

Shear Strengthening of RC T-Beams Using Mechanically Anchored Unbonded Dry Carbon Fiber Sheets

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Abstract: This research studies the feasibility and effectiveness of a new method of strengthening existing RC T-beams in shear by using mechanically anchored unbonded dry carbon fiber (CF) sheets. This method eliminates the debonding of epoxy-bonded carbon-fiber-reinforced polymer (CFRP) sheets and utilizes the full capacity of dry CF sheets. In this method, dry CF sheets are wrapped around and bonded to two steel rods. Then the rods are anchored to the corners of the web-flange intersection of the T-beam with mechanical bolts. This makes a U-shaped dry CF jacket around the web which increases the shear strength of the T-beam using the privilege of higher tensile strength and modulus of elasticity of dry CF compared to composite CFRP. A total of three RC T-beams with shear span-to-depth ratio of 2.0 were tested under increasing monotonic load till failure. The pilot tests were done as a proof-of-concept of the effectiveness of the proposed method in increasing the shear capacity of the RC T-beams. The first T-beam, which was tested as the control beam, failed in shear. The second beam was strengthened by using a U-shaped CFRP sheet that was externally bonded to the web of the beam in the shear zones. The third beam was strengthened by using anchored U-shaped dry CF sheet. The test results showed that the beam strengthened by the new mechanically anchored dry CF had about 48% increase in shear capacity as compared to the control beam and 16% increase in shear capacity as compared to the beam strengthened by CFRP epoxy-bonding method.

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Introduction

The last decade has witnessed increasing demand for strengthening or rehabilitation of existing RC bridges and buildings. This is mainly due to aging, deterioration, increase in loads, corrosion of steel reinforcement, or advancement in the design codes and knowledge.

Despite the fact that RC T-section is the most popular shape of beams and girders in buildings and bridges, most of the research efforts to study the retrofit of RC beams in shear are conducted on rectangular sections. In general, failures of RC T-beams could be categorized to be due to deficiencies in flexure capacity, shear capacity, or both.

There exist many beams that were designed based on the pre-1970s codes [e.g., American Concrete Institute (ACI) 1968; American Association of State Highway Officials (AASHTO) 1965]. Several researchers pointed out that previous design provisions did not have a comprehensive understanding of the shear behavior. As the result, pre-1970s designs might be deficient in shear according to current codes (Higgins et al. 2007). Such deficient beams, would fail in a nonductile manner once their shear

capacity is reached. Therefore, strengthening RC T-beams to increase shear capacity is deemed one of the important goals in the retrofit of RC bridges and buildings.

Prior to 1990s, most of the retrofit activities used traditional methods in increasing the capacity or ductility of the section, for example, by increasing its cross section through steel jacketing. In the 1990s researchers paid more attention to the potential applications and benefits of using fiber-reinforced polymer (FRP) composites in retrofitting RC elements. This is due to the attractive characteristics of FRP, such as high strength to density ratios, corrosion resistance, and high fatigue strength. Most of the research activities on the use of FRP in strengthening RC beams were directed to enhancing their flexural capacity. For the few researches that have been done in strengthening RC beams in shear, most of the researches were conducted on RC rectangular sections (Bousselham and Chaallal 2004).

Al-Sulaimani et al. (1994) investigated the possibility of using glass FRP sheets to rehabilitate weak concrete beams in shear. A series of small-scale rectangular RC beams deficient in shear were cast. The specimens were loaded until the first visible cracks appeared, then rehabilitated with glass-fiber sheets. Even when the beams were designed to yield at a flexural capacity of 1.5 times the shear capacity before repair, some beams still failed due to flexure, and the full potential of FRP shear strengthening could not be reached. Similar concrete beam specimens without stirrups were also tested by other researchers (Chajes et al. 1995; Triantafillou 1998), but both studies concluded that full-scale tests and tests with internal shear reinforcement should be conducted.

To date, most of the research conducted on strengthening RC T-beams in shear focused on, to the writers' knowledge, enhancing the shear strength of the beam by using the contribution of the FRP through bond with the exterior sides of the beam. Cao and

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Teng (2005) categorized different shear strengthening techniques using FRP as complete FRP wraps covering the whole cross section (i.e., complete wrapping, valid only for rectangular sections), FRP U-jackets covering the two sides and the tension face (i.e., U-jacketing), and side bonded FRP covering only the two sides of the beam. Most of the design guidelines such as, ACI-440 (ACI 2008), Fédération Internationale du Béton (fib) (2001), and CSA-S06 [Canadian Standards Association (CSA) 2006], endorsed design formulas for the shear strength contribution from FRP that is bonded to beam for the three mentioned bonding methods. Previous studies have established clearly that such strengthened beams fail in shear mainly in one of the modes: tensile rupture of the FRP and debonding of the FRP from the sides of the RC beam, depending on how the beam is strengthened. Although not widely applicable, available experimental data indicates that almost all beams strengthened by complete wrapping (referred to as FRP-wrapped beams) failed due to FRP rupture. On the other hand, almost all beams strengthened by side bonding only, and most of the ones strengthened by U-jacketing, failed due to FRP debonding (Bousselham and Chaallal 2004; Cao and Teng 2005). With the goal of improving bond in mind, Khalifa et al. (1999) tested three specimens of which one of them was strengthened with a proposed patented form of bonded anchored U-jacketing FRP application. The anchor was an embedded bent portion of the end of the FRP reinforcement into preformed groove in concrete. No FRP debonding was observed at ultimate due to anchorage, yet the amount of surface preparation needed to grind off grooves in concrete as well as the extra amount of epoxy and additional rebars needed to secure the FRP sheets in the groove limited subsequent research to explore more that technique. Lees et al. (2002) used externally prestressed pin-loaded carbon-fiber-reinforced polymer (CFRP) straps as shear reinforcement for concrete beams. Experimental results showed that a concrete beam strengthened with CFRP straps exhibits higher load capacity than an unstrengthened beam. This method is applicable to rectangular concrete beams. In order to use this method on RC T-beams, several concrete cuts should be implemented in the T-beam's flange.

From the above, it could be stated that there is a need to develop a new technique for strengthening shear-deficient RC T-beams in order to reach the ductile capacity of the beam. In this research, a new strengthening system is proposed. The new system aims at using the full capacity of the dry fiber sheets, avoiding premature failures by using mechanical anchors rather than epoxy bonding, and maintaining easiness and speed of application features of FRP-strengthening techniques. Three pilot tests were performed as a proof-of-concept of the effectiveness of the proposed method in increasing the shear capacity of the RC T-beams.

Anchored U-Shaped Dry Fiber Sheet Proposed Method

Fig. 1(a) shows a schematic diagram of the Anchored U-shaped dry fiber sheet proposed method for shear strengthening of RC T-beams. The method relies on using the full mechanical contribution of the dry carbon fiber (CF) sheets, which will be activated upon development of strain in the RC web (and subsequently initiation and widening of shear cracks), and transferring them through a longitudinal steel rod to the core of the compression web-flange zone by means of mechanical anchors. In the proposed method, strengthening RC T-beams in shear is executed in four steps: (1) the dry CF sheet is wrapped around two steel rods

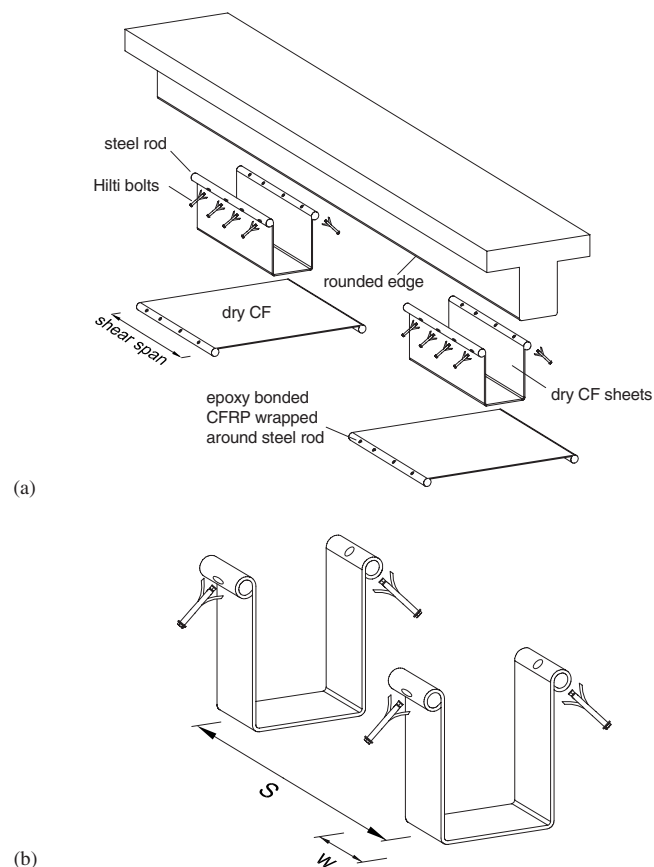


Fig. 1. Anchored U-shaped dry CF strips proposed method: (a) continuous sheet; (b) strips of width w every spacing S

(one at each end) in the site, where epoxy is applied at the wrapped parts only (in order to prevent slip of the sheet); (2) the corners of the soffit of the beam are chamfered to prevent stress concentration in this zone; (3) holes are drilled in the designed locations at the intersection of the web and flange with 45° inclination as well as in the rod; and (4) finally, the jacket is put at its proper location and the bolts are tightened in the predrilled holes, ensuring that the fibers are stretched uniformly along the strengthened zone.

Fig. 1(b) shows a possible strengthening method that uses the proposed mechanically anchored dry CF strips with specific spacing. The diameter of the steel rod and the width of the strips can be designed such that the rod would not yield before rupture of the dry CF. In order to keep the attractive feature of corrosion resistance of FRP-rehabilitation methods, it is recommended that the steel rod and anchors are made of stainless steel. Alternatively, a glass fiber insulating sheet could be used to avoid galvanic corrosion. It is anticipated that the proposed method will be less time consuming and consequently cost-effective compared to conventional epoxy-bonding methods. In addition, using mechanical anchors, rather than chemical adhesion through epoxy-based materials, is expected to have a more uniform and predictable behavior and performance of the FRP rehabilitation system with time.

Experimental Program

In order to examine the performance of the proposed method, the following pilot experimental program was conducted.

Table 1. Test Variables in the Experimental Program

Beam	Concrete compressive strength (MPa)	CF condition	Number of layers
S-C-O	41	—	—
S-E-B	43	Wet (CFRP)	1
S-M-D	43	Dry (CF)	1

Test Specimens

In order to evaluate the relative enhancement in shear capacity of a control beam when strengthened using epoxy-bonded and the proposed rehabilitation techniques, three half-scale RC T-beams were tested to failure. Table 1 shows the details of the test variables in the experimental program. The term S represents the beams strengthened in shear; C-O is for the concrete original (control) beam; E-B is for the beam strengthened using epoxy-bonding technique; and M-D is for the new mechanical-anchored dry CF technique. S-C-O was designed to have flexural capacity that is approximately 1.7 times higher than the shear resistance, in order to guarantee a shear failure in the T-beam. To reinforce the T-shape beams in flexure, two 25M bars with a 25.4 mm (1 in.) nominal diameter were used to provide the targeted flexural capacity. The beams were tested in four-point bending. The total length of the beams was 2.4 m, with a shear span of 575 mm resulting in a shear span-to-depth ratio of 2.0. 10M U-shaped stirrups, with 11.3 mm (7/16 in.) nominal diameter, at a spacing of 250 mm were used. The yield strength of the longitudinal reinforcement and stirrups was 465 MPa. Fig. 2 shows the concrete beam dimensions and details of reinforcement for the three tested beams. The beams were cast at the same time with the same concrete mixture using ready-mixed concrete. The concrete compressive strength at the test day for the three specimens is shown in Table 1.

Beam S-C-O was the control beam, while the other two beams were strengthened with CF sheets. Beam S-E-B had U-shaped bonded CFRP jackets in its shear spans as external shear reinforcement. The jackets were made of a single ply of CFRP sheet with the fiber in the direction perpendicular to the longitudinal

Table 2. Properties of Dry CF Sheet SCH-11UP (as Provided by the Supplier Fyfe Company LLC 2007)

Typical dry fiber properties	Value
Tensile strength	3.79 GPa
Tensile modulus	230 GPa
Ultimate elongation	1.60%
Density	1.80 g/cm ³
Weight per square meter	315 g/cm ³

axis of the beam. The CFRP sheets were extended below the flange to provide a minimum anchor length of 100 mm and were wrapped under the web. The type of FRP that was used to externally strengthen the T-beams was uniaxial CF (Tyfo SCH-11UP, Fyfe Company LLC 2007). Tables 2–4 show the properties of the Tyfo SCH-11UP CFRP dry and composite sheets, and the Tyfo S Epoxy adhesive used for the bonding process, respectively.

Beam S-M-D was strengthened using the proposed anchored U-shaped dry CF sheet system. Dry CF sheets were wrapped around and bonded to the two round steel rods with 25.4 mm (1 in.) diameter using Tyfo S Epoxy, which is the same adhesive epoxy used to bond the CFRP sheets in beam S-E-B. Then the concrete and steel rods were drilled in specific locations in the two corners of the web-flange intersection of the T-beam. The locations and the numbers of the anchors were highly over-designed such that the rods not fail before the CF sheet reaches ultimate capacity. The two end rods were anchored to the T-beam in the predrilled points with HSL-3 M10 Hilti bolts (Hilti Corporation 2007) to create a U-shaped dry CFRP jacket around the web in the shear-span zones. In both beams, S-E-B and S-M-D, the bottom corners of the web were chamfered at a radius of 10 mm in order to avoid stress concentration and premature rupture of the CF sheet. Fig. 3 shows the layout of strengthening schemes of beams S-E-B and S-M-D. In addition, the three tested T-beams were strengthened in the web area using 4 mm (5/32 in.) thick steel plates bolted to the beams' web above the support location in both sides by four medium duty 9.5 mm (3/8 in.) bolts to prevent any possibility of local failures above the supports. Figs. 4(a and b) shows a detailed view of the strengthened beams S-E-B and S-M-D, respectively.

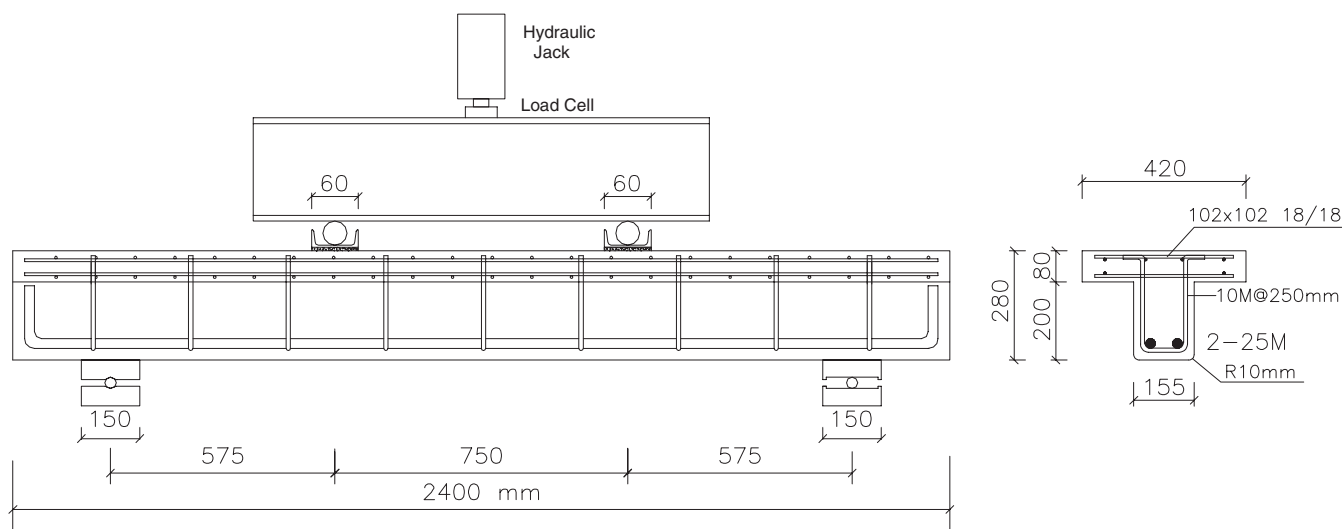
**Fig. 2.** Dimensions and details of reinforcement of the three tested beams

Table 3. Properties of Composite Gross Laminate SCH-11UP (as Provided by the Supplier Fyfe Company LLC 2007)

Composite gross laminate properties	Value
Ultimate tensile strength	903 MPa
Elongation at break	1.05%
Tensile modulus	86.9 GPa
Laminate thickness	0.27 mm

Table 4. Properties of Epoxy Adhesive Tyfo S (as Provided by the Supplier Fyfe Company LLC 2007)

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

Test Setup and Instrumentation

The test setup shown in Fig. 5, consisted of a four-point loading system that created a zone of constant moment in midspan zone. All three beams were tested under four-point loading over the span of 1,900 mm to failure. The shear span on each side is equal to 575 mm and the distance between the two loads is 750 mm. The shear span-to-depth ratio is approximately 2.0.

Deflections were measured at the midspan and the loading points' locations to record the deflections at the beginning and at the end of constant moment zone as shown in Fig. 5. The vertical displacement was recorded using three vertical linear variable differential transformers (LVDTs), each with a gauge length of 100 mm.

The strains in longitudinal reinforcement, stirrups, and several locations of CF sheets were monitored and recorded during the test. Eight strain gauges were mounted on the two longitudinal bars in the pure flexure zone, and 10 strain gauges were installed

**Fig. 4.** Detailed view of the strengthened beams before test: (a) S-E-B; (b) S-M-D

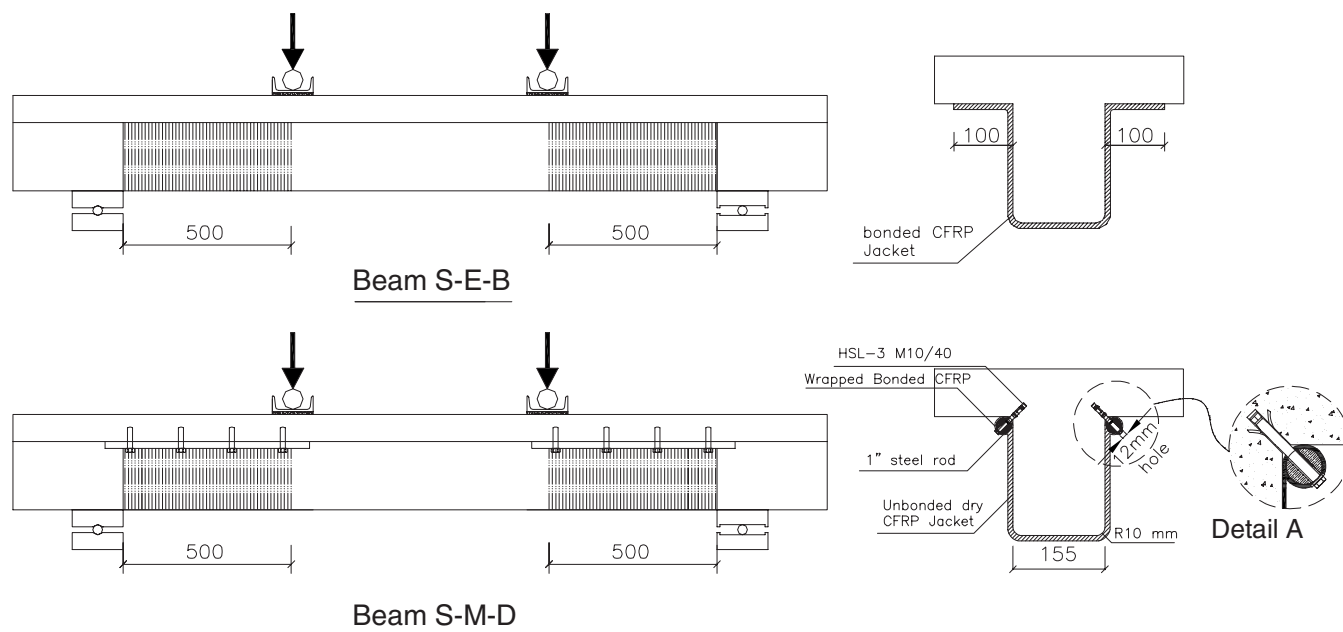
on the stirrups in the shear-span zones. In addition, eight strain gauges were installed on the CF sheets on the shear-spans of the T-beams. These strain gauges were attached to the CF sheets on one side of the beams S-E-B and S-M-D and oriented along the fiber direction. The strain gauges were mounted such that it would capture the high strains resulting from shear cracks as observed when testing the control beam S-C-O. The locations of the strain gauges mounted on CF sheets are shown in Fig. 5. The readings of the strain gauges, LVDTs, and the load cell were scanned and recorded using a data acquisition system.

Experimental Results

Failure Loads and Failure Modes

All of the three RC T-beams failed in shear as it was anticipated. Table 5 shows the load capacity and the failure modes for the three beams. Fig. 6 shows the cracking and failure patterns of the three tested beams.

Upon loading beam S-C-O, diagonal shear cracks were visible at a load of 145 kN. At this point, two main shear cracks were detected in the web within each shear span, as shown in Fig. 6(a).

**Fig. 3.** Layout of strengthening schemes of beams: (a) S-E-B; (b) S-M-D

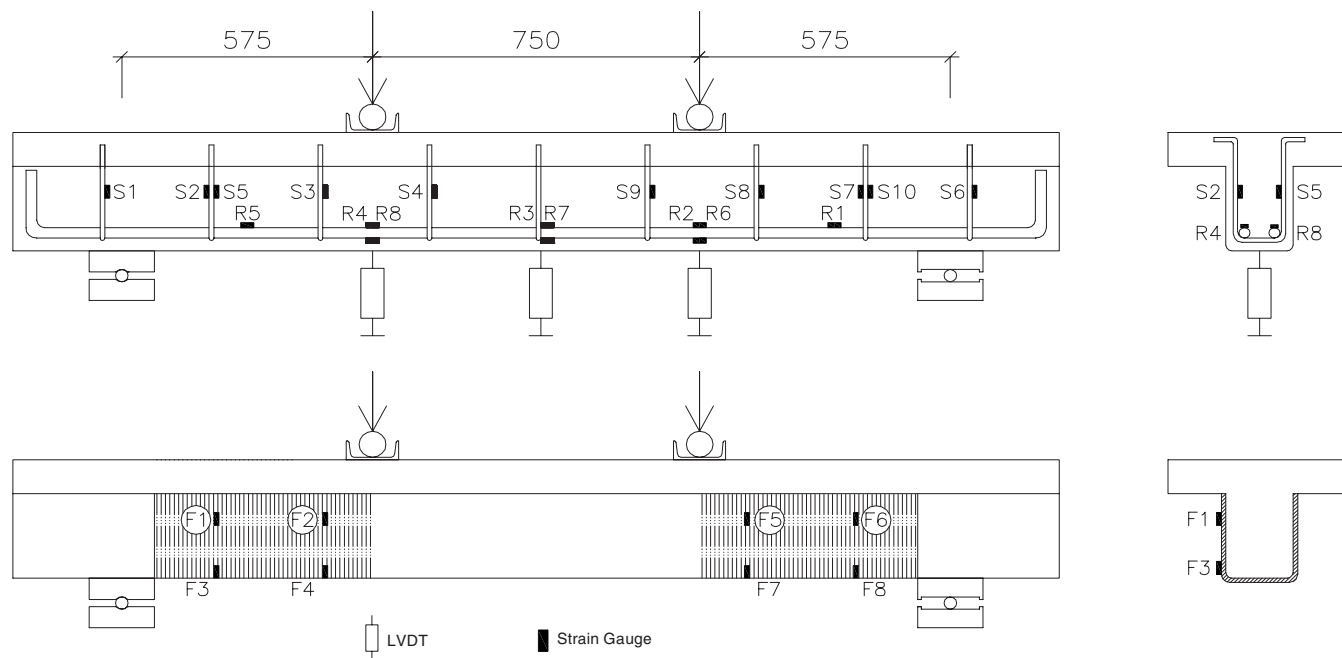


Fig. 5. Test setup and instrumentation of the T-beams

The shear cracks were initiated at the middle of both shear spans concurrently. As the load increased, the major crack started to extend and propagate upward through the flange toward the loading point leading to a brittle failure at an ultimate load of 202 kN.

While beam S-E-B was being loaded, failure started by debonding of the CFRP sheets over the major shear crack in the same location observed in beam S-C-O; though, subsequent to the web shear cracks, a vertical crack was created on the top of the flange near the support, and then it spread descending. The bonded CFRP sheet eventually unzipped vertically, as shown in Fig. 6(b). This effect is explained by the strain compatibility between the flange and the web. When the concrete strut formed in the web, it created a secondary effect in the top flange. As the applied load increased, the bottom of the beam close to the support attempted to rotate, but the large, wide flange restrained the movement. This led to horizontal tensile strains in the top part of the flange near the loading point. Finally, these strains reached the ultimate tensile strength of the concrete. A vertical crack then generated from the top of the flange and propagated downward through the flange thickness to the web and the CFRP sheets, where a vertical tearing of the fibers occurred. The same failure mode was reported by Khalifa and Nanni (2000), Adhikary and Mutsuyoshi (2004), Deniaud and Cheng (2003), and Cao and Teng (2005). The load carrying capacity of beam S-E-B was 256 kN with a 27% increase in the shear capacity compared to the control beam S-C-O. If debonding could be re-



Fig. 6. Cracking and failure pattern of beams: (a) S-C-O; (b) S-E-B; and (c) S-M-D

Table 5. Experimental Results

Beam	Shear strength (kN)	Increase (%)	Failure mode
S-C-O	101	—	Diagonal shear
S-E-B	128	27	Shear crushing+CFRP sheet debonding
S-M-D	149	48	Shear crushing+horizontal cracks at top face

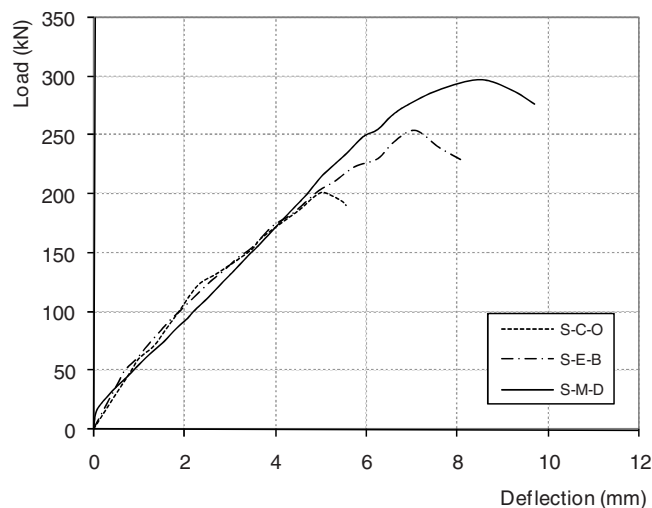


Fig. 7. Load versus midspan deflection

strained, a more effective utilization of the strengthening material and as a result a higher increase in shear capacity of the original RC beam would have been achieved.

As a result of the use of the newly proposed mechanical end-anchors, a significant increase in the shear capacity was achieved in the beam S-M-D by strengthening the web with the dry CF sheet by means of end-anchors. Although the reinforcement and strengthening of the beam S-M-D were designed such that the contribution of the anchored CF strengthening system could be quantified before reaching the yielding strength of the beam, yet the flexural steel started yielding and gained some ductility to the specimen before the failure. Due to the usage of the U-shaped dry CF sheet, the CF vertical fibers were able to bridge the diagonal shear cracks. As such, the abrupt debonding of the CFRP sheet used in the strengthening of specimen S-E-B was successfully prevented. Failure of the beam S-M-D occurred due to the formation of a major shear crack in the concrete beam, as shown in Fig. 6(c). The load carrying capacity of the beam S-M-D was 298 kN which represents an increase in the shear capacity of 48% compared to the control beam S-C-O and 16% higher ultimate failure load over the CFRP bonded specimen S-E-B. It is anticipated that the contribution of the new U-shaped dry CFRP sheet strengthening system would have been higher if the beam S-M-D did not reach yielding of the longitudinal rebars. The shear strength of T-beam S-M-D ($V_n=149$ kN), is experimentally observed to be more than the shear strength of a virtual T-beam with the same size and shape that is design adequately in shear ($V_n=141$ kN) based on shear reinforcement limits of Canadian code CSA A23.3-04 (10M at $0.7d_v=170$ mm), where d_v is the effective shear depth.

Load-Displacement Behavior

The load versus midspan deflection curves of the three beams are shown in Fig. 7. From the figure, it could be seen that the beam S-M-D strengthened by the mechanically anchored dry CF sheets demonstrated negligible higher stiffness compared to the beam S-E-B strengthened by the bonded CFRP sheets. In addition, the figure shows that the beam S-M-D experienced more ductility than the beam S-E-B. The supplemental ductility was gained from the flexural steel yielding. The midspan deflection of the beam S-M-D at the maximum load was 1.21 times (8.5 mm at

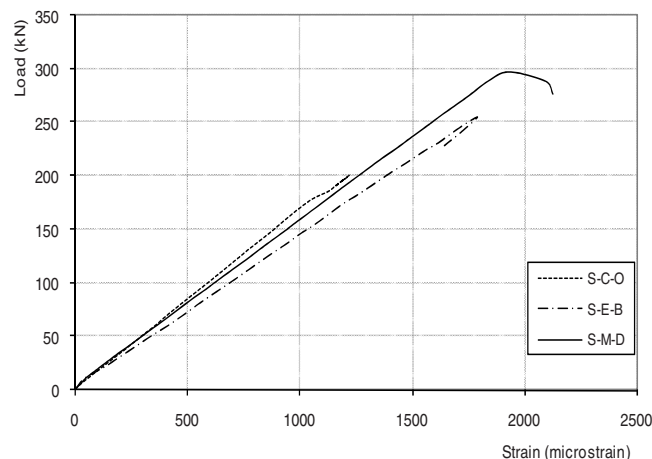


Fig. 8. Load versus strain in flexural reinforcement

load 298 kN/7 mm at 256 kN) the deflection of the beam S-E-B at the maximum load, whereas the beam S-C-O had the smallest deflection at the maximum load (5 mm at 202 kN). All the beams excluding the beam S-M-D exhibited brittle behavior, a sudden drop can be observed in the load-displacement curves after the peak load. Beam S-M-D slightly behaved in a ductile manner before it suddenly failed in shear.

Load-Strain Behavior

Fig. 8 compares the longitudinal strain in flexural reinforcement at midspan for all beams. The beam S-M-D partly showed yielding of longitudinal reinforcement at the maximum moment region, which enabled the beam to exhibit a very slight ductile behavior; even though it ultimately failed in shear. The remaining beams did not show yielding of longitudinal reinforcement. It was also observed that the beam S-E-B showed 13% lower maximum strain in longitudinal tensile reinforcement compared to beam S-M-D.

Fig. 9 shows the load versus vertical strain in the CF sheets relationships of the two strengthened beams. The shown CF strains are those of the strain gauge that had the highest recorded strains just before failure. Test measurements shown in Fig. 9 shows that there were strains developing in the FRP in beam S-M-D slightly after the beginning of loading. Therefore, it could be said that the system starts contributing at service stage.

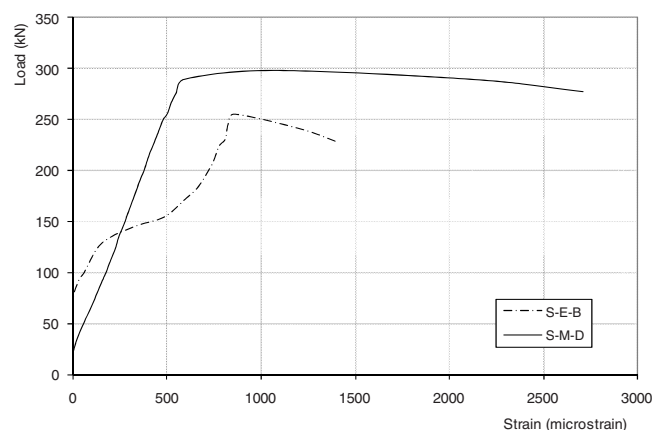


Fig. 9. Load versus vertical strain in CF sheets

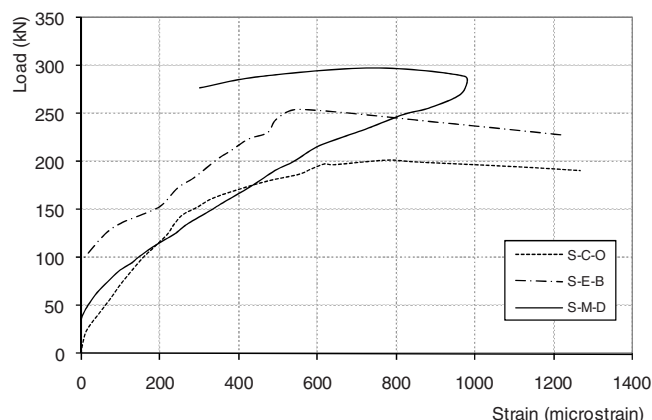


Fig. 10. Load versus vertical strain in steel stirrup

It is to be noted that, although these strain gauges are installed in the shear constant zone where the maximum shear forces and strains in CF occur, these strain gauges may not necessarily show the maximum values developed in the sheet; rather, they show the strains where the gauges were attached. In order not to alter the strength of dry CF sheets, the amount of the strain gauges were limited, since surface preparation on the dry CF to install strain gauges may influence the strength of the material. Therefore, the measured strains shown in Fig. 9 represent those only at the instrumented fiber bands. Since the used dry CF is uniaxial and there is no significant load transfer perpendicular to major axis, the behavior of the fibers could be greatly independent from each other. The only element that helps the fibers bridge the shear cracks and transfer contribution of CF in shear is the 25.4 mm (1 in.) steel rod at the end of the U-jacket.

In beam S-E-B it is seen that the measured strain increment was very small prior to diagonal cracks. The strain measurements on CFRP sheets indicate that CFRP slightly participated in resisting shear in service stage. However, once the beams developed diagonal cracks, there was a rapid increase in strain. The strain in the bonded CFRP sheets of the S-E-B specimen increased until the CFRP debonded and the beam failed at 256 kN. On the other hand, the strain in the dry CF used in strengthening beam S-M-D increased gradually from the beginning of the loading. Prior to the failure, the strain in the dry CF increased with a high rate until the beam eventually failed at 298 kN. The beam S-M-D showed relatively higher strains indicating the maximum utilization of the strengthening sheet before the failure. The beam S-E-B developed a strain of 1,402 microstrains in the sheet, which is about 13% of the ultimate strain for the CFRP (1.05%; maximum elongation for the wet CFRP) used in this study. The measured local maximum vertical strain of the dry CF anchored U-jacket was about 4,200 or 25% of ultimate strain (1.7%; maximum elongation for dry CF sheet). This value is not an absolute value because it depends greatly on the location of the strain gauge with respect to a crack since there might be higher strains in the fibers without strain gauges.

The maximum measured strain in the steel stirrup of the three specimens is shown in the Fig. 10. Comparing the strain in the stirrups of the three beams, it could be seen that beam S-C-O has relatively higher strains in its stirrups, compared to the strengthened beams, in the early stages of load application (force range from 50 to 100 kN). The vertical strain increases gradually in the steel stirrup of the beam S-C-O until it reaches its maximum shear capacity and it fails at 202 kN. A sudden increment of vertical

strain can be seen in the stirrup after the load passes 200 kN (Fig. 10). From the figure, it could be seen that for the same level of force, the stirrup with the maximum vertical strain in the beam S-C-O has noticeably higher values in comparison with that in the beam S-E-B. This behavior is expected since beam S-E-B benefits from the contribution of concrete (V_c), steel stirrup (V_s), and the CFRP sheet (V_{FRP}) in the shear resistance while the beam S-C-O resists shear through the V_c , and V_s mechanisms. Therefore the difference between these values of strain in the stirrups of the beam S-C-O and S-E-B can be explained as the contribution of bonded CFRP in shear resistance of the RC beam. After load reached 250 kN the CFRP sheets debonded and the beam S-E-B loses the V_{FRP} contribution completely. Consequently, the strain in the stirrups of beam S-E-B increased immediately and the beam fails in shear at load 256 kN.

Comparing the strains in the stirrups of beams S-C-O and S-M-D, it could be seen that the stirrup with the maximum vertical strain in the beam S-C-O and that in the beam S-M-D follow a similar pattern until the load reaches approximately 150 kN. This could be explained that although a continuous CF sheet was used in strengthening beam S-M-D, yet during the test it has been observed that the dry CFs were significantly stretched in the middle of the spacing of the internal steel stirrups in comparison to the fibers in other locations. This could be interpreted that the dry CF sheets are contributing to the shear capacity of the beam through the formation of a truss pattern, where the highest tensile stresses occurs at the middle of the spacing of the stirrups. After the load passes 150 kN the strain in the stirrup of the beam S-M-D increases until the force reaches 280 kN. A significant drop in the vertical strain of the steel stirrups of the beam S-M-D can be seen in the Fig. 10 while the force reaches 290 kN. At the same force level there is a significant increment in the strains of the U-shaped dry CF sheet (Fig. 9). At this stage, the concrete aggregates lose their integrity and the shear contribution of concrete is highly decreased. The RC T-beam is one step to final shear failure. The steel stirrups interaction with the concrete is decreased. Hence, the strain in steel stirrups drops significantly (Fig. 10). Since the dry CF U-jacket is anchored to confined concrete above the flange, the dry CF strengthening system keeps functionality even after the web concrete fails in shear. Fig. 9 shows that the CF strain at the same time increases considerably to confine and hold the concrete as much as possible. Although the process leads to beam's final shear failure, the mechanically anchored dry CF U-jacket acted as a safety mesh to hold the beam even after its shear failure.

Comparison between Experimental and Analytical Results

Due to the brittle nature of the failure in the beams strengthened in shear, the prediction of the beams' force-deformation response prior to failure was not considered. The goal was to predict the shear force capacity of each beam using analytical formulas provided by CSA A23.3-04 (CSA 2004) and CSA S6-06 (CSA 2006) design codes. According to CSA S6-06, the nominal shear resistance V_n is determined by

$$V_n = V_c + V_s + V_{FRP} \quad (1)$$

where V_c =nominal shear resistance provided by the concrete mechanism; V_s =nominal shear resistance provided by shear reinforcement; and V_{FRP} is the nominal shear resistance attributed to

Table 6. Calculated and Experimental Shear Strengths (in kN) for Beams Strengthened in Shear

Beam	Shear mechanisms			Shear strength V_n		Calculated/ experimental
	V_c	V_s	V_{FRP}	Calculated (kN)	Experimental (kN)	
S-C-O	35	83	—	118	101	1.17
S-E-B	36	83	30	149	128	1.16
S-M-D	36	83	40	159	149	1.07

the FRP. V_c and V_s are calculated using CSA A23.3-04 (CSA 2004).

In the analytical calculation of V_n a simplified analysis is used. This simplified analysis assumes the simultaneous occurrence of the nominal capacity of the steel stirrups and the dry CF. More refined model by Hoult and Lees (2009) explores relative contributions of concrete, steel stirrups, and external prestressed CFRP straps on the nominal shear resistance of RC T-sections. In the analytical calculation of V_{FRP} in beam S-M-D the same formulas by CSA S6-06 were used since the analogy is assumed to be similar

$$V_{FRP} = \frac{E_{FRP} \epsilon_{FRP} t_{FRP} w_{FRP} d_{FRP}}{s_{FRP}} \quad (2)$$

where E_{FRP} =modulus of elasticity of dry fiber in MPa (Table 2); ϵ_{FRP} =effective strain in dry fiber (measured from Fig. 9 equal to 2,500 μ strains); t_{FRP} =total thickness of externally dry fiber sheets (calculated using the data in Table 2 equal to 0.175 mm); w_{FRP} =width of dry fiber strip (equal to 1 for continuous fiber sheets); d_{FRP} =effective shear depth of dry fiber sheet; and s_{FRP} =spacing between strips (equal to 1 for continuous fiber sheets).

Table 6 summarizes both the analytical predictions and the experimental results for the beams strengthened in shear. The degree of correlation between the predicted and experimental results is calculated as a ratio of predicted values to experimental values. As can be seen from the table, the shear strength analytical values are higher than the experimental failure loads up to 17% for the three tested beams. The proposed model for calculating the V_{FRP} , and consequently V_n , for beam S-M-D showed a good correlation with the shear capacity reached during the test.

Conclusions

Three tests were conducted as a proof-of-concept in effectiveness of a proposed method of increasing the shear capacity of the RC beams strengthened with the dry CF U-wraps. In this method, dry CF sheets are wrapped around and bonded to two steel rods. Then the rods are anchored to the corners of the web-flange intersection of the T-beam with mechanical bolts. This makes a U-shaped dry CF jacket around the web which increases the shear strength of the T-beam using the privilege of higher tensile strength and modulus of elasticity of dry CF in comparison with wet (bonded) CFRP. It should be noted that, since dry CF sheet is exposed, similar to an epoxy-bonded FRP sheet, a protective cover could be used. Also, the anchors and anchor rod should be made of corrosion-resisting steel.

For a beam strengthened with the CFRP bonded wrap, the shear capacity increased but the failure was still governed by the debonding of the CFRP. In the specimen where the mechanical dry CF was used, the shear capacity of the member further in-

creased. The beam strengthened using mechanically anchored dry CF wrap showed 48% increase in the shear capacity as compared to the control beam without any strengthening. Moreover, it showed 27% increase in the shear capacity as compared to the beam strengthened using the CFRP epoxy-bonding method. The beam also sustained the load up to almost yielding of the flexural tensile steel reinforcement. A rough cost estimation on the two strengthening methods used in this research reveals that, although mechanically anchored method is cost saving regarding the time and labor needed for the surface preparation of the concrete, the two methods are approximately similar economically. While, it is believed that, industrial fabrication of mechanically anchored CF U-jacket will significantly decrease the expenses of the proposed system. From the outcomes of this experimental research, the following can be deduced:

1. The mechanically anchored dry CF jackets can increase the ultimate shear strength of RC T-beams and slightly improve the flexural stiffness.
2. The use of mechanical anchorage eliminates the debonding of the CF jacket, and consequently results in a better utilization of the full capacity of the CF sheet.
3. The proposed strengthening method using mechanically anchored U-shaped dry CF sheets needs relatively less hard work and time consuming concrete surface preparation compared to conventional externally bonding methods.

While supplementary validation tests need to be conducted to completely understand the performance of RC beams strengthened with the proposed system, there are promising evidences that the new hybrid mechanically anchored dry CF U-jacket can make the FRP strengthening more attractive yet potentially economical for the retrofit of concrete structures.

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Notation

The following symbols are used in this paper:

- d_{FRP} = effective shear depth of dry fiber sheet (mm);
- d_v = effective shear depth (mm);
- E_{FRP} = modulus of elasticity of dry fiber (MPa);
- s_{FRP} = spacing between strips (mm);
- t_{FRP} = total thickness of externally dry fiber sheets (mm);
- V_c = shear resistance provided by tensile stresses in concrete (kN);
- V_{FRP} = shear resistance provided by tensile stresses in FRP (kN);

V_n = nominal shear resistance of the beam (kN);
 V_s = shear resistance provided by shear steel reinforcement (kN);
 w_{FRP} = width of dry fiber strip (mm); and
 ϵ_{FRPe} = effective strain in dry fiber.

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