Replacing of Internal Tension Bars by External Bonded Plate

Awadh E. Ajeel
Assistant Lecturer
Highway and Transportation Dept.
College of Engineering
Al-Mustansiriya University

Rana Hashim Ghedan
Assistant Lecturer
Civil Engineering Dept.
College of Engineering
Al-Mustansiriya University

Dina Mukheef Hamza
Assistant Lecturer
Civil Engineering Dept.
College of Engineering
Al-Mustansiriya University

Abstract

This paper presents comprehensive test data on the effect of replacing internal tension bars by externally bonded steel plate at beam tension face on its cracking pattern, structural deformations and ultimate strength of concrete beams reinforced with external. The experimental work include flexural testing of $100\times150\times1500$mm concrete beams divided into two groups to investigate the mutual action of increasing the plate thickness against its width and vice versa. The test variable included the amount of conventional (internal) reinforcement, percentage of replacing the internal reinforcement with external steel plate and the dimensions of plate. Provided the adhesive layer are chosen carefully and proper gluing techniques are followed The results show that beams reinforced with external steel plate show beam action and composite behavior right up to failure and can be used successfully instead of internal reinforcement of a replacement ratios of removed bars to the original tension bars equal to 33% and 67% in which the steel plates used of wider and thiner than other used plates. There is, however, a limitation to plate thickness beyond which shear/bond failure occurs without the beams achieving their full flexural strength (premature failure).

Keywords : Steel plate, bar reinforcement, replacing, epoxy bonded, external plate, composite action RC beams, bond failure, tension face, replacement ratio

---

الخلاصة

يقدم هذا البحث دراسة عملية على تأثير استبدال تسليح الشد الداخلي بصفيحة فولاذية خارجية على التشقق , التشوهات الإنشائية و المقاومة القصوى لها. تتضمن هذه الدراسة فحص الانثناء لعتبات خرسا نية ($100\times150\times1500$ملم) مقسمة الى مجموعتين للتحقق من التأثير المتبادل لزيادة سمك الصفيحة مع العرض. متغيرات البحث تشمل كمية التسليح الداخلي , نسبة استبدال التسليح الداخلي بالصفيحة الفولاذية الخارجية و أبعاد الصفيحة المستخدمة. حيث أظهرت نتائج هذه الدراسة انه بتوفير الربط المناسب بين سطح الخرسانة و الصفيحة الفولاذية يمكن استخدام هذه الصفائح بنجاح بدلا من التسليح الداخلي التي بها نسبة الاستبدال للحديد المرفوع الى الحديد الاصلي مساوي الى 33% و 67% والتي بها الصفائح اعرض و انحف من بقية الصفائح المستخدمةفي النماذج الاخرى لذلك لابد من ضرورة تحديد سمك الصفيحة المناسب لمنع حدوث فشل في منطقة ارتباط الصفيحة بالوجه الخرساني قبل الوصول للمقاومة القصوى للعتبة.

Notation.

b_c, b_g, b_p, b_{pmax}: concrete, glue layer, plate and the maximum width of plate respectively.

c, c_b: neutral axis distance from top fiber in general at balance load conditions respectively.

d: effective depth of concrete section.

E_c, E_s: modulus of elasticity of concrete and reinforcement.

f'_c, f_r: compressive and flexural strength of concrete.

f_y, f_{yp}: yield stress of internal steel bars and external steel plate respectively.

L, L_g, L_p: clear span length of beam, length of glue layer and external steel plate respectively.

t_g, t_p, t_{pb}, t_{pmax}: glue, plate, plate at balance cond. and the max. thickness of plate respectively.

$ß_1$: factor depending on concrete compressive strength (0.65 – 0.85).

Δ_y, Δ_u: deflection at yield and ultimate load respectively.

Φ: diameter of internal steel bars.

1-Introduction.

It is clear that, the plate bonding has a considerable success in repair, strengthening and upgrading of concrete structures in spite of variety of their loadings and functions. The fame of this technique came from his capability of increasing the structural member capacity and restraining its deformability. Whoever, the technique advantages are conjugation with a worrying illicit which is the concentrated stresses at the plate ends. The end plate high stresses make the structure prone to premature failure due to plate separation, shearing or tearing of concrete cover. Thereby, taking care and rational design are recommended to utilization of the plating technique. Investigations into the performance of members strengthened by steel plate were started in the 1960s[1-3].since then, considerable experimental work has been reported on the performance of the repair technique when employed for strengthening undamaged RC beams [4-8].however , only limited data has been published on the performance of this technique when used for strengthening and repair of structurally damaged RC beams .

Macdonald [9] reported that there is no adverse effect of precracking on the structural behavior of RC beams strengthened by bonding steel plates on their tension faces. Similarly, based on an experimental study, Swamy reported that plate bonding can be used for repairing structurally damaged RC beams [10]. He witnessed the performance of RC beams strengthened by bonding steel plates on the tension faces of beams preloaded up to 30,50,and 70 percent of the ultimate load level , relatively thin plates were used in his study and no plate separation was observed at the end. More recently, work has been carried out in the united kingdom at a number of centers, including the department of Civil and Structural Engineering a Sheffield University[1-3,11].

The work at Sheffield has highlighted a number of features of this technique, some of which can be summarized as follows:

1. Full composite action can be achieved between a concrete member and a steel plate by the use of suitable epoxy glue.
2. Plating has a considerable reducing effect on both flexural crack width and deflections.
3. Where failure of a strengthened reinforced concrete member is by yielding of the bonded plate, the ultimate strength can be accurately predicted by using conventional R.C. theory.
4. When it is necessary to use relatively thick plate, failure can occur by horizontal cracking and plate separation which starts at the ends of the plates. In this case, the plate do not reach their yield stress and failure is sudden and with little warning.

2- Objective.

This paper presents tests results on the performance of plate bonding technique when used instead of internal reinforcement. The effect of reinforcement ratio and plate dimensions on mode of failure, ultimate strength and ductility of the beams is intended to discuss.

3- Steel Plate Dimensions.

Some researchers adopted expressions to attain a desirable or rational design for externally bonded plates; these expressions can be arranged as follows: The width of plate and therefore interface layer is limited by the size of concrete section and is recommended to be at most 20mm less than the concrete beam width (12), so that:

$$\mathbf{b_{p\,max} = b_g = b_c - 20} \qquad (1)$$

The thickness of the plate must be equal to / or less than the plate thickness at balance load conditions and the maximum plate thickness to ensure a ductile flexural failure (12).

$$\mathbf{t_{p\,max} \le t_{pb} = \frac{(\beta 1 c_b (0.85 f'c) b_c - A_S f_y)}{b_p f_{yp}}} \qquad (2)$$

Two tentative design criteria are suggested to ensure the full flexural capacity of the beam and ductility at failure (8):

$$\mathbf{\frac{b_p}{t_p} \ge 50 \quad (\text{for mild steel})} \qquad (3)$$

$$\mathbf{\frac{c}{d} \le 0.4} \qquad (4)$$

4. Experimental Work.

Six beams were tested up to failure. All the beams were 100 x 150 mm cross section and 1500 mm long. All beams were reinforced with 2Φ8mm in compression and tension reinforcement as shown in Figure (1), the shear spans were provided with Φ 6mm links at 60mm center to center.

These beams were divided into two groups each group was implemented by four identical reinforced concrete beams with normal strength concrete. In these two groups the tension reinforcement was replaced by different configuration steel plates to achieve aim of the study. The first group was made with external steel plates have increment thickness and constant width and length to select the sufficient thickness and equivalent area of external steel plate that can substitute the tension steel bars strength.

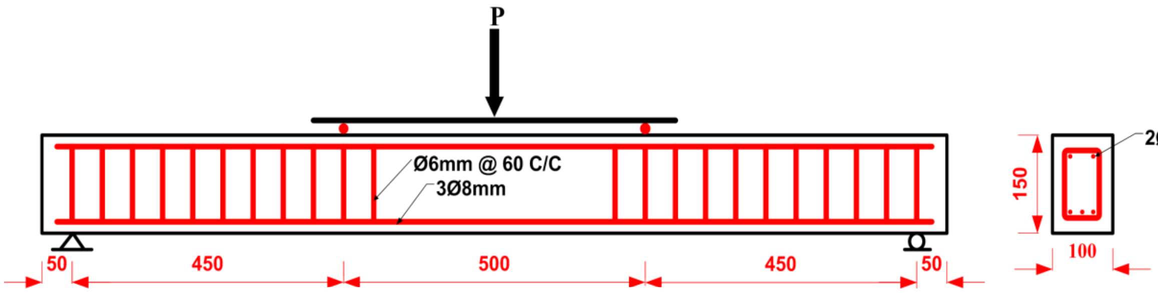
The second group was provided with constant thickness and length of external steel plates, but with variable width to determine the required width of the plate that guarantees adequate bonding and composite action between concrete beam and external steel plate. The external plates were bonded to the beams by epoxy (glue) of dimensions t_g x b_g and length L_g was equal to the plate length Lp. The details of the main variables in the test are shown in Table (1).

Table (1) Details of the Beams

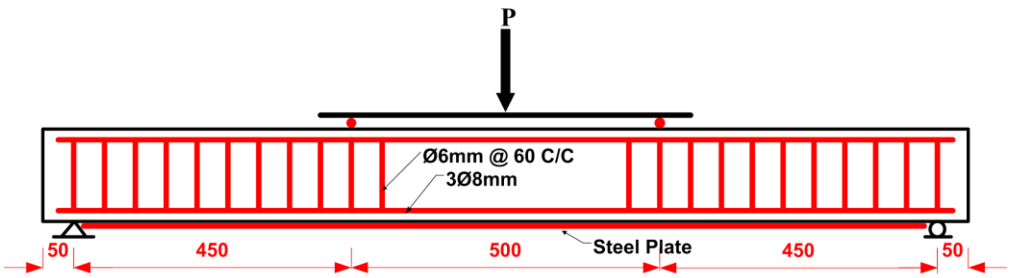
Beam Symbol	Group	Tension Bars	Dimensions of Plate	Ratio of Replacement
R1	1 & 2	3 Φ 8	-	0
R2	1	2 Φ 8	50 x 0.5	33
R3	1&2	1 Φ 8	50 x 1.0	67
R4	1	-	50 x 1.5	100
R5	2	2 Φ 8	25 x 1.0	33
R6	2	-	75 x 1.0	100

Surface preparation and the gluing process is as follows: the beam surface to be plated was ground to expose the aggregate and then cleaned of debris, the plate were shot blasted both sides to prevent warping, the glue was mixed and then applied to both the plate and the concrete surface, the two surfaces were then put together and hold in place until the glue had cured. An average glue layer thickness of 1.0 mm was maintained in all the beams reinforced with external steel plate.

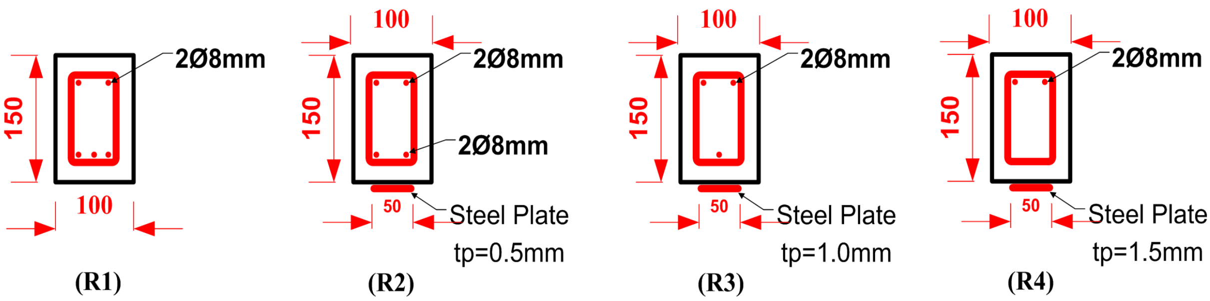
The beams were tested simply supported on a pivot bearing on one side and a roller bearing on the other, over a span of 1400mm and loaded at the third points. Load was applied , by means of a hydraulic jack, in increments of 2kN throughout the test at each load increment, concrete cracks were detected using a magnifying glass and drawn on the face of the tested beam and their width measured. Testing was continued until the beam showed a drop in load carrying capacity with increasing deflections.



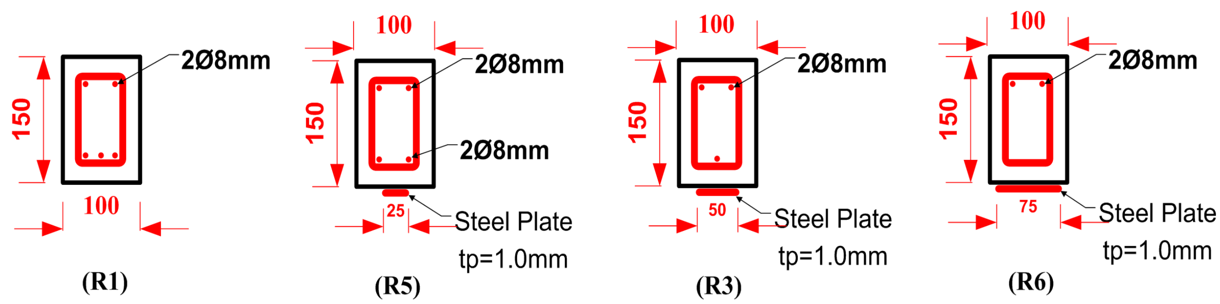
(a) Reference Beam R1



(b) Beams Reinforced Externally by Steel Plate



(c) First Group of Beams with Plate Width=50mm and Plate Thickness=0.5-1.0-1.5mm



(d) Second Group of Beams with Plate Width=25-50-75mm and Plate Thickness=1.0mm

Figure (1) Details of the Reinforced Concrete Beams

5. Materials Properties.

5.1 Concrete.

Ordinary Portland cement type I was used. The coarse aggregate was a 14mm maximum size crushed gravel and the fine aggregate was natural river sand (AL-Ukhaider), zone (2) according to IQS:45 1984 with 2.71 fineness modulus. Cylinders and prisms for control tests were cast and stored with each beam and then tested when the beam was tested. The mix proportions and the average results of cylinder strength f'c, modulus of rupture f_r for all beams are given in Table (2).

Table (2): Mix Proportions and Mechanical Properties of Concrete

Mix Properties	Weight Constituent kg/m^3					Mechanical Properties MPa		
	Cemen	Sand	Gravel	Water	SP	f'_c	f_r	†f_r
	430	645	1289	129	6.5	38.8	3.2	4.4

$\dagger f_r = 0.7\sqrt{f'c}$ (13)

5.2 Steel Bar Reinforcement.

Deformed steel bars of diameter 8mm were used for the main reinforcement and plain steel bars of diameter 6mm were used for stirrups. Properties of the steel bars are shown in Table (3).

5.3 Steel Plate Reinforcement.

Steel plates with different thicknesses of 0.5, 1.0 and 1.5 mm were used as external reinforcement instead of the tension reinforcement, this plate was bonding to the concrete surfaces by epoxy resin. Properties of the plates are shown in Table (3).

Table (3): Properties of Steel Reinforcement

Reinforcement	Bar Diameter mm	Plate Thickness (t_p) mm	Yield Stress (f_y) MPa	Ultimate Stress (f_u) MPa	Modulus of Elasticity (E_s) GPa
Steel Bars	*6*	*-*	*383*	*545*	*200*
	8	*-*	*424*	*602*	*200*
Steel Plates	*-*	*0.5*	*283*	*351*	*200*
	-	*1.0*	*280*	*347*	*200*
	-	*1.5*	*280*	*347*	*200*

5.4 Super Plasticizer.

Chloride free liquid admixture with a 1.21 specific gravity was used, commercially named Top Bond 603.

5.5 Glue.

Epoxy bonding agent with greater than concrete mechanical properties was used in the bonding of steel plate with concrete beams.

6. Theoretical Assumptions.

The basis of the theoretical assessment of the beam section reinforced with external plate outlined below.

1. The ultimate concrete compressive strain was taken as 0.003 and a rectangular stress block of 0.85 f'c maximum stress is used for all the beams.
2. The glue layer is sufficient to provide enough bonding between the concrete beam and the external plate for achieving full composite action.
3. Concrete and glue effect in tension resistance is ignored.
4. Linear stress strain curve for the bar and plates is used to determine total steel tensile force. From the above consideration it was found that all the six beams were theoretically under reinforced.

7- Test Results.

It is obvious from the present study that the removed area of tension reinforcement replaced not by a steel plate of equivalent moment capacity but by a plate of a halved area of the removed bars and that to investigate the adequacy of the steel plate of less area of original removed bars on the performance of the beams.

7.1 Failure Load.

Table (4) give a summary of failure loads and other properties for each beam. The theoretical load capacity and height of crack are based upon the same assumptions detailed earlier. Considering the ratio of experimental load capacity to the theoretical load capacity, the ratios for beams R1, R2, R3 and R5 show good agreement, which suggests that the method of analysis was satisfactory.

The low result of beam R4 may lie in the failure by separation of steel plate that occur before the beam reach its load capacity. It can be indicated when a comparison conducted between reference beam R1 with beam R2 and beam R5 that added steel plate as external reinforcement of a replacement ratio 33% increased the strength slightly by about 4.5% and 0% respectively from beam R1 so the behavior of beams R2 and R5 is as the same as beam R1.When a replacement ratio equal to 67% in beam R3 the load capacity decreased 4.5% of the reference beam R1.

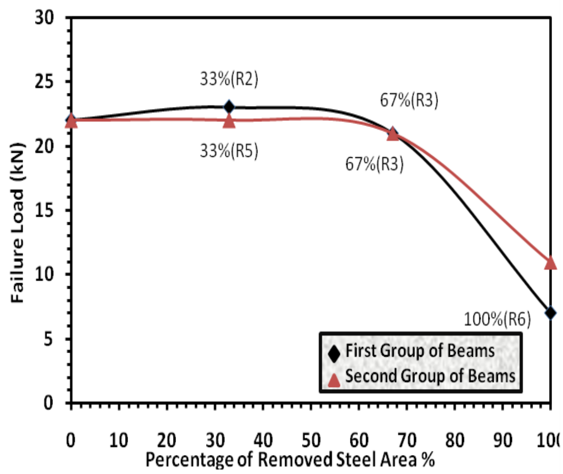
This reduction of load capacity continued in beams R4 and R6 of replacement ratio equal to 100% in which the load capacity decreased significantly by about 68%, 50% respectively of reference beam R1. In general, beams bonded with thin plate failed in flexure and reached theoretical predicted ultimate strengths based on the flexural mode of failure. Beams bonded with relatively thicker plates failed prematurely and could not reach the theoretical predicted ultimate strength.

Figures (2) and (3) show the comparison between the two groups based on failure load and Ratio of failure loads respectively. From these figures the second group show the best load capacity and good agreement between the experimental and theoretical results in beam R6 and (with ratio of steel replacement=100%), while the first group show best results in beam R2 (with ratio of steel replacement=33%).

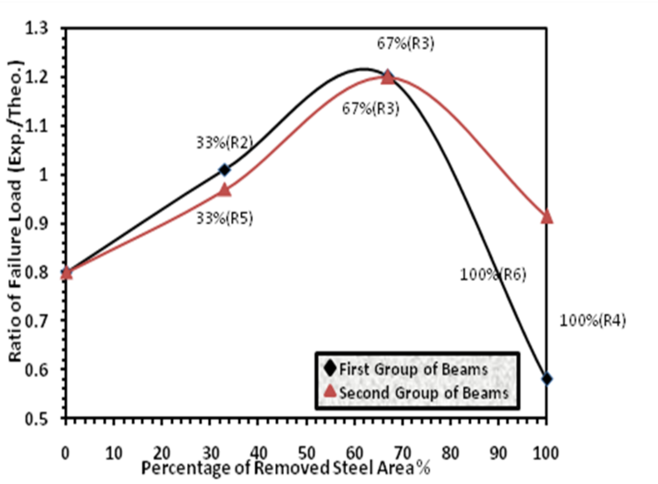
Table(4) Details of Failure Loads, Height and Width of Crack and Mode of Failure

Beams	A Maximum Load Sustained kN.m	B Theoretical Load kN.m	A/B	C Experimental Height of Crack	D Theoretical* Height of Crack	C/D	Crack Width	Mode of Failure
R1	22	27.6	0.8	115	118.32	0.97	0.6	Steel yielding
R2	23	22.8	1.011	123	121.5	1.012	1.7	Steel yielding with plate separation
R3	21	17.5	1.2	138	124.36	1.11	1.8	Plate separation
R4	7	12.1	0.58	110	127	0.87	0.8	plate separation
R5	22	22.7	0.97	125	121.53	1.028	1.9	Steel yielding
R6	11	12.1	0.915	123	127	0.97	1.0	Steel yielding with plate separation

*** based on theoretical neutral axis depth**



Figure(2) Comparison between the two groups based on failure load



Figure(3) Comparison between the two groups based on Ratio of failure loads

7.2 Mode of Failure.

From table (4) the failure mode of reference beam R1 and beam R5 is the same by yielding of the internal and external reinforcement in beam R5. The failure of beams R2 and R6 was accompanied by local separation of the steel plates. The failure was occurred at a vertical flexural crack immediately below the crushed zone of the concrete. Separation was through the concrete above the epoxy layer and occurred after spilling had begun in the concrete compression zone.

The results of these beams are reasonable and again suggest that the full composite action was achieved almost up to failure. Beams R3 and R4 failed by plate separation before yielding and crashing of the concrete. From figures (4) and (5) the comparing between the theoretical with the experimental results for the two groups, the second group shows the best agreement.

Figure(4) Experimental Results of the First Group of Beams

Figure(5) Experimental Results of the Second Group of Beams

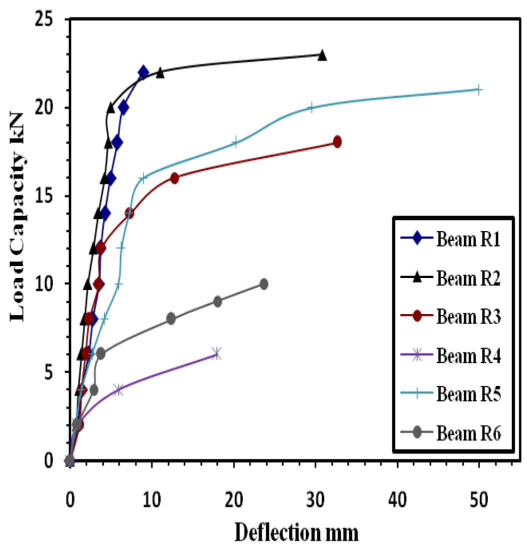
7.3 Deflection and Ductility.

The results of deflection of all the beams are shown in table (5) and the load-deflection curve is shown in figure (6) to compare between these beams. It is considered from figure (6) that the deflection of beams increased from the original beam R1 with increasing the removed tension bars. The comparison of beam R1 with beams R2 and R5 which has the same percentage of reinforcement show that the deflection increased from the original beam R1 but in different values of beams R2 and R5 of 30.86mm and 49.96mm respectively and that due to the wider and thiner steel plate of beam R2 The same thing clear in beams R4 and R6 which of the wider steel plate with 17.97mm and 23.7mm respectively.

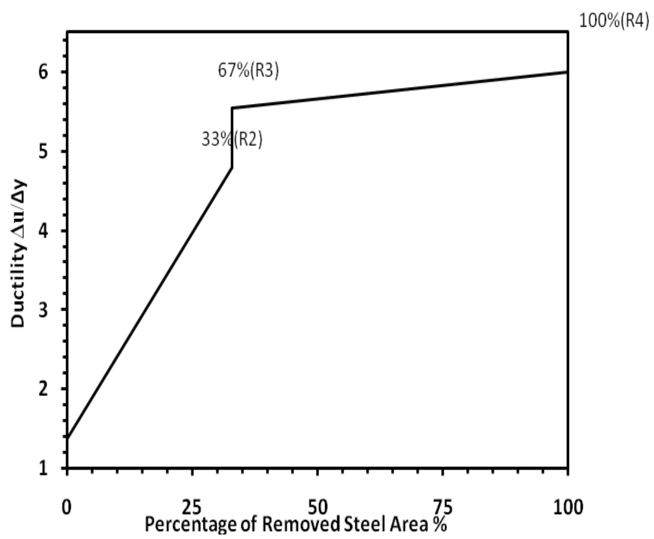
However the beams with added plates show more ductility so that the ductility is observed to decrease as the reinforcement ratio increased. The beam R1 exhibited small ductility while all the rest beams show more ductility before failure as in figure (7). The values of ductility are shown in table (5). It is so clear that the beam with a steel plate of sufficient width is more ductile as in comparison of beams R2 and R5 from the reference beam R1.

Table(5) The Results of Deflection near The Ultimate Load and Ductility

Beams	Ratio of replacement %	Δy mm	Δu mm	Δu/Δy Ductility
R1	0	6.5	8.92	1.38
R2	33	5	30.86	4.8
R3	67	4	32.63	Rupture
R4	100	1	17.97	Rupture
R5	33	9	49.96	5.55
R6	100	4	23.70	6



Figure(6) Load–Deflection Curve for all the Beams



Figure(7) Comparison of Ductility between the Beams

7.4 Cracking.

Typical crack patterns can be seen in figures (4) and (5) .The values of average crack width and height are shown in table (6). From the results, the length of cracks increased with decreased tension reinforcement as in beams R2 and R3 from original one R1 of the first group in which the crack length was 1.7mm, 1.8mm and 0.6mm for beam R1 respectively, except beam R4 of crack length decreased to 0.8mm due to rupture happened in this beam. The same results of crack length revealed in beams of second group. Again the height of cracks increased with decreased tension reinforcement in beams of the first group also fore the width of cracks. The length, height and width of cracks for beam R5 are greater than in beam R2 of wider and thiner steel plate. It is obvious that the beam R4 failed by rupture so its values of cracks smaller than in beam R6.

Table(6) The Results of Cracks

Beams	L_{cr} mm	H_{cr} mm	W_{crs} mm
R1	0.6	115	170
R2	1.7	123	240
R3	1.8	138	240
R4	0.8	110	120
R5	1.9	125	300
R6	1.0	123	80

8. Conclusions.

The present experimental work represents a preliminary study only and much more data are required before any conclusions can be made. The following remarks can be observed from current study:

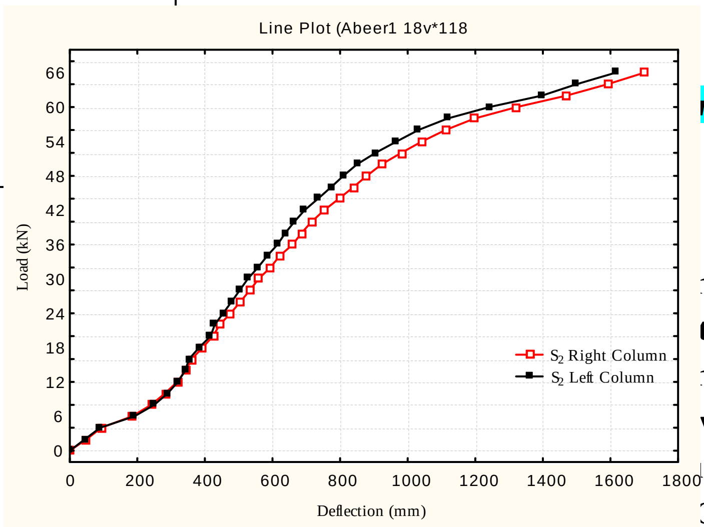
- The conventional method of analysis is satisfied in plated beams where the tension bars are replaced by a steel plate when a full composite action exists between the plate and concrete and the beam failed by yielding.

- Although the removed bars replaced not by a plate of an equivalent area but by a plate of a halved area the performance of some plated beams give a good agreement with the original beam R1.

- It is obvious from the present results of all beams that the steel plate can be used as an external reinforcement only in case of replacing specific ratio of internal tension reinforcing bars. In the present study this ratio of the area of removed bars to the area of total tension bars is equal to 33% which here 1Ø8mm removed and replaced by a steel plate and that achieved in beams R2 and R5 which both gave nearly the same loading capacity either equal or little greater than the capacity of reference beam R1 even the same mode of failure. This conclusion confirmed when the removed area increased to 2Ø8mm in beam R3 so the ratio of removed area to the total tension area is 67% exceeding 33% in which the load capacity decreased and this conclusion continued in beam R4 and beam R6 in which a considerable decreasing in the load capacity due to the replaced area increased to 3Ø8mm as shown in Figure (8).

- It can be say from the results of beams R2 and R5, this results were represented by load capacity, crack length, crack height and crack width. The beam R2 gave better results from beam R5 so that the steel plate of width 50mm and thickness 0.5mm is adequate as an external reinforcement from width 25mm and thickness 1.0mm. The explanation of these results is that when the thickness increased from certain value the concentration stresses at the plate ends increase so the beam fail in bonding between the plate and concrete before reach to ultimate capacity of the section, on the other hand the wide steel plate provide more bonding area between concrete and the plate. So the load Capacity enhanced with wider and thiner steel plate accompanied with decreased in cracks length, height and width.

- In the beams which failed in plate separation the bond stresses in the interface between concrete and plate are small and there is no a full composite action between them that depend on the thickness of the plate.



, " **Beton Arme par Collage des Armature**",RILEM
nstruction, Paris, Part 2, 172-203,1967.

t **Metal Reinforcement for Concrete Slabs**",RILEM
nstruction, Paris, Part 2, 165-173,1967.

with External Reinforcement Bonded by Gluing
ilding Construction, Paris, Part 2,142-151,1967.

Charif , A . ," **Plate Separation and Anchorage Reinforced Concrete Beams Strengthened by Epoxy Steel Plates** ", Structural Engineer , Vol. 66,No. 5, 85-94, Mar. 1,1988.

5. Irwin, C.A.K," **Strengthening of Concrete Beams by Bonded Steel Plates.** ", TRRL Supplementary Report 160 UC, Bridge Design Div., Structural Dept. Crowthorne, Berkshire, England,1975.
6. Jones , R.,Swamy, R.N., Bloxham, J. and Bouder , Balah," **Composite Behaviour of Concrete Beams with Epoxy Bonded External Reinforcement** ", Int. J. Cement Composites , Vol. 2, No.2, 91-107, May 1980.
7. Jones , R.,Swamy, R.N and Aug, T.H.," **Under and Over-Reinforced Concrete Beams with Glued Steel Plates**", Int. J. Cement Composites and Light Weight Concrete, Vol. 4, No. 1, 19-32, Feb.1982.
8. Swamy, R.N., Jones, R. and Bloxham, J.W.," **Structural Behaviour of the Reinforced Concrete Beams Strengthening by Epoxy-Bonded Steel Plates.** Structural Engineer , Vol. 65A, No. 2, 59-68, Feb.1987.
9. Macdonald ,M.D.," **Flexure Behaviour of Concrete Beams with Bonded External Reinforcement** ", TRRL Supplementary Report 415, Transport and Road Research Laboratory, Dept . of the Environment , Crowthorne, Berkshire, England, 1978.
10. Swamy, R.N., Jones, R. and Charif, A." **Effect of External Reinforcement on the Strengthening of Structurally Damaged RC Beams** ", Structural Engineer ,Vol. 67,No 3, 45-56, Feb.1989.
11. Barrett, N." **Plates Steel Leeds Floor** ", New Civil Engineer, 20 June 1985,p18.
12. Ziraba, Y. N., Baluch, M. H., Basunbul, I. A., Sharif, A. M. and Al–Sulaimani, G. J., "**Guidelines Towards The Design of Reinforced Concrete Beams with External Steel Plates**", ACI Structural Journal, V. 91, No. 6, Dec. 1994, PP. 639–646.
13. ACI Committee 318, "**Building Code Requirements for Structural Concrete (ACI 318M-05) and Commentary (ACI 318RM-05)**", American Concrete Institute, Farmington Hills, Michigan, 2005, 432 PP.