STRENGTH AND DUCTILITY OF SIMPLE SUPPORTED R/C BEAMS RETROFITTED WITH STEEL PLATES OF DIFFERENT WIDTH-TO-THICKNESS RATIOS

S M Rakgate and M. Dundu
University of Johannesburg, Department of Civil Engineering Science,
P O Box 524, Auckland Park, 2006, South Africa
Emails: srakgate@uj.ac.za and morgandundu@gmail.com

Abstract: The concept of strengthening reinforced concrete beams using epoxy-bonded steel plates (EBSP) is a well-known solution in structural engineering, however, there is little information about the effect of the width-to-thickness ratio of steel plates on the behaviour of steel-concrete composite beams. This paper presents an experimental study of the flexural behaviour of under-reinforced concrete beams, strengthened in flexure by externally-bonded steel plates (EBSP) of varying width-to-thickness ratios. A total of 23 reinforced concrete beams were tested; 6 beams in Series 1 tests and 17 beams in Series 2 tests. One beam in Series 1 tests and two beams in Series 2 tests were regarded as control specimens, whilst the remaining beams were strengthened with steel plates of different width-to-thickness ratios. In each group, the width of the bonded steel plate varied from 75 mm to 175 mm, in increments of 25 mm. The beams were tested as simply supported, under two-point static loadings until failure. From the experimental results, it was observed that the externally bonded steel plates led to substantial increase in flexural stiffness, which resulted in an increase in the capacity and cracking load of the strengthened beams and a decrease in vertical deflections and crack-widths, compared to the control beams. It was also found that the width-to-thickness ratio of steel plates as low as 12.5 can promote flexural yielding and extensive ductility in strengthened beams.

Keywords: Steel plates, width-to-thickness ratio, reinforced concrete beams, epoxy-resin externally strengthened, composite beams.

1.0 INTRODUCTION

The need for strengthening beam elements in reinforced concrete structures arises, when the capacity 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. The former is usually caused by design errors, inadequate detailing, construction faults, usage of inferior materials during construction and loss of capacity due to corrosion or other types of degradation caused by aging, or a combination of these factors. Although there are several methods for strengthening reinforced concrete beams, strengthening of reinforced concrete beams using epoxy-bonded mild steel plates on the tension face has been proven to be the most effective, efficient, economical and convenient technique to enhance the flexural and shear performance of reinforced concrete beams under service and ultimate loads [1-17]. 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 [5], does not significantly change the overall dimensions of the structure, and can be secured without causing any damage to the structure [8]. This technique has been applied successfully to strengthen reinforced concrete structures such as buildings and bridges in various parts of the world, including South Africa, France, Switzerland, Japan, Poland, Belgium and United Kingdom [4]. Although, fibre reinforced polymers (FRP) plates are preferred in other parts of the world, because of their superior strength-to-weight ratio and corrosion resistance, they are very expensive and not readily available in South Africa, and the rest of Africa. Excluding import costs, the cost of FRP can be 10 times as much as that of steel plates [18, 19]. In addition, the use of FRP poses the increased possibility of brittle failure modes.

The technique was pioneered by Fleming and King in South Africa [1] and L'Hermite and Bresson in France [2], and since then many studies have been conducted to fully understand the structural behaviour of reinforced concrete beams strengthened by externally bonded steel plate on their soffit [3-17]. However, there is limited work about the effect of the width-to-thickness ratio of steel plates on the behaviour of steel-concrete composite beams. L'Hermite and Bresson [2], Macdonald [3], Bloxham [4], Jones et al. [6], Huovinen [13], Neelamegan et al. [15] and Oh et al. [16] recognised that the width-to-thickness ratio of steel plates could have an influence on the premature failure of strengthened beams. L'Hermite and Bresson [2] tested strengthened beams with mild steel plates of width-to-thickness ratio of 24 only, and suggested that, had the width-to-thickness (w/t) ratio of the steel plate been larger, full composite action might have been possible. Several years after L'Hermite and Bresson's work, Macdonald [3] concluded that the reinforced concrete beams should be strengthened with steel plate having width-to-thickness ratio of not less than 22 in order to obtain composite action.

To ensure that full composite action is achieved, Bloxham [4] suggested that the width-to-thickness ratio of the steel plate should not be less than 50 and the neutral axis depth should not be more than 0.4 times the effective depth. Jones et al [6] tested one (1) control beam, and four (4) under-reinforced concrete beams with an adhesive thickness of 3 mm, and compressive strength, tensile strength and modulus of elasticity of 44 MPa, 5.3 MPa, and 6 GPa, respectively. From this work, it can be concluded that the width-to-thickness ratio of the steel plates should range from 26.67 to 53.33 if full composite action is to be accomplished. Huovinen [13] investigated the bond strength of glued steel plates of 6 strengthened beams, with two types of adhesive resin (Epoxy BI-R glue and Concressive 1380 glue) for each steel plate, and concluded that the width-to-thickness ratio of plates should not be less than 20 for flexural yielding of the strengthened beams to occur. In a wider investigation, Neelamegam et al [15] tested nine (9) strengthened beams with varying length bonded plates of 800 – 2200 mm long. The strengthened beams with steel plates of at least 43.5 width-to-thickness ratio maintained composite action until failure and failed by flexural yielding. Lastly, in an investigation performed by Oh et al [15], beams with width-to-thickness ratios of 37.5, 50 and 70, and shear span –to–depth (a_v/d) of 4.77 failed by yielding of the external plate followed by plate separation.

The brief review above clearly shows that there is a lack of comprehensive information about the effect of the width-to-thickness ratio of steel plates on composite beams. Added to this, there is huge inconsistency about the width-to-thickness ratio of the steel plate that should be used to promote yielding and ductility of the composite beams. The objectives of this study are to determine the effect of the width-to-thickness ratio of steel plates on the flexural capacity, deflections and flexural stiffness of beams, and to evaluate the width-to-thickness ratio of steel plates that encourage yielding of the composite beams. In addition, the experimental results are compared with the theoretical results predicted using EN 1992-1-1 [20].

2.0 MATERIAL PROPERTIES

A detailed evaluation of the capacity of strengthened beams depends heavily on the reliability of the properties of the materials used. This section describes the tensile and compressive test procedures to determine the material properties of the reinforcement bars, steel plates and concrete cubes.

2.1 Reinforcement bars and steel plates

The material properties of the reinforcement bars (6 mm, and 8 mm and 12 mm) and steel plates (4 mm, 6mm and 8 mm) were established from the tensile coupon tests. Mild steel was used for the 6mm shear links (R) in Series 2, and high yield strength (Y) deformed steel was used for the main reinforcement in all beams and as well as the shear links in Series 1. Reinforcement diameter bars and steel plates (SP) were cut using a metal cutter, and the steel plates were prepared into coupons using a Topaz steel mill machine, in accordance with EN ISO 6892-1 [21]. The reinforcement bars were cut to a length of 250 mm, and each bar was tested in their original diameters of 6 mm, 8 mm and 12 mm.

Before testing, the specimens were firmly gripped at both ends to prevent slippage. After all the necessary preparations were done, the specimens were tested using a 100 kN capacity displacement controlled Instron 1195 tensile testing machine. A calibrated extensometer, with a gauge length of 50 mm, was used to measure the axial elongation of the specimens during tensile testing. The specimens were pulled at a rate of 3 mm per minute and the stress-strain relationship for each steel coupon was derived from the load-strain relationship, using the original cross-sectional area and gauge length of 50 mm. The 0.2 % proof yield stress (f_y), yield strain (ε_y), ultimate stress (f_u), ultimate strain (ε_u) and the Young's Modulus of Elasticity (E_s) were determined from the stress-strain graphs, and are tabulated in Table 1.

2.2 Concrete

For both series of tests, a total of 5, 100 x100 x 100 mm concrete cubes each were cast from the 30 MPa ready mixed concrete mixture, donated by AfriSam. The cubes were cured by placing them inside a curing bath under a controlled temperature for 28 days, as per standard practice. After 28 days, all the cubes were tested to establish their compressive strength, in accordance with provision provided in SANS 5863 [22]. The compression load was applied at a rate of 0.3 MPa/s until failure. Table 1 shows the average compressive strength of the concrete cubes at 28 days (f'_{cu}), and the equivalent cylindrical strength at 28 days (f_{ck}), used in the code-prediction analysis.

Table 1: Material properties of reinforcement bars, steel plates and concrete

Series	Specimen	f_y (MPa)	ε_y (%)	f_u (MPa)	ε_u (%)	E_s (GPa)	f_{cu} (MPa)	f_{ck} (MPa)
Series 1	Y10-1	442.77	0.417	648.59	16.300	203.07	30.0	24.0
	Y10-2	448.95	0.419	652.64	16.200	204.07		
	Y10-3	442.90	0.448	649.92	15.800	198.34		
	Average	**444.88**	**0.428**	**650.38**	**16.100**	**201.83**		
	SP6-1	344.59	0.189	421.28	18.700	204.28		
	SP6-2	353.17	0.153	423.57	19.100	202.50		
	Average	**348.88**	**0.171**	**422.43**	**18.900**	**203.39**		
Series 2	R6-1	380.95	0.425	421.46	13.311	202.30	31.5	25.5
	R6-2	380.60	0.406	428.60	16.055	200.80		
	Average	**380.78**	**0.416**	**425.03**	**14.683**	**201.50**		
	Y8-1	353.64	0.431	386.14	3.964	201.00		
	Y8-2	356.32	0.418	387.83	5.515	200.00		
	Average	**354.98**	**0.425**	**386.98**	**4.740**	**200.50**		
	Y12-1	442.80	0.468	596.97	20.244	200.30		
	Y12-2	450.38	0.437	596.00	17.828	200.50		
	Average	**446.59**	**0.453**	**596.53**	**19.036**	**200.40**		
	SP4-1	421.34	0.335	536.73	15.006	200.90		
	SP4-2	421.44	0.380	541.11	14.946	200.10		
	SP4-3	421.48	0.385	537.21	14.940	205.20		
	Average	**421.42**	**0.367**	**538.35**	**14.964**	**202.40**		
	SP6-1	398.63	0.418	529.45	18.031	201.40		
	SP6-2	397.52	0.407	528.10	18.943	204.00		
	SP6-3	398.68	0.408	529.31	18.930	203.50		
	Average	**398.28**	**0.411**	**528.95**	**18.635**	**203.00**		
	SP8-1	326.39	0.378	426.55	17.689	201.80		
	SP8-2	326.54	0.327	429.72	18.619	200.00		
	SP8-3	321.93	0.369	426.84	18.381	204.00		
	Average	**324.95**	**0.358**	**427.70**	**18.229**	**201.90**		

3.0 PREPARATION AND TESTING PROGRAMME

3.1 Tested beams

The first phase of the experimental programme focused on the preparation of the beams. A total of 23 reinforced concrete beams were cast in rectangular wooden shutters using ready-mixed concrete. As concrete was poured, it was compacted thoroughly using a needle poker vibrator. Adequate curing of the reinforced concrete beams was achieved by spraying the beams with water, at least twice a day and then covering them with plastic sheets. As shown in Table 2, the specimens were divided into two series, viz; Series 1 (S1) and Series 2 (S2). Series 1 consisted of a total of 6, 175x250x3000mm reinforced concrete beams; one of the tested beams was used as a control beam (S1-C1), that is, it was not strengthened, while the remaining 5 beams were externally strengthened with steel plates. In this series, all the beams were strengthened with 6mm thick plates (S1-6-75 to S1-6-175), with varying width of the bonded steel plates of 75 - 175 mm, in increments of 25 mm. The beams were internally reinforced with 2, 10 mm tension bars (TBs), placed at an effective depth of 205 mm and 2, 10 mm compression bars (CBs), placed at a distance of 45 mm from the top surface of the beam. Ten (10 mm) mild stirrups were placed at a spacing of 195 mm centre-to-centre, over the whole length of the beam, to prevent the beams from failing in shear. To promote yielding and reduce the chances of debonding failure, the gap between plate end and support was reduced to 15mm.

In Series 2, the 175x300x3200 beams were split into four groups; the first group of five beams were strengthened with 4 mm steel plates (S2-4-75 to S2-4-175), the second group of five beams were strengthened with 6 mm steel plates (S2-6-75 to S2-6-175), the third group of five beams were strengthened with 8 mm steel plates (S2-8-75 to S2-8-175), and the remaining two beams (S2-CB1 and S2-CB2) were not strengthened with steel plates, as they were used as control specimens. As in Series 1, the width of the bonded steel plates varied from 75 mm to 175 mm, in increments of 25 mm. Two (2), 8 mm and 12 mm high yield deformed diameter bars, were used as compression and tension reinforcement, respectively. The tension bars were placed at an effective depth of 263 mm and compression bars, placed at a distance of 29 mm from the top surface of the beam. Vertical shear was resisted by plain mild steel shear links of 6 mm in diameter. Figure 1 shows the longitudinal and cross section of the control beam in Series 2. In both series the beams were under reinforced so that they can fail in flexure only. In Series 2, the gap between the end of the plate and support was chosen to be 50 mm. Table 2 also shows the span-to-depth (s/d) and the shear span-to-depth (a_v/d) ratios. For each series, these were kept constant. After curing the beams for period of 28 days, the beams in Series 2 were painted white so that the initiation and development of the cracks would be more visible during the tests. In order to examine the effect of the width-to-thickness ratio (w/t) of the steel plates on the beam specimens only, the size of the beams, amount of reinforcement bars and the loading arrangement for each series were kept constant.

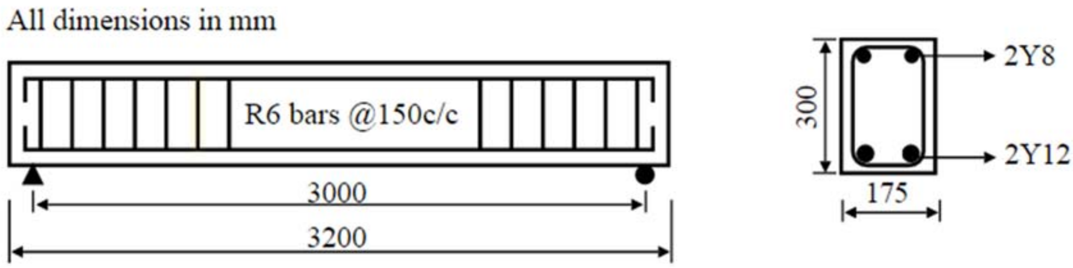


Figure 1: Longitudinal and cross section of the control beam in Series 2

Table 2: Details of the tested beams

Beam size	Specimen	$\frac{s}{d}$	$\frac{a_v}{d}$	Steel plates			Reinforcement bars		
				Plate size	w/t	Gap	TBs	CBs	Shear links
175x250x3000	S1-C1	11.6	3.87	-	-	-	2Y10	2Y10	R10
	S1-6-75	11.6	3.87	75x6x2800	12.50	15	2Y10	2Y10	R10
	S1-6-100	11.6	3.87	100x6x2800	16.67	15	2Y10	2Y10	R10
	S1-6-125	11.6	3.87	125x6x2800	20.83	15	2Y10	2Y10	R10
	S1-6-150	11.6	3.87	150x6x2800	25.00	15	2Y10	2Y10	R10
	S1-6-175	11.6	3.87	175x6x2800	29.17	15	2Y10	2Y10	R10
175x300x3200	S2-C1	10.0	3.33	-	-	-	2Y12	2Y8	R6
	S2-C2	10.0	3.33	-	-	-	2Y12	2Y8	R6
	S2-4-75	10.0	3.33	75x4x3000	18.75	50	2Y12	2Y8	R6
	S2-4-100	10.0	3.33	100x4x3000	25.00	50	2Y12	2Y8	R6
	S2-4-125	10.0	3.33	125x4x3000	31.25	50	2Y12	2Y8	R6
	S2-4-150	10.0	3.33	150x4x3000	37.50	50	2Y12	2Y8	R6
	S2-4-175	10.0	3.33	175x4x3000	43.75	50	2Y12	2Y8	R6
	S2-6-75	10.0	3.33	75x6x3000	12.50	50	2Y12	2Y8	R6
	S2-6-100	10.0	3.33	100x6x3000	16.67	50	2Y12	2Y8	R6
	S2-6-125	10.0	3.33	125x6x3000	20.83	50	2Y12	2Y8	R6
	S2-6-150	10.0	3.33	150x6x3000	25.00	50	2Y12	2Y8	R6
	S2-6-175	10.0	3.33	175x6x3000	29.17	50	2Y12	2Y8	R6
	S2-8-75	10.0	3.33	75x8x3000	9.38	50	2Y12	2Y8	R6
	S2-8-100	10.0	3.33	100x8x3000	12.50	50	2Y12	2Y8	R6
	S2-8-125	10.0	3.33	125x8x3000	15.63	50	2Y12	2Y8	R6
	S2-8-150	10.0	3.33	150x8x3000	18.75	50	2Y12	2Y8	R6
	S2-6-175	10.0	3.33	175x8x3000	21.88	50	2Y12	2Y8	R6

3.2 Surface Preparation and bonding of steel plates

The effectiveness of the epoxy-bonded steel plate technique is largely depended on the surface preparation of the reinforced concrete beams and steel plates [1, 4, 8, 13, 23]. A clean, dry and well prepared surface produces a suitable mechanical key necessary for the development of a strong bond. In order to achieve a strong bond, the tension surface of the reinforced concrete beams was scabbled using a multiple steel pin air vibrator, as shown in Figure 2, so as to expose the coarse aggregates, and thus increase the frictional grip between the reinforced concrete beam and the epoxy coated steel plate. The dust and debris left on the scabbled surfaces were then removed using compressed air. Similarly, the mild steel plates were sandblasted to 50μm in order to remove the oxide layer and thus improve the contact surface area with the epoxy adhesive. Sandblasting allows the epoxy to soak into the rough surface, and strengthen its grip on the surface thereby reducing the chance of debonding at failure [24]. Prior to bonding, the steel plates were cleaned with acetone so as to remove dust and other oily material.

(a) Scabbling process

b) Scabbled surface

Figure 2: Surface preparation of beams

There are several different types of epoxy resin available commercially, with extensive range of mechanical properties. The epoxy resin that was used in this experiment was supplied by StonCor Africa and consists of two parts, namely; primer adhesive (Pro-Struct 618LV) and an epoxy adhesive (Pro-Struct 617NS). According to the manufacturer, the shear strength, compressive strength, tensile strength and modulus of elasticity of the epoxy adhesive are at least 14MPa, 75 MPa, 55 MPa, and 1500 MPa, respectively, after 3 days of curing. Since the tensile strength of the epoxy is much larger than the strengths of the concrete, it is expected that failure will occur in the concrete. After the contact surfaces of both the concrete beams are thoroughly prepared, a two part low viscous epoxy-resin primer (Pro-Struct 618 LV) was mixed thoroughly in the ratio of 2:1 and applied to the soffit of the concrete beams. A 10-15 minutes period was allowed for the primer to penetrate through the small holes and thin surface cracks of the beam. Subsequently, a two part non-sag epoxy adhesive (Pro-Struct 617 NS) was mixed thoroughly in the ratio 1.1 until a uniform grey-coloured mixture was observed. Immediately after this process, the epoxy resin adhesive was applied to the sandblasted surface of the steel plate. An average glue thickness of 1.5 mm was maintained by bonding small crushed glasses of 1.5 mm thickness along the length of the prepared steel plate [25]. It has been proven by Olajumoke and Dundu [25] that an adhesive thickness of 1.5 mm performs better than thicker adhesives. To complete the process, the steel plate was finally pressed on the scabbled surface, using hydraulic jacks. The epoxy adhesive was allowed to cure for a period of 7 days as specified by the manufacture to ensure an effective bond.

3.3 Instrumentation and test procedure

All beams were tested under two-point static loading over simply supported spans to simulate a distributed load, as shown in the Figure 3, with equal point loads at a third of the effective length of the beam from each support. Simply supports were achieved using 30mm diameter solid steel bars. These round bars were maneuvered until the required gap between the end of the plate and support was attained. In order to record as much information as possible, the beams were extensively instrumented. Data recorded included the central deflections, steel strains and ultimate load. The mid-span deflections of all beams were measured using a single linear variable displacement transducers (LVDT) and mid-span strains in the external reinforcement were measured using an electrical strain gauges. The concrete strains were also measured at the mid-spans, along the depth of the concrete beam, at 50 mm from the edge of the beam. The information from the strain gauges and LVDT were captured using a data logger. All beams in Series 2 were painted white so as to visually observe the appearance of the first crack clearly and the corresponding load at which the crack appears, including crack development and propagation/patterns. The crack widths at the flexural zone were measured at failure, using an elcometer 143 crack-width ruler of 0.05 mm precision. A 500kN Instron testing machine was used to apply the load, at a constant rate of 2 mm/min, and testing was discontinued when the beam showed a drop in the capacity.

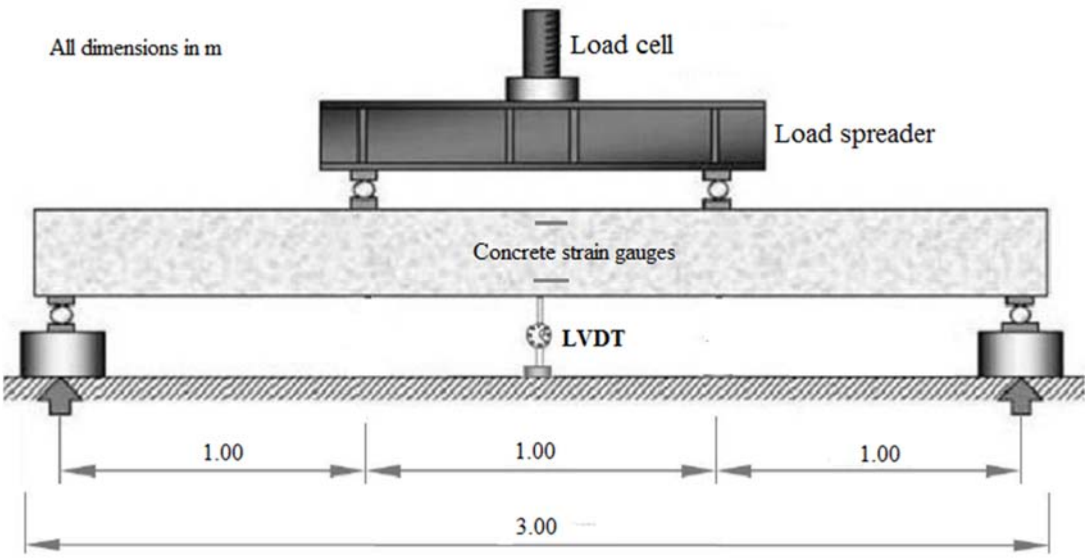


Figure 3: Typical set-up

4.0 EXPERIMENTAL AND THEORETICAL RESULST OF TESTED BEAMS

In order to discuss the experimental and theoretical results thoroughly, a decision was taken to split this section into failure modes, experimental and theoretical strengths results, moment-deflection response, moment-steel strain and beam depth-strain relationships.

4.1 Failure modes

During testing it was observed that the control beams in the two series tested, failed by flexure in the zone of high bending moment, that is, between the two applied point loads, after the formation of widely spaced flexural or vertical cracks. The cracks originated from the tension zone and propagated towards the compression zone of the beam. Loss of strength in these beams was eventually caused by yielding of the tension steel reinforcement and crushing of the concrete in the compression zone, as shown Figure 4(a). Likewise, specimens with steel plates of small width-to-thickness ratios, such as S1-6-75 and S1-6-100 in Series 1 tests, and S2-4-75 and S2-4-125 in Series 2 tests, failed in flexure by crushing of the concrete and yielding of the steel plate, after extensive plastic deformation in the zone of high moment. The vertical cracks in Figure 4(b) illustrates that the final failure was in actual fact flexural yielding. This means that the stress in the adhesive layer was strong enough to allow the plate to experience a stress of over 300MPa.

(a) Yielding and crushing of control beams (b) Yielding and crushing of strengthened beams

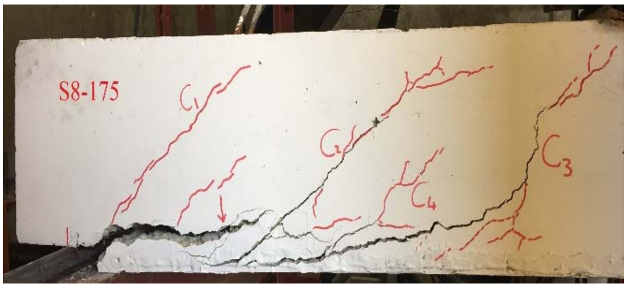
Figure 4: Yielding of the reinforcement bars/steel plates and crushing concrete

In Series 1, when the width-to-thickness ratio exceeded 20, as in specimen S1-6-125, the mode of failure shifted from full flexural yielding to premature failure by plate-end debonding or plate seperation. This mode of failure was initiated by diagonal shear cracks, in the zone of high interfacial normal and shear stresses, at the end of the plate [16, 26]. Since plate-end debonding or plate separation usually leads to the peeling off of the plate, with little or no concrete, it is associated with inadequate surface preparation of the concrete. As evidence that the contact surfaces of the concrete and steel were well prepared, plate-end debonding did not result in simple separation of the concrete and steel. It can be observed in Figure 5 (a) that as the plate separation propagated towards the mid-span, it changed into a diagonal crack, which extended towards the loading point. The latter failure mechanism is called the critical diagonal crack (CDC) debonding and usually occurs after the formation of a large crack, which may be due to insufficient shear reinforcement [26, 27]. Specimen S1-6-150 and S1-6-175 failed by shear and no debonding of the plate or cracks were noticed between the plate ends. Shear failure was initiated at the support, propagated at 55˚to the top reinforcement, and final extended along the top reinforcement. The crack is clearly illustrated in Figure 5 (b), and was caused by the low shear resistance of the links, compared to the larger stiffness provided by the steel plates.

(a) Plate-end debonding (b) Shear failure

Figure 5: Plate-end debonding and shear failure

A significant number of strengthened beams in Series 2 failed by delamination or separation of both the steel plates and concrete cover of the beam, as shown in Figure 6. Delamination originated as a small diagonal crack at the end of the plate, which then extended to the reinforcing bars, and propagated towards the mid-span of the beam. This failure mechanism usually occurs if the epoxy glue is strong enough to prevent the plate from separating from the concrete, leading to a rip-off of the concrete cover [11, 28]. A possible cause of this crack is the abrupt change in stresses from the steel plate to the concrete. It should be noted that before delamination occurred, small diagonal shear cracks developed close to the support, at an angle of about 45º. The presence of reinforcing steel bars forced the crack to propagate along the bars, towards the mid-span. Finally, the crack changed direction and propagated at about 50º, towards the loading point, due to the influence of the high shear and bending stresses (biaxial stress) at this point. Since the crack is more than 45º (typical of unplated beams), this leads to reduced shear capacity. A possible weakness in these tests is the large spacing between the support and the end of the plate, which was fixed at 50mm. In the future tests, this spacing should be reduced as much as possible. The failure modes of the control and strengthened specimen are also presented in Table 3.



(a) S2-8-100

(b) S-8-175

Figure 6: Delamination of beams strengthened with 8mm plates

Three variables were used to examine the cracking behaviour of the beams, namely; crack width, crack spacing and crack height. In all beams the average crack-spacing of the control beams of about 186 mm was found to be much larger than the crack spacing of the strengthened beams, which ranged from 123.4 - 180.4 mm. Further, the strengthened beams had a smaller crack width compared to the crack width of the control beams at failure of 6mm. The maximum experimental crack width for beams strengthened with 4 mm thick steel plate ranged from 0.3 - 2.0 mm, whereas the experimental crack width for beams strengthened with 6 mm and 8 mm thick steel plate, ranged from 0.35 - 0.40mm and 0.20 - 0.55 mm, respectively. The increased stiffness offered by the bonded steel plate delayed the formation of cracks. As for the measured average crack height, the control beams exhibited an average crack height of 285 mm at failure, which is almost the full depth of the beam of 300 mm. All beams strengthened with steel plate exhibited a reduction in average crack height, and this can be attributed to an increase in stiffness offered by the steel plate. The beams that were strengthened with 4 mm steel plates had crack heights ranging from 177 to 218.6 mm, beams that were strengthened with 6 mm steel plates had crack heights ranging from 177.2 to 197.5 mm and those that were strengthened with 8 mm steel plates had crack heights ranging from 84.5 to 180.4 mm. This means that larger bonded steel plates reduced the average crack height more than smaller bonded plates.

4.2 Experimental and theoretical strengths results

The test and code-predicted results are given in Table 3. In this table, $P_{ecrc/s}$ is the first experimental crack load of the control/strengthened beam, $P_{eserc/s}$ is the experimental serviceability load, $P_{emaxc/s}$ is the maximum experimental load of the control/strengthened beam, $M_{emaxc/s}$ is the maximum experimental moment, and $M_{tmaxc/s}$ is the maximum theoretical moment of resistance. According to Bloxham [4], the serviceability load of the reinforced concrete beam ranges from 60 to 65% of the ultimate load. In this study, the serviceability load of the reinforced concrete beams is taken as 62.5% of the ultimate load, which is the average of the minimum and maximum serviceability loads, suggested by Bloxham [4]. 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}), the experimental first crack load to the maximum experimental load of the control and strengthened beams ($P_{ecrc/s}/P_{emaxc/s}$), the maximum experimental moment of the strengthened beam to the maximum experimental moment of the control beam (M_{emaxs}/M_{emaxc}) and the maximum experimental moment of the control and strengthened beams to the code-predicted moment of resistance ($M_{emaxc/s}/ M_{tmaxc/s}$).

Table 3: Experimental and theoretical strength results

Beam	w/t	f_y	$P_{ecrc/s}$ (kN)	$P_{eserc/s}$ (kN)	$P_{emaxc/s}$ (kN)	$M_{emaxc/s}$ (kNm)	$M_{tmaxc/s}$ (kNm)	$\frac{P_{ecrs}}{P_{ecrc}}$	$\frac{P_{ecrc/s}}{P_{emaxc/s}}$	$\frac{M_{emaxs}}{M_{emaxc}}$	$\frac{M_{emaxc/s}}{M_{tmaxc/s}}$	Failure Mode
S1-C1	-	-	-	26.18	41.89	19.62	13.64	-	-	-	1.44	FY
S1-6-75	12.50	348.88	-	62.94	100.70	48.04	46.84	-	-	2.45	1.03	FY
S1-6-100	16.67	348.88	-	78.64	125.82	59.51	56.37	-	-	3.03	1.06	FY
S1-6-125	20.83	348.88	-	85.27	136.44	65.53	65.13	-	-	3.34	1.01	FY+PED
S1-6-150	25.00	348.88	-	94.91	151.86	71.59	73.13	-	-	3.65	0.98	FY+SF
S1-6-175	29.17	348.88	-	77.68	124.28	58.97	80.36	-	-	3.01	0.73	SF
S2-C1	-	-	23.56	38.81	62.09	31.05	22.72	-	0.38	-	1.37	FY
S2-C2	-	-	24.25	38.95	63.21	31.61	22.72	-	0.38	-	1.39	FY
S2-4-75	18.75	421.42	25.08	74.48	127.49	63.75	56.22	1.05	0.20	2.04	1.13	FY
S2-4-100	25.00	421.42	28.59	84.78	134.06	67.03	65.74	1.20	0.21	2.14	1.02	FY+D
S2-4-125	31.25	421.42	37.66	82.52	137.73	68.87	74.83	1.58	0.27	2.20	0.92	FY+D
S2-4-150	37.50	421.42	41.96	98.06	151.97	75.99	83.49	1.76	0.28	2.43	0.91	FY+D
S2-4-175	43.75	421.42	53.57	93.54	159.30	79.65	91.73	2.24	0.34	2.54	0.87	D
S2-6-75	12.50	398.28	46.72	79.43	129.04	64.52	70.31	1.95	0.36	2.06	0.92	FY+D
S2-6-100	16.67	398.28	43.97	87.87	142.75	71.38	83.47	1.84	0.31	2.28	0.86	D
S2-6-125	20.83	398.28	76.47	86.50	144.14	72.07	95.69	3.20	0.53	2.30	0.77	D
S2-6-150	25.00	398.28	86.16	91.80	149.13	74.57	106.96	3.60	0.58	2.38	0.70	D
S2-6-175	29.17	398.28	95.61	88.72	159.18	79.59	117.27	4.00	0.60	2.50	0.68	D
S2-8-75	9.38	324.95	52.25	70.45	114.45	57.23	74.07	2.19	0.46	1.83	0.77	SDC+D
S2-8-100	12.50	324.95	73.65	87.26	141.76	70.88	88.14	3.08	0.52	2.26	0.80	SDC+D
S2-8-125	15.63	324.95	76.47	97.98	159.18	79.59	101.08	3.20	0.48	2.54	0.79	D
S2-8-150	18.75	324.95	98.13	100.74	163.41	81.71	112.90	4.10	0.60	2.61	0.72	D
S2-8-175	21.88	324.95	98.13	105.61	171.57	85.79	123.59	4.10	0.57	2.74	0.69	D

FY – Flexural yielding; PED – Plate-end debonding; SF – Shear failure; D – Delamination; SDC = Shear diagonal cracks

Before analysing the strengths results, it should be noted that the first experimental crack load of the control/strengthened beam in Series 1 tests was not recorded. In Series 2 tests, the appearance of the first crack was observed visually, and the corresponding crack load recorded. For the control beams (S2-C1 and S2-C2), the first crack load appeared at an average load of 23.91kN. From Table 3, it can be observed that there was generally a significant increase in the experimental first crack loads of strengthened beams, compared to the control beam (P_{ecrs}/P_{ecrc}). All strengthened beams showed a delay in the appearance of the first crack as compared to the control beam. The increase of the cracking load of strengthened beams varied from 5% to 124% for S2-4-75 to S2-4-175 beams, 84% to 300% for S2-6-75 to S2-6-175 beams, and 119% to 310% for S2-8-75 to S2-8-175 beams, compared to the control beams. The relationships between the increase in capacity of the strengthened beams and the width-to-thickness ratio at first crack load are shown in Figure 7. It is clear from Table 3 and Figure 7 that the width-to-thickness ratio of the plates influenced the initiation of the first crack in all the beams tested. Beams strengthened with larger steel plates achieved larger crack loads. Figure 7 also shows that the ratio of the experimental first crack loads versus the width-to-thickness ratio are steeper for beams with plates of smaller yield strength and width-to-thickness ratio or larger thickness.

The positive impact of this strengthening technique was also noticed at serviceability and maximum capacity of the beams, where the capacity of the strengthened beams increased from 104% to 154% for S2-4-75 to S2-4-175 beams, 106% to 150% for S2-6-75 to S2-6-175 beams, and 83% to 174% for S2-8-75 to S2-8-175 beams, compared to the control beams. The relationships between the increase in maximum capacity of the strengthened beams and the width-to-thickness ratios are shown in Figure 8. Similarly to the behaviour of the beams at the first crack load, the increase of the maximum capacity of the strengthened beams versus the maximum capacity of the control beams for each plate thickness increases as the width-to-thickness ratio increases, however, the gradients are less steeper at maximum capacity than at first crack load level. Possible causes of a lower increase in strength include a lower yield strength and failure mechanism. Except for a few samples, the ratio of the first crack load versus the maximum capacity of the strengthened beams of 0.20 – 0.60 increased with an increase of the width-to-thickness ratio of the steel plates.

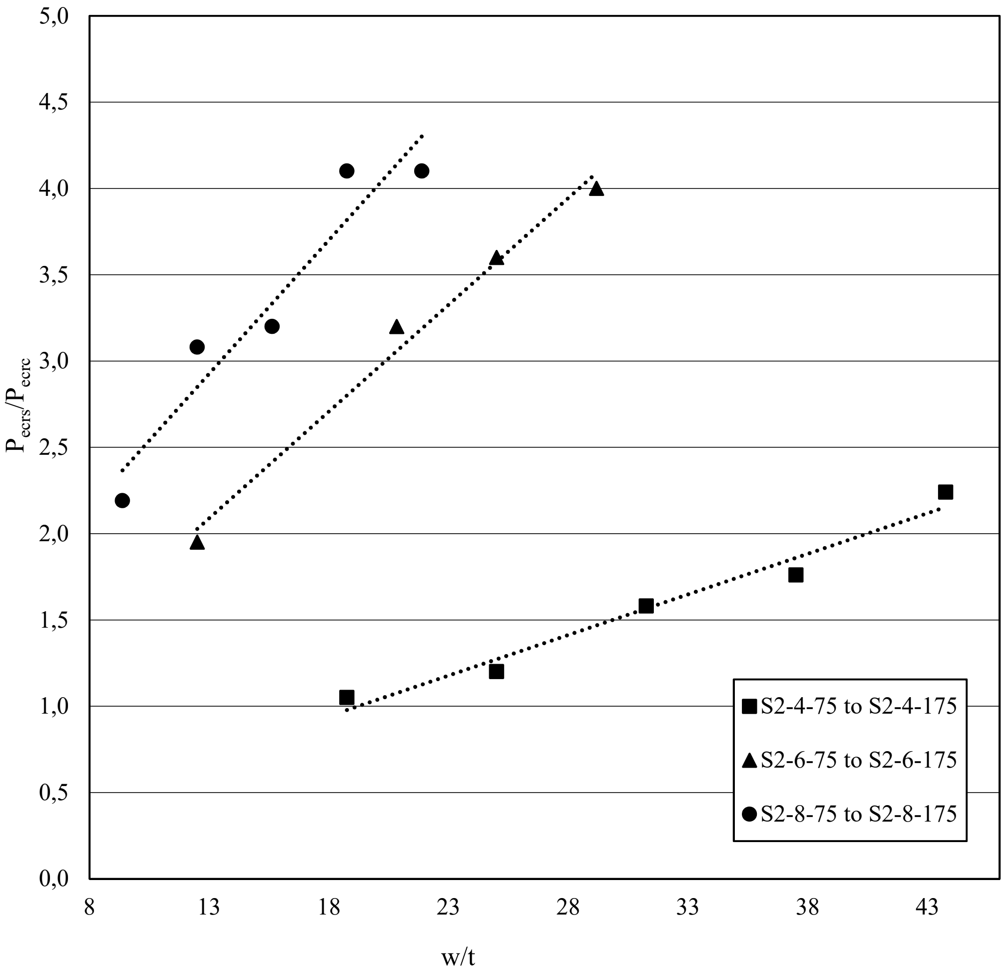


Figure 7: Ratio of experimental first crack loads vs the width-to-thickness ratio

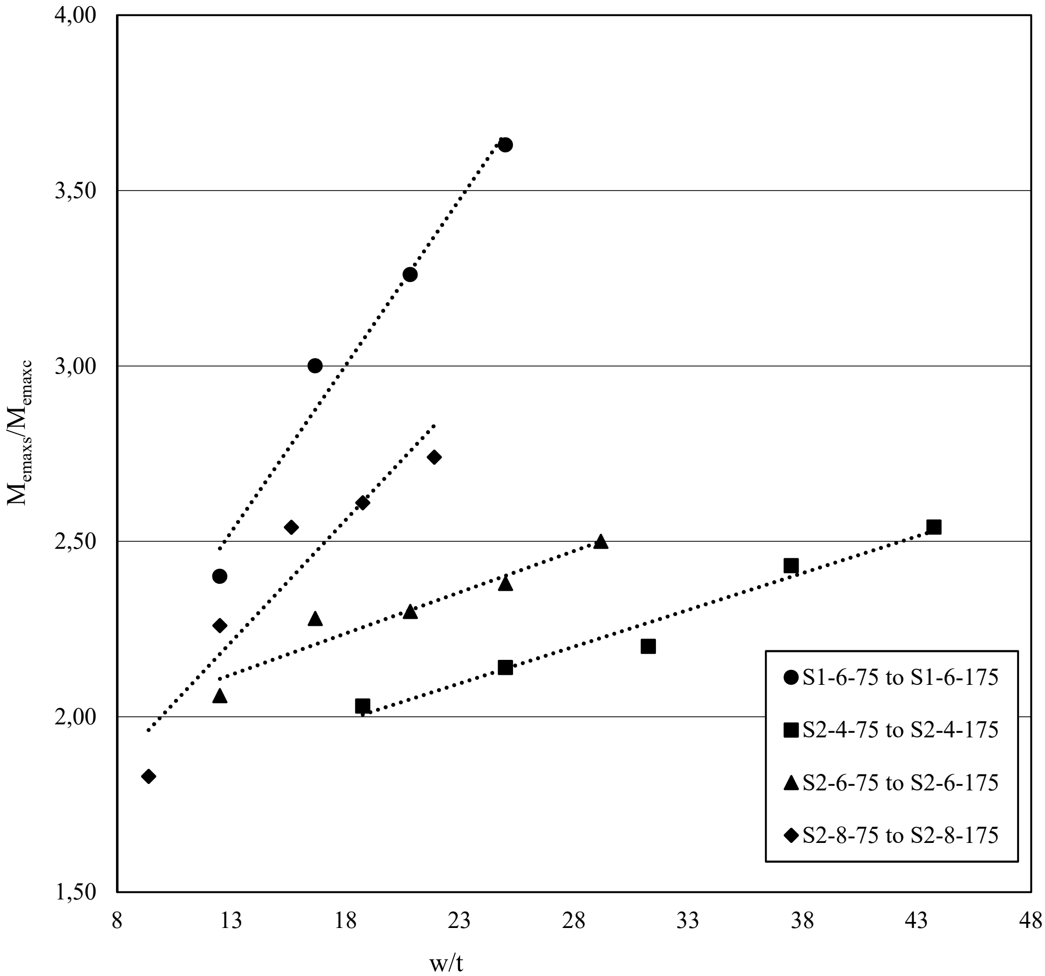


Figure 8: Ratio of experimental maximum moments vs the width-to-thickness ratio

The efficiency of this technique is demonstrated by the fact that for all graphs the maximum capacity of the beams increased with the increase in the width-to-thickness ratio. Where the delamination failure mode was dominant, as in some specimens in Series 2, the ratio P_{ecrs}/P_{ecrc} is much larger than the ratio M_{emaxs}/M_{emaxc}. This clearly shows that the beams did not reach their maximum capacity, but failed prematurely by delamination. In S2-4-75 to S2-4-175 specimens, where the ratio P_{ecrs}/P_{ecrc} is smaller than the ratio M_{emaxs}/M_{emaxc}, delamination was limited by the comparatively larger width-to-thickness ratio of the steel plates.

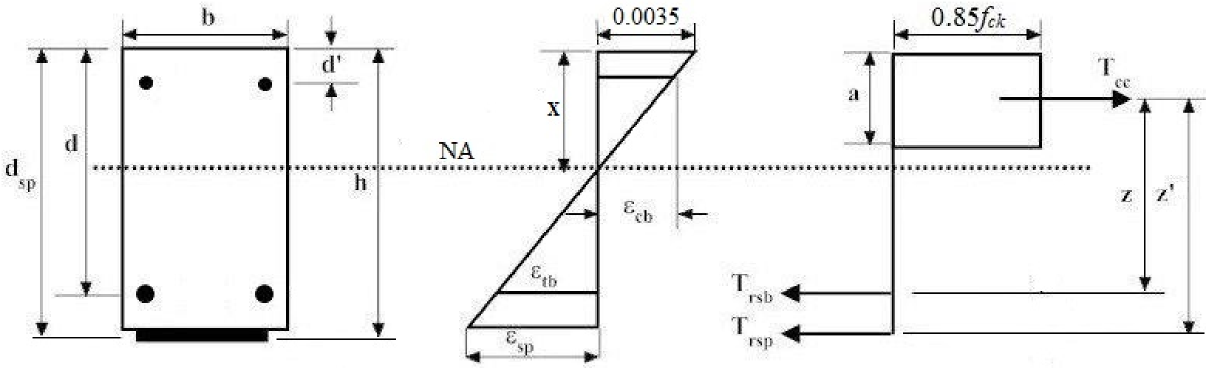
It is also apparent that at all load levels, beams strengthened with larger plate thicknesses, but same width-to-thickness ratio, performs better than beams strengthened with smaller plate thicknesses. For example, beams S2-4-75 and S2-8-150 with steel plates of width-to-thickness ratio of 18.75 have P_{ecrs}/P_{ecrc} ratios of 1.05 and 4.10, and corresponding M_{emaxs}/M_{emaxc} ratios of 2.04 and 2.61, respectively. Other examples include S2-4-100, S2-6-150 and S1-6-150 beams (steel plates of width-to-thickness ratio of 25) with P_{ecrs}/P_{ecrc} ratios of 1.20 and 3.60, and corresponding M_{emaxs}/M_{emaxc} ratios of 2.14, 2.38 and 3.65, respectively, and beams S2-6-75, S2-8-100 and S1-6-75 (steel plates of width-to-thickness ratio of 12.5) with P_{ecrs}/P_{ecrc} ratios of 1.95 and 3.08, and corresponding M_{emaxs}/M_{emaxc} ratios of 2.06, 2.26 and 2.45, respectively. It can be concluded that, where adequate surface preparation and bonding exist, beams with larger steel thicknesses perform better than beams with smaller plate thickness, even though the width-to-thickness ratios of the steel plates are the same.

As indicated before, the code-predicted moment capacity of the control and strengthened specimens were determined in accordance with the provisions provided in EN 1992-1-1 [20]. For composite beams that are subjected to a positive moment, EN 1992-1-1 [20] assumes a rectangular stress block of the composite section to calculate the ultimate moment capacity, as shown in Figure 9. Since the purpose of the compression reinforcement bars was to support the shear links, all reinforced concrete beams were designed as singly reinforced concrete sections. In the rectangular stress block, the height of the compression zone of the concrete (a) is established from the horizontal equilibrium forces of the concrete ($C_r = 0.85baf_{ck}$) and the combined resistance of the tension reinforcement ($T_{rsb} + T_{rsp}$), where $T_{rsb} = A_{sb} f_y$ denotes the tensile resistance of the reinforcement bars, $T_{rsp} = A_{sp} f_y$ denotes the tensile resistance of the steel plates, A_{sb} is the area of the reinforcement bars, A_{sp} is the area of the steel plates and f_y is the 0.2% yield strength of the reinforcement bars/steel plates. The moment of resistance of the control beam is calculated from the equilibrium of the horizontal forces acting on the reinforced concrete beam and taking the moment of the tension in the reinforcement bars about the centroid of the concrete in compression, as given in Equation 1, whilst the moment of resistance of the strengthened beam is determined by taking the moment of the tension in the reinforcement bars and steel plates about the centroid of the concrete in compression, as given in Equation 2.

$$M_{rcb} = T_{rsb} \times z \tag{1}$$

$$\mathrm{M_{cmr}} = \mathrm{T_{rsb}} \times z + \mathrm{T_{rsp}} \times z' \tag{2}$$

where, z is the lever arm from the centroid of the concrete in compression to the steel reinforcement, and z' is the lever arm from the centroid of the concrete in compression to the steel plates.



(a) Cross-section (b) Strain distribution c) Rectangular stress block

Figure 9: Strain distribution and stress block for strengthened beams

A comparison of the maximum experimental moment and the code-predicted moment of resistance of the control beams, shows that EN 1992-1-1 [20] underestimate the maximum experimental moment of the control beams by a large margin (an average of 40.0%). For all strengthened beams, the ratio of the experimental moment of resistance to the code-predicted moment of resistance ($M_{emaxc/s}$/ $M_{tmaxc/s}$) decreases as the width-to-thickness ratio of the steel plate increases (Figure 10). As this happens, the mode of failure changes from flexural yielding to premature plate-end debonding, to shear failure or delamination. Evidence of yielding of beams is shown by the ratio of the experimental moment of resistance to the code-predicted moment of resistance ($M_{emaxc/s}$/ $M_{tmaxc/s}$) of at least 1.0.

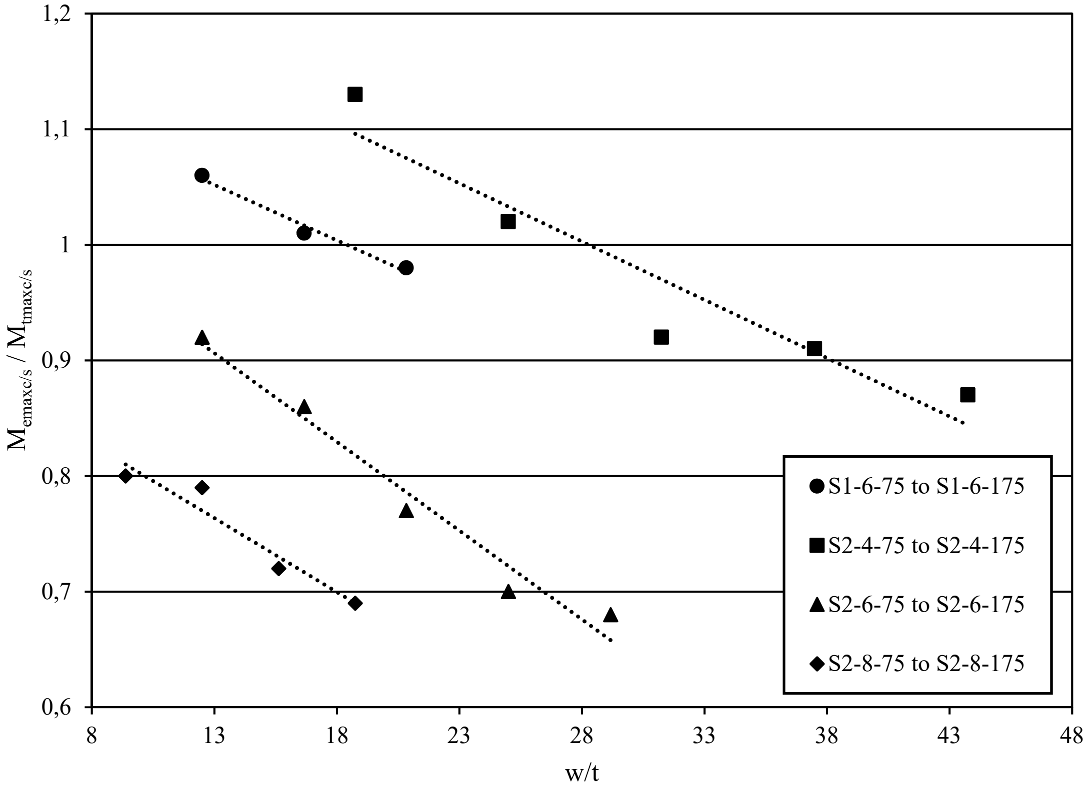


Figure 10: M_{emaxs}/ M_{tmaxs} versus w/t

4.3 Moment-deflection response

All strengthened beams, except beams S2-4-75 to S2-4-125, experienced high reduction in mid-span deflections, compared to the control beams. These mid-span reduction in deflections ranged from 48.1% to 78.2%, and decreased as the width-to-thickness of the steel plates increased. Beams that fail by flexural yielding only tended to have high deflections at failure as compared to those that failed by plate-end debonding, shear failure, delamination or a combination of these modes of failure with flexural yielding. The moment-deflection responses of all beams are illustrated in Figures 11 - 14. Initially, the behaviour of both the control and strengthened beams are linear, and the stiffnesses are similar. Depending on the size of the steel plate, this stiffness was maintained for a longer period by strengthened beams, compared to the control beams, as the steel reinforcement in the latter yielded. After losing the initial stiffness, the control beams yielded extensively, as displayed by the horizontal moment-deflection curves. The external bonded steel plate increased the overall stiffness of the strengthened sections, resulting in high cracking load and maximum capacity, and high reductions in mid-span deflections, crack width and crack spacing, compared to the control beams.

Strengthened beams with smaller width-to-thickness ratio exhibited extensive ductility or large deformations without collapsing. Most specimens, which experienced considerable ductility had steel plates with smaller width-to-thickness ratios (S2-4-75 and S2-4-100) and lower yield strength (S1-6-75 and S1-6-100). Ductility is important in continuous strengthened beams because it allows re-distribution of moments and provides warnings of imminent failures. It is a particularly important factor in seismic regions where the structural elements experience a lot of dynamic forces generated by ground movements. It should also be noted that although some of the strengthened beams did not achieve substantial ductility, they attained immense inelasticity, which should be enough to allow full composite design of such beams.

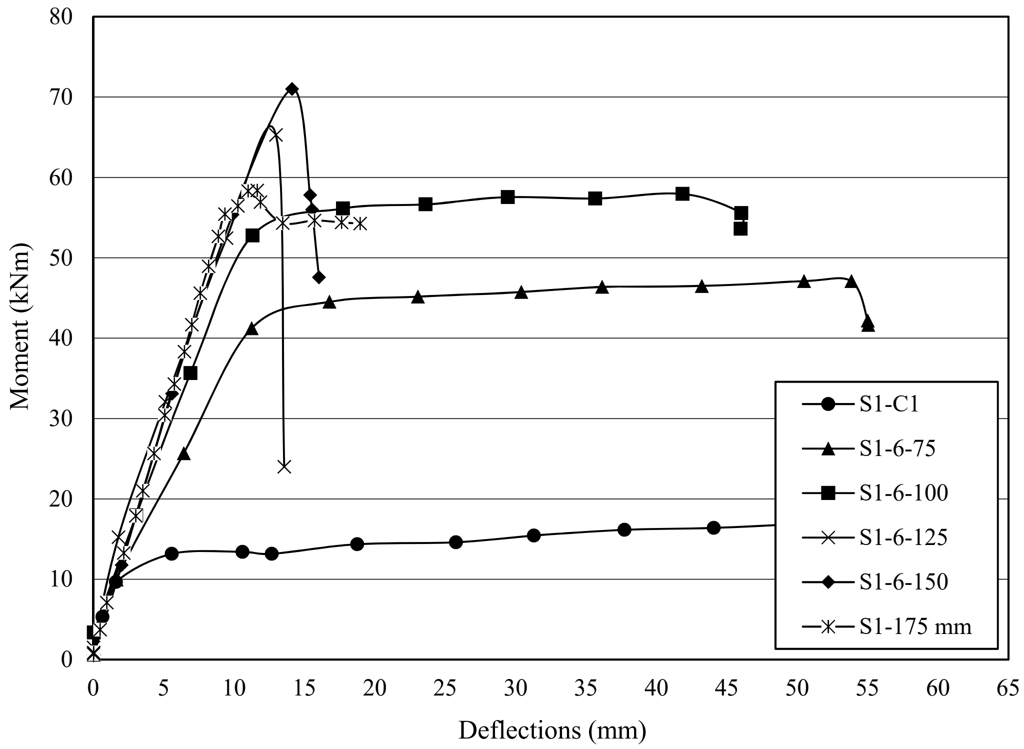


Figure 11: Moment-deflection response of S1-6-75 to S1-6-175 beams

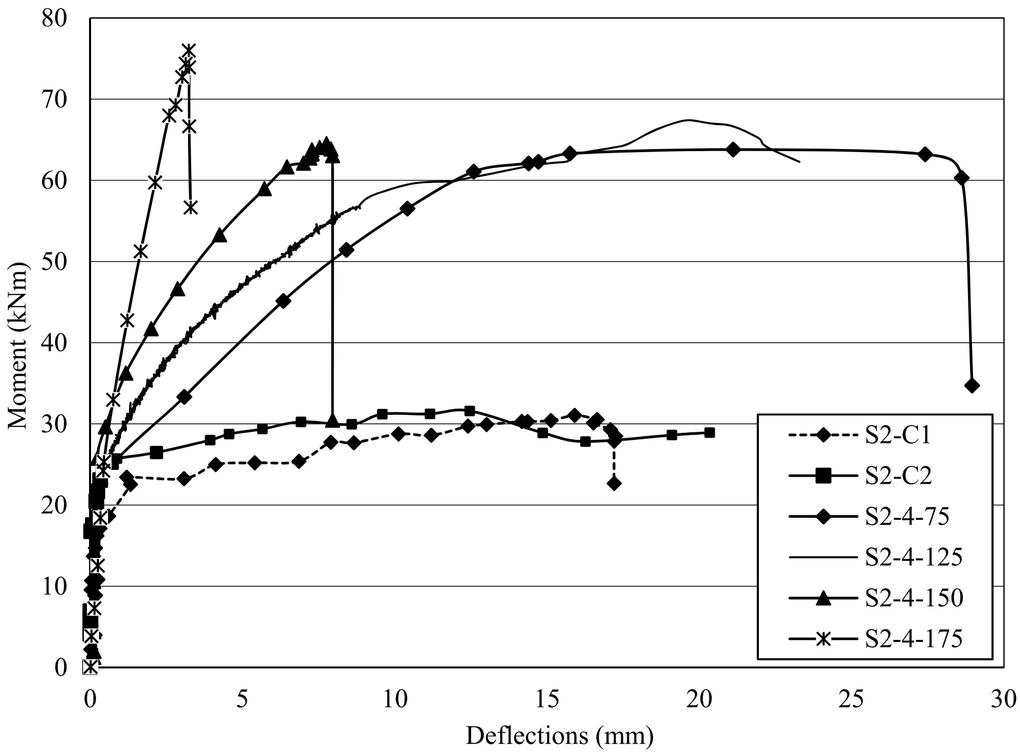


Figure 12: Moment-deflection response of S2-4-75 to S2-4-175 beams

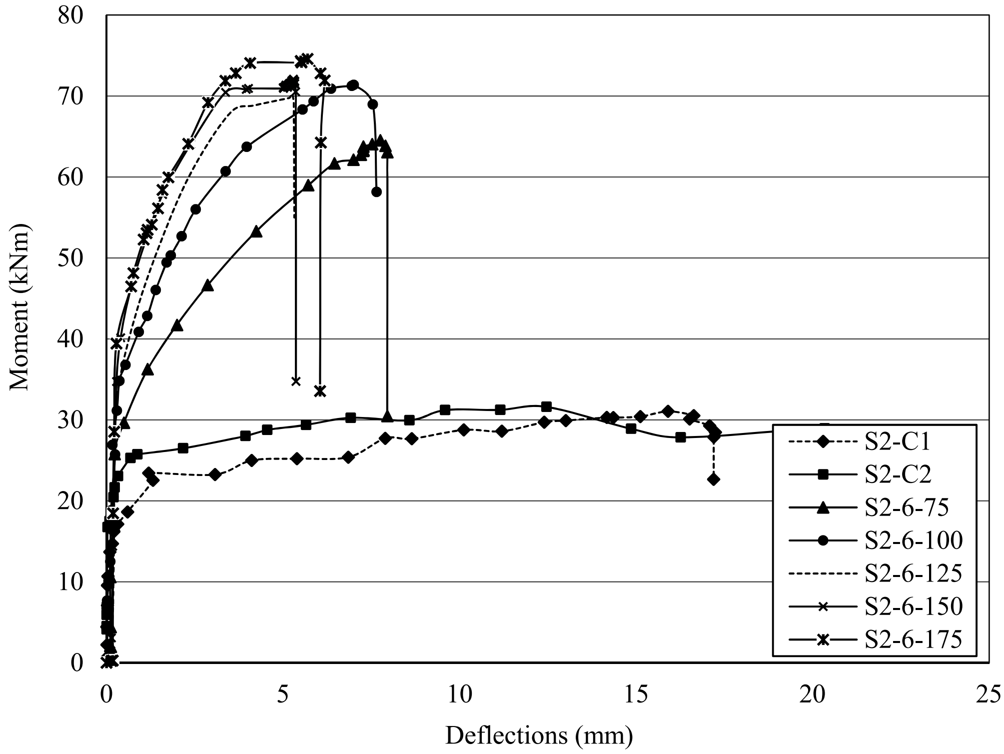


Figure 13: Moment-deflection response of S2-6-75 to S2-6-175 beams

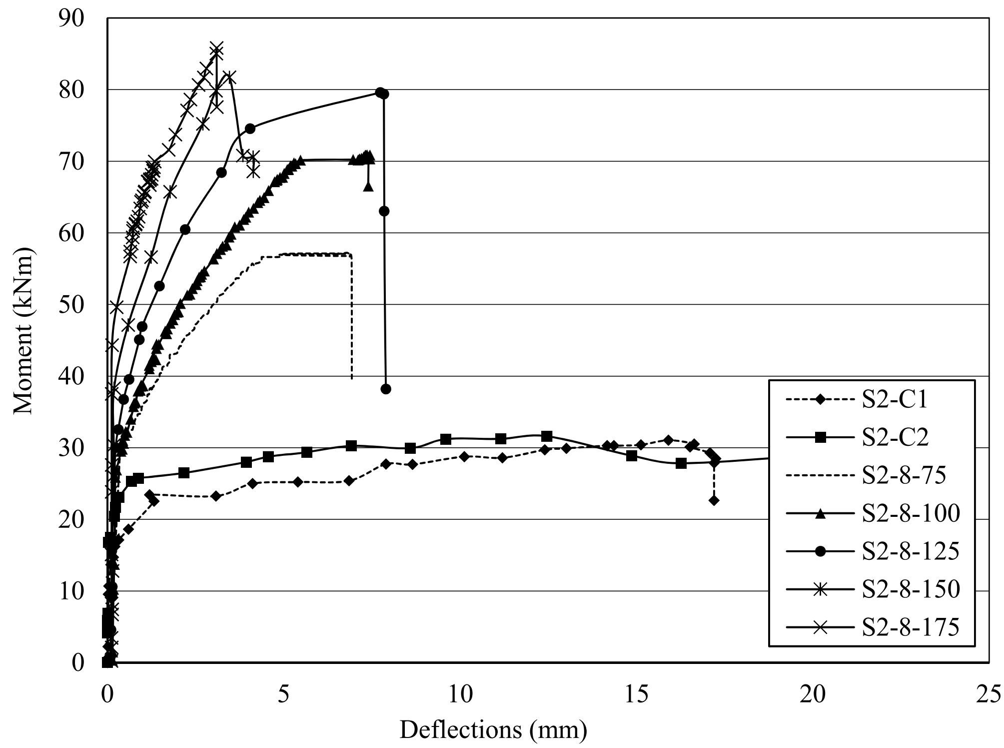


Figure 14: Moment-deflection response of S2-8-75 to S2-8-175 beams

4.4 Moment-steel strain relationships

The moment-strain curves of beams strengthened with steel plates of various width-to-thickness ratios are shown in Figures 15 - 18, and are largely elastic. The moment-strain characteristics are similar to the moment-deflection curves of each beam. In Figures 15 and 16, the steel plate of S1-6-75, S1-6-100 and S2-4-75 beams yielded extensively before failure. As for the remaining moment-strain curves, none of the steel plates showed any sign of yielding or inelastic behavior, as illustrated by the corresponding moment-deflection graphs. This is because the strain gauges probably peeled off just before any plastic deformation took place. Beams strengthened with steel plates of larger width-to-thickness ratios strained less than beams strengthened with steel plates of lower width-to-thickness ratios. This implies that beams strengthened with steel plates of larger width-to-thickness ratios are more bound to debond or fail prematurely before the bonded steel plates yield. It is clear from Figures 15 - 18 that the steel strain decreases as the width-to-thickness ratio of the plates increase.

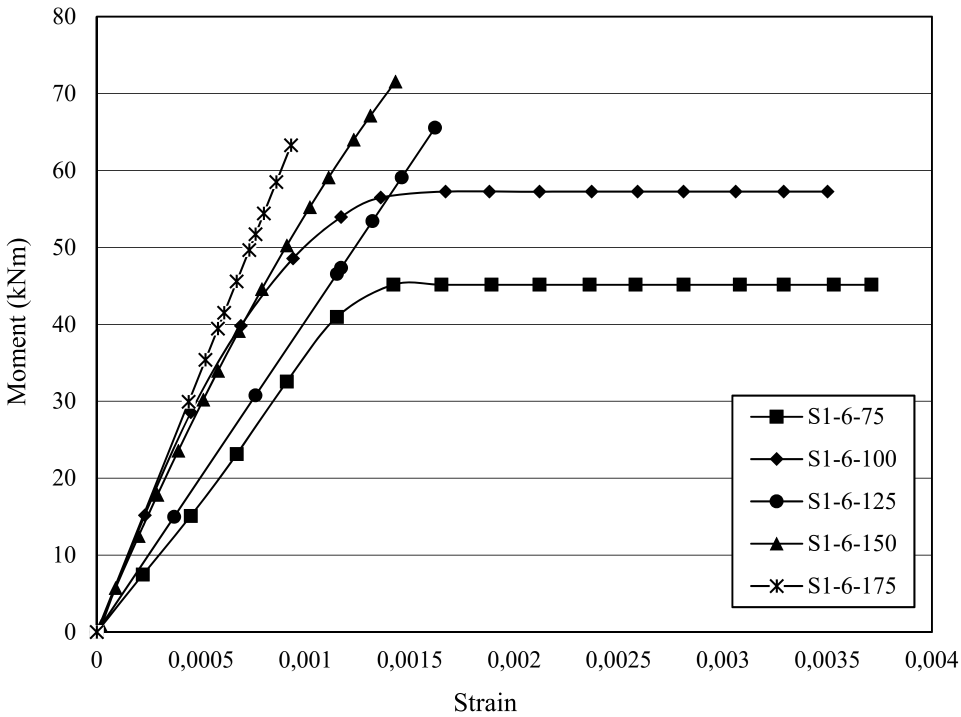


Figure 15: Moment-strain response of S1-6-75 to S1-6-175 beams

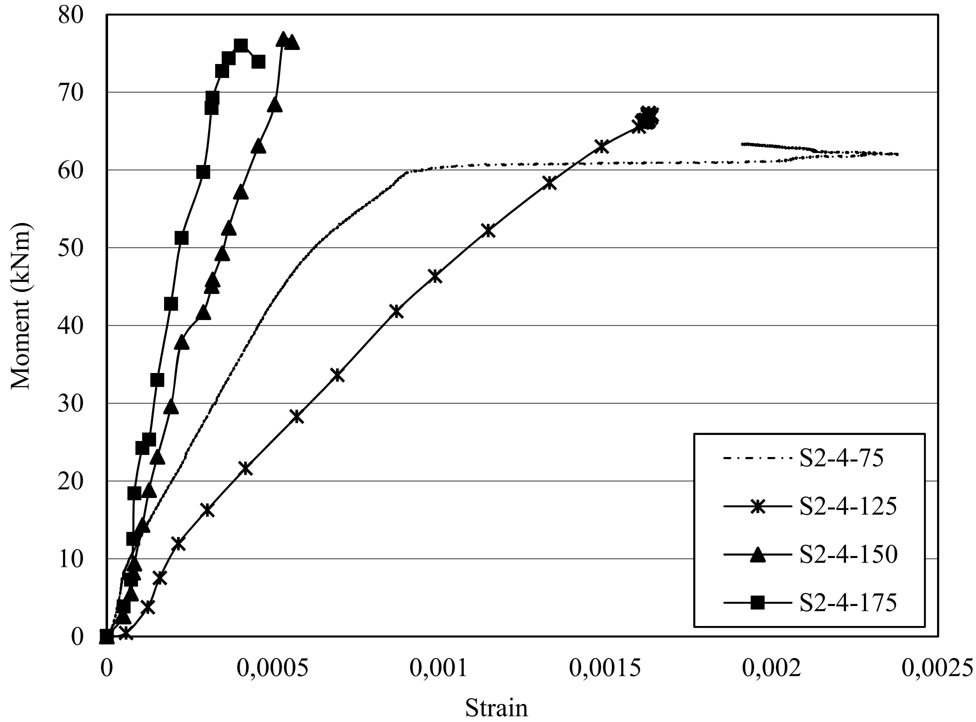


Figure 16: Moment-strain response of S2-4-75 to S2-4-175 beams

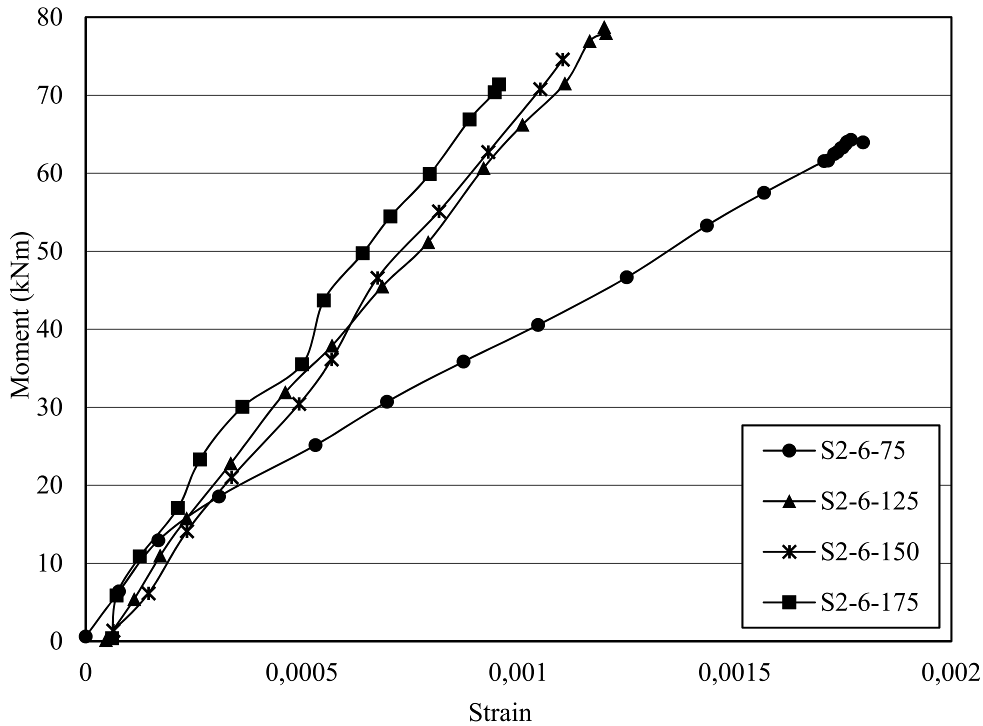


Figure 17: Moment-strain response of S2-6-75 to S2-6-175 beams

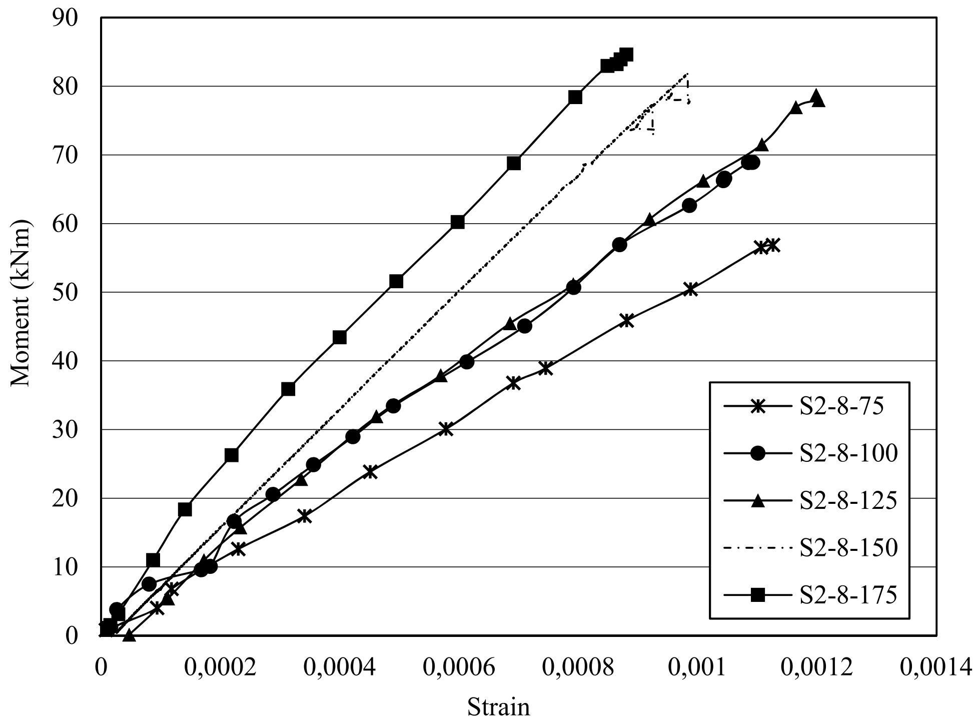


Figure 18: Moment-strain response of S2-8-75 to S2-8-175 beams

4.5 Beam depth-strains relationships

In strengthened beams, it is important to ensure that the beam preserves composite behaviour until failure. The strain results for most specimens, confirm that this can be achieved, even for beams with plates of large width-to-thickness, which eventually fail in modes other than flexural yielding. Since the neutral axis depth is one of the main governing factor in ensuring a ductile flexural failure, this was monitored for all the beams tested in this study, up to the last load stage prior to failure. Strengthened concrete beam sections, in which the tension steel reaches the yield strain and the concrete crushes simultaneously, are called balanced sections. In this case, the position of the actual neutral axis lies at the critical neutral axis of the section. When the steel reaches the yield strain at loads lower than the load at which the concrete crushes, they are called under-reinforced sections. This is the most desirable mode of failure in concrete beams, and happens when the actual neutral axis of the section lies above the critical neutral axis of the section. However, when maximum strain in concrete is reached first than the yield strain of the steel, then they are called over-reinforced beam sections. For this case the actual neutral axis lies below the critical neutral axis. As illustrated in Figures 19 – 21, the mid-span strain profile of the control beams and most of the strengthened beams shows that the beams were significantly under-reinforced. However, as the width-to-thickness ratios of the steel plates increased, resulting in an increase of the total tension steel, the strains on the tension side decreased. Except for specimens S2-6-150 and S2-6-175, there is not much difference between the neutral axis of the strengthened and unstrengthened beams. In S2-6-150 and S2-6-175 specimens, a combination of the large width-to-thickness ratio and high strength of the steel plates, caused the neutral axis to shift downwards, towards a balanced state. It is obvious from the tension strains that the steel deformed more than the concrete.

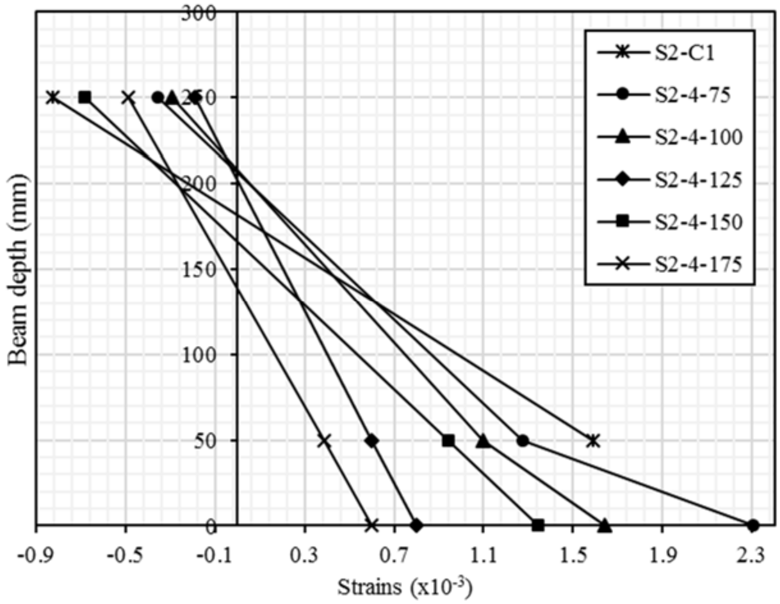


Figure 19: Strains along the depth of S2-4-75 to S2-4-175 beams

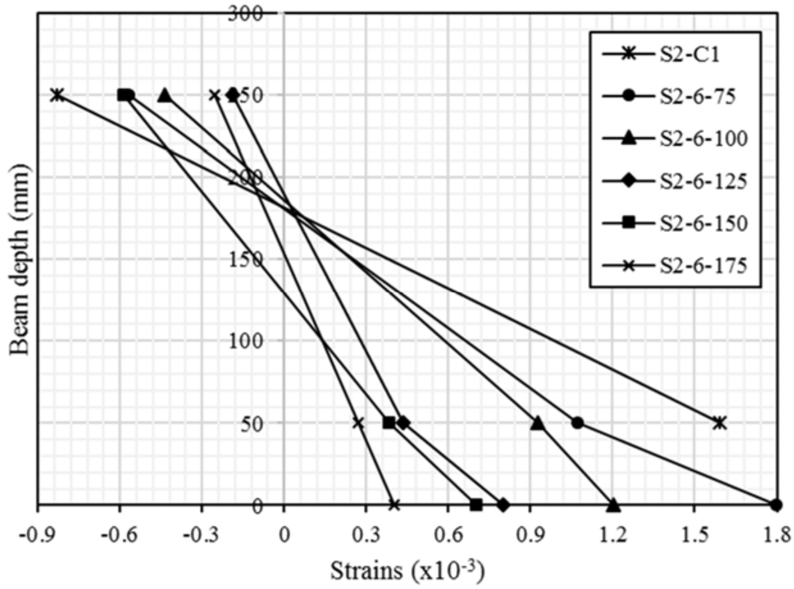


Figure 20: Strains along the depth of S2-6-75 to S2-6-175 beams

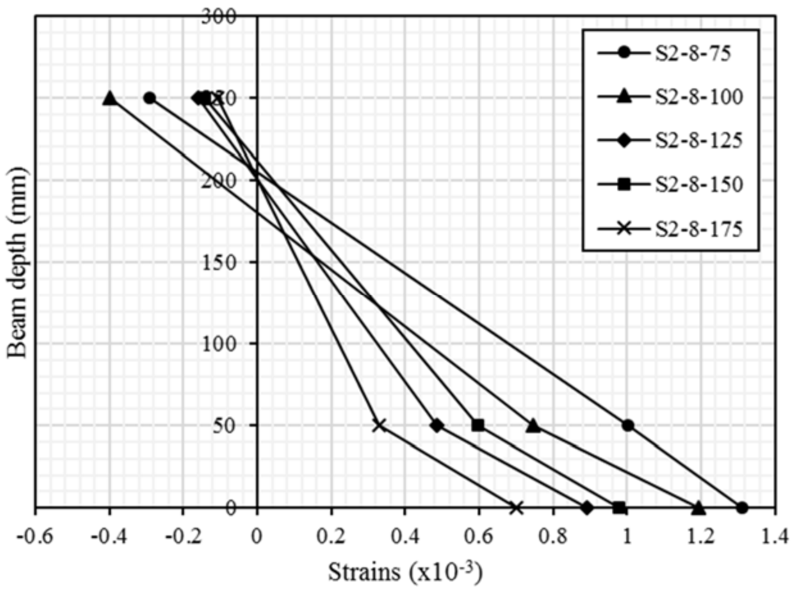


Figure 21: Strains along the depth of S2-8-75 to S2-8-175 beams

5.0 CONCLUSIONS

The purpose of this study was to evaluate the effect of the width-to-thickness ratio of steel plates on the flexural capacity, deflections and flexural stiffness of beams, and to evaluate the width-to-thickness ratio of steel plates that encourage yielding of the composite beams. Based on the overall experimental investigation results, the following conclusions can be drawn from the study:

- The use of external strengthening delayed the appearance of the first crack. Bonded steel plates caused a significant increase in the experimental first crack loads, serviceability and maximum capacity of strengthened beams, compared to the control beam. Beams strengthened with larger steel plates achieved larger loads. The graphs of the ratio of the experimental loads versus the width-to-thickness ratio are steeper for beams with plates of smaller yield strength and width-to-thickness ratio or larger thickness, however, the gradients are less steeper at maximum capacity than at first crack load level. Possible causes of a lower increase in strength include lower yield strength and failure mechanism.

- Control beams and strengthened beams with steel plates of small width-to-thickness ratios, such as S2-6-75 and S2-6-100 in Series 1 tests, and S2-4-75 and S2-4-125 in Series 2 tests, failed in flexure by crushing of the concrete and yielding of the steel plate, after extensive plastic deformation in the zone of high moment. Strengthened beams of larger width-to-thickness failed prematurely either by plate-end debonding, shear or delamination. In some cases these premature failure occurred after yielding has started.

- For all strengthened beams, the ratio of the experimental moment of resistance to the code-predicted moment of resistance decreased as the width-to-thickness ratio of the steel plate increases. As this happens, the mode of failure changed from flexural yielding to premature plate-end debonding, shear failure or delamination. Evidence of yielding of beams is shown by the ratio of the experimental moment of resistance to the code-predicted moment of resistance of at least 1.0.

- Except in S2-4-75 to S2-4-125 beams, the use of external strengthening reduced the deflections at all load levels. For all the strengthened beams, the maximum deflections at failure decreased with increasing width-to-thickness ratio. Beams that fail by flexural yielding only tended to have high deflections at failure as compared to those that failed by plate-end debonding, shear failure, delamination or a combination of these modes of failure with flexural yielding.

- The external bonded steel plate increased the overall stiffness of the strengthened sections, resulting in high cracking load and maximum capacity, and high reductions in mid-span deflections, crack width and crack spacing, compared to the control beams. It should also be noted that although some of the strengthened beams did not achieve substantial ductility, they attained immense inelasticity, which should be enough to allow full composite design of such beams.

- Beams strengthened with steel plates of larger width-to-thickness ratios strained less than beams strengthened with steel plates of lower width-to-thickness ratios. This implies that beams strengthened with steel plates of larger width-to-thickness ratios are more bound to debond or fail prematurely before the bonded steel plates yield.

- Although the authors were able to provide comprehensive data and detailed analysis about the effect of the width-to-thickness ratio of steel plates on composite beams, they were not able to resolve the huge inconsistency in the literature about the width-to-thickness ratio of the steel plate that should be used to promote yielding and ductility of the composite beams. In this paper, the authors found that the width-to-thickness ratio of steel plates as low as 12.5 can promote flexural yielding and extensive ductility in strengthened beams. To resolve this problem the authors are working on a separate paper to analyse strengthened beams based on the relationship between the total tension reinforcement and the cross-sectional area of the concrete, instead of the width-to-thickness ratio of the steel plates only. This work will include some finite element modelling.

6.0 ACKNOWLEDGEMENTS

The authors wish to acknowledge AfriSam South Africa, for donating the concrete used in this work, and Mr. Kelby Tansley for helping to carry out tests reported in Series 1.

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