FRP Strengthening of Seismically Deficient Full-Scale RC Beam-Column Joints

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SUMMARY:
This paper presents preliminary results of tests on three deficient full-scale RC beam-column T-joints strengthened with FRP composites. The joints were representative of existing RC buildings in developing countries, and therefore had inadequate detailing in the core zone. The bare joints were first tested under cyclic load to evaluate their basic seismic performance. After the initial tests, the damaged concrete in the core was removed and replaced with new concrete. The joints were subsequently strengthened using externally bonded FRP sheets and the tests were repeated. The results from the experiments were then used to compare the efficiency of the strengthening strategy at improving the seismic performance of the joints using existing guidelines. The results show that the strengthening with FRP sheets was very effective at improving the load carrying capacity of the deficient joints by up to 66%.

Keywords: beam-column exterior joints, CFRP strengthening, reinforced concrete, cyclic load

1. INTRODUCTION

Extensive damage in recent major earthquakes in developing countries has highlighted the seismic vulnerability of existing RC buildings. Many of these buildings were designed using old codes and often suffer from poor detailing in critical zones such as beam-column joints. Consequently, these structures have experienced severe damage or even catastrophic collapse. In recent years, the use of externally bonded FRP sheets has offered engineers possible solutions to strengthen existing structures and reduce their seismic vulnerability. Compared to other traditional strengthening techniques, FRP materials offer advantages such as high strength to weight ratio, high resistance to corrosion, excellent durability, ease and speed of in-situ application and flexibility to strengthen only those members that are seismically deficient. Despite the large amount of experimental research reported in literature (*fib* Bulletin 35, 2006; Bousselham, 2010), current design guidelines for FRP strengthening do not cover explicitly the seismic strengthening of deficient RC beam-column joints. Moreover, such rehabilitation guidelines need to be developed based on detailed experimental studies to establish joint performance (ACI 352R-02, 2002).

This paper investigates experimentally the efficiency of externally bonded FRP materials at improving the seismic behaviour of deficient RC beam-column joints of typical RC buildings of developing countries. Three deficient full-scale RC beam-column T-joints with poor detailing in the core zone were tested under cyclic loading. The specimens were designed to fail at the core zone where no horizontal steel stirrups were provided. After the initial tests that produced severe damage, the core zone was recast with new concrete and later strengthened using FRP sheets. This paper presents preliminary results of these experiments and compares the joint behaviour with current assessment guidelines explicitly developed for joints. These tests are part of a larger multi-stage project focused on seismic strengthening of typical substandard RC structures of developing countries still in progress at the University of Sheffield.

2. EXPERIMENTAL PROGRAMME

2.1. Geometry of specimens

The geometry of the specimens (shown in Figure 1) simulates a portion of a full-scale 2D exterior joint between contra-flexure points of a storey in a multi-storey moment-resisting frame. The column was 2700 mm long and had a cross section of 260×260 mm. Its main longitudinal reinforcement consisted of 16 mm bars. To represent typical pre-seismic construction practices, the longitudinal bars of the column were lapped just above the core zone as shown in Figure 1. A lap length of l_b=25d_b was selected as it is commonly found in many existing substandard buildings of developing countries. To prevent a shear failure, the column was reinforced with 8 mm transverse stirrups spaced at 150 mm centres. To replicate old construction practices, the core zone had no transverse stirrups.

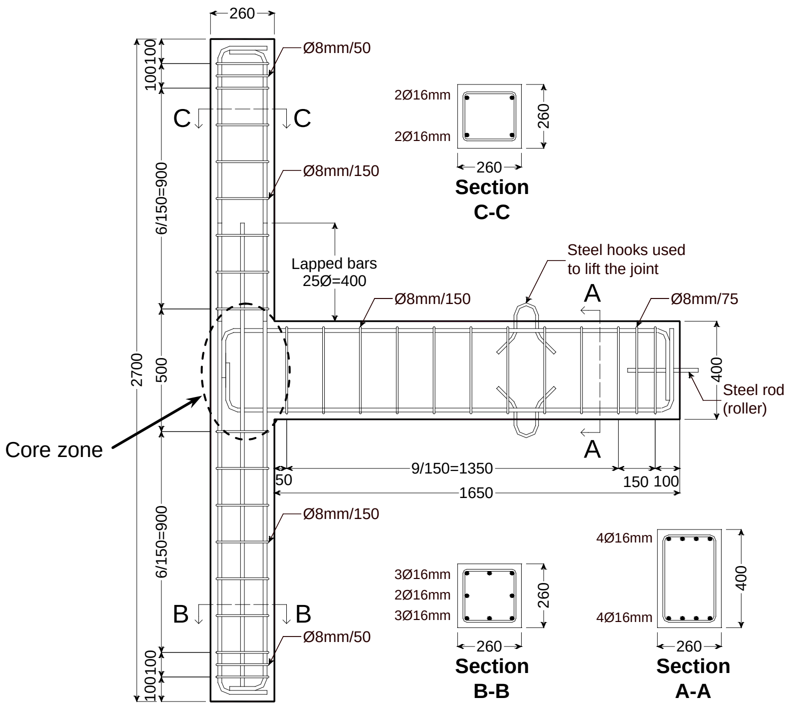

Figure 1. Geometry of beam-column joints

The beam had a length of 1650 mm and a cross section of 260×400 mm. The flexural reinforcement consisted of 16 mm bars as shown in Figure 1. The beam had sufficient transversal reinforcement consisting of 8 mm stirrups spaced at 150 mm centres. To study the effect of deficient bar anchorage, the beam reinforcement of each joint was detailed as shown in Figure 2. As can be seen, the bottom beam reinforcement was only anchored into the joint for a length of 220 mm (approximately 14d_b) and no hooks or bends were provided. This short anchorage length was estimated to be insufficient to develop the full capacity of the 16 mm bars.

To get maximum benefit of the joints, the tests were performed in two stages. In stage 1, the joints were tested in bare (as-built) condition to evaluate their basic seismic performance. As significant damage occurred at the core zone after the initial tests, the damaged concrete was removed and replaced with new concrete. After this rehabilitation, the joints were strengthened using externally bonded FRP sheets and the tests were repeated in stage 2.

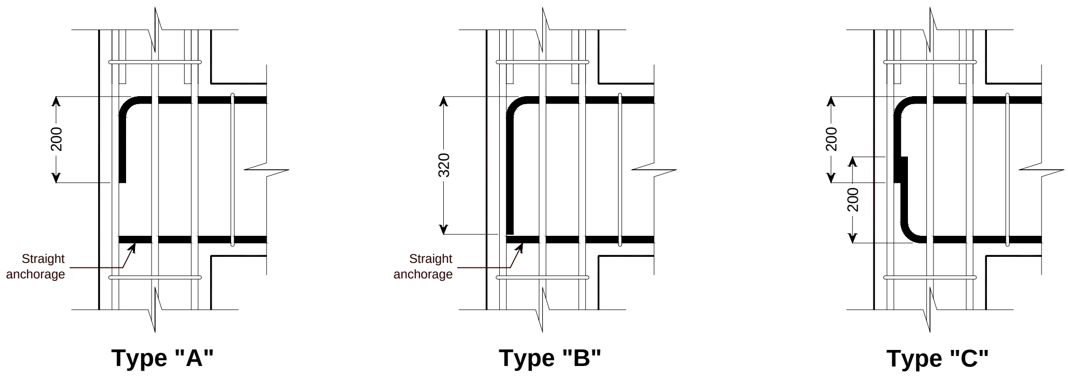

Figure 2. Types of detailing of beam reinforcement at the core zone

Table 2.1 summarises the main characteristics of the joints. The specimens were identified using an ID code (see Table 2.1). The first character of the ID identifies the joints using a “J” letter, whilst the second letter stands for the type of detailing of the beam reinforcement (e.g., letter “A” represents detailing Type “A” as shown in Figure 2). The letter following the number specifies the condition of the joints: “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 later strengthened with FRP sheets. Note that the corresponding ID of the joints tested in bare condition omits the last letters. As an example, the joint JB-2RF had detailing Type “B”, its original core was replaced with new concrete after the tests in bare condition, and was strengthened with FRP sheets. Note also that joint JB-2R was subjected to further tests (stage 3) to investigate the effect of the core replacement.

Table 2.1. Characteristics of the tested beam-column joints

Test stage	ID	$f_{c,joint}$ (MPa)	$f_{c,core}$ (MPa)	Test condition
1	JA-2	32.0	-	Bare joint
	JB-2	31.3	-	Bare joint
	JC-2	32.0	-	Bare joint
2	JA-2RF	32.0	54.2	FRP-strengthened JA-2; new recast core
	JB-2RF	31.3	55.3	FRP-strengthened JB-2; new recast core
	JC-2RF	32.0	56.9	FRP-strengthened JC-2; new recast core
3	JB-2R	31.3	53.7	Rehabilitated JB-2RF; new recast core

2.2. Material properties

The joints were cast using two batches of ready mixed concrete. For each batch, the mean concrete compressive strength of the joint ($f_{c,joint}$, see Table 2.1) was obtained from tests on three 150×300 mm concrete cylinders. The reinforcement of the joints consisted of high ductility ribbed bars Grade 500. Yield and ultimate strength of steel reinforcement obtained from three test samples were 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.

2.3. Instrumentation and test set-up

Due to the limited height of the ring frame, the joints were tested with the column in horizontal position as shown in Figure 3. The joint was supported on steel plates and rollers at both ends of the column, which in turn were supported on stiff steel I-shaped beams. The plates and rollers were held in place using high strength steel rods clamped to the stiff beams. The beam tip displacement and vertical movement at the column supports were monitored using Linear Variable Displacement Transducers (LVDTs). The strain developed along the beam and column steel reinforcement was measured using foil-type electrical resistance strain gauges.

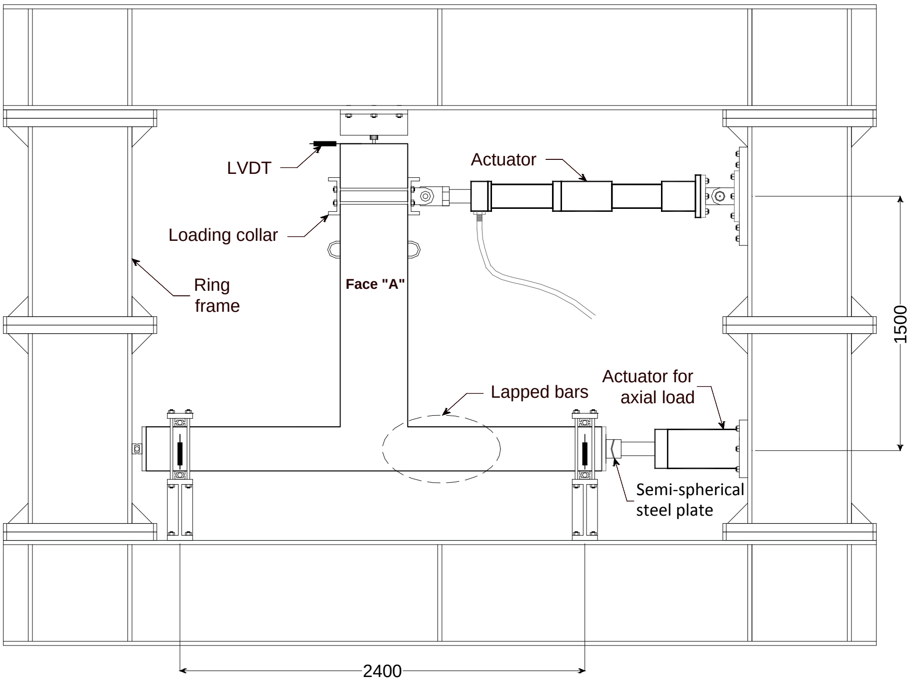

Figure 3. General test set-up for beam-column joints

2.4. Tests in stage 1

The load was applied to the beam using a servo-hydraulic actuator in displacement control (see Figure 3). Three full push-pull cycles were performed at beam tip displacements of ±4.2, ±8.4, ±12.5, ±16.7, ±25.0, ±33.4, ±50.1 and ±66.8 mm. These displacements correspond to drift ratios (defined here as the ratio of beam tip displacement to beam length) of δ = 0.25%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0% and 4.0%. A second actuator applied a constant axial load of 150 kN to the column using a semi-spherical steel plate (see Figure 3). The tests were halted when the load carrying capacity of the joint dropped to approximately 50% of the peak load capacity.

2.5. Concrete replacement in core zone

As expected, the initial tests in bare condition (stage 1) produced severe damage in the core. Hence, this concrete was removed and replaced with new concrete. The main objective of this rehabilitation was to achieve a joint with a sound core and limited damage in beam and column to perform further tests. Each joint core was recast using a different batch of concrete. The mean concrete compressive strength of the rehabilitated cores $f_{c,core}$, is summarized in Table 2.1.

2.6. Installation of FRP sheets

After the concrete replacement, the joint core was strengthened using externally bonded Carbon FRP (CFRP) sheets. In addition to preventing the premature failure of the core zone, the application of CFRPs aimed to improve the anchorage of the bottom beam reinforcement, avoid lap failure in the column reinforcement, and produce a beam mechanism (strong column-weak beam). The strengthening was done using Tyfo® SCH-41.5X Composite system and was supplied by FYFE Europe SA. The mechanical properties of the dry fibres as provided by the manufacturer are f_{fu}=4140 MPa, E_f=241 GPa, ε_{fu}=1.7% and t_f=0.185 mm. The CFRP strengthening was designed assuming that the total shear capacity of the core was the sum of concrete and CFRP shear reinforcement contributions. Current design guidelines and models for shear strengthening limit the maximum allowable strain developed in the FRP sheets to a value of 0.004 (e.g. ACI 440.2R-02, 2002). However, this work adopts a slightly less conservative strain value of 0.0045 based on previous research at the University of Sheffield (Guadagnini et al., 2006).

A general view of the strengthening strategy is presented in Figure 4. Before applying the CFRP sheets, the uneven concrete surfaces were smoothed using a grinding tool. The sharp corners of the specimens were also rounded off to a radius of approximately 10 mm. To improve the adherence between the existing concrete and the fibre sheets, the concrete surfaces were wire brushed and cleaned thoroughly with pressurised air. As shown in Figure 4, U-shaped CFRP sheets were used to strengthen the core zone of the joints (① in Figure 4). Confinement sheets (②) were then wrapped around the beam to prevent premature debonding of the U-shaped sheets. The U-shaped sheets fixed around the core were also used to restrain a possible pullout failure of the bottom beam bars. The flexural capacity of the columns was increased using CFRP sheets fixed parallel to the column axis (③ and ④ in Figure 4). Note that 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 fixed on faces A and B of the column. An additional layer of CFRP (⑤) was fixed on faces A and B to prevent movement/dropping of sheets ③ during the subsequent installation of the confinement sheets. Finally, CFRP confinement was used (⑥ and ⑦) to increase the ductile capacity of the column and to avoid premature debonding of ③, ④ and ⑤. Table 2.2 summarises the number of CFRP sheets used for each joint. All the sheets were applied using a wet lay-up procedure.

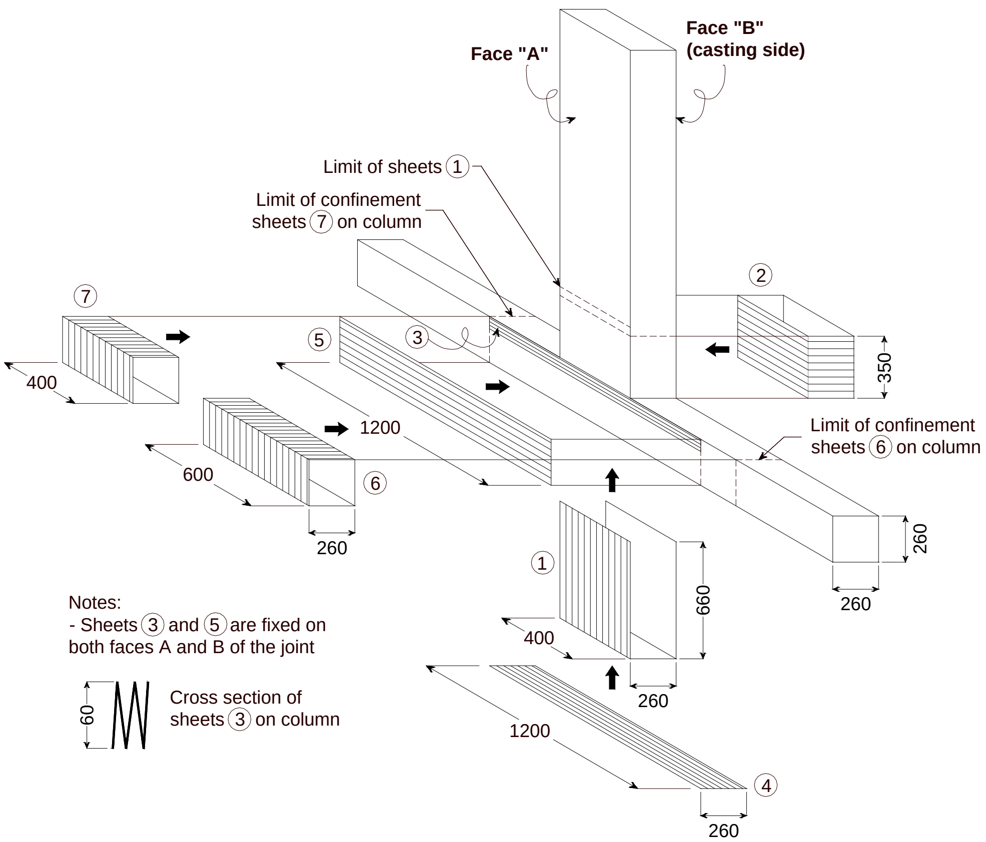

Figure 4. CFRP strengthening strategy used in beam-column joints

Table 2.2. Number of CFRP layers used to strengthen the beam-column joints

ID	Number of layers						
	①	②	③[a]	④	⑤[a]	⑥	⑦
JA-2RF	2	1	2	2	2	1	1
JB-2RF	3	2	2	2	2	2	2
JC-2RF	3	2	2	2	2	2	2

[a] One layer was applied on face A and one layer on face B

2.7. Tests in stage 2

Seven days after the installation of the CFRP sheets, the cyclic tests described in stage 1 were repeated. However, due to restrictions in the total testing time, some cycles of the pattern used in stage 1 (±12.5 and ±25.0 mm) were not applied. The same axial load used for the initial tests in bare condition (150 kN) was applied to the column.

2.8. Test in stage 3

After the test performed in stage 2, the core zone of joint JB-2RF was rehabilitated again to repeat the test. The rehabilitated joint is identified as JB-2R in Table 2.1. The rehabilitation included the stripping off the CFRP sheets, removal of the damaged concrete in the core zone and recast of the core using new concrete. The loading of the specimen and test protocol used in this test stage were similar to those used in stage 2.

3. EXPERIMENTAL RESULTS

3.1. Tests in stages 1 and 3

The performance of the joints was very similar and typical results are summarised here. The first cracks in the joints were visually detected at the beam column interface during the first load cycles at ±8.4 mm. Diagonal X-shaped cracks appeared in the concrete core at a displacement of ±12.5 mm. As the displacement increased up to the value corresponding to the maximum shear strength of the joints, the diagonal cracks widened and additional cracks appeared across the joint core. Final failure of the joints was dominated by extensive cracking and eventual spalling of concrete within the core. Typical failure modes for the joints tested in stage 1 and 3 are shown in Figures 5(a) and (b), respectively.

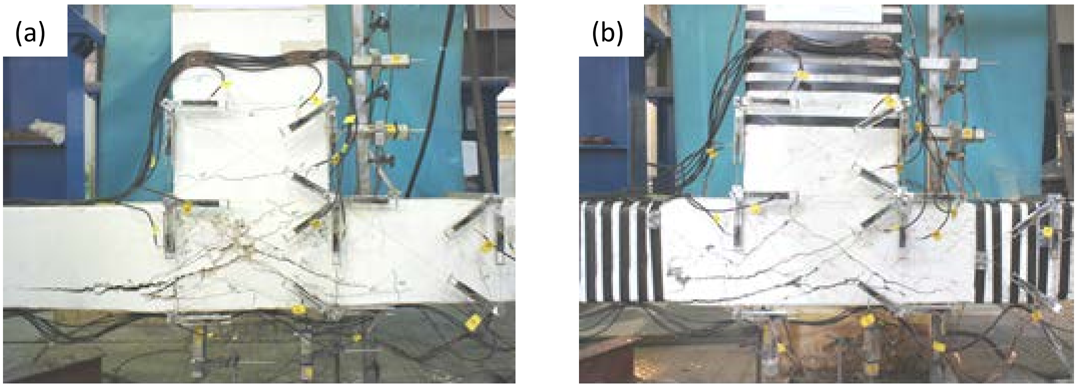

Figure 5. Typical failure modes of joints (a) JB-2, and (b) JB-2R, face A

Figures 6(a) and (b) show the load-beam tip displacement responses for joints JB-2 and JB-2R, respectively. The results in Figure 6 show that despite the lack of steel stirrups, the shear failure of the core was gradual and the joints were able to sustain significant levels of deformation. Figures 6(a) and (b) also provides information on the effect of the core replacement on the strength of the joints. It is shown that compared to the original bare joint JB-2, the core replacement increased the peak load carrying capacity of joint JB-2R by approximately 35%. Moreover, for the same level of deformation, joint JB-2R exhibited consistently higher load capacities than joint JB-2. These results confirm that, although the joints were severely damaged after the initial tests in stage 1, the new core itself was able to enhance the load carrying capacity of the joints. However, note that the concrete used in the core replacement had a higher compressive strength than the original (approximately 70%, see Table 2.1).

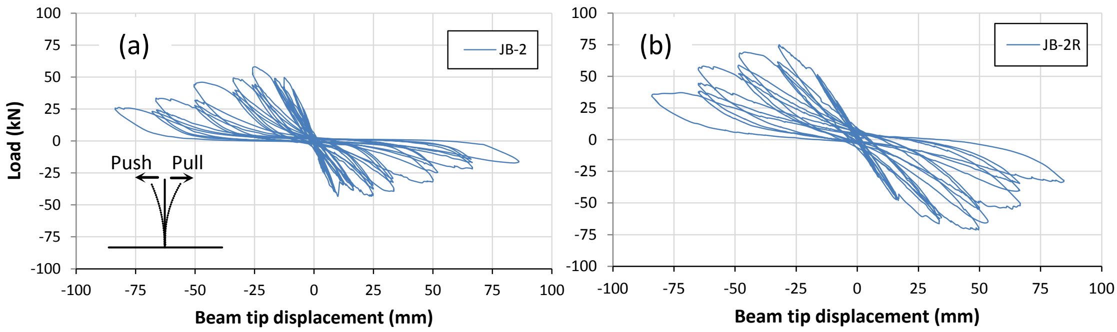

Figure 6. Typical load-beam tip displacement of joints (a) JB-2, and (b) JB-2R

3.2. Tests in stage 2

The results of joint JB-2RF are presented here, which are representative of the other CFRP-strengthened joints tested in stage 2. At the early stage of loading (±8.4 mm), the flexural cracks formed previously in the beam during test stage 1 widened further. As the cyclic displacements increased to ±16.7 mm, the CFRP sheets started to debond as evidenced by crackling sounds. At displacements of ±50.1 mm, finger tapping made evident some local debonding of sheets ③, possibly due to excessive cracking of the concrete core. Final failure was dominated by a combination of core crushing and rupture of CFRP sheets ①, ⑥ and ⑦, just above sheets ③ (see Figure 7(a)). No evident damage was observed in the columns outside the strengthened area.

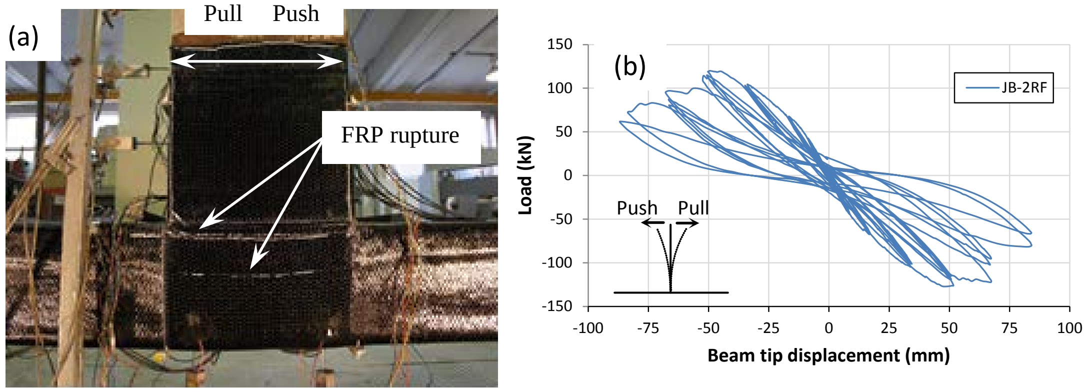

Figure 7. (a) Failure mode of joint JB-2RF, and (b) typical load-beam tip displacement of CFRP-strengthened joints

Load-beam tip displacement of joint JB-2RF is shown in Figure 7(b). As can be seen, the combination of core replacement and CFRP strengthening increased significantly the load carrying capacity of joint JB-2RF by 100% when compared to the original JB-2 joint, and by 60% when compared to joint JB-2R. Figure 7(b) also indicates that for the same levels of deformation, joint JB-2RF exhibited higher strengths than joints JB-2 and JB-2R. Readings from strain gauges also showed that some yielding of the beam reinforcement (which was one of the strengthening goals) occurred in joints JB-2RF and JC-2RF. However, beam reinforcement of joint JA-2RF remained essentially elastic. This can be due to the fewer number of CFRP layers applied to the core zone of the latter joint (see Table 2.2).

Table 3.1 summarises the test results of the tested joints. It is shown that the peak load carrying capacity (P_{max}) of the joints tested in stage 2 and 3 was always higher than the joints tested in stage 1 (bare joints). Note that the maximum load capacity of the bare joints is approximately 50% the theoretical load required to produce yielding of the beam reinforcement (P_y=106 kN). Whilst the results between joints JB-2, JB-2R and JB-2RF can be compared directly, results from the other joints tested in stages 1 and 2 are not comparable. This is because the P_{max} values include both the load carrying capacity enhancement due to the core replacement and the CFRP strengthening. Therefore, to estimate the actual load enhancement due to CFRP strengthening alone, the contribution (strength) of the recast core to the load capacity enhancement needs to be quantified first. To compute the strength of the core, this paper adopts the shear strength factor γ used by current guidelines to reflect the confinement of a joint (e.g. ACI 352R-02, 2002). Table 3.1 presents the shear strength factor γ_{exp} computed using the experimental results for the joints. Note that despite of the damage produced during test stages 1 and 2, joint JB-2R reached a shear strength factor comparable to that of the original specimens JA-2, JB-2 and JC-2 tested in stage 1. This shows that the rehabilitation of the core using new concrete was effective at restoring the initial bare strength of the joints.

Table 3.1. Summary of test results of beam-column joints (push direction)

Test stage	ID	P_{max} (kN)	γ_{exp} √MPa	γ_{P97} √MPa	γ_{ASCE} √MPa	P_{core} (kN)	P_{CFRP} (kN)	ΔP_{core} (%)	ΔP_{CFRP} (%)
1	JA-2	57.0	0.53	0.58	0.50	-	-	-	-
	JB-2	58.0	0.54	0.59	0.50	-	-	-	-
	JC-2	54.5	0.51	0.58	0.50	-	-	-	-
2	JA-2RF	86.2	0.62	-	0.50[(a)]	70.0	16.2	23%	23%
	JB-2RF	120.0	0.85	-	-	75.0	45.0	29%	60%
	JC-2RF	119.4	0.83	-	0.50[(a)]	71.7	47.7	32%	66%
3	JB-2R	75.0	0.54	0.55	0.50	-	-	-	-

[(a)] Theoretical values based on ASCE/SEI 41-06 guidelines

Table 3.1 presents the shear strength factor γ_{P97} computed using the approach suggested by Priestley (1997) for deficient joints. Table 3.1 also includes the shear strength factor γ_{ASCE} given by ASCE/SEI 41-06 (2007) to assess the strength of deficient joints. The results show that whilst the approach proposed by Priestley tends to slightly overestimate the experimental shear strength factors of joints JA-2, JB-2, JC-2 and JB-2R, ASCE 41 predicts the shear strength factor with reasonable accuracy. Based on these results, it is concluded that ASCE 41 can be used to estimate the shear strength of the joints tested in stages 1 and 3 (i.e. joints JA-2, JB-2, JC-2 and JB-2R).

Table 3 also summarises the actual load carrying capacity resisted by joints with new recast core (P_{core}) and strengthened with CFRP sheets (P_{CFRP}) tested in stage 2. For joints JA-2RF and JC-2RF, these two values were computed as follows. First, the theoretical load capacity of the joint (P_{core}) was computed based on ASCE 41. This was done using a shear factor γ=0.5√MPa and the corresponding strength of the new recast core $f_{c,core}$ (see Table 1). The actual load capacity resisted by the CFRP sheets was then calculated as the difference between the experimental load carrying capacity P_{max} and the theoretical load capacity of the joints, P_{core}. Note that for joint JB-2RF, the value P_{core} was taken directly from the results of joint JB-2R. Columns 9 and 10 of Table 3.1 present the enhancement in load carrying capacity achieved by the core replacement (ΔP_{core}) and the CFRP strengthening (ΔP_{CFRP}). The results confirm that the core replacement was effective at increasing the load carrying capacity of the joints by up to 32% when compared to the bare specimens (stage 1). Moreover, the CFRP strengthening was very effective at increasing further the capacity of the rehabilitated joints by up to 66%. It is worth mentioning that the experimental shear strength factors obtained for joints JB-2RF and JC-2RF (γ_{exp}=0.85√MPa and 0.83√MPa, respectively) are similar to the γ value used in ASCE 41 to assess the shear strength of interior joints (γ=0.83√MPa). Note also that such values are only 15 and 17% lower than the more stringent factor γ=1.0√MPa considered in ACI 352R-02 for the design of code-compliant exterior joints.

In actual existing RC frame buildings, concrete joint replacement is a rehabilitation technique seen in practice in many Mediterranean and developing countries. However, the recast of severely damaged joints would require the use of provisional supports near the rehabilitated area. Moreover, whilst one of the main advantaged of the CFRP sheets is their ease of application, it is clear that the continuity of the CFRPs along the longitudinal column axis (for instance, sheets 3 of the proposed CFRP strengthening) may be hindered by the presence of a concrete slab. In this case it would be necessary to carefully drill holes (gaps) in the slab to let the sheets pass through. In spite of the additional work, it is considered that the strengthening strategy would be justified in the case of severely damaged joints, but it is not necessarily justified for joints with limited damage. Clearly, this also depends on the importance of the building. Finally, the proposed strengthening strategy is expected to be less labour demanding than other traditional techniques such as concrete jacketing or shotcrete.

4. CONCLUSIONS

This paper presented preliminary results of tests on three deficient full-scale RC beam-column T-joints strengthened with FRP composites. The bare joints were first tested under cyclic load to evaluate their basic seismic performance (stage 1). The damaged concrete in the core was then removed and replaced with new concrete. The joints were subsequently strengthened using externally bonded FRP sheets and the tests were repeated in stage 2. From the preliminary experimental results presented in this paper, the following conclusions are drawn:

1) The initial tests (stage 1) showed that the bare beam-column joints had limited load carrying capacity of approximately 50% the plastic capacity. Final failure was dominated by extensive shear cracking at the concrete core.

2) Compared to the bare joints, the core replacement using new stronger concrete increased the load carrying capacity of the joints by more than 30%.

3) The CFRP strengthening improved further the structural behaviour of the joints rehabilitated with a new core. For the tested joints, the CFRP intervention increased the load capacity by up to 66%.

4) The results indicate that, for the joints tested in stages 1 and 3, the shear strength factor γ given by ASCE/SEI 41-06 predicts the joint strength with reasonable accuracy.

5) For the CFRP-strengthened joints JB-2RF and JC-2RF, the experimental shear factor γ is only 15 and 17% lower than the shear strength factor γ=1.0√MPa considered in ACI 352R-02 for the design of code-compliant exterior joints. Consequently, the CFRP intervention was very effective at improving the shear strength of the joints.

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