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Seismic strengthening of severely damaged beam-column RC joints using CFRP

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Abstract

This paper investigates the seismic behavior of three full-scale exterior reinforced concrete (RC) beam-column joints rehabilitated and strengthened with externally bonded Carbon Fiber Polymers (CFRP). The specimens had inadequate detailing in the core zone and replicated joints of a real substandard building tested as part of the EU-funded project BANDIT. Seven tests were performed in two successive phases. The bare joints were first subjected to reversed cyclic loading tests to assess their basic seismic performance. As these initial tests produced severe damage in the core, the damaged concrete was replaced with new high-strength concrete. The specimens were subsequently strengthened with CFRP sheets and the cyclic tests were repeated. The results indicate that the core replacement with new concrete enhanced the shear strength of the substandard joints by up to 44% over the bare counterparts. ASCE/SEI 41-06 guidelines predict accurately the shear strength of the bare and rehabilitated joints. The CFRP strengthening enhanced further the joint strength by up to 69%, achieving a shear strength comparable to that of joints designed according to modern seismic provisions. Therefore, the rehabilitation/strengthening method is very effective for post-earthquake strengthening of typical substandard structures of developing countries.

Subject headings: Full-scale tests; Reinforced concrete; Joints; Rehabilitation; Strengthening; Fiber reinforced polymer; Composite materials; Shear strength

Keywords: *beam-column RC joints; seismic strengthening; concrete rehabilitation; externally bonded CFRP*

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Introduction

Recent strong earthquakes in developing countries (Kashmir, 2005; China, 2008; Indonesia, 2009 and Haiti, 2010) caused extensive economic and human losses due to the poor behavior of many old reinforced concrete (RC) buildings. Many structural failures in these structures can be attributed to the lack of internal steel stirrups in beam-column joints, which increase the seismic vulnerability of the building. The local strengthening of these deficient elements is a feasible option for reducing the vulnerability of such substandard buildings. Over the last twenty years, externally bonded Fiber Reinforced Polymers (FRP) have been used extensively to strengthen seismically deficient elements. In comparison to other strengthening materials, FRP possess advantages such as high resistance to corrosion, excellent durability, high strength to weight ratio, adaptability to different shapes, and ease and speed of in-situ application (Gdoutos et al. 2000).

Numerous experimental studies have demonstrated the effectiveness of FRP strengthening at improving the seismic behavior of substandard RC beam-column joints (e.g. Mosallam 2000; Gergely et al. 2000; Granata and Parvin 2001; El-Amoury and Ghobarah 2002; Antonopoulos and Triantafillou 2003; Prota 2004; Said and Nehdi 2004; Ghobarah and El-Amoury 2005; Mukherjee and Joshi 2005; Engindeniz 2008; Pantelides 2008; Akguzel and Pampanin 2010; Alsayed et al. 2010; Le-Trung et al. 2010; Bousselham, 2010; Parvin et al. 2010; Al-Salloum 2011a; 2011b; Ilki et al. 2011). Most of these studies aimed at a) assessing the effectiveness of FRP at preventing premature shear failure of joints without internal confinement, and b) changing the strength hierarchy of the joints to promote yielding in the beam reinforcement. Despite the extensive research, the majority of these studies focused on undamaged specimens, whilst less research has investigated the use of FRP as a post-earthquake strengthening solution in joints that experienced severe damage. Different rehabilitation techniques have been used to repair damaged joints, including a) crack injection with epoxy resin and partial core replacement with high-strength cement paste or mortar (Karayannis et al. 1998; Karayannis and Sirkelis 2008; Sasmal et al. 2011), b) complete core replacement with new concrete (Ghobarah and Said 2001; 2002), and c) partial core replacement with high-strength mortar (Tsonos 2008; Sezen 2012). However, researchers rarely attempted to evaluate the individual contributions of the repairing technique and the FRP strengthening to the total strength of the joint. As shown by Karayannis et al. (1998), the use of high-strength materials in the rehabilitation can by itself enhance the strength of the joint considerably. Crack injection and mortar repairing have proven effective at low to moderate levels of damage, but they may be less effective when severe damage occurs (e.g. complete concrete crushing in the core) or bond between reinforcing bars and concrete is lost. In this case, the complete replacement of the core with new concrete may be necessary to recover its structural integrity before applying the FRP. Nonetheless, results of joints rehabilitated and strengthened with this solution are not available in the literature. A combination of core replacement with high-strength concrete and FRP strengthening can be suitable for rehabilitating existing substandard buildings in developing countries, where strengthening interventions are usually carried out in structures damaged after an earthquake.

This study is part of the multistage EU-funded project BANDIT (SERIES Program FP7) which focuses on the seismic strengthening of substandard RC structures typical of developing countries. The work carried out under this project comprises tests on beam-column joints and shake table tests on a full-scale RC building (Garcia 2013; Garcia et al. 2014a). This paper focuses on the former tests and investigates the seismic behavior of severely damaged full-scale RC beam-column joints rehabilitated and strengthened with externally bonded CFRP. The geometry and detailing of the tested specimens were similar to those used in the joints of the BANDIT building (Garcia et al. 2014a). Therefore, the current tests aimed at i) assessing the capacity and behavior of substandard joints under severe demands, and ii) investigating effective rehabilitation and strengthening solutions for damaged joints with FRP sheets. The experimental results are discussed and compared to predictions obtained according to existing models.

Experimental program

Three RC beam-column joints were tested in two successive phases. In phase 1, the bare joints were subjected to cyclic tests up to drifts of about 4.0%, and the tests were halted when the peak capacity dropped by 50%. As these tests produced severe damage in the joint core, the damaged concrete was fully removed and replaced with new high-strength concrete. The specimens were subsequently strengthened with externally bonded FRP sheets and retested up to failure (phase 2).

Geometry of specimens

The specimens simulated a full-scale 2D exterior joint between contra-flexure points of a floor in a multi-story moment-resisting frame, but excluding the slabs (see Fig. 1(a)). The column had a cross section of 260×260 mm and a height of 2700 mm. The longitudinal column reinforcement consisted of 16 mm bars (see Fig. 1(b)). These bars were lapped over a length l_b=25d_b (d_b= bar diameter) just above the joint core to represent typical construction practices of developing countries.

The beam had a cross section of 260×400 mm and a length of 1650 mm. The main flexural reinforcement consisted of 16 mm bars as shown in Fig. 1(b). Three types of anchorage detailing were examined for the top beam reinforcement as shown in Fig. 1(c). To study the effect of deficient bar anchorage, the bottom beam reinforcement of detailing types A and B was anchored into the joint for a length of 220 mm only (approximately 14d_b), with no hooks or bends. This short anchorage length would be deemed insufficient to develop the full capacity of the 16 mm bars according to current design recommendations. The column-to-beam relative flexural strength ratio ($\Sigma M_{Rcol}/\Sigma M_{Rbeam}$) of the specimens was approximately 1.0, and therefore the strong column-weak beam strength hierarchy intended by current design philosophy was not satisfied. Moreover, the specimens were designed to fail at the core where no confining stirrups were provided. To prevent a shear failure outside the joint core, the column and beam were reinforced with 8 mm transverse stirrups spaced at 150 mm centers. The stirrups were closed with 90° hooks instead of 135° hooks typically required by current seismic codes.

Table 1 gives some of the main characteristics of the beam-column joints including concrete strength. The specimens are identified using an ID code in which the first letter stands for “Joint” and the second for the type of beam reinforcement detailing (A, B or C), respectively. The letters after the number indicates the condition of the joints during the test: “R” stands for a joint tested in rehabilitated condition (i.e., with a new concrete core), whilst “RF” stands for a joint tested in rehabilitated condition and strengthened with FRP sheets. The core of joint JB2RF was recast again and the specimen (renamed as JB2R) was retested to examine the effect of core replacement on the joint shear strength.

Material properties

The joints were cast using two batches of ready mixed concrete. A steel roller was inserted at the center of the cross section of the beam tip during casting, as shown in Fig. 1(a). Following casting, the specimens were cured for seven days in the formwork and then stored under standard laboratory conditions. The mean concrete compressive strength (f_{cm}) was determined from tests on three 150×300 mm concrete cylinders according to BS EN 12390-3 (BSI 2009a). The indirect tensile splitting strength (f_{ctm}) was obtained from tests on three 100×200 mm cylinders according to BS EN 12390-6 (BSI 2009b). All cylinders were cast at the same time and cured together with the joints. Table 1 summarizes the mean values and standard deviations obtained from the tests.

Grade S500 ribbed bars were used as reinforcement for all joints. The yield and tensile strengths of the steel reinforcement were obtained from three test samples and were found to be f_y=612 MPa and f_u=726 MPa for the 8 mm bar, and f_y=551 MPa and f_u=683 MPa for the 16 mm bar, respectively. The elastic modulus of both bars was determined as E_s=209 GPa.

After phase 1 of testing, the damaged concrete in the core of the joints was completely removed and replaced with new highly flowable concrete. In joint JB2, the bottom bars of the beam were welded to the 90° bends of the top beam reinforcement as shown in Fig. 1(d) (JB2RF and JB2R in phase 2). Before casting the new concrete, the contact surfaces of the existing concrete were thoroughly cleaned with compressed air and moistened for 24 hours. No bonding agent was used between the new and existing concrete. At the end of casting, the new core was cured in the mold for three days, and without the mold for four additional days.

Following the core replacement, the joints were strengthened with externally bonded CFRP using a wet lay-up procedure. The unidirectional CFRP sheets had the following nominal properties provided by the manufacturer: tensile strength f_f=4140 MPa, elastic modulus E_f=241 GPa, ultimate elongation ε_{fu}=1.70%, and sheet thickness t_f=0.185 mm. Before bonding the CFRP sheets, the concrete surfaces were wire brushed and cleaned with pressurized air to improve adherence, and the corners were rounded off to a radius of approximately 15 mm.

Experimental setup and instrumentation

The joints were tested with the column in horizontal position as shown in Fig. 2. A guiding device consisting of an oiled roller inserted between two parallel steel plates was used at the beam tip to restrain possible out-of-plane movement at large displacements, but such device allowed free displacement and rotation of the beam in the direction of testing. Displacements were monitored using Linear Variable Differential Transformers (LVDTs) at the locations shown in Fig. 2. Deformations of the joint core were also measured using a set of 16 linear potentiometers. The strain developed along the beam and column steel reinforcement and CFRP sheets was monitored using strategically placed foil-type electrical resistance strain gages (see Fig. 3(a)).

The cyclic load was applied to the beam in displacement control using a servo-hydraulic actuator (see Fig. 2). Three push-pull cycles were applied at drift ratios δ (δ=beam tip displacement/beam length) of ±0.25%, ±0.5%, ±0.75%, ±1.0%, ±1.5%, ±2.0%, ±3.0%, ±4.0% and ±5.0%. Cycles at δ ±0.75% and ±1.5% were not applied in phase 2 to reduce the testing time. Each cycle was applied in the push (+) direction first, which tensioned the top beam reinforcement. A second actuator applied a constant axial load N=150 kN on the column (see Fig. 2), which corresponds to an approximate axial load ratio $v=N/(f_{cm}A_g)=0.07$, where A_g is the column gross cross sectional area. The formation and development of cracks were monitored continuously during the test. Moreover, the tests were paused at the onset of the first visible diagonal core cracking (which appeared suddenly) to record the applied load and tip displacement. The tests were halted when the load capacity of the joints dropped to approximately 50% of the peak load.

CFRP strengthening

Fig. 3(a) shows a general view of the CFRP strengthening sequence. The main goal of the strengthening was to develop the plastic capacity of the beam reinforcement. To achieve this, the premature failure of the core zone had to be prevented and the flexural capacity of the column enhanced.

Strengthening of joint core

The CFRP strengthening was designed considering the total shear capacity of the joint core as the sum of concrete and CFRP contributions. The concrete contribution (V_c) was computed according to ASCE/SEI 41-

06 (2007) using Eq. (1), a shear strength coefficient γ=0.083×6√psi=0.50√MPa and the strength of the new concrete core (see Table 1).

$$V_c = \gamma A_j \sqrt{f_c} = (0.5)(260)(260)\sqrt{55} = 250 \text{ kN} \tag{1}$$

where A_j is the effective horizontal joint area.

The theoretical shear force required to develop the plastic capacity of the beam reinforcement V_{jh} (associated to a beam load P_y=±106 kN) was computed using force equilibrium according to Eq. (2).

$$V_{jh} = T_b - V_{col} = P\left[\frac{L_b}{z} - \frac{L_b + 0.5h_c}{H_c}\right] = 106000 \cdot \left[\frac{1370}{0.875(362)} - \frac{1370 + 0.5(260)}{2400}\right] = 392 \text{ kN} \tag{2}$$

where T_b is the tension force of the top beam reinforcement; V_{col} is the column shear; L_b is the beam length to the applied load point; H_c is the distance between column supports; h_c is the height of the column cross section; and z is the lever arm of the beam flexural moment (assumed equal to 0.875 the beam effective depth).

Thus, the shear to be resisted by the CFRP sheets is V_f=392-250=142 kN. The required number of CFRP layers was determined using ACI 440.2R-08 guidelines (ACI Committee 440 2008) adopting the recommended value of effective CFRP strain (ε_{fe}=0.004) and α=90°:

$$n = \frac{V_f}{2t_f \varepsilon_{fe} E_f d_{fv}} = \frac{142000}{2(0.185)(0.004)(241000)(222)} = 1.6 \text{ layers} \Rightarrow 2 \text{ layers} \tag{3}$$

where d_{fv} is the effective depth of FRP shear reinforcement.

Accordingly, a minimum of two layers of CFRP were required to strengthen the joint core. As shown in Fig. 3(a), U-shaped CFRP sheets strengthened the joint core to increase its shear strength (layer 1). Confining sheets (layer 2) were then wrapped around the beam to prevent premature debonding of the U-shaped sheets.

Strengthening of column

The flexural capacity of the column was increased by bonding CFRP sheets parallel to the column axis (layers ③ and ④ in Fig. 3(a)). The number of layers required to satisfy a hierarchy strength $\Sigma M_{Rcol} > 1.3\Sigma M_{Rbeam}$ was determined using conventional moment-curvature analysis assuming perfect bond between CFRP sheets and concrete. The presence of the beam hindered the continuity of sheets ③ on the inner part of the column. To avoid interrupting or mechanically anchoring sheets ③ in the beam section, these sheets were folded and bonded on the column faces. As a result, sheets ③ provided slightly lower flexural strength in comparison to sheets ④. Nonetheless, detailed moment-curvature analyses indicate that such difference is less than 10%, which is acceptable for practical strengthening applications. An additional layer of CFRP (⑤) was bonded on both sides of the column to keep sheets ③ in place during the subsequent installation of the confining sheets. Finally, sheets ⑥ and ⑦ were used to increase the ductility of the column and to avoid premature debonding of ③, ④ and ⑤.

Table 2 summarizes the number of CFRP sheets used for each joint. In joint JA2RF, sheets ① had a shorter bonded length to examine its effect on the resulting confinement level of the joint core and overall joint behavior. As shown later, the relatively low capacity of joint JA2RF showed that two layers of CFRP sheets ① were insufficient to develop the plastic capacity of the beam reinforcement. Therefore, three CFRP layers were used for joints JB2RF and JC2RF (Table 2). Also, two layers of confining sheets ②, ⑥ and ⑦ were used in the latter joints as one layer did not prevent premature fiber debonding at beam and column corners of joint JA2RF. No mechanical anchors or steel plates were utilized to prevent debonding of CFRP sheets.

After the tests on joint JB2RF, the CFRP sheets were completely removed and the core was replaced again following the same procedure described previously (see Fig. 3(b)). Post-tensioned steel strapping was applied to the beam and column outside the core to promote the development of shear cracks in the core, and the joint was retested (JB2R).

It should be noted that in actual rehabilitation of damaged buildings, the removal and recast of the concrete core would require the use of temporary shoring adjacent to the joint. Shoring can be removed after the recast core sets, thus allowing the preparation of concrete surfaces for the application of CFRP sheets. In real CFRP strengthening applications adopting the layout shown in Fig. 3(a), sheets ③ could be bonded (completely unfolded) on the inner face of the column, and then secured using mechanical CFRP anchors embedded in the concrete. Such anchoring solution was proven effective at preventing debonding of the CFRP strengthening on a substandard full-scale RC building tested by the authors (Garcia et al. 2010).

Test results and discussion

Table 3 reports a) the load and drift ratio at the onset of diagonal cracking in the core (P_{cr} and δ_{cr}, respectively), b) the load and drift ratio at peak load (P_{max} and δ_{max}, respectively), c) enhancement in peak load (ΔP_{max}) over the bare specimens, and d) ultimate drift ratio (δ_u) causing a 50% drop in P_{max}. The results are presented for the push (+) and pull (-) directions. The following sections summarize the most significant observations of the testing program and discuss the results shown in Table 3.

Bare and rehabilitated joints

The progression of damage and final failure mode of the bare and rehabilitated joints were very similar. Despite the short anchorage length of the bottom beam reinforcement, pull out failure was not observed. This was confirmed by experimental observations (no cracks formed at the beam-joint interface) and by comparison of readings from strain gages fixed on the bars with displacements. Narrow splitting cracks formed along the spliced reinforcement in the column. As shown in Fig. 4(a) and (b), final failure of the bare and rehabilitated specimens was dominated by extensive diagonal cracking and partial concrete spalling in the core zone (J-type failure).

The load-drift responses of the bare joints (Fig. 5(a)-(c)) show that the specimens remained elastic until the onset of diagonal core cracking, when the load dropped slightly. Despite the lack of steel stirrups, the bare specimens failed gradually and sustained drifts of up to ±4.0% (JC2 in Table 3). Such gradual failure is attributed to progressive crushing of the diagonal concrete strut in the joint core. As shown in Table 3, the load and drift at the onset of diagonal cracking were similar for all bare joints, regardless of the type of detailing in the beam reinforcement. The peak load for all bare joints was achieved at very similar δ_{max} ±1.4-

1.5% in both the push and pull directions and no pullout failure occurred during the tests because shear failure dominated the response. The lower loads resisted in the pull direction (Table 3) can be due to damage produced by the cyclic loading regime. The influence of the different anchorage solutions used for the joints was examined in another study (Jemaa 2013).

The load-drift relationship of joint JB2R in Fig. 5(d) shows the effect of core replacement on the shear strength of the joint (see also Table 3). Compared to JB2, the rehabilitation enhanced the peak capacity of joint JB2R by an average of 44%. This confirms that, in spite of severe damage produced in the joints during testing phases 1 and 2, the high-strength concrete used in the core recast and the welding of the top and bottom beam reinforcement (JB2R only) enhanced considerably the joint capacity. Note that although the beam reinforcement did not pullout during the tests, such bars were welded in JB2R to correct excessive permanent deformations and to assess the effectiveness of this strengthening solution as pullout failure was not desired. This approach was also adopted in the joints of a full-scale building tested recently by the authors (Garcia et al. 2014a).

Whilst in this study the joint specimens were tested up to a drift level of ±4.0% to produce severe damage and help understand their vulnerability, it is evident that the residual value and capacity of an actual substandard building pushed to such drift value would be relatively low. Nonetheless, concrete core replacement can be carried out at lower levels of drift or damage and is justified when, for instance, the building experience extensive shear damage or was cast with low-strength concrete (f_{cm}<20 MPa). Poor quality concrete is a common deficiency of many low-rise substandard constructions of developing countries.

CFRP-strengthened joints

As the CFRP sheets were bonded directly onto the concrete surface, the onset of diagonal cracking in the joint core could not be observed. No damage was observed in the CFRP sheets, but extensive "crackling" noise at a drift ratio δ ±1.0% indicated that debonding was taking place at different locations. In specimen JA2RF, full debonding of sheets ① and the rupture of sheets ② at δ of ±3.0% led to premature failure of the CFRP strengthening. In contrast, total rupture of sheets ① occurred across the beam depth (just above sheets

③) at δ ±5.0% and ±4.0% in joints JB2RF and JC2RF, respectively. Although no mechanical anchors were used, sheet debonding did not occur in these joints. Fig. 6(a) and (b) show specimens JA2RF and JB2RF at the end of the tests. The removal of the CFRP sheets after the tests revealed extensive diagonal cracking in the core, but the width and extension of cracks reduced considerably in comparison to the bare specimens. No evident damage was observed in the lap splices or in the columns outside the CFRP-strengthened area. However, the beams of joints JB2RF and JC2RF experienced significant flexural cracking outside the CFRP-strengthened region.

Fig. 7(a) to (c) show the load-drift relationships for the CFRP-strengthened specimens. The combination of core replacement and CFRP strengthening enhanced significantly the load and deformation capacity of the joints. Compared to the bare counterparts, the peak load of specimens JA2RF, JB2RF and JC2RF increased by an average of 52%, 145% and 128%, respectively (see Table 3). Moreover, the peak and ultimate drift ratios of the joints also increased by up to 97% and 67%, respectively (joint JB2RF). Fig. 7(a) to (c) show that the area enclosed by the hysteretic loops is significantly larger for the CFRP-strengthened joints than for the bare specimens, which implies that these joint had a higher energy dissipation capacity.

Fig. 8 shows envelopes of the load-drift ratio relationships of the tested joints. The results indicate that the bare specimens resisted only 40-55% of the load required to develop the plastic capacity of the beam, P_y (see also Table 3). The peak load of specimen JA2RF reached 78% of P_y as premature debonding of the CFRP sheets occurred. In contrast, specimens JB2RF and JC2RF developed some yielding in the top and bottom beam reinforcement as shown by short post-yield incursions in Fig. 8. Readings from strain gages also confirmed that the beam reinforcement of the joints developed strains of up to 4000-5000 με (e.g. Fig. 9). As a result, joints JB2RF and JC2RF failed in a more ductile BJ-type mode (i.e. joint failure after yielding of beam reinforcement), thus achieving the strengthening goals. However, CFRP rupture and excessive damage in the joint core prevented the development of larger plastic strains in the beam reinforcement.

Stiffness degradation and shear stress-strain response

Fig. 10 compares the stiffness degradation of the tested specimens. The secant stiffness is defined by the slope of a line connecting the maximum drifts in the push and pull directions of the first hysteresis loop. The results indicate that the core recast was very effective at restoring the original stiffness of the CFRP-strengthened specimens. The stiffness of JB2R was not fully recovered and this can be attributed to the flexural cracks formed in the beam of this joint during the previous tests (JB2 and JB2RF, see Fig. 10). In comparison to the bare specimens, it is also evident that the CFRP strengthening of the joints reduced significantly the rate of stiffness degradation. The fast stiffness deterioration after δ ±2.0% in the CFRP-strengthened specimens can be attributed to the onset of CFRP rupture and to damage in the core.

Fig. 11 compares the experimental shear stress-strain response of joints JC2 and JC2RF, which are representative of the rest of the specimens. Shear strains were derived using average measurements of the linear potentiometers located at the joint core. Results in Fig. 11 are only shown up to the point where the potentiometers failed, after reaching the joint capacity. It is shown that average joint strains at peak load (P_{max}) of specimens JC2 and JC2RF were 0.0067 and 0.069 rad, respectively, whereas maximum joint strains were 0.025 and 0.11 rad. The considerable enhancement in joint deformation capacity of joint JC2RF is attributed to the rehabilitation/strengthening intervention. Due to space limitations, the detailed analysis of the joint strains will be published by the authors in a future paper.

CFRP strains

Typical strain readings (see Fig. 12(a)) from gages located at the core zone of the specimens indicate that, as expected, CFRP strains were negligible at the beginning of the test and increased after the onset of diagonal core cracking. In general, the CFRP strain values measured at peak load (P_{max}) varied from 2500 με (joint JA2RF) to 7300 με (JB2RF), and at ultimate drift (δ_u) varied from 11900 με (JA2RF) to 16900 με (JB2RF). The latter values correspond to 70% and 100% of the ultimate strain of the CFRP sheets, respectively. Fig. 12(b) shows typical strain readings from gages fixed on the confining sheets ⑥ at the mid-point of the lap splice length (see Fig. 3(a)). Overall, maximum strains at P_{max} varied from 200 to 570 με only, whereas strains at δ_u were always lower than 1000 με. These results confirm previous research by the authors (Garcia et al. 2013; 2014b) that showed that low CFRP strains (ε_f<1600 με) develop in CFRP-confined lap-spliced RC members dominated by bond splitting failure.

Contribution of rehabilitation and CFRP to total joint shear strength

The maximum capacities of the strengthened joints reported in Table 3 include the contributions of the replaced core and the CFRP strengthening. To decouple the individual contribution of the replaced core, the shear strength factor γ included in current guidelines (e.g. ACI-ASCE Committee 352 2002) is adopted in this study. Table 4 summarizes the experimental shear stress υ_{jh} (computed using Eq. (4)) and corresponding factor γ for the tested joints. The reported values are the average of the push and pull directions.

$$\upsilon_{jh} = V_{jh} / A_j \quad (4)$$

For comparison, Table 4 also shows the factors γ ($\gamma=\upsilon_{jh}/\sqrt{f_c}$) computed using existing predictive models: γ_P Priestley (1997); γ_{KL} Kim and LaFave (2009); γ_{HJ} Hassan (2011) for J-type failure; γ_{HS} Hassan (2011) for S-type failure due to the pullout of the straight bottom beam reinforcement; and γ_{PM} Park and Mosalam (2012). A "virtual" joint index of 0.0139 was adopted to determine γ_{KL} for joints without shear reinforcement a as suggested by Kim and LaFave (2009). Table 4 also includes the shear strength factor γ_{A41} for exterior joints given by the ASCE/SEI 41-06 (2007) guidelines.

Table 4 shows that the bare specimens have similar shear strength factors γ ranging from 0.49 to 0.52. Moreover, despite the damage produced during testing phases 1 and 2, specimen JB2R had a similar factor γ=0.53. This indicates that the replaced core resisted a shear stress comparable to that of the bare joints. It is shown that the approaches proposed by Priestley, Kim and LaFave, Hassan for J-type failure and Park and Mosalam overestimate the experimental shear strength factors γ by an average of 13%, 77%, 48% and 85%, respectively. Conversely, Hassan's S-failure model always underestimates γ by an average of 44%. This underestimation may be due to the calibration of the model, which was done using joints with short anchorage lengths of 152 mm only. Whilst recent research (e.g. Park and Mosalam 2013) suggests that ASCE 41 may yield conservative estimates for the shear strength of substandard joints, it predicted accurately the values γ of the bare and rehabilitated joints tested in this research. Consequently, ASCE 41 is used in this study to evaluate the shear strength of the recast cores.

Table 4 also shows the decoupled contributions of the recast core and CFRP strengthening for the CFRP-strengthened specimens. In this table, $\upsilon_{jh,core}$ and $\upsilon_{jh,CFRP}$ are the shear stress contributions of the recast core and CFRP strengthening to the total joint capacity, respectively; and $\Delta\upsilon_{jh,core}$ and $\Delta\upsilon_{jh,CFRP}$ are the corresponding shear stress enhancements. For specimen JB2RF, $\upsilon_{jh,core}$ was taken as the shear stress of the corresponding rehabilitated specimen JB2R. For joints JA2RF and JC2RF, $\upsilon_{jh,core}$ was computed adopting γ=0.50√MPa and the concrete strength of the replaced core listed in Table 1. The value $\upsilon_{jh,CFRP}$ was then calculated as the difference between the experimental shear stress υ_{jh} of the CFRP-strengthened joints and $\upsilon_{jh,core}$. The rehabilitation and strengthening were very effective at increasing the shear strength of the joints, with the new core contributing by up to 44% of the total and the externally bonded CFRP by up to 69% (see joints JB2RF and JC2RF in Table 4). It should be mentioned that the experimental shear strength factors obtained for joints JB2RF and JC2RF (γ=0.85√MPa and 0.83√MPa, respectively) are only 15 and 17% lower than the factor γ=1.0√MPa considered in ACI 352R-02 (2002) for the design of code-compliant exterior joints. As the capacity of the joints is not expected to increase significantly after yielding of the beam reinforcement, the experimental factors γ reported here are considered as maximum achievable values. This implies that the amount of CFRP utilized in the strengthening was sufficient to develop the full available plastic capacity of the joints.

Summary and conclusions

This paper presented test results of three substandard full-scale RC beam-column joints subjected to two successive testing phases. The geometry and detailing of the specimens were similar to those used in a full-scale substandard RC building tested on a shake table as part of BANDIT Project (Garcia et al. 2014a). In phase 1, the bare joints were subjected to cyclic tests up to a load that induced a 50% drop in peak capacity. As these tests produced severe damage in the joint core, the damaged concrete was fully removed and replaced with new high-strength concrete. The specimens were subsequently strengthened with externally bonded CFRP sheets and retested up to failure in phase 2. From the test results and analysis presented here, the following conclusions can be drawn:

1) The behavior of the bare joints was dominated by extensive cracking in the concrete core which led to premature shear failure (J-type failure). The capacity of the bare specimens was approximately 40-55% the plastic capacity of the joints. Despite the substandard anchorage used for the bottom beam reinforcement, no pullout failure occurred during the tests.

2) The final failure mode of the rehabilitated joint JB2R was similar to that observed in the bare counterparts (J-type). However, the complete core replacement using high-strength concrete restored the original stiffness of the severely damaged joints and increased their capacity by up to 44%.

3) The CFRP strengthening enhanced the capacity of the joints by up to 145% over the bare counterparts (joint JB2RF), and by up to 69% over the specimens rehabilitated with a new core (JB2RF and JC2RF). Compared to the bare joints, the ultimate drift of the CFRP-strengthened joints was enhanced by up to 66% (JB2RF). The use of CFRP strengthening also resulted in yielding of the beam reinforcement and led to a more ductile BJ-type of failure. Although the adopted strengthening layout prevented sheet debonding without the use of mechanical anchors, CFRP rupture and excessive damage in the joint core prevented the development of large plastic strains in the beam reinforcement.

4) For the bare and rehabilitated joints presented here, the approaches proposed by Priestley (1997), Kim and LaFave (2009), Hassan (2011) for J-type failure and Park and Mosalam (2012) overestimate the experimental shear strength factors γ by an average of 13%, 77%, 48% and 85%, respectively. Conversely, Hassan's model for S-type failure underestimates γ by an average of 44%. The shear strength factor γ=0.50√MPa given by ASCE/SEI 41-06 (2007) for exterior joints predicts the shear strength of the bare and rehabilitated joints with very good accuracy.

5) The amount of CFRP utilized in the strengthening was sufficient to develop the full plastic capacity of the joints. The experimental shear factors γ of joints JB2RF and JC2RF (γ=0.85√MPa and 0.83√MPa,

respectively) are only 15 and 17% lower than that considered in ACI 352R-02 (2002) for the design of code-compliant exterior joints (γ=1.0√MPa). Therefore, the rehabilitation/strengthening method proposed in this study is very effective for post-earthquake strengthening of typical substandard structures of developing countries.

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Tables

Table 1. Characteristics of beam-column joints

Phase	ID	f_{cm} (MPa)	f_{ctm} (MPa)	Test condition
1	JA2	32.0(1.61)	2.44(0.16)	Original joint
	JB2	31.3(1.20)	2.41(0.21)	Original joint
	JC2	32.0(1.61)	2.44(0.16)	Original joint
2	JA2RF	54.2(3.00)	3.67(0.17)	JA2 with new recast core and CFRP
	JB2RF	55.3(1.90)	3.91(0.18)	JB2 with new recast core and CFRP
	JC2RF	56.9(1.20)	3.61(0.19)	JC2 with new recast core and CFRP
	JB2R	53.7(3.60)	3.70(0.21)	JB2RF with new recast core

Note: standard deviations shown in parenthesis

Table 2. Number of CFRP layers used for strengthening

Sheet no.	No. of CFRP layers		
	JA2RF	JB2RF	JC2RF
①	2	3	3
②	1	2	2
③	2	2	2
④	2	2	2
⑤	2	2	2
⑥	1	2	2
⑦	1	2	2

Table 3. Load and drift ratio results of tested joints

ID	P_{cr} (kN)	δ_{cr} (%)	P_{max} (kN)	δ_{max} (%)	ΔP_{max} (%)	δ_u [(a)] (%)
JA2	+39.8	+0.47	+57.0	+1.42	-	±3.0
	-37.6	-0.50	-51.9	-1.48	-	
JB2	+42.0	+0.59	+58.0	+1.51	-	±3.0
	-43.1	-0.62	-43.3	-1.46	-	
JC2	+41.4	+0.52	+54.5	+1.40	-	±4.0
	-35.3	-0.39	-48.5	-1.49	-	
JA2RF	+49.0	+0.56	+86.2	+1.86	+51	±4.0
	-47.9	-0.54	-79.8	-2.00	+54	
JB2RF[(b)]	+63.9	+0.90	+120.0	+2.92	+107	±5.0
	-62.8	-0.87	-127.0	-2.95	+193	
JC2RF	+65.5	+0.80	+119.4	+2.91	+119	±5.0
	-53.3	-0.58	-115.0	-2.75	+137	
JB2R[(b)]	+56.7	+1.19	+75.0	+1.95	+29	±4.0
	-46.7	-1.01	-71.3	-2.87	+65	

[(a)] Ultimate drift ratio applied in the test
[(b)] The bottom beam bars were welded to the top beam bars

Table 4. Joint shear strength contributions of core rehabilitation and CFRP strengthening

Phase	ID	υ_{jh} (MPa)	γ	γ_P	γ_{KL}	γ_{HJ}	γ_{HS}	γ_{PM}	γ_{A41}	$\upsilon_{jh,core}$ (MPa)	$\upsilon_{jh,CFRP}$ (MPa)	$\Delta\upsilon_{jh,core}$ (%)	$\Delta\upsilon_{jh,CFRP}$ (%)
1	JA2	2.96	0.52	0.58	0.90	0.75	0.28	0.99	0.50	-	-	-	-
	JB2	2.75	0.49	0.59	0.90	0.75	0.29	1.01	0.50	-	-	-	-
	JC2	2.80	0.49	0.58	0.90	0.75	NA[(a)]	0.99	0.50	-	-	-	-
2	JB2R	3.97	0.53	0.55	0.90	0.75	NA[(a)]	0.77	0.50	-	-	-	-
	JA2RF	4.51	0.61	-	-	-	-		0.50[(b)]	3.68	0.83	+24	+23
	JB2RF	6.71	0.90	-	-	-	-		-	3.97	2.74	+44	+69
	JC2RF	6.36	0.84	-	-	-	-		0.50[(b)]	3.77	2.59	+35	+69

[(a)] Not applicable because the bottom beam bars cannot pullout from the core
[(b)] Theoretical values based on ASCE/SEI 41-06 (2007)

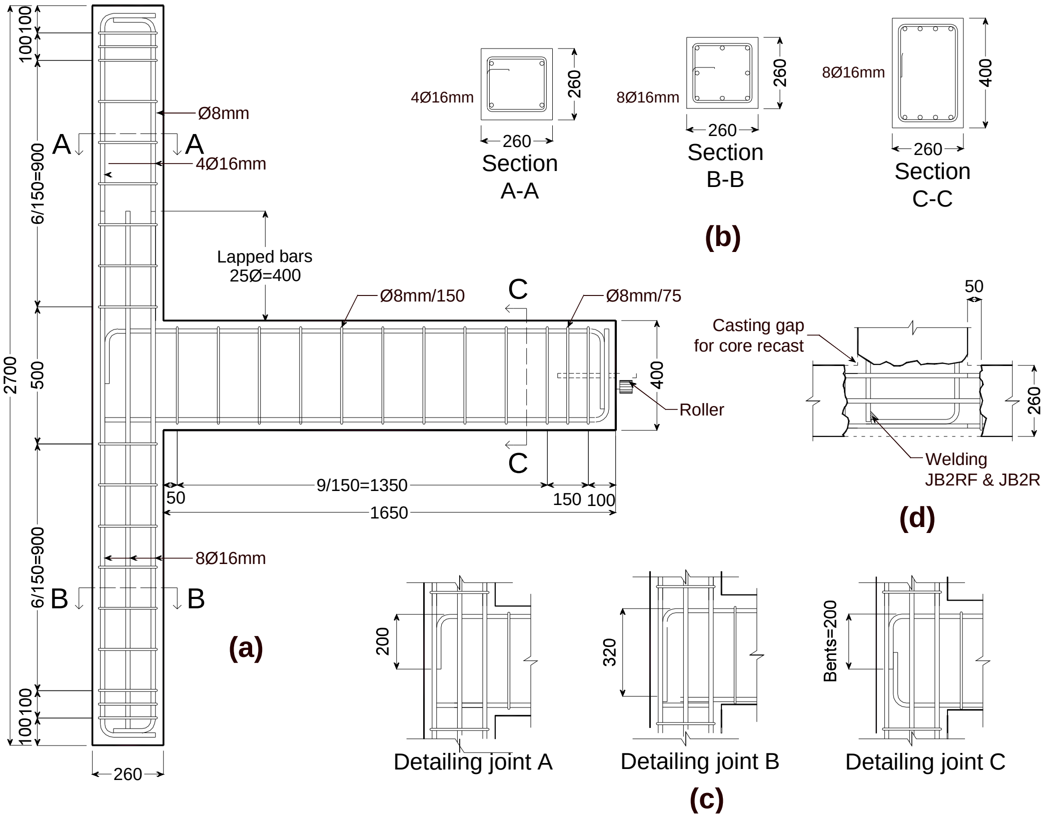

Fig. 1. General geometry and reinforcement details of tested joints (units: mm)

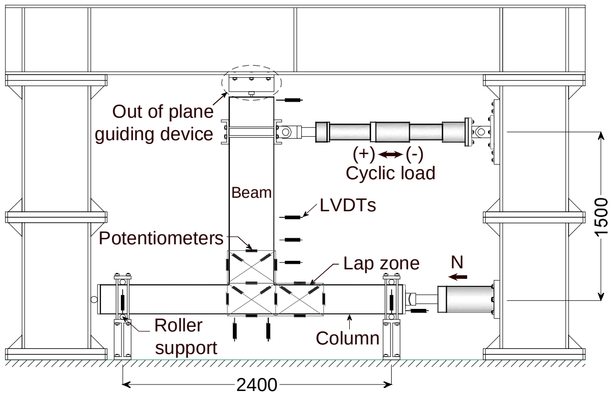

Fig. 2. Test setup and instrumentation of joints (units: mm)

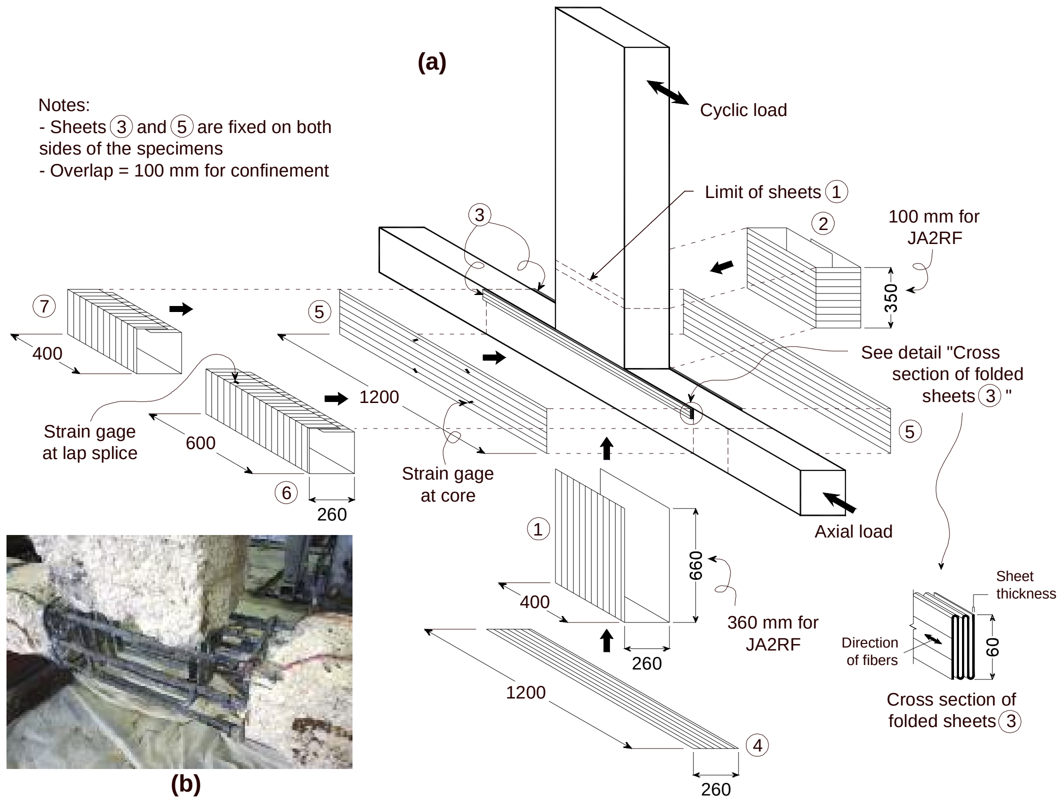

Fig. 3. (a) CFRP strengthening strategy, and (b) removal of CFRP and core replacement in joint JB2RF (units: mm)

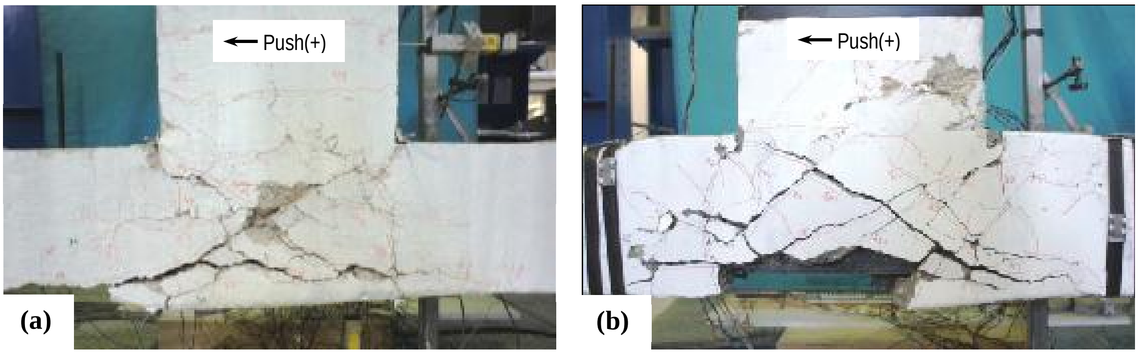

Fig. 4. Typical failure of (a) bare joints (JB2), and (b) rehabilitated joint JB2R

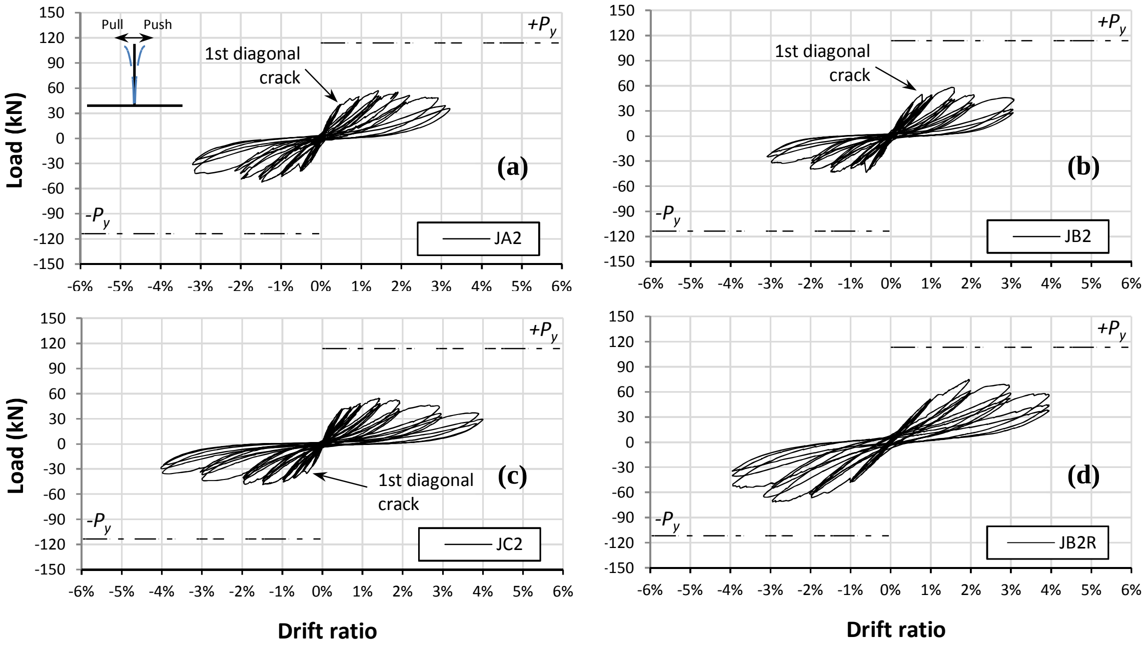

Fig. 5. Load-drift response for (a)-(c) bare joints, and (d) rehabilitated joint JB2R

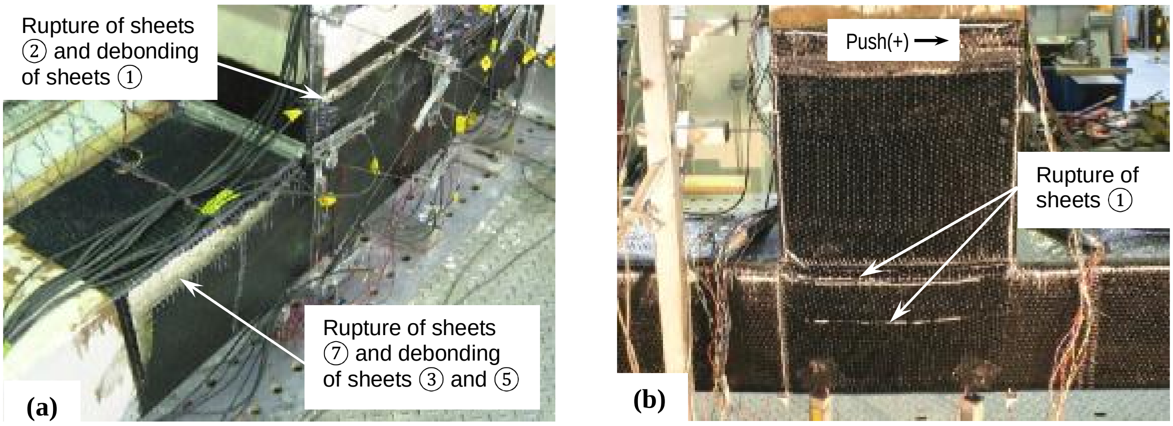

Fig. 6. Failure mode of specimens (a) JA2RF, and (b) JB2RF

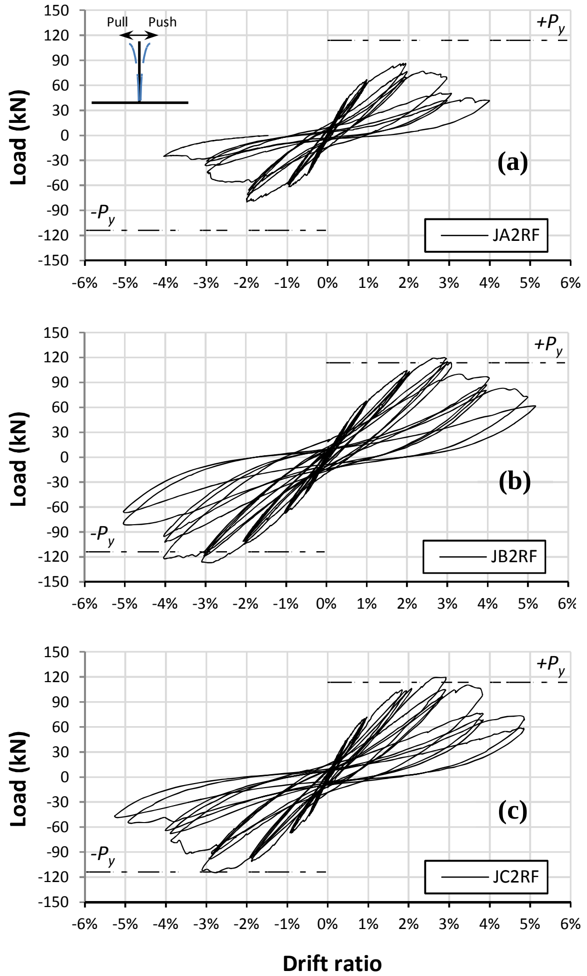

Fig. 7. Load-drift response for rehabilitated and CFRP-strengthened joints

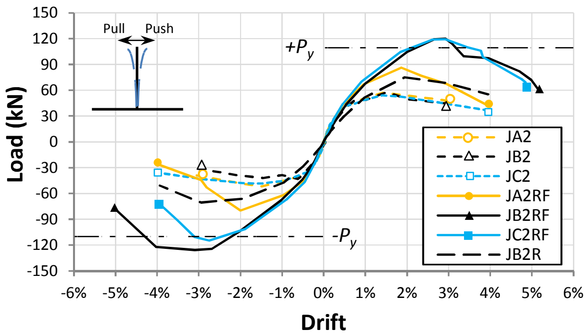

Fig. 8. Comparison of envelope from test results

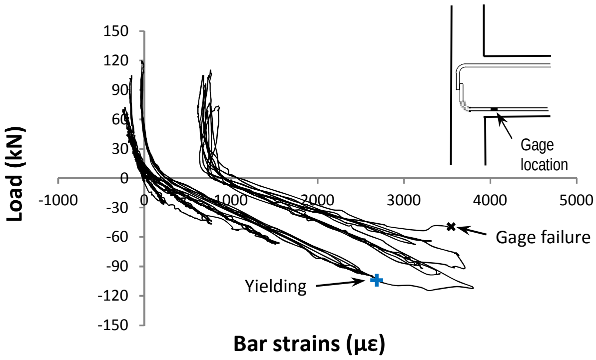

Fig. 9. Strains recorded at bottom beam reinforcement of joint JC2RF

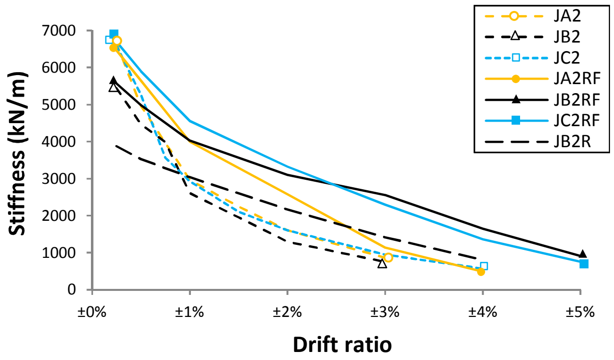

Fig. 10. Stiffness degradation of tested joints

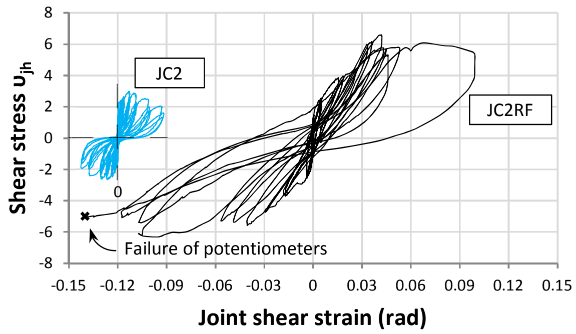

Fig. 11. Shear stress-strain of specimens (a) JC2, and (b) JC2RF

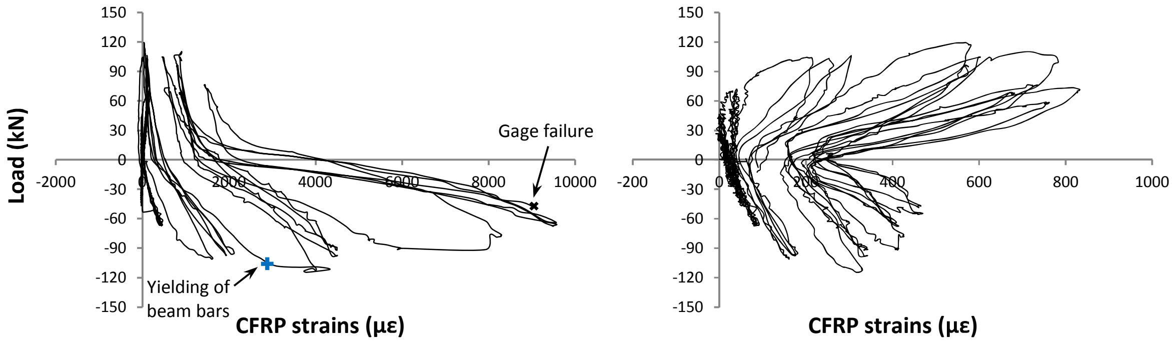

Fig. 12. Strains recorded in CFRP sheets at (a) core and (b) column lap splices of joint JC2RF