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Effects of Eccentricity on the Seismic Rehabilitation Performance of Nonseismically Detailed Interior Beam-Wide Column Joints

Bing Li¹, Qian Kai², and Weichen Xue³

Abstract: In this paper, the effects of the eccentricity on the effectiveness of fiber-reinforced polymers (CFRP) or glass fiber reinforced polymer (GFRP) in repairing the shear strength and ductility of nonseismically detailed beam-wide column joints have been evaluated. For this purpose, four (two concentric and two eccentric) full-scale nonseismically detailed interior beam-wide column joints were used as control specimens. All four subassemblages were subjected to similar cyclic lateral displacement so as to provide the equivalent of severe earthquake damage. The damaged control specimens were then repaired by filling their cracks with epoxy and externally bonding them with CFRP sheets and GFRP sheets. These repaired specimens were then re-tested and their response histories were obtained. Hence, a total of eight specimens were tested: four control, and four repaired. The response histories of the control and repaired specimens were then compared. The results were compared through hysteretic loops, load-displacement envelopes, energy dissipation capacity, secant stiffness degradation, and shear strength and damage indices. Moreover, the effectiveness of the proposed repair schemes in repairing the eccentric and concentric joints was also compared. The present study demonstrates that proposed repair schemes can recover the performance of both damaged RC concentric and eccentric beam-wide column joints effectively.

Keywords: eccentricity; beam-column joints; repair; rehabilitation; seismic; FRP

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1 **Introduction**

2 Premature failure of reinforced concrete (RC) beam-column joints may lead to large lateral
3 deformations and consequent collapse. Inadequate detailing of beam-column joints may result in
4 bursting failures, shear failures, and anchorage failures, possibly contributing to the soft storey
5 effect. As a critical element in structural design, beam-column joints play a pivotal role in
6 resisting earthquake loading. However, the BS 8110 code (1997) used in Singapore does not
7 specify any provision for the seismic design or detailing of RC structures. There are insufficient
8 transverse reinforcements, discontinuous beam bottom reinforcement or other non-ductile
9 detailing (Li *et al.* (2002)). Therefore, the structures in these regions of low to moderate
10 seismicity have to rely on its inherent ductility to respond to seismic loading.

11 Current practices generally see the demolition of the damaged structure in the instance of
12 damage, which is highly resource inefficient. Rehabilitation on the other hand could be more
13 economical than demolition of the damaged structure. However, the lack of understanding of the
14 performance and effectiveness of the repairs has seen structure rehabilitation shunned in favor of
15 complete demolition. This provides a significant impetus to illustrate the seismic performance of
16 beam-column joints repaired by different schemes. Knowledge gained from the test results in
17 this paper will be useful in developing effective and economical techniques to rehabilitate such
18 nonseismically designed beam-wide column joints.

19 Fiber reinforced polymer (FRP) composite materials have gained popularity over the past
20 decade because of their high strength-to-weight ratios, corrosion resistance, ease of application
21 and tailor-ability. Fiber orientation in each ply can be adjusted to meet specific strengthening
22 objectives (Engindeniz *et al.* (2005)). Gergely *et al.* (1998, 2000) have experimental investigated
23 the effectiveness of CFRP composites to strengthen of T-joints and have developed a simplified

analytical model for the prediction of shear capacity of FRP strengthened beam-column joints. Ghobarah and Said (2002), El-Amoury and Ghobarah (2002) developed an effective selective rehabilitation scheme for RC beam-column joints using advanced composite materials. Karayannis *et al.* (1998) proved the effectiveness of repair the damaged beam-column joints by using epoxy injection. Karayannis and Sirkelis (2002, 2008) further investigated the strengthening and rehabilitation performance of RC beam-column joints by combining the effects of the epoxy resin injection and CFRP jacketing. Their study demonstrated the damaged joints after epoxy resin injection could well restore their seismic behavior. If the damaged joints after epoxy resin injection and CFRP jacketing strengthening, their seismic resistance behavior could be significantly upgraded. Al-Salloum and Almusallam (2007) and Alsayed *et al.* (2010) developed effective rehabilitation schemes for RC beam-column interior and exterior joints respectively, using advanced composite materials. Their experimental results provided very satisfactory confirmation of the viability and reliability of the adopted retrofit solution and of the proposed analytical procedure to predict the actual sequence of events. Moreover, Tsonos (1999, and 2007) have derived an analytical model for predicting the shear capacity of RC beam-column joints. However, in actual application, there are more realistic construction difficulties in the FRP application, such as the presence of slab and transverse beams. Thus, it is paramount to investigate the effectiveness of FRP for strengthening and repairing of full-scale, complete frame structures while considering the three-dimensional (3D) and bidirectional load effects. Balsamo *et al.* (2005a and 2005b) and Di Ludovico *et al.* (2008) have conducted full-scale RC frame tests to investigate the performance of deficient full-scale frame strengthened with FRPs via the pseudo-dynamic load method. However, Pampanin *et al.* (2007) investigates the seismic behavior of 2/3 scale RC frame after retrofitting by FRPs via quasi-static

load method. Engindeniz *et al.* (2008a and 2008b) have conducted a series of full-scale corner beam-column-slab sub-assemblages to comprehensively investigate the slab, bi-directional and 3D effects on the corner joint strengthening and repairing. The above, complete frame tests and realistic 3D full-scale sub-assemblage tests significantly improved the understanding of the efficiency of FRP repair and strengthening for realistic RC frames subjected to the seismic load scenario. The observation and conclusions obtained from the above tests aiding for developing the design guidelines. The above literature review indicated that extensive experimental and analytical studies have conducted in the past decade for FRPs to repair and strengthen the RC joints. However, few studies have been carried out regarding the retrofitting of beam-wide column sub-assemblages, which is a popular practice in zones of low seismicity like Singapore. Li and Chua (2009) have conducted six full-scale experimental specimens to develop an effective and economic FRP strengthening scheme for nonseismically detailed interior beam-wide column joints. However, Li and Chua (2009) only focused on the strengthening of the beam-wide column joints before damage. Li and Kai (2011) have conducted a series of tests to evaluate whether the proposed repair schemes can effectively repair the damaged concentric beam-wide column joints. Although Li and Kai (2011) have indicated that their proposed repair schemes can effectively repair the seismic damaged concentric beam-wide column joints, the effectiveness of their proposed repair schemes to repair the eccentric beam-wide column joint, where the column centerline is offset from the beam centerline, are still not clear. It should be pointed out that, when an eccentric beam-column joint is subjected to seismic loadings, the eccentricity between beam and column centerlines will induce additional torsion moments in the joint region (as shown in Fig. 1). This produces additional joint shear stresses. For normal beam-column joints, the effect of the eccentricity on their seismic performance is limited

(Lawrance *et al.* (1991) and Shin and LaFave (2004)) possibly because the width of the beam is close to the width of the column. For beam-wide column joints, the effect of the eccentricity possibly is significant as the width of the column is significantly larger than the width of the beam. Thus, it is necessary to study the effect of eccentricity on the seismic performance of the beam-wide column joints and to assess the effectiveness of the proposed repair schemes in Li and Kai (2011) to repair the eccentric beam-wide column joints. In general, this study is a supplemental work of Li and Kai (2011). It should be noted that no slab and transverse beams were incorporated in these test models, which was a deficiency of this study. The realistic 3D bi-directional beam-wide column joints should be conducted in the future (like Engindeniz *et al.* 2008a and 2008b) to more accurate regarding the real performance of the joints.

The first part of the paper briefly presents the seismic behavior of four full-scale control joints typically found in framed structures with no seismic detailing in Singapore. After studying the failure modes of these control specimens, different FRP rehabilitation schemes were utilized to repair the different specimens. Then, the repaired specimens were re-tested under similar loading conditions.

Experimental Program

Descriptions of Control Specimens

An eight-storey reinforced concrete wide-column moment resisting frame designed according to BS 8110 code (1997) was investigated (Fig. 2). The frame was designed to sustain the gravity loading and a notional load required by BS 8110 code (1997) while no seismic loading was considered. The tested specimens were typical joints extracted from the edge frame and interior frame for eccentric and concentric joints, respectively. However, for the tested beam-wide column joints, the length of the beam span and the height of the column were modified to fit the

test setup but the dimensions of the beam and column sections and reinforcement details were left unchanged. The major deficiencies of the detailing of these control specimens are: 1) Hoop stirrups with 90 degrees bend were utilized as transverse reinforcements, 2) No transverse reinforcements were provided within the joint regions. 3) No additional transverse reinforcements were provided within the beam potential plastic hinge zone. Fig. 3 illustrates the schematic dimensions of the specimens. Specimens E1A and E1B are eccentric interior beam-wide column joints with the beam flushed with the column edge, whereas Specimens C1A and C1B are concentric interior beam-column joints with their respective centerlines of beams and columns intersecting. These specimens had a column-to-beam width ratio of approximately 3.56 and had a column cross-section dimension of 820×280 mm and a beam cross-section dimension of 230×300 mm. The ratio of column width to the effective joint width was 2.2. As shown in Table 1, the column-to-beam strength ratio $\sum M_c / \sum M_b$ is 5.23, 6.36, 5.23, and 6.36 for Control Specimens C1A, C1B, E1A, and E1B, respectively. According to ACI-08 (2008), these specimens satisfy the requirement $\sum M_c / \sum M_b > 1.20$ and meet the criteria of a strong column-weak beam design philosophy. Two levels of axial compression load $0.1 f'_c A_g$ and $0.35 f'_c A_g$ were applied on joints to investigate the effect of axial compression load on the seismic behavior of beam-wide column joints. The specimens were built with identical reinforcements and cast with Grade 20 concrete. In Singapore, the buildings are normally designed to sustain high axial loading of $0.35 f'_c A_g$. The concrete compressive strengths of Specimens E1A, E1B, C1A and C1B, on the testing days were 18.9, 18.4, 18.4 and 19.2 MPa, respectively. The average yield strength of the steel rebar is 354, 510, 508, 504, and 517 MPa for R10, T10, T13, T22, T25, respectively. The stress-strain curves for these rebar are shown in Fig. 4. The properties of FRP are as shown in Table 2.

Test Set-up

A schematic view of the loading apparatus is shown in Fig. 5. To simulate seismic cyclic loading on the specimen, a reversible horizontal load was applied in a quasi-static fashion at the top end of the column through a double-acting 1000 kN capacity long-stroke dynamic actuator, mounted on the reaction wall. The actuator was manually operated to have a better control on the load increment. The constant axial column load was applied using small hydraulic jacks placed between the column top end and bottom suffix of the steel transfer beam. Threaded rods were also fixed around the test unit to balance the applied axial load. The column end was pinned to the strong floor. The beam ends were connected to the strong floor by steel links which allowed rotation and free horizontal movement of the beams while preventing vertical movement.

Instrumentation

A number of instruments including a load cell merged in the dynamic actuator, strain gauges, linear variable displacement transducers (LVDTs) and displacement transducers were utilized to record data from the experimental setup. Details of the instrumentation layout are illustrated in Fig. 5. (1) Three LVDTs installed at the top face of the column were used to measure the horizontal displacement and the torsional deformation. (2) A total of 36 electrical resistance strain gauges were mounted on reinforcing bars at key locations in and around the connection (the specific locations of these strain gauges were shown in Fig. 3). (3) A series of displacement transducers were installed on the top and bottom of beams to estimate beam rotations in the vicinity of beam-column interfaces. (4) Two LVDTs were installed on joint region to measure the overall joint shear deformations. (5) Axial force on the top of the column was monitored by the pressure-gauge of the oil supplier.

Test Procedure

The axial load was gradually applied onto the column until the designated level, $0.1 f'_c A_g$ for Specimens E1A and C1A and $0.35 f'_c A_g$ for Specimens E1B and C1B were reached. The axial compression load was maintained during each test by manually adjusting the flat jacks after each load step. The lateral load was cyclically applied through the dynamic actuator in a quasi-static fashion at the top end of the column. The typical loading procedure consisting of displacement controlled steps are illustrated in Fig. 6 (The fractions shown in Fig. 6 are the targeted drift ratios). As shown in the figure, a displacement rate of 6 mm/min was used until the drift ratio reached 1.33 %. After 1.33 % of drift ratio, the rate was increased to 10 mm/min.

Seismic Behavior and Failure Modes of the Control Specimens

The behavior of each specimen was controlled by the flexural mechanism of the weak beam. Strain gauge results indicated that a plastic hinge was formed in the beam near to the beam-column interface while limited cracks were observed in the column during the test.

Eccentric Specimen E1A was loaded under column axial load of $0.10 f'_c A_g$. Cracks were first observed at beam top when DR of 0.4 % was exceeded. It was noticeable that cracks at beams propagated rapidly at DR of 0.67 % where the cracks in joint core area were found. At DR of 1.0 %, cracks were found to propagate rapidly at beam top as well as joint core. Even though no crack was found in column, more cracks formed further from the joint core area while new cracks were found at beam bottom after DR of 1.33 %. Due to its drastic drop in lateral resisting capacity, the specimen failed at DR of 2.0 %. Slight spalling of concrete was observed at the beam end near to the left beam-column interface, as shown in Fig. 7. The column was also found to be intact with no cracks when the test was terminated. It is noteworthy that severe joint diagonal cracks were only observed at the flushed surface of the beam-column joint while the protruded joint on the opposite face was found to be intact. This is majorly due to the width of

the column significantly larger than the column effective width. Further discussion of this phenomenon will be given in the section of “Joint Shear Stress and Principal Tensile Strength”.

Eccentric Specimen E1B was loaded under column axial load of $0.35 f'_c A_g$. Initial cracks formed at beam top during DR of 0.4 % and cracks propagated rapidly when DR of 1.0 % was attained. It is noteworthy that cracks in joint core area were first observed at DR of 1.0 %. Generally, no cracks were found on the column and no spalling of concrete was observed in the beam. More cracks formed further away from the joint core area, while limited new cracks were found at the bottom of beam after DR of 2.0 %. Specimen E1B deteriorated after DR of 3.0 %. Crack patterns are shown in Fig. 8. Comparing to Specimen E1A, the joint diagonal cracks in Specimen E1B were much milder. Compare the results of Specimen E1B ($0.35 f'_c A_g$) with that of Specimen E1A ($(0.10 f'_c A_g)$), Specimen E1B has lower lateral resisting capacity while higher energy dissipation capacity. This is due to the axial compression load having a positive effect on the anchorage resistance of the beam bars passing the joint before the anchorage resistance reached its maximum value. However, when the bond resistance reached its maximum value and started to degrade, the presence of axial compression load will accelerate the bond deterioration. This can be seen from the strain gauge results. As shown in Fig. 9, at a DR of 2.0 %, the compression reinforcement of Specimens E1B and C1B attain tensile yielding, indicating anchorage failure. However, at this drift ratio, a sufficient amount of bond resistance still exists in Specimens E1A and C1A.

Fig. 10 shows the formation of cracking patterns that occurred during the experiment on Specimen C1A. It should be noted that the crack pattern of Specimen C1B is similar to Specimen C1A. As the test observation of these two specimens has been detailed introduced in Li and Kai (2011), only the final failure mode was re-emphasized in here. For the C1 series

specimens, the failure was concentrated in the beam-column interfaces. Throughout the test, no cracks were found on the columns for both specimens due to presence of column axial loads and large column effective width compared to that of E1 series specimens. However, Specimen E1B and C1B showed similar energy dissipation capacity and lateral resisting capacity trends. At a DR of 4.0 %, the energy dissipation capacities of the Specimens E1B and C1B were 10.3 and 10.2 kNm respectively. This indicates high column axial load could override the effect of eccentricity due to the high column axial load increased the width of the concrete compression strut of the joint in resisting the joint shear force. Moreover, the high axial load reduced the tensile principal stress of the joint (explained detailed in the following).

Proposed Repair Schemes

Following the loading and failure of the Control Specimens E1A, E1B, C1A and C1B, the joints were repaired by epoxy injection together with FRP jacking. As mentioned in the introduction section, one of the objectives of this study was to evaluate the effectiveness of the proposed schemes in Li and Kai (2011) to repair the eccentric beam-wide column joints. Thus, the repair schemes utilized in this study are similar as the schemes proposed in Li and Kai (2011). The rehabilitation scheme #1 is designed as follows:

- i. **Beam flexural strength repairing.** The failure modes and strain gauge reading of all of specimens indicated that there are plastic hinges formed in the beams near the beam-column interfaces. The design thickness (t_b) was selected such that the load capacity of the sheets would be equivalent to that of all beam longitudinal bars at yield:

$$t_b = \frac{\sum A_s f_y}{w_s \times \varepsilon_{f.e} \times E_{frp}} \quad (1)$$

where w_s is the effective width of the sheet, and $\varepsilon_{f.e}$ is the design value of effective

CFRP strain (0.006 was suggested value by FYFE Asia Private Limited in Singapore), and E_{frp} is the elastic modulus of FRPs in the principal fibre orientation.

Thus, one layer of CFRP L-wrap (Sheet 1 in Fig. 11) is applied at each of the four corners of the joint. The CFRP is bent at 90 degrees and thereafter extended 1000 mm along the beam length and 500 mm along the column. The sheet with 500 mm length along the column was not only providing the additional flexural strength for the column, but also providing the well anchorage for the beam flexural strength repairing. The 1000 mm sheet along the beam was arbitrary selected (3 times of the length of the beam effective depth) to ensure well anchorage of the fibre sheets.

ii. **Column flexural strength repairing.** The 500 mm length sheet of the CFRP L-wrap could provide additional strength of the column. However, it should be noted that the column flexural strength strengthening majorly relies on the GFRP wrapping of the column (Sheet 2 in Fig. 11). For the current tests, strengthening the column was not a critical issue due to large column-to-beam strength ratio was designed and limited damage was observed in the column from the failure mode of the control specimens.

iii. **Joint shear strength strengthening.** The maximum horizontal joint shear force ($V_{jh}^{\max}$) that could develop was estimated by assuming yielding of all beam top bars and fracture of the external bonded FRP sheet (Sheet 1 in Fig. 11).

$$V_{jh}^{\max} = \sum A_s f_y + \sum A_{frp} f_{frp} - V_{col} = 2 \times 266 \times 510 + 2 \times 299 \times 575 - \frac{M_{nb}}{l_c} \quad (2)$$

where M_{nb} is the moment capacity of the beam when both the tensile rebar and FRP reached their ultimate strength and l_c is the column length.

The shear capacity of the strengthened joint can be calculated as follows:

$$V_{Rd} = V_{cd} + V_{wd} + V_{frp} \quad (3)$$

where V_{cd} is the shear capacity of the concrete compression zone according to ACI-08 (2008), V_{wd} is the shear carried by the web reinforcement through the truss mechanism according to ACI-08 (2008), and V_{frp} is the FRPs contribution to shear capacity, which can be written in the following form as suggested by Gergely *et al.* (2000):

$$V_{frp} = \varepsilon_{f,e} E_f t_{jh} (0.9 h_j) \quad (4)$$

where t_{jh} is the thickness of the FRPs oriented perpendicular to the longitudinal axial of the column (90 degrees), and h_j is the height of the joint cover with CFRP (230 mm).

As observed in the failure modes of the control specimens, the damage in the joint was related limiting. Moreover, based on the conclusion of previous research (Karayannis *et al.* 1998), the epoxy injection could repair the joint very well. Thus, the shear capacity of the compression zone (V_{cd}) was assumed as ACI-08 (2008)

$$V_{cd} = 1.0 \sqrt{f'_c} b_c h_j \quad (5)$$

where b_c is the column effective width, which may be taken as the width of the beam plus 1/4 depth of the column in each side. Thus, the effective width of E1 series and C1 series specimens is 300 mm and 370 mm, respectively (Paulay and Priestley 1992).

It should be noted that the above joint shear strengthening schemes were designed without considering the most distinguish characteristic of the test joints (wall like column). Frankly say, the authors did not believe the FRP sheets oriented perpendicular to the longitudinal axial of the column (90 degrees) could effective increase the joint shear strength of these beam-wide column joints as the beam-normal width column joints due to these FRP sheets were not directly bonded the column's effective region. More tests should be conducted in the future regarding this critical problem.

iv. **Beam shear strength repairing.** According to ACI-08 (2008), the design shear strength carried by the concrete was

$$V_c = 0.2\lambda\phi_c\sqrt{f'_c}b_wd \quad (6)$$

where V_c is the shear force carried by the concrete, λ is a factor related to the density of the concrete (here $\lambda = 1.0$), and ϕ_c is the shear reduction factor ($\phi_c = 0.6$ for concrete).

Eq. (6) indicated that concrete could provides enough shear strength to resist the beam shear force of the control specimens although diagonal shear crack was observed in the control specimens at final of the tests. For repaired specimens, as the beam flexural strength was repaired/strengthened, the shear strength of the beam should be strengthened slightly correspondingly. Thus, one layer of GFRP U-wrap extending 500 mm from the beam-column interface was applied on both beams to increase shear strength (Construction innovation (2002)). Similar as the column wrapping, the GFRP U-wrap was not only increase the beam's shear capacity, but also ensured a good anchorage of the FRP sheets using for strengthening the joint body (Sheet 3 in Fig. 11).

Scheme #2 is similar to the scheme #1; only the continuous CFRP L-wrap at each of the four corners of the joint and CFRP sheet in the direction parallel to the beam axis across the joint core are replaced by GFRP L-wrap and sheet respectively. Two different design schemes were utilized in the current tests to study the effect of FRP material on the repairing effectiveness of the damaged RC joints although. Fig. 11 illustrates the rehabilitation schemes step by step. The repaired specimens shall be referred to with a prefix of "R" to the control specimen names. For instance, Specimen RE1A is the repaired specimen E1A. Specimens RE1B and RC1A were strengthened with scheme #1 while Specimens RE1A and RC1B were strengthened with scheme #2. As shown in the Fig. 11, in all repaired specimens, the FRP sheets terminated at 70 mm from

the top of the beams to account for the presence of the floor system. Fig. 11 also shows the locations of the fibre anchors. Fibre anchors are utilized to allow the FRP sheets to delay the premature delamination of FRP sheets. The fibre anchor consists of two parts, the anchor bolt and the protruding fibres. The total length of the fibre anchors used was about 110 mm. The length and diameter of the anchor bolt were about 50 mm and 7 mm respectively. First, holes (10 mm diameter) were drilled at a depth 50 mm on beam faces before the application of FRP sheets. After the application of FRP sheets, the anchor bolts were inserted through the epoxy resin into the holes. The protruding fibres are bent and spread out in circle with radius about 60 mm to act as the base of anchorage. These anchorages were applied at intervals of 300 mm (selected arbitrary). It should be noted that in order to ensure good adherence between the FRP sheets and concrete, the surface was first cleared of dust and any deleterious substances which acted as bond barriers. Sharp edges and protrusions were removed by mechanical grinding. Corners of the specimens strengthened were rounded to a minimum radius of about 20 mm to avoid FRP cracks due to local failure and stress concentration (Columb *et al.* 2008).

Results and Discussion

Failure Modes

For Specimen RE1A, flexural cracks were first observed at beam bottom at DR of 0.13 %. At DR of 0.4 %, the cracks became more inclined in Specimen RE1A, indicating shear cracks. Also, small regions of light shades on the GFRP sheet were observed around beam-column interface. First crack at beam top for Specimen RE1A was observed at DR of 1.33 %. At DR of 2.0 %, Specimen E1A failed with a large decrease in lateral load resistance while Specimen RE1A kept increasing the lateral load resistance. At this DR stage, light shades observed previously enlarged showing greater debonding (see Fig. 12a) was occurred. At DR of 3.0 %, cracks due to breaking

of epoxy resin were heard. Epoxy cracks were observed at the corners of the joint (see Fig. 12b). At the end of the test, GFRP sheet delaminated at joint core at the flushed side with cracks of epoxy resin at the joint corners as shown in Fig. 12c. GFRP sheet delaminated within 200 mm from the beam-column interface at both top beam surfaces.

For Specimen RE1B, the first flexural crack was observed at beam bottom at DR of 0.1 %. As the DR increased, the flexural cracks propagated. At DR of 0.67 %, the first cracking sound was heard and flexure-shear cracks were observed on Specimen RE1B. First few flexural cracks at beam top occurred at DR of 1.33 % for Specimen RE1B. Slight debonding of GFRP sheets shown by small regions of lighter shades near to the beam-column joint interface was also observed. As the drift ratio increased, these regions enlarged further, accompanied by continuous audible breaking of epoxy resin. Delamination of CFRP sheet near the top beam-column joint interface caused by shear cracks (see Fig. 13a) occurred at DR of 2.0 % and was severed in subsequent drift ratios. Towards the last few cycles, the CFRP and GFRP sheet failed with a wide crack formed near the beam-column joint interface. Wide flexural crack was formed near the termination of the U-wrapping of beam (see Fig. 13b).

Specimens RC1A and RC1B failed by debonding of GFRP sheet near to the beam-column interface and delamination of CFRP/GFRP sheet at the top surface of beam. For Specimen RC1A, the first flexural crack was observed on the right beam top at a DR of 1.0 % and lighter shades of GFRP sheet, indicating debonding, were observed near the beam-column interfaces (see Fig. 14). Epoxy resin failure was initiated near the joint region as significant cracks were observed. At a DR of 3.03 %, the delamination of CFRP sheets was observed at the top beam surface. For Specimen RC1B, cracks started to propagate and became more inclined at a DR of 0.67 %. Also, first shear crack was observed on right beam top. At a DR of 1.0 %, both flexural and shear cracks started to

propagate rapidly, accompanied by cracking sound. Small regions of debonding were observed near the beam-column joint interfaces. At a DR of 2.0 %, slight concrete cover debonding due to shear crack was observed on left beam top. Finally, the beams failed in shear with GFRP sheet delamination at the top surface of beam (see Fig. 15b) and GFRP sheet debonding near beam-column joint interfaces, indicated by lighter shades (see Fig. 15a). The key results of repaired specimens are shown in Table 3.

Load-Displacement Hysteresis Loops

All control specimens have unique theoretical lateral strength P_i due to it was computed by assuming the edge-beams would reach their moment strengths at the beam-column interfaces (beam hinging) due to the column-to-beam strength ratio is larger than 5.0. As shown in Fig. 16, the theoretical lateral strength P_i can be calculated as following expression:

$$P_i = \frac{(M_{b1} \times \frac{l_{n1}}{l_1} + M_{b2} \times \frac{l_{n2}}{l_2})}{2 \times l_{c1}} \quad (7)$$

where M_{b1} and M_{b2} are the bending moment of the beam section near to the column interfaces, l_{n1} , l_{n2} are the clear span of the left and right beam, respectively, l_1 , l_2 are the span of the left and right beam, respectively, and l_{c1} is the height of the upper column.

Fig. 17 shows the load-displacement hysteresis loops of the control and repaired specimens. The hysteretic behavior was evaluated in terms of lateral resisting capacity and maximum displacement. The curves exhibited considerable pinching (the middle part of each hysteretic loop was relatively narrow). This behavior is characteristic of most RC frame connections and is typically attributed to reinforcement bond slip through the joint region. As shown in the Fig. 3, the bottom reinforcement did not continue through the interior joint. The lap splice region possible produces extra slip. The repaired specimens performed more severe pinching than the

corresponding control specimens due to no direct action was taken for repairing the bond-slip damage cumulated in the control specimens. However, in considering of the lateral resisting capacity, the repaired specimens were significantly higher than their corresponding control specimens except Specimen RE1A. However, Specimen RE1A recovered the positive lateral resisting capacity of the Control Specimen E1A at final drift ratio and had much higher negative lateral resisting capacity.

Load-Displacement Envelopes

Fig. 18 shows the envelope curves of storey shear versus horizontal displacement in the specimens, from connecting the first peak load point of each three cycle group. As shown in the Fig. 18, towards the last few cycles, all repaired specimens were able to restore strength more than that of their theoretical lateral strength.

The lateral resisting capacity of Specimen RE1A is larger than its corresponding Control Specimen E1A in both the positive and negative loading cycle by 0 % and 17 % respectively. However, for Specimen RE1B, the increased lateral resisting capacities in the positive and negative loading cycles are 37 % and 68 % respectively. Moreover, Control Specimen E1A was broken at DR of 2.0 % while the repaired Specimen RE1A could reach up than the DR of 4.0 %. This increased ductility can significant increase the energy absorption capacity of joint. The strength of Specimen RE1A does not degrade even at a DR of 4.0 %. However, for Specimen RE1B, its strength at a DR of 4.0 % only decreases by 15.2 % and 2.3 % in the positive and negative loading cycles, respectively.

The lateral resisting capacity of Specimen RC1A is larger than its corresponding Control Specimen C1A in both the positive and negative loading cycle by 17.4 % and 49.7 % respectively. Similarly, Specimen RC1B had a much higher lateral resisting capacity than its corresponding

control specimen as well. Moreover, for Specimens RC1A and RC1B, its strength at a DR of 4.0 % only decreases by 2 % and 5.5 % in the positive and negative loading cycles, respectively. This slower strength degradation of the specimens is always favorable when the structures suffer an earthquake event.

Energy Dissipation Capacity

The survival of the structures due to earthquake loading depends on its ability to dissipate the input energy. In this study, one form of energy dissipation: hysteretic damping was estimated by the area enclosed by the lateral load-displacement loops. Comparisons of cumulated energy dissipation capacity of control and repaired specimens are presented in Fig. 19. The cumulative energy dissipated was calculated by summing up the energy dissipated in consecutive loops throughout the test. All repaired specimens can recover its original energy dissipation capacity of their counterparts except Specimen RC1B. For Specimen RC1B, it only can recover almost 74 % of its original energy dissipation capacity. This is due to repairing the concentric beam-wide column joint with GFRP sheets is not as effective as repairing the eccentric joint. Moreover, CFRP sheet is much effective confining the wide-column joint region than that of GFRP sheet.

Stiffness Degradation

Fig. 20 shows the comparison of secant stiffness in degradation of both control and the repaired specimens. In the initial stage, Specimen RE1A was much stiffer than E1A. After which, there was a rapid decrease in stiffness due to concrete cracking. The FRP debonding after DR of 2.0 % explained the relatively linear curve in the subsequent cycles. Its initial stiffness started to increase until about DR of 0.2 % and decreased rapidly until DR of 1.33 % due to severe crack was formed in the beam-column interfaces. After DR of 1.33 %, the slope of the curve becomes relatively milder comparing with that of E1A. Such slower stiffness degradation is desirable as often during

an earthquake, structures collapse due to sudden loss of stiffness of beam-column joints with increasing lateral movement of the structure. For Specimen RE1B, though had lower initial stiffness, generally restored the original stiffness of Control Specimen E1B at every stage after DR of 0.2 %. Similar as Specimen RE1A, the stiffness decreased rapidly until DR of 1.33 % due to flexural and shear cracks were formed in the beam. For repaired C1 series, Specimens RC1A and RC1B managed to have higher stiffness at every drift ratio than control counterparts. The detailing of the stiffness performance of these two specimens can refer to Li and Kai (2011).

Joint Shear Stress and Principal Tensile Stress

As shown in Fig. 16, the horizontal and vertical joint shear forces can be calculated as following equations:

$$V_{jh} = T_b + C'_{sb} + C'_{cb} - V_{col} = T_b + T'_b - V_{col} \quad (8)$$

$$V_{jv} = T_c + C'_{sc} + C'_{cc} - V_b = T_c + T'_c - V_b \quad (9)$$

where V_{jh} and V_{jv} are the joint horizontal and vertical shear force, respectively.

The shear stresses in the joint region can be calculated using the following formulas:

$$\nu_{jh} = \frac{V_{jh}}{h_c \times b_c} \quad (10)$$

$$\nu_{jv} = \frac{V_{jv}}{h_b \times b_b} \quad (11)$$

where ν_{jh} and ν_{jv} are the joint horizontal and vertical shear stress, respectively, b_c and b_b are the effective width of column and beam, respectively, h_c and h_b are the effective depth of the column and beam, respectively.

The shear strength coefficient, γ is calculated and compared with ACI-08 (2008) and NZS 3101 (1995) in Table 1. It should be noted that, for simplification, the lever arm of the beam section of the repaired specimen was assumed same as that of the corresponding control

specimens (about 0.85d). As shown in the table that the joint shear stress of C1 series specimens is less than that of E1 series specimens due to the eccentricity reduced the column effective width and the torsion produces additional joint shear stresses. Moreover, the shear stress of all specimens is less than the suggested value of ACI-08 (2008). This indicated that joint failure is not critical for test specimens (consistent with the failure mode observation). Finally, the shear stress of RE1B and RC1A is larger than the upper bound of the suggested value of NZS 3101 (1995). This indicated that the FRP sheets wrapping the joint have provided additional joint shear strength.

The principal tensile stress is given by Mohr's circle and expressed as below:

$$\sigma_1 = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \nu^2} \quad (12)$$

where σ is the axial stress and $\nu = (\nu_{jh} + \nu_{jv})/2$ is the equivalent shear stress.

As shown in the Table 1, the principal tensile stress of E1B and C1B is significantly less than that of E1A and C1A, respectively. This was consistent with the crack pattern observation that mild joint diagonal cracks were observed in E1B compared to E1A. For Specimen C1A, the principal tensile stress is larger than $0.29\sqrt{f'_c}$ and supposed to form joint diagonal crack. However, no joint diagonal crack was observed during test possible due to the equation for determining the column effective width, which was proposed by Paulay and Priestley (1992), was conservative for the column-to-beam width ratio of 3.56.

Damage Index

The structure's ability to survive an earthquake may be measured in terms of the expected state of damage of the structure after the earthquake. Damage may be quantified by using any of several damage indices defined as functions whose values can be related to particular structural damage

states. The widely used damage index model proposed by Park and Ang (1985) was used to evaluate the effectiveness of the repair schemes in this study due to its simplicity and it has been validated by extensive experimental data from various buildings damaged during past earthquakes (Karayannis *et al.* 2008). This damage index is defined as the linear combination of the ultimate displacement and the dissipated energy by the following expression:

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \quad (13)$$

where δ_m is the maximum deflection attained during seismic loading, δ_u is the ultimate deflection capacity under monotonic load, β is a model parameter that depends on the value of shear and axial forces and the amount of longitudinal and confinement reinforcement, and Q_y is the calculated yield strength, and dE is the incremental dissipated hysteretic energy corresponding to the drift ratio.

It should be noted that the values of δ_m , Q_y , and dE of this model can obtained from the test results of the test joints. However, the value of δ_u was estimated using the empirical formula for the calculation of ultimate drift ratio according to Eurocode 8 (2005):

$$\delta_u = \frac{1}{\gamma_{el}} 0.016 \times 0.3^{P^*} \left(\frac{\max(0.01, \omega')}{\max(0.01, \omega)} f_{cm} \right)^{0.225} (L^*)^{0.35} \times 25^{\alpha \rho_s \frac{f_{yt}}{f_{ym}}} (1.25)^{100 \rho_d} \quad (14)$$

where $\gamma_{el} = 1.0$, $P^* = P/bhf_{cm}$ is the axial load index, b and h are the width and height of the section, respectively, w and w' are the mechanical reinforcement ratio of the tension and compression longitudinal reinforcement, respectively, f_{cm} is the concrete compressive strength, $L^* = M/Vh$ is the shear span index, $\rho_s = A_{sx}/b_w s_h$ is the transverse reinforcement ratio, s_h is the spacing of stirrups, b_w is the width of the confined concrete core, f_{yt} is the yield stress of the transverse reinforcement, ρ_d is the diagonal reinforcement ratio, and α is the confinement effectiveness factor derived from the expression:

$$\alpha = (1 - \frac{s_h}{2b_w})(1 - \frac{s_h}{2h_c})(1 - \frac{\sum b_i^2}{6b_w h_c}) \quad (15)$$

where b_i denotes the centreline spacing of the longitudinal bars laterally restrained by a stirrup corner, and h_c is the height of the confined concrete core.

Because the test specimens have similar longitudinal and confinement reinforcement arrangements, parameter β is assumed 0.15 (Cosenza *et al.* 1993). The damage indices versus different drift ratio at first cycle were determined based on the above model (as shown in Fig. 21). As shown in the figure, the repaired specimens show much lower damage factors than the corresponding control ones except Specimen E1A. For that specimen, almost same damage factors were observed in Specimens E1A and RE1A before DR of 2.0 %. This indicated that the proposed repair schemes were effective for repairing the damaged eccentric and concentric beam-wide column joints. Moreover, as expected that the damage index was enhancing with an increase of the repeating cycles due to more damage was cumulated with increasing load cycles.

Conclusions

In the present paper, the effectiveness of the two economical FRP rehabilitation schemes proposed by Li and Kai (2011) to repair the damaged eccentric beam-wide column joints has been evaluated. Based on the observations and the experimental results of this study, the following conclusions can be drawn.

1. At an axial compression load of $0.10 f'_c A_g$, Eccentric Specimen E1A obtained lower energy dissipation capacity, seismic strength and stiffness than the concentric one than expected. However, at an axial compression loading of $0.35 f'_c A_g$, Eccentric Specimen E1B could obtain similar energy dissipation capacity, seismic strength and stiffness as the corresponding Concentric Specimen C1B. This is due to the high axial compression load having overridden

the effect of eccentricity.

2. In general, both Schemes #2 and #1 were able to recover the seismic performance for both concentric and eccentric beam-wide column joints. Though, Scheme #2 recovered the energy dissipation capacity of Specimen C1B, it is not so ideal due to the bonding of GFRP sheets parallel to the beam axis across the joint core. The confinement effect of GFRP sheet is slightly weaker than CFRP sheet.

3. The joint shear stress results indicated that E1 series specimens were more severe for joint shear than that of C1 series specimens due to the eccentricity significantly reduced the column effective width and the torsion produces additional joint shear stresses. Moreover, the shear stress of all specimens is less than the suggested value of ACI-08 (2008). This indicated that joint failure is not critical for all test specimens (consistent with the failure mode observation). Finally, the shear stress of RE1B and RC1A is larger than the upper bound of the suggested value of NZS 3101 (1995). This indicated that the FRP sheets wrapping the joint have provided additional joint shear strength.

4. The principal tensile stress of E1A and E1B is $0.53\sqrt{f'_c}$ and $0.24\sqrt{f'_c}$, respectively, which was consistent with the failure modes of these two specimens. The higher compression axial force could significantly reduce the joint tensile principal stress. The tensile principal stress of C1A is $0.46\sqrt{f'_c}$, which was expected to form joint diagonal cracks. However, no crack sign was found in this specimen, which indicated that the column effective width proposed by (Paulay and Priestley 1992) is conservative for the test specimens with column to beam width ratio of about 3.56.

5. In order to quantitatively evaluate the effectiveness of the proposed repairing schemes for beam-wide column joints, the damage level of the control and the repaired specimens is

assessed using a well-established damage index model, which is properly adapted to the parameters of this experimental study. Based on the computed values of the damage indices, it can be concluded that all repaired specimens presented lower damage factors than their counterparts except Specimen RE1A. Specimen RE1A displayed approximately the similar damage level with Specimen E1A before the DR of 2.0 %.

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Table 1

Comparison of the measured shear stress and principal tensile stress of the test specimens

Specimen	$\frac{P}{f'_c A_g}$	$\frac{\sum M_C}{\sum M_B}$	v_{jh}	γ	v_{jh}/f'_c	v_{jh}/f'_c NZS (1995)	γ ACI (2008)	v_{jv}	σ_1	$\frac{\sigma_1}{\sqrt{f'_c}}$
E1A	0.10	5.23	2.99	0.69	0.158	0.11-0.17	1.0	2.99	2.30	0.53
RE1A	0.10	5.23	3.00	0.69	0.159	0.11-0.17	1.0	3.00	2.37	0.55
E1B	0.35	6.36	3.04	0.71	0.165	0.11-0.17	1.0	3.04	1.04	0.24
RE1B	0.35	6.36	3.70	0.86	0.201	0.11-0.17	1.0	3.70	1.83	0.43
C1A	0.10	5.23	2.43	0.57	0.132	0.11-0.17	1.0	2.43	1.99	0.46
RC1A	0.10	5.23	3.43	0.80	0.186	0.11-0.17	1.0	3.43	3.25	0.76
C1B	0.35	6.36	2.46	0.57	0.128	0.11-0.17	1.0	2.46	0.83	0.19
RC1B	0.35	6.36	2.99	0.70	0.156	0.11-0.17	1.0	2.99	1.49	0.34

Table 2

Properties of Tyfo® Fibwrap Composite system

Parameter	Properties ^a	
	GFRP with epoxy, Tyfo® SEH-51A Composite	CFRP with epoxy, Tyfo® SCH-41 Composite
Type of FRP	Unidirectional GFRP sheet	Unidirectional CFRP sheet
Ultimate tensile strength in primary fiber direction	575 MPa	986 MPa
Elongation at break	2.2%	1.0%
Tensile Modulus	26.1×10^3 MPa	95.8×10^3 MPa
Laminate thickness	1.3 mm	1.0 mm

^a Properties value given are based on test value by supplier (FYFE Asia Pte. Ltd in Singapore).

Table 3

Comparison key parameters between control specimens with corresponding repaired specimens

Specimen ID	Positive Peak load (kN)	Negative Peak load (kN)	Total energy Dissipation (kNm)	Failure mode
E1A	19.8	-17.3	5.4	Plastic hinge and concrete crushing was formed in the beam-column interface, and severe
RE1A	19.8	-20.3	9.4	Debonding of GFRP sheet around the beam-column interface, GFRP sheet delamination at joint corner, and epoxy cracks at the
E1B	15.8	-14.9	10.3	Plastic hinge was formed in beam with limited joint shear cracks
RE1B	21.6	-25.0	10.2	CFRP sheet delamination at the top surface of beam, and wide flexural crack formed in the end of U-wrapping of the beam
C1A	18.4	-19.1	9.8	Plastic hinge was formed in beam, no cracks were observed within the column and joint region
RC1A	21.6	-28.6	10.4	Debonding of the wrapping of the column, and delamination of the CFRP sheet at top surface of beam
C1B	15.5	-15.5	10.2	Plastic hinge was formed in beam, no cracks were observed within the column and joint region
RC1B	25.0	-27.8	7.5	Debonding of the wrapping of the column, and delamination of the GFRP sheet at top surface of beam

Figure 1

Fig. 1. Forces acting on eccentric interior beam-column joint

Figure 2

Fig. 2. Reinforced concrete beam-wide column frame

Figure 3

Fig. 3. Geometry and reinforcement details of specimens (dimensions are in mm)

Figure 4

Fig. 4. Stress-strain relationship of steel rebar

Figure 5

Fig. 5. Test setup (dimensions are in mm)

Figure 6

Fig. 6. Loading procedure

Figure 7

Fig. 7. Typical cracking patterns of Specimen E1A of 2 %

Figure 8

Fig. 8. Typical cracking patterns of Specimen E1B at DR of 4 %

Figure 9

Fig. 9. Local strains in beam top longitudinal reinforcement of Specimens E1A and E1B

Figure 10

Fig. 10. Typical cracking patterns of series C1 at DR of 4 %

Figure 11

Fig. 11. Schematic of the rehabilitation schemes (not scaled)

Figure 12

Fig. 12. Final failure modes of Specimen RE1A

Figure 13

(a)

(b)

Fig. 13. Final failure modes of Specimen RE1B

Figure 14

Fig. 14. Final failure modes of Specimen RC1A

Figure 15

Fig. 15. Final failure modes of Specimen RC1B

Figure 16

Fig. 16. Schematically determining the theoretical lateral strength of the joint

Figure 17

Figure 17-conti.

Fig. 17. Load-displacement hysteretic loops for control and repaired specimens

Figure 18

Fig. 18. Comparison of envelope of hysteretic loops of control and repaired specimens

Figure 19

Fig. 19. Comparison of energy dissipation capacity of control and repaired specimens

Figure 20

Fig. 20. Comparison of stiffness degradation of control and repaired specimens

Figure 21

Fig. 21. Comparison of the damage index factors of the control specimen with the corresponding repaired specimens