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## University of Bath

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1     **EXPERIMENTAL INVESTIGATION OF REINFORCED CONCRETE**  
2             **T-BEAMS STRENGTHENED IN SHEAR WITH EXTERNALLY**  
3                     **BONDED CFRP SHEETS**

4     Robert M. Foster<sup>1</sup>, Monika Brindley<sup>2</sup>, Janet M. Lees<sup>3</sup>, Tim J. Ibell<sup>4</sup>, Chris T. Morley<sup>5</sup>, Antony  
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6  
7     An experimental investigation was undertaken into the effectiveness of unanchored and  
8     anchored externally bonded (EB) U-wrapped carbon fibre reinforced polymer (CFRP) shear  
9     strengthening for reinforced concrete T-beams at a range of realistic sizes. The T-beam sizes,  
10    geometry and reinforcement were chosen to reflect existing slab-on-beam structures with low  
11    levels of transverse steel shear reinforcement. Geometrically similar reinforced concrete T-  
12    beams were tested across three sizes ranging from 360 to 720 mm in depth and with different  
13    amounts of EB CFRP shear reinforcement. The beams were subjected to three-point bending  
14    with a span to depth ratio of 3.5. All the beams failed in diagonal shear. The experimental  
15    results indicate significant variability in the capacity of unstrengthened control beams, and a  
16    number of these control beams showed greater shear capacity than their EB CFRP  
17    strengthened counterparts. Greater thicknesses of CFRP reinforcement did not lead to  
18    increased shear capacity compared with lesser thicknesses of unanchored or anchored EB  
19    CFRP, but anchored EB CFRP did lead to moderate increases in shear capacity compared to  
20    both control and unanchored EB CFRP strengthened beams.

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22 **Keywords:** reinforced concrete T-beam, shear strengthening, externally bonded carbon fibre  
23 reinforced polymer fabric, size effect

24

25

## INTRODUCTION

26 Accurate assessment of the actual strength of reinforced concrete structures and the need for  
27 effective strengthening are a growing concern worldwide. This applies both to buildings and  
28 to infrastructure, with infrastructure being the area of greater economic concern. The cost of  
29 assessing and strengthening deficient bridge structures alone has been estimated as being in  
30 excess of £4 billion for the UK (Middleton 2004) and \$140 billion for the US (American  
31 Association of State Highway Transportation Officials 2008).

32

33 Deficiencies in the strength of reinforced concrete infrastructure can arise due to a variety of  
34 factors including accidental damage, construction defects, deterioration, changes in  
35 understanding, changes in use and failure to design for future loading. The demolition and  
36 replacement of such structures can involve large capital expenditure, environmental impacts,  
37 interruptions to service, over-burdening of nearby infrastructure, and local opposition to  
38 construction.

39

40 Approaches to strengthening existing concrete structures in-situ are therefore of considerable  
41 interest to infrastructure owners seeking to extend a structure's useful life. Of interest as  
42 materials for use in concrete strengthening applications are fibre reinforced polymers (FRPs)  
43 and in particular carbon fibre reinforced polymers (CFRPs), primarily due to their favourable  
44 strength-to-weight ratios and resistance to various forms of corrosion. FRP strengthening for  
45 reinforced concrete structures has been the subject of extensive research (Bakis et al. 2002).  
46 FRP materials are currently in use in strengthening and repair applications, and design

47 guidance exists in a number of jurisdictions for embedded and externally bonded (EB)  
48 strengthening for axial, flexural, shear and seismic applications (RILEM 2016).

49

50 A common structural form that may require shear strengthening is that of a slab-on-beam  
51 arrangement. While there is extensive evidence that slab-on-beam structures, usually  
52 modelled experimentally by T-beams, are often stronger in shear than similar rectangular  
53 beams (Pansuk & Sato 2007), only the contribution of the web section is typically considered  
54 for the purposes of design. EB CFRP reinforcement may be preferred in many strengthening  
55 applications as it avoids the need to remove areas of concrete or drill into the section with the  
56 associated risks of exposing or damaging existing reinforcement. However, in the case of a  
57 T-beam, the presence of the flange means that such a strengthening system cannot be fully  
58 wrapped around the beam. This commonly leads to partial ‘U-wrapping’ of the accessible  
59 down-stand portion of the beam in which the CFRP anchorage relies entirely on surface  
60 bonding to the web cover concrete. The CFRP anchorage may thus terminate below the  
61 neutral axis, which in most T-beams occurs within the depth of the flange. This means that  
62 the CFRP anchorage is located in a region of tension, and that the tension and compression  
63 regions are not connected by the CFRP reinforcement.

64

65 While a large number of experimental investigations on the FRP shear strengthening of  
66 reinforced concrete have been carried out, an analysis by Lima & Barros (2011) of a database  
67 of over 250 EB CFRP shear strengthened beams indicated that the mean height of tested  
68 beams was approximately 350 mm, with 54% of beams having a concrete compressive  
69 strength between 20 and 30 MPa, and 51% having no shear reinforcement. Only half of the  
70 tests considered a U-wrapped CFRP arrangement and 83% of tests were carried out on  
71 rectangular beams. Although guidance exists for U-wrapped FRP strengthening systems,

72 evaluation of a number of these models against the beams in this data set led Lima & Barros  
73 (2011) to conclude that none of the available analytical formulations predicted the  
74 contribution of EB FRP systems for the shear strengthening with sufficient accuracy. Some  
75 recent investigations have provided experimental evidence of a lack of conservatism in the  
76 prediction of the FRP contribution to shear resistance (Dirar et al. 2012, Mofidi & Chaallal  
77 2014). Investigators have also reported results indicating that increasing the CFRP thickness  
78 in EB FRP systems may not result in increased gains in shear strength (Bousselham &  
79 Chaallal 2006) and that a strengthened beam can fail at a lower shear load than a nominally-  
80 identical unstrengthened control beam (Deniaud & Cheng 2001). Test series investigating the  
81 shear strengthening of prestressed I-girders have identified that the EB FRP contribution to  
82 be strongly influenced by the cross-sectional geometry and that the provision of EB FRP  
83 strengthening can lead to a reduction in shear capacity (Murphy et al. 2012). Investigators  
84 (Mofidi et al. 2012, Ozden et al. 2014) have reported that greater effectiveness of the external  
85 shear-strengthening system could be achieved when the CFRP sheets are anchored in the  
86 compression zone of the beam as proposed by Khalifa et al. (1999). This paper presents  
87 details of an investigation carried out in order to provide new experimental data with which  
88 to evaluate the influence of size, CFRP ratio and anchorage condition in realistically-sized  
89 CFRP-strengthened T-beams with internal transverse steel reinforcement.

90

91

## RESEARCH SIGNIFICANCE

92 This research investigates the shear behaviour of reinforced concrete T-beams with low  
93 levels of transverse steel reinforcement strengthened with U-wrapped CFRP fabrics at a  
94 range of realistic sizes. Three sizes of geometrically scaled T-beams of 360, 540 and 720 mm  
95 depth, with a shear span to depth ratio of 3.5, were tested in three-point bending until failure  
96 in shear. Unstrengthened control beams at each size were tested, as were beams strengthened

97 with varying thicknesses of CFRP. The 540 and 720 mm high beams were also tested with  
98 anchored CFRP, with the additional anchorage provided by a longitudinal near-surface-  
99 mounted bar-in-slot system. By testing multiple unstrengthened control specimens, this study  
100 provides experimental evidence of the variability of control specimens and the influence of  
101 the variability of the underlying reinforced concrete T-beam on the effectiveness of CFRP  
102 strengthening. This area has been largely unaddressed by previous investigations into CFRP  
103 shear strengthening. This research also provides important experimental evidence that, in at  
104 least some cases, the capacity of the unanchored EB CFRP strengthened beams was lower  
105 than that of unstrengthened counterparts.

106

107

## **EXPERIMENTAL PROGRAMME**

### **108 Test series**

109 The T-beam test series presented here was carried out as part of a joint experimental  
110 programme at the University of Bath and the University of Cambridge investigating the  
111 behaviour of reinforced concrete T-beams strengthened with CFRP materials. A total of 15  
112 reinforced concrete T-beams were designed to fail in shear under three-point bending. Beams  
113 are designated by a letter 'L' for large, 'M' for medium and 'S' for small followed by a 'B'  
114 indicating testing at Bath or a 'C' indicating testing at Cambridge. In the case of  
115 unstrengthened control beams, this second letter is followed by a 'C', with a subscript  
116 differentiating between multiple control beams 'C<sub>1</sub>', 'C<sub>2</sub>'. In the case of beams with CFRP  
117 strengthening, the second letter is followed by a number indicating the percentage of CFRP  
118 provided and followed by a letter 'U' indicating an unanchored U-wrapped configuration or  
119 'UA' indicating an anchored U-wrapped configuration. For example, a small beam with 1  
120 layer of 0.5 mm thick U-wrapped CFRP strengthening (0.7%) and tested in Cambridge is  
121 designated SC0.7U.

122

123 The T-beam geometry was scaled in order to investigate the effect of size on CFRP  
124 strengthened beam behaviour. The concrete cover was also scaled, with nominal cover  $c_{nom}$   
125 of 40 mm, 30 mm and 20 mm for the large, medium and small beams respectively. Aggregate  
126 size was not scaled. The specimen geometries and reinforcement arrangement are shown in  
127 Fig. 1.

128

129 The T-beams were designed with a transverse reinforcement ratio of 0.1%, in order to  
130 investigate the behaviour of structures with very low transverse reinforcement provision. In  
131 the test span, shear reinforcement was provided in the form of closed links fabricated from  
132 plain mild steel bar. Mild steel was chosen partly to reflect material properties of reinforcing  
133 steel found in many historic structures and partly to provide an adverse case for load share  
134 between the steel and the CFRP strengthening. The internal transverse steel reinforcement in  
135 the test span was spaced at  $0.6d$ . In order to ensure failure in the test span, substantial  
136 transverse reinforcement was provided to the non-test span in the form of deformed steel  
137 links at a transverse reinforcement ratio of approximately 0.5%. The main flexural  
138 reinforcement consisted of six bars arranged in two layers, as shown in **Fig. 1**. The  
139 longitudinal tension reinforcement ratio based on web area was 2.2% for the large beams,  
140 2.4% for the medium beams and 3.5% for the small beams. It should be noted that, due to a  
141 fabrication drawing error, the longitudinal reinforcement ratio for the small beams is rather  
142 higher than for the medium and large beams.

143

144 For the strengthened systems, two arrangements were considered: externally bonded  
145 continuous CFRP sheets without end anchorage and CFRP sheets anchored with a near  
146 surface mounted bar-in-slot anchorage system. The CFRP arrangements are shown in **Fig. 2**.

147 The beams were designed and constructed to reflect some of the constraints typical of  
148 existing concrete structures. A chamfered 45° haunch detail was provided, which is typical  
149 for cast-in-place slab-on-beam structures and reduces the vertical bonded length available for  
150 the CFRP sheets. This detail is provided to both strengthened and unstrengthened control  
151 beams. The externally bonded sheets were applied in a U-wrap configuration with CFRP  
152 sheets bonded to three sides of the beam. The anchored U-wrap configuration was further  
153 provided with a continuous near-surface-mounted bar-in-slot anchorage system at the base of  
154 the haunch detail. The CFRP thickness was varied in order to investigate the influence of  
155 CFRP reinforcement ratio  $\rho_{frp}$  on behaviour. Two weights of carbon fibre fabric were used in  
156 order to target  $\rho_{frp}$  of 0.7% and 1.3%. Due to the limited fabric weights available, the medium  
157 sized beam with one layer of fabric MC0.9U was provided with  $\rho_{frp}$  of 0.9%. Details of the  
158 test matrix are presented in **Table 1**.

159

160 The large beams and three medium beams were tested at the University of Bath. The small  
161 beams and three medium beams were tested at the University of Cambridge. All beams were  
162 fabricated at the same precast facility using the same concrete mix design and aggregate  
163 source. The same formwork was used for the medium-sized beams tested at both Bath and  
164 Cambridge. The longitudinal reinforcement and the transverse reinforcement in the non-test  
165 span were supplied by the precaster. Transverse reinforcement in the test span was supplied  
166 and instrumented by the authors. Fabrication of the reinforcement cages and the casting of the  
167 beams were overseen by the authors in order to ensure good quality control procedures.

168

### 169 **Material properties**

170 The concrete used in this study was made up of coarse limestone aggregate (20 mm  
171 maximum dimension), fine grit-sand aggregate and ordinary Portland cement, with a water-



172 cement ratio of 0.53. A concrete compressive cube strength of 60 MPa was targeted in line  
173 with the higher present-day concrete strengths of many historic concrete structures (Thun et  
174 al. 2006). All beams were cured for a minimum of 28 days prior to the application of CFRP  
175 strengthening. The mean concrete cube strength for each beam on test day is shown in **Table**  
176 **1**.

177

178 Plain mild steel bar, nominally S275, was used for transverse steel links in the test span. All  
179 other steel reinforcement was deformed high yield steel bar. Steel reinforcement properties  
180 were determined by direct tensile testing. The results of the direct tensile testing on steel are  
181 summarized in **Table 2**. Where direct tensile test results were not obtained, characteristic  
182 values are given following BS 4449 (2005).

183

184 The externally bonded CFRP used in this study was a commercial system comprised of one  
185 or more layers of carbon fibre fabric acting compositely with a two-part epoxy resin matrix.  
186 Two fabrics were used in this study, with dry fibre content of 644 g/m<sup>2</sup> and 393 g/m<sup>2</sup>  
187 respectively – in conjunction with an epoxy resin. In both fabrics the weave is effectively  
188 uni-directional, having only a small number of aramid or carbon fibres perpendicular to the  
189 primary carbon fibre direction, in order to maintain the integrity of the loose fabric. The  
190 CFRP bars used for anchorage were spiral-wound sand-coated bars. Material properties for  
191 the CFRP materials obtained from the manufacturers' data sheets (Tyfo 2013a, 2013b, Aslan  
192 2011) are summarised in **Table 3**. The bond strengths of the concrete and the CFRP-concrete  
193 interface for the Bath beams were determined post-test in the undamaged regions of the  
194 reaction span according to ASTM D7522. The mean values of bond strength to the concrete  
195 surface  $f_b$  were 2.6 MPa for both the large and the medium beams. The mean bond strengths  
196 of the CFRP to the concrete  $f_{bf}$  were 3.0 MPa and 3.5 MPa for the large and the medium

197 beams respectively, greatly exceeding the 1.4 MPa minimum tension adhesion strength  
198 requirements of ACI440.2R-08 (ACI 2008).

199

#### 200 **Beam fabrication and strengthening**

201 Beams were cast in high quality stiffened timber formwork which was struck after  
202 approximately 24 hours and the moulds cleaned, oiled and reused for the next beam. While  
203 pouring, the concrete mix was vibrated with pokers to ensure good compaction. The beams  
204 were cast web down – as an in-situ beam would be cast on site – with the main longitudinal  
205 tension reinforcement in the ‘good bond’ zone (BSI 2004). After a minimum 28 days, the  
206 web portion of the test span of beams to receive externally bonded CFRP was prepared to  
207 remove any loose surface material in accordance with the manufacturer’s guidance (Tyfo  
208 2013a and 2013b). Due to local constraints, differing surface preparation methods were used  
209 across the beam series. However, visual inspection indicated that there was no significant  
210 variation in the finish achieved and all methods suitably removed the external cement paste  
211 layer to expose the underlying aggregate. The large beams were prepared by ‘dry sponge  
212 blasting’; the medium Bath beams were prepared by wet grit blasting followed by a two week  
213 drying period; and the medium and small Cambridge beams were prepared by hand-held disk  
214 grinding. Discussion with the CFRP manufacturer’s technical representative indicated that, in  
215 their experience, all three preparation methods are suitable and that while surface preparation  
216 is an important consideration in the case of deteriorating or damaged concrete in existing or  
217 historic structures, it is less critical in the case of undeteriorated concrete. The web soffit  
218 corners were ground to a recommended minimum radius of 25 mm to prevent premature  
219 failure of CFRP due to stress concentrations at the corners. For the bar-in-slot anchorage  
220 system, slots were chased along the haunch detail to provide clearance of 30 mm x 30 mm

221 and 25 mm x 25 mm for large and medium beams respectively. The corners of the slot were  
222 ground to a radius of only 15 mm due to space limitations.

223

224 The CFRP was applied in a wet lay-up system. An initial priming layer of epoxy resin was  
225 brushed onto the prepared concrete surface. The carbon fibre fabric, cut to size, was saturated  
226 with epoxy by roller and then applied to the concrete with the principal fibre direction aligned  
227 perpendicular to the longitudinal axis of the beam. In order to remove air bubbles and ensure  
228 that the material was suitably bedded against the concrete substrate, a roller was applied in  
229 the principal fibre direction. A further coat of epoxy was brushed over to ensure full coverage  
230 of the fibres and provide protection. Where a second layer of fabric was applied, the epoxy  
231 coat provided a primed base for the second layer and the process was repeated. In the case of  
232 the Bath beams, the epoxy was thickened with silica fume approved by the manufacturer. For  
233 the anchored U-wrap strengthening systems, the CFRP sheets were applied as for unanchored  
234 cases and secured by continuous CFRP bars coated with thickened epoxy and inserted by  
235 hand into the prepared slots. CFRP bar diameters of 12 mm and 10 mm were used for the  
236 large and medium beams respectively. All beams tested at Bath were prepared and  
237 strengthened along the entire length of the beam by specialist contractors. Specimens  
238 strengthened at Cambridge were prepared and strengthened in-house in the test span in  
239 accordance with the manufacturer's guidance (Tyfo 2013 and 2013b) and following training  
240 by a specialist contractor. In both cases the procedures were instructed and supervised by the  
241 authors.

242

### 243 **Loading and instrumentation**

244 The loading arrangements in the two test facilities were statically equivalent, but the actual  
245 test set-up was not identical. At Bath, the load was applied through the central support from

246 above using an automatic hydraulic Instron testing machine with maximum capacity 2000 kN  
247 at a displacement rate 1 mm/min. To achieve support conditions consistent with a simply  
248 supported beam, two layers of oiled PTFE sheets were inserted between the supporting steel  
249 plates in the tested span region to create a sliding pin. At Cambridge, the beams were tested  
250 under displacement control at a manually controlled displacement rate using a 5000 kN  
251 Amsler column testing rig. Load was applied from below to the end supports through a  
252 spreader beam and the reaction was provided by the central support above. Simply supported  
253 conditions were achieved through the use of a captured pin at the central support and sliding  
254 pins at the end supports. In both arrangements the load at the central support was applied  
255 across the width of the flange. The loading and support conditions are shown in **Fig. 3**.

256

257 The transverse steel reinforcement in the test span of all beams was equipped with single-  
258 direction strain gauges on both legs of the stirrup at mid-height of the link. The strain gauges  
259 applied to the EB CFRP sheets of the Bath beams were three-directional strain gauge rosettes.  
260 The strain gauges on CFRP were located based on an assumed main shear crack location to  
261 capture debonding processes. For the Cambridge beams the strain gauges applied to the EB  
262 CFRP were single directional strain gauges aligned with the principal fibre direction of the  
263 CFRP and positioned at mid-height at the link positions. In this way the strains in the CFRP  
264 and the transverse steel reinforcement were obtained at similar locations. The strain gauge  
265 layout for the steel reinforcement and CFRP strengthening is shown in **Fig. 3**.

266

267

### TEST RESULTS AND DISCUSSION

268 All test specimens failed in diagonal shear. The failure of the CFRP strengthened beams was  
269 preceded by progressive separation of the CFRP material. Separation of the CFRP was  
270 identified post-test as having occurred through the cover concrete in all cases. The ultimate

271 shear force  $V_u$  was recorded at failure with corresponding mid-span displacement,  $\Delta_u$ . The  
272 shear force at steel yield strength  $V_{fy}$  was determined from strain gauge readings on the  
273 transverse steel reinforcement at the load where strain gauges registered the first yielding.  
274 Due to differences in the yield strength of the steel used, the yield strains obtained by direct  
275 tensile testing were 0.0016 and 0.0020 for large and medium Bath beams, and 0.0029 and  
276 0.0024 for the medium and small Cambridge beams. Corresponding mid-span displacements  
277  $\Delta_{fy}$  were also determined from the test data. A summary of the test results is presented in  
278 **Table 4**. A malfunction of the data acquisition systems during the testing of beam  $MCC_2$   
279 means that the relationship between load and measured strains and displacements cannot be  
280 reliably determined. However, the applied load was captured by a secondary system allowing  
281 the peak shear force to be given with reasonable confidence.

282

283 Significant variation in shear load capacity was observed between unstrengthened control  
284 beams. This variation was observed both between beams tested at the same facility,  $SCC_1$  and  
285  $SCC_2$ ; and between beams tested at different facilities, MBC and  $MCC_1 / MCC_2$ . In all cases,  
286 the beams provided with unanchored EB CFRP failed at lower loads than those of the  
287 stronger of their respective control specimens. Beams provided with anchored EB CFRP  
288 reached higher loads than both their respective control beams and their unanchored  
289 counterparts. However, the increase in strength associated with the anchored EB CFRP was  
290 small when considered with reference to the stronger of the relevant control beams.  
291 Increasing  $\rho_{frp}$  did not, in most cases, lead to increasing shear strength for either anchored or  
292 unanchored EB CFRP. Values of  $V_{fy}$  were significantly greater for the CFRP strengthened  
293 beams than for the unstrengthened control beams, indicating that the externally bonded  
294 strengthening delayed the onset of yield in the transverse steel reinforcement.

295

296 Fig. 4 shows the failure modes of the unstrengthened control beams, and of the strengthened  
297 beams after testing and removal of separated CFRP U-wrap for inspection. A range of critical  
298 diagonal crack inclinations were observed. Significant penetration of the flange by the  
299 eventual critical diagonal crack prior to peak load was observed for the weaker  
300 unstrengthened control beams MCC<sub>1</sub>, MCC<sub>2</sub> and SCC<sub>2</sub>. The critical diagonal web cracks in  
301 the ‘stronger’ control beams LBC, MBC and SCC<sub>1</sub> were quite shallow, with an inclination  $\beta$   
302 of approximately 22-23° to the longitudinal axis of the beam. Note that a line drawn platen-  
303 to-platen would have an inclination of 21.8° which is also the minimum strut inclination  
304 permitted by the EC2 variable inclination strut model (BSI 2004). The critical diagonal web  
305 cracks in the weakest control beams MCC<sub>2</sub> and SCC<sub>2</sub> were inclined at approximately 45°,  
306 which is also the maximum strut inclination permitted by the EC2 variable inclination strut  
307 model (BSI 2004). The critical diagonal web crack in beam MCC<sub>1</sub> developed at an  
308 intermediate inclination of approximately 31°. The CFRP strengthened beams, which could  
309 only be inspected after testing, showed evidence of critical diagonal web cracking at an  
310 inclination of approximately 37° in most cases. These observations suggest that the  
311 inclination of critical diagonal web cracking can vary considerably in otherwise-similar  
312 unstrengthened T-beams. Although a relationship between critical diagonal web crack  
313 inclination and shear capacity is indicated, it is unclear whether variation of the web crack  
314 inclination is itself a cause of a change in capacity, or a consequence of variability in some  
315 other load resisting system(s). The presence of externally bonded CFRP strengthening  
316 appears to be associated with reduced variability in both critical diagonal web crack  
317 inclination and shear capacity, for the beams considered here.

318

319 **Shear-deflection behaviour**

320 The shear-deflection behaviour of the strengthened and unstrengthened beams for the three  
321 different beam sizes is shown in **Fig. 5**. For the beams tested in Bath, a number of unloading-  
322 reloading cycles were carried out during initial loading. These cycles are not shown in Fig. 5  
323 for reasons of clarity. The full shear-deflection cycle data is included with the test data  
324 associated with this paper.

325

326 All unstrengthened control beams showed nearly linear elastic behaviour until the onset of  
327 diagonal shear cracking. A diagonal crack, initiating in approximately the middle third of the  
328 height of the beam web and propagating towards the support and loading platens, was  
329 observed in each of the unstrengthened control beams. The onset of diagonal cracking is seen  
330 in the shear-deflection plots as an abrupt change in the gradient of the ascending branch. For  
331 unstrengthened control beams LBC, MBC,  $MCC_1$  and  $SCC_1$ , the onset of diagonal cracking  
332 was followed by a further near-linear ascending portion at a reduced stiffness. For beams  
333 LBC, MBC and  $SCC_1$ , this ascending portion remained almost linear until sudden failure at  
334 peak load. These failures were observed to be very brittle and energetic, with little or no  
335 observed diagonal crack penetration of the beam flange prior to peak load. It should be noted  
336 that these were also the ‘stronger’ control beams, i.e. those that achieved greater peak shear  
337 loads than their unanchored strengthened counterparts. For beam  $MCC_1$ , failure was preceded  
338 by further softening of the ascending branch. Progressive penetration of the critical diagonal  
339 crack into the flange was observed during this period. After the onset of diagonal cracking,  
340 beam  $SCC_2$  showed a brief increase in shear load, at a similar gradient to that displayed by  
341  $SCC_1$  after cracking, prior to a further drop in load. This coincided with penetration of the  
342 flange by the diagonal crack, running almost to the central support platen. A small further  
343 increase in shear load was seen at a lower gradient before a progressive falling-off of load  
344 post-peak.

345

346 All strengthened beams showed a similar pattern of shear-deflection behaviour. Beams with  
347 one and two layers of EB CFRP U-wrap appeared to behave similarly. The beams with  
348 unanchored CFRP displayed near linear elastic shear-deflection behaviour until  
349 approximately twice the load associated with the onset of diagonal cracking for the  
350 corresponding control beam(s). This indicates that the onset of diagonal cracking was  
351 significantly delayed or inhibited by the EB CFRP. The faltering shear-deflection behaviour  
352 observed at or close to peak load corresponds to the observed progressive separation of the  
353 EB CFRP sheets from the main web concrete. The beams with anchored CFRP displayed  
354 similar shear deflection behaviour to the beams with unanchored CFRP but the peak loads  
355 associated with separation of the CFRP were higher than for the unanchored specimens. Post-  
356 test inspection indicated that the CFRP separation failure in all cases occurred through the  
357 cover concrete, with the separated material including whole aggregate, rather than through  
358 the epoxy-concrete interface. A substantial ‘wedge’ of separated concrete along the line of  
359 the main diagonal cracking was found bonded to the CFRP wrap in all sizes of beam. This  
360 separated wedge was observed to be larger for the larger beam sizes.

361

## 362 **Ductility**

363 For the purposes of comparison it can be useful to attempt to quantify ductility. While  
364 ductility is commonly expressed in terms of a ratio between displacement at failure and  
365 displacement at yield, i.e. the ratio of plastic to elastic capacity; this may not be applicable to  
366 relatively brittle failure modes such as shear. An approach adopted by Dirar (2009),  
367 following Barrera et al. (2006), is to relate the displacement at failure to a notional equivalent  
368 elastic deflection at the failure load.

369



370 Defining a displacement ductility  $\mu_{\Delta}$ :

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_{e,n}} \quad (1)$$

371 where  $\Delta_u$  is the vertical displacement at  $V_u$  and  $\Delta_{e,n}$  is a notional equivalent elastic vertical  
372 displacement **determined from the shear force deflection curve**. The displacement  $\Delta_{e,n}$  is  
373 taken as the displacement that would be achieved if behaviour remained elastic until failure at  
374  $V_u$ . Values obtained for  $\mu_{\Delta}$  are shown in **Table 4**. By this measure, the U-wrapped medium  
375 and small beams display a reduced ductility compared with the unstrengthened control beams,  
376 with U-wrapped beams obtaining values of  $\mu_{\Delta}$  in the range 1.1-1.3 and control beams  
377 obtaining values in the range 1.6-2.2. The decrease in ductility did not appear to be sensitive  
378 to the thickness of EB CFRP in these beams. Ductility of the large beams was similar for  
379 both unanchored U-wrapped and control beams, but was reduced for the beams with  
380 anchored strengthening. The ductility of the small and medium control beams was in all cases  
381 greater than that of the large control beam. This may provide an indication that, while the  
382 addition of EB CFRP may extend a beam's elastic shear-deflection behaviour, ductility may  
383 be reduced. This may be particularly true for smaller beams.

384

### 385 **Effect of size**

386 In order to compare the effect of size on the behaviour of the strengthened and  
387 unstrengthened beams, it is convenient to normalise the shear-deflection behaviour of the  
388 beams as shown in **Fig. 6**. The normalised nominal shear stress  $v/f_{cu}$  is given by:

$$\frac{v}{f_{cu}} = \frac{V}{b_w d f_{cu}} \quad (2)$$

389

390 The value  $v/f_{cu}$  represents the average shear stress across the web section relative to the  
391 compressive strength of the concrete. This is plotted against the vertical deflection  $\delta_v$

392 normalised by effective depth  $d$ . **Fig. 6** shows that the normalised ‘stiffness’ of the medium  
393 and large control beams is similar but that the small control beams are stiffer, both before and  
394 after the onset of diagonal cracking. **Fig. 6** also indicates that the small beams strengthened  
395 with unanchored CFRP have a greater normalised stiffness than the medium and large beams  
396 strengthened with unanchored CFRP, although to a lesser extent than for the unstrengthened  
397 control beams. This difference in stiffness may be at least partially attributed to differences in  
398 longitudinal reinforcement ratio (**Table 1**).

399

400 **Fig. 7** plots the peak shear stress  $v_u$  (**Table 1**) normalised by  $f_{cu}^{1/2}$  against the natural log of  $d$ .  
401 The dotted line indicates the gradient of the trend predicted by linear fracture mechanics for  
402 the size effect on shear in concrete (Yu & Bazant 2011). The pattern of results indicated both  
403 by the ‘stronger’ control beams, and by the strengthened beams is not incompatible with this  
404 trend. The similarity of the apparent size effect for both the ‘stronger’ control beams and the  
405 strengthened beams indicates that behaviour in the strengthened cases may have been  
406 dominated by the underlying reinforced concrete beam. The absence of the same trend in the  
407 weaker unstrengthened beams, particularly beam SCC<sub>2</sub>, indicates a different failure mode;  
408 with failure not precipitated by sudden fracture of the concrete. This is compatible with the  
409 observed, less brittle and less energetic failure mode of beam SCC<sub>2</sub> (**Table 4**). A size effect  
410 relating to the effectiveness of the EB CFRP strengthening is not apparent. **This is in contrast**  
411 **with the clear size effect in EB FRP strengthening reported for rectangular beams of similar**  
412 **depth to those tested in this series (Leung et al. 2007).**

413

#### 414 **CFRP behaviour**

415 CFRP behaviour was characterised in all cases by progressive separation of the CFRP above  
416 the critical diagonal crack. Peak load was associated with complete separation of the CFRP

417 sheet above the crack. For the beams tested at Bath, the separation of the U-wrapped CFRP  
418 was captured using a high definition camera. **Fig. 8** shows the progressive separation of the  
419 CFRP for beams LB1.3U and LB1.3UA. In the case of the beams with unanchored  
420 strengthening (**Fig. 8a**), vertical splitting of the sheets was particularly evident; this was also  
421 observed in the Cambridge beams. For the anchored specimens (**Fig. 8b**), differential  
422 separation at the edge of the sheets was largely prevented by the continuous bar-in-slot  
423 anchorage system, although the ultimate separation of the sheets initiated in the same region  
424 as for the unanchored U-wrap. As this fabric separation propagated towards the anchored  
425 edge of the CFRP sheet, the CFRP bar anchoring the sheets was pulled out of the slot leading  
426 to failure of the beam. Rupture of the CFRP material across the principal fibre direction was  
427 not observed in any of the tested beams, with failure of the CFRP strengthening governed  
428 entirely by separation.

429  
430 **Fig. 9** shows the strain gauge readings on the surface of the CFRP plotted against vertical  
431 deflection. Deflection at peak load  $\Delta_u$  is also indicated. The discrete peaks in strain indicated  
432 by the readings suggest that higher strains in the unanchored CFRP strengthening were only  
433 present over a limited portion of the shear span at any one stage of loading. Strain gauges for  
434 the Cambridge beams were positioned approximately at the link spacing of  $0.6d$  and a similar  
435 spacing for the Bath beams (**Fig. 3**). This indicates that peaks in strain occurred over a width  
436 smaller than the  $0.6d$  interval between gauges which suggests that the full width of the CFRP  
437 across the shear crack is not mobilised simultaneously. The peaks in strain are followed by an  
438 abrupt drop-off in strain indicating separation. The separation process can thus be seen as a  
439 relatively narrow wave front propagating from approximately the position at which the  
440 critical diagonal crack eventually intersects the underside of the flange and out towards the  
441 end support. For the beams with anchored strengthening, strain development was more

442 gradual and there was greater overlap indicating that strains developed over a greater width  
443 of CFRP than in the unanchored case with the continuous bar-in-slot anchorage system  
444 providing some bridging across vertically-split sections of CFRP. This suggests a greater  
445 width of CFRP is contributing to resisting shear in the anchored compared to the unanchored  
446 case. However, the maximum strains in the CFRP are broadly similar whether unanchored or  
447 anchored. These measurements appear to agree with the separation behaviour observed in **Fig.**  
448 **8.**

449

#### 450 **Comparison with code predictions**

451 In **Table 5** the strengthened beam capacities are compared with the predictions of TR55  
452 (Concrete Society 2012), *fib* 14 (*fib* 2001) and ACI440.2R-08 (ACI 2008) whilst the control  
453 beams are compared with the predictions of the corresponding guidance for unstrengthened  
454 beams EC2 (BSI 2004), and ACI318-14 (ACI 2014).

455

456 The design approach adopted by EC2 for reinforced concrete beams with transverse shear  
457 reinforcement is a variable angle truss model. Resistance is determined solely by the  
458 contribution of the transverse reinforcement at an assumed concrete strut inclination between  
459  $21.8^\circ$  and  $45^\circ$ , subject to a limiting concrete stress in the concrete web to prevent crushing of  
460 the concrete strut. This design approach is based on the lower bound theory of plasticity for  
461 reinforced concrete and as such is theoretically conservative. The ACI318 model considers an  
462 empirically derived concrete contribution in addition to a transverse reinforcement  
463 contribution determined by a truss model with a fixed  $45^\circ$  concrete strut inclination. TR55  
464 and *fib*14 consider a further additional contribution from the FRP strengthening using a fixed  
465 angle truss model superposed onto the underlying EC2 model. ACI440 considers an FRP  
466 strengthening contribution superposed onto the underlying ACI318 model in a similar

467 manner. Potential for contribution of the T-beam flange to shear resistance is neglected in all  
468 cases.

469

470 EC2 and ACI318 under-predict the strength of the control beams despite the setting of  
471 explicit safety factors to 1. For the stronger control beams LBC, MBC and SCC<sub>1</sub> the  
472 predictions are particularly conservative. The predictions for the capacity of the strengthened  
473 beams are generally less conservative than those for the unstrengthened beams with the  
474 unfactored values predicted by *fib*14 and ACI440 often being unconservative. Significant  
475 variation is seen between the shear capacity predicted by the EC2 and ACI318 for  
476 unstrengthened beams; and between TR55, *fib*14 and ACI440 for strengthened beams. The  
477 influence of the presence of the CFRP strengthening on the delayed onset of yield of the  
478 internal transverse steel reinforcement is shown by the increase in  $V_{fy}$  (**Table 4**) for the beams  
479 with unanchored and anchored CFRP strengthening compared to the unstrengthened control  
480 beams. Potential for interaction between steel and CFRP strains is not considered by TR55,  
481 *fib*14 or ACI440.

482

483 The principal difference between the TR55, *fib*14 and ACI440 guidance with respect to the  
484 FRP strengthening contribution, are the differing models for the determination of the  
485 effective FRP strain  $\epsilon_{fe}$ . As can be seen in **Table 6**, the effective CFRP strains predicted by  
486 TR55, *fib* 14 and ACI440 were in some cases comparable to the peak CFRP strains  $\epsilon_{fe-exp}$   
487 measured. However, at peak load these strains appear to have been limited to a width less  
488 than the  $0.6d$  link spacing. The width over which the effective strains are considered to be  
489 acting in all three models is related to the horizontal projection of the assumed  $45^\circ$  strut  
490 inclination, meaning that this width is the same as the lever arm of the idealised FRP-  
491 concrete truss adopted by each model. For all of the beams tested, the width over which the

492 effective CFRP strain is assumed to be mobilised is greater than  $0.5d$ , and in a number of  
493 cases greater than  $0.6d$ , according to TR55, *fib*14 and ACI440. This is evidence of a potential  
494 discrepancy between actual CFRP behaviour and that assumed in the guidance. It should be  
495 noted that observed crack angles for the strengthened beams were typically lower than the  
496 assumed  $45^\circ$  strut inclination for the FRP contribution, but higher than the minimum strut  
497 inclination for the unstrengthened capacity contribution given by EC2. It can also be argued  
498 that the addition of brittle CFRP material violates the assumption of ductility that is implicit  
499 in the lower-bound method of superposition of stress distributions which underpins these  
500 design approaches.

501

502

## CONCLUSIONS

503 An experimental study of unstrengthened and CFRP-strengthened reinforced concrete T-  
504 beams was undertaken to investigate the influence of the beam size, anchorage and the  
505 percentage of externally bonded U-wrap CFRP reinforcement. Based on the results, the  
506 following conclusions can be drawn:

507 • A size effect of increasing shear stress capacity with decreasing size was observed for  
508 the U-wrapped beams and for the ‘stronger’ unstrengthened beams. This size effect  
509 appears to be associated with the behaviour of the underlying reinforced concrete T-  
510 beam and is broadly compatible with the general trend predicted by fracture  
511 mechanics.

512 • The variability and significantly greater-than-predicted strength of some of the  
513 unstrengthened control beams tested indicates that more accurate assessment of  
514 existing slab-on-beam structures may obviate the need for strengthening in some  
515 cases.

- 516 • Inclinations of the critical diagonal web crack in unstrengthened control beams were  
517 observed to range from 22° to 45°. Higher shear capacities were associated with flatter  
518 critical diagonal web cracking angles and an absence of crack penetration into the  
519 flange prior to failure. Strengthened beams displayed a reduced variation in critical  
520 diagonal crack inclination, with an inclination of approximately 37° in most cases.
- 521 • Shear-deflection behaviour indicated that the CFRP U-wrap delayed the onset of  
522 significant diagonal cracking in all U-wrapped beams. Stiffer behaviour was observed  
523 in U-wrapped beams until near peak load. However, this stiffer behaviour was also  
524 associated with reduced ductility compared with unstrengthened control beams.
- 525 • As noted by others, the presence of the CFRP U-wrap delayed the strain development  
526 in the internal transverse steel reinforcement, possibly meaning that the steel had not  
527 fully yielded until after the CFRP had separated.
- 528 • The relatively small enhancement achieved by the beams with anchored EB CFRP  
529 over the stronger unstrengthened control beams indicates that the near-surface-  
530 mounted anchorage system tested may have the potential to improve CFRP  
531 effectiveness, but to a rather limited extent. This increase appears to be due to an  
532 increase in the mobilised width of the CFRP rather than the development of increased  
533 strains in the CFRP.
- 534 • Comparison of measured versus predicted effective strain levels according to current  
535 design guidelines showed that the values may be over- or under-predicted. For the  
536 beams with unanchored strengthening, the peak CFRP strains were only observed to  
537 occur over a relatively narrow width of CFRP at peak load. This width may be less  
538 than the effective width of CFRP assumed to be mobilised by the 45° truss models of  
539 TR55, *fib*14 and ACI440.

540 • The observed variation in the shear capacity of the unstrengthened control beams was  
541 significant in comparison to the magnitude of the enhancement expected from the  
542 CFRP strengthening, raising questions as to the appropriateness of the widely-adopted  
543 experimental approach for determining the experimental ‘FRP contribution’ on the  
544 basis of the tested strength of a single control beam.

545

546 The Authors recognise that the experimental finding that some strengthened beams achieved  
547 lower shear capacities than some of their respective control specimens is unusual, although a  
548 small number of similar results have been presented previously in the literature by Deniaud &  
549 Cheng (2001) and Murphy et al. (2012). Deniaud and Cheng (2001) attribute their result to a  
550 sliding shear failure along the dominant diagonal crack. Murphy et al. (2012) attribute their  
551 result to the reduction of effective web cross-sectional area due to cover concrete separation  
552 with the FRP.

553

554 The reduction in cover concrete due to the separation of the EB CFRP that was observed in  
555 the strengthened beams of this test series is likely to have played a role in the reduction of  
556 shear capacity of the strengthened beams relative to that of the stronger control beams. This  
557 cover separation may have been exacerbated by the particular pattern of web cracking  
558 behaviour observed in this test series. Diagonal cracking in the unstrengthened control beams  
559 was observed to initiate in approximately the middle third of the shear span and the middle  
560 third of the height of the beam web and propagate towards the support and loading platens.  
561 Indirect observation indicated that diagonal cracking in the strengthened beams may have  
562 also initiated at approximately this position. This suggests that diagonal cracking that initiates  
563 in the strengthened web of the beam, rather than as the more commonly observed rotating  
564 extension of flexural cracks initiated from the web soffit, may provide an adverse condition



565 for EB FRP strengthening. While this condition may be somewhat particular to the tested  
566 beam arrangement which had a high longitudinal reinforcement ratio, a low transverse  
567 reinforcement ratio with plain mild steel bars, a relatively high concrete strength and a  
568 reinforced flange; the implications of these results for the design of EB FRP shear  
569 strengthening in general should be considered.

570

571

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578 Europe S.A., Fibrwrap UK, Hughes Brothers and Ebor Concrete Ltd.

579

580 Additional data related to this publication is available at the University of Bath and the  
581 University of Cambridge’s institutional data repositories: [*permanent links will be added*  
582 *following review*].

583

584

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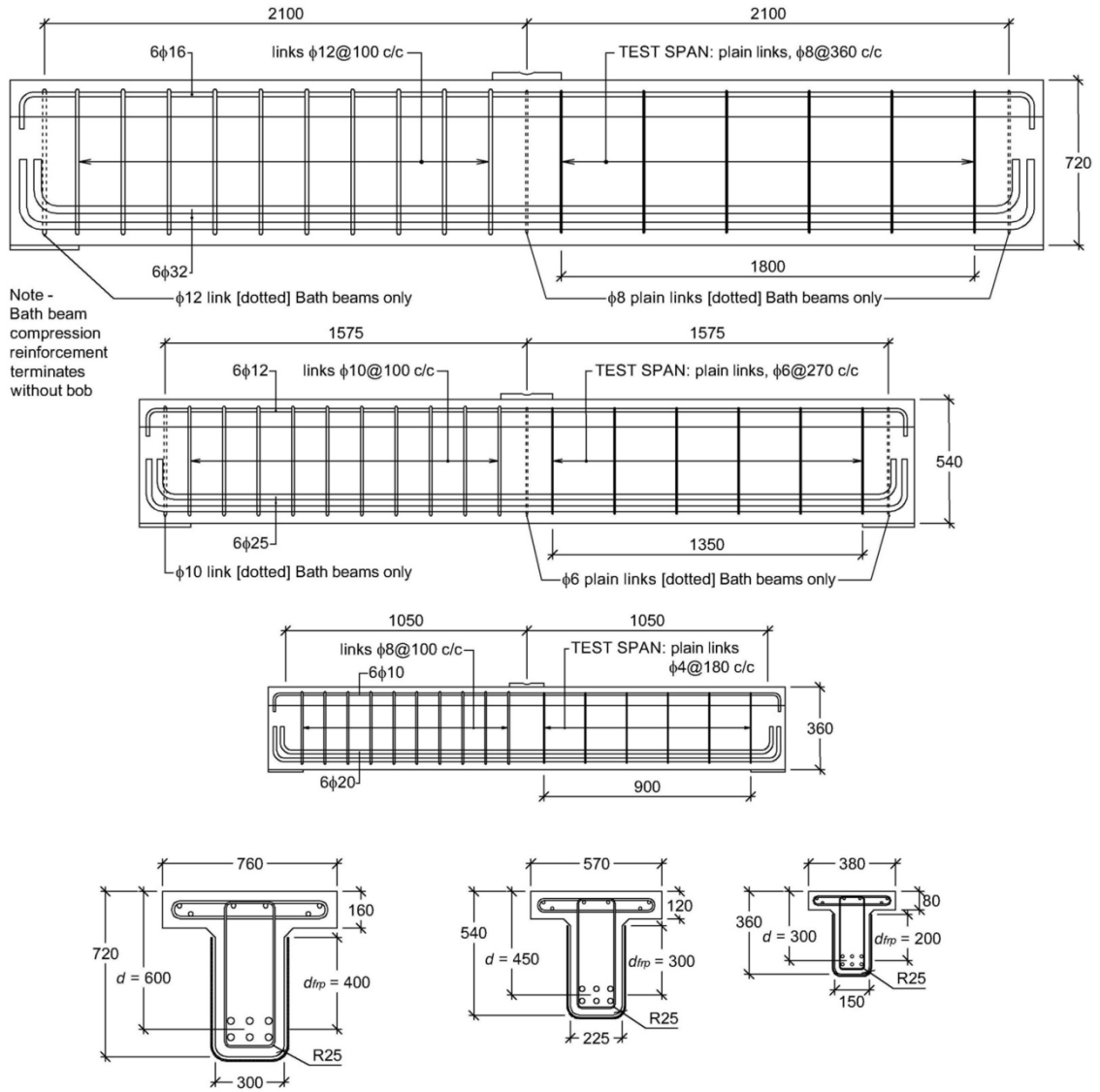
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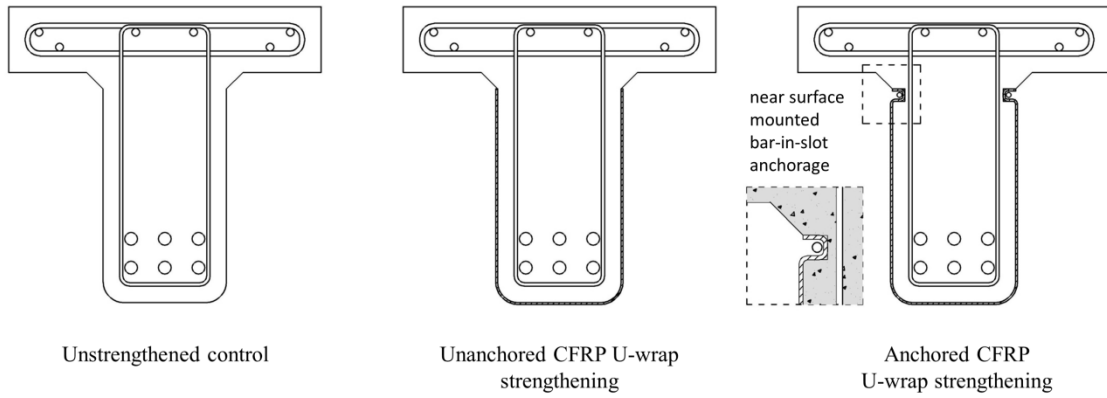


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652

653

**Fig. 1. Test specimens [mm]**

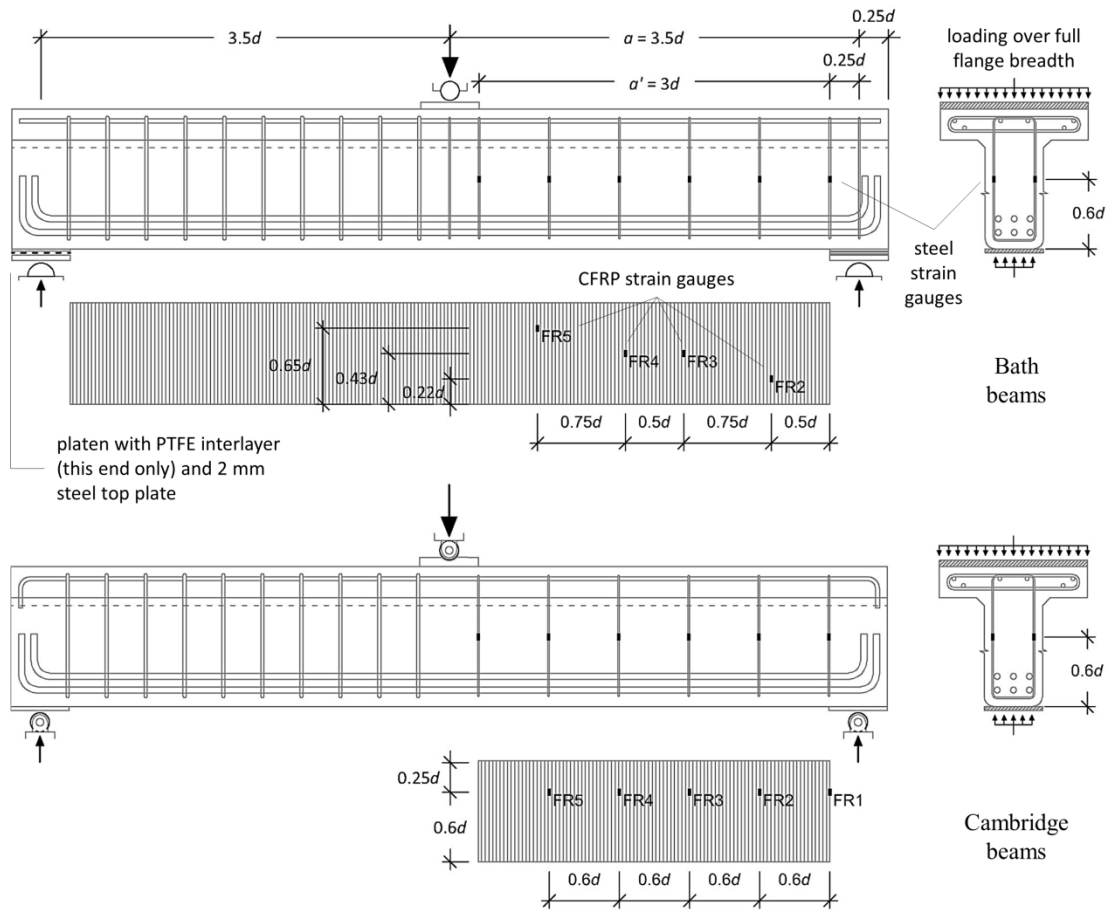


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655

**Fig. 2.** CFRP strengthening arrangements

656

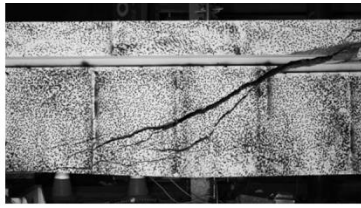


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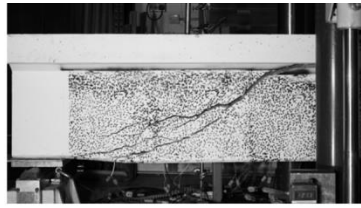
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**Fig. 3.** Loading and support conditions, and strain gauge layout

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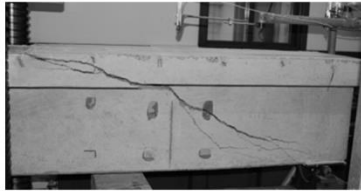
LBC ( $\beta = 22^\circ$ ,  $V_u = 472$  kN)



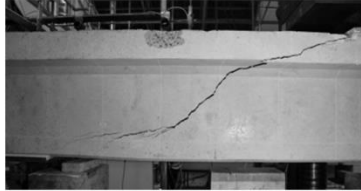
MBC ( $\beta = 22^\circ$ ,  $V_u = 322$  kN)



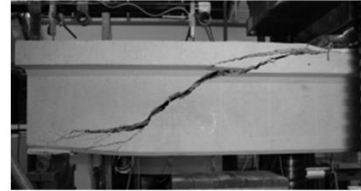
SCC<sub>1</sub> ( $\beta = 23^\circ$ ,  $V_u = 195$  kN)



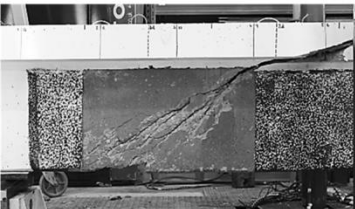
MCC<sub>1</sub> ( $\beta = 31^\circ$ ,  $V_u = 250$  kN)



MCC<sub>2</sub> ( $\beta = 45^\circ$ ,  $V_u = 225$  kN)



SCC<sub>2</sub> ( $\beta = 45^\circ$ ,  $V_u = 89$  kN)



LB0.7U ( $\beta = 37^\circ$ ,  $V_u = 458$  kN)



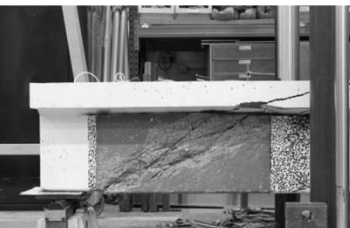
MC0.9U ( $\beta = 27^\circ$ ,  $V_u = 299$  kN)



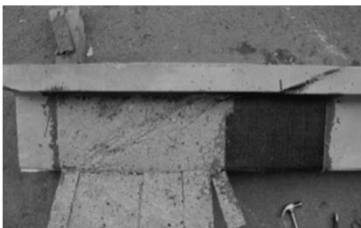
SC0.7U ( $\beta = 35^\circ$ ,  $V_u = 166$  kN)



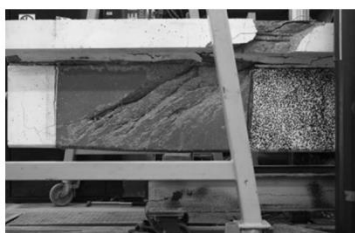
LB1.3U ( $\beta = 37^\circ$ ,  $V_u = 437$  kN)



MB1.3U ( $\beta = 37^\circ$ ,  $V_u = 306$  kN)



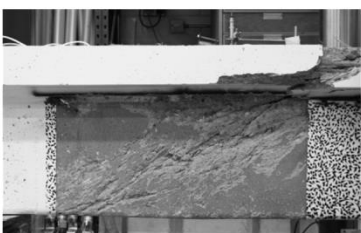
SC1.3U ( $\beta = 42^\circ$ ,  $V_u = 153$  kN)



LB0.7UA ( $\beta = 37^\circ$ ,  $V_u = 512$  kN)



LB1.3UA ( $\beta = 37^\circ$ ,  $V_u = 511$  kN)



MB1.3UA ( $\beta = 37^\circ$ ,  $V_u = 370$  kN)

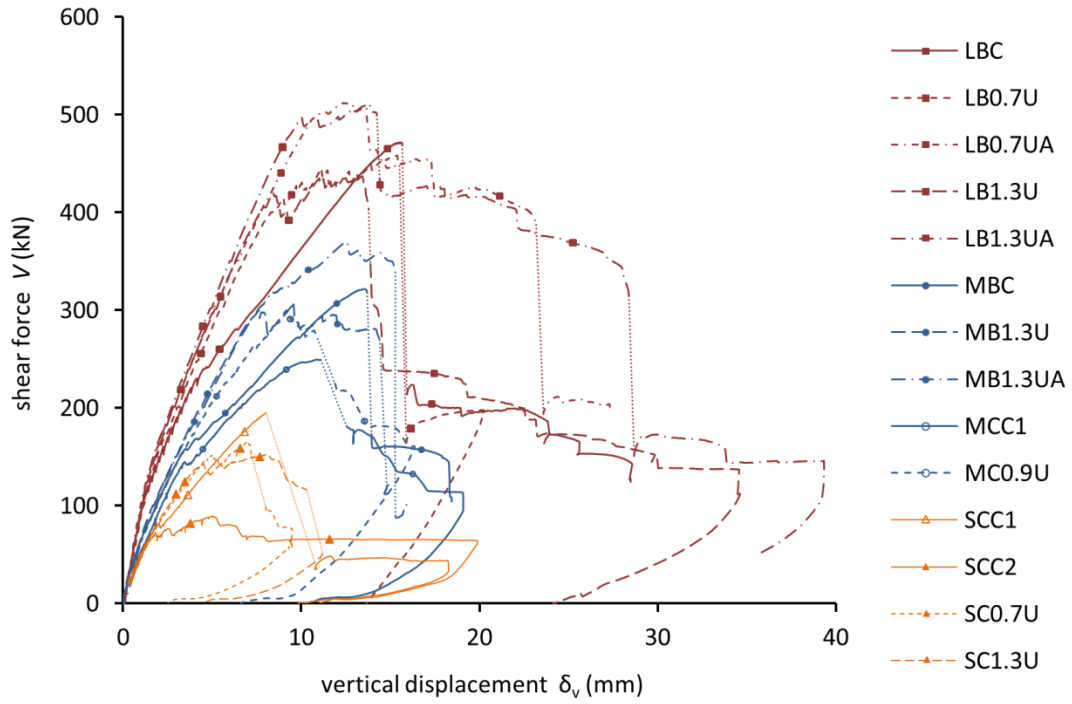
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**Fig. 4.** Failure modes, showing critical web shear crack angles  $\beta$  and peak shear  $V_u$ .



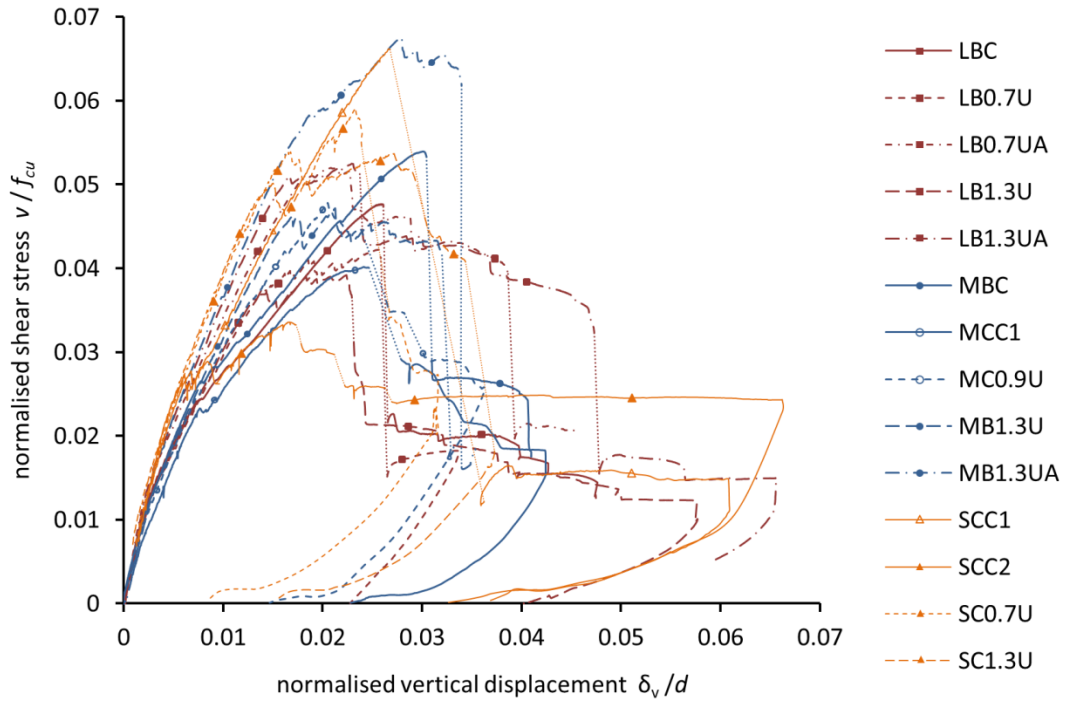


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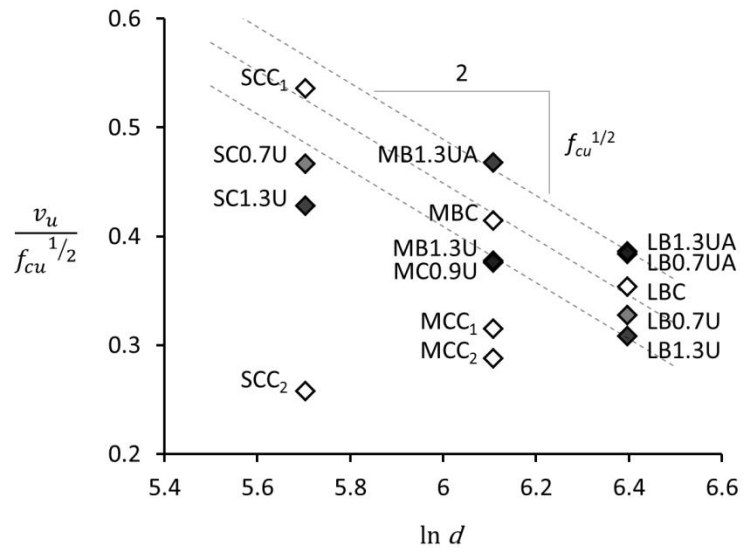
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**Fig. 5.** Shear deflection behaviour for small, medium and large beams with and without EB CFRP

665



**Fig. 6.** shear stress  $v$  normalised by  $f_{cu}$  plotted against  $\delta_v$ , normalised by  $d$

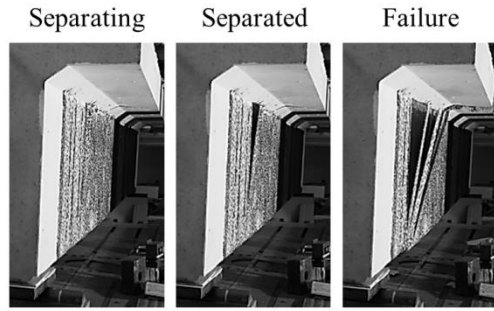


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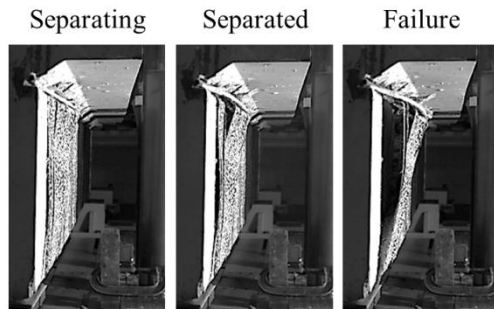
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**Fig. 7.** Peak shear stress  $v_u$  normalised by  $f_{cu}^{1/2}$  plotted against  $\ln d$

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(a) Unanchored U-wrap LB1.3U



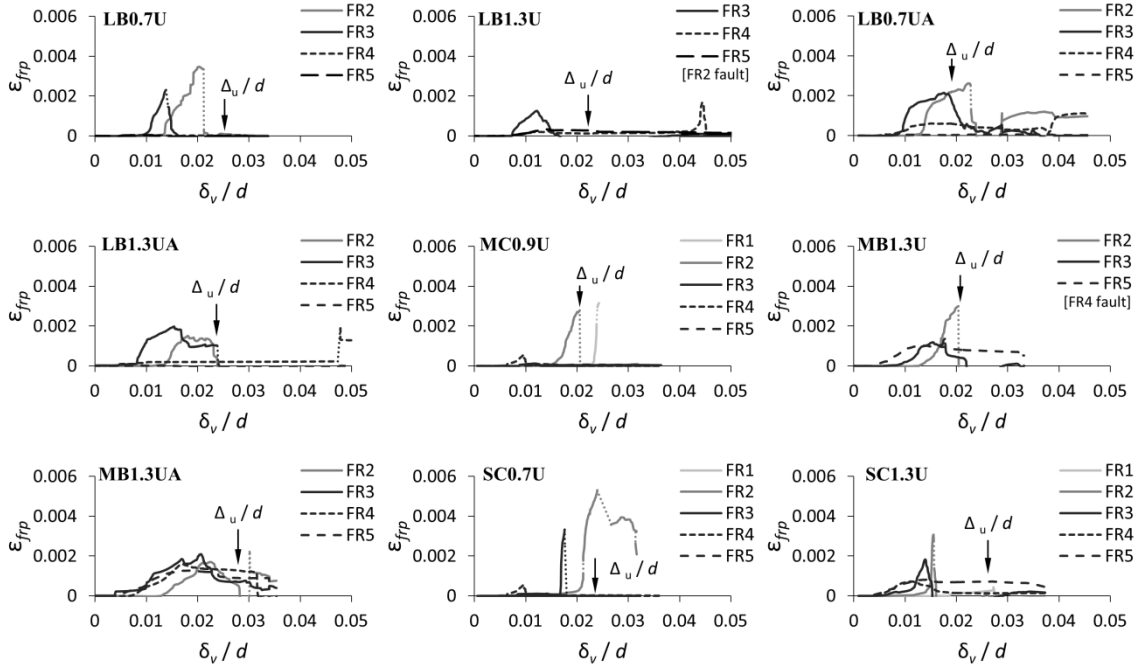
(b) Anchored U-wrap LB1.3UA

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**Fig. 8.** Progressive separation of the U-wrapped CFRP



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676 **Fig. 9.** CFRP strains measured in strengthened beams. FR5 gauges positioned closest to the central support and

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FR1 gauges closest to the end support. A detailed strain gauge layout is shown in **Fig. 3.**

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**Table 1.** Test matrix

beam	concrete	steel		CFRP		
	$f_{cu}$	$\rho_{sl}$	$\rho_{sv}$	$t_{frp}$	$\rho_{frp}$	anchor bar diameter
	MPa	%	%	mm	%	mm
LBC	55.0	2.2	0.1	–	–	–
LB0.7U	60.3	2.2	0.1	0.5 + 0.5	0.7	–
LB0.7UA	55.0	2.2	0.1	0.5 + 0.5	0.7	13
LB1.3U	62.0	2.2	0.1	1.0 + 1.0	1.3	–
LB1.3UA	54.1	2.2	0.1	1.0 + 1.0	1.3	13
MBC	58.9	2.4	0.1	–	–	–
MCC <sub>1</sub>	61.4	2.4	0.1	–	–	–
MCC <sub>2</sub>	59.7	2.4	0.1	–	–	–
MC0.9U	61.7	2.4	0.1	1.0	0.9	–
MB1.3U	64.1	2.4	0.1	1.0 + 0.5	1.3	–
MB1.3UA	61.1	2.4	0.1	1.0 + 0.5	1.3	10
SCC <sub>1</sub>	65.4	3.5	0.1	–	–	–
SCC <sub>2</sub>	59.0	3.5	0.1	–	–	–
SC0.7U	62.5	3.5	0.1	0.5	0.7	–
SC1.3U	63.2	3.5	0.1	0.5 + 0.5	1.3	–

**Table 2.** Steel properties

beams	bar diameter $d_b$ [mm]	steel grade	bar type	yield strength $f_y$ [MPa]	tensile strength $f_u$ [MPa]
Large	32	B500C	Deformed	510 <sup>a</sup>	587 <sup>a</sup>
	16	B500C	Deformed	538 <sup>a</sup>	631 <sup>a</sup>
	12	B500C	Deformed	518 <sup>a</sup>	586 <sup>a</sup>
	8	S275	Plain	336 <sup>a</sup>	438 <sup>a</sup>
Medium Bath	25	B500C	Deformed	554 <sup>a</sup>	667 <sup>a</sup>
	12	B500C	Deformed	518 <sup>a</sup>	586 <sup>a</sup>
	10	B500C	Deformed	538 <sup>a</sup>	625 <sup>a</sup>
	6	S275	Plain	434 <sup>a</sup>	536 <sup>a</sup>
Medium Cambridge	25, 12, 10	B500C	Deformed	500 <sup>b</sup>	≥ 575 <sup>b</sup>
	6	S275	Plain	570 <sup>a</sup>	637 <sup>a</sup>
Small	20, 10, 8	B500C	Deformed	500 <sup>b</sup>	≥ 575 <sup>b</sup>
	4	S275	Plain	465 <sup>a</sup>	514 <sup>a</sup>

<sup>a</sup> Average values from direct tensile testing

<sup>b</sup> Characteristic values in accordance with BS 4449: 2005

**Table 3.** CFRP composite and constituent properties

	$E_{f/p}$ MPa	$f_u$ MPa	$\epsilon_u$ %
Epoxy	3180	72	5.0
644 g/m <sup>2</sup> fabric	230000	3790	1.7
393 g/m <sup>2</sup> fabric	230000	3790	1.7
644 g/m <sup>2</sup> fabric – composite <sup>a</sup>	95800	986	1.0
393 g/m <sup>2</sup> fabric – composite <sup>b</sup>	105400	986	1.0
13 mm diameter bar	124000	2068	1.7
10 mm diameter bar	124000	2172	1.7

<sup>a</sup> nominal thickness per layer 1.00 mm

<sup>b</sup> nominal thickness per layer 0.51 mm



**Table 4.** Summary of test results

Beams	$V_u$ [kN]	$v_u$ [MPa]	$\Delta_u$ [mm]	$\Delta_{e,u}$ [mm]	$\mu_\Delta$	$V_{fy}$ [kN]	$\Delta_{fy}$ [mm]	Failure mode
LBC	472	2.6	15.6	10.5	1.5	241	4.8	very brittle shear
LB0.7U	458	2.5	15.4	10.9	1.5	409	9.0	fabric separation/shear
LB0.7UA	512	2.8	11.8	10.8	1.1	480	9.5	fabric separation/shear
LB1.3U	437	2.4	13.4	9.8	1.5	396	7.8	fabric separation/shear
LB1.3UA	511	2.8	13.7	10.3	1.3	496	10.7	fabric separation/shear
MBC	322	3.2	13.6	8.6	1.6	163	3.8	very brittle shear
MCC <sub>1</sub>	250	2.5	10.9	6.0	1.8	159	4.5	brittle shear
MCC <sub>2</sub>	225	2.2	--	--	--	--	--	brittle shear
MC0.9U	299	2.9	9.2	8.3	1.1	266	7.7	fabric separation/shear
MB1.3U	306	3.0	9.6	8.1	1.2	278	6.7	fabric separation/shear
MB1.3UA	370	3.7	12.6	9.7	1.3	305	7.8	fabric separation/shear
SCC <sub>1</sub>	195	4.3	8.0	5.0	1.6	98	3.1	very brittle shear
SCC <sub>2</sub>	89	2.0	5.0	2.2	2.3	68	2.2	shear
SC0.7U	166	3.7	7.0	5.5	1.3	151	5.1	fabric separation/shear
SC1.3U	153	3.4	8.2	5.9	1.4	139	4.6	fabric separation/shear

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**Table 5.** Comparison of tested shear strength  $V_u$  with the values predicted by design guidance. Explicit design safety factors set equal to 1.

Beam	Experimental		Predicted									
	$\rho_{fp}$	$V_u$	EC2		TR55		fib 14		ACI318		ACI440	
			$V_{EC2}$	$V_{EC2} / V_u$	$V_{TR55}$	$V_{TR55} / V_u$	$V_{fib14}$	$V_{fib14} / V_u$	$V_{ACI318}$	$V_{ACI318} / V_u$	$V_{ACI440}$	$V_{ACI440} / V_u$
[%]	[kN]	[kN]		[kN]		[kN]		[kN]		[kN]		
LBC	-	472	126	0.27	-	-	-	-	254	0.54	-	-
LB0.7U	0.7	458	-	-	322	0.70	494	1.08	-	-	563	1.23
LB0.7UA	0.7	512	-	-	351	0.69	481	0.92	-	-	536	1.05
LB1.3U	1.3	437	-	-	394	0.90	630	1.44	-	-	684	1.57
LB1.3UA	1.3	511	-	-	398	0.78	605	1.18	-	-	634	1.24
MBC	-	322	93	0.29	-	-	-	-	157	0.53	-	-
MCC <sub>1</sub>	-	250	122	0.49	-	-	-	-	172	0.53	-	-
MCC <sub>2</sub>	-	225	122	0.54	-	-	-	-	175	0.78	-	-
MC0.9U	0.9	299	-	-	247	0.83	359	1.20	-	-	368	1.23
MB1.3U	1.3	306	-	-	241	0.79	380	1.24	-	-	405	1.32
MB1.3UA	1.3	370	-	-	250	0.68	375	1.01	-	-	394	1.06
SCC <sub>1</sub>	-	195	44	0.23	-	-	-	-	73	0.37	-	-
SCC <sub>2</sub>	-	89	44	0.49	-	-	-	-	71	0.80	-	-
SC0.7U	0.7	166	-	-	102	0.61	137	0.83	-	-	149	0.90
SC1.3U	1.3	153	-	-	120	0.78	171	1.12	-	-	198	1.29

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690 **Table 6.** Comparison of maximum CFRP strains measured during testing  $\epsilon_{fe-exp}$  prior to peak load, with those  
 691 predicted by design guidance. Predictions based on measured concrete strengths with explicit design safety  
 692 factors set equal to 1.

Beams	Experimental		Predicted					
	$\rho_{frp}$ [%]	$\epsilon_{fe-exp}$	TR55		<i>fib</i> 14		ACI440	
			$\epsilon_{fe-TR55}$	$\epsilon_{fe-TR55} / \epsilon_{fe-exp}$	$\epsilon_{fe-fib14}$	$\epsilon_{fe-fib14} / \epsilon_{fe-exp}$	$\epsilon_{fe-ACI440}$	$\epsilon_{fe-ACI440} / \epsilon_{fe-exp}$
LB0.7U	0.7	0.0023	0.0027	1.16	0.0036	1.54	0.0035	1.50
LB0.7UA <sup>a</sup>	0.7	0.0023	0.0026	1.13	0.0034	1.48	0.0033	1.27
LB1.3U	1.3	0.0013	0.0019	1.46	0.0024	1.85	0.0024	1.85
LB1.3UA <sup>a</sup>	1.3	0.0020	0.0018	0.90	0.0023	1.15	0.0022	1.10
MC0.9U	0.9	0.0028	0.0027	0.98	0.0031	1.10	0.0034	1.22
MB1.3U	1.3	0.0030	0.0021	0.70	0.0025	0.83	0.0028	0.93
MB1.3UA <sup>a</sup>	1.3	0.0021	0.0022	1.05	0.0024	1.14	0.0027	1.29
SC0.7U	0.7	0.0048	0.0039	0.81	0.0036	0.76	0.0040	0.84
SC1.3U	1.3	0.0031	0.0026	0.84	0.0025	0.80	0.0033	1.06

<sup>a</sup> Predictions do not assume additional anchorage due to near-surface-mounted bar-in-slot system.

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