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Flexural capacity of bi-directional GFRP strengthened RC beams with end anchorages

Experimental and theoretical studies

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Abdurra'uf Mukhtar Gora
Department of Civil Engineering, University of Nottingham, Semenyih, Malaysia
Jayaprakash Jaganathan
School of Civil Engineering, Vellore Institute of Technology, Vellore, India
Mohammed Parvez Anwar
Department of Civil Engineering, University of Nottingham, Semenyih, Malaysia, and
Hau Y. Leung
School of Civil Engineering, Nazarbayev University, Astana, Kazakhstan

Abstract

Purpose – The purpose of this paper is to present the results of experimental and theoretical studies on the flexural capacity of reinforced concrete (RC) beams strengthened using externally bonded bi-directional glass fibre reinforced polymer (GFRP) composites and different end anchorage systems.

Design/methodology/approach – A series of nine RC beams with a length of 1,600 mm and a cross-section of 200 mm depth and 100 mm width were prepared and externally strengthened in flexure with bi-directional GFRP composites. These strengthened beams were anchored with three different end anchorage systems namely closed GFRP wraps, GFRP U-wraps and mechanical anchors. All these beams were tested with four-point bending system up to failure. The experimental results are compared with the theoretical results obtained using the relevant design guidelines.

Findings – The experimental results demonstrate a significant increase in the flexural performance of the GFRP strengthened beams with regard to the ultimate load carrying capacity and stiffness. The results also show that GFRP strengthened beams without end anchorages experienced intermediate concrete debonding failure at the GFRP plate end, whereas all the GFRP strengthened beams with different end anchorage systems failed in rupture of GFRP with concrete crushing. The theoretical results revealed no significant difference among the relevant design guidelines with regard to the predicted ultimate moment capacities of the bi-directional GFRP strengthened RC beams. However, the results show that ACI Committee 440 Report (2008) design recommendation provides reasonably acceptable predictions for the ultimate moment capacities of the tested beams strengthened externally with bi-directional GFRP reinforcement followed by FIB Bulletin 14 (2001) and eventually by JSCE (1997).

Originality/value – The research work presented in this manuscript is authentic and could contribute to the understanding of the overall behaviour of RC beams strengthened with FRP and different end anchorage systems under flexural loading.

Keywords Ductility, Anchorage systems, Bi-directional glass fibre reinforced polymer, Deflection and flexural effectiveness, Ultimate moment capacity

Paper type Research paper

1. Introduction

A significant number of reinforced concrete (RC) structures are required to be retrofitted due to one or combination of several factors including constructions faults, original design limits, alterations in usage, excessive loading and natural disasters or aggressive environmental conditions. The conventional strengthening techniques such as external steel plate bonding method, section enlargement and external post-tensioning system have been used to extend service life and retrofit the damaged RC structures. However, in recent decades, the application of externally bonded fibre reinforced polymer reinforcement has been an extensively used technique for retrofitting the damaged RC structures due to its potential advantageous

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characteristics that include high strength-to-weight ratio, high stiffness and ease of installation without any external support (Hollaway, 2010; Teng *et al.*, 2002).

Substantial experimental investigations (Anania *et al.*, 2005; Benjeddou *et al.*, 2007; Carlos *et al.*, 2018; Ceroni, 2010; Gao *et al.*, 2005; Hosen *et al.*, 2018; Jayaprakash *et al.*, 2015; Kim and Shin, 2011; Li *et al.*, 2008; Nurbaiah *et al.*, 2010; Rafi *et al.*, 2008; Spadea *et al.*, 2015; Sundarraja and Rajamohan, 2008; Toutanji *et al.*, 2006; Triantafyllou *et al.*, 2017; Yang *et al.*, 2018) have been conducted on the flexural strengthening of RC beams using fibre reinforced polymer composites. These studies have proved that external FRP reinforcement is effective in enhancing the flexural performance of the strengthened RC beams with regard to ultimate strength and stiffness. However, the strengthened beams have encountered different modes of failure that include FRP rupture, crushing of concrete, shear failure, critical diagonal crack (CDC) debonding, concrete cover separation failure, plate end interfacial debonding and intermediate crack (IC) induced interfacial debonding (or IC debonding) (Teng *et al.*, 2001, 2003; Leung and Yang, 2006; Wang and Zhang, 2008; Teng and Chen, 2007; Hollaway and Teng, 2008). It has been noted by researchers (Aram *et al.*, 2008; Bencardino *et al.*, 2002; Chahrour and Soudki, 2005; Fu *et al.*, 2018; Said and Wu, 2008) that RC beams strengthened externally with FRP experienced premature debonding failure of FRP reinforcement from the concrete surface. However, this premature debonding failure mode limits the strengthening capacity of the FRP composites.

The effect of U-jacket anchors on strength and ductility as well as on the general performance of externally reinforced beams was first discussed and clarified in Spadea *et al.* (1998) and Spadea *et al.* (2000). Recently, Pham and Al-Mahaidi (2006) investigated the behaviour and flexural performance of 260 mm × 140 mm RC beams retrofitted with CFRP composites. The CFRP strengthened beams were anchored with unidirectional CFRP U-straps of 209 GPa modulus of elasticity at the CFRP plate ends or at 180 mm spacing along the shear span. The study showed that the addition of U-jacket anchors at the FRP plate end significantly enhanced the efficiency of FRP system by preventing intermediate span and end debonding failures. The result also indicates an excellent performance of the U-jacket anchors when applied at a certain spacing to the shear span of retrofitted RC beams, as the mechanism of IC debonding failure was prevented or delayed. Subsequently, Al-Amery and Al-Mahaidi (2006) studied the coupling shear-flexural retrofitting of RC beams with CFRP U-jackets placed at 200 mm spacing along the span of the beams and tested in four- and three-point bending system. The results have shown that the presence of CFRP U-straps spaced at 200 mm along the span of the beam significantly prevented the debonding of CFRP sheet, and it limits the interfacial slip between CFRP in the beam soffit and the concrete section up to 10 per cent. It was also found that using CFRP U-straps to anchor the flexural CFRP sheets could substantially enhance the flexural strength to 95 per cent in addition to increased ductility.

Duthinh and Starnes (2004) studied the flexural effectiveness of RC beams strengthened in flexure with CFRP of 50 mm width, the thickness of 1.2 mm and elastic modulus of 155 GPa. Out of seven specimens, three were anchored with mechanically fastened steel plates of 203 mm width over the FRP plate ends. A clamping force of 15–25 kN was then applied to two bolts which are torqued to 0.4 kNm. The experimental results have shown that the combination of adhesion and clamping could enhance the FRP anchorage capacity as expected because both diagonal and transverse wraps anchored the flexural CFRP to a strain increase of 58 and 43 per cent of the rupture strain. The result also demonstrated that the improved ultimate strain of the anchored plates was up to 1.14 per cent, i.e. 60 per cent of the rupture strain, as a result of adhesion and clamping. However, the authors also concluded that the mechanism of failure was debonding of CFRP laminate usually initiating from diagonal or transverse shear cracking zone.

A new hybrid system comprising mechanically fastened system and common externally bonded systems was investigated by Wu and Huang (2008). The experimental program consisted of beam specimens strengthened with two, four and six layers of CFRP strips. The hybrid system failed by two apparent failure patterns, namely, the CFRP rupture at mid-span in beams strengthened with two and four layers of FRP strips, and the total debonding of CFRP strips which occurred in beams strengthened with six layers of CFRP strips. The results showed that beams mechanically fastened with four and six layers of FRP sheets exhibit higher flexural strength than the beams strengthened with two layers of CFRP and no fasteners. The authors further concluded that the use of hybrid plate bonding system could significantly improve the flexural capacity and bond strength in addition to the moment of resistance enhancement.

In recent decades, different anchorage systems including U-jacket anchors, mechanically fastened metallic anchors and FRP anchors in order to enhance the efficiency and to prevent the premature debonding of FRP reinforcement have been investigated (Chahrour and Soudki, 2005; Leung, 2002; Smith, 2010; Lam and Teng, 2001; Zhang and Smith, 2012; Yalim *et al.*, 2008; Smith and Teng, 2003; Buyle-Bodin and David, 2004). It has been proved that the anchoring systems could enhance the load carrying capacity and ductility of RC beams strengthened with FRP composites. Moreover, it was found that the end anchorages could provide an efficient load transfer between concrete and the bonded FRP reinforcement in addition to FRP strain level enhancement prior to failure (Grelle and Sneed, 2013; Kalfat and Smith, 2013; Baggio *et al.*, 2014). In due course, the literature review reveals that the number of experimental results of beams with anchorages is not extensively investigated and validated with the different existing codes including ACI Committee 440 Report (2008), FIB Bulletin 14 (2001) and JSCE (1997). RC beams strengthened in flexure with bi-directional glass fibre reinforced polymer (GFRP) composites and plate end anchorages are considered and reported in this study. The end anchored flexural bi-directional GFRP sheets are placed at 150 mm away from the beam support in order to avert end-of-plate failure.

In this paper, the influence of the number of bi-directional GFRP layers and different end anchorages on flexural capacity of strengthened RC beams is investigated. The investigation covers two parts. The first part includes a detailed experimental investigation to study the influence of different end anchorage systems on the ultimate load carrying capacity and failure mechanism of RC beams strengthened externally with bi-directional GFRP reinforcement. A comparison of experimental and theoretical results using three different design guidelines predictions (i.e. ACI Committee 440 Report, 2008; JSCE, 1997; FIB Bulletin 14, 2001) is presented in the second part of the research study.

2. Experimental investigation

2.1 Preparation of RC beams and material properties reluctant

A total of nine RC beams were cast in concrete mixing laboratory with a targeted concrete grade of 20 N/mm^2. All these beams were of 1,600 mm span length and a cross-section dimension of 100 mm × 200 mm. All these beams were designed to fail in flexure using Eurocode 2: design guideline for concrete structures (BS EN 1992-1-1, 2004). These beams were reinforced with a longitudinal compression reinforcement of 2–10 mm in diameter and a tensile reinforcement of 2–12 mm in diameter. The beams were firmly reinforced in the shear zones to prevent shear failure. The steel stirrups at shear and flexural zones were placed with 6 mm diameter at 50 and 100 mm centre to centre, respectively.

2.1.1 Concrete. The concrete mix was prepared in accordance with BS 1881 Part 125 (1986) design guidelines, in order to achieve the targeted compressive strength of 20 N/mm^2 in 28 days. The mix proportion by weight of cement:fine aggregate:coarse aggregate:water was 1:1.38:2.42:0.50, respectively. Ordinary Portland cement, sand, crushed coarse aggregates of 20 mm maximum size and potable water were used to prepare the concrete mix at Concrete Mixing Laboratory, University of Nottingham, Malaysia Campus. A total of nine beams were cast with four different batches of concrete and were cured for 28 days before testing. Four concrete cube specimens of 100 mm × 100 mm × 100 mm size were also prepared for each batch to determine the compressive strength of concrete cube at the age of 28 days. The results of compressive strength of concrete cubes are presented in Table I.

2.1.2 Steel reinforcement. The average tensile strength of 12 and 10 mm steel rebar was of 460.5 and 251.82 N/mm^2, respectively. Figure 1 portrays the reinforcement details of the RC beams.

2.1.3 FRP composites. All the beams were reinforced externally with bi-directional GFRP reinforcement with two-component epoxy resin. As recommended in the manufacturer's manual, the bi-directional GFRP sheet was applied using the manual wet layup technique with a resin and hardener mix ratio of 1:2. The material properties of bi-directional GFRP reinforcement and epoxy resin are presented in Table II.

2.2 Specimen description

The first specimen was retained as control beam (CB) with no external bi-directional GFRP reinforcement which is labelled as CB, and the remaining eight specimens were divided into four series. Two beams, within each series, were bonded externally with one and two layers of bi-directional GFRP reinforcement along the soffit of the beams. The specimens in first (i.e. FSB-CA1 and FSB-CA2) and second (i.e. FSB-UA1 and FSB-UA2) series used closed and

Table I.
Results of compressive and tensile strength of concrete

Specimen	Compressive strength f'_c (N/mm^2)	Average compressive strength of concrete $f'_{c,\text{ average}}$ (N/mm^2)	Average tensile strength of concrete $f_{ct} = 0.7\sqrt{f'_{c,\text{ ave}}}$ (N/mm^2)
CB	27.56	25.39	3.53
	24.55		
	29.75		
	19.71		
FSB-UA1	24.83	28.40	3.73
FSB-UA2	32.97		
FSB-CA2	32.51		
	23.29		
FSB-SA2	24.69	25.66	3.55
FSB-CA1	23.61		
	23.99		
	30.36		
FSB-C1	21.42	23.69	3.41
FSB-C2	20.48		
FSB-SA1	21.42		
	31.45		

Note: f_{ct} = concrete tensile strength (ACI Committee 318, 2008)

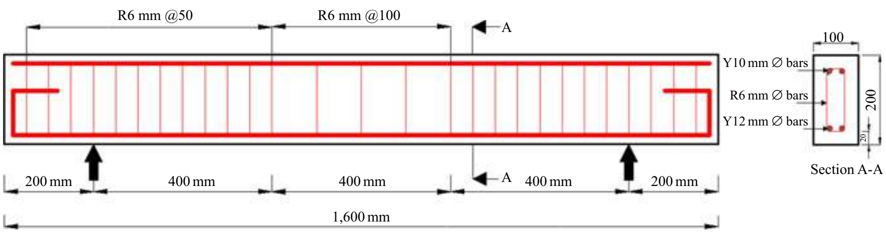


Figure 1.
Reinforcement details of RC beams

U-wrap anchorages with two layers of bi-directional GFRP (i.e. 100 mm width) strip bands at the ends of flexural reinforcement, respectively. The third series, labelled as FSB-SA1 and FSB-SA2, were anchored with 100 mm × 100 mm × 2 mm steel plate at the ends of GFRP reinforcement, and the specimens in the fourth series with one and two layers of flexural bi-directional GFRP reinforcement were designated as FSB-C1 and FSB-C2 with no plate end anchor, respectively. The outline of the test beams is presented in Table III and illustrated in Figure 2(a)–(d).

2.3 Test procedure

A schematic diagram of the experimental set-up of the RC beam is shown in Figure 3. All the beam specimens were subjected to four-point bending test. Prior to testing, the beam specimens were painted white for ease of identification of cracks. A 30-tonne capacity testing frame was used to perform the four-point bending test. The load was applied using a hydraulic jack at equal interval until failure. A dial gauge was placed at the centre of the beam to measure the mid-span deflection. The crack patterns of the beam specimens at different stages of loading were observed.

3. Results and discussion

3.1 Load-deflection curve

3.1.1 Control beam (CB). The applied load-deflection behaviour of the CB is illustrated in Figure 4. The curve shows a tri-linear response which is a typical behaviour of RC beams with no external GFRP reinforcement. Upon loading, the early stage of the deflection curve shows a region with relatively high stiffness prior to the formation of flexural cracks. The flexural cracks, in the bending zone, occurred at an applied load of 22.5 kN; however, at later stages, the stiffness of the deflection curve decreases as the concrete cracks and this region exhibits the post-cracking behaviour of the beam. The stiffness of the curve continues to decrease with an increase in the applied load up to the yielding of

Materials	Properties	
GFRP (e-glass woven fabric) (EWR 600-100)	Tensile strength (N/mm^2)	3,850
	Modulus of elasticity (N/mm^2)	70,000
	Rupture strain (mm/mm)	0.055
	Thickness (mm)	0.6
Epoxy resin	Tensile strength (kg/cm^2)	800±50
	Flexural strength (kg/cm^2)	375±50

Table II. Properties of bi-directional GFRP fabric and epoxy resin as per manufacturer's manual

Series	Specimen designation	Average compressive strength of concrete	Number of GFRP layers	Types of anchorage
–	CB	25.39	–	–
1st	FSB-CA1	25.66	1	GFRP closed-wrap
	FSB-CA2	28.40	2	
2nd	FSB-UA1	28.40	1	GFRP U-wrap
	FSB-UA2	28.40	2	
3rd	FSB-SA1	23.69	1	Fastened with steel plate and blot
	FSB-SA2	25.66	2	
4th	FSB-C1	23.69	1	–
	FSB-C2	23.69	2	–

Table III. Outline of the test beams

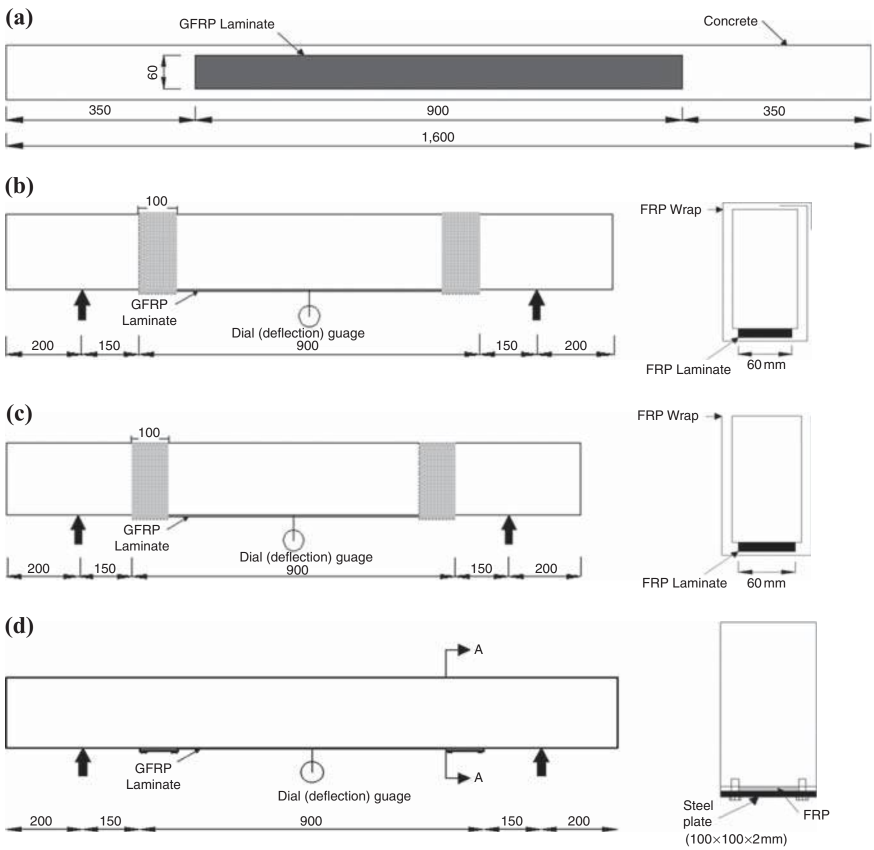


Figure 2.
Detail of beam strengthening scheme

Notes: (a) Soffit view of bi-directional GFRP strengthened beam specimen; (b) first series beam specimen; (c) second series beam specimen; (d) third series beam specimen

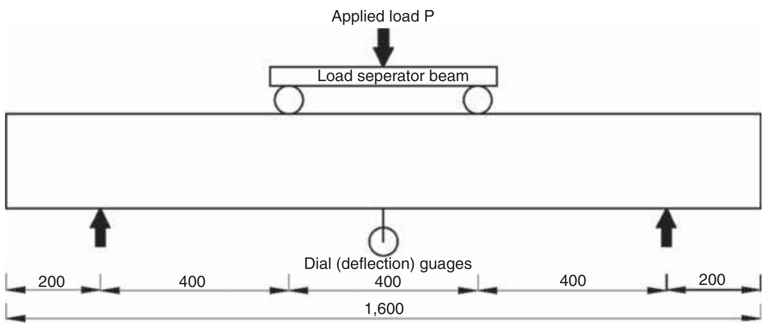


Figure 3.
Experimental set-up of reinforced concrete beam

steel reinforcement. At last, the beam failed in flexure due to the yielding of steel reinforcement at a peak load of 101.05 kN and a maximum deflection of 44.81 mm. Moreover, the beam achieved a ductile behaviour beyond yield point up to the failure load. Plate 1 shows the flexural failure pattern of CB.

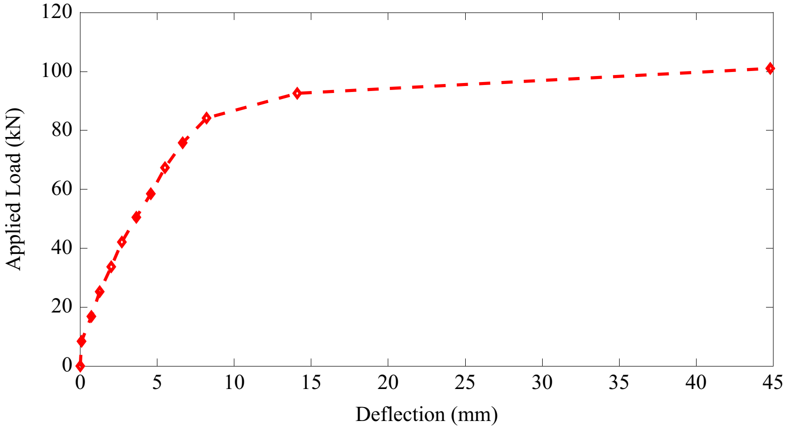


Figure 4. Load-deflection behaviour of control beam (CB)

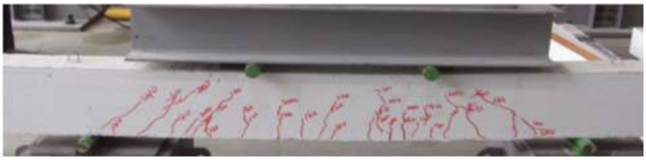


Plate 1. Flexural failure pattern of control beam (CB)

First series. The first series consisted of two beams (i.e. FSB-CA1 and FSB-CA2) strengthened externally with one and two layers of bi-directional GFRP reinforcement and these beams were anchored with two layers of closed GFRP strips with a width of 100 mm. The applied load-deflection behaviour of the control and strengthened beams is shown in Figure 5. The ductility of bi-directional GFRP strengthened beams with closed GFRP anchorage strips was relatively less as compared to the CB. These specimens FSB-CA1 and FSB-CA2 exhibited similar stiffness trend before yielding of steel reinforcement despite the fact that the beams were bonded with different GFRP layers and identical anchoring system.

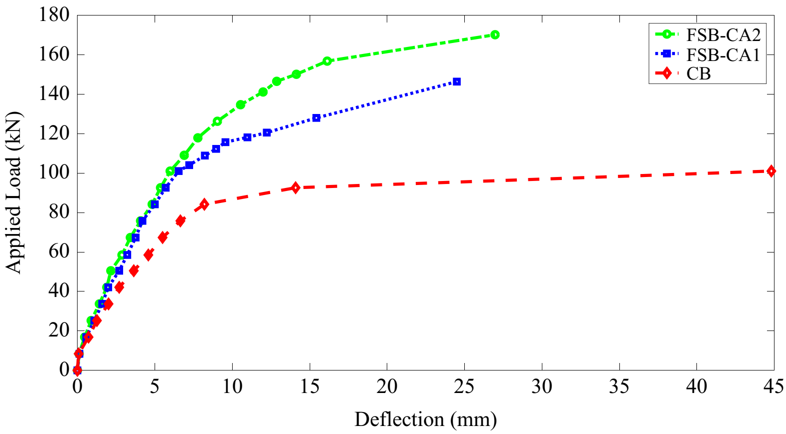


Figure 5. Load-deflection behaviour of beams FSB-CA1, FSB-CA2 and CB

From Figure 5, it should be noted that the loss of ductility of the beam FSB-CA2 is relevant compared both with that of the CB and with the beam FSB-CA1. This result shows that the stiffness of GFRP strengthened beam could be increased by increasing the thickness of GFRP reinforcement. However, it might not increase the ductility of the beam. Moreover, the beam FSB-CA2 with two layers of GFRP reinforcement attained better performance in terms of flexural capacity as compared to the beam FSB-CA1 with one layer of GFRP reinforcement. The flexural cracks in these GFRP strengthened beams were observed at a load of 42.1 kN. These FSB-CA1 and FSB-CA2 beams were failed in flexure with the rupture of GFRP reinforcement and followed by the crushing of concrete at a peak load of 146.4 and 170.18 kN, respectively. The maximum deflection at failure for FSB-CA1 and FSB-CA2 was 24.50 and 26.97 mm, respectively. No debonding failure was observed in the anchorage region of the tested beams. Plate 2 portrays the failure pattern of bi-directional GFRP strengthened beams FSB-CA1 and FSB-CA2.

Second series. Beams in the second series, designated as FSB-UA1 and FSB-UA2 (i.e. bonded with one and two layers of GFRP reinforcement), were anchored with two layers of bi-directional GFRP U-strips of 100 mm width. The load-deflection curve for FSB-UA1, FSB-UA2 and CB beams is shown in Figure 6. As seen in the first series beams, the deflection curve of beams FSB-UA1 and FSB-UA2 experienced less ductile behaviour as compared to the CB. It also demonstrated a tri-linear response (i.e. bounded by three different stages). The first stage is the region with higher stiffness before first cracking occurs; the second part is the post-cracking stage after the formation of crack with a decrease in flexural stiffness; the third stage represents a region where a loss in stiffness occurred after the yielding of internal steel reinforcement up to failure. The deflection curves of beams FSB-UA1 and FSB-UA2 experienced similar stiffness trend till the steel yielding stage. However, prior to failure, higher performance in flexural stiffness was observed in the FSB-UA2 beam as compared to the beam FSB-UA1 which was due to the effect of the number of GFRP layers. The flexural cracks in both FSB-UA1 and FSB-UA2 occurred at the

(a)

(b)

Plate 2.
Failure pattern of the first series beams with one and two layers of bi-directional GFRP reinforcement

Notes: (a) Rupture of GFRP with the concrete crushing failure of beam FSB-CA1; (b) rupture of GFRP with the concrete crushing failure of beam FSB-CA2

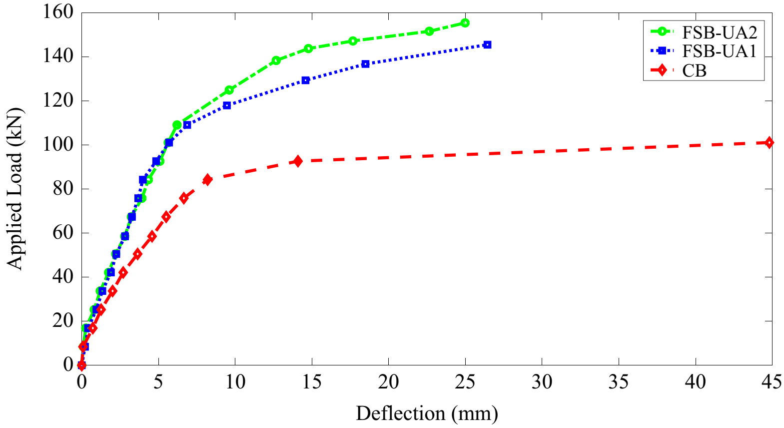


Figure 6. Load-deflection behaviour of beams FSB-UA1, FSB-UA2 and CB

constant bending zone of the beam. Upon loading, the flexural cracks of these beams occurred at a load of 42.1 kN. The failure of FSB-UA1 and FSB-UA2 beams occurred when the bi-directional GFRP reinforcement ruptured in the constant moment region of the beams at a peak load of 145.45 and 155.35 kN, with the corresponding maximum deflection of 26.44 and 24.98 mm, respectively. Similar to beams in the first series with closed GFRP anchorages, no debonding failure was observed at the anchorage zones of the beams in the second series. The failure pattern of beams FSB-UA1 and FSB-UA2 is shown in Plate 3.

Third series. The third series comprised of two beams designated as FSB-SA1 and FSB-SA2 strengthened in flexure with one and two layers of GFRP reinforcement and anchored with 100 mm × 100 mm × 2 mm steel plate at the ends of flexural GFRP reinforcement. Figure 7 portrays the load-deflection behaviour of the GFRP strengthened (i.e. third series) and CBs. The curves for both FSB-SA1 and FSB-SA2 also exhibited tri-linear behaviour as seen in the first and second series beams. However, as compared to the CB, these beams also experienced less ductile behaviour. From Figure 7, it is evident that the beam

Notes: (a) Rupture of GFRP with the concrete crushing failure of beam FSB-UA1; (b) rupture of GFRP with the concrete crushing failure of beam FSB-UA2

Plate 3. Failure pattern of the second series beams with one and two layers of bi-directional GFRP reinforcement

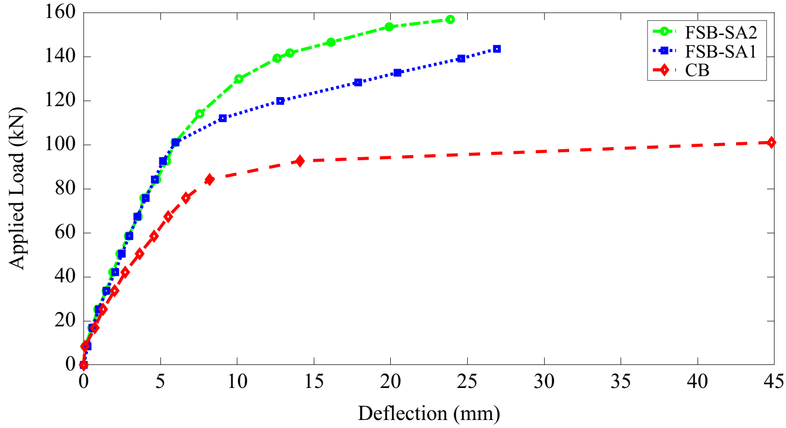
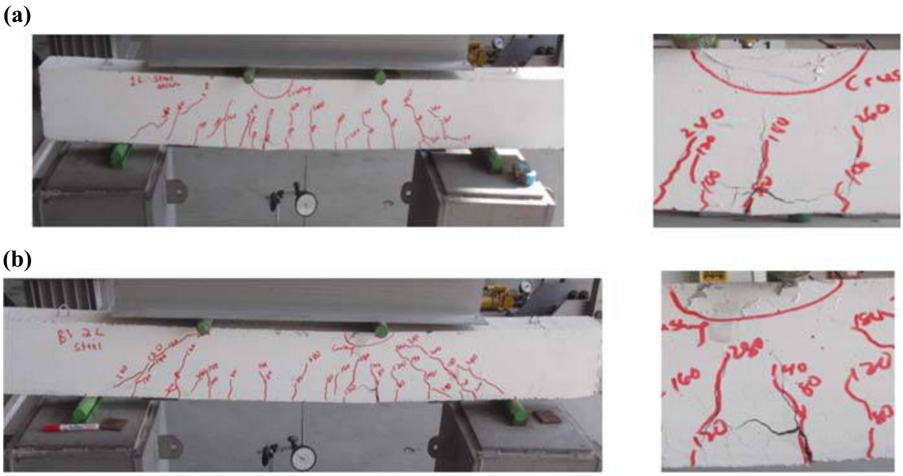


Figure 7.
Load-deflection behaviour of beams FSB-SA1, FSB-SA2 and CB

FSB-SA2 with two layers of GFRP reinforcement exhibited better performance in terms of flexural stiffness over the beam FSB-SA1 having one layer of GFRP reinforcement. Flexural cracks began to occur at a load of 33.7 and 42.1 kN for FSB-SA1 and FSB-SA2 beams, respectively. The beams failed in GFRP rupture at the constant moment region as the applied load attained a peak value of 143.55 and 156.90 kN with the corresponding maximum deflection values of 26.92 and 23.88 mm for the beams FSB-SA1 and FSB-SA2, respectively. However, no failure was observed in the region of steel plate anchors in both the specimens in the third series. The typical modes of failure of beams in the third series are shown in Plate 4.

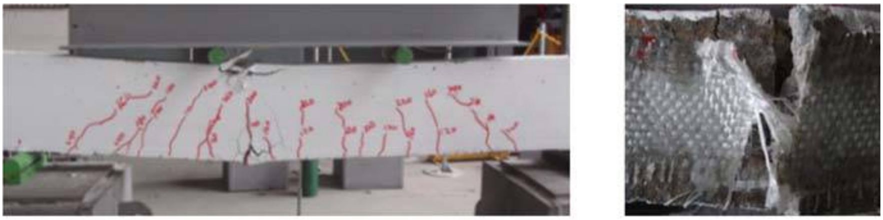
Fourth series. The fourth series beams, designated as FSB-C1 and FSB-C2, were strengthened with one and two layers of GFRP along the soffit surface with no anchorages. The beams in this series are considered as CBs for the first, second and third series beams to evaluate the effect of different anchorage systems on the GFRP strengthened beams. Plate 5 illustrates the typical rupture of GFRP with IC debonding failure of strengthened beams in



Notes: (a) Rupture of GFRP with the concrete crushing failure of beam FSB-SA1; (b) rupture of GFRP with the concrete crushing failure of beam FSB-SA2

Plate 4.
Failure pattern of the third series beams with one and two layers of bi-directional GFRP reinforcement

(a)



(b)

Notes: (a) Rupture of GFRP with intermediate concrete (IC) debonding failure of beam FSB-C1; (b) rupture of GFRP with intermediate concrete (IC) debonding failure of beam FSB-C2

Plate 5.
Failure pattern of the fourth series beams with one and two layers of bi-directional GFRP reinforcement

this series. The load-deflection behaviour for beams in the fourth series is shown in Figure 8. The curves represent a less ductile behaviour for beams FSB-C1 and FSB-C2 between steel yielding and failure as compared to that of the CB. Beam FSB-C2 bonded with two layers of GFRP reinforcement slightly achieved better performance in flexural stiffness as compared to the beam FSB-C1. Upon further loading, the flexural cracks, in beams FSB-C1 and FSB-C2, were initiated between the constant bending moment region at a load of 42.1 and 33.7 kN, respectively. The beams FSB-C1 and FSB-C2 failed in the rupture of GFRP with IC debonding at a peak load of 137.15 and 149.55 kN with the corresponding maximum deflection values of 28.16 and 27.89 mm. From Figure 9 it is clear that the GFRP strengthened beams with additional GFRP closed end anchorages, sustained higher loads and achieved excellent performance regarding flexural capacity and stiffness than the GFRP strengthened beams without end anchorages. The experimental results are summarised in Table IV.

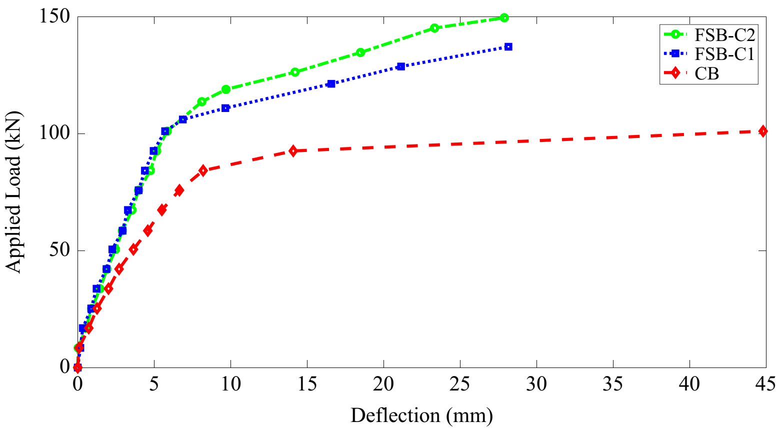


Figure 8.
Load-deflection behaviour of beams FSB-C1, FSB-C2 and CB

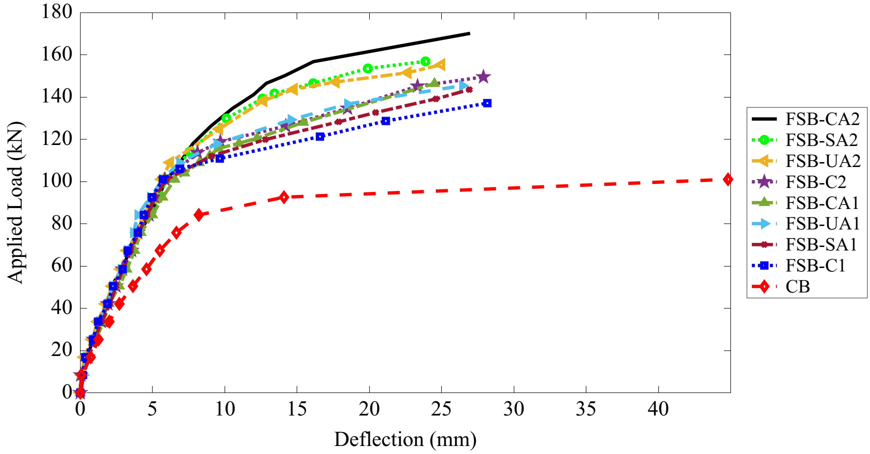


Figure 9.
Load-deflection behaviour of all the strengthened beams and the control beam

Table IV.
Summary of experimental test results

Specimen	Yield load P_y (kN)	Mid-span deflection at yield load δ_y (mm)	Failure load P_u (kN)	Mid-span deflection at failure load δu (mm)	Percentage P_u increase over the control beam (%)	Ductility index $\mu_\delta = (\delta_u)/(\delta_y)$	Modes of failure
CB	84.20	8.20	101.05	44.81	–	5.46	FF
FSB-CA1	115.70	9.56	146.40	24.50	45	2.56	R-CC
FSB-CA2	134.70	10.54	170.18	26.97	68	2.56	R-CC
FSB-UA1	117.90	9.46	145.45	26.44	44	2.79	R-CC
FSB-UA2	124.90	9.61	155.35	24.98	54	2.60	R-CC
FSB-SA1	112.05	9.06	143.55	26.92	42	2.97	R-CC
FSB-SA2	129.90	10.11	156.90	23.88	55	2.36	R-CC
FSB-C1	110.90	9.65	137.15	28.16	36	2.92	R-IC
FSB-C2	118.90	9.69	149.55	27.89	48	2.88	R-IC

Notes: FF-C, flexural failure; R-CC, rupture of GFRP reinforcement with crushing of concrete; Y-IC, rupture of GFRP with IC debonding failure

3.2 Ultimate moment capacity

The percentage increase in the ultimate moment capacity of all the bi-directional GFRP strengthened beams over the CB is presented in Table V. The experimental results have shown that the externally strengthened RC beams with different end anchorages were effective in enhancing the flexural capacity by 36–68 per cent over the CB. From Figure 10, it is clear that the beam FSB-UA1 with one layer of GFRP reinforcement and U-GFRP strips end anchor achieved 44 per cent increase in ultimate moment capacity, higher than the beam FSB-C1 that gave a moment capacity increase of 36 per cent over the CB. The U-GFRP strip anchor in beam FSB-UA1 was significant because the beam exhibited a 6 per cent increase in ultimate moment capacity over beam FSC-C1 of the same level of flexural strengthening. However, beam FSB-CA2 with closed GFRP strip anchorage achieved 68 per cent increase in ultimate moment capacity over the CB, and it is higher than all the strengthened beams. By comparing the first (FSB-CA1 and FSB-CA2) and fourth (FSB-C1 and FSB-C2) series beams, it is clearly seen that the presence of closed GFRP strip end anchorage system in the first series beams significantly improved the flexural moment capacity of the beams by 6.7 and 13.8 per cent over the beams FSB-C1 and FSB-C2, respectively. It is also clear from Figure 10 that beams FSB-SA1 and FSB-SA2 achieved an ultimate moment capacity increase of 4.7 and 4.9 per cent over beams FSB-C1 and FSB-C2. This might be attributed to the metallic rigidity of the steel plate anchorage system in beam FSB-SA1 and FSB-SA2. The experimental results show that the addition of end anchorages of steel plate or GFRP strips to RC beams strengthened in flexure enhanced the ultimate moment capacity of the beams. However, the moment capacity increase for the GFRP strengthened beams was observed to be within the strength increase of up to 40 per cent as recommended by ACI Committee 440 Report (2008).

3.3 Ductility

Ductility of RC beam can be defined as its ability to deform under loading prior to total collapse without loss in ultimate load carrying capacity (Spadea *et al.*, 2015).

The ductility of the investigated GFRP strengthened beams decreased as compared to that of the CB. Ductility is determined by considering the deflection or curvature of the beam. This study only focused on deflection ductility (μ_δ). The deflection ductility is defined as the ratio of ultimate deflection (δ_y) to yield deflection (δ_u), i.e.:

$$\mu_\delta = \frac{\delta_u}{\delta_y}. \tag{1}$$

Specimen	Experimental ultimate moment capacity (M_u) (kNm)	Percentage increase over control beam (%)	Percentage increase over FSB-C1 (%)	Percentage increase over FSB-C2 (%)
CB	20.21	–	–	–
FSB-CA1	29.28	45	6.7	
FSB-CA2	34.04	68	–	13.8
FSB-UA1	29.09	44	6.1	
FSB-UA2	31.07	54	–	3.9
FSB-SA1	28.71	42	4.7	
FSB-SA2	31.38	55	–	4.9
FSB-C1	27.43	36	–	–
FSB-C2	29.91	48	–	–

Table V. Ultimate moment capacities of GFRP strengthened and control beams

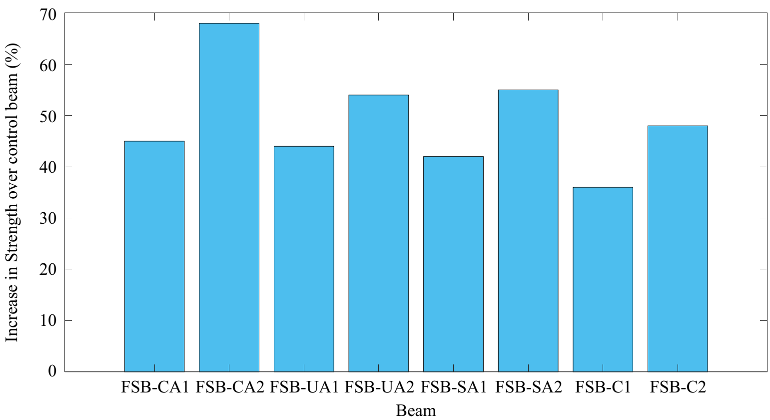


Figure 10. Ultimate moment capacity increase (%) over control beam

Table VI presents the calculated ductility index (μ_δ) and the ductility ratios of all the bi-directional GFRP strengthened beams to that of the CB. These results confirmed that strengthening of RC beams with externally bonded bi-directional GFRP reinforcement resulted in significant loss in ductility of the strengthened beam. The deflection ductility ratio of all the strengthened beams was found to be 43–54 per cent of that of the original CB. It is also observed that the ductility ratio of GFRP strengthened beams with anchorages, except the beam FSB-SA1, was relatively lower than that of GFRP strengthened beams without anchorages. The decrease in ductility was because the anchorage system at the ends controls the bond slip between the concrete and flexural GFRP reinforcement. Beam FSB-SA1 experienced the largest ductility index of 2.97. This indicates that the presence of steel anchorage has an insignificant effect on the ductility of the tested beams and results in only 2 per cent increase in ductility index over beam FSB-C1. Figure 11 illustrates the ductility ratios of the bi-directional GFRP strengthened and the CBs.

4. Theoretical evaluation

According to ACI Committee 440 Report (2008), the ultimate moment resistance of RC beams strengthened with FRP reinforcement can be determined using strain compatibility method and equilibrium equation. Figure 12 shows the internal stress, and strain distribution of a

Table VI.
Results of deflection, ductility index and ductility ratio

Specimen	Mid-span deflection at yield load δ_y (mm)	Mid-span deflection at failure load δ_u (mm)	Ductility index $\mu_\delta = (\delta_u)/(\delta_y)$	Ductility ratio $\mu_\delta/\mu_{\delta\text{control}}$
CB	8.20	44.81	5.46	1.00
FSB-CA1	9.56	24.50	2.56	0.47
FSB-CA2	10.54	26.97	2.56	0.47
FSB-UA1	9.46	26.44	2.79	0.51
FSB-UA2	9.61	24.98	2.60	0.48
FSB-SA1	9.06	26.92	2.97	0.54
FSB-SA2	10.11	23.88	2.36	0.43
FSB-C1	9.65	28.16	2.92	0.53
FSB-C2	9.69	27.89	2.88	0.53

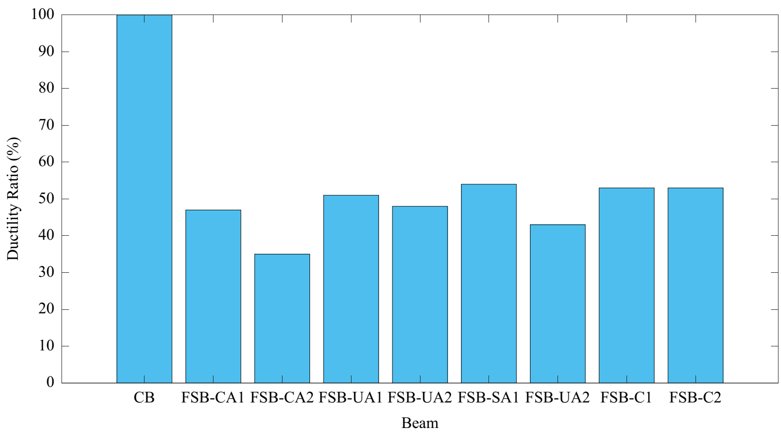
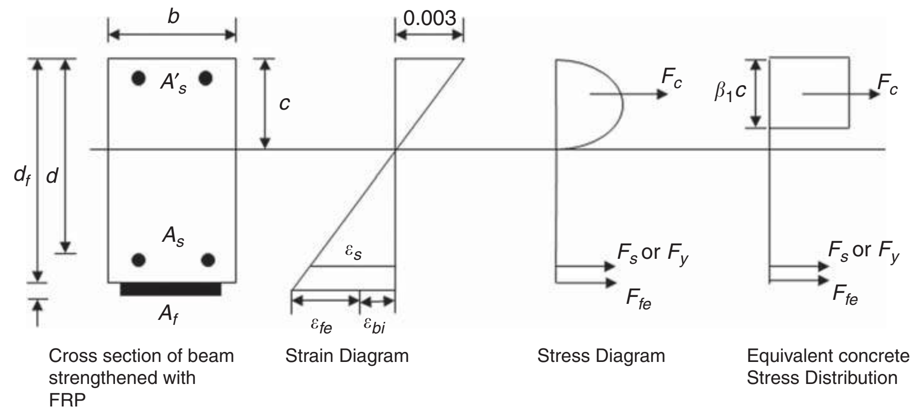


Figure 11.
Ductility ratio of control and GFRP strengthened beams



Source: ACI Committee 440 Report (2008)

Figure 12. Internal strain and stress distribution for a beam section under flexure at ultimate limit state

RC beam section strengthened in flexure with FRP. The ultimate moment resistance (M_u) of the section can be computed using the following equation:

$$M_u = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi_f A_f \varepsilon_{\mathrm{fd}} E_f\left(h - \frac{\beta_1 c}{2}\right) + A'_s f'_s\left(\frac{\beta_1 c}{2} - d'\right), \tag{2}$$

where A_f is the area of FRP reinforcement; A_s is the cross-sectional area of tension steel reinforcement; A'_s is the cross-sectional area of compression steel reinforcement; c is the extreme distance from compression fibre to the neutral axis; d is the distance from extreme compression fibre to the centroid of tensile reinforcement; d' is the depth of compression steel; E_f is the elastic modulus of FRP reinforcement; f_s is the stress in tensile reinforcement; f'_s is the stress in compression steel reinforcement; h is the overall depth of beam; $\beta_1$is the ratio of the depth of equivalent rectangular stress block to a depth of the neutral axis; ψ_f is the fibre reduction factor taken as 0.85 recommended by ACI Committee 440 Report (2008) for flexural RC members with FRP external reinforcement to account for FRP uncertainties.

The ultimate theoretical moment of resistance (M_u) for all the bi-directional GFRP strengthened beams is computed using Equation (2) by considering the debonding strain ($\varepsilon_{\mathrm{fd}}$) adopted by the relevant design guidelines.

4.1 ACI 440.2R-08 design guide

The ACI Committee 440 Report (2008) proposed a design equation for predicting the debonding strain ($\varepsilon_{\mathrm{fd}}$) to account for the premature IC debonding failure of the FRP plates. The debonding strain can be computed using the equation given below:

$$\varepsilon_{\mathrm{fd}} = 0.41\sqrt{\frac{f'_c}{nE_{\mathrm{frp}} t_{\mathrm{frp}}}} \leqslant 0.9\varepsilon_{\mathrm{fu}}, \tag{3}$$

where f'_c is the compressive strength of concrete; $\varepsilon_{\mathrm{fu}}$ is the rupture strain in FRP; t_{frp} is the thickness of FRP strip; and n is the number of FRP layers.

4.2 FIB Bulletin 14 (2001) recommendations

The design guidelines of FIB Bulletin 14 (2001) give a design formula for predicting the total debonding strain base on fracture mechanics approach. The debonding strain is predicted as follows:

$$\varepsilon_{\mathrm{fd}} = \alpha c_1 k_c k_b \sqrt{\frac{f_{\mathrm{ct}}}{nE_f t_f}}, \quad \text{for } l_b \geqslant l_{b,\,\max}, \tag{4a}$$

$$\varepsilon_{\mathrm{fd}} = \alpha c_1 k_c k_b \sqrt{\frac{f_{\mathrm{ct}}}{nE_f t_f l_{b,\,\max}} \frac{l_b}{}} \left(2 - \frac{l_b}{l_{b,\,\max}}\right), \quad \text{for } l_b < l_{b,\,\max}, \tag{4b}$$

$$l_{b,\,\max} = \sqrt{\frac{nE_f t_f}{c_2 f_{\mathrm{ct}}}}, \tag{4c}$$

$$k_b = 1.06 \sqrt{\frac{2-(b_f/b)}{1+(b_f/400)}} \geqslant 1, \tag{4d}$$

where α is the reduction factor approximately equal to 0.9, to account for the influence of inclined cracks on the bond strength ($\alpha = 1$ for beams with sufficient internal and external shear reinforcement and in slab); c_1 and c_2 can be obtained through calibrations with test results assumed to be 0.64 and 2; k_c is a factor accounting for the state of compaction of concrete (k_c can generally be assumed to be equal to 1; however, FRP bonded to concrete faces with low compaction, e.g. faces not in contact with formwork during casting, $k_c = 0.64$); and k_b is a geometry factor and is given in Equation (4d); b_f is the width of FRP laminate, with $(b_f)/(b) \geqslant 0.33$.

4.3 Japan Society of Civil Engineers (JSCE) design guidelines

The JSCE (1997) recommends a design equation for predicting the total debonding strain ($\varepsilon_{\mathrm{fd}}$) of continuous FRP sheets by interfacial fracture energy. According to JSCE, the debonding strain can be calculated from the following equation:

$$\varepsilon_{\mathrm{fd}} = \sqrt{\frac{2G_f}{nE_f t_f}}, \tag{5}$$

where G_f is the interfacial fracture energy between FRP laminate and concrete, and its value is assumed to be 0.5 N/mm.

5. Comparison of experimental and theoretical results

Table VII presents the comparison of experimental and predicted results of three design guidelines for FRP strengthened beams. The results show that ACI Committee 440 Report (2008) gave a mean ratio of experimental to the predicted ultimate moment capacities of 1.43 and a corresponding variation coefficient of 3.9 per cent. The mean ratios of the experimental to predicted ultimate moment capacities of bi-directional GFRP strengthened beams using FIB Bulletin 14 (2001) and JSCE (1997) were found to be 1.53 and 1.59 with a variance coefficient of 4.5 and 5.8 per cent, respectively. The comparison of results indicates no significant difference between the relevant design guidelines with regard to the predicted ultimate moment capacities of the bi-directional GFRP strengthened RC tested beams with and without end anchorages. However, the results indicate that ACI Committee 440 Report (2008) design recommendation provides

Specimen	$M_{u\ exp}$ (kNm)	ACI $M_{u\ ACI}$ (kNm)	ACI $(M_{u\ exp})/(M_{u\ ACI})$	JSCE $M_{u\ JSCE}$ (kNm)	JSCE $(M_{u\ exp})/(M_{u\ JSCE})$	FIB $M_{u\ FIB}$ (kNm)	FIB $(M_{u\ exp})/(M_{u\ FIB})$
FSB-C1	27.43	20.10	1.36	18.29	1.50	18.99	1.44
FSB-C2	29.91	21.55	1.39	19.05	1.57	20.02	1.49
FSB-CA1	29.28	20.24	1.45	18.32	1.60	19.10	1.53
FSB-CA2	34.04	21.92	1.55	19.14	1.78	20.31	1.68
FSB-UA1	29.09	20.68	1.41	18.36	1.58	19.22	1.51
FSB-UA2	31.07	21.92	1.42	20.96	1.48	20.31	1.53
FSB-SA1	28.71	20.10	1.43	18.29	1.57	18.99	1.51
FSB-SA2	31.38	21.83	1.44	19.10	1.64	20.15	1.56
Mean			1.43		1.59		1.53
SD			0.06		0.09		0.07
Variance coefficient (%)			3.9		5.8		4.5

Table VII. Comparison of experimental and theoretical predictions

reasonably acceptable predictions for the ultimate moment capacities of the tested beams strengthened externally with bi-directional GFRP reinforcement followed by FIB Bulletin 14 (2001) and eventually by JSCE (1997).

6. Conclusions

The experimental results of the effects of anchorages (i.e. U-shaped, closed and steel plate anchors) on RC beam strengthened in flexure with the bi-directional GFRP were presented. A comparison was made between the experimental results and theoretical predictions based on ACI Committee 440 Report (2008), JSCE (1997) and FIB Bulletin 14 (2001) design recommendations. The following findings can be summarised as follows:

(1) The flexural bi-directional GFRP strengthening of RC beams with different end anchorages (i.e. U-shaped, closed and steel plate anchors) was found to be effective for enhancing the flexural effectiveness of the beams in terms of stiffness and ultimate moment capacity (36–68 per cent).

(2) The unanchored GFRP strengthened beams were failed in GFRP rupture with IC debonding failure whereas all the GFRP strengthened beams anchored at the GFRP plate ends failed in rupture of GFRP reinforcement with concrete crushing.

(3) Experimental results confirm that the bi-directional GFRP strengthened beams with closed GFRP strip end anchors exhibited the highest performance in ultimate moment capacity of 45–68 per cent of the CB followed by two layers GFRP flexural strengthened beam with steel plate anchor archiving an ultimate capacity increase of 55 per cent.

(4) The ductility of all the strengthened beams with or without end anchorages was found to be 34–54 per cent of that of the original CB. This evidently indicates that strengthening of RC beams with externally bonded bi-directional GFRP reinforcement and end anchorages resulted in significant loss of structural ductility of the strengthened beam.

(5) The results of the comparison of experimental and theoretical predictions show that ACI Committee 440 Report (2008) design recommendation provides reasonably acceptable predictions for the ultimate moment capacities of the tested beams strengthened externally with bi-directional GFRP reinforcement followed by FIB Bulletin 14 (2001) and eventually by JSCE (1997).

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Corresponding author
Jayaprakash Jaganathan can be contacted at: jayaprakash.j@vit.ac.in