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Flexural Behaviour of Pre-cracked Reinforced Concrete Beams Repaired with Adhesive Bonded Steel Plates

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Abstract

The problem of cracked concrete structures is receiving considerable attention in the construction industry worldwide. Externally bonded steel plates are used to repair cracked reinforced concrete structures in a number of projects in various parts of the World, but their overall performance is still not fully understood. This investigation assesses the strength and deflections of 12 full-scale reinforced concrete (RC) beams of 175 mm wide x 300 mm deep x 3200 mm long that were pre-cracked, repaired with steel plate at its soffit, using strong epoxy glue and after that, tested to failure under a four-point loading. The beams were divided into three groups. Group 1 comprised of two control beams, which were tested until failure, and were not repaired with steel plates. Group 2 consisted of five beams which were pre-cracked up to the serviceability capacity of the control beams, and Group 3 consisted of five beams which were pre-cracked up to 85% of the capacity of the control beams. All the pre-cracked beams were repaired with steel plates of 6 mm thickness and widths which varied from 75 mm up to 175 mm, in increments of 25 mm. The structural behaviour of all the beams is reported in terms of flexural strength, stiffness, maximum deflections and failure modes. Finally, experimental results are compared with code-predicted results calculated using the EN 1992-1-1 (2004). Externally bonding the steel plate to the pre-cracked reinforced concrete beams resulted in increased stiffness and maximum load capacities and decreased in the maximum midspan deflections. The strength and rigidity of the repaired beams were found to increase with increasing the width-to-thickness ratio of the steel plate.

Keywords: Repairing, Flexural behaviour, Steel plates, Composite beams, Pre-cracking.

1. INTRODUCTION

Reinforced concrete structures often have to face modification and improvement of their performance during their service life. The reason for improvement of an existing structure may come from excessive deflections, cracking elements, structural damage by settlements, earthquakes and vehicle impacts. Repairing has become an acceptable way of improving their load carrying capacity and extending their service lives. In most cases, low carbon steel is used in repairing damaged concrete structures. This type of steel has a ductile stress strain properties and high deformation capacities which contribute to the overall ductility of the externally plated beam. The other advantage of using steel is low cost and wide availability of mild steel. External plating also does not require skilled labour. The epoxy bonded steel plates were extensively carried out by the researchers to strengthen and repair damaged concrete structures in a number of projects since the pioneering study of Hermite and Bresson in 1967. Macdonald (1978) reported that there is no adverse effect of pre-cracking on the structural behaviour of RC beams repaired by bonding the steel plates on their tension faces. Similarly, based on experimental study, Swamy at al (1989) reported that plate bonding can be used for repairing structurally damaged RC beams.

The present investigation focuses on the flexural behaviour of reinforced concrete beams repaired with steel plates. The aim of repairing cracked RC beams was to improve their serviceability and ultimate failure performance. This study investigates the pre-cracked reinforced concrete beams at both serviceability and ultimate loads. After pre-cracking these beams, they were then repaired with epoxy bonded steel plates to their soffits. Previous research has showed that the effectiveness of this repairing method strongly depends on the width-to-thickness ratio of the steel plates used to repair the damaged reinforced concrete structures, (Swamy et al (1989)). This investigation addresses that problem by changing the width-to-thickness ratio of the steel plates and observes the effect it have on the load carrying capacities, deflections and failure modes of the cracked specimens.

2. MATERIAL PROPERTIES

The experimental program consisted of casting 12 under reinforced concrete beams of 175 mm by 300 mm in cross-section and 3200 mm in length. The beams were under-reinforced on the tension side with 2, 12 mm high yield ribbed diameter bars of 451 MPa yield strength and 596 MPa ultimate strength, placed at an effective depth of 263 mm. In order to resist shear failure, 6 mm diameter shear links of 346 MPa yield strength and 425 MPa ultimate strength were provided at 150 mm centre to centre along the shear span. In order to hold the shear links in position, two compression bars of 8 mm in diameter were provided of 327 MPa yield strength and 389 MPa ultimate strength. The average compressive strength of concrete at 28 days was 31 MPa and the concrete cover was 25 mm.

3. SPECIMEN PREPARATION AND TESTING PROGRAMME

3.2 Testing of unrepaired beams

The beams were tested simply supported under a four point static loading system, over a span of 3000 mm. A linear variable displacement transducers (LVDT) was positioned at the mid-point of each beam in order to measure the vertical deflection. The testing configuration and instrumentation is shown in Figure 1. The tests were carried out using an Instron testing machine with an actuator capacity of 500 kN. The load was applied at a deflection rate of 2 mm/min to ensure that sufficient data was collected, until the beam failed. All measurements were recorded automatically at each load increment as the tests were performed. The control beams (Group 1) were loaded until failure attributable to the crushing of the concrete whereas the Group 2 and Group 3 beams were loaded up to their pre-cracking percentages of the load.

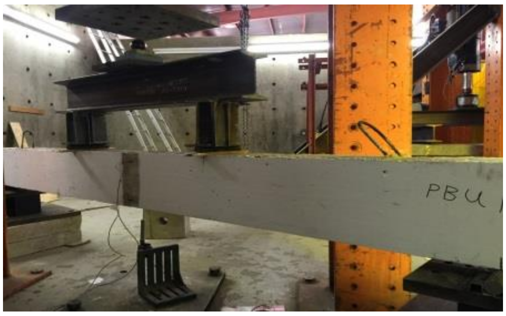


Figure 1: Test set-up

3.3 Surface preparation, bonding of the steel plates and final testing

The surface preparation of the concrete beams was achieved by removing the cement laitance on the soffit of the concrete beams, using a scabbling machine, in order to expose the aggregates and hence provide a good bonding surface for epoxy resins. The mild steel plates were sand-blasted to obtain a clean rough and white metal finish. The used epoxy resin consists of two parts, that is, a primer adhesive (Pro-Struct 618LV) and an epoxy adhesive (Pro-Struct 617NS). The primer adhesive was a two part product of base and activator component which were mixed in a ratio of 2:1, as recommended by the manufacture. It was then applied using a brush onto the prepared tension side of the beams, to allow it to penetrate through the small holes and thin hairline cracks. The epoxy adhesive is also a two part product of base and activator component. The components were mixed together for 5-10 minutes in a ratio of 1:1. The glue layer thickness of 1.5 mm was chosen for the experiment, as it has been proven to perform better by many eminent researchers (Swamy et al (1987), Swamy et al (1989), Basunbul et al (1990a), Hussain et al (1995), Olajumoke and Dundu (2015)). The steel plate was then placed on top of the prepared concrete surface and held in position using rectangular concrete slabs of dimensions of 700 x 700 x 200 mm. A minimum of 7 days of curing was allowed between plating and testing.

After 7 days of curing at ambient temperature, the repaired beams were instrumented and tested in similar fashion as the unrepaired beam. In this loading stage, electrical strain gauges were further used to measure the strain at the mid-point of the bonded steel plate. The distance between the support and the steel plate was kept constant at 35 mm for each beam. The test for each beam was stopped once the load showed a dramatic drop.

4. ANALYSIS AND DISCUSSION OF RESULTS

4.1 Experimental and theoretical maximum capacities

Table 1 shows the experimental and code-predicted results of the control and repaired beams. In this table, M_{rp} is the experimental pre-cracking moment of the repaired beams, δ_p is the experimental deflection at pre-cracking load of the repaired beam, $M_{emaxc/r}$ is the maximum experimental moment of the control/repaired beam, $M_{tmaxc/r}$ is the maximum theoretical moment of resistance, $\delta_{emaxc/r}$ is the experimental maximum deflection at failure of the control/repaired beam.

Table 1. Experimental and theoretical results

Group	Specimen	w/t	M_{rp} (kNm)	δ_p (kNm)	$M_{emaxc/r}$ (kNm)	$M_{tmaxc/r}$ (kNm)	$\delta_{emaxc/r}$ (kNm)	$\frac{M_{rp}}{M_{emaxr}}$	$\frac{\delta_p}{\delta_{emaxr}}$	$\frac{\delta_{emaxc}}{\delta_{emaxr}}$	$\frac{M_{emaxr}}{M_{emaxc}}$	$\frac{M_{emaxc/r}}{M_{tmaxr/c}}$	FM
Group 1	CB1	-	-	-	31.03	25.47	12.79	-	-	1	-	1.22	FY
	CB2	-	-	-	31.61	25.47	12.49	-	-	1	-	1.24	FY
Group 2	PBS1-75	12.50	19.47	10.57	68.07	70.61	10.58	0.29	0.99	1.21	2.17	0.96	PED
	PBS2-100	16.67	20.00	13.25	77.55	83.75	9.92	0.26	1.34	1.29	2.48	0.93	PED
	PBS3-125	20.83	19.78	9.26	79.07	95.94	9.32	0.25	0.99	1.37	2.52	0.82	PED
	PBS4-150	25.00	12.21	7.00	79.13	107.17	8.97	0.15	0.78	1.42	2.53	0.74	D
	PBS5-175	29.17	14.41	15.76	71.86	117.46	8.39	0.20	1.88	1.52	2.28	0.61	D
Group 3	PBU1-75	12.50	26.68	6.95	65.24	70.61	9.80	0.41	0.71	1.31	2.08	0.92	PED + D
	PBU2-100	16.67	25.92	6.66	73.86	83.75	9.61	0.35	0.69	1.34	2.36	0.88	PED
	PBU3-125	20.83	26.49	6.21	77.55	95.94	11.74	0.34	0.53	1.09	2.48	0.81	PED +D
	PBU4-150	25.00	26.13	11.11	77.74	107.17	9.52	0.34	1.16	1.33	2.48	0.73	D
	PBU5-175	29.17	25.99	10.48	78.04	117.46	7.83	0.33	1.34	1.63	2.49	0.66	D

FM: Failure mode; PED: Plate-end debonding; D: Delamination

(a) (M_{emaxr}/M_{emaxc}) vs w/t ratio of plates **(b) (M_{emaxr}/M_{tmaxr}) vs w/t ratio of plates**

Figure 2: The relationship between moment capacity and the width-to-thickness ratio

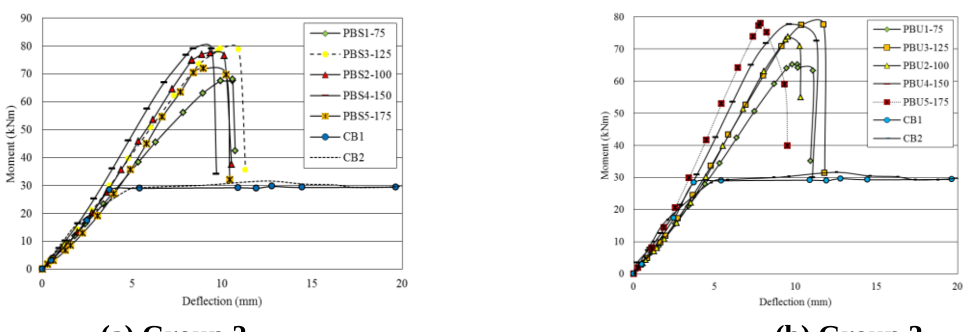
The repaired beams were able to achieve load capacities that are more than twice that of the control beams. Group 2 beams, PBS1-75 to PBS5-175, achieved a range of 117% to 153% increase in load capacity when compared to the control beams, whilst Group 3 beams, PBU1-75 up to PBU5-175, attained a range of 108% to 150% increase in load capacity when compared to the control beams. The repaired beams achieved a maximum strength that ranges from 2.17 to 2.53 and 2.08 to 2.49 as compared to the control beams, for beams in Group 2 and Group 3, respectively. Except for specimen PBS5-175, there is a generally increase in the capacity of both Group 2 and 3 beams with increase in the width-to-thickness ratios of the steel plates. It is clear that the beams that were pre-cracked at a lower load level (serviceability load) reached higher strengths than beams that were pre-cracked up to 85% of the ultimate load of the control beams. It can be concluded that the level of pre-cracking does affect the moment capacity of repaired beams. Although Group 3 beams were pre-cracked almost to failure, plating not only restored the beam’s original capacity, but doubled it.

In Figure 2(a), both graphs of Group 2 and 3 beams increased gradually from a width-to-thickness ratio of 12.5 to a peak of M_{emaxr}/M_{emaxc} = 2.55 for Group 2 beams and a peak of M_{emaxr}/M_{emaxc} 2.50 for Group 3 beams, after that the graphs started decreasing gradually. This shows that there exists a limit in the width-to-thickness ratio of steel plates for which the load can increase. For the same width-to-thickness ratio, Group 2 repaired beams (pre-cracked at lower load) show greater maximum moment capacity as compared to Group 3 repaired beams (pre-cracked at higher load) for the first half of graph but quickly sheds the strengths much more than Group 3 beams. This behaviour was influenced by the stoppage of the Instron midway through the tests.

The theoretical moments of resistance of the beams were calculated using EN 1992-1-1 (2004) by assuming a rectangular stress distribution. Table 1 shows that the control beams, CB1 and CB2, were able to reach their full predicted flexural capacity since the ratio of the experimental maximum moment to the code-predicted moment of resistance of the control beam (Memaxc/Mtmaxc) was more than unity. Group 2 and Group 3 beams did not reach the code-predicted flexural capacity, as they all failed prematurely by debonding of the bonded steel plate. This means that no full composite action was achieved. Figure 2 shows that the ratio (M_{emaxr}/M_{tmaxr}) decreased as the width-to-thickness ratio increased, implying that debonding was more dominant in the beams with larger width-to-thickness ratios. Some inelastic behaviour was found in beams repaired with steel plates of smaller width-to-thickness ratios, such as PBS1-75, PBS2-100 and PBU1-75. Such beams had a ratio of experimental moment of resistance to the code-predicted moment of resistance (Memaxc/r/ Mtmaxc/r) of just less than 1.0.

4.2 Moment deflection curves

Figure 3 shows the moment-deflection curves of the beams tested in this study. The control beams shown in Figure 3 demonstrate stiffnesses that are similar to the repaired beams, which were maintained up to a moment of 28kNm. After this moment, the control beams experienced a significant drop in stiffness. These control beams’ moment-deflection curves end with a horizontal line which indicates that the internal reinforcement was yielding. The high stiffness was maintained up to the maximum load in the repaired beams and this was caused by the increase in rigidity provided by the external bonded steel plate.



(a) Group 2 **(b) Group 3**

Figure 3: Moment-deflection curves of control and repaired beams

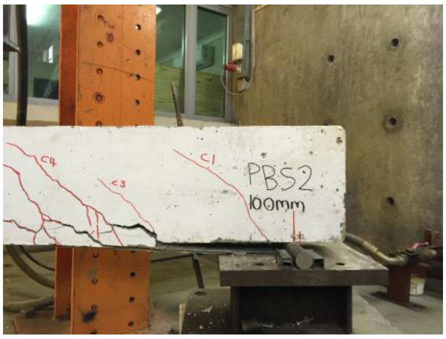
The steel plates provided additional stiffness for Group 2 and Group 3 beams to reach moment capacities that are 54% - 61% and 52% - 60% larger than the moments of the control specimens, respectively. This increase in the moment capacity is as a result of additional tensile forces and an increase in the lever arm provided by the external steel plate since it is bonded at an extreme position from the compression zone. The repaired beams show a reduction in deflections as compared to the control beams. The maximum experimental mid-span deflections was found to be 16% to 34% less than that of the control specimens for beams PBS1-75 to PBS5-175, respectively. The maximum experimental mid-span deflections for Group 3 repaired beams ranged from 7% to 38% less than that of the control specimens, for beams PBU1-75 to PBU5-175. The mid-span deflection is also observed to decrease as the width-to-thickness ratio of the steel plate increases, except for beam PBU3-125.

4.3 Failure modes

Four types of failure modes were observed in this experiment, as can be seen in Table 1. These are flexural yielding failure (FY), delamination (D) and plate-end debonding (PED) and a combination of PED and D. Both control beams, CB1 and CB2, failed by flexural yielding. These beams had flexural cracks only on the constant moment region. The cracks widened as the loading was increased, causing a loss in stiffness and strength of the beam. The beams failed by yielding of the tensile steel reinforcement and crushing of the concrete in the compression zone.

Two modes of failure were observed in the repaired beams, namely; plate-end debonding and delamination. Small diagonal shear cracks were observed, just before plate-end debonding occurred, due to interfacial shear and normal stresses at the plate-end curtailment (Oehlers (1992)). These cracks caused the end of the plate to separate from the concrete. As the plate separation propagated towards the mid-span, it transformed into shear diagonal cracks, which extended towards the loading point, at approximately 45º.

Similar to plate-end debonding, delamination started as a small diagonal crack at the plate-end curtailment, as a result of a change in stresses between the steel plate and concrete. In delamination, the crack extended to the tensile reinforcement. As the loading was increased, the crack propagated along the reinforcing bars towards the mid-span. Since the shear and bending stresses are high at the loading point, this forced the crack to change direction and to propagate at about 60º towards this point. Table 1 shows that, beams with a smaller width-to-thickness ratio failed by plate-end debonding and those with a larger width-to-thickness ratio failed by delamination. This means that the mode of failure was also dependent on the amount of external reinforcement used.



(a) PED (PBS2-100)

(b) Delamination (PBS1-150)

Figure 4: Plate-end debonding and delamination of the repaired beams

5. CONCLUSION

The maximum capacity of the beams, moment-deflection curves and failure modes were analysed and the following conclusions are drawn. The repaired beams were able to achieve load capacities that are more than twice that of the control beams. Except for specimen PBS5-175, there is generally increase in the capacity of the repaired beams with increase in the width-to-thickness ratio of the steel plates.

Beams that were pre-cracked at a lower load level (serviceability load) reached higher strengths than the beams that were pre-cracked at a higher load level (85% of the ultimate load capacity). The experimental results showed that there exists a limit in the width-to-thickness ratio of the steel plate for which the load can increase. The repaired beams showed high stiffnesses when compared to the control beams, due to the increase in rigidity provided by the external bonded steel plate. The rigidity of the repaired beams increased with increasing the width-to-thickness ratio of the steel plate. The use of external reinforcement reduced the maximum deflections of the beams and except for beam PBU3-125, this maximum deflections of the repaired beams decreased with increasing the steel plate width-to-thickness ratio. The repaired beams of smaller width-to-thickness ratio failed by plate-end debonding whereas those of larger width to thickness ratio failed by delamination.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

Basunbul IA, Gubati AA, A-L-Sulaimani GJ, Baluch MH (1990a). Repaired reinforced concrete beams, ACI Materials Journal, 87(37), 348-354.

Basunbul IA, Hussain M, Sharif AM, Al-Sulaimani G, Baluch MH (1990b). Repair of flexural cracked RC beams with bonded external steel plate, KFUPM Saudi Arabia, 81-92.

EN 1992-1-1 (2004). Eurocode 2. Design of concrete structures-Part 1-1: General rules and rules for buildings, The European Union.

Hermite RL, Bresson J (1967). Concrete reinforced with glued plates, RILEM International Symposium, Synthetic resins in building construction, Paris, 175 – 203.

Hussain M, Sharif A, Basunbul IA, Baluch MH, Al-Sulaimani GJ. (1995). Flexural behavior of precracked reinforced concrete beams strengthened externally by steel plates, ACI Structural Journal, 92(1), 14 -22.

MacDonald MD (1978). The flexural behaviour of concrete beams with bonded external reinforcement, TRRL Supplementary Report 415, Transport and Road Research laboratory, Department of the environment, Crowthone, 13.

Oehlers DJ (1992). Reinforced concrete beams with plates glued to their soffits, Journal of Structural Engineering, 118(8), 2023-2038.

Olajumoke AM, Dundu M (2015). Impact of adhesive thickness on the capacity of the surface mounting steel plates strengthening technique, International conference on structural and geotechnical engineering, December 20-22, Cairo, Egypt, ICSGE 14.

SANS 5863 (2006). Concrete tests-Compressive strength of hardened concrete, Pretoria, South African Bureau of Standard Division.

SANS 6892-1 (2010). Metallic materials-Tensile testing, Part 1: Method of test at room temperature. Pretoria, South African Bureau of Standard Division.

Swamy RN, Jones R, Bloxham JW (1987). Structural behaviour of reinforced concrete beams strengthened by epoxy-bonded steel plates, Structural Engineer, 65(2), 59 -68.

Swamy RN, Jones R, Charif A (1989). The Effect of External Plate Reinforcement on the Strengthening of Structurally Damaged R.C. Beams, Structural Engineer, 67(3), pp45- 54.