

Strengthening RC Beams in Flexure Using New Hybrid FRP Sheet/Ductile Anchor System

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Abstract: This paper explores a new hybrid fiber-reinforced polymer (FRP) sheet/ductile anchor system for rehabilitation of reinforced concrete (RC) beams. The advantages of the proposed strengthening method is that it overcomes the problem of low ductility that is associated with brittle failure mode in conventional methods of strengthening beams using epoxy-bonded FRP sheets. The proposed system leads to a ductile failure mode by triggering yielding to occur in a steel anchor system (steel links) rather than by rupture or debonding of FRP sheets, which is sudden in nature. Four half-scale RC *T*-beams were tested under four-point bending. Three retrofitted beams were strengthened using one layer of carbon FRP sheet. The results of the two beams that were strengthened with the new hybrid FRP sheet/ductile anchor system were compared with the results from the beam strengthened with conventional FRP bonding method and the control beam. The results show the effectiveness of the proposed strengthening system in increasing flexural capacity and ductility of RC beams.

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Introduction

Deterioration of the aging civil infrastructure has been well documented worldwide. In the United States and Canada, nearly 50% of the bridges were built before the 1940s or the 1950s. A large number of these are known to be structurally deficient at the present time. Mirza and Haider (2003) estimated that the cost of Canada's infrastructure deficit as well as the overall state of repair is roughly comparable to that of the U.S. According to the study, deterioration is the key determining factor in the infrastructure deficit. Three decades of deferred maintenance work have created a situation where if the deterioration is not halted, the associated costs will escalate exponentially. Structural deficiencies in bridge girders are usually the result of deterioration caused by aging, exposure to harsh environments, and higher traffic demands. Common structural deficiencies of deteriorated bridge girders are their low flexural and displacement ductility capacities. As a result, a large number of concrete highway bridges are in need of rehabilitation or replacement. Similar to the consequences of deterioration of bridges, there exist many reinforced concrete (RC) beams in need of upgrade or strengthening in older buildings, due to aging, increase in load, or deterioration.

Historically, concrete members have been repaired by post-tensioning or jacketing with new concrete in conjunction with a surface adhesive (Klaiber et al. 1987). Since the mid-1960s, epoxy-bonded steel plates have been used to retrofit flexural

members (Dussek 1980). Steel plates have a durability problem pertinent to this technique, because corrosion may occur along the adhesive interface. This type of corrosion adversely affects the bond at the steel plate/concrete interface and is difficult to monitor during routine inspection. Additionally, special equipment is necessary to install the heavy plates. As a result, fiber-reinforced polymer (FRP) composites have emerged as alternatives to traditional materials and techniques (externally bonded steel plates, steel or concrete jackets, and external post-tensioning). The advantages of using composites are mainly due to their high modulus of elasticity, lightness, corrosion resistance, and adaptable electromagnetic properties. Various FRP rehabilitation methods are being put in practice in strengthening and improvement of RC concrete beams. The oldest and mostly used method using FRP in the practice is the epoxy-bonded FRP. Since the 1990s, externally bonded FRP sheets/strips have been successfully applied to strengthen concrete structures in flexure (Meier 1992; Nanni 1995).

The FRP epoxy bonding method is capable of increasing the strength and stiffness of RC beams, conditional upon several variables, such as: FRP type, number of layers, FRP properties, skill of workers, etc. (Ritchie et al. 1991; Fanning and Kelly 2001). Although externally bonded FRP reinforcement performs well in the service stage, failure due to premature debonding was observed and identified by many researchers (Saadatmanesh and Ehsani 1989; Sharif et al. 1994). In fact, beams strengthened using the FRP epoxy bonding method are mostly susceptible to fail suddenly and without any precautions in a brittle manner due to debonding of the FRP sheet or laminate [Ritchie et al. (1991), Fanning and Kelly (2001), among others]. A number of researchers suggested the application of several different end anchors using *U*-shaped FRP to guarantee that the rehabilitated beam can reach its full capacity (Sharif et al. 1994; Swamy and Mukhopadhyaya 1999; Bencardino et al. 2002). Several details were proposed to avoid brittle failure without significant improvement in the mode of failure.

With the goal of improving bond in mind, near-surface

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mounted FRP was proposed. Use of near-surface mounted FRP rods and strips can preclude delamination-type failures, frequently observed by using externally bonded reinforcement. Blaschko and Zilch (1999) carried out bond tests on carbon FRP (CFRP) strips inserted inside grooves. Bond tests were conducted on double shear specimens. Test results showed that strengthening using near-surface mounted CFRP strips has a greater anchoring capacity compared to externally bonded CFRP strips. De Lorenzis and Nanni (2001) investigated the structural performance of simply supported reinforced concrete beams strengthened with near-surface mounted glass FRP (GFRP) and CFRP rods. Both flexural and shear strengthening were examined. Test results showed that the use of near-surface mounted FRP rods is an effective technique to enhance flexural and shear capacity of reinforced concrete beams (Hassan and Rizkalla 2003). This method also needs complicated surface preparation to prepare an appropriate groove in the concrete to install and epoxy the FRP strip in the groove.

A more recent application of FRP for flexural strengthening of reinforced concrete beams used mechanical anchors as a bonding technique. Unlike an externally epoxy bonded strengthening system, a mechanically anchored unbonded FRP strengthening system does not require surface preparation, adhesive application, or skilled labors. One method of application of mechanically anchored unbonded FRP uses a powder-actuated fastener gun to install mechanical fasteners and fender washers through holes in the FRP predrilled into the concrete substrate, "nailing" the FRP in place (Lamanna et al. 2001). Although little work has been done on the performance of RC beams strengthened with powder-actuated mechanically anchored FRP strengthening systems, nevertheless, results from preliminary studies indicate that a significant strength increase is possible through the use of such a strengthening system in specimens tested under monotonic conditions (Borowicz 2002). On the other hand, some notable disadvantages of this method have been observed, including scale effects, greater initial cracking induced by the impact of fasteners in higher-strength concrete, and less-effective stress transfer between the FRP and the concrete because of discrete attachment points. However, reported flexural performance in the tested specimens showed higher ductility compared to sudden failures associated with the externally epoxy bonded method. In cases where speed and ease of installation are major concerns, the mechanically anchored FRP method is a viable option (Borowicz 2002; Quattlebaum et al. 2005; Manos et al. 2007). El Maaddawy et al. (2005) investigated the structural performance of reinforced concrete *T*-beams strengthened with a mechanically anchored unbonded system, which was compared to that of *T*-beams strengthened with an externally epoxy bonded system. The anchor used to attach the CFRP strip to the bottom soffit of the specimen was a steel plate, which was placed below the CFRP strip and held in place using four bolts. The results showed that mechanically anchored unbonded strengthening system was effective in upgrading the *T*-beam strength but the strength gain was less than that obtained by the use of an externally epoxy bonded strengthening system.

Research Significance

In order to avoid the shortcomings of the traditional externally bonded FRP strengthening method (such as the high potential for debonding, need for surface preparation, high level of uncertainty in the FRP/concrete bond and interface characteristics, etc.), a new mechanically anchored system is proposed to increase the

RC beam's flexural strength and displacement ductility capacities. The goal of this study is to examine the effectiveness of a new FRP sheet/ductile anchor system for increasing the flexural capacity and ductility of RC beams. The advantages of this method on a conventional FRP epoxy-bonding method for RC beams is that the studied hybrid FRP/ductile anchorage system is not time consuming and does not need difficult surface preparation, high amount of adhesive application, or skilled workers. The proposed system leads to a ductile failure mode by triggering yielding to occur in the steel anchor system (steel links) rather than by rupture or debonding of FRP sheets, which is sudden in nature. In addition to its structural performance, the design of the proposed strengthening system is aimed at being a faster and cheaper intervention rehabilitation method compared to current conventional ones.

Proposed System for Increasing Flexural Strength and Ductility Capacities

Fig. 1 shows the details of the proposed strengthening system. It is mainly composed of an FRP sheet(s) that could be bonded (or not) to the soffit of the RC beam. The FRP sheets are wrapped around two steel plates at their ends (i.e., with a 180°) and then epoxy bonded (through an overlap) to the original FRP sheet. The steel plates have rounded corners in order not to have stress concentration and rupture of FRP sheets. The overlap is to avoid debonding between the FRP sheets (usually >150 mm). The steel plate is then linked to an angle that is anchored to the beam-support corner, through two steel link members (one at each side of the beam). The steel link member (looks like a conventional tensile test coupon sample) is linked to the steel plate and the anchored angle by means of high tensile threaded steel rods. According to this setup, the steel link members will always have axial tensile forces with no moments in them. The total cross-sectional area of the steel link members at one side of the beam are designed to have a yield stress that is less than the ultimate strength of the total FRP sheets. As such, the FRP sheets will transfer the stresses to the link members, than would yield before FRP ruptures. The total cross-sectional area of the steel link members, and consequently the total area of FRP, should be designed to achieve the targeted increase in the strength of the beam. Further, in order to keep the attractive feature of corrosion resistance of FRP, the steel anchorage system can be protected against corrosion without changing its mechanical properties, e.g., using galvanization, epoxy coating, or any other antirust coatings.

In the case when FRP is unbonded to the soffit of the beam in the new proposed system, work will only be needed at the ends of the RC girders to anchor the steel anchorage system and to wrap the FRP sheets around the anchors. The FRP sheets themselves could be epoxied as laminas on the work site in a way that does not interrupt the traffic (in the case of bridges). As such, FRP is preferred as a material to be the "tie" between the two ductile anchors due to its light weight, flexibility, high strength, and non-corrosiveness. Therefore, the proposed system (FRP sheets and ductile anchors) is expected to be easier to install (not many temporary supports and work platforms are needed compared to the conventional strengthening methods) and to have more quality control on the strengthening procedure.

Simplified Analysis of the Strengthening System

Fig. 2 shows the expected behavior of RC beams strengthened with the proposed system. The FRP sheets could be bonded to

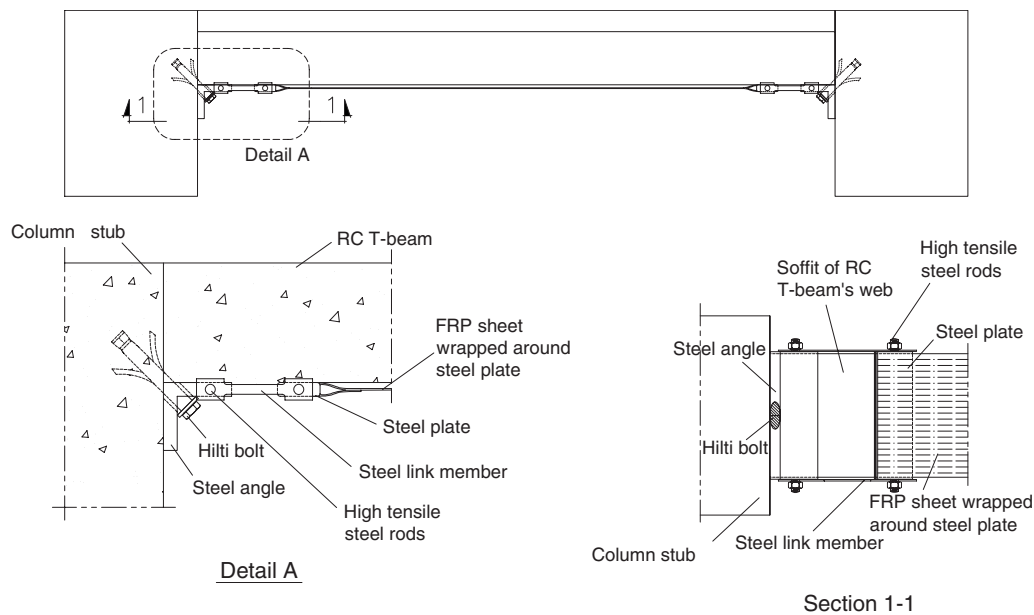


Fig. 1. Proposed hybrid FRP/ductile steel anchor system

soffit of the beam (using epoxy) or it could be unbonded. In the latter system, the FRP sheets come into effect, i.e., assist in the moment resisting capacity of the beam, by transferring the stresses to the end anchors, as the beam deforms. In the former system, the end anchors will act to eliminate the peel off and debonding of the FRP sheets, in an attempt to better utilize the capacity of the added FRP sheets. In such a case, the analysis would follow those of the design guidelines (ACI 2002; ISIS 2006; Fib 2001) and eliminate the peel off and debonding criteria.

For a strengthening system utilizing the proposed hybrid FRP/ductile steel anchor system where the FRP sheets are unbonded to the concrete beam, the total nominal resisting moment, $M_{n,total}$, of the strengthened beam is equal to the summation of the nominal resisting moment, $M_{n,orig.}$, of the control beam and added moment, ΔM , to the concrete beam due to the application of the hybrid FRP/ductile anchorage system, i.e.

$$M_{n,total} = M_{n,orig.} + \Delta M \quad (1)$$

$$\Delta M = A_{links} \times f_{y,links} \times jd \quad (2)$$

such that

$$A_{links} \cdot f_{y,links} = 3/4 \times A_{FRP} \times f_{FRP,u} \quad (3)$$

where A_{links} =total cross-sectional area of the link members at each end; jd =arm of the tension-compression couple; $f_{y,links}$ =yield stress of the steel link members; A_{FRP} =design area of the

FRP sheet(s); and $f_{FRP,u}$ =ultimate tensile strength of the FRP sheets. In Eq. (3), A_{links} should be designed to ensure that failure would occur in steel link members and not in the FRP sheet. In this research, a factor of 3/4 was arbitrarily chosen to ensure that the capacity of the steel link members is less than that of the FRP sheets. It should be mentioned that due to the nature of the mechanism, there will be a small gap, δ , between the FRP sheet and the soffit of the beam, which is equal to half of the thickness of the plate that the FRP sheets are wrapped around [Fig. 2(a)]. This gap will be closed as the beam deflects, as shown in Fig. 2(b). As long as the soffit of the beam is elongating, the FRP sheet will be experiencing tensile stresses that are transferred to the steel link members, which in turn transfer the stresses to the column stubs through the Hilti anchors.

Experimental Program

The goal of the experimental program is to evaluate the relative performance of beams strengthened with a hybrid FRP/ductile anchorage system to those strengthened using a conventional epoxy bonding application.

Test Specimens

Four half-scale RC *T*-beams with two column stubs were cast, with the main flexural steel reinforcement ratio selected to ensure under-reinforced behavior (i.e., tension-controlled section) with a reinforcement ratio of 0.92%. Fig. 3 shows the dimensions and details of reinforcement of the four tested beams. The *T*-beams were connected to two column stubs that were simply supported using hinge and roller supports, 3,000 mm apart. The flexural tension reinforcement consisted of two 15 M rebars. A mesh of 102 × 102 18/18 was used in the flange of the *T*-beam to represent typical slab reinforcement. The stirrups used were 10 M rebars spaced at 165 mm. The beams were overdesigned in shear (shear capacity is >200% higher than flexural capacity) to avoid a brittle shear failure. The column stubs were reinforced with four 20 M rebars as longitudinal reinforcement enclosed by 10 M ties.

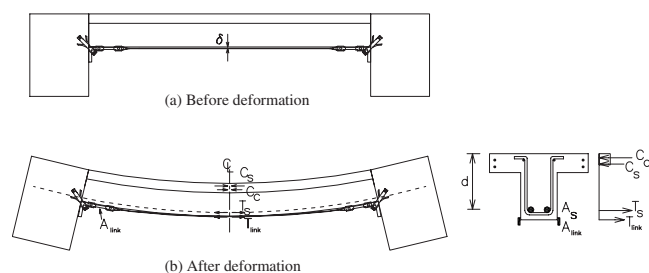


Fig. 2. Simplified analysis of the proposed mechanism

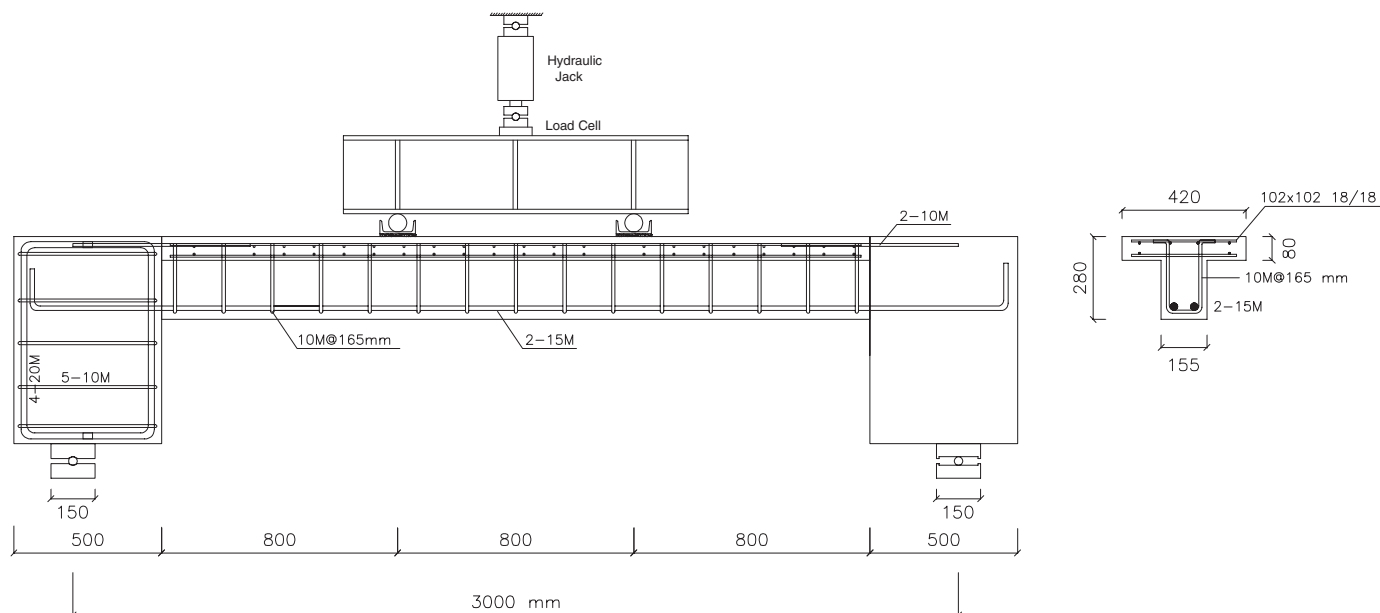


Fig. 3. Dimensions and details of reinforcement of the four tested beams

All four beams were tested under four-point loading over the span of 3,000 mm. The strains in longitudinal reinforcement, several locations of CFRP sheet, and steel link members (whenever applicable) were monitored and recorded during the tests. Further, deflections were measured at the midspan and the loading points and the midpoint of the shear span locations. The crack opening and propagation were checked by visual inspection and marked on the beams during the tests. Beam F-C-O was tested as a control beam without any strengthening systems. Beam F-E-B was strengthened with conventional epoxy-bonded FRP sheets. Beam F-M-U was strengthened with the new unbonded hybrid FRP sheet/ductile anchor system. In this beam, epoxy was applied to the FRP sheet, yet the FRP was not bonded to the beam. Beam F-M-B was strengthened using the same anchorage system of beam F-M-U, yet the FRP sheet was bonded to the soffit of the beam by epoxy. The three strengthened beams were strengthened using one layer of CFRP sheet.

Material Properties

The ready-mix concrete of the four tested beams was ordered for 28-day cylinder compressive strength of 30 MPa. Table 1 shows the concrete compressive strength of the four beams. The internal reinforcing rebars were hot-rolled steel with average measured

yield strength of 462 MPa and a tensile strength of 571 MPa with the maximum elongation of 16.9%. The stirrups were made of steel with the same mechanical properties.

The type of FRP that was used to externally strengthen the T-beams was uniaxial carbon fiber [Tyfo SCH-11UP, Fyfe Co. (2007)]. Tables 2(a)–(c) show the properties of the CFRP sheets (Tyfo SCH-11UP) and the epoxy adhesive (Tyfo S Epoxy) used for bonding process, respectively.

Table 2. Properties of CFRP Sheets and Epoxy Adhesive [as Provided by the Supplier Fyfe (2007)]

(a) Properties of dry carbon fiber sheet SCH-11UP	
Typical dry fiber properties	Value
Tensile strength	3.79 GPa
Tensile modulus	230 GPa
Ultimate elongation	1.70%
Density	1.74 gr/cm ³
Weight per m ²	298 gr/cm ³
Fiber thickness	0.127 mm
(b) Properties of composite gross laminate SCH-11UP	
Composite gross laminate properties	Value
Ultimate tensile strength	903 MPa
Elongation at break	1.05%
Tensile modulus	86.9 GPa
Laminate thickness	0.25 mm
(c) Properties of epoxy adhesive Tyfo S	
Epoxy adhesive properties	Value
Tensile strength	72.4 MPa
Tensile modulus	3.18 GPa
Elongation percent	5.00%
Flexural strength	123.4 MPa
Flexural modulus	3.12 GPa

Table 1. Test Variables of the Tested Beams

Beam	Concrete compressive strength (MPa)	CFRP sheet condition	Number of layers	External anchorage
F-C-O	33.0	—	—	—
F-E-B	37.1	Wet bonded	1	—
F-M-U	39.0	Wet unbonded	1	Hybrid CFRP/steel ductile anchor
F-M-B	39.2	Wet bonded	1	Hybrid CFRP/steel ductile anchor

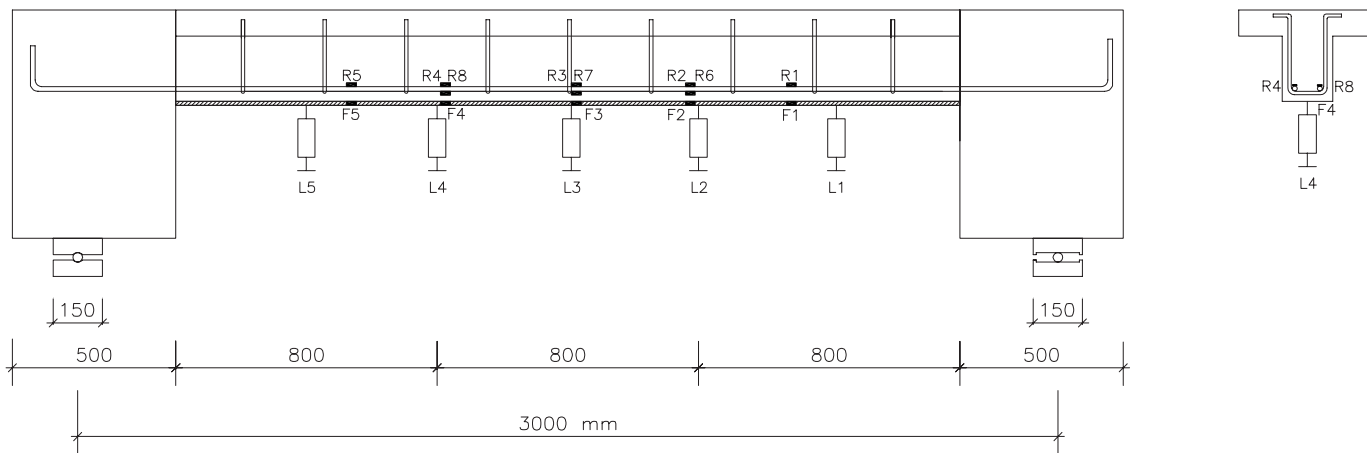


Fig. 4. Test setup and instrumentation of the tested *T*-beams

Test Setup and Instrumentation

All *T*-beams were tested under four-point bending with a span of 3,000 mm and a shear span of 1,100 mm. The beams were tested under an increasing monotonic load up to failure, or after reaching the end of the stroke of the actuator, which is 100 mm. One linear variable differential transducer (LVDT) was placed under the midpoint of the *T*-beam to measure the vertical deflection while a calibrated load cell was used to record the load. Four other LVDTs measured the vertical deflection at the loading points and the midpoint of the shear span locations.

Eight strain gauges were mounted on critical locations along the length of the two longitudinal bars, two at the midspan, four at the loading points, and two at the midpoint of the shear span as shown in Fig. 4. In addition, five strain gauges were installed on the flexural strengthening CFRP sheets of the *T*-beams, at the midspan and the loading points, and the midpoint of the shear span. These strain gauges were attached to the CFRP sheets on the beams F-E-B, F-M-U, and F-M-B and oriented along the fiber direction as shown in Fig. 4.

End Anchorage

A photo of the anchorage system used in beams F-M-U and F-M-B is shown in Fig. 5. The ductile anchor that holds the CFRP sheet under the soffit of the specimen consisted of one steel plate, 160×40×13 mm having two threaded 3/8" holes in its thickness, one steel angle, L 64×64×13 mm having one hole in the middle and two threaded 3/8" holes in its thickness (the angle is 160 mm long), two steel tensile link members (typical coupon samples for steel tension test), four high tensile 3/8" threaded rods ($f_y=724$ MPa and $f_u=862$ MPa) and one heavy duty HSL-3 M 24/60 Hilti bolt (Hilti 2007) at each end. The steel angle was fastened by the heavy duty Hilti bolt with 45° inclination located in the predrilled hole at the middle of the intersection point of the *T*-beam's soffit and the column stub. The two tensile steel link members connected the angle to the steel plate via high tensile 3/8" rods, which were located in the holes in the thickness of both the angle and the plate and it was fastened using nuts.

Experimental Results

Load and Displacement Capacities and Modes of Failure

The results obtained from the tests conducted on the four reinforced concrete beams with column stubs are discussed in this

section. The intent of this discussion is to evaluate the relative performance of beams strengthened with a hybrid FRP/ductile anchorage system to those strengthened using a conventional epoxy bonding application. Table 3 shows a summary of the test results for the four tested beams. The load versus midspan deflection relationship for the tested beams is shown in Fig. 6.

Canadian Highway Bridge Design Code (2006) (CAN/CSA-S6-06) and ISIS Canada 2006 categorized the potential flexural failure modes for externally strengthened reinforced concrete flexural members into four groups: (1) concrete crushing before yielding of the reinforcing steel; (2) steel yielding followed by concrete crushing; (3) steel yielding followed by FRP rupture; and (4) debonding of the FRP reinforcement at the FRP/concrete interface. The control *T*-beam exhibited a conventional ductile flexural mode of failure in which the *T*-beam failed by yielding of the steel reinforcement followed by crushing of the concrete. Specimen F-E-B that was strengthened with external epoxy bonding with no external anchorage failed prematurely without warning by debonding of the CFRP sheet after yielding of the steel reinforcement (failure mode 4). Beam F-E-B had only 7% higher load capacity compared to the control beam F-C-O. The CFRP sheet debonded at a displacement ductility of 1.91, after which, the response of the beam was governed by the original RC section.

On the other hand, beam F-M-U strengthened using the unbonded hybrid FRP/ductile anchorage strengthening system re-

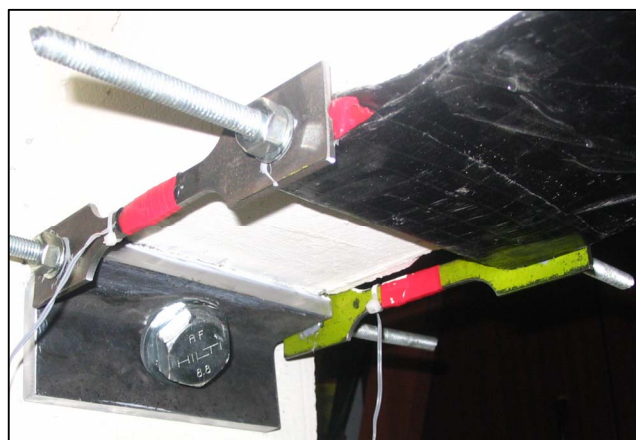


Fig. 5. Ductile anchor system used in beams F-M-U and F-M-B

Table 3. Summary of Test Results of the Four Beams

Beam	F-C-O	F-E-B	F-M-U	F-M-B
Concrete age at testing (days)	47	63	116	138
Load at yield (kN)	62.60	72.50	65.02	75.45
Deflection at yield (mm)	10.80	9.91	9.69	10.41
Maximum applied load (kN)	83.45	87.50	100.77	105.75
Deflection at maximum load (mm)	44.86	18.90	88.07	35.05
Displacement ductility	4.15	1.91	9.09	3.37
Mode of failure	Concrete crushing after rebar's yielding	FRP debonding	Concrete crushing after rebar's and steel link members' yielding	FRP rupture

sulted in about a 21% increase in the load carrying capacity compared to that of the control beam. The beam showed a high displacement ductility level that reached 9.09. The increase in strength and ductility were achieved by fully utilizing the capacity of the CFRP sheet through triggering yielding in the anchors' steel links and allowing the beam to crack and deflect without being restrained by bond with the CFRP sheet. Using the same ductile anchorage system of F-M-U, yet bonding the CFRP with epoxy to the beam soffit, beam F-M-B showed an increase in the strength capacity of 27% compared to the control beam, but failed due to the rupture of CFRP at a low displacement ductility of 3.37, mainly due to the high strains arising from being bonded at the locations of crack growths in the flexure zone. Although anchoring the CFRP sheets that are bonded to the beam, at the beams' ends, eliminated the peeling off of the sheets, the bonded CFRP sheet was subjected to high strains due to the growth and widening of the flexural cracks, which resulted in a sudden rupture of the CFRP sheets (failure mode 2) without reaching high ductility levels in the beam.

Thus, it could be said that the premature debonding and limited load capacity of F-E-B beam, and the FRP rupture with limited displacement ductility of beam F-M-B were successfully avoided in beam F-M-U that showed high load capacity and ductility.

From Table 3 it could be seen that the strengthening techniques used in F-E-B and F-M-B beams resulted in an increase in the load corresponding to the yielding of longitudinal reinforcement of the control beam by about 15 and 20%, respectively. On the other hand, the level of increase in the load corresponding to the yielding of longitudinal reinforcement in beam F-M-U was less than 5%. This could be attributed to the fact that the strengthening mechanism of F-M-U comes into effect after the beam deforms and starts to strain the FRP sheets and the steel link

members, which is not the case in the strengthening techniques of F-E-B and F-M-B where the FRP strains throughout the increase in applied loads. The midspan deflection at maximum load of the T-beam strengthened with unbonded hybrid FRP/ductile anchorage system was about 366% higher than that of specimen F-E-B, 151% higher than that of specimen F-M-B, and even 96% higher than that of the control.

Displacement ductility values for the four beams, defined as the ratio of the midspan deflection at the maximum load to the midspan deflection at the yield of longitudinal reinforcement, are shown in Table 3. From the table, it could be seen that beam F-M-U, which was strengthened with unbonded hybrid FRP/ductile anchorage method, had the most ductile behavior, with a displacement ductility of 9.09, whereas the other strengthening methods resulted in displacement ductilities that are less than that of the control beam.

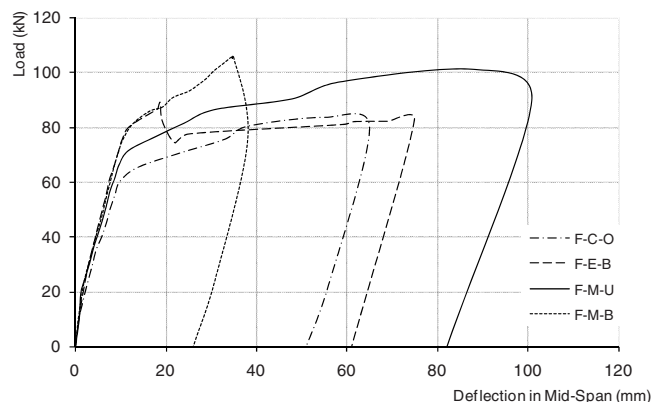
Fig. 7 shows the load and displacement ductility capacities of the three strengthened beams in relation to those of the control beam. From the figure, it could be seen that beam F-M-U that was strengthened using a ductile anchor system with unbonded FRP sheet was able to gain an increase in both load carrying capacity and displacement ductility. This was not possible by the other two strengthening systems. Fig. 8 shows the four tested beams at failure.

Role of Ductile Steel Link Members and Anchors

As explained earlier, the strengthening technique of beam F-M-U relied on increasing the moment capacity of the beam through transferring stresses in the FRP sheet (after its contact with the soffit of the beam) to the ductile steel link members that are anchored to the beam-column joints at the ends of the beam. Fig. 9 shows the load versus strain in the longitudinal rebars, steel link members, and the CFRP sheet of strengthened beam F-M-U. The concurrent measured strains at different stages of the load sequence of the beam F-M-U are distinctively marked in the three curves to help demonstrate their respective contributions at each stage. The following describes the performance of beam F-M-U at different stages up to failure as depicted by points 1 to 5 in Fig. 9.

Loading to Point 1: upon loading of beam F-M-U, the strain in the longitudinal steel reinforcement, FRP sheets and steel link members started to increase. The development of strain in the FRP sheet (even before closing the gap between the FRP and soffit of beam) could be attributed to the elongation in the soffit of the beam, and consequently the anchoring system and FRP, which shows that the hybrid FRP/ductile anchorage system is effective at service load capacity. At the end of this stage, yielding of the longitudinal rebar commences.

Point 1 to Point 2: as soon as the longitudinal steel rebars yielded, the CFRP sheet contribution to the load resistance in-

**Fig. 6.** Load versus midspan deflection (mm)

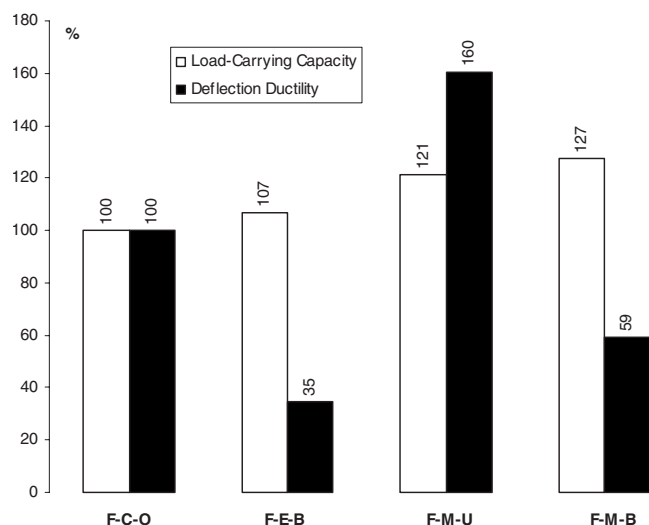


Fig. 7. Effect of strengthening technique on load and displacement ductility capacities

creased and the strain in the CFRP sheet increased with a higher rate compared to that in the previous stage. At the end of this stage, yielding of the ductile anchor system commences. It should be noted that the reduction in the measured strain in the yielded longitudinal rebar seen in the figure between Point 1' and Point 2 could be attributed to the engagement of the anchor system, as can be seen by a local increase in the strain of the steel link members at Point 1'.

Point 2 to Point 3: the anchoring system behaves in a post-yielding way. Upon strain hardening of the steel link members, the rate of increase of the strains in the CFRP sheet reduces (compared to the rate of increase in CFRP strain between Points 1 and 2). This is mainly due to the postyielding capacity of the steel link members.

Point 3 to Point 4: continued behavior similar to the previous stage with a higher rate of increase in the measured strain in the

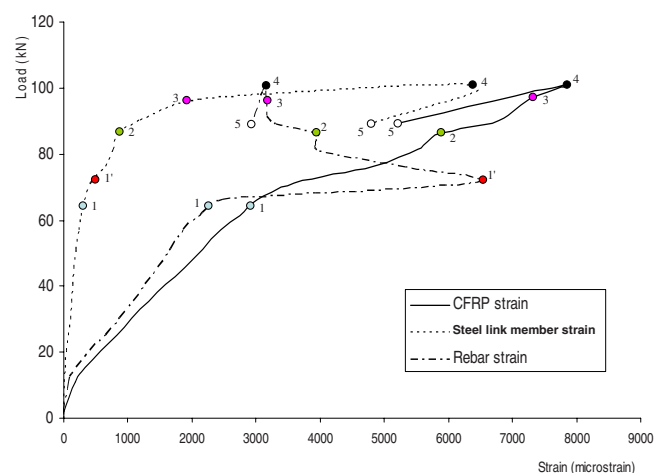


Fig. 9. Load versus strain of longitudinal rebar, CFRP sheet, and steel link members of beam F-M-U

steel link members. This results in a moderate rate of increase in the CFRP strain. The beam fails at the end of this stage (Point 4) due to compression failure of the concrete zone. It should be noted that the steel link members did not reach their limit state. The steel link members did not rupture and there were no signs of necking of the steel observed. From the figure, it could be seen that at higher load levels, the CFRP sheet was able to achieve high strain levels (almost 80% of its ultimate strain capacity). This was possible due to the elimination of the debonding failure criteria through the use of the anchoring system.

Point 4 to Point 5: after the beam failure, the measured strain in the steel link members, CFRP sheet, and longitudinal rebars decreases while the specimen is being unloaded.

Beam F-M-B, which was strengthened with epoxy-bonded CFRP along with the hybrid FRP/ductile anchors at the ends of the beam failed in a brittle manner by the CFRP sheet rupturing that was preceded by yielding of the steel reinforcement. The presence of the hybrid FRP/ductile anchors prevented debonding the CFRP sheet and, hence, the T-beam developed its full flexural capacity. Using the same ductile anchorage system of F-M-U, yet bonding the CFRP with epoxy to the beam soffit, beam F-M-B showed an increase in the strength capacity of 27% compared to the control beam, but failed due to the rupture of CFRP due to the high strains arising from being bonded at the locations of crack growths in the flexure zone. The bonded hybrid FRP/ductile anchors eliminated the end peeling off of the sheets, due to the presence of the anchorage system.

Comparison between Experimental and Analytical Results

The four tested beams were analyzed using CSA A23.3-04 (CSA 2004), ISIS Module 4 (ISIS Canada 2006), and ACI 440.2R-02 (ACI 2002) codes to predict their maximum load capacities. Table 4 shows the summary of the capacity analysis of the four tested beams. The nominal moment of beam F-C-O was calculated using CSA A23.3-04 code. The nominal moment of beam F-E-B was calculated using ACI 440.2R-02 code. ACI 440.2R-02 limits the strain in the laminate to account for debonding.

Fig. 10 shows a simplified analysis of the hybrid unbonded CFRP sheet/ductile steel anchor system of beam F-M-U.

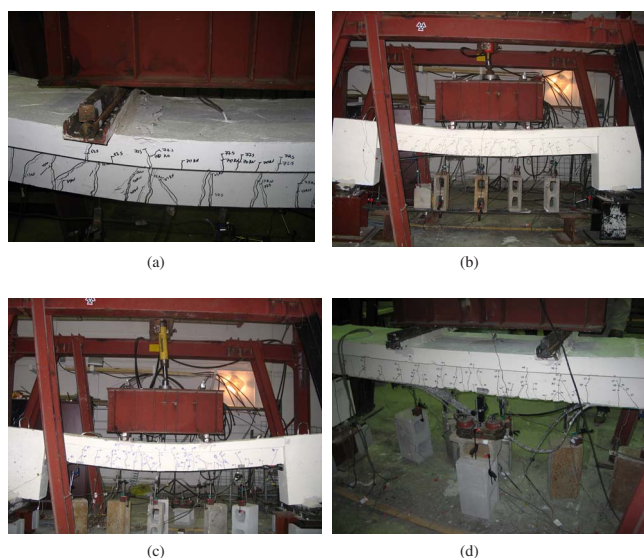


Fig. 8. Tested beams at failure: (a) F-C-O: concrete crush; (b) F-E-B: deflected shape before peel off; (c) F-M-U: maximum deflection; and (d) F-M-B: rupture of CFRP sheet

Table 4. Summary of Capacity Analysis of the Four Tested Beams

Parameters	F-C-O	F-E-B	F-M-U	F-M-B
c distance from extreme compression fiber to neutral axis (mm)	22.0	24.1	26.8	27.2
C_c compression force in the concrete (kN)	222.9	223.4	225.8	232.0
T_s tension force in the longitudinal steel reinforcement (kN)	184.4	184.4	184.4	184.4
T_{frp} tension force in FRP sheet (kN)	—	31.8	32.3	41.2
M_n nominal moment (kN m)	47.2	51.9	56.2	57.5
P_{max} maximum applied load (kN)	85.8	94.3	102.2	104.5

Note: $A_s=400\text{ mm}^2$; $A'_s=107\text{ mm}^2$; $f_y=462\text{ MPa}$; $A_{links}=80.6\text{ mm}^2$; and $f_{y,links}=462\text{ MPa}$.

From compatibility

$$\Delta_{total} = 2\Delta_1 + \Delta_2 \quad (4)$$

Thus, $\Delta_{total}=2\varepsilon_s L_1 + \varepsilon_{FRP} L_2$, where Δ_{total} =total elongation of the whole system due to vertical deformation of the beam; ε_s and ε_{FRP} =strains in the steel link members and the FRP, respectively.

From equilibrium (and ignoring friction)

$$\begin{aligned} T_{link} &= T_{frp} \\ \varepsilon_s \cdot E_s \cdot A_{links} &= \varepsilon_{FRP} \cdot E_{FRP} \cdot A_{FRP} \end{aligned} \quad (5)$$

The nominal moment of beam F-M-U was calculated based on a ductile failure controlled by yielding of the steel link members.

The nominal moment of beam F-M-B was calculated using the tensile capacity of the CFRP sheet, as was evidenced by the rupture of the CFRP sheet in the test.

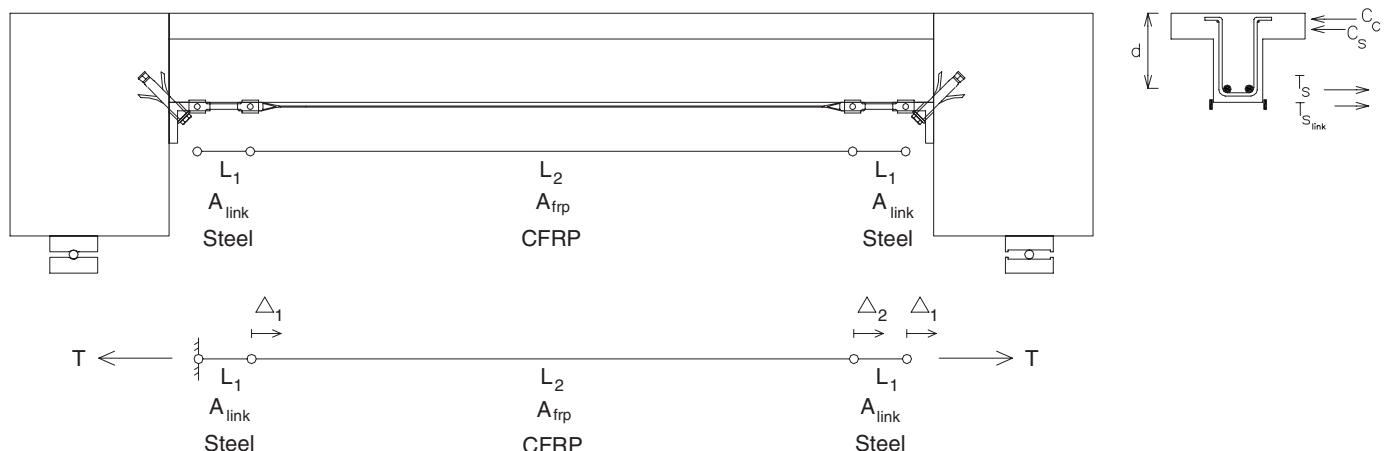
As can be seen from Tables 3 and 4, the differences between the predicted load capacities and the experimental ones are within 10% for the four tested beams.

Conclusions

This study examines the effectiveness of a new hybrid fiber-reinforced polymer (FRP) sheet/ductile anchor system for increasing the flexural capacity and ductility of reinforced concrete (RC) beams. Four half-scale RC *T*-beams each connected to two column stubs were tested under four-point bending. The first beam was used as the control beam, the second was strengthened with the conventional epoxy-bonding method, while the third and fourth beams were strengthened with a new hybrid FRP sheet/ductile anchor system. The difference between the latter two beams is whether the CFRP sheet was bonded or was unbonded to the soffit of the concrete beam.

The main conclusions from the proof-of-concept tests are as follows:

1. An externally bonded CFRP strengthening system without end anchorage increased the yield and the ultimate loads by about 16 and 5%, respectively, relative to those of the control beam. The *T*-beam failed prematurely due to peeling off of the CFRP sheet. The midspan deflection of the strengthened *T*-beam at ultimate load was 54% less than that of the control beam.
2. The presence of hybrid FRP/ductile anchorage in the externally bonded CFRP system prevented early peel off of the CFRP sheet, which enhanced the *T*-beam strength and ductility. The ultimate load of the *T*-beam strengthened with externally bonded CFRP along with hybrid FRP/ductile anchorage was about 27% higher than that of the control *T*-beam, whereas the midspan deflection at ultimate load was 19% lower than that of the control.
3. An unbonded hybrid FRP/ductile anchorage strengthening system was effective in increasing the *T*-beam strength and the strength gain was 21% higher than that of the control beam and 13% higher than the strength of the beam rehabilitated using conventional externally bonded CFRP.
4. The midspan deflection at ultimate load of the *T*-beams strengthened with unbonded hybrid FRP/ductile anchorage system was 366% higher than that of the *T*-beam strengthened with externally bonded CFRP without end anchorage, 151% higher than that of the *T*-beam strengthened with externally bonded along with hybrid FRP/ductile anchorage, and even 96% higher than that of the control.
5. An unbonded hybrid FRP/ductile anchorage system guarantees ductile failure in the strengthened concrete beams using hybrid CFRP sheet/steel anchor. The specimen F-M-U,

**Fig. 10.** Simplified analysis of the hybrid unbonded CFRP sheet/ductile steel anchor system

which was strengthened with an unbonded hybrid FRP/ductile anchorage method, exhibited the most ductile behavior, with a displacement ductility of 9.09.

It should be mentioned that, although the results and performance of the RC beams strengthened with the proposed hybrid unbonded FRP/ductile anchor system are promising, more tests could be conducted in order to understand the system's behavior for a wide range of parameters and to develop design guidelines. In addition to the enhancement to the strength and ductility capacities of RC beams, the proposed rehabilitation system is expected to be more economic, faster in applying, and have reliable performance with less variability.

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Notation

The following symbols are used in this paper:

- A_{frp} = design area of the FRP sheet (mm^2);
- A_{links} = total cross-sectional area of the link members at each end (mm^2);
- A_s = total area of main longitudinal steel (mm^2);
- A'_s = total area of compression steel (mm^2);
- C_c = compression force in the concrete (kN);
- c = distance from extreme compression fiber to neutral axis (mm);
- $f_{FRP,u}$ = ultimate tensile strength of the FRP sheets (MPa);
- f_y = specified yield strength of longitudinal steel reinforcement (MPa);
- $f_{y,link}$ = specified yield strength of steel link members (MPa);
- jd = arm of the tension-compression couple (mm);
- $M_{n,org.}$ = nominal resisting moment (kN m);
- $M_{n,total}$ = total nominal resisting moment (kN m);
- T_{link} = tension force in the steel link members (kN);
- T_s = tension force in the main longitudinal steel reinforcement (kN);
- ΔM = added moment due to strengthening (kN m); and
- δ = gap between the FRP sheet and the soffit of the beam (mm).

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