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Shear Behaviour of RC T-Beams with Externally Bonded Discrete CFF Strips – A Experimental and Finite Element Study

J. Jayaprakash[1,*], Abdul Aziz Abdul Samad [2], Mustafasanie M. Y[3], Anwar M. P[4], Mohamed A. A[5]

[1, 4 5] Department of Civil Engineering, The University of Nottingham, Malaysia
[2] Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn, Malaysia
[3] School of Civil Engineering, Universiti Sains Malaysia, Malaysia

*Corresponding E-mail: J.Jayaprakash@nottingham.edu.my (or) j_jaypee2000@yahoo.com

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Abstract

The application of fibre reinforced polymer (FRP) composites for retrofitting and strengthening of existing reinforced concrete (RC) structures has fascinated the attention of researchers and engineers in the recent decades. This paper presents the results of experimental and finite element (FE) investigation of shear behaviour of reinforced concrete T-beams repaired with externally bonded bi-directional discrete carbon fibre fabric (CFF) strips. The reinforced concrete T-beams were tested under four point bending system to investigate the performance of CFF shear strengthening scheme in terms of ultimate load carrying capacity. These beams were modelled using LUSAS software. To evaluate the behaviour of the simulated models, the predicted results were compared with the experimental results. The experimental results show that the gain in shear capacity of the CFF repaired beams ranged between 20% and 40% over the control beam. Thus, it can be concluded that the externally bonded CFF strips significantly increased the shear capacity of CFF repaired beams. It was generally observed that the developed FE model shows better agreement with the experimental results. The results of load-deflection profile, cracking pattern, modes of failure, and strain distribution in discrete CFF strips are presented.

Keywords: *CFF Strips, Concrete, Beam, Shear, Repair*

1.0 Introduction

Fibre Reinforced Polymer (FRP) has been effectively used as an external reinforcement to enhance the structural performance of deficient or damaged concrete structures. The demand to use the FRP fabrics or sheets is due to its better characteristics such as high strength in the direction of fibres, non-corrosive, high stiffness, and resistance to acids, and chemicals [1-2]. This plate bonding technique can be used to increase the load carrying capacity of the reinforced concrete members such as columns, beams, slabs, and masonry walls.

Shear failure of reinforced concrete beam is catastrophic and could occur abruptly without any advance warning. However, the addition of Fibre Reinforced Polymer composites is very complex because of the brittle nature of the material. A number of experimental investigations [3-8] have been studied the shear strengthening behaviour of reinforced concrete beams with the application of continuous carbon fibre sheets. It was found that the externally bonded carbon fibre reinforcement significantly increased the shear capacity and stiffness of the beam. Some researchers [2, 9-11] have investigated the effectiveness of the strengthened beams using discrete carbon fibre strips. The result showed that the discrete carbon shear strips increased the shear capacity of the beams.

Limited researchers have attempted to simulate the behaviour of FRP strengthened beams using finite element numerical analysis [12]. Arduini et al [13] simulated the behaviour and failure mechanism of strengthened RC beams using finite element analysis. The FRP plates were modelled using two dimensional plate elements, however the crack patterns were not predicted. Kachlakev et al [14] performed non-linear finite element analysis to model the uncracked reinforced concrete FRP strengthened beams using ANYSIS software. The smeared crack model was used to predict the non-linear behaviour of the concrete material. The stress-strain relationship of concrete was assumed to be linearly elastic up to the ultimate tensile strength. The steel rebars were assumed to be perfectly elastic-plastic material and identical in tension and compression. The fibre sheet was taken as orthotropic and transversely isotropic material. He also assumed that the bond between concrete and materials was perfect without any bond slip. From the analysis, it was found that the predicted results showed good agreement in comparison to the experimental results. Similarly, Santhakumar and Chandrasekaran [15] have developed a finite element model to study the shear behaviour of rectangular CFF strengthened beams using ANSYS software. The predicted load deflection profile was validated with the experimental results of Norris et al [16]. The assumptions were similar to the Kachlakev et al [14] finite element model. In the analysis, Newton-Raphson Equilibrium Iteration was employed at each increment of load. The numerical results agreed well with the experimental results.

Literature review reported that most of numerical analysis have dealt with shear behaviour of the strengthened rectangular beams with continuous wrapping system [14-15]. However, the reinforced concrete T-beam strengthened with discrete CFF strips has been limited. Moreover, some worthwhile parameters such as strain in fibres and cracking patterns have not been widely addressed.

This paper presents the results of experimental and finite element (FE) study of shear behaviour of reinforced concrete T-beams repaired with externally bonded bi-directional discrete Carbon Fibre Fabric (CFF) strips. The analysis used LUSAS Finite Element software. To evaluate the behaviour of the simulated models, the modelled results were compared with the experimental results. The numerical results of load-deflection profile, cracking pattern, modes of failure, and strain distribution in discrete CFF strips are presented.

2.0 Experimental Beams and Material Properties

This study used the experimental results of control (TS1a) and repaired CFF beams (TS1-1 and TS1-2) of the author [9]. These specimens were of 2980mm span with flange and web cross sections of 400mm x 100mm and 240mm x 120mm, respectively. Fig. 1 shows the cross section and internal longitudinal and transverse reinforcement details of the reinforced concrete T-beams. Beams TS1-1 and TS1-2 were precracked and repaired externally with discrete CFF strips spaced at 150 mm and 200 mm centre to centre, respectively. The width of CFF strip was 80mm with a orientation of 90 degree to the longitudinal axis of the beam. The external CFF reinforcement details are shown in Fig. 2. Tables 1 and 2 show the material properties of concrete & internal steel rebars and epoxy and discrete CFF strip reinforcement, respectively.

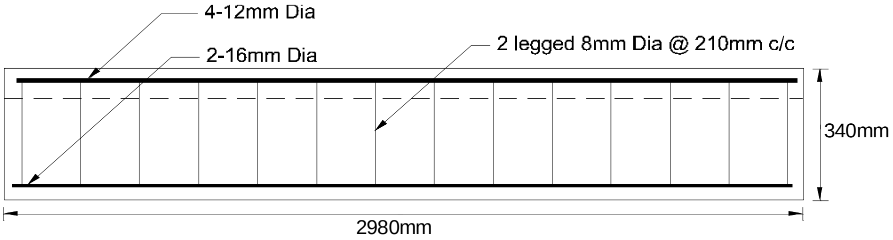
These beams were tested under four point bending system. The control beam was subjected to loading to develop precracks followed by unloading to zero and then reloaded for failure. However, the repaired beams were loaded for two cycles of loading to develop and widening of precracks prior to the application of external discrete CFF strip reinforcement (i.e. precracking phase). Subsequently, the repaired specimens were loaded to failure without any intervention.

Table 1. Details of material properties of steel and concrete

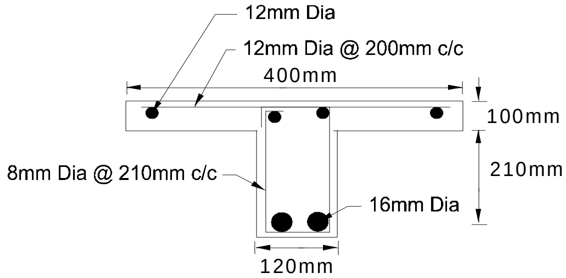
Specimen	Span (mm)	Compressive strength of concrete (N/mm^2)	Tensile strength of steel rebars (N/mm^2)		
			Stirrup	Tensile	Compression
			8mm	16mm	12mm
TS1a, TS1-1 and TS1-2	2980	16.70	620.31	311.22	565.42

Table 2. Material properties of CFF composites [17]

External Discrete CFF Strips	Thickness (mm)	Tensile strength (N/mm^2)	Modulus of Elasticity (N/mm^2)
U-strips (0/90) Bi-directional	0.09	3,800	230,000
Epoxy	-	30	3800

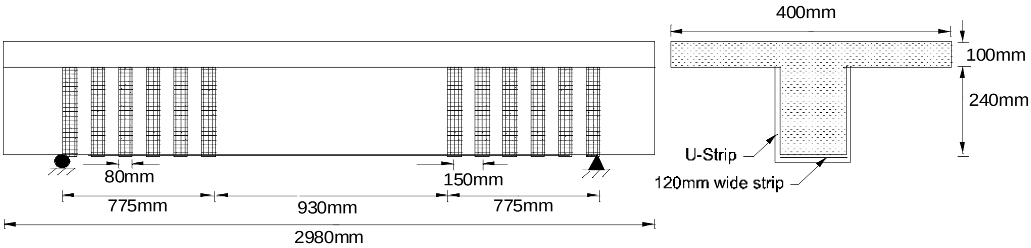


(a) Reinforcement details of T-beams

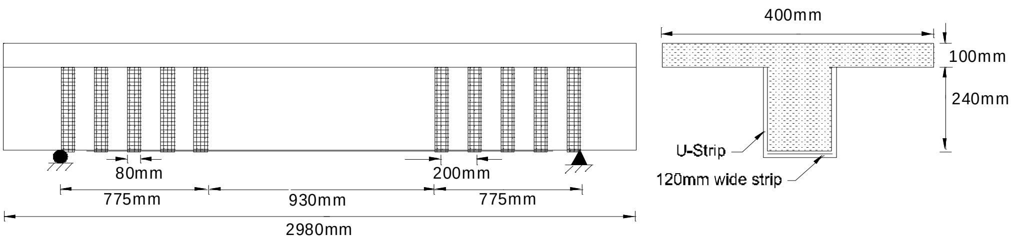


(b) Cross-section

Figure 1. Reinforcement and cross-section details of T- beam



(a) CFF discrete U-strip with spacing of 150 mm for repaired beam TS1-1



(b) CFF discrete U-strip with spacing of 200 mm for repaired beam TS1-2

Figure 2. External CFF discrete strip reinforcement details of repaired beams

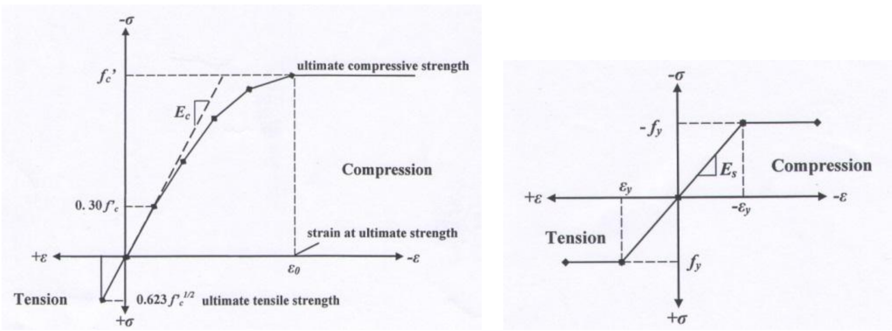
3.0 Finite Element Analysis

Concrete: Solid 82 elements were used to model the concrete. The elements are hexahedral in shape with quadratic interpolation. The stress strain curve of concrete in compression was linearly elastic up to 30 percent of maximum compressive strength. After the maximum compressive strength, the curve descends into a softening region and eventually crushing failure occurs at an ultimate strain [18]. This study used a perfectly plastic relationship instead of compressive strain-softening curve [14, 19]. Under uniaxial tension, the material was assumed to be linearly elastic up to the tensile strength. The stress-strain relationship of concrete is shown Fig. 3(a). The poison's ratio and ultimate strain values for concrete were taken as 0.2 and 0.035, respectively. The elastic modulus and tensile strength of the concrete were calculated using Eqns. 1 and 2 as recommended in AS3600: 2001 [20] (cited in Pham and Al-Mahaidi [21]).

$$E_c = 5050\sqrt{f_c^{'}} \tag{1}$$

$$f_{ct}^{'} = 0.4\sqrt{f_c^{'}} \tag{2}$$

The variables E_c, f'_c, and $f^{'}_{ct}$ represent modulus of elasticity, compressive strength, and tensile strength of concrete, respectively.



(a) (b)

Figure 3. Stress-strain curve for (a) concrete [14, 19] and (b) steel rebar [19]

Steel Reinforcement: The steel rebar was modelled as line element. Each division of line element was same as mesh element. This rebar was assumed to be elastic-perfectly plastic and identical in tension and compression [19, 22]. The stress-strain relationship of steel rebar is shown in Fig. 3(b). Furthermore, the bond between the rebar and concrete was assumed to be perfect (i.e.) no loss of bond slip [22]. The values of elastic modulus and poison's ratio of steel rebars were taken as 200,000MPa and 0.3, respectively.

Discrete CFF Strip Reinforcement: The CFF composite reinforcement used surface element. A linear elastic isotropic material was adopted for external discrete CFF strip reinforcement [23]. The bond between the discrete CFF strips and concrete was assumed to be perfect, whereas the bond interface was not considered in this model [14 15].

A half of the full beam was modelled by taking the advantage of symmetry of section and loading pattern with proper boundary condition [14]. It was modelled as uncracked beam but the precracking phase was ignored. A lagrangian isoparametric equation was employed to represent the geometric non-linearity of the model. Fig. 4 portrays the typical finite element model of the repaired beams with discrete CFF strips.

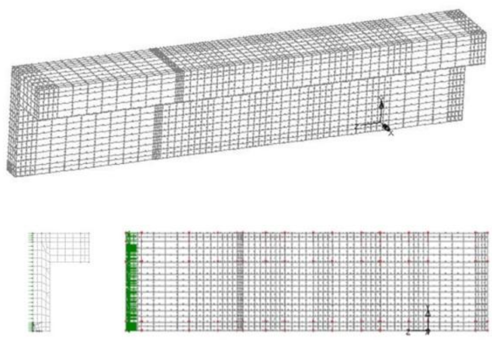


Figure 4. Elevation view of geometry model of repaired beam TS1-1

4.0 Results and Discussion

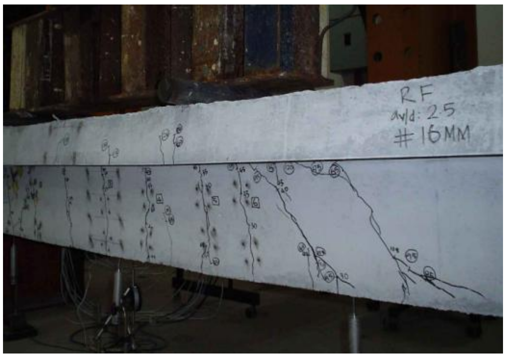
4.1 Experimental Observation

The control beam TS1a observed a diagonal shear crack at a load of 68kN. As the applied increased, these observed shear cracks propagated along the bottom of the flange towards the loading point. Moreover, some flexural cracks from the constant moment zone grew into the flange. The failure mode of the control specimen was controlled by shear at a peak load of 134.74 kN. Fig. 5(a) portrays the shear failure pattern of the control beam TS1a.

Beams TS1-1 and TS1-2 were precracked similar to the control specimen TS1a. The observed flexural and shear cracks were almost identical as in the beam TS1a. Subsequently, these beams TS1-1 and TS1-2 were repaired with CFF strips at a spacing of 150mm and 200mmc/c, respectively. The shear crack in the beam TS1-1 exhibited between the CFRP strip gaps (i.e. unwrapped zone). As the applied load increased, the CFF strips were ruptured approximately at a load of 148kN and followed by shear cracks emerging at the supports prior to the failure. However, in beam TS1-2, there was no sign of rupture in any CFF strip, however the flexural crack under the loading point grew up to bottom of flange. Both beams TS1-1 and TS1-2 were failed in flexure at a peak load of 187.95kN and 161.34kN, respectively. The flexural failure of these beams occurred under the loading points. The observed propagation and distribution of shear cracks in TS2-2 were relatively more as compared to beam TS1-1 due to the increase in spacing of CFF strips. These beams TS1-1 and TS1-2 attained a shear enhancement of approximately 40% and 20% over the control beam TS1a. Fig. 5(b) portrays the flexural failure pattern of the CFF repaired specimen TS1-1.

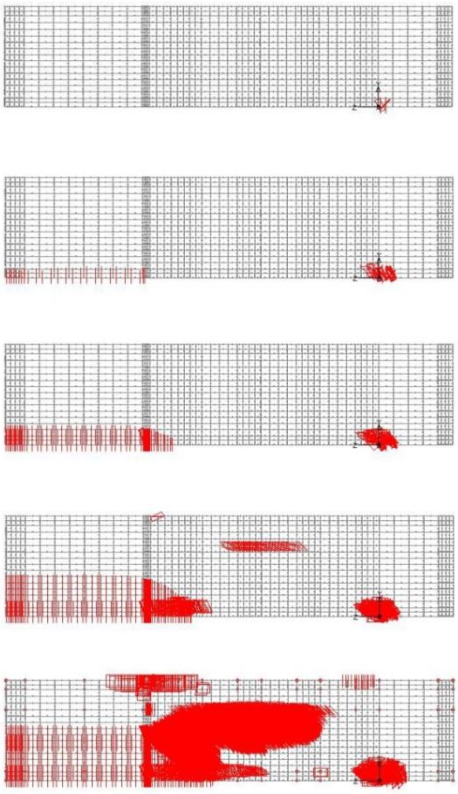
4.2 Cracking and Failure Patterns – Numerical Observation

The cracking patterns of control and CFF repaired beams have been generated using LUSAS finite element software for different loading cases. Fig. 5(c) portrays the cracking patterns of the control beam TS1a. At the early stage of loading, the local crushing of concrete was observed near the support of the beam as the support was not modelled in this study. The flexural cracks were then initiated in the flexural zone and propagated towards the flange zone. With further increase of load, the diagonal shear cracks began at the shear span and extended along the bottom of the flange. Some compressive cracks were also developed in the flange at the point of application load. The tested control beam was also accomplished similar observations except the local crushing at the support (see Fig. 5(a)).

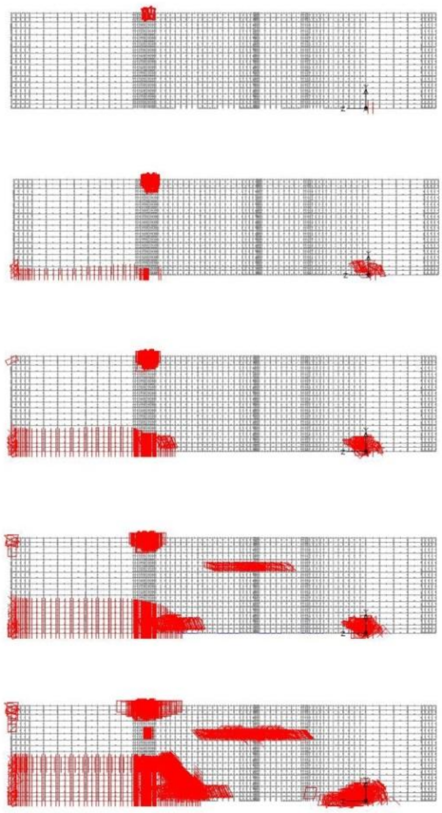


(a) Control beam TS1a

(b) CFF repaired beam -TS1-1



(c) Numerical study – Beam TS1a



(d) Numerical study – Beam -TS1-1

Figure 5. Numerical and experimental cracking patterns for (a) control (TS1a) and repaired (TS1-1) beams

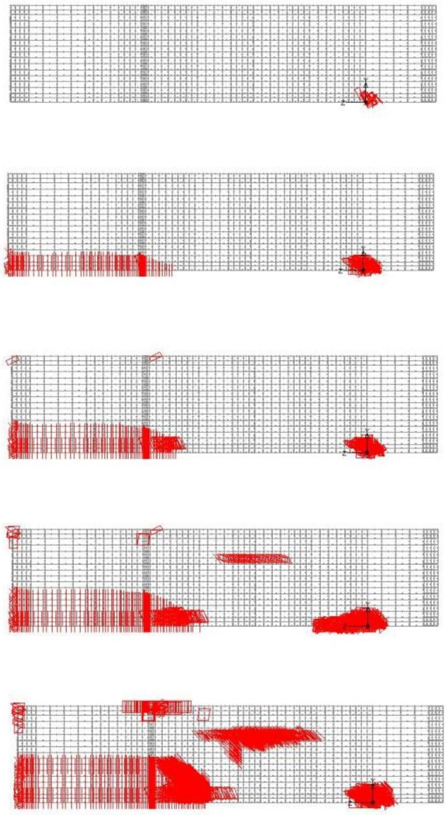


Figure 6. Cracking patterns for repaired beam TS1-2

The cracking patterns of repaired beams TS1-1 and TS1-2 at various loading stages are depicted in Figs 5(d) and 6, respectively. These CFF repaired beams had local crushing of concrete and flexural cracks similar to the control beam. The shear cracks were also emerged in the shear zone and propagated along the bottom of flange. Moreover, the flexural cracks were observed under the loading points. It can be seen from figures, the distribution of cracks in the CFF repaired beams were not same as control beam due to the application of external CFF shear reinforcement. All these observations (i.e. shear and flexural cracks) were also achieved in the experimentally repaired beams (e.g. see Figure 5(b)) prior to the failure load. From the overall discussion, it can be concluded that the cracking pattern of the modelled control and repaired beams was almost similar to the experimental beams except the local crushing failure at the support. It was also observed that the modelled control TS1a failed in shear, however the CFF repaired beams TS1-1 and TS1-2 were failed in flexural mode. The predicted failure modes of the numerical models were same as observed in the tested control and CFF repaired beams.

4.3 Load Deflection Profile

The typical deformation of the modelled beam is shown in Fig. 7. Figs. 8(a) - (c) show the comparison of experimental and predicted load-deflection profile for control and CFF repaired beams. In control beam TS1a, the stiffness of the predicted deflection curve was similar to the experimental results up to a load approximately of 110kN. However, in the later stage, there was an unexpected drop in deflection value of the predicted deflection curve. Comparing the experimental and predicted results of repaired beam TS1-1, the numerical results show better agreement with the experimental results, whereas the beam TS1-2 was relatively less agreement in comparison to the beam TS1-1. The deflection profile of beam TS1-2 shows similar trend until the load 100kN but the stiffness increased as the applied load increased. Table 3 shows the comparison of experimental and predicted FE results of control and CFF repaired T-beams. It was observed that the predicted results were varied between 25.92% and 31.76% over the experimental results. The discrepancy in predicted results is probably due to the several reasons as reported by Kachlakev et al [14]. At first, this effect is due to the presence of micro cracks in concrete which could be produced by drying shrinkage in the concrete and handling of the beams. The microcracks probably reduce the stiffness of the tested beams, whereas the finite element models do not include the microcracks. Secondly, this finite element analysis assumed that the bond between the concrete and steel is perfect but it would not be true in the actual beam. As bond slip occurs, there is no composite action between the concrete and steel reinforcement [14]. Furthermore, the experimental beams were precracked before applying the CFF shear reinforcement, however this FE analysis ignored the effect of precracking phase of the modelled beams. In due course, the assumption prefect bond between concrete surface and CFF in the model may cause the failure to simulate the premature debonding failure of the CFF. This could also lead to the overestimating the member stiffness and the ultimate load bearing capacity [24]. As a consequence, the overall stiffness and load carrying capacity of the numerical results was expected to be greater than the experimental results.

(a)

(b)

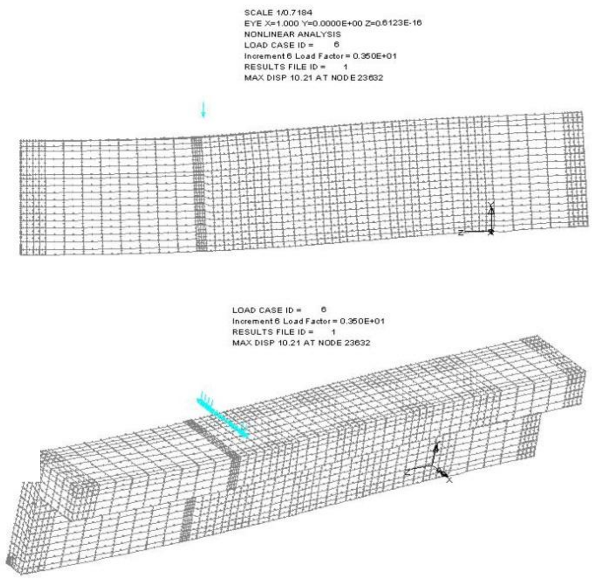
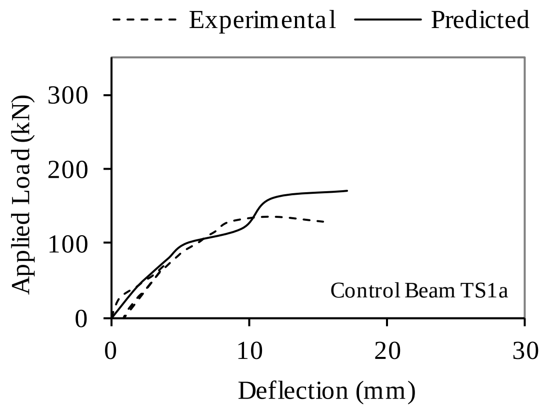


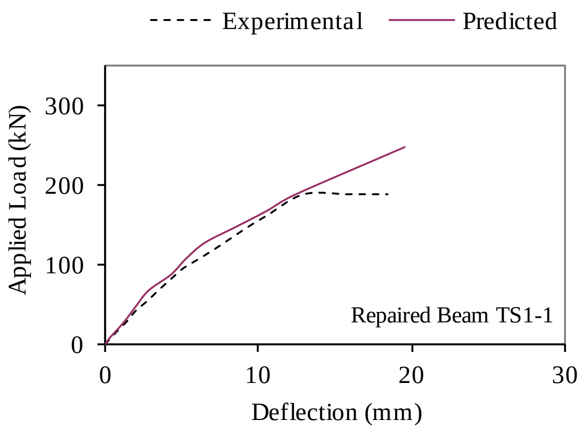
Figure 7. Typical deformation of modelled beam (a) Elevation view and (b) Isometric view

Table 3. Comparison of experimental and predicted FEM results for control and CFF repaired T-beams

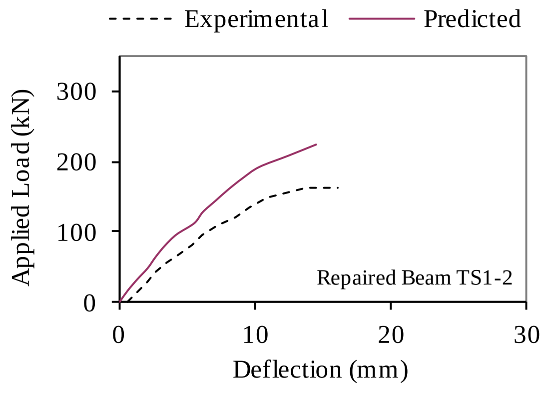
Beam	External CFF Reinforcement		Failure load		
	Width (mm)	Spacing (mm)	EXP (kN)	FEM (kN)	(%) Difference [(FEM-EXP)/EXP] x 100
TS1a (Control)	---		134.74	170.0	25.92
TS1-1 (Repaired)	80	150	187.98	247.7	31.76
TS1-2 (Repaired)	80	200	161.34	224.0	27.97



(a) Control beam TS1a



(b) CFF repaired beam TS1-1



(c) CFF repaired beam TS1-2

Figure 8. Load – deflection profile for control and CFF repaired T-beams

4.4 Strain Distribution in Discrete CFF Strips

The distribution of fibre strains in the CFF repaired T-beams predicted the location of failure zones such as debonding or fracture of discrete CFF strips where the stress is highly concentrated. Figs. 9(a) and (b) depict the development of strain distribution in discrete CFF strips of the modelled beams TS1-1 and TS1-2, respectively. From the figures, it can be seen that the discrete CFF strips placed proximity to the mid-shear span of the beams TS1-1 and TS1-2 were stressed. This was probably due to the propagation of shear crack in the concrete band (i.e. between CFF strips). It also confirms that the discrete CFF strips only began to carry the load after the formation of crack in the concrete. Moreover, the negative strain values were probably caused by the fracture of discrete CFF strips.

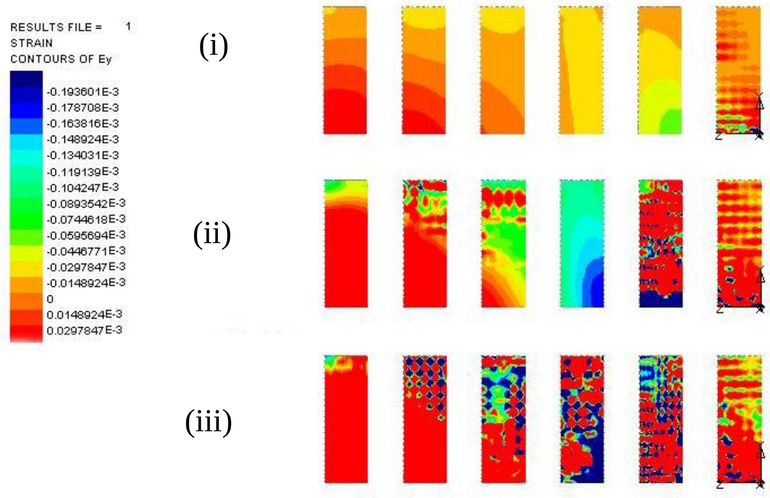


Figure 9(a). Development of strain distribution in discrete CFF strips for repaired beam TS1-1 at different stages of loading (i) 27.7 kN; (ii) 47.7 kN; (iii) 107.7 kN

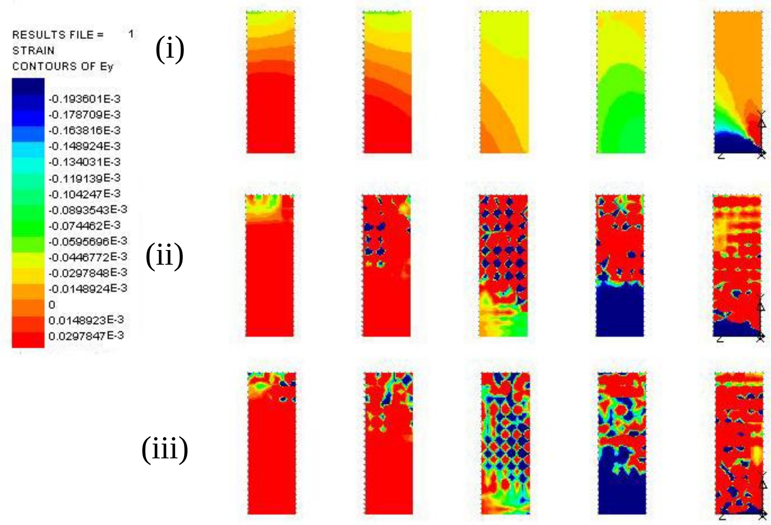


Figure 9(b). Development of strain distribution in discrete CFF strips for repaired beam TS1-2 at different stages of loading (i) 20.0 kN; (ii) 48.0 kN; (iii) 112.0 kN

5.0 Conclusions

This paper presents the experimental and finite element results of shear behaviour of reinforced concrete T-beams repaired with discrete CFF strip composites. The following conclusions can be drawn from this experimental and numerical investigation:

- The experimental results show that the gain in shear capacity of the CFF repaired beams ranged between 20% and 40% over the control beam. Thus, it can be concluded that the externally bonded CFF strips significantly increased the shear capacity of CFF repaired beams.
- The numerical results have shown that the application of external shear enhancement can enhance the shear behaviour of the repaired beams that is similar to the experimentally tested beams. The load deflection behaviour of the predicted finite element model shows better agreement with the experimental results. However, the difference in stiffness was slightly higher as assumptions taken in the numerical study may not be fully applied in the real specimens.
- This numerically predicted model can be used to depict the initiation and propagation of cracks in the CFF repaired beams but it is partially able to observe the cracking pattern in the tested beams with discrete CFF strips. Besides that, the numerical model assists to predict and identify the high risk of failure zone for discrete CFF strips.

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