# Seismic Retrofit Schemes with FRP for Deficient RC Beam–Column Joints: State-of-the-Art Review

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## Abstract

This paper aims to review and critically assess experimental research efforts on the seismic retrofit of existing reinforced concrete (RC) beam-column joints with fibre-reinforced polymer (FRP) sheets of the past 20 years. The review of the literature revealed several promising features of FRP strengthening schemes. FRP retrofits can be used to address a number of different deficiencies in non-seismically designed RC members framing into beam-column joints. A majority of studies concentrate on joint shear strengthening and strengthening in the axis of principle stress is found to be most effective. Other strategies include counter-acting the weak-column/strong-beam in non-seismically designed specimens by means of column flexural strengthening, as well as plastic hinge relocation within the beams, away from the joint. Only a limited number of studies look at combining several of these retrofit objectives into a more complete retrofit of the joint sub-assemblage. In most studies it is observed that simple FRP wrapping is used for anchorage, which is not always effective. Instead, it is shown that anchorage by means of FRP anchors or mechanical anchors is required to achieve adequate strengthening in most cases. Next to the detailed discussion of the literature, a database of all tested specimens is compiled and analysed. An assessment of shear strengthening design equations from major design guidelines is made based on the experimental results collected in this database, highlighting the need for their further improvement. Moreover, analysis of the database reveals a lack of tested specimens with realistic test set-ups, including scaled specimens, testing without axial load, as well lack of slab and transverse beams. It is found that these parameters heavily affect retrofit effectiveness and may lead to non-conservative results. Moreover, on average, the effectiveness of repairing pre-damaged specimens is found to be similar to that of retrofitting specimens without damage.

## Introduction

Reinforced concrete (RC) structures built in earthquake prone areas and designed to resist gravity loads only or before the introduction of modern seismic codes (pre-1970's or 80's), typically display several deficiencies resulting in inadequate behaviour under seismic loading. In particular, the hierarchy of strengths around beam-column joints plays a critical role in the overall cyclic behaviour of RC structures. Avoiding premature failure of said joints is hence a fundamental principle in modern seismic design to allow the framing members reach full capacity (Kappos and Penelis 1996).

Adequate energy dissipation under seismic loading also relies on an appropriate hierarchy of strengths between the framing members of the joints. To achieve larger global structural displacements and hence higher ductility under seismic loading, modern seismic design guidelines aim to ensure beam-hinging precedes column-hinging mechanisms (Fardis 2009).

For existing structures, in which an adequate seismic behaviour is not ensured, structural retrofit often allows changing the expected failure mechanism. A variety of retrofit methodologies and materials exist, traditional retrofitting techniques, such as concrete or steel jacketing, addition of shear walls and epoxy repair, have proven effective and popular, as suggested in a detailed review by Thermou and Elnashai (2006). Concrete jacketing involves addition of new longitudinal bars, ties and a layer of concrete which increase the cross-section dimensions. Steel jacketing refers to encasing the element with steel plates and filling the gap with grout or epoxy resin. These techniques aim increase both flexural and shear strength and improve concrete confinement. Yet, many of these methods present practical issues, namely, adding weight and stiffness to structural elements, increasing their cross-section dimension, as well as being and labour intensive (Engindeniz et al. 2005).

In the last twenty years, retrofitting RC structures with fibre-reinforced polymer (FRP) are growing in interest due to the light weight and high strength combined with corrosion resistance of FRP materials. The application of FRP wraps can be performed rapidly and without disrupting the building occupancy, which is another major advantage, as it reduces the down-time in businesses and the need of relocating inhabitants in residential properties (Bousselham 2010). FRP retrofits however also suffer from disadvantages, such as weak bond to concrete and low fire resistance. To improve the fire resistance of FRP, however, solutions are being investigated, for instance applying intumescent coatings (e.g.: Ji et al. 2013). Recently, beam-column joint retrofits using other composite materials using mortar instead of epoxy to bind the fibres to the concrete substrate have hence also become popular, including the use of textile reinforced mortars (Al-Salloum et al. 2011), or fibre reinforced cement composites (Del Vecchio et al. 2018), benefiting from better bond, as well as improved thermal and fire resistance.

This paper will instead focus on the use of FRP for joint shear strengthening. FRP can be used to ensure capacity design hierarchy of strengths by providing selective strengthening of members framing into the joint relative to their respective load capacities. FRP upgrades are used to address distinct strengthening objectives:

- Joint shear strengthening by means of sheets placed in the horizontal (e.g.: El-Amoury and Ghobarah 2002), vertical (e.g.: Le-Trung et al. 2010) or diagonal axis (e.g.: D'Ayala et al. 2003) across unobstructed joint panels.
- Column flexural strengthening to prevent unwanted column hinging failure, using straight FRP sheets along the column axis (Antonopoulos and Triantafillou 2003), L-shapes (Akguzel and Pampanin 2012a; Garcia et al. 2014; Yu et al. 2016), near-surface-mounted (NSM) FRP (Hasan et al. 2016; Prota et al. 2004) or FRP anchors (Shiohara et al. 2009).
- Increasing confinement and ductility (Akguzel and Pampanin 2012a; Al-Salloum and Almusallam 2007; Antonopoulos and Triantafillou 2003; Del Vecchio et al. 2014; Engindeniz et al. 2008b; Shiohara et al. 2009) or shear strengthening (Lee et al. 2010) of columns using sheets wrapped fully around the column.
- Preventing beam bar-slippage and cracks opening at the beam-joint interface using FRP sheets along the bottom face of beams (Engindeniz et al. 2008b) or L-shapes at the bottom corner between beams and columns (El-Amoury and Ghobarah 2002; Ghobarah and El-Amoury 2005).
- Shear strengthening beams with FRP U-wraps perpendicular to the beam axis (Akguzel and Pampanin 2012a; Alsayed et al. 2010; Antonopoulos and Triantafillou 2003; Engindeniz et al. 2008b).

While the number of research papers in this field is increasing rapidly in the last years, the last thorough state-of-the-art review paper dates back to 2010 (Bousselham 2010). Understanding current trends and compiling what has been tested so far is crucial to reveal promising avenues for future research, as well as uncovering gaps in available experimental data. The aim of this paper is hence to address this need to review and critically assess the state of experimental research on RC beam-column joint retrofits with FRP.

A detailed review and a database of all existing experimental research on seismically strengthened beam-column joints with FRP is presented in this paper. The retrofit schemes are assessed in terms of their effectiveness (increase in strength or ductility), as well as their practical applicability to real structures. At the end of this review, an assessment of shear strengthening design equations from major design guidelines is made based on the experimental results collected in the database. This is followed by a statistical analysis of the database addressing the experimental design in terms of type and geometry of set-ups, material properties and leading to a discussion on the parameters affecting the effectiveness of retrofits.

**Review of existing research**

Beam-column joint specimens with typical pre-1970's design deficiencies, retrofitted with FRP and tested under cyclic loading are the focus of this review. Research papers only considering static loading (push-over) are hence excluded and so are experimental specimens designed according to modern seismic design codes. Shake table tests on full frames or cyclic tests on individual RC members are also excluded from this review. Furthermore, research on retrofitting bridge pier connections is excluded in this study, due to the significant differences in terms of size, loading and desired response to seismic actions between buildings and bridges.

To facilitate a critical and systematic review of the literature a database of experimental work on the seismic FRP strengthening of RC beam-column joints in buildings is compiled. A summary of this database can be found in Appendix A. This work builds upon similar efforts by other researchers (Bousselham 2010). In the process of the review of existing literature, a number of parameters were recorded, including the type of deficiencies in the control specimen, the geometry and dimensions of specimens, the material properties (concrete, steel, FRP), details on the FRP strengthening (aim, fibre type, number of layers, dimensions, surface preparation, presence and type of anchors), as well as information on the experimental set-up, loading, instrumentation and the available results of the experiments (load and displacement ductility).

This review is organised thematically by geometry of the tested beam-column joint specimens (exterior, interior or corner joints), as well as the main strengthening objective (joint shear strengthening, column strengthening, beam strengthening, or multi-objective retrofit). Every major section is completed by a concise summary of observations, as well as a table summarising representative results obtained from the respective experiments and visually describing the different retrofits using schematics. As the achieved ductility is not always published in the studied papers, for the summary tables of each chapter, only comparisons in strength increase are given.

***Shear strengthening of two-dimensional exterior joints***

A well-researched topic in the literature is the shear strengthening of shear-deficient exterior joints without slab or transverse beams. A summary of the four main avenues for shear retrofitting is found in Fig. 1, in which schematics of the X-shaped (a), U-shaped (b), T-shaped (c) and retrofits with multi-axial sheet (d) can be seen. In Table 1, representative strength increases obtained by the different papers described in this section are also included to ease their comparison.

Ghobarah et al tested six shear deficient full-scale exterior joints and propose two different retrofit layouts (Ghobarah and Said 2002). The “U”-configuration, with a single layer of bi-directional GFRP sheet wrapped around the free sides of joint, performs well when anchored with steel plates (specimen T1R). Delayed shear failure with nearly double the ductility is observed, limited only by the tearing of the single layer of FRP. With two GFRP-layers extended above and below the joint on the column faces (T2R), the behaviour is further improved. Failure is transferred to a ductile beam hinging mechanism, achieving a five-fold increase in energy dissipation. Without anchorage (T4), early debonding occurs, impeding contribution to the shear strength.

For an “X”-configuration of unidirectional GFRP wrapped diagonally around the joint (T9), using steel angles to ease the FRP-application, initial damage is transferred to the beam, however, as the FRP debonds, the joint still fails in shear. An increase in ductility similar to T2R is achieved, with a slightly lower increase in energy dissipation. The results would suggest that adequate anchorage to resist debonding could improve this layout. Overall, the importance of anchorage to avoid debonding is particularly highlighted by these experiments.

Compared to a joint designed to modern Canadian RC guidelines (CSA A23.3 1994), 80% of the displacement ductility and 90% of the load capacity can be achieved by the best retrofit, however with about half the energy dissipation (Said and Nehdi 2004).

Antonopoulos and Triantafillou (2003) designed 18 2/3-scale exterior joints to fail in shear, so as to assess the relative contribution of different retrofit parameters on their shear capacity. In all retrofits, FRP is placed on the joint along the vertical and horizontal axis in a T-shape and delayed shear failure is observed. Using an equivalent amount of GFRP results in slightly better energy dissipation and shear strength (+45%) compared to CFRP (+41%), likely due to its higher rupture strain, as fracture is observed for the CFRP sheet. An increase in number of FRP layers is also found to enhance strength and energy dissipation, but not proportionally. Specifically, doubling the number of horizontal layers (F21) is found to achieve a higher strength increase (+65%), than in the vertical direction (F12, +15%), compared to a single-layer retrofit (F11).

Doubling the applied axial load in F22A, is found to be an important factor (+22% of strength), due to enhanced joint confinement. Bond properties are shown to be equally important, as in all unanchored retrofits, debonding is observed. Using FRP wraps for anchorage (F22W), achieves significant shear strength enhancement (+24%) compared to an unanchored counterpart F22. The effect of anchorage is dramatically stronger (+250%) for an FRP-strip retrofit anchored with steel plates (S33L). This can be attributed to the weaker bond properties of FRP strips compared to sheets.

Other important factors for retrofit effectiveness are reinforcement detailing and geometry, with just one joint shear stud significantly reducing the effectiveness of the retrofit (-48%). For specimens with a stub transverse beam, the efficiency of the retrofit is significantly reduced (up to -78%), as FRP cannot be fully applied to both sides of the joint panel. This indicates that results of retrofit efficiency inferred from scaled specimens with simplified geometry may not be transferrable to actual structures.

The effect of the steel shear reinforcement in the joint is the objective of Karayannis and Sirkelis (2008) study on 12 half-scale joints strengthened by U-shaped CFRP wraps. Anchorage is achieved perpendicular wrapping in beam and columns. The retrofit strategy is effective in strengthening the joint and relocating damage into the beam. Lower damage is observed for specimens with transverse steel reinforcement, but the retrofit effectiveness is higher for specimens with no shear studs (+88.4% vs +65.3%). This highlights the effect of steel reinforcement ratio on the FRP retrofit effectiveness. In real pre-1970’s joints, often the steel reinforcement is low or inexistent, which means the potential strength enhancement with FRP will be higher.

Le-Trung et al. (2010) tested seven 1/3-scaled exterior joints with no transverse steel reinforcement, using two different CFRP retrofit layouts to prevent joint shear failure and ensure ductile beam hinging instead. Combined with the unrealistic scale of specimens, also no axial load is applied in these experiments, meaning the results should be taken with care.

For the “T”-shaped configuration, CFRP sheets are applied on two sides of the joint, extended onto column and beam. Additional “L”-shaped FRP sheets are applied at the corners between beams and columns to prevent bar-slippage and delay crack opening, with anchorage strips at column-ends and/or beam-ends. Delamination is observed for all specimens but delayed for specimens with anchorage strips compared to the non-anchored specimen (RNS-1). This leads to higher ductility, up to three times the control specimen. For specimen RNS-5 with beam-end anchorage, debonding near the joint is observed rather than the beam-end, leading to a low strength increase (+11%). Strip anchorage at the extremities only is hence not sufficient and further strips, or mechanical anchorage, would be required. Compared to a seismically designed specimen, only specimen RNS-6, with two FRP-layers, achieves a higher strength increase (+32% vs +22%).

The “X”-shaped configuration with fibres at 45° on three sides of the joint (RNS-3 and RNS-4) is found to be most effective in increasing ductility, with a five-fold increase compared to the control specimen. Compared to Ghobarah’s (2002) retrofit, diagonal wrapping is extended onto the columns, which creates better anchorage, hence preventing debonding. Additional “L”-shaped strips without anchorage in specimen RNS-4 only achieve a delay in the onset of failure, however early debonding reduces its effectiveness at higher drift ratios. In terms of strength, the improvement is less significant than for the T-shaped retrofit, with a similar increase is obtained for both X-shaped retrofitted joints (+17% and +16%).

Realfonzo et al. (2014) tested a series of eight full-scale shear-deficient exterior joints, half with a “weak column” configuration (Type 1) and the other half with a “strong column” configuration.

While all eight specimens are two-dimensional, for six specimens, the presence of a floor slab is simulated by means of thick steel plates positioned over a depth of 300 mm from the top of the beam.

Both types of specimens are retrofitted with U-shaped CFRP sheets for joint strengthening and CFRP wrapping of the columns for confinement. For the “weak column” specimens, additional threaded steel rods for column flexural strengthening are provided. The first specimen was retrofitted with the largest amount of FRP in the joint, using two U-shapes of 150 mm width, but without anchorage. Only a slight increase in strength (+23%) was obtained and joint shear failure was not prevented due to FRP delamination. The strength is nearly doubled (+98%) for a retrofit of the same specimen with two U-shapes of only 100 mm width but adequate anchorage. In this case joint shear failure can be prevented and a ductile beam hinging mechanism is promoted. For the three “strong column” specimens, three anchored U-shaped CFRP strips of 50 mm width are applied in the joint area. Joint shear failure is delayed, but not prevented due to delamination of the U-shapes after opening of flexural cracks at the column-joint face. Due to differences in the CFRP strengthening of the exterior column face, flexural cracking occurred at different stages for the three specimens, hence leading to strength increases varying from 54% for X-shaped column strengthening to 75% for longitudinal strengthening. It is important to add that FRP strain in the U-shapes was measured throughout the tests, with maximum recorded strain values between 1/3 (0.25%) and 1/2 (0.4%) of the ultimate FRP strain (0.8%). These values are similar in magnitude to the 0.4% design strain often assumed in design codes to avoid potential debonding (Pohoryles and Rossetto 2014). Finally, while the authors made an effort to include the presence of a floor slab in their design, the presence of potential transverse beams would affect the feasibility of the retrofit scheme significantly.

Hadi and Tran (2014, 2016) proposed an unusual retrofit method for shear deficient exterior joints. Glued-on concrete covers are installed around the columns and joint to modify the square cross-sections and exploit the increased effectiveness of FRP wrapping in circular cross-sections. Three full-scale specimens with different amounts of U-shaped CFRP in the joint (one, three and six layers) are then tested. While the retrofits with lower amounts of FRP achieve delaying joint shear failure, only with six layers of FRP a successful change to beam hinging is observed. The increase in strength and ductility observed is not proportional to the amount of FRP used. With one (+84% in strength) and three layers (+116%), FRP rupture in the column and joint are observed, while for the specimen with six layers of FRP (+140%), no debonding or rupture is seen.

While the increase in strength employing this joint shear strengthening strategy is significant, it is highly impractical in real structures with transverse beams, walls or slabs present, which prevent the placement and continuity of the additional circular covers. Moreover, the effect of the additional concrete covers alone is not assessed by the authors of the study. This would however allow a fairer assessment of the effectiveness of the FRP intervention.

This section presented arguably the most prominently featured retrofit objective in the field, shear strengthening of two-dimensional exterior joints. As highlighted in Fig. 1, the suggested retrofits in this section can be split into four groups:

- the U-shaped configuration, wrapping the joint horizontally using U-shaped FRP;
- the T-shaped configuration, applying FRP in horizontal and vertical directions in the joint;
- the X-shaped configuration, applying the FRP in the diagonal of the joint, following the angle of principle stress.
- Application of bi- or quadri-axial FRP in the joint, similarly to the X-shaped configuration.

A main observation from all experiments is the need for anchorage. For U-shaped configurations, Ghobarah and Said observed no improvement in behaviour without anchorage. Similarly, Realfonzo et al (2014) obtained a modest strength increase without anchorage, compared to a very strong (+99%) increase for an adequately anchored U-shaped retrofit. Anchorage is also found to be crucial in the study by Antonopoulous and Triantafillou (2003), which compared a variety of T-shaped retrofits with different amounts of horizontal and vertical FRP. It is found that the effect of horizontal FRP layers on the strength of retrofitted joints is more important than that of vertical layers. The effect of axial load is also found to be a critical parameter.

Comparing X-shaped retrofits with U-shaped or T-shaped retrofits, despite having the fibres oriented in the axis of principle stress, a reduced strength enhancement is found by Ghobarah and Said (2002) and Le-Trung et al. (2010). In both cases, this may however be associated to debonding problems for the X-shaped configuration. In practical terms, retrofits with horizontal and vertical sheets of FRP are easier to anchor, as they can be anchored on beams and columns using steel anchors or FRP wrapping.

Hadi and Tran (2016) proposed a highly successful retrofit scheme, which consists of adding concrete covers to render the joint cross-section rounder before applying U-shaped FRP-wrapping. While the retrofit is the most effective in terms of strength increase (+140%), it´s applicability to real structure is debatable. Generally, practical applicability of the retrofit schemes presented in this section is limited to structures without RC slabs or transverse beams, as these would obstruct the retrofits and require significant changes to the proposed lay-outs or partial removal of RC members.

***Shear repair of two-dimensional exterior joints***

Five recent studies specifically investigated the effect of pre-damage on the effectiveness of interventions by repairing exterior joints with FRP. Agarwal et al. (2014) repaired a damage full-scale specimen using cement grouting and five layers of GFRP U-wrap in the joint, extended and anchored by five layers of full-wraps around the beam. The full strength of the control specimen is not recovered, as only 68% of the capacity is reached. Moreover, a reduction in initial stiffness is observed and joint shear failure is not prevented, due to debonding of the GFRP in the joint panel. An improvement in ductility is however observed (+43%).

Garcia et al. (2012, 2014) repaired three full-scale exterior joints by replacing the damaged core with concrete of nearly double the compressive strength. The joints are then rehabilitated with CFRP U-wraps around the joint extending into the column, longitudinal FRP along the column for flexural strengthening, as well as full confinement wraps of the column and beam to ensure an adequate beam hinging mechanism. While FRP rupture and significant damage are observed in the joint core of the repaired specimens, a significant strength increase of +51.2% (two layers of FRP) up to +119% (three layers) over the control specimen, corresponding to a maximum of +69% compared to a specimen repaired by concrete replacement only, is achieved. Notably, this increase in capacity corresponds to 85% of the shear strength of ACI code-compliant RC joints (ACI 2002a).

Yurdakul and Avsar (2015) employed an improved version of “X”-configuration proposed by Ghobarah and Said (2002), with two layers of CFRP and additional FRP wrapping for anchorage to the beam and column, to repair one full-scale exterior joint. After application of a strong repair mortar

(60 MPa), two layers of 200 mm wide CFRP sheets were applied in the joint region at a ±45° angle with respect to beam axis. Joint shear failure, as observed for the control specimen, is not prevented, as FRP fracture, followed by spalling of the repair mortar is observed. The repaired specimen does hence not reach the capacity of the original specimen (-25%) and the initial stiffness is reduced by 28%. The non-achievement of the retrofit objective is explained by an eccentricity in axial loading, causing larger stress in one side of the retrofit. It can however also be deduced that the repair of damaged concrete with the mortar was not done in the most effective way, as a significant reduction in strength and stiffness were obtained. The type of shear failure with rupture of the FRP is also a reminder with regards to the use of lower design strains in the design procedures, rather than assuming full development of the FRP, hence avoiding brittle rupture.

Beydokhti and Shariatmadar (2016) tested the effect of the level of pre-damage on repair effectiveness, applying cement mortar and CFRP U-wraps on the joint for four 2/3-scaled exterior joints of increasing pre-damage. L-shaped CFRP is also applied at the beam-column interface and anchored using a layer of FRP. Despite adequate anchorage length according to the ACI 440.2R.08 guidelines (ACI 2008), buckling and debonding of the beam-CFRP is observed, resulting in flexural failure of the beams at the joint-interface. With increasing pre-damage levels, an increasing deterioration of initial stiffness is observed. For the two specimens with the lowest pre-damage, the strength of the control specimens is recovered, with a slight increase of 2.5% and 5.9% respectively. For the specimens damaged to near-collapse and collapse damage states however, a reduction in strength of 19.5% and 15.3%, respectively, is obtained. These observations suggest that a full recovery of a severely damaged structure is not achievable with the proposed repair scheme and a storey drift of 1.5 % for a damaged structure is introduced as reparability storey drift based on the tested joints.

Faleschini et al. (2019) repaired three full-scale, one-way exterior joints damaged by a shear mechanism in the joint region. The repair process of the pre-damaged joints started with restoring the joint with a premixed cement-based mortar. Two CFRP retrofits were tested and compared to a textile-reinforced mortar (TRM) repair scheme. The first FRP repair used diagonal FRP in the direction of expected principal stress, while the second scheme used U-shaped FRP. In both schemes, FRP jacketing of the columns above and below the joint is applied. This provides anchorage for the X-shaped scheme, while the U-shaped scheme uses transversal FRP in the beams for anchorage. None of the repair schemes achieved restoration of the original strength of the joints. While detachment of the TRM lead to a loss in strength of 41%, for the X-shaped FRP fracture in direction of the principal stress with a strength loss of 30% was observed. For the U-shaped FRP, damage in concrete below the jacket lead to bulging of the FRP and a 34% loss in strength. Overall the performance of the two FRP schemes can be seen as similar, but in the case of the diagonal retrofit, due to the obtained fracture of fibres, it can be hypothesised, that a larger number of FRP layers may improve the repair performance.

With regards to repair of exterior joints, similar approaches to the retrofit schemes have been tested, however the behaviour of the undamaged specimen was often not recovered. Based on the limited tests, it would appear that this is mainly due to the level of repair of the damaged concrete with mortars being insufficient to rehabilitate the joint, combined with an insufficient amount of FRP calculated for the pre-damaged joints.  Looking at results by Beydokhti and Shariatmadar (2016), the level of pre-damage is the most important aspect deciding whether the strength of a damaged joint can be reinstated. This observation is of interest, when looking at the repair scheme by Garcia et al (2012, 2014), which starts with a complete replacement of the damaged joint core concrete, rather than only superficially repairing the joint with mortar. For this repair a significant increase in strength is achieved, while the repaired joints by Agarwal et al. (2014), Yurdakul and Avsar (2015) or Faleschini et al. (2019), which only repair the damaged joint core by grouting before the FRP retrofit, cannot recover the strength of the pre-damaged joint.

***Shear strengthening of corner joints with slabs and/or transverse beams***

A shear-deficient corner joint with slab and a (stub) transverse beam, retrofitted with CFRP is compared to RC-jacketing by Tsonos (2008). The retrofit scheme considers realistic geometric constraints and uses ten horizontal layers of FRP applied on the accessible joint face, as well as nine vertical and seven horizontal layers to improve flexural and shear capacities of the columns, respectively. The joint FRP is anchored with CFRP strips fully wrapped around the beam through pre-drilled slots in the slab. The strength of the joint is improved by 70% and a more ductile beam hinging mechanism is achieved, however due to the lack of anchorage along the beam, damage is concentrated at the beam-joint interface. In comparison to an equivalent RC-jacketed retrofit, similar strength is obtained, but energy dissipation and stiffness are reduced.

Ilki et al. (2011) test corner beam-column joints with slab and transverse beams with very low concrete strength (8.3 MPa) and without joint shear reinforcement, representing a typical Turkish pre-1990's structure. Three layers of bi-directional CFRP are placed on the free side of the joint and anchored using diagonal sheets of FRP wrapped across the beams, which is practically feasible for exterior joints. For the strengthened specimen, joint shear failure is obverted, but beam bar slippage becomes the dominant mechanism, hence only a slight increase in strength is achieved (+18%). To avoid this, beam bars are welded to the back of the joint, which leads to a more ductile and dissipative beam-hinging failure. Using FRP sheets in the bottom face of the beam instead of welding additional bars is not considered as an option but would be an interesting avenue to explore to achieve a full FRP retrofit.

Full-scale corner joints (Del Vecchio et al. 2014) with one transverse beam are retrofitted with quadri-axial CFRP sheets in the joint panel and FRP U-wraps at the beam-joint interface to avoid brittle joint shear failure (specimen FL1). While failure mode and ductility cannot be changed, strength is increased by 17%. A second, more complete scheme, adds perpendicular unidirectional sheets on columns and beams (specimen FS1) to change the failure mechanism, but also to increase the energy dissipation. A variation of this scheme with double the amount of FRP is also tested (specimen FS2). With confinement of the column, a ductile behaviour with large increase in strength (+32% and +49% for FS1 and FS2, respectively) is observed. For specimen FS1, shear failure in the joint is not prevented, but only delayed. Doubling the amount of FRP achieves preventing joint shear failure and moving failure to column hinging, highlighting the need for column flexural strengthening to overcome a strong-beam/weak-column strength hierarchy in pre-1970's structures. Finally, an important observation is made in terms of the recorded strain values. Larger strains, close to the ultimate strain of the material are recorded by the authors, suggesting that the limits proposed by design guidelines (e.g. 0.4% strain) are conservative.

***Shear strengthening of two-dimensional interior joints***

Mosallam (2000) compared the effectiveness of CFRP and GFRP on two half-scale interior joints retrofitted with two layers of bi-directional sheets fully wrapped around beams, columns and joint, as well as diagonal straps around the joint. The tested specimens with GFRP display a greater increase in ductility, but in terms of ultimate load, the results for both materials are identical. The retrofit is heavily limited by practical considerations in terms of retrofit layout and scale, as slabs and transverse beams would not allow placing FRP wraps around every member.

D'Ayala et al. (2003) studied the retrofit of interior joints characterised by either a weak-beam (WB) or a weak-column configuration (SB). L-strips are applied at the corners of the joint to improve the capacity of the beam, as well as vertical and horizontal FRP sheets in T-form on the joint to increase its shear strength (W1). All strips are anchored by perpendicular wraps at beams and columns, but delamination still occurs and the retrofit is not successful in shifting the failure mode. A maximum increase in strength of 17% is obtained for this strengthening scheme.

A further layout is hence tested, using diagonal strips around the joint core with full wrapping of beams and column (W2). This X-shaped retrofit is more effective than the orthogonal retrofit, not only because of the direction of fibres, but also because the joint is more confined at its corners. The most significant improvement in terms of strength (+93%), but also in energy dissipation and reduction in stiffness degradation are obtained. Still, joint cracking is observed, mainly attributed to the low concrete strength ($f_{ck}$ = 10MPa). The same retrofit schemes are also tested as repair schemes on the fully damaged control specimens, following removal of damaged concrete and grouting. In line with observations for exterior joints in this review, no improvement in behaviour is observed, even if the strength of the original joint is recovered. Finally, the authors state that their retrofit design does not include the presence of floor slabs, which would restrict the applicability of the suggested retrofit, unless the floor slab is perforated.

Lee et al. (2010), applies horizontal FRP in the joint for full-scale interior specimens, anchored to the beams with steel-angles to provide shear strengthening. The strengthening is combined with four layers of CFRP strip along the corner of the columns and through the joint. The retrofit is successful in increasing damage in the beams and delaying joint shear failure. This leads to improved ductility, with a more dissipative failure mechanism (+90%) and a higher load capacity (+36%).

***Column-strengthening in interior joints***

To achieve an adequate failure mechanism, next to joint shear strengthening, column flexural strengthening is often also required. Three distinct strengthening schemes for columns in beam-column joint sub-assemblies are identified in the literature and summarised in Table 2 and Fig. 2, in which the retrofits are also depicted schematically. Prota et al (2004) carried out tests on ten half-scale interior joints with weak-column/strong-beam configuration under three different axial load levels (v=0.1, 0.2 and 0.3). The aim of their work is to move failure from the columns to the beams. In a first stage, the effect of column confinement CFRP wrapping alone is compared to wrapping combined with flexural strengthening through near-surface mounted (NSM) CFRP rods applied continuously through the joint between both columns.

Using continuous NSM rods, as shown in Fig. 2 (b), failure can successfully be moved away from the columns, leading to higher strength increase (up to +62%) compared to confinement wrapping alone (+33%). As failure is now moved to the joint, ductility is lowered, and additional strengthening of the joint is hence applied by means of bidirectional CFRP sheets (labelled JS in Fig. 2 (b)). This full retrofit can lead to similar strength increase (+83%), but also strong increase in ductility (up to +73%). In practice, addition of the bidirectional sheets however prevents the retrofit from application in interior joints with transverse beams. The two main observations of this study are that higher axial loads lead to a larger strength increase and that the flexural strengthening of the columns (NSM rods) needs to be applied continuously through the joint, otherwise flexural failure of the columns cannot be prevented.

A similar method, feasible for interior joints with slabs, uses bundled CFRP strands with fan-shaped anchors on either end, passed through small plastic tubes at the column corners (Shiohara et al. 2009). This retrofit shown in Fig. 2 (c) has the advantage of providing continuity of the longitudinal FRP along the column, as well as contributing to the shear resistance mechanism by confinement of the joint. The retrofit is tested on one-third scale interior joints without slab and no axial load. An increase in maximum story drift (+30%) and shear force (+13%) are obtained, as yielding of the column bars is prevented, but damage in the joint core is not reduced. Slippage of the CFRP strands is observed and energy dissipation hence not improved. The retrofit shows potential for flexural column strengthening, however testing on a more realistic specimen and additional joint shear strengthening are required.

Recently, Yu et al. (2016) proposed an easily applicable retrofit using L-shaped FRP laminates at the corners of beams and columns in weak-column interior joints. The L-shapes are anchored by full wraps on the columns and U-shapes on the beams (Fig. 2 (a)). The effect of pre-damage level, axial load (v= 0.3 and 0.5), as well as FRP material (CFRP and BFRP) are tested on realistic interior joints with slab and transverse beams. Successful reversal of the hierarchy of strengths with beam-hinging is observed for all specimens. With increased extent of damage, a reduction in initial stiffness, but no significant influence on maximum strength (less than 5% difference) or ultimate drift are observed. Unlike observations by Prota (2004), the effect of increased axial load on strength is minimal (+3%).

Finally, using CFRP results in a stronger increase in capacity (up to +26%) compared to BFRP (+11%), but a lower increase in ultimate drift (24% vs 49%). This is a consequence of the higher strength and stiffness of CFRP and echoes results obtained for CFRP and GFRP by other authors (e.g. Antonopoulos and Triantafillou 2003). It is worth noting that the effective strain in both FRP materials is half the guideline predictions (ACI 2008). Yu et al. propose an improved conservative strain calculation for L-shaped laminates, highlighting that effective strain in FRP can limit retrofit effectiveness.

Column strengthening in beam-column joint sub-assemblies is a topic generally only addressed by providing confinement to the column above and below the joint. The three studies in this section however highlight the importance of flexural strengthening of columns in pre-1970´s joints that often suffer from an inadequate weak-column/strong-beam hierarchy of strengths. Prota et al. (2004) show the need for using continuous flexural strengthening through the joint in order to prevent column hinging. Their use of NSM bars at the column faces is however considerably less practical then the proposal of Shiohara et al. (2009), which instead use FRP strands passed through holes at the corners of the columns. This retrofit has the advantage of only needing minor intervention to the slab in order to be applicable in structures with slabs and transverse beams, while the NSM bars on the column face could not be continued through the transverse beam. Using L-shapes in the top and bottom columns anchored with FRP wraps around the beams (Yu et al. 2016) is also successful in preventing column hinging, while being much simpler to apply. Only in the presence of infill walls this retrofit would be less practical then Shiohara´s.

### ***Beam-strengthening in joints***

Following capacity design principles, beam-hinging is the preferred mode of failure for a joint sub-assembly. After considering retrofits against brittle joint and column failure, the focus of the research presented in this section lies on eliminating brittle beam-bar bond-slip failures and relocating the plastic hinge away from the beam-joint interface. Typical retrofit layouts for beam-strengthening in beam-column joint sub-assemblies are depicted in Fig. 3, while key results are shown in Table 3.

Mukherjee and Joshi (2005) aim to prevent brittle beam-bar anchorage failures in 1/3-scale interior joints using L-shaped FRP sheets between beams and columns (Fig. 3 (a)), or a pre-cured CFRP strip along the top and bottom of the beams (Fig. 3 (b)). For scheme (a), the type of FRP used (CFRP and GFRP) and amount of FRP (one or two layers) are varied. The three retrofit variations successfully prevent pull-out of the beam rebars. For ductile specimens with adequate bar anchorage, the CFRP-sheet retrofit leads to higher strength (+58% vs +53%), but lower ductility than GFRP. The strip retrofit (b) proves however most effective (+113%). For non-ductile specimens, the GFRP retrofitted specimen achieves a larger strength increase (+99% vs +79%), while the strip retrofit (b) proves less effective (+68%) than the L-shape retrofit (a). While the obtained strength increase is significant, it is important to re-iterate the 1/3-scale of the specimens, which may increase the retrofit effectiveness.

Ghobarah and El-Amoury (2005) propose to increase resistance to bond-slip in full-scale exterior joints using CFRP sheets along the bottom of the beam, continued in an L-shape along the interior face of the bottom column of two exterior joints (Fig. 3 (a)). Using two FRP layers and a simple steel angle with expansion anchors for anchorage, the behaviour does not improve, due to debonding and fracturing of the sheets. A reduction in strength (-11%) and the same ductility are observed. With four CFRP layers and an improved anchorage system using a curved steel plate on a concrete haunch and four external steel ties around the outside of the column, a ductile hinging mechanism is achieved, however very close to the joint interface. A large strength increase (+40%) and a nearly doubled ductility are the consequence of the improved anchorage, highlighting how important it is to be considered.

Attari et al. (2010) worked on improving the behaviour of 1/3-scaled deficient interior beam-column joints by addition of horizontal strips of FRP on the top and bottom of the beams (Fig. 3 (b)).

In a first retrofitted specimen, a strip of CFRP laminate is applied, while in a second repaired specimen, next to the CFRP, additionally two layers of GFRP L-shapes are applied at the four beam-column corners and anchored using GFRP wraps on the column (Fig. 3 (a)). For the first retrofit, initially an improved behaviour is obtained with an increased peak load of +24%. However, with increased cycling, pull-out of the FRP strip is observed, leading to abrupt reduction in resistance and sever pinching, ultimately causing failure of the specimen. For the specimen with combined CFRP plates and GFRP L-shapes, a larger increase in strength (+44%) is obtained attributed to the prevention of debonding due to additional anchorage with column GFRP wraps.

Russo and Pauletta (2012) combine a CFRP retrofit with steel-plate anchorage of plain reinforcing bars with inadequate bar anchorage length. The retrofit of exterior joints successfully prevents bond-slip, which is however mainly attributed to the steel-plate anchorage. Strength increase in 2/3-scaled specimens is more pronounced for smaller bar diameters (266% for 12 mm vs 136% for 16 mm), for which bond-slip is more critical. For full-scale joints, the strength increase is significantly lower (49%).

Choudhury et al. (2013) developed two retrofits for exterior joints with bond-slip- or shear-deficient beams, respectively. Unidirectional CFRP sheets are used at the bottom of the beam to improve bar anchorage, while bidirectional GFRP wraps around the beam aim to improve its shear capacity (Fig. 3 (b)). Bidirectional GFRP U-wraps are also used to strengthen the joint in shear. Investigating the effect of specimen scale (full-, 2/3 and 1/3-scale) for both retrofits, the gain in load capacity is found to decrease with specimen size (from 27% to 12% for the shear- and from 24% to 5% for the bond-slip retrofit). Although the findings are based on a specific layout, they still mandate caution when inferring results from scaled experiments.

Strengthening of beams near the joint to relocate the plastic hinge (PH) further along the beam is tested by Mahini and Ronagh (2007, 2011) on 1:2.2-scaled exterior joints. Bonding three layers of CFRP-sheet to the beam-web (Fig. 3 (c)), parallel to the beam-axis, anchored with CFRP along the length of the column, which also serves to improve its flexural behaviour, leads to successful PH relocation 150 mm away from the joint face.

This scheme would however not be applicable with slab and transverse beams and an improved retrofit without FRP in the joint panel and thin FRP-wraps that fit through slots drilled in a slab, is proposed (Eslami et al. 2013; Eslami and Ronagh 2014). As shown in Fig. 3 (b), to achieve PH relocation, CFRP sheets are extended along the beam top and bottom faces for 1) a short length (175 mm - one beam depth) or 2) a longer length (350 mm - two beam depths). Anchorage is provided by embedding the FRP into a 25 mm deep groove in the cover of the joint. For one test without this embedment, no improvement in the specimen's behaviour is observed. The shorter beam FRP length (1) successfully relocates PH with a strength increase of 31%. With a longer FRP length, instead, flexural failure is observed at the joint interface, attributed to the anchorage length (groove depth) not being sufficient for the length of FRP. While the strength increase is larger (45%), ductility is found to be reduced (-36%).

For beam strengthening in deficient beam-column joint sub-assemblies, two main strengthening aims can be discerned. The first one is improvement of beam-bar anchorage to prevent bond-slip failures, the second is plastic hinge relocation away from the joint. Both issues have been addressed by means of applying L-shapes along the corner of beam and column, as well as using strips or plates of FRP on bottom and/or top faces of the beams, often anchored into groves at the joint. Representative results for the strength increase obtained by the retrofit scheme layouts are summarised in Table 3.

Two issues are found to be crucial in the behaviour of these retrofitted joints. Only when adequate anchorage is provided, both types of retrofit are shown to be effective. L-shape retrofits, steel anchors or FRP wraps around the column are shown to be successful. For the simple strips or plates retrofit, embedment into 25 mm deep groves has shown good results for the retrofit scheme suggested by Eslami and Ronagh (2014). The same depth of groove provides insufficient anchorage in the scheme proposed by Attari et al. (2010). The required depth of embedment is indeed not a constant and is dependent on the length of the FRP strip (Eslami et al. 2013).

A second critical element is the effect of scale on the retrofit effectiveness. Choudhury's work showed that the efficiency of beam retrofit schemes in joint sub-assemblies heavily depends on the scale, with scaled specimens achieving larger improvements in performance. This is also confirmed by the results from Russo and Pauletta (2012), who obtain lower strength increase for full-scale specimens or specimens with larger bar diameters.

Overall all proposed retrofits are potentially applicable in real buildings. Apart from the initial retrofit by Mahini and Ronagh (2007), which would be restrained by the presence of transverse beam and slab, the other proposed retrofits do not present any practical application issues. The strip retrofit proves more effective in the case of improving ductility of the beams, while the L-shapes are more effective in addressing beam bar anchorage issues. A combination of the two with adequate anchorage and embedment (Attari et al 2010), would hence appear to be the most logical solution.

***Multi-objective retrofits of joints***

One major advantage of FRP retrofitting is the possibility of a selective upgrading, e.g. of joint shear capacity, as well as bond-improvement in bottom beam bars and confinement or flexural strengthening of the column. A number of multi-objective retrofit schemes for beam-column joints are identified in the literature and are summarised in Fig. 4, which shows schematics of typical retrofit schemes, and Table 4, highlighting key results. A two-part GFRP retrofit for exterior joints (El-Armoury and Ghobarah 2002) sets the basis for the *fib* 35 (2006) guidelines. For joint shear strengthening, U-wraps anchored with steel plates connected by steel rods are applied. Four unidirectional FRP L-shapes along the beam bottom face and inside face of the bottom column, secured using bolted steel angles, are used for rebar bond-slip enhancement (retrofit TR1). A variation of this scheme with 8 layers and two U-shaped steel anchors at the beam ends is also tested (TR2, as shown in Fig. 4 (a)). The latter prevents debonding and hence bond-slip observed for TR1, leading to much higher values of drift and an increase in load of 52% (compared to 40%), but ultimately leads to joint shear failure.

Interior joints (Al-Salloum and Almusallam 2007) and exterior joints (Alsayed et al. 2010) with a 600 mm wide slab, but no transverse beams, are retrofitted with CFRP, calculated in accordance to El-Amoury and Ghobarah (2002). Two different retrofit layouts are tested: (a) applying horizontal FRP on both faces of the joint (U-wrap for the external joint), wrapping the column above and below the joint, and finally U-wrapping the beam(s) (Fig. 4 (a)); (b) strengthening the joint only, but with steel bolts as anchorage. Both CFRP retrofit schemes delay joint shear cracking, however for the specimens without mechanical anchorage, debonding and rupture of the FRP in the beam is observed. With layout (b), the failure mode is shifted from joint shear failure, to beam hinging and hence a larger ductility. The lateral load capacities for retrofit (a) are however higher (+32% compared to +27%), as the joint and beams are also strengthened, but retrofit (b) can be seen as more efficient using less material for a similar strength gain. Finally, as mentioned for other schemes, one constraint ignored by both retrofit designs is the presence of transverse beams, which would render them difficult or impossible to apply.

Pantelides *et al.* (2008) tested cruciform interior specimen without joint shear reinforcement and insufficient bottom beam bar anchorage. A CFRP retrofit developed to address both deficiencies, consists of sheets along the bottom of the beam, U- wraps for anchorage and shear strengthening of the beams and two CFRP layers at an angle of 60° in the joint region, in the style of X-shaped retrofits in Fig.1, as well as column confinement. The retrofit amounts can be adjusted depending on the relative strengths of the members, hence joints with stronger beams, assumed to fail in joint shear, and joints with stronger columns, predicted to fail in beam bar anchorage are tested. Ductile behaviour is achieved for all retrofitted specimens, but greater strength improvements are obtained for joints with joint shear deficiency (+55% vs +50%).

A capacity design CFRP retrofit promoting beam hinging failure in pre-1970's corner joints with slabs under bi-directional loading is proposed by Engindeniz et al. (2008a). Extensive yielding of column bars, joint shear cracking, and beam rebar bond-slip are to be prevented by a two-step retrofit, first adding column rebars and then applying CFRP wrapping to columns, L-shaped CFRP around the joint and U-shaped CFRP around the beams, as shown in Fig. 4 (b), in amounts designed to ACI-440 guidelines (2002b). An extra layer of FRP is applied to the joint for a specimen with half the concrete strength. The proposed multi-objective retrofit is successful in preventing joint shear failure, achieving a shear capacity compliant with modern codes. The specimen with stronger concrete achieves a strong increase in strength (+36%) with a ductile beam hinging failure characterised by high energy dissipation. The FRP ruptures at 94% of its ultimate strain rather than debonding, indicating that the anchorage consisting of full FRP wrapping only is sufficient. The specimen with lower concrete grade, due to reduced bond with the beam bars, experiences a reduction in strength (-5%), significant reduction in stiffness (44% lower) and strongly reduced energy dissipation (37% lower), indicating a more severe and impractical retrofit, including concrete replacement may be needed for very weak concrete.

A performance-based FRP retrofit approach to provide a ductile failure is proposed by Akguzel et al. (2011; 2009). An initial test of the principle on two-dimensional interior and exterior joints without slab uses a simple system of vertical and horizontal CFRP sheets (Pampanin et al. 2004). The retrofit successfully alters the hierarchy of strength, but debonding reduces its effectiveness, as no anchorage is provided.

A similar retrofit using GFRP is used on three-dimensional corner joint without slabs under bidirectional loading (Akguzel and Pampanin 2007). With bidirectional loading, increased joint damage and faster strength degradation is observed for the control specimen. The GFRP retrofit protects the joint from excessive damage but is less effective than in an equivalent 2D specimen (+61% vs +75% strength increase). Complete relocation of damage from the joint to the end of the beam GFRP sheet is observed for both geometries, resulting in a more ductile and dissipative mechanism.

In additional tests (Akguzel and Pampanin 2010), the effect of the amount of horizontal FRP (1 or 2 layers) in the beam and joint, having one stirrup in the joint, and the effect of bidirectional or unidirectional lateral loading are investigated. A reduced shear strain in the joint at maximum drift from 0.76%, with one layer, to 0.25%, with two layers of horizontal FRP, is observed, which is in line with earlier results (Antonopoulos and Triantafillou 2003). In turn, the joint shear stud improves the retrofit effectiveness, maintaining the integrity of the core and preventing buckling of column bars. There is 10% additional stiffness degradation per drift level for 3D compared to 2D specimens.

An effort combining the GFRP retrofit, shown in Fig. 4 (c), with selective weakening of slabs is tested on a corner joint specimen with slab and transverse beam and compared to an equivalent specimen without slab (Akguzel and Pampanin 2012b). Selective weakening consists of reducing the contribution of the slab to the hogging capacity and stiffness of the beams, by cutting rebars around the beam perimeter near the joint. This intervention aims to promote a beam hinging mechanism, while protecting the assembly from brittle joint or column failure with GFRP sheets, anchored with fan-shaped FRP dowels through the slab. An increase in strength of 28%, slightly lower than for the specimen without slab (+34%), is obtained. Despite the weakening cuts, beam hinging is however not achieved for the specimen with slab. Detachment of column FRP causes ultimate failure at the column/joint interface, while the retrofit is effective in transferring damage to the beams for the specimen without slab. Anchorage of the joint and beam FRP with dowels was however effective in preventing debonding.

The idea of controlling the hierarchy of strength between columns and beams prior to retrofitting using selective-weakening is also adopted by Pohoryles et al. (2018) for retrofitting interior beam-column joints with slab and transverse beams (see Fig. 4 (d)). The retrofit uses rolled strands of CFRP sheet at the corners of the columns for continuous flexural strengthening, similar to Shiohara et al. (2009), but tested under more realistic loading conditions. Joint shear strengthening is applied by means of two CFRP strands applied through holes drilled through the transverse beams, which presents an effort to tackle joint shear strengthening in the presence of transverse beams realistically. Furthermore, next to selective weakening of the slab, continuous beam strengthening with fan-shaped FRP is applied to the beams to achieve plastic hinge relocation away from the beam/joint interface. A combination of FRP anchor fans and metallic anchors is used to prevent any potential debonding mechanisms. Overall, a behaviour close to a specimen designed to Eurocode 8 (CEN 2004) is achieved, with transfer of damage to the expected plastic hinge zone and a significant strength (+38%) and ductility increase (+90%). The same retrofit applied to a 2D cruciform specimen, however achieves an even stronger strength increase (+50%), but reduced ductility (-46%), again highlighting the importance of testing realistic specimen geometries (Pohoryles 2017).

In this section, retrofits with multiple strengthening objectives were discussed. The aim of these retrofits follows the principle of capacity design. To protect the joint, in most analysed retrofits, horizontal FRP, placed as U-shapes for exterior joints or L-shapes for corner joints is used. Pohoryles et al. (2018) use horizontal FRP strands passed through holes drilled into the transverse beams, hence explicitly considering practical considerations. Only Pantelides et al. (2008) used the X-shaped configuration. Column confinement and flexural strengthening are then applied, with vertical sheets of FRP applied on the column faces being the most common solution. In the presence of slabs or transverse beams, this is however highly impractical. Pampanin et al. (2012) suggest the use of L-shaped FRP, while Pohoryles et al. (2018) use vertical FRP strands passed through pre-drilled holes. Finally beam bar anchorage or plastic hinge relocation in the beams is the last objective in order to ensure a ductile hinging mechanism. For this generally sheets or strips of FRP placed along the beam or L-shapes are used. The proposed schemes all highlight the potential of FRP retrofits to achieve multiple objectives, with increases in lateral capacity and change in failure mechanism observed. The study by Engindeniz et al. (2008) highlights the higher effect of retrofits for larger concrete strengths, while Pampanin et al. (2012) and Pohoryles et al. (2018) show that 2D specimens tested without slab display a larger retrofit effectiveness. Overall, the two latter retrofit schemes can be deemed most realistic for practical applications, as geometric constraints of slab and transverse beams are explicitly addressed, and the retrofit schemes are shown to be effective in eliminating brittle failure, while ensuring large increases in strength.

## Analysis and Discussion

The state-of-the-art review presented in this paper highlights the vast number of experimental efforts conducted in the area of seismic strengthening using FRP. With this comes a wide variety of retrofit configurations, test set-ups and specimen geometries that may all affect the retrofit effectiveness. To understand the existing data better and assess any gaps in the literature in terms of experimental data, the statistics on the configuration of the 260 tested specimens in the database are shown in Table 5. The variety of FRP configurations proposed in the literature are in direct relation to the type of pre-1970's structural design deficiencies identified for beam-column joints. The statistics related to the studied deficiencies are presented in Fig. 5. Note that an overlap of two or more deficiencies is common in the presented studies and the percentage values are hence not in direct relation to the number of specimens tested.

It can be observed that most specimens (37%) are joint shear deficient (JTR) and retrofit layouts often only focus on this deficiency. With this in mind, it is important to indicate a heavy bias in specimen geometry in Table 5, with 82% of tested specimens presenting no transverse beams or slab, hence making the joint accessible for this type of shear strengthening. This joint geometry may however be seen as overly simplified and hides potential practical issues with the proposed retrofit solutions.

### *Assessment of joint shear strengthening equations*

As most papers in this review focus on joint shear strengthening, it is attempted to compare the experimental results to design equations from three FRP strengthening guidelines: Eurocode 8 – Part 3 (CEN 2006), CNR-DT 200.R1 (CNR 2013) and ACI-400.2R08 (ACI 2008). For the purpose of this analysis, shear retrofitted specimens from the database that failed in joint shear failure are selected. This results in a total of 41 specimens summarised in Table 6. Of the chosen specimens, most are external joints (74%), but also internal (16%) and corner joints (10%) are included. Nearly half of the specimens used for analysis are full-scale (46%), with the rest mainly being 2/3 scale (44%). Finally, the type of retrofits is also diverse, with most specimens used for this analysis retrofitted using U-shaped FRP (54%), and X-shaped and L-shaped (both 17%) also being commonly used. Strips are only used in the three specimens tested by Antonopoulos and Triantafillou. The selected specimens are hence diverse in geometry, size and retrofit layout. This gives this study a wider outlook on the issue of joint shear strengthening equations, as it allows testing the equations on a larger dataset compared to previous efforts (Bousselham, 2010).

First, to determine the experimental shear capacities of joints, the published values of ultimate force acting on beam or column (depending on the experimental set-up) are used. The horizontal shear acting at the centre of the joint, $V_{jh}$, is evaluated by equilibrium considerations. More details on this can be found elsewhere (Pohoryles and Rossetto 2014). The FRP contribution to the joint shear resistance of the tested joints is then calculated by deducting the capacity of the respective control specimen. Specimens for which a reduction in strength (e.g. heavily pre-damaged repair specimens) are excluded from this assessment.

The increase in capacity due to FRP is strengthening is then calculated for all 41 specimens for the three guidelines. In Eurocode 8-Part 3 (EC8), the contribution of FRP to the shear capacity can be calculated according to equations A.22 and A.23. It is suggested that FRP is placed with fibres in the direction of hoops (cl. 4.4.1(1)). In the Italian CNR DT-200.R1 (CNR) guidelines, for the shear strengthening of RC joints, clause 4.7.2.1.4 (1) applies, which states that FRP fibres should be placed in the direction of principal tensile stresses, rather than the hoop direction, as suggested by EC8 and ACI. Finally, for ACI 440.2R-08 (ACI) equations from section 11.4 for shear strengthening of rectangular members are taken. Equation 11-3 allows calculation of the contribution of FRP to the shear force, which is however limited by the maximum given by equation 11-11.

The equations for the strengthened joint shear capacity ($V_f$) in the three guidelines are summarised in Table 7. Note that the symbols and equations are slightly adapted in order to ease comparison. The factor $\beta$ corresponds to the angle of the fibres to the axis of the column, while $\theta$ is the angle of the principal stress in the joint (assumed to be the angle of the diagonal) to the axis of the column. In the case of strips used for retrofitting, $w_f$ is the width of FRP strip, measured in the orthogonal direction to the fibres, and $s_f$ the spacing between the strips of FRP (equal to $w_f$ for sheets). The ratio of $w_f$ to $s_f$ simplifies to 1 in the case of continuous shear strengthening is applied. In terms of the retrofit dimensions, $d_{fv}$ is the breadth of applied sheet and $t_f$ is its total thickness. The geometry of the joint is represented by $d$, the effective depth and $b_j$ the width of the joint. The partial reduction factor $\gamma_{Rd}$ in the CNR equation is ignored for the purpose of this comparison.

As it can be observed, the equations are similar in nature, albeit different in their details. In particular, the EC8 and CNR guidelines include the angle of principle stress, which is not foreseen in the ACI guidelines. Between the CNR and EC8 guidelines, the EC8 equation includes an additional sin $\beta$ factor, as well as a separate equation for FRP strengthening that is not applied in a U-shape, but only side bonded. The main differences between the guidelines lie however in the calculation in the effective strength, $f_{fe}$ of the FRP, in turn related to the effective strain developed in the material.

The ACI equations calculate effective strain based on the geometry of the retrofit, i.e. for fully wrapped members clause 11-6a is used, which limits the FRP strain to a maximum of 75% of the rupture strain, while for U-wraps or side-bonded FRP, a bond reduction factor $\kappa_v$, is first calculated. This factor takes into account the compressive strength of concrete, $f_c$, the active bond length of FRP, as well as the material stiffness and thickness of the FRP strengthening.

In EC8, in turn, the effective strain is calculated based on cl.A.4.4.2(5) for fully wrapped or properly anchored jackets, cl.A.4.4.2(6) for U-wraps, and cl.A.4.4.2(7) for side-bonded sheets and strips. The effectiveness of anchorage is hence explicitly addressed in the guideline. Other factors in the equations affecting effective strain include concrete tensile strength, FRP ultimate tensile strength, the corner radius of the element, as well as an effective bond length. In the CNR guidelines, cl. 4.3.3.2 used to calculate the effective strain is similar to the EC8 equation and also dependent on FRP shape, material strength, bond length and corner radius. Anchorage is explicitly addressed in clause 4.7.2.1.4 (1)). Finally, it is stated that the maximum tensile strain of FRP wraps in RC joint strengthening shall not exceed 4‰, which is in line with ACI.

The results of the analysis of the joint shear strengthening guideline equations are displayed in Table 6 for each specimen and summarised in Fig. 6. Looking at the results, it can be observed that all three guidelines achieve a relatively accurate prediction of the strengthened capacity. The CNR guidelines are found to be slightly conservative leading to an average ratio of 0.82 of the predicted to experimental results. The equation also delivers the lowest variance (0.52). The EC8 equation gives the best results on average (ratio of 1.08), albeit slightly non-conservative. The coefficient of variance is the highest for the EC8 equation (0.85), indicating a larger spread of predicted capacities. The ACI guidelines, are found to overestimate the contribution of the FRP by 53%, with a relatively high variance (0.78).

Looking at the equations in the guidelines, it would appear that a main reason for the CNR and EC8 guidelines performing better can be found in the use of the angle of fibres and the angle of principal stress, which allows for a more accurate calculation of the contribution of the fibres in tension. Ignoring this, leads to an overestimation of the FRP contribution as seen for the ACI equations. Furthermore, evaluating the bond strength of FRP to concrete, by explicitly taking into account anchorage in EC8, permits a relatively accurate estimation of the retrofitted joint shear capacity without overestimating the contribution of FRP.

***Factors affecting FRP retrofits***

The evaluation of design guidelines highlights the need for further improvements in order to enable a wider applicability of full FRP retrofits of RC structures. To improve guidelines and design better retrofits, it is important to understand which parameters affect the retrofits the most. As shown in this review, next to joint shear strengthening, more elaborate retrofit layouts dealing with additional deficiencies are also found in the literature. Looking at in Fig. 5., inadequate beam bar anchorage (BA) or a weak-column/strong-beam mechanisms (WC/SB) are examples of these (12% and 11% of tested specimens, respectively). To fully capture the effectiveness of these types of retrofit and their practical implementation, more complex (and realistic) sub-assemblies geometries may be required, as the presence of transverse beams or slabs affects the accessibility of the areas to be retrofitted, as well as the relative hierarchy of strengths.

The full database is further analysed to assess the influence of scale and geometry, as well as of pre-damage, on the increase in strength and ductility of retrofitted specimens (Table 8). The analysis confirms general observations from the review, namely that specimens without slab and transverse beams (2D) present a higher effectiveness of FRP retrofit in terms of average strength increase (+44% compared to +27%). Also, in terms of ductility, 2D specimens achieve a much larger improvement (+63%) then 3D specimens (+38%). The effect of scaled specimens is less pronounced for strength increase (+42% compared to +39% in full-scale) and reversed for ductility (+55% compared to +63% in full-scale specimens). Finally, repairs of pre-damaged specimens are not, as one would expect, less effective in increasing the load capacity than retrofits of existing structures (+41% compared to +39%). This may be attributed to the replacement of damaged concrete by stronger mortar. As a decrease in initial stiffness for repaired specimens is commonly observed, in terms of ductility, repaired specimens present however a lower increase (+36% compared to +67%).

The geometry, scale and pre-damage of test specimens are hence crucial aspects affecting retrofit effectiveness. Based on the wealth of experimental knowledge developed by the scientific community, the authors of this review determined other factors that have an important effect on the effectiveness of the FRP retrofit, defined in terms of increase in lateral load capacity or ductility. These factors are summarised and discussed concisely in this section.

- **FRP material**: CFRP having higher strength, but lower ultimate strain than GFRP or BFRP, the choice of FRP material often falls to the first, with 80% of all specimens retrofitted with CFRP and the rest mostly with GFRP (18%) and limited studies on BFRP (<3%). Due to the properties of the material, higher strength increases are observed for the stronger CFRP, but lower ductility (Mukherjee and Joshi 2005). As a result of higher ultimate strain in GFRP, using more layers of GFRP can achieve a higher increase in ductility (Antonopoulos and Triantafillou 2003). Similar effects are observed for BFRP (Yu et al. 2016). A variety of factors, including cost, anchorage solution and retrofit layout may however also favour one material over the other.

- **Amount of FRP**: A variety of retrofits solutions are tested in the literature, with number of layers varying from one up to ten layers (Tsonos 2008). The amount of FRP layers used affects the retrofit effectiveness, the increase in strength is however not proportional to the increase in FRP layers (e.g.: Antonopoulos and Triantafillou 2003).

- **Angle of fibres**: For shear strengthening of joints or beams, in particular, a more effective retrofit is achieved by orientating the fibres in the angle of principal stress, compared to the use of horizontal and vertical sheets. For joints, placing sheets diagonally is however often difficult or impossible due to the presence of slab or transverse beams. Using horizontal sheets with bi-directional fibres at 45° is an option, however (currently) leads to more expensive solutions.

- **Strips vs sheets**: In terms of lateral load capacity, sheets seem more effective than strips. This is however based on limited evidence and may be mainly attributed to weaker bond characteristics of the latter (Antonopoulos and Triantafillou 2003).

- **Anchorage:** In all major design guidelines FRP debonding is recognised as a brittle failure mechanism, which leads to an abrupt loss of capacity in retrofitted members. Adequate FRP development lengths and reduced levels of FRP design strain are hence generally recommended to prevent it. In the literature, premature failure through FRP delamination is observed when the FRP sheets are not anchored adequately, which can cause the retrofit to be ineffective (e.g.: Ghobarah and Said 2002). Lack of anchorage affects retrofits with FRP strips more than when using sheets of FRP (Antonopoulos and Triantafillou 2003). Using transversal sheets of FRP to wrap the main strengthening has shown little effect on its own (Akguzel and Pampanin 2010). Other anchorage solutions are hence recommended:

  - FRP dowels or fan-shaped anchors through slabs or beams (Akguzel et al. 2011; 2012a).
  - Steel plates and rods (Al-Salloum and Almusallam 2007; Ghobarah and Said 2002).
  - Steel angles connected between beams and column (El-Amoury and Ghobarah 2002; Lee et al. 2010).
  - U-shaped steel anchors (El-Amoury and Ghobarah 2002).

  Fig. 7 displays the percentage of retrofits in the literature providing anchorage and the type of anchorage provided. Anchorage using strips or sheets of FRP are labelled as ‘FRP’, while FRP dowel or fan-shaped anchors are labelled ‘FRP-anchors’. Steel anchors using plates, rods or bolts are simply labelled ‘steel’. It can be seen that a majority of studies (65%) recognises the need to provide anchorage. This anchorage is mainly provided by FRP U-wrapping or full-wrapping (‘FRP’), while only 18% of studies employ steel devices or FRP fan/dowel anchors. In a review of anchorage systems metallic anchors are shown to be the most effective method (Kalfat et al. 2013), with FRP-based anchors however being less invasive, more practical to apply and often providing a similar level of anchorage. FRP fan-shaped anchors may enhance the effective strain in FRP sheets to 75% of the rupture strain

(Bournas et al. 2015), which is significantly larger than typical design strain values in guidelines.

- **Continuous strengthening**: For effective flexural strengthening of columns, an adequate bond length is required, which needs continuity through the joint. For cruciform specimen, this is addressed by simply applying FRP sheets or NSM rods continuously from the superior to the inferior column (Antonopoulos and Triantafillou 2003; Mahini and Ronagh 2011). With a slab or transverse beams, this would however not be achievable for all sides of the column (Prota et al. 2004; Shiohara et al. 2009) without the need of cutting or drilling through adjacent members. To date, only Pohoryles et al. (2018) addressed this issue explicitly. Similarly, the continuity and anchorage through the joints for longitudinal FRP sheets in the beams is only addressed by anchorage with U-wraps (Akguzel et al. 2011; Al-Salloum and Almusallam 2007) or by bending the sheets in an L-shape onto the columns (Ghobarah and El-Amoury 2005).

- **Pre-damage**: Generally, the extent of pre-damage affects the effectiveness of FRP repairs. While moderately pre-damaged specimens can be repaired to achieve similar performance to retrofitted counter-parts, for severely pre-damaged joints, the strength may not be recovered (Agarwal et al. 2014; Beydokhti and Shariatmadar 2016). The analysis of the database highlights that on average for the many repaired specimens in the database, however, the effectiveness in terms of strength increase resembles that of retrofits closely (+39% vs +41%). Only in terms of ductility a large difference is found due to the reduced stiffness of pre-damaged specimens.

- **Concrete strength**: The distribution of concrete strengths of the experimental specimens presented in the literature is shown in Fig. 8. The majority of specimens (65%) have a concrete with a mean strength ($f_{cm}$) of 20-30 MPa. While most studies aim to replicate pre-1970s structures which often have lower concrete strengths, these are difficult to produce nowadays with modern materials. In limited studies with very weak concrete, it is observed that FRP retrofits may not be viable without partial replacement of existing concrete (D'Ayala et al. 2003; Engindeniz et al. 2008a).

- **Axial load**: The effectiveness of FRP retrofits is generally found to be strongly affected by the level of axial load. Higher levels of axial load lead to a stronger enhancement of ductility and strength for retrofitted specimens (Antonopoulos and Triantafillou 2003; Prota et al. 2004). Using an axial load varying with lateral load, however, leads to an unfavourable effect on the retrofit (Akguzel and Pampanin 2010, 2012b). In most experiments reported in the literature, one constant value of normalised axial load is used, a majority of which using values from 0.1 to 0.2, as shown in Fig. 9. This value can be deemed typical for lower storey columns. It is worth noting that no axial load is applied in nearly 15% of studies.

- **Effect of scale**: The effectiveness of retrofits is seen to be reduced for full-scale specimens compared to smaller specimens (e.g.: Choudhury et al. 2013). Non-conservative results may hence be reported when basing outcomes on experiments on scaled specimens. Looking at the statistics on specimens tested in the literature (Table 5), this effect of scale may affect a large number of experiments, as 59% of specimens are less than full-scale. Indeed, the analysis of the database has highlighted that scaled specimens have a larger improvement in strength increase, however this effect is less clear then anticipated.

- **3D vs 2D set-up**: Again looking at Table 5, most of the present studies test two-dimensional (2D) specimens without slab or transverse beams (82%). The presence of framing members, but also bi-directional loading, are however seen to reduce the effectiveness of FRP retrofits (Akguzel et al. 2011). Slabs contributing to the moment capacity of the beams can influence the failure mechanism of sub-assemblies (Kam et al. 2010; Park and Mosalam 2013), often promoting unwanted column hinging mechanisms despite retrofit (Akguzel and Pampanin 2012b). In practical terms, the placement of sheets and location of anchorage are also affected by the real geometry of the structure, with slabs and transverse beams typically being present for most moment-resisting frame (MRF) buildings (Genesio et al. 2010; Lehman et al. 2004; Pampanin et al. 2002). Regarding the tested joint geometries, as shown in Table 5, most retrofitted specimens are exterior joints (54%), which generally are more critical, as, unlike interior joints, these are not confined from four sides and are subjected to lower axial forces. Still, there is also a large interest in interior joints, corresponding to 32% of the tested specimens. Looking at limited experimental evidence for full frames (Akguzel et al. 2011;

Gallo et al. 2012), it appears that retrofitting exterior joints only may not be sufficient to improve the global structural behaviour, hence the need for investigating the retrofit of interior joints too.

The analysis of the database together with a discussion of the factors affecting the effectiveness of FRP retrofits highlights the need for assessing the practical engineering aspects of the retrofits. From the reviewed studies, many ignore the presence of practical challenges to the retrofit application. When these factors are included, it becomes clear that full FRP retrofits of structures will need to include additional drilling of holes through slabs or walls in order to enable transversal FRP strengthening or anchorage of longitudinal FRP sheets. An often-cited benefit of FRP strengthening compared to conventional retrofitting (e.g. concrete jacketing) is the reduced labour, reduced time of the intervention and being less invasive. In order to fully confirm this, it is however important to test proposed retrofitting schemes on full-scale three-dimensional set-ups that accurately reflect real structures. Ignoring these effects will lead to an over-estimation of the effect of FRP retrofits, but also lead to easier applications and anchorage of FRP, hence not giving a correct picture on the practical applicability of the scheme.

## Conclusions

A review of the state-of-the-art of FRP retrofitting of beam-column joint subassemblies presents numerous successful implementations that address various retrofit objectives. A detailed analysis of the proposed schemes and a compilation of a database of experiments allows important conclusions to be drawn from this review.

A plethora of successful implementations of joint shear strengthening schemes have been presented. While sheets in the angle of principal stress are shown to be most effective, horizontal strengthening with FRP sheets is deemed most realistic. To address other design deficiencies of pre-1970's RC frames, such as the low flexural capacity of weak columns, a range of implementations are reported in the literature. At beam-column joints, continuous flexural strengthening of columns through slabs and transverse beams is proposed, with FRP anchors deemed most appropriate, as they can be passed through small holes at the corners of the columns.

Beam plastic hinge relocation (PHR) is another application of FRP that shows potential in improving the seismic behaviour of structures. Studies show that strengthening the beam in the proximity of the joint allows relocation of damage and plastic hinge formation away from the joint. This protects the joint from yield penetration and improves the dissipative behaviour of the specimen further. Anchorage of the FRP sheets at the beam/joint interface is a practical challenge that is not sufficiently addressed.

Any type of anchorage solution is deemed to be critical in ensuring FRP retrofit effectiveness. Most studies consider anchorage; however, looking at the database of experiments, simple FRP wrapping is the most common application, which is not always effective. A combination of FRP anchor fans and metallic anchors is shown to be most successful.

With regards to the assessment of joint shear strengthening equations in design guidelines, it is found that the ACI guidelines overestimate the FRP contribution. While the CNR and EC8 equations give a better fit to experimental data, the variance in the results is still relatively large and hence require improvement. Even though the database encompasses a wide range of joint types and geometries, there is still a need for more data to validate and improve code provisions. As the aim of most retrofit interventions is to avoid brittle failure mechanisms and promote ductile failure, there is a lack of experiments on retrofitted specimens that fail in joint shear. While this demonstrates the effectiveness of joint shear retrofitting with FRP, it reduces the available data for validating or developing design equations. One conclusion from this review is hence the need for experiments with purposefully understrengthened retrofitted specimens to obtain further experimental data on the contribution of FRP to the joint shear strength.

Finally, despite the large number of conducted studies, a strong bias towards scaled cruciform test specimens is observed. The ease of construction and testing, as well as their less complex behaviour, have led to increased testing of these types of joints. This review, however, highlights the important effect of realistic size, loading and geometry of test specimens on FRP retrofit effectiveness. Without these elements, the joint region is more accessible and practical challenges for the schemes, including placement of anchors, are ignored. Future studies should hence consider more realistic test set-ups to explicitly address potential practical issues in the retrofit design.

**Acknowledgments**

The presented research is part of the Challenging RISK project funded by EPSRC (EP/K022377/1).

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## List of Figures

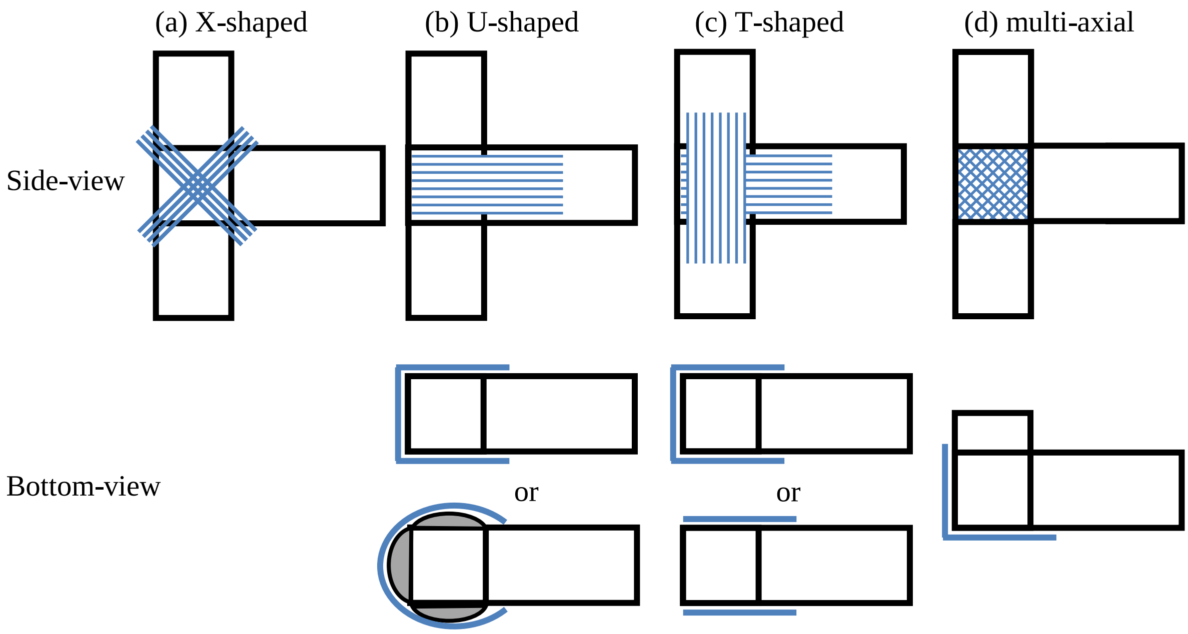


Fig. 1. Schematics of typical joint shear retrofit schemes found in the literature.

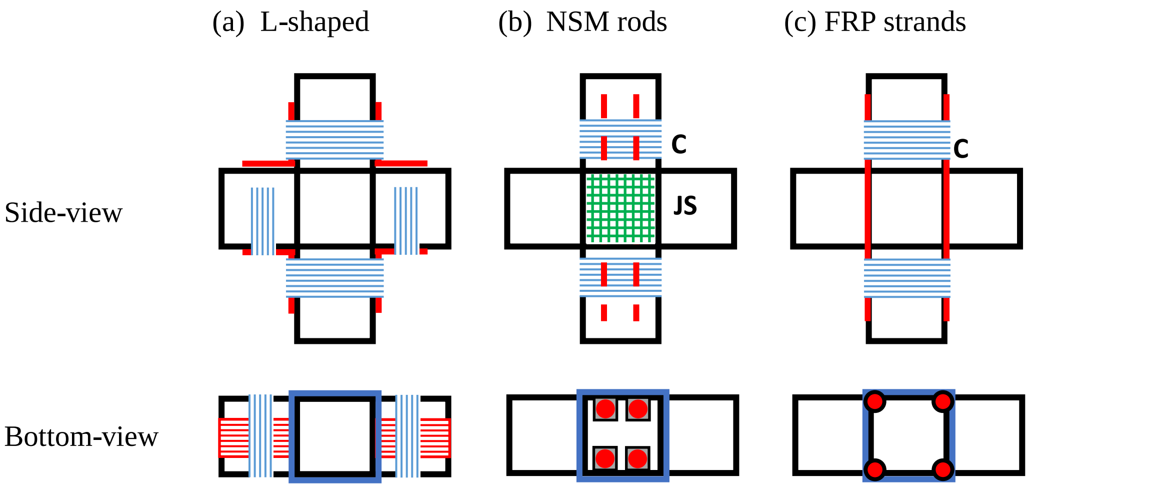


Fig. 2. Schematics of retrofit schemes for columns through beam-column joint sub-assemblies found in the literature (Note: dashed line represents embedded NSM rods; JS = Joint strengthening, C = confinement).

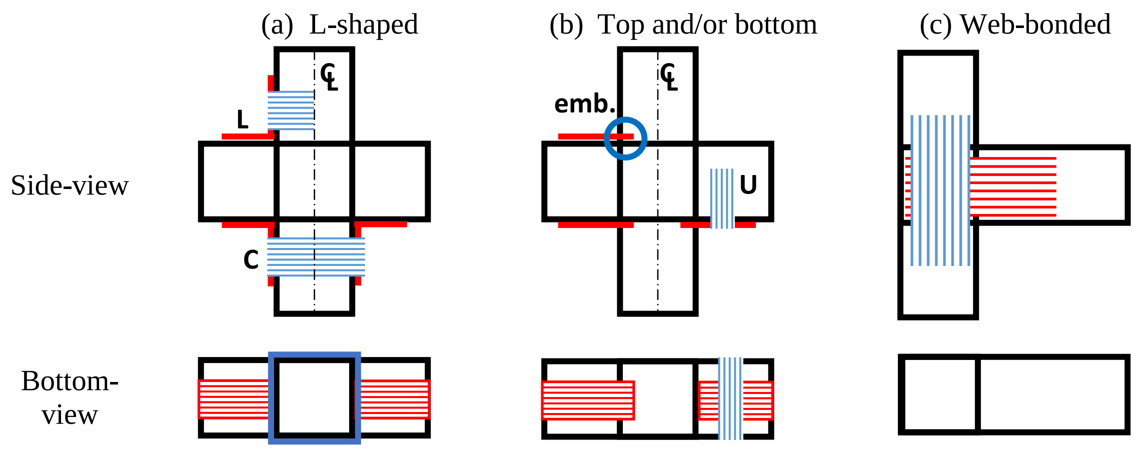


Fig. 3. Schematics of retrofit schemes for beams in beam-column joint sub-assemblies found in the literature. (Note: C = confinement, L = L-shape, U = U-shape, emb. = embedment; CL = centre line).

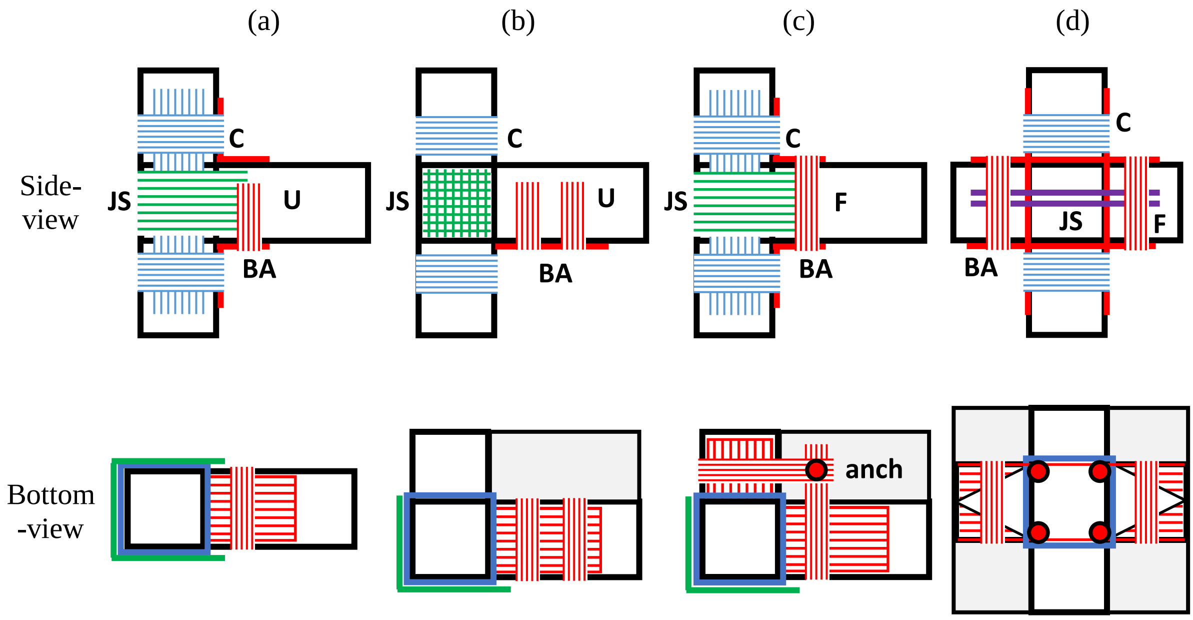


Fig. 4. Schematics of multi-objective retrofit schemes for beam-column joint sub-assemblies found in the literature: (a) El-Amoury and Ghobarah (2002) and Alsayed et al. (2010); (b) Engindeniz et al. (2008); (c) Akguzel and Pampanin (2012) and (d) Pohoryles et al (2018). (Note: JS = Joint strengthening, C = confinement, BA = beam bar anchorage, U = U-shape, F= full wrap, anch = anchor).

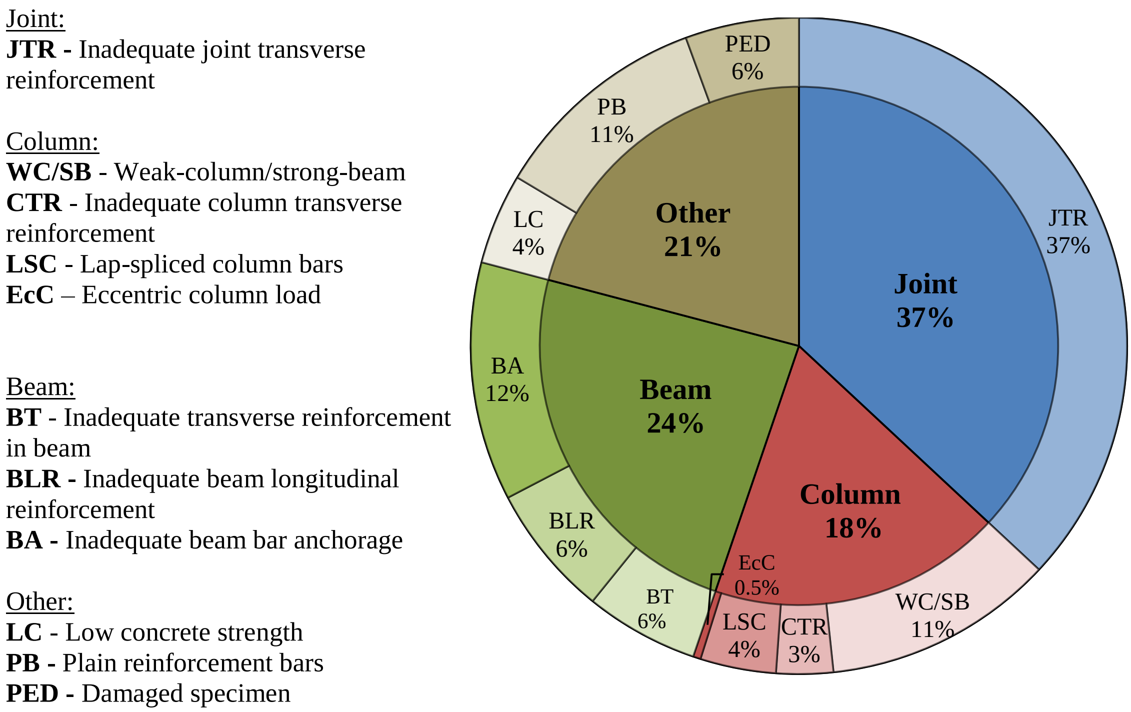


Fig. 5. Type of design deficiency analysed for tested specimens reported in the literature.

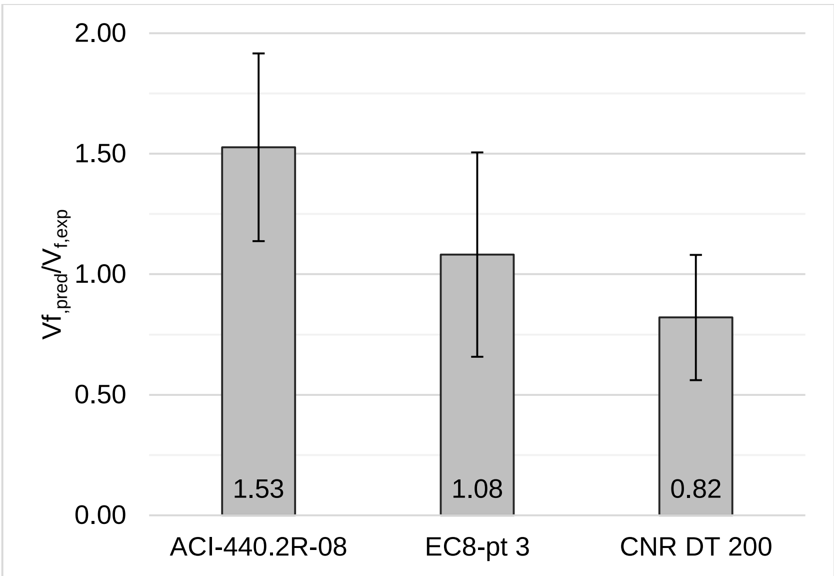


Fig. 6. Average ratio of the predicted ($V_{F,pred}$) to experimental ($V_{F,exp}$) FRP contribution to shear strength of joints for the three design guidelines.

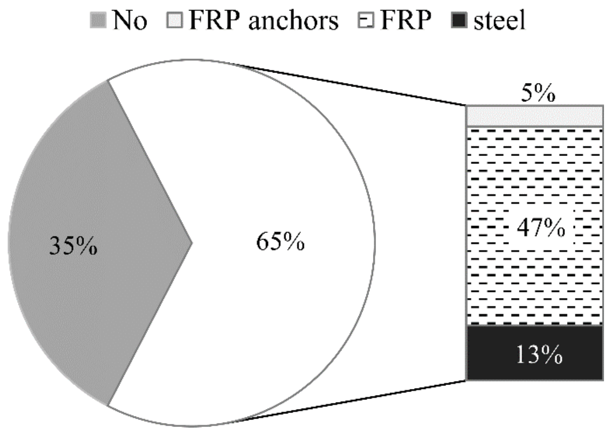


Fig. 7. Statistics of employed anchorage solutions in the experimental database.

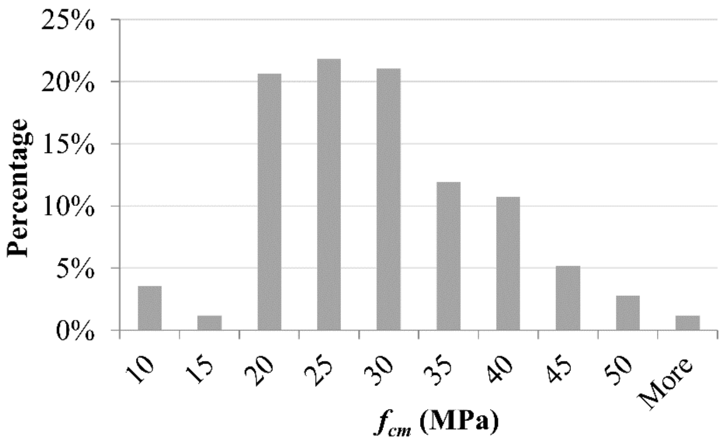


Fig. 8. Statistics of mean concrete strengths ($f_{cm}$) of tested specimens in the experimental database.

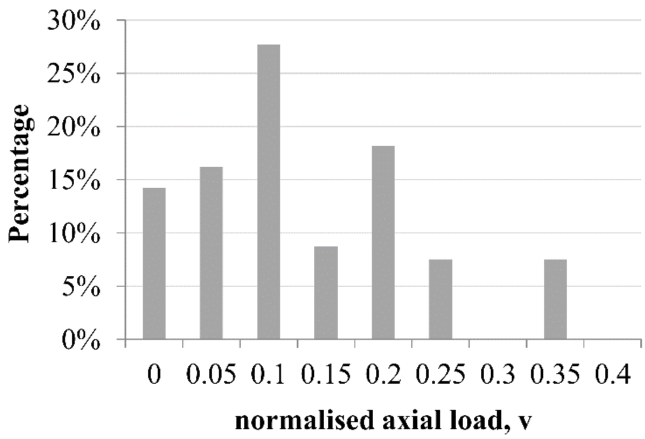


Fig. 9. Statistics of normalised axial load in the experimental database.

Table 1. Representative strength increases obtained due to the joint shear retrofits.

| Author | Main parameter | U-shaped | T-shaped | X-shaped | Multi-axial |
|---|---|---|---|---|---|
| Ghobarah and Said (2002) | | +18% | | +11% | |
| Antonopoulos and Triantafillou (2003) | CFRP<br>GFRP | | +41%<br>+45% | | |
| Karayannis and Sirkelis (2008) | | +88% | | | |
| Le-Trung et al. (2010) | | | +32% | +17% | |
| Realfonzo et al. (2014) | Unanchored<br>Anchored | +23%<br>+99% | | | |
| Hadi and Tran (2014, 2016) | Rounded | +140% | | | |
| Agarwal et al. (2014) | Repaired | -32% | | | |
| Garcia et al. (2012, 2014) | Repaired | +69% | | | |
| Yurdakul and Avsar (2015) | Repaired | | | -25% | |
| Faleschini et al. (2019) | Repaired | -34% | | -30% | |
| Beydokhti and Shariatmadar (2016) | Moderate damage<br>Near-Collapse<br>Collapse | +6%<br>-19%<br>-15% | | | |
| Del Vecchio et al. (2014) | Corner | | | | +49% |
| Ilki et al. (2011) | Corner | | | | +18% |
| D'Ayala et al. (2003) | Retrofit<br>Repair | | +17.3%<br>-5.2% | +92.6%<br>+2% | |
| Lee et al. (2010) | Interior | | +36% | | |

Table 2. Representative strength increases obtained due to the retrofit schemes for columns in beam-column joint sub-assemblies (JS = joint shear strengthening).

| Author | Retrofit | Main parameter | Strength increase |
|---|---|---|---|
| Yu et al (2016) | L-shaped | CFRP | +26% |
| | | BFRP | +11% |
| Prota et al. (2004 | NSM rods | Without JS | +62% |
| | | With JS | +83% |
| Shiohara et al (2009) | FRP strands | | +13% |

Table 3. Representative strength increases obtained due to the retrofit schemes for beams in beam-column joint sub-assemblies found in the literature.

| Author | Main parameter | L-shaped | Top and/or bottom strips | Web-bonded |
|---|---|---|---|---|
| Mukherjee and Joshi (2005) | GFRP | +99% | +68 % | |
| | CFRP | +79% | | |
| Ghobarah and El-Amoury (2005) | Simple anchor | -11% | | |
| | Well anchored | +40% | | |
| Attari et al. (2010) | | +44% | +24% | |
| Choudhury et al. (2013) | Full | | +5% | |
| | 2/3 | | +22% | |
| | 1/3 | | +24% | |
| Mahini and Ronagh (2011) | | | | +9% |
| Eslami and Ronagh (2014) | Short FRP | | +31% | |
| | Long FRP | | +45% | |

Table 4. Representative strength increases obtained using multi-objective retrofit schemes for beam-column joint sub-assemblies.

| Author | Main parameter | Strength increase |
|---|---|---|
| El-Amoury and Ghobarah (2002) | | +52% |
| Alsayed et al. (2010) | | +32% |
| Engindeniz et al. (2008) | Weak concrete | -5% |
| | Strong concrete | +36% |
| Akguzel and Pampanin (2012) | No slab | +34% |
| | With slab | +28% |
| Pohoryles et al (2018) | No slab | +50% |
| | With slab | +38% |

Table 5. Geometrical statistics of joints contained in database.

| | | Joint type | | | Contains | | Scale | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Number | Interior | Exterior | Corner | Slab | Trans. Beam | Full | 2/3 | Half | 1/3 |
| Control | 91 | 32% | 54% | 14% | 18% | 22% | 44% | 15% | 14% | 26% |
| Retrofitted | 125 | 43% | 47% | 10% | 12% | 16% | 37% | 26% | 14% | 22% |
| Repaired | 44 | 34% | 61% | 5% | 32% | 27% | 48% | 5% | 27% | 20% |
| Total | 260 | 37% | 52% | 11% | 18% | 20% | 41% | 19% | 16% | 23% |

Table 6. Experimental versus predicted contributin to shear capacity for strenghtened specimens

| | Experimental | | | | | ACI-440 | | EC8-3 | | CNR DT 200 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Specimen[1] | Geometry[2] | | Scale | FRP[3] | $V_{f,exp}$ | $V_{f,ACI}$ | $\frac{V_{f,ACI}}{V_{f,exp}}$ | $V_{f,EC8}$ | $\frac{V_{f,EC8}}{V_{f,exp}}$ | $V_{f,CNR}$ | $\frac{V_{f,CNR}}{V_{f,exp}}$ |
| Gho(T1) | Ext | 2D | 1 | U | 52.4 | 121.8 | 2.3 | 91.5 | 1.7 | 102.4 | 2.0 |
| Gho(T2) | Ext | 2D | 1 | X | 69.5 | 256.3 | 3.7 | 49.7 | 0.7 | 49.7 | 0.7 |
| Gho(T1R) | Ext | 2D | 1 | U | 233.3 | 137.1 | 0.6 | 135.9 | 0.6 | 126.8 | 0.5 |
| Gho(T4) | Ext | 2D | 2/3 | ST | 17.6 | 59.7 | 3.4 | 14.7 | 0.8 | 6.8 | 0.4 |
| Gho(T9) | Ext | 2D | 2/3 | ST | 34.9 | 85.4 | 2.4 | 25.4 | 0.7 | 6.4 | 0.2 |
| El-(T0) | Ext | 2D | 2/3 | ST | 44.6 | 60.2 | 1.3 | 65.5 | 1.5 | 43.1 | 1.0 |
| El-(TR2) | Ext | 2D | 2/3 | U | 45.0 | 87.5 | 1.9 | 48.5 | 1.1 | 35.1 | 0.8 |
| Ant(C) | Ext | 2D | 2/3 | U | 70.2 | 125.4 | 1.8 | 76.3 | 1.1 | 52.0 | 0.7 |
| Ant(C) | Ext | 2D | 2/3 | U | 74.1 | 111.9 | 1.5 | 76.0 | 1.0 | 51.8 | 0.7 |
| Ant(S33) | Ext | 2D | 2/3 | U | 51.5 | 130.6 | 2.5 | 56.4 | 1.1 | 40.9 | 0.8 |
| Ant(S63) | Ext | 2D | 2/3 | U | 89.6 | 126.8 | 1.4 | 77.2 | 0.9 | 52.7 | 0.6 |
| Ant(S33L) | Ext | 2D | 2/3 | U | 91.0 | 130.0 | 1.4 | 127.9 | 1.4 | 93.9 | 1.0 |
| Ant(F11) | Ext | 2D | 2/3 | U | 42.1 | 110.2 | 2.6 | 65.0 | 1.5 | 44.0 | 1.0 |
| Ant(F22) | Ext | 2D | 2/3 | U | 48.6 | 78.6 | 1.6 | 47.1 | 1.0 | 31.7 | 0.7 |
| Ant(F21) | Ext | 2D | 2/3 | U | 39.6 | 104.8 | 2.6 | 60.5 | 1.5 | 40.9 | 1.0 |
| Ant(F12) | Ext | 3D | 2/3 | L | 34.2 | 37.0 | 1.1 | 23.3 | 0.7 | 15.8 | 0.5 |
| Ant(F22A) | Ext | 3D | 2/3 | L | 18.0 | 37.9 | 2.1 | 16.5 | 0.9 | 14.2 | 0.8 |
| Ant(F22W) | Int | 2D | 1 | X | 443.2 | 590.4 | 1.3 | 135.2 | 0.3 | 141.1 | 0.3 |
| Ant(F22in) | Int | 2D | 1 | X | 344.7 | 590.4 | 1.7 | 167.6 | 0.5 | 183.0 | 0.5 |
| Ant(GL) | Int | 2D | 1 | X | 305.0 | 590.4 | 1.9 | 58.8 | 0.2 | 65.9 | 0.2 |
| Ant(S-C) | Int | 2D | 1 | X | 403.4 | 590.4 | 1.5 | 58.0 | 0.1 | 64.7 | 0.2 |
| Ant(S-F22) | Int | 2D | 1 | X | 452.6 | 590.4 | 1.3 | 69.0 | 0.2 | 80.9 | 0.2 |
| Ant(T-C) | Ext | 2D | 1 | U | 141.3 | 263.0 | 1.9 | 129.6 | 0.9 | 105.5 | 0.7 |
| Ant(T-F33) | Ext | 2D | 1 | U | 300.1 | 293.5 | 1.0 | 170.8 | 0.6 | 132.9 | 0.4 |
| Ant(T-F22S2) | Ext | 2D | 1 | U | 314.2 | 297.7 | 0.9 | 172.3 | 0.5 | 134.6 | 0.4 |
| Pro(L3) | Cor | 3D | 2/3 | L | 39.1 | 41.5 | 1.1 | 146.6 | 3.8 | 89.8 | 2.3 |
| Pro(H2) | Cor | 2D | 2/3 | U | 32.4 | 54.3 | 1.7 | 50.3 | 1.6 | 41.4 | 1.3 |
| Pro(H2U) | Cor | 2D | 2/3 | U | 51.2 | 52.6 | 1.0 | 49.2 | 1.0 | 40.5 | 0.8 |
| Pro(H3) | Cor | 3D | 2/3 | L | 10.8 | 15.9 | 1.5 | 58.2 | 5.4 | 47.9 | 4.4 |
| Pro(H4) | Ext | 2D | 1/2 | U | 75.7 | 94.7 | 1.2 | 51.2 | 0.7 | 39.9 | 0.5 |
| Pro(M3) | Ext | 2D | 1/2 | U | 61.1 | 94.7 | 1.5 | 60.2 | 1.0 | 45.7 | 0.7 |
| Al-(IC1) | Ext | 3D | 1 | L | 28.4 | 9.2 | 0.3 | 31.0 | 1.1 | 31.5 | 1.1 |
| Al-(IR1) | Ext | 3D | 1 | L | 92.2 | 11.0 | 0.1 | 38.5 | 0.4 | 37.6 | 0.4 |
| Al-(IC2) | Ext | 3D | 1 | L | 92.2 | 11.0 | 0.1 | 38.5 | 0.4 | 37.6 | 0.4 |
| Tso(F1) | Ext | 2D | 1/2 | X | 169.3 | 593.3 | 3.5 | 125.1 | 0.7 | 115.8 | 0.7 |
| Kar(A2) | Ext | 2D | 1 | U | 62.0 | 87.4 | 1.4 | 114.7 | 1.8 | 76.5 | 1.2 |
| Pan(24-1) | Ext | 2D | 1 | U | 158.8 | 43.7 | 0.3 | 145.2 | 0.9 | 84.8 | 0.5 |
| Pan(24-2) | Ext | 2D | 1 | U | 165.2 | 43.7 | 0.3 | 145.2 | 0.9 | 84.8 | 0.5 |
| Pan(R24-3) | Ext | 2D | 1 | U | 219.0 | 43.7 | 0.2 | 145.2 | 0.7 | 84.8 | 0.4 |
| Pan(R24-4) | Int | 2D | 1 | S | 326.2 | 269.5 | 0.8 | 453.2 | 1.4 | 354.1 | 1.1 |

[1]Specimen name based on: “First three letters of first author (Specimen)”
[2]Ext = External; Int = Internal; Cor = Corner Joint
[3]Shape of FRP joint retrofit: U=U-shape; L=L-shape; X=X-shape; ST=strip; S=Single sheet

Table 7. Summary of the equations for evaluating joint shear strengthening

| Guideline | Clause | Equation |
|---|---|---|
| Eurocode 8 – Part 3 | A.22 | $V_{f,EC8} = 0.9 \cdot d \cdot f_{fe} \cdot 2 \cdot t_f \cdot \left(\frac{w_f}{s_f}\right)^2 \cdot (\cot\theta + \cot\beta) \cdot \sin\beta$ (U-shaped) |
| | A.23 | $V_{f,EC8} = 0.9 \cdot d \cdot f_{fe} \cdot 2 \cdot t_f \cdot \frac{\sin\beta}{\sin\theta} \cdot \left(\frac{w_f}{s_f}\right)$ (side bonded) |
| CNR DT-200 | 4.19 | $V_{f,CNR} = \frac{1}{\gamma_{Rd}} \cdot 0.9 \cdot d \cdot f_{fe} \cdot 2 \cdot t_f \cdot (\cot\theta + \cot\beta) \cdot \frac{w_f}{s_f}$ |
| ACI 440.2R-08 | 11-3 | $V_{f,ACI} = 2 \cdot t_f \cdot f_{fe} \cdot (\cot\beta + \cot\beta) \cdot d_{fv} \cdot \frac{w_f}{s_f}$<br>with a maximum of (11-11): $V_S + V_f \leq 0.66\sqrt{f_{cm}} \cdot b_j \cdot d$ |

Table 8. Analysis of the literature database in terms of retrofit effectiveness.

| | Geometry | | Scale | | Retrofit type | |
|---|---|---|---|---|---|---|
| | 3D | 2D | Full | Less than full | Repair | Retrofit |
| Average strength increase | 27% | 44% | 39% | 42% | 39% | 41% |
| Average ductility increase | 38% | 63% | 63% | 55% | 36% | 67% |