STRENGTHENING OF RC T-SECTION BEAMS SUBJECTED TO CYCLIC LOAD USING CFRP SHEETS

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ABSTRACT

The results of an experimental and analytical study of the behaviour of reinforced concrete (RC) T-section beams strengthened with carbon fibre reinforced polymer (CFRP) sheets are presented. Shear deficient specimens are strengthened by using side-bonded and ⊔- jacketed CFRP sheets. The effect of CFRP sheets on strength and stiffness of the beams is considered for various orientations of the fibres. The main objective of the study is to obtain ductile flexural behaviour from the shear deficient RC beams. To verify the reliability, test results were compared with results of ACI-440 design code, Concrete Society TR55, Fib Bulletin 14 and Colotti et al. model.

KeyWords: RC Beam, Strengthening, Shear, CFRP, Cyclic Load, Anchorage.

1. INTRODUCTION

Various strengthening materials and techniques were carried out all over the world in the last two decades, from a considerable need of repairing and retrofitting of existing structures. It is obvious that steel plates and fibre reinforced polymers (FRP) are the two most popular external shear-strengthening materials. Carbon fibre reinforced polymer (CFRP) may provide more economical and technically superior alternative to the traditional methods. CFRP sheets are cost-effective, 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. Numerous studies have been undertaken in the structural flexural strengthening area. [1-3]. However, the shear strengthening studies of reinforced concrete (RC) beams by the use of CFRP have been limited and must certainly be expanded [3-6].

Many of the previous studies on shear strengthening were related to the effectiveness of CFRP as a shear strengthening material. Moreover, effects of parameters on the behaviour of shear deficient beams were examined in literature [7-9]. These parameters are shear span to depth ratio (a/d), composite materials strength and ratio, compressive strength of concrete, yield strength of shear and longitudinal reinforcement. A number of studies were also focused on the failure modes. Eventual failure of almost all test beams occurred in one of the two main failure modes: tensile rupture of the CFRP and debonding of CFRP from the concrete [10, 11].

However, monotonous loading was applied in all these studies, and thus, behaviour of the CFRP was not examined under cyclic load. Bonding is critical for the effectiveness of CFRP strengthening. For preventing this type of failure, anchorage details must be developed and applied at the end of the CFRP plates. But, limited number of studies is currently encountered about developed anchorage details [12, 13].

This paper provides the results of an experimental study conducted on shear strengthening of RC T-section beams with CFRP sheets under cyclic loads. All seven beams used in the study except the flexural reference had no internal shear reinforcements. Shear deficient specimens were strengthened by using side-bonded and ⊔- jacketed CFRP sheets, while one of the shear deficient specimens was tested without strengthening to serve as a reference to shear deficiency. Unidirectional and bidirectional CFRP sheets were used for strengthening in this paper. Developed mechanical anchorage detail was also applied to all strengthened specimens.

Ductile flexural failures of the strengthened beams were aimed in this application. The effects of type and arrangements of CFRP plates that were used for improvement on behaviour, strength, energy dissipation capacity, failure mode and stiffness of the specimens were investigated. Experimental results were compared with the analytical approaches that were suggested by ACI-440 Committee Report [14], Concrete Society TR55 [15], Fib Bulletin 14 [16] and Colotti et al. model [17].

2. EXPERIMENTAL PROGRAMME

2.1 Specimens and material properties

Seven T-section RC cantilever beams with different CFRP orientations were tested subjected to cyclic load. All specimens had same geometry and materials as shown in Fig. 1. Longitudinal reinforcement consists of three 20 mm diameter steel rebars in the bottom and top of the beam section. For flexural reference 6 mm diameter closed stirrups, spaced at 75 mm spacing, were utilized. The remaining shear deficient beams do not include shear reinforcements. The measured average cylinder compressive strengths of the concrete were 30 MPa. The yield stress of the longitudinal reinforcing steel was 414 MPa.

Beam-2 was the reference beam, to be tested without strengthening. The remaining RC beams were strengthened with CFRP sheets. Anchorage was used for all the strengthened beams. Strengthening scheme details were presented in Fig. 2. Beam-3 was strengthened with two-ply unidirectional CFRP sheets. The first ply was applied 45° inclination to the beam axis (45°) while the second was bonded perpendicular to the first ply (135°). The purpose of the plies was to investigate if they would have any affect in stopping the spread of shear cracks. Beam-4 was similar to the Beam-3 and was also strengthened with two-ply unidirectional CFRP sheets, but its strengthening scheme was applied with 60° and 120° angles to the longitudinal axis of the beam, respectively. Beam-5 was strengthened with two ply unidirectional CFRP. The first ply was bonded parallel to beam axis (0°) and the second ply was bonded perpendicular to the first ply (90°). The purpose of adding a horizontal ply in 0° direction was to determine if horizontal bonded CFRP would make any affect on increasing the beam shear capacity. Strengthening procedure was also constituted for Beam-5 to see whether it would act like a bidirectional CFRP application or not. Beam-6 was strengthened with one-ply bidirectional CFRP sheets. Different from Beam-5, CFRP was applied on the sides, tension face and under the flange throughout the beam (⊔-jacketing). Beam-7 strengthening scheme was simi-

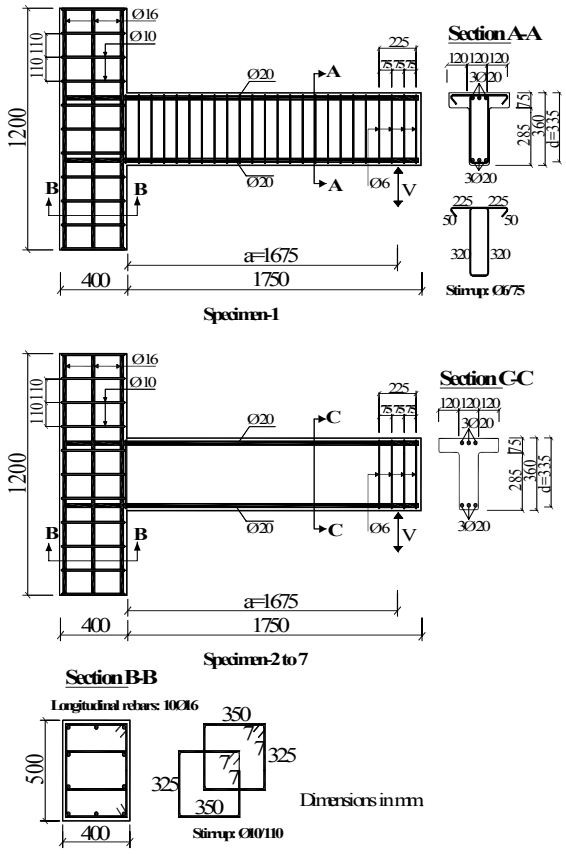


Fig.1: Reinforcement Details of Specimens

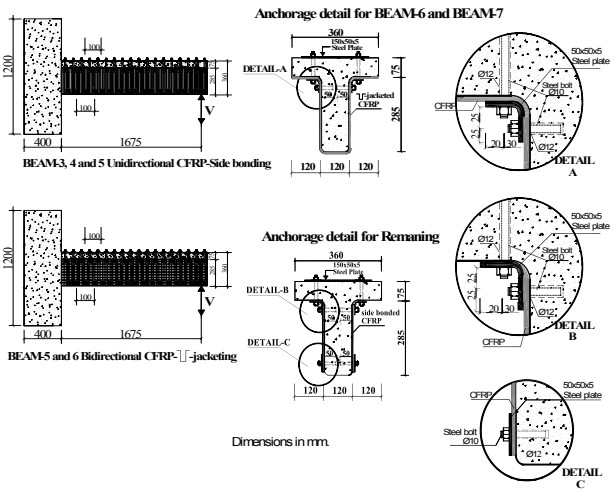


Fig.2: CFRP Arrangements and Anchorage Details for Specimens

lar to Beam-6. But, this was strengthened with two-ply bidirectional CFRP sheets in this case.

Shear strengthening of beams may be problematic as they are cast monolithically with slabs. This phenomenon increases the difficulty of anchoring the FRP at the beam-slab junction and intensifies the risk of debonding failure. However, bonding FRP on both sides, or the sides and the soffit, provide some shear strengthening for that kind of members. In this wise, different mechanical anchorage details were developed in the study. Anchorages were included as 50x50x5 mm square steel plates, 'L' shaped 50x50x5 mm 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. For the bottom, distances of 100 to 1600 mm with intermittent steps of 100 mm were marked, respectively, from the support. For the top, distances of 50 to 1550 mm with intermittent steps of 100 mm were marked, respectively, from the support. The bottom and top marks at the beams' web were drilled with 12 mm diameter drills up to 50 mm depth. Points corresponding to the top drilled holes were drilled with 12 mm diameter drills throughout the flange. All the drilled holes were cleaned with pressured air. Threaded rods of 10 mm diameter were 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. As bottom anchorage, 50x50x5 mm steel plates were used. As top anchorages, 'L' shaped 50x50x5 mm steel plates were used. To anchor 'L'-shaped plates to the beam's flange, 10 mm diameter threaded rods were used. This procedure was carried out for the beams 3, 4, and 5. For Beam-6 and Beam-7, bottom anchorage details were not applied. Even so, the top anchorage detail is exactly the same as was specified earlier.

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 [18].

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

Table 1: Properties of CFRP and Resin

Properties of CFRP	Remarks of SikaWrap 160C	Remarks of SikaWrap 230C
Fibre Orientation	$0^0/90^0$ (bidirectional)	0^0 (unidirectional)
Construction	50% Wrap, 50% weft	99% Wrap, 1% weft
Thickness (mm)	0.045	0.12
Tensile Strength (MPa)	3 800	4 100
Elastic Modulus (MPa)	230 000	231 000
Ultimate Tensile Strain (%)	1.5 %	1.7 %
Properties of Resin	Remarks of Resin	
Tensile Strength (MPa)	30	
Elastic Modulus (MPa)	3800	

primer was coated all throughout the beam according to the manufacturer's suggestions. CFRP sheets 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 Experimental setup

A schematic view of experimental setup and the location of the measurement devices are shown in Fig. 3. All specimens were tested under cyclic load. Load was applied in cycles of loading and unloading with a 1000 kN capacity hydraulic jack and was controlled with a 600 kN capacity load cell. The same loading cycles were applied to all specimens. Loading was increased up to the yield load of the flexural reinforcements or until the failure of the specimen. Four linear variable differential transformers (LVDTs) were used to monitor displacements. The LVDTs were 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 were used to determine the shear cracks before propagation. To monitor the strain changes, seven strain gauges were therefore attached at the section mid-height with distances of 150, 300, 450, 600, 750, 900, 1050 mm, respectively, from the support [19].

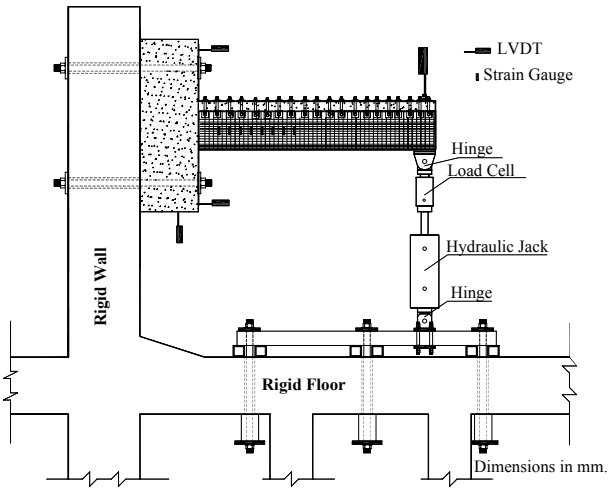


Fig.3: Test Setup and Instrumentation

3.EXPERIMENTAL 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 in entire specimens. Shear cracks were not visually examined because of fibre application. However, to specify any crack propagation, strain activity and the sound that emerges during shear, crack propagation was taken into consideration.

Beam-1 showed ductile flexural behaviour as expected. Large displacements were observed, and specimen reached an ultimate load value 11% greater than the yield load. Also a plastic hinge was developed at the maximum moment region. Beam-2, which was fabricated to designate the shear deficiency, was failed in shear. Three shear cracks were developed in the shear span at 39.50 kN load level. As a result of these cracks, the reference specimen failed in shear as expected. Two failure modes were determined from the strengthened beams, namely, tensile rupture of the CFRP and concrete cover separation over the lower rebar.

Beam-3 was reached its yielding strength at 86 kN load level both in the forwards and backwards loading cycles. Then two displacement-controlled load cycles were applied. When the load was 83 kN, initial crack development was advanced until the concrete cover separation occurred and the specimen subsequently failed.

The longitudinal compression and tension reinforcements of Beam-4 were reached at their yield strengths at 83 kN load level. In the next load cycle, the beam failed at backwards loading of the 82 kN load level. Beam-4 failed because of the concrete cover separation over the lower rebar. Beam-3 and Beam-4 had the same failure mode and the similar behaviour (Fig. 5).

Beam-5 was strengthened by two-ply unidirectional CFRP sheets in which plies were applied perpendicular to each other. While forwards loading of the 70

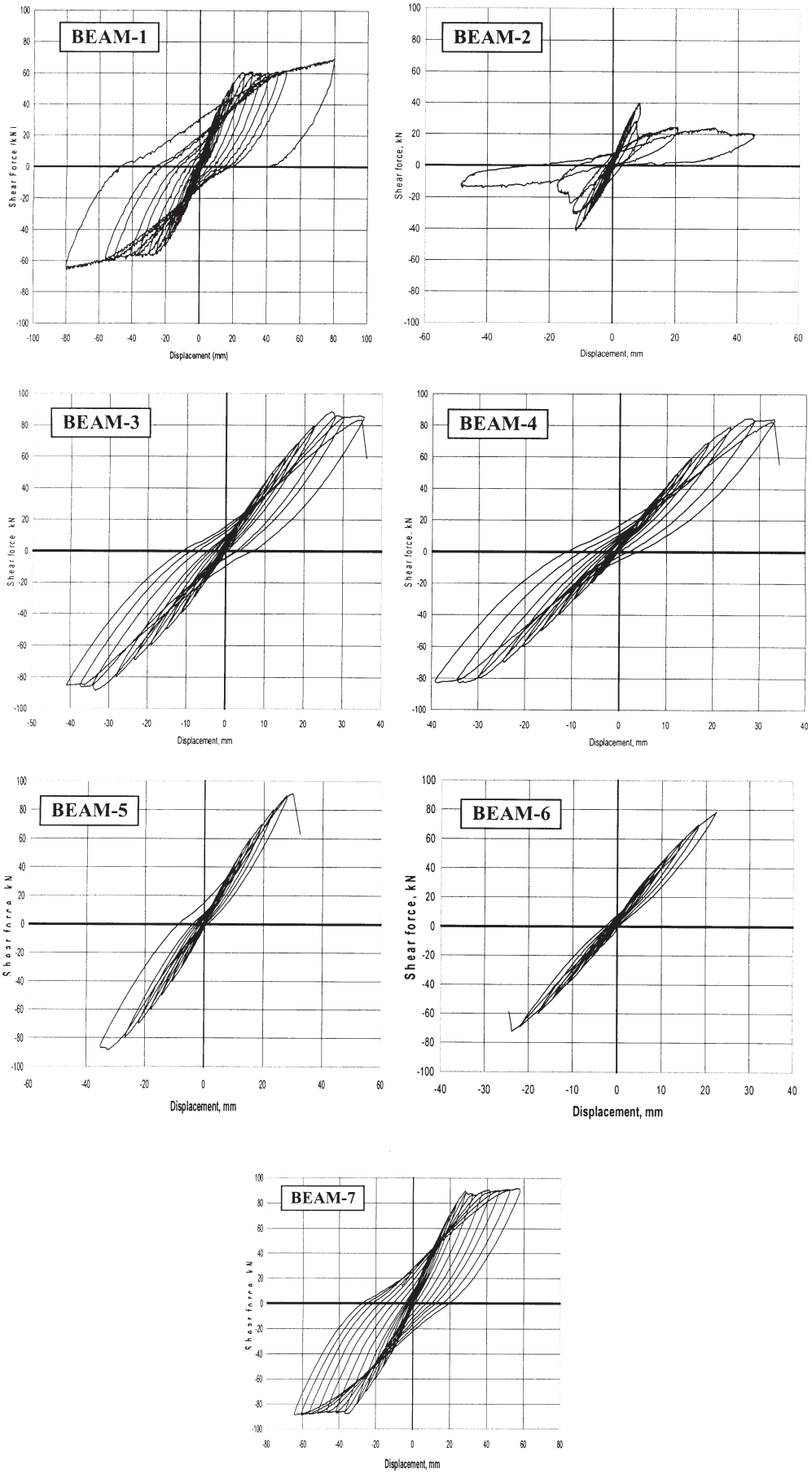


Fig.4: Load-Displacement Curves of the Specimen

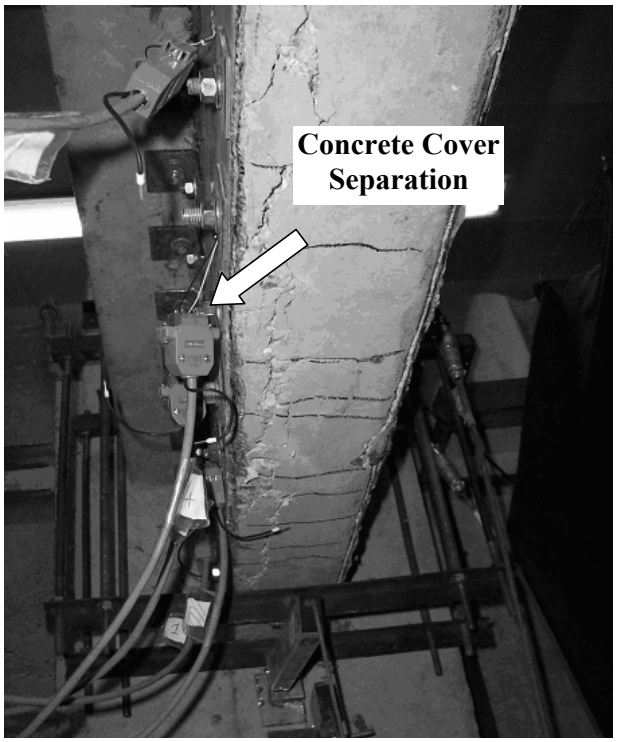


Fig.5: Failure Mode, Concrete Cover Separation

kN load cycle, a significant crack parallel to beam axis was occurred at the bottom of the web but the development of the crack was concluded in a short period of time. This crack exhibited no development until failure. The longitudinal reinforcements were reached at their yield strengths in the 90 kN forwards load cycle. In the next load cycle when the load was 91.38 kN, initial crack development at the bottom face was progressed till the concrete cover separation happened and the beam subsequently failed.

Different from the other strengthened beams, ⊔-jacketed CFRP sheets were used for Beam-6. A considerable crack was not developed up to the 80 kN. After reaching 80 kN at the forwards loading cycle, the CFRP sheet was ruptured abruptly at 72.28 kN at the backwards loading. The rupture continued with diagonal shear crack propagation and the specimen failed rapidly (Fig. 6).

To change the failure mode and to increase the strength, extra ply was added to Beam-7. Two-ply ⊔- jacketed bidirectional CFRP sheets were applied to Beam-7. Longitudinal reinforcements of the specimen were reached at their yield strengths at 88.25 and 86 kN load, at the forwards and the backwards loading cycle, respectively. Hereafter, behaviour of

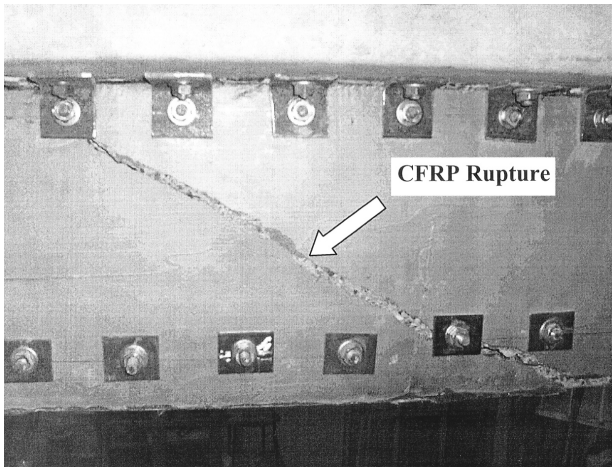


Fig.6: Failure Mode, CFRP Rupture

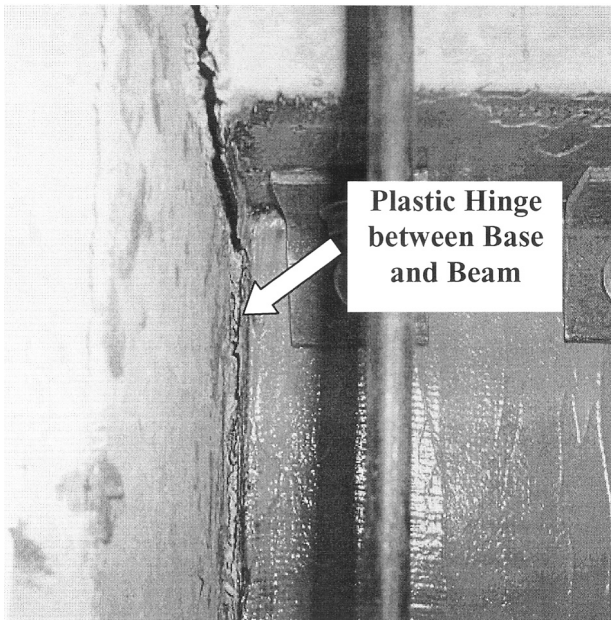


Fig.7: Failure Mode, Flexural Failure

the specimen changed. Despite new load cycles that were 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 seen from the shear force-displacement curve after yielding. It was clearly seen that plastic hinge was formed between the supporting base and the cross-section of the beam (Fig. 7). The specimen was able to act like a ductile beam without losing its load-carrying capacity. The ductility and the strength of Beam-7 were increased in comparison with Beam-6. The success depends on the CFRP orientation and the ply number. ⊔- jacketing hindered the loss of concrete cover over the lower rebar.

According to the test results, strengthening by one-ply CFRP was not adequate. Apart from other strengthened specimens, Beam-6 failed because of the CFRP rupture. Furthermore, the strength values of this specimen were smaller than the others. Longitudinal reinforcements of the specimen were not reached at their yield strengths. It was therefore more rigid and having less displacement capacity.

3.2 Evaluation of test results

Response envelope curves were drawn by connecting peak points of the loading cycles and presented in Fig. 8. Envelope curves were used for the comparison of the strength and stiffness of specimens. After yielding of longitudinal tension reinforcement at 87.38 kN, Beam-1 showed 70.6 mm displacement up to the failure. Beam-2 failed under 39.50 kN load with 8.71 mm displacement. The test results pointed out that CFRP is an efficient material to increase the shear capacity of RC beams. Table 2 summarizes of the results of specimens during test.

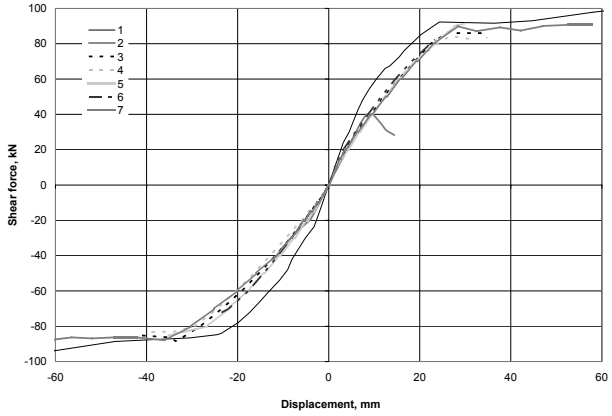


Fig.8: Response Envelopes of Specimen Beams

Comparing to the Beam-2, a minimum increase of 98% and a maximum of 131% in shear capacity was acquired from the strengthened specimens. Beam-3 showed 9% less strength than Beam-1 and 124% more strength than Beam-2. Beam-4 failed at 83 kN load level and showed 2.10 times more strength than Beam-2. The results show that 45° and 60° application types had nearly the same strength and behaviour.

Beam-5 was the strongest beam of the experimental programme. To simulate bidirectional CFRP, two perpendicular unidirectional plies were used to strengthen Beam-5. Through this orientation, an increase by 131% over Beam-2 was obtained.

The new application scheme was used to strengthen Beam-6. The specimen failed because of the CFRP rupture at 78.33 kN load level. Premature rupture caused a minimum strength on Beam-7 with 98% over the reference beam (Beam-2). Beam-6 also showed 20% less strength than the flexural control specimen (Beam-1). As ⊔-jacketing was able to prevent concrete cover separation, Beam-7 was strengthened in the same way. Two-ply bidirectional CFRP was used to strengthen Beam-7. This strengthened scheme worked so well that the specimen behaved as a ductile flexural beam. The strength of Beam-7 was also increased by 130%.

In addition to the strength, the initial and yielding stiffness values of the beams were examined. Initial stiffness values were calculated by using the

Table 2: Experimental Results

Specimen		Flexural Yield Load (kN)	Failure Load (kN)	Initial stiffness (kN/mm)	Stiffness at Yield (kN/mm)	Ductility Ratio	Energy dissipation capacity (kNmm)	Failure Mode at Ultimate
BEAM-1	**Control**	-----	39.5	6.02	3.80	3.55	7112	Shear
BEAM-2	**Control**	-----	-75.6	4.47	-----	-----	1163	Shear
BEAM-3	**Strengthening**	86	83	4.07	2.58	1.30	1997	Flexure-Shear
BEAM-4	**Strengthening**	83	82	3.75	2.65	1.22	1681	Flexure-Shear
BEAM-5	**Strengthening**	90	91.4	4.91	2.69	1.02	1219	Flexure-Shear
BEAM-6	**Strengthening**	-----	72.3	4.20	-----	-----	531	Shear
BEAM-7	**Strengthening**	88.3	90.7	4.63	3.08	2.01	4946	Flexure

slope of the lines that connected the origin and the load, the point at which first flexural crack occurred. The biggest initial stiffness value was calculated from Beam-5. Perpendicular plies postponed crack propagations; therefore, the beam's initial stiffness became 10% greater than that of Beam-2. Except for Beam-5 and Beam-7, initial stiffness value of all strengthened beams exhibited lower values.

The specimen's ductility ratios were examined to decide what kind of CFRP arrangement is more suitable for increasing the ductility. For Beam-6, it was not possible to obtain their ductility ratios. Beam-7 failed with ductile flexural behaviour and its ductility ratio was calculated as 2.01.

Energy dissipation of the beams was calculated by using area under load-displacement curves for each cycle. Dissipations for each cycle were summed for computing total cumulative energy dissipation. Those values were presented in Table 2. Maximum energy dissipation was seen from Beam-1. Beam-6 dissipated minimum amount of energy. Longitudinal reinforcements of the Beam-6 were not reached at their yield strengths and they failed with CFRP rupture. Strengthening with CFRP plates significantly increased the cumulative energy dissipation capacity of specimens. When CFRP ply number was doubled, energy dissipation increased meaningfully.

Totally seven strain measurement were taken from the specimens. Strain values can help to determine the contribution of CFRP to the shear resistance. Strain values that were measured Beam-6 at which shear cracks localized or passed under, so that values larger than the others. Besides, the strain values that were measured from Beam-3 were distributed more homogenously. The strain values presented herein are not necessarily the greatest values. They are completely connected to the position of the gages with respect to that of the shear cracks. Actually, largest strain graphs of the specimens were presented in Fig. 9.

Strain values of the inclined strengthened beams were a bit different. Measured maximum strain value of Beam-3 was 0.0033 which corresponds to 83% of the expected value outlined in the ACI-440 committee report. But Beam-4's same value was 0.0048. This value was 20% higher than the expected ACI-440 committee report value. This distinction may be explained by fewer shear crack propagations or success of the 45° inclination strengthening scheme.

The maximum strain value of Beam-5 was 0.0027 mm/mm. That corresponds to 68% of the expected value outlined in the ACI-440 committee report. The low level strain of Beam-5 can be explained through general strain behaviour. Strain activity exactly depends on crack spreading and development. As the load increases, small size shear cracks propagate through the beam. These cracks cause no or slight differences at strains. In the next load cycles, CFRP starts to play a great function for obstructing the development of the cracks that widen. The contribution of CFRP can be assessed that time by the increase in strain. A low strain level of the specimen can therefore be explained by fewer shear crack propagations, deficient bond or success of the strengthening scheme.

Despite the fact that Beam-7 reached its yield load at the forwards and backwards cycle, CFRP did not contribute to the resistance as much as Beam-6. However, the maximum strain gauged at failure was 0.0023 mm/mm. It is clear that Beam-7's strengthening scheme worked so well that no obvious contribution was needed to change failure mode. Maximum strain values of the beams were presented in Fig. 10.

4.COMPARISON OF EXPERIMENTAL AND ANALYTICAL RESULTS

Results from both analytical and experimental study are shown in Table 3. The analytical shear force strengths of specimens were calculated according to guidelines. For all of them, the nominal shear strength of the beam is:

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

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 equa-

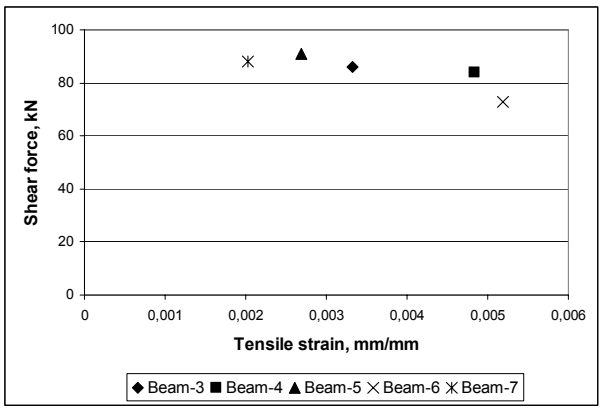


Fig.9: Measured Maximum Tensile Strains of CFRP

tion (1). It is not guaranteed that concrete and/or steel stirrups can exploit their maximum strengths, as exhibited in equation (1). But, all common codes still use the relative contribution of these materials to calculate the total shear strength. It is clear that these parameters interact with each other. Hence, experimental and analytical results must be considered according to former points.

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

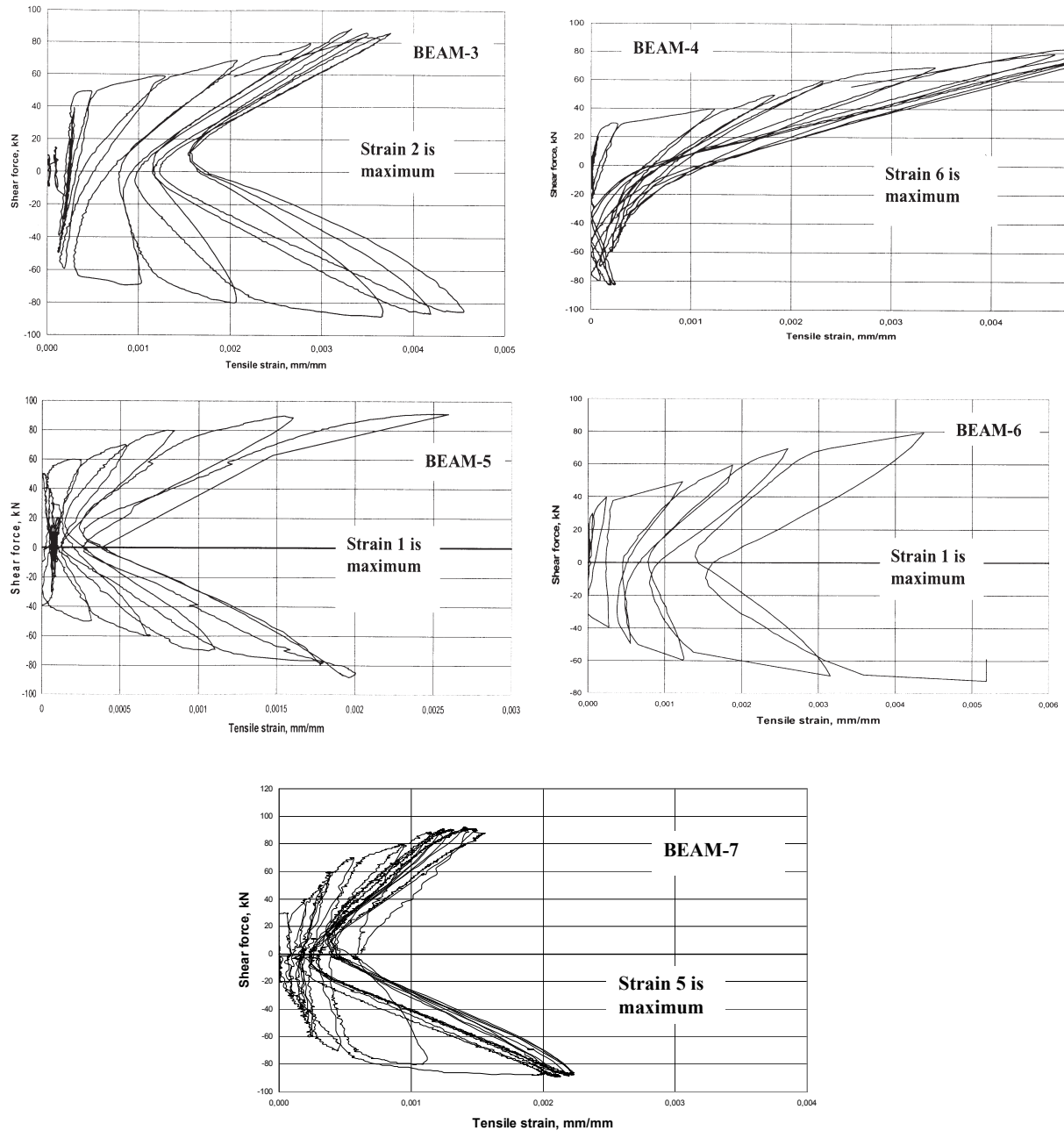


Fig.10: Load-Maximum CFRP Tensile Strain Curves of BEAM-3, 4, 5, 6 and 7.

et al. suggestions. As this result indicates, ACI-440, Concrete Society and Fib Bulletin 14 results were not too close to the shear capacity of the specimens, whereas Colotti et al. model results showed nearly the same shear capacity as obtained from the experimental program.

The closest results of the experimental programme were obtained for Beam-5. The shear capacity for Beam-5 was 7% more than that of the experimental result for ACI-440 and 2% more than The Concrete Society TR55. Analytical shear load-carrying capacities that were calculated with ACI-440, for the specimens, showed the biggest distinction with the range of 7% and 33% from experimental results. Besides, The Concrete Society TR55 and Fib Bulletin 14 calculated closer results for the specimens [20]. Different from the above design guidelines, Colotti et al. model results caused maximum 10% difference between experimental and analytical shear force values. For Beam-3, Beam-5 and Beam-7, Colotti et al. model results closely the same shear capacity as obtained from the tests. These close results were based on the effect of a/d which was considered in Colotti et al. model procedure. Influences of anchorages to shear force capacity must also consider in the analytical equations that were offered by the guidelines and researchers. The proposed design results would be more realistic if all the parameters that affect the behaviour were obtained from the guidelines.

5.CONCLUSIONS

The shear behaviour of the strengthened T-section RC beams with shear deficiencies was investigated in this experimental programme. Test results showed that the externally bonded bidirectional CFRP could improve the ultimate shear strength and change the failure mode from shear to flexure.

• Developed anchorage detail worked efficiently under cyclic load. Top anchorages prevented the separation of beam flange and web. It also prevented peeling of the CFRP from concrete. However, bottom anchorages prevent the premature peeling of the CFRP and it was very successful. Hence, top and bottom anchorages helped to increase the shear strength of the beams.

• CFRP applications improved the specimens' strength in a different way. Beam-5 showed the biggest strength among the specimens. Ultimate shear capacity of Beam-7 was 90.71 kN, which corresponds to an increase of 130% over the shear deficient control beam. More important from the strength, the beam behaved like a ductile beam. Ac-

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	Calculated Strengths for Colotti et al. model	Experimental/Calculated			
		$V_{exp.}$ (kN)	$V_{cal.}$ (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	$V_{exp.}/V_{cal.}$ for Colotti et al.
BEAM-3	Side bonding	88.33	97.29	82.96	103.9	87.95	0.91	1.06	0.85	1.00
BEAM-4	Side bonding	83.00	109.84	97.78	109.3	90.07	0.76	0.85	0.76	0.92
BEAM-5	Side bonding	91.38	97.92	93.72	81.5	89.27	0.93	0.98	1.12	1.02
BEAM-6	U-jacketing	78.33	58.84	55.57	67.8	86.85	1.33	1.41	1.16	0.90
BEAM-7	U-jacketing	90.71	74.22	68.85	75.7	86.27	1.22	1.32	1.20	1.05

cording to the results, two-ply bidirectional CFRP with ⊔- jacketing seems to be the best way for shear strengthening.

• Strengthening with CFRP plates was significantly improved the cumulative energy dissipation capacity of beams. When CFRP ply number was increased, energy dissipation capacity also increased. Energy dissipation increased 9.33 times more, when CFRP ply number was doubled. Beam-6 dissipated minimum amount of energy among the strengthened specimens because of its ply number.

• The purpose of adding a horizontal ply in 0° direction was to determine if it would have any effect on increasing the beam shear capacity. According the test results, the horizontal ply helped to increase the contribution of the concrete and it postponed the crack propagation at the bottom of the section. At 70 kN load level a considerable crack, parallel to beam axis, was formed at tension face, but with the help of horizontal ply, the development at that crack was postponed until 90 kN.

• Significant differences were determined between analytical shear load capacities that were suggested by ACI-440 committee report, The Concrete Society TR55 and Fib Bulletin 14 and experimental results. The reasons of obtaining different results from analytical calculation and experiments than expected was due to the fact that the guidelines did not consider to a/d ratio and effects of anchorage. Different from these guidelines, Colotti et al. model nearly the same shear strength results as obtained from the tests. The similarity occurs because of the influence of a/d ratio, which was included in the procedure.

• It is not guaranteed that concrete and/or steel stirrups can exploit their maximum strengths when CFRP is used as a strengthening material. But principal design equations still indicate that this is the case. The contribution of concrete may be decreased if large strains are required to calculate the contribution of CFRP. To prevent any improper approaches, however, some design codes set limit on the effective strain i.e. ACI-440 committee report. But the test results showed that this limit could be exceeded. So, the efficiency and acceptability of this limit must be examined in further studies.

The study was an initial step to cope with experimental data shortage. It is intended with this study that is clarified further, the shear mechanism of a RC beam strengthened with externally bonded CFRP. More in depth studies may still be needed to support the step of efficiencies of each orientation and arrangement including the use of anchorage for shear strengthening of RC beams.

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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