

# Experimental and Finite Element Studies on CFF Strengthened RC Rectangular Beams in Shear

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**Abstract:** This paper presents the experimental and finite element (FE) results of Reinforced Concrete (RC) rectangular beams in shear bonded externally using Carbon Fibre Fabric (CFF) reinforcement. In the experimental study, all, except control, beams were strengthened/repared with CFF reinforcement at laboratory environment. These beams were tested under four point bending system for failure. Moreover, to validate the obtained experimental results, the study used two-dimensional model to simulate the shear behaviour of the CFF repaired/strengthened beams using LUSAS finite element analysis software. The experimental results confirm that the CFF repaired and initially strengthened beams attained a gain of 37.28% - 77.95% and 60.71% - 77.34% for shear span to effective depth ratio of 2.5 and 4.0, respectively. The results of FE prediction were compared against the experimental results obtained from the experimental investigation. It was generally found that the comparison between the predicted FE and experimental results show satisfactory correlation in terms of load-displacement profile.

**Keywords:** Carbon, beam, shear, repair finite element

## 1. Introduction

Different repairing techniques such as steel plate bonding method and external section enlargement have been used for improving the structural performance of the existing reinforced concrete structures. Externally bonded Fibre Reinforced Polymer (FRP) reinforcement has attracted the interest of researchers and engineers for different structural application in civil engineering field. The failure mechanism of RC beams in shear is very complex as compared to the flexural failure since it occurs abruptly without any advance warning. Moreover, shear strengthening of RC beams is very complex as it cast monolithically with slab. Researchers [1-6] have attempted with different wrapping layouts (i.e. closed wrapping, U-wraps, and sides of web), types of FRP, amount and orientation of FRP reinforcement. It was observed that the externally bonded CFF reinforcement enhanced the shear capacity of the beams. These experimental investigations have demonstrated different modes of failure, i.e. shear with FRP rupture, splitting of concrete, flexural, and debonding of FRP.

Moreover, theoretical investigations by Triantafillou and Antonopoulos [7], Khalifa and Nanni [4], Zhang and Hsu [8], ACI Committee 440 [9] established design equations to evaluate shear capacity of the FRP strengthened beams using the existing experimental database. Literature review reported that significant experimental and theoretical investigations have been conducted to study the shear behaviour of externally bonded FRP beams. However, the analysis of externally bonded FRP strengthened RC beams using finite element method has not been widely addressed [10-13].

Kachlakev et al. [13] used ANSYS software to model shear behaviour of the GFRP strengthened beams, which replicated the transverse beams of the Horsetail Creek Bridge. It was found that the linear and non-linear behaviour up to failure of the FE models show good agreement with observation and data obtained from the experimental results. Similarly, Santhakumar and Chandrasekaran [12] have simulated the behaviour of the CFRP retrofitted RC shear beams using ANSYS. The effect of two different orientation (i.e.  $\pm 45^\circ$  and  $90^\circ$ ) of CFRP reinforcement were simulated and compared with experimental results. Moreover, researchers [10-13] have studied the finite element analysis of RC beams with the application of continuous FRP reinforcement; however the FE analysis of beams in shear with discrete FRP strips has not been addressed.

This paper presents the results of experimental and finite element (FE) modelling of Reinforced Concrete (RC) rectangular beams in shear bonded externally using Carbon Fibre Fabric (CFF) reinforcement. In order to study the behaviour of the CFF repaired/strengthened beams, the predicted finite element results are compared with the experimental results obtained from the past literature of the author (Jayaprakash et al. [14]). The software used in this study is LUSAS Finite Element Analysis software. The computational results of load-displacement profile and failure load are addressed.

## 2. Experimental Study

Seven full-scale rectangular shear beams bonded externally using bi-directional discrete CFF strips were tested under three and four point bending systems at the

structural engineering laboratory by Jayaprakash et al [14]. The cross section and overall span of beams was 120mm x 340mm and 2980mm, respectively. These beams were divided into two divisions: namely, BT1 and BT2 representing the shear span to effective depth ratio ( $a_v/d$ ) of 2.5 and 4.0, respectively. Fig. 1 shows the reinforcement and cross section details of beams in divisions BT1 and BT2. The discrete CFF strips of width 80mm was applied at two different orientations  $0/90^0$  and  $45/135^0$ , however the spacing of CFF strips remained as 150mm c/c. In order to prevent the flexure failure, a strip of width 120mm was applied along the soffit of the beam. The CFF strips were applied using two components epoxy. The internal and external reinforcement details of the tested beams are illustrated in Table 1. The control beam was subjected to loading to develop precracks followed by unloading to zero and then reloaded for failure. However, the CFF 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, these CFF repaired specimens were loaded to failure without any intervention. On the other hand, the initially strengthened beams (i.e. BT1-II and BT2-2I) were strengthened and loaded to failure without any precracking phase. The displacement was measured by using LVDT placed at the centre of the beam.

For the beams in division BT1 representing a shear span to effective depth ratio ( $a_v/d$ ) of 2.5, the control beam BT1a observed a diagonal crack within the shear span at a load of approximately 48kN and the shear failure was attained at a peak load of 94.82kN. The repaired (BT1-1) and initially strengthened (BT1-II) beams wrapped using discrete CFF U-strips (i.e. orientation of CFF strips:  $0/90^0$ ) along the shear zone. After repairing, the beam BT1-1 attained a diagonal shear crack at a load of 108kN and the shear with CFF rupture failure occurred at a peak load of 134.73kN. Fig. 2 shows the failure patterns of control and CFF repaired beams. In the case of beam BT1-II, the diagonal shear cracks were exhibited at 95kN and their corresponding failure load occurred at 174.64kN. Analogous to beam BT1-1, the initially strengthened BT1-II was also failed in conjunction with shear and CFF rupture. These beams BT1-1 and BT1-II attained a gain of 37.28% and 77.95% over the control beam BT1a. The attained shear capacity of the BT1-II was 30% greater than the repaired beam BT1-1.

In the case of beams with  $a_v/d$  ratio of 4.0, the diagonal shear crack in the control beam BT2a emerged at a load of 55kN and failed catastrophically at right shear span of the beam with an ultimate load of 64.88kN. For the beam BT2-1 repaired with vertical (i.e.  $0/90^0$ ) CFF strips, the diagonal cracks originated in between the CFF strips at a load of approximately 95kN and failed abruptly in shear with CFF rupture at a ultimate failure load of 134.73kN. The repaired beam BT2-2 with inclined CFF strips attained a diagonal crack with a load of 108kN. As the applied increased, the beam failed in shear with CFF rupture in the left shear span at a peak load of

121.42kN. The shear capacities of these beams were increased by 67.36% and 60.71% over the control beam BT2a. Fig. 3 portrays the failure patterns of repaired beams with vertical and inclined CFF strips. The initially strengthened beam BT2-2I also used similar orientation as in BT2-2 which attained a failure load of 154.68kN. The mode of failure of beam BT2-2I was similar to beam BT2-2. The shear crack occurred at a load of 108kN respectively. The attained enhancement of the initially strengthened beam BT2-2I was 77.34% greater than the control beam BT2a. All these beams in division BT2 were also attained shear with CFF rupture failure which is similar to that of CFF repaired beams in division BT1. Moreover, no debonding of CFF strip from the concrete surface or peeling of CFF strip was observed in any of beams in divisions BT1 and BT2 which is probably due to the application of bi-directional CFF reinforcement.

From the experimental investigation, it was observed that the externally bonded CFF reinforcement enhanced the shear capacity of rectangular beams in shear. Results also show that the shear capacity of the CFRP repaired beams was varied with respect to the test variables (i.e. shear span to effective depth ratio, and orientation of CFF reinforcement). Table 3 shows the summary of experimental results.

### 3. Finite Element Study

A two-dimensional finite element model was developed using LUSAS software [15]. The concrete material was modelled as surface element (QTS4). Fig. 4(a) portrays the surface element QTS4. The stress-strain curve of concrete in compression was linearly elastic up to 30 percent of maximum compressive strength. Subsequently, the stress increased gradually up to the maximum compressive strength. After the maximum compressive strength, the curve descends into a softening region and eventually crushing failure occurs at an ultimate strain [16]. This study used a perfectly plastic relationship instead of the compressive strain-softening curve [13, 17]. Under uniaxial tension, the material was assumed to be linearly elastic up to the tensile strength. After this point, the concrete cracks and the strength decreases to zero [13, 16, 17]. Fig. 5(a) shows the behaviour of stress-strain relationship of concrete material under uniaxial loading. The modulus of elasticity and tensile strength of concrete for each beam can be calculated using the following equations (ACI 318-99) [18]:

$$E_c = 4750\sqrt{f'_c} \quad (1)$$

$$f_r = 0.623\sqrt{f'_c} \quad (2)$$

where  $E_c$  is the modulus of concrete, MPa;  $f'_c$  is the ultimate compressive strength of concrete, MPa; and  $f_r$  is the ultimate tensile strength of concrete, MPa. The poison's ratio and the ultimate strain value for the concrete were taken as 0.2 and 0.035 as used [19].

The line element BAR2 was used to model the steel rebars. Fig. 4(b) shows the bar or line element. Similarly, the flexural CFF reinforcement along the soffit of the beam was modelled as line element. The steel rebar was assumed to be elastic-perfectly plastic manner and identical in tension and compression. The stress-strain behaviour of steel rebar is shown in Fig. 5(b). The tensile strength and modulus of elasticity of steel rebar was 554.17N/mm<sup>2</sup> and 200,000N/mm<sup>2</sup> respectively. A poisson's ratio of 0.3 was used for steel rebar. The bond between the rebar and concrete was assumed to be perfect (i.e. no bond loss).

The CFF reinforcement on the sides of web was modelled as surface element. The CFF reinforcement was assumed to be an isotropic material and modelled as linearly elastic [20]. In order to simulate the e CFF strips with concrete the nodes of concrete were connected to the nodes of CFF strips at the interface therefore two

materials shared the same nodes. Moreover, perfect bond was assumed between the concrete and CFF reinforcement and no loss of bond between them [12, 17]. Table 2 shows the material properties of the CFF reinforcement.

In this FE analysis, only half of the beam was modelled as the dimension of beam and the loading pattern is symmetrical. The control and repaired specimens were modelled as uncracked (i.e. no cracks) beams by ignoring the precracked phase. Figs. 6(a)-(c) portray the developed finite element model of control and CFF strengthened beams. The failure load of the control, CFF repaired, and initially strengthened beams obtained from the numerical finite element study is compared with the experimental results reported by the author [14] and presented in Table 3.

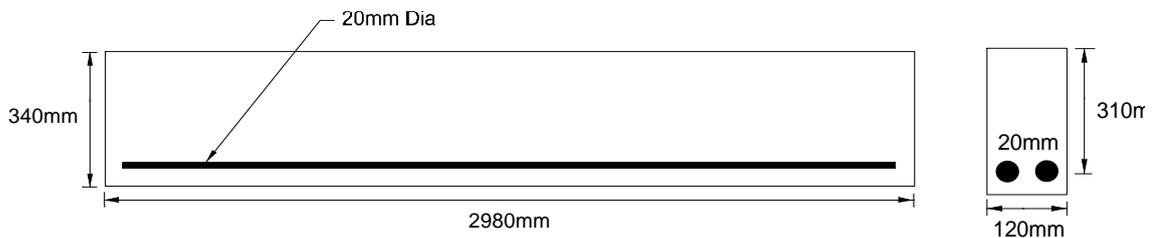


Fig. 1 Reinforcement and cross-section details of beams in divisions BT1 and BT2

Table 1 Internal and external reinforcement details of beams

Division	Specimen	a <sub>v</sub> /d	f <sub>c</sub> (N/mm <sup>2</sup> )	External CFRP Reinforcement			
				Type of Wrapping	Width (mm)	Spacing (mm c/c)	Orientation
BT1	BT1a	2.5	27.38	---	---	---	---
	BT1-1			U-strip	80	150	0/90°
	BT1-1I			U-strip	80	150	0/90°
BT2	BT2a	4.0	16.73	---	---	---	---
	BT2-1			U-strip	80	150	0/90°
	BT2-2			Inclined L-strip	80	150	45/135°
	BT2-2I			Inclined L-strip	80	150	45/135°

Table 2 Material properties of CFF composites [21]

External CFF Reinforcement	Thickness (mm)	Tensile strength (N/mm <sup>2</sup> )	Modulus of Elasticity (N/mm <sup>2</sup> )	Poisson's ratio
U-strips (0/90) Bi-directional	0.09	3,800	230,000	0.184



(a) BT1a



(b) BT1-1

Fig. 2 Failure patterns of control and CFF repaired beams (a) Shear failure (b) Shear with CFF rupture failure



(a) BT2-1



(b) BT2-2

Fig. 3 Failure patterns of CFF repaired beams (a) U-CFF strips – Shear failure (b) Inclined CFF strips- Shear with CFF rupture failure

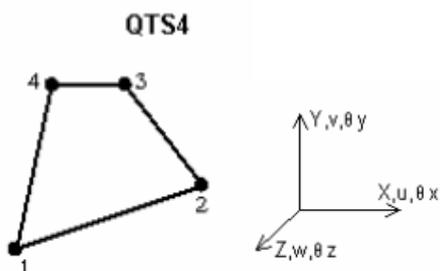


Fig. 4(a) Surface element

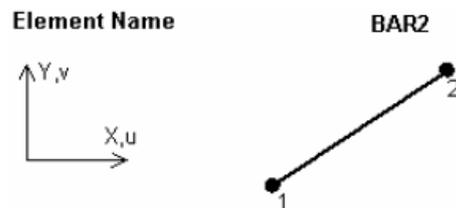


Fig. 4(b) Bar element

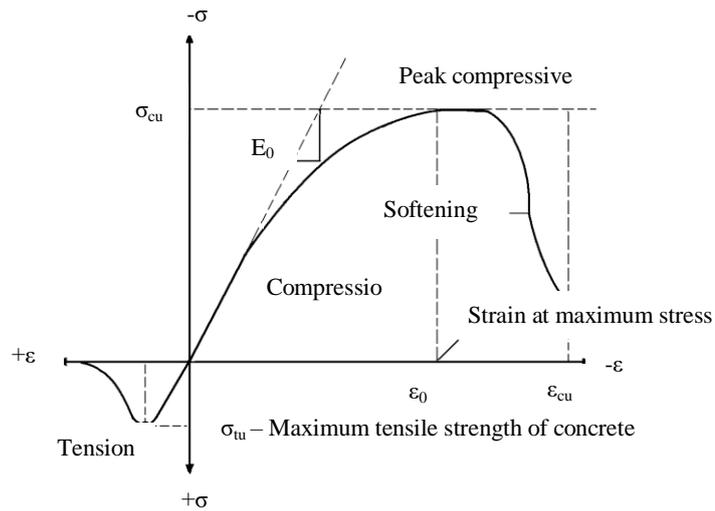


Fig. 5(a) Stress-strain curve for concrete [17]

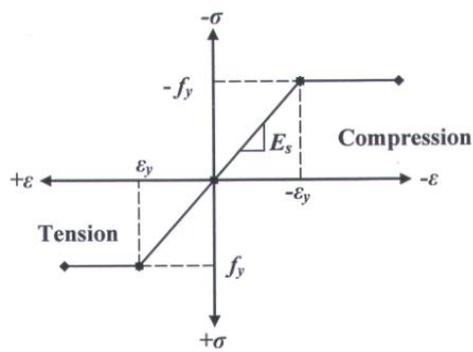


Fig. 5(b) Stress-strain curve for steel rebar [17]

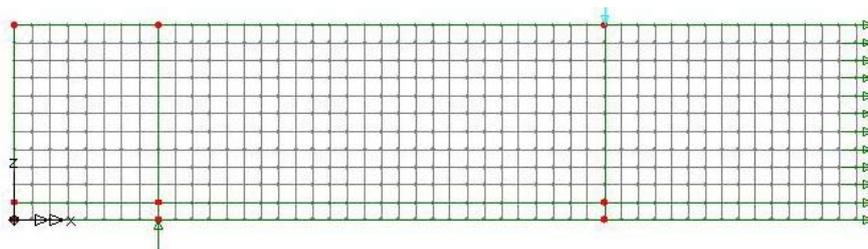


Fig. 6(a) Finite element model of control beam BT1a

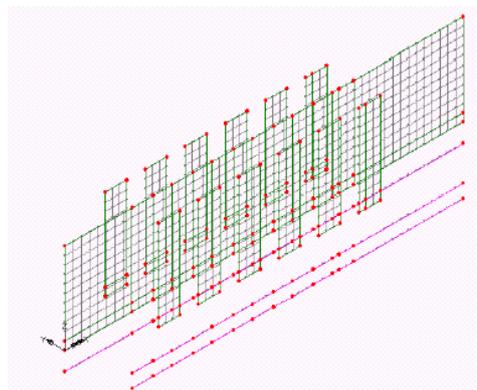


Fig. 6(b) Finite element model of CFF repaired beam BT1-1 (Orientation of CFF strip:  $0^0$ )

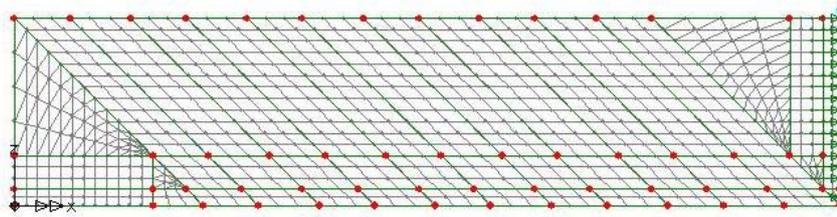


Fig 6(c) Finite element model of CFF repaired beam BT2-2 (Orientation of CFF strip: 45°)

Table 3 Comparison of experimental and predicted FEM results for beams in divisions BT1 and BT2

Specimen	Failure load (kN)			Deflection (mm)		
	Exp	FEM	Exp/FEM	Exp	FEM	Exp/FEM
BT1a	94.82	101.27	---	12.64	6.90	---
BT1-1	134.73	167.00	0.80	12.86	12.44	1.03
BT1-1I	174.64	167.00	1.04	15.54	12.44	1.24
BT2a	64.88	103.28	---	6.21	7.60	---
BT2-1	134.73	132.07	1.02	23.57	15.20	1.55
BT2-2	121.42	112.97	1.08	14.76	8.90	1.65
BT2-2I	154.68	112.97	1.36	15.18	8.90	1.70

#### 4. Load-Deflection Profile

The load-displacement profile of the experimental and predicted results of the control, CFF repaired, and initially strengthened beams in division BT1 is presented in Fig. 7. The predicted displacement trend of the control beam BT1a was similar to the experimental results. For repaired beam BT1-1, the stiffness of the predicted FE model shows good correlation in the linear limb of the experimental results, however in the later stage, there was a deviation was observed from the test curve. It was also observed that the observed experimental result was 19% less than the predicted failure load. The deformed shape of the beam BT1-1 is shown in Fig. 8. On the other hand, the initially strengthened beam BT1-1I exhibits better agreement between the predicted and experimental results. The obtained experimental result was 4% greater than the predicted result. Moreover, the predicted displacement curve attained similar stiffness trend up to 29% of ultimate peak load of the experimentally drawn curve.

Fig. 9 portrays the load-displacement profile of the experimental and predicted results of the control, CFF repaired, and initially strengthened beams in division BT2. The stiffness of the test results of the control beam BT2a was similar up to 47% of ultimate failure load; however the attained predicted failure load was 37% greater than the experimental results. The behaviour of repaired beam BT2-1 from FE analysis demonstrates excellent correlation with the experimental results. Similarly, for the beam BT2-2 repaired at 45/135° orientation, a good agreement exists between the experimental and predicted results, whereas the predicted failure load was 8% less than experimental results. Fig. 10 shows the deformed shaped

of beam BT2-2 repaired with CFF strips placed at 45/135° orientation. When comparing with the experimental curve of initially strengthened beam BT2-2I, the predicted deflection trend was similar prior to the failure load, however, at ultimate stage, the numerical results were 36% less over the experimental results.

The discrepancies between the experimental and FE results is probably due to the effect of micro cracks in the concrete and handling of beams. The microcracks could reduce the stiffness of the tested beams whereas the finite element analysis does not include the microcracks. Moreover, this finite element analysis assumed that the bond between the concrete and steel was perfect but this assumption would not be true in the actual beam. As bond slip occurs, there was no composite action between the concrete and steel reinforcement [13].

From the overall behaviour of displacement profile, it can be seen that the predicted FE result shows good correlation with the existing experimental results. Fig. 11 illustrates the comparison of experimental and predicted FE results. The experimental displacement values of beams in divisions BT1 and BT2 were 3.4% - 25% and 35% - 70.56% greater over the predicted FE values. It was also observed that the predicted deflection values of the CFF repaired and initially strengthened beams at ultimate failure load were relatively less as compared to the experimental values due to the catastrophic shear with CFF rupture failure. Since the failure of CFF repaired/strengthened beams in catastrophic nature, the obtained displacement values were abruptly increased when compared to the predicted results. The repaired and initially strengthened CFF beams attained a shear failure with rupture which was similar to the experimental results.

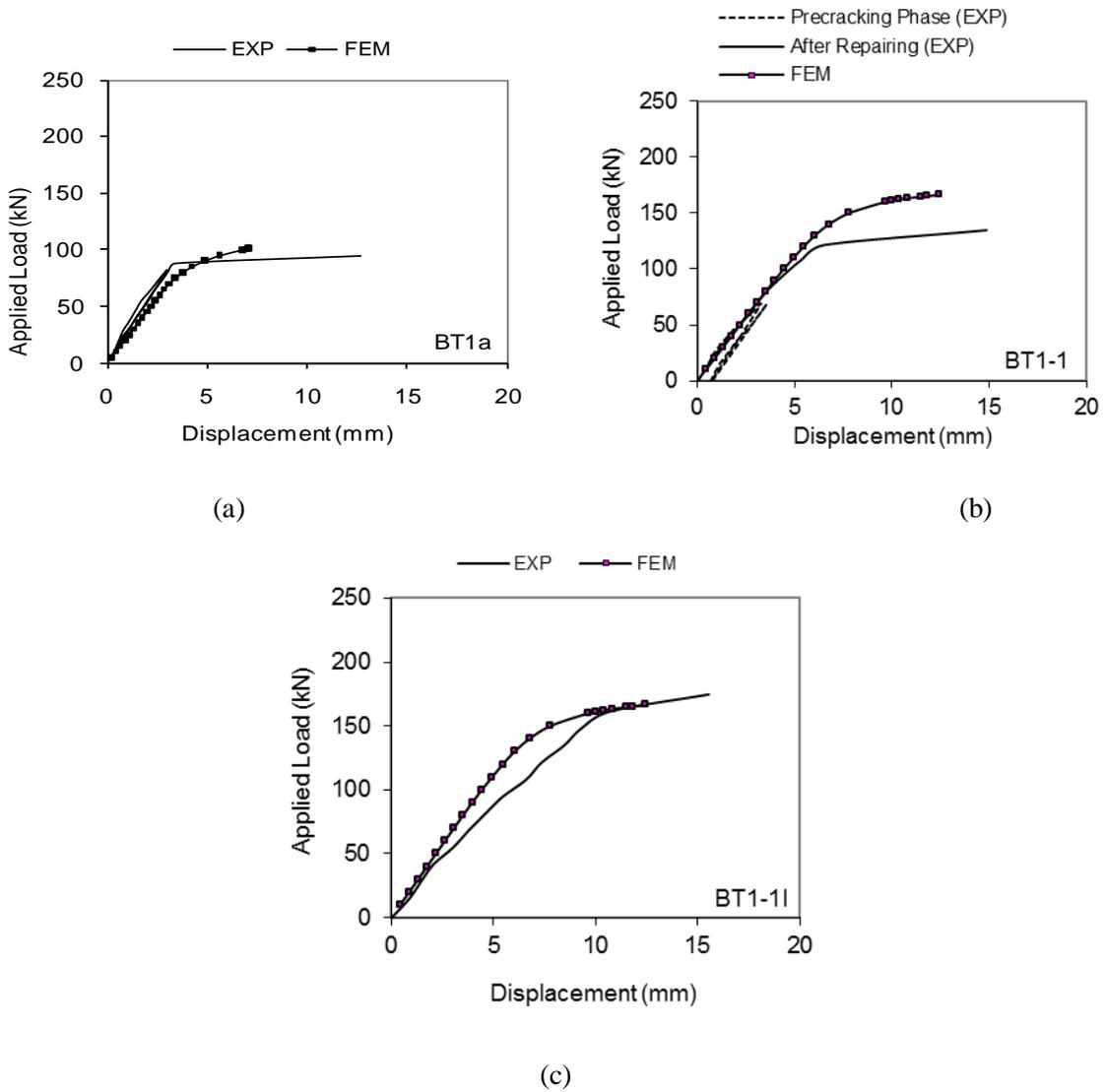


Fig. 7 Load – displacement curves for control and CFRP repaired/strengthened beams in division BT1

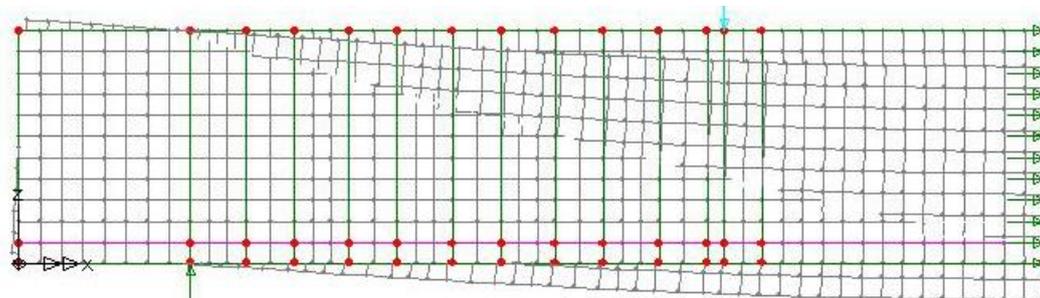


Fig. 8 Deformed shape of repaired beam BT1-1 (i.e. U-CFF strips)

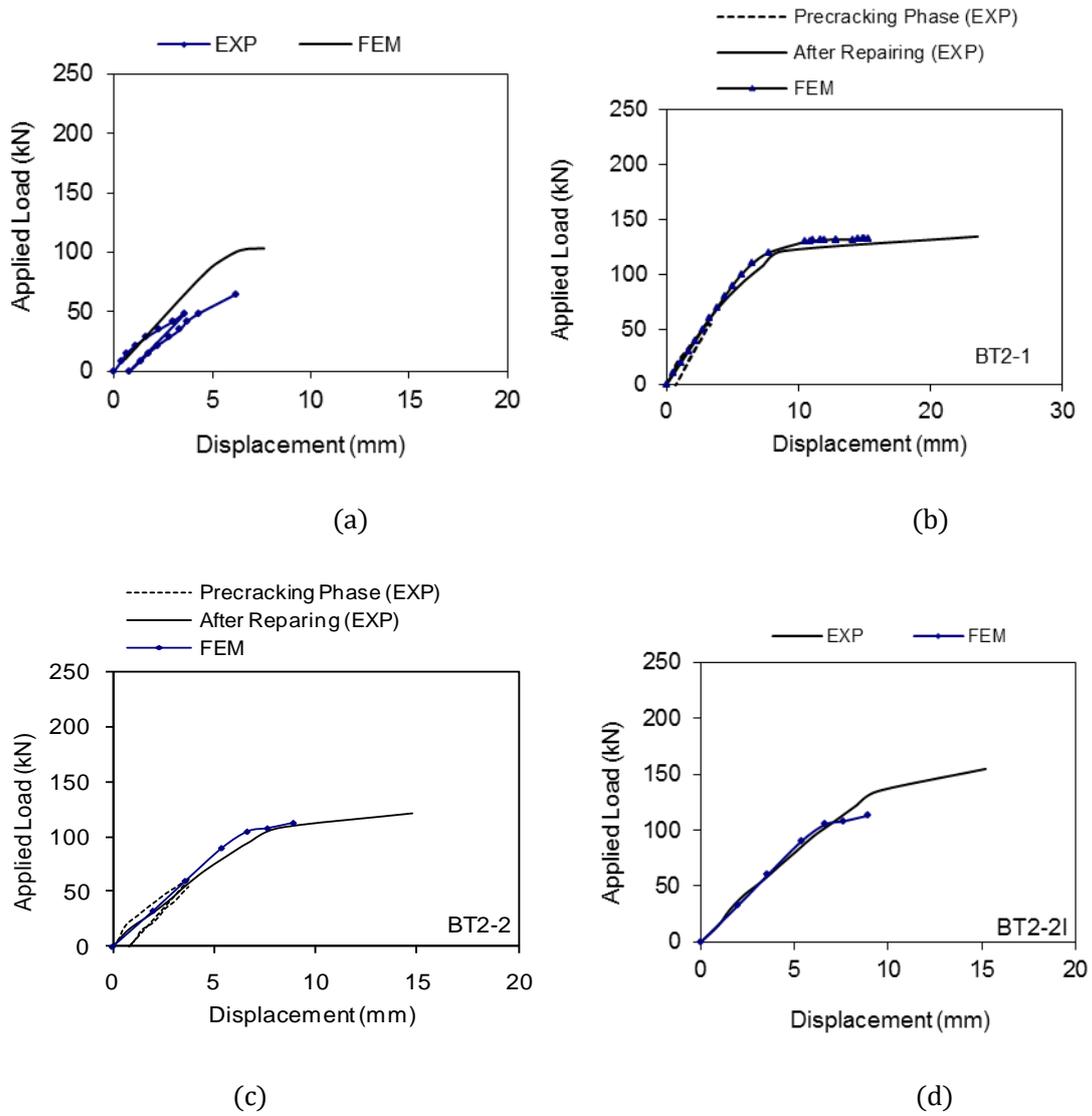


Fig. 9 Load – displacement curves for control and CFRP repaired/strengthened beams in division BT2

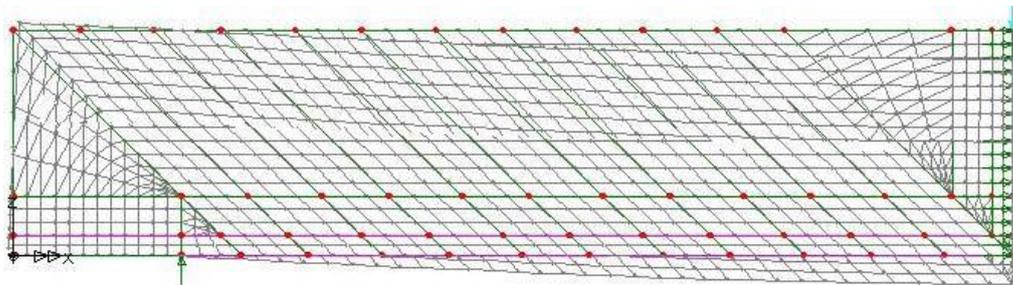


Fig. 10 Deformed shape of repaired beam BT1-1 (i.e. orientation: 45/135 Degree)

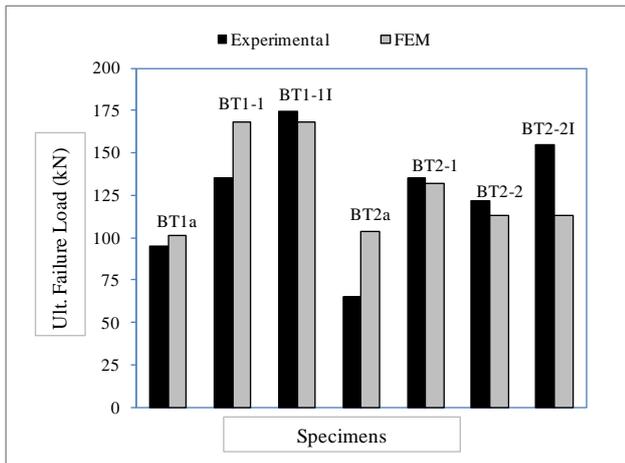


Fig. 11 Comparison of experimental and FEM results

## 5. Conclusions

This study presented the Finite Element (FE) modelling of Reinforced Concrete (RC) rectangular beams in shear bonded externally using CFF reinforcement. The experimental results confirm that the CFF repaired and initially strengthened beams attained a gain of 37.28% - 77.95% and 60.71% - 77.34% for shear span to effective depth ratio of 2.5 and 4.0, respectively. The developed finite element model could be used in predicting ultimate strength and stiffness with acceptable accuracy. It was generally found that the comparison between the predicted FE and experimental results show satisfactory correlation in terms of load-displacement profile.

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