SEISMIC STRENGTHENING OF BEAM-COLUMN JOINTS WITH MULTIDIRECTIONAL CFRP LAMINATES

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Abstract

An experimental program was carried out to analyse the potentialities of a technique based on the use of multidirectional CFRP laminates (MDL-CFRP) for the seismic repair and strengthening of reinforced concrete (RC) beam-column joints. This experimental program comprises cyclic tests on three full-scale RC joints, representative of interior beam-column connections in buildings. The joints were initially submitted to a cyclic test inducing a damage pattern representative of a seismic event. Subsequently, they were repaired and strengthened with MDL-CFRP. The strengthened joints were then tested for the same loading history of the original ones up to their failure. The adopted strengthening technique uses the

MDL-CFRP that are simultaneously glued and anchored to the concrete surfaces. This technique is called Mechanically Fastened and Externally Bonded Reinforcement (MF-EBR). In the present study, the effectiveness of two different strengthening configurations was investigated. The tests are described and the main results are presented and analyzed.

Keywords: multidirectional CFRP laminates; RC beam-column joint; seismic strengthening; MF-EBR strengthening technique.

1. Introduction

The main existent techniques for repairing and strengthening reinforced concrete (RC) beam-column joints can be grouped as follows [1]: repair with epoxy (injection of epoxy resin in the cracks of lightly damaged elements); removal and replacement of concrete in more damaged areas; jacketing with RC layers, masonry blocks or steel plates; use of composite materials. Epoxy repair techniques have demonstrated limited success, whereas concrete jacketing of columns and encasing the joint region is an effective strengthening method, but requiring intensive labour [1]. The techniques based on the use of externally bonded FRP composites (EBR) can reduce some important limitations of other methods, since for this solutions the used materials are relatively easy to apply and do not modify the original geometry of the elements to strengthen, but premature debond can compromise the effectiveness of EBR technique [1].

Recently, a repair and strengthening technique that uses multidirectional carbon fiber laminates (MDL-CFRP), simultaneously glued and anchored to concrete, called Mechanically Fastened and Externally Bonded Reinforcement (MF-EBR) was proposed [2]. The efficiency of near surface mounted (NSM), EBR and MF-EBR strengthening techniques was compared by means of four-point bending tests with RC beams submitted to monotonic and fatigue loading [3]. When compared with the EBR strengthening technique, the MF-EBR has shown an important performance improvement in terms of load carrying capacity (of about 40%) and deflection capacity (of about 140%) [3].

To assess the potentialities of the MF-EBR technique for seismic retrofitting, three full-scale interior RC beam-column joints were strengthened and tested under cyclic loading. These joints were representative of interior beam-column connections of the buildings construction practice in Southern European countries until the early 1970s. The joints were initially submitted to a cyclic test inducing a damage pattern representative of a seismic event. Then, they were repaired, strengthened with MDL-CFRP and tested following the same loading history imposed to the original ones up to their failure.

In this paper the tests are described and the results are presented and discussed.

2. Experimental Program

2.1 Specimen and test configuration

Figure 1 presents the geometry of the joints, as well as the detailing of the beam and column cross sections adopted for all the specimens. In the beams, with a cross section of 0.30 m wide and 0.40 m height, the longitudinal reinforcement was composed of 2 steel bars of 12 mm diameter (2Ø12) at the top and 4Ø12 at the bottom. The transverse reinforcement consists of 8 mm diameter stirrups spaced 0.20 m. In the columns, with square cross-section of 0.30 m edge, the longitudinal reinforcement was composed by 4Ø12 and the transverse reinforcement was formed by 8 mm diameter stirrups spaced 0.25 m. The concrete cover was 20 mm thick for the beams and columns of all elements.

Figure 2 presents the adopted test setup and the instrumentation applied. The beams are simple supported at their extremities, and the “lower” column is supported in both directions. The transverse and axial loads at the top of the “upper column” were applied by two hydraulic actuators equipped with two load cells C1 and C2 (see Figure 2b) to measure the corresponding forces. Additionally, one load cell (C3 in Figure 2b) was used to register the horizontal reaction at the base of the column and another one to register the vertical reaction at the same point (C4 in Figure 2b). Several inductive linear position sensors and linear variable differential transducers were used to measure relative displacements along the specimen. Further details about the test configuration and instrumentation can be found elsewhere [4, 5].

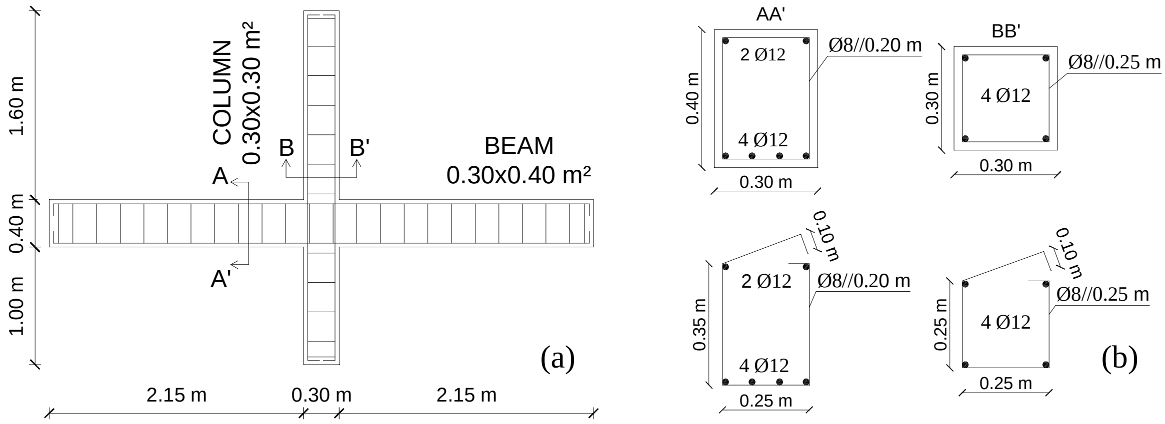

Figure 1. Geometry and reinforcement detailing: (a) joint; (b) cross-sections of the beams and columns.

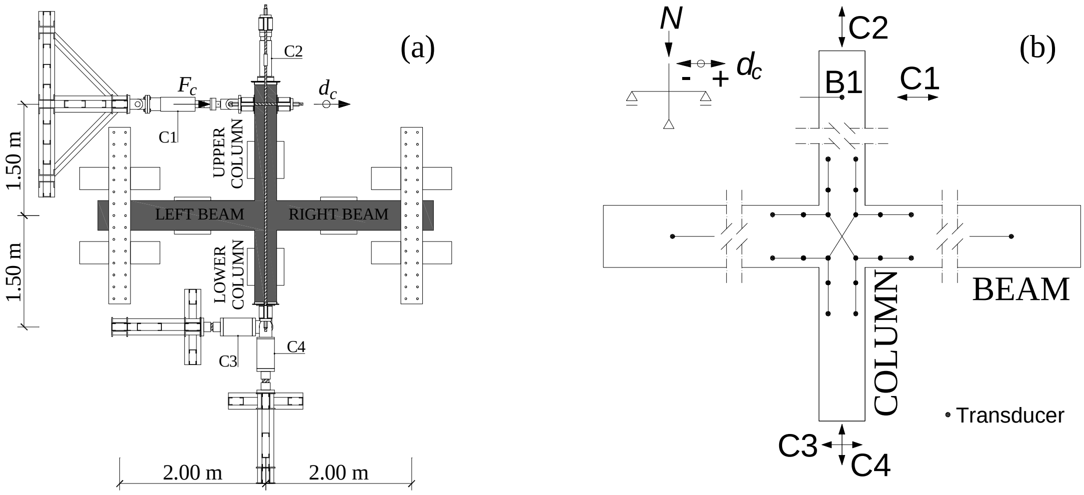

Figure 2. (a) Testing setup; (b) instrumentation.

All the tests were carried out under displacement control at B1 (see Figure 2b). The imposed law consisted on applying complete reversal cycles throughout eighteen displacement levels of increasing amplitude: ±1 mm, ±2 mm, ±4 mm, ±6 mm, ±10 mm, ±15 mm, ±20 mm, ±25 mm, ±30 mm, ±40 mm, ±50 mm, ±60 mm, ±70 mm, ±80 mm, ±90 mm, ±100 mm, ±110 mm and ±120 mm. From level ±1 mm to ±4 mm only one complete cycle per displacement level was performed. From level ±6 mm to the end of the test three complete cycles per level were applied.

The experimental program was developed in two different phases. Initially, the three joints (denominated JD, JPA1 and JPA2) were submitted to a cyclic loading in order to introduce typical damages in these elements due to seismic actions. All the details about this testing phase can be found elsewhere [5]. In the second phase, all the joints were strengthened with MDL-CFRP according to the strengthening configurations and detailing represented in Figure 3. In this phase, the nomination adopted for the retrofitted joints is JDR, JPA1R and JPA2R.

In both phases, an axial force of 200 kN was applied at the top of the column (by actuator C2) before starting the cyclic test. This axial force corresponds to a reduced axial force (ν) of about 10%, which is a typical value for RC columns in external frames with spans of about 4 m, of buildings with 2 or 3 storeys. The axial force is kept constant during the entire cyclic test for all specimens tested.

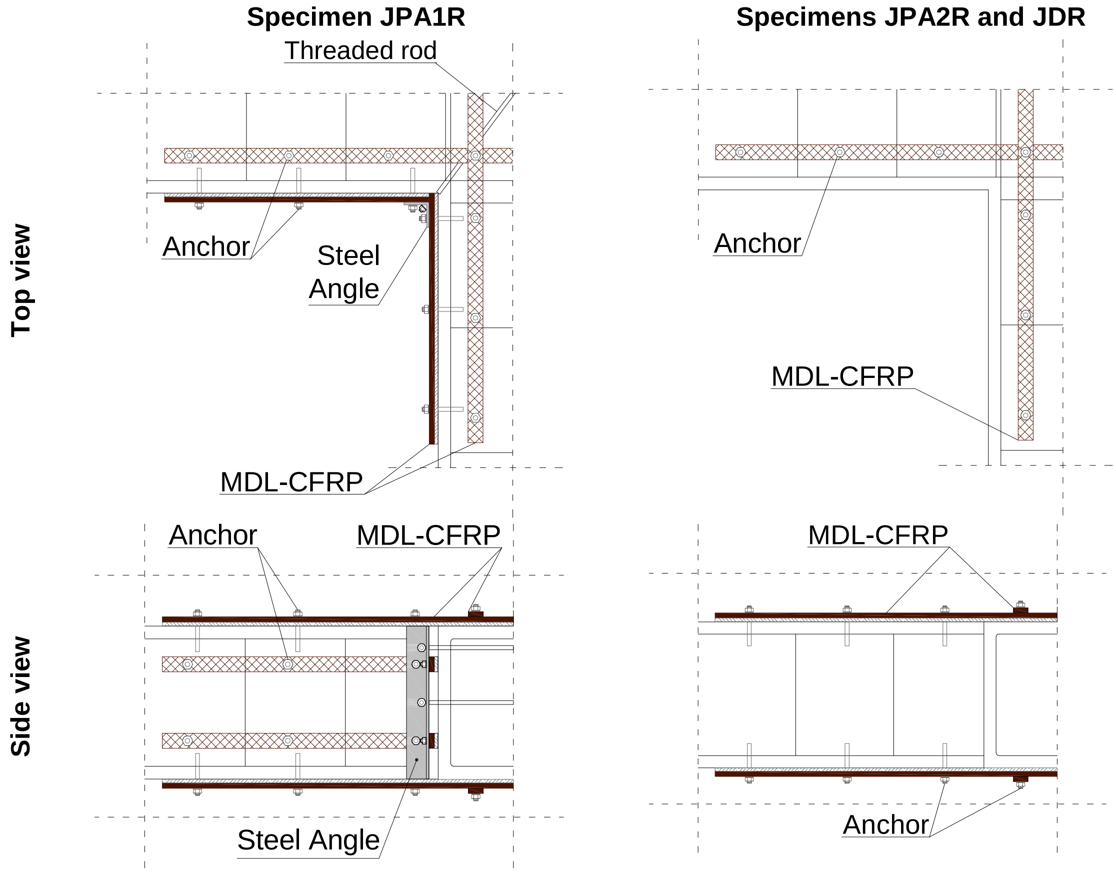

Figure 3. Strengthening solutions detailing adopted in the context of the present work.

2.2 Material characterization

The mechanical characterization of the concrete used in this work was assessed by means of compression tests on cubic concrete specimens (150 mm wide). From these tests, an average compressive strength of 23.5 MPa was obtained.

The mechanical properties of the plain steel rebars (specimens JPA1, JPA2, JPA1R and JPA2R) were evaluated through tensile tests according to EN 10002-1:1990 (PT). From these tests, an average value of 590 MPa, 640 MPa and 198 GPa was obtained for the yielding and ultimate strengths, and for the modulus of elasticity, respectively. The ribbed rebars properties (specimens JD and JDR) were assumed equal to those of current A400 NR steel used in RC buildings construction (NP EN 1992-1-1:2010).

The MDL-CFRP used to strengthen the joints was designed and produced in the framework of the current research project. All the information related to its development and characterization is available elsewhere [2]. From this characterization, the following average values were obtained: tensile strength of 1866 MPa; modulus of elasticity of 118 MPa; strain at failure of 1.58%; bearing unclamped resistance of 316 MPa; bearing clamped resistance of 604 MPa; thickness of 2.07 mm.

The S&P Resin 220 epoxy adhesive® was adopted to glue the MDL-CFRP to concrete surfaces. To mechanically fix the MDL-CFRP to concrete, a Hilti system composed by the resin HIT-HY 150 MAX, the HIT-V M8 8.8 threaded anchors and DIN 9021 washers was adopted. The anchors were pre-stressed using a torque of 40 N·m. The main properties of these materials can be found elsewhere [2].

2.3 Specimens repair and strengthening

Repair and strengthening of the joints involved three main phases (see Figure 4): joint reconstruction with mortar, crack sealing with a chemical adhesive and MDL-CFRP application. All the details about these steps can be found in [3].

Figure 4. Specimens repair and strengthening: (a) joint reconstruction; (b) crack sealing; (c) MDL-CFRP application.

3. Results

Figure 5 shows, for all the tested specimens, the global response in terms of lateral force (C1 – see Figure 2a) *versus* lateral displacement at the top of the column (B1 – see Figure 2b), as well as the corresponding envelopes. Table 1 presents the main results obtained in terms of maximum force reached and the corresponding displacement (in both directions) for the original (JD, JPA1 and JPA2) and strengthened (JDR, JPA1R and JPA2R) specimens.

For JDR and JPA1R joints, the adopted repair/strengthening strategy provided an increase of load carrying capacity, when compared with the obtained for the corresponding original specimens (first phase). In fact, the load carrying capacity increase was about 10% and 35% for JDR and JPA1R specimens, respectively. The strengthening strategy applied in the damaged JPA2R specimen, restored the stiffness and the lateral force load capacity of the original JPA2 specimen, which is notable, considering the damage level previously installed.

Comparing the responses for the original and strengthened joints, in terms of force-drift envelopes, two distinct results can be observed. For JPA1R and JPA2R, the initial stiffness of the corresponding reference specimens was recovered, but not for JDR, which can be justified by the repairing solution adopted. In fact, for the specimens with plain rebars (JPA1R and JPA2R), apart from concrete spalling at the joint corners in the first phase of the tests, it was observed only a single large crack at the extremity of each beam and column element (JPA1 and JPA2). Thus, these damages were easily detected and repaired for theses joints. However, for the specimen with ribbed rebars (JDR), in addition to the concrete crushing, several cracks with distinct widths occurred during the test of the first phase. In this case, the crack sealing strategy adopted was unable of restoring the integrity of the original joint.

Comparing the response of JDR and JPA2R joints in terms of loading carrying capacity, it is observed a better performance for the former. In fact, the location of concrete crushing of JPA2R specimen was closer to the corners of the joint, when compared to the obtained in JDR (see also Figure 6). This aspect is critical, since the flexural carrying capacity of a section is assured by the reinforcement under tension and the concrete under compression. Since premature concrete crushing of JPA2R was observed, an inferior performance is expected for this joint.

JPA1R and JPA2R presented very different responses. For the case of JPA1R insignificant stiffness degradation was observed up to about 35 kN, whereas for the specimen JPA2R a successive stiffness degradation is observed from the beginning of the test. The better performance of JPA1R is justified, by the threaded rods that avoid the concrete crushing at the corners of the joint and confine the core of the joint, and also by the strengthening strategy solution adopted for this specimen (see Figure 3).

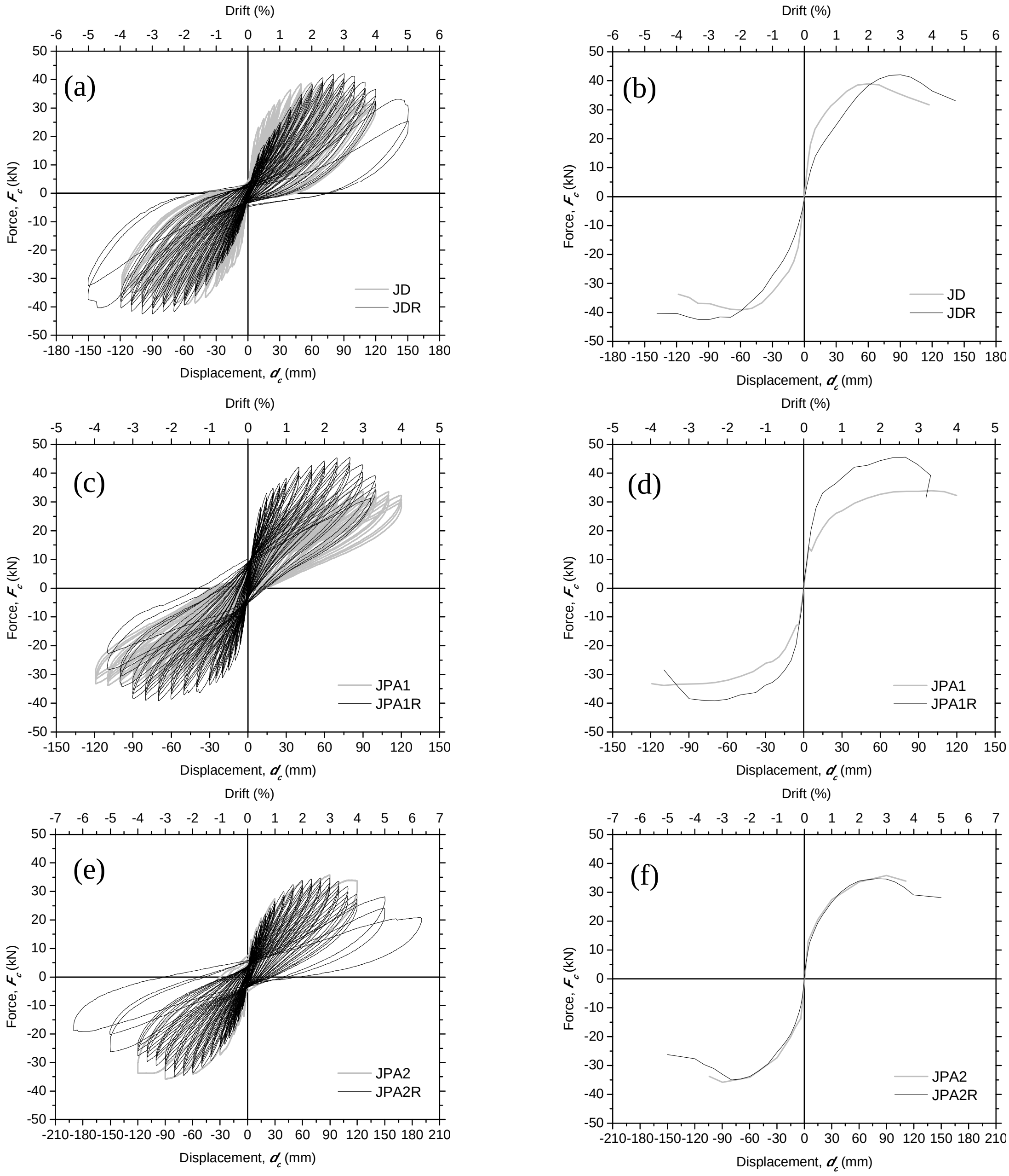

Figure 5. Force *vs*. displacement: (a) JD and JDR; (b) JD and JDR envelopes; (c) JPA1 and JPA1R; (d) JPA1 and JPA1R envelopes; (e) JPA2 and JPA2R; (f) JPA2 and JPA2R envelopes.

Table 1. Summary of the main results.

Specimen	**Negative direction**		**Positive direction**	
	$F_{c,max}$ [kN]	$d_{c,max}$ [mm]	$F_{c,max}$ [kN]	$d_{c,max}$ [mm]
JD	-39.14	-59.04	38.9	59.81
JDR	-42.48 (9%)	-89.51	42.11 (8%)	90.30
JPA1	-33.85	-109.72	33.85	100.13
JPA1R	-39.22 (16%)	-69.76	45.54 (35%)	79.68
JPA2	-35.85	-89.73	35.84	89.98
JPA2R	-35.06 (-2%)	-79.80	34.78 (-3%)	79.55

Note: the values in brackets represent the increase relatively to the original non-strengthened specimens.

Figure 6 presents the failure modes of the joints, which were characterized basically by flexural cracks and concrete spalling at the corners of the joints. Comparing the damages for the tested strengthened joints, JPA2R presented the higher level of concrete spalling at the corners of the joint. This damage justifies the inferior performance of the strengthening strategy adopted in this specimen. On the other hand, the use of diagonal threaded rods (see also Figure 3) may have contributed for the delay of concrete spalling, justifying a better performance observed for specimen JPA1R.

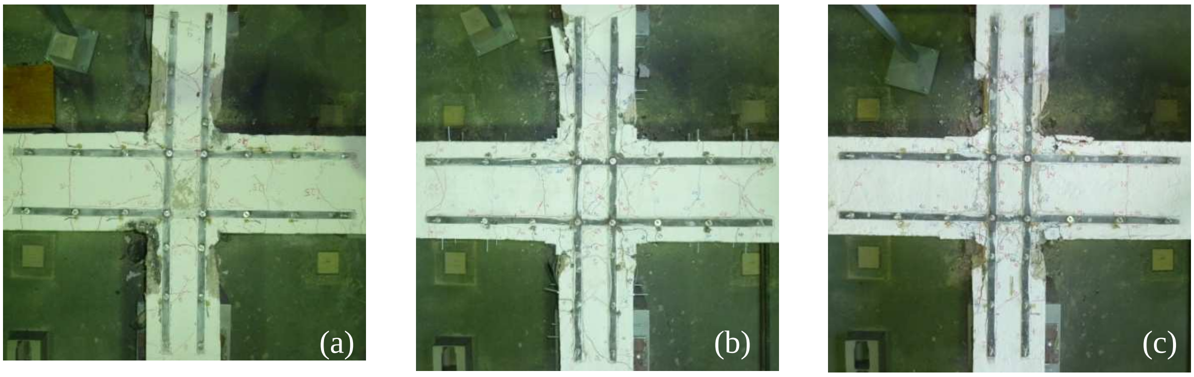

Figure 6. Failure modes of the tested joints: (a) JDR; (b) JPA1R; (c) JPA2R.

the evolution of the stiffness degradation of the specimens for different drift levels. The secant stiffness was calculated for the maximum strength, positive and negative, for each half-cycle of the curves presented in Figure 5. This simplified procedure to evaluate the stiffness degradation evolution provides valuable information on the influence of the strengthening strategy on this phenomenon. As was expected, specimens JPA1R and JPA2R yielded better performance.

Table 2 presents the evolution of the dissipated energy for each imposed drift level in the tests. Until the imposed drift level of 4%, corresponding to the lateral displacement of ±120 mm, JD specimen dissipated more energy than JDR. On the other hand, specimens JPA1R and JPA2R dissipated more energy than the corresponding unstrengthened specimens. As Figure 7 evidences, the recovery of the initial stiffness and the observed inferior stiffness degradation contributed determinedly to this result. In this figure is represented the evolution of the stiffness degradation of the specimens for different drift levels. The secant stiffness was calculated for the maximum strength, positive and negative, for each half-cycle of the curves presented in Figure 5. This simplified procedure to evaluate the stiffness degradation evolution provides valuable information on the influence of the strengthening strategy on this phenomenon. As was expected, specimens JPA1R and JPA2R yielded better performance.

Table 2. Dissipated energy.

Drift [%]	**Dissipated Energy** [kN·m]				
	JD	**JDR**	**JPA1**	**JPA1R**	**JPA2R**
1	2.65	2.06 (-22%)	2.84	3.98 (40%)	2.83 (-1%)
2	8.08	6.90 (-15%)	7.63	10.54 (38%)	8.75 (15%)
3	18.13	16.64 (-8%)	16.02	25.56 (60%)	19.49 (22%)
4	31.42	26.39 (-16%)	28.32	33.96 (20%)	34.99 (24%)

Note: the values in brackets represent the increment for JDR relatively to JDR and JPA1R and JPA2R to JPA1.

4. Conclusions

In the present work three intensively damaged RC beam-column joints were repaired and strengthened using the MF-EBR strengthening technique. The strengthening technique adopted uses multidirectional laminates of CFRP that are simultaneously glued and anchored to concrete. The experimental program comprises cyclic tests on full-scale RC joints, representative of building' interior beam-column connections, two with smooth plain bars and the other with ribbed bars.

In general, the repairing technique, which consists on joint reconstruction with mortar and crack sealing with a chemical adhesive, was very efficient for the case of the joints with plain bars. The main reason for that was the simplicity of sealing the cracks in the joints with smooth bars from previous characterization tests, since only a single large crack exists. In these joints the initial stiffness was recovered. The strengthening technique which avoided the concrete crushing at the corners was very efficient, leading to an increase of strength capacity (up to 35%). For this case the increase in terms of dissipated energy was very significant.

The failure modes of the strengthened joints included basically flexural cracks and concrete spalling at the corners of the joints (for the joints where concrete damage at corners was not avoided).

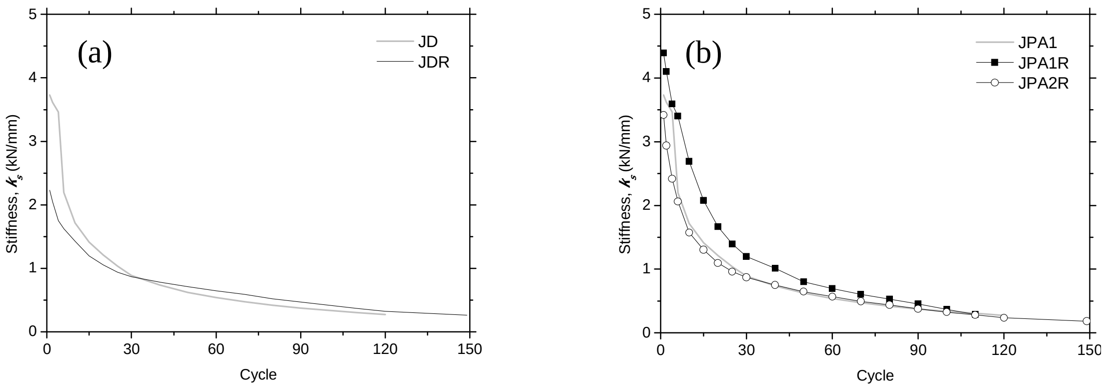

Figure 7. Stiffness degradation: (a) JD/JDR; (b) JPA1/JPA1R/JPA2R.

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