SEISMIC REHABILITATION OF RC FRAME INTERIOR BEAM-COLUMN JOINTS WITH FRP COMPOSITES

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ABSTRACT :

An experimental research program is described regarding the use of externally applied Carbon Fiber Reinforced Polymer (CFRP) jackets for seismic rehabilitation of reinforced concrete interior beam-column joints, which were designed for gravity loads. The joints had steel reinforcement details that are known to be inadequate by current seismic codes in terms of joint shear capacity and bond capacity of beam bottom steel reinforcing bars at the joint. Lap splicing of beam bottom steel reinforcement at the joint using externally applied longitudinal CFRP composite laminates is investigated. Improvement of joint shear capacity using diagonal CFRP composite laminates is another strengthening scheme employed. The test results indicate that CFRP jackets are an effective rehabilitation measure for improving the seismic performance of existing beam-column joints with inadequate seismic details in terms of increased joint shear strength and inelastic rotation capacity. In addition, CFRP laminates are an effective rehabilitation measure for overcoming problems associated with beam bottom steel bars that have inadequate embedment into the beam-column joints.

KEYWORDS: Composites, Concrete structures, Experiments, Fiber reinforced polymers (FRP), Joints, Seismic rehabilitation.

1. INTRODUCTION

Reinforced concrete (RC) buildings constructed before the 1970’s where typically no steel hoops are provided in the beam-column joint region, and longitudinal bars have inadequate anchorage passing through or terminating in the joint have inadequate shear strength. Distress in beam-column joints leading to building collapse has been observed in past earthquakes (Moehle and Mahin 1991). The cause of collapse in certain cases has been attributed to inadequate joint confinement. The inadequacy of building joints designed according to earlier standards has been the cause for severe damage or collapse. This type of damage has been observed in many earthquakes. Shear failure or bond slip failure are considered undesirable since they lead to degradation of the strength and stiffness of the frame (Park and Paulay 1975). Failure of interior beam-column joints can also be initiated by pullout of the beam bottom steel reinforcement at the joint typically embedded for a short and insufficient length into the column. Beres et al. (1992) and Pantelides et al. (2002) have demonstrated the pullout failure mode for interior and exterior beam-column joints; typically, beam bottom steel reinforcement at the joint pulls out at stress levels below the yield stress.

Seismic rehabilitation, strengthening and repair of beam-column joints in existing RC structures includes epoxy repair, reinforced concrete (RC) jackets, steel jackets, and FRP jackets. Rehabilitation using FRP composites offers advantages such as fast and easy installation, high strength/weight ratio, and resistance to corrosion. Rehabilitation of RC beam-column joints using FRP jackets for improving joint shear strength has been studied by several researchers. Fewer studies address the issue of pullout failure of beam bottom steel bars at beam-column joint regions. In RC building frames, pullout failure at a joint can result when the beam bottom steel bars do not have adequate embedment into the column. A knee-joint was retrofitted with vertical steel U stirrups within the joint and additional diagonal reinforcement within the core, across the inside corner of the joint by Mazzoni et al. (1991); test results indicated that the retrofit measures were successful in improving the performance of the joint subjected to cyclic loads. Pantelides et al. (1999, 2007) successfully used an externally applied carbon FRP (CFRP) composite U-strap to postpone pullout of column longitudinal steel reinforcement extending into a bridge T-joint. Ghobarah and El-Amoury (2005) successfully used glass FRP jackets in the joint zone and steel rods or steel plates for exterior joints of RC frames.

In this research, strengthening of RC beam-column interior joints in building frames which are deficient under seismic loads either in joint shear, pullout failure of the beam bottom steel bars at the joint, or both is addressed using CFRP composite jackets. The column vertical steel ratio was 2% with lap splices located immediately above the floor level and column ties widely spaced. The objectives of the seismic rehabilitation were to prevent joint shear failure, postpone pullout failure of the beam bottom steel bars at the joint, and promote plastic hinge formation in the beams to increase inelastic rotation capacity.

2. EXPERIMENTAL PROGRAM

2.1. Experimental Program

The deflected shape of a plane frame from earthquake loading results in the deformation of the subassembly shown in Fig. 1(a). The orientation of the specimens in the experimental program, as shown in Fig. 1(b), is different in that the column ends were held in a vertical line for testing. Loads applied at the beam ends were measured with load cells attached in series with the two actuators that applied the quasi-static cyclic loads, as shown in Fig. 1(c). The top and bottom ends of the column were pinned supports. A constant axial load equivalent to 0.1 *Agf'c* was applied to the column through an actuator at the column bottom, as shown in Fig. 1(c), which was maintained constant during the tests. Beam tip displacements were measured using linear potentiometers. These measurements allowed the calculation of story shear *V*, and drift ratio δ , defined in the structure of Fig. 1(a) as $\delta = \Delta / h$, to the equivalent expressions for the subassemblage of Fig. 1(b) as:

$$V = \frac{(R_E + R_W)L}{2h}; \quad \delta = \frac{(\Delta_E + \Delta_W)}{L} \tag{1}$$

The tests were carried out using displacement control at increasing drift levels to failure. The two actuators were displaced by the same amount in opposite directions simultaneously, as shown in Fig. 1(b); the quasi-static application of displacement was repeated for three cycles for each drift ratio δ defined in Eq. (1). The actuator forces R_E and R_W corresponding to displacements Δ_E and Δ_W were used to calculate the story shear *V* as defined in Eq. (1). The applied displacements and actuator forces were applied slowly in a quasi-static fashion and were recorded at one second time-steps. Two types of beam-column joints were tested; Type I had a beam 406 mm wide and 610 mm deep as shown in Fig. 2(a), and Type II had a beam 406 mm wide and 406 mm deep as shown in Fig. 2(b). Both joint types had a column with dimensions of 406x406 mm. The reinforcement details are shown in Fig. 2; for both joint types, the beam bottom steel bars at the joint have an embedment of only 127 mm on each side from the face of the column, which creates unfavorable conditions for bond development of these bars. In addition, there is no horizontal hoop steel in the joint itself, which creates unfavorable conditions for developing the shear strength of the joint. The column vertical steel ratio was 2% with lap splices located immediately above the floor level in the zone of maximum seismic moment, along with widely spaced column ties. These details are typical of older standards, such as the *ACI Building Code Requirements for Reinforced Concrete 318-63* (ACI 1963).

2.2. Type I Joints: 610 mm (24 in.) Beam Depth

Two "as-built" (24-1 and 24-2) and two rehabilitated (24R-3 and 24R-4) beam-column Type I joints were tested in this research. According to the ACI 352 *Recommendations* (2002), the three Φ13 mm beam bottom steel bars shown in Section B-B of Fig. 2(a) have an embedment which is only 29% of the required development length for beam-column connections that are required to dissipate energy through reversals of deformation into the inelastic range; in addition, these bars have no hooks. To limit bar slippage within the joint, the ACI 352 *Recommendations* (2002) require that all straight beam bars passing through the joint should have a diameter smaller than 1/20 of the column depth and a similar requirement exists for column bars compared to the beam depth; in both cases the reinforcement details do not conform to this requirement. According to current codes regarding joint transverse reinforcement (ACI 352 2002), the center-to-center spacing between layers of horizontal hoop reinforcement, required for connections that are part of the primary system for resisting seismic lateral loads, should not exceed the least of ¼ of the minimum column dimension, six times the diameter of the

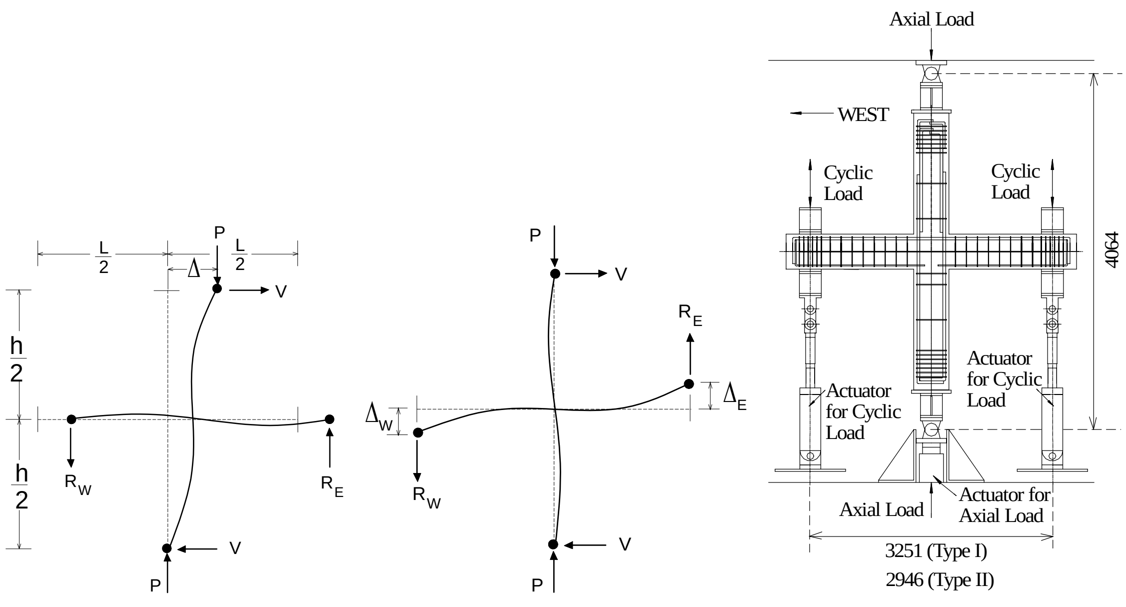


Figure 1. (a) Idealized deflected shape of beam-column joint, (b) joint tested in the laboratory, (c) test setup.

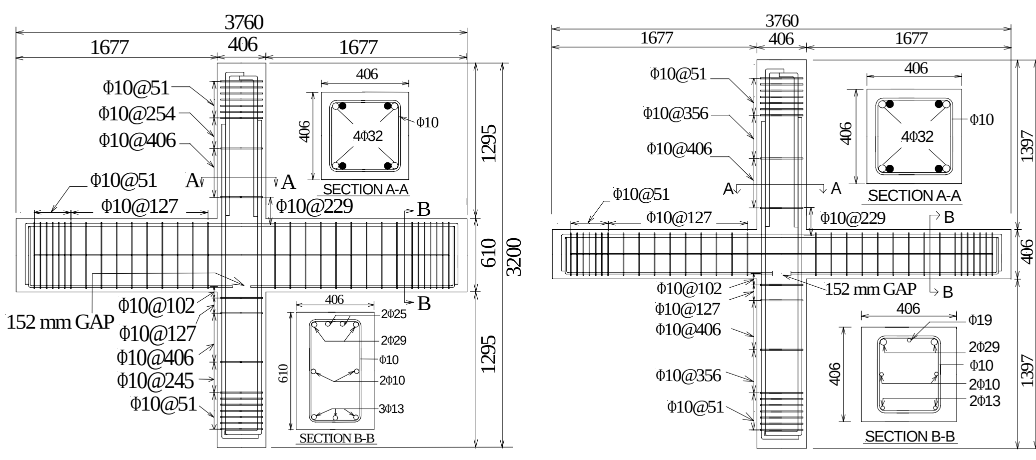


Figure 2. Specimen dimensions and reinforcement details: (a) Type I, (b) Type II.

longitudinal column bars to be restrained, or 150 mm; this requirement is not met since no horizontal hoop reinforcement was provided in the joint. The joint shear demand, based on the beam flexural capacity was 1.6 times the capacity of interior joints without transverse beams and no horizontal confinement reinforcement in the joint suggested in the *ASCE/SEI 41-06 Standard* (ASCE 2007). The dimensions and reinforcement details of the column and beam sections, under the column axial compressive load of 0.1 *Agf'c*, gave a ratio of the summation of flexural capacity of the columns to flexural capacity of the beams framing into the joint equal to 1.0, which is less than the 1.2 ratio required by current codes for strong column-weak beam behavior (ACI 318 2005).

2.3. Type II Joints: 406 mm (16 in.) Beam Depth

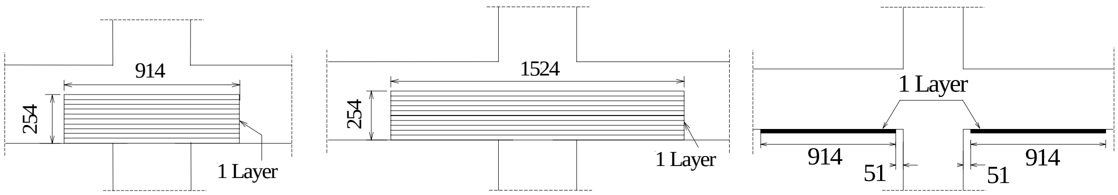
One "as-built" (16-1) and three rehabilitated (R16-2, R16-3, and R16-4) beam-column Type II joints were tested in this research. The Type II joints had a smaller beam depth (406 mm) compared to the 610 mm depth of the Type I joints as shown in Fig. 2(b); Type II joints have a smaller joint shear demand because of the smaller theoretical beam flexural capacity compared to Type I joints. Similar to Type I joints, the joints are deficient regarding the required development length of the beam bottom steel bars at the joint, and the size of the beam bars compared to the column depth. The joints are also deficient regarding the joint horizontal hoop reinforcement. For these beams, the joints conform to the strong-column weak-beam requirement and have a horizontal joint shear demand less than the capacity of interior joints without transverse beams reinforced with horizontal confinement reinforcement in the joint, as suggested in the ASCE/SEI 41-06 Standard (ASCE 2007).

3. CFRP COMPOSITE STRENGTHENING

CFRP composites from three manufacturers were used in the rehabilitation of the beam-column joints. They were applied using a wet layup and cured at ambient conditions. The surface preparation consisted of a high-pressure water jet and application of a structural epoxy adhesive. The goals of the CFRP composite seismic rehabilitation were to improve the story shear and inelastic rotation capacity of the joints under simulated seismic loads. The CFRP composite design was governed by the following criteria: (1) postpone pullout failure of beam bottom steel bars at the joint; (2) improve joint shear capacity; (3) protect the beam critical regions against shear failure, and (4) satisfy the strong-column weak-beam design criterion to a great extent and protect the column from shear failure. The above rehabilitation measures are intended to form plastic hinges in the beams and move any plastic hinges away from the joint area. The same details were implemented for both Type I and Type II joints.

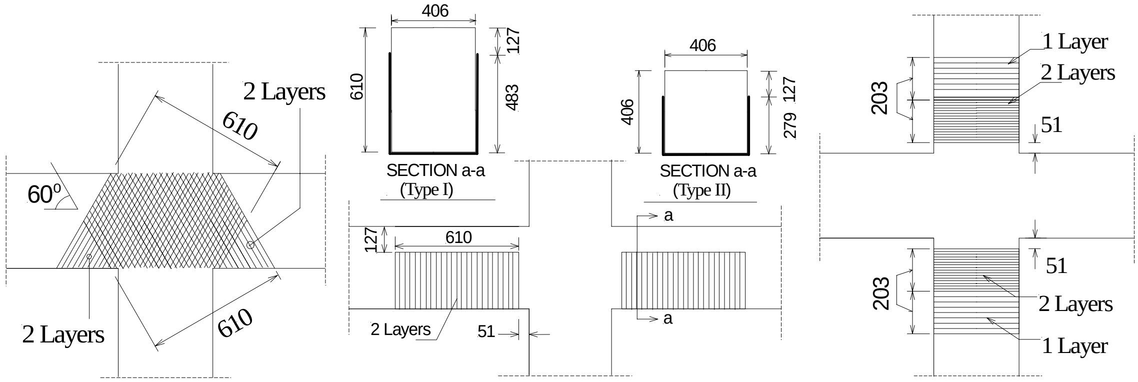
In order to improve the bond of beam bottom steel bars at the joint, and to limit bar slippage, an external CFRP "lap splice" technique was implemented. This consisted of applying two 254 mm-wide CFRP sheets on each of the unconfined faces in the lower portion of the joint. The first CFRP sheet applied was 914 mm long as shown in Fig. 3(a), and the second sheet was 1.52 m long as shown in Fig. 3(b); in both cases the fibers were aligned with the direction of the beam axis and were designed to carry the equivalent tension in the internal reinforcing bars as if they had been fully developed. Calculations showed that two CFRP layers would have been sufficient to carry this force in the beam bottom steel bars from one side of the joint to the other. The short 914 mm sheets were applied first to capture the immediate stresses transferred from the internal steel bars through the concrete, and the long 1.52 m sheets were applied to distribute these bond stresses over a wider participating concrete surface. From previous beam and slab tests conducted by the authors, it was observed that an FRP sheet applied at the bottom surface of the beam postpones the growth of flexural cracks that eventually become shear cracks. A CFRP composite sheet 406 mm wide and 914 mm long was placed on the bottom surface of each of the two beams near the joints, as shown in Fig. 3(c), to postpone the growth of flexure/shear cracks to a higher drift ratio.

The design of the CFRP composite rehabilitation for improving the joint shear capacity was determined by a procedure similar to that developed for T-joints in bridges (Pantelides et al. 1999, Gergely et al. 2000, Pantelides and Gergely 2002). The width and thickness of the CFRP composite fabric was designed to increase the capacity of the joint in the direction of principal tension from a value known to cause cracking. Two CFRP layers were placed at an angle of ±60° from the horizontal in the joint region, as shown in Fig. 4(a), on both faces of the joint to resist diagonal tensile stresses. To minimize the potential for shear failure from occurring in the beams of the rehabilitated subassembly, two layers of CFRP U-stirrups were used in the critical regions of the beams, as shown in Fig. 4(b). The required number of layers was found using established principles (ACI 440 2002) by ignoring the contribution of the concrete in the plastic hinge regions. The two layers were terminated 127 mm from the beam tops to account for the typical presence of a RC slab. No mechanical anchoring of the CFRP U-stirrups was used. The CFRP U-stirrups also served as anchorage of the CFRP lap splice sheets. A general principle in seismic retrofit is that intervention for strengthening one portion of the structure should not force an undesirable brittle failure. To satisfy the strong-column weak-beam design criterion, the columns required strengthening for shear enhancement and confinement. Following design principles for column shear strengthening and confinement, two layers were found to be sufficient as shown in Fig. 4(c); two layers were applied in the hoop direction for the first 203 mm and one layer for the subsequent 203 mm to distribute the stresses. The CFRP composite was terminated 51 mm from the beam face to allow independent rotation between the column and beams. The corners of all beams and columns were beveled to a 25 mm radius.



(a) (b) (c)

Figure 3. (a) CFRP “lap splice” short bond sheet, (b) long bond sheet, (c) CFRP sheets applied at the bottom surface of the beam.



(a) (b) (c)

Figure 4. (a) CFRP joint shear reinforcement, (b) CFRP beam shear reinforcement, (c) CFRP column confinement and shear reinforcement.

4. BEHAVIOR AND FAILURE MODES

The observed behavior of the eight beam-column joint specimens tested is described briefly. For Type I joints, two specimens (24-1 and 24-2) were in the as-built condition, and two (R24-3 and R24-4) were rehabilitated with CFRP composites. For Type II joints, one specimen (16-1) was in the as-built condition, and three (R16-2, R16-3, and R16-4) were rehabilitated with CFRP composites.

4.1. As-Built Type I Specimens

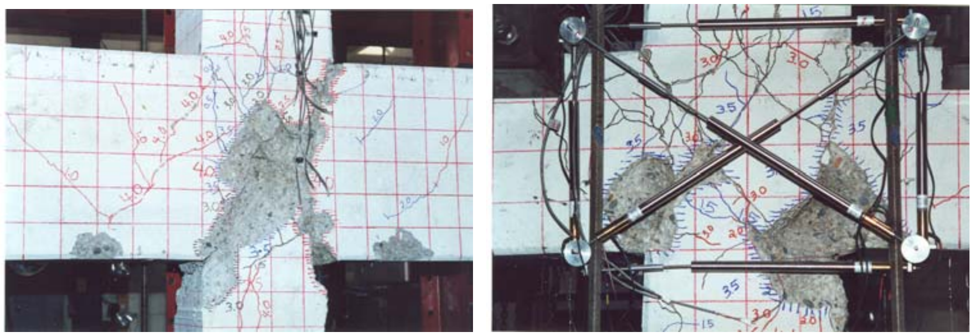
Specimens 24-1 and 24-2 both developed a significant diagonal concrete shear crack, which remained open after unloading, in the joint region with a width of 1.0 mm, at a drift ratio of 2.5%. A shear failure mechanism was observed with a main diagonal crack approximately 2.5 mm-wide which developed at a 3.0% drift ratio for both specimens. Longitudinal beam bottom steel bars were exposed at the joint and spalling reached 25 percent of the joint depth at a drift ratio of 3.5%. The shear failure mechanism is shown in Fig. 5(a) for specimen 24-1.

4.2. As-Built Type II Specimens

Specimen 16-1 developed the first significant shear crack, which remained open after unloading, in the joint region with a width of 1.0 mm at a drift ratio of 2.5%; the diagonal cracks in the joint region reached a width of 2.5 mm at a drift ratio of 3.5%. A bond failure mechanism developed at a drift of 3.5% at the bottom corners of the joint accompanied by spalling with a height larger than 10 percent of the cross-section, as shown in Fig. 5(b).

4.3. Rehabilitated Type I Specimens

CFRP rehabilitated specimens R24-3 and R24-4 developed delamination of the CFRP composite at the joint at a drift ratio of 2.0%, which spread to the joint-beam interface on both sides at a drift ratio of 3.0%. At a drift ratio of 3.0% for R24-4 and 3.5% for R24-3, bulging at the bottom of the joint was observed due to volumetric



(a) (b)

Figure 5. (a) As-built specimen 24-1 (Type I): joint shear failure mechanism, (b) As-built specimen 16-1 (Type II): bond failure mechanism of beam bottom steel bars at the joint.

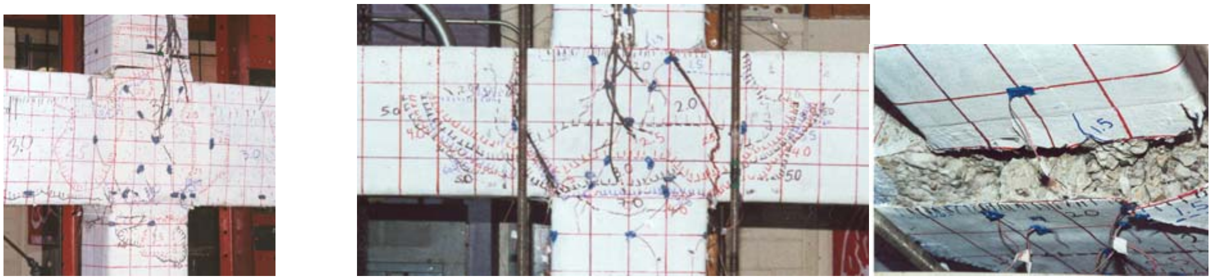
expansion and fracturing of the concrete inside the joint. At a drift ratio of 4.0%, bulging of the joint region reached the bottom of the beam for both R24-3 and R24-4. At a drift ratio of 5.0%, a bulging failure mechanism occurred whereby bulging of the concrete had spread over the entire joint region, as shown in Fig. 6(a) for specimen R24-3. The CFRP composite jacket created a "basketing" effect of the fractured concrete inside the joint which allowed the specimen to carry compression forces well after cracking had developed in the joint concrete. The CFRP rehabilitated specimens achieved a higher drift ratio of 5% before a ductile failure occurred compared to the as-built specimens which failed in a brittle manner at a 3% drift ratio.

4.4. Rehabilitated Type II Specimens

Rehabilitated specimens R16-2 and R16-3 developed delamination of the CFRP composite at the top corners of the joints on the east and west side at a drift ratio of 2.0%, and R16-4 at 2.5%. Delamination of the CFRP composite continued downwards up to ¾ of the beam depth and spread to the joint-beam interface; at a drift ratio of 2.5%, there was a CFRP composite tensile failure of the wrap at the column to beam interface for R16-2. Significant bulging of the joint was observed at a drift ratio of 2.5% for specimens R16-2 and R16-3 due to fracturing of the concrete inside the joint. At a drift ratio of 3.0%, CFRP delamination had reached the full joint depth for all three rehabilitated specimens, as shown in Fig. 6(b) for specimen R16-2. At a drift ratio of 3.5% the CFRP composite wrap opened fully and vertical fractures in the CFRP composite developed at the beam to column interface. Significant bulging of the joint was observed for R16-4 at a drift ratio of 4.0% due to fracturing of the concrete inside the joint. CFRP delamination spread into the beam region, and a bulging failure mechanism occurred at 4% drift ratio for R16-3 (5% drift ratio for specimens R16-2 and R16-4). The bulging failure mechanism is shown in Fig. 6(b) for R16-2. The CFRP composite jacket created a "basketing" effect of the fractured concrete in the joint which allowed the specimen to carry compression forces well after cracking had developed in the joint concrete. The CFRP rehabilitated specimens achieved a higher drift ratio of 4% to 5% before ductile failure occurred compared to the as-built specimen that failed suddenly at a 3.5% drift ratio.

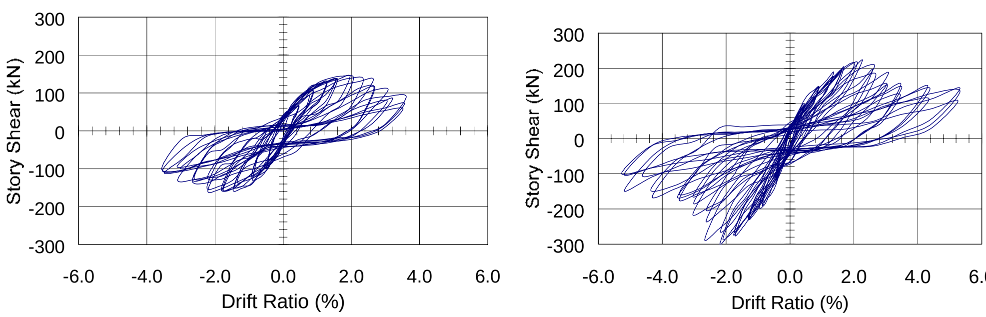
5. HYSTERETIC PERFORMANCE

Figure 7 shows the hysteresis curves for two Type I specimens. Specimen 24-1 in Fig. 7(a) reached a peak story shear of 144 kN. At a drift ratio of 2.7%, the story shear dropped 20% from its peak value. This drift ratio is close to the 3.0% drift ratio at which a full shear mechanism had developed. The rehabilitated joint specimen R24-3 in Fig. 7(b) reached a peak story shear of 216 kN; this is 1.50 times the story shear reached by the as-built specimen 24-1. Figure 8 shows the hysteresis curves for two Type II specimens. Joint specimen 16-1 reached a peak story shear of 84 kN as shown in Fig. 8(a). At a drift ratio of 3.5%, the story shear dropped 20% from its peak value, and a bond failure mechanism developed. The rehabilitated joint specimen R16-2 reached a peak story shear of 115 kN as shown in Fig. 8(b); this is 1.37 times the story shear reached by the as-built specimen 16-1. The story shear dropped 20% from its peak value at a drift ratio of 5%, which is the same drift



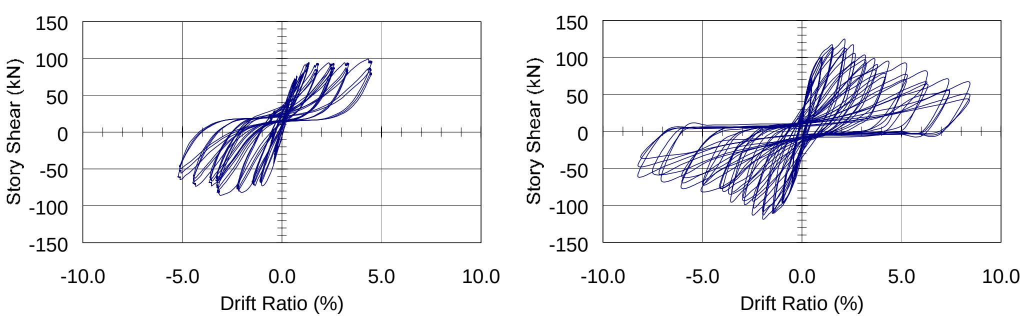
(a) (b)

Figure 6. (a) Rehabilitated specimen R24-3 (Type I): CFRP composite delamination and bulging failure mechanism, (b) Rehabilitated specimen R16-2 (Type II): CFRP composite delamination, confinement failure and bulging failure mechanism.



(a) (b)

Figure 7. Hysteresis curves for Type I specimens: (a) As-built 24-1, (b) Rehabilitated R24-3.



(a) (b)

Figure 8. Hysteresis curves for Type II specimens: (a) As-built 16-1, (b) Rehabilitated R16-2.

ratio at which a bulging failure mechanism occurred. Similar behavior for both as-built and CFRP rehabilitated joint specimens was observed for the remaining specimens.

5. CONCLUSIONS

Rehabilitation measures were successful in promoting ductile behavior by delaying brittle joint shear failure and pullout of the beam bottom steel bars at the joint. The "basketing" effect of the CFRP composite jacket allowed the fractured concrete in the joint to carry compression forces well after concrete cracking had developed. These measures delayed the propagation of damage in the joint panel and postponed the loss of stiffness and strength. The CFRP seismic rehabilitation of the joints included: (1) an external CFRP "lap splice" technique for bridging the gap between beam bottom steel bars at the joint to prevent early debonding and pullout, and (2) application of CFRP composite joint shear reinforcement to prevent brittle joint shear failure.

The as-built specimens demonstrated a diagonal shear failure mechanism with a main diagonal concrete shear crack, and some debonding of the beam bottom steel bars at the joint in the case of Type I specimens. For Type II specimens, significant spalling of the concrete at the bottom corners of the joint was observed, indicating early debonding and pullout failure of the beam bottom steel bars at the joint. The rehabilitated Type I and Type II specimens had a story shear force capacity 1.5 times that of the as-built specimens on average. The Type II CFRP rehabilitated specimens reached a maximum drift ratio 2.7 times that of the Type II as-built specimens and a plastic beam rotation 1.1 times that of the Type II as-built specimens. The Type I CFRP rehabilitated specimens achieved 2.2 times the drift ratio of the Type I as-built specimens and 1.5 times the plastic beam rotation of the Type I as-built specimens. CFRP delamination was the initiating cause of failure for the rehabilitated beam-column joints, which was the development of a bulging failure mechanism due to fracturing of the concrete inside the joint. However, this failure occurred at higher drift ratios and was ductile. The rehabilitation measures achieved a relatively greater improvement of the performance for shear deficient specimens (Type I).

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