Proc. 1st International Conference on Structural Engineering Research (iCSER 2017)
20-22 Nov 2017, Sydney, Australia
ISBN: 978-0-6480147-6-8

Steel Contribution Effect on Beams Bonded With Steel Plates of Different Width-To-Thickness Ratios

Sylvester M Rakgate[1] and Morgan Dundu[2]

[1]Lecturer, University of Johannesburg, Johannesburg, South Africa
[2]Senior Lecturer, University of Johannesburg, Johannesburg, South Africa
Corresponding author's E-mail: srakgate@uj.ac.za[1] and mdundu@uj.ac.za[2]

Abstract

The concept of strengthening reinforced concrete (RC) beams using epoxy-bonded steel plates (EBSP) is a well-known solution in structural engineering. Experimental investigations conducted in the past has proved that strengthening RC beams with steel plates are the most efficient, effective, and cost-effective technique of increasing the flexural performance of these elements. However, the focus has been on effect of the external bonded steel plate, and not on the effect of the overall steel contribution ratio on the behaviour of the strengthened beams. Several codes give the minimum reinforcement ratios for concrete beams in order to encourage/improve their flexural behaviour such as cracking and ductility failure. The purpose of this present study is to investigate the effect of the steel contribution ratio on the flexural behaviour of concrete beams strengthened on their tension face with epoxy bonded steel plate, using the experimental results obtained by various researchers in this field. The outcomes of strengthening RC beams are decrease in mid-span deflections, decrease in crack-widths, and increase in first crack load, and consequently increase in both serviceability load and ultimate load, thus making it to be the most feasible strengthening technique

Keywords: Steel plates, Steel contribution factor, RC beams, Externally strengthened, Composite beams.

1. INTRODUCTION

Strengthening of beam elements in reinforced concrete (RC) structures is required when the strength of an existing structure is no longer adequate to resist the current design loads or when the structure is now required to resist larger ultimate loads. There are several methods for strengthening RC beams, however, strengthening of RC beams using epoxy-bonded mild steel plates on the tension face has proven to be the most effective, efficient, economical and convenient technique to enhance the flexural and shear performance of RC beams under service and ultimate loads. This technique has been applied successfully to strengthen RC structures such as buildings and bridges in various parts of the world, including South Africa, France, Switzerland, Japan, Poland, Belgium and United Kingdom (Bloxham, 1980). The technique has several advantages, compared to other strengthening methods, which include the fact that steel plates are relatively cheaper and readily available, has uniform material properties (isotropic), high ductility and high fatigue strength, can be secured easily whilst the structure is in use (Raithby, 1982), does not significantly change the overall dimensions of the structure, and can be secured without causing any damage to the structure (Swamy et al. 1987). However, much of the focus in these studies has been on the effect of the external bonded steel plate on the overall behavior of the strengthened beams and whether the width-to-thickness ratios of the bonded steel plate encourage ductility or not. The purpose of the present study is to investigate the effect of the steel contribution ratio (ρ) on the flexural behavior of rectangular concrete beams strengthened on their tensile with epoxy bonded steel plate. In this paper, the steel contribution ratio (ρ) is defined as the ratio of total area of the internal and external steel reinforcement to the area of the concrete.

2.0 SPECIMENS AND TESTS CONDUCTED.

A decision was taken to group all the material properties according to the author reviewed. The material properties of the reinforcement bars, steel plate, concrete and epoxy resin used in the various studies are given in Table 1, where f_y is the 0.2% proof yield stress of the steel reinforcement bars and steel plates, f_u is the ultimate stress, E is Young's modulus of elasticity, F_{cu} is the average 28-day compressive strength of the concrete cubes, and Cs, f_t and E_s are the compressive, tensile strength and Young modulus of the epoxy resin, respectively.

Table 1: Material properties

Author	Specimen	Steel and reinforcement bars			Concrete	Epoxy resin		
		f_y (MPa)	f_u (MPa)	E_s (GPa)	F_{cu} (MPa)	C_s (MPa)	f_t (MPa)	E_s (GPa)
Fleming and King (1967)	6Y14	No material properties provided						
	2Y6							
	SP0.6							
Huovinen (1996)	2Y9	No material properties provided						
	SP2							
	SP5							
	SP10							
Bloxham et al (1980)	3Y20	450.0	507.0	200	63.0	-	15.0	-
	R 6	250.0	336.0	-	73.0			
	SP 1.5	236.0	310.0	200	-			
	SP3.0	258.0	316.0	200	-			
	SP6.0	248.0	308.0	200	-			
Jones et al. (1982)	2Y10	530.0	597.0	200	63.4	44	5.3	6.0
	SP1.5	216.6	359.6	192	-			
	SP3.0	263.0	346.2	198	-			
	SP5	217.5	445.2	200	-			
	SP10	240.0	434.6	200	-			
Oh et al. (2003)	2Y16	365.0	536.0	200	28.7	180	70.0	2.3
	2Y13	345.0	503.0	200	-			
	R 8	420.0	600.0	200	-			
	SP2	292.0	410.0	200	-			
	SP3.0	292.0	410.0	200	-			
	SP4	292.0	410.0	200	-			
	SP5	292.0	410.0	200	-			
Neelamegam et al. (1998)	2Y9	475.0	-	-	44.0	No epoxy properties provided		
	R 6	275.0	-	-	-			

Fleming and King (1967) conducted an experimental investigation to determine the effect of strengthening RC beams by bonding steel plates on the tension side. To achieve this, a total of 2, 150 x 280 x 2800 mm beams, reinforced with 6, 14 mm tension reinforcement bars and 2, 6 mm compression bars were tested. Shear failure was limited using 5 mm shear links, spaced at 200 centre-to-centre mm. The author did not provide the properties of the materials. In a related study, Bloxham et al. (1980) examined the effect of epoxy-bonded steel plate on the structural deformations, serviceability loads, ultimate loads and first crack loads of RC beams. Twelve (12) RC beams of 155 x 255 mm in cross section and 2500 mm long were tested; one of the RC beams was used as a control beam. All beams were under-reinforced on the tension side with 3, 20 mm diameter placed at an effective depth of 220 mm. To avoid shear failures, 6 mm diameter shear links were provided at 75 mm center-to-center along the shear span. The shear links were held in compression by 2, 6 mm diameter bars.

Jones et al. (1982) tested 5 beams, one control beam and four (4) under-RC beams of 100 x 150 x 2400mm size, with glued mild steel plates of 80mm width, 2150mm length and varying thickness of 1.5, 3, 5, and 10mm. All the beams were internally reinforced on the tension side with 2, 10 mm diameter and 2, 8 mm diameter bars in compression. The beams were reinforced for shear with 6 mm shear links, spaced at 70 mm centre-to-centre, to ensure flexural failure. In order to establish the bond strength of glued steel plates and the effect of the steel plate on the overall flexural capacity of the strengthened beams, Huovinen (1996) tested 8 RC beams of 300 x 300 and 3200 mm in size. Two (2) were used as control beams and 6 beams were externally strengthened by bonding steel plates of 100 mm width, 2800 mm length and varying thicknesses of 2 mm, 5mm and 10 mm.

The RC beams were internally reinforced with two compressive bars and two tension-reinforcing bars of 9 mm in diameter.

In a wider study, Neelamegam et al. (1998) tested ten RC beams of 100 x 200 x 2400 mm in size, under incrementally two-point static loading, over a span of 2300 mm between the supports. The beams were internally reinforced on the tension side with 0.58% tension reinforcements and the shear links were of 6 mm diameter mild stirrups, spaced at 100 mm centre-to-centre. The bonded steel plates had a constant width of 100 mm, thicknesses varying from 0.8 mm to 6.8 mm, and length of 2200mm, 1800mm, 1400, and 800mm for S2-S7, S8, S9 and S10, respectively. Oh et al. (2003) tested 12 rectangular concrete beams under two-point static loading, with one (1) control beam and 11 strengthened beams. The beams were of 150 x 250 x 2100 mm in size, and each beam was strengthened internally with 2, 16 mm diameter tension bars and 2, 13 mm compression bars. To avoid shear failure, 8 mm diameter shear links, spaced at 110 mm centre-to-centre were provided. The bonded steel plate had constant length of 2000 mm, width of 150 mm and thicknesses varying from 2 to 5 mm. The adhesive thicknesses used and the shear-span-to-depth ratio (a/d) also varied from 1 to 7 mm and from 1.36 to 4.77, respectively.

Table 2: Details of the tested specimen

Author	Specimen	L/h	a/d	Steel plate	Reinforcement bars			
				Plate size	Gap	TBs	CBs	Shear links
Fleming and King (1967)	150x280x2800	10.0	0.50	150x6	-	6Y14	2Y6	R5
Huovinen (1996)	300x300x3000	10.0	3.33	100x2.0x2800	100	2Y9	2Y9	-
				100x5.0x2800				
				100x10x2800				
Bloxham *et al.* (1980)	155x255x2500	9.80	3.01	125x1.5x2200	50	3Y20	2R6	R6
				125x3.0x2200				
				125x6.0x2200				
Jones *et al* (1982)	100x150x2400	16.0	5.33	80x1.5x2150	50	4Y16	2R8	R6
				80x3.0x2150				
				80x5.0x2150				
				80x10x2150				
Oh *et al*. (2003)	150x250x2400	9.6	2.80	150x2.0x2000	50	2Y16	2Y13	R8
				150x3.0x2000				
				150x4.0x2000				
				150x5.0x2000				
Neelamegam *et al*. (1998)	100x200x2400	12.0	3.83	100x0.8x2200	50	2Y9	-	R6
				100x1.2x2201				
				100x2.3x2202				
				100x3.2x2203				
				100x4.3x2204				
				100x6.8x2205				

All beams were tested under two-point static loading over simply supported spans to simulate a distributed load, with equal point loads applied at a third of the effective length of the beam from each support. Almost all the beams were tested with a gap of 50 mm between the bonded plate end and the support to delay plate separation and maximize the strengthening effect.

3.0 EXPERIMENTAL RESULTS

The experimental results of the tested specimen are given in Table 3. In this table, ρ is the steel contribution ratio, $P_{ecrc/s}$ is the first experimental crack load of the control/strengthened beam, $P_{emaxc/s}$ is the maximum experimental load of the control/strengthened beam, $M_{emaxc/s}$ is the maximum experimental moment of the control/strengthened beam. Table 3 also compares the experimental first crack load of the strengthened beam to the experimental first crack load of the control beam (P_{ecrs}/P_{ecrc}), and the maximum experimental moment of the strengthened beam to the maximum experimental moment of the control beam (M_{emaxs}/M_{emaxc}). The failure modes of the respective specimen for each author are also recorded in Table 3.

Table 3: Experimental results

Author	Beam	ρ	t (mm)	$P_{ecrc/s}$ (kN)	$P_{emaxc/s}$ (kN)	$M_{emaxc/s}$ (kN)	$\delta_{c/s}$ (mm)	$\frac{P_{ecrs}}{P_{ecrc}}$	$\frac{M_{emaxs}}{M_{emaxc}}$	Failure modes
Fleming and King (1967)	C1	0.0220		40.0	86.0	80.27	-	-	1.00	Flexure
	B1	0.0430	-	90.0	98.0	91.47		2.25	1.14	Shear failure
Bloxham *et al* (1980)	C-1	0.0239	-	35.0	232.0	88.90	-	-	1.00	Flexure
	203	0.0286	1.5	50.1	270.0	103.50	-	1.43	1.16	Flexure
	204	0.0333	1.5	50.1	270.0	103.50	-	1.43	1.16	Shear failure
	205	0.0428	1.5	54.8	213.0	81.70	-	1.57	0.92	Delamination
	207	0.0286	3.0	55.0	262.0	100.60	-	1.57	1.13	Flexure
	208	0.0333	3.0	49.0	264.0	101.20	-	1.40	1.14	Shear failure
	209	0.0428	3.0	52.4	220.0	84.30	-	1.50	0.95	Delamination
	210	0.0428	3.0	50.1	215.0	82.40	-	1.43	0.93	Delamination
	216	0.0286	6.0	54.0	262.0	100.60	-	1.54	1.13	Flexure
	217	0.0333	6.0	48.0	257.0	98.60	-	1.37	1.11	Shear failure
	218	0.0428	6.0	51.1	194.0	74.50	-	1.46	0.84	Delamination
	219	0.0428	6.0	55.0	220.0	84.30	-	1.57	0.94	Delamination
Jones et *al* (1982)	URB1	0.0105	3.0	-	28.1	10.54	-	-	1.00	Flexure
	URB2	0.0185	3.0	-	40.0	15.00	-	-	1.42	Flexure
	URB3	0.0265	3.0	-	55.0	20.63	-	-	1.96	flexure+ plate separation
	URB4	0.0371	3.0	-	57.5	21.56	-	-	2.05	Plate separation
	URB5	0.0638	3.0	-	53.1	19.91	-	-	1.89	Plate separation
Neelamegam *et al* (1998)	CB	0.0058		8.0	30.0	23.00	7.5	-	1.00	Flexure
	S2	0.0098	-	20.0	37.0	28.37	3.4	2.50	1.23	Flexure
	S3	0.0118	-	24.0	43.0	32.97	2.8	3.00	1.43	Flexure
	S4	0.0173	-	28.0	59.9	45.92	2.1	3.50	1.98	Flexure
	S5	0.0218	-	36.0	60.0	46.00	1.7	4.50	2.00	Debonding
	S6	0.0218	-	44.0	48.0	36.80	1.8	5.50	1.60	Debonding
	S7	0.0273	-	48.0	56.0	42.93	1.3	6.00	1.87	Debonding
	S8	0.0398	-	36.0	42.5	32.50	1.8	4.50	1.42	Debonding
	S9	0.0218	-	20.0	35.0	26.83	2.1	2.50	1.17	Flexural peeling
	S10	0.0218	-	10.0	28.0	21.47	7.5	1.25	0.93	Flexural peeling
Oh *et al* (2003)	CB	0.0107	-	-	89.0	31.15	34.7	-	1.00	Flexure
	B23	0.0187	3	-	136.0	47.60	8.15	-	1.53	Flexure
	B33	0.0227	3	-	137.0	47.95	7.02	-	1.54	Flexure
	B43	0.0267	3	-	126.0	44.10	4.35	-	1.42	Plate separation
	B53	0.0307	3	-	142.0	49.70	5.00	-	1.60	Plate separation
	B41	0.0267	1	-	125.0	43.75	4.68	-	1.40	Plate separation
	B45	0.0267	5	-	134.0	46.90	4.97	-	1.51	Plate separation
	B47	0.0267	7	-	150.0	52.50	5.35	-	1.69	Plate separation
	B43B1	0.0267	3	-	132.0	69.30	5.94	-	1.48	Flexure
	B43B2	0.0267	3	-	128.0	57.60	5.61	-	1.41	Plate separation
	B43B3	0.0267	3	-	135.0	33.80	4.67	-	1.51	Plate separation
	B43B4	0.0267	3	-	221.0	33.20	5.13	-	2.58	Shear failure
Huovinen (1996)	CB1	0.00142	-	-	77.3	77.3	-	-	1.00	Flexure
	B1	0.00364	1	-	102.5	102.5	-	-	1.33	Flexure
	B2	0.00697	1	-	145.6	145.6	-	-	1.88	Flexure
	B3	0.0125	1	-	107.7	107.7	-	-	1.39	Plate separation

In the investigation reviewed, only three authors (Fleming and King (1967), Bloxham et al (1980) and Neelamegam *et al* (1998)) recorded the first experimental crack load of the control/strengthened beam. In Fleming and King (1967)'s work, the first crack of the control beam appeared at a load of 40.0kN, whilst that of the strengthened beam appeared at a load of 90 kN, to yield a mammoth increase of 125% compared to the crack load of the control beam. This shows a significant delay in the appearance of the first crack as compared to the control beam. In Bloxham et al's (1980) study, the first crack of the control beam appeared at a load of 35.0kN, whilst that of the strengthened beam appeared at loads varying from 48.0 – 55.0 kN. The increase of the cracking load of strengthened beams varied from 37% – 57%, compared to the control beams. This increase is much lower than the beam tested by Fleming and King (1967), despite the fact that the contribution ratio of this beam is the same as some of the beams tested by Bloxham et al (1980). No distinct trend could be established between steel contribution factor and the ratio of the first crack loads (P_{ecrs}/P_{ecrc}) or the modes of failure of the beams. Lastly, in Neelamegam et al's (1998) beams, the first crack of the control beam occurred at a load of 8.0kN, whilst that of the strengthened beam occurred at loads varying from 10.0 – 48.0 kN. The increase of the cracking load of strengthened beams varied from as little as 25% to as high as 500%, compared to the control beams. Except beams S8 – S10, the ratio of the first crack loads (P_{ecrs}/P_{ecrc}) increased with increase in the steel contribution ratio. It is also observed that beams with low steel contribution ratio failed by flexure, whilst those with high steel contribution ratio failed by debonding. The relationships between the ratio of the first crack loads (P_{ecrs}/P_{ecrc}) and the steel contribution ratios are shown in Figure 1. Although the trend is not clear in some results, it is clear from Table 3 and Figure 1 that the steel contribution influenced the initiation of the first crack in all the beams tested.

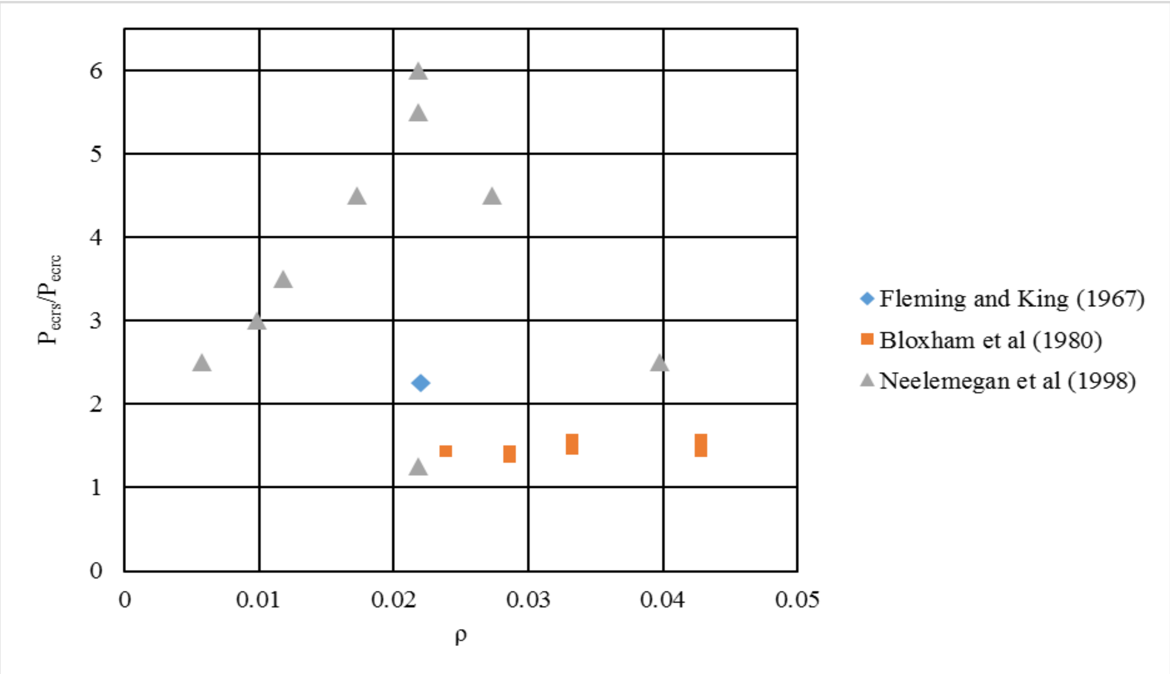

Figure 1: Ratio of experimental first crack loads vs the width-to-thickness ratio

As for Huovinen (1996), all the specimens with reinforcement ratio ranging from 0.00364 to 0.0125 registered a rise of 33 to 88% of ultimate capacity as compared to the control beam with reinforcement ratio of 0.00142, and this is due to the restraining effect offered by the bonded steel plate. Huovinen (1996) experimental result prove that, the load carrying capacity increases as the steel contribution factor increases. Furthermore, all the strengthened beams with steel contribution ratio (ρ) of 0.00364 and 0.00697 failed in flexure and the remaining beam with reinforcement ratio of 0.0125 failed prematurely by plate separation.

The drop in load of beam B3 with steel contribution factor of 0.0125 suggest that from the ductility and ultimate strength point of view, there is a limiting amount of reinforcement ratio that would be structurally reliable. It was concluded that the reinforcement ratio of the concrete beam that can encourages composite action until failure should not be more than 0.00697.
The author cannot conclude with such a bold statement after testing three epoxy bonded steel plate, to make a bold statement like that, a thorough and well defined experimental investigation must be done. In addition to that, the results obtained in this investigation, are only adequate to conclude about the increase in stiffness offered by the bonded steel plate as compared to the control.

4.0 CONCLUSIONS

From the literature conducted, it is apparent that epoxy-bonded steel plate can be successfully used as the strengthening and repairing technique provided the operation be cautiously implemented. Based on the results obtained in the literature, the epoxy-bonded steel plate can increase the flexural stiffness at all load levels and thus reducing the mid-span deflections significantly, it can increase the ductility at failure, increase the maximum flexural capacity, it delays the appearance of the first visual cracks thus resulting in increased service loads, it increases the range of elastic behaviour. Provided the surface preparation and the gluing process are carried out properly, under a given load the epoxy-bonded steel plate can decrease the tensile strains in the concrete due to the composite behaviour of the bonded steel plate, adhesive resin and concrete, in comparison with those in the unplated concrete beam.
Regardless of the advantages the epoxy-bonded steel plate offers, only a limited amount of efficient research has been done regarding the reinforcement ratio (ρ) that must be provided to encourage yielding of the externally bonded steel. Several investigators compliment the effectiveness of the epoxy-bonded steel plate technique by comparison of the manner at which the strengthened beams perform as compared with the unplated beams, however, there is lack of experimental investigation regarding the various reinforcement ratio (ρ) that can lead to composite action to be maintained until failure. The reported range of reinforcement ratio (ρ) that can encourage composite action until failure can be confusing as there is no agreement among the researchers regarding the primary factors that controls the behavior of composite beams

5.0 REFERENCES.

Fleming CJ, King GEM. The development of structural adhesives for three original uses in South Africa. BULLETIN RILEM 1967; 37:240-251.

L'Hermite R, Bresson J. Concrete reinforced with glued plates. RILEM International Symposium. Synthetic resins in building construction, Paris. 1967. p.175 – 203.

Bloxham JW. Investigation of the Flexural Properties of RC Beams Strengthened by Externally Bonded Steel Plate. PhD thesis, Sheffield University; 1980.

Jones R, Swamy RN, Ang TH. Under- and over-RC beams with glued steel plates. International Journal of Cement Composite and Lightweight Concrete 1982; 4:19–32.

Swamy RN, Jones R, Bloxham JW. Structural behaviour of RC beams strengthened by epoxy-bonded steel plates. The Struct. Engr. 1987; 65A (2):59–68.