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5 **RESPONSE OF BEAM-TO-COLUMN WEB CLEATED JOINTS FOR FRP PULTRUDED**
6 **MEMBERS**

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18 **ABSTRACT**

19 Physical testing is used to characterise the structural properties of beam-to-column joints, comprising
20 pultruded Fibre Reinforced Polymer (FRP) H-shapes of depth 203 mm, connected by 128 mm long
21 web cleats and two M16 bolts per leg. Testing is performed on two batches of nominally identical
22 specimens. One batch had web cleats of pultruded FRP and other had structural steel. The structural
23 behaviour of the joints is based on their moment-rotation responses, failure modes, and serviceability
24 vertical deflection limits. Joints with FRP cleats failed by delamination cracking at top of cleats, and
25 when cleats were of steel the FRP failure occurred inside the column members. Neither failure mode
26 is reported in the design manuals from pultruders. At the onset of FRP damage it was found that the
27 steel joints were twice as stiff as the FRP joints. Based on a characteristic (damage) rotation,
28 calculated in accordance with Eurocode 0, the serviceability deflection limits are established to be
29 span/300 and span/650 for the joints with FRP and steel cleats, respectively. This finding suggests
30 that appropriate deflection limits, in relation to cleated connections, should be proposed in
31 manufactures' design manuals and relative design standards and design codes. Failure to address the
32 serviceability, by the Engineer of Record could lead to unreliable designs.

33
34 **Keywords:** Web cleats; pultruded joints; damage onset; moment-rotation response; deflection limit.

35 INTRODUCTION

36 The traditional structural materials of stone, timber, steel and concrete have historical presence in
37 construction. Although steel and reinforced concrete have emerged to be the leading materials it is
38 recognized that when exposed to a chemically aggressive environment they are both susceptible to
39 degradation and deterioration over time. Construction is responsible, in 2012, for almost a third of the
40 global carbon emissions. In order to minimise the ecological impact on the built environment, there is
41 a need to promote and develop the use of structural materials with a sustainable credibility. Fibre
42 Reinforced Polymer (FRP) is such a construction material possessing high strength, lightweight,
43 improved chemical and corrosion resistance, and of equal importance, a low (ecological) impact
44 (Daniel 2003). FRP is a two-part composite material (Bank 2006) comprising of high strength (often
45 continuous) fibres embedded in a lower strength polymer based matrix. Members of FRP have been
46 used in primary structural engineering applications for more than two decades (Bank 2006). Due to
47 quicker installation and an expected durable performance, FRP can be the cost-effective structural
48 material in applications such as, cooling towers, chemical plants and railway footbridges. However, a
49 major hurdle to the wider usage of FRP components is a lack of recognised and verified structural
50 design guidance.

51

52 Pultrusion is the cheapest composite manufacturing process for the continuous production of FRP
53 thin-walled shapes. One category of pultruded profiles possess the same cross-sectional shapes (I, H,
54 Leg-angle, channel, box, etc.) as found in structural steelwork, but standard profiles of FRP have
55 very different mechanical and structural properties (Bank 2006). They consist of E-glass fibre
56 reinforcement having layers of unidirectional rovings and continuous mats in a thermoset resin based
57 matrix, usually having the polymer of polyester or vinylester. Having a weight of only 25% of steel
58 FRP materials are lightweight. Like steel, the tensile strength in the longitudinal direction is more
59 than 200 MPa. The longitudinal modulus of elasticity lies in the range 20-30 GPa, which is 10-6
60 times lower than steel. The elastic modulus in the transverse direction is 0.3 of the longitudinal value
61 (Anonymous 2013a; 2013b; 2013c).

62

63 It is recognized that as much as 50% of the cost of executing frame structures can be for the
64 fabrication of connections and joints. Current practice is to construct pultruded FRP frames that are
65 of simple (non-swayed braced) construction. Simple joint details are expected to behave as nominally
66 pinned when subjected to moment. They must be capable of transmitting internal forces without
67 developing significant moments. Furthermore, they need to rotate sufficiently to meet the severability
68 vertical deflection limits for the simply supported beam subjected to a uniformly distributed load.
69 Joint details commonly have web cleats (or clip angles) that connect the beam and column members
70 with conventional steel bolting. Information found in the design manuals from two American
71 pultruders (Anonymous 2013a; 2013b) are for the web cleats to be fabricated from pultruded FRP
72 equal leg-angle. Design strengths are based on a (relatively) high factor of safety of 4 in an
73 Allowable Stress Design (ASD) approach (Anonymous 2013a; 2013b). Because there are concerns
74 (Mosallam 2011) that the fibre architecture in FRP cleats is inappropriate to resist prying action
75 deformations an alternative material for cleating can be of structural steel.

76

77 The moment-rotation responses and properties of joints with pultruded members is characterised
78 through full-sized physical testing (Bank *et al.* 1990; Bank *et al.* 1992; Bass and Mottram 1994;
79 Mosallam *et al.* 1994; Qureshi and Mottram 2012), because theoretical and numerical methods
80 cannot reliably analyse the initiation and progression of FRP material damage. Turvey and Cooper
81 (2004) presented a review of 59 individual joint tests, out of which only two pairs of specimens had
82 nominally identical joint details. Reported test results from the 1990s were therefore based on a batch
83 with a single specimen. Due to lack of specimen repetition, the variability in a joint's rotational
84 stiffness could not be statistically quantified to establish a characteristic value for design. Turvey
85 (1997) developed an analytical treatment to utilise the inherent non-zero rotational stiffness of
86 (simple) joints to quantify the increase in load carrying capacity of beam members. Utilizing the
87 semi-rigid joint action he formulated closed-form equations for calculating vertical deflection that
88 were functions of the joint's initial rotational stiffness (S_i). Inserting into these equations a value of S_i

89 established from too few test results is going to be unreliable. To characterise the key joint properties
90 for their variability it is necessary to conduct tests on batches with more nominally identical joints.
91 One of the objectives of this paper is to report test results from two batches that can be statistically
92 analysed to obtain information that can be used to prepare improved design guidelines for simple
93 construction.

94

95 The moment-rotation ($M-\phi$) response of beam-to-column joints with pultruded FRP web cleats have
96 been investigated in previous studies. Bank, Mosallam and Gonsior (1990) were first to report
97 experimental test results. They characterised one single-sided joint using 203×203×9.53 mm
98 members and cleats (without dimensions) cut from a 152×152×12.7 mm leg-angle. At mid-depth of
99 the double-sided cleating there was a single row of two 19 mm diameter FRP bolts. Mottram (1996)
100 presented $M-\phi$ results from four double-sided joint tests (three of major axis and one for minor axis
101 configurations) in an appendix to the EUROCOMP Design Code and Handbook. Two key findings
102 from his work, using the same research methodology as for the test results reported in this paper,
103 were that adhesive bonding cannot be used on its own, and there needs to be a gap of 6-12 mm
104 between a beam-end and column face to accommodate ‘free’ rotation between the connected
105 members. Two major and one minor axis joint test with leg-angle cleats and steel bolting and 254
106 mm deep members were conducted by Mottram and Zheng (1999a). The aim of this test series was to
107 confirm the design guidance in the EUROCOMP appendix (Mottram 1996). A major concern of
108 using cleats of FRP material was that the onset of delamination failure (Bank 2006) at the top of the
109 cleating could occur before the simply supported beam achieves the serviceability vertical deflection
110 limit of span/250, taken from EUROCOMP (Clarke 1996). Because many FRP structures are
111 constructed for a chemically hostile environment, delamination fractures initiating under
112 serviceability loading could have a serious detrimental effect on the service life. For this reason
113 Mottram and Zheng (1999a) and Mosallam (2011) both recommended using other composite
114 manufacturing processes to manufacture FRP connection components that should, without FRP
115 failure, accommodate joint rotations in excess of 25 mrad.

116

117 Owing to the uncertainty of having cleats of FRP it is known that fabricators can prefer steel for the
118 connection components. Pultruders provide no design guidelines (Anonymous 2013a; 2013b; 2013c)
119 when the cleating is of steel, and to establish their joint properties there are few test results too.
120 Mottram and Zheng (1999b) carried out two one-off tests for flange-cleated steel joints for study on
121 semi-rigid action. Turvey (2000) test series was with specimens having web, flange and web, and
122 flange only cleats of steel leg-angles. A shortcoming in the work by Turvey (2000) is that the beam
123 was connected directly to a relatively stiff steel support that (completely) eliminated the flexibility of
124 the pultruded FRP column; which is part of the joint zone (BS EN 1993-1-8:2005). Because of the
125 specific test configuration the measured joint stiffness would be too high. To reliably quantify joint
126 properties, it is essential to take into account the flexibility of the pultruded column. Characterisation
127 of a joint's properties using the test configuration and method in Mottram and Zheng (1999a; 1999b)
128 represents the construction of pultruded frames when there are no seismic actions.

129

130 The main objective of this paper is to study the $M-\phi$ responses of nominally pinned joints focusing on
131 two key test parameters. The first of these parameters is specimen repetition and the second is to have
132 web cleat material of either FRP or steel. One test batch will consist of five specimens having 10
133 joints and FRP cleating, and the second batch will have three specimens for six joints with steel
134 cleats. Using the batch results there will be a discussion on joint properties, moment-rotation
135 responses, failure modes, damage onset criteria and vertical deflection limit for Serviceability Limit
136 State (SLS) design. Finally, an important insight towards the preparation of design guidelines is
137 gained from an evaluation of the findings.

138

139 **TEST CONFIGURATION AND TEST PROCEDURE**

140 Figs. 1-4 illustrate the test configuration consisting of two back-to-back cantilever beams connected
141 to a central column. A pair of web cleats and steel bolts is used to connect each beam to the major-
142 axis of the column. The web cleat material is either pultruded FRP or structural grade steel. A joint is

143 defined as the zone where two or more members are interconnected. For design purposes (BS EN
144 1993-1-8:2005) it is the assembly of all the basic components required to represent the behaviour
145 during the transfer of the relevant internal forces and moments between the connected members. A
146 beam-to-column joint consists of a web panel, from the column side, and either one connection
147 (single sided joint configuration) or two connections (double sided joint configuration). The latter
148 configuration is for the test configuration in Figs. 1-4 and so the joint moment (M) is to be
149 determined at the column's centroidal axis.

150

151 Each test specimen gives two joints, called the Left and the Right joint. Similar test arrangement has
152 previously been used by Qureshi and Mottram (2012) and Mottram and Zheng (1999a; 1999b). The
153 beams and columns are 1.5 m long and are of size 203×203×9.53 mm from the Pultex®
154 SuperStructural 1525 series of Creative Pultrusions Inc (Pultex® pultrusion design manual 2013). From
155 this pultruder's Design Manual (Anonymous 2013a) the shape's flexural strength is 228 MPa and the
156 second moment of area about the Major axis is $4.18 \times 10^7 \text{ mm}^4$. Based on conventional linear elastic
157 beam theory the flexural moment of resistance for the section could be 94 kNm. For a laterally
158 unrestrained beam the ULS mode of failure is likely to be local flange buckling. A lower bound
159 estimate for the uniform compression stress for critical elastic local buckling can calculated from (is
160 Equ. (6) in Mottram (2004a)):

161

$$\sigma_{c,cr} = \frac{G_{LT}}{\left(\frac{b}{2t_f}\right)^2} \quad (1)$$

162 In Equ. (1) G_{LT} is the in-plane shear modulus of the flange material, taken to be 4.0 GPa, b is the
163 flange width of 203 mm and t_f is the flange thickness of 9.53 mm. The critical local buckling stress
164 ($\sigma_{c,cr}$) is 35 MPa and using beam theory, again, the moment resistance of the section for local
165 buckling failure is 14.5 kNm.

166

167 Standard size leg-angles are used to fabricate the web cleats, with the FRP angle at 75×75×9.53 mm
168 and the steel at 75×75×10 mm. The cleats are 128 mm long (Fig. 2) for the 203 mm deep beam
169 member.

170

171 The 10 joints with pultruded FRP cleats are denoted by label Wmj203_2M16_FC and the six with
172 steel cleats by Wmj203_2M16_ST. This joint labelling convention continues from that used by
173 Qureshi and Mottram (2012) and Mottram and Zheng (1999a). Label Wmj203_2M16_FC specifies
174 the joint as *Web-cleated* with a *major axis column*, 203×203×9.53 mm wide flange sections using a
175 single row of 2 *M16* bolts with pultruded *FRP web Cleating*. Similarly, the label Wmj203_2M16_ST
176 is used for the batch with *STeel* cleats.

177

178 **Connection detailing**

179 Fig. 2 shows a web cleated joint that corresponds to Detail 2 illustrated on Page 19-6 of the Strongwell
180 Design Manual (Anonymous 2013b). This detailing satisfies the minimum requirements for bolted
181 connection geometries as permitted in a standard under preparation (Anonymous 2013d). The
182 detailing in the drawing has steel bolting and the provision of a 10 mm gap between the beam end and
183 column flange. The gap, bolting, etc., in the Wmj203_2M16_FC and Wmj203_2M16_ST joint
184 specimens are presented in Figs. 1-4.

185

186 Bolting has steel bolts of M16 grade 8.8 and 3 mm thick by 35 mm diameter steel washers. The length
187 of the bolt shank in contact with FRP is plain to avoid any localised FRP failure due to bolt thread
188 bearing stresses. In order to bring connected FRP panels into firm contact the bolts are tightened to
189 the snug fit condition, which is achieved when the bolt or nut will not turn any further with the full
190 effort of a construction worker using a standard hand wrench (Gorenc *et al.* 2005). Firm contact is
191 defined as “the condition that exists on a faying surface when the plies are solidly seated against each
192 other, but not necessarily in continuous contact” (Anonymous 2000). One important feature in these
193 tests is that clearance hole size is kept minimal (on beam side) to ensure that joint rotation (ϕ) is

194 dominated by prying action from the applied M (Qureshi and Mottram 2012). To achieve this test
195 condition, precision holes of 16 mm diameters were drilled into the web cleats, and beams and
196 column members using a CNC machine with a geometric tolerance ± 0.1 mm. Bolt clearance hole
197 could not be eliminated altogether because ‘off the shelf’ M16 bolts have a diameter in the range of
198 15.6 to 15.9 mm.

199

200 The approach to bolt tightening used follows the guidance in Anonymous (2011). It also corresponds
201 to the description of what is ‘snug-tight’ in the well-known monograph for steel structures by Kulak
202 *et al.* (1987). The main reason for not using calibrated torque wrench is that the bolt torque will lie in
203 the range $\pm 30\%$ of a mean value (Kulak *et al.* 1987). A second reason is that to ensure the same
204 (initial) clamping pressure in the bolted connection with changes in FRP material, FRP thicknesses,
205 bolt material, bolt sizes (diameter and pitch), washer type, etc, would require an extensive list of
206 specified bolt torques. This is not realistic for practice. Another important reason for not needing to
207 use a calibrated torque wrench is that FRP is a viscoelastic material, and as shown by Mottram
208 (2004b), the bolt tension will disappear (exponentially) with time, and might be reduced to half by
209 the end of a structure’s service life. At the time of testing the frictional force that exists between the
210 connected FRP panels cannot therefore be known with certainty. Moreover, the test results, after
211 compensation for ‘secondary’ slippage, will not change if bolt tightening is lower or higher. It is
212 important to appreciate that the purpose of the research reported herein is to establish the onset of
213 damage in the FRP web cleats or members when the joint assembly gives the stiffest $M-\phi$ response
214 that could exist.

215

216 Although the additional ϕ due to slippage (from having clearance holes) will be beneficial in the field
217 (Anonymous, 2013a; Anonymous 2013b), it cannot be guaranteed for the reason now explained. The
218 magnitude of slip rotation depends on where the bolts are placed in their holes. There could be
219 assemblies where bolting is positioned in such a way that no slip can occur before the joint
220 experiences its ultimate moment of resistance, which is defined by the maximum joint moment, M_{\max} .

221 This worst case in the field was the justification for the slip rotation to be eliminated in the testing.
222 To minimise the contribution to joint rotation from slippage the clearance hole size was made
223 minimal for the beam side connections. For ease in assembling there is a clearance hole of 2 mm to
224 the bolting on the column side. The presence of clearance in the column connections does not
225 influence overall joint rotations.

226

227 **Loading Procedure**

228 As seen in Figs. 1 and 3 loading is applied, at a horizontal distance of 1.016 m from the centre of the
229 column, into the two beams by means of a hanger assembly. This moment lever arm distance is
230 controlled by the layout of the anchor points on a strong floor, which are 408 mm (16 in.) apart
231 (Mottram and Zheng 1999a). To ensure vertical alignment of the load it is transferred through a steel
232 ball bearing, of 12.7 mm diameter, located in a hemi-spherical steel socket at the centre of the two
233 steel loading plates. For the Left and Right joints the applied load is measured through tension load
234 cells having a capacity of 9 kN with a resolution of ± 0.01 kN. A rocker base fixture is used
235 underneath the column member to alleviate effects of flexure, and to accommodate free in-plane
236 rotations. Two independent manual hydraulic pumps are used to operate the two tension jacks. It is
237 operationally difficult to guarantee equal pressure (load) to the Left and Right sides. Even if the
238 applied load is not equal, the rocker base fixture at the bottom of the column ensures the same joint
239 moment (M) on both sides. Fig. 1 shows the longitudinal centreline of the two beams is at a vertical
240 distance of 1094 mm from the base of the column. This distance is dictated by the height of hydraulic
241 tension jacks and is enough to allow a downward stroke of 150 mm on the jacks.

242

243 The specimens are loaded under load control in increments of 0.1 kN. For visual inspection of the
244 joint, a time interval of 5 minutes is maintained throughout the loading regime. This time gap is
245 essential to observe any cracking and progressive damage. Load, rotation and displacement readings
246 are taken instantly after load is applied and after a time lapse of 5 minutes. The loading increments
247 are continued until rotation increases rapidly without a corresponding increase in M or when further

248 loading would cause instability of the specimen. To observe permanent rotations, the specimens were
249 loaded and unloaded after overall rotations of about 10, 20 and 30 mrad.

250

251 **Instrumentation**

252 Joint properties are measured using the instrumentation shown in Figs. 3 and 4. To record the beam
253 rotations the inclinometers C1 and C3 are positioned 100 mm from the connected end of the Left side
254 and Right side, respectively. The rotation of the column is measured by C2 placed at the centre of the
255 joint, and the Left and Right joint rotations are determined from the difference between the beam and
256 column rotations. Relative slip between a pair of cleats and the beam is measured via two
257 displacement transducers, labelled in Fig. 4 as LTL and LBL, and LTR and LBR. The first letter in
258 LTL is for the centre-to-centre vertical distance of 64 mm between two horizontal transducers, and
259 the second and third letters are for the *Top* of cleat and for the *Left-sided* joint. Rotations are
260 measured to a resolution of 0.02 mrad (linear to $\pm 1\%$ over a 10° range) and displacements to ± 0.01
261 mm. Slip rotation due to relative horizontal slip between a pair of web cleats and the beam web has to
262 be subtracted from the measured joint rotation in order to obtain the required ϕ . This ‘secondary’ slip
263 rotation (ϕ_{slip}) is calculated from:

$$264 \quad \phi_{slip} = \tan^{-1} \left(\frac{lb - lt}{l} \right) \times 1000 \quad (\text{mrad}) \quad (2)$$

265 where lt and lb are the horizontal slips measured by the displacement transducer pair of either LTL
266 and LBL for Left joint or LTR and LBR for Right joint.

267

268 When web cleats are of FRP, failure is by way of delamination cracking at top of cleats near the fillet
269 radius (Mottram and Zheng 1999a; Mosallam 2011; Qureshi and Mottram 2012). With change of
270 material to steel, the web cleating in itself is not the weak link. The structural steel has characteristic
271 yield strength of 275 MPa that is many times higher than the through-thickness tensile strength of
272 FRP and the modulus of elasticity is 10-20 times higher. These significant differences in material
273 properties ensure that the steel cleating, of 10 mm thickness, cannot fail first under the prying action.

274 The resulting tension from the joint moment force acting at top bolt level can be expected to produce
275 significant flexural deformation in the column flange outstands. In order to monitor these outstand
276 deformations the change in column depth, given by $(h_{\text{prying}} - h)$ is measured after each load
277 increment. Fig. 2 defines h to be the undeformed depth of the column member and h_{prying} to be its
278 deformed depth. Throughout the testing h_{prying} is measured both at the top and bottom bolt levels.

279

280 **RESULTS AND DISCUSSION**

281 The modes of failure, joint properties and moment-rotation (M - ϕ) responses will be presented in a
282 discussion of results in two parts. The first part is for the joint tests with FRP cleats, while the second
283 part is for the tests with steel cleats. Joint properties that are dependent on ϕ have been compensated
284 for slip rotation ϕ_{slip} using Equ. (2). Tables 1 and 2 report the joint properties for the 10
285 Wmj203_2M16_FC joints and the six Wmj203_2M16_ST joints. Each specimen has a Left and
286 Right-sided joint and this is identified in the tables. When two values from a single specimen are
287 given in the discussion the first will always be for the Left-sided joint and the second for the Right-
288 sided joint. To highlight the minimum and maximum measurements they are given in bold text.
289 Column (1) gives the specimen label using the scheme introduced earlier in the paper. Columns (2) to
290 (4) report the linear joint properties of initial moment (M_i), initial joint rotation (ϕ_i) and initial joint
291 stiffness S_i ($= M_i/\phi_i$). As soon as the M - ϕ response is observed to go non-linear M_i and ϕ_i are
292 established. The same three properties at (FRP material) damage onset of M_j , ϕ_j and ($S_j = M_j/\phi_j$) are
293 given in columns (5) to (7). In this study subscript ‘j’ is for the key properties of a joint immediately
294 after initiation of damage onset due to FRP failure. A specific definition for damage onset is to be
295 given for both cleat materials. Maximum joint properties of M_{max} and ϕ_{max} are given by columns (8)
296 to (9). Mean and coefficient of variation (CV) for the eight joint properties are given at the bottom of
297 the tables.

298

299

300 **Joint tests with pultruded FRP cleats**

301 Failure patterns and a definition for damage onset are discussed first, followed by an evaluation of
302 the joint properties presented in Table 1, the M - ϕ curves and the relationship between damage
303 rotation and SLS vertical deflection limits. Fig. 5 has four parts, with (a) and (b) for the undeformed
304 ($\phi = 0$) Left and Right joints in Wmj203_2M16_FC1.3 with (c) and (d) for these joints after ϕ_{\max}
305 (column (9) in Table 1) had been applied.

306

307 An appropriate definition for onset of FRP failure is crucial in establishing the serviceability rotation
308 for design of the beam section in bending. For joints with FRP cleats it is defined as a point on the M -
309 ϕ where hairline delamination cracking first becomes visible at top of cleating and near the fillet
310 radius. This failure pattern is well-known when using pultruded leg-angles for the web cleats (Bank
311 *et al.* 1990; Qureshi and Mottram 2012). Using a dentist's mirror to view the top surface clearly, the
312 photograph in Fig. 6 shows the failure mode on testing Wmj203_2M16_FC1.4. It is noted that
313 initiation of the delamination cracks can happen on either side of the junction between a pair of legs.
314 At each load increment, careful observations were made to detect the extent of FRP damage
315 progression. As can be seen in Figs. 5(c) and 5(d) the increase in M from M_j to M_{\max} caused the FRP
316 legs to become visually separated from column flanges. At this stage of the test, existing cracks are
317 widened and the new delamination cracks are formed. Loud, and audible noises signalling crack
318 propagation following an instant increase in ϕ , without corresponding enhancement in M , were signs
319 of impending ultimate failure. The ultimate failure of all 10 joints with FRP cleats was due to
320 excessive delamination damage. Because the positioning of layers of E-glass reinforcement are not
321 constant through the leg-angle's thickness either the Left or Right cleat pair experienced more FRP
322 damage, and thus joint rotation, than the other. This helps to explain why ϕ_{\max} in column (9) of Table
323 1 for the Left and Right-sided joint pair is often significantly different. This difference in rotation can
324 be seen by comparing in Figs. 5(c) and 5(d) the deformations of the joints in specimen
325 Wmj203_2M16_FC1.3.

326

327 The 10 entries in column (2) of Table 1 inform us that the M - ϕ response remains linear up to a mean
328 M_i of 0.32 kNm with a Coefficient of Variation (CV) of 12%. The range for M_i is for a minimum of
329 0.26 kNm to a maximum of 0.35 kNm. Initial rotations (ϕ_i) in column (3) are seen to range from a
330 minimum of 3.2 mrad to a maximum of 5.2 mrad, with mean and CV of 4.2 mrad and 16%. From
331 column (4) the minimum and maximum initial joint rotational stiffnesses (S_i) are 63 and 87 kNm/rad.
332 The mean S_i of 76 kNm/rad has a CV of 9%. Columns (8) and (9) give M_{\max} and ϕ_{\max} and their means
333 are 1.0 kNm and 43 mrad respectively. It is found that the mean M_{\max} of 1 kNm is < 7% of the lower
334 bound estimate for the ULS moment of resistance (14.5 kNm) due to elastic local (flange) buckling.
335 This result informs us that in accordance with Clause 5.2.3.2(3) in Eurocode 3 Part 1-8 (BS EN 1993-
336 1-8:2005) the FRP cleated joints can be classified as nominally pinned by strength. In terms of the
337 flexural moment of resistance (94 kNm) for the 203×203×9.53 mm shape the M_{\max} (1 kNm) is just
338 above 1%.

339

340 Whilst the M_{\max} from the batch of 10 joints has a relatively low CV at 4% there is a very high CV of
341 32% with ϕ_{\max} . Two reasons can be given for this significant variation in maximum rotation. One of
342 these is that it depends on when the testing was stopped, and the termination criterion used was either
343 excessive FRP failure or when there could be instability of the specimen. The second of the reasons
344 existed when either the Left or Right joint had rotated considerably more than the other. The
345 difference in ϕ_{\max} is seen to be associated to a significantly different level of delamination cracking
346 on the two sides, as seen in Figs. 5(c) and 5(d).

347

348 Figs. 7 and 8 present the M - ϕ curves for Wmj203_2M16_FC1.3, with and without the slip rotation
349 compensated for. In these figures, the Left joint's M - ϕ is represented by a solid line curve and the
350 Right joint by a dashed line curve. On each curve a solid circle symbols is used to indentify M_j and
351 ϕ_j . The saw-tooth shape to the M - ϕ curves is due to taking sets of readings immediately after load

352 application and 5 minutes later, before the next increment is applied. The measured reduction in M is
353 because the joints are undergoing relaxation with time. The test results indicate that response remains
354 linear elastic until web cleats start to delaminate causing loss of joint stiffness and increased local
355 deformation. Beyond a moment of 0.35 kNm the M - ϕ response goes non-linear. For this specific joint
356 pair the value of ϕ at ultimate failure on Left side is double that on Right side. It was observed that
357 the Left joint experienced more FRP progressive failure and this observation can be explained by the
358 inhomogeneous nature of the pultruded leg-angle, as discussed earlier. Figs. 5(a) and 5(b) show the
359 undeformed Left and Right joints, and Figs. 5(c) and 5(d) are for when they were fully deformed. It is
360 very clear from the latter two images that the Left side rotated most in order to maintain the same
361 level of M . At damage onset, the secondary slip rotations for specimen Wmj203_2M16_FC1.3 were
362 0.9 and 5.5 mrad. This leads to an artificially higher ϕ_j (for damage onset) of 14.5 and 20 mrad and
363 different M - ϕ curves in Figs. 7 and 8 for what are nominally identical joints. When the slip rotation
364 (ϕ_{slip}) is compensated for in Fig. 8, the two joints now give the same trends and similar ϕ_s at 13.6 and
365 14.4 mrad. To be able to propose improved design guidance the comparison of the M - ϕ curves in
366 Figs. 7 and 8 justifies why slip rotation had to be accounted for so that the reported joint responses
367 are primarily due to prying action deformation in the cleated connections.

368

369 As can be seen from the plots in Figs. 7 and 8 that specimen Wmj203_2M16_FC1.3 was thrice
370 unloaded and reloaded to assess the extent of permanent deformation in the joints. This next
371 discussion will be specific to the M - ϕ results reported in Fig. 8. First unloading took place when ϕ
372 first attained 10 mrad, before FRP damage had appeared. Measured permanent rotations were 3.5
373 mrad on both joint sides. Second unloading stage was taken when ϕ was about 20 mrad and this gave
374 permanent rotations of 7.5 and 8.5 mrad. When the planned third unloading stage of 30 mrad was
375 reached the jack operator could no longer control the rotation, and the 40 mrad on the Left side was
376 16 mrad higher than on the Right side. Unloading from joint rotations of 40 and 26 mrad resulted in

377 permanent rotations of 22 and 11 mrad. On unloading from M_{\max} the permanent joint rotation was
378 significant at 43 and 13 mrad, respectively.

379

380 A SLS is the condition beyond which a whole structure or part thereof fails to satisfy its intended
381 purpose under unfactored design loading, but has not reached an ultimate limit state (BS EN
382 1990:2002). For a simply supported steel beam having a span of L subjected to a uniformly
383 distributed load, a common deflection limit is $L/360$. This is for the structural situation where beam
384 members are carrying plaster or other brittle finish, and is found for example, in the NA to BS EN
385 1993-1-1:2005. For design of beams the Design Manual from Creative Pultrusions Inc. has allowable
386 uniform load tables for a number of shapes (Anonymous 2013a). The table on page 29 of Chapter 4 is
387 specific to the Pultex® SuperStructural Wide Flange section of size 203×203×9.53 mm (Pultex®
388 pultrusion design manual 2013) used in the testing. It presents a number of vertical deflection limits
389 that are acceptable for this shape when used as a simply supported beam member. The table allows
390 for a maximum deflection limit of $L/150$ when L ranges from 5 to 7.25 m. Moreover, it gives uniform
391 distributed loads for the deflection limits of $L/180$ (3.25 to 7.25 m), $L/240$ (2.75 to 7.25 m) and $L/360$
392 (2.5 to 7.25 m). The values in brackets are for the span range specific to the deflection limit. There
393 are no notes with the Creative Pultrusions tables to recommend when the different limits are to be
394 adopted. Creative Pultrusions lets this task up to the engineer of Record. It is noteworthy that more
395 than a single limit could be required to account for different structural situations, environmental
396 conditions and/or loading cases. Irrespective of the FRP beam's size, the EUROCOMP Design Code
397 and Handbook (Clarke 1996) recommends a SLS deflection limit of $L/250$. These different limits for
398 vertical deflection show that work is needed to find out a reliable SLS design approach.

399

400 The bar chart in Fig. 9 presents the ϕ_s s from testing the 10 joints having FRP cleats (see column (6) in
401 Table 1). Higher than the measured ϕ_s s, the 'SLS' deflection limit of 17.8 mrad (for $L/180$) from
402 Creative Pultrusions Inc. is given by the horizontal dashed line. Note that when determining the end
403 rotation (e.g. 17.8 mrad) for a deflection limit (e.g., $L/180$) the Pultruded FRP beam member is

404 assumed to be shear rigid and the properties for the 203×203×9.53 mm shape are taken from the
405 Pultex® SuperStructural table of mechanical properties in Chapter 3 of Anonymous (2013a). Using
406 the expression Mean – 1.72×SD, from Annex D of Eurocode 0 (BS EN 1990:2002), and assuming
407 the CV is known, the characteristic ϕ_j for the batch of joints is calculated to be 10.9 mrad. SD is for
408 the Standard Deviation of the batch of results, and is given by Mean×CV. Analysis therefore
409 indicates that the SLS vertical deflection limit for the FRP cleated joint could be $L/300$. This $L/300$
410 limit is given in Fig. 9 by a solid horizontal line and, clearly, this EC0 determined limit is
411 significantly below all, but $L/360$, of the four limits in the load table on page 22 of Chapter 4
412 (Anonymous 2013a). For a nominally pinned joint a rotation of 17.8 mrad (for $L/180$) has been
413 shown to be too liberal since FRP cracking can be present this deflection can be reached in practice.
414 Clearly there will be severe FRP damage (at cleat tops) when the vertical deflection attained $L/150$
415 (for a ϕ of 21.3 mrad). Even the lower SLS limit of $L/250$ from the EUROCOMP Design Code and
416 Handbook (Clarke 1996) could be unacceptable because durability will be impaired when cleats have
417 delamination damage.

418

419 Based on an evaluation of the test results presented in Table 1 a mid-span vertical deflection of $L/300$
420 can be proposed to ensure satisfactory performance during the service life. It is to be recognized that
421 a SLS limit of $L/300$ could be relaxed when the environmental conditions surrounding the FRP
422 cleating are benign (i.e. there is minimal moisture/water to attack exposed glass fibres at the
423 delamination crack surfaces (Zafari and Mottram 2012)). This more favourable serviceability
424 condition could, for example, exist if the simple constructed frame is enclosed by, say weather
425 protecting panelling.

426

427 **Joint tests with steel cleats**

428 The same test method was carried out with a batch of three nominally identical specimens having
429 replaced the FRP cleats with steel cleats possessing virtually the same dimensions. Table 2 reports

430 the results from the six steel joints using the same format as in Table 1. Because failure is different
431 and new, there is a need to develop a specific definition for what constitutes damage onset. As for the
432 test series with the 10 joints with FRP cleats there follows a discussion on the moment-rotation
433 results and what could be the SLS vertical deflection limit for a (simply supported) beam subjected to
434 a uniformly distributed load.

435

436 Defining damage onset with steel cleats is more complex than was the case with FRP cleating.
437 Because steel cleats are not the weak link, failure in the FRP occurs close to the web-flange junction
438 in the pultruded column member. Because this initial damage is internal it could not be observed by
439 visual inspection. In the absence of visible FRP cracking, damage onset was signalled by the first
440 audible acoustic emissions emanating from the source of internal fracturing. Additional evidence for
441 this approach to establishing ϕ_i is that audible noises were found to coincide with a significant
442 outward flexural deformation of the flange outstands at the top bolt level. This deformation was
443 signalled by the commencement of nonlinearity in $M-\phi$ response. Damage onset is, therefore,
444 specifically defined with steel cleating as the point on the $M-\phi$ curve when acoustic emissions were
445 first heard, followed by measurement of considerable flexural deformation of column flanges. It is
446 noteworthy that acoustic emission had previously been established from FRP joint testing (Mottram
447 and Zheng 1999a) to be a reliable indicator for onset of FRP failure.

448

449 Figs. 10(a) and 10(b) show the jointing region in specimen Wmj203_2M16_ST1.3 before testing and
450 after M_{\max} had been attained. Comparing the two images shows that there was, at the end of testing,
451 significant outward flexural deformation of the flange outstands level with the top bolts. The depth of
452 the column at bottom bolt level (h_{BOTTOM}) essentially remains constant, and is unaffected by the
453 resultant compressive force from the moment generated by the prying action. Fig. 11 presents the
454 variation in column depth h_{prying} , due to prying action, corresponding to M . Column depth at the top
455 bolt level is denoted by $h_{\text{prying(TOP)}}$ and is plotted with a solid line. The dashed curve in Fig. 11 is for
456 column depth at the bottom bolt level, represented by $h_{\text{prying(BOTTOM)}}$. The column depth at bottom bolt

457 level of web cleat shows a marginal decrease of 0.1-0.2%, as the moment approaches M_{\max} . When M
458 exceeds 1.4 kNm, h_{prying} at the top bolt level is found to increase rapidly from 1 to 4% of the
459 measured undeformed depth, h (i.e., 202.4 mm). This non-linear response is a signal of impending
460 ultimate joint failure. In the three tests with a pair of steel cleat joints the maximum increase in
461 column depth was found to be $1.05h$.

462

463 Presented in Table 2 are the initial (M_i , ϕ_i and S_i), damage onset (M_j , ϕ_j and S_j) and maximum joint
464 properties (M_{\max} and ϕ_{\max}). The properties at damage onset were determined using the specific
465 definition for steel cleating introduced above. M - ϕ curves for the six joints were found to remain
466 linear to a mean M_i of 0.64 kNm. Because this joint property varies from 0.61 to 0.66 kNm it has a
467 relatively low CV of 4%. ϕ_i is found to range from 3.2 to 4.6 mrad, giving a mean and CV of 3.8
468 mrad and 13% respectively. The batch of steel joints gave a mean initial rotational stiffness ($S_i =$
469 M_i/ϕ_i) of 169 kNm/rad, with a CV of 11%, and the minimum and maximum stiffnesses are 144 and
470 194 kNm/rad. At the onset of FRP damage in the column member the mean moment (M_j), rotation
471 (ϕ_j) and rotational stiffness (S_j) are 0.88 kNm, 5.9 mrad and 150 kNm/rad, respectively. As
472 established by their CVs being $\leq 10\%$ these joint properties do not vary too much. The mean M_{\max}
473 and ϕ_{\max} are 1.7 kNm and 42 mrad with corresponding CVs of 8% and 51%. The reasons for why
474 there is considerable variation in reported ϕ_{\max} values in Table 2 are the same as for the detailing with
475 the FRP cleating. To demonstrate that joint detailing with steel cleats can be classified as nominally
476 pinned for their strength the mean M_{\max} (1.72 kNm) is found to be $< 12\%$ of the estimated moment
477 resistance of the section (14.5 kNm) for the ULS failure mode of local (flange) buckling.

478

479 Moment-rotation (M - ϕ) curves for the Wmj203_2M16_ST1.2 joints are plotted in Fig. 12 (with slip
480 rotation included) and Fig. 13 (with slip rotation compensated for). Both figures show that there is
481 virtually a linear response to the damage rotation (ϕ_j), which is characterised by loss of rotational
482 stiffness and the increasing outward flexural deformation of the column flange outstands. After

483 reaching M_j of 0.9 kNm (as given by the solid circular symbols), the M - ϕ curves go increasingly non-
484 linear. The measured rotations from slippage were 1.1 and 0.6 mrad at ϕ_j . With the slip rotation taken
485 into account ϕ_j for Left and Right joints were 5.3 and 6.5 mrad.

486

487 Specimen Wmj203_2M16_ST1.2 was unloaded and reloaded to determine the extent of permanent
488 deformation. First unloading took place when ϕ approached 16 mrad and gave a permanent rotation
489 of 5 mrad for both joints. Because of progressive internal material damage, it was hard to keep both
490 joint rotations roughly the same. On reloading to the same (unloading) moment it was observed that ϕ
491 increased to 35 mrad on Left side whilst the Right side rotation stayed constant at 16 mrad. This
492 change in joint response indicates that the Left joint was deteriorating more rapidly. This finding was
493 confirmed by different permanent rotations of 10 and 5 mrad when Wmj203_2M16_ST1.2 was
494 unloaded and reloaded again when the Left and Right ϕ s were 35 and 16 mrad. Unloading after M_{max}
495 had been surpassed gave permanent ϕ s of 15 and 10 mrad.

496

497 Replacing cleats of pultruded FRP with structural steel gives a stiffer and stronger joint. As listed in
498 column (6) in Table 2 the mean ϕ_j with steel is almost half its mean in Table 1 for the FRP joints.
499 Using a bar chart construction Fig. 14 presents the six joint ϕ s using the damage onset criterion for
500 steel cleating. Following the presentation in Fig. 9 the SLS vertical deflection limit of $L/180$ is given
501 by a horizontal dashed line. The characteristic rotation for the steel joints is calculated to be 4.9 mrad,
502 from Mean - 1.77 \times SD and assuming the CV is known. For a simply supported beam with uniformly
503 distributed load an end rotation of 4.9 mrad results in a mid-span vertical deflection of only $L/650$.
504 The predicted characteristic value is seen to be below one-third of the recommend SLS rotation of
505 17.8 mrad for a deflection limit of $L/180$ taken from pultruder's Design Manual (Anonymous 2013a).
506 It is moreover found to be less than half of the 12.8 mrad recommended by the guidance in the
507 EUROCOMP Design Code and Handbook (Clark, 1996).

508

509 **CONCLUDING REMARKS**

510 Test results are presented for the moment-rotation characteristics of two batches of 10 and six
511 nominally identical (nominally pinned) joints having FRP or steel web cleats, respectively. In all
512 other respects the joint detailing and test method are identical. The variation found in rotational
513 properties from a batch of nominally identical joints shows why the testing was necessary. An
514 evaluation of the results was made using the key joint properties, the moment-rotation responses, the
515 failure modes, damage onset criteria and limits on vertical mid-span deflection for Serviceability
516 Limit State (SLS) design.

517

518 The main findings from the experimental study are:

- 519 • There are distinct failure modes for the batches of the joints with FRP and steel web cleats.
520 For the FRP situation failure is always due to excessive delamination cracking at top of the
521 cleats. When cleating is of structural grade steel FRP failure happens within the column
522 member as significant outward flexural deformation causes internal (non-visible) fracturing.
- 523 • It is noted that there is no mention of these failure modes in any of the pultruders' design
524 manuals (Anonymous 2013a; 2013b; 2013c). The authors recommend that all joint failure
525 modes and their design implications should be given for acceptable guidelines.
- 526 • The average initial rotational stiffness of 169 kNm/rad for the steel joints is found to be
527 double the stiffness of 76 kNm/rad for the FRP joints. In both cases, the average initial
528 rotation at which the moment-rotation response goes non-linear is similar, and is about 4
529 mrad.
- 530 • The magnitude of slip rotation (at bolt holes) in the measured joint rotation was successfully
531 minimised by having minimal bolt clearance holes for the beam-side cleat connections.
532 Owing to 'off-the-shelf' M16 grade 8.8 bolts having a diameter in the range of 15.6 to 15.9
533 mm, tight-fitting bolting on specimen assembly was impractical. By compensating for
534 slippage the test methodology ensured that reported joint rotations are due primarily to the
535 deformation caused by the (damaging) prying action.

- 536 • Using the statistical method in Annex D of Eurocode 0 the characteristic rotation at the onset
537 of FRP damage (for material fracturing) is determined to be 10.9 mrad for the batch of FRP
538 joints. When a simply supported beam having span L is subjected to uniformly distributed
539 load, this nominally pinned joint rotation corresponds to a mid-span deflection limit of $L/300$.
540 It is found that the characteristic rotation is only 4.9 mrad from the batch of steel joints. The
541 corresponding deflection limit is only $L/650$; under half that established with FRP cleating.
- 542 • It is recommended that the vertical deflection limits shall be carefully scrutinized by the EOR.
543 Current manufacturers' manuals, codes and standards do not address the serviceability in
544 relation to cleated connections. The governing service limit state may be dictated by joint
545 rotation.
- 546 • Although the presence of clearance holes allows there to be slip rotation that is beneficial,
547 even essential, in the field, it cannot be relied upon to ensure there is no FRP failure when
548 satisfying SLS design. Depending on the positioning of the bolts in their clearance holes there
549 is a likelihood that it might not occur. In the field, it is not practical to locate the bolts with
550 precision that ensure the necessary slippage contribution to the SLS joint rotation is always
551 going to be guaranteed.
- 552 • Based on an evaluation of the test results reported in this paper it can be recommended to
553 designers of pultruded frame structures that they need to be careful when specifying the
554 combination of cleat material and other joint details. The reason for this guidance is that the
555 solution chosen must enable a nominally pinned joint to rotate, without FRP failure, to satisfy
556 the required SLS vertical deflection limit, especially when the surrounding environment is
557 aggressive as exposed fractured surfaces will cause longer-term durability issues.

558

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