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Shear Strengthening of Deep T-Section RC Beams with CFRP Bars

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Abstract: Deep T-section beams have been widely used in engineering structures due to their high bearing capacity, high construction efficiency and economic benefits, while the current beam design theory can hardly interpret reasonably the mechanical behaviors of deep beams. The performance features of the deep T-beam were investigated, involving in strain distribution and principal stress trace using experimental tests. Different near surface mounted (NSM) reinforcement schemes were proposed for deep T-beams aiming at improving the shear capacity. The results show that the behaviors of deep T-beams dissatisfy the assumption of plane cross-section, and the ‘strut-and-tie’ model is applicable in such structures. The reinforcement systems can significantly relieve the strain concentration, mid-span deflection and crack width in deep T-beams, consequently improving the shear capacity range from 45 to 65%. The scheme is preferential for the reinforcement of deep T-beams when the applied angles, positions and lengths of CFRP bars are optimized based on the ‘strut-and-tie’ model.

Keywords: deep T-beam; ‘strut-and-tie’ model; near surface mounted; shear capacity; strain distribution



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1. Introduction

A T-beam with a depth-span ratio less than two (i.e., $l_0/h \leq 2$ shown in Figure 1) is defined as a deep T-beam [1]. Deep T-beams have superiorities such as strong bearing capacity, high construction efficiency and economic benefits. This leads to their wide applications in conversion layer of high-rise structures, sidewalls of large shallow silos, high-speed rails, bridges and offshore structures [2].

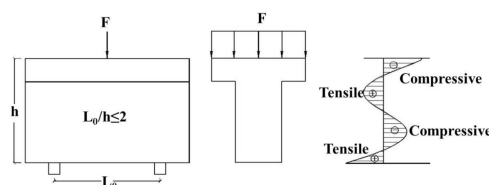


Figure 1. Strain distribution on mid-span section of deep T-cross section beam. Note: F: Load, h: the height of T beam, l_0 : clear span of T beam.

However, previous research [3–5] found that the mechanical behaviors of deep T-beam in real projects differ from the predicted results based on the current design code and the corresponding calculation model. The strain in cross-section distributed non-linear under an external force, which conflicts with the plane-section assumption [3,4]. The deep T-beams were dominated by shear deformation, which led to a high risk of shear failure, while such deformation is neglected in the ordinary shallow beam theory [5]. Moreover, when the ‘truss-arch’ model applied in the current beam theory is adopted for deep T-beam, the arch effect appears. This conflicts with the compatibility equation for strain. Moreover, it is difficult to quantify the contribution of this effect in ultimate bearing capacity of deep

beams [6]. This makes the stress distribution unclear when subjected to tensional and compressive loads. Thus, the practice based on the Code for Design of Concrete Structure can hardly reasonably describe the true stress distribution in deep T-beam structures [1]. It increases the difficulties in formulating the reinforcement strategy for deep T-beams. In view of these considerations, the study on reinforcement methodology is necessary for in-service deep T-beams with insufficient shear capacity.

There are many traditional methods of strengthening structures [7,8] including (1) increasing the section of the component, (2) replacing the original concrete with higher-grade concrete, (3) increasing reinforcement rebar, (4) grouting mortar, (5) sticking structural steel. However, these methods may bring about a higher self-weight and a smaller usable space. In addition, some methods become unavailable when construction conditions can not be satisfied. In consideration of these limitations, the Fiber Reinforced Polymer (FRP) reinforcement method is suggested by the engineering community [9,10]. Among many FRP materials, in spite of the project cost of Carbon Fiber Reinforced Polymer (CFRP) being slightly higher than other materials, CFRP is preferable due to the high strength-to-weight ratio and strong corrosion resistance.

The strengthening methods using CFRP reinforcement involve (1) the surface pasted reinforcement (SPR) method and (2) near surface mounted reinforcement (NSM) method. The SPR method is carried out by pasting CFRP laminates on the surface of the beam. This method can not be implemented in a targeted manner according to the stress flow distribution and structural deformation characteristics. Moreover, CFRP laminates are liable to peel off from the substrate. The NSM method involves creating a groove at a specific position on the beam surface based on a certain reinforcement path, and then uses a structural adhesive to embed CFRP bars into the groove. This method may enhance the structural shear capacity more efficiently by improving the interfacial adhesion and providing more interfacial areas between the CFRP bars and the substrates.

A series of studies have surveyed the shear capacity of deep beams. Tanaka, T. [11] conducted shear tests on CFRP-reinforced deep beams. They found that the effective bond length of CFRP bars influenced the failure mode and the bond between the CFRP bars and the concrete substrate. Maeda, T. [12] studied similar CFRP-reinforced deep beams and formulated the bond-slip law. Yao, J. [13] analyzed the SPR method and formulated the bond bearing capacity of CFRP-reinforced deep T-beams. Dias, S.J.E. [14] studied the shear strengthening of deep beam using the NSM method and proposed a theoretical calculation of shear reinforcement by virtue of simulation and genetic algorithm. Alwash, D. [15] researched the application of a double shear strengthening theory in the deep beams strengthened by using the NSM method.

The above research set forth the significance of shear strengthening methods and the calculation methodology of shear capacity for deep beams. However, the maximum bond length and stress flow model still fail to receive enough recognition for pre/post-reinforced deep T-beams. In this context, the shear strengthening methodology [16–24] will be stressed for deep T-beams in the following sections.

2. Materials Tests and Interface Mechanics Analysis

In this section, CFRP bars were manufactured and selected, followed by an investigation of the interface between the CFRP bars and the reinforced concrete (RC) T-beam substrate. The interfacial properties were studied experimentally, including the bond-slip law, interface strain distribution and load-transfer law in the reinforced system.

2.1. Tests and Selection of CFRP Bars

CFRP bars were made of CFRP filaments (see Figure 2a) and Epoxy Adhesive. Their physical and mechanical properties are listed in Table 1, according to the manufacturers' data.



Figure 2. Original materials and CFRP bars test: (a) Carbon fiber filaments; (b) Manufactured CFRP bars; (c) Setup in tensile tests.

Table 1. Properties of Epoxy Adhesive parameters and CFRP filament.

Epoxy Adhesive	Compressive Strength/MPa		Bonding Strength/MPa	Shear Modulus/GPa	
	32		10	3.8	
CFRP filament	Brand	Type	Density/kg · m ^{−3}	Tensile Strength/MPa	Elasticity Modulus/GPa
	RORAY	T-300	1500	3500	230

Six groups of CFRP bars containing a different monofilament number (i.e., 16, 24, 32, 40, 48, and 60 strands) were prepared with the same fiber volume fraction of 60%. Referring to [25], the tensile strength of CFRP bars were tested using a universal material testing machine (Type: DNS-100, Jilin Guan teng, China). In order to avoid the damage of CFRP bars resulting from clamping stress, both ends of the CFRP bars were wrapped in aluminum sheets before tests, as shown in Figure 2b. The test setup is shown in Figure 2c.

The test results are shown in Table 2. A big difference is presented among different groups due to the variation of CFRP filament strand number. The 32-strand monofilament bars have the highest tensile strength of 1572.79 MPa, with a 20% increase compared to the average value of 1308.12 MPa. Therefore, the 32-strand CFRP bar is preferable.

Table 2. Properties of CFRP bars.

Strand Number	Sectional Area/mm ²	Tensile Strength/MPa	Elasticity Modulus/GPa
16	10 × 1.8	885.73	119.71
24	10 × 2.1	1312.45	119.18
32	10 × 2.5	1572.79	120.05
40	10 × 2.8	1424.36	119.87
48	10 × 3.1	1345.26	119.78

The test results show that the CFRP bars' strength decreases with respect to the strand number. The possible reason is that the CFRP bars contain inclusions and voids that distribute disorderly. With the increase in CFRP filament layers, the epoxy adhesive around the carbon fiber filaments becomes more nonhomogeneous, which results in more defects and, thereby, the CFRP bars' strength decreases.

2.2. Determination of Bond Thickness in Reinforcement System

The 32-strand CFRP bars were adopted to improve the shearing capacity of deep T-beams using the NSM method. A series of pull-out tests were conducted in order to discover the interface properties between CFRP bar and concrete substrate. Considering that the adhesive layer thickness soundly influences the efficiency of the reinforcement system [26], different specimens were prepared with the adhesive thickness ranging from 0.1 to 2.0 mm. The details of specimens and experimental results are shown in Figure 3.

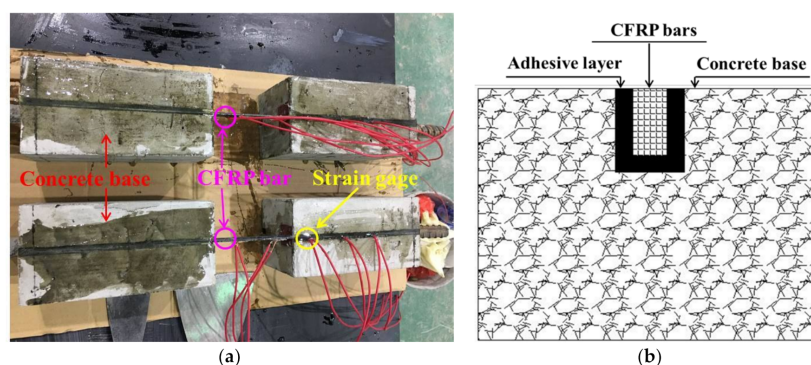


Figure 3. The Optimization coefficient of adhesive layer thickness: (a) Specimens; (b) Schematic diagram of cross-section by NSM.

The tests results (see Table 3) demonstrate that the 1.5-millimeter-thick adhesive layer provides an optimal effect. Therefore, 1.5-millimeter-thickness will be followed in all reinforcement systems.

Table 3. Properties of adhesive layer.

Adhesive Layer Thickness/mm	Tensile Strength/MPa	Adhesive Layer Thickness/mm	Tensile Strength/MPa
0.1	348.78	1.1	1088.18
0.2	418.53	1.2	1130.04
0.3	502.24	1.3	1213.74
0.4	585.94	1.4	1255.59
0.5	655.70	1.5	1325.35
0.6	697.55	1.6	1227.69
0.7	767.31	1.7	1143.99
0.8	837.06	1.8	1088.18
0.9	906.82	1.9	1032.38
1.0	1004.48	2.0	962.62

2.3. Determination of Bond-Slip Law at CFRP/Concrete Interface

As the tensile load increases, the CFRP bars often slip off from the concrete substrate; thereby resulting in an unattainable synergic effect between the CFRP bars and RC T-beam. With the viewpoint that the slip of CFRP bars is highly dependent on the interface properties, the bond-slip relationship in the reinforcement system will be focused on and experimentally determined based on pull-out tests. The layout of the strain gauges is shown in Figure 4. The resistance gauges typed by BX 120-5AA had a size of $3.5 \times 4.5 \text{ mm}^2$ and a sensitivity coefficient of $2.08 \pm 1\%$. They were adhered to the CFRP bar surface to measure longitudinal strains. The experimental details are shown in Figure 5. Three groups of specimens were used during the tests. The ultimate loads and CFRP bar strains were measured, as listed in Table 4.

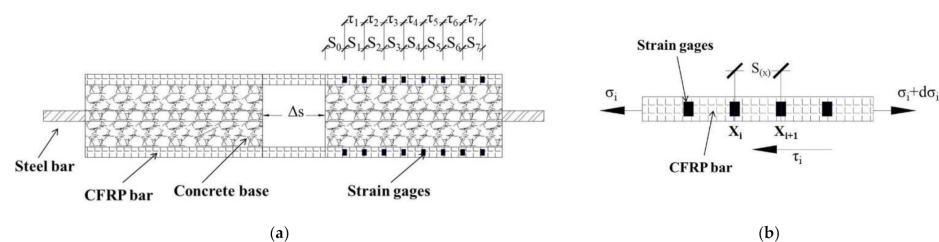


Figure 4. Distribution of strain gauges on bonding slip specimen: (a) Layout of strain gauges in pull-out tests; (b) Schematic diagram of free body.

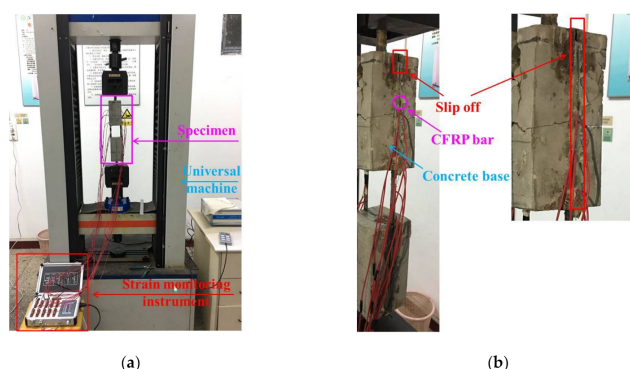


Figure 5. Test details in pull-out tests: (a) Setup and specimen; (b) Failed specimen.

Table 4. Results from pull-out tests.

No.	Embedded Length/mm	Embedded Depth/mm	Embedded Width/mm	Ultimate Load/kN	Relative Displacement/mm
SJ-1	160	11.5	5	35.54	4.52
SJ-2	160	11.5	5	35.18	4.31
SJ-3	160	11.5	5	34.89	4.17

A free body in the bonding zone was separated to analyze the stress states, as shown in Figure 4b. From the equilibrium conditions of the forces, we obtain

$$\tau_i = \frac{A_{cfrp} E_{cfrp}}{b_p + 2h_p} \frac{d\varepsilon_{pi}}{dx} \quad (1)$$

The bond stress between two adjacent strain gauges can be approximated as

$$\tau \left(\frac{s_i + s_{i+1}}{2} \right) = \frac{A_{cfrp} E_{cfrp}}{b_p + 2h_p} \frac{\varepsilon_{i+1} - \varepsilon_i}{s_{i+1} - s_i} \quad (2)$$

The relative slip between the concrete and CFRP bars is obtained by integral

$$s(x) = \int_x^{x_{i+1}} \varepsilon(x) dx \quad (3)$$

The bond stress versus relative slip curve can be experimentally determined based on Equations (2) and (3), and the result is plotted in Figure 6a. According to the previous conclusions from Refs. [27,28], the interface performance is influenced by the maximum bond stress and the corresponding slip as well as interfacial fracture energy, rather than the shape of bond stress–slip curves [27,28]. In this way, the bond stress vs. slip curve is simplified to be trilinear, as shown in Figure 6b. The bond–slip law at CFRP/concrete interface will be combined with the interface stress between the CFRP bars and concrete, which are used to calculate the effective CFRP bars length in Section 3.

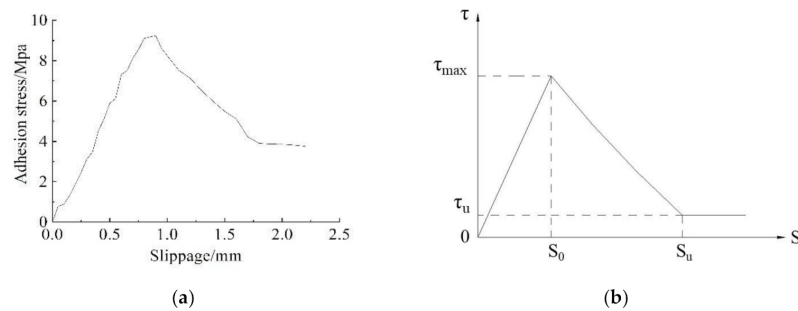


Figure 6. Relationship between bond stress and slip at interface: (a) Measured bond stress vs. slip curve; (b) Simplified bond stress vs. slip law.

2.4. Theoretical Analysis of Interface Stress Distribution

The stress concentration at the interface between the CFRP bar and concrete substrate often leads to premature failures of the reinforcement system. Therefore, the interfacial stress distribution is particularly worthy of concern. The theoretical models of the interfacial bonding stress and CFRP bar's tensile stress are derived on the basis of the elastic mechanics theory and the following assumptions:

The embedded CFRP bars are linear elastic, and their normal stress is distributed evenly in cross-sections.

No initial strain occurs in the reinforcement system.

The adhesive layer wrapping the CFRP bars is evenly distributed and only shear deformation is taken into consideration.

An orthogonal coordinate system is set up in Figure 7b. The force equilibrium condition of CFRP bar in the x -axis direction yields.

$$d\sigma_p(x) \cdot b_p \cdot 2h_p = \tau(x) \cdot 2(b_p + 2h_p) \cdot d(x) \quad (4)$$

$$dF_p(x) = d\sigma_p(x) \cdot b_p \cdot 2h_p \quad (5)$$

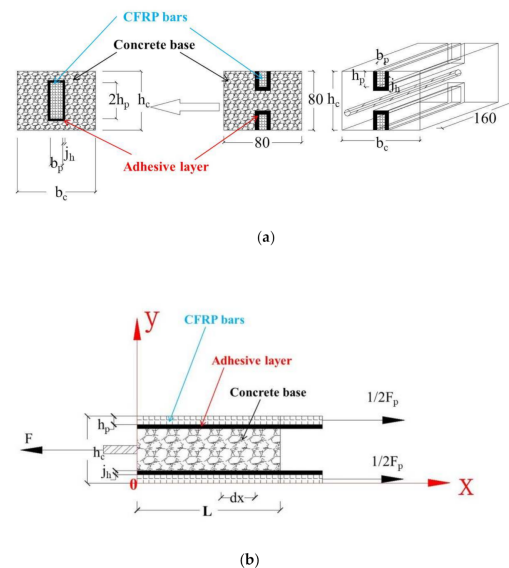


Figure 7. Interface bond model: (a) Mechanical equivalent model; (b) Boundary conditions model.

For the epoxy adhesive layers, the shear strain follows

$$\gamma_j = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \quad (6)$$

From the geometrical equation of bonding layer, we obtain

$$\tau(x)_j = \gamma_j \cdot G_j = \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) \cdot G_j \quad (7)$$

Combining Equation (4) to Equation (7) yields

$$\frac{d\sigma_p(x)}{dx} = \tau(x) \cdot \frac{(b_p + 2h_p)}{b_p h_p} = \frac{(b_p + 2h_p)}{b_p h_p} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) G_j \quad (8)$$

Finding the second derivative of Equation (8) with respect to x yields

$$\frac{d^2\sigma_p(x)}{dx^2} = \frac{(b_p + 2h_p)}{b_p h_p} G_j \left(\frac{\partial^2 u}{\partial x \partial y} + \frac{\partial^2 v}{\partial x^2} \right) \quad (9)$$

According to the assumptions, the displacement in the y -axis v equals 0 approximately. Equation (9) can be simplified as

$$\frac{d^2\sigma_p(x)}{dx^2} = \frac{(b_p + 2h_p)}{b_p h_p} G_j \frac{\partial^2 u}{\partial x \partial y} \quad (10)$$

$$\frac{du}{dy} = \frac{1}{j_h} (u_p - u_c) \quad (11)$$

Substitution of Equation (11) into Equation (10) yields

$$\frac{d^2\sigma_p(x)}{dx^2} = \frac{(b_p + 2h_p)}{b_p h_p} G_j \left(\frac{du_p}{dx} + \frac{du_c}{dy} \right) = \frac{(b_p + 2h_p)}{b_p h_p} G_j [\varepsilon_p(x) - \varepsilon_c(x)] \quad (12)$$

For the reinforced beam structure, we obtain

$$\sigma_c(x) b_c h_c + \sigma_p(x) b_p 2h_p = F \quad (13)$$

The substitution of Equations (13) and (14) into Equation (12) produces

$$\frac{d\sigma_p(x)}{dx^2} = \left(\frac{G_j}{b_p + 2h_p} + \frac{2G_j b_p}{E_c b_c h_c} \right) \sigma_f(x) - \frac{G_j F}{b_p h_p E_c t_c} \quad (14)$$

The general solution of Equation (14) can be expressed as

$$\sigma_f(x) = C_1 \cdot e^{(-\sqrt{A} \cdot x)} + C_2 \cdot e^{(\sqrt{A} \cdot x)} - \frac{B}{A} \quad (15)$$

where C_1 and C_2 are constants to be determined based on boundary conditions, and

$$A = \frac{G_j}{j_h^2 \cdot E_j} + \frac{2G_j \cdot b_f}{E_c \cdot b_c \cdot h_c \cdot j_h}, B = -\frac{G_j \cdot F}{j_h^2 \cdot E_c \cdot b_c \cdot h_c} \quad (16)$$

The boundary conditions are

$$\begin{cases} x = 0, & \sigma_f(0) = 0 \\ x = L, & \sigma_f(L) = \frac{\frac{1}{2}F_p}{b_p \cdot t_p} = \frac{F}{b_p \cdot 2h_p} = C \end{cases} \quad (17)$$

As a result, the coefficients C_1 and C_2 can be solved based on Equation (16) as

$$C_1 = \frac{-C \cdot A - B + B \cdot e^{(\sqrt{A} \cdot L)}}{A \cdot (e^{(\sqrt{A} \cdot L)} - e^{(-\sqrt{A} \cdot L)})}, C_2 = \frac{C \cdot A + B - B \cdot e^{(\sqrt{A} \cdot L)}}{A \cdot (e^{(\sqrt{A} \cdot L)} - e^{(-\sqrt{A} \cdot L)})} \quad (18)$$

The combination of Equations (4) and (16) yields

$$\tau(x) = \left(-C_1 \sqrt{A} \cdot e^{(-\sqrt{A} \cdot x)} + C_2 \sqrt{A} \cdot e^{(\sqrt{A} \cdot x)} \right) \cdot \frac{b_p + 2h_p}{b_p h_p} \quad (19)$$

Equations (4) and (17) feature the distribution of the interfacial bond stress and CFRP bars' tensile stress in the bond zone.

3. Mechanical Behaviors of Reinforced System

The mechanical properties of the steel bars and concrete in reinforced concrete (RC) beams were tested and are shown in Tables 5 and 6, respectively.

Table 5. Properties of steel bars.

No.	Diameter/mm	Type	Average Elastic Limit/kN	Average Yield Strength/MPa
1	8	HRB335	20.72	412.45
2	10	HRB335	34.26	436.37
3	18	HRB335	118.48	465.83

Table 6. Mechanical properties of concrete.

Cube Crushing Strength/MPa	Axial Compressive Strength/MPa	Tensile Strength/MPa	Elasticity Modulus/GPa	Poisson Ratio/MPa
40.5	38.6	3.6	25.2	0.2

In order to determine the difference between the deep T-beams and shallow T-beams in strain fields and principal stress trajectory, three-point bending experimental tests on the RC T-beams with a 1300-millimeter length and 450-millimeter height (with a shear span ratio $\lambda = 1.13$) were carried out. The local strain and full strain fields were measured by strain gages (the resistance strain gauges (BX 120-10AA) with the size of $10 \times 3 \text{ mm}^2$ and the sensitivity coefficient of $2.08 \pm 1\%$.) and digital image correlation (DIC) technology, respectively [29]. The reinforcement layout of the deep T-beam specimens in the test are shown in Table 7, Figures 8 and 9.

Table 7. The reinforcement layout of the deep T-beam specimens.

Order	Reinforcement Layout	Order	Reinforcement Layout
①	The upper longitudinal steel bar $8\phi 10$	②	The lower longitudinal steel bar $3\phi 18$
③	Horizontally distributed steel bar $\phi 8@150$	④	Vertically distributed steel bar $\phi 8@150$
⑤	Vertically distributed steel bar $\phi 8@150$	⑥	Tie steel bar $\phi 8@300$

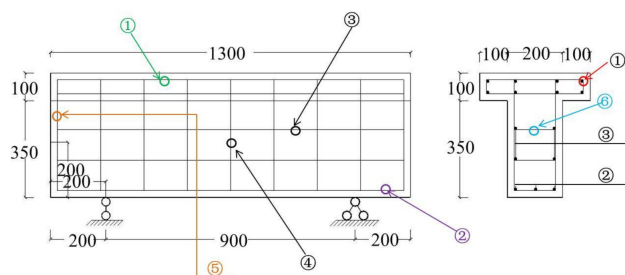


Figure 8. Layout of deep T-beam. Note: Size unit: mm.

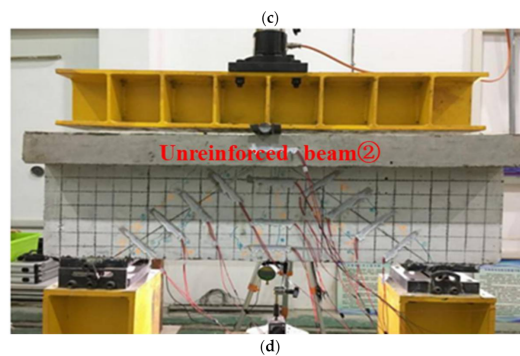
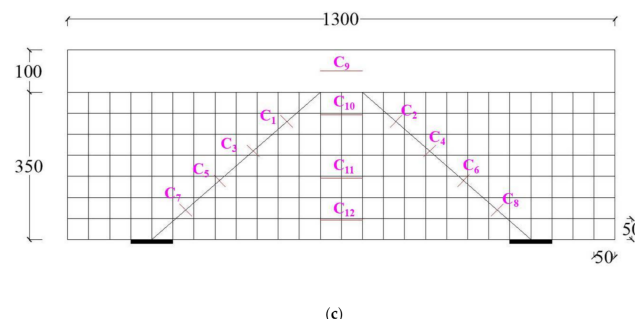
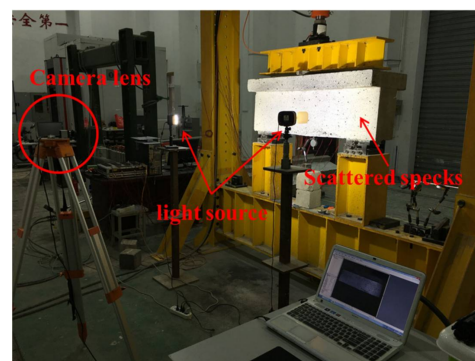
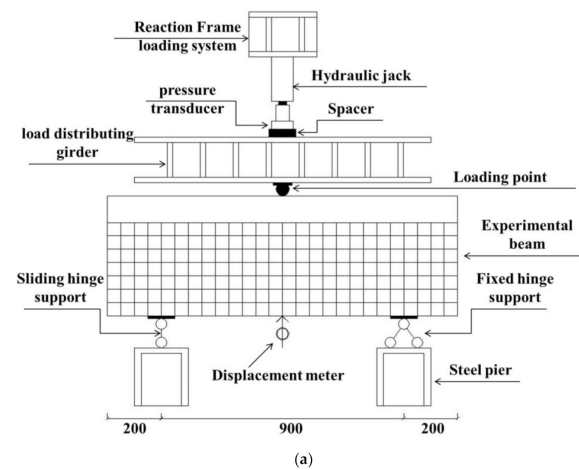


Figure 9. The setup of unreinforced deep T-beams under three-point bending loading: (a) Diagram of three-point bending test setup; (b) Diagram of three-point bending tests by DIC; (c) The location of strain gages in unreinforced beam. Note: strain gages C1-C12 are pasted on beam surface; (d) Unreinforced beam.

The distributions of normal strain and shear stress in the mid-span cross-section are plotted in Figure 10. The result reveals the phenomena that (1) the shear strain cannot be neglected and (2) the strain distributes nonlinearly in cross-sections. Consequently, the behaviors of deep T-beams do not abide by the assumption of the plane cross-section that is followed in shallow beams.

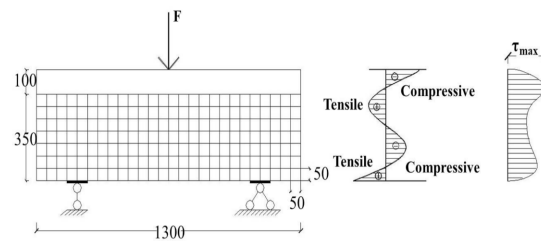


Figure 10. Normal strain and shear stress in cross-section.

The experimental full strain field is shown in Figure 11a. The principal stress trajectory of the deep T-beam is shown in Figure 11b.

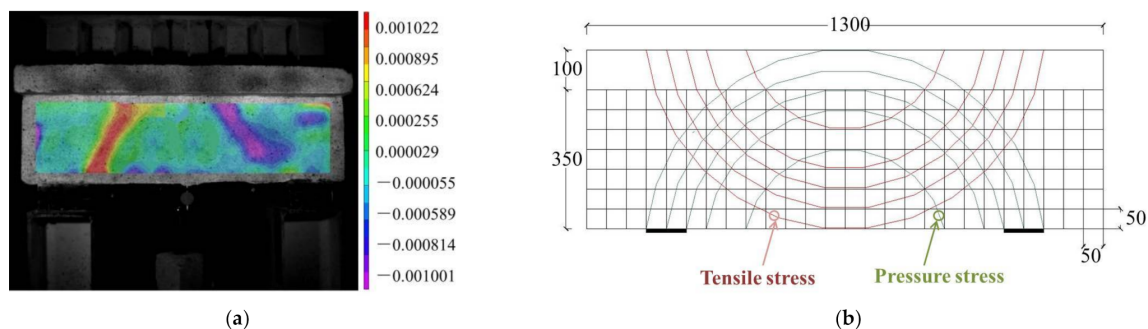


Figure 11. The full field strain distribution and principal stress trajectory of the beam by DIC of unreinforced deep T-section beam: (a) The full field strain distribution of the beam by DIC; (b) Principal stress trajectory.

According to Figure 11a, the high strain zone in the beam is also concentrated in the area between the loading points and supports. The compressive force flow is concentrated along the transmission path, which can, consequently, be considered as a strut. In contrast, the transverse tensile force flow is intense at the bottom of the beam, which may be treated as a tie accordingly. Thus, the ‘strut-and-tie’ model [30,31] can clearly represent the distribution of force flow. This interprets the occurrence of the arching effect in deep beams.

Based on the principal stress trajectory curves shown in Figure 11b, the reinforcement can be carried out reasonably to enhance the shear capacity of deep T-beams. The two reinforcement designs are herein proposed as follows:

Scheme I. The CFRP bars are applied to be tangent to the tensile stress trace, which provides a 45° angle to the beam belly horizontal line. The two ends of the CFRP bars align with the belly upper and the bottom, respectively (Figure 12a).

Scheme II. The CFRP bars are located with their centers fixed to the positions where the maximum tensile stress in the tensile stress trace appears. The embedded lengths are calculated by Equation (17). The angles of the bars are tangent to the tensile stress trace. The geometric parameters and the positions of the bars are listed in Table 8, and the reinforced scheme is shown in Figure 12b.

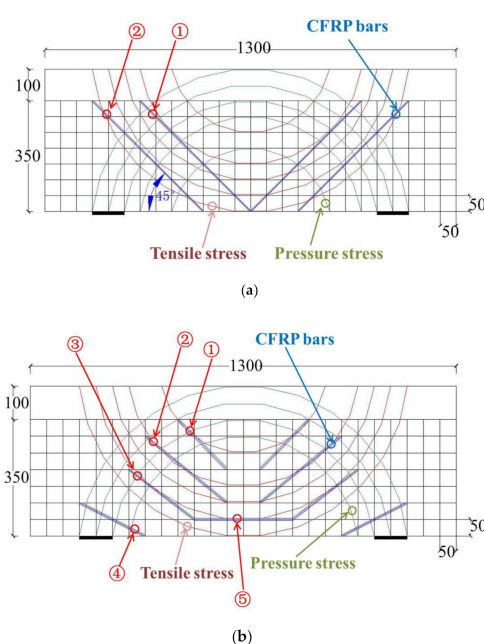


Figure 12. Reinforcement schemes: (a) Scheme I. Note: ①,② are CFRP bars; (b) Scheme II. Note: ①,②,③,④,⑤ are CFRP bars.

Table 8. CFRP bars on the two reinforced schemes (Scheme II).

Order	Central Position of CFRP Bars/mm	Maximum Tension Stress/MPa	Maximum Effective Length/mm	Angle/°
1	(525,275)	3.85	212	45.0
2	(475,200)	5.61	320	38.7
3	(400,125)	4.79	250	36.9
4	(250,12)	4.25	224	26.6
5	(653,55)	5.24	300	0

(Note: The coordinate position takes the axial direction of the beam to the right as the positive X axis, vertical direction upward as the positive Y axis, and coordinates origin as the lower left end of the beam.)

4. Experimental Validation

The reinforced deep T-beams based on Schemes I and II were tested under three-point bending condition to analyze the reinforcement efficiency of the schemes. (see Figure 13a,c). The surface strain of the beam and CFRP bars' normal stress were monitored by the resistance strain gauges (BX 120-10AA) with the size of $10 \times 3 \text{ mm}^2$ and the sensitivity coefficient of $2.08 \pm 1\%$. The strain gauges were located according to Figure 13b,d. The real-time surface strain field of the beam was observed via DIC technology.

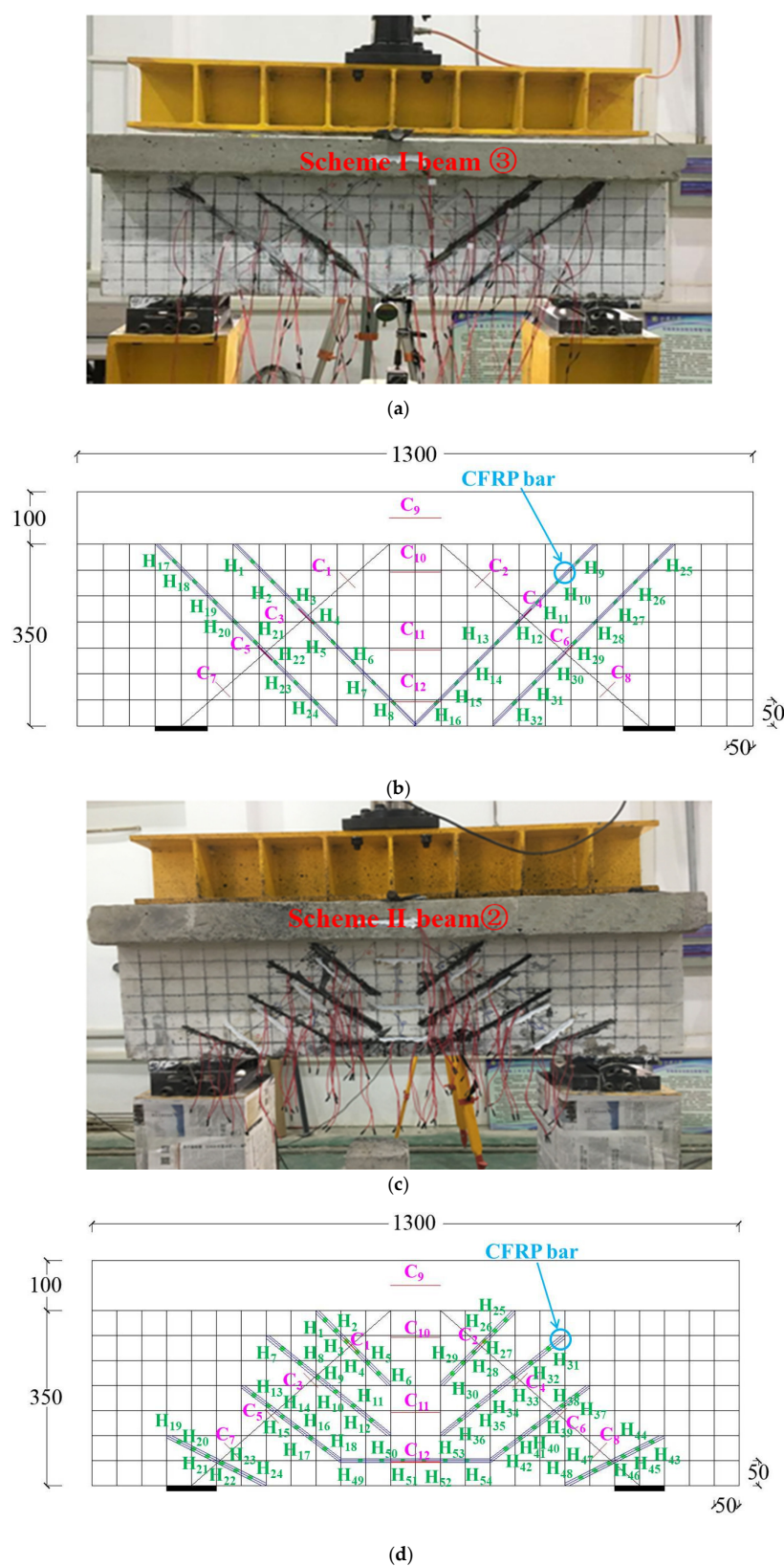


Figure 13. Deep T-beams under three-point bending loading: (a) Reinforced beam by Scheme I; (b) The location of strain gages in scheme I beam. Note: strain gages C1–C12 are pasted on beam surface, strain gages H1–H32 are pasted on CFRP bars surface; (c) Reinforced beam by Scheme II; (d) The location of strain gages in scheme II beam. Note: strain gages C1–C12 are pasted on beam surface, strain gages H1–H54 are pasted on CFRP bars surface.

The ultimate loading in the Unreinforced/Scheme I/Scheme II beams is listed in Table 9. The surface strain field, normal stress of the CFRP bars, mid-span deflection of the beams, crack widths and shear capacity of the beams under different loading conditions are plotted in Figures 14–18.

Table 9. Experimental average of three level loads on the three working conditions.

Working Conditions	Cracking Load/kN	Inclined Crack Load/kN	Shear Failure Load/kN
Unreinforced	80	200	270
Scheme I	160	260	390
Scheme II	190	300	445

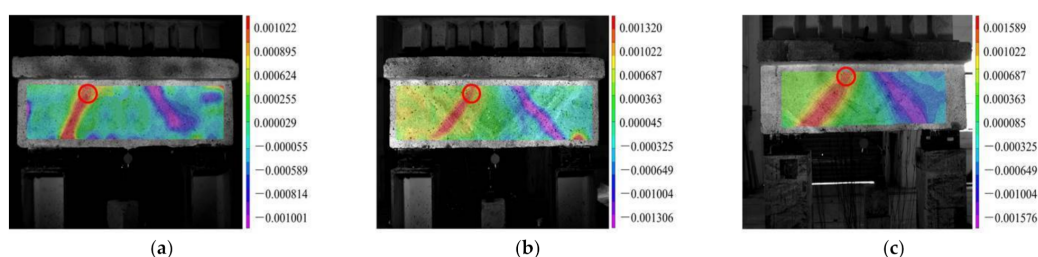


Figure 14. Failure strain distribution in different beams: (a) Failure strain distribution of unreinforced beam; (b) Failure strain distribution in Scheme I; (c) Failure strain distribution in Scheme II.

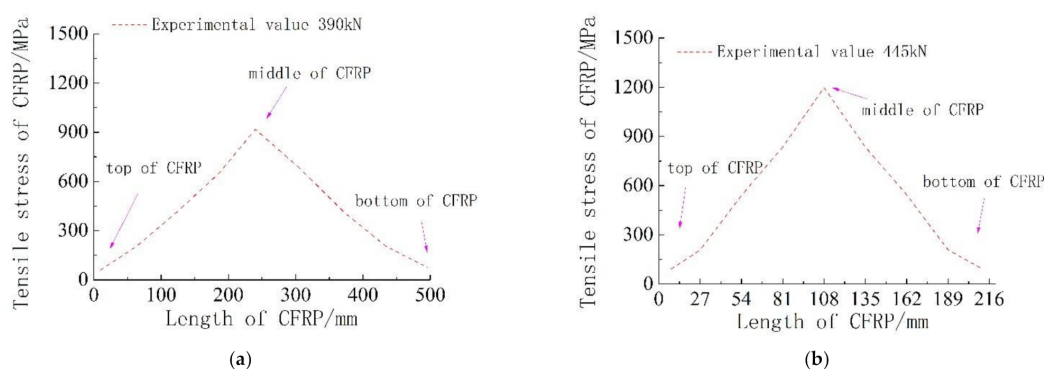


Figure 15. Tensile stress of middle CFRP bar in strengthened beams: (a) Middle CFRP bar's tensile stress in Scheme I; (b) Middle CFRP bar's tensile stress in Scheme II.

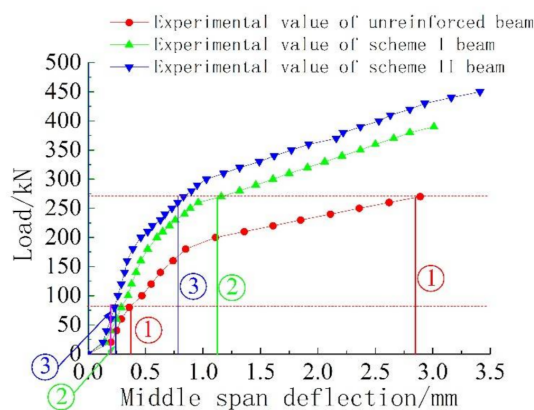


Figure 16. The experimental value of mid-span deflection under three working conditions. Note: ① ② ③ represent Unreinforced beam, Scheme I beam and Scheme II beam, respectively.

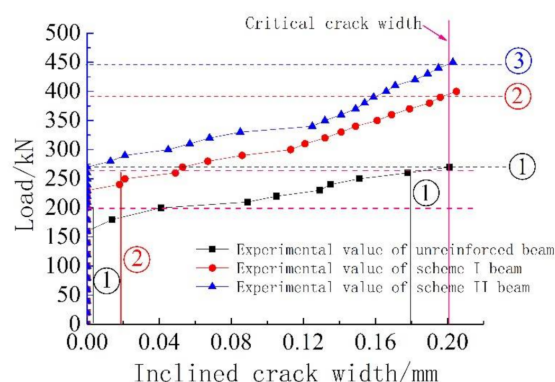


Figure 17. The experimental value of inclined crack width under three working conditions. Note: ① ② ③ represent Unreinforced beam, Scheme I beam and Scheme II beam, respectively.

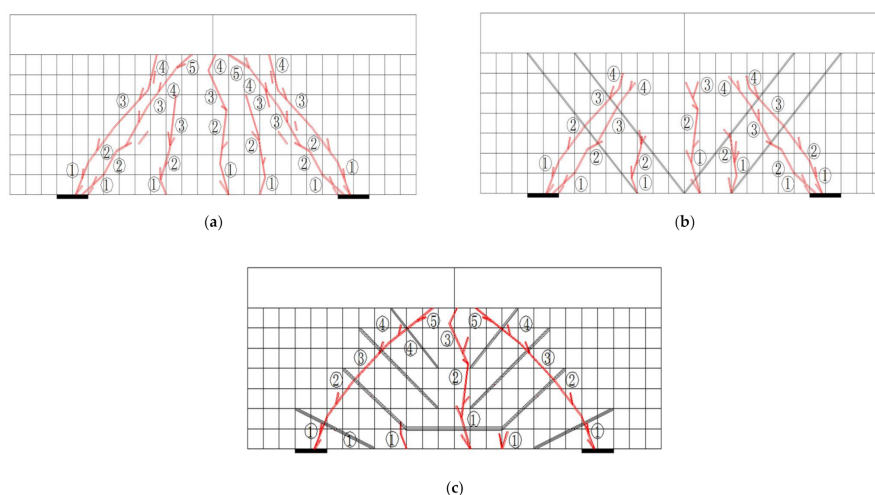


Figure 18. Crack distributions in three-point bending tests: (a) Unreinforced beam; (b) Scheme I; (c) Scheme II.

4.1. Shear Capacity

It can be observed from Table 9 that the vertical cracking loads of unreinforced, Scheme I and Scheme II are 80, 160 and 190 kN, respectively, i.e., the cracking load of the beam in Scheme I is increased by 100% and that of Scheme II is 140% higher when compared to the unreinforced beam.

As for the shear failure load, 270, 390 and 445 kN are presented during the tests for the unreinforced beam, Scheme I and Scheme, respectively. These reinforcement measures significantly improve the failure loads, with a 45% increase for Scheme I and 65% for Scheme II. Meanwhile, the Scheme II has a 15% higher failure load than Scheme I. These comparisons provide the positive responses of the proposed reinforcement schemes on the shear capacity of deep T-beams. Moreover, it is obvious that Scheme II plays a more preferable role when compared to Scheme I.

4.2. Surface Strain Field

The comparisons between the reinforced beams and the unreinforced beam in Figure 14 show that the strain concentration in the beam webs can be relieved by the reinforcement systems. For example, the unreinforced beam was damaged when the load was up to 270 kN, accompanied with a maximum strain of 1011 $\mu\epsilon$; while the maximum strain in Scheme I is 721 $\mu\epsilon$ and 630 $\mu\epsilon$ in Scheme II, with 41 and 59% reductions, respectively. These failure strain differences in the three deep T-beams indicate that the beam strain is restrained and optimized by mounting the CFRP bars. Moreover, the optimization makes

the strain in deep T-beams concentrated in the stress disturbance zones, which is consistent with the conclusions in Section 3.

4.3. Tensile Stress of CFRP Bars

As can be seen in Figure 15, the beam in Scheme I has a failure load of 390 kN and the maximum tensile stress presented in the CFRP bar is 978 MPa. Compared to the strength of the CFRP bar, only 42% of the material strength is utilized. In contrast, the failure load is 445 kN in Scheme II, which is much higher than in Scheme I. The maximum tensile stress of CFRP bar is 1189 MPa. The peak normal stress value of the CFRP bars in Scheme II is 22% higher than the counterpart in Scheme I, which indicates a better utilization of CFRP material in Scheme II than in Scheme I.

The normal stress of the CFRP bars distributes unevenly along the length direction, keeping the stress peaks at the central part of the bars and gradually decreasing toward both ends, regardless of the reinforcement schemes. This implies that the external load exerting on the beam is physically transferred to the central part of the CFRP bars, while the other part of the bars exerts a smaller action in bearing capacity.

4.4. Mid-Span Deflection of Beam

Figure 16 shows the differences of mid-span deflection in different beams. When the load is up to 80 kN, the unreinforced beam cracks firstly with the mid-span deflection of 0.36 mm. Under a similar load level, the mid-span deflection is 0.29 mm in Scheme I and 0.23 mm in Scheme II. The deflections reduce to 24 and 56%, respectively.

The unreinforced beam fails under the load of 270 kN with a mid-span deflection of 2.89 mm. In contrast, both the reinforced beams remain intact at this level. Their mid-span deflections are only 40 and 29% of the counterpart of the unreinforced beam, namely, 1.16 and 0.83 mm, respectively.

These results imply that the CFRP bars have a positive action in improving the stiffness of the beams and, consequently, restrict the deflection growth.

4.5. Crack Width and Distribution

From Figure 17, it can be observed that the width of the first major inclined crack reaches 0.014 mm in the unreinforced beam when the load is up to 200 kN, while the beams in Scheme I and II do not crack at this level. When the load increases to 260 kN, a 0.017-millimeter inclined crack appears firstly in Scheme I; while the inclined crack in the unreinforced beam increases to 0.178 mm and the beam in Scheme II is still in perfect condition. This shows that the CFRP bars have a synergic effect with the RC beam and, therefore, the CFRP strength can be utilized fully. As a result, the beam rigidity is enhanced and the crack propagation in beams is delayed.

As the load increases, the cracks in the beams in Schemes I and II widen gently, followed by an accelerated growth when the load exceeds a threshold. This change implies that the synergic effect between the CFRP bars and the RC beams is weakened. Finally, the CFRP bars peel from the beam and the structure stiffness is degraded accordingly.

Figure 18 shows that there are, in total, seven major cracks on the beam surface of the unreinforced beam: four inclined cracks in the stress disturbed zones and three vertical cracks in the middle of the beam. Among them, one vertical crack and two inclined cracks separately penetrate thoroughly from the bottom to the flange of the beam.

In Scheme I, seven main cracks appear on the beam surface as well. However, no crack penetrates thoroughly the beam. The longest crack extends from the bottom to three fourths of the beam height. This phenomenon indicates that the damage propagation was effectively restrained by the CFRP bars.

As for Scheme II, one vertical crack and two inclined cracks distribute in the mid-span and the stress disturbed zones, respectively. They pass through the beams from the bottom to the flange. The other cracks are still in an initial cracking stage. The above crack distribution patterns and quantities in three beams support the viewpoint that CFRP bars

can effectively restrict crack propagation. Moreover, Scheme II has a better reinforcement effect compared to Scheme I.

5. Discussion

The ‘strut-and-tie’ model features the stress distribution in deep T-beams, and further interprets the stress disturbance zones and the force transfer path of tensile and compressive stress in deep beams. In Scheme II, the CFRP bars were aligned in the tangent direction of the tensile stress trace. It is obvious that the bars in this direction have the strongest resistance against shear stress (see Figure 19). This is the reason why Schemes I and II can achieve a significant improvement in the shear capacity of the beams.

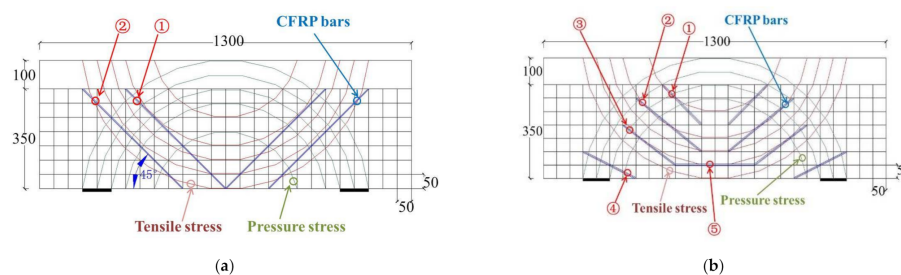


Figure 19. Scheme I and Scheme II: (a) Simple reinforced scheme I distribution of CFRP bars; (b) Optimized reinforced scheme II distribution of CFRP bars.

Furthermore, in Scheme II, the angle of the bars was adjusted according to the stress traces so that all the bars were aligned with the tangent directions. This led to a better effect on the improvement of shear capacity in Scheme II, rather than in Scheme I.

The two reinforced Schemes I and II are developed under the condition of no cracking. However, most structures work with cracks in practice. Therefore, how to arrange the positions of the CFRP bars based on the cracking trend and stress distribution in a more reasonable manner is worth further research.

6. Conclusions

In this study, an investigation based on experiments of the reinforcement strategy was carried out. The main characteristics of structural behavior were researched before and after reinforcement, including the bond-slip relationship at the CFRP/concrete interface, stress distribution in CFRP bars and RC deep T-beams, mid-span deflection, crack width and the ultimate shear capacity. The main conclusions are summarized as follows.

The behaviors of deep T-beams do not abide by the assumption of plane cross-section, which is followed in shallow beams. The strain disturbed zone and mechanism model in deep T-beams can be explained by the ‘strut-and-tie’ model.

The peak normal stress value of CFRP bars in Scheme II is 22% higher than the counterpart in Scheme I, which indicates that the former has a better efficiency and effectively utilizes the tensile strength of the CFRP bars.

The strain concentration, mid-span deflection and crack width in the deep T-beams can be relieved by the provided reinforcement schemes.

Scheme I and Scheme II improve the shear performance of the RC beam significantly, with a 45 and 65% increase separately compared to the reference beam. Scheme II has a 15% higher failure load than Scheme I, resulting from the optimization in Scheme II based on the ‘strut-and-tie’ model.

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Abbreviations

Symbol	Interpretation	Symbol	Interpretation
A_{cfpr}	cross-sectional area of CFRP bars	E_{cfpr}	elasticity modulus of CFRP bars
τ_i	shear stress of CFRP bars	ε_{pi}	Strain of CFRP bar
h_c	height of concrete test blocks	h_p	height of CFRP bars cross section
b_c	width of concrete test blocks	b_p	width of CFRP bars cross section
j_h	the thickness of epoxy adhesive	j	epoxy adhesive
c	concrete	p	CFRP
γ_j	shear strain of the bonding adhesive layer	G_j	the shear modulus of epoxy adhesive
u_p	displacement of CFRP bars	u_c	displacement of concrete
F_p	external force of CFRP bars	σ_p	Stress of CFRP bars
E_j	elasticity modulus of epoxy adhesive		

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