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PRECRACKED RC T-BEAMS REPAIRED IN SHEAR WITH BONDED CFRP SHEETS

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ABSTRACT

This paper investigates the structural behavior of precracked reinforced concrete (RC) T-beams strengthened in shear with externally bonded carbon fiber reinforced polymer (CFRP) sheets. It reports on seven tests on unstrengthened and strengthened RC T-beams identifying the influence of load history, beam depth and percentage of longitudinal steel reinforcement on the structural behavior. The experimental results indicate that the contributions of the external CFRP sheets to the shear force capacity can be significant and depend on most of the investigated variables.

This paper also investigates the accuracy of the prediction of the FRP contribution in ACI 440.2R-08; UK Concrete Society TR 55 and *fib* Bulletin 14 design guidelines for shear strengthening. Comparison of predicted values with experimental results indicates that the guidelines can overestimate the shear contribution of the externally bonded fiber reinforced polymer (FRP) system.

Keywords: beam; fiber reinforced polymer; precracking; reinforced concrete; shear; strengthening

INTRODUCTION

Throughout the world many existing reinforced concrete (RC) structures are deemed no longer able to sustain current capacity demands. In the United Kingdom (UK) alone, it has been estimated that there are about 10,000 bridges on the motorway and trunk road network (the majority of which are reinforced and prestressed concrete structures) and 150,000 bridges on local roads of which a considerable number need strengthening or replacement¹. The estimated cost of assessing and strengthening these strength-deficient structures is in excess of £4 billion². Other countries are faced by the same problem. In the United States (US) for example, of the 600,905 bridges across the country, 72,868 bridges (12.1%) were categorized as structurally deficient and 89,024 bridges (14.8%) were categorized as functionally obsolete. The estimated cost of strengthening and repairing both categories is about US\$140 billion³.

Several factors can cause a RC structure to be judged as having insufficient capacity. The need to sustain heavier loads is one important factor, particularly in the case of bridges. Further factors that can have detrimental effects on capacity include corrosion of internal steel reinforcement, changes in use, poor initial design and more stringent assessment codes. One viable solution is to use fiber reinforced polymers (FRPs) as external strengthening reinforcement for RC structures. The use of FRPs is advantageous since the combination of high-strength, high-stiffness structural fibers with low-cost, lightweight, environmentally resistant polymers results in composite materials with excellent mechanical and durability properties.

During the past two decades, several research studies have considered RC beams strengthened in shear with externally bonded FRP systems. However, there are areas where further research is still needed. T-beams have not been considered as extensively as rectangular beams and in general studies investigating the effect of load history on the strengthened behavior have been scarce. Therefore, an attempt is made in this paper to investigate the effect of load history on the behavior of precracked RC T-beams strengthened in shear with carbon FRP (CFRP) sheets. Other parameters that may influence behavior, namely the effective depth of the beam and the longitudinal steel reinforcement ratio, are also discussed. Finally, the reliability of the prediction of the FRP shear contribution in three international shear strengthening design guidelines, namely ACI 440.2R-08⁴; UK Concrete Society TR 55⁵ and *fib* Bulletin 14⁶, is examined.

RESEARCH SIGNIFICANCE

Many researchers^{e.g. 7-11} have investigated the technique of strengthening RC beams in shear using FRPs and established its effectiveness. Published research studies have provided valuable findings, particularly with regard to the effects of the type, stiffness and configuration of the composite material on the shear strength enhancement. However, other parameters that may also influence the shear resisting mechanisms, such as the load history, have not yet been sufficiently studied. In addition to investigating the effect of load history on the shear strength enhancement, this study simulates aspects of the in-service behavior of FRP-strengthened beams including precracking and strengthening under load.

EXPERIMENTAL INVESTIGATION

The experimental investigation consisted of two unstrengthened control beams along with five other beams that were precracked prior to the application of the CFRP sheets. All specimens were T-shaped beams having a significant difference between their unstrengthened shear capacity and their flexural capacity. The T-shaped cross-section was favored because it adequately simulates the slab-on-beam construction method. The gap between the shear capacity and the flexural capacity was deemed necessary in order to provide a sufficient range over which the level of shear enhancement could be measured.

Each specimen had a four part designation given as $X/d/LP\#/p$ where X indicates that the beam was either unstrengthened (U) or strengthened with CFRP fabrics (F), d is the effective depth of the beam in mm, $LP\#$ indicates the loading pattern to which the beam was subjected and p indicates the longitudinal reinforcement ratio ($A_s/b_w d$) of the beam. Hence, the designation F/295/LP1/4.5 refers to a beam that was strengthened with CFRP fabrics (sheets), had an effective depth of 295 mm (11.61 in.), tested under loading pattern 1 (LP1) and had a longitudinal reinforcement ratio of 4.5%.

All tested shear spans had 6 mm (0.24 in.) internal steel transverse reinforcement spaced at 250 mm (9.84 in.) c/c. For the strengthened beams, the external shear reinforcement on a strengthened shear span consisted of three layers of CFRP sheets.

The two T-shaped cross-sections considered in this experimental investigation are detailed in **Fig. 1**. Additional details of the test specimens are given in **Table 1**.

Loading patterns

Three loading schemes were adopted for testing. Loading Pattern 0 (LP0), which was only applied to test the control specimen U/295/LP0/4.5, consisted of loading the beam up to

1 failure. The remaining two loading patterns, namely Loading Pattern 1 (LP1) and Loading
2 Pattern 2 (LP2), involved the pre-cracking of the test specimens in order to model the state of
3 damage that may exist in RC structures requiring strengthening. Tests on the unstrengthened
4 beams showed that the state of damage caused by a load level of about 70% of the
5 unstrengthened shear force capacity can be representative of the state of damage that may
6 exist in some RC structures requiring strengthening. Hence, that load level was used in both
7 LP1 and LP2 described below. Since RC structures may also be carrying dead loads while
8 being strengthened, specimens were unloaded to a proportion of the unstrengthened capacity
9 before the CFRP strengthening system was applied. The final phase involved loading the
10 strengthened specimens up to failure.

11 Specimens subjected to LP1 were loaded, as shown in **Fig. 2**, to 70% of the unstrengthened
12 capacity of the corresponding control beam. Specimens were then unloaded to 40% of the
13 unstrengthened capacity of the corresponding control beam and the strengthening system was
14 installed. Loading then continued up to failure. Under this loading pattern, the shear cracks
15 formed prior to strengthening are likely to be mobilized once strengthened.

16 LP2 aims to stimulate a set of shear cracks after strengthening that are different from those
17 formed prior to strengthening. Specimens subjected to LP2 were initially loaded at position
18 (B), as illustrated in **Fig. 2**, to 70% of their unstrengthened capacity. Specimens were then
19 unloaded to 40% of their unstrengthened capacity and the strengthening system was installed.
20 Load was then shifted gradually to position (A) and continued up to the failure of the
21 specimens. The total load was kept constant at 40% of the unstrengthened capacity of the test
22 beam as it was shifted from position (B) to position (A).

23

24 **Test setup**

25 All the beams except U/295/LP2/4.5 and F/295/LP2/4.5 represented a single specimen tested
26 in four-point bending (see **Fig. 3**). However, in order to speed up the final stages of the
27 testing process, U/295/LP2/4.5 and F/295/LP2/4.5 were tested in three-point bending as this
28 type of loading allowed two tests to be carried out on a single beam. This was achieved by
29 testing one beam end zone while keeping the other end overhung and unstressed and vice
30 versa (see **Fig. 3**). The 925 mm (36.42 in.) long shear span was reinforced with additional
31 transverse steel reinforcement (6 mm [0.24 in.] shear links spaced at 100 mm [3.94 in.] c/c) to
32 ensure that failure always occurred in the 1125 mm (44.29 in.) long shear span. The shear
33 span to effective depth ratio (a/d) in all beams was maintained at a value of 3.8.

34

1 **Materials**

2 The concrete used to cast the specimens consisted of coarse gravel aggregate (10 mm [0.39
3 in.] maximum size), fine aggregate (sand) and ordinary Portland cement (ASTM C150 Type
4 D). The mix proportions by weight were cement : sand : gravel = 1 : 2.75 : 3.36. The
5 water/cement ratio was 0.7. The targeted cube compressive strength after 28 days was 25
6 MPa (3.63 ksi). This value was favored because it simulates the deterioration in the concrete
7 compressive strength that may exist in deficient RC structures.

8 Tensile tests were carried out on the steel reinforcement used in this study in order to quantify
9 its mechanical properties. The test results for the strength and stiffness properties of the steel
10 reinforcement are given in **Table 2**.

11 The CFRP fabrics (sheets) used in this investigation were the commercially available
12 SikaWrap-230C. These are unidirectional woven carbon fiber fabrics that are usually used in
13 conjunction with an epoxy laminating resin, in this case Sikadur-330, to provide a composite
14 strengthening system. The fabric, adhesive and laminate (i.e. fabric + adhesive) properties, as
15 obtained from the manufacturer's data sheets^{12,13}, are presented in **Table 3**.

16

17 **Instrumentation**

18 The deflections of all specimens were measured with linear resistance displacement
19 transducers (LRDTs). The LRDTs were positioned either at mid-span for the beams tested in
20 four-point bending or at position (A), i.e. at $a = 3.8d$ (see **Fig. 2**), for the specimens tested in
21 three-point bending.

22 The strain in the transverse steel reinforcement and in the CFRP sheets was measured with
23 strain gauges. The strain gauges were bonded to the internal steel reinforcement before
24 casting whereas the strain gauges on the CFRP sheets were bonded to the surface of the sheet
25 after it had cured but before starting the final phases of LP1 and LP2. **Fig. 3** illustrates the
26 locations of these strain gauges. The strain gauges on the transverse steel reinforcement are
27 designated TR# where # indicates the strain gauge number. Similarly, CF# indicates the
28 strain gauge number for the strain gauges on the CFRP sheets. Strain gauges on the steel and
29 CFRP shear reinforcement are spaced 250 mm (9.84 in.) c/c.

30 The LRDTs and strain gauge readings were acquired using an automatic data logging system.

31

32 **EXPERIMENTAL RESULTS AND DISCUSSION**

33 **Shear force capacity**

1 The unstrengthened shear capacity of each specimen as well as the shear force at failure and
2 the gain in shear capacity above the corresponding unstrengthened control beam are
3 presented in **Table 4**. The unstrengthened control beam tested by Hoult and Lees¹⁴ is
4 nominally identical to F/215/LP1/4.6 and F/215/LP2/4.6. Hence, it will be used as a basis of
5 comparison for these two beams. It failed in shear at a shear force of approximately 88 kN
6 (19.78 kips).

7 Specimen U/295/LP0/4.5 was an unstrengthened control beam designed to fail in shear in
8 order to create a baseline reading for the 350 mm (13.78 in.) deep specimens. The
9 unstrengthened specimen U/295/LP2/4.5 was tested to examine the effect of LP2 on the shear
10 carrying capacity of an unstrengthened beam.

11 The shear force carried by U/295/LP0/4.5 at failure was 107 kN (24.05 kips). The other
12 unstrengthened specimen, i.e. U/295/LP2/4.5, attained a shear force of 116 kN (26.08 kips) at
13 failure. The difference in shear force capacity between the two specimens was approximately
14 8%, suggesting that using LP2 had little significant effect on the shear force capacity of
15 U/295/LP2/4.5.

16 The 350 mm (13.78 in.) deep strengthened specimens F/295/LP1/4.5 and F/295/LP2/4.5
17 failed at a shear force of 135 kN (30.35 kips) and 133.5 kN (30.01 kN) respectively, attaining
18 increases in shear force capacity of 26.2% and 24.8% respectively. The corresponding 270
19 mm (10.63 in.) deep strengthened specimens (F/215/LP1/4.6 and F/215/LP2/4.6) failed at a
20 shear force of 102.5 kN (23.04 kips) and 96.5 kN (21.69 kips) respectively, achieving
21 increases of 16.5% and 9.7% respectively.

22 Specimen F/295/LP1/3.3 attained a shear force of 122.5 kN (27.54 kips) at failure
23 corresponding to 14.5% shear enhancement. This specimen failed in flexure. Although not
24 reported in detail in this paper, the readings of strain gauges on the longitudinal steel of
25 F/295/LP1/3.3 showed clearly that yielding had occurred. Due to the flexural failure of
26 F/295/LP1/3.3, it is possible that the difference in the shear carrying capacities between
27 F/295/LP1/3.3 and F/295/LP1/4.5 is a consequence of specimen F/295/LP1/3.3 attaining its
28 flexural capacity rather than a consequence of the change in the longitudinal reinforcement
29 ratio. Hence, it can only be concluded that the capacity of F/295/LP1/3.3 was increased by at
30 least 15.5 kN (3.49 kips).

31 The two load histories investigated, LP1 and LP2, did not generally seem to have a
32 significant effect on the load carrying capacity of the strengthened beams. There was less
33 than a 7% capacity difference in the 270 mm (10.63 in.) deep strengthened beams and the 350
34 mm (13.78 in.) deep strengthened beams did not show any significant difference in capacity.

1 During testing, it was clear that pre-existing cracks were interacting with subsequent crack
2 formation yet interestingly this interaction did not seem to impact greatly on the peak load at
3 failure. As will be explained later in this paper, the strengthened beams failed due to the
4 debonding of the CFRP sheets. Such debonding failures could conceal any possible load
5 effects that may have affected the shear force capacity at a further loading stage. This
6 possibility may be further investigated by preventing debonding at a fairly early stage. This
7 can be achieved either by fully wrapping the beam or, more practically, by using fasteners to
8 secure the CFRP sheets and so exploit its tensile strength more effectively.

9 The CFRP contribution of the strengthened beams was clearly affected by the change in beam
10 depth. It increased with increasing depth from 14.5 kN (3.26 kips) to 28 kN (6.30 kips) in the
11 strengthened beams subjected to LP1 and from 8.5 kN (1.91 kips) to 26.5 kN (5.96 kips) in
12 the strengthened beams subjected to LP2. These results suggest that the bonded fabric system
13 is more effective when used on the “deeper” 350 mm (13.78 in.) beams. In the “shallower”
14 270 mm (10.63 in.) deep specimens, the fabric strengthening was not fully effective since
15 only a fairly short bonded length is available for force transfer. Another explanation could be
16 that the “deeper” beams had more CFRP area bridging a shear crack compared to the
17 “shallower” beams.

18

19 **Shear force-deflection relationship**

20 The shear force-deflection curves for the specimens considered in this investigation are
21 presented in **Fig. 4**. Except for F/295/LP1/3.3 which failed in flexure, all specimens
22 experienced a drop in load at peak shear force which is a characteristic of brittle (shear)
23 failure. The unstrengthened specimens were more brittle compared to the corresponding
24 strengthened specimens. The deflection ratio between the strengthened beams that failed in
25 shear and the corresponding unstrengthened beams, however, is of the same order of
26 magnitude, approximately 1.22.

27 In the initial loading stage (up to 70% of the unstrengthened capacity), the strengthened
28 beams, except F/215/LP2/4.6 as it was initially loaded at a shorter shear span, behaved
29 similarly to the corresponding unstrengthened specimens.

30 In the final loading stage, all the strengthened beams attained slightly higher stiffness, which
31 deteriorated gradually with increased loading due to cracking until failure occurred.
32 Specimens U/295/LP2/4.5 and F/295/LP2/4.5 show stiffer shear force-deflection
33 relationships as they had shorter lengths.

1 Specimen F/295/LP1/3.3 had the same geometrical dimensions as the other 350 mm (13.78
2 in.) deep specimens. However, its longitudinal steel ratio was 27% less. That is why, after the
3 initial flexural cracking of the concrete at a shear force of about 20 kN (4.5 kips), this beam
4 experienced more deflection at a given shear force compared to U/295/LP1/4.5 and
5 F/295/LP1/4.5. This difference in deflection at the end of the pre-cracking process was in
6 excess of 2mm (0.08 in.). Except for the extra deflection, at reloading the behavior of
7 F/295/LP1/3.3 was similar to that of F/295/LP1/4.5 up to a shear force of 122.5 kN (27.54
8 kips). At that load level, the beam developed ductile behavior as illustrated by the
9 approximately 10 mm (0.39 in.) long yield plateau seen in **Fig. 4** and then failed in flexure.

11 **Failure mode**

12 The two unstrengthened beams failed in shear as shown in **Fig. 5**. U/295/LP0/4.5 failed due
13 to an inclined crack that ran from the support to the load point. This inclined crack followed a
14 path at an angle of approximately 24° in the web and a much shallower path in the flange.

15 Specimen U/295/LP2/4.5 failed due to an inclined crack that penetrated the flange and
16 propagated towards the load pad. This was accompanied by the excessive opening of one of
17 the inclined cracks in the web as shown in **Fig. 5**. Of importance is that the inclined shear
18 crack that formed in the first stage of loading remained stable and did not contribute to the
19 failure mechanism. This may explain why load case LP2 had little effect on the shear
20 carrying capacity.

21 Specimens F/295/LP1/4.5 and F/295/LP2/4.5 failed due to an inclined crack that extended
22 into the flange and ran to the load pad. This was preceded by the debonding of the CFRP
23 sheets located between the splitting zones shown in **Fig. 6** and the load pads. The fabric
24 splitting was caused by a set of vertical cracks that formed initially in the flange and extended
25 downward to the web. The formation of such cracks in the flange can be explained by strain
26 compatibility between the flange and the web. With increased loading, the web portion
27 between the support and the major shear crack attempts to rotate. However, the flange
28 restrains its movement. Consequently, horizontal tensile strains and stresses develop in the
29 top part of the flange. Eventually, the tensile stresses exceed the tensile strength of the
30 concrete and vertical cracks form. Hence, the fabric splitting close to the support region of
31 the beam may be prevented by applying a layer of the unidirectional CFRP sheets parallel to
32 the longitudinal axis of the beam.

1 Specimen F/215/LP2/4.6, which was tested before F/215/LP1/4.6, is pictured at failure in
2 **Fig. 7**. Initially, the fabrics started to peel off at the web-flange interface closer to the support.
3 The beam failed due to peeling off of the fabrics and concrete failure in the end region of the
4 beam before the inclined cracks could reach the load pad. However, the penetration of the
5 inclined cracks well into the flange and their progress toward the load pad were signs that
6 shear failure was imminent. It is possible that the reduced support area in this specimen due
7 to chamfering and the relatively short overhang might have led to concrete failure in the end
8 region of the beam.

9 When the support area was not chamfered and the overhang length was increased in specimen
10 F/215/LP1/4.6, the beam failed in shear as shown in **Fig. 7**. The fabrics started to peel off in a
11 similar way to that of F/215/LP2/4.6 and the inclined cracks continued to propagate towards
12 the load pad and backwards above the support and into the overhang until, eventually, they
13 led to beam failure. This was accompanied by separation between concrete and fabrics as can
14 also be seen in **Fig. 7**.

15 Specimen F/295/LP1/3.3 failed in flexure due to the crushing of the concrete in the
16 compression zone at the middle of the beam as shown in **Fig. 8**. This result is important as it
17 shows that the externally bonded CFRP sheets can change the mode of failure from a brittle
18 shear failure to a ductile flexural failure. The CFRP composites in the two shear spans of
19 F/295/LP1/3.3 were still intact and bonded to the beam web at failure.

20 It was not possible to measure the width of shear cracks in the strengthened specimens as
21 these cracks were covered by the CFRP sheets. Nevertheless, It was expected that very
22 limited size effects, if any, existed due to the limited increase in beam depth from 270 mm
23 (10.63 in.) to 350 mm (13.78 in.).

24

25 **Strain in the steel shear reinforcement**

26 This section reports on the strain in the transverse steel reinforcement in the shear spans
27 where failure occurred. **Fig. 3** shows the positions of the strain gauges on the transverse steel
28 reinforcement. For the purpose of interpreting results, the shear links are categorized into
29 “outer links” (TR1), “middle links” (TR2 and TR3 in the 350 mm [13.78 in.] deep specimens,
30 and TR2 in the 270 mm [10.63 in.] deep specimens) and “inner links” (TR4 in the 350 mm
31 [13.78 in.] deep specimens and TR3 in the 270 mm [10.63 in.] deep specimens).
32 Unfortunately, some strain gauges failed during testing and their results were discarded.

33 The outer and middle shear links in the unstrengthened beams started to function only after a
34 shear force of between 30 kN (6.74 kips) and 45 kN (10.12 kips) (see **Fig. 9**). Thereafter, the

1 strain in the stirrups increased significantly with increasing load. Most of the shear links in
2 this group attained their yield strain after a shear force of approximately 90 kN (20.23 kips).
3 This was expected since these links were crossed by the major shear cracks.
4 The outer and middle shear links in the strengthened beams experienced five phases during
5 loading. In the initial phase, which is bounded on the upper end by a shear force between 35
6 kN (7.87 kips) and 45 kN (10.12 kips), the contribution of the shear links to the resistance
7 was negligible. In the second phase, which included loading to 70% of the unstrengthened
8 shear capacity, the shear links started to develop strain due to the initiation and propagation
9 of inclined cracks. In the third phase, unloading to 40% of the unstrengthened shear capacity
10 reduced the strain in the outer and middle shear links. The fourth phase is marked by the
11 addition of the CFRP sheets and the stiffer response shown by the transverse steel
12 reinforcement on further reloading. The transverse steel strain showed limited increases with
13 increasing load until, in the final stage, yielding was achieved in most cases. The transverse
14 steel reinforcement that yielded is easily identified by the plateaus featured in **Fig. 9**.
15 **Fig. 9** also shows that the inner links carried the least amount of strain in all test specimens.
16 The strains in this group of transverse reinforcement developed at a relatively low rate even
17 after the formation of shear cracks. This is mainly because the beam region close to the load
18 pad, where the inner links were located, did not experience significant inclined cracking.

19

20 **Strain in the CFRP sheets**

21 The shear force-strain curves for the externally bonded CFRP sheets in the shear spans where
22 failure occurred are shown in **Fig. 9**. The positions of the strain gauges are given in **Fig. 3**.
23 The fabrics are categorized into “outer fabrics” (CF1), “middle fabrics” (CF2 and CF3 in the
24 350 mm [13.78 in.] deep specimens, and CF2 in the 270 mm [10.63 in.] deep specimens) and
25 “inner fabrics” (CF4 in the 350 mm [13.78 in.] deep specimens and CF3 in the 270 mm
26 [10.63 in.] deep specimens). Some strain gauges failed during testing and their results were
27 discarded due to the erroneous data they provided.

28 The curves feature two phases. In the first phase, the fabrics started to resist the further
29 opening of existing shear cracks at the inception of the final reloading stage. They continued
30 to develop tensile strain with increased load up to approximately the peak loads. In the
31 second stage, the fabrics started to debond and finally peeled off. Debonding is indicated by
32 the reversing of the shear force-strain curves.

33 In a given beam, the middle fabrics – represented by CF3 in the 350 mm (13.78 in.) deep
34 specimens and CF2 in the 270 mm (10.63 in.) deep specimens – developed the highest strain.

1 The sheets bonded to the 350 mm (13.78 in.) deep specimens developed higher strains
2 compared to those bonded to the 270 mm (10.63 in.) deep specimens. This increase in the
3 effectiveness of the fabrics can be explained by the increase in bond length. This result
4 highlights the fact that the deeper the section, the higher the potential of the sheets to
5 experience strain and hence provide shear enhancement.

6 7 **COMPARISON OF EXPERIMENTAL RESULTS WITH PREDICTIONS** 8 **OF SHEAR STRENGTHENING DESIGN GUIDELINES**

9 Design guidelines for externally bonded FRP shear reinforcement have been developed in the
10 UK and elsewhere. In the UK, the Concrete Society Technical Report 55⁵ (TR 55) is the first
11 – and currently the sole – standard document to give guidance on the design of externally
12 bonded FRP shear reinforcement. The design procedure adopted by TR 55⁵ is based upon that
13 proposed by Denton et al.¹⁵ and assumes that the ultimate shear capacity of an FRP
14 strengthened beam can be expressed as the sum of the shear forces carried by the concrete,
15 the internal steel shear reinforcement and the external FRP shear reinforcement. Similarly,
16 the ACI 440.2R-08⁴ shear strengthening design model, based on work by Khalifa et al.¹⁶, and
17 *fib* Bulletin 14⁶ shear strengthening design guidelines, based on work by Triantafillou and
18 Antonopoulos¹⁷, use the same approach as TR55⁵, assuming that the shear capacity of a
19 strengthened RC beam can be expressed as the sum of the concrete, steel and FRP
20 contributions. Further, the FRP contribution in the three aforementioned design guidelines is
21 determined by adopting the truss analogy and assuming the inclination angle of the shear
22 cracks to be 45°. The main difference among the three models is in the method of evaluating
23 the effective strain in the FRP reinforcement.

24 A database of eight experimental results against which to compare the predictions of ACI
25 440.2R-08⁴, TR55⁵ and *fib* Bulletin 14⁶ has been assembled. The database beams had T-
26 shaped cross-sections, internal steel shear reinforcement, and shear span to effective depth
27 ratios greater than or equal to 2.5. Although the design guidelines should be validated with a
28 larger database, there have not been so many tests on RC Beams that meet the above criteria.
29 RC beams that do not meet the above criteria are deemed to lie beyond the scope of this study
30 and hence are not included in the database.

31 Three of the database beams are the fabric-strengthened beams F/295/LP1/4.5, F/295/LP2/4.5,
32 and F/215/LP1/4.6 detailed in this study. The other five beams are SB-S1-2L-175 and SB-S1-
33 0.5L-350 tested by Bousselham and Chaallal^{7,8}, Specimen No. 2 tested by Sato et al.¹⁸, and
34 T4S2-C45 and T6S4-C90 tested by Deniaud and Cheng^{19,20}. All beams included in the

1 database, except those tested by Deniaud and Cheng^{19,20}, were strengthened with continuous
2 U-shaped externally bonded CFRP shear reinforcement. The externally bonded shear
3 reinforcement in the beams tested by Deniaud and Cheng^{19,20} consisted of CFRP U-strips
4 spaced 100 mm (3.94 in.) c/c. All beams included in the database failed in shear due to the
5 peeling off of the CFRP reinforcement.

6 **Table 5** compares the contributions of the externally bonded CFRP system predicted by the
7 ACI 440.2R-08⁴, TR55⁵ and *fib* Bulletin 14⁶ design guidelines with the experimental results
8 from the literature and the testing described in this paper. All safety factors are set equal to
9 1.00 for the purpose of the comparison except for the ACI 440.2R-08⁴ FRP strength reduction
10 factor, ψ_f , which is set equal to 0.85 as it is an integral part of the nominal shear capacity
11 expression. The experimental contributions of the CFRP sheets were calculated by
12 subtracting the experimental unstrengthened shear capacity from the experimental
13 strengthened shear capacity for each beam. It should be noted that some of the strengthened
14 beams reported in this paper had slightly higher concrete compressive strength than the
15 corresponding unstrengthened beams. However, further analyses carried out by the authors
16 (not reported in this paper) showed that this slight difference in the concrete compressive
17 strength had little significant effect on the predicted FRP contribution.

18 The total predicted shear force has not been compared to the total experimental shear force
19 because such a comparison can lead to erroneous conclusions. Such a comparison requires
20 the use of conventional design codes such as the Eurocode 2²¹ (EC 2) to calculate the
21 concrete and steel contributions to the total shear force capacity. Such codes often
22 underestimate the concrete and steel contributions to the total shear force because they
23 assume that only the web of the beam is effective when calculating the shear force capacity of
24 a T-beam. For example, the total predicted shear force capacity of F/215/LP1/4.6 using the
25 EC 2²¹ and the TR 55⁵ design equations is 94.1 kN (21.15 kips). As the total experimental
26 shear capacity of F/215/LP1/4.6 is 102.5 kN (23.04 kips), this would lead to the conclusion
27 that the design model of the TR 55⁵ which overestimates the FRP contribution to the shear
28 force capacity of F/215/LP1/4.6 by a factor of 2.08, is safe. Hence, while comparing the total
29 experimental shear force capacity to the total predicted shear force capacity could result in a
30 conservative prediction; such a comparison may lead to the erroneous conclusion that an
31 over-conservative CFRP design model is safe.

32 The ACI 440.2R-08⁴ design model is statistically the best model among the three design
33 models investigated. However, the ACI 440.2R-08⁴ design model has a mean predicted to
34 experimental ratio of 1.41 and a standard deviation of 0.53. This is probably due to the fact

1 that the bond model used in the ACI 440.2R-08⁴ design model is based on limited
2 experimental data.

3 The TR 55⁵ predictions overestimated the contributions of the externally bonded CFRP sheets
4 to the shear force capacity with a mean predicted to experimental ratio of 1.64 and a standard
5 deviation of 0.62. The inaccuracy of the TR 55⁵ predictions stems from the inaccuracy of its
6 effective strain model. TR55⁵ predicts that the effective strain in the CFRP sheets of
7 F/295/LP1/4.5, F/295/LP2/4.5 and F/215/LP1/4.5 is approximately 2560 micro-strain, 2650
8 micro-strain and 2810 micro-strain respectively. The experimentally measured CFRP strain
9 for these beams varied between 125 micro-strain and 1200 micro-strain (see **Fig. 9**).

10 The *fib* Bulletin 14⁶ design model is statistically the worst model among the three design
11 models investigated. It has a mean predicted to experimental ratio of 2.52 and a standard
12 deviation of 0.76. The deficiency of the model is probably due to the fact that the equations
13 for the effective FRP strain were obtained by regression analysis with limited experimental
14 data. Hence, the effective FRP strain equations do not consider the bond mechanism which
15 affects the mode of failure.

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SUMMARY AND CONCLUSIONS

18 This study investigates the structural behavior of precracked RC T-beams strengthened in
19 shear with externally bonded CFRP sheets. The influence of load history, effective depth of
20 the beam and longitudinal steel ratio on the strengthened behavior was studied. The
21 predictions of three international shear strengthening design guidelines were compared with
22 experimental results. Based on the results of this study, the following conclusions are drawn:

23 1. All strengthened specimens exhibited greater capacities than equivalent
24 unstrengthened control beams, with capacity enhancements ranging from 9.7% to
25 26.2%, confirming the potential effectiveness of the CFRP sheets.

26 2. The two loading patterns investigated, LP1 and LP2, did not generally seem to have a
27 significant effect on the shear capacity of the strengthened beams. During testing it
28 was clear that pre-existing cracks were interacting with subsequent crack formations
29 yet this interaction did not seem to impact greatly on the peak load at failure.

30 3. The increase in beam depth positively affected the contribution of the CFRP sheets to
31 the shear force capacity through providing additional bond length to better exploit the
32 sheets' tensile strength.

33 4. The decrease in the longitudinal reinforcement ratio from 4.5% to 3.3% changed the
34 mode of failure from brittle shear failure to ductile flexural failure.

1 5. The ACI 440.2R-08⁴, TR55⁵ and *fib* Bulletin 14⁶ shear strengthening design
2 guidelines overestimated the contribution of the externally bonded CFRP sheets with
3 mean predicted to experimental ratios of 1.41, 1.64, and 2.52, and standard deviations
4 of 0.53, 0.62, and 0.76 respectively.
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8 (EPSRC) through grant GR/S55101/01 is gratefully acknowledged.
9

10 NOTATION:

11 A_s = area of steel tension reinforcement
12 a = shear span
13 b_w = web width
14 d = effective beam depth
15 f_{cu} = cube compressive strength
16 p = longitudinal reinforcement ratio
17 ψ_f = FRP strength reduction factor used in the nominal shear capacity expression
18 suggested by the ACI 440.2R-08
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Table 1–Summary of test specimens

Specimen	f_{cu} , MPa (ksi)	a , mm (in.)	d , mm (in.)	A_s , mm ² (in. ²)
U/295/LP 0/4.5	24 (3.48)	1125 (44.29)	295 (11.61)	1383 (2.14)
U/295/LP 2/4.5	28 (4.06)	1125 (44.29)	295 (11.61)	1383 (2.14)
F/295/LP 1/4.5	24 (3.48)	1125 (44.29)	295 (11.61)	1383 (2.14)
F/295/LP 2/4.5	27 (3.92)	1125 (44.29)	295 (11.61)	1383 (2.14)
F/295/LP 1/3.3	28 (4.06)	1125 (44.29)	295 (11.61)	1030 (1.60)
F/215/LP 1/4.6	32 (4.64)	820 (32.28)	215 (8.46)	1030 (1.60)
F/215/LP 2/4.6	25 (3.63)	820 (32.28)	215 (8.46)	1030 (1.60)

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Table 2–Steel reinforcement properties

Bar diameter, mm (in.)	Yield strength, MPa (ksi)	Yield strain	Ultimate strength, MPa (ksi)
6 (0.24)	580 (84.12)*	0.0050*	586 (84.99)
8 (0.31)	520 (75.42)	0.0028	594 (86.15)
16 (0.63)	500 (72.52)	0.0032	593 (86.01)
20 (0.79)	580 (84.12)	0.0038	680 (98.63)
25 (0.98)	440 (63.82)*	0.0044*	540 (78.32)

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* Using the 0.2% offset method.

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Table 3–CFRP sheets and adhesive properties

Material	Tensile strength, MPa (ksi)	Ultimate strain	Elastic modulus, MPa (ksi)
CFRP sheets*	4300 (623.66)	0.0180	238000 (34519)
Epoxy resin	30 (4.35)	0.0090	4500 (653)
Composite material**	350 (50.76)	0.0125	28000 (4061)

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* Nominal thickness per layer = 0.131 mm (0.0052 in.).

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** Nominal thickness per layer = 1 mm (0.0394 in.).

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Table 4–Experimental results

Specimen	Unstrengthened shear capacity, kN (kips)	Shear force at failure, kN (kips)	Gain in shear strength, kN (kips)	Gain in shear strength, %
U/295/LP0/4.5	107.0 (24.05)	107.0 (24.05)	0 (0)	0
U/295/LP2/4.5	107.0* (24.05)	116.0 (26.08)	9 (2.03)	8.4
F/295/LP1/4.5	107.0 (24.05)	135.0 (30.35)	28.0 (6.30)	26.2
F/295/LP2/4.5	107.0 (24.05)	133.5 (30.01)	26.5 (5.96)	24.8
F/295/LP1/3.3	107.0 (24.05)	122.5 (27.54)	15.5 (3.49)	14.5
F/215/LP1/4.6	88.0** (19.78)	102.5 (23.04)	14.5 (3.26)	16.5
F/215/LP2/4.6	88.0** (19.78)	96.5 (21.69)	8.5 (1.91)	9.7

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* For purpose of comparison with U/295/LP0/4.5.

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** Based on the control beam, tested by Hoult and Lees¹⁴, which is nominally identical to F/215/LP1/4.6 and F/215/LP2/4.6.

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Table 5–Experimental versus predicted shear resistance due to FRPs

Specimen	Experimental, kN (kips)	ACI 440.2R -08 ⁴ , kN (kips)	TR55 ⁵ , kN (kips)	<i>fib</i> Bulletin 14 ⁶ , kN (kips)
F/295/LP1/4.5	28.0 (6.30)	51.4 (11.56)	63.1 (14.19)	70.1 (15.76)
F/295/LP2/4.5	26.5 (5.96)	56.7 (12.74)	66.7 (14.99)	74.1 (16.66)
F/215/LP1/4.6	14.5 (3.26)	31.6 (7.11)	30.1 (6.77)	57.5 (12.93)
SB-S1-2L-175 ⁷	12.2 (2.74)	14.9 (3.34)	21.8 (4.90)	26.6 (5.98)
SB-S1-0.5L-350 ⁸	19.2 (4.32)	23.5 (5.30)	23.0 (5.17)	49.0 (11.02)
Specimen No. 2 ¹⁸	24.0 (5.40)	21.8 (4.90)	23.9 (5.37)	50.2 (11.29)
T4S2-C45 ¹⁹	17.8 (4.00)	19.2 (4.32)	30.2 (6.79)	52.2 (11.74)
T6S4-C90 ²⁰	85.3 (19.18)	61.0 (13.72)	50.2 (11.29)	95.8 (21.54)

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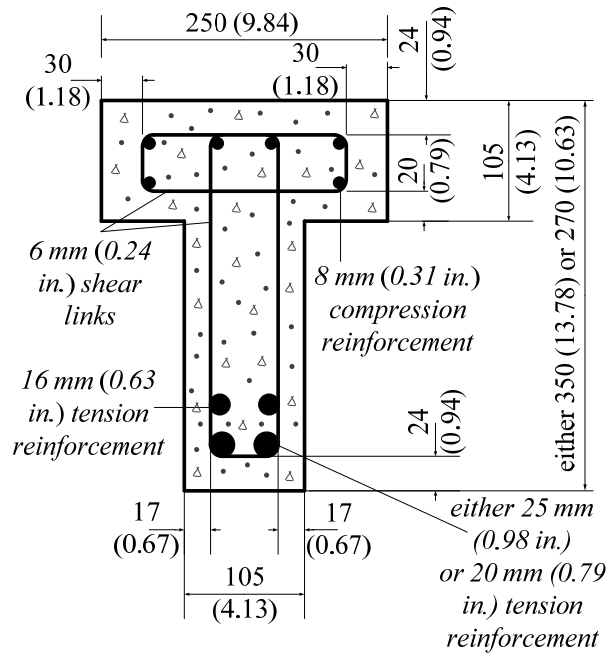


Fig. 1– Cross-sections details – dimensions in mm (in.).

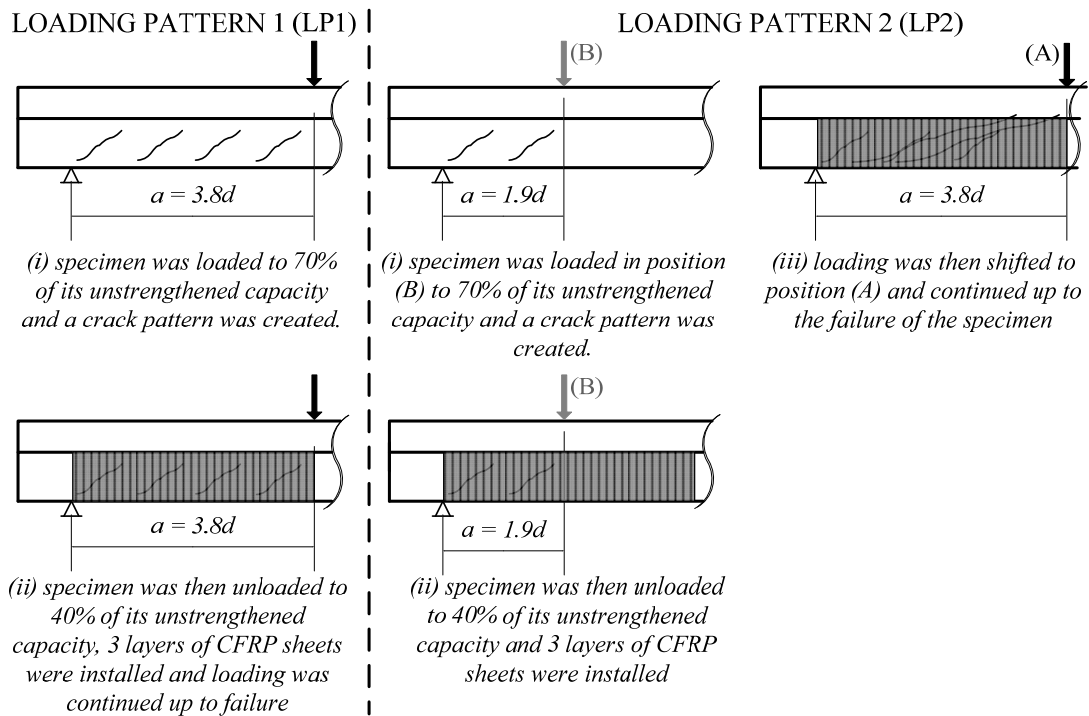


Fig. 2–Loading patterns.

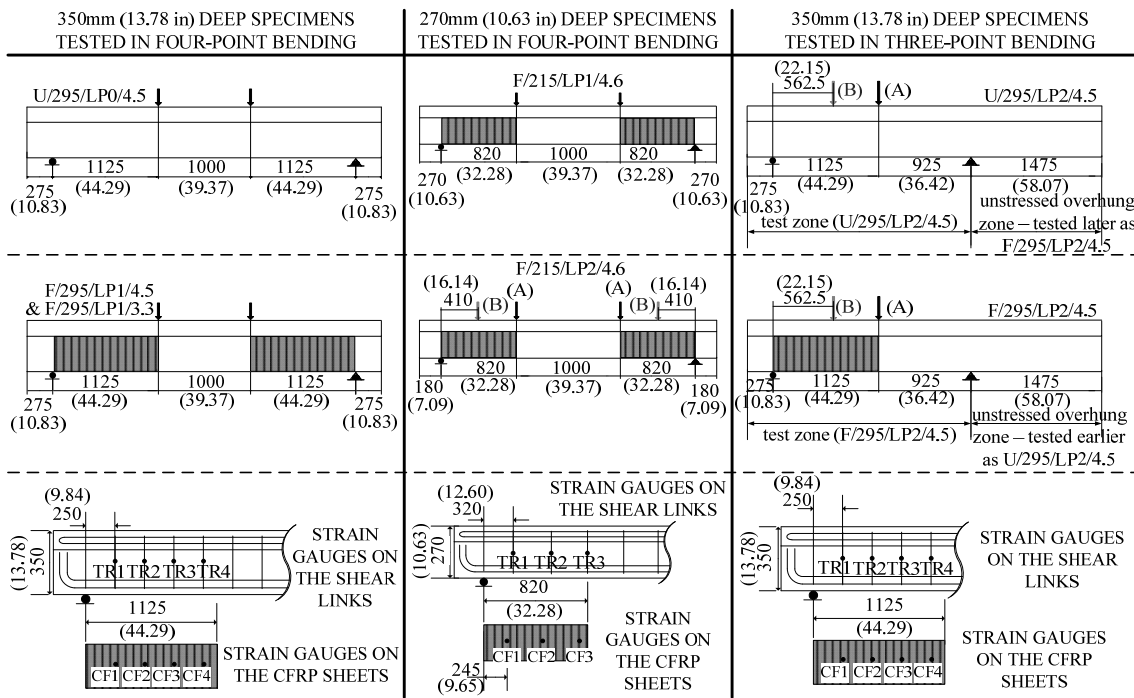


Fig. 3– Details of test specimens – dimensions in mm (in.).

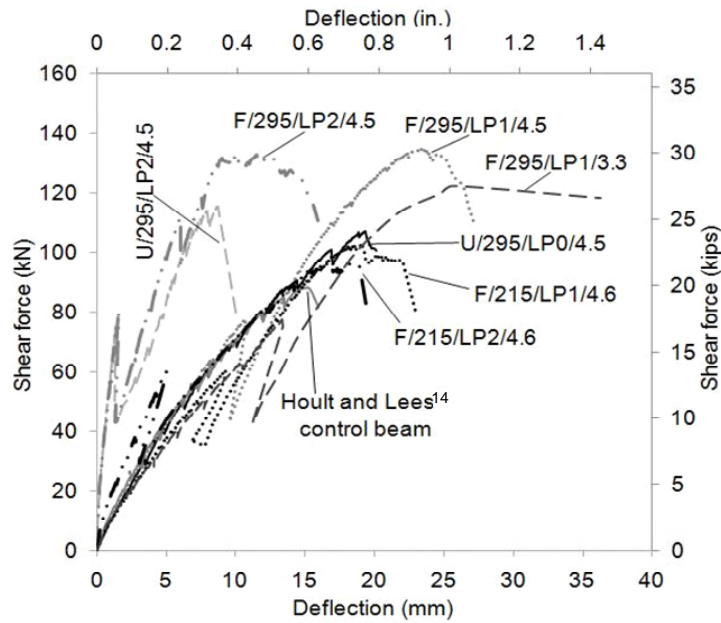


Fig. 4– Shear force-deflection curves.

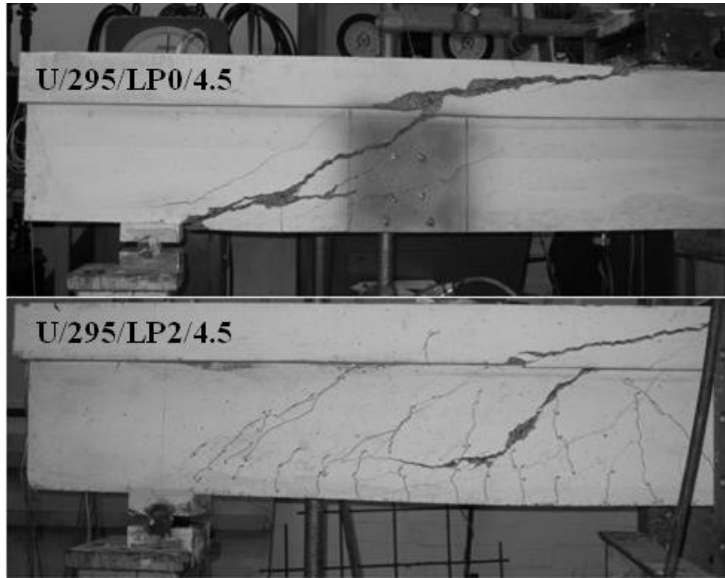


Fig. 5–Unstrengthened specimens at failure.

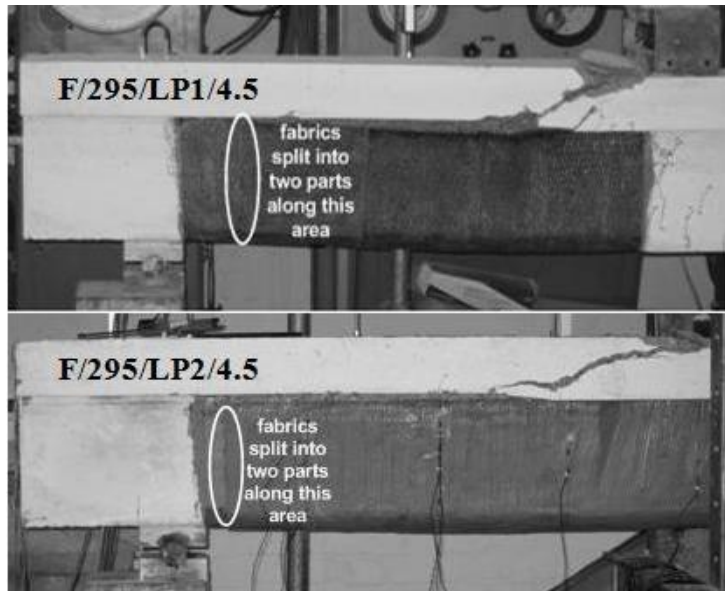


Fig. 6–F/295/LP1/4.5 and F/295/LP2/4.5 at failure.



Fig. 7–F/215/LP1/4.6 and F/215/LP2/4.6 at failure.



Fig. 8–Flexural failure of F/295/LP1/3.3.

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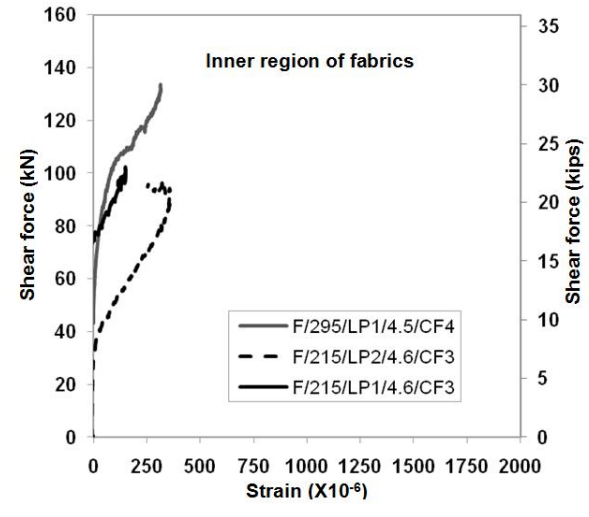
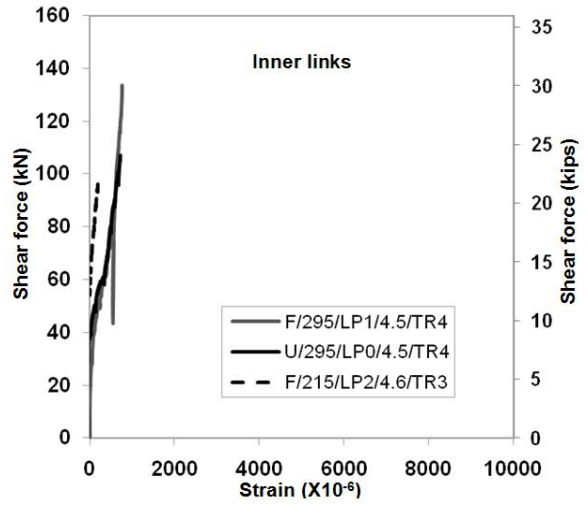
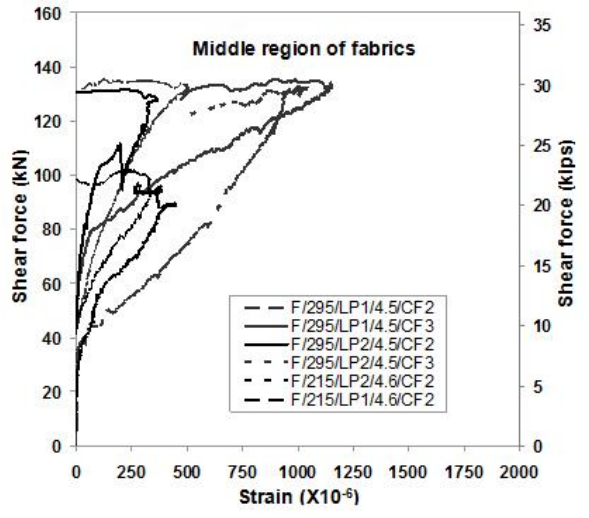
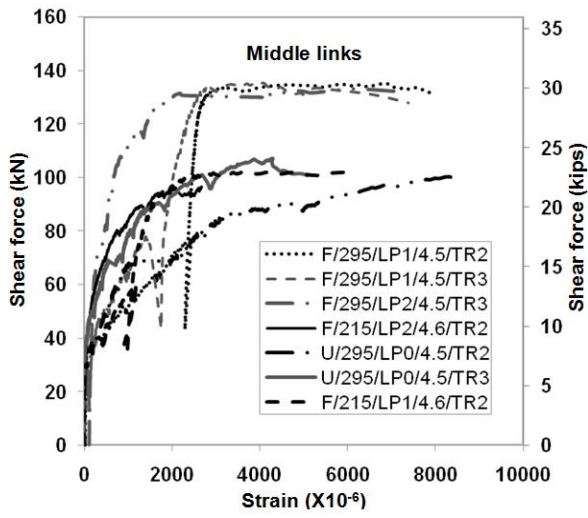
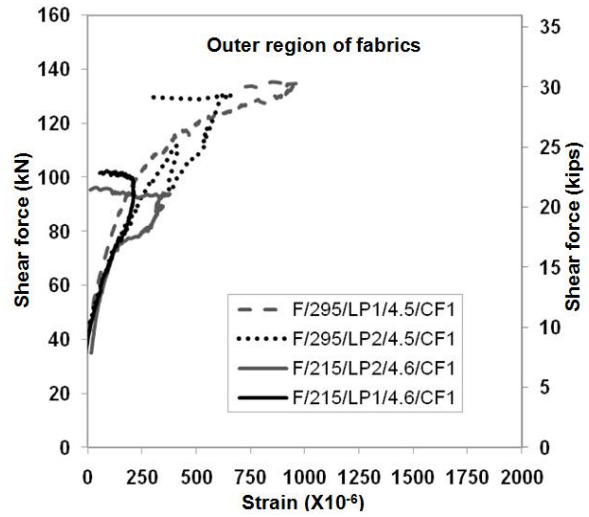
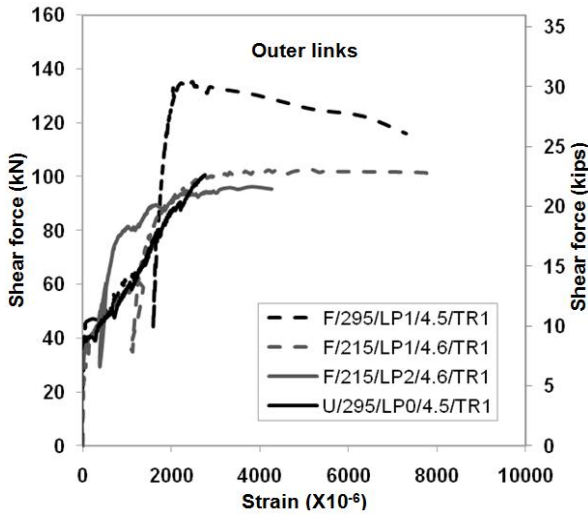


Fig. 9–Shear force versus strain in the internal and external shear reinforcement.