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Seismic rehabilitation of reinforced concrete beam-column joints by bonding with concrete covers and wrapping with FRP composites

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Seismic rehabilitation of reinforced concrete beam-column joints by bonding with concrete covers and wrapping with FRP composites

Abstract

This study evaluates the effectiveness of a new method for seismic strengthening of exterior RC beam-column connections and develops a joint shear strength model for the strengthened connections. Four RC exterior connections without transverse reinforcement at the joints were cast and tested under reverse cyclic loading. The first connection was tested as the control specimen while the three remaining connections were glued with concrete covers around the columns at the joint area to modify them from square to circular sections and then they were wrapped with different ratios of CFRP for strengthening. The joint shear strength model was developed based on average plane stress concept and it was evaluated with a collected database containing 32 connections strengthened with the conventional FRP methods and three connections strengthened with the new method. Experimental results showed that the variation of the CFRP ratios wrapped around the specimens led to different failure modes of the examined connections; despite the differences in the failure modes, shear capacity and seismic performance of the strengthened connections improved significantly. The new strengthening method could eliminate two of the primary disadvantages (debonding and/or bulging of FRP from the concrete surface and a low confinement effect) of the existing method. The experimental and analytical results showed that the new strengthening method improved the ability of FRP and concrete working together to resist the joint shear forces. The proposed model predicted the joint shear strength of the FRP-strengthened connections accurately. Due to their superior performance, the proposed strengthening method and the proposed model are expected for practical application.

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1 **Seismic Rehabilitation of Reinforced Concrete** 2 **Beam-Column Joints by Bonding with** 3 **Concrete Covers and Wrapping with FRP** 4 **Composites**

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13 **Abstract**

14 Purpose

15 This study evaluates the effectiveness of a new method for seismic strengthening of exterior RC
16 beam-column connections and develops a joint shear strength model for the strengthened
17 connections.

18 Methods

19 Four RC exterior connections without transverse reinforcement at the joints were cast and tested
20 under reverse cyclic loading. The first connection was tested as the control specimen while the
21 three remaining connections were glued with concrete covers around the columns at the joint
22 area to modify them from square to circular sections and then they were wrapped with different
23 ratios of CFRP for strengthening. The joint shear strength model was developed based on average
24 plane stress concept and it was evaluated with a collected database containing 32 connections
25 strengthened with the conventional FRP methods and 3 connections strengthened with the new
26 method.

27 Results

28 Experimental results showed that the variation of the CFRP ratios wrapped around the specimens
29 led to different failure modes of the examined connections; despite the differences in the failure
30 modes, shear capacity and seismic performance of the strengthened connections improved
31 significantly. The new strengthening method could eliminate two of the primary disadvantages
32 (debonding and/or bulging of FRP from the concrete surface and a low confinement effect) of the
33 existing method. The experimental and analytical results showed that the new strengthening
34 method improved the ability of FRP and concrete working together to resist the joint shear
35 forces. The proposed model predicted the joint shear strength of the FRP-strengthened
36 connections accurately.

37 Conclusions

38 Due to their superior performance, the proposed strengthening method and the proposed model
39 are expected for practical application.

40 **Keywords** FRP, Beam-column, Connections, Strengthening, Shear failure

41

42 Introduction

43 A significant portion of reinforced concrete (RC) buildings, which are the most popular
44 structures in the world, were constructed without properly considering the seismic
45 effects that pose a significant risk to human beings, especially when earthquakes occur.
46 Recent earthquakes showed that beam-column connection with insufficient transverse
47 reinforcement often failed by brittle shear failure with 'X' shape cracks [1]. This type of
48 failure made the beam-column connections become the most susceptible component of
49 RC frames because it generally leads to the whole structure collapsing. Therefore, to
50 improve the safety of the existing RC structures, it is essential to improve the strength
51 and ductility of beam-column connections so that the ductile flexural hinge formed in
52 the beam was archived under cyclic loading.

53 Some techniques including epoxy repair, removal and replacement, concrete jacketing,
54 concrete masonry unit jacketing and steel jacketing have been proposed for
55 strengthening RC beam-column connections in recent years. The application of these
56 conventional techniques could improve the performance of the strengthened
57 connections but they also have some limitations [2]. Shannag, et al. [3] proposed a
58 strengthening method using high performance fiber reinforced concrete while Hadi [4]
59 proposed a method using steel straps. However, their methods were complicated and
60 could not fully restore the performance of the destructed RC connections. Strengthening
61 beam-column joints by planar joint expansion [5] could improve seismic performance of
62 connections but it was complicated and significantly increased the joint volume.
63 Preponderantly, externally bonded FRP has been recognized as the ideal technique for
64 strengthening beam-column joints.

65 Many researchers [6-11] have presented experimental studies on retrofitting beam-
66 column joints using FRP composites. They concluded that the application of externally
67 bonded FRP was able to improve the shear resistance, the stiffness and the ductility of
68 beam-column connections. The efficiency of the FRP retrofitting method depends on
69 the ability of the anchorage systems to delay or prevent debonding and/or bulging of
70 FRP from the concrete surface at the joint area. Unexpectedly, FRP debonding was
71 observed in most of the experimental studies that used FRP for strengthening beam-
72 column connections. The debonding of FRP on specimens tested by Ghobarah and Said
73 [12], Antonopoulos and Triantafillou [8], Li and Chua [13], Akguzel and Pampanin [9] and
74 Al-Salloum, et al. [14] are typical examples. Although many methods have been
75 proposed to stop or delay debonding of externally bonded FRP layers, it was still one of
76 the reasons for failure of the tested specimens [10].

77 **Beside experimental studies, significant effort has been paid on developing models for**
78 **predicting the shear strength of FRP-strengthened beam-column connections.** Gergely,
79 et al. [15] and Ghobarah and Said [7] introduced two design methods for calculating the
80 required amount of externally bonded FRP where the role of externally bonded FRP was
81 proposed to be equivalent to external steel stirrups and therefore the shear strength of
82 a retrofitted joint could be predicted relatively simply. Based on Pantazopoulou and
83 Bonacci [16] model, Antonopoulos and Triantafillou [17] and Almusallam and Al-Salloum
84 [18] developed models to account for the contribution of FRP on the shear behaviour of

85 joints but while they could predict the joint shear behaviour at any loading states, they
86 were too complicated. Tsonos [19] proposed an analytical model where the confined
87 effect of FRP on the shear performance of a retrofitted connection was exaggerated.
88 Bousselham [20] proposed a design model to predict the contribution made by FRP
89 based on deriving an expression for the effective FRP strain. Akguzel and Pampanin [21]
90 developed an analytical model in which the principal tension stress at the joint was
91 suggested to be the key criterion that controls the shear strength of the retrofitted
92 joints. Crushing of the concrete caused by principal compression stress at the joint,
93 therefore, was inadequately considered. Besides analytical modelling, computational
94 modelling was also developed for predicting the joint shear strength but it is
95 complicated for practical implication.

96 It is a fact that the increased confinement of a joint caused by externally bonded FRP
97 lead to improvement in the performance of the strengthened beam-column connections
98 under cyclic loads. However, in most of the previous FRP-strengthening techniques, FRP
99 was wrapped at the beam-column connections around square or rectangular sections of
100 the beams and the columns, which created little confinement effect. Recently, a new
101 method of retrofitting exterior beam-column connections (T connections), which could
102 improve the confined effect of CFRP, has been proposed by Tran et al. [22] and Hadi and
103 Tran [23]. In these studies, to retrofit a RC T connection, the column of the joint was
104 firstly circularized by bonding concrete covers and then it was wrapped with CFRP. The
105 test results of these studies showed that wrapping CFRP around the modified circular
106 section increases the efficiency of CFRP confinement and reduces the possibility of CFRP
107 bulging at the joint area. It is noted that the advantages of shape modification have
108 been successfully proven by recent studies carried out by Hadi et al. [24] and Pham et al.
109 [25] on strengthening square columns by circularizing and wrapping with CFRP.

110 Although to be considered as a promising technique, the effectiveness of Hadi and Tran
111 method [23] for strengthening T connections was not evaluated thoroughly. It is
112 because only two retrofitted specimens (one strengthened and one repaired) and no
113 reference specimen were tested. In order to validate the effectiveness of this method, a
114 total of four identical RC T connections were tested in this study, one was used as the
115 control specimen and the three remaining connections were strengthened with the
116 proposed technique, but with different CFRP ratios. The variation of the CFRP ratio aims
117 at creating different failure modes (i.e. joint shear (JS) failure, flexural beam hinge (BH)
118 failure, and a combination of JS and BH failure mode) of the tested specimens. The focus
119 on the specimen that failed in BH mode was mainly on how affective the strengthening
120 method was with regards to the behaviour of the entire beam-column connection,
121 including the beam, the columns, and the joint, but for the remaining specimens, the
122 focus was mainly on how the joints reacted. It is noted that experimental investigations
123 focusing on FRP retrofitted specimens that failed in JS mode are rare and thus, they are
124 need [20].

125 In addition, an analytical model for estimating the shear strength of FRP-retrofitted T
126 connections was also developed in this study for practical application. It was applied to
127 predict the shear strength of the connections tested in this study. The comparisons
128 between the predicted and the experimental results have shown that the predicted joint

129 shear strength fitted well with the experimental ones. In order to evaluate the reliability
130 of the proposed model, comparisons with experimental results of 32 connections
131 strengthened using the conventional FRP methods were also made. These comparisons
132 demonstrated the accuracy of the proposed model.

133 Description of the specimens

134 Four **full-scale** RC T connections were cast and tested. The first connection, denoted as
135 Specimen T0, was used as the control specimen, while the three remaining connections,
136 denoted as Specimens TS, TS1 and TS2 were strengthened using the proposed method.
137 The connections were identical and were designed without any joint transverse
138 reinforcement detail at the joints area to represent those connections designed without
139 considering the seismic effect. The dimensions and details of reinforcement used on the
140 four specimens are shown in Fig. 1. The column was 2800 mm high with cross sectional
141 dimensions of 200x200 mm. The beam's length was 1400 mm from the face of the
142 column to the free end and had a cross section of 200x300 mm. Six N16 (16 mm
143 deformed bars with 500 MPa nominal tensile strength) bars were used to reinforce the
144 column longitudinally, while the beam was reinforced longitudinally at the top and
145 bottom with four N12 bars. **The amount of column ties and beam stirrups was calculated**
146 **and detailed based on AS-3600 [26] to ensure that a shear failure mechanism could not**
147 **occur in the beam and columns outside the joint center when their ultimate flexural**
148 **capacity were reached. Based on this design principle, R10 (10 mm plain bars with 250**
149 **MPa nominal tensile strength) were used for the ties and the stirrups of the column and**
150 **the beam. The column ties were at 100 mm intervals, the beam stirrups were 75 mm**
151 **apart for the first 650 mm from the column, 150 mm apart for the next 450 mm. As the**
152 **end of the beam was designed to connect with the hydraulic actuator, the amount of**
153 **beam stirrups was increased and it was 83 mm apart for the last 300 mm of the beam.**

154 Normal strength concrete supplied by a local supplier, with a 10 mm maximum
155 aggregate size and a 120 mm slump was used for casting the specimens. All of the
156 specimens were cast horizontally rather than vertically as in a real building construction.
157 **Compression tests were performed in accordance with AS 1012 [27] on three 100 mm**
158 **diameter and 200 mm high cylinder samples for each batch of concrete. The average**
159 **cylindrical compressive strengths of the concrete of Specimens T0, TS, TS1 and TS2 at**
160 **the time of testing each specimen were 41 MPa, 50 MPa, 41 MPa and 44 MPa,**
161 **respectively. The average yield stresses of the N16, N12, R10 were 550, 551 and 322**
162 **MPa, while their average ultimate stresses were 647, 654 and 485 MPa, respectively.**

163 **It is noted that, with the aim of casting strong beam and columns but weak joint**
164 **connections, the concrete compressive strength was designed to be 40 MPa. This value**
165 **as well as the actual concrete strength may be higher than that of the real existing RC**
166 **structures but with the above design, the 90-degree hooks at the end of the longitudinal**
167 **beam bars had a straight extension at the tail of the hooks of a length less than the**
168 **required value given by ACI-352R-02 [28]. Moreover, the joint shear stresses were $v_{jh} =$**
169 **$1.12 \sqrt{f'_c}$ (calculated using the design material properties and a strength reduction**
170 **factor of unity) whereas, for RC exterior joints, ACI-352R-02 [28] required $v_{jh} \leq$**

171 $1.0\sqrt{f_c'} MPa$. Additionally, the joint shear strength of the designed connections
172 calculated using Tran et al. [29] model was $0.52\sqrt{f_c'} MPa$. As the designed specimens
173 did not meet the requirement of the ACI-352R-02 [28] and the calculated joint shear
174 stress is higher than the predicted shear strength, the designed original specimens were
175 expected to fail in joint shear mode, and thus they need to be strengthened at the joint
176 location.

177 Unidirectional CFRP was used to strengthen the T connections. The type of CFRP used in
178 this study was Carbon-uni-fabric sheet and the applied resin was a mixture of epoxy
179 resin R105 and hardener R206 at a weight ratio of 5:1. The coupon tests showed that
180 the fibre has an elastic modulus of $E_f = 235$ GPa and an ultimate strain $\varepsilon_{fu} = 1.79$, while
181 these values provided by the manufacturer are 240 GPa and 1.8%, respectively. The
182 strengthening procedure of Specimens TS, TS1 and TS2 included two stages. In the first
183 stage, the columns of the connections were bond with the segmental circular concrete
184 covers using a mix of epoxy resin and 20% thickener to modify the columns of the
185 connections at the joint area from square to circular sections. Before they were bonded,
186 the surfaces of the specimen around the beam-column joint area and of the segmental
187 circular concrete covers were ground with an electric grinder and then cleaned with air
188 blasting to ensure the contact surfaces were smooth. Details of the bonding technique
189 can be seen in Hadi and Tran [23]. In the second stage, the modified connections were
190 externally bonded with CFRP sheets in a four-steps procedure as follow. Firstly, two
191 CFRP sheets were wrapped vertically onto two opposite faces of the segmental circular
192 concrete covers for a distance of 900 mm, secondly CFRP layers were wrapped around
193 the column for the width of the joint (300 mm) extending 300 mm into the length of the
194 beam, then the additional sheets of CFRP were applied onto two opposite sides of the
195 beam. Finally, CFRP sheets were applied around the beam and columns close to the joint
196 area to anchor the vertical and horizontal CFRP. The strengthening configuration and
197 process for Specimen TS1 and TS2 were the same as Specimen TS but more sheets of
198 CFRP were applied at the beam-column interface, but carbon fibre with different
199 thicknesses. The additional sheets of CFRP were t_{fhi} thick, and were applied onto
200 opposite sides of the beam and extended 100 mm into the column (Fig. 2) to help
201 prevent the CFRP from breaking at the beam-column interface, and thus increasing its
202 effectiveness. A summary of the CFRP application on Specimens TS1, TS2 and TS is
203 shown in Fig. 2 and Table 1. In this figure, the horizontal Carbon fibre wrapped around
204 the joint was t_{fjh} thick, t_{fjv} and t_{fhc} thick along and around the columns, respectively. The
205 anchorage fibre around the beam was t_{fb2} thick for the first 100 mm close to the column,
206 and t_{fb1} thick for the next 100 mm. For easy identification, images of the specimen after
207 completion of the first and the second stages are presented as shown in Fig. 3.

208 It can be seen from Fig. 3 that the above method can be applied to strengthen the
209 beam-column connections of RC bridges or RC frames of buildings in which the
210 transverse beams are not connected directly to the beam-column joint but are
211 connected to the main beam at a certain distance from the column face, for example
212 when there are openings in slabs or when the edge-side columns of the building are not
213 connected directly to the transverse beams. These features of the connections may
214 eliminate the application of the proposed strengthening method in practice but it is

215 noted that under seismic load, this type of exterior joint is weaker than joints with
216 transverse beams connected directly into the column because the existence of these
217 elements at the joint location improve the joint performance. Therefore, this type of
218 beam-column joint shows a real need to be strengthened.

219 **Test set up**

220 A strong frame was used to test the specimens and the test set up is shown in Fig. 4. The
221 connections were tested in the column vertical position. Two restrainers placed at a
222 distance of 2200 mm were provided to resist the horizontal forces near the top and
223 bottom ends of the columns. No compression load was applied on the columns to
224 evaluate the worst-case circumstance of the connections as explained by Quintero and
225 Wight [30] because an increase in column axial load leads to an improvement in joint
226 shear performance [31], [32]. The vertical cyclic load was applied 1100 mm away from
227 the beam-column interface using a 600 kN hydraulic actuator. The cyclic load was
228 applied slowly with a deflection rate of 5 mm per minute and the beam's free end
229 displacement was controlled. The amplitudes of the peaks in the displacement loading
230 history were increased from 10 mm to 90 mm with 10 mm steps as shown in Fig. 5.

231 **Test Results**

232 **Behaviour of the specimens**

233 *Specimen T0*

234 Specimen T0 exhibited, as expected, premature shear failure during the early stages of
235 cyclic loading. A maximum load of 35.8 kN was reached at the peak of the 2nd loading
236 cycle when diagonal cracks developed at the centre of the joint. In the next loading
237 cycles, the peak loads reduced rapidly, the diagonal cracks opened widely and spread
238 into the columns to connect with the bond-splitting cracks along the column longitudinal
239 bars, which extended approximately 450 mm both above and below the centre of the
240 joint. These cracks opened widely and connected with the joint diagonal cracks to cause
241 a serious shear failure of the connection. This development of diagonal and bond-
242 splitting cracks caused a serious shear failure of the connection. No flexural cracks
243 occurred in the beam or the columns near the joint core, but when testing ended, the
244 concrete at the joint core was completely crushed. The measured shear force versus
245 beam end deflection and the final cracking patterns of Specimen T0 are shown in Fig. 6
246 (a) and Fig. 7 (a), respectively.

247 *Specimen TS*

248 Specimen TS firstly failed by formation of a beam flexural hinge at the column face due
249 to the rupture of the horizontal CFRP around the joint at the beam-column interface in
250 the first six loading cycles. The specimen then continued in the last three loading cycles
251 with the joint shear failure mode. A maximum load of 77.2 kN was reached at the peak
252 of the fourth loading cycle, an increase of 116% as compared to the control specimen.
253 After reaching the maximum load, the horizontal CFRP around the joint ruptured

gradually at the beam-column interface, thus in the next two loading cycles the beam flexural hinge was formed at the beam-column interface and the load reduced slowly together with the CFRP rupturing process. In the last three loading cycles when the CFRP around the joint was completely ruptured, the anchorage system was inactive. Debonding of CFRP at the joint area, therefore, occurred leading to the joint shear failure and the load reduction. The measured shear force versus beam end deflection and the final cracking patterns of Specimen TS are shown in Fig. 6 (b) and Fig. 7 (b), respectively. Large flexural cracks were observed at the beam column interface and diagonal cracks were also recorded at the joint area, especially at the area close to the beam-column interface.

Specimen TS1

Specimen TS1 exhibited joint shear failure mode after the first four loading cycles because the vertical and horizontal CFRPs ruptured. A maximum load of 65.8 kN was recorded at the peak of the 4th loading cycle. Compared to the control specimen, the shear resistance of the joint improved about 84%. With the application of the additional CFRP (Fig. 2), rupture of the horizontal CFRP did not occur at the beam-column interface but was initiated at the joint region instead. The CFRP rupture led to the reduction of the load in the latter loading cycles. The measured shear force versus beam end deflection and the final cracking patterns of Specimen TS1 are shown in Fig. 6 (c) and Fig. 7 (c), respectively. No considerable cracks were observed in both the beam and the columns outside the strengthened area during the test. At the peak of the fourth loading cycles, the CFRP at the top and the bottom of the joint region and the CFRP around the column close to the top and bottom faces of the beam started to break. At the same time, CFRP debonding were initiated in a small area at the beam column interfaces. In the fifth loading cycle, the rupture of CFRP around the column developed faster, the load, therefore, reduced rapidly. CFRP rupture at the top and bottom of the joint area did not develop and the CFRP near the center of the joint did not rupture during the test. During the sixth loading cycle, the sudden rupture of the anchorage CFRP around the beam led to the development of CFRP debonding area at the beam column interface. After the anchorage CFRP around the beam was ruptured, in the last three loading cycles, the CFRP at the joint was mostly debonded and the contribution of CFRP to the shear behaviour of the joint was negligible.

Specimen TS2

Specimen TS2 exhibited beam flexural failure mode as a beam flexural hinge formed approximately 250 mm away from the beam-column interface. Hairline flexural cracks appeared on the beam and the columns close to the strengthened area during the second loading cycle when the load was greater than 60 kN. The flexural cracks on the beam developed faster and were also wider than those on the columns during the third loading cycle, and they led to a flexural hinge forming on the beam during the fourth loading cycle. A maximum load of 86 kN was reached at the peak of the fourth loading cycle, which is an increase of 140% compared to the control one, and the load continued to be maintained at a high level until the ninth loading cycle. During this test, no debonding or breakage of CFRP at the joint region occurred, which means that the

amount of CFRP applied onto this specimen was adequate and it could cause the connection to shift to beam failure from joint shear. The measured shear force versus beam end deflection and the final cracking patterns of Specimen TS2 are shown in Fig. 6 (d) and Fig. 7 (d), respectively.

Joint stiffness

The stiffness of the tested specimens depended on the stiffness of the columns, the beam, and the joint. In this study, joint stiffness was used to evaluate the efficiency of the proposed strengthening method. The stiffness of the joint was calculated as the ratio between the beam tip load and joint rotation. Three inclinometers were used to measure the rotation of the beam and the columns during the test. This rotation of the joints was calculated as the difference between the rotation of the beam and the columns. The plots of joint stiffness versus the beam tip displacement of the tested specimens are shown in Fig. 8. In general the joint stiffness of the strengthened specimens had improved significantly such that Specimen TS was stiffer than Specimen TS1 but softer than Specimen TS2 at the same beam tip displacement. These results were reasonable because the amount of CFRP wrapped around Specimen TS2 was adequate, whereas Specimen TS was higher than TS1, both were inadequate.

The significant improvement in the joint stiffness of the strengthened specimens over the control specimen during the first two loading cycles may have resulted from the concrete covers being glued in place. It is because at these loading stages, the strain of the CFRPs was very small (Fig 11.) so the contribution of CFRP towards joint stiffness was insignificant. The joint stiffness of Specimen TS and TS1 was very close during the first three loading cycles, but during the next loading cycles Specimen TS had significantly higher joint stiffness. This may be due to difference in their failure modes; because Specimen TS failed in HB mode in loading cycles 4-6 and then JS mode, with a significant contribution of CFRP in the last three loading cycles, whereas Specimen TS1 failed completely in JS mode during the last six loading cycles. Because the CFRP ruptured, the joint stiffness of Specimens TS1 and TS rapidly decreased to relatively low levels during the last four loading cycles, while Specimen TS2 remained stable at the same time. During the last four loading cycles the CFRPs in Specimen TS1 ruptured completely but its joint stiffness was significantly higher than the control specimen, indicating that the glued concrete covers still worked well together with the existing concrete to resist the shear load.

Displacement ductility

One of the main goals of this method of strengthening was to improve the ductility of the connections. The displacement ductility index μ was used to evaluate the improvement in ductility of the strengthened specimens. Displacement ductility is defined as the ratio between the ultimate displacement Δ_u and the yield displacement Δ_y . The ultimate displacement was determined corresponding to a 15% drop in the loading capacity, while the estimation of yield displacement was based on equivalent elasto-plastic energy absorption [33]. All the estimated displacements were based on the positive and negative branches of envelopes of hysteretic curves (Fig. 9) of the tested

specimens. The displacement ductility index μ of each specimen is calculated as means of the ductility indexes corresponding to its positive and negative branches of the hysteretic curve. The calculated ductility index of Specimens T0, TS, TS1 and TS2 is 1.71, 2.53, 2.28 and 3.20, respectively. These figures show that the displacement ductility of the strengthened specimens improved significantly. Although joint shear failure mode was observed in Specimen TS1, the increase in its displacement ductility was 33% compared to Specimen T0. With Specimen TS, which first failed by beam hinge and then by joint shear mode, the improvement in displacement ductility was 48%, and 87% for Specimen TS2, which failed by beam hinge formation.

Energy dissipation

The dissipated energy of a specimen was defined as the area enclosed by a hysteretic loop at a given loading cycle. It is an important criterion for the behaviour of beam-column connections under cyclic loading. Fig. 10 shows the cumulative energy dissipated by the tested specimens such that the capacity of the beam-column connections to dissipate energy improved significantly with the proposed strengthening method. Compared to Specimen T0, the increase in the cumulative dissipated energy for Specimens TS, TS1 and TS2 was 220%, 99% and 407%, respectively. Fig. 10 also shows that in the last four loading cycles, although the CFRP ruptured at the joint area, the dissipated energy of Specimen TS1 was significantly higher than the control specimen, which again proved that the bonded concrete covers worked well with the existing concrete to resist the shear load during the test, even when the CFRPs had ruptured.

Strain of external CFRP

The effectiveness of this proposed technique depends on how well the CFRP can confine the concrete at the joint core. To evaluate the effect of CFRP confinement the strains of the CFRP around the joint were measured by strain gauges numbered from 11 to 18 (Fig. 11). The measured strains of CFRP around the joint at the peak of loading cycles from 1st to 6th (beam tip displacement $\Delta = 10, 20, \dots 60$ mm) of Specimens TS, TS1 and TS2 are presented in Fig. 11 (a), (b) and (c), respectively. The location of the strain gauges is also attached in the figure for easy identification. In general, as it was expected, the CFRPs around the joints were under tension during the test and thus the concrete at the joint was affected by a relatively high confined stress.

For Specimen TS (Fig 11 a), where no additional CFRP was installed, the strains of CFRP at the beam-column interface (Strain Gauges 11 and 15) were about 10 to 20% higher than at the center of the joint (Strain Gauges 12 and 16). Whereas, in Specimen TS1 and TS2 the strains recorded from Strain Gauges 12 and 16 were higher than that of Strain Gauges 11 and 15. These are the reasons explained why break of the CFRP at the beam-column interface occurred just in Specimen TS. This indicates that the application of the additional CFRP could prevent effectively the break of the horizontal CFRP at the beam-column interface. For Specimen TS1 (Fig. 11 b) the CFRP strains measured from Strain Gauges 12 and 16 reached the values of about 8900 and 7000 micro strain, corresponding to 1/2 and 2/5 of the CFRP ultimate strain f_{fu} , respectively. This shows the effectiveness of the CFRP in this method, because the strains of only about 1/3 to

1/5 f_{fu} were reached in the conventional CFRP strengthening methods. Fig. 11 (b) also shows that CFRP strains of Specimen TS1 reduced in loading Cycles 5 and 6 after reaching the maximum values at the peak of the fourth loading cycle. This could be caused by the reduction in the load due to the disruption of the CFRPs around the columns and the beam. In specimen TS2 (Fig. 11 c) the CFRP strain increased rapidly from the first to the fourth loading cycles and then maintained at high level in the next two loading cycles. These results coincided with the measured beam tip load (Fig. 6 d).

Although the CFRPs near the back of the column had lower strains than those at the center and at the beam-column interface, the distribution of the CFRP confined stress in the tested specimens was quite uniform if compared to when CFRP was applied around the square or rectangular sections, where confinement effect was only generated at the corners of the sections. Moreover, relatively high confinement stresses were created at the centre of the tested joints, whereas in the conventional methods, the CFRP confinement effect at this area was insignificant because with the conventional methods, the concrete surface at the joint center was flat and a relatively lower FRP strains was reached. The confined stress created at the joint center could be the main reason why CFRP did not bulging or debonding from the concrete in this area, which was a common occurrence in conventional FRP methods. This also helped to maintain the bond between the existing and the concrete covers and thus it improved the possibility of them working together when resisting the cyclic shear forces. Harajli [34] proved that FRP confinement substantially improved the reinforcement-concrete bond under the cyclic loading. Therefore, the absence of bond-splitting cracks on Specimens TS, and TS2 (Fig. 7) can be explained as due to the relatively high CFRP confinement stress created at the joint core. By preventing the risk of bulging and/or debonding as well as increasing the confinement effect, it is believed that the possibility of the concrete and the externally reinforcement working together to resist the shear forces was also increased with this proposed method.

The proposed model for shear strength of FRP-strengthened T connections

Based on average plane stress concept, a model for predicting the shear strength of T connections is developed as presented in the following section. In the model, biaxial failure criterion of concrete is adopted to predict the failure of the joint core caused by a combination of tensile and compressive stresses. The contribution of shear reinforcement is accounted for by increasing the tensile strength of the concrete while the axial stress was accounted for by its contribution on the direction and magnitude of the mean tensile and compressive stresses.

Failure criterion and shear strength of the FRP retrofitted T connections

Fig. 12 shows an illustration of the schematic, forces and failure mechanism of a RC T connection retrofitted with FRP. When a T connection fails in shear, the center of the joint is the critical location because most of the serious failures, which are caused by diagonal tension and compression stresses, occur at this place. Therefore, it is assumed that the joint shear strength of the retrofitted joint is reached when the principal tensile

and compressive stresses (σ_1 and σ_2 in Fig. 12 b) of the concrete at the center of the joint reach the failure envelope. It is noted in Fig 12 that the average plane stress approach was applied. This application may not fully illustrate the stress states of the joint especially when the concrete at the joint core are under high confinement. However, due to its simplicity and the relatively low confinement of the concrete at the joint core, this approach is currently used by the research community for the derivations of most of the existing models for joint shear strength of beam. Further discussion on the influence of the confinement effect on the joint shear strength of the shape-modified specimens is discussed in the following sections of the paper.

The tension-compression branch of Tasujiet *al.* [36] failure envelope is employed as the failure criterion of the concrete at the center core of the joint. The equation of the assumed criterion can be expressed as follows:

$$\frac{\sigma_1}{f_{ct}^*} - \frac{\sigma_2}{f_c'} = 1 \quad (1)$$

where f_c' is the compressive strength of the concrete and f_{ct}^* is the nominal modified tensile strength of the concrete considering the contribution of the shear reinforcements (steel and FRP); σ_1 and σ_2 are the principal tensile and compressive stresses, which can be determined by Eqs. (2) and (3) using Mohr's circle (Fig. 12 c):

$$\sigma_1 = \tau_{jh} \tan \theta \quad (2)$$

$$\sigma_2 = \sigma_y - \tau_{jh} \tan \theta \quad (3)$$

where $\sigma_y = N_c / (A_c)$ is the normal vertical compressive stress; $\tau_{jh} = V_{jh}^* / (A_j)$ is the joint shear stress; V_{jh}^* is the shear force resisted by the joint core defined in Eq. (4); A_c is area of the column section; A_j is the effective joint area; N_c is the column axial load and θ is the inclination of the principal compressive stress from the column longitudinal which

can be determined using Fig. 12 as $\tan \theta = \sqrt{\frac{\sigma_1}{\sigma + \sigma_1}}$. Different from the existing

models, in this study the role of the column axial stress is accounted for by its influence on both the magnitude and the direction of principal tensile and compressive stress. Assuming that when the shear strength of the joint is reached, the nominal principal tension stress at the center of the joint reached the modified tensile strength, f_{ct}^* , the

inclination angle θ , therefore, can be calculated as $\tan \theta = \sqrt{\frac{f_{ct}^*}{\sigma + f_{ct}^*}}$. However, for

simplicity in this model $\tan \theta = \sqrt{\frac{1.15 f_{ct}}{\sigma_y + 1.15 f_{ct}}}$ is proposed, where f_{ct} is the nominal

tensile strength of concrete.

454 In most of the existing modes, the joint shear force which is calculated as
 455 $V_{jh} = A_{sb}f_{sb} - V_c \approx A_{sb}f_{sb}(1 - 0.85h_b/H)$ is assumed to be resisted totally by the
 456 concrete at the joint. This model however, assumed that only a fraction of this force is
 457 resisted by the joint core and the remaining is transferred and resisted by the column
 458 near the joint. Therefore, the shear force resisted by the joint can be calculated as:

$$459 \quad V_{jh}^* = \beta V_{jh} = \beta(A_{sb}f_{sb} - V_c) \approx \beta A_{sb}f_{sb}(1 - 0.85h_b/H) \quad (4)$$

460 where β is a fraction factor that illustrates the ratio of shear force resisted by the joint
 461 core. The fraction factor β depends on the bond condition between the beam
 462 longitudinal reinforcement and the surrounding concrete at the joint with an average
 463 value of 0.84, for simplicity, $\beta = 0.84$ is proposed in this study; V_{jh} is the joint shear force;
 464 A_{sb} and f_{sb} are the area and the stress of the beam tensile reinforcement at the beam-
 465 column interface, respectively; h_b is the total height of beam cross section.

466 The combination of Eqs. (1), (2), (3) and (4) yields the following expression for the shear
 467 strength of the retrofitted joint:

$$468 \quad V_{jh} = \frac{b_j h_c}{\beta \tan \theta} \frac{1 + \sigma_y / f_c'}{1 / f_{ct}^* + 1 / f_c'} \quad (5)$$

469 **The nominal modified tensile strength of concrete**

470 The contribution of the vertical and horizontal joint reinforcement (FRP and steel) is
 471 accounted for by improving the nominal tensile strength of concrete. The nominal
 472 tensile strength, f_{ct} of concrete is reached at cracking strain, ε_{cr} is given by ACI 318R-08
 473 [37]:

$$474 \quad f_{ct} = 0.556 \sqrt{f_c'} \quad (\text{MPa}) \quad (6)$$

475 The average nominal tensile strength, f_{jsr} of the joint shear steel reinforcement reached
 476 at its yield strain ε_{sjy} and can be determined as follows:

$$477 \quad f_{js} = \frac{A_{jsh} f_{jhy} \cos \theta + A_{jsv} f_{jvy} \sin \theta}{A_j / \sin \theta} \quad (7)$$

478 where A_{jsh} , A_{jsv} are the total area of the horizontal and the vertical steel shear
 479 reinforcement with yield strength f_{jhy} , f_{jvy} , respectively.

480 The average nominal tensile strength, f_{jfr} of the joint shear FRP reinforcement reached at
 481 FRP debonding or at FRP rupture strain and can be determined as follows:

$$482 \quad f_{jf} = \frac{A_{jfh} f_{fe} \cos \theta + A_{jfv} f_{fe} \sin \theta}{A_j / \sin \theta} \quad (8)$$

483 where A_{jfh} is the area of the joint horizontal fibre applied between the top and the
 484 bottom beam bars; A_{jfv} is the area of the applied intermediate vertical fibre at the joint;

485 $f_{fe} = E_f \varepsilon_{fe} = E_f \min(\varepsilon_{fr}, \varepsilon_{fd})$ is the effective strength of FRP; $\varepsilon_{fe}, \varepsilon_{fr}, \varepsilon_{fd}$ are the
 486 effective strain, the rupture strain and the debonding strain of FRP, respectively and E_f
 487 is the FRP elastic modulus.

488 The FRP debonding strain can be calculated based on Holzenkampfer model [38] as
 489 follows:

$$490 \quad \varepsilon_{fd} = \varepsilon_{fd, \max} = c_1 \sqrt{f_{ct} / (t_f E_f)} \quad \text{for } l_b \geq l_{b, \max} \quad (9)$$

$$491 \quad \varepsilon_{fd} = \varepsilon_{fd, \max} \frac{l_b}{l_{b, \max}} \left(2 + \frac{l_b}{l_{b, \max}} \right) \quad \text{for } l_b < l_{b, \max} \quad (10)$$

492 where c_1 is the FRP constant, t_f (in mm) is thickness of the bonded FRP, l_b (in mm) is the
 493 development bond length of FRP, and

$$494 \quad l_{b, \max} = \sqrt{t_f E_f / (c_2 f_{ct})} \quad (11)$$

495 where c_2 is another constant, in the case of CFRP the constants in Eqs. (9) and (10) take
 496 values 0.64 and 2, respectively.

497 Due to the non-uniform stress distribution caused by localised deformation in cracked
 498 concrete and the curvature of the FRP jacket, the rupture strain of FRP in the
 499 strengthened connection ε_{fr} was smaller than its ultimate strain obtained from the
 500 coupon tests ε_{fu} . Based on experimental calibrations [39] proposed a value of
 501 $\varepsilon_{fr} = 0.586 \varepsilon_{fu}$ for the FRP confined columns, but when FRP was wrapped on beam-
 502 column connections, the FRP fracture strain was not calibrated due to the lack of
 503 experimental data. Thus, in this study the value of $\varepsilon_{fr} = 0.5 \varepsilon_{fu}$ was assumed based on
 504 the measured breaking strain of FRP in Specimen TS1. It was noted that the error caused
 505 by applying this assumption was small because in most cases ε_{fr} was larger than ε_{fd}
 506 which was calculated from Eqs. (9) and (10) and thus ε_{fe} depends mostly on ε_{fd} .

507 When considering the role of the joint shear reinforcement most of the existing models
 508 assumed that the concrete and the reinforcement work synchronously. However, as the
 509 concrete at the joint core, the joint shear steel reinforcement and the joint shear FRP
 510 reinforcement cannot reach their tensile strength simultaneously and the tensile
 511 cracking strain of concrete is much lower than the yield strain of steel reinforcement or
 512 the FRP effective strain, the following equation is proposed to determine the nominal
 513 modified tensile strength of the concrete:

$$514 \quad f_{ct}^* = \text{Max} \left\{ f_{ct} + \frac{\varepsilon_{cr}}{\varepsilon_{jsy}} f_{js} + \frac{\varepsilon_{cr}}{\varepsilon_{fe}} f_{jf}, f_{js} + f_{jf} \right\} \quad (12)$$

515 where a strain ratio of $\varepsilon_{cr} / \varepsilon_{jsy} = 0.5$ was proposed based on the calibration using a
 516 database of experimental tests on RC exterior joints and a strain ratio of $\varepsilon_{cr} / \varepsilon_{fe} \leq 1$ are

517 proposed in the equation to consider the asynchronization of the concrete and the FRP
 518 shear reinforcement. The magnitude of the $\varepsilon_{cr} / \varepsilon_{fe}$ ratio is discussed in the following
 519 sections of the paper.

520 Verification of the proposed model

521 The experimental and predicted joint shear strength of specimens T0, TS, 522 TS1 and TS2

523 In the FRP-strengthened schematic illustrated in Fig. 12 (a) the FRP applied at the joint is
 524 extended into the beam thus the beam flexural capacity at the beam-column interface is
 525 increased. If the section capacities (flexural and shear capacities) of the columns are
 526 larger than the beam, when increasing shear load V_b , the beam-column connections can
 527 fail either by formation of the beam flexural hinge at the joint face or at a distance l_b
 528 from the joint face (Section *P-P* in Fig. 12a) or by joint shear failure. Consequently, the
 529 predicted joint shear force ($V_{jh,pred}$) of the retrofitted beam-column connections is $V_{jh,pred}$
 530 $= \min(V_{jh}, V_{jh,1}, V_{jh,0})$, where $V_{jh,1}$ and $V_{jh,0}$ are the joint shear forces corresponding to
 531 formation of flexural hinges at section *P-P* and at the beam-column interface,
 532 respectively, and which can be determined using Eqs. (13) and (14), respectively.

$$533 \quad V_{jh,1} = M_n \frac{L_0}{L_1} \left(\frac{1}{j_d} - \frac{L_0 + h_c / 2}{HL_o} \right) \quad (13)$$

$$534 \quad V_{jh,0} = (M_n + M_{FRP}) \left(\frac{1}{j_d} - \frac{L_0 + h_c / 2}{HL_o} \right) \quad (14)$$

535 where $M_n = \alpha M_y$ (M_y is beam moment capacity corresponding to the yield of beam
 536 reinforcement) is the nominal moment strength of the beam before being strengthened,
 537 $\alpha = 1.25$ is an over strength factor for the beam longitudinal reinforcement; $j_d \approx 0.8 h_b$ is
 538 the lever arm of the beam cross section; M_{FRP} is the increased flexural capacity of the
 539 beams provided by the horizontal CFRP on the beams close to the columns [23]; the
 540 distances H , L_0 and L_1 are illustrated in Fig. 12 (a).

541 A comparison between the experimental and the predicted results calculated by the
 542 proposed model for specimens T0, TS, TS1 and TS2 is summarized in Table 2. It is noted
 543 that for Specimens TS, TS1 and TS2, the predicted joint shear forces $V_{jh,pred}^{0.75}$, $V_{jh,pred}^0$,
 544 $V_{jh,pred}^{0.25}$ and $V_{jh,pred}^1$ are calculated with strain ratio $\varepsilon_{cr} / \varepsilon_{fe} = 0.75, 0, 0.25$ and 1 ,
 545 respectively and the effective joint area A_j is assumed to be equal to the modified
 546 column section because experimental results showed that the bonded and existing
 547 concrete worked simultaneously to resist the shear forces. The experimental joint shear
 548 force $V_{jh,est}$ was derived based on the maximum applied loads measured from the tests
 549 and by assuming that the moment lever arm of the beam cross section was $j_d = 0.8 h_b$.

550 The predicted joint shear strength of Specimen T0 was $V_{jh,pred} = 155.2$ kN, which was 5%
 551 higher than the experimental result, and since no FRP was applied on this specimen, A_{jfn}

552 $=A_{fV} = 0$ was used to calculate $V_{jh,pred}$ and thus this predicted joint shear strength did
 553 not depend on the strain ratio $\varepsilon_{cr} / \varepsilon_{fe}$. Table 2 shows that the predicted joint shear
 554 strength of the strengthened specimens (Specimens TS, TS1 and TS2) increased as the
 555 strain ratio $\varepsilon_{cr} / \varepsilon_{fe}$ increased. When $\varepsilon_{cr} / \varepsilon_{fe} = 0$ the average of the test-to-predicted
 556 joint shear strength was 1.39, it then reduced to 1.22, 1.01, and 0.99 corresponding to
 557 $\varepsilon_{cr} / \varepsilon_{fe} = 0.25$; 0.75 and 1.0, respectively. It is worth mentioning that for specimens TS
 558 and TS2 the predicted joint shear strength corresponding to $\varepsilon_{cr} / \varepsilon_{fe} = 0.75$ and 1.0 was
 559 equal because when $\varepsilon_{cr} / \varepsilon_{fe} \geq 0.75$ the predicted failure mode of both specimens was
 560 BH and the shear capacity of the connections was governed by the flexural strength of
 561 the beam close to the joint core. From the above analysis it can be concluded that the best
 562 predicted joint shear force corresponds to the strain ratio $\varepsilon_{cr} / \varepsilon_{fe} = 0.75$.
 563 Consequently, this strain ratio is proposed in the model to predict the joint shear
 564 strength of a T connection strengthened using the proposed strengthening method.

565 **The experimental and predicted joint shear strength of specimens** 566 **strengthened with the conventional FRP methods**

567 To validate the proposed analytical model for predicting the joint shear capacity of the
 568 conventional FRP-strengthened T connections, 32 experimental results of testing T
 569 connections available in the literature were used. All the specimens were subjected to
 570 cyclic lateral loading; the FRP was wrapped on the existing square or rectangular
 571 sections of the beams and the columns and were at least one-third scale. The
 572 comparison between the experimental and predicted results for the collected T
 573 connections is summarized in Table 3. In this table, the experimental shear force, $V_{jh,test}$,
 574 is either collected from the reported values or derived based on the maximum applied
 575 loads measured from the tests with the assumption that the moment lever arm of the
 576 beam cross section $j_d = 0.8h_b$. The predicted joint shear forces $V_{jh,pred}^{0.25}$, $V_{jh,pred}^{0.75}$ and $V_{jh,pred}^1$
 577 are calculated with strain ratio $\varepsilon_{cr} / \varepsilon_{fe} = 0.25$, 0.75 and 1, respectively. It can be seen
 578 from the table that the best prediction corresponds to the strain ratio $\varepsilon_{cr} / \varepsilon_{fe} = 0.25$, thus
 579 this strain ratio is proposed in the model to predict the joint shear strength of a T
 580 connection strengthened using the conventional methods.

581 A question arises as why the strain ratio of Specimens TS, TS1 and TS2 was higher than
 582 that of the other specimens strengthened using conventional methods? The difference
 583 in the strain ratio would be caused by the difference between the proposed and the
 584 conventional strengthening methods, because the experimental results showed that by
 585 reducing the risk of bulging and debonding as well as increasing the confinement effect,
 586 the concrete and FRP can work together to resist the shear forces in the proposed
 587 method better than the conventional methods. This was a reasonable assumption
 588 because it is recognized that the improvement in FRP confinement improved the joint
 589 shear behaviour of the strengthened connections and the effectiveness of the FRP
 590 strengthening method was governed by the bond between the FRP and the concrete
 591 surface. As mentioned above, for simplicity plane stress approach was applied in the

592 proposed model. This idealization may lead to errors in the predicted joint shear
593 strength, however, by calibrating the strain ratio $\varepsilon_{cr} / \varepsilon_{fe}$ using experimental results these
594 errors may be reduced and the good prediction of the proposed model as presented
595 above proved this point.

596 Conclusion

597 Four RC exterior beam-column connections were cast and tested under cyclic loading to
598 evaluate the effectiveness of a proposed strengthening method using CFRP and concrete
599 covers. The variation of the joint failure mode with the applied CFRP ratio has been
600 presented. Despite the difference in the failure modes, the test results have proved that
601 the proposed strengthening method not only helped in upgrading shear strength but
602 also helped in improving the seismic performance of the connections. The bonded
603 concrete covers work well together with the existing concrete to resist shear load. They
604 also helped in improving the FRP confinement effect and thus improved the
605 effectiveness of the applied CFRP. The supplement of the additional CFRP sheets at the
606 beam-column interface effectively prevents the horizontal CFRP from breaking at the
607 beam-column interface. Furthermore, the experimental results also showed that the
608 proposed method can help in relocating the failure of the T connection from the joint
609 area to the preferred place at the beam.

610 In addition, an analytical model to predict the shear strength of FRP-strengthened T
611 connections was developed based on plane stress concept. In the model, the possibility
612 of the FRP joint shear reinforcement bonded at the joint area to work simultaneously
613 with the existing concrete to resist principal tensile stress at the joint core was
614 calibrated. This calibration showed that the proposed FRP-strengthening method can
615 help in increasing the effectiveness of the wrapped FRP by improving the possibility of
616 the CFRP and the concrete to work together in resisting the shear load. The reliability of
617 the proposed model was evaluated by comparing the predicted and the experimental
618 joint shear strength of 3 connections strengthened using the proposed technique and 32
619 connections strengthened using the conventional methods. These comparisons have
620 shown the accuracy of the proposed model.

621 Finally, due to their effectiveness, it is believed that the proposed strengthening method
622 and the proposed model presented in this paper are suitable for practical implications.

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734

Figure legends

735

736 Fig. 1. Reinforcement details of Specimens T0, TS, TS1 and TS2

737 Fig. 2. Application of CFRP sheets for strengthening of Specimens TS, TS1 and TS2

738 Fig. 3 Images of a strengthened specimen

739 Fig. 4. The schematic of the test set up

740 Fig. 5. Displacement history of the beam's free end

741 Fig. 6. Load versus displacement responses hysteresis

742 Fig. 7. The final failures of Specimens T0, TS, TS1 and TS2

743 Fig. 8. Joint stiffness versus beam tip displacement

744 Fig. 9. Envelope of hysteretic loops of Specimens T0, TS, TS1 and TS2

745 Fig. 10. Cumulative energy dissipation

746 Fig. 11. Strain of the CFRP around the joint of Specimen TS, TS1 and TS2

747 Fig. 12. Joint shear resisting mechanism of an FRP-strengthened RC T connection

748

749 Table1. Variation of fibre thickness on Specimens TS, TS1 and TS2

Specimen	Fibre thickness (mm)					
	t_{jfh}	t_{fhi}	t_{jfv}	t_{fhc}	t_{fb1}	t_{fb2}
TS	0.33	0	0.33	0.33	0.33	0.66
TS1	0.11	0.11	0.11	0.11	0.11	0.22
TS2	0.66	0.66	0.66	0.66	0.66	1.32

t_{jfh} , t_{jfv} , t_{fhi} , t_{fhc} , t_{fb2} and t_{fb1} are the thicknesses of the CFRPs defined in Fig. 2

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751 Table 2. The experimental and predicted joint shear strength of specimens T0, TS, TS1 and TS2

Specimen	f'_c (MPa)	A_j (m ²)	E_f (GPa)	ε_{fu}	A_{jfh} (mm ²)	t_{jfh} (mm)	A_{jfv} (mm ²)	t_{jfv} (mm)	$V_{jh,pred}$ (kN)	$V_{jh,test}$ (kN)	$\frac{V_{jh,test}}{V_{jh,pred}^{0.75}}$	$\frac{V_{jh,test}}{V_{jh,pred}^0}$	$\frac{V_{jh,test}}{V_{jh,pred}^{0.25}}$	$\frac{V_{jh,test}}{V_{jh,pred}^1}$
T0	41	0.0400	-	-	-	-	-	-	155.2	146.7	0.95	0.95	0.95	0.95
TS	50	0.0628	235	0.018	158.4	0.33	105.6	0.33	327.3	339.4	1.04	1.37	1.21	1.04
TS1	41	0.0628	235	0.018	52.8	0.11	35.2	0.11	274.5	277.2	1.01	1.25	1.16	0.95
TS2	44	0.0628	235	0.018	316.8	0.66	211.3	0.66	354.5	355.5	1.00	1.54	1.29	1.00

A_j is the effective joint area; A_{jfh} , t_{jfh} are the total area and the thickness of the horizontal fibre applied between the top and the bottom beam bars, respectively; A_{jfv} , t_{jfv} are the total area and the thickness of the intermediate vertical fibre, respectively.

752

753 Table 3.Comparison of analytical model predictions with experimental test results

Reference	Specimen ID	f'_c (MPa)	$N_c/(b_c h_c)$ (Mpa)	A_{sb} (mm ²)	E_f (GPa)	ε_{fu}	A_{jfh} (mm ²)	t_{jfh} (mm)	A_{jfv} (mm ²)	t_{jfv} (mm)	$V_{jh,pred}$ (kN)	$V_{jh,test}$ (kN)	$\frac{V_{jh,test}}{V_{jh,pred}^{0.25}}$	$\frac{V_{jh,test}}{V_{jh,pred}^{0.75}}$	$\frac{V_{jh,test}}{V_{jh,pred}^1}$
Ghobarah and Said [12]	T1R	38	6.00	1195	19.5	0.017	498	1.10	498	1.1	552	572.9	1.04	1.04	1.04
	T2R	30.6	6.00	1195	19.5	0.017	996	2.20	996	2.2	552	589.6	1.07	1.07	1.07
	T4	30.8	6.00	1195	19.5	0.017	498	1.10	498	0.6	552	531.0	0.96	0.96	0.96
El-Amoury and Ghobarah [39]	TR1	43.5	6.00	1195	30.1	0.015	569	1.22	218	0.0	555	581.7	1.05	1.05	1.05
	TR2	39.5	6.00	1195	30.1	0.015	569	1.22	218	0.0	555	604.5	1.09	1.09	1.09
Antonopoulos and Triantafillou [8]	S33	26	1.15	462	150.0	0.016	158	1.05	158	1.1	166	136.5	0.82	0.69	0.65
	S63	24.2	1.15	462	150.0	0.016	315	1.05	158	1.1	170	155.6	0.92	0.72	0.66
	S33L	26.3	1.15	462	150.0	0.016	158	1.05	158	1.1	167	172.5	1.03	0.86	0.81
	F11	22.8	1.15	462	230.0	0.015	62	0.13	42	0.1	160	165.3	1.03	0.84	0.79
	F22	27.2	1.15	462	230.0	0.015	125	0.26	83	0.3	181	193.4	1.07	0.83	0.76
	F21	27	1.15	462	230.0	0.015	125	0.26	42	0.1	179	197.5	1.11	0.86	0.79
	F12	29.5	1.15	462	230.0	0.015	62	0.13	83	0.3	181	171.8	0.95	0.76	0.71
	F22A	27.8	2.88	462	230.0	0.015	125	0.26	83	0.3	224	221.8	0.99	0.78	0.71
	F22W	29.2	1.15	462	230.0	0.015	125	0.26	83	0.3	186	215.9	1.16	0.90	0.82
	F22 in	21	1.15	462	230.0	0.015	125	0.26	83	0.3	162	162.1	1.00	0.77	0.71
	GL	19.5	1.15	462	70.0	0.031	200	0.42	134	0.4	149	170.6	1.14	0.93	0.87
	S-F22	19	1.15	462	230.0	0.015	125	0.26	83	0.3	162	170.4	1.05	0.82	0.75
Karayannis and Sirkelis [40]	A2R	36.4	1.75	157	230.0	0.015	58	0.12	0	0.0	143	139.1	0.97	0.97	0.97
	A3	36.4	1.75	157	230.0	0.015	58	0.12	0	0.0	143	137.3	0.96	0.96	0.96
	B2R	36.4	1.75	157	230.0	0.015	58	0.12	0	0.0	143	135.6	0.95	0.95	0.95
	B3	36.4	1.75	157	230.0	0.015	58	0.12	0	0.0	143	139.1	0.97	0.97	0.97
Shrestha, et al [41]	SC 1	25.8	2.00	1809	243.0	0.011	0	0.00	47	0.2	387	366.1	0.95	0.90	0.90
Akguzel and Pampanin [9]	2DR1	24.8	1.42	314	76.0	0.028	167	0.36	133	0.4	176	157.5	0.90	0.90	0.90
	2DR2	18.9	2.10	314	76.0	0.028	167	0.36	133	0.4	139	143.4	1.03	1.03	1.03
	2DR3	18	4.35	314	76.0	0.028	167	0.36	133	0.4	139	160.8	1.16	1.16	1.16
	2DR4	18.7	2.10	314	76.0	0.028	334	0.72	133	0.4	139	160.8	1.16	1.16	1.16
Le-Trung, et al. [42]	RNS-1	36.5	0.00	300	240.5	0.021	132	0.33	110	0.3	105	103.1	0.98	0.98	0.98
	RNS-2	36.5	0.00	300	240.5	0.021	132	0.33	110	0.3	105	103.8	0.99	0.99	0.99
	RNS-3	36.5	0.00	300	170.0	0.021	264	0.66	220	0.7	105	101.6	0.97	0.97	0.97
	RNS-4	36.5	0.00	300	170.0	0.021	264	0.66	220	0.7	105	103.8	0.99	0.99	0.99
	RNS-5	36.5	0.00	300	240.5	0.021	132	0.33	110	0.3	105	100.1	0.95	0.95	0.95
	RNS-6	36.5	0.00	300	240.5	0.021	264	0.66	220	0.7	105.2	102.3	0.97	0.97	0.97
Total	32										Average:		1.01	0.93	0.90
											Coefficient of variation:		7.8%	12.5%	15.6%

754

(a) Specimen TS

(b) Specimen TS1

(c) Specimen TS2

Note: $\Delta=10$ $\Delta=20$ $\Delta=30$ $\Delta=40$ $\Delta=50$ $\Delta=60$

The specimen after completion of the first stage

The specimen after completion of the second stage

(a) Specimen T0

(b) Specimen TS

(c) Specimen TS1

(d) Specimen TS2

(a) Specimen T0

(b) Specimen TS

(c) Specimen TS1

(d) Specimen TS2