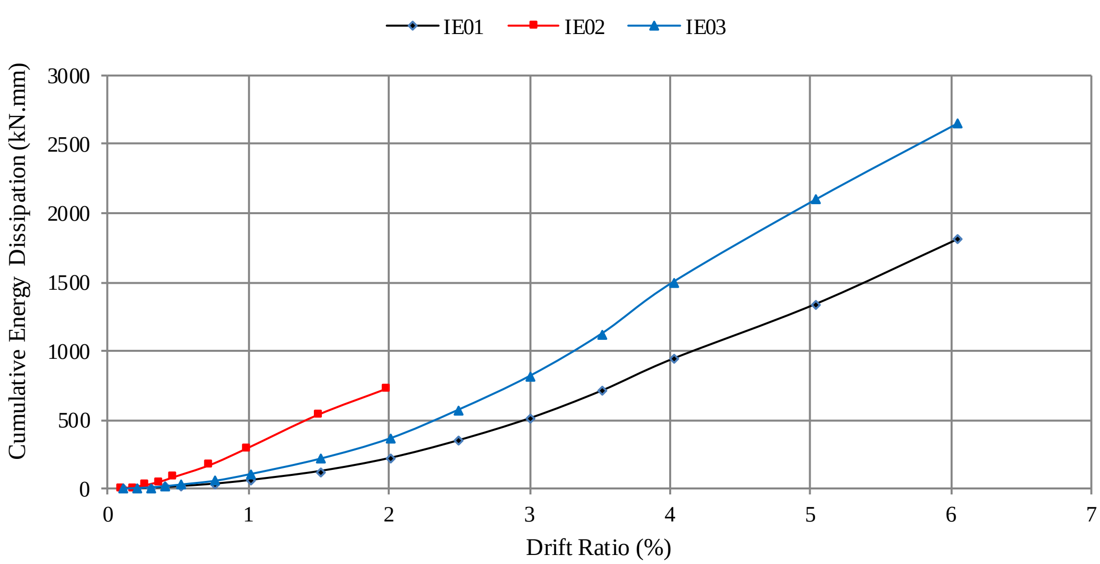


Fig. 12 - Cumulative energy dissipation capacity of the tested specimens

5. Conclusion

The results of the tests performed on RC frame repaired with CFRP have been presented in this study. The interpretation of the results indicates that CFRP wrapping on the damaged captive columns found to be effective in terms of several response parameters such as strength, displacement capacity, dissipated energy and stiffness degradation. The structural system can be further in service if an effective repairing was applied for the damaged captive columns.

The experimental outcomes point out that the repaired frame without infill wall has a lower lateral load capacity than the second specimen with the partial infill wall. On the other hand, the lateral load capacity of the repaired specimen is almost twice the one for reference frame. This result revealed that, repairing the damaged captive columns with CFRPs can enhance the performance of its original undamaged bare frame considerably in terms of lateral load capacity.

The findings show that wrapping the severely damaged columns by CFRP sheets improved their ultimate displacement capacity considerably. The repaired specimen can be able to reach the 6% drift ratio without any excessive strength deterioration however the test of IE02 is terminated at 2% drift ratio due to the severe damage in the captive columns.

Based on the observations and the measured data from the tests IE02 specimen has higher initial stiffness than the reference and repaired specimen. The repaired specimen has higher initial stiffness than the reference specimen. A severe stiffness degradation occurs at the IE02 specimen after the formation of shear cracks in the captive columns. It was also observed that after the 2% drift ratio, the stiffness of the specimens becomes closer to each other.

Energy dissipation capacity for the IE02 specimen is the largest among the three tested specimen up to 2% drift ratio, but after that point capacity is almost lost because of the severe damage in the columns. The cumulative energy dissipation capacity in the last step of tests show that the repaired specimen with CFRPs has more energy dissipation capacity than the one for the reference specimen. Therefore, it can be concluded that repairing technique is effective in terms of energy dissipation capacity.

The results show that the repairing procedure against the shear failure for columns caused by captive column defect is an applicable method because the investigated response quantities of the repaired specimen improved considerably compared to the reference specimen. On the other hand, it would be much more practical and cost-effective to prevent the captive column defects by simple structural and architectural precautions before the occurrence of captive column damage.

6. Acknowledgements

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7. References

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