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Performance evaluation of different strengthening measures for exterior RC beam-column joints under opening moments

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Abstract. Devastating RC structural failures in the past have identified that the behavior of beam-column joints is more critical and significantly governs the global structural response under seismic loading. The congestion of reinforcement at the beam-column joints with other constructional difficulties has escalated the attention required for strengthening RC beam-column joints. In this context, numerous studies have been carried out in the past, which mainly focused on jacketing the joints with different materials. However, there is no comparative study of different approaches used to strengthen RC beam-column joints, from efficiency and cost perspective. This paper presents a detailed investigation carried out to study the various strengthening schemes of exterior RC beam-column joints, viz., steel fiber reinforcement, carbon fiber reinforced polymer (CFRP) strengthening, steel haunch strengthening, and confining joint reinforcement. The effectiveness of each scheme was evaluated experimentally. These specimens were tested under horizontal loading that produced opening moments on the joints and their behavior was studied with emphasis on strength, displacement ductility, stiffness, and failure mechanism. Special attention was given to the study of crack-width.

Keywords: beam-column joint; RC member; strengthening; strength; ductility; stiffness; CFRP

1. Introduction

The quality of steel structural construction is better than that of RC structures, mainly due to better material homogeneity of steel compared to concrete. However, concrete/reinforced concrete (RC) is a versatile constructional material, second- only to water in quantity used in the world, and has advantages like moulded into any shape using semi-skilled labour. But the use of RC frames, especially in earthquake prone areas, has resulted in several failures of framed structures. This is mainly attributed to the poor performance of beam-column joints, which have failed either due to non-provision of sufficient shear-reinforcement within the beam-column joint or congestion of reinforcement resulting in improper consolidation of concrete within the joint (Subramanian and Prakash Rao,2003, Subramanian, 2013) Hence, several codes have stipulated specific rules for the detailing of reinforcements in the beam-column joint in order that they have sufficient strength, ductility, and satisfactory structural performance, even under adverse loading. Previous research in beam-column joints has identified the need for (1) proper anchorage of longitudinal beam reinforcement into the column, (2) adequate shear reinforcement, and (3) proper concreting and compaction of joints. When these are not considered, vulnerable weak beam-column joints result, leading to catastrophic structural failures (Subramanian, 2013 and 2015). Under lateral loading, moments acting on beam- column joints may lead either to the opening or closing of the joints. Previous studies on the behaviour of beam-column joints have confirmed that opening joints lead to adverse effects and hence considered with care to avoid failures (Subramanian and Prakash Rao,2003, Uma and Meher Prasad, 2006, Kaur and Lal, 2012a,b, Ahmed *et al.* 2019). Thus, there is an urgent need to strengthen existing weak beam-column joints to meet the desired performance levels. Past studies on various strengthening measures have confirmed significant improvement in their performance (Dubey *et al.* 2015, Elmasry *et al*. 2017, Dar *et al.* 2015, 2017a-b, 2019). In many strengthening schemes, jackets made of Carbon Fiber-Reinforced Polymer (CFRP), Ferro-cement, or even reinforced concrete have been suggested.

CFRP jacketing is considered better, as it results in improvement of strength, yield load resistance, stiffness, and desirable mode of failure (Singh *et al.* 2014a-b, Elmasry *et al*. 2017, Sharma and Sharma, 2017, Balasubramanian *et al.* 2011, Sheela &Geetha, 2012, Vijayalakshmi *et al.* 2010, Gnanapragasam *et al.* 2016, Shahbazpanahi *et al*. 2018, Prota *et al.* 2014, Kumara *et al*. 2019). It has to be noted that the orientation of CFRP greatly influences the performance of beam-column joints, particularly in the absence of proper joint detailing (Mahmoud *et al*. 2014). Ferrocement jacketing is also found to improve strength and energy absorption characteristics. However, it affects the ductility of the beam-column joint (Bansal *et al.* 2016). In addition, the improvement in the load-carrying capacity of the beam-columns joints is not substantial at stress levels ranging from 50-100% damage levels (Dubey *et al*. 2015). Incorporation of U-bars with

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conventional RC jacketing is found to drastically improve the moment resistance of RC beam-column joints (Sivakumar *et al.* 2015). Additionally, the cracking of the joint was also controlled largely (Kannan *et al.* 2014). Addition of steel fibers in the RC beam-column joint designed and detailed without shear reinforcement, offered an observable enhancement in ductility, when the fiber volume fraction was limited to 1.5% (Bansal *et al.* 2013). The tensile strength, toughness, as well as ductility significantly increased due to the addition of steel fibers in limited volumes to ensure adequate workability (Liu, 2006). A few researchers have also found that the incorporation of a haunch element at the beam-column joint showed great improvement in strength as well as stiffness (Genesio *et al.* 2010, Rao *et al.*2013, Rahmi *et al*. 2017).

A thorough review of past research on beam-column joints, shows that a comparison of the different strengthening schemes, to is lacking. It will be beneficial to the owners, contactors, and designers, if a study which reveals not only the advantages of different strengthening schemes but also their efficiency and cost. Hence, an attempt is made in this paper to present an experimental investigation to compare the performance and effectiveness of steel fiber reinforcement, carbon fiber reinforced polymer (CFRP) s, steel haunch, and confining joint reinforcement, on the behavior of exterior RC beam-column joints. These specimens were tested under horizontal loading creating opening moments and their behavior studied with emphasis on strength, displacement ductility, stiffness, and failure mechanism. Special attention was also given to the formation of cracks and their width. Since the testing was done in India, Indian Standards were followed for material testing; however comparable ACI codes are also cited wherever necessary.

2. Objectives and scope of this study

To assess the performance and effectiveness of different strengthening schemes a RC beam-column joint with no strengthening scheme was also tested, which will give benchmark values for strength, stiffness, and ductility. For future references, this specimen will be referred as BMS (benchmark specimen). Two more specimens similar to BMS were prepared and strengthened with CFRP (named as CFRPS) and steel haunch (named as SHS). Two more specimens were tested in order to study the effect of ductile detailing of joints (as detailed in codes like IS 13920:2016, ACI 318:2019), and referred as DDS. For future references this specimen will be referred as DDS. One more specimen was prepared by incorporating discontinuous steel fiber reinforcement (2%), in addition to the conventional steel reinforcement that was adopted in BMS, this is referred as SFRS. The strength, stiffness and ductility enhancement in all these specimens were investigated in terms of percentage improvement to assess the efficiency of each adopted scheme.

In total, five specimens were prepared to achieve the objective of this study, which was followed by the testing of the materials involved and then the testing of the specimens under lateral loading so that the joints are subjected to opening moments.

3. Experimental study

This section presents the details of preparation of the five RC beam-column joint specimens, testing of the different materials used and the test set-up adopted for the detailed testing.

3.1 Preparation of specimens

Five half-scale specimens were prepared with the size of the beam as 200 mm × 250 mm (b × d) and that of the column as 200 mm × 200 mm. The length of the beam and the height of the column were fixed at 1900mm and 1000mm respectively. The reinforcement for the beam and the column was quantified as per the conventional code of practice for reinforced concrete (IS 456:2000) and the beam-column joint was designed for a lateral load of 17kN. The joint was designed in such a way that 'strong column and weak beam' condition is satisfied. The primary reinforcement (longitudinal reinforcement) for the beams consists of four bars of 16mm diameter on the tension side and two bars of the same diameter on the compression side. Two-legged stirrups of 8 mm diameter with a uniform spacing of 100 mm centre-to-centre were adopted as shear reinforcement. The primary reinforcement for the columns was four bars of 16mm diameter provided at the corners of the section; Two-legged 8 mm stirrups at 75 mm centre-to-centre were used as secondary reinforcement. The development length adopted for the beam and the column was 480mm and 600mm respectively. This reinforcement detailing was used in all the five specimens and shown in Figure 1 (a) & (b). Furthermore, except for the benchmark specimen (BMS), each strengthened specimen was strengthened as already discussed. For the DDS, apart from the common reinforcement, a special confining reinforcement (10mm bars) in the form of hoops was provided over the entire joint at a spacing of 25mm centre-to-centre (as per the usual ductile detailing practice) and shown in Figure 1 (c). CFRPS, CFRP sheets of 0.12mm thick (and density = 1.8 g/cm3) were adopted for strengthening the beam-column joint and were extended into the beam and column over and extra length of 100mm. Proper surface treatment was carried out to ensure proper bonding between the CFRP sheets and the RC beam-column joint by using epoxy adhesives. For SFRS, hook ended dramix steel fibers (with an aspect ratio of 65 and 45 degree hooked ends) of size 62mm × 0.95mm were added to the concrete mix (1.5% by volume) at the corner joints over the same region as was in the CFRPS. For SHS, a steel haunch was fabricated using a mild-steel plate 10mm thick, 277m long, inclined at 45o and welded to two base plates of the same thickness and 200mm long. Two such haunches were fixed at the corners of the SHS by means of 6 black bolts of 9.8 grade and 13mm diameter on each face. A clear cover of 40 mm was adopted on all the four sides for both beams as well as columns. A rebar detector was used to avoid cutting of reinforcement bars during drilling process. Furthermore, a thin layer of rich mortar was placed between the interface of beam-column joint and steel plates to resolve any possibility of non-perfect orthogonality between the beam and the column. For preparing all the five

Table 1 Material properties of ordinary Portland cement used

S.No.	Characteristics	Values obtained from tests	Values specified by IS 269
1	Standard consistency (%)	31.5	-
2	Fineness of cement as retained on 90 μm sieve-residue by weight (%)	0.75	10% (Maximum)
3	Setting time (minutes)		
	Initial	28	30 (Minimum)
	Final	421	600 (Maximum)
4	Specific gravity	3.15	
5	Compressive strength (MPa)		
	7-days	24	22 (Minimum)
	28-days	35	33 (Maximum)

specimens, M20 grade concrete was used and was prepared in accordance with IS 10262:2019(similar to ACI 211.1-91) at ambient temperature in the Structural Engineering Laboratory of National Institute of Technology Srinagar. Ordinary Portland cement (OPC) of grade 53 was used as the binder. For the preparation of concrete, locally available aggregates restricted in size to 12-14 mm and locally available river sand passing through 4.75mm sieve (as per IS 383:2016) were used as coarse and fine aggregates respectively. Water to binder ratio of 0.53 was adopted. Potable water was used for the preparation as well as curing of the specimens. A 2mm thick cold-formed steel sheet was used for creating a uniform plane surface for the smooth commencement of the testing. Polytetrafluoroethylene (PTFE Pad) of 10mm thickness was used to provide a frictionless surface for free sliding of specimens during the testing process as shown in Figure 2. Figure 3 shows a schematic view of SFRS, CFRPS, and SHS.

3.2 Material testing

To determine the actual properties of the different materials used in the preparation of various specimens, relevant tests were performed on each of them.

3.2.1 Material tests on cement, aggregates and concrete

OPC bearing the brand name Khyber Cement of grade 53, conforming to Indian Standard (IS -269:2015) was adopted in the current study (see also ACI 301-10). Standard consistency, fineness -, initial and final setting time and specific gravity tests were conducted on the cement specimens, and the average values of these tests are presented in Table 1. Both the coarse, as well as fine aggregates that were locally available and confirming the Indian Standards, were adopted. All the relevant tests pertaining to the mechanical properties like specific gravity, bulk density, fineness modulus, etc., were carried out. The average of the test results is given in Table 2.

As discussed earlier, M20 grade concrete was prepared for the casting of specimens. The mix design was followed in accordance with the Indian Standard IS- 10262:2019 (Similar to ACI 301-10)at ambient temperature. The details of the mix design are given in Table 3. Concrete cubes were prepared in accordance with the Indian Standard IS516:1989, and tested for their compressive strengths after 7-days and 28-days, using a universal testing machine. Their compressive strengths are given in Table 1.

3.2.2 Tensile tests on reinforcement steel bars and structural steel plates

The reinforcement steel bars with a nominal strength of 500 MPa and mild steel plate with a nominal strength of 250 MPa were used in the preparation of the specimens. The size of the tensile coupons considered for mild steel was 1250mm × 2500mm × 10mm and the length of the coupons for the reinforcement steel bars was 2500mm, both conforming to Indian Standard (IS 1608: 2005, also ACI 301-10). Five specimens were prepared for each case. The procedure of testing, prescribed in the Indian Standard (IS 1608-2005), was adopted for the testing of the tensile coupons using a fully automatic universal testing machine. Table 4 presents the average of the tensile coupon test results.

3.2.3 Material tests on cement, aggregates and concrete

The material properties of CFRP used in the tests are given in Table 5. An epoxy adhesive by the brand name “BONDINSUL 52 (A)” was used for bonding the CFRP with the RC beam-column joint, which had a viscosity ranging between 2500-3500 centipoise at room temperature as per the company’s test certificate. It had a hydrolysable chlorine content of 0.5%, with an epoxy equivalent weight of 225-250 eq/gm. A hardener by the company name “BONDINSUL 71 (B)” was used with resin to hardener ratio of 10:1. Table 6 shows the relevant material properties of the epoxy adhesive used.

3.3 Test set-up and loading procedure

The portal type shape of the specimen was selected for this study as it allows the specimen to be tested in the horizontal position, lying on frictionless supports on the ground which permits better monitoring of the specimens as shown in Figure 2. A hydraulic jack of 1000 kN capacity was fixed between the ends of the columns for the lateral load application in force-controlled mode, which will

Table 2 Material properties of aggregates used

	Fine aggregate		Coarse aggregate	
S.No.	Characteristics	Test values	Characteristics	Test values
1	Specific gravity	2.63	Category	Crushed
2	Bulk density loose (kg/l)	1.31	Maximum nominal size (mm)	14
3	Fine modulus	2.32	Specific gravity	2.61
4	Water absorption (%)	2.41	Water absorption (%)	1.87
5	Grading zone (based on percentage passing 600μm sieve) as per IS: 383	II	Fineness modulus	6.55

Table 3 Mix proportions of concrete (M20 grade)

Ingredient	Quantity (kg/m^3)
Cement (OPC)	695.72
Fine aggregate	1043.61
Coarse aggregate	2087.19
Water	292.25

Table 4 Material properties of reinforced steel bars and structural steel plates used

S.No.	Sectional size of the steel specimens (mm)	Yield strength (MPa)	Ultimate Strength (MPa)	Elongation (%)	Modulus of Elasticity (GPa)
1	Reinforced steel bar of 10 mm diameter	503.2	567.4	21.3	196.4
2	Reinforced steel bar of 8 mm diameter	514.8	576.3	25.4	197.2
3	Structural steel plate of 10 mm thickness	292.3	427.8	23.2	197.8

Table 5 Material properties of CFRP used

S.No.	Physical characteristics	Values
1	Tensile strength (MPa)	4000
2	Modulus of Elasticity (GPa)	220
3	Density (g/cm^3)	1.8
4	Weight of carbon fiber before stitching (gsm)	210
5	Weight of CFRP after stitching (gsm)	240
6	Thickness (mm)	0.118
7	Elongation (%)	1.7

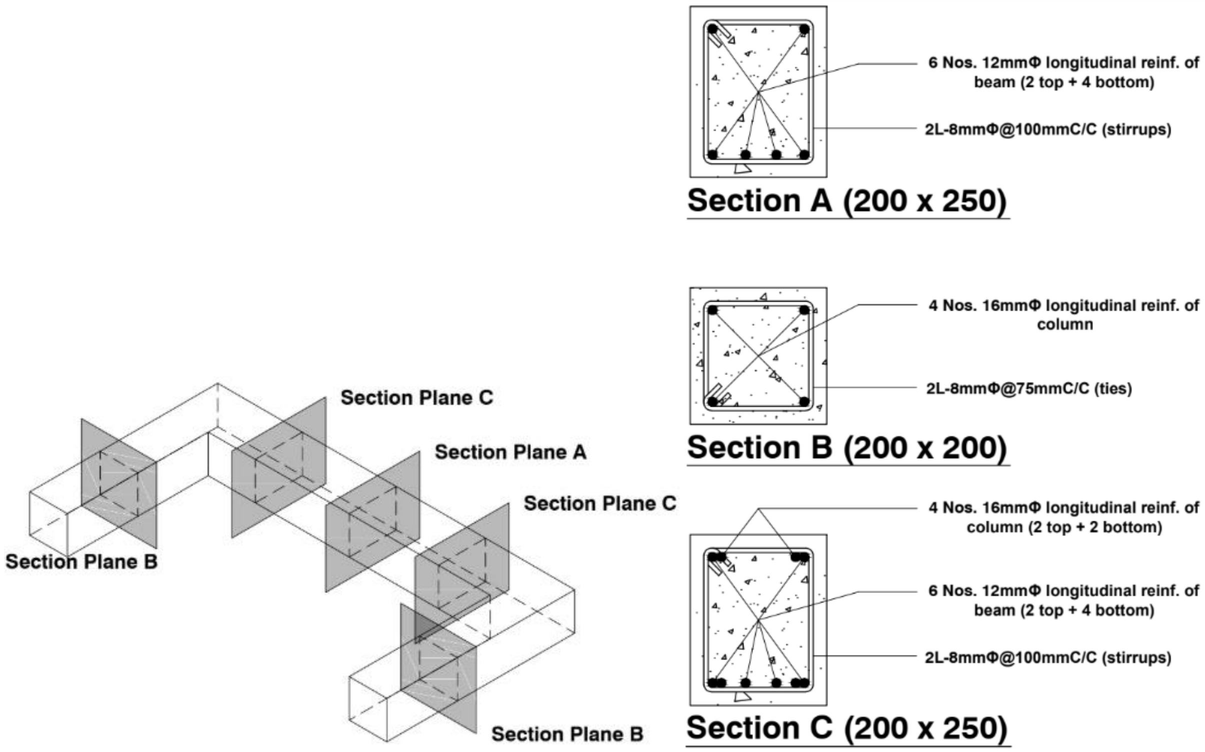
Table 6 Material properties of the epoxy adhesive used

S.No.	Aspect	Values
1	Volume solids (%)	90
2	Density of the mix	1.15 ± 0.05
3	Ratio of the resin: hardener mix (by weight)	100-10
4	Mixed viscosity (cps at 25°)	3000 ± 500
5	Pot life (in minutes)	45-60 minutes at 27°
6	Setting time	<3 h at 25°C
7	Full cure	7 days at 18°C
8	Compressive strength	>40 MPa at 1 day > 60 MPa at 7 days
9	Tensile strength	>17 MPa
10	Flexure strength	>35 MPa
11	Density	0.8–1.0 kg/m^2
12	Filament Diameter (μm)	7
13	Hydrolysable chlorine content (%)	0.5 (Maximum)
14	Epoxy equivalent weight	225-250 eq/gm
15	Physical State	Viscous liquid
16	Appearance	Transparent
17	Color	Pale yellow/colorless

subject the beam-column joints under opening moments, which will, in turn, replicate seismic forces on the beam-column joints. A sensitive proving ring of 100 kN capacity was employed for recording the load applied to the specimen. To record the deflections at the corners and at the ends of the columns, six dial gauges of least count 0.01 mm were employed, as shown in Figure 4 (a). A crack detection microscope of least count 0.01 mm was employed for measuring the cracks formed on specimen during the testing. During the progression of tests, loads, as well as the displacements, were noted at regular intervals of load increments (2kN), which was subsequently followed by the monitoring of initiation and propagation of cracks. Further, the loading at the first crack as well as failure modes of specimens were noted.

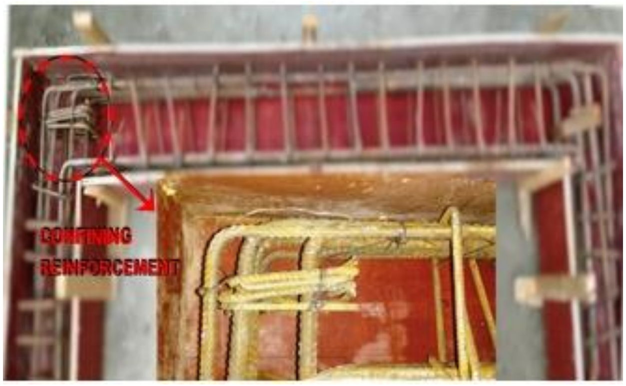
4. Results and discussion

The performance evaluation in terms of strength, stiffness, and ductility of the various strengthening schemes adopted, were compared. The detailed test results of all the specimens are given in Table 7. Figure 4(b) presents the load vs. displacement for BMS. The first visible crack originated from the inner face of the beam-column joint at a lateral load of 3.32 kN, which propagated diagonally towards the outer corner of the beam-column joint, and occurred simultaneously at both the joints. With further



(a) Cross-sectional reinforcement details

(b) Normal reinforcement in the specimens



(c) Confining reinforcement in DSS

Fig. 1 Reinforcement details of various specimens

Table 7 Test results

S. No.	Specimen	P_{cr} (kN)	P_y (kN)	P_u (kN)	Strength enhancement (%)			Δ_{cr}	Δ_y	Δ_u	Ductility ratio Δ_u / Δ_y	Energy absorbed (kN-mm)
					P_{cr}	P_y	P_u					
1	BMS	3.32	6.64	14.94	-	-	-	1.17	3.1	8.7	2.8	70
2	DDS	4.98	11.62	18.26	50	40	22	0.72	3.0	9.7	3.2	120.6
3	SFRS	3.32	8.3	19.92	0	20	33	0.24	4.8	20	4.2	259.3
4	CFRPS	-	11.62	23.24	-	40	55	-	3.2	19	5.9	313.9
5	SHS	6.64	11.62	36.52	100	40	144	1.05	3.1	16	5.2	326.9

increase in the lateral loading, the crack propagation advanced and more cracks started developing within the joint region, as shown in Figure 5. This behaviour continued until the joint failed by a combination of flexural and shear failure in the joint region leading to the collapse of the corner at a lateral loading of 14.94 kN. The maximum average displacement of 9.77 mm was recorded at the time of failure and the crack width of 1 mm was measured at failure load. Yield strength was noted as 6.64kN. The stiffness of the load vs displacement plot dropped beyond the load of magnitude 6.64kN.

Figure 4(c) presents the load vs. displacement for DDS. The first visible crack was initiated at a load of 4.98kN and progressed further upon the increment of lateral loading. This behaviour continued until the specimen failed at a load of 18.26 kN with a maximum displacement of 9.70 mm at

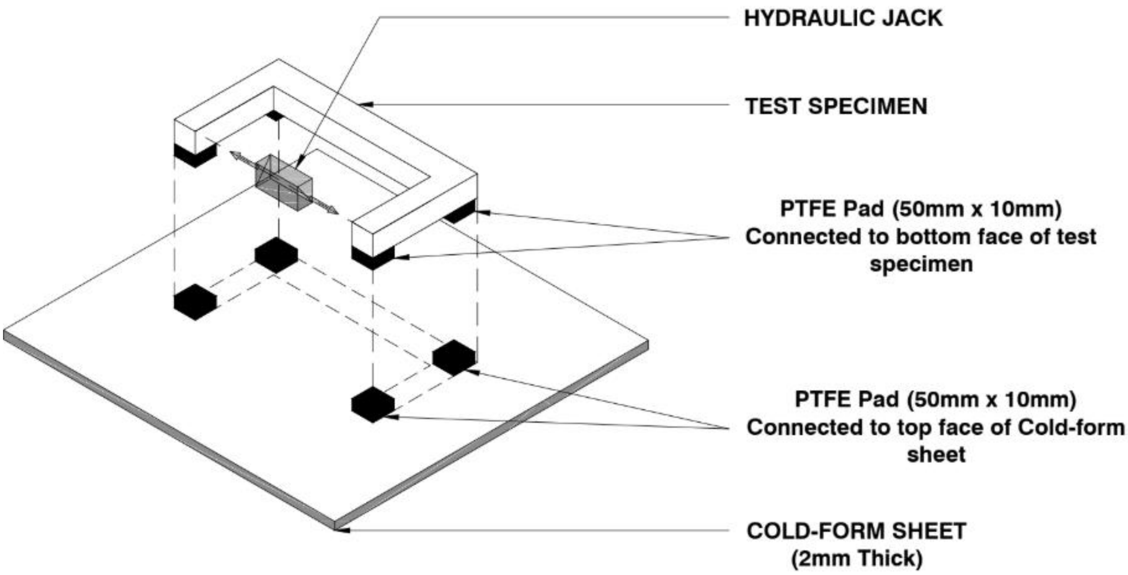


Fig. 2 Position of PTFE pads

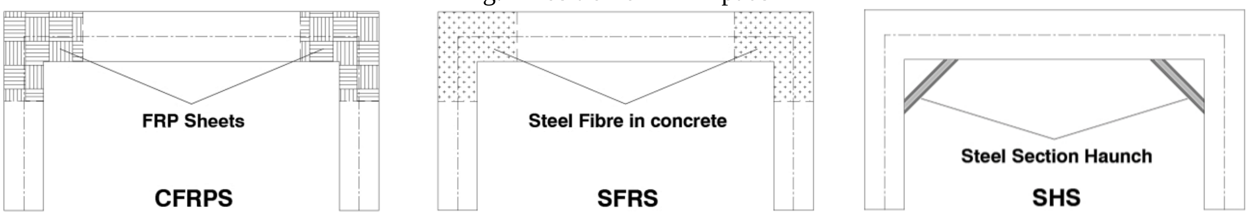


Fig. 3 Schematic of the strengthened specimens

the column ends. The crack width at failure measured 1.1mm. Yield strength was noted as 11.62 kN.

Figure 4(d) presents the load vs. displacement for SFRS. The first visible crack in SFRS was initiated at a load of 3.32 kN which widened upon further loading till the specimen failed at a load of 19.92 kN, with a maximum lateral displacement of 20.065 mm at the column ends. A crack width of 0.72 mm was recorded at failure and the crack pattern is shown in Figure 6. Yield strength of 8.30kN was noted.

Figure 4(e) presents the load vs. displacement for CFRPS. The specimen failed at a load of 23.24kN with a maximum lateral displacement of 19.35 mm at the column ends. A crack width of 0.92 mm was recorded at failure and the crack pattern is shown in Figure 7. Yield strength of 11.62kN was noted.

Figure 4(f) presents the load vs. displacement for SHS. On the application of lateral loading, the first visible crack was observed at a load of 6.64 kN which widened upon further increment in the lateral loading. This behaviour continued till the specimen failed at a load of 36.52 kN with a maximum lateral displacement of 15.975 mm at the column ends. A crack width of 0.75 mm was recorded at failure and the crack progression is shown in Figure 8. Yield strength of 11.62 kN was noted.

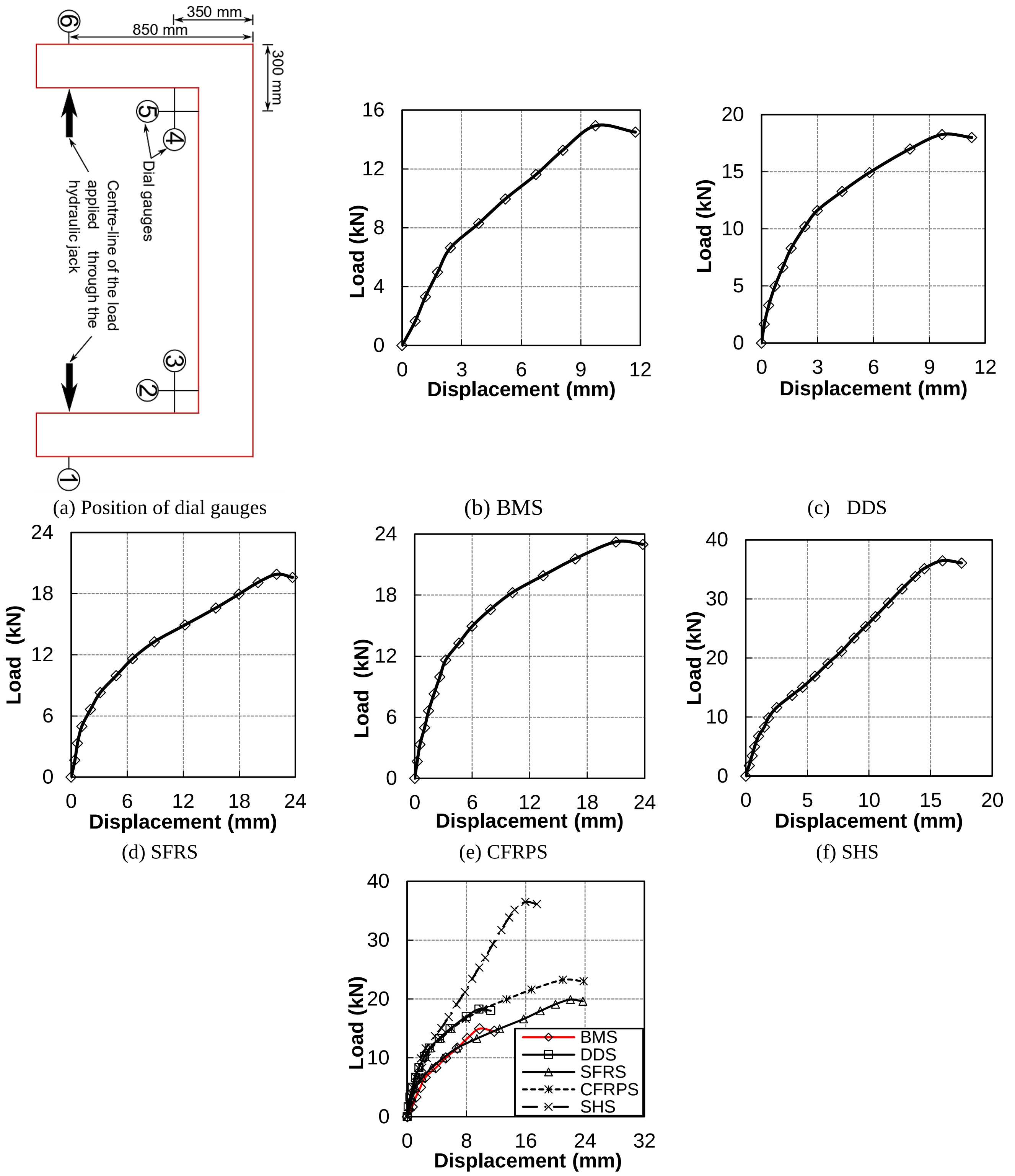
Figure 4(g) shows the comparison of the load vs. displacement curves of all the five specimens. It can be seen that the trend of the load vs. displacement response for CFRPS and DDS is the nearly same, with the former carrying higher loading. A similar behaviour was observed in BMS and SFRS, with the later carrying higher loading.

4.1 Effect on the ultimate capacity

With respect to strength behaviour, it was observed that all the strengthening measures improved the ultimate capacity of RC beam-column joints, as compared with BMS, and is shown in Figure 9. The decreasing order of the strength enhancement for the various measures with comparison to the BMS is given below

SHS > CFRPS > SFRS>DDS

This order is strictly valid for the current dimensions of the strengthening systems adopted. The strength enhancement in SHS, CFRPS, SFRS, and DDS was found to be 144.4%, 55.5%, 33.3% and 22.2% respectively. The entire cross-section of the inclined plate in the steel haunch was subjected to tensile forces under the lateral loading of the SHS specimen. Since, tensile forces led to strength failure, it offered high resistance to axial deformation, thus enhancing the load-carrying capacity of the specimen substantially. Further, the yield strength of the specimen improved. approximately by 75%. However, due consideration must be given in practice to anchorage the steel haunch in the beam-column joint, to prevent stability failure due to pulling out of the connecting bolts. The CFRP sheets wrapped around the beam-column joint comprised of high tensile performance carbon fibers which were subjected to tensile forces on the inner corner regions.



(a) Position of dial gauges (b) BMS (c) DDS

(d) SFRS (e) CFRPS (f) SHS

(g) Comparison

Fig. 4 Load *vs.* displacement response of the various specimens

Since the tensile strength of CFRP is high, it developed large resistance against deformation, which delayed the crack development at the joint and greatly improved the capacity of the joint. Also, an improvement of approximately 75% in the yield strength was observed. The confining reinforcement in the DDS specimen resulted in better confinement of the corner reinforcement, which prevented the disintegration of the corner joint and improved the joint's performance apart from improving the yield strength by approximately 75%. The incorporation of discontinuous steel fibers improved the overall strength of SFRS specimen due to better bonding of concrete from

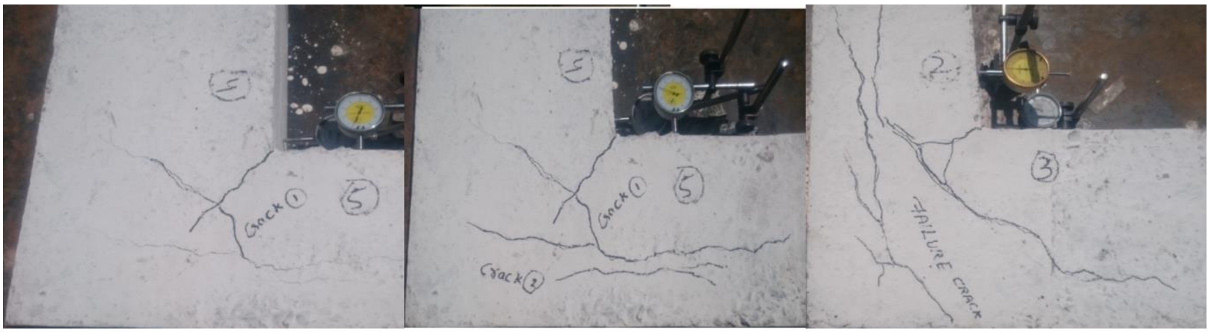


Fig. 5 Crack progression in BMS

Fig. 6 Crack pattern in SFRS

Fig. 7 Crack pattern in CFRPS

Fig. 8 Crack progression in SHS

within due to their unique bridging action. This was mainly due to its hooked ends and the irregular configuration which resulted in the performance enhancement of the beam-column joints. There was an improvement of around 33% in the yield strength.

4.2 Effect on energy absorption, displacement ductility ratio and initial stiffness

Since the force-controlled mode of loading was adopted, the lateral displacements for the various specimens were obtained up to their ultimate capacity only. Therefore, idealization of the observed load vs. displacement curves to equivalent tri-linear curves was carried out to compute the energy absorption. The energy absorbed by the specimens was determined by computing the area under the idealized tri-linear load vs. displacement curves. Figure 10 shows the comparison of the energy absorption characteristics of various specimens. Specimens SHS and CFRPS displayed good energy absorption characteristics and were nearly in the same range (above 310 kN-mm), with the former performing slightly better. The improvement in the energy absorption of SHS was attributed to its enhanced ultimate strength, while that of the CFRPS was due to its large observed lateral displacement. The energy absorbed by SFRS specimen was 259.3 kN-mm and was due to both its strength enhancement as well as its large lateral displacement. As both the strength as well as the lateral displacement exhibited by the DDS specimen was small, it had small energy absorption of 120.6 kN-mm.

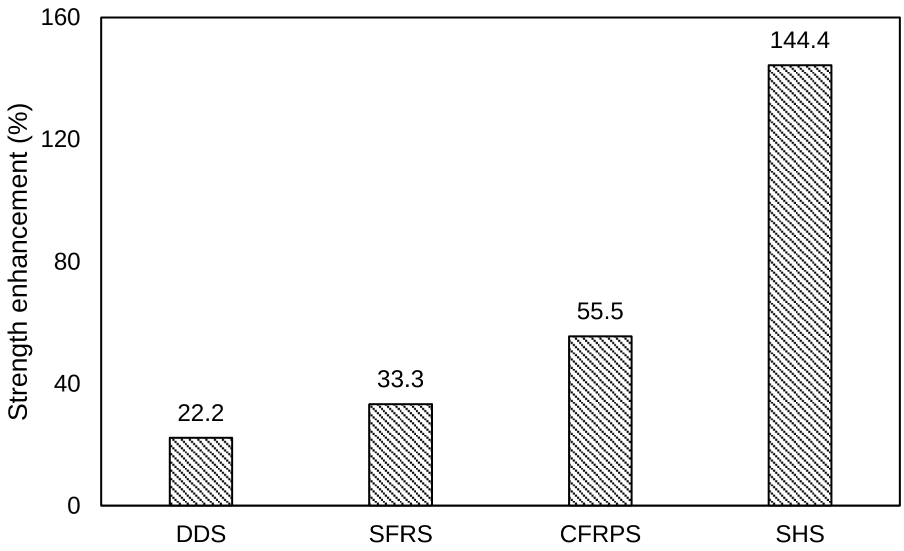


Fig. 9 Strength enhancement comparison

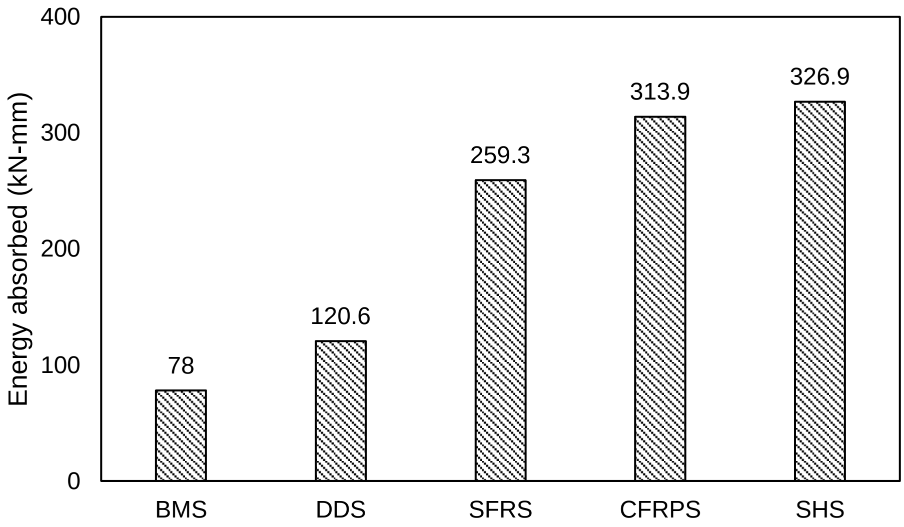


Fig. 10 Energy absorption comparison

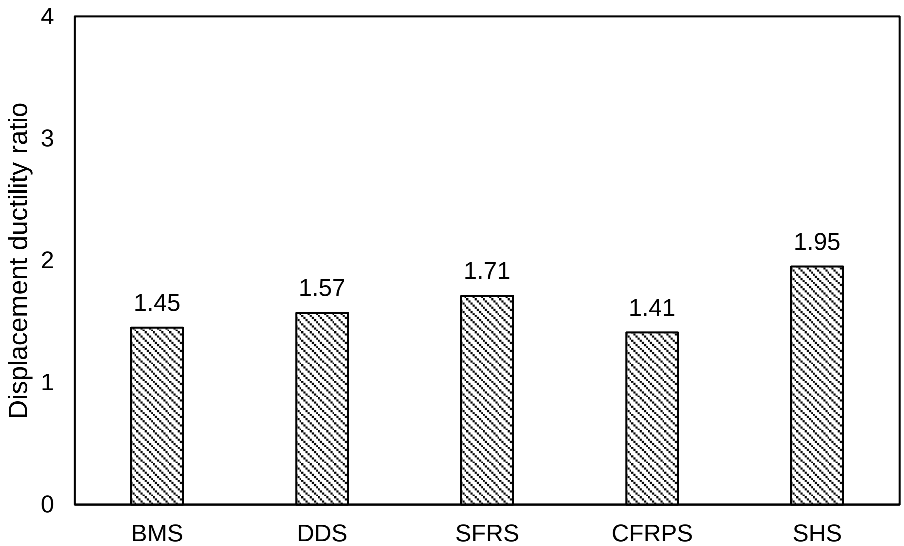


Fig. 11 Displacement ductility ratio comparison

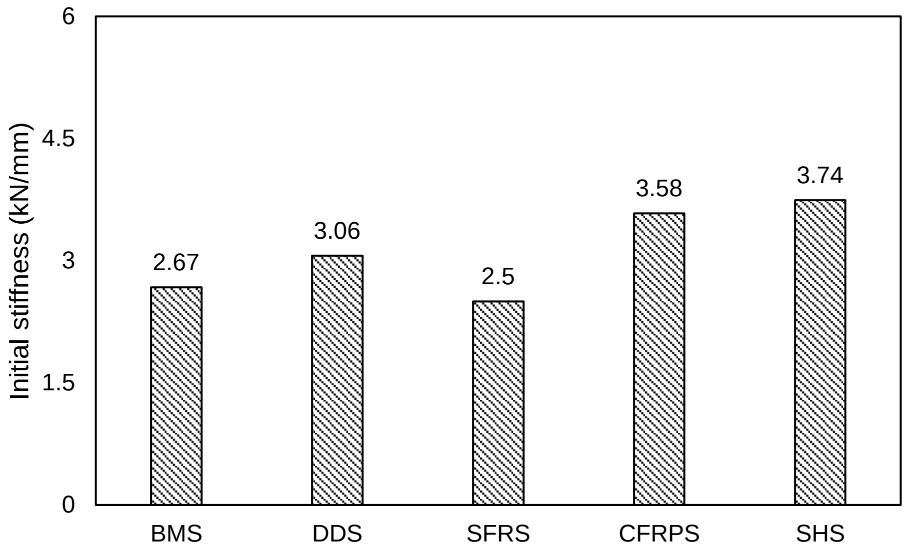


Fig. 12 Initial stiffness comparison

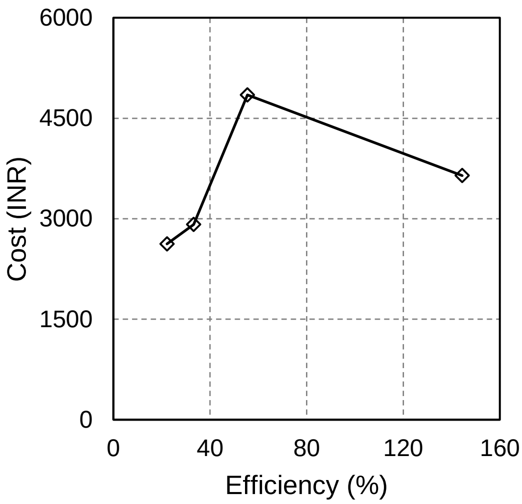


Fig. 13 Cost-benefit comparisons

The displacement ductility ratio was quantified as the ratio of displacement at the ultimate load to the displacement at the yield load. Figure 11 shows the comparison of the displacement ductility ratio for various specimens. SHS and CFRPS displayed good displacement ductility behaviour with the ratio being greater than 5.00. This was primarily due the steel haunch in SHS, which imparted good ductility to the beam-column joint and CFRP wrapping that improved the ductility characteristics of the joint. The SFRS also displayed good ductility behaviour with the displacement ductility ratio being 4.2 and was attributed to the ductility offered by the high-performance steel fibers. Further, the ductility of DDS was higher than that of BMS, as the special confinement reinforcement extended additional ductility. As ductility is an important and preferred seismic feature, it has to be given consideration while choosing strengthening schemes.

As the slope of load vs. displacement curves was nearly constant up to a lateral displacement value of approximately 3 mm, the initial stiffness was determined as the ratio of the load resisted by the specimens at that displacement to their corresponding lateral displacement. Figure 12 shows the comparison of the initial stiffness offered by various specimens. SHS and CFRPS displayed good initial stiffness behaviour with their ratio being 3.74 and 3.58 respectively.

4.2 Cost-benefit analysis

The primary objective of this study was also to compare the performance of different strengthening schemes with respect to their cost. The cost of construction plays an important role in the selection and implementation of strengthening scheme. Hence, a comparison of the cost of different strengthening schemes with their corresponding efficiencies was made. Figure 13 presents the plot of cost vs. efficiency of the various adopted schemes. It was observed that the cost of the different strengthening schemes was proportional to their efficiency, except for SHS. The overall structural performance of SHS was promising. However, there are certain limitations of each strengthening scheme. The steel haunch needs to be carefully adopted without cutting the main reinforcement of the beam-column joint, and hence needs skilled worker. Also, it may not be pleasing from aesthetic point of view. Steel fiber reinforcement affects the workability of concrete and needs careful mixing and additional compaction. CFRP improves the strength as well as the energy absorption but affects the ductility of the joint, which may be an important factor in severe earthquake zones.

5. Conclusions

Experimental investigations were carried out to study the performance of various strengthening schemes on exterior RC beam-column joints, viz., using steel fiber reinforcement, carbon fiber reinforcement polymer (CFRP), steel haunch, and confining joint reinforcement. These specimens were tested under horizontal loading that created opening moments at the joints and their behavior was discussed with emphasis on strength, displacement ductility, stiffness, and failure mechanism, and led to the following conclusions:

Providing steel haunch improves the strength, displacement, ductility ratio, energy absorption, and initial stiffness of beam-column joints substantially. Furthermore, it is cost-effective and can be adopted for increasing the already built and capacity deficit beam-column joints. However, it may not offer an aesthetically pleasing solution and needs to be installed carefully without cutting the main reinforcement of the beam-column joints.

- Wrapping CFRP plates around the beam-column joint sufficiently enhances the energy absorption, initial stiffness, as well as strength of the joint. However, these improvements are achieved with a slight reduction in the displacement ductility and result in a costlier option than other strengthening schemes. Moreover, it may reduce the aesthetic appearance as well.
- The addition of high-performance steel fibers to the concrete mix in the beam-column joint region offers good displacement ductility and energy absorption characteristics in addition to improving its strength. But, it may affect the workability of the concrete mix when added in higher quantities and cannot be adopted for strengthening already built capacity deficit beam-column joints.
- The provision of special confining reinforcement at the beam-column joint considerably improves its initial stiffness, but at the cost of energy absorption characteristics. Furthermore, it did not improve the displacement ductility much and cannot be adopted in the already built beam-column joints. In addition, installing the confining transverse reinforcement in the joint at the site may pose practical problems and may increase the congestion at the joint, which in turn may result in problems of concreting and consolidation of concrete. However, it is the most cost-effective strengthening measure.
- The conventional (non-ductile) code of practice for reinforced concrete (IS 456) predicted the strength of the beam-column joint un-conservatively (~10-15%) and needs to be revised for better and reliable strength prediction.

Recommendations and future scope

From the investigation carried out in this study, the following recommendations are proposed:

- From strength consideration, steel haunch can safely be adopted for strength enhancement up to 140%, CFRP for up to 50%, steel fibers for up to 30% and special confinement reinforcement for up to 20%.
- From energy absorption consideration, steel haunch and CFRP can safely be adopted for energy absorption enhancement of nearly 300%, steel fibers for up to 230% and special confinement reinforcement for up to 70%.
- From displacement ductility consideration, steel haunch can safely be adopted for a displacement ductility enhancement of up to 35%, steel fibers for around20% and special confinement reinforcement for up to 8%.
- From initial stiffness consideration, steel haunch can safely be adopted for an initial stiffness enhancement of up to 40%, CFRP for around 35%, and special confinement reinforcement for around 15%.
- From cost consideration, special confinement reinforcement should be preferred over steel haunch, CFRP and steel fibers. From an aesthetic point of view, steel fibers and special confinement reinforcement should be preferred. This study mainly focused on the behaviour of RC beam-column joints under lateral loads subjected to opening moments. The behaviour of SFRS by adopting different steel fibers needs to be investigated. The study to optimize the thickness and configuration of CFRP sheets for improved structural performance also needs to be investigated. The effect of the thickness of steel haunch and its connections needs to be studied in detail for optimized performance. The conclusions drawn from this study are limited to the dimensions of the strengthening systems used.

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