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Ductility of carbon fiber-reinforced polymer (CFRP) strengthened reinforced concrete beams: Experimental investigation

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Abstract. Strength of reinforced concrete beams can easily be increased by the use of externally bonded CFRP composites. However, the mode of failure of CFRP strengthened beam is usually brittle due to tension-shear failure in the concrete substrate or bond failure near the CFRP-Concrete interface. In order to improve the ductility of CFRP strengthened concrete beams, critical variables need to be investigated. This experimental and analytical research focused on a series of reinforced concrete beams strengthened with CFRP composites to enhance the flexural capacity and ductility. The main variables were the amount of CFRP composites, the amount of longitudinal and shear reinforcement, and the effect of CFRP end diagonal anchorage system. Sixteen full-scale beams were investigated. A new design guideline was proposed according to the effects of the above-mentioned variables. The experimental and analytical results were found to be in good agreement.

Key words: CFRP; CFRP anchorage system; flexural strength; ductility.

1. Introduction

Some of the existing concrete bridges are in need of upgrade due to increase in traffic loads, aging and deterioration. Several traditional strengthening methods have been used to strengthen reinforced and prestressed concrete bridges; for example, external post-tensioning, externally bonded steel plate, enlarging section size, and addition of extra structural elements.

CFRP composite systems are effective, alternative methods for strengthening existing reinforced and prestressed concrete bridges, as compared to conventional strengthening systems used for bridge retrofits. A CFRP composite system has several advantages; corrosion resistance, light weight, easy installation, high strength to weight ratio, and low maintenance cost. In addition, CFRP systems could be used for enhancing the performance of concrete bridges under traffic loading.

Many researchers have investigated the behavior of CFRP strengthened reinforced concrete beams. Arduini and Nanni (1994), Bencardino *et al*. (2002), Matthys and Taerwe (2000) presented the modes of failure of CFRP strengthening reinforced concrete beams. The most common failure mode was the premature debonding between concrete surfaces and CFRP composites. This failure mode causes wasteful application of CFRP composite materials due to un-development of their full capacities. To

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Table 1 Comparison of the design moments and reduction factors for an FRP strengthened member

	Design moment	Reduction factor for FRP
Manual No.4 (Canadian code)(2001)	$M_r = T_s\left(d - \frac{a}{2}\right) + T_{frp}\left(h - \frac{a}{2}\right)$ $T_{frp} = \phi_{frp}E_{frp}\varepsilon_{frp}A_{frp}$; for $\varepsilon_{frp} \le \varepsilon_{frpu}$ $T_{frp} = 0$; for $\varepsilon_{frp} > \varepsilon_{frpu}$	$\phi_{frp} = 0.75$
fib (European code) (2001)	$M_{RD} = A_{s1}f_{yd}(d - \delta_G x) + A_f E_f \varepsilon_f (h - \delta_G x)$	$\gamma_f = 0.67$ to 0.83
ACI 440 Report (2000)	$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(h - \frac{\beta_1 c}{2}\right)$	$\psi_f = 0.85$

prevent this premature brittle failure, several different anchoring systems were investigated by Ritchie *et al*. (1991), Sharif and Baluch (1994), Spadea *et al*. (2000). However, it is still necessary to develop a simple and more effective anchorage system, e.g. diagonal CFRP anchors.

Several institutes and agencies have developed design guidelines for the use of CFRP systems for rehabilitation of concrete members. (Neale 2001; *fib* Task Group 9.3 FRP Reinforcement for Concrete Structures 2001, ACI Committee 440 Report 2000). Table 1 shows the flexural equations and the reduction factors of these guidelines. The Canadian guideline (Manual No. 4, Neale 2001) suggests the several assumptions and recommendations for flexural design: plane sections remain plane; perfect bond exits between the steel and concrete and between the FRP and concrete; shear strain is neglected for flexural design; and a proper anchorage or development length is provided for FRP reinforcement. In addition, the Canadian guideline suggests resistant factors for concrete, steel, and FRP, and ultimate compressive strain in concrete of 0.0035 for structures and 0.003 for bridges.

In the European guideline (*fib* Manual, 2001), the recommended design of externally bonded reinforcement using FRP composites reflects the effects of the additional FRP reinforcement on the section designed assuming full composite action and the capability of transferring forces through the bonded interface. The use of end anchoring of the longitudinal FRP reinforcement is recommended. If the shear capacity of the strengthened member is larger than the acting shear forces, the guideline recommends shear strengthening along with flexural strengthening. In addition, the guideline recommends the load combinations and partial safety factors for concrete, steel, and FRP. The suggested ultimate strain for concrete in the compression zone is 0.0035.

The most popular design guideline for FRP strengthened concrete member in the United States is the American Concrete Institutes (ACI) 440 report (2000). The ACI 440 report is based on several assumptions for flexural design: the actual dimensions and material properties are considered; the strains in the strengthened member are proportional to the distance from the neutral axis; there exists no slip between the concrete and FRP reinforcement; the shear strain is neglected within the interface layer; the tensile strength of concrete is neglected; and the stress-strain curve of the FRP composite is linear up to failure. In addition, the guideline gives the recommended strength reduction factor for FRP composites and an ultimate compressive strain for concrete of 0.003. When FRP composites are used for flexural strengthening, the guideline recommends that the shear capacity with the flexural strengthened member be less than the existing shear forces. If the additional shear strengthening is needed, FRP composites can be used to provide additional shear forces. Due to a potential for delaminating at the interface between the concrete and FRP composites, the guideline recommends the additional strength reduction factor; however, it is recommended the use of mechanical anchoring systems.

The above mentioned guidelines did not much consider the effects of end anchorage, multiple layers of FRP application; amount of longitudinal steel reinforcement, and amount of steel stirrup on the behaviors of the FRP strengthened members. This research is conducted to investigate on the effects of the amount of CFRP composites, the amount of the longitudinal and shear reinforcement, and CFRP end diagonal anchorage system on the behavior of the flexural strengthening and ductility of CFRP strengthened reinforced concrete beams. In this paper, all experimental beams were designed and fabricated following the ACI 318-02 code (2002) and ACI 440 report.

2. Experimental program

2.1. Test specimens

To investigate the effects of the main variables mentioned above, sixteen full-scale under-reinforced concrete beams were constructed and tested. The sixteen beams were divided into four groups. Groups 1 and 2 each consisted of four-rectangular beams, as shown in Fig. 1. Beam B1 was tested up to its ultimate load. After testing, Beam B1 was repaired and tested as Beam B1R. Groups 3 and 4 consisted of four T-shaped beams and six T-shaped beams, respectively. Group 4 adopted two beams (TBB1 and TBB2) from Group 3. The details of test beams are summarized in Table 2. In comparison with rectangular beams, T-shaped beams have larger concrete compression zones. T-shaped beams allow increasing the amount of CFRP composites without premature failure of the concrete compression zones. Geometries of CFRP applications are shown in Fig. 1. Cross section of all rectangular beams was 12 in. (304.8 mm) wide by 16 in. (406.4 mm) high. Cross section of all T-shaped beams is 30 in. (762 mm) in flange width, 12 in. (304.8 mm) in web width, and 16 in. (406.4 mm) in height. The total length of the beam was 11 ft. (3352.8 mm), and the span between the supports was 9 ft. and 6 in. (2895.6 mm).

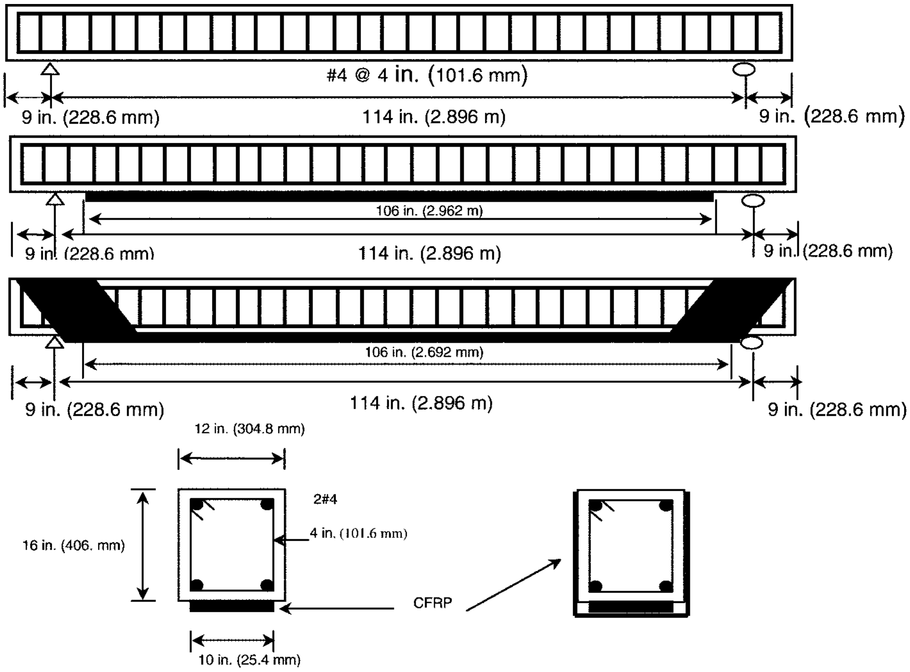
The average concrete compressive strengths were 5250 psi (36.2 MPa) for rectangular beams in Groups 1 and 2, 4435 psi (30.6 MPa) for T-shaped beams in Group 3, and 2000 psi (13.8 MPa) for T-shaped beams in Group 4, respectively. Grade 60 steel (414 MPa) was used. The average tensile yielding strengths were varied from 50 to 79 ksi (345 to 545 MPa).

2.2. CFRP composites

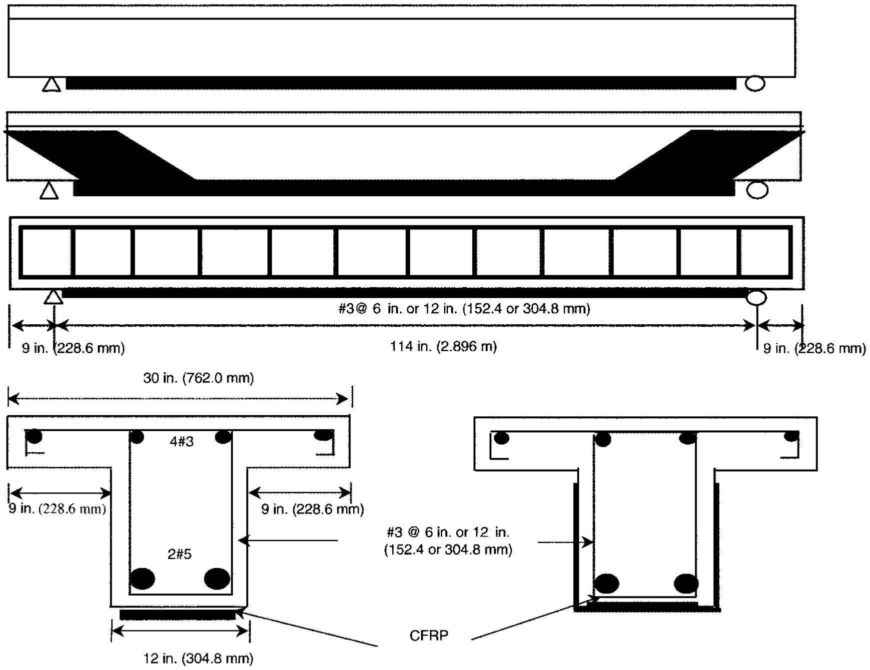
The cross section of each unidirectional CFRP composite sheet was 0.0066 in. thick by 10 in. wide (0.168 mm × 254 mm). The average tensile strength is shown in Table 3. CFRP coupon specimens were tested, and elongation was measured by an extensometer according to standard method (ASTM D 3039/D 3039M–95a). From the coupon test results, the ultimate strain and elastic modulus of the CFRP coupons decreased as the number of layers of CFRP composite sheet increased. Similar results were observed by the manufacturer. This phenomenon may occur due to stress concentrations between layers or at the anchoring zones of each CFRP coupon. The more fibers in a specimen, the higher the probability of failing elements, and the lower the average strength.

2.3. CFRP application

There are six main steps in CFRP application: surface preparation, primer application, putty application, resin undercoat application, CFRP sheet application, and resin overcoat application. The main purpose



(a) Rectangular specimens in Groups 1 & 2



(b) T-shape specimens in Groups 3 & 4

Fig. 1 Details of test specimens

Table 2 Details of all test specimens

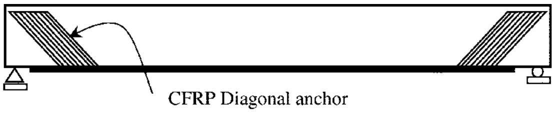
Group	Beam ID	Tension steel reinforcement	Actual f_y (ksi) (MPa)	Stirrup spacing (in.) (mm)	No. of longitudinal CFRP layers	No. of diagonal CFRP layers	Comments
1	B1	2#6	65 (448)	4 (101.6)	0	0	Reference
	B2				1	0	Strengthened
	B3				1	1	Strengthened w/anchors
	B1R				1	0	Repaired
2	B4	2#4	66(455)	4 (101.6)	1	0	Strengthened and tensile reinforcement ratio
	B5	2#5	68(487)		1	0	
	B7	2#7	71(490)		1	0	
	B8	2#8	68(487)		1	0	
3	TBA1	2#5	50(345)	6 (152.4)	1	0	Strengthened and shear reinforcement ratio w/o or w/ anchors
	TBA3		79(545)		1	1	
	TBB1		74(510)	12 (304.8)	1	0	
	TBB2		65(448)		1	1	
4	TBB1	2#5	74(510)	12 (304.8)	1	0	Strengthened and number of layers of CFRP sheet w/o or w/ anchors
	TBB2		65(448)		1	1	
	TBB3		64(441)		2	0	
	TBB4				2	1	
	TBB5				3	0	
	TBB6				3	3	

Table 3 Properties of CFRP composites

No. of layers of CFRP composite sheet	Average tensile strength (ksi) (MPa)	Tensile modulus (ksi) (MPa)	Rupture strain
1*	550 (3792)	33400 (230293)	0.017
1	750 (5171)	39000 (268905)	0.019
2	450 (3103)	37000 (255115)	0.012
3	400 (2758)	35000 (241325)	0.011

*Provided by a manufacturer

for using the primer is to have it penetrate into the concrete surface to enhance the strength of concrete and improve the bonding between the concrete surface and the CFRP sheet. After primer application, putty was applied to fill cavities, gaps, or pinholes, and uneven surface. After putty application, resin was applied and the first CFRP sheet was attached on the concrete surface. The resin was then over-coated on the CFRP sheet. The functions of resin are to act as an adhesive to bond the CFRP sheet to the concrete surface and to penetrate into the CFRP sheet and bond the carbon fibers together. The CFRP system was allowed to cure for a week before testing.



(a) Straight CFRP sheet along with end diagonal CFRP anchors

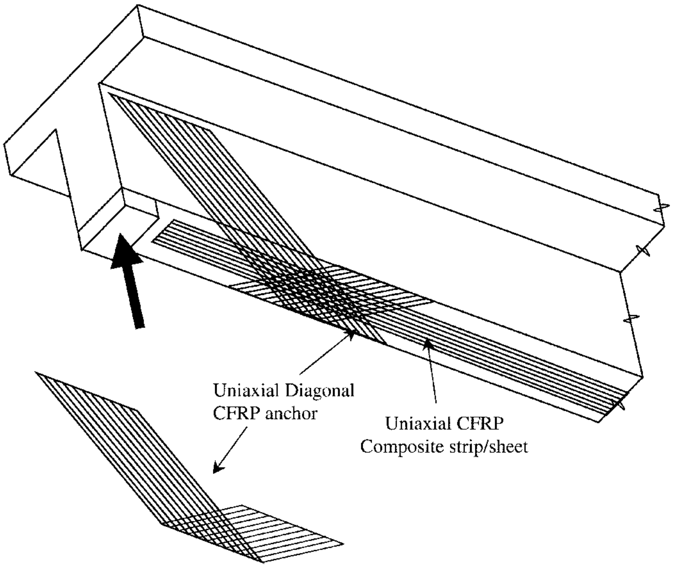


Fig. 2 Flexural strengthening of reinforced concrete beam with straight CFRP system, and end diagonal CFRP anchors

In this experimental test program, two CFRP details were adopted: external CFRP sheets applied to the tension side only, and external CFRP sheets applied to the tension side along with end diagonal CFRP anchors, as shown in Fig. 2. The side and bottom view of an end diagonal CFRP anchor is shown in Fig. 2(b). The end diagonal unidirectional CFRP anchor has better performance than the end vertical unidirectional CFRP anchor because the end diagonal CFRP anchor has vertical and horizontal components, and the horizontal component can resist against the acting tension force at the bottom of the beam.

2.4. Instrumentation

In the experimental tests, the beams were tested under a three-point static loading as shown in Fig. 3. The deflection was measured at the mid-span of the beams by linear motion transducers (LMT). Strains were measured on the steel rebars and on the CFRP sheets.

3. Test results

3.1. Load-deformation relationship

The load-deflection relationship of the test beams varied due to the configuration of the internal steel reinforcement and the detail and amount of the external CFRP reinforcement.

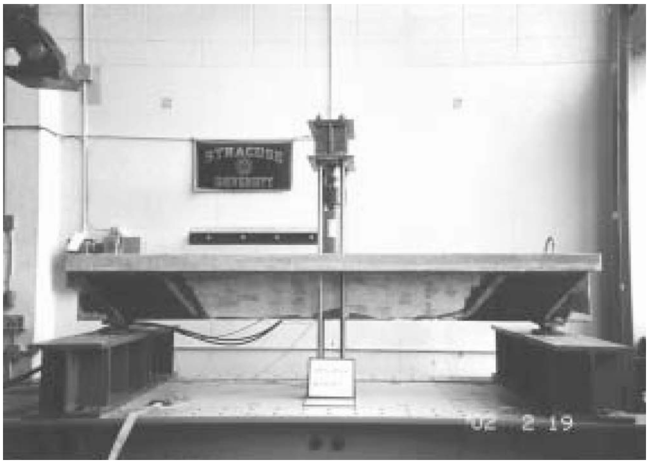


Fig. 3 Test setup

3.2. Group 1 beams

Group 1 beams consisted of four beams; reference beam (B1), strengthened beam (B2), strengthened beam with end CFRP diagonal anchors (B3), and repaired beam (B1R). Fig. 4(a) shows the load-deflection curves for Group 1 beams. The main goal of this test was to investigate the effect of the end CFRP diagonal anchors on the behavior of CFRP strengthened reinforced concrete beams.

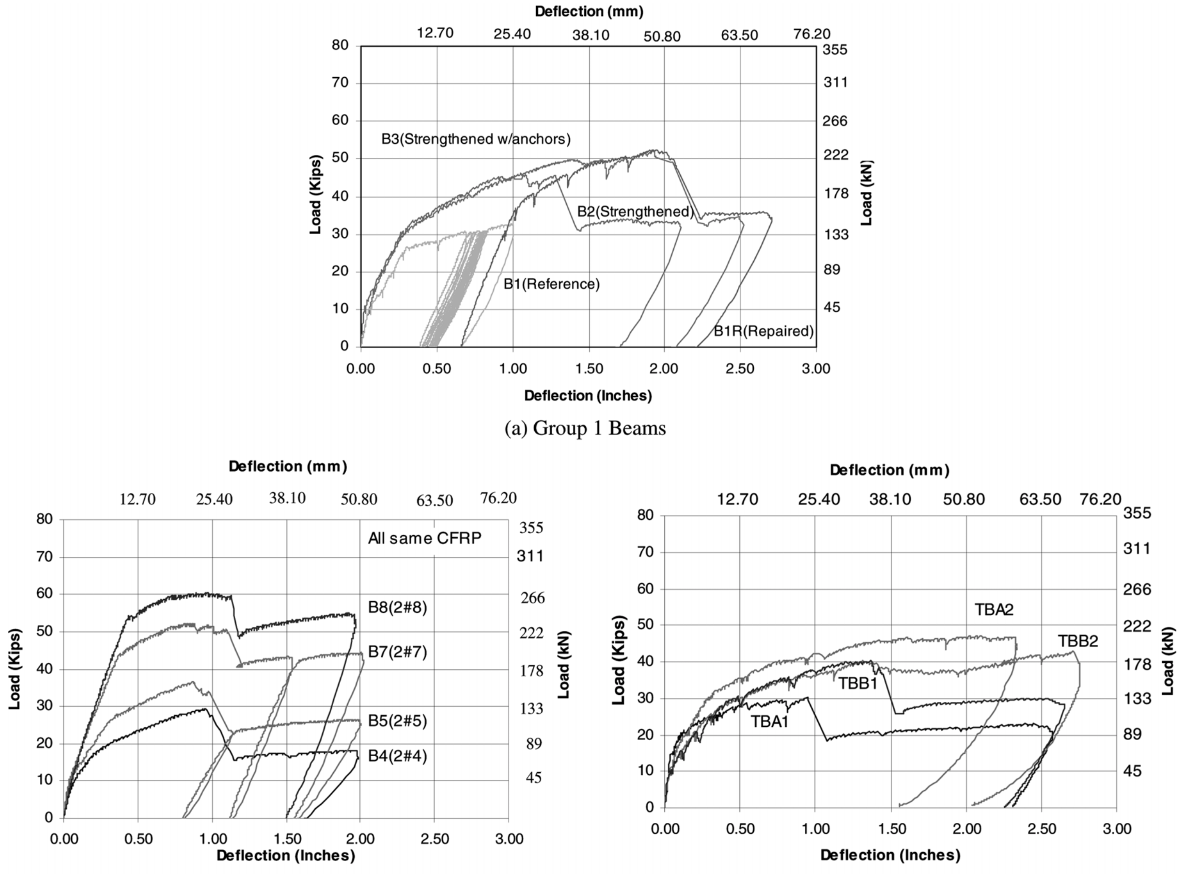
The response of Beam B1 presents typical behavior of an under-reinforced concrete beam: cracking, yielding, and ultimate stages. Repeated loading and unloading was applied to Beam B1 around the ultimate state in order to damage the bond between the steel reinforcement and the concrete. Beam B1 was later repaired and tested as Beam B1R.

Beams B2 and B3 showed higher strength and stiffness than Beam B1 due to the effect of external CFRP reinforcement. The loading continued up to a certain displacement, 2.1 in. (53.3 mm). After the separation of the CFRP sheet, Beam B2 maintained an ultimate strength equivalent to that of an unstrengthened ordinary reinforced concrete beam, the strength of which is equivalent to that of the reference beam, Beam B1. Before the complete rupture of the CFRP sheet, the debonding occurred over the full length except the end diagonal anchor zones of Beam B3, which had a displacement of 2.05 in. (52.0 mm) Beam B3 returned to the ultimate state of the unstrengthened beam, the strength of which is equivalent to that of the reference, Beam B1.

Beam B1R was the repaired Beam B1. After the separation of the concrete cover from the internal steel bars due to bond deterioration in previous test, the strength of Beam B1R dropped to that of the unstrengthened beam, the strength of which was equivalent to that of the reference beam, Beam B1. From the Group 1 test results, it has been shown that end CFRP diagonal anchorage system was very effective in increasing flexural strength and ductility, and developing the ultimate capacity of the longitudinal CFRP sheet.

3.3. Group 2 beams

Group 2 beams consisted of four rectangular strengthened beams, which had the same amount of the CFRP sheet, but different amount of internal tensile steel reinforcement: 2#4, 2#5, 2#7, and 2#8 rebars



(a) Group 1 Beams

(b) Group 2 Beams

(c) Group 3 Beams

Fig. 4 Load-Deflection curves for Groups 1, 2, and 3 Beams

for Beams B4, B5, B7, and B8, respectively. The primary goal of this test was to investigate the effect of the amount of internal tensile steel reinforcement on the behavior of CFRP strengthened reinforced concrete beams.

Fig. 4(b) shows the load-deflection curves for Group 2 beams. The stiffness of the strengthened beamsí curves was the same up to cracking point. Between the points of cracking of concrete and yielding of the internal steel bars, the stiffness of the strengthened beams' curves varied according to the ratios of CFRP to steel. As expected, the internal steel reinforcement controlled the stiffness between cracking and yielding points on the load-deflection curves even though the external CFRP reinforcement had limited. The slopes between the yielding and ultimate points in the load-deflection curves were mainly controlled by the external CFRP reinforcement. As the ratio of CFRP to steel increased, the flexural strength was increased and the deformation at failure decreased.

3.4. Group 3 beams

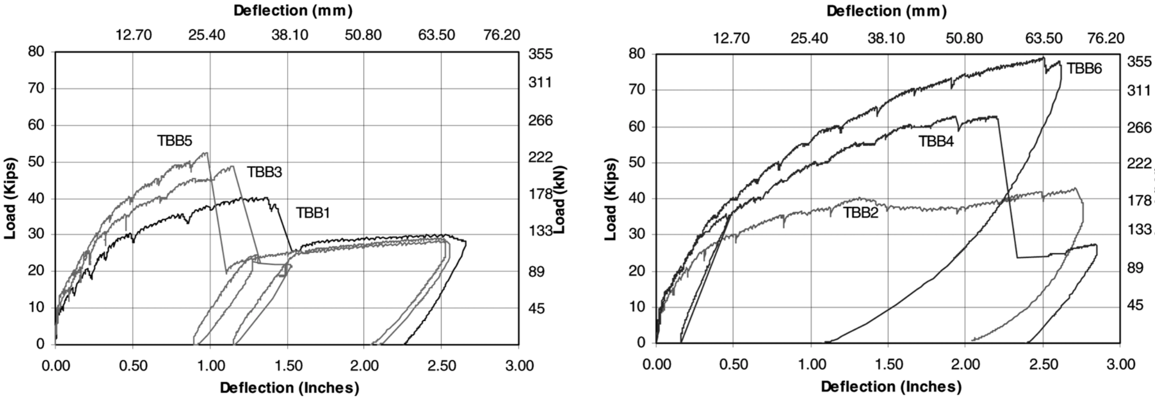
Group 3 beams consisted of four strengthened beams: TBA1, TBA2, TBB1 and TBB2. TBA2 and TBB2 were strengthened using CFRP sheet with end CFRP diagonal anchors. The primary goal of this test was to investigate the effects of both the amount of internal steel shear reinforcement and the various tensile strength of the internal longitudinal steel reinforcement on the behavior of the CFRP strengthened beams.

Fig. 4(c) shows the load-deflection curves for Group 3 beams. The average tensile strengths of internal steel reinforcement were 50, 78, 74, and 64 ksi (344.8, 537.8, 510.2, and 441.3 MPa). The yielding points were different because the yielding strengths of the longitudinal steel reinforcement of all Group 3 beams were different. The beams with low shear reinforcement (TBB1 and TBB2) exhibited high deformation. The spacing of flexural-shear cracks in the beams with low shear reinforcement was wider, and the slope of flexural-shear cracks was less steep. From the results of this test, it was demonstrated that the tensile yield strength of the longitudinal steel directly affected the flexural strength, and the amount of the shear reinforcement influenced the deformation and failure modes.

3.5. Group 4 beams

Group 4 beams consisted of six strengthened beams. Three of them were strengthened using CFRP sheet(s) to the tension side only: TBB1, TBB3, and TBB5. The last three beams were strengthened using CFRP sheet(s) to the tension side along with end CFRP diagonal anchors: TBB2, TBB4, and TBB6. The main goal of this test was to examine the effect of the amount of CFRP reinforcement and the effect of end diagonal anchors on the behavior of the CFRP strengthened beams.

Figs. 5(a) and (b) show the load-deflection curves for Group 4 beams. The beams with the high ratio of CFRP to steel and without end diagonal anchors exhibited the highest strength and stiffness and the lowest deformation at failure, as shown in Fig. 5(a). After the separation of the CFRP sheets, the loading continued up to a displacement of 2.5 in. (63.5 mm). Beams, TBB1, TBB3, and TBB5, maintained an ultimate strength equivalent to that of the unstrengthened beams. From Fig. 5(b), the beams with the high ratio of CFRP to steel and with end CFRP diagonal anchors showed the highest strength and stiffness. The displacement of Beams TBB2, TBB4, and TBB6 reached over 2 in. (50.8 mm). The beams with end CFRP diagonal anchors showed a higher strength and stiffness and a larger deformation at failure. From the results of this test, the diagonal anchoring system was very effective in increasing the flexural strength and the deformability. It is clearly seen that the limitation of the amount of CFRP sheet should exist for the efficiency of CFRP application and the demanding of shear reinforcement.



(a) Group 4 Beams w/o anchors (b) Group 4 Beams w/ anchors

Fig. 5 Load-Deflection curves for Group 4 Beams

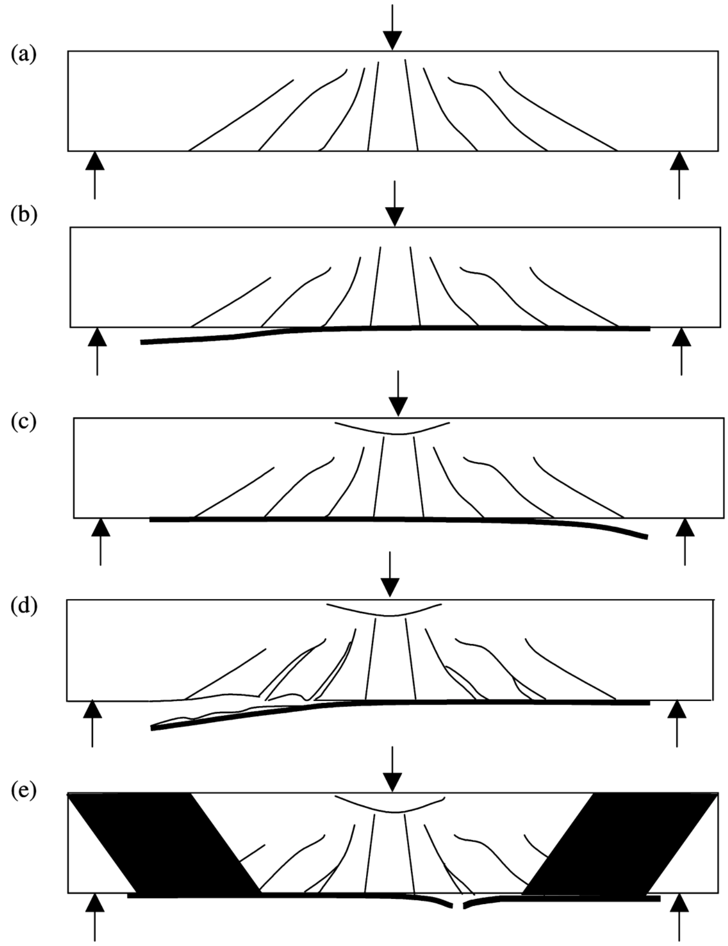


Fig. 6 Failure modes of unstrengthened and strengthened RC beams for Group 1 and 2: (a) Flexural failure; (b) Tension-shear failure between concrete surface and the CFRP sheet; (c) Crushing of compressive zone in concrete and then tension-shear failure; (d) Concrete cover separation; (e) CFRP rupture

3.6. Failure mode

Several failure modes were observed in the experimental tests. The typical failure modes can be classified into six types: (a) flexural failure, (b) tension-shear failure between the concrete surface and the CFRP sheet, (c) crushing of the compressive zone in the concrete and then tension-shear failure, (d) concrete cover separation, (e) CFRP rupture, and (f) tension-shear failure and partial or complete CFRP fracture along the diagonal edge. Schematic modes of failure are shown in Figs. 6 and 7. The total modes of failure were observed in beams from Groups 1 through 4. The failure modes of the strengthened beams with the diagonal anchorage systems occurred in ductile manners; whereas the failure modes of the strengthened beams without the diagonal anchorage systems occurred in abrupt and brittle manners with some catastrophic sounds.

However, flexural cracks started vertically in the tension region of all beams tested. These cracks then propagated into the shear region and approached the compression zone of the concrete until failure

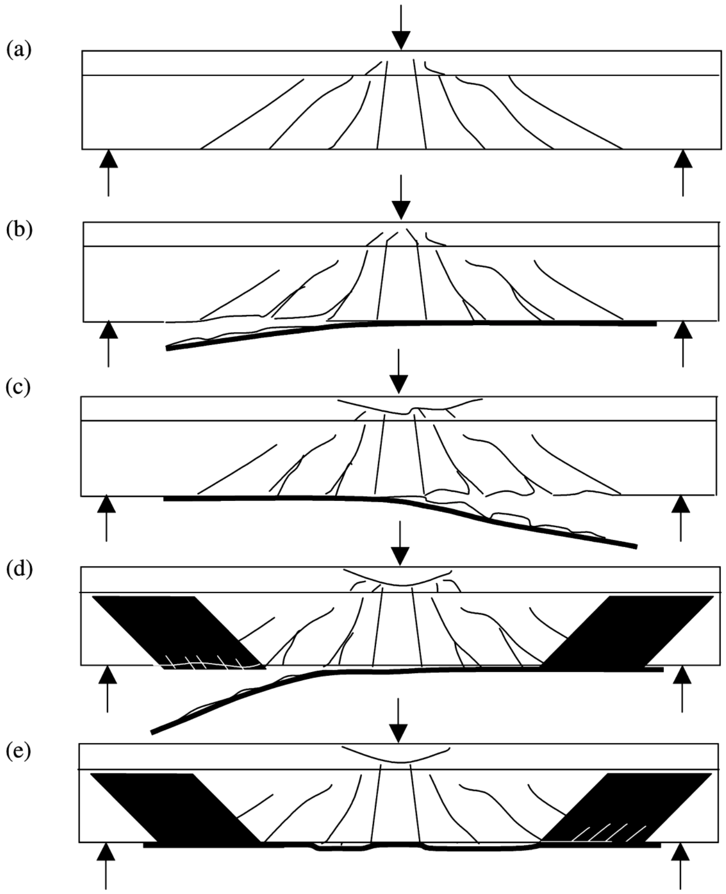
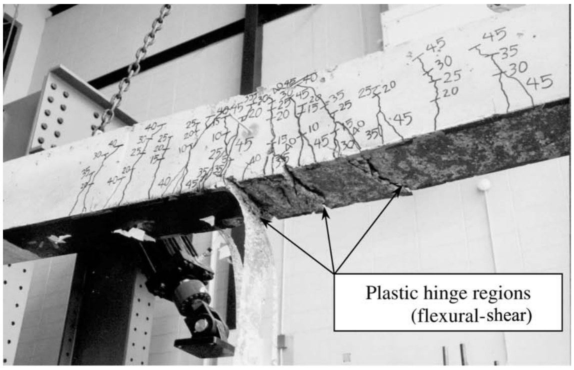


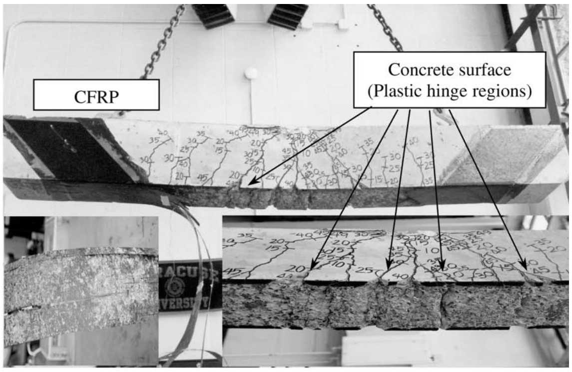
Fig. 7 Failure modes of unstrengthened and strengthened RC beams for Group 3 and 4: (a) Flexural failure; (b) Tension-shear failure between concrete surface and the CFRP sheet; (c) Crushing of compressive zone in concrete and then tension-shear failure; (d) Fracture of CFRP along the diagonal edge; (e) Tension-shear failure and partial CFRP rupture along the diagonal edge

occurred as shown in Figs. 6 and 7. Beam B1 (reference beam) behaved and failed in flexure as an ordinary reinforced concrete beam without any noticeable mode of shear failure, as shown in Fig. 6(a). After the cracks reached the compression zone, flexural and shear cracks developed at a distance between "d" (effective depth of the tested beams) and "2d" from the center of the beams, which are the plastic hinge regions. Beams B2 and B3 exhibited similar behavior. Photographs of the plastic hinge regions in Beams B2 and B3 are shown in Fig. 8.

The vertical movements of the flexural-shear cracks caused the separation of the CFRP sheets as shown in Fig. 9. The most CFRP strengthened beams without diagonal anchors failed by the tension-shear model of failure of the concrete cover in the vicinity of the longitudinal CFRP sheet with no crushing of concrete, as shown in Figs. 6(b) and 7(b). However, Beam B8 and TBB5 exhibited crushing of the concrete in the compression zone prior to the tension-shear failure, as shown in Figs. 6(c) and 7(c), respectively. The CFRP strengthened beams with the diagonal anchors held the CFRP sheets until crushing of the concrete in the compression zone occurred, as shown in Fig. 10.

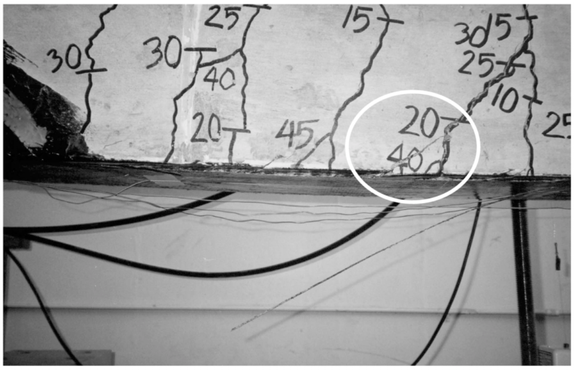


(a) Plastic hinge regions at Beam B2



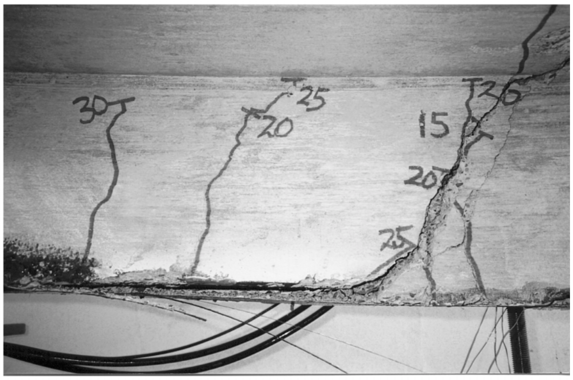
(b) Plastic hinge regions at Beam B3

Fig. 8 Plastic hinges of strengthened beams



(a) Vertical movement at Beam B3

(b) Magnified circle region of Beam 3



(c) Vertical movement at Beam TBB2

Fig. 9 Vertical movement of strengthened beams with diagonal anchors

Tension-shear failure occurred in the CFRP strengthened beams along the CFRP sheet on the bottom of the strengthened beams due to the vertical movement of the flexural-shear cracks in the plastic regions before crushing of the concrete in the compression zone occurred. After crushing of the concrete in the compression zone, the CFRP sheet was completely ruptured near the diagonal

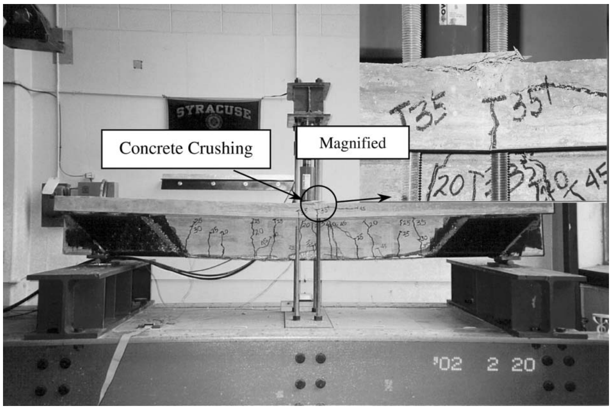
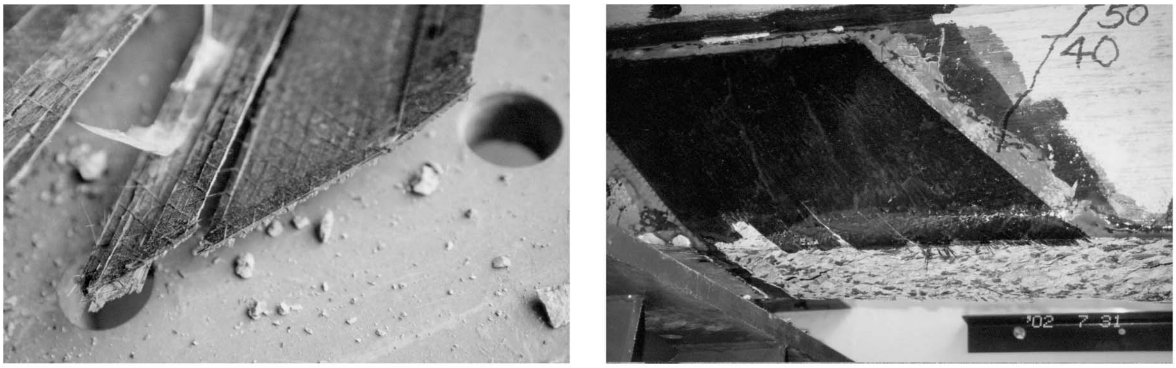


Fig. 10 Crushing of compression zone in concrete of Beam TBA2



(a) Rupture of CFRP sheet on the tension side of Beam B3

(b) Complete fracture of CFRP sheet on end diagonal anchors of Beam TBB4

Fig. 11 Fracture failures of the CFRP strengthened beams

anchorage on the right side of Beam B3, as shown in Figs. 6(e) and 11(a). The failure of Beams TBA2 and TBB2 was the partial fracture of the longitudinal CFRP sheet at the edge of the end CFRP diagonal anchors, as shown in Fig. 7(e). The CFRP sheet of the beams was debonded over the full length of the beam except the end anchoring zones. Beam TBB4 failed by the complete rupture of the left side of the diagonal anchors because the diagonal anchors had only a layer of CFRP sheet, as shown in Figs. 7(d) and 11(b). Beam TBB6 failed due to the partial fracture of the longitudinal CFRP sheets at the edge of the end of the CFRP diagonal anchors. In summary, diagonal anchoring systems changes the failure mode of the CFRP strengthened beams from brittle modes to ductile modes because the diagonal anchors sustained the longitudinal CFRP reinforcement even though it was totally debonded from the concrete surface. Table 4 presents a summary of the mode of failure of test specimens.

3.7. Ductility

The ductility of a beam can formally be defined as its capability to sustain deformation without losing load-carrying capacity before failure. According to ACI 440 report, classical ductility is defined as the ratio of the deformation at failure to the deformation at yielding. In this paper, ductility was investigated in

Table 4 Summary of mode of failure of test specimens

Group	Beam ID	Tension steel reinfor-cement	Stirrup spacing (in.) (mm)	No. of longitu-dinal CFRP layers	No. of diagonal CFRP layers	Comments	Mode of failure
1	B1	2#6	4 (101.6)	0	0	Reference	Yielding of rebars then crushing of concrete
	B2			1	0	Strengthened	Tension-shear failure in concrete cover
	B3			1	1	Strengthened w/ anchors	Crushing of concrete compression zone
	B1R			1	0	Repaired	Tension-shear failure in concrete cover
2	B4	2#4	4 (101.6)	1	0	Strengthened and tensile reinfor-cement ratio	Tension-shear failure in concrete cover
	B5	2#5		1	0		Tension-shear failure in concrete cover
	B7	2#7		1	0		Tension-shear failure in concrete cover
	B8	2#8		1	0		Tension-shear failure in concrete cover
3	TBA1	2#5	6 (152.4)	1	0	Strengthened and shear reinforcement ratio w/o or w/ anchors	Tension-shear failure in concrete cover
	TBA3			1	1		Crushing of concrete compression zone
	TBB1		12 (304.8)	1	0		Tension-shear failure in concrete cover
	TBB2			1	1		Crushing of concrete compression zone
4	TBB1	2#5	12 (304.8)	1	0	Strengthened and number of layers of CFRP sheet w/o or w/anchors	Tension-shear failure in concrete cover
	TBB2			1	1		Crushing of concrete compression zone
	TBB3			2	0		Tension-shear failure in concrete cover
	TBB4			2	1		Crushing of concrete compression zone along with localized damage of CFRP diagonal anchor
	TBB5			3	0		Tension-shear failure in concrete cover
	TBB6			3	3		Crushing of concrete compression zone

two forms: deformation model and energy model. The deformation model is based on classical ductility. For the energy model, ductility is defined as the ratio of the total energy up to ultimate to the inelastic energy up to yielding. Structural ductility is calculated at the yielding of the tension steel and at ultimate failure using Benardino *et al.*'s report (Blaschko *et al.* 1998). The ductility indices are defined as:

$$\text{Deflection ductility, } \mu_\Delta = \Delta_u / \Delta_y \tag{1}$$

$$\text{Energy ductility, } \mu_E = E_u / E_y \tag{2}$$

Where:
Δ_u = mid-span deflection at ultimate load
Δ_y = mid-span deflection at yielding load
E_u = area under the load deflection curve at ultimate load
E_y = area under the load deflection curve at yielding load

Table 5 Calculated ductility of the tested beams

Group	Beam ID	Ductility	
		Deflection ductility (μ_Δ)	Energy ductility (μ_E)
1	B1	3.1	4.6
	B2	4.1	7.8
	B3	5.8	11.2
	B1R	4.1	10.4
2	B4	3.7	6.4
	B5	3.2	5.1
	B7	2.9	5.0
	B8	2.5	4.3
3	TBA1	7.3	14.6
	TBA2	8.0	10.9
	TBB1	3.7	6.6
	TBB2	8.8	18.9
4	TBB1	3.7	6.6
	TBB2	8.8	18.9
	TBB3	3.6	6.6
	TBB4	6.7	16.1
	TBB5	2.8	4.5
	TBB6	6.6	17.3

For each beam, both deflection and energy models were used to calculate ductility, as shown in Table 5. In Group 1, the strengthened beams were more ductile than the reference beam (B1). In Group 2, the ductility was increased as the ratio of CFRP to steel bars was increased. In Group 3 and 4, the T-beams without the diagonal anchors were less ductile than those with the diagonal anchors. It can be concluded that the diagonal anchoring system remarkably increases the ductility of the CFRP strengthened reinforced concrete beams.

4. Comparisons of experimental and analytical results

Experimental and analytical results are compared in Table 5. For the comparison, analytical ultimate strength was calculated based on equations provided by ACI 440 report (2000). The ACI 440 design practice considers two additional reduction factors: the long-term environmental reduction factor (C_E) and additional strength reduction factor (ψ_f) for calculation of ultimate strength. C_E is considered for the CFRP composite due to the long-term environmental effects: a value of 0.95 was for interior exposure and a value of 0.85 was for exterior exposure. ψ_f, the value of 0.85, recommended in the ACI 440 report was used, because the bond condition between the CFRP composite sheets and the concrete was not as reliable as that between deformed steel bars and concrete, which results in a limited ductility.

In Table 6, analytical results assumed that C_E was 1.0 due to short-term environment effect, and ψ_f varied from 1 to 0.85. The experimental results showed in good agreement with the analytical results calculated based on the ACI 440 report. However, the ultimate capacities of the strengthened beams

Table 6 Experimental and analytical results

Group	Beam ID	Experiment (ultimate load)	ACI 440 (Paci, ultimate load) (kips), (kN)		Comparison	
		Pu (kips), (kN)	Paci-1.0 (C_E = 1.0, Ψ_f =1.0)	Paci-0.85 (C_E = 1.0, Ψ_f = 0.85)	Pu/Paci-1.0	Pu/Paci-0.85
1	B1	32.3(143.7)	29.0(129.0)	29.0(129.0)	1.11	1.11
	B2	45.5(202.4)	46.3(205.9)	43.5(193.5)	0.98	1.05
	B3	52.1(231.7)	46.3(205.9)	43.5(193.5)	1.13	1.20
	B1R	52.5(233.5)	46.3(205.9)	43.5(193.5)	1.13	1.21
2	B4	29.2(129.9)	33.1(147.2)	30.3(134.8)	0.88	0.96
	B5	36.0(160.1)	39.9(177.5)	37.1(165.0)	0.90	0.97
	B7	52.0(231.3)	55.2(245.5)	52.5(233.5)	0.94	0.99
	B8	60.0(266.9)	65.4(290.9)	62.7(278.9)	0.92	0.96
3	TBA1	30.1(133.9)	35.5(157.9)	32.6(145.0)	0.85	0.92
	TBA2	46.8(208.2)	44.2(196.6)	41.4(184.1)	1.06	1.13
	TBB1	40.3(179.3)	43.0(191.2)	40.2(178.8)	0.94	1.00
	TBB2	42.9(190.8)	40.1(178.4)	37.3(165.9)	1.07	1.15
4	TBB1	40.3(179.3)	43.0(191.2)	40.2(178.8)	0.94	1.00
	TBB2	42.9(190.8)	40.1(178.4)	37.3(165.9)	1.07	1.15
	TBB3	48.6(216.2)	57.6(256.2)	52.0(231.3)	0.84	0.93
	TBB4	63.0(280.2)	57.6(256.2)	52.0(231.3)	1.09	1.21
	TBB5	52.6(234.0)	76.2(338.9)	67.8(301.6)	0.69	0.78
	TBB6	79.0(351.4)	76.2(338.9)	67.8(301.6)	1.04	1.17

with diagonal anchors in the experimental results are 13 to 21% and 4 to 13% higher than those in the analytical results with an additional reduction factor (ψ_f = 0.85) recommended in the ACI 440 report and an additional reduction factor (ψ_f = 1.0), respectively. From the experimental and analytical results, it is suggested that an additional reduction factor for the strengthened beams with diagonal anchoring systems of 1.0 be used instead of 0.85 recommended in the ACI 440. In addition, the limitation of application of the amount of CFRP composite needs to be investigated in the future.

5. Practical design recommendations

Based on the ACI 440 report, the flexural design recommendations are modified and several limitations on flexural strengthening using CFRP composites. The concrete compressive stress model used is Whitney's stress block. Several assumptions are presented in the flexural analysis of the CFRP strengthened reinforced concrete beams, as follows:

(1) Plane sections before loading remain plane after loading.
(2) Tensile strength of concrete is neglected.
(3) Maximum compressive strain of concrete is 0.003.
(4) Perfect bonding exists between the concrete and steel reinforcement.
(5) Perfect bonding exists between the concrete and CFRP reinforcement.

Before installation of the CFRP reinforcement, the beam's initial strain (ε_{bi}) should be considered due to existing loads. The initial strain can be calculated from a conventional flexural analysis of the existing beam with existing loads. Whenever the beam is strengthened using CFRP reinforcement, shear forces should not exceed the beam's maximum shear capacity. If shear forces exceed the shear capacity of the beam, the external CFRP shear reinforcement should be considered according to ACI 440 report.

5.1. Reinforcement ratio of CFRP composites at balanced condition of ultimate state

The balanced condition at ultimate state is one in which the steel reinforcement exceeds the yield strain, the compression zone in the concrete is crushed, and the CFRP reinforcement is separated or ruptured. The strains at the balanced condition are:

$$\varepsilon_c = \varepsilon_{cu} = 0.003$$
$$\varepsilon_s > \varepsilon_y$$
$$\varepsilon_{fu} = f_{fu}/E_f \tag{4}$$

Where:
ε_{fu} = the rupture strain of the CFRP composite considered the environmental effects
f_{fu} = the rupture stress of the CFRP composite considered the environmental effects (=C_E* rupture stress provided by manufacturer)
C_E = the long-term environmental reduction factor (0.95 for interior exposure and 0.85 for exterior exposure are recommended by ACI 440 report).
E_f = the elastic modulus of the CFRP composite

The reinforcement ratio of the CFRP composites (ρ_f) is defined as follows:

$$\rho_f = \frac{A_f}{b_w d} \tag{5}$$

Where:
A_f = the cross-sectional area of the CFRP composites

From the internal equilibrium condition, the balanced reinforcement ratio of CFRP reinforcement is obtained as:

$$\rho_{f,b} = \frac{0.85 f_c' a - \rho f_y d}{f_{fe} d} = \frac{0.85 f_c' a - \rho f_y d}{\left(\frac{h-c}{c}\varepsilon_{cu} - \varepsilon_{bi}\right) E_f d} \tag{6}$$

$$f_{fe} = \varepsilon_{fe} E_f$$
$$\varepsilon_{fe} = \frac{h-c}{c}\varepsilon_{cu} - \varepsilon_{bi} \tag{7}$$

Where:
ε_{bi} = initial strain of the dead load

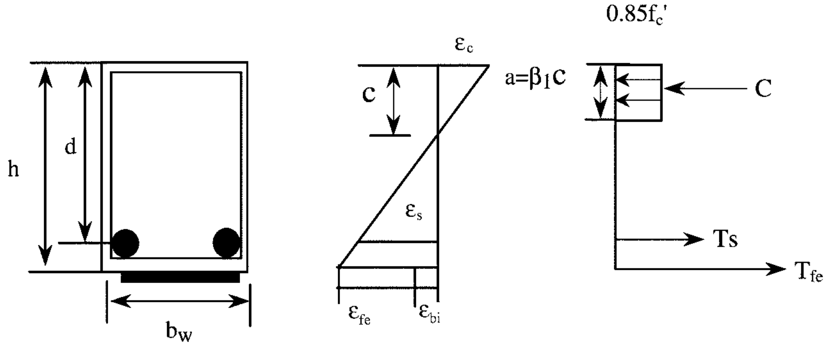


Fig. 12 Stress-strain diagram in a strengthened rectangular beam

The maximum ratio of CFRP reinforcement to avoid an over-reinforcement is less than the balanced ratio of CFRP reinforcement, as follows:

$$\rho_f < \gamma\rho_{f,\,b} \tag{8}$$

Where:
γ = factor of safety is recommended as same as that of ACI 318-02 (0.75 is recommended).

The factor of safety should be further investigated to determine the most appropriate value.
As shown in Fig. 12, the nominal flexural strength equation without the CFRP diagonal anchorage given by ACI 440 report (Equation 9.2) is:

$$M_n = A_s f_s\left(d-\frac{a}{2}\right) + \psi_f A_f f_{fe}\left(h-\frac{a}{2}\right) \tag{9}$$

Where:
f_{fe} = the effective stress of CFRP reinforcement ($= E_f\,\varepsilon_{fe}$)

$$\varepsilon_{fe} = 0.003\left(\frac{h-c}{c}\right) - \varepsilon_{bi} \le k_m \varepsilon_{fu}$$

$$k_m = 1 - \frac{nE_f t_f}{2{,}400{,}000} \tag{10}$$

Where:
κ_m = the factor in order to prevent debonding
ψ_f = an additional reduction factor (0.85 is recommended)
a = calculated from internal force equilibrium as followed:

$$a = \frac{A_s f_s + A_f f_{fe}}{0.85 f_c' b} \tag{11}$$

f_s = the stress of steel reinforcement
($=E_s\,\varepsilon_s \le f_y$)

The amount of increase in the flexural strength for CFRP strengthened beams without diagonal anchors should be limited by the ultimate strength of the beam without the CFRP reinforcement, due to the premature debonding, as follows:

$$M_{ss} < M_{uo} \tag{12}$$

Where:
M_{uo} = the ultimate strength without the CFRP reinforcement
M_{ss} = the maximum moment due to external loads at the service state for CFRP strengthened beam
ψ_f = an additional reduction factor (0.85 is recommended)

The amount of increase in the flexural strength for CFRP strengthened beams with diagonal anchors should be greater than the required strength of the beam, as follows:

$$\phi M_n \geq M_u \tag{13}$$

Where:
M_u = the required strength
M_n = the nominal strength with the CFRP reinforcement
Ψ_f = the strength reduction factor (1.0 is recommended due to the effect of diagonal anchors)

6. Conclusions

The application of CFRP composite sheet(s) to tension side only for strengthening ordinary reinforced concrete beams was investigated under a three-point static loading. Experimental research was conducted on full-scale beams with and without the addition of CFRP diagonal anchorage systems. All beams tested were designed according to the ACI 318-02 code and ACI 440 report. The variables in the test results are reflected in the design guideline. A practical design guideline was proposed following a modified ACI 440 model.

According to the experimental research presented in this paper, the following conclusions can be drawn:

(1) CFRP composite sheets can effectively be used to strengthen and repair existing ordinary reinforced concrete beams.
(2) The use of the CFRP strengthening system results in increasing shear demand. Shear resistance of the strengthened beam must be examined.
(3) The use of the additional diagonal anchorage system can further increase the flexural strength and ductility. It can also change the mode of failure from a brittle manner to a ductile manner.
(4) The same amount of CFRP layers for the diagonal anchorage as those for the longitudinal CFRP sheets is needed to prevent premature fracture failure of the diagonal anchorage.
(5) The ratio of the shear reinforcement in the CFRP strengthened reinforced concrete beams can change the crack patterns: those are from close crack spacing in higher shear reinforcement to wide crack spacing in lower shear reinforcement.

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Conversion factors

1 in. = 25.4 mm
1 kip = 4.448 kN
1 ksi = 6.895 MPa

Notation

Δ_u : mid-span deflection at ultimate load
Δ_y : mid-span deflection at yielding load
E_u : area under the load deflection curve at ultimate load
E_y : area under the load deflection curve at yielding load
f : Factor that controls contact compatibly

E	: Elastic modulus (if two different materials are contacted, smaller value of E should be chosen)
H	: Characteristic contact length should be a contact target length or a typical element size
ε_{fu}	: the rupture strain of the CFRP composite considered the environmental effects
f_{fu}	: the rupture stress of the CFRP composite considered the environmental effects
C_E	: the long-term environmental reduction factor (0.95 for interior exposure and 0.85 for exterior exposure as recommended by ACI 440 report)
E_f	: the elastic modulus of the CFRP composite
A_f	: the cross-sectional area of the CFRP composites
ε_{bi}	: the initial strain of the dead load
γ	: the factor of safety recommended by that of ACI 318-02 (0.75 is recommended)
f_{fe}	: the effective stress of the CFRP reinforcement (= $E_f\,\varepsilon_{fe}$)
κ_m	: the factor in order to prevent debonding
a	: calculated from internal force equilibrium as followed:

$$a = \frac{A_s f_s + A_f f_{fe}}{0.85 f_c' b}$$

f_s	: the stress of steel reinforcement ($=E_s\,\varepsilon_s \le f_y$)
M_{uo}	: the ultimate strength without the CFRP reinforcement
M_{ss}	: the strength with the CFRP reinforcement at the service state
M_u	: the required strength
M_n	: the nominal strength with the CFRP reinforcement