
Experimental Investigation on Behavior of Reinforced Concrete Beam Column Joints Retrofitted with GFRP-AFRP Hybrid Wrapping

Robert Ravi.S¹, Prince Arulraj.G²

1- Assistant Professor, School of Civil Engineering, Karunya University, Coimbatore.

2- Professor and Dean, Department of Civil Engg., S.N.S College of Tech., Coimbatore.

srobertravi@yahoo.com

ABSTRACT

Upgradation to higher seismic zones of several cities and towns in the country has necessitated in evolving new retrofitting strategies. Recent earthquakes have demonstrated that most of the reinforced concrete structures were severely damaged during earthquakes and they need major repair works. One of the techniques of strengthening the reinforced concrete structural members is through external confinement by high strength fiber composites which can significantly enhance the strength and ductility which will result in large energy absorption capacity of structural members. Fiber materials are used to strengthen a variety of reinforced concrete elements to enhance the flexural, shear, and axial load carrying capacity of elements. Beam-column joints, being the lateral and vertical load resisting members in reinforced concrete structures are particularly vulnerable to failures during earthquakes and hence their retrofit is often the key to successful seismic retrofit strategy. The existing reinforced concrete beam-column joints which were designed as per code IS 456:2000 must be strengthened since they do not meet the requirements given in the ductility code IS 13920:1993. In this paper an attempt has been made to study the behavior of reinforced concrete beam-column joints retrofitted with glass-carbon hybrid fiber sheets. Three exterior reinforced concrete beam-column joint specimens (control) were cast and tested to failure. Two specimens had reinforcement details as per code IS 456:2000. The other specimen had reinforcement details as per code IS 13920:1993. An axial load was applied on the column. Push and pull load was applied at the free end of the cantilever beam till failure. The failed two beam-column joint specimens designed as per code IS 456:2000 were retrofitted with GFRP-AFRP/AFRP-GFRP hybrid fiber sheets wrapping to strengthen the specimens. The performance of the retrofitted beam-column joints was compared with the control beam-column joint specimens and the results are presented.

Keywords: Beam-column joint; Retrofitting; GFRP sheet; AFRP sheet.

1. Introduction

Earthquakes have exposed the vulnerability of existing reinforced concrete beam-column joints to seismic loading. Concrete jacketing and steel jacketing were the two common methods adopted for strengthening the deficient reinforced concrete beam-column joints. This type of retrofitting results in substantial increase in the cross sectional area and the self-weight of the structure, more over the retrofitted joints have poor resistance for

weather attacks and are labour intensive. A new technique has emerged recently which uses fiber reinforced polymer (FRP) sheets to strengthen the beam-column joints which have a number of favorable characteristics such as ease to install, immunity to corrosion and high strength. The simplest way to strengthen the joints is to wrap fiber sheets in the joint region in two orthogonal directions. An attempt has been made to carry out experimental investigation on behavior of retrofitted reinforced concrete beam-column joint specimens retrofitted with GFRP-AFRP/AFRP-GFRP hybrid wrapping sheets.

2. Experimental Investigation

The experimental program consisted of testing three reinforced concrete beam-column joint specimens named as C1, C2 and C3 . Specimens C1 and C2 had reinforcement details as per code IS 456:2000. Specimen C3 had reinforcement details as per code IS 13920:1993. The columns had a cross section of 200 mm x 200 mm with an overall length of 1500 mm and the beams had a cross section of 200 mm x 200 mm with a cantilevered portion of length 600 mm. The column portion was reinforced with 4 numbers of 12mm diameter rods and the beam portion was reinforced with 2 numbers of 16 mm diameter rods at tension and compression zones. The lateral ties in the columns of the specimens C1 and C2 were 6 mm diameter bars with the spacing of 180 mm c/c as per code IS 456:2000, cl 26.5.3.2(c). Beam had vertical stirrups of 6 mm diameter bar at 120 mm c/c as per code IS 456:2000, cl.26.5.1.6. The development length of the tension and compression rods in beam were also provided as per clause 26.2.1 of code IS 456:2000. The lateral ties in the columns of the specimens C3 consisted of 8 mm diameter bar at 75 mm c/c for the central distance of 1100 mm as per code IS 13920:1993, cl 7.4.6 and 6mm diameter bars at 100 mm c/c for the remaining length of the column. Beams had vertical stirrups of 6 mm diameter bar at 40 mm c/c. up to a distance of 340 mm from the face of the column as per code IS 13920:1993, cl 6.3.5 and 6 mm diameter bar at 80 mm c/c for remaining length of the beam. The development length of the beam rods were also provided as per code IS 13920:1993, cl 6.2.5. The concrete mix was designed for a target strength of 20 MPa at the age of 28 days. The load carrying capacity of the column was estimated to be 440 kN. The details of the test specimens are given in Figure.1 (a) & Figure.1 (b). The load reversal (push and pull) tests were conducted on the control and retrofitted reinforced concrete beam-column joint specimens. Generally, when the axial load on the column exceeds 50 to 60% of its capacity, the effect of axial load will be more predominant on the joint. In the case of the seismic forces, the effect of lateral load will be more predominant. Hence in order to truly reflect the behavior of the joint under seismic load conditions, it was decided to restrict the axial loads of column to a maximum of 200 kN which is less than 50 % of load carrying capacity of the column. A point load was applied at the free end of the cantilever beam portion till the failure of the specimen. The loading was continued till the joint failed by crushing of concrete in the case of control specimens and by the rupture of wrap in the case of retrofitted specimens. The failed specimens C1 and C2 were retrofitted with hybrid wrapping sheets. The specimen C1 had GFRP sheet as inner layer and AFRP sheet as outer layer. The specimen C2 had AFRP sheet as inner layer and GFRP sheet as outer layer. The specimens were redesignated as retrofitted specimens R1 & R2 and again tested to failure.

The performance of the retrofitted beam-column joint specimens was compared with that of the control beam-column joint specimens. The physical properties of GFRP and AFRP sheets used for the retrofitting of the specimens are given in Table.1

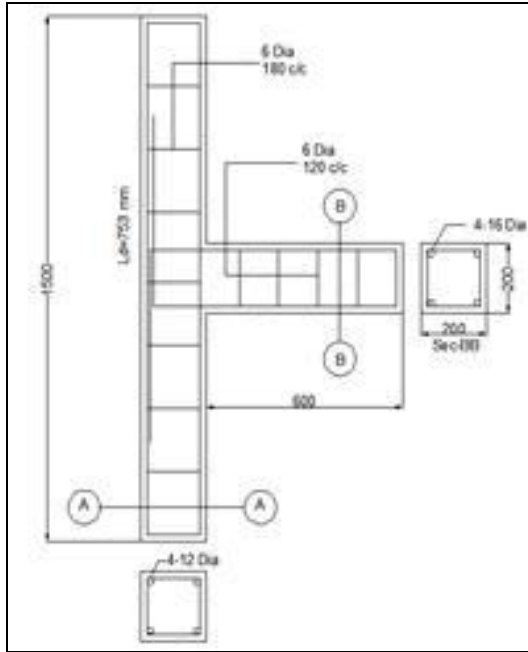


Figure 1.a: Reinforcement Details for Specimen as per code IS 456:2000

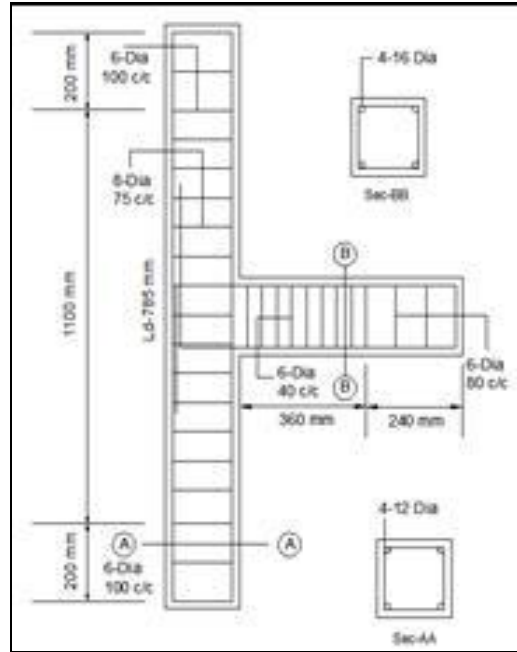


Figure 1.b: Reinforcement Details for Specimen as per code IS 13920:1993

Table.1 Typical Properties of GFRP & AFRP sheet

Name	MBrace G Sheet EU 750	MBrace A sheet
Colour	White	Yellow
Technical data	E-Glass, 750 gsm	450 gsm
Modulus of elasticity	73 kN/mm ²	120 kN/mm ²
Tensile strength	3400 N/mm ²	2900 N/mm ²
Total weight of sheet	750 g/m ²	450 g/m ²
Density	2.6 g/cm ³	1.45 g/cm ³
ε Ultimate %	4.5	2.5
Thickness	0.285 mm	0.29 mm
Safety factor for static design	1.5 (recommended)	1.2 (recommended)

3. Preparation of Specimens

The reinforced concrete beam-column joint specimens were cast using fabricated steel moulds. Reinforcement was prepared and placed inside the mould. The grade of concrete used was M20. IS method of mix proportioning was adopted to arrive design mix as

1:1.57:2.97 with the w/c ratio of 0.49. Concrete was mixed in a tilting type mixer machine and was properly placed and compacted. The sides of the mould were removed 24 hours after casting and the test specimens were cured in water for 28 days. The failed specimens C1&C2 were retrofitted and redesignated as specimens R1&R2. The concrete near the area of failure was removed completely. After applying cement paste in this area, the portion was filled and compacted with the same grade of concrete. The specimens were cured for 28 days. Before wrapping fiber sheet, the faces of the specimens were ground mechanically to remove the laitance. All the voids were filled with putty. Then a two component primer system was applied on the concrete surface and allowed to cure for 24 hours. A two component epoxy coating was then applied on the primer coated surface and inner layer of fiber sheet was immediately wrapped over the entire surface of the reinforced concrete beam-column joint. A hand roller was then applied gently over the wrap so that good adhesion was achieved between the concrete surface and allowed to cure for seven days. Another coat of the two component epoxy was applied over the inner layer of fiber sheet. Then the outer layer of wrap was applied following the same procedure and allowed to cure for a further period of seven days. Both the wrapped layers were orthogonal to each other. Figure .2 a) Figure. 2 b) & Figure.2 c) show the typical control and retrofitted specimens.



Figure.2a: Typical view of Control Specimen

Figure.2b: Typical view of GFRP -AFRP Specimen

Figure.2c: Typical view of AFRP-GFRP specimen

4. Description of Test Programme

The specimens C1,C2 & C3 were tested in a loading frame in the horizontal plane. Both the ends of the column were hinged using roller plates. The axial load of 200 kN was applied at one end the column using a hydraulic jack of 500 kN capacity and the load was measured using an electrical load cell. The other end of the column was supported by the steel bulkhead attached to the loading frame. A transverse push and pull load was applied at the free end of the beam through a push and pull hydraulic jack of capacity 400 kN to develop a bending moment at the joint. The load on the beam was also measured using

load cell. The deflection at the free end of the beam was recorded at regular load intervals up to a control deflection of 50 mm. The retrofitted specimens R1 & R2 were also tested following the same procedure. Figure 3.a), Figure 3.b) & Figure 3.c) show the typical view of the failed control and retrofitted specimens.



Figure 3.a: Typical view of failed Control Specimen **Figure 3.b:** Typical view of failed(GFRP-AFRP)Specimen **Figure 3.c:** Typical view of failed(AFRP-GFRP)Specimen

5. Results and discussion

In the case of the specimen C1, during pushing, first crack was formed in the beam portion approximately at a distance of 40 mm from face of the column at a load of 15 kN. At a load of 16 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 17 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 18.5 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 20 kN. During pulling, first crack was formed in the beam portion at a load of 13 kN. At a load of 14 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 15 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 16 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 18 kN.

In the case of the specimen C2, during pushing, first crack was formed in the beam portion approximately at a distance of 45 mm from face of the column at a load of 15.5 kN. At a load of 16.5 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 17.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 19 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 20.5 kN. During pulling, first crack was formed in the beam portion at a load of 13.5 kN. At a load of 14.5 kN, another crack was formed

in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 15.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 16.5 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 18.5 kN.

In the case of the specimen C3, during pushing, first crack was formed in the beam portion approximately at a distance of 50 mm from face of the column at a load of 17 kN. At a load of 18 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 19 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 20.5 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 22 kN. During pulling, first crack was formed in the beam portion at a load of 15 kN. At a load of 16 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 17 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 18 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 20 kN.

In the case of the specimen R1, during pushing, first crack was formed in the beam portion approximately at a distance of 50 mm from face of the column at a load of 18.5 kN. At a load of 19.5 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 20.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 22 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 23.3 kN. During pulling, first crack was formed in the beam portion at a load of 16.5 kN. At a load of 17.5 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 18.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 19.5 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 21.3 kN.

In the case of the specimen R2, during pushing, first crack was formed in the beam portion approximately at a distance of 40 mm from face of the column at a load of 19.5 kN. At a load of 20.5 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 21.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 23 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 24.3 kN. During pulling, first crack was formed in the beam portion at a load of 17.5 kN. At a load of 18.5 kN, another crack was formed in the beam-column joint of the test specimen. The cracks in the beam started to widen at a load of 19.5 kN. Spalling of concrete occurred in the tension zone of the beam at a load of 20.5 kN. The application of the load was stopped when the deflection at the free end of the beam reached 50 mm. The load corresponding to this deflection was 22.3 kN. The load deflection curves of control & GFRP-AFRP/AFRP-GFRP wrapped reinforced concrete beam-column joint specimens are shown in Figure.4 The load carrying capacity

and energy absorption capacity of the control and retrofitted beam-column joint specimens are given in Table.2

Table.2 Load Carrying Capacity & Energy Absorption Capacity of Specimens

Specimen ID	% increase in Load Carrying Capacity		% increase Energy absorption capacity	
	Push	Pull	Push	Pull
C1	--	--	--	--
C2	--	--	--	--
C3	10.0	11.0	10.0	13.8
R1	16.5	18.3	22.7	26.6
R2	21.5	23.8	28.9	33.2

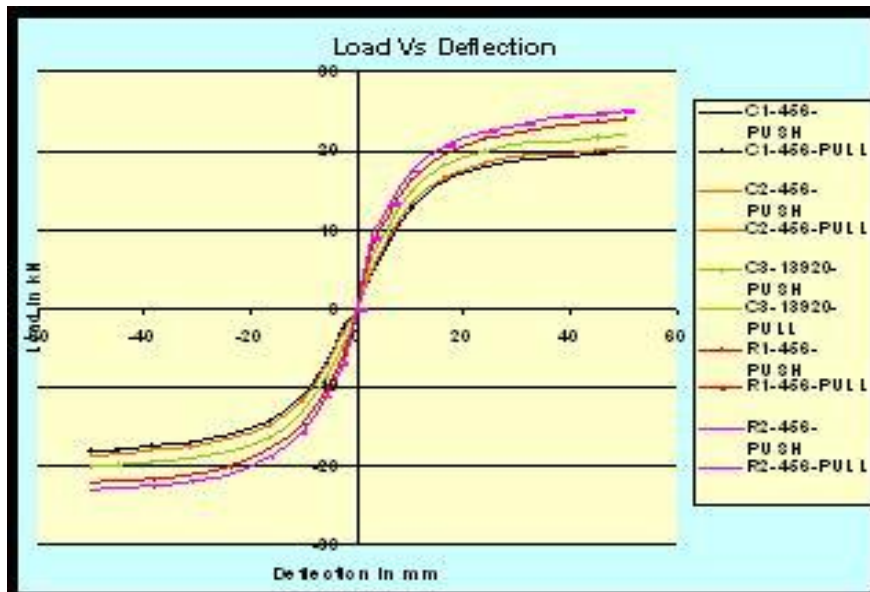


Figure.4 Load Deflection Curve for Control and Retrofitted Specimens

It is seen from the Table. 2 that the load carrying capacity of the beam-column specimen retrofitted with GFRP-AFRP/AFRP-GFRP hybrid sheet increased about 21.5 % during pushing and 23.8 % during pulling. The area of the load-deflection curve up to the limiting deflection increased by 28.9 % for the beam-column specimen retrofitted with GFRP-AFRP/AFRP-GFRP hybrid sheet during pushing and 33.2 % during pulling. The load deformation characteristics also improved to a larger extent in the case of wrapped specimens over the control specimens. This resulted in a substantial increase in the energy absorption characteristics of the specimens that were wrapped with GFRP-AFRP/AFRP-GFRP sheet.

5. Conclusions

Based on the experimental investigations carried out on the control and retrofitted beam-column joint specimens using GFRP-AFRP/AFRP-GFRP wrapping, the following conclusions were drawn:

- The load carrying capacity of the reinforced concrete beam-column joint specimens designed and detailed as per code IS 13920:1993 was found to be 10 % - 11 % more than the specimens detailed as per code IS 456:2000
- The energy absorption capacity of the reinforced concrete beam-column joint specimens designed and detailed as per code IS 13920:1993 was found to be 10 % - 13.8 % more than the specimens detailed as per code IS 456:2000
- The load carrying capacity of the reinforced concrete beam-column joint specimen retrofitted with GFRP-AFRP hybrid sheet was found to be 18.3 % more than the control specimens.
- The load carrying capacity of the reinforced concrete beam-column joint specimen retrofitted with AFRP-GFRP hybrid sheet was found to be 23.8 % more than the control specimens.
- The energy absorption capacity of the reinforced concrete beam-column joint specimen retrofitted with GFRP-AFRP hybrid sheet was found to be 26.6 % more than the control specimens.
- The energy absorption capacity of the reinforced concrete beam-column joint specimen retrofitted with AFRP-GFRP sheet was found to be 33.2 % more than the control specimens.
- The failure was in the column portion of the joint for the control specimen which is to be avoided. In the case of the wrapped specimens, the failure was noticed in the beam portion only and the column was intact and this is the most preferred type of failure under seismic loads which will prevent progressive collapse of the structure.

6. References

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