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## Seismic Retrofit Schemes with FRP for Deficient RC Beam-Column Joints

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1 **Review of experimental research on seismic retrofit schemes with FRP for deficient RC beam-column joints**

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8 **Abstract**

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

30 **Introduction**

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

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

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

51 In the last twenty years, retrofitting RC structures with fibre-reinforced polymer (FRP) are growing in  
52 interest due to the light weight and high strength combined with corrosion resistance of FRP  
53 materials. The application of FRP wraps can be performed rapidly and without disrupting the building  
54 occupancy, which is another major advantage, as it reduces the down-time in businesses and the  
55 need of relocating inhabitants in residential properties (Bousselham 2010). FRP retrofits however also  
56 suffer from disadvantages, such as weak bond to concrete and low fire resistance. To improve the fire  
57 resistance of FRP, however, solutions are being investigated, for instance applying intumescent  
58 coatings (e.g.: Ji et al. 2013). Recently, beam-column joint retrofits using other composite materials  
59 using mortar instead of epoxy to bind the fibres to the concrete substrate have hence also become

60 popular, including the use of textile reinforced mortars (Al-Salloum et al. 2011), or fibre reinforced  
61 cement composites (Del Vecchio et al. 2018), benefiting from better bond, as well as improved  
62 thermal and fire resistance.

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

- 67 • Joint shear strengthening by means of sheets placed in the horizontal (e.g.: El-Amoury and  
68 Ghobarah 2002), vertical (e.g.: Le-Trung et al. 2010) or diagonal axis (e.g.: D’Ayala et al.  
69 2003) across unobstructed joint panels.
- 70 • Column flexural strengthening to prevent unwanted column hinging failure, using straight FRP  
71 sheets along the column axis (Antonopoulos and Triantafillou 2003), L-shapes (Akguzel and  
72 Pampanin 2012a; Garcia et al. 2014; Yu et al. 2016), near-surface-mounted (NSM) FRP  
73 (Hasan et al. 2016; Prota et al. 2004) or FRP anchors (Shiohara et al. 2009).
- 74 • Increasing confinement and ductility (Akguzel and Pampanin 2012a; Al-Salloum and  
75 Almusallam 2007; Antonopoulos and Triantafillou 2003; Del Vecchio et al. 2014; Engindeniz  
76 et al. 2008b; Shiohara et al. 2009) or shear strengthening (Lee et al. 2010) of columns using  
77 sheets wrapped fully around the column.
- 78 • Preventing beam bar-slippage and cracks opening at the beam-joint interface using FRP  
79 sheets along the bottom face of beams (Engindeniz et al. 2008b) or L-shapes at the bottom  
80 corner between beams and columns (El-Amoury and Ghobarah 2002; Ghobarah and El-  
81 Amoury 2005).
- 82 • Shear strengthening beams with FRP U-wraps perpendicular to the beam axis (Akguzel and  
83 Pampanin 2012a; Alsayed et al. 2010; Antonopoulos and Triantafillou 2003; Engindeniz et al.  
84 2008b).

85 While the number of research papers in this field is increasing rapidly in the last years, the last  
86 thorough state-of-the-art review paper dates back to 2010 (Bousselham 2010). Understanding current

87 trends and compiling what has been tested so far is crucial to reveal promising avenues for future  
88 research, as well as uncovering gaps in available experimental data. The aim of this paper is hence to  
89 address this need to review and critically assess the state of experimental research on RC beam-  
90 column joint retrofits with FRP.

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

99

## 100 **Review of existing research**

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

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

115 as information on the experimental set-up, loading, instrumentation and the available results of the  
116 experiments (load and displacement ductility).

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

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

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

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

138 For an “X”-configuration of unidirectional GFRP wrapped diagonally around the joint (T9), using steel  
139 angles to ease the FRP-application, initial damage is transferred to the beam, however, as the FRP  
140 debonds, the joint still fails in shear. An increase in ductility similar to T2R is achieved, with a slightly  
141 lower increase in energy dissipation. The results would suggest that adequate anchorage to resist

142 debonding could improve this layout. Overall, the importance of anchorage to avoid debonding is  
143 particularly highlighted by these experiments.

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

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

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

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

167 The effect of the steel shear reinforcement in the joint is the objective of Karayannis and  
168 Sirkelis (2008) study on 12 half-scale joints strengthened by U-shaped CFRP wraps. Anchorage is  
169 achieved perpendicular wrapping in beam and columns. The retrofit strategy is effective in

170 strengthening the joint and relocating damage into the beam. Lower damage is observed for  
171 specimens with transverse steel reinforcement, but the retrofit effectiveness is higher for specimens  
172 with no shear studs (+88.4% vs +65.3%). This highlights the effect of steel reinforcement ratio on the  
173 FRP retrofit effectiveness. In real pre-1970's joints, often the steel reinforcement is low or inexistent,  
174 which means the potential strength enhancement with FRP will be higher.

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

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

189 The "X"-shaped configuration with fibres at 45° on three sides of the joint (RNS-3 and RNS-4) is found  
190 to be most effective in increasing ductility, with a five-fold increase compared to the control specimen.  
191 Compared to Ghobarah's (2002) retrofit, diagonal wrapping is extended onto the columns, which  
192 creates better anchorage, hence preventing debonding. Additional "L"-shaped strips without  
193 anchorage in specimen RNS-4 only achieve a delay in the onset of failure, however early debonding  
194 reduces its effectiveness at higher drift ratios. In terms of strength, the improvement is less significant  
195 than for the T-shaped retrofit, with a similar increase is obtained for both X-shaped retrofitted joints  
196 (+17% and +16%).

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



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

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

220 Hadi and Tran (2014, 2016) proposed an unusual retrofit method for shear deficient exterior  
221 joints. Glued-on concrete covers are installed around the columns and joint to modify the square  
222 cross-sections and exploit the increased effectiveness of FRP wrapping in circular cross-sections.  
223 Three full-scale specimens with different amounts of U-shaped CFRP in the joint (one, three and six  
224 layers) are then tested. While the retrofits with lower amounts of FRP achieve delaying joint shear  
225 failure, only with six layers of FRP a successful change to beam hinging is observed. The increase in  
226 strength and ductility observed is not proportional to the amount of FRP used. With one (+84% in

227 strength) and three layers (+116%), FRP rupture in the column and joint are observed, while for the  
228 specimen with six layers of FRP (+140%), no debonding or rupture is seen.

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

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

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

242 A main observation from all experiments is the need for anchorage. For U-shaped configurations,  
243 Ghobarah and Said observed no improvement in behaviour without anchorage. Similarly, Realfonzo  
244 et al (2014) obtained a modest strength increase without anchorage, compared to a very strong  
245 (+99%) increase for an adequately anchored U-shaped retrofit. Anchorage is also found to be crucial  
246 in the study by Antonopoulous and Triantafillou (2003), which compared a variety of T-shaped retrofits  
247 with different amounts of horizontal and vertical FRP. It is found that the effect of horizontal FRP  
248 layers on the strength of retrofitted joints is more important than that of vertical layers. The effect of  
249 axial load is also found to be a critical parameter.

250 Comparing X-shaped retrofits with U-shaped or T-shaped retrofits, despite having the fibres oriented  
251 in the axis of principle stress, a reduced strength enhancement is found by Ghobarah and Said (2002)  
252 and Le-Trung et al. (2010). In both cases, this may however be associated to debonding problems for  
253 the X-shaped configuration. In practical terms, retrofits with horizontal and vertical sheets of FRP are

254 easier to anchor, as they can be anchored on beams and columns using steel anchors or FRP  
255 wrapping.

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

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

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

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

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

282 (60 MPa), two layers of 200 mm wide CFRP sheets were applied in the joint region at a  $\pm 45^\circ$  angle  
283 with respect to beam axis. Joint shear failure, as observed for the control specimen, is not prevented,  
284 as FRP fracture, followed by spalling of the repair mortar is observed. The repaired specimen does  
285 hence not reach the capacity of the original specimen (-25%) and the initial stiffness is reduced by  
286 28%. The non-achievement of the retrofit objective is explained by an eccentricity in axial loading,  
287 causing larger stress in one side of the retrofit. It can however also be deduced that the repair of  
288 damaged concrete with the mortar was not done in the most effective way, as a significant reduction  
289 in strength and stiffness were obtained. The type of shear failure with rupture of the FRP is also a  
290 reminder with regards to the use of lower design strains in the design procedures, rather than  
291 assuming full development of the FRP, hence avoiding brittle rupture.

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

305           Faleschini et al. (2019) repaired three full-scale, one-way exterior joints damaged by a shear  
306 mechanism in the joint region. The repair process of the pre-damaged joints started with restoring the  
307 joint with a premixed cement-based mortar. Two CFRP retrofits were tested and compared to a  
308 textile-reinforced mortar (TRM) repair scheme. The first FRP repair used diagonal FRP in the direction  
309 of expected principal stress, while the second scheme used U-shaped FRP. In both schemes, FRP  
310 jacketing of the columns above and below the joint is applied. This provides anchorage for the X-

311 shaped scheme, while the U-shaped scheme uses transversal FRP in the beams for anchorage.  
312 None of the repair schemes achieved restoration of the original strength of the joints. While  
313 detachment of the TRM lead to a loss in strength of 41%, for the X-shaped FRP fracture in direction of  
314 the principal stress with a strength loss of 30% was observed. For the U-shaped FRP, damage in  
315 concrete below the jacket lead to bulging of the FRP and a 34% loss in strength. Overall the  
316 performance of the two FRP schemes can be seen as similar, but in the case of the diagonal retrofit,  
317 due to the obtained fracture of fibres, it can be hypothesised, that a larger number of FRP layers may  
318 improve the repair performance.

319           With regards to repair of exterior joints, similar approaches to the retrofit schemes have been  
320 tested, however the behaviour of the undamaged specimen was often not recovered. Based on the  
321 limited tests, it would appear that this is mainly due to the level of repair of the damaged concrete with  
322 mortars being insufficient to rehabilitate the joint, combined with an insufficient amount of FRP  
323 calculated for the pre-damaged joints. Looking at results by Beydokhti and Shariatmadar (2016), the  
324 level of pre-damage is the most important aspect deciding whether the strength of a damaged joint  
325 can be reinstated. This observation is of interest, when looking at the repair scheme by Garcia et al  
326 (2012, 2014), which starts with a complete replacement of the damaged joint core concrete, rather  
327 than only superficially repairing the joint with mortar. For this repair a significant increase in strength is  
328 achieved, while the repaired joints by Agarwal et al. (2014), Yurdakul and Avsar (2015) or Faleschini  
329 et al. (2019), which only repair the damaged joint core by grouting before the FRP retrofit, cannot  
330 recover the strength of the pre-damaged joint.

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

332           A shear-deficient corner joint with slab and a (stub) transverse beam, retrofitted with CFRP is  
333 compared to RC-jacketing by Tsonos (2008). The retrofit scheme considers realistic geometric  
334 constraints and uses ten horizontal layers of FRP applied on the accessible joint face, as well as nine  
335 vertical and seven horizontal layers to improve flexural and shear capacities of the columns,  
336 respectively. The joint FRP is anchored with CFRP strips fully wrapped around the beam through pre-  
337 drilled slots in the slab. The strength of the joint is improved by 70% and a more ductile beam hinging  
338 mechanism is achieved, however due to the lack of anchorage along the beam, damage is

339 concentrated at the beam-joint interface. In comparison to an equivalent RC-jacketed retrofit, similar  
340 strength is obtained, but energy dissipation and stiffness are reduced.

341 Ilki et al. (2011) test corner beam-column joints with slab and transverse beams with very low  
342 concrete strength (8.3 MPa) and without joint shear reinforcement, representing a typical Turkish pre-  
343 1990's structure. Three layers of bi-directional CFRP are placed on the free side of the joint and  
344 anchored using diagonal sheets of FRP wrapped across the beams, which is practically feasible for  
345 exterior joints. For the strengthened specimen, joint shear failure is obverted, but beam bar slippage  
346 becomes the dominant mechanism, hence only a slight increase in strength is achieved (+18%). To  
347 avoid this, beam bars are welded to the back of the joint, which leads to a more ductile and dissipative  
348 beam-hinging failure. Using FRP sheets in the bottom face of the beam instead of welding additional  
349 bars is not considered as an option but would be an interesting avenue to explore to achieve a full  
350 FRP retrofit.

351 Full-scale corner joints (Del Vecchio et al. 2014) with one transverse beam are retrofitted with  
352 quadri-axial CFRP sheets in the joint panel and FRP U-wraps at the beam-joint interface to avoid  
353 brittle joint shear failure (specimen FL1). While failure mode and ductility cannot be changed, strength  
354 is increased by 17%. A second, more complete scheme, adds perpendicular unidirectional sheets on  
355 columns and beams (specimen FS1) to change the failure mechanism, but also to increase the  
356 energy dissipation. A variation of this scheme with double the amount of FRP is also tested (specimen  
357 FS2). With confinement of the column, a ductile behaviour with large increase in strength (+32% and  
358 +49% for FS1 and FS2, respectively) is observed. For specimen FS1, shear failure in the joint is not  
359 prevented, but only delayed. Doubling the amount of FRP achieves preventing joint shear failure and  
360 moving failure to column hinging, highlighting the need for column flexural strengthening to overcome  
361 a strong-beam/weak-column strength hierarchy in pre-1970's structures. Finally, an important  
362 observation is made in terms of the recorded strain values. Larger strains, close to the ultimate strain  
363 of the material are recorded by the authors, suggesting that the limits proposed by design guidelines  
364 (e.g. 0.4% strain) are conservative.

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

366 Mosallam (2000) compared the effectiveness of CFRP and GFRP on two half-scale interior  
367 joints retrofitted with two layers of bi-directional sheets fully wrapped around beams, columns and

368 joint, as well as diagonal straps around the joint. The tested specimens with GFRP display a greater  
369 increase in ductility, but in terms of ultimate load, the results for both materials are identical. The  
370 retrofit is heavily limited by practical considerations in terms of retrofit layout and scale, as slabs and  
371 transverse beams would not allow placing FRP wraps around every member.

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

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

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

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

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

404 Using continuous NSM rods, as shown in Fig. 2 (b), failure can successfully be moved away  
405 from the columns, leading to higher strength increase (up to +62%) compared to confinement  
406 wrapping alone (+33%). As failure is now moved to the joint, ductility is lowered, and additional  
407 strengthening of the joint is hence applied by means of bidirectional CFRP sheets (labelled JS in Fig.  
408 2 (b)). This full retrofit can lead to similar strength increase (+83%), but also strong increase in  
409 ductility (up to +73%). In practice, addition of the bidirectional sheets however prevents the retrofit  
410 from application in interior joints with transverse beams. The two main observations of this study are  
411 that higher axial loads lead to a larger strength increase and that the flexural strengthening of the  
412 columns (NSM rods) needs to be applied continuously through the joint, otherwise flexural failure of  
413 the columns cannot be prevented.

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



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

433 Finally, using CFRP results in a stronger increase in capacity (up to +26%) compared to BFRP  
434 (+11%), but a lower increase in ultimate drift (24% vs 49%). This is a consequence of the higher  
435 strength and stiffness of CFRP and echoes results obtained for CFRP and GFRP by other authors  
436 (e.g. Antonopoulos and Triantafillou 2003). It is worth noting that the effective strain in both FRP  
437 materials is half the guideline predictions (ACI 2008). Yu et al. propose an improved conservative  
438 strain calculation for L-shaped laminates, highlighting that effective strain in FRP can limit retrofit  
439 effectiveness.

440 Column strengthening in beam-column joint sub-assemblies is a topic generally only  
441 addressed by providing confinement to the column above and below the joint. The three studies in  
442 this section however highlight the importance of flexural strengthening of columns in pre-1970's joints  
443 that often suffer from an inadequate weak-column/strong-beam hierarchy of strengths. Prota et al.  
444 (2004) show the need for using continuous flexural strengthening through the joint in order to prevent  
445 column hinging. Their use of NSM bars at the column faces is however considerably less practical  
446 than the proposal of Shiohara et al. (2009), which instead use FRP strands passed through holes at  
447 the corners of the columns. This retrofit has the advantage of only needing minor intervention to the  
448 slab in order to be applicable in structures with slabs and transverse beams, while the NSM bars on  
449 the column face could not be continued through the transverse beam. Using L-shapes in the top and  
450 bottom columns anchored with FRP wraps around the beams (Yu et al. 2016) is also successful in  
451 preventing column hinging, while being much simpler to apply. Only in the presence of infill walls this  
452 retrofit would be less practical than Shiohara's.

453 ***Beam-strengthening in joints***

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

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

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

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

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

490 Russo and Pauletta (2012) combine a CFRP retrofit with steel-plate anchorage of plain  
491 reinforcing bars with inadequate bar anchorage length. The retrofit of exterior joints successfully  
492 prevents bond-slip, which is however mainly attributed to the steel-plate anchorage. Strength increase  
493 in 2/3-scaled specimens is more pronounced for smaller bar diameters (266% for 12 mm vs 136% for  
494 16 mm), for which bond-slip is more critical. For full-scale joints, the strength increase is significantly  
495 lower (49%).

496 Choudhury et al. (2013) developed two retrofits for exterior joints with bond-slip- or shear-  
497 deficient beams, respectively. Unidirectional CFRP sheets are used at the bottom of the beam to  
498 improve bar anchorage, while bidirectional GFRP wraps around the beam aim to improve its shear  
499 capacity (Fig. 3 (b)). Bidirectional GFRP U-wraps are also used to strengthen the joint in shear.  
500 Investigating the effect of specimen scale (full-, 2/3 and 1/3-scale) for both retrofits, the gain in load  
501 capacity is found to decrease with specimen size (from 27% to 12% for the shear- and from 24% to  
502 5% for the bond-slip retrofit). Although the findings are based on a specific layout, they still mandate  
503 caution when inferring results from scaled experiments.

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

509 This scheme would however not be applicable with slab and transverse beams and an improved  
510 retrofit without FRP in the joint panel and thin FRP-wraps that fit through slots drilled in a slab, is

511 proposed (Eslami et al. 2013; Eslami and Ronagh 2014). As shown in Fig. 3 (b), to achieve PH  
512 relocation, CFRP sheets are extended along the beam top and bottom faces for 1) a short length (175  
513 mm - one beam depth) or 2) a longer length (350 mm - two beam depths). Anchorage is provided by  
514 embedding the FRP into a 25 mm deep groove in the cover of the joint. For one test without this  
515 embedment, no improvement in the specimen's behaviour is observed. The shorter beam FRP length  
516 (1) successfully relocates PH with a strength increase of 31%. With a longer FRP length, instead,  
517 flexural failure is observed at the joint interface, attributed to the anchorage length (groove depth) not  
518 being sufficient for the length of FRP. While the strength increase is larger (45%), ductility is found to  
519 be reduced (-36%).

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

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

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

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

#### 545 ***Multi-objective retrofits of joints***

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

558 Interior joints (Al-Salloum and Almusallam 2007) and exterior joints (Alsayed et al. 2010) with  
559 a 600 mm wide slab, but no transverse beams, are retrofitted with CFRP, calculated in accordance to  
560 El-Amoury and Ghobarah (2002). Two different retrofit layouts are tested: (a) applying horizontal FRP  
561 on both faces of the joint (U-wrap for the external joint), wrapping the column above and below the  
562 joint, and finally U-wrapping the beam(s) (Fig. 4 (a)); (b) strengthening the joint only, but with steel  
563 bolts as anchorage. Both CFRP retrofit schemes delay joint shear cracking, however for the  
564 specimens without mechanical anchorage, debonding and rupture of the FRP in the beam is  
565 observed. With layout (b), the failure mode is shifted from joint shear failure, to beam hinging and  
566 hence a larger ductility. The lateral load capacities for retrofit (a) are however higher (+32% compared  
567 to +27%), as the joint and beams are also strengthened, but retrofit (b) can be seen as more efficient

568 using less material for a similar strength gain. Finally, as mentioned for other schemes, one constraint  
569 ignored by both retrofit designs is the presence of transverse beams, which would render them  
570 difficult or impossible to apply.

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

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

595 A performance-based FRP retrofit approach to provide a ductile failure is proposed by  
596 Akguzel et al. (2011; 2009). An initial test of the principle on two-dimensional interior and exterior

597 joints without slab uses a simple system of vertical and horizontal CFRP sheets (Pampanin et al.  
598 2004). The retrofit successfully alters the hierarchy of strength, but debonding reduces its  
599 effectiveness, as no anchorage is provided.

600 A similar retrofit using GFRP is used on three-dimensional corner joint without slabs under  
601 bidirectional loading (Akguzel and Pampanin 2007). With bidirectional loading, increased joint  
602 damage and faster strength degradation is observed for the control specimen. The GFRP retrofit  
603 protects the joint from excessive damage but is less effective than in an equivalent 2D specimen  
604 (+61% vs +75% strength increase). Complete relocation of damage from the joint to the end of the  
605 beam GFRP sheet is observed for both geometries, resulting in a more ductile and dissipative  
606 mechanism.

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

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

626           The idea of controlling the hierarchy of strength between columns and beams prior to  
627 retrofitting using selective-weakening is also adopted by Pohoryles et al. (2018) for retrofitting interior  
628 beam-column joints with slab and transverse beams (see Fig. 4 (d)). The retrofit uses rolled strands of  
629 CFRP sheet at the corners of the columns for continuous flexural strengthening, similar to Shiohara et  
630 al. (2009), but tested under more realistic loading conditions. Joint shear strengthening is applied by  
631 means of two CFRP strands applied through holes drilled through the transverse beams, which  
632 presents an effort to tackle joint shear strengthening in the presence of transverse beams realistically.  
633 Furthermore, next to selective weakening of the slab, continuous beam strengthening with fan-shaped  
634 FRP is applied to the beams to achieve plastic hinge relocation away from the beam/joint interface. A  
635 combination of FRP anchor fans and metallic anchors is used to prevent any potential debonding  
636 mechanisms. Overall, a behaviour close to a specimen designed to Eurocode 8 (CEN 2004) is  
637 achieved, with transfer of damage to the expected plastic hinge zone and a significant strength  
638 (+38%) and ductility increase (+90%). The same retrofit applied to a 2D cruciform specimen, however  
639 achieves an even stronger strength increase (+50%), but reduced ductility (-46%), again highlighting  
640 the importance of testing realistic specimen geometries (Pohoryles 2017).

641           In this section, retrofits with multiple strengthening objectives were discussed. The aim of  
642 these retrofits follows the principle of capacity design. To protect the joint, in most analysed retrofits,  
643 horizontal FRP, placed as U-shapes for exterior joints or L-shapes for corner joints is used. Pohoryles  
644 et al. (2018) use horizontal FRP strands passed through holes drilled into the transverse beams,  
645 hence explicitly considering practical considerations. Only Pantelides et al. (2008) used the X-shaped  
646 configuration. Column confinement and flexural strengthening are then applied, with vertical sheets of  
647 FRP applied on the column faces being the most common solution. In the presence of slabs or  
648 transverse beams, this is however highly impractical. Pampanin et al. (2012) suggest the use of L-  
649 shaped FRP, while Pohoryles et al. (2018) use vertical FRP strands passed through pre-drilled holes.  
650 Finally beam bar anchorage or plastic hinge relocation in the beams is the last objective in order to  
651 ensure a ductile hinging mechanism. For this generally sheets or strips of FRP placed along the beam  
652 or L-shapes are used. The proposed schemes all highlight the potential of FRP retrofits to achieve  
653 multiple objectives, with increases in lateral capacity and change in failure mechanism observed. The  
654 study by Engindeniz et al. (2008) highlights the higher effect of retrofits for larger concrete strengths,  
655 while Pampanin et al. (2012) and Pohoryles et al. (2018) show that 2D specimens tested without slab



656 display a larger retrofit effectiveness. Overall, the two latter retrofit schemes can be deemed most  
657 realistic for practical applications, as geometric constraints of slab and transverse beams are explicitly  
658 addressed, and the retrofit schemes are shown to be effective in eliminating brittle failure, while  
659 ensuring large increases in strength.

## 660 **Analysis and Discussion**

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

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

### 676 ***Assessment of joint shear strengthening equations***

677 As most papers in this review focus on joint shear strengthening, it is attempted to compare the  
678 experimental results to design equations from three FRP strengthening guidelines: Eurocode 8 – Part  
679 3 (CEN 2006), CNR-DT 200.R1 (CNR 2013) and ACI-400.2R08 (ACI 2008). For the purpose of this  
680 analysis, shear retrofitted specimens from the database that failed in joint shear failure are selected.  
681 This results in a total of 41 specimens summarised in Table 6. Of the chosen specimens, most are  
682 external joints (74%), but also internal (16%) and corner joints (10%) are included. Nearly half of the  
683 specimens used for analysis are full-scale (46%), with the rest mainly being 2/3 scale (44%). Finally,

684 the type of retrofits is also diverse, with most specimens used for this analysis retrofitted using U-  
685 shaped FRP (54%), and X-shaped and L-shaped (both 17%) also being commonly used. Strips are  
686 only used in the three specimens tested by Antonopoulos and Triantafillou. The selected specimens  
687 are hence diverse in geometry, size and retrofit layout. This gives this study a wider outlook on the  
688 issue of joint shear strengthening equations, as it allows testing the equations on a larger dataset  
689 compared to previous efforts (Bousselham, 2010).

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

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

706 The equations for the strengthened joint shear capacity ( $V_f$ ) in the three guidelines are summarised in  
707 Table 7. Note that the symbols and equations are slightly adapted in order to ease comparison. The  
708 factor  $\beta$  corresponds to the angle of the fibres to the axis of the column, while  $\theta$  is the angle of the  
709 principal stress in the joint (assumed to be the angle of the diagonal) to the axis of the column. In the  
710 case of strips used for retrofitting,  $w_f$  is the width of FRP strip, measured in the orthogonal direction to  
711 the fibres, and  $s_f$  the spacing between the strips of FRP (equal to  $w_f$  for sheets). The ratio of  $w_f$  to  $s_f$   
712 simplifies to 1 in the case of continuous shear strengthening is applied. In terms of the retrofit

713 dimensions,  $d_{fv}$  is the breadth of applied sheet and  $t_f$  is its total thickness. The geometry of the joint is  
714 represented by  $d$ , the effective depth and  $b_j$  the width of the joint. The partial reduction factor  $\gamma_{Rd}$  in the  
715 CNR equation is ignored for the purpose of this comparison.

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

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

727 In EC8, in turn, the effective strain is calculated based on cl.A.4.4.2(5) for fully wrapped or properly  
728 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  
729 effectiveness of anchorage is hence explicitly addressed in the guideline. Other factors in the  
730 equations affecting effective strain include concrete tensile strength, FRP ultimate tensile strength, the  
731 corner radius of the element, as well as an effective bond length. In the CNR guidelines, cl. 4.3.3.2  
732 used to calculate the effective strain is similar to the EC8 equation and also dependent on FRP  
733 shape, material strength, bond length and corner radius. Anchorage is explicitly addressed in clause  
734 4.7.2.1.4 (1)). Finally, it is stated that the maximum tensile strain of FRP wraps in RC joint  
735 strengthening shall not exceed 4‰, which is in line with ACI.

736 The results of the analysis of the joint shear strengthening guideline equations are displayed in Table  
737 6 for each specimen and summarised in Fig. 6. Looking at the results, it can be observed that all three  
738 guidelines achieve a relatively accurate prediction of the strengthened capacity. The CNR guidelines  
739 are found to be slightly conservative leading to an average ratio of 0.82 of the predicted to  
740 experimental results. The equation also delivers the lowest variance (0.52). The EC8 equation gives  
741 the best results on average (ratio of 1.08), albeit slightly non-conservative. The coefficient of variance

742 is the highest for the EC8 equation (0.85), indicating a larger spread of predicted capacities. The ACI  
743 guidelines, are found to overestimate the contribution of the FRP by 53%, with a relatively high  
744 variance (0.78).

745 Looking at the equations in the guidelines, it would appear that a main reason for the CNR and EC8  
746 guidelines performing better can be found in the use of the angle of fibres and the angle of principal  
747 stress, which allows for a more accurate calculation of the contribution of the fibres in tension.

748 Ignoring this, leads to an overestimation of the FRP contribution as seen for the ACI equations.

749 Furthermore, evaluating the bond strength of FRP to concrete, by explicitly taking into account  
750 anchorage in EC8, permits a relatively accurate estimation of the retrofitted joint shear capacity  
751 without overestimating the contribution of FRP.

#### 752 ***Factors affecting FRP retrofits***

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

763 The full database is further analysed to assess the influence of scale and geometry, as well as of pre-  
764 damage, on the increase in strength and ductility of retrofitted specimens (Table 8). The analysis  
765 confirms general observations from the review, namely that specimens without slab and transverse  
766 beams (2D) present a higher effectiveness of FRP retrofit in terms of average strength increase  
767 (+44% compared to +27%). Also, in terms of ductility, 2D specimens achieve a much larger  
768 improvement (+63%) than 3D specimens (+38%). The effect of scaled specimens is less pronounced  
769 for strength increase (+42% compared to +39% in full-scale) and reversed for ductility (+55%  
770 compared to +63% in full-scale specimens). Finally, repairs of pre-damaged specimens are not, as

771 one would expect, less effective in increasing the load capacity than retrofits of existing structures  
772 (+41% compared to +39%). This may be attributed to the replacement of damaged concrete by  
773 stronger mortar. As a decrease in initial stiffness for repaired specimens is commonly observed, in  
774 terms of ductility, repaired specimens present however a lower increase (+36% compared to +67%).

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

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

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

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

- 799       • **Strips vs sheets:** In terms of lateral load capacity, sheets seem more effective than strips.  
800       This is however based on limited evidence and may be mainly attributed to weaker bond  
801       characteristics of the latter (Antonopoulos and Triantafillou 2003).
- 802       • **Anchorage:** In all major design guidelines FRP debonding is recognised as a brittle failure  
803       mechanism, which leads to an abrupt loss of capacity in retrofitted members. Adequate FRP  
804       development lengths and reduced levels of FRP design strain are hence generally  
805       recommended to prevent it. In the literature, premature failure through FRP delamination is  
806       observed when the FRP sheets are not anchored adequately, which can cause the retrofit to  
807       be ineffective (e.g.: Ghobarah and Said 2002). Lack of anchorage affects retrofits with FRP  
808       strips more than when using sheets of FRP (Antonopoulos and Triantafillou 2003). Using  
809       transversal sheets of FRP to wrap the main strengthening has shown little effect on its own  
810       (Akguzel and Pampanin 2010). Other anchorage solutions are hence recommended:
- 811           ○ FRP dowels or fan-shaped anchors through slabs or beams (Akguzel et al. 2011;  
812           2012a).
  - 813           ○ Steel plates and rods (Al-Salloum and Almusallam 2007; Ghobarah and Said 2002).
  - 814           ○ Steel angles connected between beams and column (El-Amoury and Ghobarah 2002;  
815           Lee et al. 2010).
  - 816           ○ U-shaped steel anchors (El-Amoury and Ghobarah 2002).

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

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

829 • **Continuous strengthening:** For effective flexural strengthening of columns, an adequate  
830 bond length is required, which needs continuity through the joint. For cruciform specimen, this  
831 is addressed by simply applying FRP sheets or NSM rods continuously from the superior to  
832 the inferior column (Antonopoulos and Triantafillou 2003; Mahini and Ronagh 2011). With a  
833 slab or transverse beams, this would however not be achievable for all sides of the column  
834 (Prota et al. 2004; Shiohara et al. 2009) without the need of cutting or drilling through adjacent  
835 members. To date, only Pohoryles et al. (2018) addressed this issue explicitly. Similarly, the  
836 continuity and anchorage through the joints for longitudinal FRP sheets in the beams is only  
837 addressed by anchorage with U-wraps (Akguzel et al. 2011; Al-Salloum and Almusallam  
838 2007) or by bending the sheets in an L-shape onto the columns (Ghobarah and El-Amoury  
839 2005).

840 • **Pre-damage:** Generally, the extent of pre-damage affects the effectiveness of FRP repairs.  
841 While moderately pre-damaged specimens can be repaired to achieve similar performance to  
842 retrofitted counter-parts, for severely pre-damaged joints, the strength may not be recovered  
843 (Agarwal et al. 2014; Beydokhti and Shariatmadar 2016). The analysis of the database  
844 highlights that on average for the many repaired specimens in the database, however, the  
845 effectiveness in terms of strength increase resembles that of retrofits closely (+39% vs  
846 +41%). Only in terms of ductility a large difference is found due to the reduced stiffness of  
847 pre-damaged specimens.

848 • **Concrete strength:** The distribution of concrete strengths of the experimental specimens  
849 presented in the literature is shown in Fig. 8. The majority of specimens (65%) have a  
850 concrete with a mean strength ( $f_{cm}$ ) of 20-30 MPa. While most studies aim to replicate pre-  
851 1970s structures which often have lower concrete strengths, these are difficult to produce  
852 nowadays with modern materials. In limited studies with very weak concrete, it is observed  
853 that FRP retrofits may not be viable without partial replacement of existing concrete (D'Ayala  
854 et al. 2003; Engindeniz et al. 2008a).

- 855       • **Axial load:** The effectiveness of FRP retrofits is generally found to be strongly affected by the  
856       level of axial load. Higher levels of axial load lead to a stronger enhancement of ductility and  
857       strength for retrofitted specimens (Antonopoulos and Triantafillou 2003; Prota et al. 2004).  
858       Using an axial load varying with lateral load, however, leads to an unfavourable effect on the  
859       retrofit (Akguzel and Pampanin 2010, 2012b). In most experiments reported in the literature,  
860       one constant value of normalised axial load is used, a majority of which using values from 0.1  
861       to 0.2, as shown in Fig. 9. This value can be deemed typical for lower storey columns. It is  
862       worth noting that no axial load is applied in nearly 15% of studies.
- 863       • **Effect of scale:** The effectiveness of retrofits is seen to be reduced for full-scale specimens  
864       compared to smaller specimens (e.g.: Choudhury et al. 2013). Non-conservative results may  
865       hence be reported when basing outcomes on experiments on scaled specimens. Looking at  
866       the statistics on specimens tested in the literature (Table 5), this effect of scale may affect a  
867       large number of experiments, as 59% of specimens are less than full-scale. Indeed, the  
868       analysis of the database has highlighted that scaled specimens have a larger improvement in  
869       strength increase, however this effect is less clear than anticipated.
- 870       • **3D vs 2D set-up:** Again looking at Table 5, most of the present studies test two-dimensional  
871       (2D) specimens without slab or transverse beams (82%). The presence of framing members,  
872       but also bi-directional loading, are however seen to reduce the effectiveness of FRP retrofits  
873       (Akguzel et al. 2011). Slabs contributing to the moment capacity of the beams can influence  
874       the failure mechanism of sub-assemblies (Kam et al. 2010; Park and Mosalam 2013), often  
875       promoting unwanted column hinging mechanisms despite retrofit (Akguzel and Pampanin  
876       2012b). In practical terms, the placement of sheets and location of anchorage are also  
877       affected by the real geometry of the structure, with slabs and transverse beams typically  
878       being present for most moment-resisting frame (MRF) buildings (Genesio et al. 2010; Lehman  
879       et al. 2004; Pampanin et al. 2002). Regarding the tested joint geometries, as shown in Table  
880       5, most retrofitted specimens are exterior joints (54%), which generally are more critical, as,  
881       unlike interior joints, these are not confined from four sides and are subjected to lower axial  
882       forces. Still, there is also a large interest in interior joints, corresponding to 32% of the tested  
883       specimens. Looking at limited experimental evidence for full frames (Akguzel et al. 2011;



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

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

## 899 **Conclusions**

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

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

911 Beam plastic hinge relocation (PHR) is another application of FRP that shows potential in improving  
912 the seismic behaviour of structures. Studies show that strengthening the beam in the proximity of the  
913 joint allows relocation of damage and plastic hinge formation away from the joint. This protects the  
914 joint from yield penetration and improves the dissipative behaviour of the specimen further.

915 Anchorage of the FRP sheets at the beam/joint interface is a practical challenge that is not sufficiently  
916 addressed.

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

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

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

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- 1139
- 1140

1141 **List of Figures**

1142 Fig. 1. Schematics of typical joint shear retrofit schemes found in the literature.

1143 Fig. 2. Schematics of retrofit schemes for columns through beam-column joint sub-assemblies found  
1144 in the literature (Note: dashed line represents embedded NSM rods; JS = Joint strengthening, C =  
1145 confinement).

1146 Fig. 3. Schematics of retrofit schemes for beams in beam-column joint sub-assemblies found in the  
1147 literature. (Note: C = confinement, L = L-shape, U = U-shape, emb. = embedment; CL = centre line).

1148 Fig. 4. Schematics of multi-objective retrofit schemes for beam-column joint sub-assemblies found in  
1149 the literature: (a) El-Amoury and Ghobarah (2002) and Alsayed et al. (2010); (b) Engindeniz et al.  
1150 (2008); (c) Akguzel and Pampanin (2012) and (d) Pohoryles et al (2018). (Note: JS = Joint  
1151 strengthening, C = confinement, BA = beam bar anchorage, U = U-shape, F= full wrap, anch =  
1152 anchor).

1153 Fig. 5. Type of design deficiency analysed for tested specimens reported in the literature.

1154 Fig. 6. Average ratio of the predicted ( $V_{F,pred}$ ) to experimental ( $V_{F,exp}$ ) FRP contribution to shear  
1155 strength of joints for the three design guidelines.

1156 Fig. 7. Statistics of employed anchorage solutions in the experimental database.

1157 Fig. 8. Statistics of mean concrete strengths ( $f_{cm}$ ) of tested specimens in the experimental database.

1158 Fig. 9. Statistics of normalised axial load in the experimental database.

1159

1160 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 GFRP		+41% +45%		
Karayannis and Sirkelis (2008)		+88%			
Le-Trung et al. (2010)			+32%	+17%	
Realfonzo et al. (2014)	Unanchored Anchored	+23% +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 Near-Collapse Collapse	+6% -19% -15%			
Del Vecchio et al. (2014)	Corner				+49%
Ilki et al. (2011)	Corner				+18%
D'Ayala et al. (2003)	Retrofit Repair		+17.3% -5.2%	+92.6% +2%	
Lee et al. (2010)	Interior		+36%		

1161



1162 Table 2. Representative strength increases obtained due to the retrofit schemes for columns in beam-  
1163 column joint sub-assemblies (JS = joint shear strengthening).

1164

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%

1165

1166

1167 Table 3. Representative strength increases obtained due to the retrofit schemes for beams in beam-  
 1168 column joint sub-assemblies found in the literature.  
 1169

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%	

1170

1171 Table 4. Representative strength increases obtained using multi-objective retrofit schemes for beam-  
 1172 column joint sub-assemblies.  
 1173

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%

1174

1175 Table 5. Geometrical statistics of joints contained in database.

	Number	Joint type			Contains			Scale		
		Interior	Exterior	Corner	Slab	Trans.	BeamFull	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%

1176

1177

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

Specimen <sup>1</sup>	Experimental				ACI-440		EC8-3		CNR DT 200		
	Geometry <sup>2</sup>	Scale	FRP <sup>3</sup>	$V_{f,exp}$	$V_{f,ACI}$	$\frac{V_{f,ACI}}{V_{f,exp}}$	$V_{f,EC8}$	$\frac{V_{f,EC8}}{V_{f,exp}}$	$V_{f,CNR}$	$\frac{V_{f,CNR}}{V_{f,exp}}$	
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	

<sup>1</sup>Specimen name based on: "First three letters of first author (Specimen)"

<sup>2</sup>Ext = External; Int = Internal; Cor = Corner Joint

<sup>3</sup>Shape of FRP joint retrofit: U=U-shape; L=L-shape; X=X-shape; ST=strip; S=Single sheet

1179

1180 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	$V_{f,ACI} = 2 \cdot t_f \cdot f_{fe} \cdot (\cot \beta + \cot \beta) \cdot d_{fv} \cdot \frac{w_f}{s_f}$ with a maximum of (11-11): $V_s + V_f \leq 0.66 \sqrt{f_{cm}} \cdot b_j \cdot d$

1181

1182 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%

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