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Retrofitting of RC beams predamaged in shear using CFRP sheets

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This paper presents results from an experimental study on the behavior of reinforced concrete beams retrofitted or strengthened using Carbon Fiber Reinforced Polymers (CFRP) sheets. A total of eighteen reinforced concrete beams were tested. Tested beams were divided into two main groups. The first group included fourteen beams having a shear span to depth ratio (a/d) equals to 2.57. Two of these beams were tested to failure in their original condition without any strengthening, thus considered as control beams for this group. The other twelve beams were loaded first until shear cracks appeared. Three different levels of shear cracking were considered. Such preloaded beams were then retrofitted using four different schemes of CFRP sheets. Following that the retrofitted beams were loaded to failure. The second group included four beams having a/d ratio equals to 1.71. Such beams were strengthened first using the same four schemes of CFRP sheets and then they were loaded to failure. Test results showed the efficiency of applying CFRP sheets to reinforced concrete beams after being cracked in shear. Applying such sheets to beams resulted in a significant enhancement in the shear force capacity even if the beams were previously cracked by preloading to a load level near the failure load. The failure mode of retrofitted beams varied depending on the scheme of CFRP sheets applied. Test results also revealed that for strengthened beams having small values of shear span to depth ratio, the beams failed in a diagonal compression shear failure mode. Finally, test results of the contribution of the CFRP to the total shear capacity of beams were compared to theoretical results obtained using models found in the literature.

هذا البحث يعرض نتائج دراسة معملية عن سلوك الكمرات الخرسانية المسلحة المعيبة في القص و ذلك بتقويتهـا باسـتخدام لفات من ألياف الكربون. وقد تم إختبار عدد ثمانية عشر كمرة. تم تقسيم الكمرات المختبرة إلى مجمـوعتين رئيـسيتين. تضمنت المجموعة الأولى أربعة عشر كمرة بنسبة بحر القص إلى العمق ٢,٥٧. وقد تم اختبار أثنين من هذه الكمـرات فـي حالتهم الأصلية بدون تقوية حتى الإنهيار ليكونا بذلك كمرات مرجعية لهذه المجموعة. أما الإثنا عشر كمرة الأخرى فقـد تـم تحميلهم أولاً حتى ظهور شروخ القص لثلاث مستويات مختلفه. ثم تم بعد ذلك إصلاح تلك الكمرات بإستخدام أربعة أشـكال مختلفة من لفات ألياف الكربون. أخيرا تم تحميل الكمرات المقواة مرة أخرى حتى الإنهيار. وقد تضمنت المجموعـة الثانيـة أربعة كمرات بنسبة بحر القص إلى العمق الفعال ١,٧١. هذه الكمرات تم تقويتها أولاً بإستخدام نفس الأربعة أشكال من لفات ألياف الكربون ثم تم تحميلهم حتى الإنهيار. وقد أوضحت النتائج المعملية كفاءة إستخدام لفات ألياف الكربـون فـى إصـلاح الكمرات الخرسانية المسلحة السابق تشريخها فى القص. وقد أدى إستخدام تلك اللفات إلى تحسناً كبيراً فى قوة القـص التـى تتحملها الكمرات حتى لو كانت الكمرة قد سبق تحميلها لحمل يقارب من حمل الإنهيار. وقد إختلفت طبيعة إنهيـار الكمـرات تبعاً لشكل لفات ألياف الكربون الملصقة على الكمرات. وقد أوضحت أيضاً النتائج المعملية أن الكمرات المقـواة ذات نـسبة صغيرة لبحر القص إلى العمق الفعال تنهار بسبب مركبة القص فى إتجاة الضغط القطرى. أخيراً تم مقارنة النتائج المعمليـة لمساهمة ألياف الكربون فى قوة القص التى تتحملها الكمرة إلى النتائج النظرية لإستخدام بعض النماذج الموجودة فى الأبحاث السابقة.

Keywords*:* Fiber reinforced polymer, Reinforced concrete beams, Retrofitting, Shear strength, Strengthening

1. Introduction

Generally, reinforced concrete beams fail in either flexural or shear failure mode. In the case of flexural failure mode, the beam gives enough warning in the form of cracks and large deflection. However, brittle shear failure mode takes place in the case of beams having little amount of shear reinforcement. For this reason, codes of practice recommend that reinforced concrete beams should have enough shear reinforcement in order to ensure the occurrence of ductile flexural failure rather than a brittle shear failure.

Reinforced concrete beams may be deficient in shear due to many reasons such as: (i) design mistakes; (ii) improper detailing; (iii) construction faults; (iv) the application of larger loads due to the change in the function of the building; (v) the loss of web reinforcement due to corrosion [1]. Practically, repairing or strengthening such beams by adding internal shear reinforcement is very difficult. It was found that such strengthening may be easily achieved externally by bonding either steel plates or fiber reinforced polymers (FRP) to the beam surface using suitable epoxies.

Experimental investigations found in the literature [2] indicated a basic difference in the mode of failure for externally strengthened beams than that in the case of beams having internal stirrups. In the case of beams reinforced with internal stirrups, the shape and position of those stirrups placed inside the concrete ensure sufficient anchorage, thus failure is controlled by the tensile strength of stirrups. However, in contrast, in the case of externally strengthened beams, the failure is always controlled by the loss of anchorage in the form of debonding of strengthening materials.

Different materials were used through previous experimental studies for the external strengthening and retrofitting of RC beams deficient in shear. These materials were bonded to the external surface of the beam using suitable epoxies. These studies included the application of either traditional steel plates [2,3] or fiber composites [4,5]. Different types of fiber composites were used such as Glass fiber [1,2] and Carbon fiber [4-7]. The fibers were oriented in one direction or hybrid [5] and it may be in the form of fabric [4,5] or laminate [6,7]. Also, through these previous studies the method of external shear strengthening was applied either to beams having internal stirrups [4,5] or without any stirrups [3]. Externally strengthened beams were either uncracked [2,4,6,7] or precracked [1,3,5]. Furthermore, in some of previous studies, shear strengthening was applied in combination with flexural strengthening [1,4]. Generally, previous studies revealed that the method of external strengthening of beams could significantly enhance their shear strength. However, the degree of such enhancement is greatly influenced by the configuration of strengthening schemes.

Most of the available experimental studies found in the literature have concentrated on strengthening of uncracked beams. Little work was directed towards retrofitting precracked beams. Al-Sulaimani et al. [1] used fiber glass plates of 3 mm thickness for the external retrofitting of RC beams having internal stirrups and preloaded to the first shear crack. They applied different strengthening configurations such as: strips, wings, and U-jackets. They concluded that the application of strips or wings gives almost the same degree of enhancement. However, better anchorage was obtained in the case of U-jacket. Such good anchorage prevents the premature failure. Similar investigation was presented by Sharif et al. [3]. However, in this case beams were without internal stirrups and steel plates were applied instead of glass fibers. Another study was conducted by Norris et al. [5]. They tested six beams having internal stirrups, preloaded up to the appearance of the first shear crack. The beams were then retrofitted using CFRP sheets wrapped along the entire beam. The results of the study revealed that CFRP fibers placed perpendicular to the cracks resulted in a significant increase in the stiffness and strength of the beam and a brittle failure took place. However, when CFRP fibers were obliquely placed with respect to cracks, less increase in the stiffness and strength was obtained and mode of failure was more ductile.

Several analytical models were found in the literature considering the contribution of external strengthening to the total shear force capacity of a beam. One of the models found was proposed by Malek et al. [8]. The model was based on the anisotropic (orthotropic) behavior of the composite plate or fabric in order to calculate the shear force resisted by such plate before and after the formation of flexural cracks. Most of the analytical models found considered the effect of the external bonding material in analogy with internal stirrups [1,3,9-12]. The model proposed by [1,3], which considered the application of steel plates or fiberglass plates, was based on assuming an average shear stress value between bonding plates and concrete surface. It was found that this model did not consider the effect of the plate thickness. However in other models [9-12] CFRP fabric was considered and the contribution of the fibers to the total shear force carried by the beam was calculated based on the effective tensile strain or stress of these fibers. Such tensile strain or stress was estimated using empirical formulas developed statistically using test results from experimental studies. Fracture and debonding of fibers were found to limit the effective tensile strength of the fibers [12].

The literature survey presented herein revealed that little investigations have dealt with the external strengthening of RC beams predamaged (precracked) in shear. The previous work did not include the effect of the level or degree of shear cracking. Moreover, the effect of external shear strengthening was not covered in the case of beams having a shear span to depth ratio less than 2.

In this paper, an experimental study was conducted on reinforced concrete beams precracked in shear and then retrofitted using CFRP sheets. In retrofitting such beams, four different schemes of CFRP sheets were applied and then were compared. Three levels of shear cracking were reached for different beams before retrofitting was applied. Also, the behaviour of RC beams having a shear span to depth ratio less than 2 and strengthened with the same CFRP schemes was discussed. Finally, the contribution of CFRP external strengthening to the total shear force capacity of the beam was evaluated experimentally and was compared to results from available theoretical models found in the literature.

2. Experimental study

Eighteen simply supported reinforced concrete beams were tested in the current experimental program. All tested beams had a rectangular cross section of 120 mm width and 200 mm height. All tested beams had a bottom reinforcement of 3-16 mm diameter high tensile steel and a top reinforcement of 2-10 mm diameter high tensile steel. The percentage of the bottom tensile reinforcement was 2.87%. The yield stress and the ultimate strength of the steel reinforcement used were 400 MPa and 610 MPa, respectively for diameter of 16 mm and were 380 MPa and 600 MPa, respectively for diameter 10 mm.

Tested beams were divided into two main groups. In the first group (I), fourteen beams were tested. All beams in group (I) had a span length of 1500 mm, and were tested to failure using two symmetric concentrated loads 600 mm apart. The length of the shear span was equal to 450 mm and therefore the shear span to depth ratio (a/d) was 2.57. Two of these beams (C-1) and (C-2) were tested to failure without CFRP strengthening and were considered as control beams in order to study the contribution of both concrete and internal vertical stirrups to the total shear force capacity of the beam. The control beams were provided by vertical stirrups of 8 mm diameter and 50 mm spacing at their right shear span in order to prevent the occurrence of shear failure at this region. However, vertical stirrups of 6 mm diameter and 160 mm spacing were provided between the two concentrated loads. The left shear span was provided with vertical stirrups of 6 mm diameter and 120 mm spacing for control beam (C-1) whereas in the case of the control beam (C-2) such span was left without any vertical stirrups. The dimensions and reinforcement details for the two control beams are shown in fig. 1.

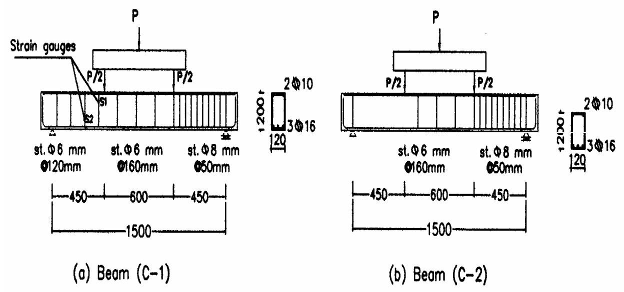

Fig. 1. Details of control beams.

The other twelve beams in group (I) were provided with vertical stirrups with the same arrangement as those provided for the control beam (C-1). These twelve beams were divided into three subgroups, depending on the level of preloading they were subjected to, as shown in table 1. Beams given the symbol (-2) were preloaded to total load of 70 kN which was the load at which the first obvious shear crack appeared within the left shear span. Beams given the symbols (-3) and (-4) were preloaded to a total load of 90 kN and 110 kN, respectively. Such loads represent 72% and 88% of the ultimate shear failure load of the control beam (C-1) respectively. Following preloading of beams the load was then removed and the beams were retrofitted using CFRP sheets. All beams were retrofitted using two layers of CFRP sheets within the precracked left shear span only. The twelve precracked beams were divided to four subgroups depending on the scheme of CFRP sheets applied as shown in table 1. Each subgroup included three beams with different precracking levels. Beams given the symbols (S-) and (US-) were retrofitted with CFRP sheets in the form of Strips and U-Strips, respectively as shown in fig. 2. The Strips or U-Strips used had a width of 50 mm, height of 20 mm and spacing of 100 mm. Beams given the symbols (W-) and (UW-) were retrofitted with CFRP sheets in the form of Wings and U-Wing, respectively as shown in fig. 2. Such Wings and U-Wing had a height of 200 mm and were applied continuous to cover the whole left shear span of the beam. It is to be noted that in all CFRP schemes applied unidirectional CFRP sheets were used and the direction of the fiber was oriented vertically making an angle 90 degrees with the beam horizontal axis. After retrofitting, beams were reloaded to failure.

In the second group (II) four beams, having the same arrangement of vertical stirrups as

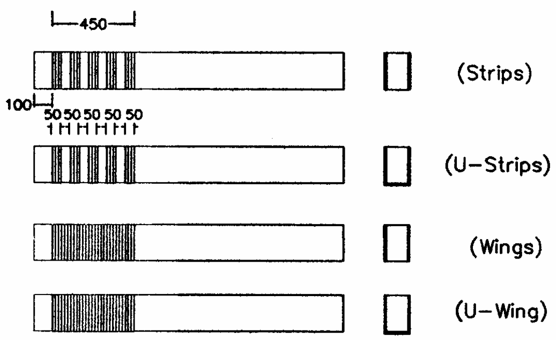

Fig. 2. CFRP schemes used for retrofitting and strengthening.

provided for the control beam (C-1) were considered. The four beams were first strengthened using CFRP sheets with the same different schemes explained for group (I) and then the beams were tested to failure. For the four strengthened beams the two supports were shifted inside 150 mm each, thus the span length was reduced to 1200 mm. However, the distance between the two concentrated loads was kept as that for beams in group (I), 600 mm, therefore the shear span was reduced to be 300 mm and consequently the shear span to depth ratio (a/d) was reduced to 1.71.

Mild steel of diameter 8 mm and 6 mm was used for vertical stirrups. The yield stress and the ultimate strength were 250 MPa and 400 MPa, respectively for diameter of 8 mm and were 280 MPa and 390 MPa, respectively for diameter 6 mm. The concrete mix used for all tested beams was made using ordinary Portland cement, natural sand, and broken stones having a maximum size of 25 mm. The mix proportions were 1.0: 1.6: 2.55, respectively by weight. The water cement ratio w/c was kept in the range of 0.4. The average concrete cube compressive strength was 40 MPa. High strength carbon fiber reinforced polymer sheets (CFRP) were used for retrofitting and strengthening test beams. The sheets were supplied by Sika Egypt under the commercial name (Sikawrap Hex-230C). The thickness of the CFRP sheets was 0.13 mm. The tensile strength and modulus of elasticity of CFRP sheets were 3500 and 230000 MPa, respectively. It should be noted that these mentioned properties of the CFRP sheets were taken from the product data sheet provided by Sika Egypt company. Two-component epoxy adhesive (Sikadur 330), supplied by the same company, was mixed according to the proportions recommended by the manufacturer to bond the CFRP sheets to the target surfaces of the tested beams

The load was applied to tested beams through a hydraulic jack of 500 kN capacity and it was monitored using an electrical load cell. In order to measure the strain in the vertical stirrups within the left shear span of the control beam (C-1), two electrical strain gauges of 6 mm gauge length were installed on the vertical branch of two of the vertical stirrups. The two strain gauges were installed on the first two stirrups left to the concentrated load as shown in fig. 1-a. Also, longitudinal strain in the bottom flexural reinforcement was measured at the mid-span of each beam using electrical strain gauges having 10 mm gauge length. Deflection under the two concentrated load points were measured using mechanical dial gauges with a travel sensitivity of 0.01 mm. Fig. 3 shows the loading set-up for tested beams of the two groups.

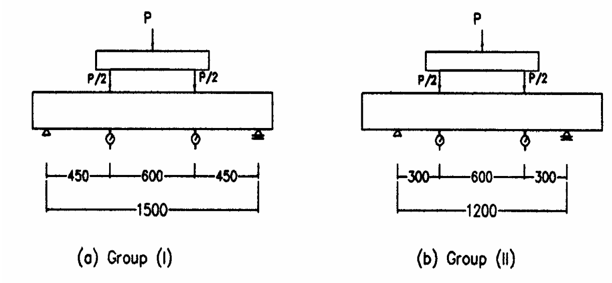

Fig. 3. Loading set-up.

3. Test results and discussions

3.1. Control beams

The two control beams were designed to fail in shear within the left shear span. Such shear span was made without shear reinforcement in the case of the control beam (C-2) and was provided with low percentage of shear reinforcement in the case of the control beam (C-1). The main objective of testing the control beam (C-2) was to evaluate the contribution of the concrete alone to the total shear capacity. Also, the objective of testing the control beam (C-1) was to estimate the contribution of vertical stirrups alone to the total shear force carried by the beam. This was done by subtracting the ultimate shear force carried by beam (C-2) from the ultimate shear force carried by beam (C-1).

For both control beams the first flexural crack formed at the position of the maximum positive moment between the two by concentrated loads at a total load P = 35 kN. As the applied load was further increased, more flexural cracks appeared. In the case of the control beam (C-2) two diagonal cracks, upper and lower (see fig. 4), formed within the left shear span at a total load P = 60 kN. Following that at a total load P = 65 kN these two diagonal cracks intersected at their both ends and suddenly a brittle explosive shear failure occurred. Thus it was concluded from this result that the ultimate shear force carried by the concrete only is 32.5 kN. In the case of the control beam (C-1), the first diagonal shear crack formed at a total load P = 65 kN. As the applied load was increased another lower diagonal shear crack formed. These two upper and lower diagonal shear cracks were more closer to each other than those in the case of beam (C-2), as shown in fig. 4. It was found that the presence of the internal stirrups resulted in a significant enhancement in the beam ductility in comparison to the case of beam (C-2). As the applied load was increased more flexural cracks formed. Besides, both the upper and lower diagonal shear cracks propagated towards the compression side. Also, the upper diagonal shear crack propagated towards the tension side till it reached the level of the bottom tensile reinforcement and extended parallel to it towards the support. In the same time, more diagonal shear cracks formed between the two upper and lower diagonal shear cracks. At a total load P = 125 kN the beam suddenly failed in a shear failure mode along the upper diagonal shear crack. The failure was more ductile than that in the case of beam (C-2). Comparing the result of the ultimate shear force carried by beam (C-1) to that of beam (C-2), it was concluded that the contribution of the internal stirrups to the total shear capacity is 30 kN. Fig. 4 shows the cracking patterns of both control beams (C-1) and (C-2) after failure.

3.2. Retrofitted precracked beams group (I)

As explained earlier tested beams in group (I) were first loaded to three levels of shear cracking corresponding to three load levels of 70 kN, 90 kN and 110 kN. These three load levels were chosen based on the results of testing the control beam (C-1) to represent three different stages of precracking.

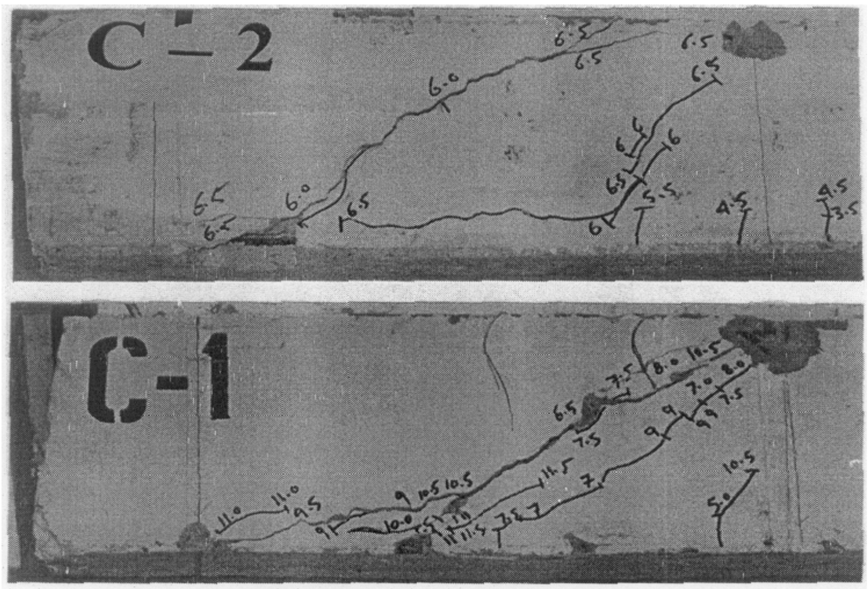

Fig. 4. Cracking patterns for control beams after failure.

The first load level represented the load at which the first obvious shear crack formed in the left shear span. The second load level represented the load at which the strain in the vertical stirrups almost reached the yield strain. Also, the third load level represented the case of post-yielding of the vertical stirrups. After the beams were loaded the load was then removed. Following that the cracked beams were retrofitted using four different schemes of CFRP sheets as shown in fig. 2 and explained in table 1. The beams were then reloaded to failure.

The test results of retrofitted beams in terms of the ultimate failure load P, the ultimate shear force and the corresponding modes of failure are shown in table 2. The table also shows the effectiveness of the CFRP sheets in increasing the ultimate shear force capacity of the beams. It is obvious from the results presented in the table that the application of CFRP sheets is an efficient method for retrofitting reinforced concrete beams precracked in shear. It can be also observed from the table that CFRP sheets in the form of strips were less efficient than all other retrofitting schemes. Both U-Strips and Wings retrofitting schemes had almost the same efficiency in retrofitting beams. However, it seems that Wings retrofitting scheme was more efficient since beam (W-2) and (W-3) developed their full flexural strength whereas the corresponding beams (US-2) and (US-3) failed in a shear mode. The U-Wing retrofitting scheme is the best among the four schemes applied. Cracked beams retrofitted with U-Wing scheme developed their full flexural capacity. This is due to the large retrofitting area and enough anchorage length obtained in the case of using this scheme.

Examining the results presented in table 2 revealed that the effect of preloading level significantly affects the efficiency of retrofitting in the case of using Strips scheme. As preloading level increased the efficiency of retrofitting decreased. However, in the case of U-Strips and Wings schemes such reduction in the efficiency of retrofitting decreased significantly. Moreover, in the case of U-Wing scheme no reduction was observed in the efficiency of retrofitting as the preloading level increased. These observations can be explained by the following facts: (i) the increase in the preloading level decreases the contribution of concrete to the total shear capacity; (ii) when the strain in the vertical stirrups exceeds the yield strain it causes an excessive strain in the concrete. This results in a loss in the aggregate interlocking which leads to a dramatic decrease in the contribution of the concrete to the total shear force; (iii) when the CFRP sheets used for retrofitting the beam are installed on a large area of concrete and have enough anchorage length, such as U-Wing scheme, it can successfully arrest the cracks and the U-shape confines the tensile reinforcement. This results in an increase in the dowel action. Consequently, the beam can restore higher shear force even if it was precracked to any stage.

Table 1
Details of tested beams

Group	Beam	Preloading level (kN)	Retrofitting scheme
(I) (a/d = 2.57)	C-1	0.0	---
	C-2	0.0	---
	S-2	70.0	Strips
	S-3	90.0	Strips
	S-4	110.0	Strips
	US-2	70.0	U-Strips
	US-3	90.0	U-Strips
	US-4	110.0	U-Strips
	W-2	70.0	Wings
	W-3	90.0	Wings
	W-4	110.0	Wings
	UW-2	70.0	U-Wing
	UW-3	90.0	U-Wing
	UW-4	110.0	U-Wing
(II) (a/d =1.71)	S-1	0.0	Strips
	US-1	0.0	U-Strips
	W-1	0.0	Wings
	UW-1	0.0	U-Wing

Table 2
Test results

Group	Beam	Failure load, P_u (kN)	Ultimate shear force, V_u (kN)	Mode of failure	CFRP effectiveness (%)
(I)	C-1	125	62.5	Shear	--
	C-2	65	32.5	Shear	--
	S-2	165	82.5	Debonding shear	32
	S-3	157.5	78.75	Debonding shear	26
	S-4	152.5	76.25	Debonding shear	22
	US-2	175	87.5	Debonding shear	40
	US-3	172.5	86.25	Debonding shear	38
	US-4	170.0	85	Debonding shear	36
	W-2	180	90	Flexure	***
	W-3	175	87.5	Debonding shear- Flexure	40
	W-4	172.5	86.25	Debonding shear	38
	UW-2	180	90	Flexure	***
	UW-3	177.5	88.75	Flexure	***
	UW-4	177.5	88.75	Flexure	***
(II)	S-1	200	100	Debonding shear	25
	US-1	230	115	Diagonal compression	43.75
	W-1	230	115	Diagonal compression	43.75
	UW-1	230	115	Diagonal compression	43.75

*** Efficiency of CFRP is not calculated since the failure is a pure flexural one.

From the results presented in table 2, it can be observed that three different modes of failure took place within tested beams of group (I). For beams retrofitted using CFRP sheets in the form of Strips or U-Strips, the failure started when the diagonal cracks, previously formed during preloading, propagated between and underneath the strips. Following that sounds were heard due to concrete shifting underneath the strips. Finally, the failure suddenly occurred by debonding of strips at their ends showing a debonding shear failure mode. It should be noted that as the level of preloading increases the failure becomes less explosive. Beams retrofitted using CFRP sheets in the form of Wings showed different modes of failure depending on the preloading level. In the case of beam (W-2) the failure mode was flexural whereas a debonding shear failure mode took place in the case of (W-4). However, the failure was hybrid for beam (W-3). Such failure started by debonding of CFRP sheets near their ends. Following that the tensile reinforcement yielded and crushing of concrete was observed at the beam compression side. Much more ductile failure was observed for beams retrofitted using CFRP in the form of U-Wing. Beams failed in this case in a pure flexural mode of failure. Fig. 5 shows modes of failure for some of the retrofitted beams.

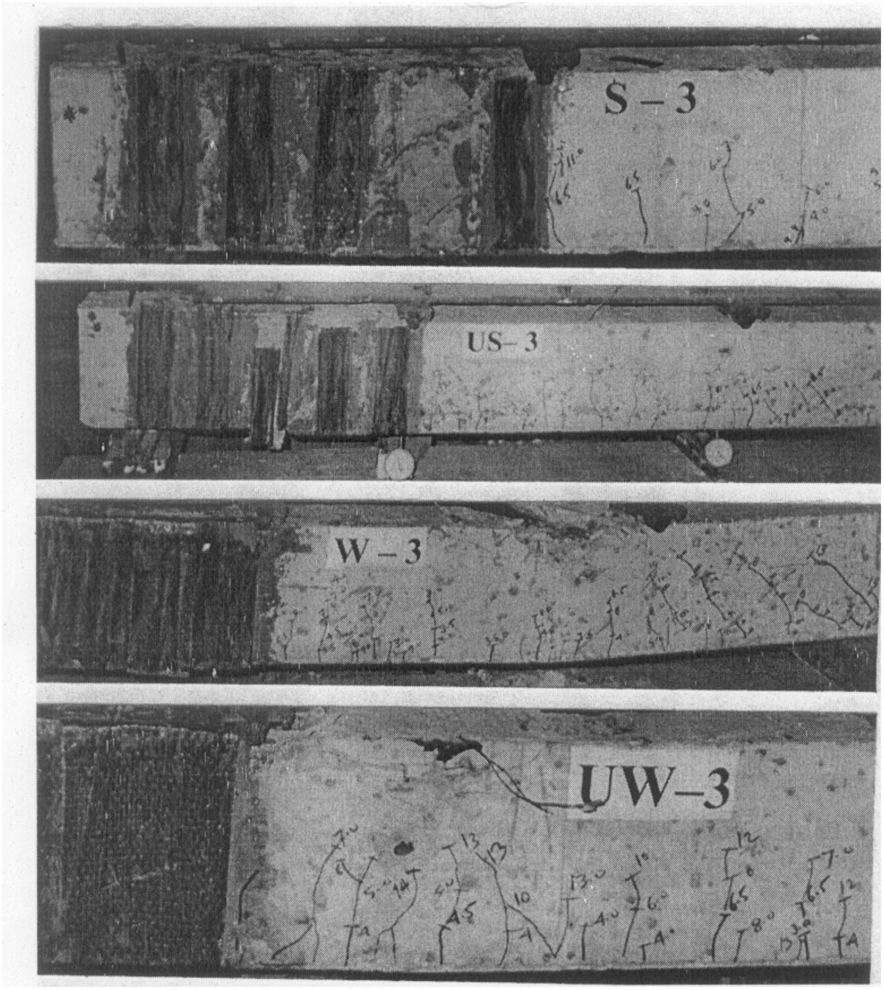

Fig. 5. Cracking patterns for some of the retrofitted beams group (I).

3.3. Strengthened beams group (II)

Test results of strengthened beams of group (II) are shown in table 2. No control beams were tested for this group. Therefore, an approximate value for the ultimate shear force carried by the concrete only was estimated using test results found in previous experimental investigations [13,14]. It was found that the ultimate shear force carried by concrete in this case (a/d = 1.71) is approximately 1.55 times that in the case of a/d = 2.57, control beam (C-2). The contribution of vertical stirrups to the total shear force was considered the same as that in the case of group (I). Based on that it was found that the total value of the ultimate shear force capacity of a control beam for group (II) can be assumed equal to about 80 kN. The effectiveness of using CFRP sheets in enhancing the shear strength of tested beams of group (II) was evaluated using such estimated result for the control beam and is presented in table 2. It is observed from the table that strengthening test beams using CFRP sheets in the form of Strips resulted in a 25% increase in the ultimate shear force of the beam. However, all other forms of CFRP strengthening (U-Strips, Wings, U-Wing) resulted in about 44% enhancement in such shear force. It can be also observed from the table that the failure of beam strengthened with strips of CFRP, beam (S-1), occurred by debonding of the CFRP sheets. However, in all other forms of strengthening, failure of beams occurred by concrete crushing due to the diagonal compression component of shear forces. Fig. 6 shows mode of failure of strengthened beams (S-1) and (UW-1).

3.4. Deflections

For all tested beams deflection was measured at the positions of concentrated loads, thus load-deflection relationships were developed. It was observed that generally the use of CFRP sheets for retrofitting the beams resulted in a significant improvement in their stiffness, although they were previously loaded to different cracking levels. Fig. 7 shows such relationships for retrofitted beams (S-3, US-3, W-3 and UW-3) in comparison to that for the control beam (C-1). This improvement in the beam stiffness resulted in a corresponding significant reduction in the beam deflection over the whole range of loading up to failure. However, such reduction in the beam deflection was affected by the scheme of CFRP used for retrofitting. The use of U-Wing scheme was the most efficient one in controlling beam deflection. Less reduction in beam deflection was observed in the case of all other retrofitting schemes.

Fig. 6. Cracking patterns for strengthened beams (S-1) and (UW-1).

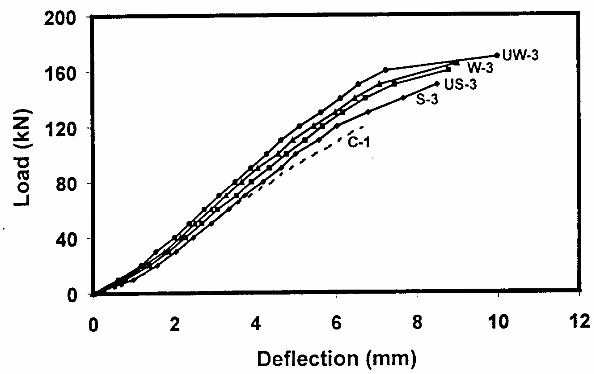

Fig. 7. Effect of CFRP retrofitting schemes on load-deflection relationships.

Fig. 8 shows the effect of preloading level on the load-deflection relationships of beams retrofitted using the Wing scheme. In this case preloading level has a marginal effect on beam deflection since large area of concrete was covered with CFRP sheets. Moreover, results not shown herein for brevity revealed that the effect of prelaoding level on the beam deflection in the case of using U-Wing scheme was negligible. The load-deflection curves were almost the same. This is because CFRP sheets covered large area of concrete and enough anchorage length was provided by the U-shape. However, the effect of preloading level was more significant in the case of other retrofitting schemes.

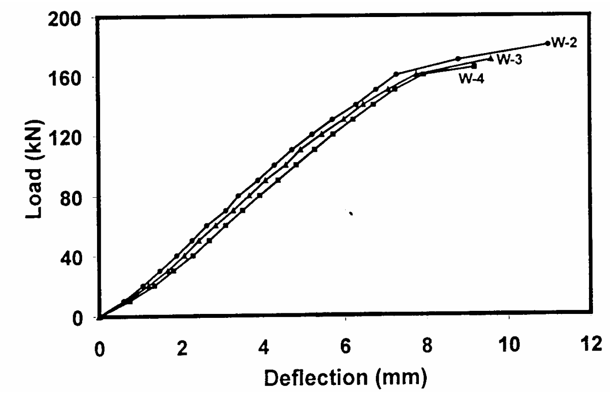

Fig. 8. Effect of precracking level on load deflection relationship for beams retrofitted using wings scheme.

Fig. 9 shows load-deflection relationships for strengthened beams of group (II). It is clear from the figure that the use of U-Wing scheme was the most efficient scheme in controlling beam deflection. Also, using U-Strips or Wings schemes had almost the same effect in reducing beam deflection.

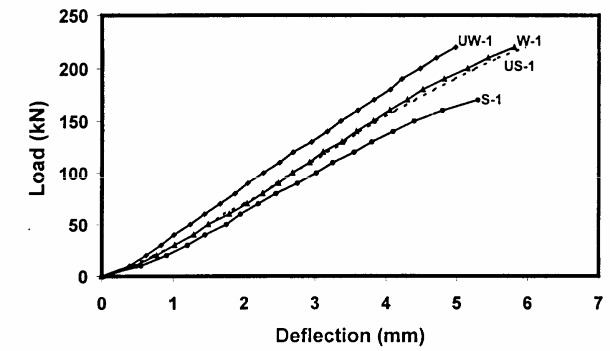

Fig. 9. Effect of CFRP strengthening schemes on load-deflection relationships.

3.5. Strains

Fig. 10 presents load-strain relationships for the control beam (C-1). Such strain was measured in the first two vertical stirrups within the left shear span left to the concentrated load, as shown in fig. 1-a. It can be seen from the figure that there is almost no contribution from the vertical stirrups to the shear strength of the beam until the first shear crack was formed at a total load of P =65 kN. However, significant contribution can be observed in the post cracking loading stage up to failure. Also, the strain in the stirrup (S2) was much greater than that for stirrup (S1) in the post cracking stage, since the stirrup (S2) was located to intersect the main diagonal shear crack.

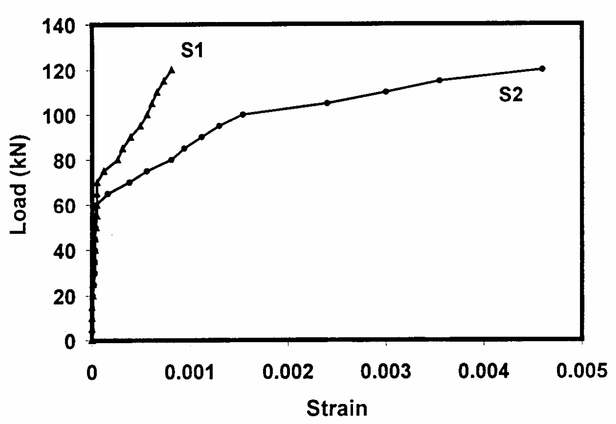

Fig. 10. Load-strain relationships for the vertical stirrups of the control beam (C-1).

Fig. 11 shows the relationships between the total applied load (P) and the strain in the main bottom flexural reinforcement at the position of maximum moment. Strains were measured during loading of retrofitted cracked beams group (I). The modes of failure of beams are reflected in these relationships. Beams failed in flexural mode showed a yield plateau. However, other beams failed in shear mode by debonding showed nearly linear strain behavior up to failure. For some of the beams, the load strain relationship was slightly affected by the cracks formed during preloading. In the case of strengthened beams group (II), the load strain relationships were almost linear up to failure as shown in fig. 12. This is because of the fact that the shear span to depth ratio was in this case (a/d=1.71) which resulted in a dominant shear failure rather than a flexural one.

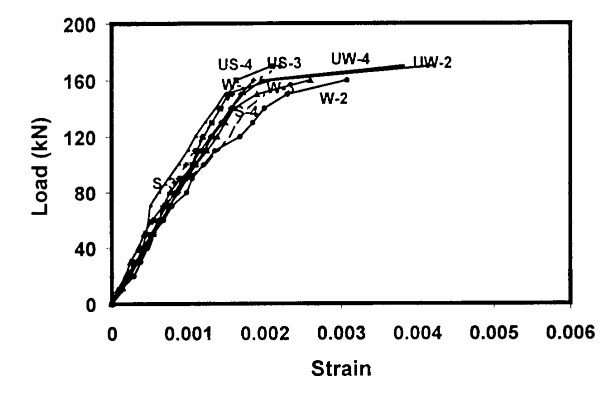

Fig. 11. Load-strain relationships for the tensile reinforcement of retrofitted beams.

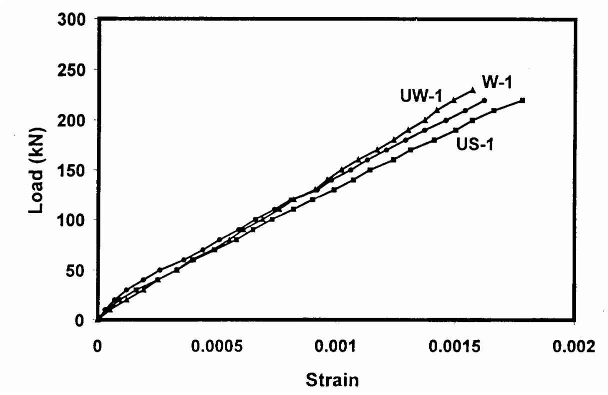

Fig. 12. Load-strain relationships for the tensile reinforcement of strengthened beams.

4. Theoretical analysis

Generally, the shear force capacity of any reinforced concrete beam, taking into account the effect of CFRP strengthening, can be expressed in the form of:

$$V_u=V_c+V_s+V_F\,, \quad (1)$$

where V_c, V_s and V_F are the contributions of concrete, web reinforcement, and CFRP to the total shear force carried by the beam section V_u. The contribution of CFRP may be evaluated theoretically in analogy with steel stirrups in the form of:

$$V_F= A_F f_{Fe}\frac{d_F}{S_F}(\sin\alpha+\cos\alpha)\,, \quad (2)$$

and

$$A_F=2t_F B_F. \quad (3)$$

Where: A_F = area of CFRP; t_F = thickness of CFRP strips at each side; B_F = width of CFRP strips; d_F = effective depth of CFRP; S_F = the spacing between CFRP strips; and, α = angle between beam longitudinal axis and direction of fibers.

When the CFRP sheets applied so that the direction of fibers being in the vertical direction (α = 90°), then eq. (2) may be rewritten in the form of:

- for Strips and U-strips schemes:

$$V_F=2t_F B_F f_{Fe}\frac{d_F}{S_F}\;, \quad (4)$$

- for Wings and U-Wing schemes:

$$V_F=2t_F f_{Fe} d_F\,. \quad (5)$$

Where f_{Fe} is the effective tensile strength of CFRP. It was found that unless CFRP sheets are closely wrapped or their ends are mechanically anchored, CFRP sheets always do not reach their ultimate tensile strength. Fracture of CFRP or debonding of CFRP sheets always takes place before CFRP sheets develop their full ultimate tensile strength. Different analytical models were found in the literature [9-12,15,16]. In some of these models the effective tensile strength of the fibers is estimated by limiting the ultimate tensile strain to a certain value [15,16]. In other models a reduction factor is used to reduce the ultimate tensile strength of the fibers [9-12]. One of these latter models was proposed by Khalifa et al. [12]. Such model was used in the current theoretical analysis. The effective tensile strength of CFRP f_{Fe} was presented by Khalifa et al. [12] in the form of:

$$f_{Fe} = R\, f_{Fu}\,, \quad (6)$$

where : f_{Fu} = the ultimate tensile strength of the CFRP; and, R = reduction factor.

The reduction factor R was proposed by Khalifa et al. [12] based on two conditions. For the first condition, R was proposed considering the case of CFRP fracture failure and was written in the form of:

$$R = 0.5622\ (\rho_F\ E_F)^2\ -\ 1.2188\ (\rho_F\ E_F) + 0.778 \leq\ 0.50, \quad (7)$$

and

$$\rho_F = (2t_F/b_w)\ (B_F/S_F)\ . \quad (8)$$

It should be noted that Khalifa et al. [12] proposed that eq. (7) is applicable only if the value of the term $(\rho_F\ E_F)$ should not exceed 1.1 GPa. Where ρ_F = CFRP ratio; E_F = modulus of elasticity of the fibers (GPa); and b_w = width of the beam.

For the second condition, R was proposed considering the case of CFRP debonding failure and was written in the form of:

$$R = \frac{0.0042(f_c')^{2/3} W_{Fe}}{(E_F t_F)^{0.58} \varepsilon_{Fu} d_F}\ . \quad (9)$$

Where f_c' = concrete cylinder compressive strength of concrete; E_F = modulus of elasticity of the CFRP (GPa); ε_{Fu}= ultimate tensile strain of the CFRP; and W_{Fe}= factor based on the effective bond length of the CFRP and the CFRP scheme. This factor W_{Fe} was expressed by Khalifa et al. [12] in the form of:

- For the case of Strips or Wings schemes:

$$W_{Fe} = d_F - 2L_e\ . \quad (10)$$

- For the case of U-Strips or U-Wing schemes:

$$W_{Fe} = d_F - L_e\ , \quad (11)$$

where, L_e = effective bond length of CFRP expressed as:

$$L_e = e^{6.134 - 0.58\,\ln(t_F E_F)}\ . \quad (12)$$

Therefore, using the least value of R obtained from the two conditions of CFRP failure, the shear force carried by CFRP can be estimated using eqs. (4) and (5) together with eq. (6). However, in order to avoid web crushing another condition was presented based on the ACI 318-95 code [17]. Such condition limits the shear force carried by the CFRP as follows:

$$V_F \leq \left(\frac{2\sqrt{f_c'}\, b_w d}{3} - V_s \right)\ . \quad (13)$$

Applying the above presented equations, the contribution of CFRP to the total shear force capacity was theoretically calculated for tested beams and the theoretical results were compared to the experimental ones. The comparison is presented in table 3. It should be noted that the experimental shear forces carried by the CFRP for precracked retrofitted beams in group (I) were calculated by subtracting the ultimate shear force of the control beam (C-1) from the total ultimate shear force of each beam. Only tested beams (S-2), (US-2), (W-2) and (UW-2) were considered in the comparison since they were precracked only to the first obvious crack in shear. Also, for strengthened beams in group (II), the experimental shear forces carried by the CFRP were calculated by subtracting the predicted value of the ultimate shear force of the assumed control beam (V = 80 kN) from the ultimate experimental shear force of each beam.

It can be observed from the table that for tested beams (S-2) and (US-2), in group (I), good agreement is found between the experimental and theoretical results. However, theoretical results were always found to be greater than the experimental ones. For tested beams (W-2) and (UW-2), the experimental results were much less than the theoretical ones. This is because these two beams failed in a flexural mode, therefore they did not reach their full shear strength. Good correlation was also found between the experimental and theoretical results in the case of tested strengthened beams in group (II). However, it is recommended herein that more reliable theoretical models be developed for the estimation of the contribution of CFRP to the total shear force of beams having low values of shear span to depth ratio.

Table 3
Comparison between experimental and theoretical contribution of the CFRP sheets to the total shear force capacity

Group	Beam	$[V_F]_{exp}$ (kN)	$[V_F]_{the}$ (kN)
(I)	S-2	20	21
	US-2	25	31
	W-2	27.5	42
	UW-2	27.5	49
(II)	S-1	20	21
	US-1	35	31
	W-1	35	42
	UW-1	35	49

5. Conclusions

An experimental study was conducted and presented in this paper in order to investigate the behaviour of CFRP retrofitted or strengthened reinforced concrete beams. Firstly, two control beams, having a shear span to depth ratio a/d = 2.57, were tested to failure in their original condition without any strengthening. Secondly, twelve beams, having a shear span to depth ratio a/d = 2.57, were preloaded to different shear cracking levels. Following that the beams were retrofitted using four different schemes of CFRP. Then the beams were reloaded to failure. Another four beams having shear span to depth ratio a/d =1.71 were first strengthened with the same four different schemes of CFRP and then they were loaded to failure. Finally, the experimental results for the contribution of the CFRP sheets to the total shear force capacity of beams were compared to theoretical results from an analytical model found in the literature. Based on this study the following conclusions can be drawn:

1- The application of CFRP sheets is an efficient method for retrofitting preloaded cracked reinforced concrete beams. The application of such CFRP sheets significantly enhanced the ultimate shear force capacity of the beams and increased their stiffness thus significantly reduced the deflection.

2- The efficiency of CFRP in retrofitting or strengthening reinforced concrete beams deficient in shear depends mainly on the scheme of CFRP applied to the beam. Such efficiency increases as the area of CFRP sheets bonded to the beams increases. Also, retrofitting schemes that provide enough anchorage length are more efficient than other schemes since debonding failure of CFRP is prevented or delayed.

3- The preloading level that the beam is subjected to significantly affects the efficiency of retrofitting in the case of Strip scheme. As the preloading level increases the efficiency of retrofitting decreases.

4- The decrease in the efficiency of CFRP retrofitting with the increase in the preloading level is not significant in the cases of U-Strips, Wings, and U-Wing schemes. This is due to the enough anchorage length (U-Strip) or the large area of CFRP sheets applied to the beam (Wings) or both (U-Wing). The application of large area of CFRP sheets can successfully arrest the cracks and the use of enough anchorage length can confine the tensile reinforcement and thus increase the contribution of the dowel action. Consequently, the beam can restore higher shear strength even if it was precracked to any stage.

5- The failure mode of retrofitted reinforced concrete beams depends mainly on the scheme of CFRP applied. A sudden explosive failure mode always takes place in the case of Strips or U-Strips scheme by debonding of the strips at their ends showing a debonding shear failure mode. Also, as the level of preloading increases the failure becomes less explosive.

6- A more ductile mode of failure takes place in the case of beams retrofitted using CFRP sheets in the form of U-Wing. The failure mode in this case is a pure flexural one.

7- In most of the cases, failure of reinforced concrete beams, having a shear span to depth ratio less than 2, strengthened using CFRP sheets takes place by concrete crushing due to the diagonal compression component of shear forces.

8- The existing models found in the literature for estimating the contribution of CFRP to the ultimate shear force capacity of reinforced concrete beams are reliable. Results from applying these models showed good agreement with the current test results. However, theoretical results are always found to be greater than the experimental ones. Moreover, it is recommended herein that more theoretical models be developed for the estimation of the contribution of CFRP to the total shear capacity of beams having low values of shear span to depth ratio.

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