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Shear strengthening of reinforced concrete T-beams with fully or partially bonded fibre-reinforced polymer composites

A series of 10 reinforced concrete T-beams, designed deficient in shear, were tested in order to investigate the shear performance achieved through externally applied U-shaped FRP composite strips. Key variables of the study were: type of FRP composite, type of surface bonding and type of end anchorage for the strips. Carbon fibre-reinforced polymer (CFRP), glass fibre-reinforced polymer (GFRP) and high modulus of elasticity carbon fibre-reinforced polymer (Hi-CFRP) strips were the special composite types with different elastic moduli, full or partial bonding of the strips to the beam surface were the variables for the type of surface bonding. All partially bonded FRP strips were free from surface bonding, whereas epoxy-bonded FRP anchors were used at their ends close to the slab-to-beam connection. Those strips with full surface bonding have either epoxy-bonded FRP anchors at their ends or the strip ends were without anchorage. The test results revealed that shear-deficient beams may well be strengthened by the externally applied FRP strips. However, the level of strength enhancement and the failure pattern is closely influenced by the composite's elastic modulus, the type of surface bonding and the type of end anchorage for the FRP strip itself. The enhancement of the Hi-CFRP strips did not live up to expectations. The used of unbonded FRP for shear strengthening yielded promising results.

Keywords: reinforced concrete, beam shear strengthening, fibre-reinforced polymer, anchorage, partially bonded FRP, modulus of elasticity, composite

1 Introduction

Owing to their numerous advantages over traditional construction materials, fibre-reinforced polymer (FRP) composites have been extensively investigated with regard to strengthening or repairing concrete structures. Good durability, superior corrosion resistance, high strength-to-weight ratio, ability to cope with different reinforced concrete sectional shapes and corners, ability to solve different deficiencies such as shear or flexural and the capacity to address an occupant-friendly retrofitting technique may be listed among the inherent advantages regarding materials or ease of application [1, 2].

The shear mechanism of reinforced concrete (RC) beams depends on many variables and yields a very complex failure pattern without adequate warning [3]. The equations for the beam shear capacity predictions in most codes of practice are mostly empirical or semi-empirical. The need for shear strengthening in beams may arise due to either misinterpretation of this mechanism, resulting in inferior load capacity, or higher performance expectancies. Cases of regular steel corrosion or accidental overloads may also be the reasons for applying FRP to existing RC beams [4].

There are several methods for strengthening existing RC beams in shear, including RC jacketing, surface bonding of external steel plates using epoxy or steel bolts, or FRP bonding, either surface or near-surface [5–9]. Various studies have been conducted to date and various strengthening methods have been introduced for shear-deficient beams [9–16]. In these studies, concrete strength, flexural reinforcement ratio, sectional geometry, shear span-to-depth ratio a/d, type of loading, strengthening methodology, strengthening material and amount and configuration of strengthening composite were investigated as test variables. FRP material is generally applied either through wet lay-up sheets or strips bonded to the external faces of the member [10, 15, 17–24], or through inserting FRP bars, strips or dry carbon fibre sheets into grooves cut in free concrete surfaces of the member for FRP strengthening of beams [9, 25–27]. Previous studies in the literature concluded that the use of FRP, externally bonded or near-surface-mounted, is an effective method for improving member strength. They also concluded that the level of strengthening may also be influenced by the FRP strip orientation and configuration [11, 16, 24, 26]. The effectiveness of the external FRP bonding has also been addressed and investigated previously [9, 17, 27–31]. Near-surface mounting of the FRP strips was reported to be more efficient in terms of exploitation of the FRP tensile strength due to the apparent larger bonding area [28, 29, 31].

2 Synopsis

The shear strength of an RC beam is mainly influenced by the yield strength and configuration of the shear reinforcement along with the concrete strength. The shear reinforcement in the form of steel stirrups has a finite yield strength, whereas the externally applied FRP composites exhibit almost brittle behaviour without a finite yield plateau. Although the tensile strengths of FRP composites

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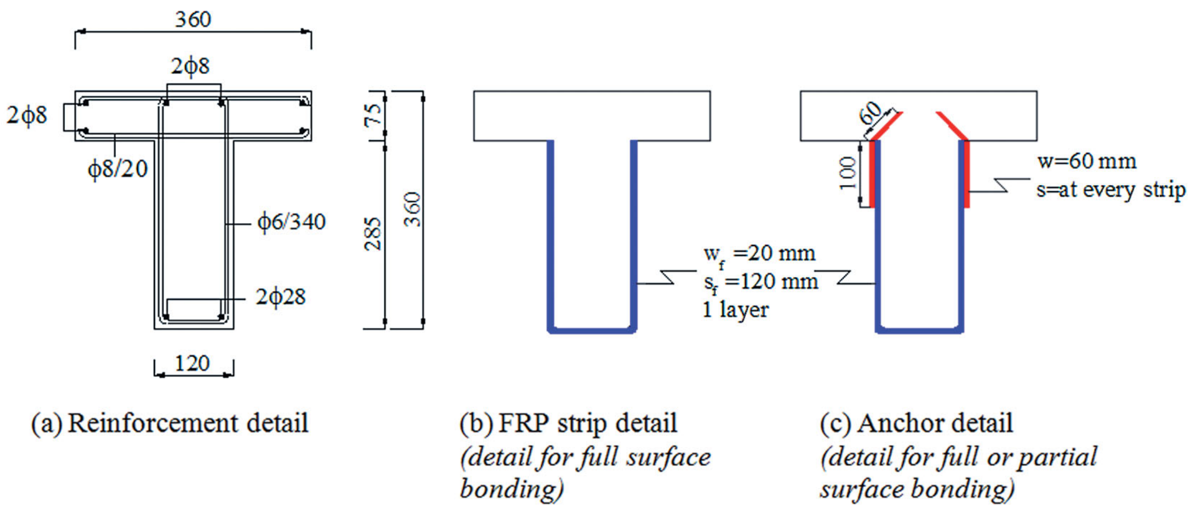


Fig. 1. Details of test specimens

are much higher than the yield strength of a regular steel stirrup, the contribution of the FRP to the beam shear capacity is not directly proportional to its strength, but mainly influenced by the FRP's ability to transfer its strength to the member. This transfer is largely influenced by the surface bonding and boundary conditions. Consequently, a strain limit for the FRP composites should be specified for the shear capacity calculations. This strain limit for different end anchorage types and different FRP types may vary significantly and results in different safety limits.

In this study, three main variables, namely FRP type, end anchorage and surface bonding, were investigated to establish how they enhance the shear strength capacity of RC beams. All the variables were critically investigated for their effects on the failure strain of the FRP and, hence, their contribution to the overall shear strength.

3 Experimental programme

In this research programme, a series of tests was carried out to investigate the shear behaviour of beams with T-sections strengthened with externally bonded FRP composite strips and loaded to failure under a monotonically increasing four-point gravity load. How the type of FRP strip and its type of surface bonding, together with its anchorage system, affect the shear capacity of beams was investigated in the light of the capacity predictions of various code approaches. The key variables were:

(1) Type of FRP composite (carbon fibre-reinforced polymer, CFRP; glass fibre-reinforced polymer, GFRP; and high modulus of elasticity carbon fibre-reinforced polymer, Hi-CFRP)
(2) Type of surface bonding of FRP strips on beam (fully bonded, partially bonded)
(3) The anchorage system used for the FRP strips (no anchorage, use of FRP anchors at beam to slab interface)

3.1 Specimens and material properties

Ten practically full-scale, shear-deficient T-section RC beams were tested to failure in the experimental programme. Cross-sectional dimensions, shear reinforcement and anchor details of the specimens are shown in Fig. 1a. The specimens were simply supported and tested under monotonically increasing four-point loading (Fig. 2). The distance between supports for all specimens was 2750 mm, the distance between point load and support in all specimens 1290 mm, resulting in a shear span-to-depth ratio $a/d = 3.8$.

The flexural reinforcement consisted of two 28 mm diameter steel bars on the positive moment side and two 8 mm diameter bars on the compression side of the beam. The shear reinforcement consisted of 6 mm diameter closed stirrups, spaced at 340 mm centre to centre throughout the beam, whereas the FRP shear reinforcement has a constant width $w_f = 20$ mm, evenly spaced at $s_f = 120$ mm (Fig. 1). The FRP anchors, with a strip width $w = 60$ mm, were designed in such a way that there would be no anchorage failure before the tensile failure of the FRP shear reinforcement. The yield strength, tensile strength and the ultimate strain values of the reinforcing bars and the compressive strength of the concrete are given in Table 1.

3.2 Loading and experimental setup

All specimens were tested as simple beams subjected to four-point loading as illustrated in Fig. 2. A hydraulic, manually driven 300 kN capacity loading system was used in order to apply a concentrated load to a steel spreader beam, resulting in two equal point loads applied 85 mm either side of the centre-line. Strain gauges recorded the load vs. mid-deflection values along with the strain variations in the FRP strips, and surface-mounted LVDTs (linear variable differential transducers) the average crack openings. Fig. 2 shows the locations of the strain gauges and LVDTs.

The strain gauges were oriented parallel with the FRP filaments in the vertical direction and located at the strip mid-height at distances of 487.5, 707.5, 927.5 and 1147.5 mm with respect to the mid-point of the beam specimen. At regular load intervals during each test, the applied load was kept constant for a short period to allow detailed monitoring of the shear crack distribution.

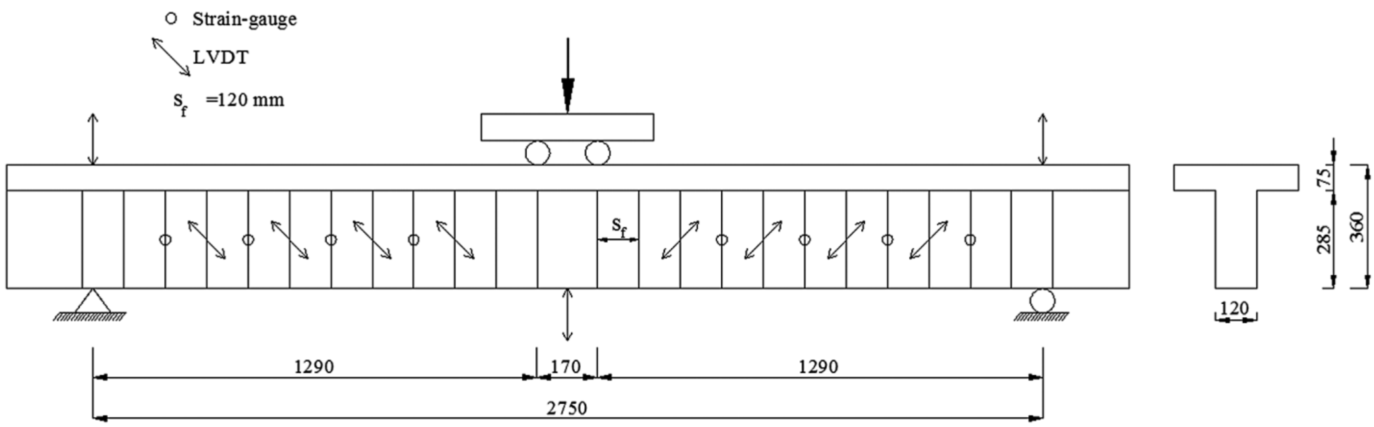


Fig. 2. Test setup and instrumentation

Table 1. Material properties of concrete and reinforcing steel

Material	Compressive strength f'_c (MPa)	Yield strength f_y (MPa)	Tensile strength f_u (MPa)	Strain at rupture (%)
Concrete	12.4	–	–	–
Steel				
D = 28 mm	–	486.3	598.9	17.7
D = 8 mm	–	478.6	716.8	28.8
D = 6 mm	–	249.0	410.8	30.2

* E_s = 200 GPa for all steel sizes

Table 2. Mechanical properties of FRP and epoxy

Property	CFRP	GFRP	Hi-CFRP	Epoxy resin I	Epoxy resin II
Design thickness (mm)	0.131	0.157	0.140	N/A	N/A
Modulus of elasticity (MPa)	238.000	73.000	640.000	3.800	3.010
Tensile strength (MPa)	4.300	3.400	2.600	30	72.4
Fibre density (g/cm^3)	1.76	2.54	2.12	N/A	N/A
Fibre weight (g/m^2)	230	400	300	N/A	N/A
Strain at rupture (%)	1.80	4.66	0.40	N/A	N/A

* Epoxy resin II used with Hi-CFRP only

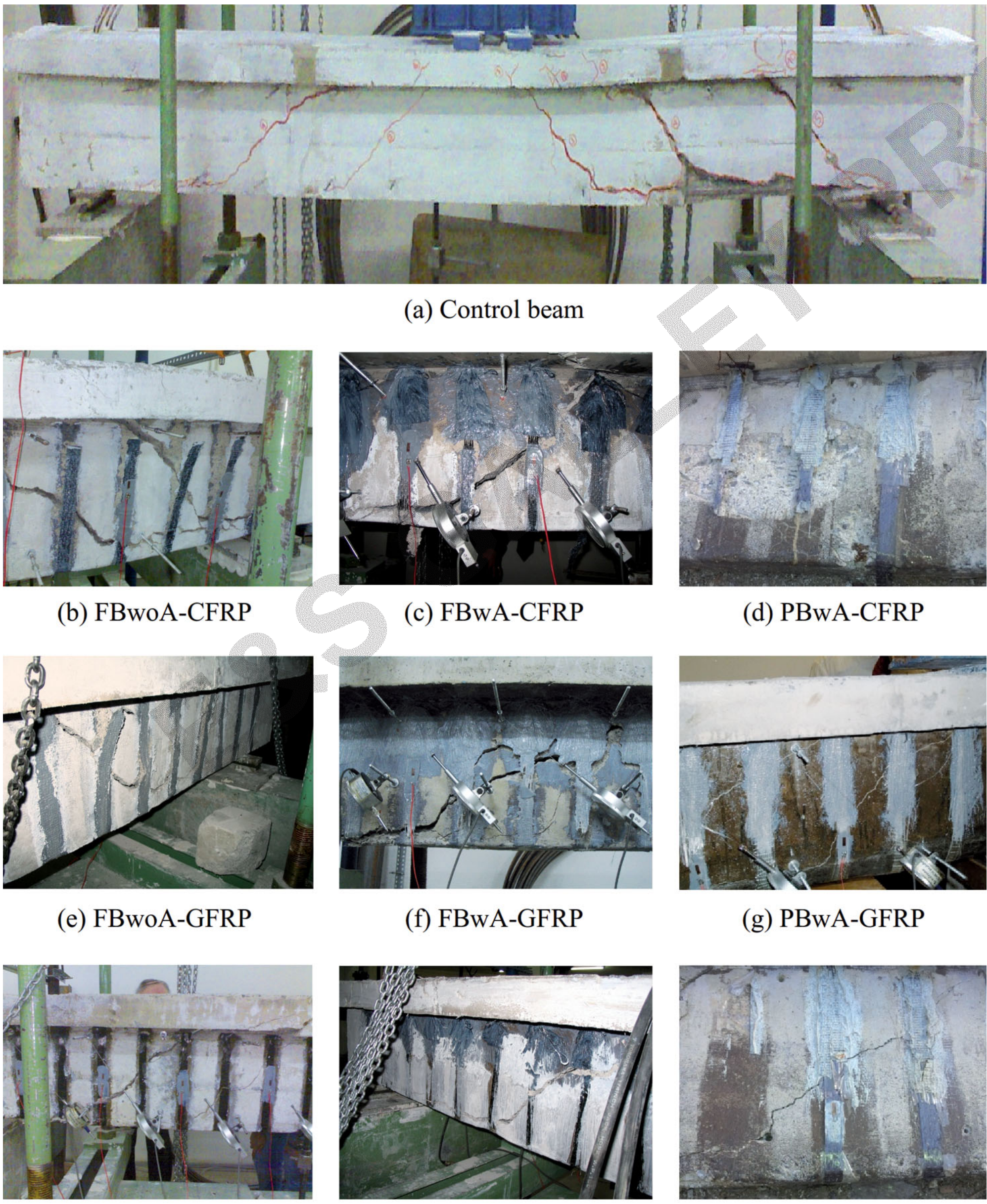
3.3 Strengthening procedure

The beam specimens were designed deficient in shear; thus, shear failure was the dominant mode of failure for both strengthened and non-strengthened specimens. Details of FRP strips and strengthening schemes are illustrated in Figs. 1b and 1c. Three different strengthening materials and three strengthening methods were used in the experiments. The mechanical properties of the strengthening materials are shown in Table 2. The strengthening was basically in the form of applying U-strips to the specimens.

The specimens with U-strips bonded to the beam (both sides and bottom surface) without a strip end anchorage are called "fully bonded without anchor", FBwoA, whereas the specimens with bonded U-strips and anchors are called "fully bonded with anchor", FBwA. On the other hand, the specimens where the U-strips were not bonded to the beam surfaces and the load transfer was only possible through the anchors are called "partially bonded with anchor", PBwA (Table 3). In all FBwA and PBwA specimens, the anchorage was in the form of an FRP strip which is initially epoxy-washed and anchored into the predrilled and properly cleaned slab hole. The free ends of the FRP anchors were later spread over the U-strips for proper force transfer (Fig. 1). The length and diameter of this 45° inclined anchor hole, right at the beam-to-slab interface, were 60 and 8 mm respectively (Fig. 1c). The anchors were made from 160 mm long x 60 mm wide FRP sheets. Anchor dimensions and configuration were the same for all specimens. After the epoxy bonding process, the strengthened beams were cured for at least 10 days in the laboratory (60–85 % relative humidity, 24 °C ambient temperature) before testing.

Table 3. FRP strip configurations

Specimen	Strip type	Strip width (mm)	Strip angle with respect to horizontal (deg)	Strip spacing (mm)	End anchor	Surface bonding of U-strips
Control	–	–	–	–	–	–
FBwoA-CFRP					no	full
FBwA-CFRP	CFRP				yes	full
PBwA-CFRP					yes	none
FBwoA-GFRP					no	full
FBwA-GFRP	GFRP	20	90	120	yes	full
PBwA-GFRP					yes	none
FBwoA-Hi-CFRP					no	full
FBwA-Hi-CFRP	Hi-CFRP				yes	full
PBwA -Hi-CFRP					yes	none



(a) Control beam

(b) FBwoA-CFRP (c) FBwA-CFRP (d) PBwA-CFRP

(e) FBwoA-GFRP (f) FBwA-GFRP (g) PBwA-GFRP

(h) FBwoA-Hi-CFRP (i) FBwA-Hi-CFRP (j) PBwA-Hi-CFRP

Fig. 3. Beam failures

Table 4. Experimental and numerical results

	Experimental results				$\varepsilon_{FRP\text{-}calc}$			
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Specimen	P_{cr} (kN)	P_u (kN)	Δ_u (mm)	ε_{FRP} (mm/m)	TEC-07 [33] (mm/m)	ACI-440 [34] (mm/m)	FIB B.N.14 [35] (mm/m)	CNR-DT 200/2004 [36] (mm/m)
Control	28.0	54.5	14.20	–	–	–	–	–
FBwoA-CFRP	35.0	62.0	9.87	1587	4000	2217	5232	941
FBwA-CFRP	40.0	82.6	15.50	3375	4000	4000	6000	3412
PBwA-CFRP	35.5	66.6	15.00	4138	4000	4000	6000	3412
FBwoA-GFRP	35.0	61.3	10.10	1873	4000	3032	6000	1632
FBwA-GFRP	42.0	77.5	14.80	4260	4000	4000	6000	9588
PBwA-GFRP	36.0	77.3	19.30	4969	4000	4000	6000	9588
FBwoA-Hi-CFRP	35.0	55.0	7.60	381	2000	1366	1365	505
FBwA-Hi-CFRP	35.0	61.8	12.40	1282	2000	3045	1365	829
PBwA -Hi-CFRP	35.0	70.3	16.32	3309	2000	3045	1365	829

(contd.)

		$V_{f\text{-}calc}$				$V_{f\text{-}exp}/V_{f\text{-}calc}$			
	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Specimen	$V_{f\text{-}exp}$ (kN)	TEC-07 [33] (kN)	ACI-440 [34] (kN)	FIB B.N.14 [35] (kN)	CNR-DT 200/2004 [36] (kN)	(5)/(10)	(5)/(11)	(5)/(12)	(5)/(13)
Control	–	–	–	–	–	–	–	–	–
FBwoA-CFRP	7.5	14.1	7.8	16.64	2.99	0.53	0.96	0.45	2.51
FBwA-CFRP	28.1	14.1	14.1	19.08	10.85	1.99	1.99	1.47	2.59
PBwA-CFRP	12.1	14.1	14.1	19.08	10.85	0.86	0.86	0.63	1.12
FBwoA-GFRP	6.8	5.2	3.9	6.88	1.91	1.31	1.74	0.99	3.56
FBwA-GFRP	23.0	5.2	5.2	6.88	11.21	4.42	4.42	3.34	2.05
PBwA-GFRP	22.8	5.2	5.2	6.88	11.21	4.38	4.38	3.31	2.03
FBwoA-Hi-CFRP	0.5	20.3	13.9	12.48	4.61	0.02	0.04	0.04	0.11
FBwA-Hi-CFRP	7.3	20.3	30.9	12.48	7.57	0.36	0.24	0.58	0.96
PBwA -Hi-CFRP	15.8	20.3	30.9	12.48	7.57	0.78	0.51	1.27	2.09

4 Experimental results and evaluations

The experimental results are presented and discussed below in terms of the observed mode of failure, load vs. mid-span deflection and load vs. FRP strains. The ultimate load-carrying capacity, the mid-span deflections and the FRP strains at failure are listed in Table 4. The failure modes of the beams are shown in Fig. 3, the load vs. mid-span deflection curves in Fig. 4.

4.1 Failure modes

The specimens were shear-deficient, even with the existence of FRP strengthening; hence, no flexural failure was observed in any test.

A shear crack, hereinafter called the main shear crack, was initially observed on either side of the loading point, and secondary cracks started to develop afterwards. The main shear crack was the widest crack observed on the specimen at the end of testing. The load was satisfactorily transferred from the region where the main shear crack occurred to the adjacent sections and maintained until the failure of the FRP strip passing through this region by either rupture or peeling off. The test was terminated when the shear crack was also observed in the flange of the T-section. This behaviour was successfully observed in all strengthened specimens.

During the test of the control specimen, initial shear cracks were concurrently observed on both shear spans, close to the centre of the half span, at a shear force of 28.0 kN. As the test load was increased, the main shear crack propagated further and more shear cracks formed within the test region as shown in Fig. 3. These cracks in the web reached the flange until failure occurred at a shear force of 54.5 kN. The shear force is defined practically as one-

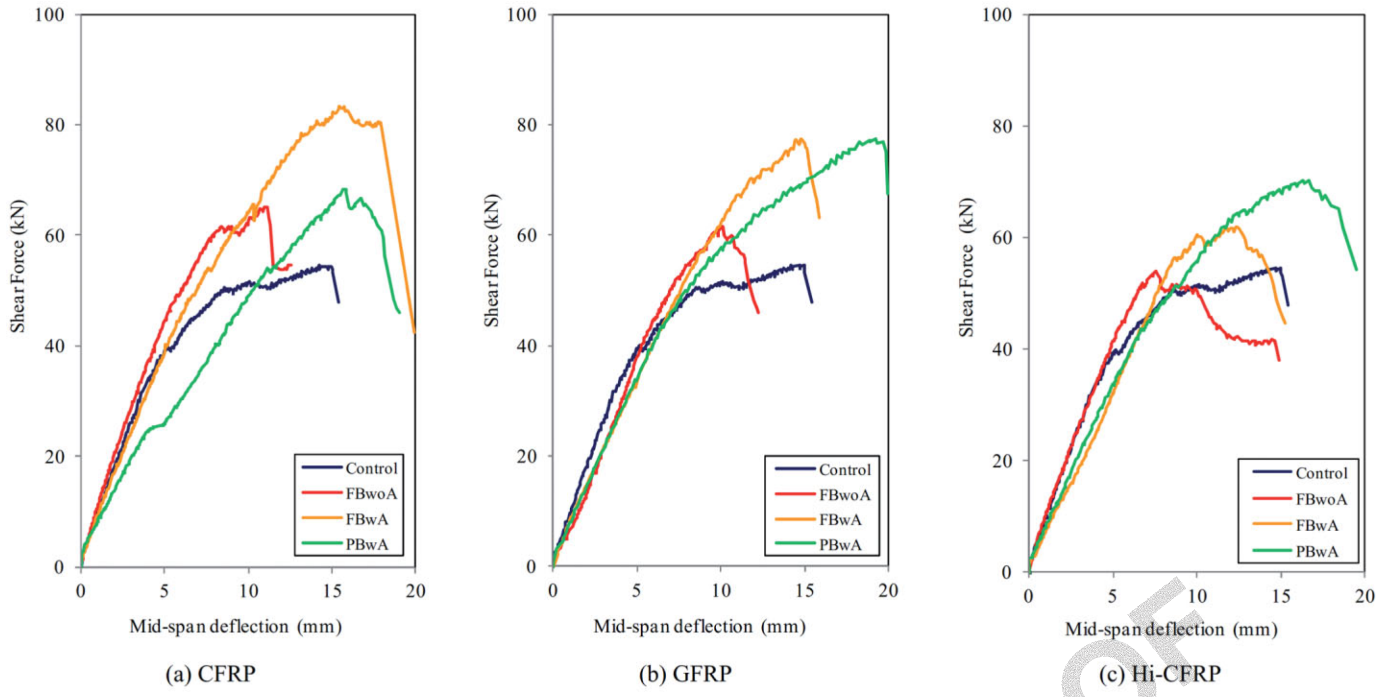


Fig. 4. Shear force vs. mid-span deflection curves for beam specimens

half of the vertical load. The failure of the beam specimen was abrupt and brittle.

The first shear crack in all strengthened specimens was observed between the FRP strips at a load level varying between 33 and 38 kN. The first cracking load of FRP-strengthened specimens was approx. 15–25 % higher than the corresponding value for the control specimen as shown in Table 4. The increase in the first cracking load level indicates that the application of FRP provides a positive contribution to the first cracking strength of beams, regardless of the type of strip end anchorage.

Two types of failure mode were observed in the strengthened specimens: the first was "debonding of the FRP" from the concrete surface, the second "FRP rupture". The FRP-strengthened beams without end anchorage (woA series), except the ones with Hi-CFRP (specimens FBwoA-Hi-CFRP and FBwA-Hi-CFRP), underwent a progressive shear failure, as described above, through a gradual FRP debonding that started from the free end of the U-strips. On the other hand, specimen FBwoA-Hi-CFRP failed through the rupture of FRP strip that passes over the main shear crack as shown in Fig. 3, contrary to its companion specimens (woA series) in the same set.

Both sides of the main shear crack covered by the FRP strip in specimens FBwA-Hi-CFRP and FBwoA-Hi-CFRP were closely investigated. It was observed that there was no visible debonding between the Hi-CFRP strips and the concrete surface on either side of the shear cracks in both specimens, even after total failure of the specimen itself. The undamaged bonding on both sides of the main shear cracks for the strips in specimens FBwA-Hi-CFRP and FBwoA-Hi-CFRP caused an abrupt and local increase in FRP shear and normal strains, leading to a sudden premature failure at a small deformation level. It should be recalled that the CFRP itself has an inferior response under shear-induced deformations. This might also be due to the remarkably low producer-specified rupture strain of the Hi-CFRP, supported by the high-quality bonding agent. It should be noted that the material characteristics of the epoxy used are better for Hi-CFRP specimens.

The failure type of all specimens with anchor and either fully or partially bonded strips was through "FRP rupture". No pull-out type of failure was observed in any of the anchors.

4.2 Improvement in shear failure load

The strengthened beams experienced significant shear capacity increases with respect to the control beam as reported in Table 4. It was observed that the shear capacity contribution of different FRP strengthening schemes depends on all the variables in the current investigation, i.e. type of bonding, anchorage detail and the fibre material characteristics. The load vs. mid-span displacement curves of the specimens are shown in Fig. 4, where the effect of the variables can be compared. Among all the specimens, specimen FBwoA-Hi-CFRP was the only one that experienced no increase in capacity, although the FRP used has the highest elastic modulus but the lowest ultimate strain capacity.

The capacity increase in specimens with full surface bonding and without anchorage was consistently below the increases for the other two types of strengthening scheme. The failure of such specimens was mostly due to the FRP peeling off near the shear failure load level. On the other hand, the shear capacity of specimens with FRP end anchorage increases remarkably for specimens with full surface bonding.

Comparing partially bonded specimens against fully bonded specimens with anchorage yielded better results in the Hi-CFRP set, where the strength enhancement dou-

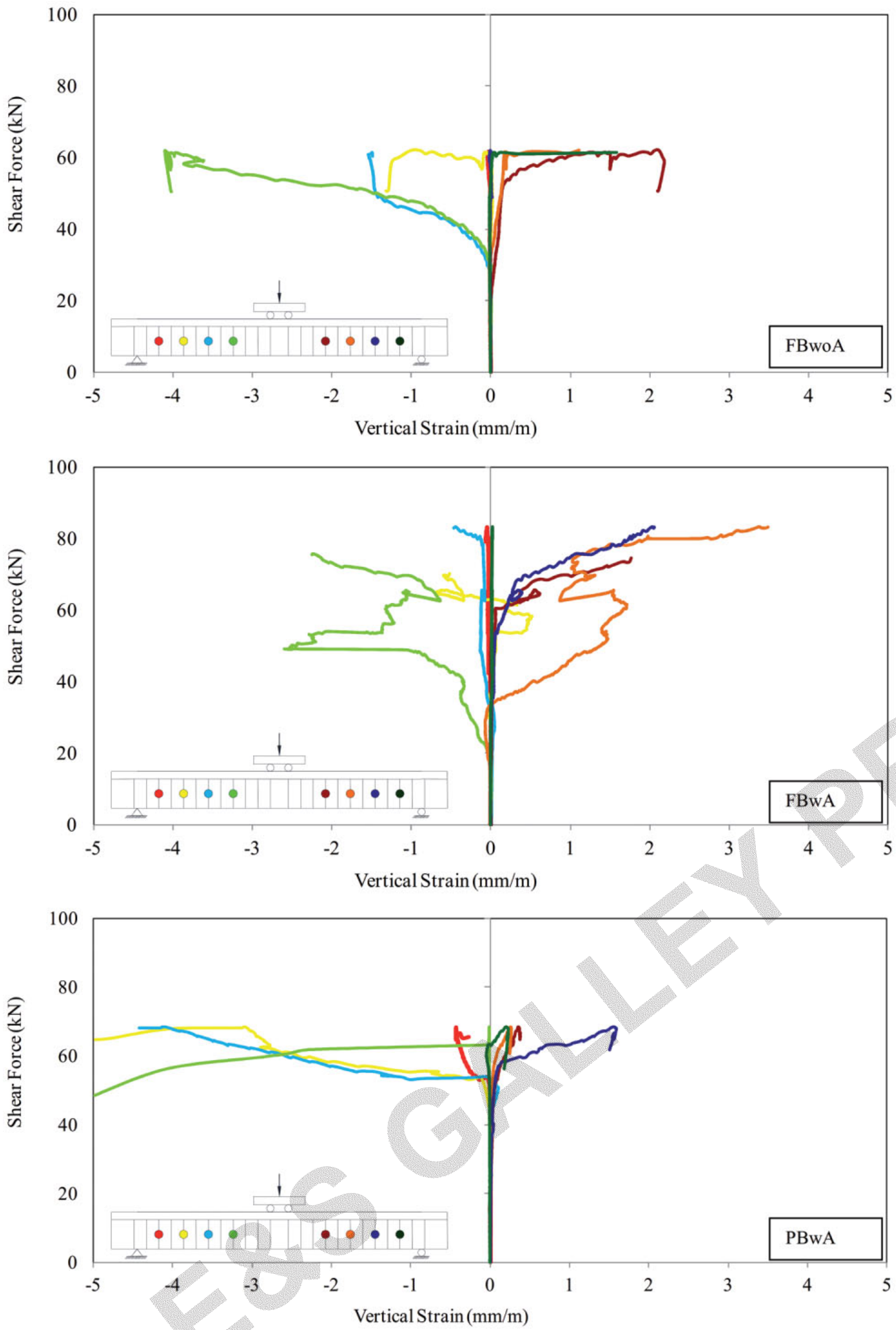


Fig. 5. Load-strain curves for beams strengthened with CFRP

bles. Partially bonded FRP may well be considered as a promising alternative to keep the FRP strains below the rupture level. Besides, partial bonding yielded comparable enhancements for both GFRP and CFRP specimens. The use of anchors, regardless of FRP material type, reduces the abruptness of the failure.

4.3 Load vs. strain behaviour of FRP strips

The measured strains of the FRP materials for each specimen at predetermined locations are given in Figs. 5, 6 and 7. The maximum FRP strain at failure of FBwoA specimens are consistently lower than the FRP strains measured for FBwA and PBwA specimens. The strain graphs reveal that the FRP strips of fully bonded specimens with end anchorage (FBwA) experienced large and sudden strain increases at the main shear crack locations, and the rupture occurs mainly in the FRP strip around the main shear crack. On the other hand, the strains in all strips developed gradually in PBwA specimens, and the strip around the main shear crack experienced the highest strain.

Figs. 5–7 reveal that the fibre strains attained in PBwA specimens are higher than those of the companion FBwA and FBwoA specimens. Even in the Hi-CFRP set, the measured strain in the FRP strips reached the ultimate strain capacity through partial bonding (PBwA). The measured fibre strains are presented in Table 4.

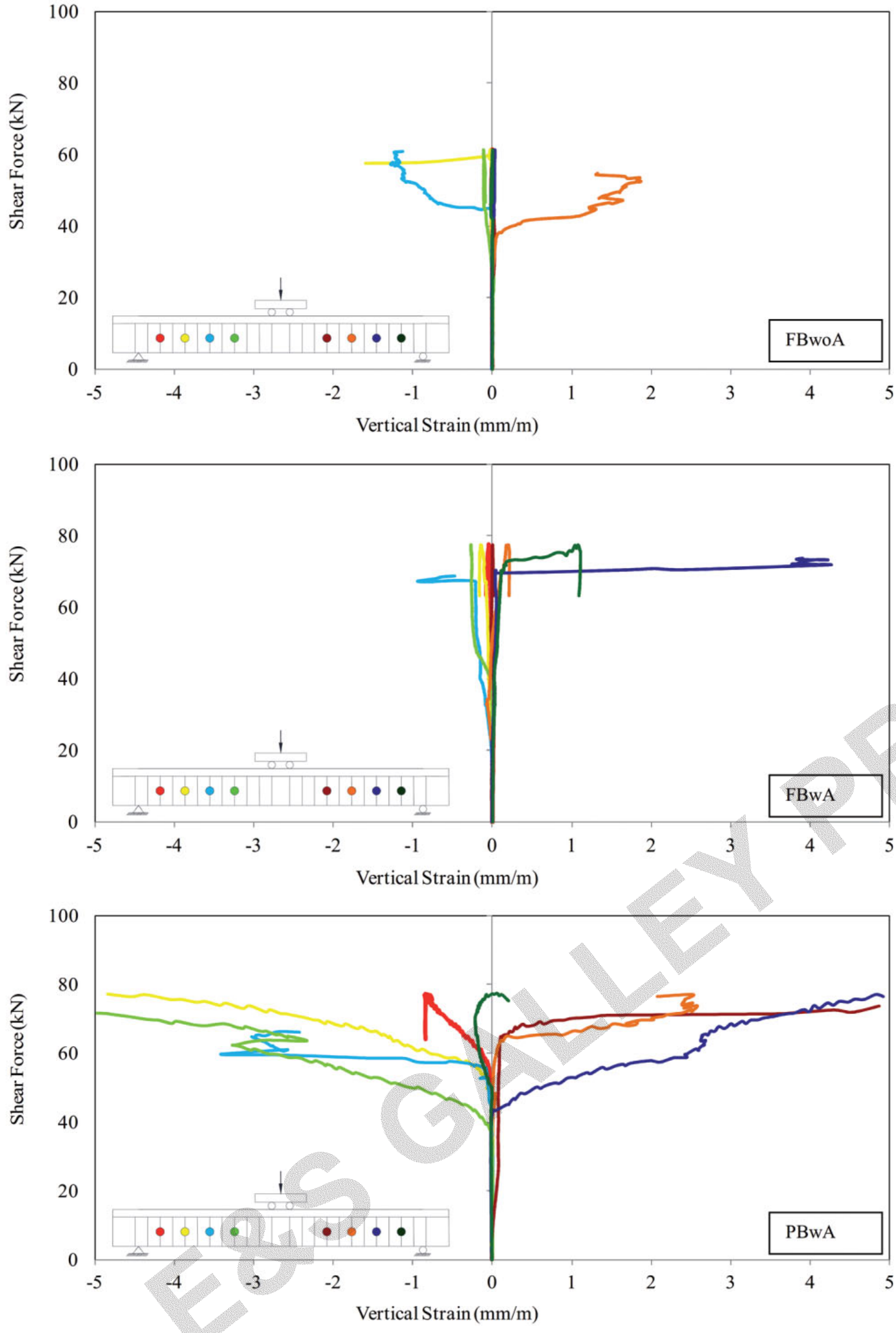


Fig. 6. Load-strain curves for beams strengthened with GFRP

4.4 Capacity predictions with available codes of practice

The shear strength contribution of the FRP strips is predicted by four code equations: Turkish Earthquake Code 2007 (TEC-07) [33], ACI-440 [34], FIB Bulletin No. 14 [35] and the Italian guideline CNR-DT 200/2004 [36]. The values measured experimentally and the code predictions are summarized in Table 4. The code equations are given elsewhere [33–36].

The shear capacity calculation approach of the three codes (TEC-07, ACI-440 and FIB B.N.14) is based mainly on predicting the effective FRP strain, possibly attained at the design failure load level. The effective FRP strain given in TEC-07 is either $\varepsilon = 0.004$ or 50 % of the producer-specified ultimate FRP strain, whichever is smaller. On the other hand, the upper bound of the effective strain values are $\varepsilon = 0.004$ and $\varepsilon = 0.006$ in the ACI-440 and FIB B.N.14 codes respectively. The effective strain equations in these latter two codes use concrete strength, FRP strength characteristics and final FRP thickness. The effective strain limitations in ACI-440 are based on the work done by *Priestley* et al. [37], *Triantafillou* et al. [21] and *Khalifa* et al. [38], whereas the work done by *Triantafillou* and *Antonopoulos* [39], besides the aforementioned researchers, is also considered for the ones used in FIB B.N.14. The CNR-DT 200/2004 code is based on the work

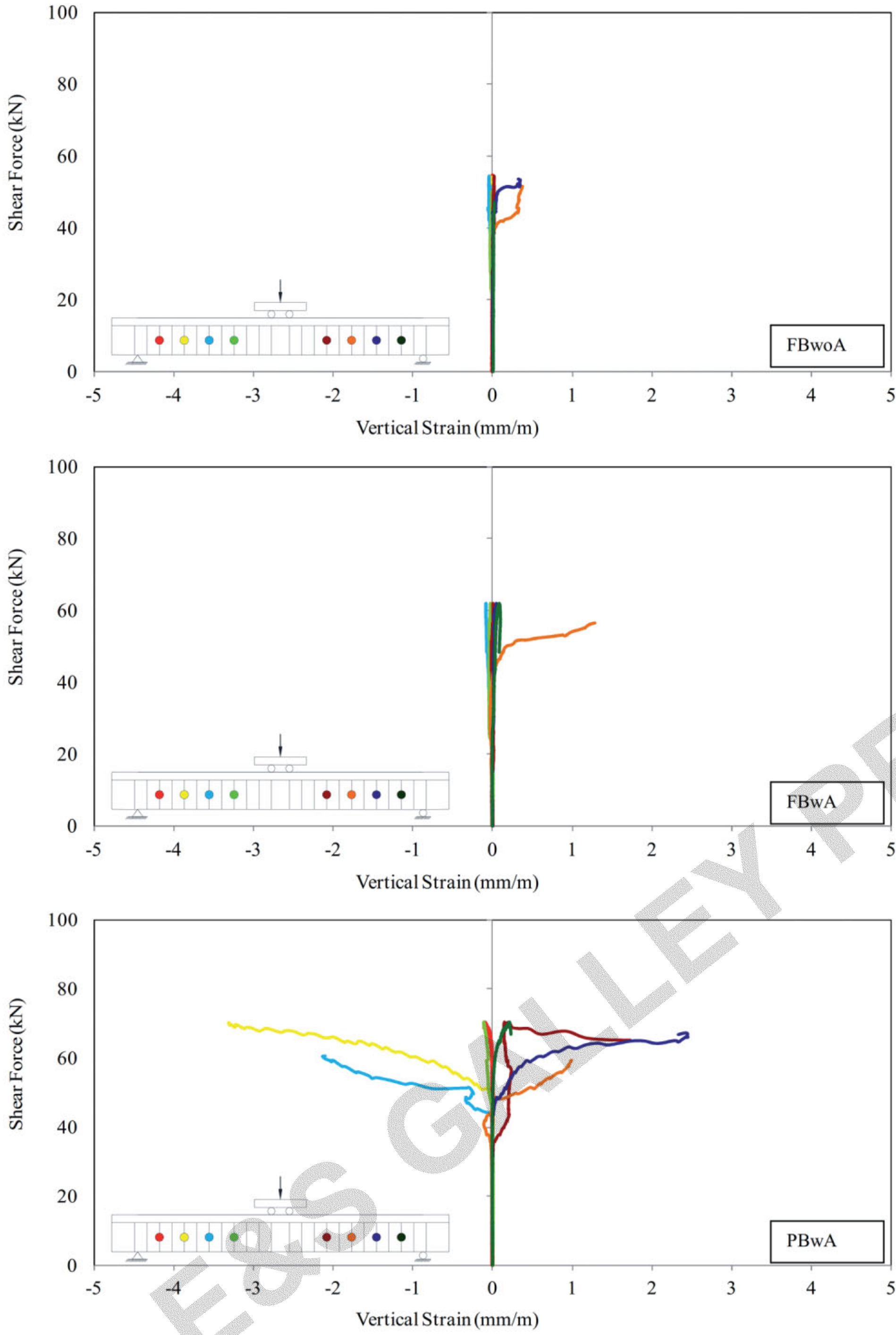


Fig. 7. Load-strain curves for beams strengthened with Hi-CFRP

of *Monti* et al. [40], where the aim was to determine the effective strength of FRP. The model uses the bond fracture energy to predict the effective FRP design strength.

The calculated FRP strain values according to TEC-07, ACI-440 and FIB B.N.14 are given directly in Table 4, whereas the effective FRP strength calculated according to CNR-DT 200/2004 is divided by the corresponding dry modulus of elasticity of that FRP and in Table 4 the resulting strains are given as the effective FRP strain. The material reduction factors in TEC-07 and ACI-440 are set to unity, similarly the partial factors for materials in FIB B.N.14 and CNR-DT 200/2004, as well as the partial factors for resistance. The only exception was the reduction factor $k = 0.8$ in FIB B.N.14, which is applied to the FRP effective strain.

A comparison of the experimental results and the code predictions reveals that the effective strains at the failure load level are best predicted by CNR-DT 200/2004 for specimens without anchorage. The upper bound FRP strain of TEC-07 ($\varepsilon = 0.002$) yielded results comparable with the experimental ones for the high-modulus CFRP, whereas the TEC-07 and ACI-440 limiting strains ($\varepsilon = 0.004$) are compatible for the normal-modulus CFRP and the GFRP when the FRP strips are anchored. On the other hand, the limiting effective strain given in FIB B.N. 14 ($\varepsilon = 0.006$) is consistently above the measured values. The

CNR-DT 200/2004 effective strain for the GFRP case with anchorage yielded very high results compared with the experimental ones. ACI-440, FIB B.N.14 and CNR-DT 200/2004 equations differentiate the anchorage type and reduce the effective strains for cases without anchorage. Neither of these codes foresees the effect of partial bonding.

When the strength contribution equations of the four approaches are evaluated, instead of the effective strain predictions discussed above, all codes yield very conservative results for the anchored GFRP cases, even with material factors equal to unity. If the Hi-Mod CFRP set is evaluated, all the code predictions yield unsafe predictions, especially for the case without anchorage. Only the unbonded specimen's strength enhancement was properly predicted by FIB B.N. 14 and CNR-DT 200/2004. With normal-modulus CFRP, ACI-440 yielded better results for capacity predictions.

5 Conclusions

A series of tests was carried out to investigate the shear behaviour of T-beams strengthened with FRP strips. The following conclusions can be drawn with respect to the current investigation:

- FRP strips, regardless of type of strip end anchorage, enhance the ultimate shear capacity and the first cracking load level of the reinforced concrete T-beams, whereas the sets with anchorage yielded higher capacities.
- It is believed that the limiting strain specified in the design codes beyond $\varepsilon = 0.004$ should be revised, especially for Hi-Mod CFRP and even for unbonded applications.
- The design codes should be revised for Hi-Mod CFRP, since the predictions of the existing codes constantly overestimate the capacity enhancement.
- The performance of partially bonded specimens is promising for all FRP types. The case of unbonded applications should be included in the code equations. For sound equations, further tests are needed. The authors believe that the increase in depth of a beam having an identical shear span-to-depth ratio results in smaller shear or axial strains under comparable shear crack widths for the partially bonded case [41]; therefore, further tests should consider the cross-sectional aspect ratio as one of the test variables.
- The capacity prediction for cases without anchorage in CFRP and Hi-Mod CFRP are consistently overpredicted by the design codes. This may result in unsafe shear strengthening applications.

Notation

a	shear span (mm)
d	effective depth of cross-section (mm)
L	length of beam (mm)
P_{cr}	first cracking shear force (kN)
P_u	ultimate shear force (kN)
s_f	spacing of FRP strips (mm)
t_f	thickness of FRP strips (mm)
$V_{f\text{-}exp}$	shear strength contribution of FRP with respect to control specimen, experimental (kN)
$V_{f\text{-}calc}$	shear strength contribution of FRP according to the code, calculated (kN)
w	width of dowel anchor (mm)
w_f	width of FRP strips (mm)
ε_{FRP}	strain on FRP at ultimate shear force (mm/m)
f'_c	28-day compressive strength of standard cylinder (MPa)
f_y	yield stress of reinforcing steel (MPa)
f_u	ultimate strength of reinforcing steel or FRP (MPa)
Δ_u	deflection at ultimate shear force (mm)

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