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Structural behavior of beam column joint retrofitted using carbon fiber reinforced polymer

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ABSTRACT

Beam column joints are one of the most critical components of reinforced concrete (RC) structures since it is subjected to large forces during severe ground shaking. The present study comprises four exterior beam-column joint specimens having different reinforcement arrangements detailed as per IS 13920: 1993, tested under reversed cyclic loading up to failure. The test was force-controlled and the specimen was loaded by increasing the load level during each cycle. The load was applied forward cyclic and reverse cyclic and deflection, were measured from every 5kN by using a linear variable digital transducer (LVDT) with the digital arrangement. The deflection was measured at the loading point and at the centre of the beam. Damaged specimens were repaired and retrofitted with carbon fibre reinforced polymer (CFRP) to prevent shear damage and strength deterioration and to achieve a more ductile response. Retrofitted specimens were subjected to similar cyclic loading. Results for displacement were obtained. Hysteresis behaviour of non-retrofitted and retrofitted specimens were studied with respect to ultimate load, maximum displacement, energy dissipation capacity, stiffness degradation and general failure pattern. The comparisons showed that CFRP sheets improved the shear resistance of the joint and increased its energy dissipation capacity. Retrofitting makes the joint so strong that failure is directed towards the beams as it helps the structure in energy dissipation through plastic hinge formation in the beam.

1 Introduction

Beam column joints are one of the most critical components of reinforced concrete moment resisting frames, since it is subjected to large forces during severe ground shaking. In the last few decades moderate and severe earthquake have struck different places in the world, causing severe damage to RC structure. Retrofitting of existing structures is one of the major challenge that modern civil engineering structures has demonstrated that most of them will need major repairs in the near

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future. Shear failure and bonding deterioration are considered as primary cause of failure of beam column joints in moment resisting framed structures. Evidences from past earthquakes have shown that failure of beam column joints often leads to partial or total collapse of structures. Hence the work of retrofitting of beam column joint using CFRP will be useful to maintain structural safety and reliability.

A new technique has emerged recently which uses fibre reinforced polymer (FRP) sheet to enhance the strength of beam column joint. Strengthening refers to the reconstruction or renewal of any part of an existing building to provide better structural capacity like higher strength and ductility than the original building [1]. The various strengthening techniques involves steel plate bonding, polymer injection, concrete jacketing, use of advanced composite materials like glass fibre reinforced polymers (GFRP), CFRP and ferrocement etc. To enhance the safety of existing RC frame buildings, it is essential to increase the strength of columns and beam-column joints so that strong column-weak beam behavior can be achieved under cyclic loading. Increased strength of columns and joint regions will allow for weaker beams to experience ductile damage or failure under extreme loading conditions. Use of FRP composites, as a lightweight material, is one of the easiest and most economical strengthening methods.

In the past, numerous research works were reported on the applications of advanced concrete materials for enhancing the strength of RC structural elements. Prota et al. [2], have studied seismic upgrade of beam column joint using FRP reinforcement. They proposed a new technique for seismic upgrading of RC beam column connections. This technique is based on the combined use of FRP laminates and near surface mounted bars. The FRP rods provide flexural strengthening, whereas the lay-up laminates provide confinement and shear strengthening. Mukherjee and Joshi [3] have carried out experimental study to find the effect of FRP in improving shear strength and ductility of damaged RC beam column joints in Gujarat earthquake. Antonopoulos and Triantafillou [4] have presented analytical model to study analysis of RC joints strengthened with externally bonded reinforcement comprising unidirectional strips or flexible fabrics. A number of case studies are examined in this article, indicating that even low quantities of FRP materials may provide significant enhancement of the shear capacity. Antonopoulos and Triantafillou [5] have conducted a comprehensive experimental program through 2/3-scale testing of 18 exterior joints. They evaluated the role of various parameters includes area of fraction of FRP, distribution of FRP and column axial load on the shear strength of exterior joints. Also, the importance of mechanical anchorages in limiting the premature debonding has been reported by authors.

Tsonos [6] has studied experimentally and analytically the effectiveness of RC jacket and a high strength fiber jacket for cases of the post-earthquake and pre-earthquake retrofitting of beam-column joints. He also compared the effectiveness of the two jackets. Ghobarah and El-Amoury [7] developed effective rehabilitation systems using composite materials and steel elements to upgrade the resistance to bond-slip of bottom steel bars anchored in the joint zone and to upgrade the shear resistance of joints. Tsonos [8] has provided the structural engineers with useful information about the safety of new RC frame structures that incorporate seismic details from current building codes. Almusallam and Al-Salloum [9, 10] have presented analytical models for the prediction of the shear capacity of the FRP strengthened beam-column joints and also showed the efficiency and effectiveness of CFRP sheets for upgrading the shear strength and ductility of a seismically deficient exterior beam column joint through experimental study.

In the present work, CFRP sheets were used to retrofit the beam column joint specimens to increase strength and stiffness of joint. The specimens were designed having different reinforcement arrangement. All the specimens were subjected to axial loading and reverse cyclic loading upto failure. Damaged specimens were repaired by filling its cracks with the application of epoxy and putty and then retrofitted using externally bonding the CFRP sheets to beam, column and beam-column joint region. The retrofitted specimens were also subjected to similar type of loading as that of non-retrofitted specimens.

2 Experimental program

2.1 Details of beam column joint specimen

To study the behaviour of retrofitted CFRP wrapped exterior beam column joint, four specimens were cast using M30 grade of concrete with water to cement ratio (W/C) of 0.48. The cross section of beam was 200×165 mm without slab thickness and beam length of 400 mm and column cross section was 220×165 mm. Height of the column was 800 mm. The reinforcement detail is designed according to IS 13920: 1993 and IS 1893(Part 1): 2002 [11, 12].

Specimen 1 (S1): The reinforcement details of specimen 1 are shown in Fig. 1(a). Main reinforcement provided in the beam was 12 mm diameter bars, 2 numbers at top and 2 numbers at bottom. Also, 12 mm diameter bars, 1 at center of top beam face and 1 at center of bottom beam face. The stirrups of 6 mm diameter bars at 75 mm c/c in beam portion, from face of column were provided. The longitudinal reinforcement provided in the column was 4 No. of 12 mm diameter bars, each at corner and 4 No. of 10 mm diameter bars, at the center of each column face. The column confinement was 6mm diameter bars at 75 mm c/c.

Specimen 2 (S2): The reinforcement details of beam column joint for this case is same as per the specimen 1, only the centre face bars from the beam have been provided diagonally. All four corner bars of the beam are extended in the column for distance Development length + (10xDiameter) from the inner face of column. For 12 mm diameter this length is 660 mm.

Specimen 3 (S3): The reinforcement details of beam column joint for this case is same as per the specimen 1, only all the bars from the beam are Headed bars up to outer face of the column. All the beam bars were provided with heads of diameter 50.4 mm and thickness 12 mm.

Specimen 4 (S4): The reinforcement details for this case is same as per the specimen 3, only the top and bottom centre Headed bars are bend in column up to 100 mm. The detailing of reinforcement of beam column joint for specimen 1 to 4 is as shown in Fig. 1(a)-(d).

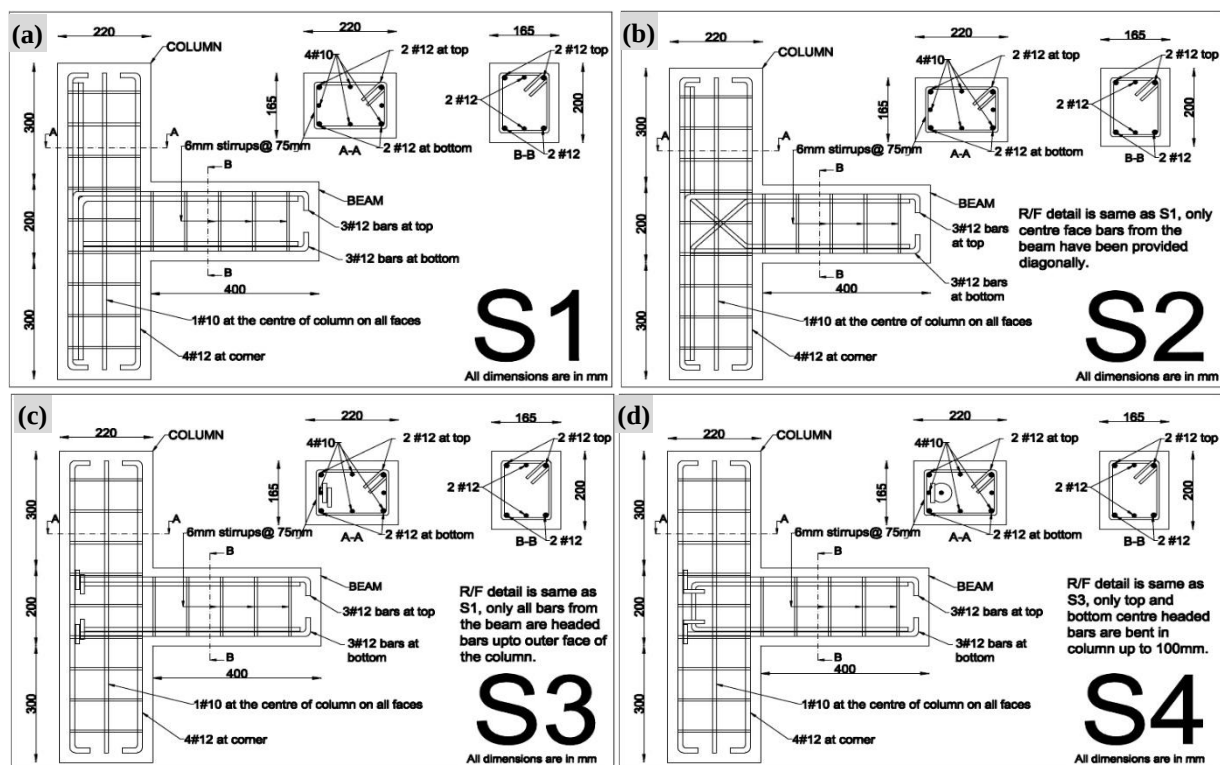


Fig. 1 – (a) Specimen1 (S1), (b) Specimen2 (S2), (c) Specimen3 (S3), (d) Specimen4 (S4),

2.2 Casting and Curing

For casting the specified specimens, the reinforcement cages were placed in the mould and cover between cage and form was provided by 25 mm cement mortar blocks. Then concreting is done and all the specimens are allowed for water curing for 28 days. After 28 days of curing, the specimens were dried in air and whitewashed.

2.3 Testing Arrangement

After curing all the test specimens were subjected to axial load and reverse cyclic loading. A constant axial load of 100 kN was applied with a hydraulic jack to the column, which is about 20% of the axial capacity of the column for holding the specimen in position and to simulate column axial load. Two ends of the column were given external axial hinge support.



Fig. 3 – Testing Arrangement

Another two hydraulic push and pull jack was used to apply reverse cyclic load to the beam portion. The point of application of the cyclic load was 50 mm from the free end of the beam. The test was force-controlled and the specimen was loaded by increasing the load level during each cycle. The load is applied forward cyclic and reverse cyclic and deflection measured from every 5 kN by using LVDT with the digital arrangement. The deflection was measured at the loading point and at the centre of the beam. Fig. 3 shows the test set up.

2.4 Preparation of Retrofitted Specimens

In the case of any damaged members, the first step is to rebuild the damaged member. Damaged specimens S1, S2, S3, and S4 as shown in Fig. 4 were first cleaned and made ready for retrofit. All cracks, voids, and irregular surfaces are finished with the help of putty. Putty is applied to smooth out the discontinuity, contaminants. When the putty was cured, a low viscosity primer coat of JSRprime is applied with the help of roller and brush, within the pot life period of 45 minutes of the mixture. JSRprime is a two-component adhesive system comprises a colorless base epoxy resin and amber-colored hardener as shown in Fig. 5(a), both are mixed in a ratio of 100(Base):50(Hardener) by weight as per the manufactures instruction manual. Primer is applied to promote adhesive bonds for saturating resin. The inappropriate ratio would result in inferior end properties. The mixture is stirred to ensure full homogeneity. It is then allowed to cure as shown in Fig. 6(a).

When primer was fully cured, unidirectional JSRwrap (carbon fibre reinforced composite) was applied with JSRepoxyS Resin system. In a clean area away from resin carbon fibre reinforced composite is measured and cut as shown in Fig. 6(b). JSRepoxyS Resin system is a moderate viscosity liquid Epoxy Resin. It is comprised of base resin (saturant) and hardener system mixed in ratio of 100(Base):2(Hardener) by weight as shown in Fig. 5(b). The resin provides good pigment wetting and fibre cloth wetting with high level of mechanical and chemical resistance properties in the cured state.



Fig. 4 – Damaged Specimens after testing (S1, S2, S3, S4)

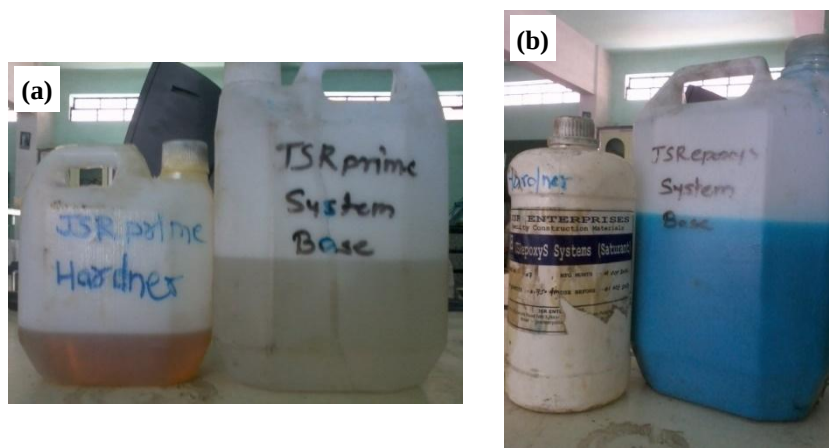


Fig. 5 – (a) JSRprime, (b) JSReposyS resin system

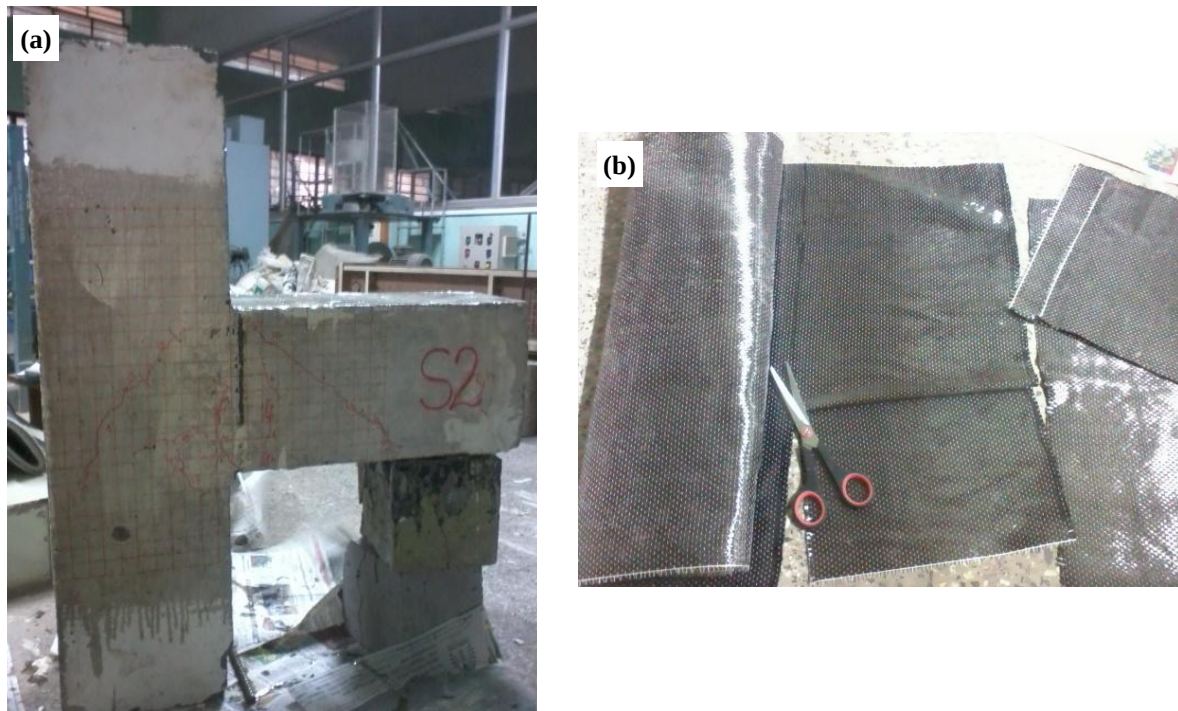


Fig. 6 – (a) Primer application, (b) Fibre cutting

For wrapping, a wet layup process was adopted to ensure better wetting of fibres in the resin. These pre-wetted sheets were then carefully placed on the primed surface of beam-column joint specimen by giving a little hand pressure. Care must be taken to remove all air pockets therefore rolling is done in the direction of fibre. Wrapping must be completed within the pot life period of the resin that is 15 minutes. Therefore, a small quantity of resin was mixed at a time. Retrofitted specimens as shown in Fig. 7, were cured in the air for a period of 7 days and then tested as per the same procedure adopted before. Damaged specimens S1, S2, S3 and S4 were retrofitted and named as RS1, RS2, RS3 and RS4 respectively.



Fig. 7 – Retrofitted beam column joint specimen

3 Results and Discussions

3.1 General Behavior

Non-retrofitted specimen: The general cracking pattern for the non-retrofitted specimens S1, S2, S3 and S4 under reverse cyclic loading is as shown in Fig. 4, from which it is observed that, flexural cracks appeared at the beam bottom face close to the beam column joint interface during the positive loading cycles. The same happened at the top surface of beam

during the negative loading cycle. The cracks were almost symmetrical on both the faces of the beam column joint. At the lower stages of loading for the entire four specimens, minor shear cracks initiated diagonally at the joint interface and propagated in the joint region. Also, some inclined shear cracks were observed in the beam portion. However, cracks in the column were much less than those in the beam. This may be due to the column is subjected to a high axial compressive load; the net tensile stresses which may lead cracking in the column are quite small. Further, some vertical cracks were also observed in the beam, which may be occurred due to high flexural moments in the beam at higher stages of loading. All specimens behaved in the same manner with widening of diagonal shear cracks at the joint interface and cracks in the beam portion, ended at joint interface. In case of S1 and S4 specimen, more number of finer cracks was developed in joint region, at the joint interface and in the beam portion. The joints were finally failed due to expansion of these cracks. However, in S1 specimen, crushing of concrete at the top of column was observed where the column was hinged. Now, in S2 specimen major crack widening was observed at the loading point which resulted in the failure of the S2 specimen. Similarly, S3 specimen also failed due to crack occurred at the loading point. This type of failure was different than failure of S1 and S4 specimen.

Retrofitted specimen: Above mentioned damaged specimens were retrofitted by full external wrapping CFRP sheets to beam column joints. As the specimens were failed in shear, the beam and the column faces were provided U-wrap. U-wrap delays the failure of concrete; an increase in shear capacity and also provides better confinement to the concrete [4]. It was observed in the retrofitted specimen using CFRP sheets, the shear failure of the joint was delayed significantly and the failure was mainly due to the crushing of concrete inside the sheets. This failure was not visible because of external wrapping of sheets. In case of RS1 and RS4 specimen, concrete crushing was followed by CFRP fracture in the beam portion which was shown in Fig. 8 for RS1 specimen.

However, RS3 specimen failed very poorly due to CFRP debonding at the point of loading in initial period of loading as shown in Fig. 9. This may be attributed to poor bonding of CFRP in that region. But it is not the same case in RS2 specimen. Though the S2 specimen was failed at the loading point but after retrofitting as it was perfectly bonded to concrete, it has taken the load similar to that of parent specimen. Therefore, it may be concluded that proper bonding should be achieved for better confinement to the concrete and to enhance the shear strength of joint. No damage was observed in the column and joint region. Thus it proves that strong column weak beam philosophy. Thus, CFRP wrapping to the beam column joint shifted the joint interface failure in the beam portion.

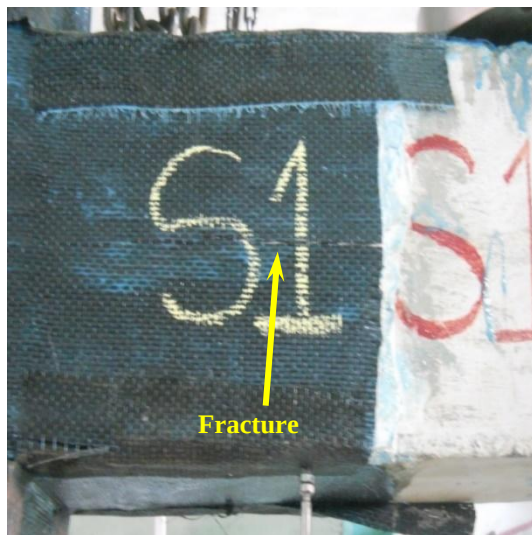


Fig. 8 – Failure of RS1 Specimen



Fig. 9 – Debonding of CFRP in RS3 Specimen

3.2 Hysteresis Behavior

The hysteresis behaviour was examined in terms of shear strength i.e. measured ultimate load, maximum displacement, energy absorption and stiffness degradation. Fig. 10, 12 and 13 shows that the ultimate load for specimen RS1, RS3 and RS4 were little less than its corresponding original specimens S1, S3 and S4 respectively. It does not mean that CFRP has not

increased the shear strength of the joint; in fact the repairing was done when original specimens lost its strength to take any further significant load. This increase in strength was up to the level which is comparable to its original specimen. However, for RS2 specimen as shown in Fig. 11, the ultimate load was similar to the ultimate load of S2 specimen.

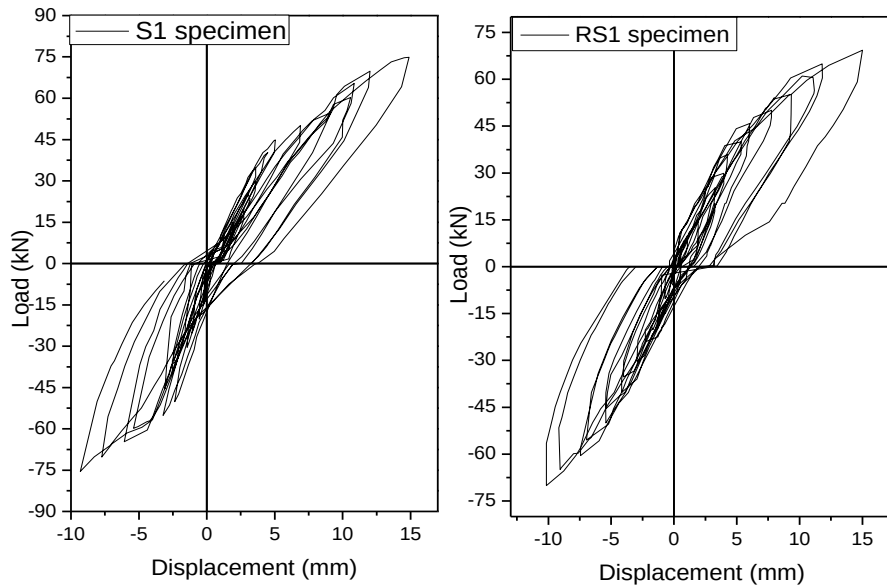


Fig. 10– Hysteresis behavior of S1 and RS1 specimen

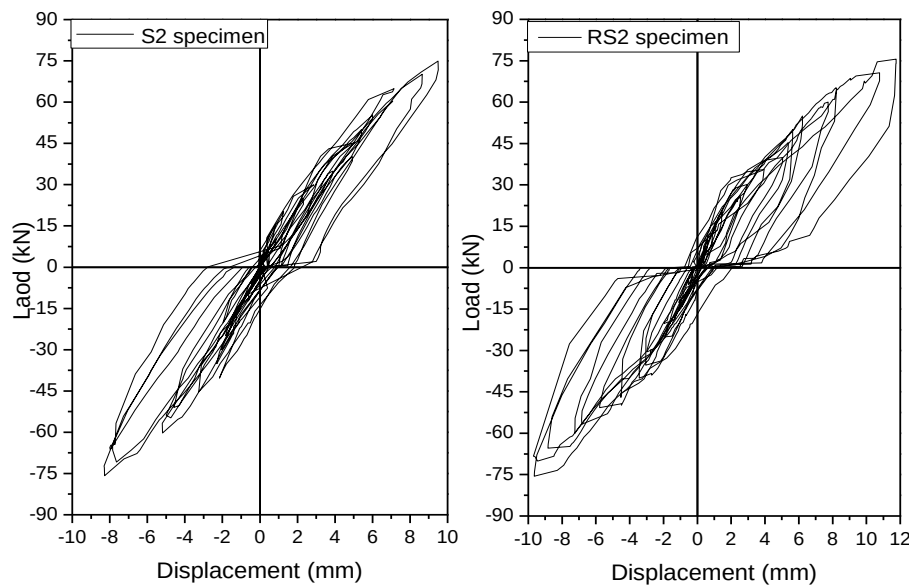


Fig. 11– Hysteresis behavior of S2 and RS2 specimen

Further, the comparison of measured maximum displacement for all the specimens from the hysteresis behaviour illustrates that, maximum displacement of retrofitted specimens were much greater than non-retrofitted specimens. RS1 specimen was able to reach maximum displacement of 15.01 mm, which was little larger than the maximum displacement of 14.89 mm of S1 specimen, i.e. it showed 0.8% increase in maximum displacement. For RS2 specimen maximum displacement was 11.76 mm, which is 24% larger than the maximum displacement of 9.49 mm of S2 specimen. Now, as RS3 specimen was poorly confined by CFRP at its bottom face, it has taken the ultimate load of 40 kN with maximum displacement of 4.97 mm; whereas the ultimate load of S3 specimen was measured as 65 kN having maximum displacement of 7.41 mm. RS4 specimen showed large increase in maximum displacement of 75%, with 15.12 mm maximum displacement of RS4 specimen

and 8.64mm for S4 specimen. Hysteresis behaviour of retrofitted specimen RS1, RS2, RS3 and RS4 shows better response over their non-retrofitted specimen S1, S2, S3 and S4.

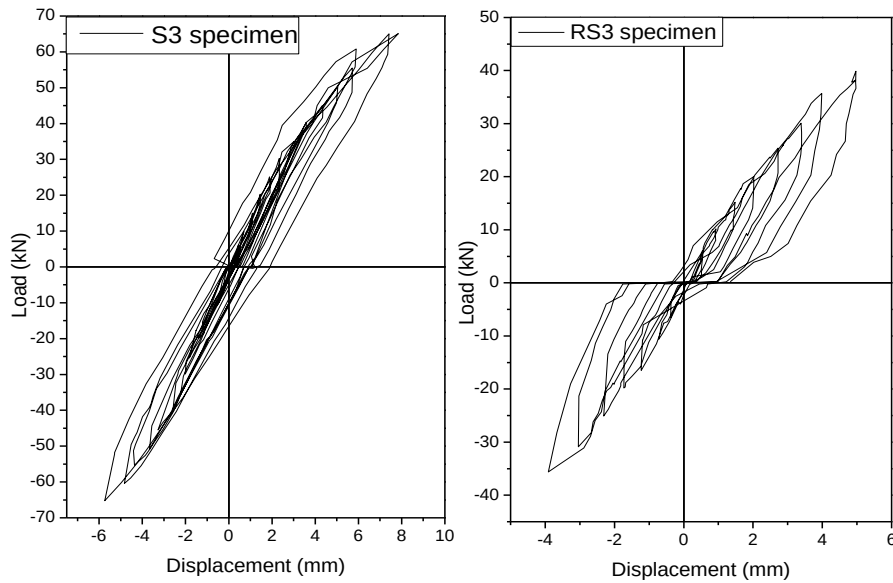


Fig. 12– Hysteresis behavior of S3 and RS3 specimen

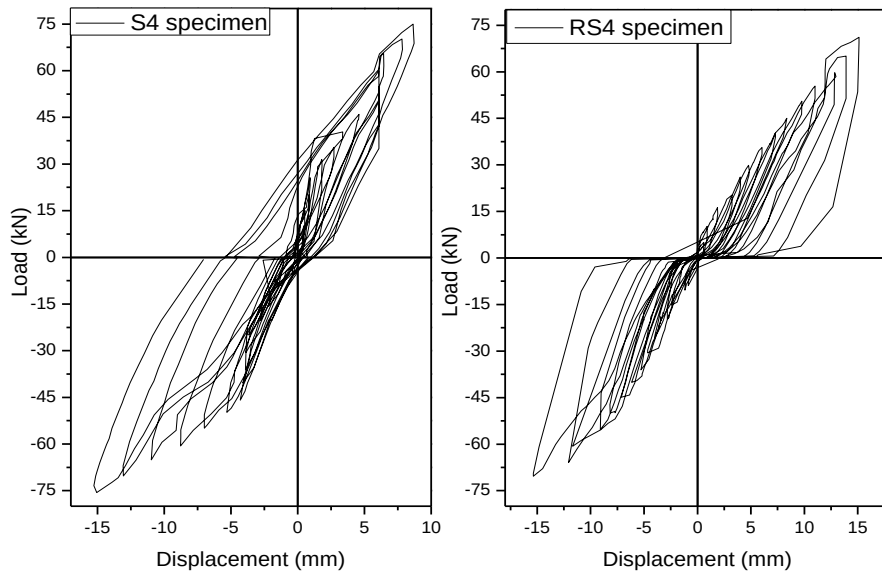


Fig. 13– Hysteresis behavior of S4 and RS4 specimen

3.3 Energy Dissipation Capacity

The energy dissipation capacity could be obtained from the area enclosed by the load-displacement plot at a given cycle. The capability of a structure to dissipate energy has a strong influence on its response to earthquake loading. The total energy dissipated by a structure consists of (1) energy dissipated by the steel reinforcement (2) energy dissipated by friction along existing cracks in concrete and (3) energy dissipated during the formation of new cracks [13]. The values of energy dissipation capacity of each specimen under cyclic loading were calculated and plotted as energy dissipation Vs number of cycles which is shown in Figs. 14-17. It is observed that energy dissipation capacity of all the retrofitted specimens were greater than that of non-retrofitted specimens.

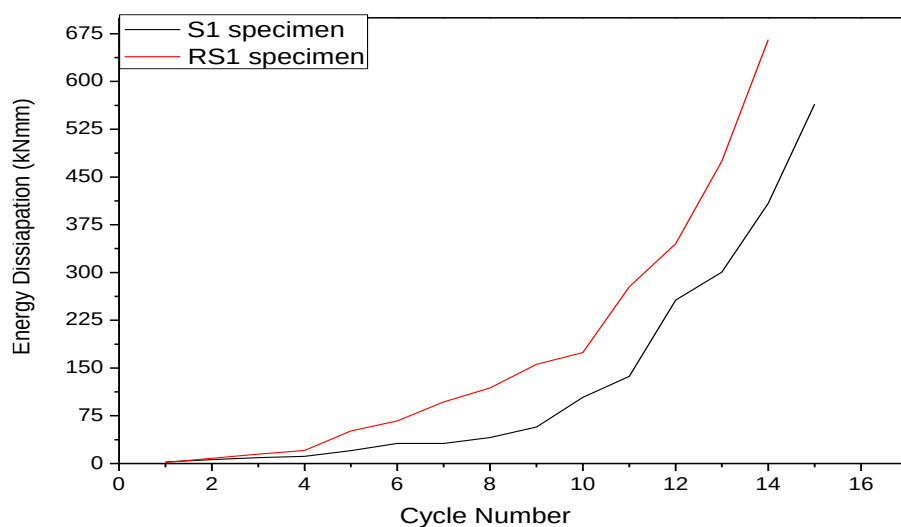


Fig. 14 – Energy Dissipation of S1 and RS1 specimen

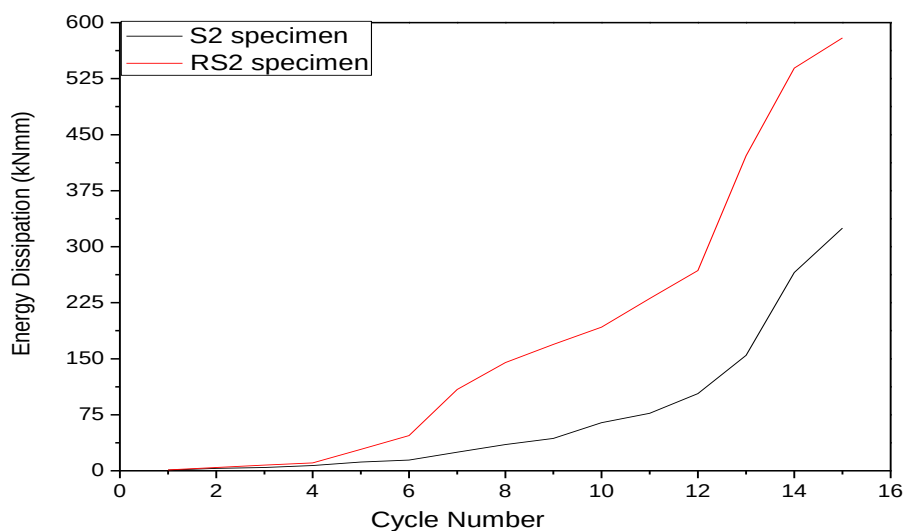


Fig. 15 – Energy Dissipation of S2 and RS2 specimen

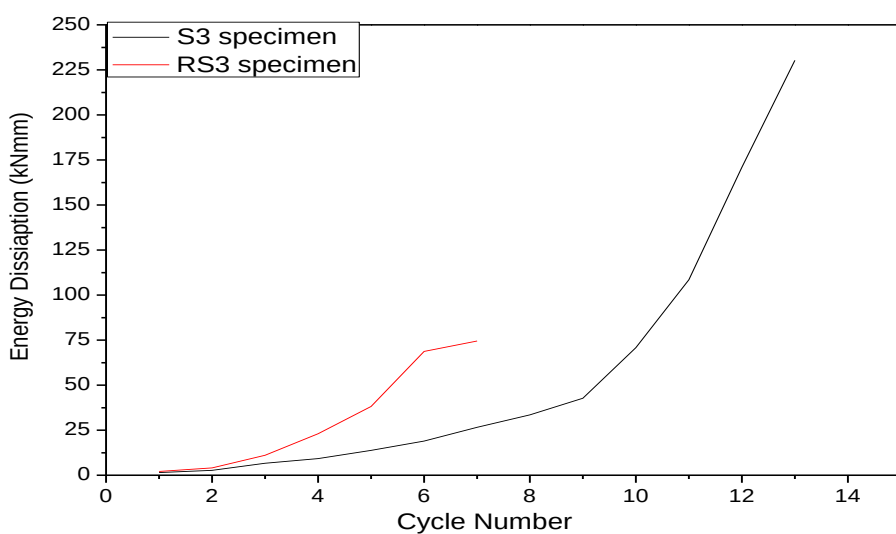


Fig. 16– Energy Dissipation of S3 and RS3 specimen

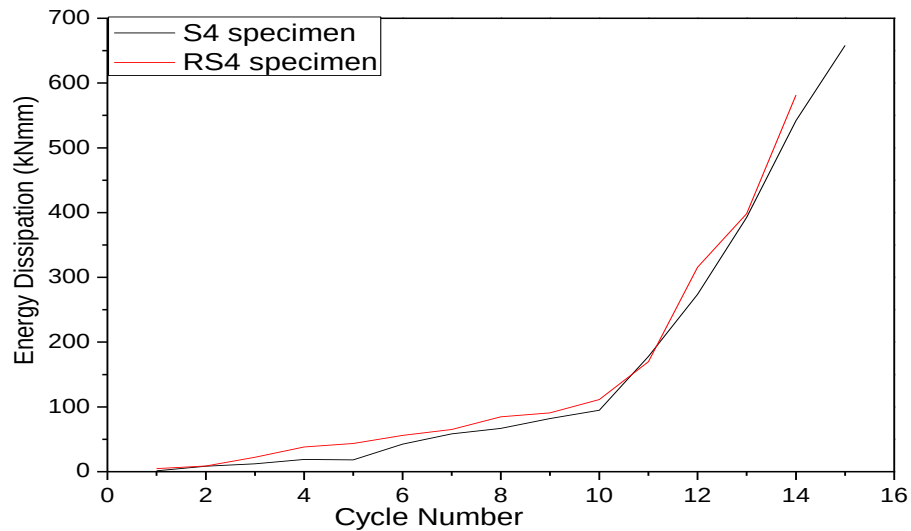


Fig. 17 – Energy Dissipation of S4 and RS4 specimen

RS1 specimen showed 24.65% increase in energy dissipation as compared to S1 specimen. RS2 specimen showed almost 100% increase in energy dissipation than its original S2 specimen. RS4 specimens have almost similar energy dissipation to that of S4 specimen as shown in Fig. 17. However, RS3 was failed earlier due to poor confinement therefore the continuation in further loading was not possible.

3.4 Stiffness Degradation

The beam-column joint stiffness was approximated as the slope of the peak-to-peak line in each loop [13]. Figs. 18-21 show the stiffness degradation with number of cycles for non-retrofitted and retrofitted specimen. This degradation may be due to concrete nonlinear deformations, flexural and shear cracking, distortion of the joint panel, slippage of reinforcement, loss of cover, debonding or delamination of CFRP, etc. A comparison of retrofitted specimen curve with corresponding non-retrofitted specimen curve shows that the initial stiffness of retrofitted specimen was less than its corresponding original specimen. This lower initial stiffness may be attributed to initial damage, lower elastic modulus of the CFRP sheet, and/or yielded steel bars at the initiation of the test as reported Al-Salloum and Almusallam [9].

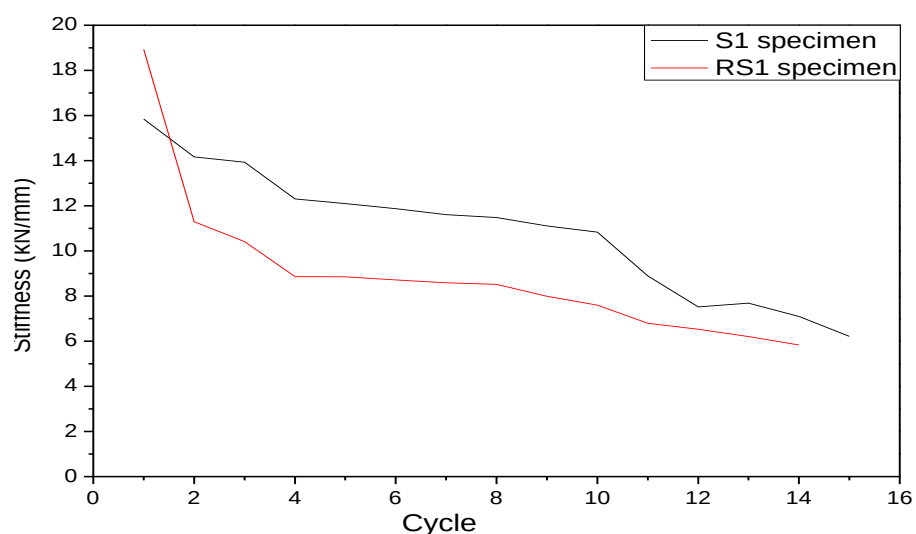


Fig. 18 – Stiffness degradation of S1 and RS1 specimen

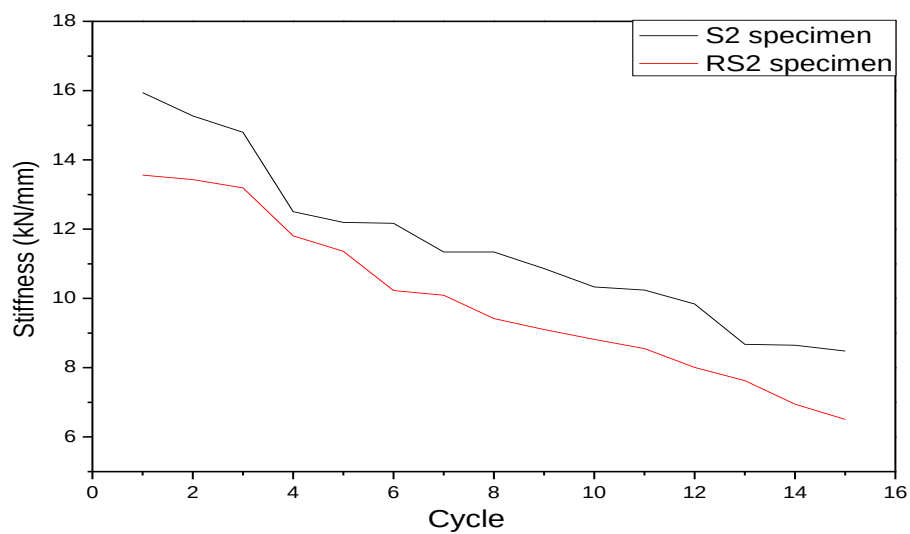


Fig. 19– Stiffness degradation of S2 and RS2 specimen

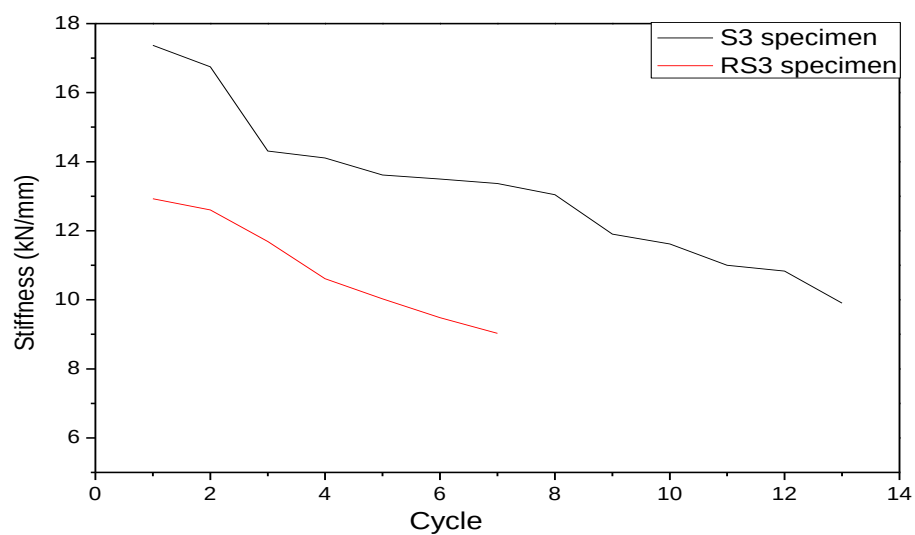


Fig. 20– Stiffness degradation of S3 and RS3 specimen

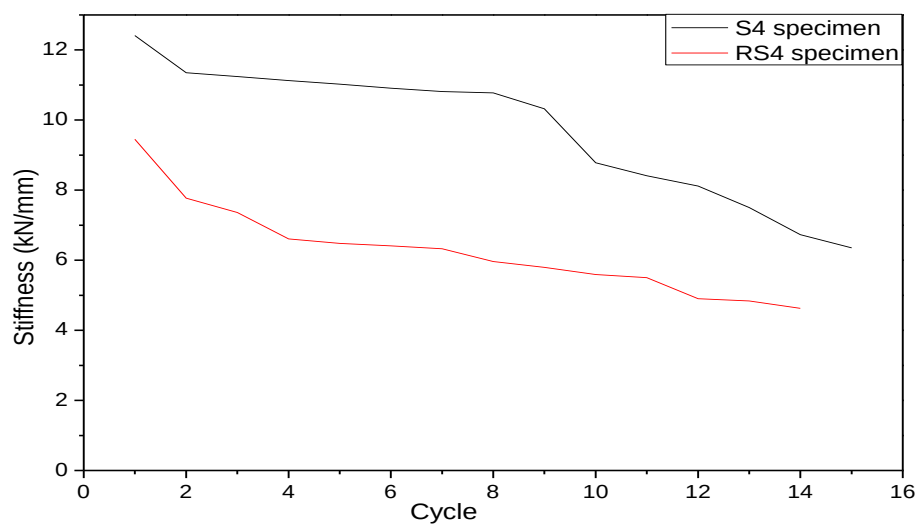


Fig. 21 – Stiffness degradation of S4 and RS4 specimen

4 Conclusions

From the experimental investigation and discussion, following conclusions are drawn.

- Seismic retrofitting method by using CFRP sheets used in this work, involved repairing the damaged specimen and upgrading the strength of joint. Overall, the retrofitting method almost restored the strength of the three beam column joint specimens and increased the deformation capacity in all specimens.
- The retrofitting method using CFRP sheets increased the displacement capacity of the specimens, and increased the energy absorption capacity of specimens thus improving the ductility.
- As the non-retrofitted specimens were damaged at a considerable level and then retrofitted. Therefore, retrofitted specimens resulted into more stiffness degradation as compared to non-retrofitted specimen because of initial damage, lower elastic modulus of the CFRP sheet, and/or yielded steel bars at the initiation of the test.
- This method has changed the failure mode from shear failure inside the joint region to ductile failure in the beam. Moreover, retrofitting makes the joint so strong that failure is directed to the beams as it helps the structure in energy dissipation through plastic hinge formation in beam.

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