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Seismic Performance of Strengthened Reinforced Concrete Beam-Column Joints Using FRP Composites

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Abstract:

Three nonseismically detailed interior reinforced concrete beam-column joints, namely, one eccentric and two concentric joints, strengthened with proposed fiber-reinforced polymer (FRP) wrapping configurations using glass fiber-reinforced polymer and carbon fiber-reinforced polymer strips and sheets, were tested under constant axial compression load and reversed cyclic loading which simulated low to moderate earthquake forces. Seismic performance of the strengthened beam-column joints in terms of their hysteresis response, stiffness, and energy dissipation capacity is evaluated and compared to those of the original and unstrengthened beam-column joints. Results indicate that applying strips at 45° on a flushed eccentric joint core and as cross bracing on the beam and confinement round the column is very effective. All specimens failed with gradual strength deterioration, bond degradation, and debonding of FRP sheets was observed near the joint core. The proposed strengthening schemes were found to be efficient and economical for mass repair or upgrading of nonseismically detailed structures.

CE Database subject heading:

Reinforced concrete; Beam columns; Joints; Seismic effects; Fiber reinforced polymers

Introduction

In low to moderate seismicity regions, such as Singapore, which has a peak ground acceleration of about 0.1 g with a 10% probability of exceedance in 50 years and many other parts of Asia and Europe, engineers do not normally include seismic considerations in building design and detailing. The structures have to rely on its inherent ductility to respond to acceptably to unexpectedly seismic excitations. As such, existing buildings are vulnerable to damage and collapse in an earthquake. Postearthquake investigations show that beam-column joints are the next weakest link after columns. With insufficient transverse reinforcement, discontinuous beam bottom reinforcement or other nonductile detailing (Engindeniz et al. 2005), beam-column joints and the adjacent framing members are susceptible to significant damage with ensuring reduction in strength and ductility. This has provided significant impetus in the research field of understanding the seismic performance of beam-column joints repaired and strengthened with various rehabilitation methods.

Recently, the use of fiber-reinforced polymer (FRP) composite materials in construction industry has greatly increased. This is primarily due to their high strength-to-weight ratios, corrosion resistance, ease of application and tailorability; fiber orientation in each ply can be adjusted to meet specific strengthening objectives (Engindeniz et al. 2005). By reviewing the previous studies by Gergely et al. (1998), Clyde and Pantelides (2003), Ghobarah and Said (2002), El-Amoury and Ghobarah (2002), Tsonos and Stylianidis (2002), Karayannis and Sirkelis (2002), and Antonopoulos and Triantafillou (2003) on the repair and strengthening of non-seismically designed RC beam-column joints with FRP composites, the following were found: externally bonded FRP composites eliminate limitations at site and the need to increase member sizes while at the same time improve joint shear capacity and shift the failure toward ductile beam hinging mechanisms. This was done by placing the fibers in $\pm 45^\circ$ directions in the joint region and wrapping the member ends to clamp the $\pm 45^\circ$ sheets and increase confinement. Lateral load capacity, joint shear strength and energy dissipation capacity have increased considerably. Most studies have shown that the behavior is dominated by the debonding of FRP composites from the concrete surface, and indicated the need for a thorough surface preparation and reliable mechanical anchorage for effective joint confinement and full development of fiber strength (Engindeniz et al. 2005).

It is noted that the beneficial effects of FRP composites on seismic behavior of nonseismically detailed special interior beam-column joints were not studied in the previous research. An experiment has been undertaken at Nanyang Technological University, Singapore, to better understand the effects of different configurations of glass fiber-reinforced polymer (GFRP) and carbon fiber-reinforced polymer strip (CFRP) strips and sheets on strengthening nonseismically detailed beam-column joints subjected seismic loadings.

Considering the increasing number of buildings severely damaged due to joint failures, RC beam-column joints require seismic strengthening and the need for economical techniques for upgrading. This paper describes the proposed strengthening schemes as an effective and economical alternative. The investigation results will be useful in developing effective and economical techniques to enhance the performance of such type of beam-column joints. Furthermore, the utilization of fiber composites in this study provides an insight into different wrapping configurations in mass repair or upgrading of nonseismically designed structures in low to moderate seismic regions where the seismic performance of rehabilitation of such structures is not widely known.

The first part of the paper briefly presents the seismic behavior of three typically as-built nonseismically RC beam-column joints subjected to constant axial compression load and reversed cyclic loading simulating low to moderate earthquake forces. Based on studying the failure modes of these specimens, different FRP strengthening schemes were proposed. The second part of the paper examines the strengthening schemes through the experimental studies. Three strengthened interior beam-column joints having identical detailing as the as-built beam-column joints were tested under similar loading conditions.

Experimental Program

Specimens and Test Setup

Six full-scale nonseismically detailed interior beam-column joints designed based on the code of BS 8110 (BS 1997) were constructed and tested. These specimens were typical as-built joints abstracted from the existing buildings in Singapore. Specimen E1C, Specimen C1C, and Specimen C2C were previously tested. Their failure modes were studied before strengthening Specimen SE1C, Specimen SC1C, and Specimen SC2C. The E1 series have an eccentric interior beam-column joint with beam flushed with the column edge, whereas the

C1 and C2 series are concentric interior beam-column joint with their respective centerlines of beams and columns intersecting. Note that C2 series are beamwide-column joints. Figs. 1 and 2 illustrate the schematic dimensions of all the specimens. Both E1 and C1 series have a column to beam width ratio about 3.56 while that of C2 series is about 7. The E1 and C1 series have a cross section dimension of 820×280 mm and a beam cross section dimension of 300×230 mm. The C2 series have a cross section dimension of $1,600 \times 300$ mm and a beam cross section dimension of 600×230 mm. Table 1 summarizes the details of the specimens. Table 2 shows steel reinforcement strength. Table 3 shows the concrete compressive strength of each specimen.

A schematic of the experimental setup is shown in Fig. 3. Each specimen was subjected to constant axial compression load and reversed cyclic loading that simulated low to moderate earthquake forces. The axial compression load of $0.35f'_cA_g$ was applied using small hydraulic jacks placed between column top end and bottom suffix of the steel transfer beam. Threaded rods were fixed around the test unit to balance the applied column axial load. A reversible horizontal load was applied in a quasistatic fashion at the top of the column through a double-acting 1,000 kN capacity long-stroke dynamic actuator mounted on the reaction wall. It was pinned at the end to allow rotation during the test. The actuator was manually operated to have a better control on the load increment. The column was pinned to a strong floor and the beam ends were connected to this strong floor by steel links which allow rotation and free horizontal movement of the beams. No vertical movement was allowed.

Materials

The specimens were built with identical reinforcement and cast with concrete grade G20. High deformed reinforcement bars Y10, Y13, Y20, Y22, Y25, and Y28 were used as main bars in the test specimens whereas mild steel bar R10 was used as stirrups.

Test Procedure and Instrumentation

Before the start of each test, the column axial load was slowly applied to the column and was balanced in steps until the designated level $0.35f'_cA_g$ was achieved. During each test, the column axial load of $0.35f'_cA_g$ was maintained by manually adjusting the flat jacks after each

load step. The lateral load was applied cyclically through the dynamic actuator in a quasistatic fashion at the top end of the column. The typical loading procedure consisting of displacement controlled steps is illustrated in Fig. 4.

The test results were explained qualitatively by using instrumentations such as dynamic actuator, strain gauges, linear variable displacement transducer (LVDT) and displacement transducers which were installed in the test setup. The horizontal displacement at the top column face was measured using a LVDT with 300-mm travel. The LVDT and displacement transducers were installed to observe the behavior of joint core area, beam, and column. Local strains in the reinforcing bars were measured using electric resistance wire strain gauges (TML FLA-5-11-5LT) which were installed on the bars before casting of specimens.

Strengthening Techniques

Seismic Behavior and Failure Modes of Specimen E1C, Specimen C1C, and Specimen C2C

Specimen E1C

Initial cracks formed at beam top during drift ratio of 0.4% and cracks propagated rapidly when drift ratio of 1.0% was attained. It is noteworthy that crack in joint core area was first found at drift ratio of 1.0%. Joint core cracks were only observed at the flushed surface of the beam-column joint while no cracks were found on the protruded joint at the opposite face. More cracks formed further away from the joint core area, while limited new cracks were found at the beam bottom after drift ratio of 2.0%. Specimen E1C deteriorated after drift ratio of 3.0% with crack patterns shown in Fig. 5. As the drift ratio increased, more cracks were found to propagate rapidly at beam top as well as joint core. Generally, no cracks were found in column and no spalling of concrete was observed throughout the test. It was declared failure and test was halted when a drift ratio of 4.0% was attained.

Specimen C1C

When a drift ratio of 1.33% was attained, cracks were observed to propagate rapidly at both beam top and bottom. Most of the crack damage was concentrated in the beams near the column. After drift ratio of 2.0%, the specimen sustained additional horizontal load with good energy dissipation. The largest flexural cracks occurred at the interfaces of the beam

ends. By the end of the test, these cracks were excessive, and the beam flexural bars were observed to have slipped through the joint due to loss of bond. This attributed to the gradual strength deterioration and low attainment of structural stiffness of the specimen during the drift ratios of 3.0 and 4.0%. The specimen began to deteriorate with a significant decrease in lateral resisting capacity after a drift ratio of 3.0% was attained. During then, cracks at beam propagated rapidly. Fig. 5 shows its crack patterns at drift ratio of 3%. No cracks were observed in column and joint core area when the test was completed.

Specimen C2C

When drift ratio of 0.67% was exceeded, diagonal flexural cracks were found at beam bottom. Limited new cracks were observed at beam bottom whereas more cracks were formed in joint core area after drift ratio of 1.33% was attained. During then, flexural cracks on the beams were also found to propagate rapidly. More new cracks at beam top were found when the drift ratio was increased to 2.0 and 3.0%, respectively. Severe punching cracking was observed on the side face of the column. These cracks, which were formed mainly at the lower portion of column, were caused by the fixed end moment of beams which rotated about the long column. The shear cracks at the column front face were believed to be the extension of these cracks formed at column. At a drift ratio of 4.0%, crushing of concrete at the fixed end of the beams was observed. Fig. 5 shows its crack patterns at drift ratio of 3%.

Proposed Strengthening Schemes

The strengthening schemes proposed for Specimen SE1C, Specimen SC1C, and Specimen SC2C were based on the failure modes of the damaged specimens, which were designed based on BS 8110 [British Standard (BS) 1997] where column longitudinal bars were just lap spliced above the floor level, beam bottom bars were lap spliced within the joint, and no transverse reinforcement was provided within the joint core. Fig. 6 illustrates the proposed FRP strengthening schemes for each of the specimens. Before the application of FRP sheets and strips, the specimens were carefully prepared by grinding of several areas to achieve a fully smooth surface and rounding of the corners at a radius of about 20 mm to avoid FRP cracks due to local failure and stress concentration (Columb et al. 2008). The wet lay-up FRP application was employed. It involved the use of epoxy resin for bonding and impregnation of the FRP sheets and strips. Putty was applied to prevent debonding due to unevenness. The following describes the procedures for the FRP strengthening schemes proposed.

To upgrade the joint in all specimens, one layer of GFRP L-wrap at each of the four corners of the joint was applied; at each corner, the GFRP strip was bent at 90° and thereafter extended 500 mm along the beam length and 400, 200, and 500 mm along the column height for Specimen SE1C, Specimen SC1C, and Specimen SC2C, respectively. Due to the severe punching cracks on the side face of the column of Specimen C2C, a relatively longer extension of GFRP strip along the column height of 500 mm was applied on Specimen SC2C compared to Specimen SC1C. Though Specimen E1C did not have column cracks, the 400-mm GFRP strip extension along column height in Specimen SE1C served mainly to upgrade its joint. The fibers were along the axes of the members.

To better bond the GFRP L-wrap to the column and enhance the confinement of the column. A GFRP sheet was wrapped round the column of Specimen SE1C and Specimen SC1C. CFRP strips (50-mm width each) were wrapped round the column at 50-mm intervals for Specimen SC2C. CFRP, having relatively higher ultimate tensile strength than GFRP, is used in strips to prevent excessive strengthening and to assess if such economical use of CFRP can improve the performance of the beam-column joint considerably. The direction of the confining fibers was perpendicular to the axis of the column.

For Specimen SE1C, two layers of CFRP strips (100-mm width each) were applied at the flushed surface; first layer was aligned at $+45^\circ$ while the second layer was aligned at -45° . This was similarly applied at the protruded side, at 500 mm along the beam face from the beam-column interface. Shear strengthening of the beam and joint is enhanced, eliminating or delaying the possibility of shear failure at the joint, especially at the flushed area. This will create the opportunity for a ductile plastic flexural hinging in the beam to occur (Ghobarah and Said 2002). As most of the crack damage was concentrated at the beam near the column of Specimen C1C, one layer of CRFP U-wrap extending 500 mm from beam-column interface was applied to cover the beam bottom of Specimen SC1C to increase shear strength (Construction Innovation 2002). This also prevents lap splices in the beam to slip. To serve the same purpose, CFRP strips (100-mm width each) formed cross bracings at both sides of the beam and column faces were applied on Specimen SC2C. This is relatively more economical than U-wrapping when strengthening a beam-wide-column joint is concerned.

To ensure better bond, one layer of continuous GFRP in the direction parallel to the beam axis was applied starting from 1,000-mm distance away from the beam-column interface on the following: across the two layers of CFRP strips at the flushed surface of Specimen SE1C, across the CFRP U-wrapping of beam bottom of Specimen SC1C, and across CFRP cross bracings on beam and column faces of Specimen SC2C. This increases the flexural strength; the fibers were along the axis of the beam. Finally, one layer of GFRP U-wrap with direction of fibers perpendicular to beam axis was applied to cover the bottom of the beams to increase shear strength (Construction Innovation 2002) and confinement of GFRP and CFRP strips and sheets of Specimen SE1C and Specimen SC1C.

In all specimens, the laminates terminated at 75 mm from the top of the beams to account for the presence of the floor system. Fiber anchors were placed at various locations along the development length of the sheets. This allowed the fibers to develop their full capacity and prevent premature delamination of the FRP wrap. In all specimens, the anchors were placed at the top of the beams and on both sides of the beam faces. In particular for Specimen SC2C, anchors were placed at the center and corners of each cross bracing. Fig. 6 also shows the locations of the anchors. The anchor consisted of a bundle of main fibers properly formed to size, and fully impregnated with epoxy before insertion to the drilled holes in the wall. The drilled holes were cleaned and kept free of dust. The embedded length of the anchor from the surface of the wall had a minimum length of 50 mm.

After the application and hardening of the epoxy resin binder, the GFRP sheets had a Young's modulus in tension equal to 20.9 GPa, ultimate tensile strength of 460 MPa, and ultimate strain of 2.2%. The ultimate tensile strength 90° to the primary fiber was 25.8 MPa and the laminate thickness around 1.3 mm. CFRP sheets had a Young's modulus in tension equal to 82 GPa, ultimate tensile strength of 834 MPa, and ultimate strain of 0.85%. The laminate thickness was around 1.0 mm. The CFRP strip used had a Young's modulus in tension equal to 139 GPa, ultimate tensile strength of 2.51 GPa, and an ultimate strain of 1.8%.

Results and Discussion

Failure modes and Response under Cyclic Loading

For all specimens, the failure modes and response under cyclic loading were generally similar. The final failure in all specimens involved the debonding of FRP sheets at the beam-column interface. Delamination of FRP strips on beam near the beam-column interface joint only occurred in Specimen SC1C and Specimen SC2C. There were no cracks on the column for Specimen SE1C and Specimen SC1C. Anchorage failure was only observed near the beam-column interface at beam top of Specimen SC2C.

During the first few cycles of loading, hairline cracking distributed along the beam, developed in all specimens; the first crack initiated at drift ratios: 0.20, 0.67, and 0.10% for Specimen SE1C, Specimen SC1C, and Specimen SC2C, respectively. Most cracks developed were approximately constant inclination, due to diagonal tension with few cracks with variable inclination. At this stage, the slope of the hysteresis loops (see Fig. 7) was steep, indicating a tremendous increase in strength. More flexural and shear cracks on the beams propagated as drift ratio increased. Fig. 8 shows the final crack patterns of the strengthened specimens. It was observed that both Specimen SE1C and Specimen SC1C had more cracks at the top beam compared to bottom beam. This is due to the smaller bars at the top beam resisting the tension during the reversed cyclic loading.

As the imposed displacement increased, widening of cracks was accompanied by audible sounds from the breaking resin, indicating initiation of the epoxy resin failure near the joint region where crack marks on the epoxy resin were observed. Significant widening of cracks was observed at beam top at drift ratio of 1.33, 3, and 2% for Specimen SE1C, Specimen SC1C, and Specimen SC2C, respectively. In particular, such cracks were apparent near the joint core corners for Specimen SE1C. It was clear evidence that most inelastic behavior occurred here. Also, it indicated advanced deformation of the yielded steel. At this stage, the slope of the hysteresis loops (see Fig. 7), was nearly horizontal, indicating that the peak strength of the beam-column joints was reached. However, for Specimen SC2C, the slope of the hysteresis loops was still increasing, though at a slower rate.

Toward the last few cycles of loading, rapid continuous audible sounds from the breaking of resin and debonding of FRP sheets were heard. The slope of the hysteresis loops had a negative slope for Specimen SC1C, indicating that strength of specimens was deteriorating. It was not apparent for Specimen SE1C, where the slope of the hysteresis loops was nearly horizontal; suggesting that after drift ratio 4%, the strength of the specimen might decrease.

Also, compared to the unstrengthened Specimen E1C, there were no severe pinching for Specimen SE1C after drift ratio 3%; implying that there was good energy dissipation at the beam ends. This phenomenon was similar for Specimen SC1C and Specimen SC2C. Final failure occurred due to fracture of epoxy resin and debonding of FRP sheets near the joint region in all specimens (refer to Fig. 9). Debonding of FRP sheets was justified with its lighter shade and hollow echo upon knocking. The debonding observed on the column face near the beam-column interface could be due to the weakening of FRP sheets due to the discontinuous FRP sheets terminated at the beam-column interface to account for the presence of floor system.

Slight delamination was observed at the following: the edge of FRP sheets at the flushed surface of Specimen SE1C, beam top near beam-column interface for Specimen SC1C, and edge of FRP sheet at the right beam face nearest to the beam pinned end. More severe FRP delamination was observed at top and bottom beam near beam-column interface for Specimen SC2C (refer to Fig. 9), indicating that FRP U-wrapping was an important technique to strengthen beam which was absent in Specimen SC2C. The initiation of delamination occurred when the fiber lost its compressive load-carrying capacity when the concrete was in compression. Few crack lines were observed at the intervals of CFRP strips of its column. There was a slight delamination of the first confining CFRP strip on the column near the beam bottom. Despite so, the slope of hysteresis loops for Specimen SC2C continued to increase after drift ratio 3%, which suggested that it was capable of higher strength and the FRP strips and sheets were not fully used. Visual methodology could not be applied to joint as it was covered with FRP sheets. However, no rupture of FRP sheets and hairline cracks on the epoxy resin at the joint corners of all specimens were indicative of low level of shear in this area. The strength of the strengthened specimens was definitely higher than the corresponded unstrengthened specimens (see Table 4), hence, in this sense, the strengthening methods used could be deemed successful.

The ductility of all strengthened specimens has improved; this was measured by the comparison of the percentage of fall in strength at each drift ratio. The strength of Specimen E1C started to fall by 8.6 and 15.5% at drift ratio of 3.0 and 4.0%, respectively. For Specimen SC1C, its strength only started to fall by 4.8% at drift ratio of 4.0%. Similar to Specimen E1C, Specimen C1C started to fall by 11.6 and 17.5% at drift ratio of 3.0 and 4.0%, respectively. For Specimen SC1C, its strength started to fall by 15.5% at drift ratio of 4.0%. It was observed

the percentage fall in the strength of the strengthened specimens was less than that of the as-built specimens at each drift ratio. For Specimen C2C, a 9.4% fall in strength was observed at drift ratio of 4.0%. At this stage, there was no fall in strength for Specimen SC2C.

Energy Dissipation Capacity and Stiffness Degradation

Fig. 10 displays the energy dissipation capacity versus drift ratio relationship for as-built and strengthened specimens. All strengthened specimens showed very good energy dissipation characteristics with almost doubled that of the as-built specimens. Toward the last few cycles, the energy gradients are relatively steeper due to the widening of cracks observed on the beams and the debonding of FRP sheets near the beam-column interface. The energy gradients for Specimen SE1C and Specimen SC1C [see Figs. 10(a and b)] during the last two cycles did not have significant change. However, it was noted that for Specimen SC2C [see Fig. 10(c)], the energy gradient during drift ratio of 4% was higher than that during drift ratio 3%, indicating it may be capable of dissipating more energy after attaining drift ratio of 4%. This could be due to the multidiffuse cracking between the bands of CFRP strips round the column which permits considerable energy dissipation.

With the increase in displacement and the number of cycles, the hysteresis loops tend to be inclined. Fig. 11 shows the stiffness versus drift ratio relationship for as-built and strengthened specimens. In all strengthened specimens, it was observed that there was a tremendous increase in stiffness during the first few cycles. After which, there was a gradual decrease in stiffness for the subsequent cycles. This explains the phenomenon of concrete cracking, steel yielding and failure of steel-concrete adherence taking place. The loss of stiffness may be primarily attributed to concrete deterioration in the beam-column joint region. As the concrete degrades, the load on the FRP strips and sheets increases; this explains the relatively linear curve in the subsequent cycles. It is noteworthy that the increase in stiffness at every drift ratio for Specimen SE1C [see Fig. 11(a)] was significant; suggesting that the two layers of $\pm 45^\circ$ CFRP strips at the joint and beam faces had contributed to the stiffness. On the other hand, there was severe stiffness degradation until drift ratio of 1% for Specimen SC1C and Specimen SC2C [Figs. 11(b and c)]. Toward the last cycle, the stiffness for all specimens is fairly low. This is due to the cracks, which are caused by the yielding of the beam longitudinal bars, remaining unclosed.

Distribution of Strains in Longitudinal Reinforcement of Beam

Fig. 12 illustrates the strain profiles of the beam top longitudinal reinforcement at the peak lateral load of each drift ratio for the original and strengthened specimens. These strain profiles represent the local strains along the length of beam top. The strains are directly obtained from the readings measured by electrical strain gauges. In all specimens, distributions of strain along the reinforcement vary considerably with the increase in lateral load. When the longitudinal reinforcement was in compression, the compressive strains measured were initially small since most of the compression was transferred through the concrete. However, as the loading progressed toward the inelastic range, the strains measured are in positive values, indicating that portion was subjected to tension.

For both Specimen E1C and Specimen SE1C, the first yield of longitudinal reinforcement occurred on beam-column interface at drift ratio 1/75 and 1/50, respectively. Similarly, the largest tensile strain was detected on the same location at drift ratio 1/50 and 1/33 for Specimen E1C and Specimen SE1C, respectively; however, as the drift ratio increased, the recorded maximum tensile strain at the same position was much smaller. This could be attributed to severe cracking and shearing of concrete in the plastic hinge region which had caused the loss of bond strength between the reinforcement and the concrete, resulting in bar slippage. Degradation of anchorage resistance in the reinforcement passing through the joint occurred at drift ratio 1/50 for Specimen E1C; Specimen SE1C displayed good anchorage resistance up to drift ratio 1/25. Strains within the beam-column joint region have never exceeded the elastic range, implying that the plastic hinge had been successfully confined near beam end. The application of two layers of CFRP strips aligned at $\pm 45^\circ$ on both sides of beam faces had delayed the yielding of longitudinal reinforcement.

The first yield of longitudinal reinforcement for Specimen C1C occurred within the joint region at drift ratio 1/50, whereas that for strengthened Specimen SC1C occurred on the beam-column interface at drift ratio 1/75. The largest tensile strain was similarly detected at the same corresponding locations for Specimen C1C and Specimen SC1C at the same drift ratio of 1/33; however, the strengthened specimen had relatively much smaller largest tensile strain value compared to that of the original specimen. It was further noted that all strains in the inelastic range were observed at the beam-column interface in Specimen SC1C; however, that of Specimen C1C were observed within the joint. This implied that the plastic

hinges of SC1C were successfully confined to the beam end. Degradation of anchorage resistance in the reinforcement passing through the joint occurred at drift ratios 1/150 and 1/75 for Specimen C1C and Specimen SC1C, respectively.

The first yield of longitudinal reinforcement for Specimen C2C occurred within the joint region at drift ratio 1/50, whereas that for strengthened Specimen SC2C occurred outside the joint area near the beam-column interface at drift ratio 1/33. At drift ratio 1/25, the largest tensile strain was similarly detected at the same location for Specimen SC2C; however, for Specimen C2C, it was located at beam-column interface. It could be concluded that the plastic hinges of C2C and SC2C were successfully confined to the beam end. Note that two strain gauges were not functional at the extreme left of Specimen SC2C. They could have been damaged during casting. Specimen C2C and Specimen SC2C still displayed a good anchorage resistance characteristic up to a drift ratio 1/33 as explained in Fig. 12(c). This shows that the column depth was not deep enough for beam bars to develop the full anchorage.

Bond deterioration occurred along the beam bars of all specimens at a drift ratio of 4.0%. Bond condition is determined mainly by the ratio of the beam and column bar diameter to the column and beam depths. The ratio of beam bar diameter to the column depth of all specimens $d_b/h_c=1/22$ does not satisfy the requirement given by NZS 3101 (Standard Association of New Zealand 1982). Hence, it is not surprising that bond deterioration occurred along the beam bars at the drift ratio of 4%.

With reference to these strain profiles in Fig. 12, yielding of the longitudinal bars spread over a distance of approximately $1.4d$ from the column face for all specimens, thus indicating the concentration of plastic hinges in the vicinity of this region. This indicated that the proposed FRP wrapping schemes were effective in confining the plastic hinges in the vicinity of this region. Anchoring the GFRP strip near the beam-column interface at beam top added to the effectiveness.

Distribution of Strains in Reinforcement along Column Height

Fig. 13 illustrates the local strains of the longitudinal reinforcement along the column height for original and strengthened specimens. For all specimens, the column strains were in the elastic range when the beam reached its flexural strength, indicating a “strong column-weak

beam” response. Fig. 13(a) shows that Specimen SE1C had relatively smaller strains compared to Specimen E1C. However, there was not much change for Specimen SC1C compared to Specimen C1C, as shown in Fig. 13(b). Note that the negative region in Fig. 13(b) has no readings as two strain gauges for Specimen SC1C were not functional during the test. They could have been damaged during casting of specimen. The small strains observed implied that there were little flexural cracks, in particular lesser in Specimen SE1C compared to the original specimen, at the column. Confining the columns with GFRP sheet at 400 and 200 mm along the column height from the beam top for Specimen SE1C and Specimen SC1C, respectively, was sufficient and effective.

For Specimen SC2C, the column bar strains were comparatively similar compared to Specimen C2C; however, it had considerably reduced strains at its bottom column compared to the original specimen. In column longitudinal reinforcement passing through beam-column joints, bond stress is imposed due to change in column moment over joint depth. If no slippage of column longitudinal reinforcement occurs, the column bar strains should change from tension on one side of the joint and compression on the other side. Slippage of column bars occurred at drift ratios $1/33$ and $1/50$ at bottom column for Specimen SC2C and Specimen C2C, respectively. However, the slippage was found only within 200 mm along the column height from beam bottom for Specimen SC2C. This implied that using CFRP strips for confining the column was effective and economical.

Decomposition of Interstory Drift

The total interstory drift recorded at the top of the column consisted of several components, comprising lateral displacements due to the beam flexure, beam shear, column flexure deformations, and joint shear distortion. Measurements by LVDTs mounted on the specimens were used to derive the different deformations using the procedures described by Wu (2001). In general, the total calculated lateral displacements due to the contributing components were less than the measured interstory drift. The uncounted lateral displacement could mainly be attributed to the rigid body rotation, which was unable to be captured during the test.

Fig. 14 illustrates the displacement decomposition versus drift ratio relationship for original and strengthened specimens. A decrease in contribution to total drift from beam displacement

was observed in all strengthened specimens. The FRP sheets and strips applied on the beam faces could have increase the stiffness of the beams. Furthermore, the FRP sheets and strips were well anchored near the beam-column joint interface, adding to the stiffness of the beams. To ensure equilibrium, a larger amount of load was imposed on the columns, resulting in an increase in contribution to total drift from column flexure observed in all strengthened specimens as compared to the unstrengthened ones.

For both Specimen E1C and Specimen SE1C, the major source of the story drift was beam displacement [see Fig. 14(a)], indicating a strong-column weak-beam response. However, the contributions to the total drift from beam flexure had decreased considerably; it varied from 57.7 to 95.6% in Specimen E1C and 30 to 38% in Specimen SE1C. The contributions to the total drift from beam flexure, beam shear, column flexure, and joint shear at drift ratio of 4% for Specimen SE1C were 30, 22, 5, and 15%, respectively. The contribution to the total drift from column flexure did not change significantly during the testing while that from joint shear distortion increased gradually.

Similar to Specimen SE1C, the same trend was also observed in Fig. 14(b). The contribution to the total drift from beam flexure had decreased tremendously for Specimen SC1C; it varied from 51.6 to 79.2% in Specimen C1C and from 1.5 to 13% in Specimen SC1C. Column flexure deformation was predominant instead in the strengthened specimen; it varied from 2 to 5% in Specimen C1C and from 27 to 36% in Specimen SC1C. The contributions to the total drift from beam flexure, beam shear, column flexure, and joint shear were 1.5–13%, 0.4–6%, 27–36%, and 0–1% respectively. Both Specimen C1C and Specimen SC1C had insignificant contribution to total drift from joint shear distortion. Similar to Specimen SE1C, the contribution to the total drift from column flexure for SC1C did not change significantly during the testing.

For Specimen SC2C, according to Fig. 14(c), the contribution of beam flexure had decreased slightly compared to the unstrengthened specimen; it varied from 23.2 to 36.1% in Specimen C2C and from 16 to 28% in Specimen SC2C. The contributions to the total drift from beam flexure and column flexure deformation were predominant in Specimen SC2C. The contribution from column flexure was relatively higher for Specimen SC2C than C2C. The gradual increase in the contribution to total drift from column flexure was supported by the increase in multidiffuse cracking between bands of CFRP strips confining the column.

The contribution to the total drift from beam flexure, beam shear, column flexure, and joint shear at drift ratio of 4% were 28, 5, 34, and 1%, respectively. Due to the three dimensional nature of the specimen, the transducers placed diagonally in the joint panel especially for the C1 and C2 series, the joint shear deformation could not be captured.

Conclusions

From the results of the experimental program, effective and economical FRP strengthening schemes are developed for existing nonseismically detailed interior RC beam-column joints. A comparison between the performance of original specimens and strengthened ones shows a tremendous increase in strength, stiffness and energy dissipation capacity. This is attributed to the following reasons: The use of two layers of $\pm 45^\circ$ CFRP strips at the joint and beam area was effective in strengthening eccentric joint. The use of CFRP strips on strong-column weak-beam is effective in flexural strength: CFRP strips round the column and CFRP strips as cross bracings on the beam and column face. Good anchorage in the form of fiber anchors is effective in anchoring FRP sheets and strips; they contributed much to the shear strengthening of beam-column joint and beam. Such anchorages also ease constructability on site.

To develop the strength of the fiber, it is recommended that the anchorages be installed at beam bottom near the beam-column interface and at the edges of FRP strips and sheets near the joint core. On the other hand, it is noted that beam jacketing in the form of FRP U-wrapping is necessary in preventing shear failure in the beam and allowed a flexural hinge to develop.

The proposed strengthening schemes were successful in eliminating or delaying the shear mode of failure. Instead, flexural hinging of the beam, a ductile mode of failure, occurred in the form of cracks near the joint core corners for Specimen SE1C and delamination of FRP strips at beam-column interface near joint core region for Specimen SC1C and Specimen SC2C. Since the behavior of beam-column joints is complex and still not completely understood. Thus, adopting a direct extension of the FRP strengthening strategies for beams and columns on beam-column joints would be difficult. The proposed strengthening schemes will help develop FRP strengthening strategy for beam-column joints and that are potentially

efficient for mass repair or upgrading of structures not suitably designed to withstand earthquakes.

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Notation

The following symbols are used in this paper:

A_g = gross sectional area of column;

d = effective depth;

d_b = beam bar diameter;

h_c = column depth;

f'_c = concrete compressive strength;

f_y = steel strength at yielding; and

g = acceleration due to gravity.

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Specimen		E1C, C1C, SE1C, and SC1C	C2C and SC2C
Beam	Size	230 × 300	230 × 600
		2Y10 ($\rho'_t=0.23\%$) mid span	
	Top bar	2Y13 ($\rho'_t=0.38\%$) lap spliced within joint	3Y25 ($\rho'_t=1.07\%$)
	Bottom bar	2Y13 ($\rho'_t=0.38\%$) midspan and lap spliced within joint	3Y20 ($\rho'_t=0.68\%$)
Column	Stirrups	R10 at 230	R10 at 300
	Size	280 × 820	300 × 1600
	Main bar	4Y22+4Y25 ($\rho_c=1.52\%$)	24Y28 ($\rho_c=3.08\%$)
	Hoops	2R10 at 125	9R10 at 300
Joint	Hoops	None	None

Table 1

Reinforcement type	Bar size	f_y (MPa)
Beam longitudinal (E1C, C1C, SE1C, and SC1C)	10	510
	13	508
Beam longitudinal (C2C and SC2C)	20	513
	25	517
Column longitudinal (E1C, C1C, SE1C, and SC1C)	25	517
Column longitudinal (C2C and SC2C)	32	504
Stirrups	10	354

Table 2

As-built specimens	f'_c (MPa)	Strengthened specimens	f'_c (MPa)
E1C	18.4	SE1C	18.8
C1C	19.0	SC1C	18.5
C2C	20.0	SC2C	18.4

Table 3

Specimen	As-built		Specimen	Strengthened	
	Negative loading (kN)	Positive loading (kN)		Negative loading (kN)	Positive loading (kN)
E1C	-14.9	15.8	SE1C	-42.8	36.9
C1C	-15.5	15.5	SC1C	-30.6	31.8
C2C	-139	139	SC2C	-223	214

Table 4

Plan view of Specimens EIC
and SEIC

Fig. 1

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

Fig. 8

Specimen SE1C

Specimen SC1C

Specimen SC2C

Fig. 9

Fig. 10

Fig. 11

Specimen E1C

(a)

Specimen SE1C

Specimen C1C

(b)

Specimen SC1C

Specimen C2C

(c)

Specimen SC2C

Fig. 12

Specimen E1C

Specimen SE1C

Specimen C1C

Specimen SC1C

Specimen C2C

Specimen SC2C

Fig. 13

Fig. 14