

SHEAR STRENGTHENING OF R. C. T- BEAMS BY MEANS OF CFRP SHEETS

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This research aimed to experimentally investigate the contribution of bonded web carbon fiber reinforced polymer (CFRP) sheets to the shear strength of reinforced concrete (RC) T-beams. Two strengthening techniques using CFRP strips were applied along the shear-span zone: the first one is vertical U-jacket and the later is vertical strips bonded to the beam sides only. Fibers of both U-jacket and side sheets were vertically oriented ($\theta = 90^\circ$). Test results showed that the strengthening technique with U-jacket CFRP sheets improved the shear strength particularly. Three mechanisms of failure were recognized for the tested beams depending upon the end condition of the bonded CFRP sheet. Although the failure mode for the different beams was a brittle one, the strengthened beams provided with U-jacket CFRP sheets showed more or less a ductile behavior at a higher loading level up to a load level just before failure. As a consequence, these beams approved an acceptable enhancement in the structural ductility. Moreover, the obtained results concerning the strains induced in the CFRP sheets are used to study the applicability of the mathematical model suggested by the ACI code [1, 2].

Keywords: Carbon Fiber Reinforced polymer (CFRP); delamination; strengthening; anchorage.

1. INTRODUCTION

The technique of externally bonding carbon fiber reinforced polymer (CFRP) is applied more and more, and becomes an attractive solution for strengthening/retrofitting the reinforced concrete (RC) structures. This due to the good properties and characteristics of carbon fiber reinforced polymer (CFRP) sheets such as: non-corrosiveness; easy to be bent and wrapped; lightweight; higher strength in comparison to the traditional reinforcement (steel plates). Moreover, the simplicity of application in a wide variety without any special precautions and conditions, which are considered so necessary to apply the conventional strengthening technique, e.g. steel plate technique. Also, under certain environmental condition, steel plating has been demonstrated to be vulnerable to corrosion especially at steel plate-epoxy interface, as well as the limited available length of steel plates may cause some problems in typical situations [3]. Hence, externally bonded CFRP sheets technique has been the adequate substitute for that of steel plates and it can solve problems and difficulties, which obstruct the application of steel plate technique in some situations.

In multistory building, generally, the beams used in the lower floors resist high shear forces during earthquakes. This results in close spacing of internal stirrups that may violate spacing requirements given in codes for proper placing of concrete. By using FRP reinforcement, especially CFRP sheets, bonded to the web of RC, the designers can increase the spacing of internal stirrups, and both the shear capacity and the cracking behavior can be improved. This technique can be used in repair and retrofit of existing beams, as well as in designing new RC beams. Several studies [4-15] have been conducted to study experimentally and analytically the behavior of RC beams having a rectangular cross-section and strengthened in shear with FRP reinforcements, and few is known about those dealing with the behavior of strengthened beams having T- cross-section, e.g. [16, 17]. Moreover, the different studies derived different expression to predict the effective strain induced in the bonded web sheets; the contribution of the bonded web reinforcement to the shear strength depends mainly on such effective strain.

In this paper, tests on six RC beams with T- cross-section and strengthened in shear with externally bonded CFRP sheets were conducted to study the contribution of the bonded sheets to the shear strength of RC T-beams. Also, the influence of the examined strengthening technique on both the ductility and failure mode was defined. Moreover, the induced strain values in the CFRP sheets were utilized to study the applicability of the mathematical model suggested by the ACI code [1, 2] to predict the effective strains induced in the CFRP sheets.

EXPERIMENTAL STUDIES

Layout of Experiments and Materials

The behavior of RC T-beams deficient in shear strength and strengthened with externally bonded web CFRP sheets was investigated by conducting two-point loading bending test on six beams over a simple span of 2000 mm, in which the effective depth (d) is 280-mm and shear-span to depth ratio (a/d) equals 2.5. All the tested models (*CS.O*, *CS.1*, *CS.2*, *DS.O*, *DS.1* and *DS.2*) have an identical T-cross-section: flange width and thickness are 450 and 70-mm, respectively, web width is 150-mm, and total section height is 320-mm. Main reinforcement of all tested beams were 4 Φ 18 mm (Steel 360/520) in the tension side; in addition, four rebars of 8-mm-diameter (Steel 240/350) are used as compression reinforcement. Young's modulus of the tension and compression rebars is 215 GPa and 208 GPa, respectively. No internal stirrups was provided along the shear-span of the samples *CS.O*, *CS.1* and *CS.2*; on the other hand, internal stirrups of 6-mm-diameter and 140-mm spacing (Steel 240/350 and Young's modulus \approx 205 GPa) were provided along the shear-span in case of *DS.O*, *DS.1* and *DS.2*, see Figs. 1 and 2 as well as Table 1.

CS.O and *DS.O* were tested in their original condition to serve as control samples (without strengthening). These beams were designed to fail mainly due to shear. Beams *CS.1* and *DS.1* were strengthened with twelve CFRP strips of 100-mm width and 200-mm spacing (six strips per shear-span). These strips of CFRP sheets were bonded to the two sides of the beam. However, Beams *CS.2* and *DS.2* were strengthened with the same effective cross-sectional area of fiber sheets of 78 mm² per shear-span (three U-jacket strips each of 100 mm in width). Both the U-jacket and strips were with the

fibers oriented vertically ($\theta = 90^\circ$) and distributed uniformly along the shear-span of the beam, Figs. 3 and 4.

The tested beams were cast using a concrete mix achieving mean splitting strength and Young's modulus of 2.85 and 23000 N/mm², respectively. The mean compressive strength for the standard cylinder (f'_c) and cube (f_c) after 28 days for the tested beams were listed in Table 1.

Table 1: Data of tested beams

Beam No.	Beam Data				Shear Strengthening per Shear Span
	f_c (N/mm ²)	f'_c (N/mm ²)	A_s	Internal Stirrups	
CS.0	31.0	26.7	4 Φ 18 mm	No stirrups along shear-span	Control Beam 1
CS.1	30.5	26.2			6 strips, each of 100 mm width, see Fig.3
CS.2	30.5	26.2			3 U-jacket strips, each of 100 mm width, see Fig.4
DS.0	31.0	26.7	4 Φ 18 mm	Stirrups ϕ 6-mm of 140-mm spacing	Control Beam 2
DS.1	31.5	27.1			6 strips, each of 100 mm width, see Fig.3
DS.2	31.5	27.1			3 U-jacket strips, each of 100 mm width, see Fig.4

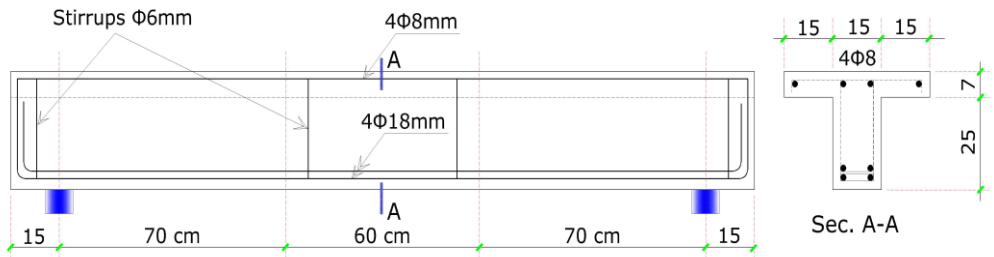


Fig. 1: Details of internal reinforcement for tested beams: CS.0, CS.1 & CS.2.

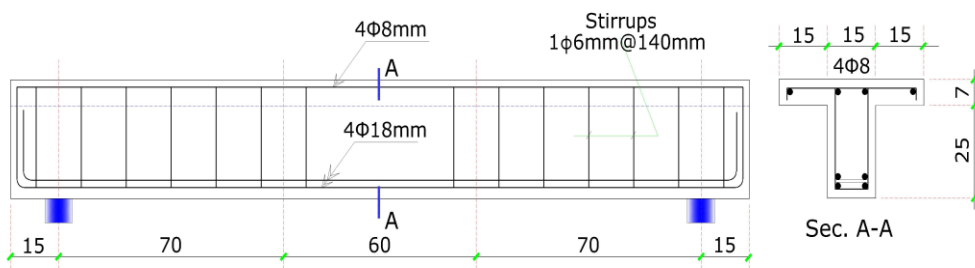


Fig. 2: Details of internal reinforcement for tested beams: DS.0, DS.1 & DS.2

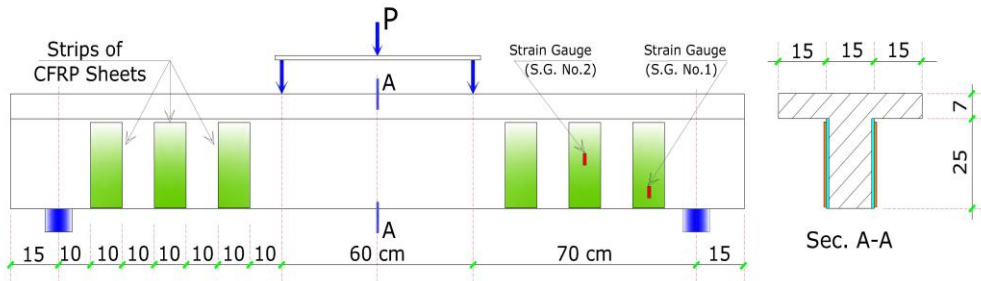


Fig. 3: Details and arrangements of bonded CFRP sheets for tested beam CS.1 & DS.1

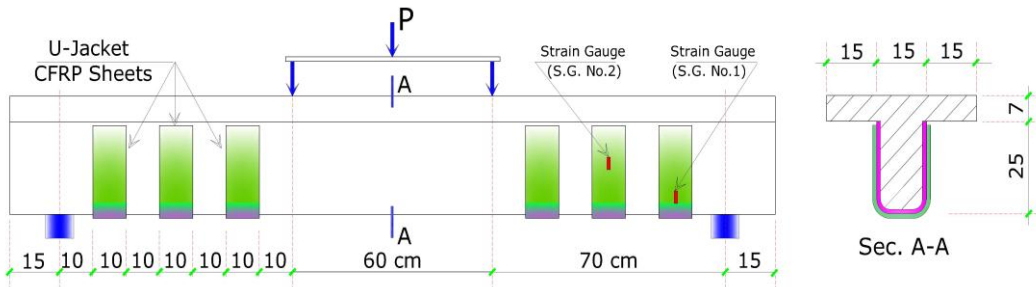


Fig. 4: Details and arrangements of bonded CFRP sheets for tested beam CS.2 & DS.2

The external reinforcement was a CFRP sheet [18]. Such CFRP sheet is available in rolled sheet of 0.13-mm effective thickness, 300-mm width and 50-m length. The effective thickness gives the section of the fibres in each single ply. The rupture strength, ultimate (= failure) strain and Young's modulus of such CFRP sheet are 3500 N/mm², 15.2 mm/m (= 15.2‰) and 230000 N/mm², respectively.

An epoxy mortar layer of about 2-mm thickness was applied to all the strengthened beams as a substratum to the CFRP sheets [18]. This epoxy mortar is completely cured within a period of 24 hours after application. The compressive, bending and tensile strengths of this mortar are 80, 20 and 6.5 N/mm² respectively.

The beams were prepared for bonding after a period of three weeks from casting. The surfaces to be strengthened were roughened using a grinding technique. Moreover, in case of strengthened beams provided with U-jacket sheets, the corners where the U-jacket sheets were applied had been rounded in a curved shape of about 50-mm-diameter. Before the application of epoxy mortar layer, the roughened surfaces were cleaned by brushing and compressed air to remove any attached fine materials.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Table 2 includes a summary of the observed behavior for the tested models at cracking and maximum conditions. For instance, load at which the flexural crack initiated ($P_{cr,flex}$), load corresponding to the commencement of a significant inclined crack ($P_{cr,sh}$), and the increase in the service load (R_{ser}) over the counterpart in the control beams are listed in Table 2. The service load P_{ser} ($=P_{cr,sh}$) is expressed by the

load at which the significant inclined crack initiated. Moreover, Table 2 contains other results concerning the ultimate condition such as: maximum load (P_{max}); increase in the maximum load over that of the control beam (R_{max}); ductility index based on the service load (μ_D); mid-span strain in the bottom fiber (at tension reinforcement's level) corresponding to the maximum load ($\epsilon_{s,max}$); maximum strain induced in the bonded CFRP strips corresponding to the maximum load ($\epsilon_{f,max}$); mean strain in the bonded CFRP strips corresponding to the maximum load ($\epsilon_{f,mean}$); and failure mode.

Table 2: Cracking and maximum load conditions of the tested beams

Beam No.	Cracking Load Condition			Maximum Load Condition					Ductility μ_D (Δ_1/Δ_2)	Failure Mode
	$P_{cr,flex}$ (kN)	$P_{cr,sh} = P_{ser}$ (kN)	R_{ser} (%)	P_{max} (kN)	R_{max} (%)	$\epsilon_{s,max}$ (‰)	$\epsilon_{f,max}$ (‰)	$\epsilon_{f,mean}$ (‰)		
CS.0	80	90	---	160	---	2.27	---	---	2.16	Mech.1
CS.1	90	100	11.1	190	18.8	2.81	2.81	2.65	2.45	Mech.2
CS.2	90	110	22.2	201	25.6	3.31	4.85	4.21	2.86	Mech.3
DS.0	80	110	---	215	---	3.01	---	---	2.94	Mech.1
DS.1	80	120	9.1	254	18.1	3.58	2.34	2.27	3.58	Mech.2
DS.2	90	130	18.2	278	29.3	3.95	6.60	4.15	4.06	Mech.3

Note: Δ_1 & Δ_2 = deflections corresponding to peak and service loads respectively, Mech.1, Mech.1, and Mech.1 = the first, second, and third failure Mechanism respectively, see the following section.

Failure Behavior and Strength

For each group of the tested beams, both the control and strengthened beams showed, at first, a linear elastic behavior. Afterward, commencement of cracks was only within the shear-span (shear crack) of the control beam *CS.0*; however in case of the other tested beams (*DS.0*, *CS.1*, *CS.2*, *DS.1*, and *DS.2*) the first crack initiated as a flexural one within the central region of the constant moment. Thereafter, a non-linear stage was recorded with the development of numerous flexural cracks along the middle region of a constant moment. Also, as the applied load increased, a number of flexural shear and shear cracks was developed along the shear spans of the examined models. As a matter of fact, at the same loading levels, the number of shear cracks was fewer in case of the strengthened beams in comparison to the corresponding control one, see Figs. 5 and 6.

Three failure mechanisms were recognized for the tested beams. The first one was the traditional brittle shear failure in case of the control samples (*CS.0* & *DS.0*). The second mechanism was noticed in the samples *CS.1* & *DS.1* where shear failure was accompanied with a partial cover delamination of the outer and middle CFRP sheets. The third mechanism was a shear one accompanied with cut off the outer CFRP sheet

as well as crushing of concrete in the shear crack path's region. Such a failure mechanism was observed in U-jacketed beams (*CS.2* & *DS.2*), see Figs. 5 and 6.

All the strengthened beams *CS.1*, *CS.2*, *DS.1* and *DS.2* exhibited the first flexural crack approximately at the same load level and shared the same flexural stiffness as their corresponding control beams *CS.0* & *DS.0*. Results showed that the behavior of the strengthened beams were quite comparable to that of the corresponding control beam up to a load level approximately corresponding to the shear cracking load level, see Figs. 7 and 8. In other words, the flexural stiffness was not influenced significantly by the applied strengthening technique, particularly at lower loading levels. This could be attributed to the fact that the applied strengthening system, either U-jackets sheets or that bonded to the sides only, was not only have a small flexural stiffness but was applied not continuously along the beam length as well. As a result, the flexural stiffness in terms of the mid-span deflection was not affected reasonably. But, at higher loading levels, since the applied strengthening technique decreased the number of cracks and constrained their propagations, the strengthened beams showed a slight enhancement in the flexural stiffness, particularly in case of beams provided with U-jackets CFRP sheets, see Figs. 7 and 8.

It is interesting to mention that the strengthened beams with U-jacket showed a slight ductile behavior before failure. As a consequence, the strain induced in tension bars was approaching considerably to the yielding value, especially in case of beams provided with internal stirrups (*DS.2*), see Fig. 8.

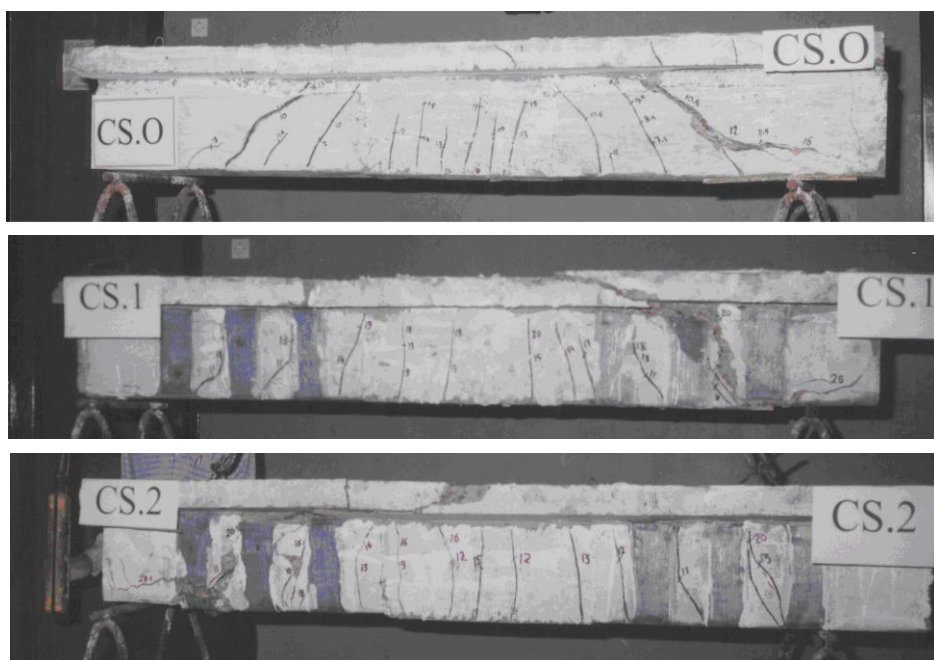


Fig. 5: Cracks pattern of beams *CS.0*, *CS.1* & *CS.2*.

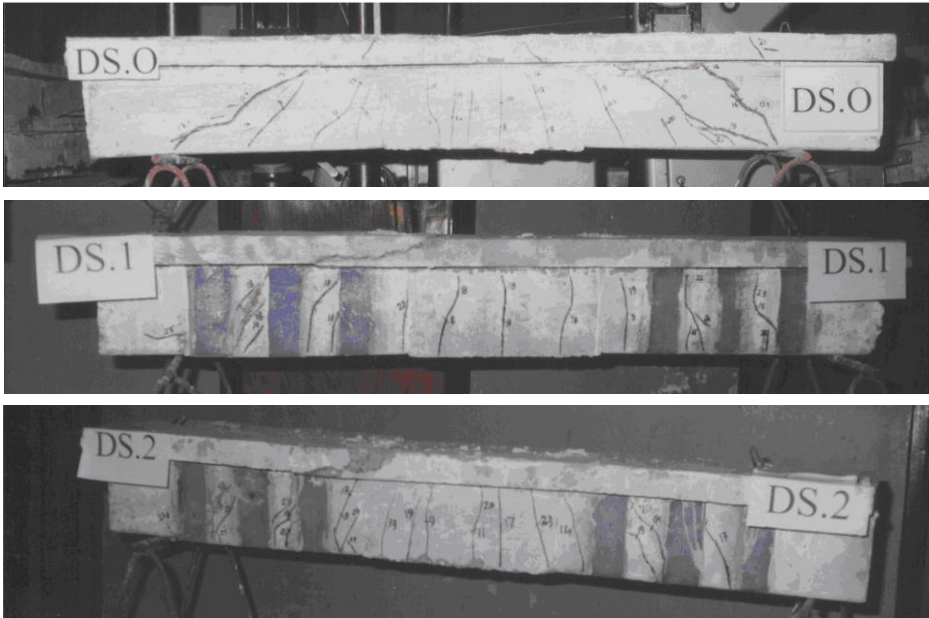


Fig. 6: Cracks pattern of beams *DS.0*, *DS.1* & *DS.2*.

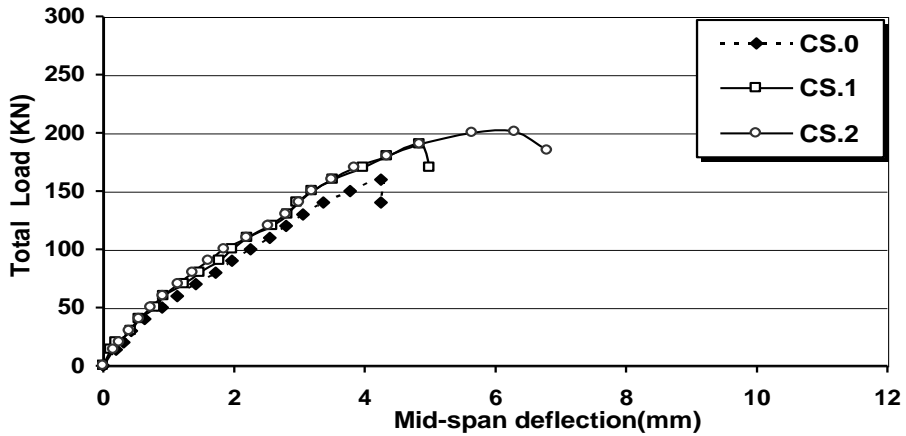


Fig. 7: Load mid-span deflection for beams: *CS.0*, *CS.1* & *CS.2*.

With respect to service load condition, the strengthened beams showed an improvement in the service load P_{ser} values. For instance, the service loads of the beams *CS.1*, *CS.2*, *DS.1* and *DS.2* are higher than that of the corresponding control samples by 11.1, 22.2, 9.1 and 18.2 %, respectively, see Table 2. On the other hand, when considering the maximum load condition, the strengthened beams achieved maximum load (P_{max}) higher than the corresponding control one by 18.8, 25.6, 18.1 and 29.3 % in case of beams *CS.1*, *CS.2*, *DS.1* and *DS.2* respectively, see Table 2 and Fig. 9.

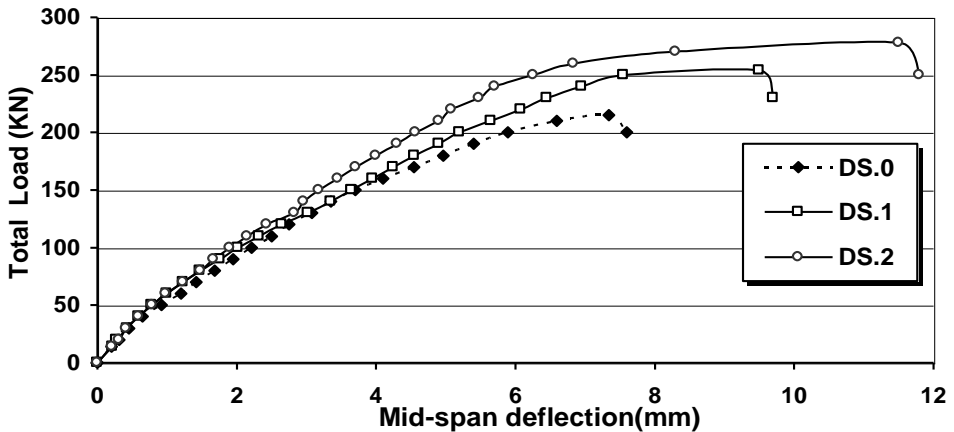


Fig. 8: Load mid-span deflection for beams: *DS.0*, *DS.1* & *DS.2*

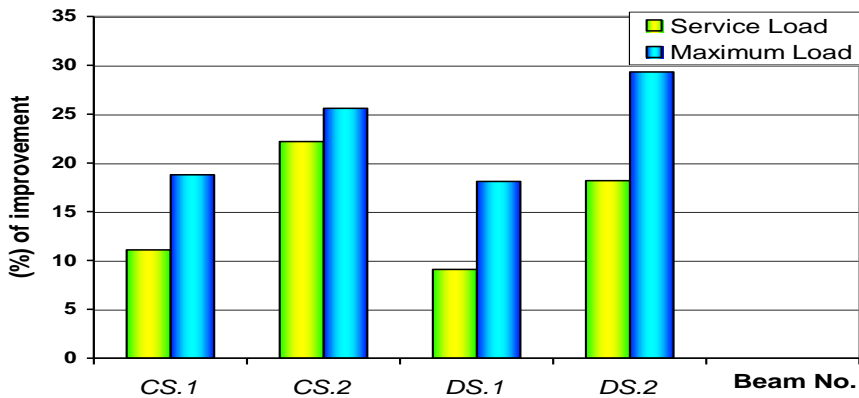


Fig. 9: Increase in both service and maximum loads for strengthened beams.

Strains in CFRP Sheets and Strengthening Efficiency

By means of strain gauges fixed on CFRP sheets at the positions of the shear crack occurred in case of the corresponding control one, the values of the strains induced in the CFRP sheets, either U-jacket sheets or that bonded to the sides only, were measured for the different CFRP strips (strain gauge No.1 (SG.1) for the outer strips, strain gauge No.2 (SG.2) for the neighbored strips), see Figs. 3 and 4. The results of these measurements are shown in Figs. 10 and 11. These figures clearly illustrate the variation of the strain induced in CFRP strips from zero loading up to failure. It is clear that SG.1 recorded the highest strain values, because the main shear crack initiated and propagated close to the location of the gauge; on the other side, lower values were recorded in SG.2, particularly for beams provided with U-jacket sheets "*CS.2* & *DS.2*". It is worthwhile to notice that the *CS.2* & *DS.2* showed higher CFRP strains in comparison to that provided with CFRP sheets bonded to the sides of the samples *CS.1* & *DS.1*. The outer strips in case of beams provided with U-jacket sheets showed a

maximum efficiency where a cut-off in CFRP sheets occurred just before failure; however, for *CS.1* & *DS.1*, a local cover delamination was occurred in the outer strips. This result could be attributed to the anchorage length of the bonded strips, where U-jacket provided a reasonable anchorage length for the bonded strips in comparison with the strips bonded to the beam sides only. It is worth mentioning that, for the models *CS.2* & *DS.2*, the records of SG.1 did not measure the maximum strain values induced in the CFRP sheets because the location of such strain gauges was a way from the main shear crack's path where a cut-off in CFRP sheet occurred.

From Table 2, it is clear that the strengthened beams provided with internal stirrups showed an increase in the maximum load higher than that provided with no internal stirrups. For instance, the maximum load of the strengthened beams *CS.1* & *CS.2*, with no internal stirrups, was higher than that of the reference beam by 30 and 41 kN, respectively. However, 39 and 63 kN were the difference in maximum load between the strengthened beams and provided with internal stirrups and the corresponding control beam, respectively. From the author's point of view, this effect is related to the existing internal stirrups which insure a good performance and contribution for the applied strengthening technique through controlling both the shear crack width and the loss of the aggregate interlock.

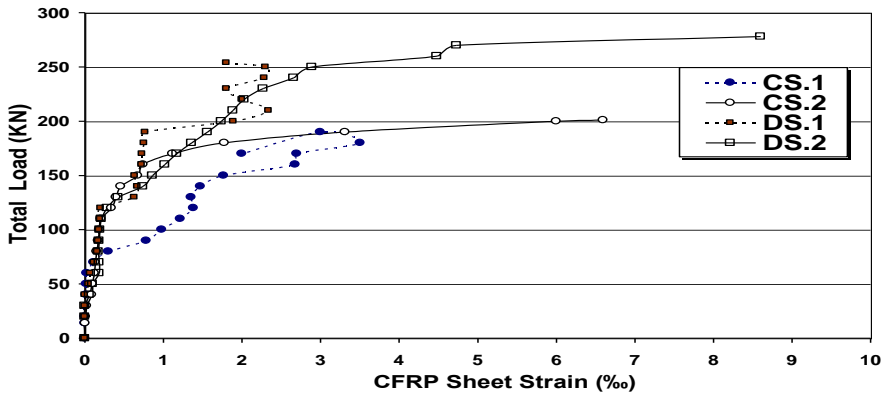


Fig.10: Load versus strain induced in the outer CFRP strips (SG. No.1)

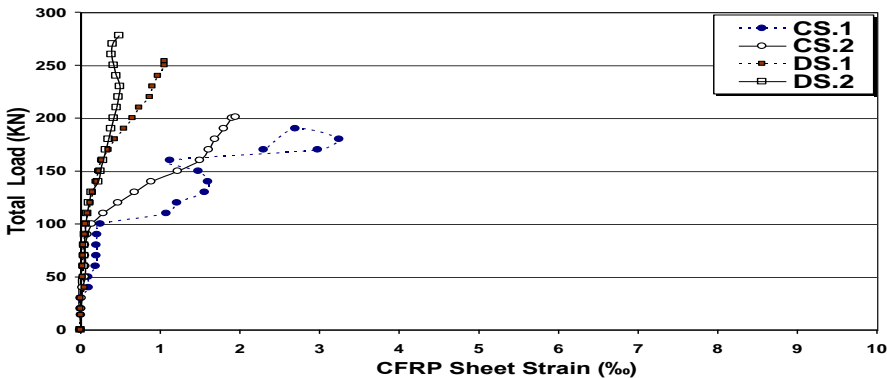


Fig. 11: Load versus strain induced in the middle CFRP strips (SG. No.2)

The real contribution of the bonded CFRP strips to the total load P_f ($= 2V_f$) may be calculated according to Eq. (1), by considering that the effective CFRP strains $\varepsilon_{f,eff}$ equals the measured mean CFRP strain $\varepsilon_{f,mean}$ recorded in Table 2, where $\varepsilon_{f,mean}$ is the mean CFRP strain for all strips located along the main shear crack at failure. The mean strain of the bonded CFRP strips can be estimated from the measured CFRP strains by taking into account the mean value of the maximum recorded strain of the CFRP strips located in the path of the main shear crack "the outer two CFRP sheet strips". It is interesting to mention here that, due to the fact that the anchorage length of CFRP strips bonded to the sides only (CS.1 & DS.1) was not enough, the maximum strains of the CFRP sheets did not reach its maximum value "15.2 mm/m ($= 15.2\%$)" and thus the contribution of the externally bonded CFRP sheets did not fulfill the maximum possible efficiency of the strengthening technique used.

$$P_f = 2V_f = 2 E_f \times \varepsilon_{f,eff} \times A_{f,eff} \quad (1)$$

where $A_{f,eff} = A_f (d_s / s_f) = n \times b_f \times t_f \times (d_s / s_f)$

in which V_f is the contribution of the bonded CFRP strips to the shear strength, d_s is the effective depth of the beam, s_f is the spacing between bonded strips, $A_{f,eff}$, n , E_f and t_f are the effective cross-section area, number of layers, modulus of elasticity and the effective thickness of the CFRP strip, respectively.

The real contribution of the bonded CFRP strips P_f to the total load calculated according to the Eq.(1) were 45, 71, 39 and 70 kN for strengthened beams CS.1, CS.2, DS.1 and DS.2 respectively. However, the corresponding results obtained experimentally were 39, 57, 39 and 63 kN respectively. What was mentioned confirms that Eq. (1) generally overestimates the contribution of the bonded CFRP strips to the shear strength of RC strengthened beams. This is due to the fact that, the concrete reaches its full contribution before the bonded CFRP sheets maintains their effective contribution. As a result, when CFRP sheets reached their effective contribution, the concrete lost a part of its contribution (lost a part of aggregate interlock). And this interprets the local concrete crushing along the path of the main shear cracks: the concrete reached its ultimate strain before the failure occurred. As a result, a local crushing in concrete along the path of main shear crack was observed in case of strengthened beams provided with U-jacket sheets. Consequently, the different codes take that into considerations. Whereas, ACI code [2] limits the effective strain of the CFRP sheets ($\varepsilon_{f,eff}$) with the value 0.004 ($\varepsilon_{f,eff} \leq 0.004$) in order to control shear crack width and loss of aggregate interlock.

Structural Ductility

Ductility of R.C. beams strengthened externally by means of bonded reinforcement could be measured based on structural characteristics [19] such as: mid-span deflection, curvature, or energy absorption capacity as represented by the area under the load-deflection curve. The different codes don't mention any ductility requirements for beams failed mainly due to shear. Although the control beams failed mainly due to shear (traditional shear) don't provide any ductile behaviour, the beams strengthened with U-jacket CFRP sheets showed an acceptable ductile behaviour, especially for beams provided with internal stirrups.

Based on the service load, the ductility index μ_D could be measured as the mid-span deflection at peak load Δ_1 over that corresponding to the service load Δ_2 . Table 2 shows the ductility index μ_D for the different tested beams based on the service load. From Table 2, it is concluded that the strengthened beams provided with U-jacket CFRP sheets showed a reasonable improvement in the measured ductility in comparison with the corresponding control beam, particularly in case of beam provided with internal stirrups.

Calculated Effective CFRP Strain

The shear capacity of the strengthened beam may be calculated in a similar way as the control beam (without strengthening). As it is known that, according to ACI code [1, 2], the shear capacity of the control beam is estimated based on truss analogy. As a consequence, the nominal shear strength of a RC beam strengthened in shear by means of externally bonded CFRP side strips, v_n , can be predicted as a sum of the shear resisting contributions of the concrete (v_c), the steel stirrups (v_s) and the bonded fiber (v_f) according to Eq. (2):

$$v_n = v_c + v_s + v_f \tag{2}$$

where: $v_c = 0.1578\sqrt{f'_c} + \frac{17.24 \rho_s F_u d_s}{M_u}$ (3)

$$v_s = \frac{A_{st} f_{y,st}}{s_s b_w} (\cot \theta + \cot \alpha_s) \sin \alpha_s \tag{4}$$

$$v_f = \frac{A_{wf} E_f \varepsilon_{f,eff}}{s_f b_w} (\cot \theta + \cot \alpha_s) \sin \alpha_f \times \frac{d_f}{d_s} \tag{5}$$

in which A_{st} and A_{wf} are the cross-sectional area of the internal and external web reinforcement respectively, $f_{y,st}$ is the yield strength of internal web reinforcement, f'_c is the concrete compressive strength for cylinder, M_u/F_u represents the shear span (a), d_s & b_w are the effective depth and web width of the beam respectively, s_s & s_f are the spacing between internal and external web reinforcement respectively, E_f & $\varepsilon_{f,eff}$ are the modulus of elasticity and effective strain of the bonded fibre strips, α_s & α_f are inclination of major diagonal shear crack, steel stirrups and bonded fibers strips respectively, d_f is the depth of the FRP sheet [= the effective depth d_s incase of rectangular section and = $(d_s - t_s)$ in case of T-section] and t_s is the thickness of the slab.

As it is mentioned before, the shear strength contribution of the externally bonded web FRP reinforcement (v_f) depends mainly on its effective strain $\varepsilon_{f,eff}$, see Eq. (5). As a matter of fact, there are several factors affect the induced effective strain of the bonded web FRP reinforcement, such as: the configuration of the formed main shear crack along its length, the bond strength between the FRP strips and the concrete surface, and the anchorage length. The last two parameters, i.e. the bond strength and the anchorage length, represent the anchoring force of the bonded web FRP reinforcement. Also, the local debonding of the bonded FRP web reinforcement at both sides of the main shear crack plays a significant role on the induced effective strain

$\varepsilon_{f,eff}$. Bonded U-wraps or bonded face plies – FRP systems that don't enclose the entire section (two- and three- sided wraps) have been observed to delaminate from the concrete before the loss of aggregate interlock of the section. For this reason, bond stresses should be analyzed to determine the usefulness of these systems and the effective strain level that can be achieved [20]. An upper limit of the effective strain $\varepsilon_{f,eff}$ is recommended ($\varepsilon_{f,eff} = 0.004$) in order to control shear crack width and loss of aggregate interlock [22]. Based on the extensive and specific research data by Triantafillou [20] and Khalifa [21], ACI code [2] proposed a more deterministic approach to predict the effective strain $\varepsilon_{f,eff}$ as a function of the bond-reduction coefficient k_v and the ultimate strain of the bonded fibre strips $\varepsilon_{f,u}$, see Eq. (6). As a consequence, the effective strain of the bonded fibre strips is calculated using a bond-reduction coefficient k_v applicable to shear as follows:-

$$\varepsilon_{f,eff} = k_v \times \varepsilon_{f,u} \leq 0.004 \quad (6)$$

in which $\varepsilon_{f,u}$ is the ultimate strain of the bonded fibre strips.

The bond-reduction coefficient k_v is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the bonded fibre sheet. The bond-reduction coefficient k_v can be computed from Eqs.(7) through (10) [21]. The active bond length L_e obtained according to Eq.(8) is defined as the length over which the majority of the bond stresses is maintained. The bond-reduction coefficient k_v also depends on two modification factors, k_1 and k_2 , that account for the concrete strength and the type of wrapping scheme used, respectively, see Eqs. (9) and (10). The obtained expressions were derived based on the fact that the contributions of the bonded web FRP reinforcement equal to the difference between the maximum load of the strengthened beam and that of the control beam, and assuming diagonal shear crack with $\theta = 45^\circ$.

$$k_v = \frac{k_1 k_2 L_e}{11900 \varepsilon_{f,u}} \leq 0.75 \quad (7)$$

$$L_e = \frac{23300}{(n t_f E_f)^{0.58}} \quad (8)$$

$$k_1 = \left(\frac{f'_c}{27} \right)^{\frac{2}{3}} \quad (9)$$

$$k_2 = \frac{d_f - L_e}{d_f} \rightarrow \dots (\text{for U - Jacket sheets}) \quad (10-a)$$

$$k_2 = \frac{d_f - 2L_e}{d_f} \rightarrow (\text{for sheets bonded to sides only}) \quad (10-b)$$

in which n is the number of layers the bonded fibre strips, t_f is the effective thickness of the bonded fibre sheet, and $\varepsilon_{f,u}$ is ultimate strain of the bonded fibre strips.

The effective strain of the CFRP bonded sheets required to estimate the shear strength of the strengthened beams was calculated according to the proposed ACI model [2], see Eq. (6). The predicted results showed a considerable approach to the

obtained experimental results "mean strain of CFRP sheets $\varepsilon_{f,mean}$ ", see Fig. 12. As a consequence, the predicted results approved the applicability of the proposed expressions suggested to predict the effective strain of the bonded CFRP sheets.

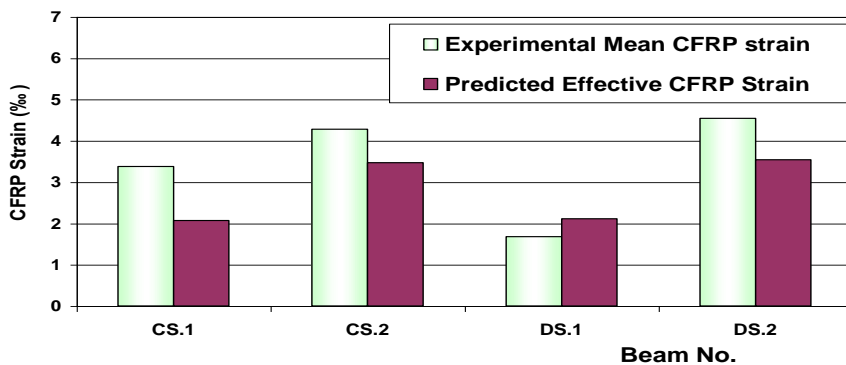


Fig. 12: Predicted effective CFRP strain and experimental mean CFRP strain

CONCLUSIONS AND RECOMMENDATIONS

From the analysis of both the experimental and the analytical results, the following conclusions and recommendations can be stated:

- * The strengthening technique applied to enhance the shear strength of R.C. beams showed an acceptable efficiency, especially in case of beams strengthened by means of U-jacket CFRP sheets.
- * Although the failure mode for the different beams was a brittle one, the strengthened beams provided with U-jacket CFRP sheets showed more or less a ductile behavior at a higher loading level up to a load level just before failure. As a consequence, these beams approved an acceptable enhancement in the structural ductility.
- * Three mechanisms of failure were recognized for the tested beams. The first one was the traditional brittle shear and occurred in the control beams. The second mechanism was a shear one accompanied with a partial delamination of the outer CFRP sheets and occurred in case of the strengthened beams with sheets bonded to their sides only. The third mechanism was a shear one accompanied with cut off the outer CFRP sheet as well as crushing of concrete in the shear crack path's region. Such a failure mechanism was recorded in case of the U-jacketed beams.
- * The suggested model by ACI code to predict the effective strain of the bonded side CFRP sheets, which is necessary to estimate the shear strength of strengthened beams, showed a considerable approach to the obtained experimental results.

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تقوية الكمرات الخرسانية المسلحة ذات المقطع على شكل حرف T والمقواة في القص باستخدام شرائح ألياف الكربون البوليمرية

في هذا البحث تم عمل دراسة معملية لسلوك الكمرات الخرسانية المسلحة ذات القطاع المستعرض على شكل حرف T والمقواة باستخدام رقائق ألياف الكربون البوليمرية CFRP sheets المصقوفة على السطح الجانبي فقط أو المصقوفة على شكل حرف U على السطح الجانبي و القاع و ذلك تحت تأثير الأحمال المستعرضة. هذه التقنية المستخدمة لتحسين كل من مقاومة القص و مطولية تلك الكمرات. كما تم دراسة المؤثر *Effective strain* المتولد في الألياف الكربونية و الضروري لحساب مساهمة رقائق ألياف الكربون البوليمرية في مقاومة القص للكمرة المقواة و بالتالي حساب مقاومة القص للكمرة المقواة. النتائج المعملية أظهرت كفاءة التقنية المستخدمة لتقوية الكمرات الخرسانية المسلحة في القص و خاصة تلك المقواة باستخدام رقائق ألياف الكربون على شكل حرف U حيث أظهرت النتائج المعملية تحسن واضح لمقاومة القص (بكل من الحمل الأقصى وحمل الخدمة *Service Load*) للكمرات المقواة بالمقارنة بالكمرات المرجعية *Control beams* و خاصة بتلك المقواة باستخدام شرائح ألياف الكربون على شكل حرف U. و أظهرت النتائج المعملية أيضا تحسن نسبي لمطولية للكمرات المقواة بالمقارنة بالكمرات المرجعية *Control beams*. أظهرت النتائج أن هناك ثلاث نماذج للإنهيار *failure mechanisms* بالكمرات تم تمييزها معمليا. الأول وهو نموذج الإنهيار القصي التقليدي *traditional brittle shear* (القص القاصف المفاجئ *Sudden brittle shear*) وحدث ذلك بالكمرات المقواة باستخدام رقائق ألياف الكربون البوليمرية المصقوفة على السطح الجانبي فقط. أما الثالث فهو نموذج الإنهيار القصي المصاحب له تمزق وقطع *cut off* لشرائح ألياف الكربون الخارجية (المجاورة للركائز) و متزامن مع تهشيم للخرسانة على طول مسار الشرخ المائل الرئيسي وحدث ذلك بالكمرات المقواة باستخدام رقائق ألياف الكربون البوليمرية المصقوفة على شكل حرف U. تم مقارنة النتائج الخاصة بالإنفعال الفعال المؤثر *Effective strain* المتولد في الألياف الكربونية التي تم حسابها باستخدام الموديل الرياضي للكواد الأمريكي ومقارنتها بالنتائج التي تم الحصول عليها معمليا وقد أثبتت المقارنة تقارب جيد بين النتائج المعملية والنتائج التي تم حسابها باستخدام الموديل الرياضي للكواد الأمريكي.