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Performance of reinforced concrete beams strengthened by hybrid FRP laminates

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Abstract

In the last two decades, the use of advanced composite materials such as Fiber Reinforced Polymers (FRP) in strengthening reinforced concrete (RC) structural elements has been increasing. Research and design guidelines concluded that externally bonded FRP could increase the capacity of RC elements efficiently. However, the linear stress–strain characteristics of FRP up to failure and lack of yield plateau have a negative impact on the overall ductility of the strengthened RC elements. Use of hybrid FRP laminates, which consist of a combination of either carbon and glass fibers, or glass and aramid fibers, changes the behaviour of the material to a non-linear behaviour. This paper aims to study the performance of reinforced concrete beams strengthened by hybrid FRP laminates.

This paper presents an experimental program conducted to study the behaviour of RC beams strengthened with hybrid fiber reinforced polymer (HFRP) laminates. The program consists of a total of twelve T-beams with overall dimensions equal to $460 \times 300 \times 3250$ mm. The beams were tested under cyclic loading up to failure to examine its flexural behaviour. Different reinforcement ratios, fiber directions, locations and combinations of carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) laminates were attached to the beams to determine the best strengthening scheme. Different percentages of steel reinforcement were also used. An analytical model based on the stress–strain characteristics of concrete, steel and FRP was adopted. Recommendations and design guidelines of RC beams strengthened by FRP and HFRP laminates are introduced.

Keywords: Ductility; Flexure; FRP; Repair; Hybrid; Rehabilitation and strengthening

1. Introduction

In the last few decades, moderate and severe earthquakes struck different places in the world causing severe damage to reinforced concrete (RC) structures. This requires upgrading of RC structures to resist more loads. In addition to the several advantages of FRP in strengthening, such as no corrosion characteristic, lightweight and high strength, this technique does not result in an increase in the stiffness of the structure and consequently no more seismic loads are added to the strengthened structural elements. Research and design guidelines concluded that externally bonded FRP could increase the capacity of RC elements efficiently. Unlike the better ductility achieved for columns wrapped by FRP due to confinement of concrete, beams strengthened with FRP have low ductility. The linear stress–strain characteristics of FRP up to failure and lack of yield plateau are the reason of the negative impact on the overall ductility of the strengthened RC beams. In seismic zones, ductility of concrete element is a main design criterion. Several proposals were suggested to enhance this performance:

1. Confinement of concrete at the compression zone [4], however, this method is only practical for beams with rectangular section where no slabs are attached.
2. Partial debonding of FRP at different locations [3].

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3. Design the beam to fail by compression in concrete [5], however this may not be achieved for beams with T-section.
4. Use hybrid FRP [1].

Strengthening of RC beams with HFRP laminates is introduced in this paper to increase both of their capacity and ductility. HFRP laminates, which consist of a combination of either carbon and glass fibers or glass and aramid fibers have a non-linear stress–stain behaviour [1]. An experimental program was conducted to study the behaviour of RC beams strengthened with HFRP laminates. The program consists of a total of twelve T-beams with overall height of 300 mm and length of 3250 mm. The beams were tested in a four-point loading configuration under cyclic loading to evaluate their ductility and energy dissipation. Different combinations of CFRP and GFRP laminates were attached to the beams to predict the best scheme of strengthening. Reinforcement ratio and location of FRP as well as fiber direction were also varied. Different steel reinforcement ratios were used in the study. An analytical model based on the stress–strain characteristics of concrete, steel and FRP was adopted. Different recommendations and design guidelines of RC beams strengthened by FRP and HFRP laminates were introduced.

2. Experimental work

The experimental program consists of testing twelve RC T-beams with overall depth and length of 300 mm and

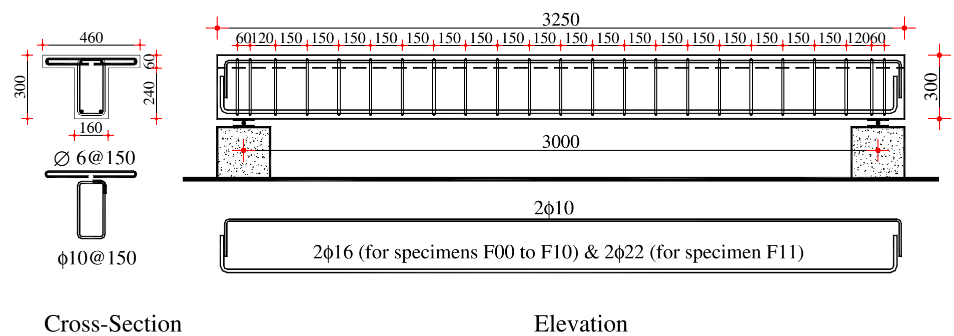

Fig. 1. Steel reinforcement detail of the test specimens.

Table 1
Details of the different specimens

Specimen no.	Type of FRP	Properties of CFRP		Properties of GFRP		Fiber direction with respect to beam axis	Beam condition at time of strengthening	Anchorage type	Notes
		Ratio (%)	Location	Ratio (%)	Location				
F00	Control specimen								–
F01	CFRP	0.073	At bottom	–	–	0°	–	–	–
F02	CFRP + GFRP	0.020	On sides	0.117	At bottom	0°	Unloaded	–	CFRP just above bottom edge
F03	CFRP + GFRP	0.020	On sides	0.124	At bottom	0°	Unloaded	6-U GFRP	CFRP above bottom edge by 20 mm
F04	GFRP	–	–	0.124	At bottom	0°	Unloaded	2-U GFRP	–
F05	GFRP	–	–	–	–	0/90°/+45° 0/90°/−45°	Unloaded	–	Two layers
F06	CFRP + GFRP	0.015	At bottom	0.169	On sides	0°	Unloaded	6-U GFRP	GFRP above bottom edge by 20 mm
F07	CFRP + GFRP	0.030	At bottom	0.338	On sides	0°	Unloaded	6-U GFRP	GFRP above bottom edge by 20 mm
F08	CFRP + GFRP	0.020	On sides	0.124	At bottom	0°	Loaded	6-U GFRP	CFRP above bottom edge by 20 mm
F09	CFRP + GFRP	0.015	At bottom	0.169	On sides	0°	Loaded	6-U GFRP	GFRP above bottom edge by 20 mm
F10	CFRP	0.073	At bottom	–	–	0°	Loaded	–	–
F11	CFRP + GFRP	0.030	At bottom	0.338	On sides	0°	Unloaded	6-U GFRP	GFRP above bottom edge by 20 mm

3250 mm, respectively. The top flange was 460 mm wide and 60 mm thick, as shown in Fig. 1. The beams were simply supported with a clear span of 3000 mm. The top and bottom longitudinal reinforcement of specimens F00 to F10 were two 10 mm and two 16 mm diameter bars, respectively, with a ratio of 0.9% for the main reinforcement. The stirrups were 10 mm diameter bars every 150 mm with a volumetric percentage of 0.63%. The main reinforcement of the last specimen, F11, was two 22 mm diameter bars with a ratio of 1.76%.

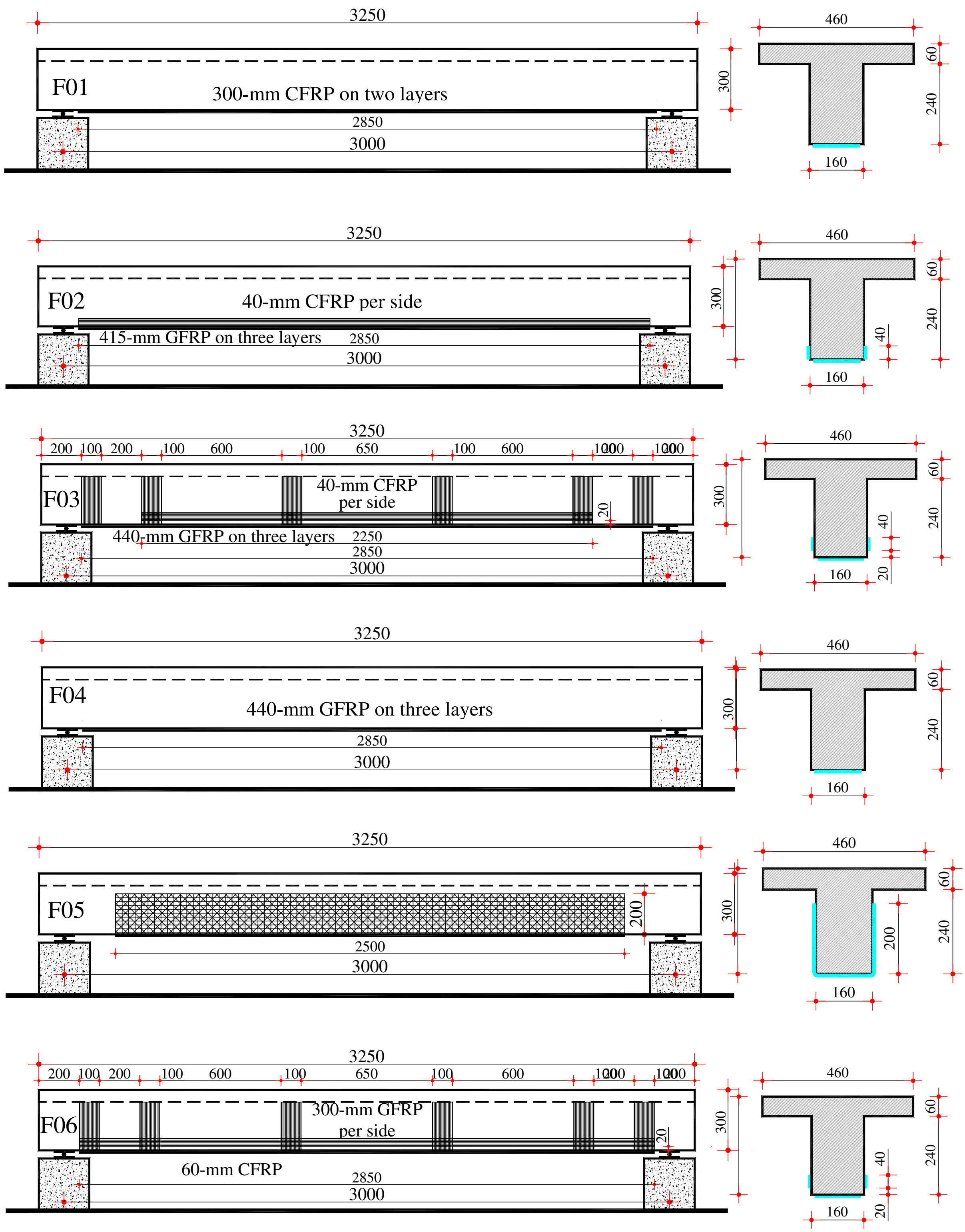

Fig. 2. Strengthening schemes of F01, F02, F03, F04, F05 and F06.

Hybrid systems consist of a combination of CFRP and GFRP laminates were used to strengthen the beams. The laminates were attached with the fibers in the longitudinal direction to increase the flexural capacity of the beams. The thickness of CFRP laminates was 0.117 mm, while its ultimate strain and modulus of elasticity were 1.55% and 240 GPa, respectively. The thickness of GFRP laminates was 0.135 mm, with ultimate strain and modulus of elasticity of 2.88% and 65 GPa, respectively. The characteristic compressive strength of the concrete cubes after 28 days was 25 MPa, while the yield stress of the steel was 415 MPa for the longitudinal reinforcement and 240 MPa for the stirrups.

2.1. Test specimens

The variables considered in this study, were the percentage of steel reinforcement, type of HFRP, type and location of FRP and ratio between CFRP and GFRP as given in Table 1. Seven beams were tested until the time of writing the paper. Beam F00 was a control specimen, while the strengthening scheme of the other three specimens was as follows:

- Specimen F01 had two longitudinal CFRP laminates of 150 mm width each attached on the bottom chord of the specimen. The volumetric percentage of CFRP laminates is 0.073%, as shown in Fig. 2.
- Specimen F02 had a combination of longitudinal CFRP and GFRP laminates. GFRP laminates with 415 mm width attached on the bottom chord of the specimens on three layers. 40-mm width of CFRP laminates attached at each side of beam just above its bottom chord. The volumetric percentages of GFRP and CFRP laminates are 0.117% and 0.02%, respectively, as shown in Fig. 2.
- Specimen F03 had a combination of longitudinal CFRP and GFRP laminates. GFRP laminates with 440 mm width were attached on the bottom chord of the specimens on three layers. CFRP laminates of 40 mm width were attached at each side of the beam above its bottom chord by 20 mm. Six GFRP U-shape laminates with 100 mm width were attached on top of the other laminates to prevent debonding of FRP. The volumetric percentages of GFRP and CFRP laminates are 0.124% and 0.02%, respectively, as shown in Fig. 2.
- Specimen F04 had longitudinal GFRP laminates with 440 mm width attached on the bottom chord of the specimens on three layers. The volumetric percentage of GFRP laminates is 0.124%, as shown in Fig. 2.
- Specimen F05 had GFRP laminates with three directions $(0°, 90°, 45°)$ attached on the bottom chord and sides of the specimens on two layers with different orientation. The first layer was attached with orientation $(0°, 90°, 45°)$, while the second layer was attached with orientation $(0°, 90°, -45°)$, as shown in Fig. 2.
- Specimen F06 had a combination of longitudinal CFRP and GFRP laminates. CFRP laminates with 60 mm width were attached on the bottom chord of the specimens. 300-mm width of GFRP laminates on five layers were attached at each side of the beam above its bottom chord by 20 mm. Six GFRP U-shape laminates with 100 mm width were attached on top of the other laminates to prevent debonding of FRP. The volumetric percentages of GFRP and CFRP laminates are 0.169% and 0.015%, respectively, as shown in Fig. 2.

FRP laminates were wrapped after the concrete reached an age of 28 days. The wrapping procedure included surface preparation of the beams using a hammer and blower and two-component epoxy adhesive as per the instructions of the manufacturing company of the laminates. The corners of the beams were rounded at a radius of 15 mm to attach the U-shape GFRP laminates.

2.2. Test set-up

The beams were subjected to cyclic loading up to failure using a hydraulic machine of 300 kN capacity. The load was measured using a load cell of 1000 kN capacity, as shown in Fig. 3. Two concentrated loads at 375 mm from mid span were applied on the beam using a stroke control

Fig. 3. Test set-up.

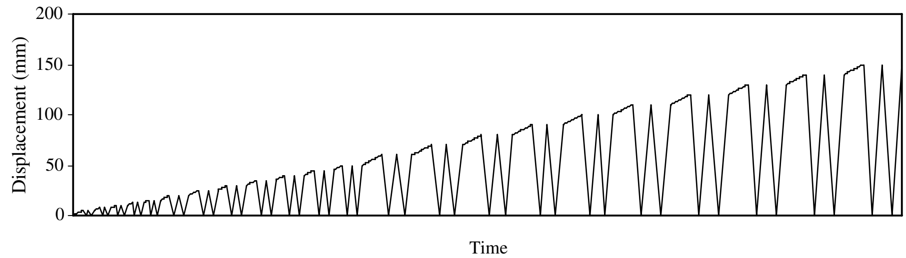

Fig. 4. Pattern cyclic loading.

system. The cyclic loading was achieved by increasing the stroke at 2.5 mm increments up to 15 mm and 5 mm increments up to 50 mm and finally with 10 mm increments, as shown in Fig. 4. Two cycles were applied at each increment. The minimum load for each cycle was 5.0 kN to maintain the stability of the test rig. The data was collected using a data acquisition system and a ''lab view'' software at a rate of one sample per second.

The longitudinal and transverse strains of the specimens were measured by two different methods; linear variable differential transducers, (LVDT), and electric strain gauges. The strains of the concrete, steel and FRP laminates were

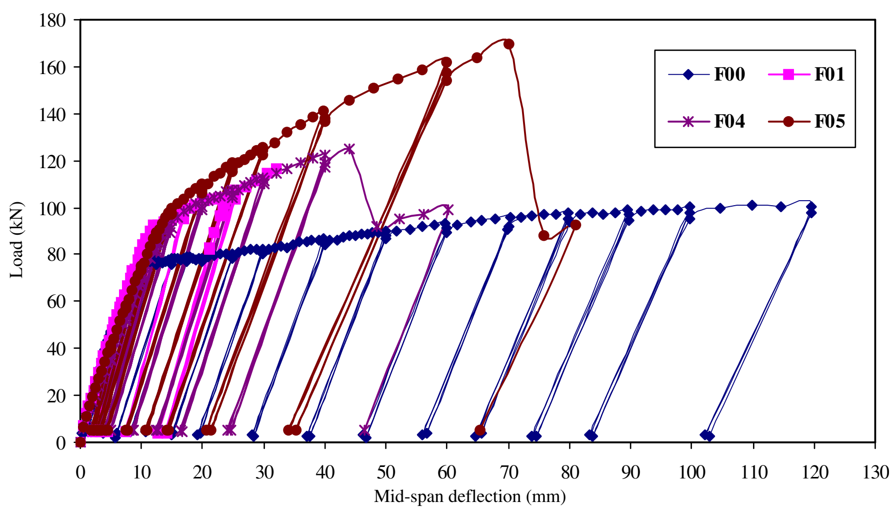

Fig. 5a. Load–deflection curves for specimens (F00, F01, F04 and F05).

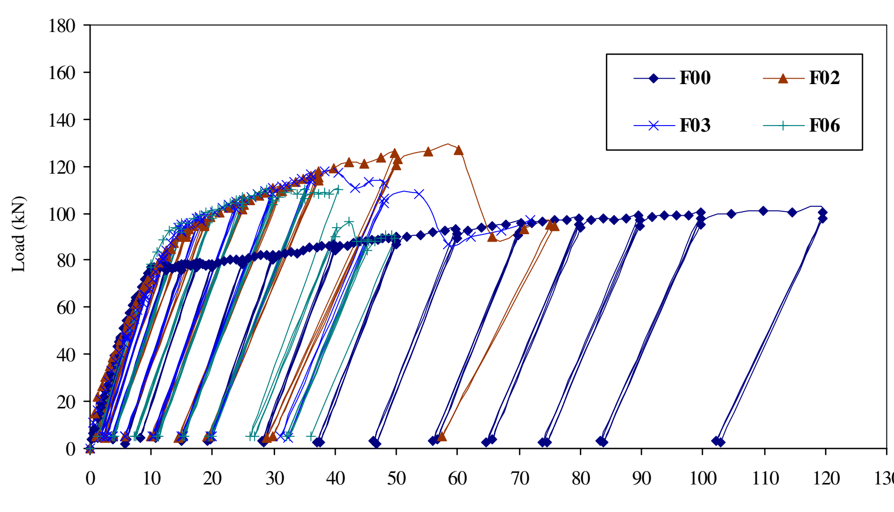

Fig. 5b. Load–deflection curves for specimens (F00, F02, F03 and F06).

measured in the longitudinal direction. Deflection was measured at center of the section and two opposite points at the mid-span of the beam.

3. Experimental results and analysis

The results presented in this paper are for seven specimens, (F00, F01, F02, F03, F04, F05 and F06), which were tested at the time of submitting the paper. Fig. 5a shows the load–deflection curves for specimens F00, F01, F04 and F05. Fig. 5b shows the load–deflection curves for specimens F00, F02, F03 and F06. Table 1 shows the details of each tested beam. The failure loads of the specimens were 100, 116.5, 127.2, 117.3, 125.25, 169.7 and 110.25 kN, respectively. The ultimate carrying capacity of the beams F01 to F06 increased by 16.5%, 27.2%, 17.3%, 25.3%, 69.7% and 10.3% respectively compared to the control specimen F00. It should be noted that the stress in the steel reinforcement at onset of failure of the control specimen was higher than the yield stress due to the large measured strain at flexural failure and the small yield plateau of the used steel. The maximum measured strain in the steel reinforcement was 2.8% for the control specimen, while it ranged from 1.0% to 1.5% for strengthened specimens. This indicated that at failure of concrete beams, the stress in the steel reinforcement was less in the strengthened specimens than that of the control specimen.

Maximum deflection measured for the specimens were 120, 32.1, 60.1, 53.6, 44.1, 70.2 and 42.1 mm, respectively. The maximum deflections of the strengthened specimens were measured at complete rupture of HFRP, which coincides with a drop in the load-carrying capacity of the beams, while deflection of the control specimen was measured at crushing of the concrete at the top flange. Failure of the control specimen was ductile since the deflection at failure was 12 times the deflection at yield of steel reinforcement. Using CFRP laminates resulted in a much less ductile failure, where the deflection at ultimate to deflection at yield of steel was only 2.5. Adding combination of CFRP and GFRP could reach a ductility of 6.0, which is 2.4 times the ductility of beams with one type of FRP laminates.

The failure of specimen F01, which is strengthened with CFRP only, was brittle. Flexural cracks appeared at the mid-span followed by yield of steel reinforcement and rupture of CFRP laminates, as shown in Fig. 6. Failure of specimen F02 was ductile since the deflection at failure was six times the deflection at yield of steel reinforcement. Flexural cracks appeared at the mid-span followed by yield of steel reinforcement and rupture of CFRP laminates at

Fig. 6. Failure of specimen F01.

Fig. 7. Failure of specimen F02.

Fig. 8. Failure of specimen F03.

Fig. 9. Failure of specimen F04.

Fig. 10. Failure of specimen F05.

Fig. 11. Rupture of CFRP laminates without any damage in GFRP for specimen F06.

Fig. 12. Failure of specimen F06.

one side. Longitudinal cracks were observed parallel to the GFRP laminates due to the large elastic energy released at onset of rupture of CFRP laminates. This energy also caused a local damage at the interface between the GFRP laminates and concrete. The final failure occurred by debonding of GFRP, as shown in Fig. 7. The failure was progressive and the load dropped gradually, which is attributed to use a combination of CFRP and GFRP laminates. Failure of specimen F03 was ductile since the deflection at failure was 5.5 times the deflection at yield of steel reinforcement. Flexure cracks appeared at the mid-span followed by yield of steel reinforcement and rupture of CFRP laminates at one side followed by rupture of CFRP laminates at the other side. The large elastic energy released at onset of rupture of CFRP laminates resulted in longitudinal cracks parallel to the GFRP laminates. It is believed that the 6-U GFRP anchors contributed to the integrity of the beam after rupture of CFRP. The final failure occurred by rupture of the GFRP as shown, in Fig. 8. Similar to specimen F02, the failure was progressive and the load dropped gradually, which is attributed to use a combination of CFRP and GFRP laminates. Failure of specimen strengthened with GFRP F04 was brittle since the deflection at failure was only three times the deflection at yield of steel reinforcement. Flexure cracks appeared at the mid-span followed by yield of steel reinforcement and rupture of GFRP laminates, as shown in Fig. 9. Failure of specimen F05 was brittle. The failure occurred by debonding of the GFRP as shown in Fig. 10, which is attributed to the large thickness of the used GFRP laminates. For specimen F06, the deflection at failure was five times the deflection at yield of steel reinforcement. Flexure cracks appeared at the mid-span followed by yield of steel reinforcement and rupture of CFRP laminates, as shown in Fig. 11. At onset of CFRP rupture, the load dropped from 110 to 100 kN. The load picked up again to a level of 110 kN until rupture of GFRP laminates occurred, as shown in Fig. 12. The failure was progressive and the load dropped gradually, which is attributed to use a combination of CFRP and GFRP laminates. A progressive failure of the beam was observed due to the un-synchronized rupture of CFRP and GFRP, as well as the progressive

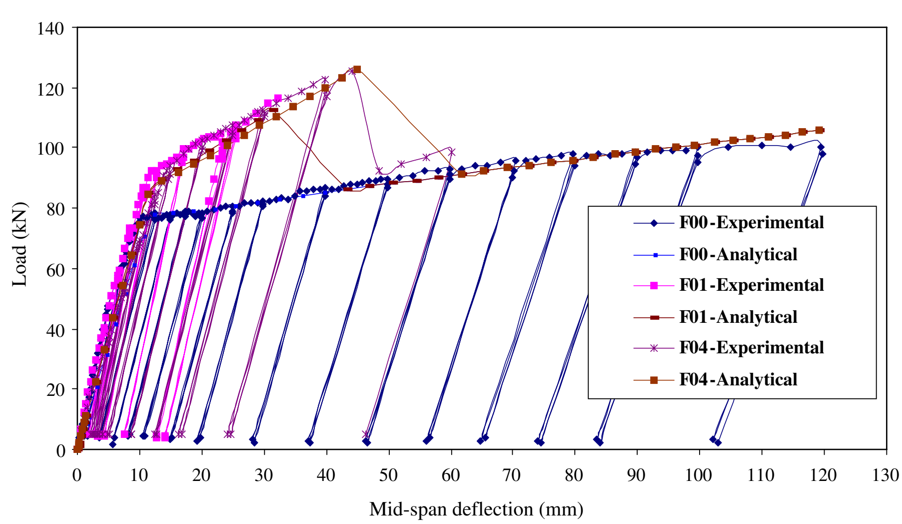

Fig. 13. Comparison between experimental and analytical load–deflection curves for specimens (F00, F01 and F04).

rupture of GFRP attached on the sides of the beams. The 6-U GFRP anchors prevented the damage of the FRP–concrete interface that may have occurred due to rupture of CFRP. It should be mentioned that for all the tested beams, after rupture of FRP laminates the load–deflection behaviour is similar to that of the control beam.

4. Analytical study

Strain compatibility approach was used to predict the ultimate carrying capacity of the beams. A parabolic stress–strain curve was assumed for the concrete [2], while elastic–plastic behaviour was assumed for the steel reinforcement. Linear elastic behaviour of the FRP laminates was assumed based on the data provided by the manufacturer. Deflection of the beam was calculated using integration of the curvature along the span of each beam. Fig. 13 shows a good correlation between the measured and predicted deflection of the beams. It should be noted that a maximum strain of 50% of the ultimate strain values reported by the manufacturer was used as an upper limit to coincide with the maximum measured strain on the FRP laminates.

5. Conclusions

Based on testing seven beams and the analytical study using the strain compatibility approach, the following conclusions can be drawn:

1. Use of CFRP or GFRP laminates for strengthening RC T-beam is an effective method to increase its ultimate carrying capacity. However, the ductility of the beams is significantly reduced. Using a combination of CFRP and GFRP laminates is an effective method to enhance the ductility of the strengthened beams.
2. The released elastic energy due to rupture of CFRP laminates negatively affects the interface between the concrete and GFRP laminates.
3. The U-shape FRP laminates used to anchor longitudinal GFRP laminates succeeded to prevent the damage occurred due to rupture of CFRP laminates.
4. The best scheme used to strengthen concrete beams is to attach CFRP laminates on the sides of the beam 20 mm above the bottom surface and anchored GFRP laminates at its bottom surface.
5. A strain compatibility approach can predict accurately the behaviour of the beams. A limit of 50% of the reported maximum strain values for both CFRP and GFRP was used in the analysis.

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