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Effect of Steel Stirrups on the Shear Resistance Gain Due to EB FRP Strips and Sheets

Amir Mofidi¹ and Omar Chaallal, MACI²

ABSTRACT

This paper presents the results of an experimental investigation of reinforced concrete (RC) T-beams strengthened in shear with externally bonded (EB) carbon-fiber-reinforced polymer (CFRP) strips and sheets. The main objective of this study was to gain insight, by varying the test parameters, into the interaction between the internal transverse-steel reinforcement and the externally bonded CFRP used for strengthening of RC beams in shear. The test parameters of this study were: 1) the CFRP ratio (i.e., the spacing of the CFRP strips); 2) the presence or absence of transverse steel; 3) the transverse-steel ratio (that is, the spacing of the stirrups); and 4) the use of CFRP strips versus CFRP sheets. In total, 10 tests were performed on 4520-mm-long (14 feet and 10 inches) T-beams. The study showed that the presence of internal transverse-steel reinforcement resulted in a significant decrease in the gain due to CFRP for all the strengthened specimens. It also showed that the steel yielded before failure for all the test specimens.

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specimens with transverse steel. Finally, the presence of CFRP did not result in a significant
decrease in transverse-steel strain. It can be concluded that the contribution of transverse steel to
shear resistance is not affected by the addition of EB CFRP. These results are in good agreement
with the assumptions made by existing codes and design guidelines, which are based on the
yielding of transverse steel at ultimate strain for RC beams strengthened in shear with EB CFRP.

Keywords: Carbon-fiber-reinforced polymer; Reinforced concrete beam; CFRP sheet; Strip;
Shear strengthening; Transverse steel reinforcement.

INTRODUCTION

In recent years, shear strengthening of reinforced concrete (RC) beams with externally
bonded (EB) fiber-reinforced polymer (FRP) material has attracted attention and has been studied
by several researchers (e.g., Uji 1992; Chaallal et al. 1998; Triantafillou 1998; Khalifa et al.
1998; Bousselham and Chaallal 2004; Chaallal et al. 2011; Mofidi and Chaallal 2011a,b). These
experimental and analytical studies have provided valuable insights and results. However, several
questions still linger in the area of shear strengthening of RC beams with FRP composites
(Bousselham and Chaallal 2004; Mofidi and Chaallal 2011a).

For instance, a comparison between the experimental shear resistance due to FRP and the
values predicted by the analytical models used in existing codes and guidelines shows that major
aspects of shear strengthening with FRP material are still not captured by the predictive models used
in the codes and guidelines (Bousselham and Chaallal 2008; Mofidi and Chaallal 2011a). This is
mainly because the calculated shear contribution of FRP according to the codes and guidelines does
not account for the effect of certain parameters that have been found experimentally to have a major
influence on the contribution of FRP to shear resistance. The effect of internal transverse-steel reinforcement is one of these influential parameters. The adverse effect of transverse steel on the effectiveness of externally bonded FRP used for shear strengthening and retrofit of RC beams is well established (Bousselham and Chaallal 2008; Chaallal et al. 2011; Mofidi and Chaallal 2011a,b). In contrast, the effect of adding EB FRP for shear retrofit on the performance of internal steel stirrups has not been thoroughly documented. Moreover, in most modern codes and guidelines, the contribution of steel stirrups to the shear resistance $V_s$ at the ultimate state is calculated on the premise that the steel stirrups have yielded. However, the premature debonding failure observed in RC beams strengthened in shear with FRP has prompted legitimate questions and concerns as to whether the assumption that the steel stirrups yield before failure holds true (e.g., Chen et al. 2010).

RESEARCH SIGNIFICANCE

So far researchers have not reached a commonly accepted agreement on the effect of transverse steel in RC beams retrofitted with FRP. Therefore, this effect is not yet considered in the design codes and guidelines, including ACI 440.2R-08. Consequently, current design codes and guidelines may overestimate the shear resistance of RC beams with transverse steel that are strengthened using EB FRP sheets/strips. Such uncertainties in shear strengthening of RC beams using EB FRP have provided the key impetus for conducting the current research study, the objective of which was to gain insight into the interaction between internal transverse-steel reinforcement and externally bonded FRP strips and sheets used for shear strengthening of RC beams. This insight has been achieved based on results obtained from an experimental program carried out on full-size T-beam specimens, as described below.
EXPERIMENTAL PROGRAM

The experimental program (Table 1) involved 10 tests performed on full-scale RC T-beams. The control specimens, which were not strengthened with carbon FRP (CFRP), were labeled NF (for No Frp), whereas the specimens retrofitted with a layer of EB CFRP sheet were labeled SH (for SHEet), and the specimens strengthened with FRP strips (strip width = 87.5 mm = 3 7/16 inches) were labeled ST (for STRips). Specimens strengthened with narrowly spaced FRP strips (spacing equal to 125 mm- approximately 5 inches) were labeled HF (for Heavily-strengthened with FRP), whereas the specimens strengthened with widely spaced strips (spacing equal to 175 mm or 6 7/8 inches) were labeled LF (for Lightly-strengthened with FRP). Series NR (Not Reinforced with transverse steel) consisted of specimens with no internal transverse-steel reinforcement (that is, no stirrups). Series HR (Heavily Reinforced with transverse steel) and MR (Moderately Reinforced with transverse steel) contained specimens with internal transverse-steel stirrups spaced at $s = d/2$ and $s = 3d/4$ respectively, where $d = 350$ mm = 13 3/4 inches represents the effective depth of the cross section of the beam. Therefore, for instance, specimen NR-ST-HF featured a beam with no transverse steel retrofitted using CFRP strips spaced at 125 mm (approximately 5 inches). The specimen details are provided in Table 1, together with the identification codes used hereafter.

Description of specimens

The T-beams were 4520 mm (14 feet and 10 inches) long, and their T-section had overall dimensions of 508 mm (20 inches) by 406 mm (16 inches). The width of the web and the thickness of the flange were 152 mm (6 inches) and 102 mm (4 inches) respectively (Fig. 1a-b). Note that the web of the strengthened beams is chamfered at the outer corners. The longitudinal
Steel reinforcement consisted of four 25M bars (diameter 25.2 mm or 1 inch, area 500 mm² or 0.78 square inches) laid in two layers at the bottom and six 10M bars (diameter 10.3 mm or approximately 0.4 inches, area 100 mm² or 0.16 square inches) laid in one layer at the top. The bottom bars were anchored at the support with 90-degree hooks to prevent premature anchorage failure. The internal steel stirrups (where applicable) were 8 mm or 5/16 inches in diameter (area 50 mm² or 0.08 square inches).

To apply the EB FRP sheets/strips to the RC specimens the following steps were implemented: (1) the area of the specimens where the CFRP sheets/strips was to be epoxy-bonded was sand-blasted to remove any surface cement paste and to round off the beam edges; (2) the specimen corners were chamfered to provide a radius of 12.7 mm (0.5 inches) in order to avoid stress concentration in the FRP sheets during the tests; (3) residues were removed using compressed air; and (4) layers of U-shaped CFRP sheets/strip were glued to the bottom and lateral faces of the RC beam using a two-component epoxy resin.

Materials

A commercially available concrete delivered to the structural laboratory by a local supplier was used in this project. The average 28-day concrete compressive strength was 25 MPa (1 MPa is equal to 145 psi), which is very close to the average compressive strength of 27 MPa obtained during the tests. It should be noted that the specimens of the MR series were cast using a different concrete batch, the compressive strength of which was 35 MPa. The scatter between the results of compression tests on the cylinder specimens was insignificant.

The longitudinal steel reinforcement consisted of 25 M bars (modulus of elasticity 187 GPa (1 GPa is equal to 145000 psi), and yield stress 500 MPa), and the transverse-steel reinforcement
consisted of deformed 8-mm or 13/16 inches bars (modulus of elasticity 206 GPa and yield stress 650 MPa).

The composite material was a unidirectional carbon-fiber fabric epoxy-bonded over the test zone in a U-shape around the web (Fig. 2). The dry CFRP sheet had an ultimate tensile strength of 3,450 MPa, an elastic modulus of 230 GPa, and an ultimate strain of 1.5%, as reported by the manufacturer. The thickness of the CFRP fabric used was 0.11 mm.

Test setup

All ten tests were conducted in three-point load flexure (Fig. 1-b). This loading configuration was chosen because it enabled two tests to be performed on each specimen. Specifically, while one end zone was being tested, the other end zone was overhung and unstressed. The load was applied at a distance $a = 3d$ from the nearest support, a configuration which was representative of a slender beam.

Instrumentation

The measuring equipment used in this research study was carefully designed to meet the objectives of this study. The vertical displacement was measured at the position under the applied load and at midspan, using a linear variable differential transformer (LVDT) with a 150-mm (5 7/8 inches) stroke. Different types of strain gauges were installed on the longitudinal reinforcement, on the steel stirrups, and embedded in the concrete to measure the strains experienced by the various materials as the loading increased and to monitor thereby the yielding of the steel. The strain gauges on the stirrups were installed along the anticipated plane of the major shear crack. Displacement sensors, also known as crack gauges, were used to measure the
strains experienced by the CFRP strips and sheets (Figs. 2 and 3). These gauges were fixed vertically onto the lateral faces of the specimens at the same location as the transverse steel. The load was applied using a 2000-kN (449 kips) capacity MTS hydraulic jack. All tests were performed under displacement-control conditions at a rate of 2 mm/min (approximately 3/16 inches).

ANALYSIS OF RESULTS

All the specimens failed in shear. The control specimens failed due to diagonal tension failure of concrete cross section. The specimens strengthened with CFRP failed by premature FRP debonding followed by diagonal tension failure (Figs. 4a-d). Local CFRP fracture was observed in few specimens (NR-ST-LF and NR-ST-HF); this local failure is attributed to stress concentration at the web corners.

Deflection response

Figure 5 compares the deflection response for RC beams without transverse-steel reinforcement. It reveals that the NR-SH and NR-ST-HF specimens exhibited slightly greater overall stiffness than the other beams. Specimen NR-ST-HF exhibited the highest deflection at the loading point and a higher maximum load at failure than the other specimens (Fig. 5). The beams strengthened with FRP strips exhibited more deformability than the beams strengthened with FRP sheet. This occurred mainly because in RC beams strengthened with FRP strips, local FRP debonding did not result in a complete debonding failure. Each local strip-debonding event resulted in a drop in the load-carrying capacity of the beam (see Figure 5), but the load continued to increase as the cracks propagated in the RC beams web, engaging the unloaded CFRP strips in
their path. In specimens strengthened with FRP strips, and unlike beams strengthened with FRP sheets, local debonding of FRP cannot propagate from one FRP strip to the next. Therefore, using FRP strips results in a more progressive type of failure, and a sudden and brittle failure is prevented.

Figure 6 shows the load versus maximum deflection curves for RC beams with transverse-steel reinforcement. It reveals that each of the specimens in the HR and MR series exhibited an overall stiffness relatively similar to that of the other beams. The maximum load at failure and the maximum deflection attained at the loading point for each specimen are provided in Table 1. Specimen HR-ST-HF reached the highest maximum load at failure. Meanwhile, the HR-ST-LF and HR-SH specimens exhibited a slightly higher deflection at the loading point than the other strengthened and unstrengthened specimens (Table 1). It should be mentioned that in Table 1, the shear contributions of concrete and steel were calculated based on the measured experimental results for the control beams.

**Strain analysis**

This part of the study investigated the behavior of CFRP and transverse steel during loading of the specimens. As mentioned previously, extensive instrumentation for strain monitoring was carefully planned and implemented to provide the data needed to gain a better understanding of the effect of transverse steel on the contribution of FRP to the shear resistance of RC beams retrofitted in shear with EB FRP.

**CFRP strain**

The distribution of the maximum strains attained in the CFRP is shown in Figure 7 for all the strengthened test specimens. Note that these strain values are the maximum measured values,
but not necessarily the absolute maximum values, experienced by the CFRP U-jackets. The two values may differ in cases where the strain gauges did not intercept the main cracks. From Figure 7, the following observations can be made:

1. All the curves presented in these figures show that the CFRP did not contribute to load-carrying capacity in the initial stage of loading.

2. For specimens of series NR, the measured strains were greater for specimens strengthened with FRP strips than for similar beams strengthened with FRP sheet.

3. For the beams strengthened with a layer of FRP sheet (NR-SH, MR-SH, and HR-SH), the maximum strain in the CFRP increased as the amount of transverse reinforcement was increased. In fact, for specimen HR-SH, the maximum strain attained by the FRP sheet was approximately 48% of the ultimate strain value, whereas the corresponding maximum strain values for specimens NR-SH and MR-SH were 17% and 27% respectively of the ultimate strain.

4. For specimens strengthened with FRP strips in the NR series (no steel stirrups), the maximum measured FRP strain values were approximately equal. This also holds true for the specimens strengthened with FRP strip in the HR series.

5. For all specimens strengthened with FRP strips in both NR and HR series, the maximum FRP strain was greater than 5000 με. It may be of interest to note that ACI 440.2R-08 limits the maximum design FRP strain value to 4000 με. On the basis of the results achieved in this study, this limit appears to be conservative for RC beams strengthened with FRP strips.

Transverse-steel reinforcement strain

Figure 8 shows the measured strain in the transverse-steel reinforcement for the test specimens with internal steel reinforcement. The vertical line identifies the strains corresponding
to the yielding of the transverse steel, as obtained by tests ($\varepsilon_y = 3250 \mu\varepsilon$). From Figure 8, the following observations can be made:

1. Like the CFRP, the steel stirrups did not contribute to load-carrying capacity in the initial stage of loading. The transverse-steel contribution to shear resistance started after the formation of diagonal cracking initiated.

2. All the monitored stirrups were significantly strained. This is also reflected by the cracking pattern observed in the beams with transverse steel (see Figure 4a-d).

3. Yielding of transverse steel was observed in all cases. This observation is in good agreement with existing code specifications and guidelines, which assume that the transverse steel yields at ultimate strain for RC beams strengthened in shear with EB FRP.

Figure 8 shows that addition of EB FRP did not result in a decrease of transverse-steel strain. For all the specimens with transverse steel, the steel yielded well before the RC beam reached ultimate failure. Therefore, it can be concluded that at the ultimate state the contribution of internal steel stirrups to shear resistance was not affected by the addition of externally bonded FRP. It follows that the shear contribution of internal steel reinforcement ($V_s$) should be calculated using the same formula for both FRP-strengthened and unstrengthened RC beams, which confirms the assumptions of the design guidelines (ACI 440.2R-08; CSA S806-02; HB 305-08).

These results are not in agreement with those based on finite-element simulations reported by Chen et al. (2010) and Chen et al. (2012); these researchers found that for RC beams strengthened in shear with FRP, the internal steel stirrups did not reach the yield point. Based on their finite-element model, they concluded that the yield strength of the internal steel stirrups in such strengthened RC beams cannot be fully utilized. The models by Chen et al. (2010) and
Chen et al. (2012) were originally generated based on a single crack failure pattern assumption. Single crack failure pattern was adopted by most design models to simplify the calculation and design of strengthening FRP sheets/strips. However, experimental observations clearly show that for RC beams strengthened with EB FRP the cracking pattern on the FRP/concrete interface is rather distributed (Mofidi and Chaallal 2011a). Eventually, the distributed cracks at the concrete cover merge together at the concrete core to form one major shear crack at ultimate. Therefore, it is believed that assuming a single crack pattern is overly simplistic when considering such a precise finite element modeling tool. Considering the fact that the crack width plays a governing role in the Chen et al. (2010) and Chen et al. (2012) models, the discrepancies between the results produced by Chen et al. (2010) and Chen et al. (2012) models and the experimental results are to be expected.

Shear resistance under increasing load

In accordance with most codes and standard guidelines, the nominal shear resistance ($V_n$) can be expressed as follows:

$$V_n = V_c + V_s + V_f$$  \hspace{1cm} (1)

The experimental contribution of transverse steel ($V_s$) is calculated as the sum of the contributions corresponding to the stirrups crossing the plane of rupture using the following equation:

$$V_s = A_s E_s \sum \varepsilon_{s,i}$$  \hspace{1cm} (2)

where $A_s$ is the section area of one stirrup; $E_s$ is the elastic modulus of the transverse steel; and $\varepsilon_{s,i}$ ($\leq \varepsilon_y$) is the measured strain in stirrup $i$ in the failure zone, where $\varepsilon_y$ is the yield strain of the stirrups.
The experimental contribution of FRP ($V_f$) can be calculated as follows:

$$V_f = 2E_f t_f \sum (w_i \epsilon_{f,i})$$  \hspace{1cm} (3)

where $E_f$ is the elastic modulus of the CFRP; $t_f$ is the thickness of the CFRP; $\epsilon_{f,i}$ is the measured strain in the CFRP corresponding to instrumented section $i$ in the failure zone; and $w_i$ is the tributary width of the strengthening FRP strips intercepted by the major shear crack, where the CFRP strain $\epsilon_{f,i}$ is assumed constant. The CFRP strengthening width represents the portion of the CFRP that effectively contributes to shear resistance.

Figure 9 shows the experimental evolution under increasing load of the contributions to the shear resistance of the two components ($V_s$ and $V_f$) for beams HR-NF, HR-ST-LF, HR-ST-HF, and HR-SH. The specimens shown in this figure had the same degree of transverse-steel reinforcement (highly reinforced), but were strengthened using different amounts of externally bonded FRP strips and sheet. The behavior of the transverse steel under increasing load exhibited a similar pattern for both the unstrengthened beam (HR-NF) and the beams retrofitted with different amounts of EB FRP strips and sheet (HR-ST-LF, HR-ST-HF, and HR-SH). This result indicates that strengthening of RC beams with EB FRP does not alter the behavior of internal transverse steel. It also reveals that addition of EB FRP does not attenuate the shear contribution of the transverse-steel reinforcement.

Figure 10 shows the experimental progression of the shear contributions of the two components ($V_s$ and $V_f$) under increasing load for specimens HR-NF, MR-SH, and HR-SH. The strengthened specimens illustrated in this figure were both retrofitted with one layer of CFRP sheet, but reinforced with different amounts of transverse-steel reinforcement. The behavior of the FRP under increasing load followed different patterns for the strengthened beams depending on the amount of internal transverse-steel reinforcement (MR-SH and HR-SH). This clearly
shows that the behavior of EB FRP depends on the amount of transverse steel used in RC beams. This result confirms that increasing the amount of transverse steel leads to a reduction in the contribution of the FRP during loading and at the ultimate state.

5 COMPARISON OF TEST RESULTS WITH SHEAR DESIGN EQUATIONS

The shear resistance due to CFRP as obtained by tests ($V_{f,exp}$) is compared in Table 2 to the nominal shear resistance ($V_{f,cal}$) predicted by ACI 440.2R (2008), HB 305 (2008), and Mofidi and Chaallal (2011a).

Mofidi and Chaallal (2011a) proposed a model for calculating the contribution of FRP to shear resistance, taking into consideration the attenuating effect of transverse steel as well as of the cracking pattern on the EB FRP contribution in shear. Based on their study, it was determined that the presence of transverse steel favors the formation of a multi-line shear-cracking pattern in the RC beam, which decreases the anchorage length of the FRP fibers and hence the available effective width of FRP ($w_{fe}$) and bonding area between the FRP and the concrete. In the calculation of $w_{fe}$, it is assumed that the cracking pattern of the RC beam becomes more propagated with the increase in the amount of internal steel and external FRP shear reinforcement as measured by their respective rigidities. On the other hand, the cracking pattern influences the anchorage length of the fibers. As the cracking pattern becomes more propagated, fewer fibers will provide the minimum effective anchorage length. Therefore, the effective width, that is the width of the fibers long enough to attain the effective anchorage length, decreases. Using a computational analysis based on the available test results in the literature (see Mofidi and Chaallal (2011a)) the effective width is defined as a function of the
sum of the rigidities of transverse steel reinforcement and that of transverse FRP sheets (Eqs. 9 and 10).

\[ w_{je} = \frac{0.6}{\sqrt{\rho_f \cdot E_f + \rho_s \cdot E_s}} \times d_f \] for U-Jacket (4)

\[ w_{je} = \frac{0.43}{\sqrt{\rho_f \cdot E_f + \rho_s \cdot E_s}} \times d_f \] for side bonded (5)

with \( w_{je} \) defined, the cracking modification factor can then be calculated as \( k_c = \frac{w_{je}}{d_f} \), i.e.,

\[ k_c = \frac{w_{je}}{d_f} = \frac{0.6}{\sqrt{\rho_f \cdot E_f + \rho_s \cdot E_s}} \] for U-Jackets (6)

\[ k_c = \frac{w_{je}}{d_f} = \frac{0.43}{\sqrt{\rho_f \cdot E_f + \rho_s \cdot E_s}} \] for side bonded (7)

The effect of \( k_L \) for beams with an anchorage length less than the effective length and that of \( k_f \) counting for the \( w_f/s_f \) ratio of the FRP strips are considered in the equation for effective strain as follows:

\[ \varepsilon_{fe} = \frac{k_c k_L k_f \cdot \tau_{ef} \cdot L_c}{t_f E_f} = 0.31 k_c k_L k_f \sqrt{\frac{\tau_{ef}}{t_f E_f}} \leq \varepsilon_{fu} \] (8)

Note that \( k_L \) and \( k_f \) can be calculated using Eqs. (9) and (11), as follows:

\[ k_L = \begin{cases} 1 & \text{if } \lambda \geq 1 \\ (2 - \lambda) \lambda & \text{if } \lambda < 1 \end{cases} \] (9)

where

\[ L_{\text{max}} = \begin{cases} \frac{d_f}{\sin \beta} & \text{for U-jackets} \\ \frac{d_f}{2 \sin \beta} & \text{for side plates} \end{cases} \] (10)

\[ k_f = \frac{1 + \left(\frac{w_f}{s_f} - \frac{1}{2}\right)^2}{1 - \left(\frac{w_f}{s_f} - \frac{1}{2}\right)^2} \] (11)
In addition, $V_f$ can be calculated as a function of $\varepsilon_{fe}$ using the following equation that accounts for the effect of the cracking angle, $\theta$ (The cracking angle can be taken equal to 45° for simplicity):

$$V_f = \frac{2t_f \cdot w_f \cdot \varepsilon_{fe} \cdot E_f \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha \cdot d_f}{s_f} = \rho_f \cdot E_f \cdot \varepsilon_{fe} \cdot b \cdot d_f \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha$$  \hspace{1cm} (12)

In all, Mofidi and Chaallal (2011a) model justifies premature FRP debonding in RC beams with internal transverse-steel reinforcement compared to beams with no transverse steel and explains the superior gain achieved due to FRP in beams with few or no steel stirrups compared to beams with moderate to high amounts of internal transverse steel.

In order to test the correlation between the experimental results and the predicted results by the models the best fit of the nominal predicted results versus the experimental results was considered. The following assumptions were made when calculating the experimental results: (i) the shear resistance due to concrete was assumed constant for beams with or without transverse steel reinforcement; (ii) the shear resistance due to concrete was assumed constant for both retrofitted and unstrengthened specimens; and (iii) the contribution of the transverse-steel was assumed constant for both retrofitted and unstrengthened specimens.

For specimens with no transverse steel strengthened using FRP strips (NR-ST-LF and NR-ST-HF), all three models underestimated the shear resistance due to FRP. This effect was more significant when using ACI-440.2R-08, where for NR-ST-HF for example, the shear resistance predicted was 28 kN, compared to 69.3 kN obtained by test (1 kN = 0.225 kips). On the one hand, for the specimen with no transverse steel strengthened using FRP sheet (NR-SH), the ACI-440.2R-08 and HB 305-08 models slightly overestimated the shear resistance due to FRP. On the other hand, the Mofidi and Chaallal (2011a) model slightly underestimated the FRP contribution.
to shear resistance (see Table 2). For all the strengthened specimens with transverse-steel reinforcement, the ACI-440.2R-08 and HB 305-08 models provided unconservative predictions and therefore overestimated results (Table 2). In contrast, results produced by Mofidi and Chaallal (2011a) for specimens strengthened with transverse-steel reinforcement correlate fairly well with the test results. Figure 11 shows that, the Mofidi and Chaallal (2011a) model predicted the experimental shear contribution of FRP \( R^2 = 0.81 \) with a high level of accuracy. However, the ACI-440.2R-08 and HB 305-08 models produced low coefficients of determination (0.04 and 0.03 respectively). In general current design guidelines models (including ACI-440.2R-08 and HB 305-08) fail to consider the effect of the transverse steel (see Mofidi and Chaallal 2011a-b). Therefore, they may predict conservative results for beams without transverse steel reinforcement. In contrast, current design guidelines models may overestimate the shear contribution of FRP for the specimens with transverse steel reinforcement and hence the shear resistance. Such unconservative results are exemplified in the results predicted by the ACI-440.2R-08 and HB 305-08 for the specimens HR-ST-LF, HR-ST-HF, HR-SH, MR-SH in the current study. The presence of the transverse steel has a significant effect on the shear resistance of RC beams strengthened with FRP, and therefore, should ultimately be considered in the design models.

CONCLUSIONS

This paper presents the results of an experimental investigation involving 10 tests on RC T-beams strengthened in shear with EB FRP strips and sheets. The effects of the following parameters were studied: 1) the CFRP ratio (i.e., the spacing of the CFRP strips); 2) the presence or absence of transverse steel; 3) the transverse-steel ratio (that is, the spacing between the
stirrups); and 4) the use of CFRP strips versus CFRP sheets. The following conclusions can be
drawn:

1. The addition of internal transverse-steel reinforcement resulted in a significant
decrease in the gain due to FRP for all the strengthened specimens;

2. For all the test specimens with transverse-steel reinforcement, the steel yielded before
the specimen failed. The presence of externally bonded FRP for shear retrofit did not
cause a significant decrease in transverse-steel strain. Overall, the contribution of steel
stirrups to shear resistance was not adversely affected by the addition of FRP;

3. Comparison of the resistance predicted by the ACI 440.2R-08, HB 305-08, and Mofidi
and Chaallal (2011a) models with test results showed that the guidelines failed to
capture the influence of transverse steel on the shear contribution of FRP. The model
proposed by Mofidi and Chaallal (2011a) showed a better correlation with
experimental results than the guidelines mentioned;

4. The maximum measured strain values in CFRP strips, and hence the gain in shear
strength due to CFRP strips, were significantly greater than for CFRP continuous
sheets. In addition, the maximum deflection was slightly greater for beams retrofitted
with CFRP strips than for beams strengthened with continuous CFRP sheets;

5. In all the specimens strengthened with FRP strips, the maximum attained FRP strain
was greater than 5000 µε. It follows that the ACI 440.2R-08 limit for maximum FRP
strain (i.e., 4000 µε) seems conservative for RC beams strengthened in shear with FRP
strips.
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REFERENCES


Table 1 – Experimental results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRP type</th>
<th>$w_f/s_f$</th>
<th>Load at rupture (kN)</th>
<th>Total shear resistance kN</th>
<th>Resistance due to concrete kN</th>
<th>Resistance due to steel kN</th>
<th>Resistance due to CFRP kN</th>
<th>Gain due to CFRP %</th>
<th>Deflection at loading point (mm)</th>
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<td>NR-NF</td>
<td>-</td>
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<td>150.6</td>
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<td>HR-NF</td>
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<td>0</td>
<td>350.6</td>
<td>232.2</td>
<td>81.2</td>
<td>151.0</td>
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<td>0</td>
<td>11.9</td>
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<td>HR-ST-LF</td>
<td>Strips</td>
<td>87.5/175</td>
<td>372.5</td>
<td>246.7</td>
<td>81.2</td>
<td>151.0</td>
<td>14.5</td>
<td>6</td>
<td>15.9</td>
</tr>
<tr>
<td>HR-ST-HF</td>
<td>Strips</td>
<td>87.5/125</td>
<td>383.4</td>
<td>253.9</td>
<td>81.2</td>
<td>151.0</td>
<td>21.7</td>
<td>9</td>
<td>15.7</td>
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<td>HR-SH</td>
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<td>250.6</td>
<td>81.2</td>
<td>151.0</td>
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<td>8</td>
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<tr>
<td>MR-NF</td>
<td>-</td>
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<tr>
<td>MR-SH</td>
<td>Sheet</td>
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<td>222.0</td>
<td>96.2</td>
<td>98.5</td>
<td>27.3</td>
<td>14</td>
<td>11.3</td>
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Note: conversion rates: 25.4 mm = 1 inch, 1 kN = 0.225 kips.

Table 2 – Coefficient of determination ($R^2$) between the values of $V_f$ as calculated by each of the models and the experimental values of $V_f$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_f^{exp}$</th>
<th>$V_f^{cal}$ by ACI 440.2R (2008)</th>
<th>$V_f^{cal}$ by HB 305 (2008)</th>
<th>$V_f^{cal}$ by Mofidi and Chaallal (2011a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NR-ST-LF</td>
<td>53.3</td>
<td>20.5</td>
<td>32.5</td>
<td>43.7</td>
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<td>NR-ST-HF</td>
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<td>37.9</td>
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<td>NR-SH</td>
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<td>45.9</td>
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<td>HR-ST-HF</td>
<td>21.7</td>
<td>28.6</td>
<td>39.8</td>
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<td>27.3</td>
<td>44.1</td>
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</table>

$V_f^{exp}$ = Experimental shear resistance due to FRP.
$V_f^{cal}$ = Calculated shear resistance due to FRP (not factored).
1 kN = 0.225 kips.
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