2019 SECED Conference
Earthquake risk and engineering towards a resilient world
9-10 September 2019, Greenwich, London

SEISMIC RETROFIT OF BEAM-COLUMN JOINTS: FROM FRP SYSTEMS TO NOVEL FRCC JACKETING

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Abstract: *Existing reinforced concrete buildings designed with old code provisions in the Mediterranean area commonly have poor seismic performance. This is due to the lack of proper seismic detailing which is triggering premature brittle failures. Field observations in the aftermath of major seismic events showed that the shear failure of beam-column joint is one of the major weakness of existing RC buildings. In this context, experimental tests demonstrated that fiber reinforced polymers (FRP) systems are effective in increasing the shear strength of beam-column joints and improving the global building performance. More recently, the development of high performance fiber reinforcing cement composites (FRCC) opened to new frontiers in the seismic retrofit of structural members. This paper deals with the seismic strengthening of a real existing building damaged by the L'Aquila earthquake. The seismic performance of the structural system are limited by the shear failure of beam-column joints. To increase its the seismic capacity two different retrofit solutions by using classic FRP systems and a novel FRCC thin jacketing are proposed and compared. A novel design procedure to define the required thickness of the FRCC jacketing of the joint panel is proposed and experimentally validated.*

Introduction

Recent seismic events showed the high vulnerability of existing reinforced concrete (RC) structures to lateral loads. The occurrence of brittle failures at level of the joint panels or in the columns are the main sources of vulnerability of RC frames designed with old code provisions (Priestley, 1997; Marco Di Ludovico *et al.*, 2008; Del Vecchio *et al.*, 2014; Frascadore *et al.*, 2015; Del Vecchio *et al.*, 2016). Indeed, the lack of proper seismic detailing and transverse reinforcement does not allow the fully exploit of the member flexural capacity often resulting in a premature shear failure.

The seismic response of existing RC beam-column joints without transverse reinforcement in the joint panel was widely investigated in the recent years. Several experimental tests (Calvi, Magenes and Pampanin, 2002; Pantelides, Clyde and Reaveley, 2002; Mitra and Lowes, 2007) and analytical studies (Hakuto, Park and Tanaka, 1995; Priestley, 1997; Shiohara and June, 1998; Pagni and Lowes, 2006) were conducted with the scope of quantifying the strength and deformation capacity of such members. Reliable capacity models, design formulations and refined non-linear numerical models were proposed. They allow to include the response of such member in the seismic performance assessment of existing RC frames (Frascadore *et al.*, 2015; Del Vecchio *et al.*, 2016; Del Vecchio, Di Ludovico, Pampanin, *et al.*, 2018).

These studies outlined that when the joint is subjected to reverse cyclic actions (bending moment and shear transmitted by the surrounding RC members) a complex stress-field characterizes the joint panel shear response. The principal directions of tension and compression stresses change depending on the shear force transmitted by the surrounding members and the axial load acting at the top of the column. The principal stress approach based on the Mohr Circle approach can be an effective tool to accurately reproduce the joint response. Furthermore, the stress limitations suggested by Priestley et al. (1997) can be employed to accurately predict the seismic response of joints without transverse reinforcements as demonstrated by several experimental tests (Calvi,

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Magenes and Pampanin, 2002; Del Vecchio *et al.*, 2014; Del Vecchio, Di Ludovico, Balsamo, *et al.*, 2018). This procedure is suggested by several international standards to assess the capacity of existing joint panels (CEN, 2005; Fardis, 2009; NZSEE/MBIE, 2017; MIT, 2018; MIT 2019, 2019) or to design the amount of joint transverse reinforcements (CEN, 2004).

By contrast, other international standards and guidelines suggest design formulations to compute the joint strength in terms of global joint shear (AIJ, 1999; ASCE/SEI 41-13, 2013). These approaches, which are of simple application, neglect the contribution of the axial load to the joint shear capacity often leading to not accurate estimation of the joint shear strength. Furthermore, the diagonal compression failure of the concrete core is not directly considered.

The poor seismic performance of the poorly detailed joints commonly limits the seismic response of the entire structural systems. For this reason, several strengthening solutions were proposed and experimentally validated (Antonopoulos and Triantafillou, 2003; Prota *et al.*, 2004; Marco Di Ludovico *et al.*, 2008; Tsonos, 2008; Bedirhanoglu, Ilki and Kumbasar, 2013; Del Vecchio *et al.*, 2014; Kalogeropoulos *et al.*, 2016; Del Vecchio, Di Ludovico, Balsamo, *et al.*, 2018). Within the available strengthening techniques fiber reinforced polymers (FRP) have gained popularity since they are an effective and easy to apply solution (Balsamo *et al.*, 2012). Indeed, their lightweight, durability and low degree of disruption makes these materials suitable for the seismic strengthening of existing RC structures. Several experimental tests demonstrated the effectiveness of FRP-based strengthening solutions. They increase the joint shear capacity promoting a ductile failure mode (Antonopoulos and Triantafillou, 2003; Prota *et al.*, 2004; M Di Ludovico *et al.*, 2008; Del Vecchio *et al.*, 2014). This result in a significant increase of the global seismic capacity as demonstrated by experimental tests and analytical studies. Proper design formulations and analytical models were later developed to support the design procedure of the FRP strengthening of the joint panel (Antonopoulos and Triantafillou, 2002; Tsonos, 2008; Bousselham, 2010; Del Vecchio *et al.*, 2015). Furthermore, design guidelines were developed to support practitioners in the design and application of these strengthening solutions (DPC-ReLUIS, 2011; Frascadore *et al.*, 2015). This strongly promoted the use of FRP materials in the recent post-earthquake reconstruction process followed to the L'Aquila earthquake (Di Ludovico *et al.*, 2017a, 2017b).

These studies also determined that the low quality of the concrete substrate may significantly reduce the effectiveness of this strengthening, anticipating FRP debonding. For this reason, in the case of poor-quality, damaged, or deteriorated concrete substrate (i.e., mean concrete compressive strength, f_{cm} < 15 MPa), replacement with a shrinkage-free cement grout is suggested before the application of the FRP strengthening (DPC-ReLUIS, 2011; CNR-DT 200, 2013).
This may significantly reduce the advantage of a simple and fast application, typical of the FRP systems. The widespread use of fiber-reinforced cement (FRC) composites and high-performance fiber-reinforced cementitious composites (HPFRCC) (Naaman and Reinhardt, 2007) opened new frontiers in the design of the shear capacity of RC members. The high tensile strength, toughness, and tolerance for damage make these materials attractive for the use in earthquake-resistant structures, with emphasis on RC members with shear-dominated response (Parra-Montesinos, 2005). The reduced thickness allows keeping the jacketing within the original cross-section dimensions. The effectiveness of the FRC/HPFRCC jacketing as a seismic strengthening solution for old-style RC beam-column joints has been recently demonstrated by means of experimental tests on full-scale specimens (Del Vecchio, Di Ludovico, Balsamo, *et al.*, 2018). These tests showed that a thin FRCC jacketing is a promising technique for the strengthening of existing RC members. However, lack of reliable design formulations do not allow to properly design the thickness of the FRCC jacketing.

In this context, this research work deals with the FRP and FRCC strengthening of poorly detailed beam-column joints. The design formulation proposed by the authors and recently included in the fib bulletin 90 is used to design the joint FRP strengthening of a case study existing RC building. The same formulation is later extended, with proper changes, to design the FRCC strengthening of the joint panel. A direct comparison with the results of an experimental tests is used to have a first validation of the proposed design procedure.

Design approaches for FRP strengthening of beam-column joints

Different methodologies are currently available in literature to design the FRP strengthening of the joint panel. A first attempt was made by (Antonopoulos and Triantafillou, 2002) that proposed a strain-based approach for the design the joint panel shear strengthening along with the flexural strengthening of surrounding members. Recently, Tsonos (2008) presented a different theoretical model where it is assumed that the FRP fibers oriented in the direction of the beam axis are equivalent to the steel hoops. In recent years, the Antonopoulos and Triantafillou (2002) model was simplified by Akguzel and Pampanin (2012) on the basis of experimental observations (Akguzel and Pampanin, 2010). They also provided a simplified non-iterative procedure, to calculate the strength capacity of beam–column joints with plain round bars and end hooks. A simple design procedure has been suggested within the ReLUIS guidelines to assist the practitioners involved in the L'Aquila post-earthquake reconstruction process (DPC-ReLUIS, 2011; Frascadore *et al.*, 2015). It is based on the approach proposed in the Eurocode 8 and NTC 2008 (CEN, 2004) to design the joint stirrups assuming that the externally bonded FRP system behaves as the interior steel stirrups. The maximum joint shear at the yielding of the beam-longitudinal reinforcement is used to determine the joint shear demand. It can be used in a local strengthening approach without performing a global analysis of the entire structural system (Frascadore *et al.*, 2015). The design strain of FRP is limited to 0.4% to account for debonding.

In the recent years, based on the experimental evidence, Del Vecchio et al. (2015) proposed a novel design formulation for the efficient design of the joint panel FRP strengthening. It relies on the capacity design principles to assess the actual demand of joint shear (and in turn, principal stresses). It can be used to design the strengthening layout considering fibers in one or multiple directions, continuous fabric or discrete strips, different type of fibers, number of joint sides strengthened in shear, presence of initial damage or end-anchorages. The contribution of the FRP strengthening is calculated considering the effective debonding strain calibrated on a wide database of experimental tests accounting for the FRP debonding on the joint panel due to multiaxial strain field. This procedure has been recently included in the new fib bulletin 90 (fib - Task Group 5.1 *et al.*, 2019) (approach 2) for the seismic strengthening of deficient beam-column joints.

According to this procedure the shear demand in the joint panel can be computed considering the target bending moment demand in the surrounding column or beams. In the case that a global analysis is not carried out, according to a local strengthening approach the joint shear demand can be computed at the yielding of the beam or columns. In particular, the minimum between the sum of the yielding moment of the beams ($\sum M_{yb}$) and the yielding moment of the columns ($\sum M_{yc}$) framing into the joint is used. Then, these moments can be converted into a joint shear stress demand, $v_{j,h}$, by using the following formulations:

$$v_{j,h} = \frac{\sum M_{yb}\left(\frac{1}{jd_b} - \frac{1}{H_n}\frac{L_{b,n}}{L_b}\right)}{(b_c h_c)} \leq \eta \cdot f_{cm}\sqrt{1 - \frac{\nu_d}{\eta}} \tag{1}$$

where: jd_b is the internal lever arm of the beam cross-section, $L_{b,n}$ and L_b are the theoretical and clear shear span of the beam, H_n is the theoretical storey height, b_c and h_c are the dimensions of the column cross-section, $\eta = 0.6\,(1 - f_{ck}/250)$ and $f_{ck} = f_{cm} - 8$, ν_d is the normalized axial load in the column above the joint that is equal to $N/(b_c h_c f_{cm})$.
It is worth checking that the joint shear demand does not exceed the maximum shear corresponding to crushing of the concrete core (the second term of Eq. (1)).
In the case that the $\sum M_{yc} < \sum M_{yb}$ the columns will yield first and $\sum M_{yb}$ can be set equal to $\sum M_{yc}$ in the Eq. (1).

Once that the demand joint shear is computed, the principal tensile stress needed for designing the required FRP amount is given by Eq. (1):

$$p_{t,f}^{dem} = -\frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{j,h}^2} - k\sqrt{f_{cm}} \tag{2}$$

where: $f_a = N/(b_c h_c)$ is the axial stress on the joint panel, k is a numerical coefficient representing the capacity of the as-built joint at the first cracking (it is equal to 0.30 for beam-column joints with deformed bars and 0.20 for beam-column joints with smooth bars) and f_{cm} is the mean concrete compressive strength.

The contribution of the FRP strengthening system, which should be greater than the demand, can be calculated as:

$$p_{t,f}^{cap} = \frac{A_f \cdot E_f \cdot \varepsilon_{fd}}{b_c \cdot (h_c/sin\theta)} \tag{3}$$

where A_f is the required FRP area on the joint panel. Different formulations were proposed to account for all the possible strengthening layouts (i.e. uniaxial fabric, bidirectional fabrix, quadriaxial fabric, FRP strips).
In the most common case of a FRP strengthening layout with quadriaxial fabric applied on the joint panel with fibers oriented in the directions 0°, 90° and ± 45° respect to the beam axis, the FRP area, Af can be calculated as:

$$A_f = n_s \cdot t_f \cdot h_c \cdot cos\theta(1 + tan\theta + 2tan^2\theta) \tag{4}$$

n_s is the number of joint panel sides strengthened in shear with FRP (1 or 2 sides).
θ is the inclination of the concrete compressive strut with respect to the beam axis. It can be assumed with sufficient accuracy that θ = arctan(h_b/h_c).
In the more general case that the joint panel is strengthened with uniaxial fibers in the direction β, measures respect to the beam axial, the area of the FRP fabric in the direction orthogonal to the direction of principal tensile stress can be computed as:

$$A_f = n_s \cdot t_f \cdot h_c / \sin\beta \tag{5}$$

The design FRP strain, ε_{fd}, is defined according to Eq. (6) and it cannot exceed the ultimate FRP strain, ε_{fu}.

$$\varepsilon_{fd} = 34 \left(\frac{f_{cm}^{2/3}}{A_f \cdot E_f}\right)^{0.6} \tag{6}$$

The proposed formulation was calibrated by Del Vecchio et al. (2015) based on experimental tests on FRP strengthened beam-column joints. It complies with the safety requirements suggested by the Eurocode (CEN, 2002). The proposed formulation allows to account for the influence of the number of layers and the quality of the concrete support on the debonding strain of the FRP system as commonly found in other applications (i.e. flexural or shear strengthening). When the ratio of concrete compressive strength of the support and the stiffness (A_fE_f) of the FRP strengthening system is high, Eq (6) provides design strain higher than the 0.4% suggested by the CNR DT 200 (2013). By contrast, when multiple layers of FRP are used this ratio decrease resulting in a design strain significantly lower than 0.4%.

Design procedure for the joint panel FRCC strengthening

A viable alternative to the FRP strengthening of deficient beam-column joint is the thin jacketing of the joint panel by mean of high performance Fiber Reinforced Cement Composites. The high tensile strength of fiber reinforced concrete may provide a significant contribution to the principal tensile stress in the joint panel avoiding the joint panel shear failure. The effectiveness of this innovative solution has been recently investigate through experimental tests on full-scale beam column joints by Del Vecchio et al. (2018). The experimental test showed that the proposed strengthening solution could be a viable and resilient alternative to the classic RC jacketing or concrete cover replacement before the application of an externally bonded FRP system in the case of poor-quality concrete substrate. The high tensile strength avoided joint panel shear cracking and contained the jacketing within the original cross-section dimensions.
Although the proposed strengthening solution is effective in the seismic strengthening of the deficient beam-column joints, reliable design formulations are still lacking.

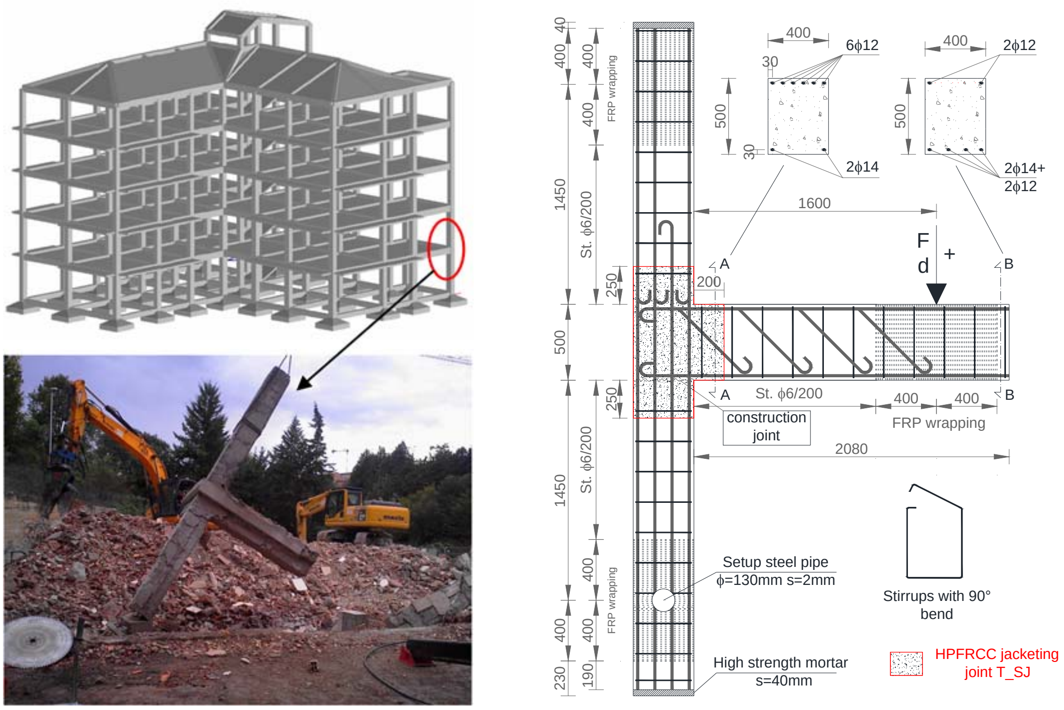
In this study the design approach proposed by Del Vecchio et al. (2015) and described above is extended to help the designer in the estimation of the required thickness of the RCC strengthening of the joint panel. In particular, the contribution of the FRCC in the direction of principal tensile stresses can be calculated by using the Eq. (3). In this equation the area of the strengthening system, A_f, can be taken as the area of FRCC measures in the direction orthogonal to the direction of the principal tensile stress and the elastic modulus of the FRCC in tension. Since the FRCC strengthening covers the entire joint panel, it is as a continuous reinforcement working in all the directions. As first tentative, the equivalent area of the FRCC on the joint panel contributing in the direction of the principal tensile stress can be estimated by using the Eq. (5) and assuming that the inclination, β, is equal to the inclination of principal tensile stresses (90°-θ). This allows to simplify the mechanical problem assuming that the FRCC strengthening reacts mainly in the direction of the principal tensile stress. Neglecting the contribution of the FRCC jacketing in the other directions obviously leads to a conservative design of the strengthening system. The elastic modulus, E_s, can be taken as the elastic modulus of the FRCC in tension, while the design strain, ε_{fd}, can be taken as the strain at the peak strength. It is worth noting that in case of strain hardening FRCC, the cracking of the cementitious matrix is expected before the achievement of the peak strength. Where the scope of the joint panel shear strengthening is to avoid the cracking of the joint panel the design strain should be limited to the first cracking of the FRCC. Based on these assumptions, the contribution of the FRCC jacketing in the principal directions can be calculated as:

$$p_{t,f}^{cap} = \frac{t_{FRCC} \cdot b_{FRCC} \cdot E_{FRCC} \cdot \varepsilon_{fd}}{b_c \cdot h_c / sin\theta} \tag{7}$$

It is worth noting that this formulation assumes that the effective area of the FRCC strengthening system is the product of thickness of the jacketing, t_f, and the width of the FRCC strengthening measured in the direction orthogonal to the direction of principal tensile stress, b_{FRCC}. As first tentative b_{FRCC} is set is equal to the full width of the FRCC system on the joint panel (i.e. b_{FRCC} = h_c/cosθ). However, this is a strong assumption and it needs to be checked against experimental observations.

Case study building: application and validation

A five-story RC building damaged by the L'Aquila earthquake was selected for this study. Because of severe damage to structural and nonstructural members (Fig. 1), the building was rated E according to the AeDES form (Baggio *et al.*, 2007). A detailed seismic assessment and design of retrofit alternatives, carried out during the reconstruction process, established that demolition and reconstruction was the most suitable solution. This was due to the poor quality concrete (f_{cm} < 8 MPa in relevant portions of the building) and the economic inconvenience of structural retrofitting (Di Ludovico *et al.*, 2017b). The structural system consisted of lateral resisting frames designed in the 1963 according to Regio Decreto (R.D., 1937) with moderate seismic actions [horizontal force equal to 7% of the gravity load, according to (D.M., 1962)]. Lack of seismic reinforcement detailing and smooth bars characterized the structural members.



(a) (b)

Figure 1. Case study building demolished after the L'Aquila 2009 earthquake and reference joint subassembly (a); geometry and reinforcement details of the reference joint.

Two corner joints were extracted from a perimetral frame and tested in the laboratory at University of Napoli (see Fig. 1a) under a constant axial load on the column (412 kN) and reverse cyclic displacement at the beam's tip. The geometry of the sampled specimens is repoted in fig. 1b. The total clear height of the column is about 3.4 m, while the cler length of the beam is about 1.6m. The beam cross-section is 400 mm width and 500 mm height. The beam longitudinal reinforcements at the interface with the joint panel are 6ϕ12 at the top and 2ϕ14 at the bottom. The column cross-section is 550 mm width and 400 mm height reinforced with 2ϕ14+1ϕ12 both at the top and bottom side plus 4ϕ16 intermediate bars.
Destructive characterization tests through core drilling and sampling of longitudinal reinforcement were conducted to identify the material mechanical properties of the two joint subassemblies in laboratory. The mean concrete compressive strength measured from compression tests on cylindrical samples, f_{cm}, was approximately 12 MPa. The yielding stress of internal steel reinforcement obtained by the experimental test is about 390 MPa and 361 MPa for the 16 mm and 12/14mm diameter bars, respectively.
These joints will be used in this research work to design proper strengthening solutions by using FRP and FRCC (see Figure 2) according to a local retrofit strategy. Furthermore, the experimental results discussed in Del Vecchio et al. (2018) will be used to validate the proposed design assumptions.

Design of the FRP strengthening

According to the design procedure described in the previous section, to allow for the development of the maximum beam flexural strength in both the positive (M_{max} = 77 kNm) and negative (M_{max} = 128 kNm) load direction the joint panel should be capable of resisting the joint shear stress demand, $v_{j,h}$, about:

$$v_{j,h} = \frac{\sum M_{yb}\left(\frac{1}{jd_b} - \frac{1}{H_n}\frac{L_{b,n}}{L_b}\right)}{(b_c h_c)} = \frac{128000\left(\frac{1}{0.43} - \frac{1}{3.4}\frac{1.8}{1.6}\right)}{(400 \cdot 400)} = 1.60\ MPa \leq 0.8 \cdot 0.59 \cdot 11.92\sqrt{1 - \frac{0.16}{0.59}} = 4.81\ MPa$$

obtained from Eq. (1) setting $\sum M_{yb}$ equal to M_{max} since $\sum M_{yb} < \sum M_{yc}$.

The joint shear demand can be converted into principal tensile stress demand by using Eq (2). In this case the total demand is p_t^{dem} = 0.87 MPa. Considering that the resisting tensile strength of the as-built joint is about $0.2\sqrt{f_{cm}} = 0.69$ *MPa* a joint shear failure is expected when the beam is subjected to a negative bending moment. This was confirmed by the experimental test on the as-built joint (Del Vecchio, Di Ludovico, Balsamo, *et al.*, 2018).

Thus, an FRP strengthening is needed to sustain the target principal tensile stress demand in the joint panel. The principal tensile stress demand in the FRP strengthening can be calculates as the difference between the total tensile stress demand and the concrete contribution as specified in Eq (2). In this case it results in $p_{t,f}^{dem} = p_{t,}^{dem} - 0.2\sqrt{f_{cm}} = 0.87\text{-}0.69 = 0.18$ *MPa.*

The design of the FRP shear strengthening of the joint panel was carried out by using the step-by-step procedure described above. In this case a quadriaxial CFRP fabric with t_f = 0.053 mm, in each of the fibre direction (0°, 90° and ± 45°), elastic modulus E_f = 230000 MPa and ultimate strain ε_{fu}.= 1.5%. is used. The FRP area on the joint panel can be computed by means of Eq. (4). Assuming n_s = 1, since it is a corner joint, h_c = 400 mm and $\theta = \arctan(h_b/h_c)$ =0.90 rad, the area A_f is equal to 71.2 mm^2.

The FRP design strain can be computed by using the Eq. (5) resulting in this case in ε_{fd} = 0.43% which is significantly lower than the ultimate FRP strain.

The FRP contribution to the principal tension stress can be computed by using Eq. (3), resulting in $p_{t,f}^{cap}$ = 0.34 MPa which is higher than the demand $p_{t,f}^{dem} = 0.18$ *MPa.* Thus, one layer of quadriaxial CFRP is useful to avoid the joint panel shear failure promoting the full development of the maximum beam flexural capacity.

It is worth noting that in case of a concrete substrate with a concrete compressive strength lower than 15 MPa, the concrete cover replacement prior to the application of FRP strengthening is recommended to have an effective functioning of the strengthening system. This activity may be time-consuming and may significantly reduce the advantage of a simple and fast application, typical of the FRP systems.

(a) (b)

Figure 2. Seismic strengthening of deficient RC beam-column joints: FRP strengthening (a); FRCC strengthening (b).

FRCC thin jacketing as novel strengthening solution

An alternative solution to the FRP strengthening of the joint panel is the FRCC thin jacketing. In this case study, the FRCC material used for the thin jacketing of the joint panel (see Figure 1) contained (1) high-strength cement and selected fine aggregates (maximum size 2.5 mm), (2) 13-mm-long and 0.21-mm-diameter straight steel fibers in proportion by weight 1:0.065, and (3) water in proportion by weight 1:0.13. The strength and modulus of elasticity for the fiber material were 2,750 MPa and 200 GPa, respectively. A high-range water-reducing agent was also added to ensure good workability of the mixture. The mechanical properties of the FRCC are: concrete compressive strength about 103 MPa, tensile peak strength (corresponding to the first cracking) about 4.9 MPa, a tensile stress at the first cracking about 0.025% and elastic modulus in tension about 20.000 MPa.

The required thickness of the FRCC jacketing can be calculated assuming that principal tensile stress in the joint core does not exceed the strength of FRCC strengthened joint. To avoid the joint panel shear failure due to diagonal tension the target principal tensile stress, $p_{t,f}^{dem}$, is set equal to *0.18 MPa (*corresponding to the fully exploit of the maximum beam flexural strength*). By* means of the Eq. (7) and fixed that $p_{t,f}^{cap} = p_{t,f}^{dem}$= 0.18 MPa, t_f can be calculated. In this example, the design results in a thickness of the FRCC jacketing about 12 mm. It is worth noting that a very small thickness comes out from the proposed design approach. This is because the effective are of the FRCC strengthening is considered as the product of t_f and the full width of the FRCC system covering the joint panel, measured in the direction orthogonal to the direction of principal tensile stress. This relies on the assumption that joint panel is subjected to a uniform stress field, which is obviously not completely trustable. Indeed, it is difficult to believe that the FRCC jacketing at the corner of the joint panel are subjected to the same stress field of the concrete core, at least when the joint panel is not cracked.

This is confirmed by experimental observation on the T_SJ joint tested by Del Vecchio et al. (2018). This is the same joint used in the design example illustrated above. It was strengthened with a 40 mm thick FRCC jacketing and tested under cyclic load. The outcomes of the test showed the effectiveness of the FRCC jacketing in avoiding the joint panel shear failure and promoting a more ductile beam hinging. The ductile failure of the beam resulted in an increase of the energy dissipation about the 85%. Although, the joint was strengthened with a FRCC jacketing about 4 times thicker than the one obtained from the previous design procedure, the local strain measurements on the joint panel showed that a strain close to the FRCC rupture (about 0.021%). Thus, further research studies are needed to investigate on the effectiveness of the FRCC jacketing and to better calibrate an effective design procedure.

Conclusions

This research work illustrates the available design procedures for the seismic retrofit of deficient beam-column joints by using composite materials. First, the design procedure currently available in the fib Bulletin 90 for the FRP strengthening of beam column joints is described. The same procedures relying on the principal stress approach is later extended to design of the FRCC strengthening of the joint panel. It allows to calculate the required thickness of the FRCC jacketing.

An existing RC case study building is selected to shows the capabilities of the proposed design approaches. The reference building was demolished after the L'Aquila earthquake and two beam-column joint were extracted and tested in laboratory. The test results of the FRCC strengthened joint are used to validate the novel design procedure.

The proposed procedure, originally developed for the design of the FRP strengthening of the joint panel is suitable for extension to the FRCC jacketing. Proper changes are proposed to consider the different behaviour of the FRCC jacketing. The comparison with experimental results shows that further research studies are needed to improve the reliability of the proposed procedure that currently may lead to an un-conservative design of the required thickness of the FRCC jacketing.

Acknowledgment

This research was carried out within the framework of the reSTARt-school research project funded by the UNINA - Compagnia San Paolo STAR 2018 program.

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