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Overview Shear Strengthening of RC Beams with Externally Bonded FRP Composites

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Abstract: Shear strengthening is required when an RC beam is found deficient in shear, or when its shear capacity falls below its flexural capacity after flexural strengthening. This study presents a state-of-the-art review of existing research on this topic. The research programs conducted to investigate the shear performance and to evaluate the shear capacity of the strengthened beams are reviewed in this paper by details. The research is a good reference for all engineers and researchers in fields of shear strengthening and repair RC beams in building and bridges.

Key words: RC beams, shear strengthening, shear capacity, FRP

INTRODUCTION

The use of externally bonded Fiber-reinforced Polymer (FRP) composite plate or sheets for the strengthening of Reinforced Concrete (RC) has become very popular in recent years. This popularity has been due the well-known advantages of FRP composite over conversational materials when they used to strengthening projects. Since the early 1990s, tests on a wide variety of shear strengthening schemes have been undertaken with the goal to increase shear capacity of reinforced concrete beams. Shear is actually a very complex problem and is not completely solved for simple reinforced-concrete (RC) beams. However, to find a reasonable method to estimate the contribution of externally bonded FRP in shear are not easy task. Several researchers have published design equations and analytical models to specifically evaluate FRP shear strengthening of reinforced concrete beams, which have already reviewed by details in this a study. Strengthening schemes have consisted of strengthening applied to side faces, U-jacketing to both sides and soffit and wrapping FRP around entire cross-section of a beam. The orientations angles of the fibers and different anchorage system have also considered as shown in Fig. 1.

A summary of research on shear strengthening with FRP: Uji (1992) carried out the tests of eight simply supported RC beams strengthened for shear with CFRP sheets using two different wrapping schemes; total wrap or two sides of the beam. He concluded that the application of CFRP substantially improves the shear capacity of RC members. He also found that the strains in the stirrups and the CFRP are different even at the same location. This is because a stirrup stretches evenly over its length, while only a limited area of CFRP stretches at the crack. Thus, the strain in CFRP is greater than in stirrups at the crack location. In his study, the maximum shear force carried by CFRP was assumed to be the product of the bond area assumed as the triangle above the middle point of the diagonal crack and the bond stress of 1.27 MPa, which was determined based on his test results.

Al Sulaimani *et al*. (1994) investigated the behaviour of reinforced concrete beams deficient in shear capacity and strengthened with GFRP. The beams were designed to have 1.5 times greater flexural strength than shear. Sixteen reinforced concrete beams were cast of 150 mm square and 1250 long. The beams were damaged to predetermined level defined subsequent to testing of the control beams which were unrepaired. the beams were damaged by preloading them under four-point load with a 1200 mm span and a 400 mm shear span up to the appearance of the first shear crack. The load was removed and the beams were repaired. Different shear repair schemes were used with GFRP strips on both sides, continues GFRP on both sides and U-jacket used in the shear spans of the beams. The results showed that the repairs increased the shear capacity and restored the degraded stiffness of the beams. The study also showed that the increase in shear capacity was almost identical for both strips and side face bonded repairs. However, U-jacket system enabled the strength of the beams to be increased to the extent necessary to achieve a flexural failure.

Chajes *et al*. (1995) tested 12 under-reinforced concrete T beams to study the effectiveness of external bonded composite for shear capacity. Woven composite

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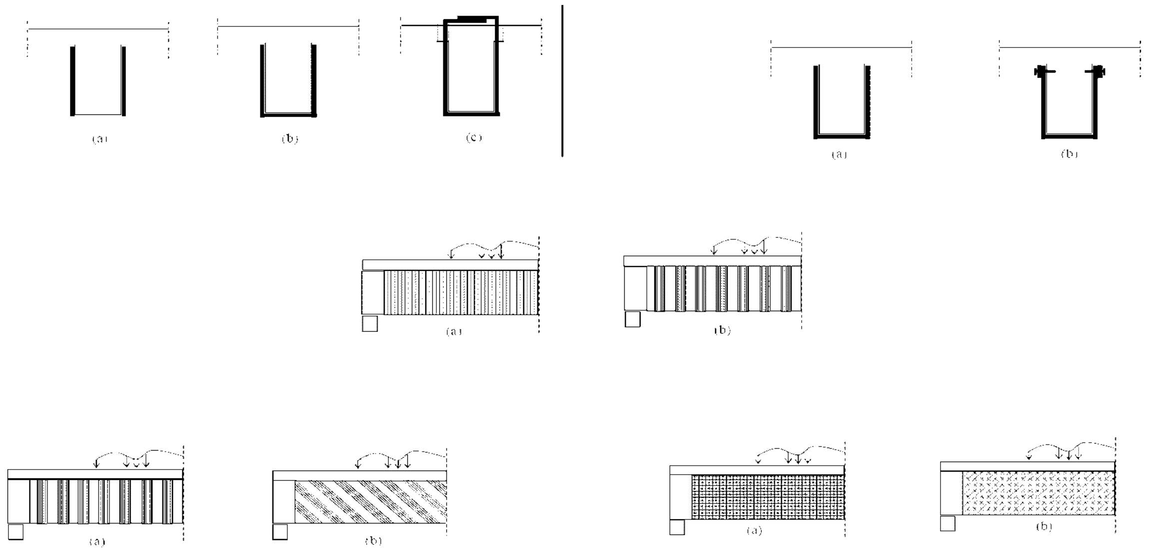


Fig. 1: Different FRP shear strengthening configuration, anchorage system and orientation angle

fabric made of armiad, E-glass and carbon fibres were bonded to the web of the T- beams using a tow-component epoxy. The three different fabrics were chosen to allow various fabrics stiffness and strengths to be studied. No internal steel shear reinforcement was used in any of the beams, and, thus the beams were under-designed in shear compared to with their flexural capacities. The beams were tested in flexure and the performance of eight beams with external shear reinforcement was compared to results of four control beams with no shear reinforcement. All the beams failed in shear due to the diagonal tension of concrete followed by the rupture of FRP and no debonding of the fabric from concrete surface occurred. The fiberes did not reach their ultimate strength. For beams with external shear reinforcement, the average increases in ultimate strength of 60 to 150% were achieved. The authors proposed a method to calculate the FRP contribution, which is similar to that used to evaluate the stirrup contribution in the ACI code. The maximum strain of FRP was taken as the ultimate tensile strain of concrete. However, the specimens used in this study were very small and only one wrapping scheme was used (i.e., U-wrap) which limited more general conclusions.

Sato *et al.* (1996) six beams were fabricated, 200 mm wide mm 300 deep and 2200 mm long and tested under four-point loading with a shear span 700 mm and a clear span 1600 mm. The specimens demonstrated that CFRP increased the shear strength significantly. The U-jacket FRP is more effective than CFRP attached only to the sides. The strains of CFRP on two beams were measured during the test. One beam used U-jacket CFRP, while the other one had CFRP on sides only. It shows the CFRP strain along the major shear crack is not uniform and has the same trend for both of the beams: CFRP strain is large at the middle of the shear crack, small at the ends of the crack. The strain distribution of CFRP is similar to that of stirrups. The strains of stirrups in one beam were also measured, which show shear force carried by FRP was greater than that by stirrups. The failure was by declamation of the CFRP sheets a long of the span shear.

Umezu *et al.* (1997) carried out an extensive experimental program in order to determine the effects of aramid and carbon FRP sheets on the shear capacity of twenty six simply supported RC beams. They used total wrap as strengthening scheme for all of their test beams. Most test pieces exhibited peeling of sheets around diagonal cracks when cracks appeared in the beams. The peak loading occurred at the moment when diagonal cracking penetrated to the upper edge of beam causing significant impairment to the shear resistance mechanism of concrete; usually it did not occur at the moment when the FRP ruptured. A truss model was used to predict the FRP contribution with a reduction factor of the ultimate FRP strength ffu, which is between 0.4 and 1.2 according to the test.

Araki *et al.* (1997) thirteen beams were tested with different types of fibers, different amounts of FRP and steel stirrups. FRP was wrapped around the beams totally. The failure mode was diagonal tension failure. In every specimen strengthened with FRP, rupture of sheets was not observed when maximum load was achieved. After the maximum load, the sheets ruptured in most of the beams. Tests indicated the average FRP stress reached an almost constant value in each type of sheets (carbon and aramid) and steel stirrups yielded when the maximum load was achieved. The contribution of FRP to the shear capacity was evaluated similar to calculation of stirrups contribution. a reduction factor to the tensile strength of the sheets was proposed. In their study the values of 0.6 and 0.45 were adopted for CFRP and AFRP sheets, respectively.

Triantafillou (1998) aimed to increase the experimental database on FRP shear strengthened reinforced concrete beams. Eleven beams deficient in shear were constructed of which nine were strengthened in shear with epoxy-bonded CFRP fabrics attached to the row sides. A further tow beams were tested as control specimens. The 1000 mm long beams were loaded in four-point bending with a span of 800 mm and shear span of 320 mm each beam had a cross-section 70 mm wide and 110 mm deep. All the beams tested experienced brittle tensile cracks in the shear span in those externally reinforced with CFRP, diagonal cracking was followed by CFRP debonding with the failure occurring at loads significantly higher than that of the unstrengthened beams. The increased of strength ranged from 65 to 95% above that of the control beams. Also, a design model for computing the shear capacity of RC beams strengthened in shear with FRP composites was presented in this study. In this model, the external FRP shear reinforcement was treated similar to the internal reinforcement. It was assumed that at the ultimate shear limit state the FRP develops an effective strain, ε_{fe} which is less than the ultimate tensile strain, ε_{fu}, of FRP. The expression for computing the FRP contribution to the shear capacity of an RC beam, V_f, was written as follows:

$$V_f = \frac{0.9}{\gamma f} \rho_f E_f \varepsilon_{fe} b_w d(1+\cos\beta)\sin\beta \tag{1}$$

where γ_f is the partial safety factor for FRP in uniaxial tension (taken 1.15 for CFRP), ρ_f is the FRP area fraction (equal to (2tf/bw) (w_f/s_f)), t_f-the FRP reinforcement thickness and w_f is the width of FRP strip, S_f is the spacing of strips, b_w is the beam width, E_f is the elastic modulus of FRP, d is the effective depth of the beam and β is angle between principal fiber orientation and longitudinal axis of the beam.

The application of Eq. (1) requires the quantification of the effective strain, ε_{fefe}.

Triantafillou observed the effective strain to be a function of the axial rigidity of the FRP sheet expressed by $\rho_f E_f$. The effective strain was, therefore, determined by finding V_f experimentally for several rigidities of FRP sheet. Based on the experimental results, the effective strain was back calculated and plotted versus the axial rigidity. A relationship between effective strain and axial

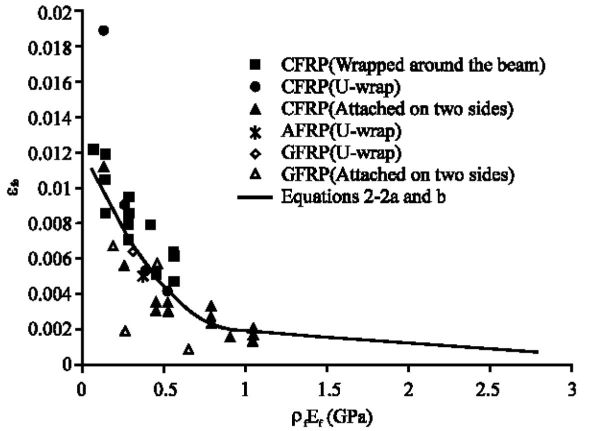


Fig. 2: Comparison between the correct experimental results and Eq. 2a and b

rigidity was derived experimentally through curve fitting of about 38 test results found in the literature and he proposed the Following Eq. (2a, b).

$$\varepsilon_{fe} = 0.0119 - 0.0205(\rho_f E_f) + 0.0104(\rho_f E_f)^2 \quad \text{for} \quad 0 \le \rho_f E_f \le 1\text{GPa} \tag{2a}$$

$$\varepsilon_{fe} = 0.00245 - 0.000065(\rho_f E_f) \quad \text{for} \quad \rho_f E_f \,\phi 1\text{GPa} \tag{2b}$$

The modelling approach of Triantafillou had the following shortcomings:

- Equation (2b) was based on fitting some wrong data considering the value of $\rho_f E_f$ equals to 2.76 instead of 0.25 for two of the test specimens. A comparison between the correct experimental results and Eq. 2a and b is shown in Fig. 2. As shown in this graph, there is no data of $\rho_f E_f = 2.76$.
- The data used to produce Eq. (2a and b), 38 test results, included three types of FRP (CFRP, AFRP and GFRP), whereas the fracture capacity of each type could be different.
- The wrapping schemes (totally wrapped, U-wrap and FRP on two beam sides), that have a significant affect on FRP contribution and mode of failure were not considered as design variables.
- The concrete strength, which is expected to affect the bond behavior, was not considered.
- One equation was used to describe both modes of failure (FRP fracture and debonding)
- The partial safety factor for CFRP material (Eurocode design format) was suggested to be equal to the partial safety factor for steel, γ_f = 1.15. A more

conservative partial safety factor should be considered for the relatively new material.

- In Eq. (1), the depth of concrete section, d, should be modified to be the effective depth of FRP reinforcement, d_f.
- The control of shear crack was not addressed or considered.
- No limit on the maximum amount of additional shear strength provided by FRP to preclude the web crushing was addressed.
- The maximum spacing of FRP strips was not addressed.

In spite of the above shortcomings, Triantafillou's model was the first systematic attempt to characterize the contribution of externally bonded FRP to the shear capacity. In addition, most of the shortcomings may be due to the relative lack of suitable experimental results available at that time.

Three series of 1.3 m long RC beams shear strengthened with CFRP strips were tested by Chaallal *et al*. (1998). The beams in one series were fully reinforced in shear with steel stirrups (FS), while beams in the second series were under-reinforced in shear (US). In the third series the beams were fabricated in the same manner as in the second series and then bonded in the shear span with 50 mm wide CFRP side strips either perpendicularly (RS90) or diagonally (RS135) to the axis of beam (Fig. 3). The RS series was designed to achieve the same shear capacity as the series FS. The beams in the US series failed in shear, whereas, the beams in FS series and RS series achieved the yielding load of the tensile reinforcement. For the tow RS series, the CFRP strips reduced extend and severity of the shear cracks thereby increasing the shear strength and stiffness of the beams. The vertical strips in series RS90 forced the diagonal cracks to bend less than in conventional RC beams, while the diagonal strips in RS135 series limited the propagation of the shear cracks. Most of the beams in the RS series failed due to peeling of laminate initiated longitudinal and transverse cracking towards the bottom of the beams. These cracks were attributed to high peeling (normal tensile) stresses developed at ends of the CFRP strips near the bottom of the beams, especially at high load levels. Chaallal *et al*. (1998) they concluded that although strips oriented at 135 degrees to the beam axis outperformed the perpendicular strips, U-strips and U-jackets should be utilized to minimize the peeling stresses at the ends of the strips leading to premature failure.

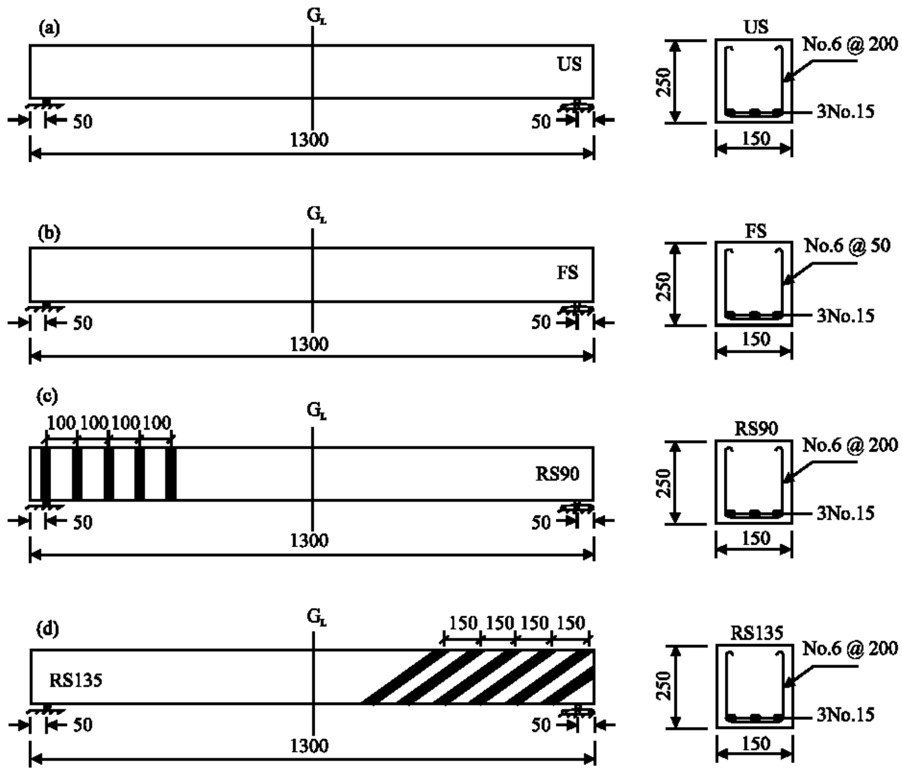


Fig. 3: Test specimens (a) US, (b) FS, (c) RS90 and (d) RS135 (Chaallal *et al*., 1998)

Grace *el al*. (1998) tested 14 simply supported rectangular beams. All beams had identical dimensions and flexural and shear reinforcement. The beams were 152 mm wide, 292 mm deep and had a clear span of 2720 mm. Each beam was initially loaded above its cracking load. The cracked beams were strengthened with FRP laminates and then tested with a concentrated load applied at mid-span until failure. Five FRP strengthening systems were used in this experimental program: tow types of CFRP sheets, tow types of GFRP sheets and CFRP plates. Four types of epoxies were used to fix the FRP to concrete surface in these systems. It was concluded that, in addition to the longitudinal layers, the fibres oriented in the vertical direction forming a U-shape around the beam cross section significantly increase the load carrying capacity and reduces deflections. Furthermore, the presence of vertical FRP sheets along the entire span length eliminates the potential for rupture of the longitudinal sheets. The combination of vertical and horizontal sheets, together with a good epoxy, can lead to a doubling of the failure load of beam. However, all the strengthened beams experienced brittle failure and Grace *el al*. (1998) suggested mandating a higher factor of safety in design.

Khalifa *et al*. (1998) modified the equation presented by Triantafillou (1998), the equation to compute CFRP shear contribution is similar to that used for steel shear reinforcement. The question to compute v_f is given below:

$$V_f = \frac{A_f\ f_{fe}\ (\sin\beta + \cos\beta)\, d_f}{s_f} \tag{3}$$

In Eq. (3), A_f is the area of one strip of transverse FRP reinforcement covering two sides of the beam. This area may be expressed as follows: $A_f = 2w_f.\ t_f$. Where t_f is the FRP reinforcement thickness and w_f is the width of the strip. The dimensions used to define the area of CFRP are shown in Fig. 4. The spacing between the strips, s_f, is defined as the distance from the centreline of one strip to the centreline of an adjacent strip. Note that, for continuous vertical FRP reinforcement, the spacing of the strip, s_f and the width of the strip, w_f, are equal. The angle β is angle between principal fiber orientation and longitudinal axis of the beam. d_f the effective depth of FRP strip, d_f, is the vertical projection of the shear crack (assumed to be 45°) minus the distance from the top of the crack to the end of the sheet. f_{fe} is the effective tensile stress of CFRP sheet when the beam failed in shear.

Equation (3) can also be expressed as follows:

$$V_f = \frac{A_f\ E_f\ \varepsilon_{fe}\ (\sin\alpha + \cos\alpha)\, d_f}{s_f} \tag{4}$$

In question (4), E_f is the elastic modulus of FRP. ε_{fe} Is the effective tensile strain of FRP The effective tensile stress in FRP at failure, taken smaller than its ultimate FRP tensile strength this is from experimental evidence, so it is calculated by applying a reduction coefficient, R, to the ultimate FRP tensile strength, f_{fu}, as expressed in following equation:

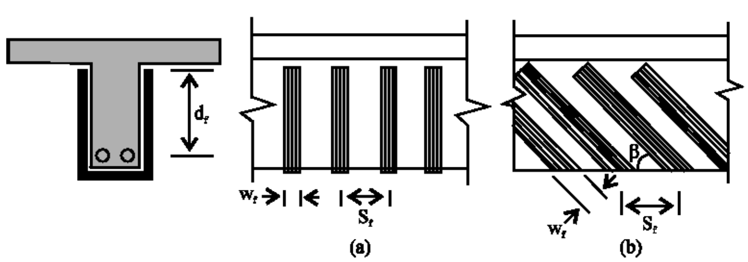
$$F_{fe} = R.F_{fu} \tag{5}$$

To apply questions (3), it is necessary to get the actual value of effective tensile strain of FRP, since the ultimate tensile strain of FRP can be obtained from the material properties supplied by the manufactures. A reductions factor is needed to compute the effective strain in the FRP at the failures.

$$e_{ef} = Re_{fu} \tag{6}$$

The proposed the reduction coefficient is determined based on the possible failure modes.

The failure of FRP reinforcement may occur either by debonding of the FRP sheets from concrete substrate, or the fracture of the FRP sheets. In either case, an upper



(a) Vertical CFRP strips (b) Inclined strips

Fig. 4: Definition of area of CFRP in shear reinforcement

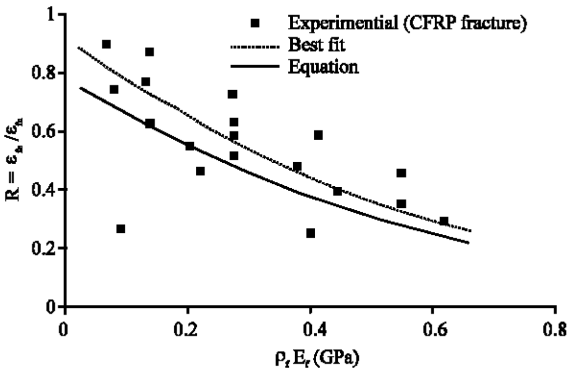


Fig. 5: Strength reduction coefficient in terms of ρf Ef for FRP fracture

limit of reduction coefficient is established to control the shear crack width and the loss of aggregate interlock. Therefore, Khalifa and Nanni (2000) suggested three questions to compute the R, one for CFRP rupture and second one for the delamination of the CFRP from the concrete surface. third one is based on limiting the shear crack width. The reduction factor assumed as the lowest of the three values.

Reduction coefficient based on CFRP Sheet Fracture Failure: the data from the experimental results of twenty-tow tests in which failure in rupture were used to compute the reduction factor. Whereas, Khalifa et al. they plotted the ratio of effective strain to ultimate strain against axial rigidity as shown in Fig. 5. Then polynomial function is used to fit the data in for the case of $\rho_f E_f \le 0.7$ GPa .

This polynomial is given in equation following

$$R = 0.56(\rho_f E_f)2 - 1.22(\rho_f E_f) + 0.78 \tag{7}$$

In this equation, ρ_f is the CFRP shear reinforcement ratio, which is defined as?

$2t_f w_f / b_w s_f$; t_f = CFRP thickness ; w_f= CFRP strip width; b_w cross-section width of the RC beam: and s_f = CFRP strip spacing.

Reduction Coefficient based on CFRP Debonding Failure: this format id derived from his analysis.

$$R = \frac{0.0042(f_c')^{2/3} w_{fe}}{(E_f t_f)^{0.58} \varepsilon_{fu} d_f} \tag{8}$$

In this equation, w_{fe} is defined as the effective width of CFRP sheets. The effective width of CFRP sheet, w_{fe}, may be computed according to the suggested bonded surface configuration (assuming a shear crack angle of $\alpha = 45°$) as shown in Fig. 6 W_{fe} = df Le If the sheet is in the form of a U-jacket without anchorage end.

W_{fe} = df - 2Le If the sheet is bonded to only sides of the beam.

Where, Le is the effective bonded length and equal to 75 mm.

Upper Limit of the Reduction Coefficient: based on limiting the shear crack width.

$$R = \frac{0.006}{\varepsilon_{fu}} \tag{9}$$

In addition, in this model if the FRP is applied in strips, as opposed to continuous FRP sheet, the maximum band spacing in defined by:

$$s_f \le w_{fe} + \frac{d}{4} \tag{10}$$

Where d is the depth of the internal steel reinforcement finally, the maximum allowable shear strengthening id described by the following expression:

$$V_f \le \frac{2\sqrt{f'_c}\, b_w d}{3} - v_s \tag{11}$$

Khalifa *et al.* (1999) investigated the effectiveness of U-jacket strengthening with and without a mechanical end anchorage system for RC beams strengthened in shear with CFRP sheets. researchers were tested three-T shaped RC beams with a total depth of 105 and 2340 mm clear span. The beams were reinforced only with longitudinal steel bars, no stirrups were used in the test region. The wraps were made of single-ply CFRP sheets with the fibre 900 degree to the longitudinal reinforcement axis of the beam. The results showed that the shear strengthened beams without mechanical anchorage system had a 72% in the shear capacity with the beam failing in shear after

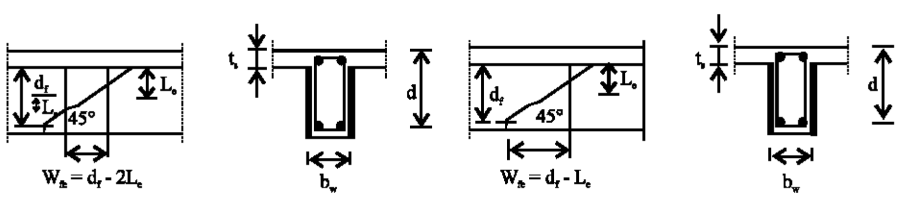


Fig. 6: Effective width of FRP reinforcement

debonding of the CFRP. For the shear strengthened beams with mechanical anchorage system, the shear capacity of the beam was further increased and the failure model was converted to tension failure without FRP debonding of the FRP.

Khalifa and Nanni (2000) investigated the performance of T-beam strengthened in shear with CFRP sheets. They found that by using various configurations of CFRP sheets, the shear capacity of the beams could be increased by 35 to 135%. It was found that an optimum quantity of FRP exists, beyond which strengthening effectiveness is uncertain. Strips of FRP applied only to the beam sides provided less strength enhancement than those bonded in U-shaped configuration. Although strips proved to be as effective as continues sheets, researchers recommended that sheet be utilized in field applications since damage to an individual strips is more detrimental to its behaviour.

Triantafillou and Antonopoulos (2000) proposed three different expressions to calculate the effective FRP strain by calibrating with 75 experimental data.

In this procedure, the FRP shear strength is calculated with the same equation used in Khalifa's procedure:

$$V_f = \frac{A_f\ f_{fe}\ (\sin\alpha + \cos\alpha) d_f}{s_f} \tag{12}$$

Where

$f_{fe} = E_f \varepsilon_{fe},A$

$\varepsilon_{fe},A = {}^{0.9}\varepsilon_{fe} \le 0.006$. For fully wrapped CFRP, Shear failure governing by FRP rupture and the corresponding effective strain ε_{fe} is:

$$\varepsilon_{fe} = 0.17\left(f_c^{2/3}\ / \rho_f E_f\right)^{0.3} \varepsilon_{fu} \tag{13}$$

For the other wrapping schemes (U-wrap or two sides), not only the above failure mode but also the failure due to FRP delamination needs to be considered, for which:

$$\begin{aligned} \varepsilon_{fe} &= \min(0.65(f_c^{2/3}\ / \rho_f E_f)^{0.65} \times 10^{-3} \\ \varepsilon_{fe} &= 0.17\left(f_c^{2/3}\ / \rho_f E_f\right)^{0.3} \varepsilon_{fu} \end{aligned} \tag{14}$$

For fully wrapped AFRP:

$$\varepsilon_{fe} = 0.048\left(f_c^{2/3}\ / \rho_f E_f\right)^{0.47} \varepsilon_{fu} \tag{15}$$

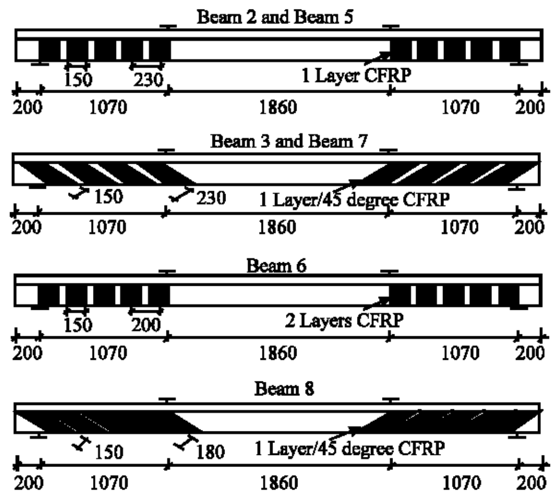


Fig. 7: Test specimens of Neto *et al.* (2001)

After computing the effective strain, ε_{fe}, the contribution of the FRP sheet to the shear capacity can be calculated from Eq……… with multiplying safety factor ϕ_f.

ϕ_f = 0.75 is used if FRP debonding dominates while 0.80 is recommended if the failure is governed by FRP fracture. 0.75 is used if ε_{fe} = 0.006.

Neto *et al.* (2001) investigated the shear strength of eight T-reinforced concrete simply supported beams. Six of the beams were strengthened with unidirectional layers of CFRP. The main variables investigated were the direction of the CFRP layer (vertical and inclined at 45 degrees) and the number and width of the layers Fig. 7. Four beams were pre-loaded to service load before being strengthened. The beams were 4400 mm in total length, 400 mm of overall height and had a 150 web width. The clear span was 4000 mm. The beams had identical flexural reinforcement. Beams 1 to 3 did not have stirrups in the shear span while beams 4 to 8 had steel stirrups. The results showed that ultimate loads of the strengthened beam were from 7 to 35% higher than the control beams. All the strengthened beams failed due to peeling of the CFRP laminates at the ends close to the flange with a thin layer of concrete attached to the laminate.

Deniaud and Cheng (2001) tested 3700 mm long concrete T-beams strengthened externally using FRP sheets to study the interaction between FRP sheets and steel stirrups in carrying shear. The beams were designed to provide a flexural capacity 2.0 to 3.5 times greater than shear capacity without the FRP contribution. Three types of FRP were applied externally to strengthen the web of the T-beams: uniaxial glass fibre, uniaxial carbon fiber and triaxial glass fibre. The test setup consisted of a four-point loading system. Test results showed that FRP

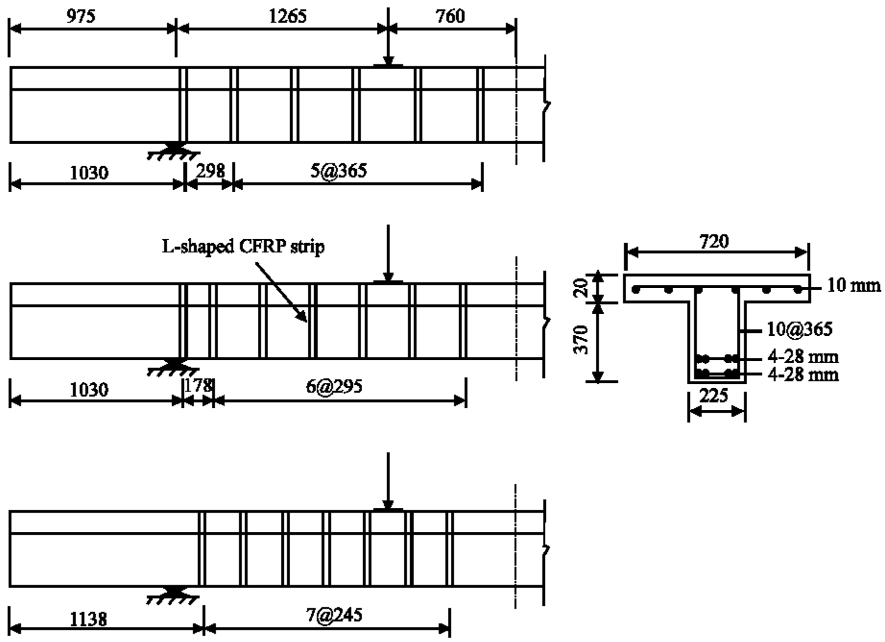


Fig. 8: Details of Lee and Al-Mahaidi (2003) T-beams

reinforcement increased the shear strengths from 77 to 117 above that of the beams without strengthening. The magnitude of the increase in shear capacity is dependent not only on the type of FRP but also on the amount of internal shear reinforcement. The FRP strains were found to be uniformly distributed for the fibres crossing the concrete at the shear crack.

Lee and Al-Mahaidi (2003) tested four large-scale-T-beams with identical reinforcement details (Fig. 8). The beams were 6000 mm long, 490 mm deep, the web was 225 mm wide and flange was 720 mm wide. All beams were designed to exhibit shear failure. The first beam was used as the control beam with no strengthening carried out. The second and third and fourth beams were strengthened with L shaped laminate strips spaced at 0.75D, 0.6D AND 0.5D where D is the overall depth of the beam. The nominal width of the plates was 40 mm with thickness of 1.2 mm. the beams were subjected to four point-loading with a shear span to depth ratio of 3.0. The test results showed that the stiffness of all beams tested were similar to each other in the early stage of loading. The presence of the external shear reinforcement did not affect significantly the initial stiffness of the strengthened beams compared to the control beam at first loading. The control beam failed in a gradual manner whereas the strengthened beams failed in an abrupt manner with a significant drop in load almost immediately after reaching the peak load. From the results, an increase in shear capacity of 54 to 81% was achieved in the T-beams strengthened with the external CFRP reinforcement with all beams failing in shear. The presence of the CFRP external reinforcement did not delay the initial formation of shear cracks but impeded their propagation and growth.

Cheng and Teng (2003) presented method to computing the contributions of FRP to shear strength in reinforced concrete beam. Also, this design approach based on models of failures so it can be predicted model of failure strengthened beam. Summary of proposals for FRP contribution to ultimate shear capacity:

For practical design, it can be assumed that shear cracks angle 45 degree. The contribution of FRP to the shear capacity can be expressed as:

$$V_f = 2f_{fe} \cdot t_f \frac{h_{fe}(\sin\beta + \cos\beta)}{s_f} \tag{16}$$

$$h_{fe} = d_{fi} - d_{ft} - (h - 0.9d) \tag{17}$$

d_{fi}, d_{ft} = distances from beam compressions face to lower and top edge of FRP on sides, respectively.

The effective stress in the FRP intersected by a shear crack at the ultimate limit state is given as:

$$f_{fe} = D_{FRP} \cdot \sigma_{F,max} \tag{18}$$

D_{FRP} = Stress distribution factor reflecting the no uniformity of stress distribution in the FRP along the critical shear crack.

$\sigma_{F,max}$ = maximum design stress of FRP intersected by the critical shear crack.

Researchers defined the D_{FRP}, $\sigma_{F,max}$ based on model of failure. Failure by FRP fracture in wrapping and U- jacketing schemes the factors will be need to calculated the effective stress at the failure are given as:

$$D_f = \frac{1+\zeta}{2}, \zeta = \frac{d_{ft}}{0.9d + d_{ft} + h} \quad (19)$$

Where the maximum design stress in FRP is defined as:

$$\sigma_{f,max} = \{0.8\, f_f/\gamma_f \text{ .... } f_f/E_f \le \varepsilon_{max} \text{ ....} \text{ or.. } 0.8\varepsilon_{max} E_f/\gamma_f \text{ .... } f_f/E_f \ge \varepsilon_{max} \quad (20)$$

f_f = tensile strength of FRP,
E_f = elastic modulus of FRP,
ε_{max} = 1.5% strain maximum.
γ_f = partial safety factor for FRP rupture = 1.25.

Failure by debonding the FRP from surface of concrete the following steps needs to calculate the D_{FRP}, $\sigma_{F,max}$ the first step:

$$\sigma_{f,max} = \min\left(\frac{f_f}{\gamma_f}, \frac{0.3}{\gamma_b} \beta_w \beta_l \sqrt{\frac{E_f}{t_f}\sqrt{f_{cu}}}\right) \quad (21)$$

γ^b = partial safety factor for FRP debonding = 1.25

$$\beta_L = 1 \text{........................} \lambda \ge 1$$
$$\beta_L = \sin\frac{\pi\lambda}{2} \text{...............} \lambda \le 1 \quad (22)$$

$$\beta_W = \sqrt{\frac{2w_t/(s_f \sin\beta)}{1 + w_f/(s_f \sin\beta)}} \quad (23)$$

$$\lambda = \frac{L_{max}}{L_e} \quad L_e = \sqrt{\frac{E_f t_f}{\sqrt{0.8 f_{cu}}}} \quad (24)$$

$$L_{max} = \frac{h_{fe}}{\sin\beta}. \quad \text{U-jackets} \quad (25)$$

$$L_{max} = \frac{h_{fe}}{2\sin\beta} \quad \text{Side-strips} \quad (26)$$

$$D_f = \frac{2}{\pi\lambda}\frac{1-\cos(\pi\lambda/2)}{\sin(\pi\lambda/2)} \quad \lambda \le 1$$
$$D_f = 1 - \frac{\pi - 2}{\pi\lambda} \quad \lambda \ge 1 \quad (27)$$

f_{cu} = cube compressive strength of concrete.

The researchers have verified their methods against the experimental data collected in the literature.

Deniaud and Cheng (2004) used a significantly different approach in predicting the capacity of beams strengthened in shear using FRP. This model is based on the shear friction theory with the lowest shear strength among all potential failure planes governing the shear strength of the beam. The shear strength of a beam reinforced with steel stirrups is

$$V_r = 0.25k^2 f'_c b_w h.\tan\theta + T_v(n-1) + nT_{FRP}, \quad \tan\theta = \frac{d_s}{ns} \quad (28)$$

Where the experimentally determined factor, K, is given the (Loov and Peng, 1998)

$$K = 2.1\,(f'_c)^{-0.4} \quad (29)$$

Where, n is the number of stirrups crossing the potential failure plane.

Where bw = width of the web; h = hieth of the beam;ds = height of the stirrups and s = stirrup spacing. Tv is the tension force in a stirrup and can be expressed as

$$T_v = A_v f_{vy} \quad (30)$$

Where Av and fvy = area and yield strength of the stirrups, respectively.

A continuous equation utilizing these modifications was developed to eliminate the need to determine the critical shear path, is expressed by

$$V_r = 2.1(f'_c)^{-0.4}\sqrt{f'_c A_c (A_v f_{vy} - T_{FRP})\left(\frac{d_s}{s}\right)} - A_v f_{vy} \quad (31)$$

In this equation TFRP, is calculated using equations developed in a parametric study using the strip method. The shear contribution of the FRP sheets bonded to the side of the beam web is given as:

$$T_{FRP} = \frac{S}{d_s} h_{FRP}.tE_{FRP}\varepsilon_{max} R_L \quad \text{with} \quad \varepsilon_{max} \le \varepsilon_{ultFRP} \quad (32)$$

Hfrp = height of the FRP sheets, t.EFRP and ultimate tensile strain of FRP can be obtained from the materials properties provided by the manufacturers. The maximum strain in the FRP sheets in millistrain is determined as.

$$\varepsilon_{max} = 3\sqrt{f'_c}..h_{FRP}^{0.16}(t.E_{FRP})^{-0.67}(K_a \sin\alpha)^{-0.1} \quad (\%) \ (33)$$

In this equation, ka represents the number of free edges for debonding.

Ka = 2 the sheets applied to each side of the web separately.

Ka = 1 the sheets wrapped around the soffit.

Ka = 0.79 the sheet extended underearth the beam flange no lessen that 100 mm minimum.

α : The orientation of fibre FRP to longitudinal axis of the beam.

The ratio of bonded to total length of the FRP, RL, is calculated in dimensionless form,

$$R_L = 1 - 1.2\exp\left(-(\frac{h_{FRP}}{K_e L_{eff} \sin\alpha})^{0.4}\right) \quad (34)$$

Where ke = integer describing the number of debonding ends, The effective length of the FRP sheets, Leff, is calculated using equation given by Maeda *et al.* (1997).

$$L_{eff} = \exp(6.134 - 0.58 \ln(t.E_{FRP})) \quad (35)$$

The same maximum sheet spacing of Khalifa *et al.* (1998) is used.

Adhikary and Mutsuyoshi (2004) tested eight 150 mm wide, 200 mm deep by 2600 mm long concrete beams strengthened in shear. The beams had no internal shear reinforcement in the desired shear failure region to ensure shear failure even after the application of carbon fibre sheets. The effects of various CFRP sheets configuration and layouts were studied. The test variables were the number of the layers of the CFRP sheet, Sheet depth, direction of fibre (vertical or horizontal) and side application or U-wrap. All the beams failed in shear. The ultimate loads for the shear strengthened beams were 29 to 119% higher than the control beam. The greatest shear strength was obtained for the beam with full U-wrapped sheets having vertically aligned fibers. Researches concluded that the results confirm the effectiveness of the U-wrap configuration in strengthening beams in shear. They recommended that the shear strengthened beam should be reinforced with CFRP sheets up to the maximum possible section depth to achieve the best strengthening effects.

Zheng *et al.* (2005) tested three controls beams and eight RC beams without shear reinforcement strengthened externally by using CFRP strips and fabrics. Eleven RC beams having cross- sectional dimension of 152.4×228.6 mm^2. the five beams had 1.22 mm a long and the other six beams had 1.83 m a long all the beams designed to fail in shear even after used externally shear reinforcement. In this study used five batches of concrete, externally bonded carbon fibre reinforced polymers were applied on both sides of the beams at various orientations with respect to the axis of the beam. The goals of this research: to increase the test database of the shear strengthening, to study the shear performance and modes of failure of RC beams with shear deficiencies after strengthening with CFRP laminate and to investigate the effect of the various CFRP types and shear reinforcement configurations on the shear behaviour of the beam. results of the test shown that the feasibility of using an externally bonded CFRP laminate system to strengthen or increase the ultimate shear capacity in the shear RC beam, also the CFRP system improved the ductility all beams after strengthened in shear. Researchers concluded that the beams with CFRP strips and the beam with CFRP fabric have completely different failure mechanisms. The strips failed by debonded while the fabrics failed by rupture of fibre. The experiment indicated that used strips increase the shear strength more than fabric In addition; Zheng *et al.* (2005) proposed design approaches to computing the CFRP contribution in the shear strength. Since more 60 available test results came up recently Zhichao Zheng updated Khalifa's equation for calculate the stress- reduction factor by used the same way Khalifa has used to derive this format in his design approach Fig. 9. This equation can be used to calculate the reduction factor for both failure fracture and debonding is shown below as.

$$R = 0.0387\ (\rho_f E_f)^2 - 0.5236\ (\rho_f E_f)^2 + 0.6393 \quad (36)$$

$$\rho_f E_f \leq 1.1\text{GPa}$$

Also, for more realistic analysis he found tow equations for computing reduction factor after separated data depends on the failure of mode.

R based on 30 test results of rupture:

$$R = -2.3156\ (\rho_f E_f)^2 + 1.5098\ (\rho_f E_f) + 0.3505 \quad (37)$$

R based on 30 test rest result of delamination:

$$R = 0.1466\ (\rho_f E_f)^{-0.8193} \quad (38)$$

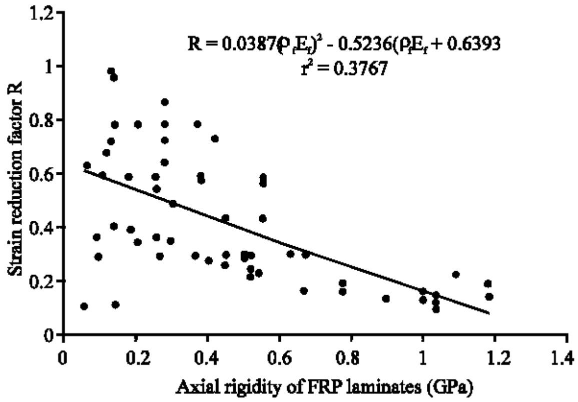


Fig. 9: Strain reduction factor R, base on all available test results

Also, Zheng *et al.* (2005) suggested that the reduction factor R can be plotted versus of axial rigidity divided into concrete compressive strength. R based on this idea gives as equation :

$$R = 1.8589.(\rho_f E_f / f'_c)^{-0.7488} \tag{39}$$

R after multiplied a safety factor 0.8 to equation:

$$R = 1.4871.(\rho_f E_f / f'_c)^{-0.7488} \tag{40}$$

Zheng *et al.* (2005) also suggested method to computing the stress- reduction factor value based on bonding mechanism. The reduction factor can be expressed as:

$$R = \frac{\tau_{max}.L_e}{2.f_{fu}.t_f} \tag{41}$$

τ_{max} is the ultimate direct shear strength MPa between the FRP and surface of concrete (Hsu *et al.*, 1997).

$$\begin{aligned}\tau_{max} &= (7.64 \times 10^{-4} \times f'^2_c) \\ &-(2.73 \times 10^{-2} \times f'_c) + 6.38\end{aligned} \tag{42}$$

Le is 75 mm the effective bonding length and tf is rthe thickness of the CFRP laminate.

The maximum reductions factor is suggested to be 0.4 in this research.

$$Rmax = 0.4 \tag{43}$$

In this design approach Eq. (2-40, 2-41 and 2-42) should be used together and the lower value of the these equations will be applied to computing effective stress in CFRP at shear failure.

The proposed design approach provides an acceptable prediction model. However, the effect of CFRP anchorage is not considered in the design approach.

Andors and Taljsten (2005a) studied the shear strengthening of reinforced concrete beams for increased shear capacity. The objectives of their study are investigating the effect of different parameters when restore RC beams for increased shear capacity with externally bonded carbon fiber composites. Twenty three rectangular larger scales RC beams were tested for this purpose. All the beams designed to fail in shear even after use CFRP for shear strengthened. The specimens divided tow groups, group A consisted of 20 RC without internal shear reinforcement. The beams in this group had 4500 mm long and 180.500 mm^2. Group B consisted of 3 RC beams with 3500 totally length, 180.400 mm^2 cross-section and internal shear reinforcement. Some of the beams in group a, they preloaded to predetermined level of damage in shear before apply the shear strengthening for Simulate real condition the beams in real life. The CFRP fibers with various thicknesses (1.2, 3 mm) were bonded in different directions about the longitudinal axis of beams (+45, -45, 90 and 0) degrees for study which directions it is more effective in shear strengthening. From this study, researchers concluded that use externally bonded CFRP is an effective method for increasing the shear capacity in shear. A pre-cracked Beam may be repaired not only its original capacity but to a capacity above what it had before. The strains are not uniformly distributed over the cross section of a rectangular beam.

Andors and Taljsten (2005b) presented new approach mode to predict the shear strength in strengthened beam by CFRP sheets for increased shear capacity, which depends on fact which came up from test results in the first work the strains are not uniformly distributed over the cross section of a rectangular beam. In this approach design the truss model have been used to quantity the contribution from externally bonded carbon fiber polymer. In their analysis three angles needed as show in Fig. 10.

α For shear crack (inclination cracks), β for fibre directions in relations to the beams longitudinal axis and θ for the angle between principle tensile stress and fiber direction $\theta = \beta + \alpha - 90$.

In this method, the contribution from the FRP composite, can be calculated as following expression which have derived by Andors Carolin:

$$V_f = \eta \varepsilon_{cr} E_f t_f r_f z \frac{\cos\theta}{\sin\theta} \tag{44}$$

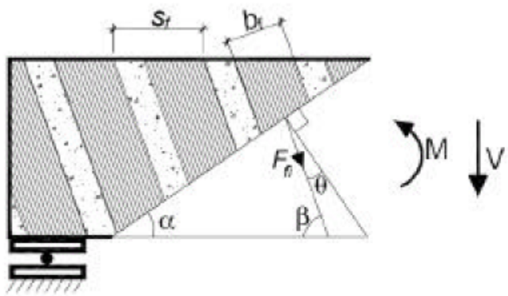


Fig. 10: Fibre direction and crack angle

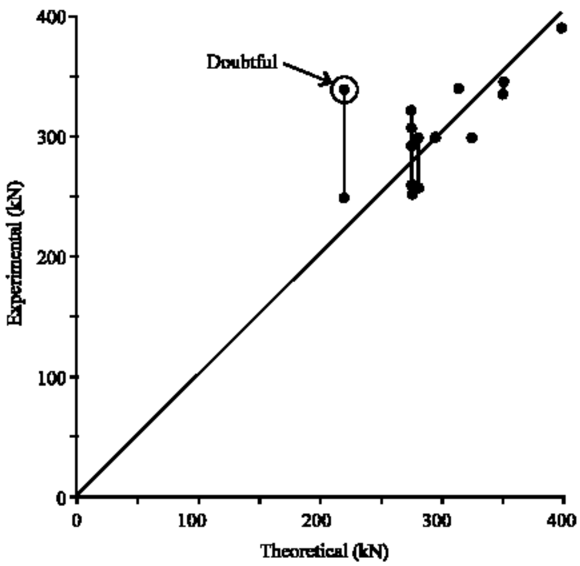


Fig. 11: Tested shear loads plotted versus theoretical capacity

E_f, t_f modulus elasticity of the composite, thickness of sheets or laminate, respectively.

The strain reduction factor, η, is dependent on the loading condition and fibre orientation.

The value of this factor is based on the analysis has done by authors, suggested to be 0.6.

$$\varepsilon cr = \min \begin{Bmatrix} \varepsilon_{fu} \\ \varepsilon_{bond} \cos^2 \theta \\ \varepsilon_{cmax} \cos^2 \theta \end{Bmatrix} \tag{45}$$

The critical strain, ε_{cr}, is limited by ultimate fibre strain ε_{fu}, anchorage of fibre ε_{bond} and the level of principal strain when a contribution from the concrete $\varepsilon_{c\,max}$.

Where z is the length of a vertical tension tie in the truss z = h composite bonded over the whole height. In others case z = 0.9d where d the effective depth of strengthened cross-section.

The factor r_f depends on the strengthening configuration.

$$r_f = \begin{Bmatrix} \sin\beta & \text{Complete coverage} \\ \dfrac{b_f}{s_f} & \text{Composites in strips} \end{Bmatrix} \tag{46}$$

The researchers have verified and compared this model against experimental data from tests in the first work as shown in Fig. 11. Fairly good agreement is found between results from tests and calculated values from theory with regard to both shear bearing capacity and average fiber utilization.

CONCLUSION

The literature clearly demonstrated that the FRPs can be used for rehabilitation and strengthening RC beam in shear. The degree of increase in shear capacity is dependent on the type of FRPs and amount of internal shear reinforcement. Of the various shear configuration tested, using strips was shown to be efficient if the direction of the strips is closer to being perpendicular to the principle tensile stress direction. The U-shape wrap or U jacketing of the sheets is more effective than strengthening only the tow- vertical side. In most cases the failure was by debonding of the sheets from the concrete surface. Also, The Literature reviewed a number of Design Approaches for Predicting the shear Capacity of FRP Strengthened Beams. Some from them have been become reliable methods for find the contribution from the FRP in shear-bearing capacity strengthened beam. From author's opinions, there are more than six methods to computing the contribution of the FRP in shear capacity strengthened beam in shear, this is mean, there is no good understood the strengthening RC beams in shear and no one is good enough for design. Therefore, this field from study needs more and more study.

From all the experiments found in the literature, the following conclusions can be drawn:

- The externally bonded FRP sheets can significantly increase the shear capacity of reinforced concrete beams;
- In all the cases, FRP fiber did not reach its ultimate tensile strength due to the bond to the concrete and stress concentrations;
- Generally, the failure mode of concrete beams with FRP sheets bonded to sides only or with U-jacket

FRP sheets is debonding of FRP from the concrete, which occurs along the shear crack; The failure mode of concrete beams with FRP totally wrapped around is FRP rupture or concrete failure, usually FRP rupture at the corner of the cross section or along the shear crack due to stress concentration;

- The bond between FRP and concrete is critical, which makes it possible to increase the shear strength of beams with U-jacket sheets or with sheets bonded to
- The sides. The bond also plays an important role in beams with FRP sheets wrapped around the section;
- Generally, wrapping FRP sheets around the cross section is the most effective way to increase the beam shear capacity, while bonding FRP on beam sides is the least effective way;
- The fiber orientation does not appear to have a great influence on the development of shear capacity;
- The effectiveness of the FRP diminishes as the volume of internal steel stirrups increases.

RECOMMENDATION FOR FUTURE WORK

The main limitation of the available experimental data is the relatively small size of the specimens tested, which do not represent the size of structural members in practice. The size effect may or may not, affect the results obtained but without data from large scale specimens no conclusion can be draw on this point.

Finally, to be more consistent with practice, the strengthening of the experiments should be undertaken under serviceability loading in their pre-cracked or damaged conditions. The database from the experimental researches in above this section shows that only Limited set of experimental studies per-damaged members, strengthening in shear CFRP sheets exists

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