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# Effect of transverse and longitudinal reinforcement ratios on the behaviour of RC T-beams shear-strengthened with embedded FRP bars

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1	EFFECT OF TRANSVERSE AND LONGITUDINAL REINFORCEMENT
2	RATIOS ON THE BEHAVIOUR OF RC T-BEAMS SHEAR-
3	STRENGTHENED WITH EMBEDDED FRP BARS
4	
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#### 26 Abstract

Seven reinforced concrete (RC) T-beams, comprising two unstrengthened (control) beams and 27 five beams strengthened in shear with embedded FRP bars, were tested to failure. The test 28 29 parameters were steel-to-FRP shear reinforcement ratio and tension reinforcement ratio. A nonlinear finite element (FE) model was developed, validated and used to conduct parametric 30 studies. The experimental and FE results showed that the concrete and FRP contributions to 31 shear resistance as well as the total shear force capacity all decrease with increasing steel-to-32 FRP shear reinforcement ratio. The tension reinforcement ratio influenced the failure mode of 33 34 the tested and modelled beams but had insignificant impact on shear strength enhancement. The experimental results were compared with the FE and Concrete Society Technical Report 35 55 predictions. The FE model correctly reproduced the experimental results and gave accurate 36 37 predictions, with a mean predicted-to-experimental ratio of 1.04, whereas TR55 gave conservative predictions, with a mean predicted-to-experimental ratio of 0.42. 38

40	Keywords: beam; concrete; embedded bars; fibre reinforced polymer; finite element; shear;
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#### 51 **1. Introduction**

Heavier traffic loads, poor initial design, aggressive exposure conditions, natural or man-made 52 extreme events and steel reinforcement corrosion can all deteriorate the shear strength of 53 existing reinforced concrete (RC) structures [1]. Many cost-effective, practical and durable 54 fibre reinforced polymer (FRP) shear strengthening solutions have emerged in response to the 55 increasing number of shear-deficient concrete structures. For example, externally bonded (EB) 56 57 [2] and near-surface mounted (NSM) [3] FRP strengthening techniques have been verified to enhance the shear strength of existing RC beams. However, EB and NSM FRP shear 58 59 strengthening systems require laborious surface preparation and, unless properly anchored, debond prematurely from the concrete. The Deep Embedment (DE) [4], also known as the 60 embedded through-section (ETS) [5], shear strengthening technique used in this study consists 61 of glass FRP (GFRP) or carbon FRP (CFRP) bars embedded into the concrete core to act as 62 additional shear reinforcement. The FRP bars are inserted into epoxy-filled holes drilled 63 throughout the entire depth of the beam, thereby connecting the compression chord to the 64 tension chord and ensuring that truss action can be fully developed. It is acknowledged that it 65 can be difficult to drill holes in members with congested internal steel reinforcement. However, 66 exiting concrete members requiring shear strengthening usually include relatively low amounts 67 of steel reinforcement. The locations of existing steel bars can be obtained from as-built 68 drawings and/or by using cover metres. Besides, core drilling machines with steel bar sensing 69 70 function can be used to drill holes. Such drilling machines automatically shut off when they touch a steel bar, thereby ensuring integrity of the steel bars. 71

Previous research work on DE FRP shear strengthening provided valuable findings, particularly with regard to the effects of the presence of internal steel shear reinforcement [6]. An installation technique that does not require access to the top surface of the beam was also developed [4, 5, 7]. The effects of the DE bar diameter and spacing [8], shear link corrosion

level [9], shear span-to-effective depth ratio [10] and moment-shear interaction [11] were 76 examined. Analytical models for predicting the contribution of the DE bars to the shear strength 77 were also proposed [6, 8, 12]. However, the effects of other parameters that may also influence 78 the strengthened behaviour have not yet been fully understood. It has been recognised that 79 steel-to-FRP shear reinforcement ratio is one of the main parameters governing the 80 strengthened behaviour. However, experimental and numerical research [e.g. 7, 13]. examining 81 82 the effect of steel-to-FRP shear reinforcement ratio is limited. Similarly, tension reinforcement ratio has been demonstrated to influence the behaviour of EB FRP-strengthened beams [2]. 83 84 However, the effect of tension reinforcement ratio on the behaviour of DE FRP-strengthened beams has not yet been identified. 85

Using physical tests and nonlinear finite element (FE) modelling, this paper critically investigates the effect of steel-to-FRP shear reinforcement ratio and tension reinforcement ratio on the behaviour of RC T-beams strengthened in shear with DE FRP bars. The experimental and FE results are used to assess the accuracy of the Concrete Society's Technical Report 55 (TR55) [14] design model for DE FRP shear strengthening.

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#### 92 2. Research Significance

Shear strengthening of existing RC structures with embedded FRP bars is an area with great 93 potential, particularly in situations when the flange and/or web are inaccessible. The embedded 94 95 FRP bars are less susceptible to debonding issues when compared with unanchored EB and NSM shear strengthening systems. Yet, the effect of transverse and longitudinal reinforcement 96 ratios is not fully understood. This study provides valuable insights into the effect of these two 97 98 parameters on the strengthened behaviour. Moreover, it identifies limitations in current shear strengthening design guidance and presents an accurate predictive tool that has been 99 demonstrated to be an improvement over existing design practice. 100

#### **3. Experimental Programme**

#### 102 3.1. Specimens

The experimental programme consisted of two unstrengthened (control) and five DE FRP-103 strengthened RC T-beams. The unstrengthened beams had a two-part designation whereas the 104 strengthened beams had a three-part designation. The first part indicates that a beam was either 105 a control (C) or a strengthened (S) specimen. The second part denotes the percentage of tension 106 107 reinforcement (either 2.0 or 2.7) in the maximum moment zone. The percentage of tension reinforcement is defined as  $(100\% A_s/b_w d)$  where  $A_s$  is the area of tension reinforcement,  $b_w$  is 108 109 the web width and d is the effective depth. The third part refers to the type (either glass (G) or carbon (C)) of DE FRP bars and the steel-to-FRP shear reinforcement ratio. The steel-to-FRP 110 shear reinforcement ratio is defined as  $(E_sA_{sw}/b_ws)/(E_fA_{f'}b_ws_f)$  where  $E_s$  and  $E_f$  are the elastic 111 moduli of steel and FRP, respectively; Asw and Af are the areas of steel and FRP shear 112 reinforcement, respectively; and s and sf are the spacing of steel and FRP shear reinforcement, 113 respectively. Hence, the designation S/2.7/C1.35 refers to a strengthened beam with a tension 114 reinforcement ratio of 2.7% and steel-to-CFRP shear reinforcement ratio of 1.35. 115

As can be seen in Fig. 1, the RC T-beams had a flange width of 200 mm, flange depth of about 116 63 mm, web width of 75 mm and overall height of 325 mm. All beams had an effective depth 117 and a shear span-to-effective depth ratio of about 300 mm and 3.0, respectively. The beams 118 were longitudinally reinforced in compression with one layer of two 8 mm diameter steel bars. 119 120 The tension reinforcement comprised either four 12 mm diameter steel bars or four 12 mm and two 10 mm diameter steel bars (see Fig. 1), resulting in a tension reinforcement ratio in the 121 maximum moment zone of either 2.0 or 2.7%, respectively. The steel shear reinforcement 122 consisted of 4 mm diameter shear links spaced at 300 mm centre-to-centre (c/c), resulting in a 123 steel shear reinforcement ratio (Asw/ bws) of 0.11%. This reinforcement arrangement is 124 representative of earlier design practice in the UK [9]. The DE shear strengthening system 125

126 consisted of 6 mm diameter sand-coated FRP bars spaced as shown in Fig. 2. The beams with

127 3 DE bars had an FRP shear reinforcement ratio  $(A_{f'} b_w s_f)$  of 0.125% whereas the beams with

128 6 DE bars had an FRP shear reinforcement ratio of 0.25%.

129

130 3.2. Material Properties

Each RC T-beam was cast from a single batch of ready-mixed concrete with a maximum 131 132 aggregate size of 10 mm. The concrete mixture proportions were cement: water: sand: coarse aggregate = 1: 0.42: 1.30: 2.65. A superplasticiser (Alphaflow 420) dosage of 0.75% by weight 133 134 of cement was added to ensure adequate workability of the concrete mix. Standard compression tests were conducted on the day of beam testing on 100 mm cube and 100 mm diameter  $\times$  200 135 mm long cylinder specimens. The results showed that the concrete had average cylinder and 136 cube compressive strength values of 41 and 49 MPa, and standard deviation values of 1.82 and 137 1.94 MPa, respectively. 138

The mechanical properties of the steel and DE FRP reinforcement, as declared by the manufacturer, are reported in Table 1. A commercially available high viscosity epoxy resin was used to bond the FRP bars to the concrete. The bond strength, compressive strength, compressive modulus, tensile strength and elongation at failure of the epoxy resin were 12.4 MPa, 82.7 MPa, 1493 MPa, 43.5 MPa and 2%, respectively, as certified by the manufacturer.

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145 3.3. Strengthening Procedure

To install the FRP bars, 9 mm diameter vertical holes were formed in the shear spans of the beams, through the centreline of the cross-section, at the FRP bar locations shown in Fig. 2. The vertical holes were created by installing PVC rods at the required positions within the steel reinforcement cage before casting the concrete. The PVC rods were removed from the concrete beams after casting. A 10 mm diameter drill bit was then used to enlarge the cast-in-place holes. Prior to installing the FRP bars, the drilled holes were roughened by a wire brush and cleaned with compressed air. The lower ends of the holes were sealed and the epoxy resin was used to fill two-thirds of the holes. The FRP bars were covered with a thin layer of the adhesive and inserted into the holes. Any excess epoxy was removed. Many research studies [e.g. 4, 5, 7] have already demonstrated that it is possible to install FRP bars by drilling vertical or inclined holes rather than by using cast-in-place holes. The installation procedure explained above was used for simplicity.

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159 3.4. Test Setup and Instrumentation

The RC T-beams were tested in a three-point bending configuration as shown in Fig. 1. This 160 setup allowed two tests to be conducted on one beam by testing one beam end zone while 161 keeping the other end unstressed and vice versa. The load was applied monotonically using a 162 1000 kN hydraulic cylinder and measured by a 1000 kN load cell. The vertical deflection under 163 applied load was measured by displacement transducers. The strain in the tension 164 reinforcement at the position of the maximum bending moment was measured using 6 mm 165 strain gauges. The strain in the steel and FRP shear reinforcement was measured using 3 mm 166 strain gauges positioned along the line joining the support and loading point (see Fig. 2). The 167 readings of the 1000 kN load cell, displacement transducers and strain gauges were obtained 168 using a data logger connected to a personal computer. 169

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#### 171 **4. Test Results and Discussion**

172 4.1. Shear Strength

Table 2 gives the unstrengthened shear force capacity, shear force at failure, gain in shearresistance due to the DE FRP bars, and failure mode of each beam.

The control beams (C/2.0 and C/2.7) had a shear strength of about 65.5 and 75.5 kN, 175 respectively. The higher shear strength of C/2.7 is attributable to the higher tension 176 reinforcement ratio, which enhanced dowel action. The tension reinforcement ratio also 177 influenced the failure mode of the strengthened beams. S/2.0/G1.91 failed in flexure whereas 178 S/2.7/G1.91 failed in shear. However, the increase in tension reinforcement ratio from 2.0 to 179 2.7% had insignificant effect on the gain due to DE GFRP bars. S/2.0/G1.91 and S/2.7/G1.91 180 181 had a comparable strength gain of 13 and 15 kN, respectively, whereas the strength gain in both S/2.0/G3.82 and S/2.7/G3.82 was negligible. 182

183 The steel-to-FRP shear reinforcement ratio had a clear effect on the shear strength gain. As stated above, the shear strength gain was negligible at a steel-to-FRP shear reinforcement ratio 184 of 3.82. The decrease in steel-to-FRP shear reinforcement ratio from 3.82 to 1.91, through the 185 provision of additional GFRP bars, resulted in a shear strength gain of about 19.8% (13-15 kN) 186 in both S/2.0/G1.91 and S/2.7/G1.91. The further decrease in steel-to-FRP shear reinforcement 187 ratio from 1.91 to 1.35, by using CFRP bars, increased the shear strength gain from 19.8% to 188 37.2% (28.1 kN). Due to the higher elastic modulus of steel (200 GPa) compared with that of 189 CFRP (130 GPa) or GFRP (46 GPa), steel shear links attract higher forces than DE FRP bars. 190 A low steel-to-FRP shear reinforcement ratio, attainable via a high FRP axial stiffness ( $E_fA_f$ ) 191 and/or low FRP spacing, is therefore required to achieve significant shear strength 192 enhancement in existing RC structures. 193

194

195 4.2. Deflection Response

Fig. 3 compares the shear force-deflection curves of the tested beams. Except for S/2.0/G1.91,
all beams featured a quasi-linear shear force-deflection response up to peak shear force.
S/2.0/G1.91 had a ductile failure featuring an approximately 60 mm long plateau. The
remaining beams had a brittle shear failure with a sudden drop in load at peak shear force.

All beams had comparable elastic (i.e. uncracked) stiffness. The cracked stiffness of all beams was also comparable up to the formation of inclined cracks at a shear force of about 25 kN. After the formation of inclined cracks, the unstrengthened (control) and GFRP-strengthened beams had comparable cracked stiffness whereas the CFRP-strengthened beam had a stiffer response. The axial stiffness of a CFRP bar (3674 kN per bar) is 2.83 times higher than that of a GFRP bar (1300 kN per bar). The CFRP bars are therefore more effective in resisting inclined crack opening and controlling deflection.

207

208 4.3. Cracking and Failure Mode

The crack patterns at failure of the tested beams are depicted in Fig. 4. Except for S/2.0/G1.91, 209 all beams had comparable cracking behaviour. The formation of flexural cracks at the soffit of 210 the beams under the load started at a shear force of about 10 kN. With increased loading, the 211 flexural cracks extended into the shear span. The outermost flexural crack in the shear span 212 turned into an inclined crack at a shear force of about 25 kN. Upon further loading, more 213 inclined cracks appeared in the shear span and, eventually, an inclined crack instigated a 214 diagonal tension failure. Of note is that the inclination of shear cracks at failure increased with 215 decreasing the spacing of the DE GFRP bars. It is well known that decreasing the spacing of 216 transverse reinforcement in a RC beam results in steeper cracks [7, 15]. Moreover, C/2.7/C1.35 217 had less inclined cracks than C/2.7/G3.82, although both beams were strengthened with 3 DE 218 219 bars. This can be attributed to the higher elastic modulus of CFRP bars as explained in the preceding section. 220

The cracking behaviour of S/2.0/G1.91 was comparable to that of the remaining beams up to a shear force of about 60 kN. Upon further loading, the inclined cracks became stable whereas the flexural cracks in the maximum moment zone started to propagate. Eventually, failure took place due to crushing of the concrete adjacent to the loading point.

#### 4.4. Components of Shear Resistance

Strain gauge readings were used to calculate the contributions of the steel and FRP shear 226 reinforcement to the shear force capacity (see Fig. 5). All steel shear links attained or exceeded 227 the yield strain of 0.0027. As the shear crack that caused failure always intersected the steel 228 shear links, the steel contribution  $(V_s)$  to shear resistance was calculated as the yield strength 229 of the shear links (540 MPa) multiplied by the cross-sectional area of the two shear links (50.2 230 231  $mm^2$ ). The FRP contribution (V<sub>f</sub>) to shear resistance was based on the strain in the DE FRP bars intersected by the main shear crack that caused failure. The strain in these bars was 232 233 multiplied by the axial stiffness (E<sub>f</sub>A<sub>f</sub>) of the FRP bars. For example, V<sub>f</sub> in S/2.7/G1.91 was calculated based on the strain in the DE FRP bars G3, G4, G5 and G6 whereas the strain in G1 234 and G2 was ignored because these two bars were not intersected by the main shear crack. The 235 concrete contribution (V<sub>c</sub>) was calculated as the total shear force capacity minus the sum of V<sub>s</sub> 236 and V<sub>f</sub>. 237

Fig. 5 presents the components of shear resistance versus shear force for the beams that failed 238 in shear. The steel shear links were inactive up to the formation of inclined cracks and the shear 239 force was resisted by concrete only. After the formation of inclined cracks, the concrete 240 contribution started to diminish with increased loading. Before yielding, the shear links in the 241 strengthened beams resisted lesser shear force than those in the corresponding control beam. 242 This is attributable to the presence of the DE FRP bars, which resisted crack opening and thus 243 reduced the forces in the shear links. Similar to the steel shear links, the DE FRP bars were 244 inactive up to the formation of inclined cracks. However, the DE FRP bars started to contribute 245 significantly to shear resistance only after the yield of the steel shear links. There were no signs 246 of FRP bar debonding up to peak shear force in all tested beams. 247

An important implication of the results shown in Fig. 5 is that the subtraction of the unstrengthened shear force capacity from the strengthened shear force capacity does not always

give a correct estimate of V<sub>f</sub>. The overall shear strength gain for S/2.0/G3.82 and S/2.7/G3.82 250 was negligible (see Table 2). Yet, the strain-based V<sub>f</sub> values for these two beams were 28.6 and 251 32.3 kN, respectively. Similarly, the overall shear strength gain for S/2.7/G1.91 and 252 S/2.7/C1.35 was 15 and 28.1 kN, respectively, whereas the strain-based V<sub>f</sub> values were 53.9 253 and 41.6 kN, respectively. This discrepancy occurs because Vc in the strengthened beams 254 reduces with increased loading. The shear strength gain reported in Table 2 is therefore the 255 256 difference between the strain-based V<sub>f</sub> values and the reduction in V<sub>c</sub> values (compared with the corresponding control beam). For example, the shear strength gain for S/2.0/G3.82 is 28.6 257 258 kN - (38.4 kN - 13.0 kN) = 3.2 kN.

Figs. 6(a) and 6(b) show the experimental variations in Vc and Vf, respectively, with steel-to-259 FRP shear reinforcement ratio. The concrete contribution decreased from 34.9 to 9.3 kN with 260 increasing the steel-to-FRP shear reinforcement ratio from 1.35 to 3.82. The relatively higher 261 axial stiffness of the CFRP bars (3674 kN per bar) in S/2.7/C1.35 controlled crack opening and 262 resulted in a relatively high V<sub>c</sub> value (34.9 kN). On the other hand, the GFRP bars in the 263 remaining 3 beams that failed in shear were much less effective in controlling crack width and 264 maintaining aggregate interlock. As a result, the GFRP-strengthened beams had relatively low 265 V<sub>c</sub> values that varied from 9.3 to 13.0 kN. 266

The FRP contributions for S/2.7/C1.35 and S/2.7/G1.91 were 41.6 and 53.9 kN, respectively. 267 These values are significantly higher than the corresponding values of 32.3 and 28.6 kN 268 obtained for S/2.7/G3.82 and S/2.0/G3.82, respectively. This result shows that the FRP 269 contribution also decreases with increasing steel-to-FRP shear reinforcement ratio. As 270 explained earlier, this is due to the higher elastic modulus of steel, which resulted in the steel 271 shear links attracting higher forces than the DE FRP bars. Given its implication for the shear 272 strength enhancement, the influence of steel-to-FRP shear reinforcement ratio is further 273 investigated numerically. 274

#### 275 **5. Finite Element Modelling**

A two-dimensional nonlinear FE model was developed using VecTor2 software package. VecTor2 is based on the Disturbed Stress Field Model (DSFM) [16], an extension of the Modified Compression Field Theory (MCFT) [17]. It utilizes a rotating smeared-crack approach to predict the structural behaviour of RC membrane elements. Further details on VecTor2 can be found elsewhere [18].

281

282 5.1. Mesh and Element Details

Two-dimensional four-node rectangular plane stress elements with two degrees of freedom at each node were used for the concrete. The concrete mesh size in each direction was taken as  $2.5d_a$  (where  $d_a$  is the maximum aggregate size). This is broadly consistent with the recommendation of Bažant and Oh [19] to use a concrete mesh size of  $3d_a$ . Moreover, Dirar et al. [20] successfully used a mesh size of  $2.5d_a$  to model FRP shear-strengthened RC T-beams comparable to those reported in this paper. For convenience, the loading and support steel plates had the same element type and size as the concrete.

The steel bars were modelled as discrete reinforcement using two-node truss elements with two 290 degrees of freedom at each node. Bond failure between the concrete and the steel bars was not 291 the governing failure mode of the tested beams. Perfect bond was therefore assumed between 292 the concrete and the steel reinforcement. A similar approach was successfully used by Qapo et 293 al. [13] to model RC T-beams strengthened in shear with DE FRP bars. There is still the 294 potential for localized slip between the reinforcement and surrounding concrete in the tested 295 beams. However, this does not affect the overall predicted behaviour as demonstrated by the 296 comparison between the experimental and numerical results (see FE Model Validation section). 297 The DE FRP bars were modelled using two-node truss elements. Two-node interface elements 298 were used to link the truss elements representing the DE FRP bars to the plane stress elements 299

300 representing the concrete. This allowed the bond behaviour between the DE FRP bars and301 surrounding concrete to be modelled.

302

#### 303 5.2. Material Modelling

The concrete in compression was modelled by Thorenfeldt's et al. [21] stress-strain curve, which is given by:

$$306 f_{ci} = -\left(\frac{\varepsilon_{ci}}{\varepsilon_p}\right) f_p \frac{n}{n-1+\left(\frac{\varepsilon_{ci}}{\varepsilon_p}\right)^{nk}} (1)$$

where  $f_{ci}$  (MPa) represents the concrete compressive stress at a given strain  $\varepsilon_{ci}$  (mm/mm);  $f_p$ (MPa) is the concrete cylinder compressive strength and  $\varepsilon_p$  (mm/mm) is the corresponding strain; *n* is a parameter equal to  $0.8 + (f_p/17)$  and *k*, taken as  $0.67 + (f_p/62)$ , is a parameter governing the descending branch of Eq. (1). The softening of concrete in compression caused by lateral cracking was incorporated by adopting the model developed by Vecchio and Collins [22]. Poisson's ratio of concrete was taken as 0.15 based on the recommendation of CEB-FIB Model Code 1990 [23].

The concrete behaviour in tension was assumed linear-elastic prior to concrete cracking. A bilinear tension softening model [23] was used to model the post-cracking behaviour of concrete. The fracture energy ( $G_F$ ) of concrete was calculated according to Eq. 2, also given by CEB-FIB Model Code 1990 [23].

318 
$$G_F = G_{Fo} \left(\frac{f_{cm}}{f_{cmo}}\right)^{0.7} (N/mm)$$
(2)

where  $G_{Fo}$  is the base value of fracture energy (taken as 0.026 N/mm for a maximum aggregate size of 10 mm);  $f_{cm}$  is the concrete compressive strength (MPa) and  $f_{cmo}$  is equal to 10 MPa. Bentz's model [24] was used to simulate tension stiffening.

322 An explicit model for shear transfer in cracked concrete was not required because, in the 323 rotating crack model, crack direction changes with the change in direction of the principal tensile stress. It follows that any crack plane in the rotating crack model is a principal planewith no shear stress.

The stress-strain model for the steel reinforcement as well as the loading and support plates had an initial linear-elastic response followed by a yield plateau and a nonlinear strainhardening phase up to rupture. For the DE FRP bars, a linear-brittle stress-strain model, based on the ultimate strength values reported in Table 1, was used.

The bond-slip results reported by Valerio et al. [6] were used to represent the FRP-to-concrete bond behaviour. These results were selected because they were based on the same epoxy adhesive type as that used in this study. The considered bond-slip tests were carried out on 7.5 mm diameter FRP bars, which are slightly larger than the 6 mm diameter FRP bars used in the tested beams. However, this had insignificant implications for the modelled behaviour (see *FE Model Validation* section).

336

337 5.3. Solution algorithm

VecTor2 utilizes an incremental-iterative algorithm to solve the nonlinear equations. A
displacement control approach was used where the load was applied in increments of 0.1 mm.
For each increment, the secant stiffness was used to iteratively search for equilibrium.
Convergence was successfully achieved at the end of each increment using this procedure.

342

#### 343 6. FE Results and Discussion

344 6.1. FE Model Validation

The FE predictions in terms of shear force at failure and deflection at peak load together with the corresponding experimental results are presented in Table 3. The FE predictions were in good agreement with the experimental results. The mean value of the predicted-toexperimental shear force at failure is 1.01 with a standard deviation of 0.07, demonstrating the accuracy of the FE model. The corresponding values for the deflection at peak shear force,
excluding S/2.0/G1.91 which failed in flexure, were 0.97 and 0.10, respectively.

Fig. 7 shows that the FE model successfully predicted the overall deflection response of the 351 tested beams. Similar to the experimental results, the predicted shear force-deflection curves 352 were quasilinear prior to cracking. The pre-cracking stiffness was accurately predicted, 353 indicating that the boundary conditions and elastic constants were well modelled. 354 355 Subsequently, the shear force-deflection curves turned nonlinear due to stiffness deterioration caused by cracking. Upon further loading, the post-cracked stiffness continued to deteriorate 356 357 up to failure. The ductile failure of S/2.0/G1.91, characterised by a plateau at peak load, as well as the brittle failure of the remaining six beams, characterised by a sudden drop in load, were 358 accurately predicted. 359

Following the successful validation of the FE model, it was used to obtain further insight intothe effect of test parameters on the strengthened behaviour.

362

363 6.2. Effect of Steel-to-FRP Shear Reinforcement Ratio

Making use of the validated FE model, a parametric study was conducted to further examine the interaction between DE FRP bars and existing steel shear links. The beams considered in the parametric study were nominally identical to the tested beams but had steel-to-FRP shear reinforcement ratios in the range from 0.17 to 4.35, obtained by changing the diameter and spacing of existing steel shear links and/or the dimeter, spacing and type of DE FRP bar. All modelled beams failed in shear after yielding of the existing steel shear links.

Fig. 8 shows that the variation of steel-to-FRP shear resistance ratio  $(V_s/V_f)$  with steel-to-FRP shear reinforcement ratio is bilinear. The DE FRP contribution far exceeds the steel contribution, resulting in  $V_s/V_f$  values in the range from 0.08 to 0.22, for steel-to-FRP shear reinforcement ratios in the range from 0.17 to 0.78. With increasing the steel-to-FRP shear reinforcement ratio from 0.78 to 2.75, the gap between the DE FRP and steel contributions decreases, resulting in  $V_s/V_f$  values in the range from 0.22 to 0.53. The steel-to-FRP shear resistance ratio remains almost constant for steel-to-FRP shear reinforcement ratios higher than 2.75. This implies that the steel-to-FRP shear reinforcement ratio should be designed to be well below 2.75 in order to maximise the DE FRP contribution.

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#### 380 6.3. Effect of Tension Reinforcement Ratio

The experimental results showed that tension reinforcement ratio influenced failure mode but not gain due to DE FRP bars (see Table 2). The effect of tension reinforcement ratio on failure mode was further investigated by modelling DE FRP shear-strengthened beams nominally identical to the tested beams but with tension reinforcement ratios in the range from 0.45 to 4.15%, obtained by changing the diameter of tension steel bars.

Fig. 9 shows that the variation of normalised moment capacity (i.e. moment at failure  $(M_u)$ divided by flexural capacity  $(M_f)$ ) with tension reinforcement ratio is also bilinear. A normalised moment capacity of 1.00 denotes flexural failure whereas  $M_u/M_f$  values less than 1.00 denote shear failure. All strengthened beam models with a tension reinforcement ratio less than 2.0% failed in flexure. On the other hand, all strengthened beam models with a tension reinforcement ratio more than 2.0% failed in shear. This finding clarifies the effect of tension reinforcement ratio on failure mode of DE FRP shear-strengthened beams.

393

#### 394 7. Evaluation of TR55 Design Model

395 TR55 [14] is currently the sole standard document covering design of DE FRP shear 396 strengthening systems. TR55 ignores the concrete contribution and assumes that the total shear 397 force capacity (V<sub>t</sub>) is composed of the steel and DE FRP contributions, V<sub>s</sub> and V<sub>f</sub>, respectively. 398  $V_t = V_s + V_f$  (3)

399 The steel contribution is given by:

$$400 \quad V_s = 0.78 \frac{A_{sw}}{s} df_y \cot\theta \tag{4}$$

401 where *d* is the beam effective depth,  $f_y$  is the yield strength of the steel shear reinforcement and 402  $\theta$  is the inclination angle of the concrete struts.

403 The DE FRP contribution is given by:

$$404 V_f = \frac{\varepsilon_{fse}E_{fd}A_f}{s_f} W_{eff} (5)$$

where  $\varepsilon_{fse}$  is the effective strain in the DE FRP bars (taken as 0.004 mm/mm),  $E_{fd}$  is the design Young's modulus of the DE FRP bars (MPa) and  $W_{eff}$  is the effective width (mm) over which the DE FRP bars may act and given by:

408 
$$W_{eff} = (h - 2l_{b,max})$$
 (6)

409 where *h* is the strengthened depth (mm) and  $l_{b,max}$  is the maximum anchorage length (mm) 410 beyond which no additional capacity gain can be achieved, given by:

411 
$$l_{b,max} = \frac{\varepsilon_{fse}E_{fd}A_f}{(\pi * d_b * \frac{\tau_b}{\gamma_A})}$$
(7)

412 where  $d_b$  is the DE FRP bar diameter (mm),  $\tau_b$  is the average bond stress (MPa) over the 413 anchored length and can be taken as 15 MPa in the absence of test data and  $\gamma_A$  is a partial safety 414 factor for the adhesive material.

TR55 suggests that the steel and DE FRP shear contributions should be evaluated concurrently as they are integral parts of the total shear force capacity. Table 4 compares the FE results  $(V_{t,FE})$  and TR55 predictions  $(V_{t,TR55})$  with the total experimental shear force capacity  $(V_{t,Exp})$ . All safety factors are set equal to 1.00 for the purpose of comparison. Both the FE results and TR55 predictions had comparable standard deviations values (0.08 and 0.05, respectively), indicating small scatter of the predictions. However, TR55 design model significantly underestimated the total shear force capacity with a mean predicted-to-experimental value of 422 0.42. As previously demonstrated, the FE model provided accurate predictions of the total shear
423 force capacity with a mean predicted-to-experimental value of 1.04.

One reason for the conservative predictions of TR55 design model is that it takes the effective strain in the DE FRP bars as 0.004 mm/mm. However, the experimentally measured strain values in the DE FRP bars intersected by the main shear cracks ranged from 0.005 to 0.015 mm/mm. Another reason is that TR55 assumes a fixed value for average bond stress whereas Caro et al. [25] demonstrated that it depends on many variables (e.g. concrete strength, DE FRP bar diameter and elastic modulus, adhesive type and embedded length).

430 A further shortcoming of the TR55 design model is that it does not consider the effect of steelto-FRP shear reinforcement ratio. Fig. 10 shows the variations of total shear force capacity, 431 predicted by the FE and TR55 models, with steel-to-FRP shear reinforcement ratio. The FE 432 model, which has been demonstrated to accurately represent the experimental results, predicted 433 a 25.1% decrease in total shear force capacity (from 104.7 to 78.4 kN) with the increase in 434 steel-to-FRP shear reinforcement ratio from 0.17 to 4.35. The decrease in total shear force 435 capacity is caused by the reduction in DE FRP contribution with increasing steel-to-FRP shear 436 reinforcement ratio (see Effect of Steel-to-FRP Shear Reinforcement Ratio section). On the 437 other hand, TR55 predictions did not show clear sensitivity to steel-to-FRP shear reinforcement 438 ratio. 439

440

#### 441 8. Conclusions

This paper presents results of an experimental and FE investigation on the structural behaviour of RC T-beams strengthened in shear with embedded FRP bars. Moreover, it provides insights into the influence of longitudinal and transverse reinforcement ratios on the structural behaviour of the strengthened beams. Furthermore, it evaluates the accuracy of the Concrete Society's Technical Report 55 shear strengthening design model for embedded FRP reinforcement. Based on the experimental and numerical results, the following conclusions aredrawn:

- The total shear force capacity as well as the DE FRP and concrete contributions to shear
   strength decreased with increasing steel-to-FRP shear reinforcement ratio. Thus,
   calculating the DE FRP shear resistance as the difference between the strengthened and
   unstrengthened shear force capacities can lead to erroneous results.
- DE FRP bars should be designed in such a way that the steel-to-FRP shear
   reinforcement ratio is less than 2.75 in order to exploit DE FRP shear strengthening
   systems.
- The tension reinforcement ratio had a clear effect on failure mode. Tested and modelled
   strengthened beams with a tension reinforcement ratio equal to or less than 2.0% failed
   in flexure whereas tested and modelled strengthened beams with higher tension
   reinforcement ratios failed in shear. However, the tension reinforcement ratio did not
   influence the gain due to FRP bars.
- The control and GFRP-strengthened beams had comparable cracked stiffness whereas
   the CFRP-strengthened beam had higher cracked stiffness, demonstrating the
   effectiveness of CFRP bars in controlling crack opening.
- TR55 design model did not consider the effect of steel-to-FRP shear reinforcement ratio
   and underestimated the total shear force capacity with a mean predicted-to experimental ratio of 0.42. On the other hand, the proposed FE model had a mean
   predicted-to-experimental ratio of 1.04 and correctly captured the reduction in shear
   strength with increasing steel-to-FRP shear reinforcement ratio.
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			8, 10 and 12	4 mm steel	6 mm sand-coated	6 mm sand-coated
	Property	Concrete	mm steel bars	bars	GFRP bars	CFRP bars
	Elastic Modulus (MPa)	-	200000	200000	46000	130000
	Cylinder / cube					
	Compressive	41 / 49	-	-	-	-
	Strength (MPa)					
	Ultimate Strain				0.019	0.017
	(mm/mm)				0.017	0.017
	Yield Strength		580	540		
	(MPa)		500	510		
	Ultimate Strength	-	680	680	900	2300
	(MPa)			000		
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### Table 2. Experimental Results

Beam	Unstrengthened shear force capacity (kN)	Shear force at failure (kN)	Gain due to DE FRP bars (kN)	Gain due to DE FRP bars (%)	Failure mode
C/2.0	65.5	65.5	-	-	Shear
S/2.0/G3.82	65.5	68.7	3.2	4.8	Shear
S/2.0/G1.91	65.5	78.5	13	19.8	Flexur
C/2.7	75.5	75.5	-	-	Shear
S/2.7/G3.82	75.5	68.7	0	0	Shear
S/2.7/G1.91	75.5	90.5	15	19.8	Shear
S/2.7/C1.35	75.5	103.6	28.1	37.2	Shear

**Table 3.** Comparison between experimental results and FE predictions

Beam	Shear force at failure (kN)			Deflection at peak shear force (mm)		
-	Experimental	FE Prediction	FE/Exp.	Experimental	FE Prediction	FE/Exp.
C/2.0	65.5	63.5	0.97	10.7	10.3	0.96
C/2.7	75.5	70.4	0.93	10.1	10.5	1.04
S/2.0/G3.82	68.7	72.2	1.05	11.2	10.4	0.93
S/2.7/G3.82	68.7	78.2	1.14	11.6	10.6	0.91
S/2.0/G1.91*	78.5	82.1	1.05	>50.0*	>25.0*	_ *
S/2.7/G1.91	90.5	91.0	1.01	14.7	12.4	0.84
S/2.7/C1.35	103.6	97.4	0.94	11.3	12.6	1.12
Mean			1.01			0.97
Standard			0.07			0.10
deviation						
6 *Flexural failur	re					
7						
o						
0						
9						
0						
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	Total shear force capacity (kN)					
Beam	V <sub>t,Exp</sub>	$V_{t,FE}$	V <sub>t,TR55</sub>	V <sub>t,FE</sub> /	V <sub>t,TR55</sub> /	
				$V_{t,Exp}$	V <sub>t,Exp</sub>	
S/2.0/G3.82	68.7	72.2	31.5	1.05	0.46	
S/2.7/G3.82	68.7	78.2	31.5	1.14	0.46	
S/2.7/G1.91	90.5	91.0	36.5	1.01	0.40	
S/2.7/C1.35	103.6	97.4	37.3	0.94	0.36	
Mean				1.04	0.42	
Standard deviation				0.08	0.05	

Table 4. Comparison between experimental, numerical and TR55 results