SHEAR STRENGTHENING OF RC BEAMS SUBJECTED TO CYCLIC LOAD USING CFRP STRIPS

Gokhan Sakar

Department of Civil Engineering, Dokuz Eylul University, Izmir, Turkey

Received 29 December 2008; accepted 17 March 2009

ABSTRACT

An experimental investigation was conducted to study the effect of composite carbon fabric shear reinforcement on the ultimate strength and behaviour of reinforced concrete beam. Shear deficient specimens were strengthened by using side-bonded and wrapped CFRP straps. Eight beams were fabricated and tested under the cyclic loads. The main objective of the study is to obtain ductile flexural behaviour from the shear deficient RC beams. To verify the reliability of shear design equations and guidelines, experimental results were compared with all common guidelines and published design equations.

Keywords: Shear strengthening, CFRP strip, Cyclic Load, Reinforced concrete beam, Anchorage.

1. INTRODUCTION

The use of externally bonded fibre-reinforced polymer (FRP) composite strips or sheets for the strengthening of reinforced concrete (RC) members has become popular in recent years. This popularity has been due to well known advantages of FRP composites. CFRP strips are durable, have good corrosion resistance, are non-magnetic, have excellent fatigue behaviour, ease of handling and installation, are tailorable, present minimal disruption to the structure function and have a high strength to weight ratio. FRP shear strengthening has been more limited compared with FRP flexural strengthening of RC beams [1-5].

FRP materials are available in the form of rods, grids, sheets and winding strands. General review on class of materials including FRP used in civil construction was presented by Bakis et al [6]. They divided the whole review into structural shapes, internal reinforcement, externally bonded reinforcement, bridge, standards and codes. Also, Pendhari et al. classify the available analytical and experimental literature and to discuss the effects of different parameters to FRP materials [7].

RC beams fail in two types in general: Flexural failure and shear failure. The shear failure is sudden and brittle in nature. It is fewer predictable because it gives no advance warning prior to failure. So, it is more dangerous than flexural failure.

Many of the earlier studies about shear strengthening were concerned with the proof of effectiveness of CFRP as shear strengthening material. These studies indicated that using CFRP for shear strengthening is an effective and convenient method for improving member's strength and/or stiffness. In these studies, the behaviour of shear deficient beams strengthened with CFRP was also investigated [8].

The shear capacity of the strengthened RC beam depends on the compressive strength of concrete, the yield strength of shear and longitudinal reinforcement, the shear span to depth ratio and the area of composite fabrics [9, 10].

Bonding is critical for the effectiveness of CFRP strengthening. The effectiveness of strengthening techniques employing FRP relies on the adhesion between the FRP strap/sheet and the concrete surface of the element to be strengthened, both in uncracked and cracked concrete zones. One important aspect, peculiar to this technique, concerns the anchorage failure that occurs in a strap/sheet bonded to a concrete surface. Many studies, both theoretical and experimental, have been carried out on FRP-concrete adhesion [11]. For preventing debonding failure, anchorage details must be developed and applied at the end of the CFRP strips. But, limited

number of studies is currently encountered about developed anchorage details [12, 13].

However, behaviour of the CFRP was not examined under cyclic load as monotonic loading was applied in all these studies. Experimental results were compared with the analytical approaches that were suggested by ACI-440 Committee Report [14], Concrete Society TR55 [15] and Fib Bulletin 14 [16].

2. EXPERIMENTAL PROGRAMME

2.1 Specimens and material properties

Eight rectangular RC cantilever beams with different CFRP orientations were tested subjected to cyclic load. All specimens had the same geometry and materials as shown in Fig. 1. Longitudinal reinforcement consists of four 20 mm diameter steel rebars in the bottom and top of the beam section. Beams had no internal shear reinforcing rebars in the shear span. Stirrups were, however, utilized outside the shear span to block local cracks that might occur under the applied load. A single concrete mix was used for all specimens to achieve a similarity in strength. The concrete was a mixture of water, cement, sand and aggregate with the ratio of 0.68:1:2:3 by mass. The measured average cylinder compressive strengths of concrete cylinder were approximately 25 MPa. The yield stress of the longitudinal reinforcing steel was 414 MPa.

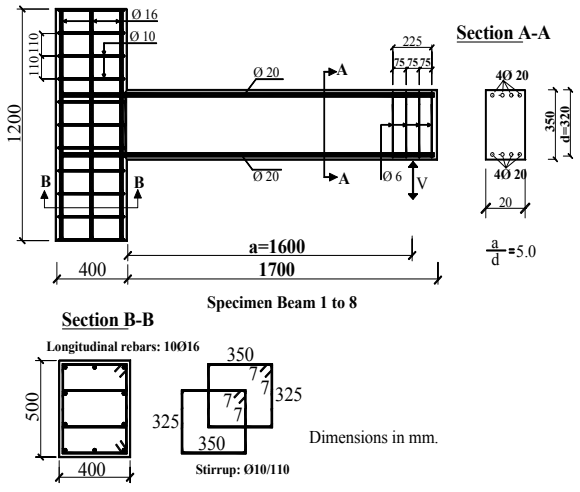


Fig.1: Reinforcement Details of Specimens

Beam-1 was the reference of shear deficient beams, tested without strengthening, and used as a baseline comparison to evaluate the enhancement in strength provided by the strengthening material. The remaining shear deficient RC beams were strengthened with CFRP strips.

Beam-2 was strengthened with 50 mm wide CFRP strips spaced at 80 mm. Strips were applied to the both sides of the beam symmetrically. Beam-3 was strengthened with 50 mm wide CFRP strips spaced at 120 mm. The beam-4 strengthening scheme was identical to beam-3. But the top and bottom ends of each of the strips were anchoraged to the beam with two anchorages. Beam-5 was strengthened with 50 mm wide strips spaced at 160 mm and anchorages were applied to both the top and bottom ends of the beam web. Strengthening scheme of beam-6 was identical to beam-5 but without any anchorage applied for strengthening material. Beam-7 was strengthened with 100 mm wide CFRP strips spaced at 160 mm. Beam-8 was strengthened with 50 mm wide CFRP strips spaced at 120 mm. CFRP was wrapped around the entire cross-section. CFRP strips were applied to both sides for all except beam-8.

Different mechanical anchorage details were developed in the study. Anchorages were included as 50x50x5 mm square steel plates and 10 mm diameter threaded rods with the anchorage details presented in Fig. 2. The location at which threaded rods were to be placed was marked for the anchorage application. The bottom and top marks at the beams' web were drilled with 12 mm diameter drills up to 50 mm depth. All the drilled holes were cleaned with pressured air. Threaded rods of 10 mm diameter were

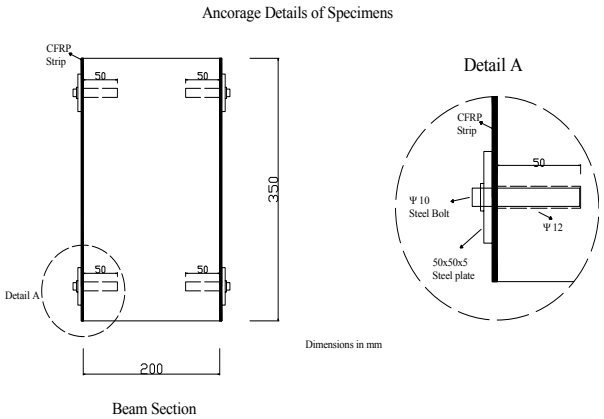


Fig.2: Anchorage Details of Specimens

placed at the bottom and top drilled holes by taking the bonding procedure of CFRP into account. While placing threaded rods, extra care was taken not to damage the continuity of fibre directions. After curing at room temperature, developed anchorage details were performed on the beam specimens.

There are several parameters that affect the capacity of adhesively bonded CFRP anchorages. The most important parameters are the anchor hole diameter, the CFRP sheet width, the embedment depth and the concrete compressive strength. Adhesive anchors transfer the applied load to the concrete through bond along the embedment depth. To have an effective bonding, manufacturers usually suggest 2–3 mm free space between concrete and anchor for the adhesive. The tensile capacity of CFRP anchorages is proportional to sheet width. However, the increase in tensile capacity of CFRP anchorages is not linearly proportional to the sheet width: tensile capacity of the anchors does not increase in the same amount with the sheet widths. There are several methods in the literature that can be used to predict the strength of adhesive steel anchors. However, there is not any successive way of predicting the tensile capacities of CFRP anchors yet [11].

2.2 Bonding procedure

Before applying to composite, some pre-treatment steps for the beams were necessary. First, the beams were sand-blasted to remove outer weak surface of the concrete and then the surfaces were vacuum cleaned to remove loose particles and dust. Epoxy primer was coated all throughout the beam according to the manufacturer's suggestions. CFRP strips were then placed on the coated epoxy primer and constant pressure was applied on the sheet surface by a roller. Another layer of epoxy was then put on top of the fabric and the extreme resin was cleaned. Specimens were cured for a minimum of 15 days under laboratory conditions. The properties of CFRP and the epoxy resin are presented in Table 1.

2.3 Test setup

A schematic view of experimental set-up and the arrangement of the measurement devices are shown in Fig.3. All cantilever specimens were tested under

Table 1: Properties of CFRP and Resin

Properties of CFRP	Remarks of SikaWrap 23^0C
Fibre Orientation	0^0 (unidirectional)
Construction	99% Wrap, 1% weft
Thickness (mm)	0.12
Tensile Strength (MPa)	4 100
Elastic Modulus (MPa)	231 000
Ultimate Tensile Strain (%)	1.7 %
Properties of Resin	Remarks of Resin
Tensile Strength (MPa)	30
Elastic Modulus (MPa)	3800

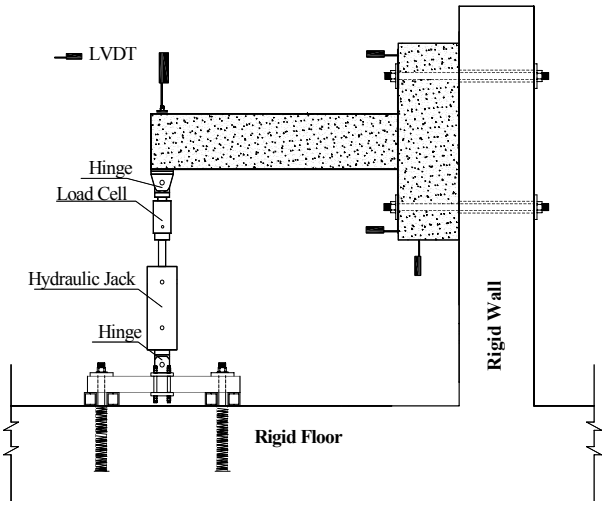


Fig.3: Test Setup and Instrumentation

cyclic load. To perform cyclic load to the specimen, a loading column was designed with hinges by the beam's free end. The loading column contained two hinges, a load cell and a hydraulic jack. The capacity of the hydraulic jack was 500 kN while the load cell's capacity was 400 kN. Load was applied in cycles of loading and unloading. The same loading cycles were applied to all specimens. Loading was increased up to the yield of the flexural reinforcements or until the failure of the specimen. For each load increment, displacements, loads and strains were recorded simultaneously by means of an automatic data acquisition system. Four linear variable differential transformers (LVDTs) were used to monitor displacements. The LVTDs are located at the end of the beam for maximum displacement, under the rigid support to calculate the undesired displacement and finally on the rigid support to calculate the rotation.

Strain gauges help to determine the shear cracks be-

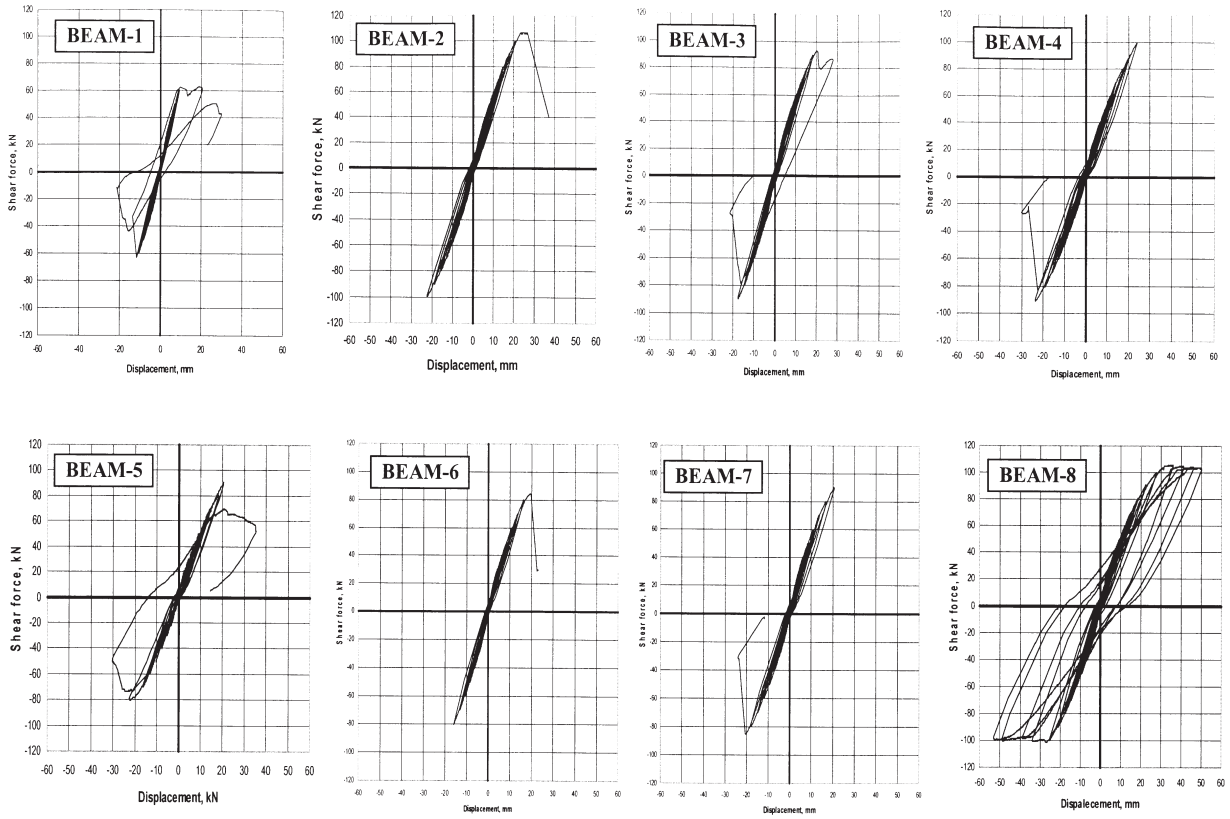


Fig.4: Load-Displacement Curves of the Specimen

fore propagation with the help of increase in strains. Strain gauges were attached to CFRP strap along the fibre direction and were attached to straps that were situated at distances between 80 to 1600 mm apart from the rigid support. Eight strain gauges were used for each specimen.

3. TEST RESULTS

3.1 Observed behaviour and failure modes

Load-displacement curves for the specimens are shown in Fig. 4. All strengthening arrangements significantly increased the shear strength of the beams. The first crack always appeared as a flexural crack at the maximum bending moment region for all beams. Shear cracks followed the flexure cracks at the unstrengthened part of the specimens, between CFRP strips. Later, shear cracks developed towards the strips because of the increasing load. In spite of the different strengthening scheme, first flexure crack propagation was observed between 34 kN and 40 kN load level for all specimens.

Beam-1 failed in shear because of a critical shear crack that developed at 61.7 kN load level (Fig. 5). First shear cracks developed on beam-2 at 69 kN load level. Beam-2 failed 108 kN load because of debonding. Splitting of one or more CFRP strips (with a layer of concrete adherent to them) caused main shear crack propagation.

First shear crack developed on beam-3 at 65 kN load level. Shear cracks were propagated and passed under the CFRP strips with increasing load. At the forwards loading cycle of 100 kN, load was decreased suddenly at 92 kN load. At the same backwards loading cycle, the beam failed without warning at 76 kN load level. Specimen failed due to the peeling of

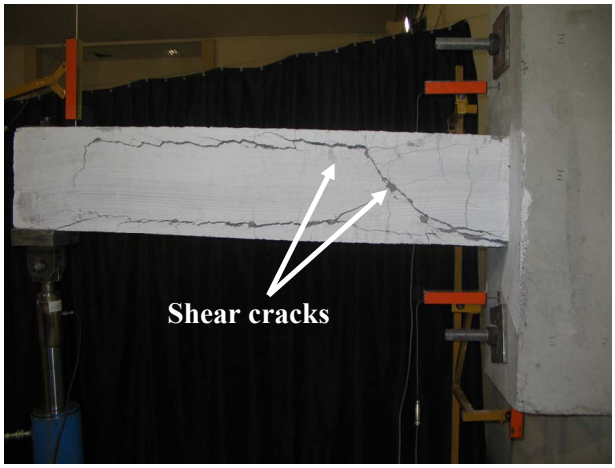


Fig.5: Failure Mode, Shear Failure

some CFRP strips from concrete surface, the shear crack was propagated through the beam web and separated the concrete cover and the beam abruptly failed in shear.

Beam-4 was strengthened with the same arrangements of CFRP straps for beam-3 but anchorages were applied in this case. At the 83 kN load, specimen failed suddenly due to the shear failure. A few CFRP strips above the shear crack debonded from the concrete surface. The beam failed due to peeling of CFRP strips from concrete surface (Fig. 6).

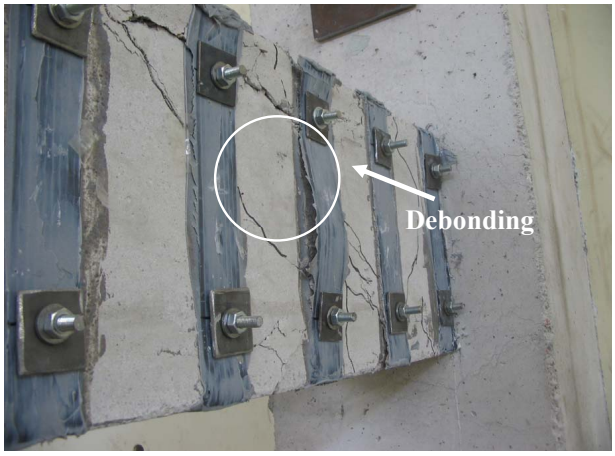


Fig.6: Failure Mode, Debonding

Beam-5 failed because of the concrete cover separation over the longitudinal reinforcement (Fig. 7). Specimen finished the 90 kN forwards cycle with success, but failed in backwards loading of the next cycle when the load was 74 kN. Some straps tried to peel from the concrete surface, but that was obstructed by the anchorages. Anchorage delayed splitting of strips at lower loads.



Fig.7: Failure Mode, Concrete Cover Separation

In beam-6, failure was initiated by debonding of CFRP strips (with a layer of concrete adhered to them) over the main shear crack. It was followed by shear failure at a load of 84 kN. In the absence of end anchorages, beam-6 had less displacement capacity. Specimen reached the maximum 19.61 mm displacement which was 25% less than that of the beam-5.

Different from beam-6, wider strips were used to strengthen beam-7. Despite the different strengthening scheme, they had the same failure mode and the similar behaviour. Brittle failure occurred at a load of 85 kN by debonding of CFRP strips followed directly by shear compression failure.

To change the failure mode and to increase the strength, different strengthening scheme was applied to beam-8. Complete wrapping in which the FRP is wrapped around the entire cross-section was used to strengthen beam-8. Longitudinal reinforcements of the specimen were reached at their yield strengths at 102.2 kN and 101.1 kN loads at the forwards and the backwards loading cycle, respectively. Hereafter, the behaviour of the specimen changed. Despite the new load cycles that was applied to the beam, no remarkable increase in the number and size of cracks was observed after yielding of longitudinal reinforcements. Yet, a small change in stiffness was observed from the shear force-displacement curve after yielding. The specimen was able to act like a ductile beam without losing its load-carrying capacity.

3.2 Evaluation of test results

The test results pointed out that CFRP is an efficient

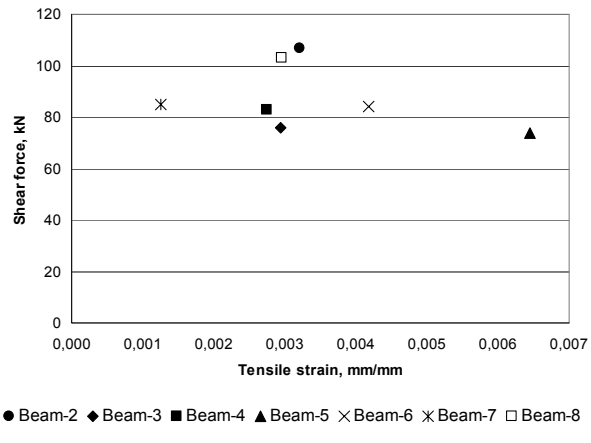


Fig.8: Measured Maximum Tensile Strains of CFRP

Table 2: Experimental Results

Specimen		f_c (MPa)	W_f (mm)	S_f (mm)	Anchorage	Arrangements	Failure Load (kN)	Maximum load at the forward cycle	Maximum load at the backward cycle	Failure Mode at Ultimate
Specimen-1	**Control**	25.40	------	------	------	------	61.7	62.4	-63.1	Shear
Specimen-2	**Strengthening**	27.30	50	80	------	Side bonding	108.0	108.0	-99.2	Shear
Specimen-3	**Strengthening**	27.90	50	120	------	Side bonding	-76.1	92.5	-89.0	Shear
Specimen-4	**Strengthening**	24.80	50	120	Yes	Side bonding	-83.0	99.6	-89.7	Shear
Specimen-5	**Strengthening**	23.90	50	160	Yes	Side bonding	-74.0	89.5	-79.8	Shear
Specimen-6	**Strengthening**	27.10	50	160	------	Side bonding	84.0	84.0	-80.1	Shear
Specimen-7	**Strengthening**	25.20	100	160	------	Side bonding	-85	89.5	-85	Shear
Specimen-8	**Strengthening**	25.40	50	120	------	Wrapped	102.0	102.0	-100.6	Flexure

*Forward loading step was described with positive mark and backward loading step was described with negative mark in Table 2

material to increase the shear capacity of RC beams. Table 2 summarizes of the results of specimens during test.

Comparing to the beam-1, a minimum increase of 20% and a maximum of 75% in shear capacity was acquired from the strengthened specimens. Beam-2 showed 75% more strength than beam-1. Beam-3 showed 23% more strength than the reference beam. By decreasing the amount of the strengthened area (increasing the spacing of CFRP strips from 80 to 120), a decrease of 42% at contribution to the shear resistance was obtained according to beam-2. The beam-4 was strengthened in a manner similar to that of beam-3. The ends of the strips in beam-4 were anchoraged. Apart from beam-3, beam-4 had reached 100 kN loading cycle but failed in 83 kN load. The specimen showed 9% more strength than beam-3. Anchorages did not provide any substantial increase in shear strength but they obtained remarkable increment in the displacement capacity of beam-4.

Beam-5 had more strip space than aforementioned strengthened specimens. Hence, the least strength improvement was observed at that beam among the strengthened specimens. It was showed 20% more strength than the control beam. Beam-5 was showed 14% less strength than beam-6 that was strengthened with same CFRP arrangement without anchorages. This result might be arising from abrupt debonding failure of the specimen. Beam-6 showed 36% more strength than beam-1. It was minimum displacement capacity beam of the strengthened specimens. Beam-7 was nearly the same shear capacity as beam-6. The specimen had double strip width as to beam-6 but that application did not improve the shear strength.

As a result of the use of wrapping, a significant increase in the shear capacity was achieved in beam-8. Furthermore, the failure mode at ultimate changed from CFRP debonding to flexural failure mode.

3.3 Load-strain behaviour

Strain values can be used to indicate the contribution of CFRP to the shear resistance. It is therefore essential to evaluate the strain activity of the strengthened specimens to determine the contribution. The strain values reported herein are not necessarily the maximum values (Fig. 8). They are strictly related to the location of the strain-gauges with respect to that of the shear cracks [11].

Strain values that were measured for beam-5 at

which shear cracks localized or passed under were larger than the others. These values were %61 larger than the expected ACI 440 committee report value [14]. The largest strain value of that was measured at CFRP straps of beam-7 was 0.0013 mm/mm.

Beam-3 and beam-4 was strengthened with 50 mm wide CFRP strips spaced at 120 mm. The maximum strain values of these beams were 0.0032 mm/mm and 0.0029 mm/mm, respectively and were very close. Anchorages did not decrease the maximum strain values of the specimens. Differently, beam-5 and beam-6's strain values were not close to each other. Values were 0.0064 mm/mm and 0.0042 mm/mm which correspond to 160% and 105% of the expected value outlined in the ACI-440 committee report, respectively. Wider strip space increased the strain values significantly. Besides, developed anchorages caused larger strain measurements from the CFRP strips of beam-5.

Beam-7 was strengthened with 100 mm wide CFRP strips spaced at 160 mm that was nearly the same shear capacity as of beam-6. Apart from the shear strength, the maximum strain level of the specimen showed 3.32 times less than beam-6. A low strain level of the beam can either be explained by fewer shear crack propagations, deficient bond or success of the strengthening scheme. The measured local maximum strain of the CFRP wrap was 0.0029 mm/mm. 50 mm wide CFRP straps spaced at 120 mm were used in the beams 3, 4 and 8 with the dissimilar application types. Despite the discrepant application scheme, strain levels of the specimens were nearby.

4. COMPARISON OF EXPERIMENTAL AND ANALYTICAL RESULTS

Results from both analytical and experimental study are shown in Table 3. The analytical shear force capacities of specimens were calculated according to guidelines. For all these methods, shear strength of a strengthened RC section could be expressed as the sum of the three components.

$$V_n = V_c + V_s + V_f \quad (1)$$

Where, V_c is the contribution of concrete, V_s is the contribution of internal steel shear reinforcement and finally V_f is the contribution of CFRP in equation (1). Concrete and/or steel stirrups can not utilize their ultimate strengths when CFRP is used as a strengthening material, as demonstrated in equation (1). Common design codes, however, still use the relative contribution of these materials to compute the total shear strength. It must be considered that these parameters interact with each other. For this reason, analytical and experimental results must be evaluated according to the aforementioned considerations.

In the literature, there are several methods to predict the tensile capacities of anchors embedded into concrete. These methods can be subdivided into two groups: concrete cone models and bond models. These two groups can also be sub grouped such as bond models neglecting the shallow concrete cone or combined cone-bond models. In concrete cone models, the tensile capacity of the anchor depends on the embedment depth and the compressive strength of the concrete where anchors are installed. The effect of anchor diameter is ignored in concrete cone models. Similar to cone models, bond models also depend on the embedment depth in addition with the effects of bond strength between concrete and adhesive, and anchor diameter. Besides these, some researchers revealed that the compressive strength of the concrete is another parameter influencing the capacity while some others did not. Bond models, neglecting the effect of shallow concrete cone, differ from the other models by the embedment length definition. The length of the anchor is reduced to account for the shallow concrete cone [11]. Differences were determined between analytical shear load capacities that were suggested by ACI-440 committee report, The Concrete Society TR55, Fib Bulletin 14 and experimental results. The reasons for obtaining different results in analytical calculation and experiments from the expected were that the guidelines did not consider a/d ratio and the effects of anchorage.

The analytical shear force capacities of beams were calculated according to ACI-440 committee report, Concrete Society TR55 and Fib Bulletin 14 sugges-

Table 3: Comparison of Experimental and Analytical Results

Specimen	Experimental Strengths	Calculated Strengths for ACI-440	Calculated Strengths for Concrete Society, TR55	Calculated Strengths for Fib Bulletin 14	Experimental/calculated		
	$V_{exp.}$ (kN)	$V_{cal.}$ (kN)	$V_{cal.}$ (kN)	$V_{cal.}$ (kN)	$V_{exp.}/V_{cal.}$ for ACI-440	$V_{exp.}/V_{cal.}$ for TR55	$V_{exp.}/V_{cal.}$ for Fib
BEAM-2	108	66.15	114.15	77.29	1.63	0.95	1.40
BEAM-3	76	60.32	93.89	73.81	1.26	0.81	1.03
BEAM-4	83	61.04	95.10	74.53	1.36	0.87	1.11
BEAM-5	74	53.21	76.83	67.19	1.39	0.96	1.10
BEAM-6	84	52.16	75.21	66.08	1.61	1.12	1.27
BEAM-7	85	68.60	80.87	79.53	1.24	1.05	1.07
BEAM-8	102	62.84	128.41	84.59	1.62	0.79	1.21

tions. Maximum divergences between analytical and experimental shear load capacities were calculated from ACI-440 results. Moreover, The Concrete Society TR55 calculated closer results for the specimens. The closest results of the experimental programme were obtained for beam-7. The shear capacity of beam-7 was 24% less than that of the experimental result for ACI-440 and 5% and 7% less than The Concrete Society TR55 and Fib Bulletin 14, respectively. The experimental and analytical results were not too close. Thus, the impact of the ratio a/d, the anchorage effect and the interaction of internal shear reinforcement to shear capacity due to CFRP were not included in the guideline. The proposed design results would be more realistic if the unconsidered parameters were measured while improving the preceding equations.

5. CONCLUSIONS

Strengthening of reinforced concrete beams in shear using CFRP straps appears to be a highly efficient technique. Developed anchorage detail worked efficiently under the cyclic load. Anchorages prevented the peeling of the CFRP from concrete. However, bottom anchorages were efficient as they prevent the premature peeling of the CFRP.

All CFRP applications improved the strength and behaviour of the specimens in a different way. The strength was increased between 1.20 and 1.75 times compared to reference beam. Besides the displacement capacity of the strengthened beams was improved meaningfully. In addition, Beam-8 that was strengthened with the wrapped CFRP strips behaved like a ductile flexural beam.

Significant differences were determined between the analytical shear force capacities that were suggested by ACI-440 committee report, The Concrete Society TR55, Fib Bulletin 14 and experimental results. The reasons for obtaining different results in analytical calculation and experiments from the expected were that the guidelines did not consider a/d ratio and the effects of anchorage.

ACKNOWLEDGMENTS

The author wishes to thank Dokuz Eylul University for supporting this project. The study was also supported by Afaprefabrik Precast Concrete Ind. & Trade Co., Inc., Izmir, Turkey.

References:

1. **Norris, T., Saadatmanesh, H., and Ehsani, M.,** "Shear and Flexural Strengthening of R/C Beams

with Carbon Fiber Sheets", *Journal of Structural Engineering,* **123**/7, (1997), 903-911.

2. **Kachlakev, D., McCurry, C.C.,** "Behavior of Full Scale Reinforced Concrete Beams Retrofitted for Shear and Flexural With FRP Laminates", *Composites: Part B,* **31**/6-7, (2000), 445-452.
3. **Saadatmenaesh, H., Ehsani, M.,** "Fibre Composite Plates can Strengthen Beams", *Concrete International ACI*, **12**/3, (1990), 65-71.
4. **Khalifa, A., Nanni, A.,** "Improving shear capacity of existing RC T-section beams using CFRP composites", *Cement and Concrete Composites,* **22**/3, (2000), 165-174.
5. **Chajes, M., Januska, T., Mertz, D., Thomson, T., and Finch, W.,** "Shear Strengthening of Reinforced Concrete Beams Using Externally Applied Composite Fabrics", *ACI Structural Journal,* **92**/3, (1995), 295-303.
6. **Bakis, C.E., Bank, L.C., Brown, V.L., Cosenza, E., Davalos, J.F. et al.** "Fiber Reinforced Polymer Composites for Construction-State of the Art Review", *ASCE Journal of Composites for Construction,* **6**/2, (2002), 73-87.
7. **Pendhari, S.S., Kant, T., Desai, Y.M.,** "Application of Polymer Composites in Civil Construction: A General Review", *Composite Structures,* **84**/2, (2008), 114-124.
8. **Sakar, G., Tanarslan, H. M., Alku, O.Z.,** An Experimental Study on Shear Strengthening of RC T-section Beams with CFRP Plates Subjected to Cyclic Load, *Magazine of Concrete Research,* **61**/1, (2009), 43-55.
9. **Triantafillou, T.C.** "Shear Strengthening of Reinforced Concrete Beams Using Epoxy-Bonded FRP Composites", *ACI Structural Journal,* **95**/2, (1998), 107-115.
10. **Swamy, R. N., Mukhopadhyaya, P., Lynsdale C. J.,** "Strengthening for Shear of RC Beams by External Plate Bonding", *Structural Engineering,* **77**/12, (1999), 19-30.
11. Fib, *"Retrofitting of Concrete Structures by Externally Bonded FRPs, with Emphasis on Seismic Applications",* Fib Bulletin **35**, (2006), Technical Report.
12. **Khalifa, A., Alkhrdaji, T., Nanni, A., Lansburg, A.** "Anchorage of Surface Mounted FRP Reinforcement", *Concrete International, ACI,* **21**/10, (1999), 49-54.
13. **El-Mihilmy, M., Tedesco, J.,** "Prediction of anchorage failure of RC beams strengthened with fibre-reinforced polymer plates", *ACI Structural Journal,* **98**/ 3, (2001), 301-314.
14. American Concrete Institute, *"Guide for the Design and Strengthening of Externally Bonded FRP Systems for Strengthening Concrete Structures",* ACI, Detroit, (2001), ACI Committee 440 Report.
15. Concrete Society, *"Design Guidance for Strengthening Concrete Structures Using Fibre Composite Materials"*, The Concrete Society, Camberley, (2000), Technical Report 55, UK.
16. Fib, *"FRP as Externally Bonded Reinforcement of RC Structures: Basis of Design and Safety concept",* Fib Task Group 9.3, Fib Bulletin 14, International Federation for Structural Concrete, Switzerland, (2001).

CONVERSION FACTORS

1 mm =0.039 in

1 mm^2 =0.00152 in2

1 kN =0.2248 kips

1 MPa =145 psi

SYMBOLS

a : Shear span

d : Effective height of the cross section

f_c : Compressive strength of concrete

L : Length of the beam

$V_{exp.}$: Experimental shear forces of specimens

V_{cal} : Calculated shear forces of specimens

ϕ : Diameter of reinforcement

w_f : CFRP strip width

s_f : Center-to-center spacing between CFRP strips