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Retrofitting of beam-column exterior joint with HPFRC jacketing

C. Beschi[1], G. Metelli[2] and P. Riva[3]

ABSTRACT: For RC frames designed before the '70s it is widely recognized that beam-column joints represent critical regions in presence of seismic loads, so that the joint behaviour influences the response of the whole structural system. Furthermore, a large number of existing buildings need to be strengthened with respect both to vertical and seismic loads. In the present research work the effectiveness in strengthening existing R/C structures by means of an innovative technique based on the application of a High Performance Fiber Reinforced Concrete jacket is investigated. Analytical evaluations are performed in order to show the increase in strength and ductility provided by the proposed technique. A simple numerical model is also used to design an experimental campaign on full scale specimens representing exterior beam-column joints subjected to cyclic loads, designed according to the structural details typical of the Italian construction practice of the 60's-70's.

1 INTRODUCTION

The strengthening of existing RC structures has become an urgent issue in Italy: the Abruzzo earthquake (6th April 2009) dramatically demonstrated that a large number of existing RC buildings were not able to sustain earthquake actions, due to many reasons: deficient material properties, improper overall structural configuration, absence of capacity design principles and poor reinforcement detailing, insufficient amount of column transverse reinforcement, inadequate anchorage detailing, lower quality of materials, in particular use of smooth bars and low-strength concrete.

Furthermore, 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. The joint behavior influences the response of the whole structural system, in terms of both deformation and strength (Pampanin et al., 2003; Riva et al., 2011).

Several retrofitting techniques can be adopted for the seismic retrofitting of existing RC elements (Fib Bulletin n. 24, 2003; Fib Bulletin n. 32, 2006; Fib Bulletin n. 35, 2006). One possible technique is the casting of RC jackets, which can increase the members' strength and ductility; a high jacket thickness (70÷100 mm) is generally needed due to the steel cover requirements, with a consequent increase in section geometry and hence in both mass and stiffness. Another possibility, recently developed, is the use of FRP wrapping, which can easily enhance the member ductility; this technique is not suitable when a significant strength increase of the column is also needed.

[1] *PhD student, Department of Civil Engineering, Architecture, Land and Environment, University of Brescia, via Branze 43, 25123 Brescia, Italy. E-mail: consuelo.beschi@ing.unibs.it*
[2] *Assistant Professor, Department of Civil Engineering, Architecture, Land and Environment, University of Brescia, via Branze 43, 25123 Brescia, Italy. E-mail: gmetelli@ing.unibs.it*
[3] *Professor, Department of Design and Technologies, University of Bergamo v.le Marconi 5, 24044 Dalmine (BG), Italy. E-mail: paolo.riva@unibg.it*

In recent years, a new technique based on the use of thin jackets made of High Performance Fiber Reinforced Concrete (HPFRC) has been developed (Martinola et al. 2007, Meda et al. 2008). This technique consists in encasing structural concrete elements in a thin layer of HPFRC (30-40 mm). The HPFRC material adopted exhibits a hardening behaviour in tension coupled with a high compression strength, strain capacity and toughness greater than traditional FRC ones.
The effectiveness in the seismic upgrade of the mentioned retrofitting technique for exterior beam-column joints is analytically investigated in the present research work. Full scale specimens are designed and they will be tested in order to validate the theoretical previsions and to evaluate the technological applicability.

2 JOINT MODELING

The test carried out by Calvi et al. (2001) on a RC frame with typical details of Italian construction practice in the '60-'70s showed significant damage to the exterior joints between the first and second floor and the formation of plastic hinges at the base of the columns at the ground floor. The development of a failure mechanism markedly different from that provided in the case of a rigid joint behavior, for which a soft floor mechanism would be expected, was evident (Figure 1).
Despite the experimental evidence, the deformability of the beam-column joints is commonly neglected in numerical analyses, because the nodal panel is assumed infinitely rigid. In the last two decades, different f.e.m. models have been proposed in order to evaluate the behaviour of beam-column joints subjected to cyclic loads, but the complexity of these models has limited their applicability in the assessment of existing structures.
A simple f.e. model for external joints of RC frames has been developed and proposed in Riva et al. (2011). The joint shear strength has been estimated using two alternative approaches, chosen for their simplicity and reliability among the many models available in the literature: in the first approach, called in this paper 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). The second approach, called MSSTM (Modified Softened Strut-and-Tie Model), is an adaptation of the strut-and-tie model proposed for confined joints (Hwang and Lee 1999).
The overall deformation of the joint is considered as the sum of two non linear contributions: the first is related to the shear deformability of the panel zone, the second to the localized rotations at the interface between the joint and the elements, due to the slip of the longitudinal reinforcement within the joint.

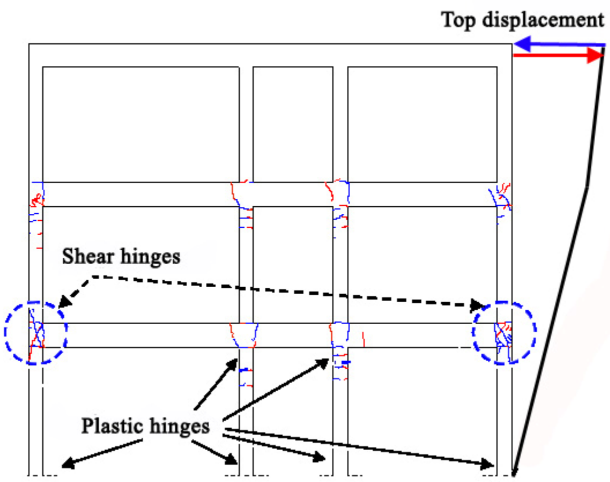


Figure 1. Global failure mechanism for a '50s-'70sRC frame (Calvi et al. 2001).

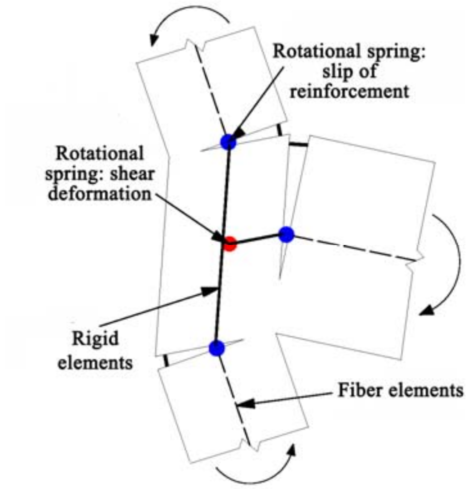


Figure 2. Proposed analytical model for joint behaviour: rotational springs.

The numerical model involves the use of a rotational spring, placed between the column and the beam, to model the shear deformation of the panel and three rotational springs, located at the interface between the joint and beam/column, to model the localized rotations, due to the slip of the longitudinal reinforcement within the joint region (Figure 2). The structural elements converging in the joint are modeled as one-dimensional elements with diffuse plasticity (fiber elements), while rigid elements are adopted to model the portion of beam and column within the panel zone region. The developed f.e. model has been adopted to model the cyclic behavior of the experimental RC frame tested by (Calvi et al., 2001). The numerical results are in good agreement with the experimental ones in term of resistance, stiffness, ductility and hysteretic behavior. The presented model can be considered as an effective tool for the assessment of the existing RC frames designed before ’70 without any seismic detailing design (Riva et al., 2011).

3 DESIGN OF A FULL SCALE JOINT TEST SPECIMEN

The above mentioned analytical and numerical models have been used to design a full scale specimen to be tested under cyclic horizontal loads. The test specimen is representative of an exterior joint of the first level of a RC reference frame designed according the construction practice of the 60’s-70’s. The materials commonly used in that period were C16/20 concrete with a f_{cm} strength equal to approximately 24 MPa and FeB32k steel bars with a yield strength of 380 MPa (Figure 3).
The elements have been designed only for gravity loads: the columns carry a centered normal axial action 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 loads. The column cross section is 30x30 cm, while the beam has a dimension equal to 30x50 cm (Figure 3). The specimen is characterized by the absence of stirrups in the joint and smooth reinforcements with hook ends, as required by the design provisions of the 50’s-70’s national standards (Regio Decreto 1939) and suggested by the technical literature of the time (Santarella 1945).
The analytical methods previously described have been used to predict the failure mode of the test specimen, designed in order to ensure the joint shear failure in at least one of the two loading directions. The test results will be an effective tools also for the model validation.

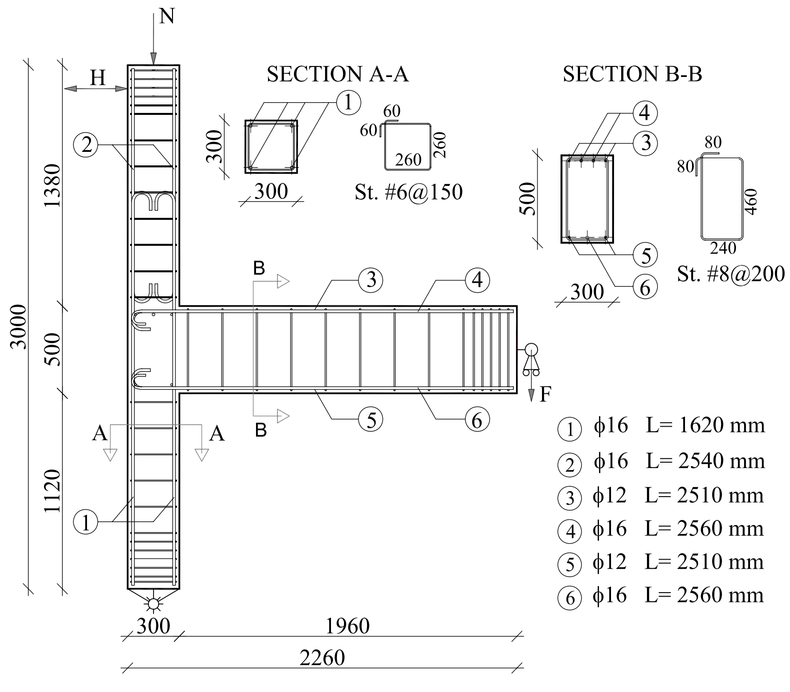


Figure 3. Geometrical dimensions of the test specimen.

Table 1. Calculation of the specimen strength (Riva et al., 2011).

NEGATIVE DRIFT				
METHOD	V_j [kN]	V_b [kN]	V_c [kN]	FAILURE
PSLM	30.9	38.4	35.7	**JOINT**
MSSTM	24.7			**JOINT**
POSITIVE DRIFT				
METHOD	V_j [kN]	V_b [kN]	V_c [kN]	FAILURE
PSLM	30.9	26.1	35.7	**BEAM**
MSSTM	24.7			**JOINT**
V_j, V_b e V_c are the shear floor values which determine the maximum shear strength of the joint, the beam and the column respectively.				

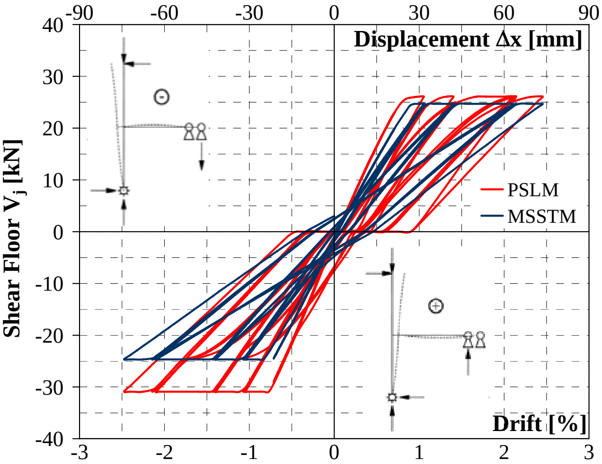


Figure 4. Numerical evaluation of the cyclic behaviour of the designed joint according to ‘50s-‘70s practice.

As shown in Table 1, the results of the PSLM and MSSTM methods show a shear failure mode in the joint in at least one direction of loading. For the same specimen, numerical analyses were carried out to simulate the experimental test and to study the joint behavior when subjected to a cyclic loading history. In Figure 4 the shear floor V - drift ratio curve is plotted for both methods. By the MSSTM the failure is always governed by the joint strength, while by the PSLM the system behavior is characterized by the joint failure for negative drift and by beam bending failure for positive drift. Further details are published in (Riva et al, 2011).

4 STRENGTH EVALUATION OF THE RETROFITTED MEMBERS

In this section, a strength evaluation of the structural members and of the joint before and after retrofitting is presented.

The application of a 30 mm thick jacket is considered, being the smallest value which can be adopted for technological limits. For the column, a solution with a 40 mm jacket is also considered.

Before the application of the jacket, the member surface needs to be sandblasted to achieve a 1÷2 mm roughness to ensure a good adhesion between new and old concrete even in the absence of chemical bonding agents, due to the high bond property of the HPFRC (Martinola et al., 2007).

For the column retrofitting, a self-compacting and self-leveling HPFRC material will be used, with the mechanical properties summarized in Table 2. For the beam retrofitting two solutions are shown: in the first U-shaped solution, a 30 mm jacket is cast using the HPFRC at the beam intrados and a thixotropic HPFRC at the sides (Figure 6). It is worth pointing out that the thixotropic material can be applied directly on the beam surfaces, without the need of formworks. The bottom and side HPFRC layers are characterized by the same type and volume of fibers (Table 2).

A second retrofitting solution is eventually proposed to avoid the strong beam – weak column failure mechanism, with a minor increase in beam strength, applying only a 30 mm side jacket of the thixotropic HPFRC. This solution allows the demand of the beam shear strength to be achieved with a controlled bending strength increase (Figure 6). Since a thixotropic material is used, the retrofitting operations are quite easy and fast, but the use of stud connections is highly recommended to ensure the bond between the old and the new concrete.

4.1 Column verification

As far as the column verification, the bending moment M – axial force N interaction diagrams are plotted for the section before and after the retrofitting.

Table 2. Material characteristics.

HPFRC		
Cubic compressive strength	130	MPa
Tensile strength	6	MPa
Average elastic modulus	38	GPa
THIXOTROPIC HPFRC		
Cubic compressive strength	90	MPa
Tensile strength	5	MPa
Average elastic modulus	33	GPa
STEEL FIBERS		
Fibers length	15	mm
Fibers equivalent diameter	0.18	mm
Fibers volume	3.9%	

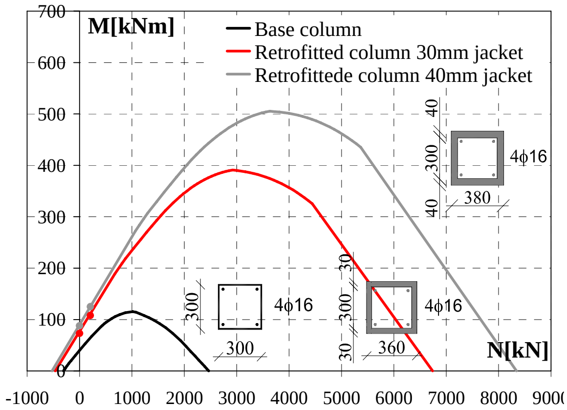


Figure 5. M-N interaction diagram for the column.

For the non retrofitted section, the M-N envelope is calculated following the classical assumptions: Navier-Bernoulli behavior; tensile strength of concrete neglected; perfect bond between steel and concrete; compression in concrete modeled by a constant stress distribution (EN1992-1-1, 2004); perfect bond between the FRC layer and the substrate (CNR-DT 204, 2006). Furthermore, since the HPFRC exhibits a hardening behavior in tension, the tensile strength of concrete has been considered by assuming a constant tensile stress distribution with a medium tensile strength of 4 MPa, and an ultimate tensile deformation equal to 1%. The theoretical curves have been computed by adopting experimental average material characteristics. For an axial force N equal to 1000 kN the bending strength of the column reaches up to 263 kNm for a 40mm jacket, starting from 115 kNm bending strength of the bare column. A very large strength increase due to the jacket application is evident also for a 30 mm thick jacket (Figure 5).

4.2 Beam verification

To study the increase in beam flexural strength of the two retrofitting proposed solutions, analytical analyses have been performed.

For the concrete, a simplified parabolic-rectangular scheme has been assumed with an ultimate compressive strain of 0.35%; the same scheme with an ultimate strain of 1% has been adopted for the HPFRC concrete, for which also the tensile contribution has been considered, adopting a constant tensile distribution with an ultimate tensile deformation equal to 1%. An average tensile strength of 4 and 3 MPa has been adopted for the normal and thixotropic HPFRC material, respectively. As for the reinforcing steel, an elastic perfectly plastic stress-strain relationship has been considered, adopting an ultimate tensile strain ε_{su} of 10%, which is a realistic value for '70s plain bars (Verderame et al. 2001).

In Figure 7, a comparison of the flexural behavior for positive and negative moments between the bare element and the two retrofitted solutions is shown.

Firstly it can be noticed that the retrofitting solutions proposed does not increase the section ductility, since the bare beam is characterized by a very low reinforcement ratio (0.30% and 0.45% respectively for positive and negative moments).

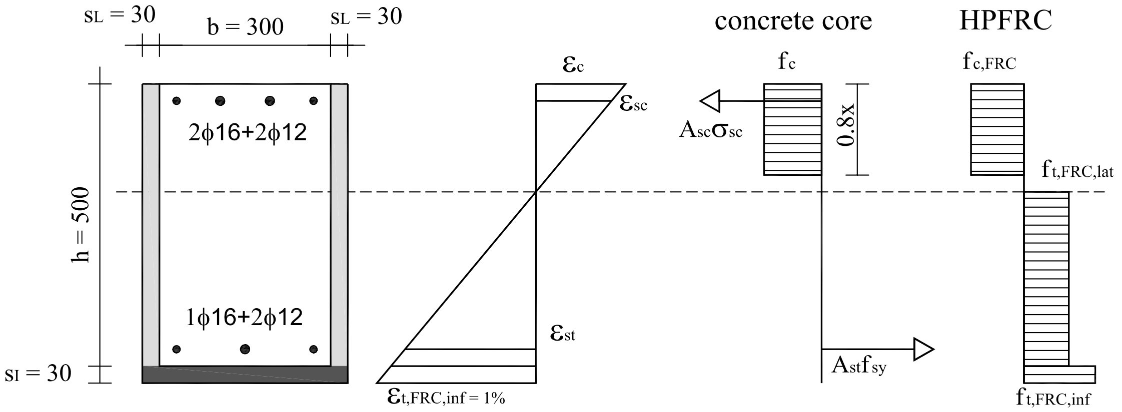
As for the flexural strength, for the positive direction the bare beam maximum bending moment is equal to 76 kNm, the peak bending moment for the retrofitted solution with a U jacket and with only a side jacket is equal to 115 kNm and 94 kNm, respectively. The bending strength increase ranges between 51% and 24%.

For the negative direction, the maximum bending moment of the bare beam is equal to 108 kNm, while the peak bending moment for the retrofitted solution with U jacket and with only side jacket is equal to 140 kNm and 127 kNm respectively (1.30 and 1.18 times the bare one). For the positive moment it can be noticed that, after reaching the peak value, the strength decreases towards the value of the non retrofitted section. For the negative moment, the curve trend after the peak is the same of that of the original beam for the retrofitted solution with only lateral jacket, while for the retrofitted solution with U jacket the curve is higher than the bare one due to effectiveness of the HPFRC in the compressive zone.

In Figure 6 the calculation scheme for an analytical evaluation of the positive peak bending moment for the retrofitted solution with U jacket is shown.

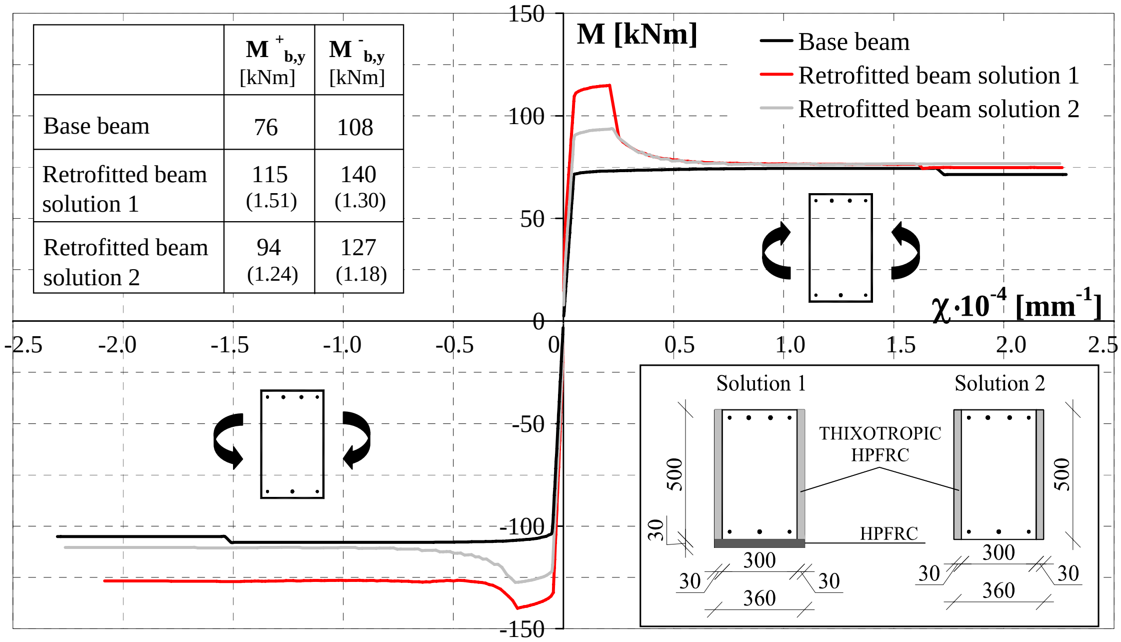
As far as the beam shear strength verification is concerned, a comparison between the bare and the retrofitted beam is given in the following. The shear strength of the retrofitted solution can be calculated as the sum of two factors: the shear resistance given by the stirrups placed in the bare element and the resistance given by the HPFRC side layers, assumed to act like equivalent vertical steel legs.

As an example, this approach is applied to the test specimen, where the stirrups have been designed to avoid a brittle shear failure. In a real '60s beam, the stirrups spacing could be larger, so the percentual increment in shear strength provided by the HPFRC jacket could be more significant.



f_c	average compressive stregth for base concrete
$f_{c,FRC}$	average compressive strength for thixotropic HPFRC
$f_{t,FRC,lat}$	average tensile strength for thixotropic HPFRC
$f_{t,FRC,inf}$	average tensile strength for HPFRC
σ_{sc}	compressive stress in top reinforcement
f_{sy}	yield stress in bottom reinforcement

Figure 6. Calculation scheme for the peak bending moment of the retrofitted beam with U jacket.



	$M^{+}_{b,y}$ [kNm]	$M^{-}_{b,y}$ [kNm]
Base beam	76	108
Retrofitted beam solution 1	115 (1.51)	140 (1.30)
Retrofitted beam solution 2	94 (1.24)	127 (1.18)

Figure 7. Flexural behaviour for the beam before and after retrofitting.

Assuming an average tensile stress for the thixotropic concrete ($f_{t,FRC,lat}$) equal to 3 MPa, the amount of equivalent stirrups $a_{s,eq}$, assuming a yield stress for steel (f_{sy}) equal to 380 MPa, can be calculated as follows:

$$a_{s,eq} = 2 \cdot (s_L \cdot 1000) \frac{f_{t,FRC,lat}}{f_{sy}} = 473 \frac{mm^2}{m} \qquad [1]$$

where s_L is the 30mm lateral thickness of the beam jacket.
The $a_{s,eq}$ area is equivalent to #8 @200 mm stirrups, corresponding to 500 mm^2/m.
The shear strength $V_{R,b}$ can be calculated for the bare beam, where #8@200 mm spacing stirrups have been placed:

$$V_{R,b} = 0.9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_{sy} = 80.4\, kN \qquad [2]$$

where A_{sw} is the area of a stirrup, s is the stirrup spacing and d is the effective depth of the beam.
Due to the presence of the lateral jacket, the retrofitted beam shear strength can be reasonably taken as:

$$V_{R,retr} = 0.9 \cdot d \cdot \left(\frac{A_{sw}}{s} + a_{s,eq} \right) \cdot f_{sy} = 156.4\, kN \qquad [3]$$

With a 30 mm HPFRC jacket the shear resistance of the retrofitted beam is about twice the shear strength of the bare beam. If the contribution of a steel mesh placed within the jacket before the HPFRC casting is also taken into account, an additional equivalent stirrup area of #8@200 mm spacing can be achieved.
In the test specimen, the maximum acting shear force at beam-column interface for the retrofitted solution is equal to:

$$V_{S,retr} = \frac{M_{b,y}}{L_{bn}} \qquad [4]$$

where L_{bn} is the clear span of the beam equal to 2.10 m and $M_{b,y}$ is the peak bending strength of the beam. For the beam retrofitting solution with the U jacket, $V_{S,retr}$ is equal to 66.67 kN and the over strength factor in terms of shear resistance is 2.35, while for the beam retrofitting solution with only the lateral jacket, the shear action associated to the peak bending moment is equal to 60.48 kN and the over-strength coefficient is about 2.59.

4.3 Strong column – weak beam

In Table 3, the resistant bending moment $M_{c,R}$ of the reinforced column with 30 or 40 mm jacket is compared to the $M_{c,E}$ moment acting in the column at the beam flexural failure: for the verification the peak negative moment is considered for both solutions.

Table 3. Comparison between column acting and resistant moments.

		Retrofitted beam solution 1		**Retrofitted beam solution 2**	
		N=0 [kN]	N=206 [kN]	N=0 [kN]	N=206 [kN]
Jacket thickness	$M^-_{c,E}$	70		63.5	
30 mm	$M_{c,R}$	73	108	73	108
	ψ	1.04	1.54	1.15	1.70
40 mm	$M_{c,R}$	88	125	88	125
	ψ	1.26	1.79	1.39	1.97

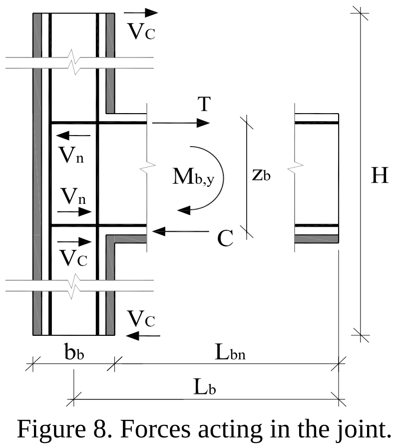


Figure 8. Forces acting in the joint.

For the evaluation of the column resistant moment, an axial load of 206 kN is considered; this value corresponds to the axial force acting at the first story column of the reference building during the seismic event. The acting moment on the column ($M_{c,E}$) is equal to $M_{b,y}/2$, where $M_{b,y}$ is the peak negative bending moment in the beam.
As shown in Table 3, considering an axial load of 206 kN, the over strength factor ψ, calculated as the ratio between the resistant ($M_{c,R}$) and the acting moment ($M^{-}_{c,E}$) in the retrofitted column, varies from 1.54 to 1.97. The solution with a jacket thickness of 30 mm and without axial load gives a safety factor lower than 1.2 for the two solutions proposed for the beam retrofitting. As a result, it is pointed out that the retrofitted solutions allow to realize a strong column – weak beam system in both cases.

4.4 Beam – column joint

According to the new Italian Code (Circolare 2 Febbraio 2009), the strength verification of an unretrofitted beam-column joint may be performed as follows:

$$\sigma_{nt} = \left| \frac{N}{2A_g} - \sqrt{\left(\frac{N}{2A_g}\right)^2 + \left(\frac{V_n}{A_g}\right)^2} \right| \leq k\sqrt{f_c} \qquad [5]$$

where A_g is the column section; V_n is the total shear acting on the joint and N is the axial load in the upper column; k factor equal to 0.3.
For the experimental specimen, the strength of both original and retrofitted joints has been evaluated by adopting the average material properties and unit safety factors.
In addition, the total shear acting on the joint has been computed by the application of Capacity Design rules as:

$$V_n = \frac{M_{b,y}}{z_b} - V_c \qquad [6]$$

where M_{by} is the peak resistant moments of the beam, as shown in Figure 8 and z_b is the flexural level arm.
The column shear V_c is defined as:

$$V_c = M_{b,y} \frac{L}{L_{bn}} \cdot \frac{1}{H} \qquad [7]$$

where L is the span of the beam; L_{bn} is the clear span of the beam and H is the interstorey height (Figure 8).
As far as the test specimen is concerned, for the un-retrofitted joint assessment an axial load of 206 kN is considered, resulting from the seismic combination of vertical loads.
For a beam positive bending strength $M_{b,y}$ equal to 76 kNm, the column shear V_c is equal to 26 kN and the total shear acting on the joint V_n is equal to 136 kN. The joint in the un-retrofitted condition is verified because the stress in the joint according to Equation 5 is equal to 0.75 MPa, smaller than the joint strength, equal to 1.47 MPa.
For a beam negative bending strength $M_{b,y}$, equal to 108 kNm, the column shear V_c is equal to 37 kN and the total shear acting on the joint V_n is equal to 197 kN. The stress in the joint is equal to 1.33 MPa, smaller than the joint strength, equal to 1.47 MPa. Also in this case the joint is verified according to Italian Code.
While the joint is verified according to DM2008, the verification by means of the analytical models previously mentioned (PSLM end MSSTM) clearly indicates that in the negative direction the joint verification is not satisfied and according to the MSSTM method, the joint is not satisfied also in the positive direction.
It is suggested by the authors that a reduction of the tensile strength factor k for the evaluation of the shear joint strength should be recommended: in the PSLM model (Pampanin et al., 2003) a k factor equal to 0.2 was used, on the basis of experimental evidence for exterior joints.
The joint tensile strength, calculated from Eq.5 is equal to 0.98 MPa. Hence, according to this formulation, the joint in the unretrofitted condition is verified for the positive moment, but the verification is not satisfied for the negative moment.

For the retrofitted specimen, Equation 5 may be modified to account for the tensile strength contribution of the HPFRC jacket as follows:

$$\sigma_{nt} = \left| \frac{N}{2A_T} - \sqrt{\left(\frac{N}{2A_T}\right)^2 + \left(\frac{V_n}{A_T}\right)^2} \right| \leq \frac{k\sqrt{f_c} \cdot A_g + \dfrac{f_{t,HPFRC}}{\gamma_{HPFRC}} \cdot A'_g}{A_T} \qquad [8]$$

where: $A_T = A_g + A'_g$, with A_g area of the section of the unretrofitted column and A'_g area of the HPFRC applied to the column; γ_{HPFRC} safety factor for HPFRC, taken equal to 1 and $f_{t,HPFRC}$ average tensile strength of HPFRC.

In the verification of the retrofitted joint k is taken equal to 0.3, according to the Italian Code (Circolare 2 Febbraio 2009) or equal to 0.2, according to PSLM (Pampanin et al., 2003).

In Table 4, the comparison between the tensile stress in the joint and its strength is given for the two different retrofitted beam solutions. The joint verification is performed for both positive and negative direction of the beam peak moments. As for the column the two jacket thickness possibilities (30 or 40 mm) are analyzed.

In addition the verification is carried out considering both the axial load acting in the joint of the first level of the reference building, and both without axial load, to evaluate the joint behavior in the upper floors where the axial load is smaller.

In the case of presence of the axial load, in all the analyzed solutions, the tensile stress in the joint ($\sigma_{n,t}$) is smaller than the joint strength ($\sigma_{n,t,R}$) and so the joint verification is satisfied. The axial load is beneficial to the joint strength: the joint verification is not satisfied when the axial load is not considered for a column jacket thickness of 30 mm and only in the case of k_1 equal to 0.2. With k_1 equal to 0.3 the joint is always satisfied.

Table 4. Beam-column joint verification.

Column jacket thickness	Beam moment	joint resistance $\sigma_{n,t,R}$ [MPa] k_1=0.3	joint resistance $\sigma_{n,t,R}$ [MPa] k_1=0.2	N = 206 kN, **Retrofitted beam sol. 1**: tensile stress in the joint $\sigma_{n,t}$ [MPa]	N = 206 kN, **Retrofitted beam sol. 2**: $\sigma_{n,t}$ [MPa]	N = 0 kN, **Retrofitted beam sol. 1**: $\sigma_{n,t}$ [MPa]	N = 0 kN, **Retrofitted beam sol. 2**: $\sigma_{n,t}$ [MPa]
30 mm	+ -	2.24	1.90	1.31 1.58	1.11 1.53	**1.95** **2.24**	1.74 **2.18**
40 mm	+ -	2.42	2.12	1.18 1.42	1.00 1.37	1.75 2.01	1.56 2.12

5 CONCLUSIONS

The present paper summarizes the first part of a research aiming at studying the seismic behaviour of exterior beam-column joints of existing RC frames, designed only for gravity loads without any Capacity Design principles. A seismic upgrade is supposed to be realized by means of encasing the RC elements in a thin layer of HPFRC. The first aim is to evaluate the effectiveness of a thin HPFRC jacket to increase the seismic response of a R.C. frame designed with ‘50s-‘70s construction practice. A 30 mm thick HPFRC has been adopted, the thinnest layer which can be applied for technological reasons.

The assessment of a R.C frame before and after retrofitting have been performed by simple analytical and numerical models, proposing two different retrofitting solutions: a column jacket with U shaped beam strengthening or with a partial jacket applying the lateral HPFRC layers only on the beam sides.

The analytical model points out the proposed technique allows to address the retrofitting design of a RC frame in order to avoid any brittle failure: by applying a HPFRC jacket

to the column and only lateral jackets to the beam, the shear over strength factor can be sufficiently increased according to the seismic demand, thus allowing the RC retrofitted frame to be characterized by strong column and joint but with weak ductile beam.
In the next research stage, the experimental results of full scale tests will allow the analytical and numerical model to be validated, in order to further provide design guidelines for the seismic upgrade of exterior beam-column joints in existing RC structures.

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