Corner Beam-Column Joints Seismic Retrofitting With High Performance Fiber-Reinforced Concrete Jacketing

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SUMMARY:
The experimental results on full-scale corner beam-column joints tests are presented herein, with the aim of studying the effectiveness in strengthening existing RC existing structures of the application of a HPFRC jacket.
The specimens subassemblies have been designed with structural deficiencies typical of the Italian construction practice of the 60's-70's: absence of any capacity design principle, use of smooth bars, inadequate reinforcement detailing, such as total lack of stirrups in the joint panel and hook-ended anchorage.
Both unretrofitted and retrofitted specimens have been tested under simulated seismic loads.
The results underline the significant vulnerability of the joint panel region and the critical role of the slippage phenomena related to the use of smooth bars and show that, with the application of a HPFRC jacket, it is possible to increase the bearing capacity of the columns, reaching also an adequate level of ductility and strength of the beam column joints.

Keywords: existing RC structures, seismic retrofitting, HPFRC jacketing, beam-column joint.

1. INTRODUCTION

The Abruzzo earthquake (6th April 2009) dramatically demonstrated that a large amount of the Italian existing RC structures, designed only for gravity loads, was not able to sustain the earthquake actions, mainly due to structural deficiencies related to scarce material properties, usually low-strength concrete; absence of capacity design principles; inadequate confinement in the potential plastic regions, typically no transverse reinforcement in the joint regions; poor reinforcement detailing, such as insufficient amount of column longitudinal reinforcement, inadequate anchorage detailing, lapped splices of column reinforcement just above the floor level, use of smooth bars for both longitudinal and transverse reinforcement.
From the observation of the effects of past earthquakes, it is widely recognized that beam-column joints represent a critical region in frame buildings subjected to seismic loads of high intensity. At the global level, a weak-column/strong-beam system results, with the risk to develop soft-storey mechanisms. At the local level, inadequate protection of the panel zone region within beam-column joint subassemblies is expected as well as brittle failure mechanisms of structural elements.
The strengthening of existing RC structures and the evaluation of the seismic response of existing RC buildings designed before the introduction of adequate seismic code provisions have thus become an urgent issue in Italy.
During the last decades, several techniques have been proposed for the seismic retrofitting of RC elements (Fib Bullettin 24 2003, Fib Bullettin 35, 2006, Fib Report 1991). Concerning the strengthening of existing columns, the possibility of using RC jackets is usually considered, in particular when the elements are made of low strength concrete. Traditional jacketing presents some inconvenience, due to the jacket thickness being governed by the steel cover, which often leads to a jacket thickness higher than 70-100 mm, with a consequent increase of both mass and stiffness,

requiring special attention with respect to the overall seismic response of the retrofitted structure.
The application of external bonded FRP composites provides a practical solution to improve the overall performance of a RC frame structure, offering several advantages, related to its high strength-to-weight ratio, resistance to corrosion, fast and relatively simple application. Furthermore, FRP wrapping is useful to enhance ductility, but is not suitable when a noticeable strength increase of the column is also needed. On the other hand FRPs may constitute a problem for fire resistance.
An alternative technique based on the use of thin jackets made with High Performance Fiber Reinforced Concrete (HPFRC) has been developed by Martinola et al. (2007), Maisto et al. (2007). This technique has been demonstrated effective for the strengthening of existing columns if compared with other techniques, particularly when the structure is made of low strength concrete (Beschi et al. 2011). The proposed solution consists in encasing structural concrete elements in a thin layer of HPFRC (30-40 mm), after sandblasting of the existing concrete surface. Due to the high bond properties of the HPFRC material, no bonding agent is usually needed (Martinola et al. 2007). The HPFRC material adopted exhibits a hardening behaviour in tension coupled with a high compression strength, larger strain capacity and toughness when compared to traditional FRCs, which makes them ideal for use in members subjected to large inelastic deformation demands. Furthermore, the proposed solution would use a retrofitting material which is more similar to the host material than any of the solutions seen above.

2. EXPERIMENTAL TESTS

In the following the results of cyclic experimental tests on four corner beam-column joints (two unretrofitted and two retrofitted) are presented.

2.1 Specimens geometry and materials details

2.1.1. Unretrofitted specimens

The unretrofitted specimens CJ1 and CJ2 are representative of a corner joint of the first level of a RC four-storey frame designed according to the Italian design provisions in force before the '70s provided by the national standards (R.D. 16/11/1939) and suggested by the technical literature of that time (Santarella 1945).

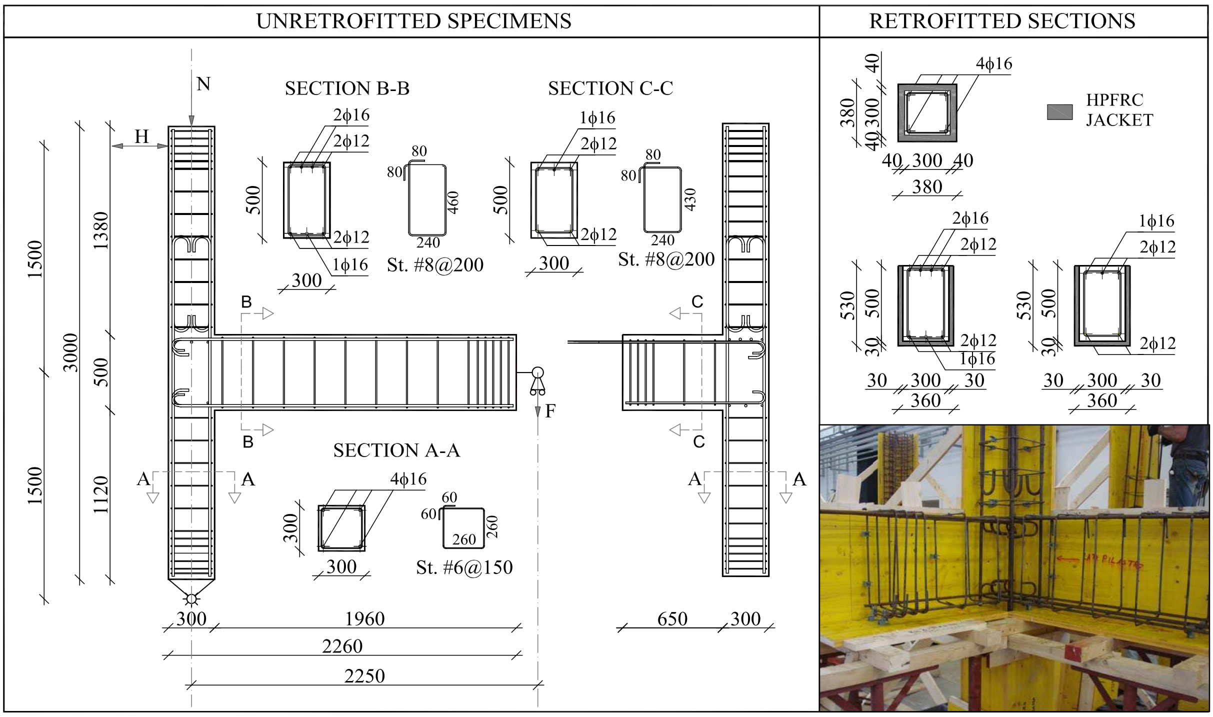


Figure 2.1. Geometry and reinforcement details for specimens CJ1 and CJ2

The structural elements have been designed only for gravity loads: the columns carry only axial force and the beams are designed according to the scheme of continuous beam on multiple supports, with upper reinforcements at the beam ends to control the crack width for service load.
The beams are characterized by a 30 × 50 cm cross section, with smooth reinforcing bars with hooked-ends anchorages. In the main beam 2 Ø12 and 2 Ø16 mm diameter longitudinal rebars were placed at the top and 2 Ø12 and 1 Ø16 mm diameter rebars were placed at the bottom with Ø8@200 mm stirrups . In the secondary beam 2 Ø12 and 1 Ø16 mm diameter have been placed at the top and 2 Ø12 mm diameter at the bottom.
The column cross section is 30 × 30 cm, with 4 Ø16 mm diameter longitudinal rebars and lap splices with hooked-end anchorages and with Ø6@150 mm stirrups. No transverse reinforcement have been placed inside the joint, as it was a common practice in the '60s-'70s.
The geometry and reinforcement details are shown in Figure 2.1.
As for the materials used, the concrete was characterized by an average compressive strength of about 38.7 MPa, while the characteristics of the reinforcing steel bars are listed in Table 2.1.

Table 2.1. Reinforcement characteristics

REINFORCEMENT			
Bar diameter	Yield strength [MPa]	Ultimate strength [MPa]	Ultimate strain [MPa]
ϕ12	365	558	15.91
ϕ16	445	546	13.66
ϕ6	493	556	16.14
ϕ8	337	440	21.03

Table 2.2. HPFRC characteristics

HPFRC			
Cementitious matrix	Compressive strength [MPa]	Tensile strength [MPa]	Elastic modulus [GPa]
19 gg	99.68	4.8	36
41 gg	112.55	4.8	36
Steel fibers	Equivalent length [mm]	Equivalent diameter [mm]	Fibers volume [%]
	15	0.18	1.2

2.1.1. Retrofitted specimens

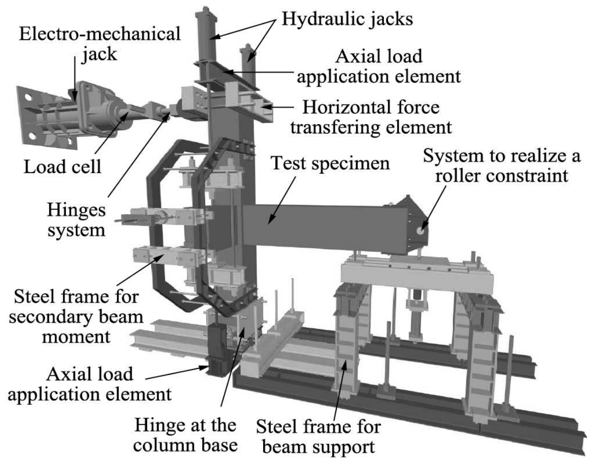
The retrofitting solution concerns the application of a HPFRC jacket to specimens with the same geometry and detail of the unretrofitted ones (Figure 2.1).
After casting and a curing period of one month, the specimens surface was sandblasted for the successive jacketing. The column was encased in a HPFRC jacket 40 mm thick while for the beam a U-shaped solution 30 mm thick was adopted (Figures 2.2 and 2.3).
Even if the mechanical characteristics of the reinforcement were the same as for specimens CJ1 and CJ2, the base concrete was characterized by an average compressive strength of about 27.01 MPa, while HPFRC characteristics are listed in Table 2.2.
The retrofitted specimens will be labelled in the following as RCJ1 and RCJ2.

2.2 Test set-up and procedure

The test set-up intended to reproduce the configuration of a beam-column subassembly in a frame subjected to reversed cyclic lateral loads. The size of the specimens is determined by the distance between the contra-flexure points (assumed to be at mid-span of the beam and at mid-height of the column) for linear elastic lateral load response of a generic moment frame.

Figure 2.2. Specimens RCJ1 surface before and after sandblasting

Figure 2.3. HPFRC jacket casting



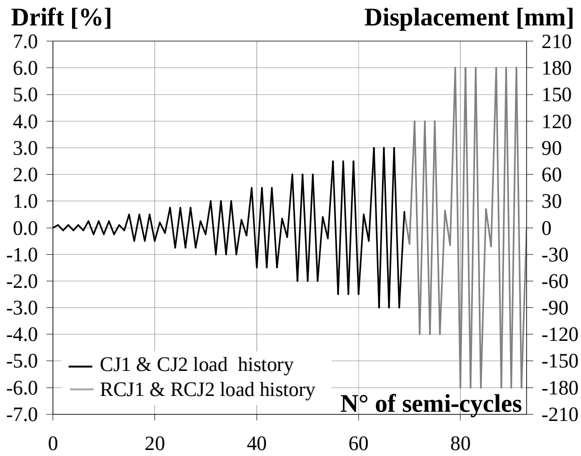


Figure 2.4. Test set-up

Figure 2.5. Load history

To this aim, a test bench has been designed in order to develop hinges at the top of the column and at the column base and a roller constraint at the end of the main beam, as shown in Figure 2.4.
The test procedure started with the application of an axial load equal to 206 kN, representing the service load acting on the column of the first level of a reference building, by means of two hydraulic jacks: the axial load was then maintained constant.
Hydraulic jacks were also used to apply a vertical force of at the main beam's end and a couple of forces at the secondary beam's end, to simulate respectively the serviceability combination of shear and moment and the serviceability moment in the joint.
The loading history consisted of cycles characterized by drift increments: 0.25% up to a drift of 1%, 0.5% up to a drift of 3% and 1% up to failure, as shown in Figure 2.5. Three fully reversed cycles were applied at each drift ratio. The test continued until a drift ratio equal to 3%, corresponding to a 90 mm top displacement for the unretrofitted specimens and ended at a drift equal to 6%, corresponding to a 180 mm top displacement for the retrofitted specimens.
Displacements and rotations were measured by potentiometric transducers. The horizontal load and the couple of forces applied to the secondary beam were monitored by means of load cells, while the vertical load applied to the main beam was measured by strain gauges applied on the bar at the beam end.

2.5. Test results

2.5.1. Unretrofitted specimens (CJ1 & CJ2)
The results in terms of horizontal load versus displacement at the level of the load application point are shown for both the unretrofitted specimens in Figure 2.6.
In the positive direction, the specimens reached their maximum strength, equal to 31.28 kN for specimen CJ1 and 34.7 kN for specimen CJ2 at a drift equal to 2% and 2.5% respectively. In the following loading cycles, the specimens exhibited a little strength degradation and at the final loading cycles, the residual load carrying capacity was approximately 98% of the maximum load for specimen CJ1 and 96% of the maximum load for specimen CJ2.
In the negative load direction, both the specimens achieved the maximum load at a 1% drift, equal to 35.98 kN and 35.41 kN for CJ1 and CJ2 respectively. After the peak value, the strength decreased more significantly for specimen CJ1 than for specimen CJ2 (63% and 76.5% of the peak value, respectively at a drift equal to 3%).
The experimental results confirmed the high vulnerability of corner beam-column joints, with significant damage in the joint core. In addition, the pronounced cyclic stiffness degradation, with pinching effect in the hysteresis loops, showed the fundamental role played by bar-slip phenomena.
The failure was characterized by the beam failure in the positive load direction, with a wide flexural crack at the interface with the joint, due to the slippage of smooth reinforcing bars and joint shear failure in the negative load direction, with the formation of a concrete wedge, combined with the

effects of stress concentration at the beam bar end-hooks, leading to a brittle local failure and a sudden loss of bearing capacity.
As shown in Figure 2.8, which represents the evolution of the cracks pattern only for specimen CJ1 (column on the left), both the specimens showed early flexural cracks in the main beam, corresponding to a drift of 0.25% in the negative direction and 0.5% in the positive one, in agreement with the test set-up, which started with the application of a top-down vertical force at the beam's end, and as consequence preliminary negative moment acting on the beam. On the outer side of the joint in correspondence with the secondary beam, no cracks appeared up to a drift of 0.75%.
The first diagonal crack in the joint panel zone started in the negative direction in the first cycle at a drift equal to 1%. In the second positive cycle, at a drift equal to 2%, two diagonal cracks appeared in the opposite direction and the concrete wedge began to take shape.
At a drift equal to 3%, severe cover spalling occurred, in particular along the vertical crack at the beam-joint interface, the upper side of the concrete wedge in the joint and in a wide area at the bottom of the joint at the secondary beam's side, due to the thrust of the hooked-end anchorages of the beam longitudinal reinforcement, while in the inner side of the joint cover spalling didn't occur due to the confinement provided by the secondary beam.

2.5.2. Retrofitted specimens (RCJ1& RCJ2)

Figure 2.7 plots the results in terms of horizontal load versus displacement for the retrofitted specimens. It can be observed that the shape of the envelope curves are well comparable for the two specimens.
It is worth pointing out that the shape of the envelope curves is typical of the behavior of a section characterized by a RC core with a HPFRC jacket. The peak value corresponds to the achievement of the maximum tensile strength in the HPFRC jacket for the fiber farther from the neutral axis. After reaching the maximum positive load, the strength of the specimens suddenly decayed until the reinforcing bars yielded, as it is evident from the plateaus in the diagram at a drift equal to +2% and +1.5% for specimen RCJ1 and RCJ2 respectively. As the horizontal displacements increased, the tensile contribution of the HPFRC jacket gradually disappeared and the section strength tended to that of the RC core.
In the positive direction, specimen RCJ1 reached its maximum capacity, equal to 44.25 kN in the first cycle at a drift equal to 0.75%, while for specimen RCJ2 the peak load was equal to 49.5 kN at a 0.5% drift.
In the negative direction the two specimens reached approximately the same peak load of about 40kN at a drift of -0.75%.. For specimen RCJ1at a drift equal to -1% it is possible to recognize a short plateau, which was not due to the bottom reinforcement yielding, which remained in the elastic range, but to the fibers pull-out at the beam-joint interface, once the bridging effect vanished.
If a comparison between the residual strength and the peak loads is considered, specimen RCJ2 decayed less in the positive direction (66% against 61% of specimen RCJ1) and slightly more for negative displacements (39% against 51%).

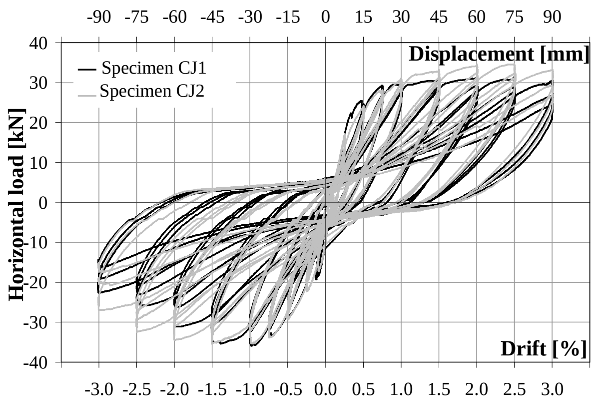


Figure 2.6. Horizontal load versus horizontal displacement curves for specimens CJ1 and CJ2

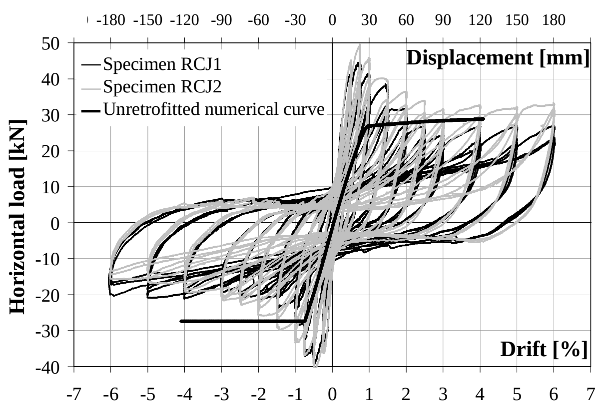


Figure 2.7. Horizontal load versus horizontal displacement curves for specimens RCJ1 and RCJ2

Figure 2.8. Cracks pattern for specimens CJ1 (on the left) and RCJ1 (on the right)

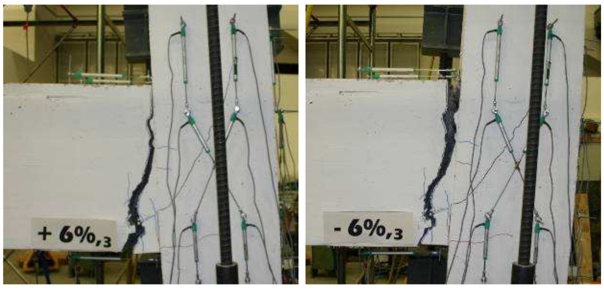


Figure 2.9. Specimen RCJ1 at a drift of 6%

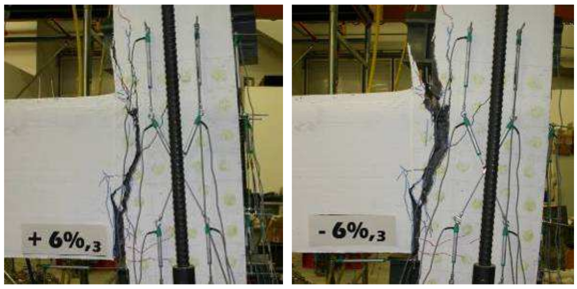
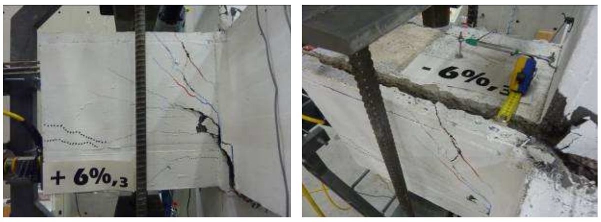


Figure 2.10. Specimen RCJ1 at a drift of 6%



(a) (b)

Figure 2.11. Damage in the secondary beam of specimen RCJ1 at the end of the test: (a) outer side of the joint; (b) inner side of the joint

(a) (b)

Figure 2.12. Damage in the secondary beam of specimen RCJ2 at the end of the test: (a) outer side of the joint; (b) inner side of the joint

From Figure 2.8, which represents the evolution of the cracks pattern for specimen RCJ1 (column on the right),it can be observed the formation at a early drift of a diagonal crack inside the joint panel, which didn’t develop significantly during the test and a vertical flexural crack.

The initial location of the vertical crack is different for the two specimens, being at the beam-joint interface for specimen RCJ1 and inside the joint for specimen RCJ2. As a matter of fact for specimen RCJ2 the flexural lever arm was higher in the positive direction, as the vertical crack was located inside the joint and hence also a compressive zone in the column collaborated in increasing the flexural strength.

For this reason, for positive displacements the first crack appeared at a drift equal to 0.25% for specimen RCJ1 and 0.5% for specimen RCJ2, due to the higher value of the flexural lever arm. This also justifies the higher peak load values reached at each drift by specimen RCJ2, with differences on average in the order of 10% up to a drift of +1% and of 20% for high drift up to the end of the test.

In the following cycles, for specimen RCJ1, the damage localized at beam-joint interface with an increasing opening of the interface crack that reached values of about 45 mm at the end of the test for a drift of about 6%.

For specimen RCJ2, the damage localized at beam-joint interface with an increase of the crack width for positive moments while for negative moments, at a drift equal to -2%, a HPFRC wedge at the top of the joint began to spall off. Starting from a drift equal to 4% the internal crack developed reaching the beam-column interface at the top of the joint.

Figure 2.9 and Figure 2.10 show the final positive and negative cycles at a drift of 6% for both the specimens.

Finally it is evident that no damage was observed in the outer side of the joint panel in correspondence with the secondary beam, while some cracks developed in the secondary beam at the inner side of the joint as shown in Figures 211(a) and 2.12(a) for RCJ1 and RCJ2 respectively. As shown in Figure 2.11(b), for specimen RCJ1 the detachment of the HPFRC jacket in the secondary beam was evident, while for specimen RCJ2 the detachment between the HPFRC and the host concrete happened in the column inside the joint and no detachment could be observed in the HPFRC jacket of the secondary beam (Figure 2.12(b)).

2.6. Comparisons

This paragraph deals with the comparison between the experimental results of the tests on the unretrofitted and the retrofitted specimens.

It is worth pointing out that comparisons in terms of force – displacement or moment – rotation results can't be performed, as the concrete used to cast the unretrofitted specimens has quite different properties than that used for the retrofitted ones. In order to compare the improvement in the joint performance deriving from the HPFRC contribution, the joint strength of an unretrofitted specimen with the same concrete strength of the retrofitted ones was evaluated by using the numerical method described in Riva et al. (2011).
In this approach, labeled PSLM (Principal Stress Limitation Model), the joint strength is governed by the maximum principal tensile stress reached in the panel zone (Pampanin et al. 2003).
From Figure 2.7, which shows the comparison between the experimental results of specimen RCJ1 and RCJ2 and the numerical curves obtained by monotonic analyses on the unretrofitted specimens, it can be observed that the application of a HPFRC jacket increases the shear strength of about 40%-45% for positive displacements, while the joint shear strength increases of about 30% for negative displacements.
With respect to the residual strength, it can be noticed that for both positive and negative directions the behavior of the retrofitted joints tended to the behavior of the unretrofitted ones.
For specimen RCJ2 the residual strength given by the analytical models underestimates the residual strength of the experimental results, due to the higher flexural lever arm which enhanced the strength of the specimen for positive displacements.
In the negative direction, the analytical evaluations overestimate the experimental residual strength. This is because the application of a preliminary negative moment to the secondary beam gives a transverse confinement at the bottom of the joint, so for positive moments applied to the main beam this contribution is favorable. On the other hand, for negative moments, the preliminary transverse moment yields to an unfavorable contribution, because it involves the raise of tensile forces at the interface between the HPFRC and the base concrete, so that the joint is not allowed to develop its full strength for negative displacements.
It is important to underline that in a real building the tensile forces acting in the joint due to the serviceability negative moment applied to the secondary beam are already present at the moment of the HPFRC jacket casting, while in the test the service moment was applied after jacket casting. The experimental set-up is hence worse than the real situation where the interface between the HPFRC and the base concrete is unloaded. However, the application of a moment after the HPFRC jacket casting can be representative of a real seismic event with lateral loads not only in the main beam direction, but also in the orthogonal one.
A comparison between the experimental results of unretrofitted and retrofitted specimens can be performed in terms of dimensionless dissipated energy, as shown in Figure 2.13. From the diagram, it can be noticed that the energy dissipation of specimens CJ1 and CJ2 are approximately comparable, with specimen CJ1 which dissipated 10% more energy than specimen CJ2. The retrofitted specimens dissipated on average 25% more energy than the unretrofitted ones at each drift value. However, unlike the unretrofitted specimens, for which dissipated energy decreased starting from a drift equal to 2%, for the retrofitted specimen RCJ1 energy dissipation always increased.
This phenomenon can be observed also in the horizontal force – displacement curves, where it is evident that for the unretrofitted specimens the hysteresis loops progressively exhibited a pronounced pinching effect, due to the bond-slip effect and the damage in the joint panel. In the retrofitted specimen curves, the pinching effect is less pronounced also at high drift levels, due to a minor contribution of both the previous effects.
For specimen RCJ2 the amount of dissipated energy started to decrease from a drift equal to 4%, but reaching higher drift values with respect to the unretrofitted specimens. It's nevertheless true that from a certain drift value the energy dissipation decreased, but with respect to the unretrofitted specimens, specimen RCJ2 reached higher drift values (6% against 3%).
Figure show the four specimens at the end of the tests. For the unretrofitted specimens CJ1 and CJ2 the three failure mechanisms could be clearly identified: beam failure with the vertical crack at the beam-joint interface, joint shear failure with the diagonal cracks in the panel zone and the thrust of the hooked end beam bars in the column at the bottom of the joint (Figures 2.14(a) and (b)).
For the retrofitted specimen RCJ1, even if some thin cracks could be observed on the outer face of the joint in correspondence with the main beam, the damage localized mostly in the beam-joint interface crack passing through the entire beam section (Figure 2.14(c)).

For specimen RCJ2 the vertical crack started a few centimeters inside the joint and developed externally only at high drift values (Figure 2.14(d)). Thus, the joint resulted more damaged than in the case of specimen RCJ1, even if the opening of the other cracks on the joint surface was limited to a few tenths millimeters.
For the retrofitted specimens the diagonal crack in the joint panel appeared in the negative direction only at a drift equal to 0.25% with a width of about 0.06÷0.07 mm, reached the maximum value of 0.35÷0.4 mm and then tended to close up to a width of 0.1÷0.2 mm for the remaining part of the test, while for the unretrofitted specimens CJ1 and CJ2 the cracks opening reached values of about 3 mm.
In both cases, no cracks were observed in the outer face of the joint in correspondence to the secondary beam, allowing to state that encasing the joint in a HPFRC jacket avoid the damage due to the thrust of the hooked end beam bars in the column at the bottom of the joint.

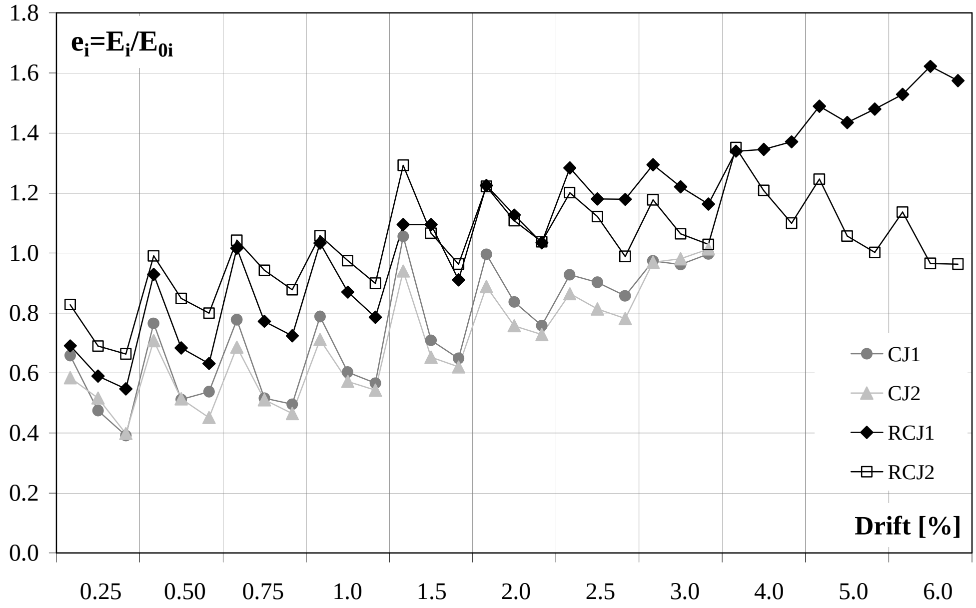


Figure 2.13. Comparison between dimensionless dissipated energy

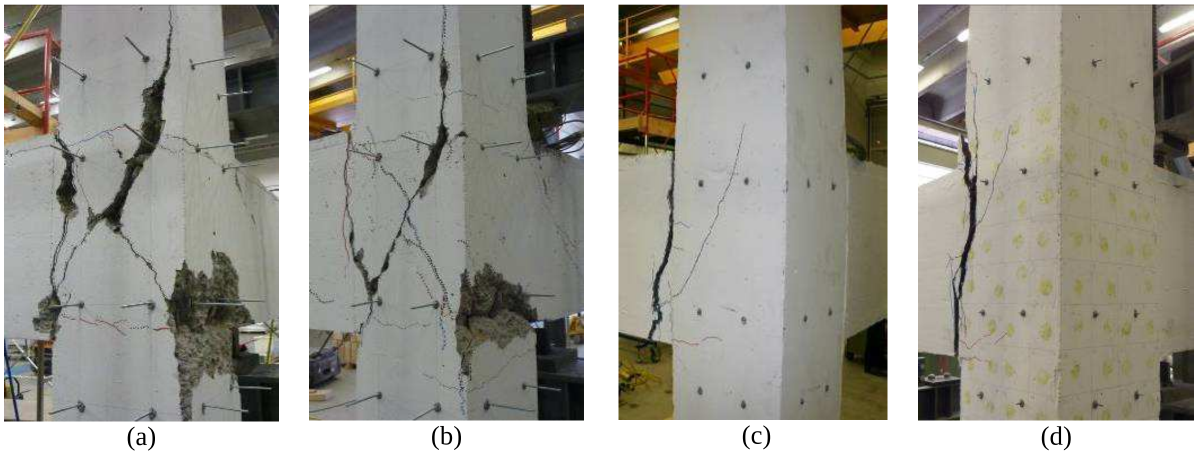


Figure 2.14. The specimens at the end of the tests: (a) CJ1; (b) CJ2; (c) RCJ1; (d) RCJ2

3. CONCLUSIONS

The experimental results confirmed the seismic vulnerability of corner beam-column joints, designed with details typical of the Italian construction practice of the ‘60s-‘70s. Two main aspects were evident: the deformability of the joint panel region and the slip of the beam reinforcing bars, which

strongly influenced the joint behavior.
This leads to the conclusion that structures, built with the same construction details adopted for the test specimen (smooth bars with hooked-end anchorage, no stirrups in the joint region, poor concrete) need to be retrofitted with respect to the lateral loads, in order to change the failure mechanism from a brittle joint shear failure to a more ductile beam flexural failure, with the development of a plastic hinge. Thus, the retrofitting operation aims at realizing a strong/column-weak/beam system, according to the principles of Capacity Design, which are at the base of the modern seismic codes.
The experimental results also confirmed an increase in the seismic performances of specimens retrofitted with a HPFRC jacket with respect to the unretrofitted ones, with no wide damage in the joint panel and, although the specimens exhibited a significant stiffness degradation after the peak value, it also showed a more limited pinching effect in the hysteresis loops, due to some bar-slip effects, after the opening of the crack at the beam-joint interface.
The application of a HPFRC jacket allows to improve also the ductility of the joint: the retrofitted specimen reached a drift equal to 6% against the 3% drift reached by the unretrofitted specimens. Also the dissipation capacity of the retrofitted joint is up to 30% higher than the unretrofitted joint, testifying a significant performance increase in case of seismic actions.
Moreover it is worth paying attention to the fact that the joint behavior was approximately symmetric in the positive and negative direction, which can be favorable if the joint is subjected to load reversal, as in the case of a seismic event.
To avoid the problem of the HPFRC detachment the adoption of stud connections between the host and the new concrete is suggested. The use of studs can be also useful for the placement of a metallic mesh if an added increase in shear strength is needed for the beam.
The use of a wire mesh around the joint, extending into the beam, might also help in controlling the crack opening at the beam-joint interface.

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