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Retrofitting of exterior RC beam-column joints using ferrocement jackets

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Abstract. Beam-column joints are recognized as one of the most critical and vulnerable zones of a Reinforced Concrete (RC) moment resisting structure subjected to seismic loads. The performance of the deficient beam-column joints can be improved by retrofitting these joints by jacketing them with varied materials like concrete, steel, FRP and ferrocement. In the present study strength behavior of RCC exterior beam-column joints, initially loaded to a prefixed percentage of the ultimate load, and retrofitted using ferrocement jacketing using two different wrapping schemes has been studied and presented. In retrofitting scheme, RS-I, wire mesh is provided in L shape at top and at bottom of the beam-column joint, whereas, in scheme RS-II along with wire mesh in L shape at top and bottom wire mesh is also provided diagonally to the joint. The results of these retrofitted beam-column joints have been compared with those of the controlled joint specimens. The results show an improvement in the ultimate load carrying capacity and yield load of the retrofitted specimens. However, no improvement in the ductility and energy absorption has been observed.

Keywords: RCC beam-column joints; retrofitting; ferrocement; ductility; energy absorption; moment and rotation

1. Introduction

In various parts of the world, Reinforced Concrete (RC) structures, even in seismic zones are still being designed only for gravity loads. Such structures, though performing well under the conventional gravity load case, could lead to questionable structural performance under seismic or wind loads. In most cases, these structures are highly vulnerable to any moderate or a major earthquake. In addition to their vulnerable behavior in the seismic prone zones like Himalayan region in India, Iran, Turkey, New Zealand and fault regions in US etc., devastations from earthquake have also been seen at places believed to be seismically not-so-active (as shown in Fig. 1).

Beam-column joints are recognized as one of the most critical and vulnerable zones of a Reinforced Concrete (RC) moment resisting structure subjected to seismic loads. During an

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earthquake, the global response of the structure is mainly governed by the behaviour of these joints. Under the action of seismic forces, beam-column connections are subjected to large shear stresses in the joint region. These shear stresses are a result of moments and shear forces of opposite signs on the member ends on either side of the joint core. Typically, high bond stresses are also imposed on reinforcement bars entering into the joint. The axial compression in the column and joint shear stress result in principal tension and compression stresses that lead to diagonal cracking and/or crushing of concrete in the joint core. (Paulay and Preistley 1992, Hakuto *et al*. 2000). In the analysis and design of reinforced concrete frames beam-column joints are sometimes assumed as rigid. This simplifying assumption can be unsafe because it is likely to affect the distributions of internal forces and moments, reduce drift and increase the overall load-carrying capacity of the frame. The parametric studies of a simple sub-frame model reveal that the quasi-static monotonic behavior of unbraced regular reinforced concrete frames is prone to be significantly affected by the deformation of beam-column joints (Ricardo *et al*. 2013).

Extensive research has been carried out in the last two decades to improve the seismic performance of reinforced concrete beam-column joints. This has resulted in use of different materials like Fiber-Reinforced Polymers (FRP) and ferrocement as external laminates for retrofitting of structures. These materials provide continuous confinement of the joint area, thereby enhancing the yield load, initial stiffness and energy dissipation capacity considerably.

Akguzel and Pampanin (2010) examined the effects of axial load variation, on the column due to the frame lateral sway, on the performance of retrofitted beam-column joints and concluded that retrofit solution designed under constant load conditions could be inadequate under varying axial load condition.

Alsayed *et al*. (2010) studied the efficiency and effectiveness of Carbon Fiber-Reinforced Polymer (CFRP) sheets in upgrading the shear strength and ductility of four half-scale seismically deficient (inadequate joint shear strength with no transverse reinforcement) exterior beam-column joints. It was revealed that, in CFRP repaired or strengthened specimens, the degradation of stiffness with lateral movement was slow as compared to that of the corresponding control specimens. This is a desirable property in earthquake like situations.

Bousselham (2010) presented a comprehensive review and synthesis of published experimental

Fig. 1 Earthquake Damage of RC building inter-storey collapse in Bhuj, India

studies on the seismic rehabilitation of RC frame beam-column joints with FRP. The test results as per the published literature showed substantial enhancement in terms of strength, ductility, and energy dissipation of the FRP wrapped joints. On the other hand, it was reported that stiffness degradation significantly reduced in the presence of FRP.

Karayannis *et al*. (2008) studied the effect of retrofitting of RC exterior beam-column joints with reinforced concrete jackets. The beam-column joints were initially subjected to cyclic loading and then retrofitted using thin RC jackets and retested under the same load sequence. Test results indicated that the seismic performance of the retrofitted specimens was fully restored and in some cases substantially improved with respect to the performance of the same specimens in the initial loading, since they exhibited higher values of load capacity and hysteretic energy dissipation.

Tsonos (2008) studied the behavior of failed beam-column joint sub assemblages repaired with RC jacketing and carbon and epoxy jacketing. It was concluded that the employed repair and strengthening techniques were effective in transforming the brittle joint shear failure mode of original specimens into a more ductile failure mode with the development of flexural hinges into the beams. Damage of the RC jacketing strengthened specimens was concentrated in both the beam's critical region and in the joint area.

The conventional local retrofitting techniques such as concrete jacketing, steel plate jacketing or FRP jacketing, although, improve the performance of beam-column joints, but they exhibit substantial disadvantages such as the requirement of skilled labour for execution of work, increase in member sizes and higher cost of retrofitting. Although FRP retrofitting technique does eliminate many of the previous mentioned important limitations that other type of jacketing induce. However, Bosselham (2010) reported that studies had shown that the failure due to, or initiated by, debonding of the FRP sheets, including delamination, represented a potential scenario of rupture. On the other hand, the test results clearly demonstrated the important role of mechanical anchorage systems in limiting such undesirable mode of failure. Moreover, Karayannis and Sirkelis (2008), suggested that there were more realistic construction difficulties in the FRP application, such as the presence of orthogonal beams and slabs. Hence, ferrocement jacketing technique with small thickness can be a good alternative for FRP. Ferrocement is a composite material consisting of rich cement mortar matrix uniformly reinforced with one or more layers of very thin wire mesh with or without supporting skeletal steel. It has been widely used for construction of various structures such as water tanks, silos, boats, and shell and folded plate structures. ACI committee 549 has presented the current state-of-the-art report on ferrocement properties and its potential applications. The presence of wire mesh reinforcement in ferrocement improves crack resistance, impact strength, and toughness. Paramasivam *et al*. (1988) reported that a reinforcement arrangement in which the wire mesh is bundled and placed near the surfaces is preferred from the point of view of first crack strength and crack characteristics. Kazemi *et al*. (2005), performed a study to evaluate the retrofit technique for strengthening shear deficient short concrete columns. Ferrocement jacket reinforced with expanded steel mesh was used for retrofitting in the study. It was concluded that expanded meshes were more effective in shear strengthening of concrete columns and also specimens strengthened with expanded meshes did show distributed fine shear cracking even at large displacement levels indicating an increase in the ductility capacity.

Li *et al*. (2013) studied a method for rehabilitating reinforced concrete interior beam-column joints using ferrocement jackets with embedded diagonal reinforcements. 2/3 scale interior beam-column joints were prepared and tested under quasi-static cyclic loading. Test results indicated that the proposed rehabilitation method, using ferrocement with high strength mortar, could improve the seismic performance of interior beam-column joints. Strength of mortar was found to be the vital factor affecting the performance of strengthened specimens. Anchor bolts installed at the interface between ferrocement and concrete substrate improved the bonding and overall performance. The performance of ferrocement jackets was further found to improve with use of high strength cement mortar with suitable flowability. Shannag and Mourad (2012) developed such high strength flowable mortar using various combinations of silica fume and fly ash and concluded that such ferrocement can be considered as promising material for repair and rehabilitation of structures. Chalioris *et al*. (2014) used self compacting concrete jackets for repair and strengthening of beams that were designed to fail mostly in shear. It was concluded that with proper anchorage, the repaired members approach the strength, ductility, deformation capacity and model of failure of the ideal monolithic member having identical reinforcement details. Experimental results have indicated that ferrocement as a composite material can enhance the seismic performance of deficient beam-column joints in terms of peak horizontal load, energy dissipation, stiffness and joint shear strength. Shear distortions within the joints are significantly reduced for the strengthened specimens. High axial load has a detrimental effect on peak horizontal load for both control and ferrocement-strengthened specimens (Li *et al*. 2015).

In this paper, the effect of different wrapping techniques on retrofitting of RCC beam-column joints using ferrocement has been presented. It is very common in modern in situ construction techniques that the beams and the supporting columns are cast monolithically. But even in the old-fashioned way of concreting, a part of the column is usually cast with the beams in order to avoid construction joints in heavily stressed areas. Therefore, top T-joints are always formed during construction of frames, Karyannis and Chalioris (2000). Herein, in the present study a detailed experimental program had been devised to study the effect of different wrapping techniques on the behavior of exterior T shaped beam-column joints initially damaged to predetermined level and then retrofitted using ferrocement jackets. Since the standard codes of practice provide procedures to calculate the equivalent static load for the dynamic/cyclic loading, a monotonic static loading mechanism was used for the testing of the beam-column joints.

2. Experimental program

2.1 Materials

2.1.1 Cement:

Ordinary Portland cement of 43 grade with a specific gravity of 3.14 and conforming to IS: 8112 has been used in the present study. The average 7 and 28 days compressive strengths of standard cubes of surface area 50 cm^2 have been found to be 35.60 MPa and 45.50 MPa, respectively.

2.1.2 Aggregates

River bed sand with a fineness modulus of 2.09, confirming to zone - III and specific gravity of 2.54 has been used as fine aggregate. 20 mm nominal size coarse aggregates with a specific gravity of 2.65 have been used for designing the concrete mix for the beam-column joint.

2.1.3 Concrete mix

M20 grade concrete mix has been designed as per IS code design procedure using the constituent

Table 1 Physical properties of steel bars and steel wire mesh

Sr. No	Diameter of bars/ mesh wire	Grade of Steel	Yield Strength (N/mm^2)	Ultimate Strength (N/mm^2)	Percentage Elongation at failure (%)
1	10 mm	Fe415	445.55	509.20	15.50
2	8 mm	Fe415	559.50	634.13	20.30
3	6 mm	Fe250	442.42	612.70	32.90
4	2.4 mm wire mesh	Fe250	400.00	511.36	2.52

materials with properties as listed in the preceding sections. The water-cement ratio achieved for the mix came out to be 0.48, whereas the proportions of materials used were in the ratio 1: 1.46: 2.94 (cement: sand: coarse aggregate). The compressive strength of 150 mm cubes prepared for the designed mix proportions, after 7 days and 28 days of curing has been found to be 21.5 MPa & 29.0 MPa, respectively.

2.1.4 Reinforcing steel and wire mesh

The properties of the reinforcing bars and the steel wire mesh used as a part of the ferrocement jacket are presented in Table 1.

2.2 Beam column joints - Detailing and testing arrangement

To study the proposed behavior, five exterior beam-column joint specimens have been cast using M-20 grade concrete. 10mm and 8mm diameters bars of Fe-500 grade steel have been used as reinforcement and 6mm diameter mild steel bars have been used as ties in the beam part of the joint. The column was rectangular in shape with dimensions 225 mm×150 mm and a length of 1000 mm, and the beam has dimensions of 225 mm×150 mm in all test specimens with a length of 500 mm. In all five joints the column main reinforcement consisted of 4 no’s of 8 mm diameter bars, whereas, in the beam portion, the reinforcement consisted of 2 no’s of 10 mm diameter bars in tension zone and 2 no’s of 8 mm diameter in the compression zone. An anchorage length of 650 mm from the face of beam is provided to both sides of column. The RCC beam-column joint was designed using limit state method considering it to be an under-reinforced section. The ties for the specimens consisted of rectangular hoops of 6 mm diameter of size 185 mm×110 mm placed 100 mm c/c in the column portion as well as in the beam portion. The reinforcement detailing is shown in Fig. 2.

The beam-column joint specimen was fixed on to the loading frame using an arrangement as shown in the Fig. 3. The joints were subjected to a point load at a distance of 300 mm from the face of column. The value of deflection has been taken with the help of three LVDT’s, wherein, one LVDT had been set at the free end of a beam, the second at a distance of 150 mm from free end and third at 100 mm from column face to note the respective deflections in the beam. The specimens were attached to the frame with the help of nut-bolts and have been tested using a hydraulic loading jack facility. The control specimen was loaded to failure and the ultimate load for the same was noted. The remaining specimens have then been subsequently loaded up to 80% of the ultimate load, so obtained for the control specimen. After retrofitting the 80% stressed RC beam-column joints with ferrocement jacketing, the ultimate strength of the remaining four specimens has been evaluated.

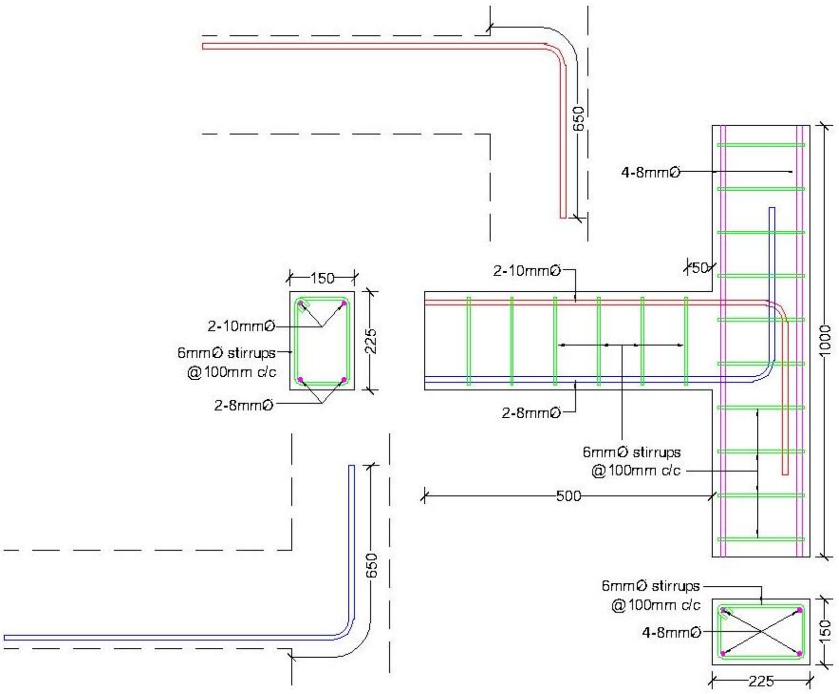


Fig. 2 Reinforcement detailing of beam column joint

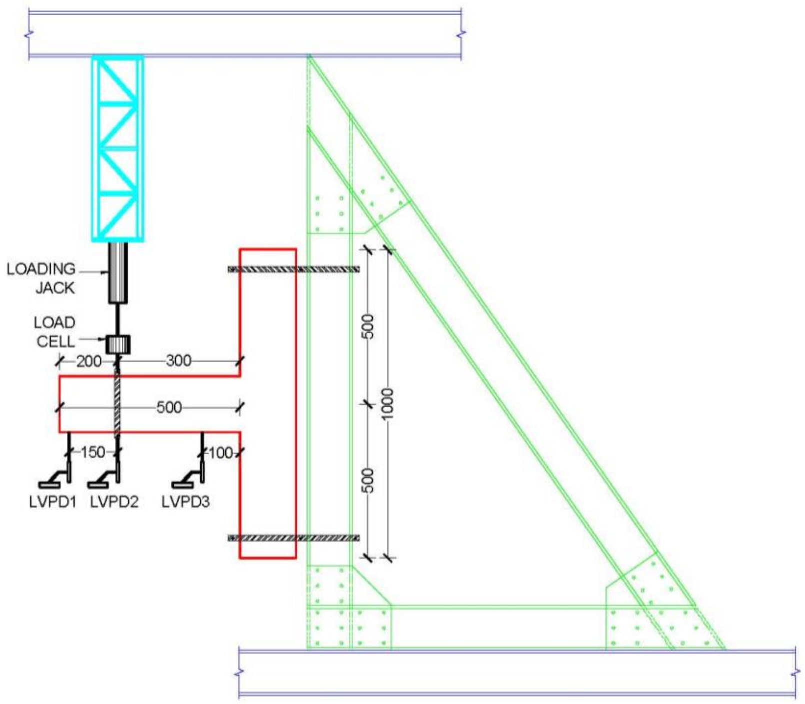


Fig. 3 beam column specimen attached with frame

2.3 Casting of composite beam-column joints

A steel mould of dimensions 225 mm×150 mm having a length of 500 mm for the beam portion and a 225×150 mm mould with a length 1000 mm for the column portion was used for the casting of beam-column joints (See Fig. 4). Blocks of 20 mm thickness were placed under the reinforcement to provide uniform cover.

After placing the desired reinforcement, concrete mix, obtained using the designed proportions, was poured in the mould. The compaction of the concrete mix has been done using a needle vibrator. The beam-column joints were then removed from the mould after 24 hours and subsequently cured for the remaining period of 27 days.

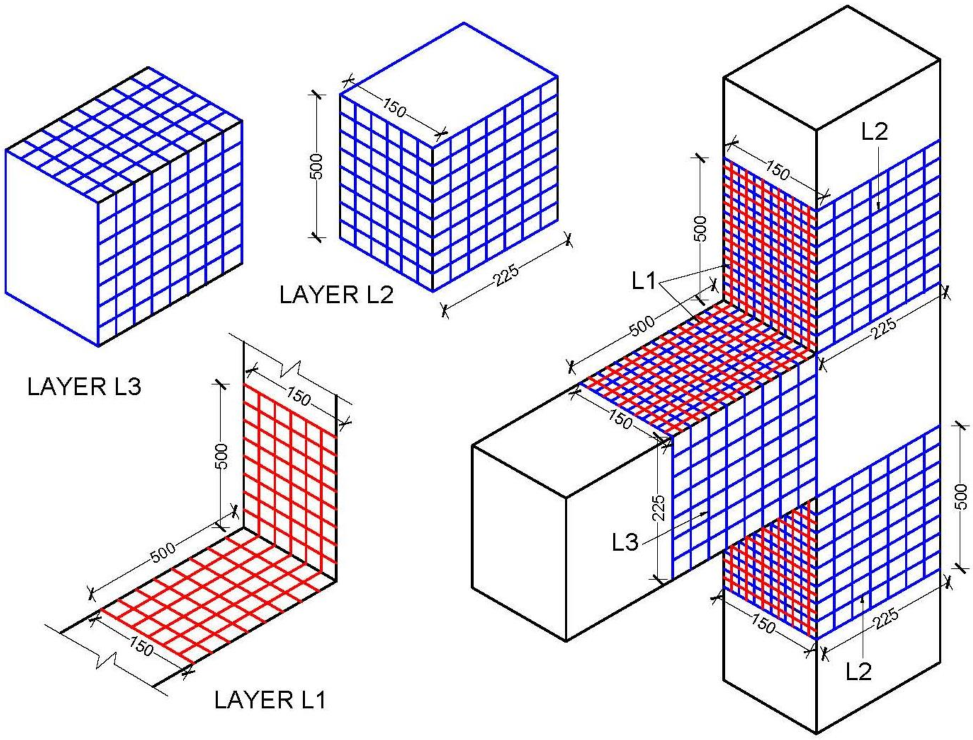
2.4 Process of retrofitting

The four beam column joints which were loaded up to 80% of the ultimate load were retrofitted using two different schemes. The retrofitting schemes consisted of wrapping the beam portion and column portion with the help of rectangular wire mesh. Firstly, the surfaces of specimens were cleaned, and then the specimens were wrapped with wire mesh using specific wrapping technique. The cement slurry was applied as a bonding agent to the surfaces of beam-column joints. The 20 mm thick cement mortar mixed in proportions of 1:3 and having water cement ratio (w/c) 0.45 was applied to the stressed beam-column joint specimen. The beams were cured for 7 days, using moist jute bags, before testing. They have been tested using the same procedure as adopted for testing of control joint specimen.

2.4.1 Retrofitting schemes

Two types of retrofitting schemes were used for wrapping of wire mesh as a part of the jacketing process. In the first scheme, RS-I, two L-shaped wire mesh pieces of appropriate size have been wrapped on the lower and upper faces of the beam at the joint. Then cement mortar of thickness 20 mm has been applied on the wire mesh bonded on the beam-column joint as shown in Fig. 5. In the second retrofitting scheme, RS-II, along with two L-shaped wire mesh pieces at top and bottom

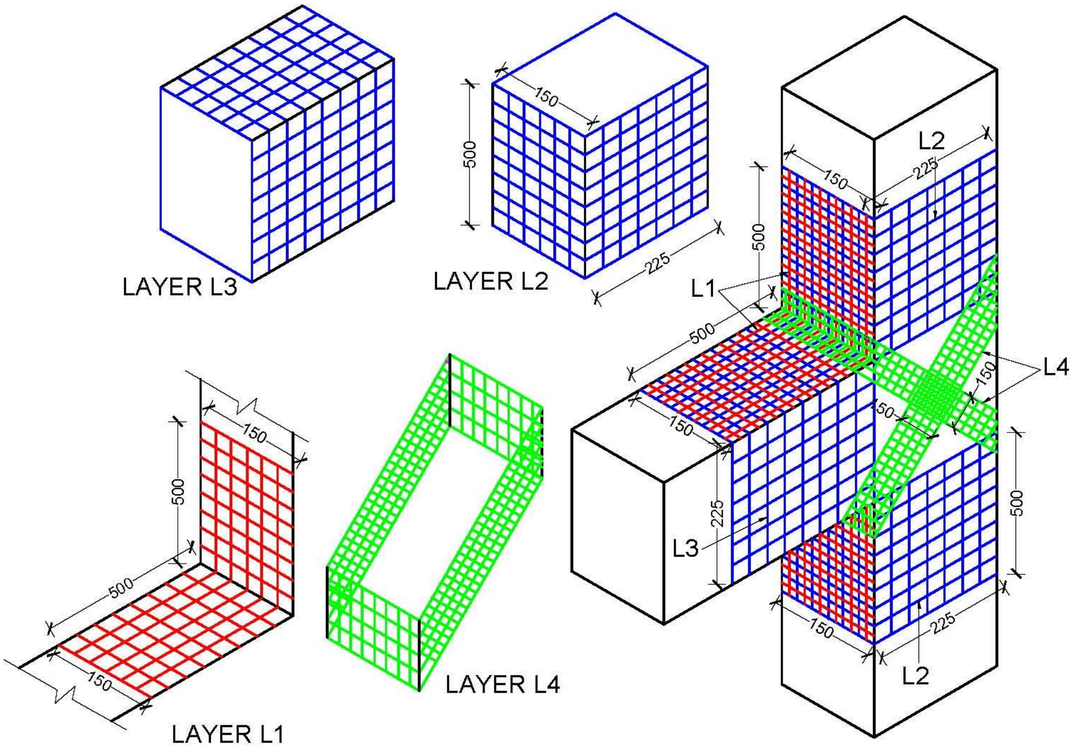
Fig. 4 Steel mould for casting of specimens



(a) Retrofitting Scheme - I

(b) Specimen retfofitted using Scheme - I

Fig. 5 Specimen retrofitted with mesh wire (Scheme I and Scheme II)



(a) Retrofitting Scheme - II

(b) Specimen retfofitted using Scheme - II

Fig. 6 Specimen retrofitted with mesh wire (Scheme I and Scheme II) faces, wire mesh has also been applied diagonal to the joint. Subsequent to this, cement mortar of thickness 20 mm has been applied on the wire mesh bonded on the beam-column joint as shown in Fig. 6. The advantage of providing diagonal beam reinforcement in beam-column joints had been studied by Chalioris *et al.* (2008) and Tsonos *et al.* (1992) and the concluded that joints with X-bars exhibited enhanced cyclic performance and improved damage mode since a distinct flexural hinge was developed in the beam-joint interface. Further, the combination of crossed inclined bars and stirrups in joint area resulted in enhanced hysteretic response and excellent performance capabilities of the specimens.

3. Results and discussions

3.1 Testing methodology

Out of the five specimens cast, one specimen, which was designate as the control specimen, has been loaded to ultimate load and the data corresponding to it was recorded through data acquisition system. The rest four specimens have subsequently been loaded to 80% of the ultimate load and then retrofitted using different wrapping techniques.

The ultimate load of the control beam-column joint comes out to be 64.1 kN, with a maximum deflection equal to 24.1 mm at free end of the beam. The rest four beam-column joints have been loaded to 80% of ultimate load of control specimen i.e., 51.28 kN. Subsequently, the retrofitting of the beam-column joints has been done with cement mortar of thickness 20 mm along with wire mesh bonded on the four beam-column joints. Out of the four, two beam-column joints have been retrofitted using retrofitting scheme RS-I and the other two retrofitted using retrofitting scheme RS-II, as per the variation in mesh wire wrapping as shown in Figs. 5 and 6 in the previous section. After 7 days of curing, the beam-column joints have been tested again adopting a similar procedure to the one used for testing of control beam-column joint. The corresponding results have been recorded in the form of loads and deflections. The test results of these retrofitted specimens have been compared with controlled specimen and a comparative analysis of the retrofitting schemes has also been carried out.

The beam-column joints designations provided are as under:-

1. Control Specimen - Control beam column joint (CS)
2. Retrofitted Beam-column joint 1 - R1 (80% loaded-type I Retrofitting Scheme)
3. Retrofitted Beam-column joint 2 - R2 (80% loaded-type I Retrofitting Scheme)
4. Retrofitted Beam-column joint 3 - R3 (80% loaded-type II Retrofitting Scheme)
5. Retrofitted Beam-column joint 4 - R4 (80% loaded-type II Retrofitting Scheme)

3.2 Control beam-column joint

For the beam-column joint, tested as a control specimen, the load is applied and deflection is noted at the three locations with the help of LVDT's. During the initial part of the loading process, the deflection up to 10 kN is very less and subsequently it increases almost linearly with the increase in load. However, after reaching the load value of 55 kN it increases at much higher rate till the ultimate load of 64.1 kN, as shown in Fig. 7. The first crack in the control specimen is observed at a load of 27.56 kN; thereafter the number of cracks increased and the same have been observed to spread over the entire area of the beam-column joint as shown in Plate 1.

3.3 Effect of method of wrapping technique

The effect of the different retrofitting schemes on the strength and ductility parameters of the beam-column joint is discussed as below:

3.3.1 Effect on ultimate load

The effect on strength of the retrofitted RCC beam-column joint R1 loaded to a predefined 80% stress level is shown in Fig. 7. The crack patterns in retrofitted beam column joints are shown in Plates 2 & 3.

It is observed from the experimental data and the corresponding graph that retrofitting leads to a significant increase in the ultimate load carrying capacity from 64.1 kN (control specimen) to 81.45 kN for the R1 specimen, whereas the deflection corresponding to ultimate load of 81.45 kN is reduced to 16.23 mm as compared to 24.1 mm for the control specimen at 64.1 kN load. Also there is a considerable increase in the yield load carrying capacity from 55 kN(control specimen) to 75 kN for the retrofitted specimen. For the R2 specimen an exactly similar trend is observed and increase in load is also of almost of the same order i.e., from 64.1 kN (control specimen) to 81.52 kN with deflection of about 16.61 mm. The yield load also increases from 55 kN (control specimen) to 75 KN. Thus, on an average, for beam-column joints stressed to 80% of the ultimate load and retrofitted using the scheme RS-1, the ultimate load increase is of the order of 27.12 % and yield load increases by 36.36 %.

It is observed from the experimental data and the corresponding graph that retrofitting leads to increase in the ultimate load carrying capacity from 64.1 kN (control specimen) to 102.21 kN for the R3 specimen, whereas, the deflection corresponding to ultimate load of 102.21 kN is reduced to

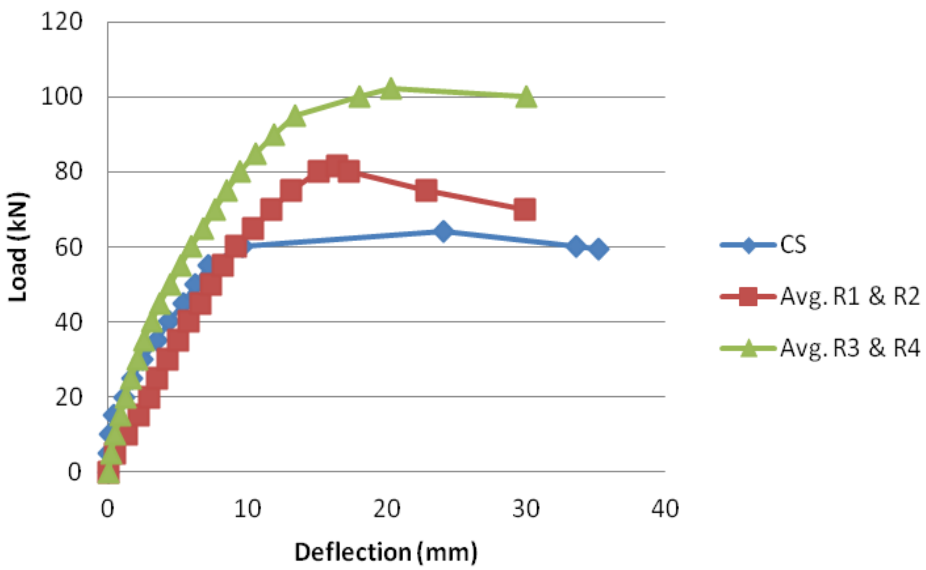


Fig. 7 Average values of load and deflection at free end of beam of controlled and retrofitted specimens

20.31 mm as compared to 24.1 mm for the control specimen at 64.1 kN. Also, there is a considerable increase in the yield load from 55 kN (control specimen) to 95 kN for the retrofitted specimen. For the R4 specimen a very similar trend is observed and increase in load is also of almost of the same order i.e. from 64.1 kN (control specimen) to 102.35 kN with deflection of about 20.35 mm. The yield load increases from 55 kN (control specimen) to 95 kN. Thus, on an average, for beam-column joints stressed to 80% of the ultimate load and retrofitted using the scheme RS-2, the ultimate load increase is of the order of 59.56% and yield load increases by 72.73%.

From the comparative analysis, it can be observed from Fig. 7 that the beam-column joints retrofitted using different wrapping techniques, show different behavior. The specimens with type two retrofitting scheme show maximum improvement in their ultimate load from 64.1 kN (control specimen) to 102.28 kN without much increase in the deflection, as well as in the yield load. The ultimate load for the type two retrofitting scheme is higher by 25.52% and the yield load by 26.67% when compared with the beam-column joints retrofitted using the type one retrofitting scheme.

3.3.2 Effect on ductility

The values of ductility ratio, which is the ratio of deflection at ultimate load to yield load, are shown in Table 3. The ductility ratio of the controlled specimen is 1.16 and that achieved for beam-column joints retrofitted using type one retrofitted specimen R1 is 1.086, indicating a reduction in the ductility ratio after retrofitting. The ductility ratio of type one retrofitted specimen R2 is 1.087 which is also less than the value of controlled specimen.

The ductility ratio achieved for beam-column joints retrofitted using type two retrofitted specimen R3 is 1.076, which is less than the ductility ratio of controlled specimen which is equal to 1.165. Similarly, the ductility ratio of type two retrofitted specimen R4 is also less than the value of controlled specimen.

On comparing the average values of ductility ratio of type one retrofitting with type two retrofitting, the ductility ratio of type two retrofitting is less than type one retrofitting.

3.3.3 Effect on energy absorption

The values of energy absorption, which is the area under the load deflection tri-linear curve, are presented in Table 3 and the Figs. 8 to 10 show the tri-linear curve for control specimen as well as for retrofitted specimens. The value of energy absorption in case of type one retrofitted specimen R1 decreases by nearly 8.176 % than the control specimen, whereas, in the case of type one retrofitted specimen R2, the value of energy absorption decreases by nearly 4.82 % than the control specimen.

Table 3 Ductility Ratio and Energy Absorption at Free End of Beam of Controlled and Retrofitted Specimens R1, R2, R3 and R4

S. No.	Beam-Column Joint Designation	P_y (kN)	$P_{ult.}$ (kN)	Ductility Ratio	Energy Absorption (kN-mm)
1	CS	55.00	64.10	1.16	1892.23
2	R1	75.00	81.45	1.09	1748.60
3	R2	75.00	81.52	1.09	1804.61
4	R3	95.00	102.21	1.08	2774.26
5	R4	95.00	102.35	1.08	2790.77

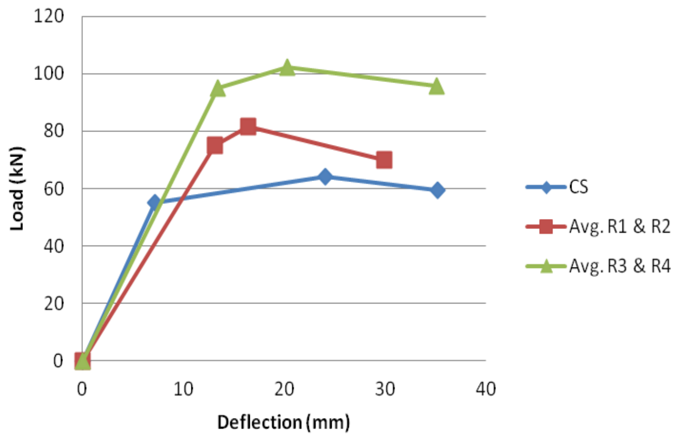


Fig. 8 Trilinear curves for average values of load and deflection at free end of beam of controlled and retrofitted specimens

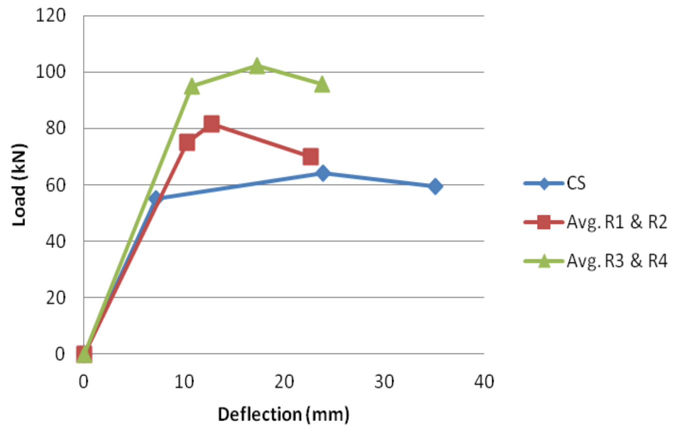


Fig. 9 Trilinear curves for average values of load and deflection at 150 mm from free end of beam of controlled and retrofitted specimens

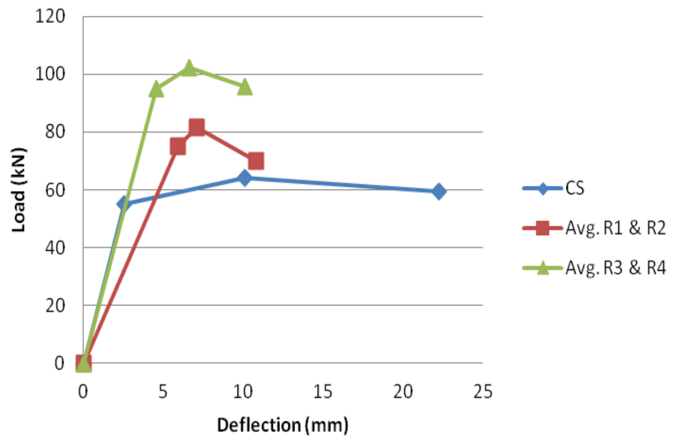


Fig. 10 Trilinear curves for average values of load and deflection at 100 mm from column face of controlled and retrofitted specimens

On an average, the value of energy absorption, in case of type one retrofitting technique, decreases by nearly 6.498% as compared to the control specimen.

The value of energy absorption in case of type two retrofitted specimen R3 increases by nearly 46.617 % than the control specimen. And in case of type two retrofitted specimen R4, the value of energy absorption increases by nearly 47.516% than the control specimen. On an average the value of energy absorption in case of type two retrofitted specimen increases by nearly 47.07% than the control specimen.

On comparing the average values of energy absorption of type one retrofitted specimen with type two retrofitted specimen, the energy absorption of beam-column joint retrofitted using the type one retrofitting scheme is nearly 56.584% less than the type two retrofitting specimen. It is worth mentioning here that the testing of the specimens has been carried out under the load control conditions due to the constraints of the set-up available in the laboratory. However, it is expected that after the failure of the retrofitted specimen at ultimate load, it will follow the path of un-retrofitted specimen. Thus, it can be said that there will be no loss of ductility and energy absorption capacity after retrofitting.

3.3.4 Damage mode and cracking pattern

From the crack patterns observed during testing and shown in plate 1 to 5, it can be concluded that in control beam-column joints and joints retrofitted using scheme RS-I there is a mixed mode of failure, i.e., failure in both beam-column junction as well as at the junction of beam and column was observed. However, when the beam-column joints are strengthened by providing diagonal wire mesh along with L shaped wire mesh on top and bottom in scheme RS-II the failure in the joint body is relatively very less and is observed to have shifted to beam-column junction, thereby, improving the performance of retrofitted joint. Further, it can be concluded from the failure modes that by increasing the amount of diagonal reinforcement beam-column joint area can be protected by shifting the plastic hinge formation to the beam.

4. Conclusions

Based on the experimental results presented in the preceding sections, the following conclusions can be drawn.

• The load carrying capacity of retrofitted beam-column joints for both types of retrofitting techniques increases significantly as compared to control beam-column joint.

• Specimens with mesh wire wrapped diagonally show maximum improvement in their ultimate load.

• There is an increase in the yield load also in both types of retrofitting; however, in case of specimens with mesh wire wrapped diagonally the increase in the yield load is more significant.

• The ductility ratio of retrofitted specimen is less than the ductility ratio of control specimens and also the ductility ratio of those specimens in which mesh wire is wrapped diagonally is less than those specimens in which mesh wire is wrapped in L shape only.

The value of energy absorption, in case of those specimens in which wire mesh is wrapped in the shape of L decreases as compared with control specimen, but the value of energy absorption in case of those specimens increases in which wire mesh is wrapped diagonally than the control specimen, indicating that the diagonal wrapping of the wire mesh is a better alternative.

Plate 1 Control Specimen

Plate 2 Specimen R1

Plate 3 Specimen R2

Plate 4 Specimen R3

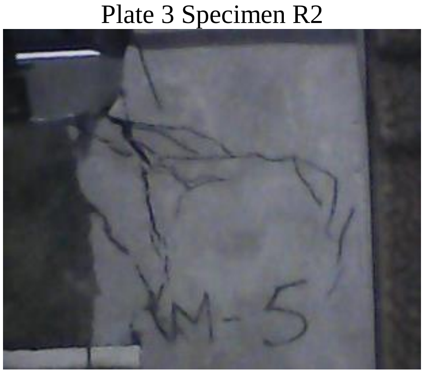


Plate 5 Specimen R4

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