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SEISMIC STRENGTHENING OF EXTERIOR RC BEAM-COLUMN CONNECTIONS USING CONCRETE COVERS AND CFRP

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ABSTRACT

This study investigates the effectiveness of a new technique for seismic strengthening of substandard reinforcing details of exterior beam-column connections (T connections) using segmental circular concrete covers together with carbon fibre reinforced polymer (CFRP). Four reinforced concrete T connections without transverse reinforcement at the joints were cast and tested under cyclic loading. The first connection was tested as the control specimen. The remaining three connections were glued with concrete covers around the column at the joint area to modify them from square to circular sections and then they were wrapped with different ratios of CFRP. The test results showed that the variation of the CFRP ratios led to different failure modes of the examined connections. Despite the differences in the failure modes, the shear performance of the retrofitted specimens improved significantly. The modification of the column from square to circular sections not only helped in reducing the debonding possibility of the CFRP but also improving the confinement effect of the CFRP.

KEYWORDS

FRP, rehabilitation, reinforced concrete, joints, beam-columns, cyclic tests, shear failures.

INTRODUCTION

Beam–column connections in general and T connections in particular are one of the most critical regions of reinforced concrete (RC) frames as their failure under seismic loading often leads to partial or total collapse of the whole structures. Many RC frames, especially those designed before the introduction of modern seismic codes, were originally constructed with inadequate reinforcement details at the joint areas that provide strength and ductility. Therefore, it is essential to improve strength and ductility of such RC T connections.

In recent years, some techniques have been proposed to strengthen beam-column T connections. They include epoxy repair, removal and replacement, concrete jacketing, concrete masonry jacketing, steel jacketing, using steel straps and externally bonded FRP (Tran *et al.* 2013, Hadi and Tran 2014). Among these methods, externally bonded FRP has been recognized as the ideal technique for strengthening beam-column joints as it can eliminate some disadvantages of the remaining methods (complicated, expensive construction and corrosion problems). Many experimental studies have demonstrated that the FRP can be effectively used to strengthen beam-column connections (Gergely *et al.* 2000, Ghobarah and Said 2001, Karayannis *et al.* 2008, Sezen 2012).

A new method of retrofitting beam-column T connections has been proposed by Hadi and Tran (2014). In this method, the column of the connection was firstly circularized by bonding with concrete covers and then it was wrapped with CFRP. This method could improve the FRP confinement effect and reduce the possibility of the FRP from debonding and/or bulging of FRP from the concrete surface at the joint core so that it improves the effectiveness of the FRP. However, as only two specimens (one strengthened and one repaired) and no control specimen were tested, the effectiveness of this method was not evaluated thoroughly. Therefore, an extensive experimental investigation is conducted in this study to better understand the behaviour of RC T connections strengthened with this method. In the experiment, a total of four identical RC T connections were tested, one is the control specimen while the three remaining connections were strengthened with the above technique but with different CFRP ratios applied on the joint, thus the CFRP ratio is the test variable. The main results of this experimental investigation which show the effectiveness of the proposed strengthening technique are introduced in this paper.

DESCRIPTION OF THE SPECIMENS

Four identical RC T connections were cast and tested. The first connection, referred to as Specimen T0, was used as the control specimen, the remaining connections, referred to as Specimens TS, TS1 and TS2 were strengthened. The connections were designed with no joint transverse reinforcement details at the joints area. The dimensions and reinforcement details of the specimens are shown in Figure 1. The column was 2800 mm high with a cross sectional dimensions of 200x200 mm. The beam’s length was 1400 mm from the face of the column to the free end with a cross section of 200x300mm. Six N16 (16 mm deformed bars with 500 MPa nominal tensile strength) bars were placed as the longitudinal column reinforcement while the top and bottom longitudinal reinforcement of the beam were four N12 bars each. R10 (10 mm plain bars with 250 MPa nominal tensile strength) were used for the ties and the stirrups of the column and the beam.

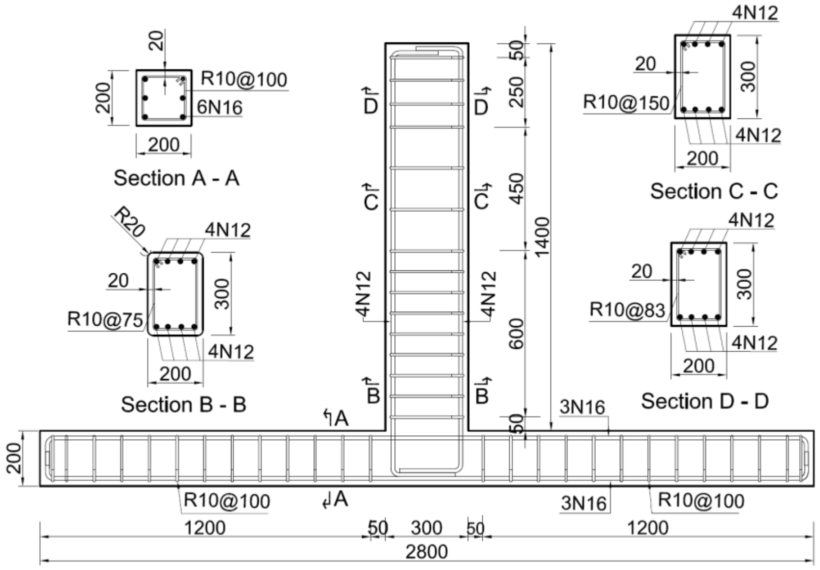
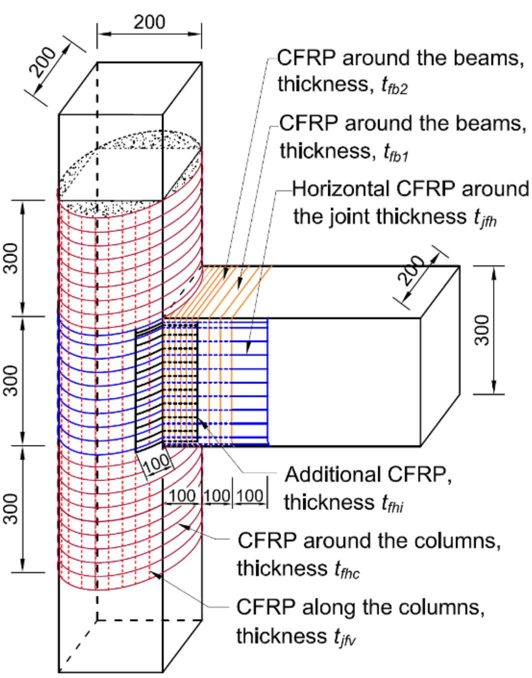


Figure 1. Reinforcement details of Specimens T0, TS, TS1 and TS2 (all dimensions in mm)

All of the specimens were cast horizontally rather than vertically as in a real building construction. The compressive strengths of the concrete of Specimens T0, TS, TS1 and TS2 at the time of testing each specimen were 41 MPa, 50 MPa, 41 MPa and 44 MPa, respectively. The average yield stresses of the N16, N12, R10 were 550, 551 and 322 MPa, respectively. Unidirectional CFRP, which has an elastic modulus of E_f =235 GPa and ultimate strain ε_{fu} =1.79%, was used to strengthen Specimens TS, TS1 and TS2.

The strengthening procedure of Specimens TS, TS1 and TS2 included two stages. In the first stage, the columns of the connections were bonded with the segmental circular concrete covers, which were cast using the same concrete as the tested specimens, to modify them from square to circular sections. Before they were bonded, the surfaces of the specimen around the beam-column joint area and of the segmental circular concrete covers were ground using an electric grinder and after that they were cleaned by using air blasting to ensure smooth and clean contact surfaces. In the second stage, the modified connections were externally bonded with CFRP sheets. The strengthening configuration and strengthening process of Specimen TS1 and TS2 were identical to that of Specimen TS but with the supplement of the additional CFRP sheets at the beam-column interfaces and with different thicknesses of the Carbon fibre. A summary of the CFRP application on Specimens TS1, TS2 and TS is shown in Figure 2. The variable is the thickness of the CFRPs (t_{fjh}, t_{fjv}, t_{fhc}, t_{fb1} and t_{fb2} in Figure 2). For easy identification, images of the specimen after completion of the first and the second stages are also presented in this figure. Details of the strengthening process can be seen in Hadi and Tran (2014).



The connection after circularisation

The connection after strengthening

Specimen	Fibre thickness (mm)					
	t_{fjh}	t_{fjv}	t_{fhi}	t_{fhc}	t_{fb1}	t_{fb2}
TS	0.33	0.33	0	0.33	0.33	0.33
TS1	0.11	0.11	0.11	0.11	0.11	0.66
TS2	0.66	0.66	0.66	0.66	0.66	0.66

Schematic illustration of the strengthening of Specimens TS, TS1 and TS2

Thicknesses of the CFRPs wrapped on Specimens TS, TS1 and TS2

Figure 2. Strengthening of Specimens TS, TS1 and TS2 (all dimensions in mm)

TEST SET UP

A strong steel frame was used to test the specimens. The connections were tested in the column vertical position. Two hinge supports placed at a distance of 2200 mm were provided to resist the horizontal forces near the top and bottom ends of the column. No compressive load was applied on the columns. A 600 kN hydraulic actuator was used to apply a vertical frequency cyclic loading onto the beam by slowly displacing the beam's free end (1100 mm from the beam-column interface) to create bending moment within the RC connection. The cyclic load was applied slowly with a deflection rate of 5 mm per minute and the beam's free end displacement was controlled. The amplitudes of the peaks in the displacement loading history were increased from 10 mm to 90 mm with an increment of 10 mm for each step. In order to measure the rotation of the beam and the columns, three inclinometers and three LVDTs were used. One inclinometer was placed on the beam at the beam-column interface; the others were placed on the top and bottom columns at the column-beam interfaces. In order to examine the behaviour of the CFRP during the tests, a total of 14 strain gauges were installed on each of the strengthened specimen. The locations of the strain gauges are shown in Figure 3 where the strain gauges are labeled from 11 to 24.

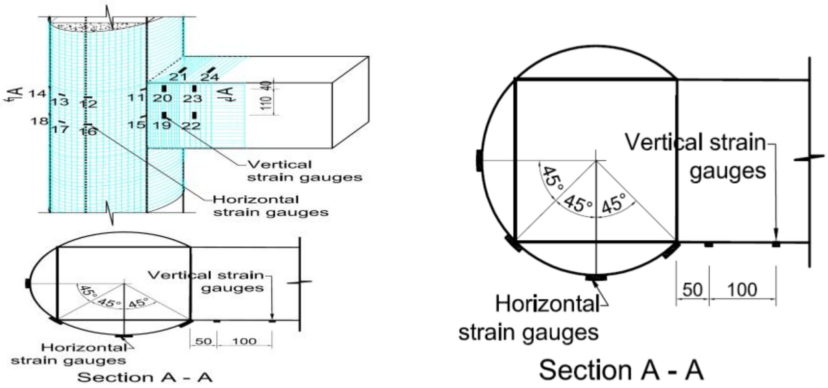


Figure 3. Position of strain gauges (all dimensions in mm)

EXPERIMENTAL RESULS AND DISCUSION

Behaviour of the Specimens

The measured shear force versus beam end deflection and the cracking patterns of the Specimen T0, TS, TS1 and TS2 at the end of testing are shown in Figures 4 and 5, respectively. Specimen T0 exhibited, as expected, premature shear failure during the early stages of cyclic loading. Hair cracks at the joint core occurred at the first loading cycle when the load was higher than 25 kN. A maximum load of 35.8 kN was reached at the peak of the 2nd loading cycle when diagonal cracks at the centre of the joint significantly developed. Specimen TS firstly failed by formation of a beam flexural hinge at the column face due to the rupture of the horizontal CFRP around the joint at the beam-column interface in the first six loading cycles. After breaking of the horizontal CFRP around the joint, the contribution of CFRP on joint shear strength reduced significantly. This reduction led to shear failure of the joint in the last three loading cycles. A maximum load of 77.2 kN was reached at the peak of the fourth loading cycle, an increase of 116% as compared with the control specimen. Specimen TS1 exhibited joint shear failure mode after the first four loading cycles due to the rupture of CFRP. The load increased rapidly in the first four loading cycles then reduced slowly in the fifth loading cycle and declined rapidly in the last four loading cycles together with the breaking of CFRP. A maximum load of 65.8 kN was recorded at the peak of the 4th loading cycle. Compared to the control specimen, the shear resistance of the joint improved about 84%. This failure mode of Specimen TS1 indicated that the amount of CFRP was inadequate. The applied CFRP can significantly improve the shear strength of the joint but this improvement could not shift the failure from joint shear to beam hinge failure. Specimen TS2 exhibited beam flexural failure mode with the formation of a beam flexural hinge at a distance of approximately 250 mm from the beam-column interface. Flexural cracks occurred on the beam in the third loading cycle. The development of these cracks led to the formation of a flexural hinge on the beam on the fourth loading cycle. A maximum load of 86 kN was reached at the peak of the fourth loading cycle, an increase of 140% compared to the control one. Favourably, the load continued to maintain at a high level until the ninth loading cycle. Debonding and breaking of CFRP at the joint region was not observed during the test. This means that the amount of CFRP applied on this specimen was adequate and the strengthening technique could shift the failure of the connection from joint shear to beam failure.

Strain of External CFRP

The effectiveness of the proposed technique depends on the capacity of the CFRP to confine the concrete at the joint core. To evaluate the CFRP confinement effect, the strains of the CFRP around the joint were measured by strain gauges numbered from 11 to 18 (Figure 3). In general, the experimental results showed that the CFRPs around the joints were under tension during the test and thus the concrete at the joint was affected by a relatively high confined stress. The CFRPs near the back of the column had lower strains than at the centre and at the beam-column interface, the distributions of the CFRP confined stress in the tested specimens were quite uniform when compared to that when CFRP was applied around the square or rectangular sections, where confinement effect only generated at the sections' corners. For Specimen TS, where no additional CFRP was installed, the strains of CFRP at the beam-column interface (Strain Gauges 11 and 15) were about 10 to 20% higher than at the centre of the joint (Strain Gauges 12 and 16). Whereas, in Specimen TS1 and TS2 the strains recorded from Strain Gauges 12 and 16 were higher than that of Strain Gauges 11 and 15. These are the main reasons behind the breakage of the CFRP at the beam-column interface that occurred just in Specimen TS but not in Specimens TS1 and TS2. This indicates that the application of the additional CFRP could prevent effectively the break of the horizontal CFRP at the beam-column interface. For Specimen TS1 the CFRP strains measured from Strain Gauges 12 and 16 reached the values of about 8900 and 7000 micro strain, corresponding to 1/2 and 2/5 of the CFRP ultimate strain f_{fu}, respectively. This shows the effectiveness of the CFRP in this method, as strains of only about 1/3 to 1/5 f_{fu} were reached in the conventional CFRP strengthening methods (Bousselham 2010). In specimen TS2 the CFRP strain increased rapidly from the first to the fourth loading cycles and then maintained at high level (from 2800 to 3000 and from 3500 to 4000 micro strain at the location of Strain gauges 16 and 12 respectively) at the peak of the next two loading cycles. These results coincide with the measured beam tip load.

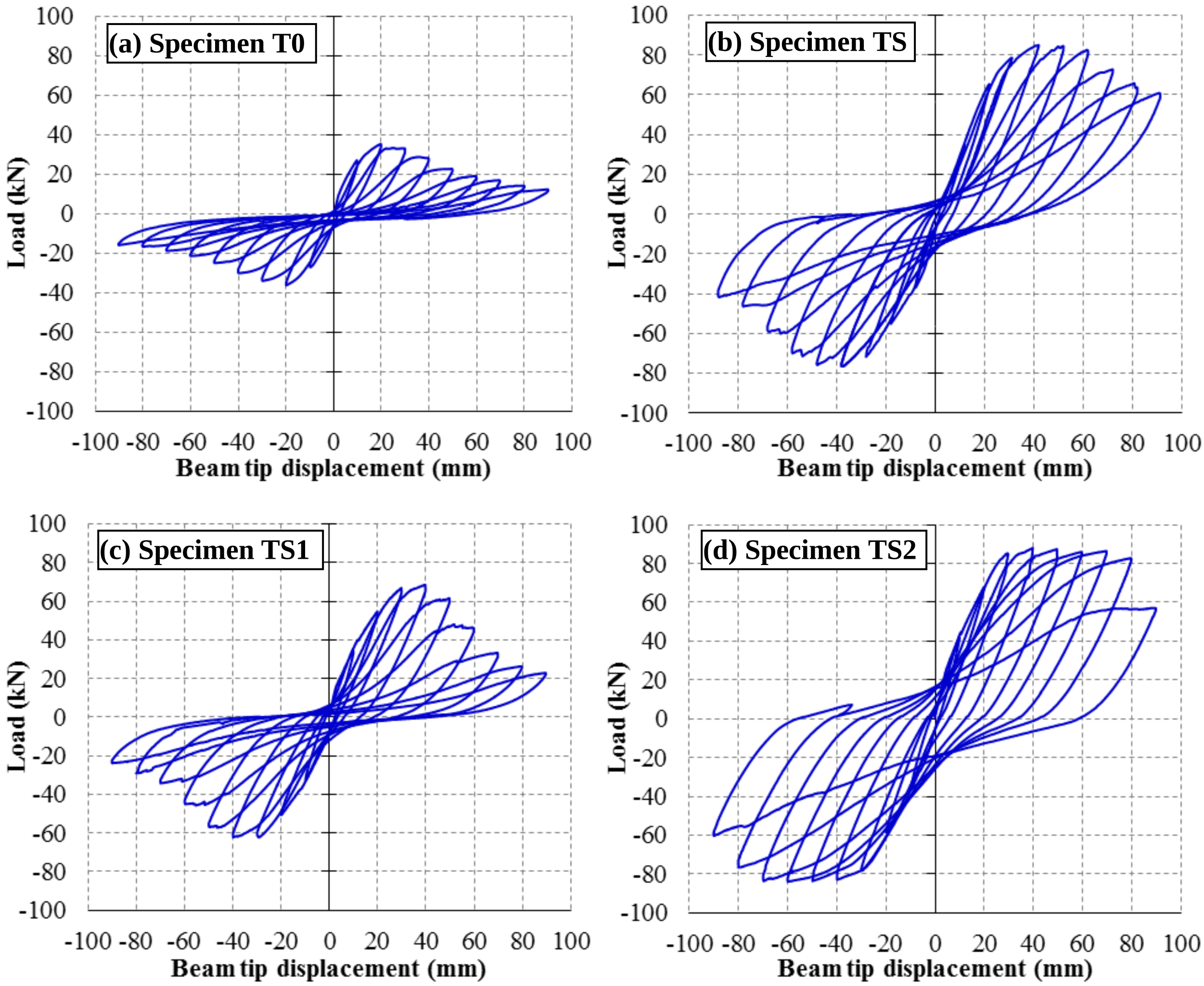


Figure 4. Load versus displacement response hysteresis of Specimens T0, TS, TS1 and TS2

The confining stress created at the joint centre could be the main reason that kept the CFRP from bulging or debonding from the concrete at the joint centre, which occurred commonly in the conventional FRP methods. This also helped maintaining the bond between the existing and the concrete covers so that improving the possibility of the existing and the glued concretes to work simultaneously in resisting the cyclic shear forces. By reducing the risk of bulging and debonding as well as increasing the confinement effect, it is believed that the possibility of the concrete and the CFRP to work well together to resist the shear forces was improved in the proposed method.

(a) Specimen T0 (b) Specimen TS (c) Specimen TS1 (d) Specimen TS2

Figure 5. Failures of Specimens T0, TS, TS1 and TS2

CONCLUSIONS

Four RC exterior beam-column connections have been cast and tested under cyclic loading to evaluate the effectiveness of a new strengthening method using CFRP and concrete covers. The variation of the joint failure mode with the applied CFRP ratio has been presented. Despite the difference in the failure modes, the test results have proved that this strengthening method effectively improve the seismic performance of the substandard details T connections. By modifying the columns of the connections from square to circular sections before wrapping with CFRP, this method improved the CFRP confinement effect and eliminated debonding and/or bulging of the CFRP at the joint centre, leading to the improvement in the effectiveness of the applied CFRP.

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