CONTRIBUTION OF EXTERNALLY BONDED CFRP TO THE SHEAR CAPACITY OF RC BEAMS

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ABSTRACT: Rehabilitation of deteriorated and aged structures with fiber reinforced polymer has been proven as a successful technique due to its outstanding properties such as resistance to corrosion and high stiffness-to-weight ratio. The purpose of this study is to investigate the contribution of the externally bonded carbon fiber reinforced polymer to the shear capacity of the reinforced concrete beams. For this purpose, three reinforced concrete beams strengthened with carbon fiber reinforced polymer in form of side bonded sheet was investigated. The experimental program consisted of three beam specimens that were strengthened with carbon fiber reinforced polymer. One of the beam specimens was not reinforced with transverse steel reinforcement, while the other two were reinforced with stirrups at 200 and 100mm spacing. The strain of the transverse steel reinforcement and the externally bonded CFRP was recorded and compared. In addition, shear capacity predicted by code provisions was compared to the test results. It was indicated that the presence of the transverse steel reinforcement would reduce the contribution of the CFRP to the shear capacity. In addition, the investigated code provisions underestimate the shear capacity of the RC beams strengthened with CFRP.

Key words: CFRP, Shear strengthening, Shear capacity, FRP, Side bonding

INTRODUCTION

Fiber Reinforced Polymer (FRP) has been greatly used to strengthen deteriorated structures in the last few decades. Externally Bonded Fiber Reinforced Polymer (EB-FRP) and Near Surface Mounted (NSM) are the most popular strengthening methods. Due to its easy implementation, externally bonded strengthening technique has been widely used to increase flexural and ultimate shear capacity of Reinforced Concrete (RC) beams (Pellegrino et al. 2009, Pellegrino & Modena 2009, Chen et al. 2015, Oehlers et al. 2015, Bukhari et al 2010, Pellegrino et al. 2002, Pellegrino & Modena 2008)

Shear behavior of RC beams strengthened by EB-FRP has attracted a great attention due to its significance to prevent shear failure. Shear strengthening is performed in form of side bonding, U-wrapping, and complete wrapping. FRP can be applied in one or more than one layer. Numerous studies (Bukhari et al. 2010, Pellegrino et al. 2002, Pellegrino & Modena 2008, Pellegrino 2006, Bousselham & Chaallal 2006, Bousselham & Chaallal 2008, Pellegrino & Vasic 2013, Chen et al. 2010) made important contributions to this subject. Yet, there are still some gaps need to be filled. The contribution of the FRP to the shear capacity is one of these gaps. Design guidelines such as ACI 440.2R, fib, and CNR-DT quantify the shear capacity of the RC beams strengthened in shear with EB-FRP by the simple sum of concrete contribution, transverse steel contribution, and FRP contribution. Nevertheless, these models sometimes lead to inaccurate assessment because they ignore the interaction between transverse steel and FRP (Pellegrino et al. 2002, Pellegrino & Modena 2008, Pellegrino 2006, Bousselham & Chaallal 2006, Bousselham & Chaallal 2008, Pellegrino & Vasic 2013, Chen et al. 2010). Therefore, more experimental tests are needed to validate these models. In this paper, the contribution of the externally bonded Carbon Fiber Reinforced Polymer (CFRP) to the shear capacity of RC beams was investigated. In addition, the shear capacity of the RC beams strengthened with CFRP obtained from the experimental program was compared with the values predicted by some code provisions.

DESIGN GUIDELINES FOR THEPREDICTION OF THE SHEAR CAPACITY

ACI 440-08

The American Concrete Institute (ACI 440.2R 2008) provisions are based on a study by Khalifa et al. (1998). Shear capacity of RC beams strengthened with FRP can be determined as follows:

$$V_{Rd} = V_c + V_s + \psi_f V_f \quad (1)$$

$$V_c = 0.17\,\lambda\,\sqrt{f'_c}\,b_w\,d \quad (2)$$

$$V_s = \frac{f_{wy}\,.A_{ws}\,.d}{S} \quad (3)$$

$$V_f = \frac{A_{fv}\,.f_{fe}\,.(\sin\alpha+\cos\alpha).d_f}{s_f} \quad (4)$$

$$A_{fv} = 2\,.\,n\,.\,t_f\,.\,w_f \quad (5)$$

$$f_{fe} = \varepsilon_{fe}\,.\,E_f \quad (6)$$

For complete wrapped member

$$\varepsilon_{fe} = 0.0004 \le 0.75\,.\,\varepsilon_{fu} \quad (7)$$

For two or three side-bonded FRP

$$\varepsilon_{fe} = k_v\,.\,\varepsilon_{fu} \le 0.004 \quad (8)$$

$$k_v = \frac{k_1\,.\,k_2\,.\,L_e}{11{,}900\,.\,\varepsilon_{fu}} \le 0.75 \quad (9)$$

$$L_e = \frac{23{,}300}{(n\,.\,t_f\,.\,E_f)^{0.58}} \quad (10)$$

$$k_1 = \left(\frac{f'_c}{27}\right)^{2/3} \quad (11)$$

$$k_2 = \frac{d_{fv}-2\,L_e}{d_{fv}} \quad \text{For two sides bonded} \quad (12)$$

$$k_2 = \frac{d_{fv}-2\,L_e}{d_{fv}} \quad \text{For U wrapping} \quad (13)$$

Where V_c, V_s, *and* V_f are the concrete, transverse steel, and FRP contribution to the shear strength; f'_c is the compressive strength of the concrete; b_w and d are the width and the depth of the beam cross section, respectively; f_{wy} and A_{ws} are the yielding strength and cross section area of the transverse steel reinforcement; S is the spacing between transverse reinforcement; ψf is a reduction factor equals to 0.85 for two side-bonded FRP scheme and 0.95 for complete wrap scheme; A_{fv}, f_{fe},

s_f, are the FRP shear reinforcement area, effective stress and spacing respectively; a is the FRP inclination angle with respect to the longitudinal axis of the member; d_f is the FRP effective depth; E_f is the FRP tensile modulus of elasticity; ε_{fu} is the ultimate rapture strain calculated by dividing the tensile strength (f_{fu}) on the tensile modulus of elasticity (E_f); ε_{fe} is FRP effective strain at the failure point; k_v bond-reduction coefficient.

FIB 2001

The European code fib (2006) derived its model to predict the shear strength of FRP strengthened RC beams based on regression of experimental results conducted by Triantafillou & Antonopoulos (2000). The shear capacity can be determined using the following Equations:

$$V_{Rd} = \min(V_{cd} + V_{wd} + V_{fd}\,,\ V_{Rd,2}) \quad (14)$$

$$V_{cd} = 0.9\,.\,k_v\,.\frac{\sqrt{f_{cm}}}{\gamma_c}\,.\,b_w\,.\,d \quad (15)$$

$$V_{wd} = 0.9\,.\ \frac{A_{sw}}{s_w}\,.\,f_{ywd}\,.\,d\,.\,(\cot\theta + \cot\beta)\sin\beta \quad (16)$$

$$V_{fd} = 0.9\,.\,\varepsilon_{fd,e}\,.\,E_{fu}\,.\,\rho_f\,.\,b_w\,.\,d\,.\,(\cot\theta + \cot\alpha).\sin\alpha \quad (17)$$

$$\varepsilon_{fd,e} = \frac{\varepsilon_{fk,e}}{\gamma_f} \quad (18)$$

$$\varepsilon_{fk,e} = k.\,\varepsilon_{f,e} \qquad \text{Where } \boldsymbol{k=0.8} \quad (19)$$

For fully side-bonded and U-jacketing FRP

$$\varepsilon_{f,e} = \min\left[0.65\,.\left(\frac{f_{cm}^{2/3}}{E_{fu}\cdot\,\rho_f}\right)^{0.56}.\ 10^{-3}\,,\ 0.17\,.\left(\frac{f_{cm}^{2/3}}{E_{fu}\cdot\,\rho_f}\right)^{0.3}.\,\varepsilon_{fu}\right] \quad (20)$$

For continuous bonded FRP

$$\rho_f = \frac{2\,.t_f\,.\sin\alpha}{b_w} \quad (21)$$

For FRP sheet or strips of width b_w at spacing s_f:

$$\rho_f = \left(\frac{2\,.\ t_f}{b_w}\right).\left(\frac{b_f}{s_f}\right) \quad (22)$$

Where V_{cd} and V_{wd} are the concrete and transverse steel contribution to the shear strength, respectively; V_{fd} is the FRP contribution; k_v is a model parameter defined by the levels of approximation and represents the ability of the web to resist aggregate interlock stresses (fib 2010); γ_c is a partial safety factor for concrete; f_{ywd}, A_{sw} is the yielding strength and cross section area of the transverse steel, respectively; s_w is the spacing between transverse steel; γ_f is the partial safety factor which is 1.30 for the bonding failure; $\varepsilon_{fd,e}$ design value of the effective FRP strain; ε_{fu} is the FRP ultimate strain; E_{fu} modules of elasticity for the FRP in the principal fiber orientation in GPa; f_{cm} is the cylindrical compressive strength of concrete in MPa; ρ_f is the FRP reinforcement ratio; d and b_w are the depth and the width of the cross section, respectively; a is the angle between the longitudinal axis of the member and principal fiber orientation; θ is the angle of the diagonal crack with respect to the member axis (assumed 45); β is the angle between transverse steel reinforcement and longitudinal axis of the member.

CNR-DT200 (2013)

The Italian CNR-DT proposed the following expression to determine the shear capacity of the RC beams strengthened by externally bonded FRP:

$$V_{Rd} = \min\left[V_{Rd,s} + V_{Rd,f};\ V_{Rd,c}\right] \tag{23}$$

For FRP side bonding configuration

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} \, .0.9\, .\, d\, .\, f_{fed}\, .\, 2\, .\, t_f\, .(\cot\theta + \cot\beta)\, .\frac{b_f}{p_f} \tag{24}$$

Where $V_{Rd,s}$, and $V_{Rd,f}$ are the steel and FRP contributions to the shear capacity, respectively; $V_{Rd,c}$ is the maximum shear force the section can resist before crashing of the concrete in the compression zone; f_{fed} is the FRP effective design strength; b_f and p_f are the FRP width and spacing, respectively, measured orthogonally to the fiber direction (for FRP strips installed one next to each other, the ratio of b_f /p_f equals to 1.0); γ_{Rd} is a partial factor for resistance models (For shear and torsion, its value equals 1.20); β is the angle between the FRP sheet and the longitudinal axis; θ is the angle of the shear crack (assume equals to 45^{o} unless more calculations are made).

$$f_{fed} = f_{fdd}\left[1 - \frac{1}{3}\, .\, \frac{l_{ed}\, .\sin\beta}{\min[0.9d;\, h_w]}\right] \tag{25}$$

$$f_{fdd} = \frac{1}{\gamma_{f,d}} \sqrt{\frac{2\, .\, E_f\, .\, \Gamma_{Fd}}{t_f}} \tag{26}$$

$$l_{ed} = \frac{1}{\gamma_{Rd} \cdot f_{bd}} \sqrt{\frac{\pi^2\, .\, E_f\, .\, t_f\, .\, \Gamma_{Fd}}{2}} \tag{27}$$

$$f_{bd} = \frac{2\, .\Gamma_{Fd}}{s_u} \tag{28}$$

(s_u = 0.25mm is the design bond strength between FRP and concrete)

Where h_w is the cross section depth; f_{fdd} is the design debonding strength of FRP; l_{ed} is the effective bond length; $\gamma_{f,d}$ is partial factor ranged between 1.20 to 1.50; E_f and t_f are the FRP modules of elasticity and thickness, respectively. For effective bond length evaluation, γ_{Rd} equals to 1.25. Γ_{Fd} is the design value of the specific fracture energy which can be determined as follows:

$$\Gamma_{Fd} = \frac{k_b\, .\, k_G}{FC}\, .\sqrt{f_{cm} \cdot f_{ctm}} \tag{29}$$

$$k_b = \sqrt{\frac{2-\frac{b_f}{b}}{1+\frac{b_f}{b}}} \geq 1.0 \tag{30}$$

Where f_{ctm} is the concrete average tensile strength; f_{cm} is the concrete compressive strength; FC is a confident factor (assumed 1.0); k_G is a corrective factor equal to 0.023mm or 0.037mm for pre-cured and wet lay-up schemes, respectively. k_b is the geometric coefficient factor depending on both of the strengthened beam and FRP. The following points should be considered when calculating k_b value:

* $b = p_f$ for discrete FRP strip application

* $b = b_f = \min[0.9d;\ h_w] . \frac{\sin(\theta+\beta)}{\sin\theta}$ For FRP system installed continuously along the length

EXPERIMENTAL PROGRAM

Specimen and Material

The experimental program consisted of three rectangular RC beam specimens. They were designed such that their ultimate shear capacity was reached before their ultimate flexural capacity as shown in Table 1. Except for specimen B1 that did not have transverse steel reinforcement, the other two beams were reinforced with transverse steel reinforcement at 100mm and 200mm spacing. The specimens were cast in the laboratories of the Civil Engineering Department of Wasit University using an electric mixer of 0.15 m^3. The properties of the concrete were as follows: cement = 412 kg/m^3, fine aggregate = 648 kg/m^3, coarse aggregate = 1152 kg/m^3, and water-cement ratio (w/c) = 0.45. Cylindrical specimens of 150 mm diameter and 300 mm length were used to determine the compressive strength. The mean compressive strength at the age of 28 days was 23.5 MPa.

Table 1. Details of Specimens

Specimen	*b* (mm)	*d* (mm)	a/d	Tension Steel	ρ	Compression steel	Stirrups diameter (mm)	Stirrups spacing, (mm)
B1	150	200	2.75	2ϕ25 mm	0.041	2ϕ10 mm	N/A	N/A
B2	150	200	2.75	2ϕ25 mm	0.041	2ϕ10 mm	6	200
B3	150	200	2.75	2ϕ25 mm	0.041	2ϕ10 mm	6	100

Strengthening Procedure

All beam specimens were strengthened with side-bonded CFRP sheets along the entire length of the beams as shown in Figure 1. First, concrete surface was prepared using a grinder and sand paper. Then, it was cleaned with a wet piece of cloth to remove all the remaining dust. Finally, one layer of unidirectional Sika-Wrap-301C carbon fabric was applied using Sikadur- 330 impregnating resin as an adhesive. Table 2 shows the geometric and mechanical properties of the CFRP.

Table 2. Geometric and Mechanical Properties of the CFRP

Type	Width (mm)	Thickness (mm)	Tensile strength (MPa)	Tensile modules of elasticity (MPa)	Elongation at break %
Sika-Wrap-301C	500	0.167	4,900	230,000	1.7

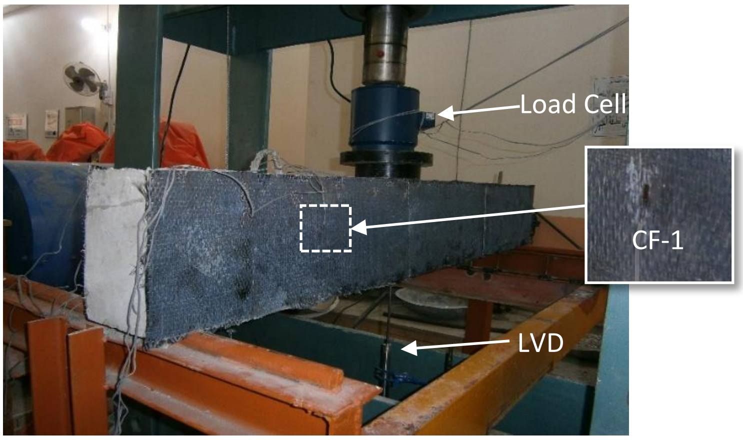

Figure 1. Test Setup

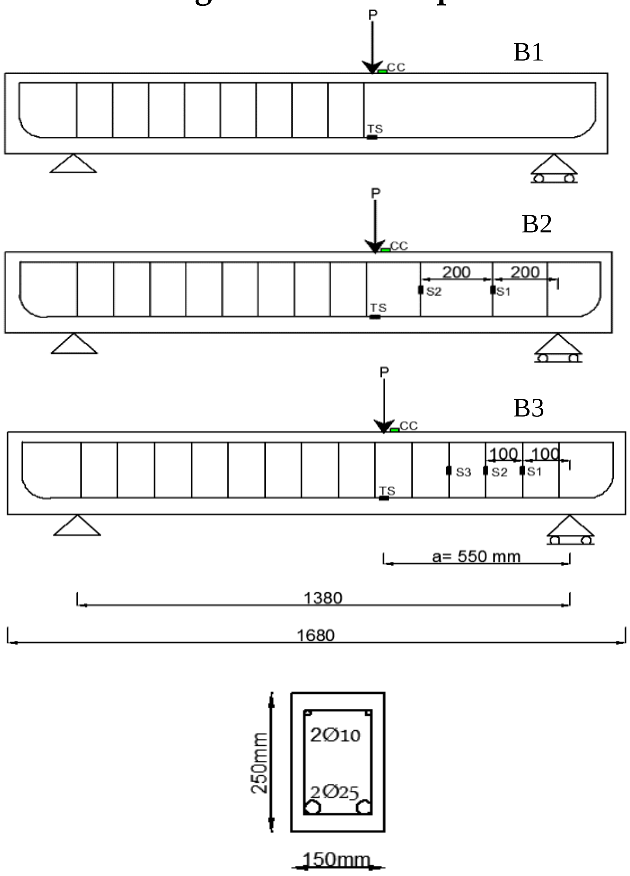

Figure 2. Specimens' Details and Instruments Position

Test Setup and Instruments

Simply supported scheme with three-point loading was carried out in this study as shown in Figure 1 and 2. Signals from the instruments were monitored and recorded by an automatic data acquisition system. The load was applied using 500 kN hydraulic jack with manual control. The applied load was measured using load cell of 1000 kN capacity. The deflection of the beam beneath the point of the applied load was captured with Linear Variable Displacement Transducer (LVDT). Two strain gauges, CF-1 and CF-2, located at 250 mm from the support were used to measure the strain of the CFRP on the both sides of the beam as shown in Figure 1. Waterproof strain gauges were used to measure the strain in the stirrups as shown in Figure 2. Strain gauges, TS and CC, were also used to measure the strain at the tensile steel reinforcement and compression concrete, respectively.

RESULTS AND DISCUSSIONS

Load-Deflection Relationship

Load-deflection relationship was utilized to assess the enhancement of the shear capacity of the RC beams strengthened with the CFRP. The load-deflection relationship obtained from the experimental program is presented in Figure 3. It can be noticed a brittle failure developed in specimen B1at a load of 160.0 kN due to the absence of the transverse steel reinforcement. However, a ductile failure was developed in specimens B2 and B3 due to the presence of the internal transverse shear reinforcement at a load of 164.9 kN and 254.9 kN respectively.

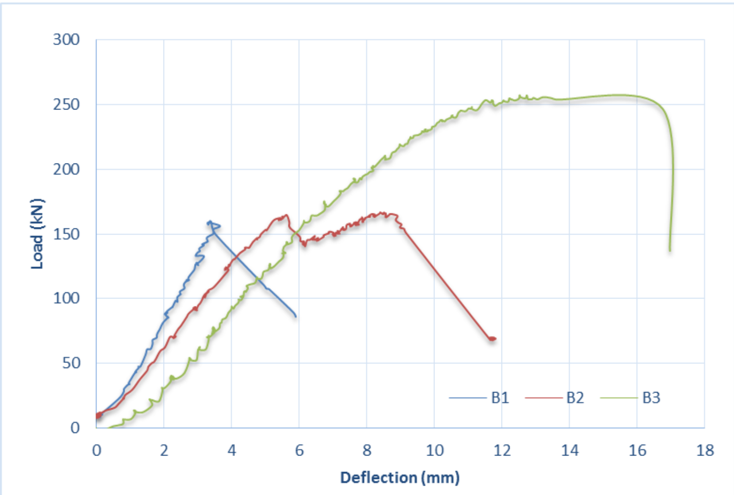

Figure 3. Load-Deflection Curves

All the beams failed by shear with CFRP delamination. First diagonal shear crack initiated at the point of the applied load then it was propagated toward the closest support. After that, CFRP delamination, combined with a small layer of concrete, was developed at the top and the bottom of the beams as shown in Figure 4. Finally, CFRP debonding propagated toward the inclined shear crack causing beams' failure.

Strain Comparison

Strain developed in the CFRP within the shear span was measured by two strain gauges (CF-1, and CF-2) placed on both faces of each specimen and located at 250 mm from the support. Figure 5 compares between the strains developed within the three specimens. It can be observed that though the shear capacity of specimen B1 is almost equal to B2, CFRP in B1 was more strained than B2. This result attributed to the higher contribution of the CFRP in the case of B1 due to the absence of the transverse steel reinforcement. In other words, since specimen B1 was not reinforced with transverse steel, its CFRP contribution to the shear strength was higher. For specimen B3, shear capacity was increased by decreasing the spacing between transverse steel reinforcement within the shear span. However, the strain captured in the CFRP at the failure was almost equal to the one captured in B2 indicating that the presence of transverse steel reinforcement can reduce CFRP contribution to a constant value.

Figure 4. Failure Mode

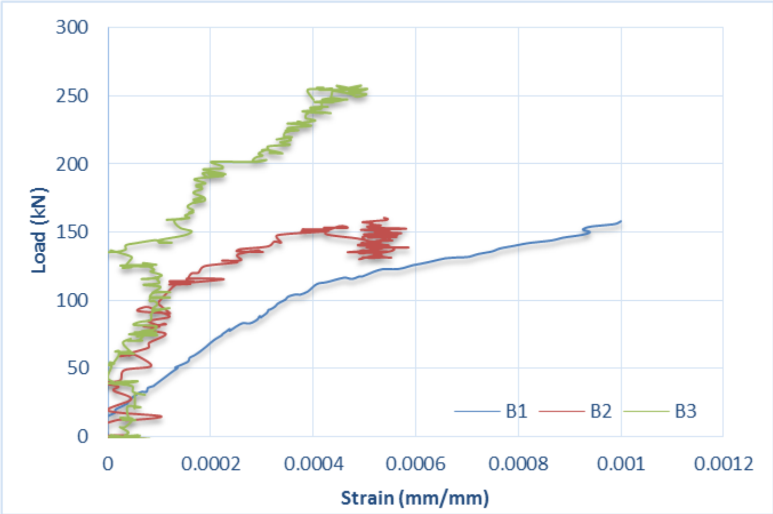

Figure 5. Load-Strain of the Three Specimens

Figure 6 compares between strains measured by S1, S2, and CF-1 for specimen B3. Results highlighted that strains developed in both stirrups were higher than the one developed in the CFRP. This result supports the finding obtained earlier that CFRP contribution to the shear capacity of beam reinforced with transverse steel would be lower than the unreinforced one.

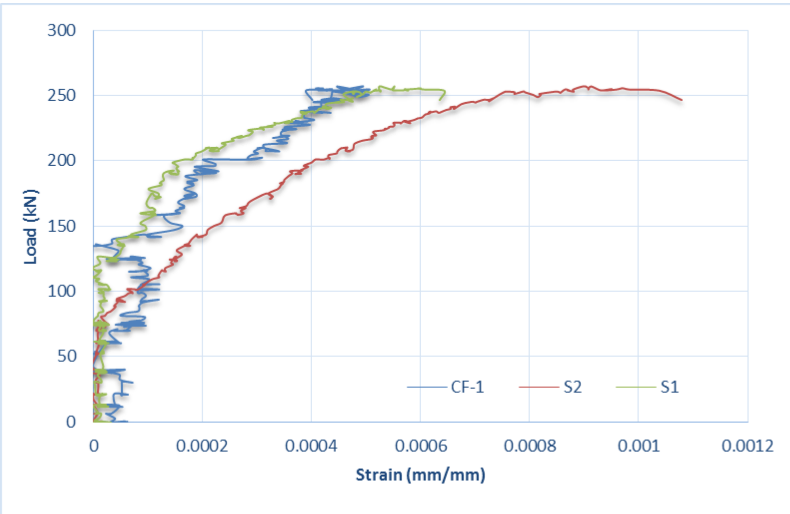

Figure 6. Load-Strain of Specimen B3

Comparison between measured shear capacity and predicted shear capacity

Shear capacity obtained from the experimental program was compared to the one predicted by code provisions in Table 3. From the comparison, it can be detected that code provisions had conservative predictions for the shear capacity of the RC beams strengthened with CFRP. The ACI 440-08 had the closest predictions, especially for specimen B2. However, the CNR-DT had the most conservative predictions, especially for specimen B1. This result can be attributed to the fact that CNR-DT neglect the contribution of the concrete to the shear capacity. Based on that, shear capacity of specimen B1 represents the contribution of the CFRP component only. This comparison proves that some expression adopted by design guidelines need to be improved more.

Table 3. Comparison between Measured Shear Capacity and Predicted Shear Capacity

		Shear capacity (V_{Rd}) MPa		
		B1	B2	B3
Experimental results $V_{Rd,\,exp.}$		160.0	164.9	254.9
Code predictions $V_{Rd,\,code}$	ACI 440-08	82.3	121.7	161.2
	fib	66.1	100.3	135.8
	CNR-DT	41.5	77.0	112.4
$V_{Rd,\,exp.}/V_{Rd,\,code}$	ACI 440-08	2.0	1.4	1.6
	fib	2.4	1.6	1.9
	CNR-DT	3.9	2.1	2.3

CONCLUSION

In this paper, the interaction between the transverse steel reinforcement and the externally bonded CFRP was investigated. The experimental results showed that using CFRP in conjunction with transverse steel reinforcement shifted failure mode from a brittle failure to a ductile failure. Moreover, the strain developed in the externally bonded CFRP was reduced, which indicates a lower contribution to the shear capacity. Comparing shear capacity estimated by code provisions with the test results indicated that the current code provisions have a conservative prediction formulas, especially the CNR-DT (2013). Therefore, more studies are advised to have more accurate models.

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