Shear strengthening of reinforced concrete continuous beams using external steel plates

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A series of five reinforced concrete continuous beams was tested to failure in order to investigate the effectiveness of using externally applied steel plates as a method of enhancing the shear capacity of the beams. The steel plates were bonded to the sides of the beams using a two-component epoxy. The variables studied were the external steel plate thickness and the longitudinal reinforcement ratio. The beams were tested under two concentrated loads and the performance of four beams with external side plates was compared to that of a control beam without side plates. For all beams tested the initiation of cracks and its propagation was observed and recorded. Deflections and steel strains were measured thus load-deflection and load-strain relationships were detected. Finally, failure loads and mode of failure were observed for all beams tested. Results have revealed the significant effect of using external steel plates bonded to the sides of the beams. The presence of such plates greatly affected the mode of failure of the beams and significantly enhanced the ultimate shear strength of the beams. Such enhancement was between 43 % and about 80 %.

يعرض هذا البحث نتائج إختبار مجموعة مكونة من خمس كمرات خرسانية مسلحة مستمرة لدراسة فعالية إستخدام ألواح حديد خارجية كطريقة لتحسين مقاومة الكمرات فى القص. وقد تم تثبيت الألواح الحديدية على جوانب الكمرات بإستخدام مادة إيبوكسية. والعوامل التى تم دراستها كانت سمك الألواح الحديدية الخارجية ونسبة حديد التسليح الطولى. وقد تم إختبار الكمرات تحت تأثير حملين مركزين وتم مقارنة سلوك الكمرات ذات الألواح الخارجية بسلوك كمرة بدون ألواح خارجية. وقد تم ملاحظة بداية ظهور الشروخ وتتبع تطورها. وتم قياس سهم الإنحناء والإنفعال فى الحديد ومن ثم إيجاد العلاقة بين الحمل وكل من سهم الإنحناء والإنفعال. أخيراً تم ملاحظة أحمال الإنهيار وشكل الإنهيار لجميع الكمرات المختبرة. وأظهرت النتائج التأثير الفعال لإستخدام ألواح حديد خارجية مثبتة على جوانب الكمرات. إن وجود هذه الألواح أثر تأثيراً كبيراً على شكل وطبيعة إنهيار الكمرات كما أدى إلى تحسن مقاومة القص القصوى للكمرات تحسناً كبيراً. وقد كان هذا التحسن بين ٤٣ % وحوالى ٨٠ %.

Keywords: Epoxy adhesive, R C continuous beams, Shear strengthening, Steel bonding, Steel plates.

1. Introduction

A substantial amount of work in the field of construction worldwide is devoted to the repair and strengthening of existing structures, especially in earthquake-prone regions where seismic strengthening is very common. In other regions, deterioration, aging and underestimated design loads are the most frequent reasons for strengthening of structures.

Strengthening of reinforced concrete (RC) members to improve their flexural and/or shear strength is usually accomplished by different methods such as: using external prestressing, adding extra reinforcement by stapling and shortcreting (shortcrete jacketing), and adding external epoxy bonded plates to the concrete beams.

Steel plate bonding (SPB) by epoxy adhesive, to restore or to upgrade the beam capacity, is considered as the common technique and can be applied in a wide variety of strengthening problems. SPB technique has become popular because it is inexpensive and unobstructive, it has little effect in the overall dimensions of the structure, and it allows the bonded plates to be easily inspected for degradation. Furthermore, adhesive bonding provides a full interactive mechanical bond between the steel plates and concrete, as compared to partial interactive bond when bolting is used. The disadvantage of SPB technique, which is the possibility of deterioration of bond between the steel plate and the concrete due to prolonged exposure to weather, had led to the technique of externally bonded advanced composite materials; e.g. fiber reinforced polymers (FRP) in the form of thin laminates or fabrics.

The majority of researches and applications carried out to date, using either steel plates or FRP, have been aimed at flexural strengthening of RC beams. The plates were glued to the tension face of the beams (i.e. beams soffits). References [1-4] present an extensive experimental study on the use of SPB for flexural strengthening.

Much research [5-8] has gone into studying the premature failure of beams strengthened at their soffits due to separation between the plate and the concrete at the plate ends before achieving the full flexural strength of the beam. It was found [2] that the plates have the tendency to peel away due to the action of vertical shear stresses, that is, shear peeling. Ohelers et al [8] found that debonding can be prevented by bonding side plates to the region adjacent to the ends of the soffit plates. A design procedure was suggested [8] for plating the tension faces of simple and continuous RC beams.

In the last ten years, shear strengthening of RC beams has attracted an increasing attention from researchers. A reinforced concrete beam can be deficient in shear capacity due to a variety of factors such as: mistakes in design calculations, improper detailing of shear reinforcement, construction faults or poor construction practice, and reduction or total loss of shear reinforcing steel due to corrosion in severe environment conditions. In addition, RC beams must have sufficient shear strength to insure ductile flexural failure. For beams deficient in shear, there is practical difficulty in adding internal shear reinforcement to the beams, especially when the beams are not isolated but are instead part of a floor-beam system.

Little work has been done on shear strengthening of RC beams by SPB technique. Sharif et al [9] tested ten RC simple beams with dimensions 150 x 150 mm, 1200 mm span, and shear span-to-effective depth ratio (a/d) of 3.2. The beams had no internal shear reinforcement. The beams were first loaded until shear cracks were visible, then the load was removed. Following that steel plates with 1.5 mm thickness were bonded to the beams, and the load was applied up to failure. Four different repair schemes were used (strips, U-strips, wings, and Jackets). All repair schemes had improved the shear capacity and restored the stiffness of the damaged beams. For the jacketed beam, substantial increase in shear strength and ductility occurred and flexural type of failure took place. They [9] presented an expression for the ultimate shear resistance V_u as follows:

$$V_u = V_c + V_s + V_p. \tag{1}$$

where, V_c is the concrete shear resistance as proposed in the national design codes, V_s is the shear provided by internal stirrups, if any, and V_p is the shear resistance provided by externally bonded steel plates calculated based on the adhesive failure rather than on yielding of the plate (which is more likely to occur in practical cases). Simple expressions for V_p were given [9] based on maximum shear stress of plate-concrete interface, which was assumed as 3.5 N/mm^2.

Swamy et al [10] tested twelve simply supported RC beams 150 x 250 mm in cross section, 2800 mm span, and a/d ratio of 3. All beams were provided with externally bonded plates (either steel or GFRP) at their tension face (bottom surface), since it was found [5] that such plates can increase the beam shear capacity by up to 16%. In addition, they provided different types of externally bonded side plates including, (i) end anchorage plates at the end of the tension plates to prevent premature debonding of the tension plate and to arrest the initiation and propagation of shear cracks at the support; (ii) U-shaped plates in the shear span, and (iii) confinement plates under the load points and/or in the compression zone to enhance the contribution of the concrete compression zone. Generally, the results indicated that externally bonded plates changed the brittle shear failure into a ductile flexural failure. They [10] concluded that the results were limited to permit the development of rational and sound theoretical basis to predict the contribution to shear strength of the bonded plates.

In spite of the limited research on the use of SPB technique for shear strengthening of RC beams, strengthening with FRP plates was extensively studied both experimentally and analytically [11-15]. Also, strengthening of concrete beams severely cracked in torsion by SPB has been presented as a case study [16].

A literature search showed that there is little available research data that dealt with shear strengthening of RC continuous beams. In continuous beams, the position of the maximum bending moment and maximum shearing force coincide. Therefore, the application of SPB technique for RC continuous beams is severely restricted by the premature debonding of the ends of plates through the action of both shear forces and bending moments [8].

This paper presents the results of an experimental study conducted on RC continuous beams with low shear capacity (without shear reinforcement) and strengthened by epoxy bonding of external side steel plates over the middle shear span. The main parameters taken into account in this study were the ratio of external shear reinforcement in terms of external steel plate thickness and the longitudinal flexural reinforcement ratio.

2. Experimental work

Five reinforced concrete continuous beams having a rectangular cross section of 100 mm width and 200 mm height and had two spans of 1725 mm each were tested. An increasing concentrated load was applied in each span at a distance of 600 mm from the central support (shear span to effective depth ratio, a/d = 3.5). No internal steel shear reinforcement was used in any of the beams except a 6 mm diameter stirrup was only provided at each support to hold the longitudinal reinforcement in position. As a result, the beams were underdesigned in shear compared to their flexural capacity.

To study the effect of the ratio of internal flexural reinforcement on the strength of the strengthened beams, two different reinforcement ratios were used. The reinforcement of three beams consisted of 2-10 mm diameter high tensile steel bars at the bottom and 2-10 mm diameter bars at the top (the percentage of tension steel μ was 0.0093, calculated for beam width b = 100 mm and effective depth d = 170 mm). The other two beams were reinforced with 2-13 mm diameter high tensile steel bars in both the bottom and the top (μ = 0.0155). It should be noted that the percentage of steel reinforcement was between the upper and the lower bounds for an under-reinforced section specified by the Egyptian Code [17] (μ_{min} = 0.00275 and μ_{max} = 0.019). The yield stress of the steel reinforcement of diameter 10 and 13 mm was 338 and 330 N/mm^2 respectively and the ultimate stress was 516 and 550 N/mm^2 respectively. Figure 1 shows dimensions and reinforcement details of tested beams.

The concrete mix used for the tested beams consisted of Ordinary Portland Cement, natural sand, and broken stone with 20 mm maximum nominal size, the mix proportions by weight were 1: 1.5: 2.55 respectively, and the respective w/c ratio was 0.4. Control specimens of 150 mm cubes were cast from each concrete batch.

The first beam was tested in its original condition (i.e. unplated) as a control one. Each of the other four beams was strengthened with two side mild steel plates. The side plates were of 190 mm height and 1250 mm length and of thickness, t_p, 2 mm or 4 mm as shown in Fig. 2. The ratio of the external shear reinforcement (calculated as the thickness of the two side steel plates divided by the beam width) was 4 and 8 % for plates with thickness of 2 and 4 mm respectively. The average value of the yield stress for the steel plates was 240 and 294 N/mm^2 for plates having thickness of 2 mm and 4 mm, respectively, and the average value of the ultimate stress was 331 and 460 N/mm^2, respectively. The loads were applied to the beams by using hydraulic jack of 200 kN capacity with a load cell to monitor the load. Deflections under the load points were recorded by means of dial gauges. Longitudinal strains in flexural top reinforcement in the beams as well as in the external steel plates were measured by means of electrical resistance strain gauges with 10 mm gauge length. The plate gauges were fixed after the bonding operation. Positions of both dial gauges and strain gauges are shown in Figs. 1 and 2. Table 1 presents details of the tested beams.

An epoxy based two-component adhesive mortar (Sikadur-30 product) of 2 mm

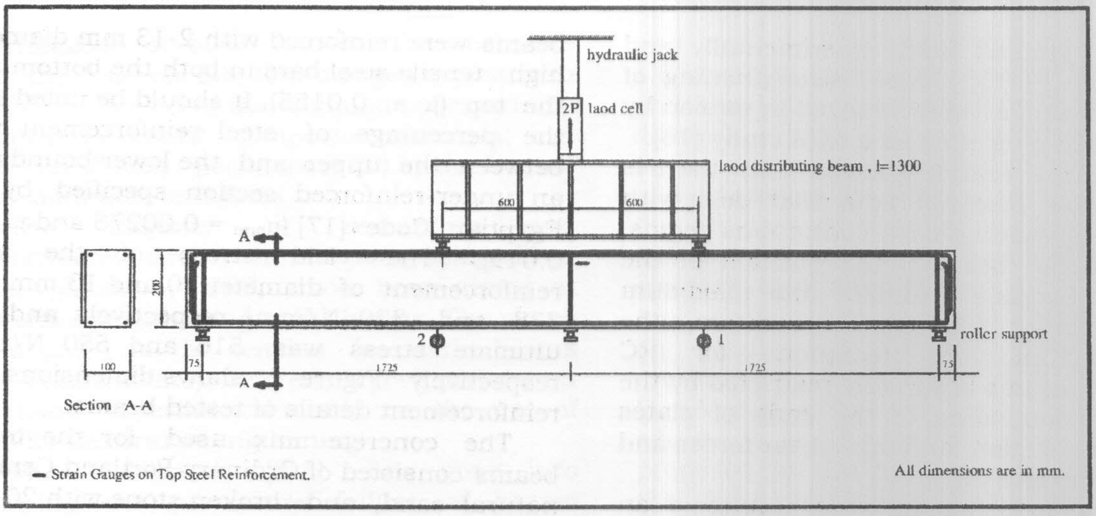


Fig. 1. Dimensions and reinforcement details of tested beams.

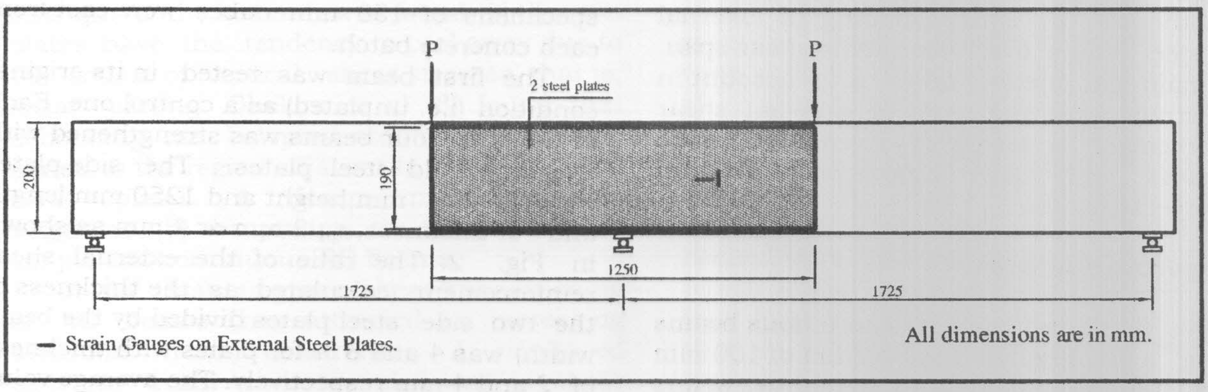


Fig. 2. External plates and location of loads on beams.

Table 1. Details of Tested Beams

Beam	Age of beam (Days)	Cube strength f_{cu} (N/mm^2)	Flexural reinforcement	Thickness of steel plate (mm)
B1	40	44.0	2 ϕ 10 mm	-----
B2	42	42.0	2 ϕ 10 mm	2
B3	55	42.0	2 ϕ 10 mm	4
B4	36	36.3	2 ϕ 13 mm	4
B5	51	36.3	2 ϕ 13 mm	2

f_{cu} = average of three 150 mm cubes from the concrete batches used to cast one beam thickness was applied to all the strengthened beams as a substratum to the steel plates. The concrete was cured for at least 28 days before bonding the plates. Each beam was turned on its side and the two parts epoxy was hand mixed to the specified proportions and hand applied to both the concrete and the steel. After each side plate was positioned, it was held down with steel weights during curing. This epoxy mortar is completely cured within a period of 50 hours after application. The compressive strength, tensile strength, shear strength and young's modulus of this epoxy mortar is 100, 25, 15, and 12800 N/mm^2, respectively.

3. Test results

The results of tested beams included the initiation of cracks and its propagation, the deformations in terms of deflection, steel strains, and final mode of failure. The experimental values of load at first crack P_{cr}, and the corresponding value of deflection δ_{cr}, ultimate load P_u, and maximum recorded deflection δ_u, are given in Table 2.

3.1 General behavior of tested beams, modes of failure, and ultimate loads

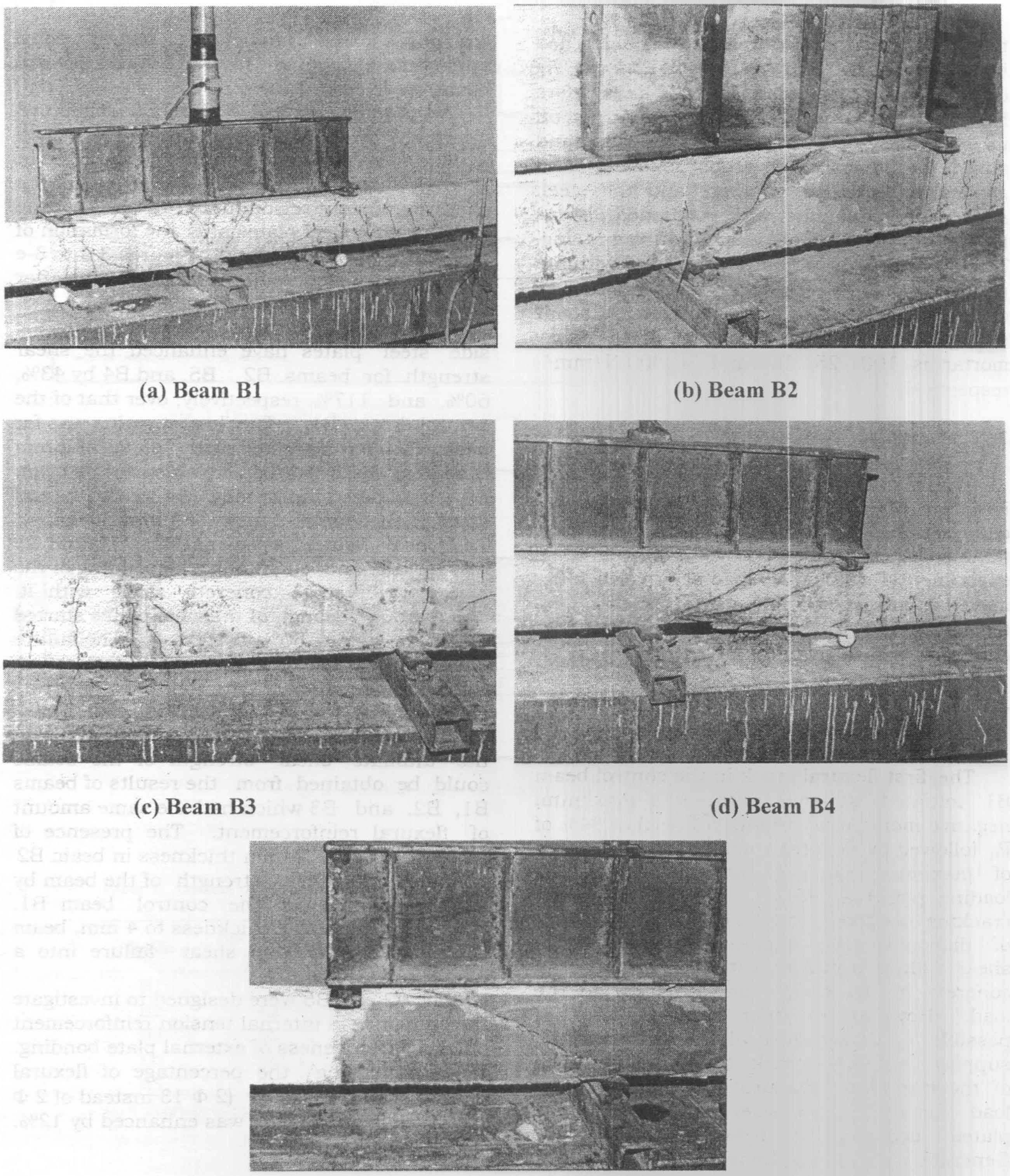
The first flexural crack in the control beam B1 initiated at the position of maximum negative moment (at middle support) at 34% of P_u followed by flexural cracking at the position of maximum positive moment (under the loading point) at 46% of P_u. Diagonal shear cracking occurred at 80% of P_u and the width of diagonal cracks increased until diagonal shear failure occurred with the crushing of concrete in the shear span adjacent to the load. However, in other beams it was not possible to detect flexural cracks at middle support and shear cracks due to the presence of the steel plate. Flexural cracking under the load for the beams strengthened with side plates occurred at 16% to 36% of P_u. Generally, the strengthened beams showed a cracking load under the loading point approximately equal to that of the control beam, as given in Table 2.

All beams, except beam B3 reinforced internally by 2 Φ 10 mm and strengthened with a side plate of 4 mm thickness, failed in shear. Beam B3 failed in flexure in the positive moment region (under the load) at the ends of the steel plates after the formation of the diagonal shear cracks. Figures 3-a to 3-e show failure modes for all tested beams after the removal of the side steel plates.

Results of the ultimate loads show that the side steel plates have enhanced the shear strength for beams B2, B5, and B4 by 43%, 60%, and 117%, respectively, over that of the control beam B1. The lowest value was for beam B2, strengthened with plates of 2 mm thickness and the highest value was for beam B4, reinforced internally by 2 Φ13 and strengthened with plates of 4 mm thickness. The shear failure of beams B2, B4, and B5 was brittle and sudden by debonding of the plates with some concrete along with it. Separation (peeling) of the steel plates started at plate ends at 50% to 100% P_u. The failure did not occur at the plate-glue interface but fracture took place in the concrete cover as shown in Fig. 4.

The effect of the steel plate thickness on the ultimate shear strength of the beams could be obtained from the results of beams B1, B2, and B3 which had the same amount of flexural reinforcement. The presence of plates having a 2 mm thickness in beam B2 enhanced the shear strength of the beam by 43% over that of the control beam B1. Increasing the plate thickness to 4 mm, beam B3, transformed the shear failure into a ductile flexural failure.

Beams B2 and B5 were designed to investigate the influence of internal tension reinforcement on the effectiveness of external plate bonding. With increasing the percentage of flexural reinforcement by 67% (2 Φ 13 instead of 2 Φ 10), the shear capacity was enhanced by 12%.



(a) Beam B1

(b) Beam B2

(c) Beam B3

(d) Beam B4

(e) Beam B5

Fig. 3. Failure modes of tested beams after the removal of the steel plates

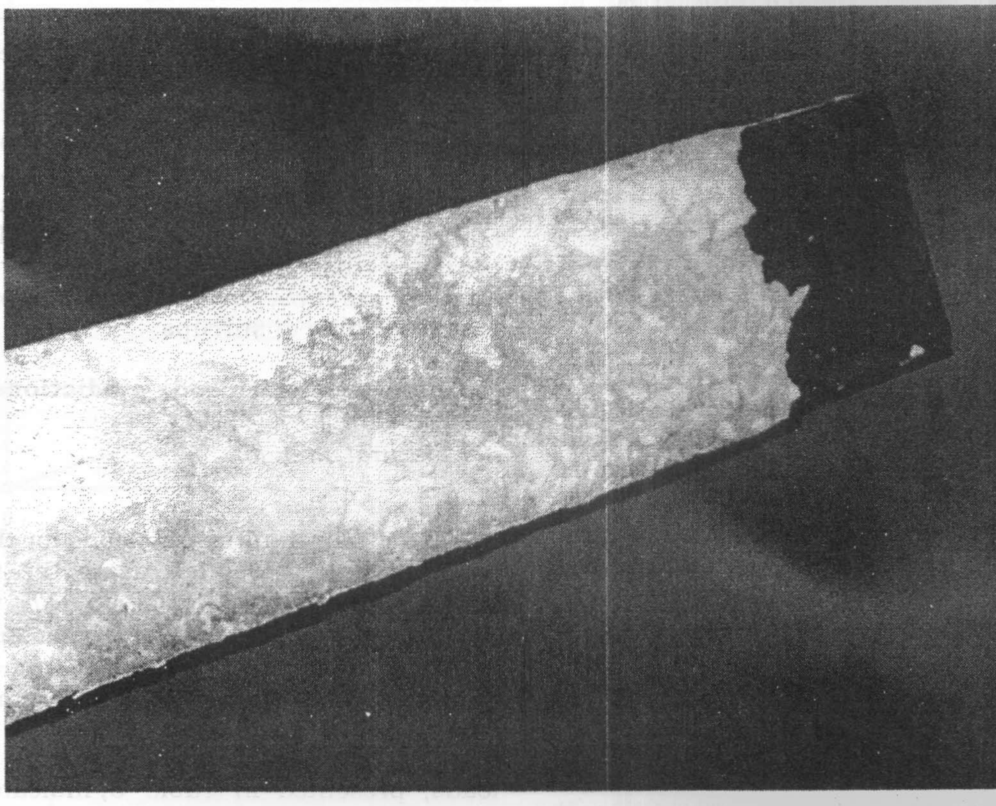


Fig. 4. Steel plate bonded to beam B4 after failure of the beam.

Table 2 Experimental results

Beam	Cracking load P_{cr} (kN)	Failure load P_u (kN)	Deflection at cracking load δ_{cr} (mm)	Deflection at failure load δ_{max} (mm)	Mode of failure
B1	15.00* 20.00*	43.75	1.57	4.82	Shear
B2	20.00* 22.50**	62.50	2.24	8.39	Shear
B3	12.50**	77.50	1.46	12.95	flexure
B4	20.00**	95.00	2.66	9.11	shear
B5	20.00**	70.00	3.58	12.92	shear

* Load at top flexural cracking at the middle support.
** Cracking load within the positive moment region.

The enhancement in the shear strength of the externally strengthened beams can be detected examining Fig. 5. The figure shows the relationship between the ratio of the ultimate load of a tested beam to that of the control beam (in terms of $(P_u/\sqrt{fcu})/(P_u/\sqrt{fcu})_{control}$ and the ratio of external steel plates ($2t_p/b$).

3.2 Deflection of tested beams

Figure 6 shows the relationships between the applied load P and the measured deflection at the point under the load for all the beams, whereas Table 2 presents the values of deflection at P_{cr} and P_u.

It is observed from both Fig. 6 and Table 2 that the deflection under the load at flexural cracking load (δ_{cr}) was approximately equal for all the strengthened beams, except B3. Such deflection values were slightly higher than that for the control beam. The results indicated that the steel plate thickness had no effect on δ_{cr}. However, at failure load the strengthened beams showed very high values of deflection (δ_u). Such deflection values were up to about 2.7 times the deflection of the control beam. At any level of loading (P/P_u) the strengthened beams showed a greater deflection than that of the control beam at the same relative load.

The load-deflection relationship for B4, reinforced internally by 2 Φ 13 and strengthened by plates of 4 mm thickness, see Fig. 6, showed stiffer behavior than that for other beams.

3.3 Strain in flexural steel reinforcement and on side steel plates

Figure 7 displays the relationship between the applied load and measured strain on the top flexural steel reinforcement at the middle support. Yielding of steel in beam B3, which failed in flexure, occurred at 71% of P_u. Yielding of steel in beam B2, reinforced with 2 Φ 10 mm, occurred at 84% of P_u with the full development of the flexural cracking at the middle support along the beam height and across its width. Strain measurements for beam B5, reinforced with 2 Φ 13 mm and strengthened with plates of 2 mm thickness showed yielding of steel at shear failure of the beam with the appearance of fine flexural cracking at the middle support. No yielding of steel was recorded for beam B4, reinforced with 2 Φ 13 mm and strengthened with 4 mm thickness steel plates. In this case severe shear failure have occurred.

In all cases, failure did not occur in close proximity to the strain gauges placed on the external steel plates and therefore gauges did not record reliable data. However, the small values of strain recorded (about 0.00044 for beam B2 strengthened by plates of 2 mm thickness) indicated that the plates had not reached their full tensile strength. It should be noted that in RC beams reinforced with internal stirrups, the shear failure occurs by tensile yielding of stirrups.

4. Comparison with code predictions

4.1 Flexural strength

The flexural ultimate loads for the tested beams were calculated using the formulas recommended by the ECP-95 [17] with the removal of the material safety factors (γ_c and γ_s) and assuming f_{cu} = 40 N/mm^2, as an average value, and f_y = 334 N/mm^2. The calculated values are given in Table 3.

The experimental values for the ultimate loads, presented in Table 3, indicate that all beams, including the control beam, have attained the full calculated capacity in flexure. Beam B3 failed in flexure in the positive moment zone under the load with P_u = 1.37 of the calculated flexural strength. It is evident that the presence of the external steel plates enhanced the flexural strength at the middle support. Such enhancement may be due to the lateral confinement provided to the compression zone by the externally bonded steel plates even without anchorage of the plates. Such anchorage may be provided by bonding plates in the shape of U-section around the beam cross-section. Another explanation of the flexural strength enhancement is that the presence of externally bonded steel plates provided a restraining medium against flexural crack propagation at the middle support.

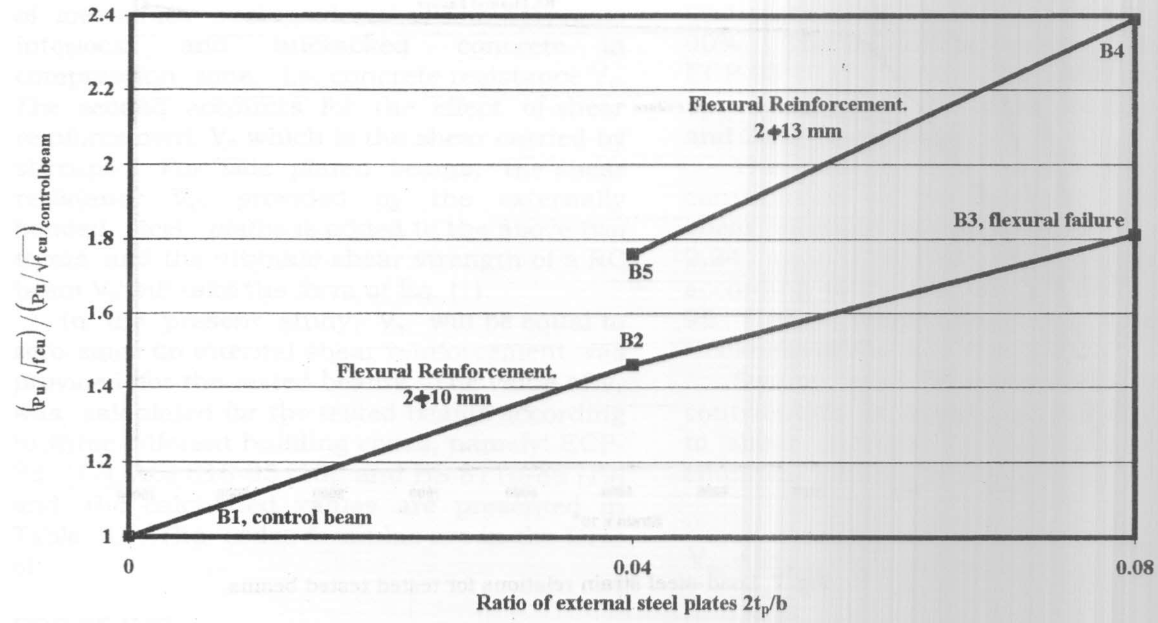


Fig. 5. The Enhancement in shear strength of externally strengthened beams.

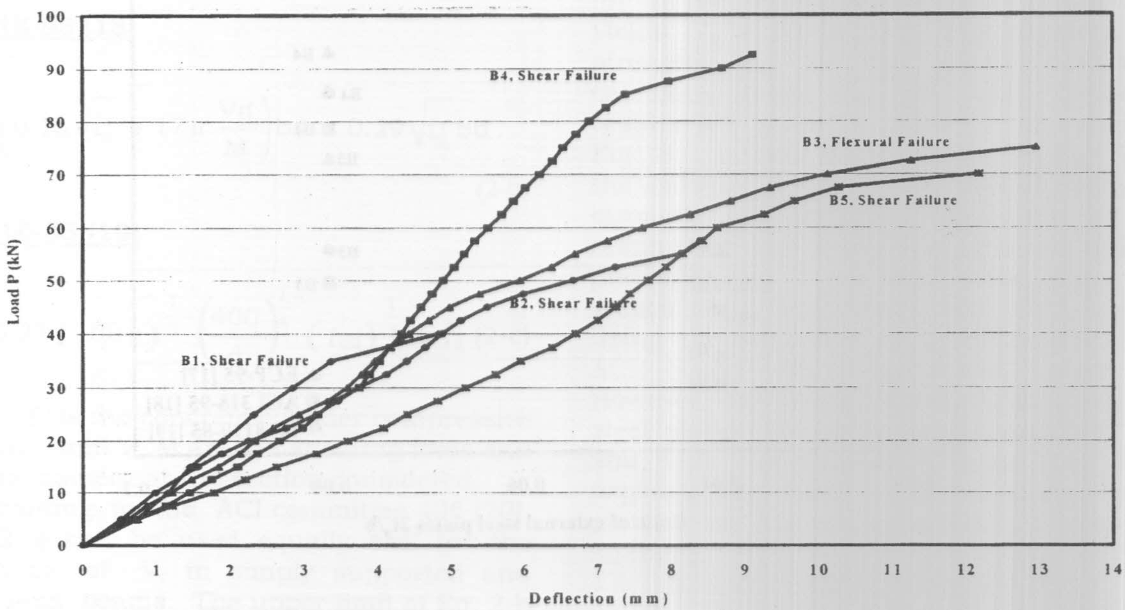


Fig. 6. Load-deflection relationships for tested beams.

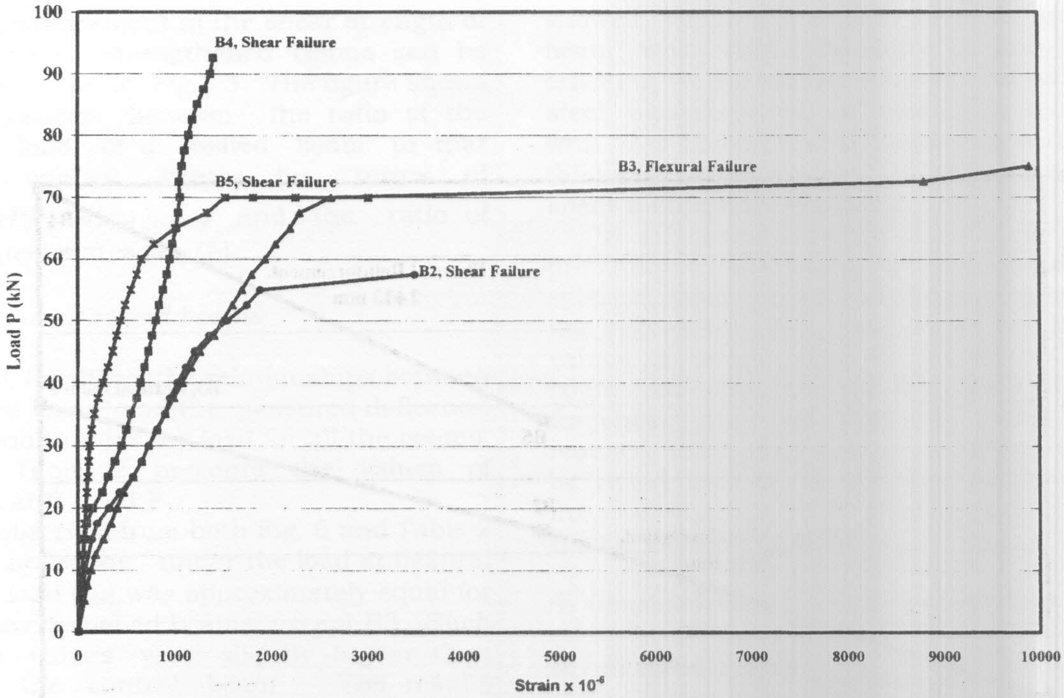


Fig. 7. Load-steel strain relations for tested tested beams.

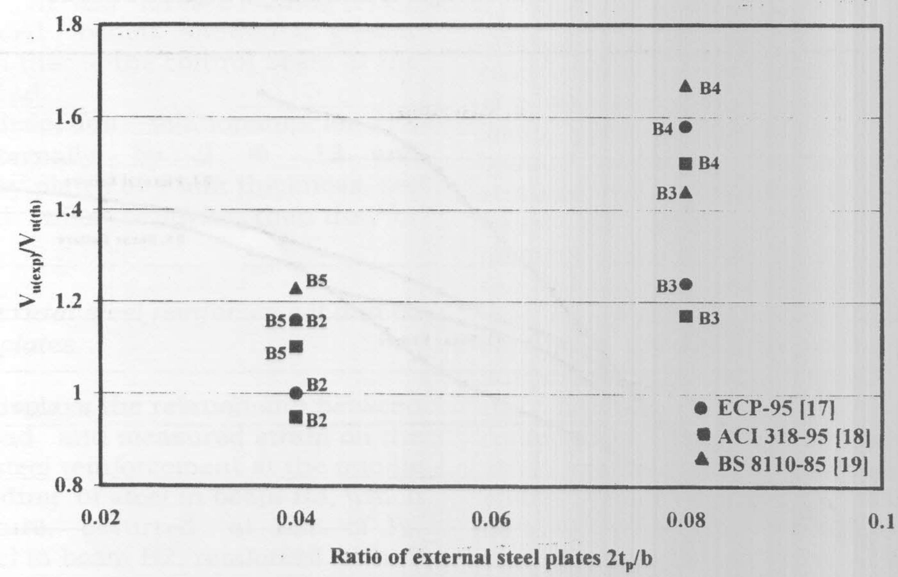


Fig. 8. Comparison of Experimental ultimate loads with code predictions.

4.2 Shear strength

According to design codes, the design of RC beams in shear is typically based on the assumption that the total contribution to shear capacity is given as the sum of two terms. The first term accounts for the action of mechanics such as dowel action, aggregate interlock, and uncracked concrete in compression zone, i.e. concrete resistance V_c. The second accounts for the effect of shear reinforcement V_s which is the shear carried by stirrups. For side plated beams, the shear resistance V_p, provided by the externally bonded steel plates is added to the above two terms and the ultimate shear strength of a RC beam V_u will take the form of Eq. (1).

In the present study, V_s will be equal to zero since no internal shear reinforcement was provided for the tested beams. The value of V_c was calculated for the tested beams according to three different building codes, namely: ECP-95 [17], ACI 318-95 [18], and BS 8110-85 [19] and the calculated values are presented in Table 3. The codes formulas are in the form of:

ECP-95 [17]

$$V_c = 0.24\sqrt{f_{cu}}\,bd \quad \text{(2-a)}$$

ACI 318-95 [18]

$$V_c = \left(0.16\sqrt{f'_c} + 17\mu\frac{Vd}{M}\right)bd \le 0.29\sqrt{f'_c}\,bd\,. \quad \text{(2-b)}$$

BS 8110-85 [19]

$$V_c = 0.27\,(100\mu)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{\frac{1}{4}}(f_{cu})^{\frac{1}{3}}\,bd\,, \quad \text{(2-c)}$$

where; f_c' is the concrete cylinder compressive strength, and V, M are the shearing force and bending moment at the section considered.

According to the ACI committee 426 [20], Eq. (2-b) can be used equally well for the calculation of V_c in simply supported and continuous beams. The upper limit of Eq. 2-b (i.e. $V_c = 0.29\ \sqrt{f'c}$, bd) was used since diagonal tension failure occurred in the tested beams near inflection points as shown in Fig.3.

Comparing the values of V_c as predicted by Eq. (2) with the experimental shear strength of the control beam B1, indicated that the BS 8110-85 [19] approach, Eq. (2-c), greatly underestimated the shear strength by about 90%. The equations presented by both the ECP-95 [17] and the ACI 318-95 [18] underestimated the shear strength by 36% and 22%, respectively.

The results in Table 3 show that the contribution of the side steel plates to the shear strength of the beams, V_p, was 0.98-2.24 and 0.78-1.91 of the values of V_c according to the ECP-95 [17] and the ACI 318-95 [18], respectively, depending on the thickness of the side steel plates.

Swamy et al. [9] attempted to estimate the contribution of externally bonded side plates to shear strength V_p, and they suggested an empirical formula in the form of:

$$V_p = 2\tau_{avg}\left(\frac{dh_w}{2}\right), \quad (3)$$

where: τ_{avg} is the average interface shear stress along the plate height = 0.8 N/mm^2, and h_w is the depth of side plate. Applying Eq. (3) to the tested beams in the present study yields V_p = 25.84 kN. The ultimate shear strength, V_u, for the tested beams were calculated using Eqs. (2) and (3) and the results are given in Table 3 and plotted in Fig. 8. It can be observed from Fig. 8. that the calculated values for the ultimate shear strength are in good agreement with test results for the beams strengthened with steel plates having 2 mm thickness. The average values of $V_u/V_{exp.}$ were 1.08, 1.03, and 1.2, using the values of V_c predicted by the ECP, ACI, and BS approaches, respectively. However, for beams strengthened with 4 mm steel plates, the above values were 1.41, 1.34, and 1.56. It should be noted that Eq. (3) neglects the effect of the thickness of side plates on its contribution to the shear strength.

Table 3. Experimental and theoretical strength results.

Beam number	Experimental			Theoretical[(b)]					$V_{u(exp)}/V_c$			$V_{u(exp)}/V_{u(th)}$[(d)]		
	P_u kN	V_u[(a)] KN	v_u N/mm²	Flexural Strength P_u, kN		Shear strength V_c kN			ECP [17]	ACI [18]	BS [19]	ECP [17]	ACI [18]	BS [19]
				At middle support	Under the load	ECP [17]	ACI[(c)] [18]	BS [19]						
B1	43.75	36.71	2.16	31.60	56.70	27.09	30.15	19.60	1.36	1.22	1.87	1.36	1.22	1.87
B2	62.50	52.44	3.08	31.60	56.70	26.44	29.46	19.29	1.98	1.78	2.72	1.00	0.95	1.16
B3	77.50	65.02	3.83	31.60	56.70	26.44	29.46	19.29	2.46	2.21	3.37	1.24	1.17	1.44
B4	95.00	79.71	4.65	42.10	75.60	24.58	27.38	21.78	3.24	2.91	3.66	1:58	1.50	1.67
B5	70.00	58.73	3.45	42.10	75.60	24.58	27.38	21.78	2.39	2.14	2.70	1.16	1.10	1.23

a at middle support: $V = 0.84\ P$.
b material safety factors and strength reduction factor have been taken as unity.
c f_c' was taken as $0.85\ f_{cu}$.
d $V_{u(th)} = V_c + V_p$, $V_p = 25.84$ kN.

5. Conclusions

From the results reported herein on the feasibility of using external bonded steel plates for shear strengthening of reinforced concrete continuous beams, the following conclusions can be drawn:

1- Reinforced concrete continuous beams can be strengthened in shear efficiently as in the case of simply supported beams by the external bonding of steel plates to the critical shear zone. Using such technique, it is possible to transfer the brittle shear failure of a beam having no internal stirrups into a ductile flexural failure away from the middle support.

2- External plate bonding in reinforced concrete continuous beams increased the ultimate shear capacity at the middle support by up to about 2 times compared to a beam without external steel plates.

3- The presence of external steel plates bonded to the sides of continuous RC beams at the middle support zone enhanced the negative flexural strength of the beam, since failure of tested beams occurred either by shear or at the positive moment region. Also, extending such plates to the positive moment region increased the positive flexural strength by about 37% over the calculated strength.

4- Shear failure in beams having externally bonded steel plates occurred suddenly and was accompanied by tearing and separation of the steel plates along with the concrete cover.

5- The simple empirical formula proposed by [9] for the estimation of the contribution of externally bonded steel side plates to the shear strength does not take into account the effect of the thickness of the side plates. The formula well predicted such contribution for beams strengthened with 2 mm thickness steel plates. However, for beams strengthened with 4 mm thickness steel plates the formula becomes inappropriate.

6. Acknowledgement

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