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Experimental and Analytical Investigations on the Structural Behaviour of Steel Plate and CFRP Laminate Flexurally Strengthened Reinforced Concrete Beams

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Abstract: This study presents an experimental and analytical investigation to compare the structural behaviour of externally bonded steel plates and Carbon Fiber Reinforceed Polymer (CFRP) laminates flexurally strengthened r.c. beams. For the experimental investigation, three r.c. beams were cast. One beam was tested in the un-strengthened condition to act as the control beam. The second beam was strengthened using steel plate while the third beam was strengthened using CFRP laminate. The strengthened beams were designed to have the same strength with the assumption that they would fail in a ductile manner. The test results indicated that although both beams were designed for the same strength, the CFRP laminate strengthened beam recorded a slightly higher failure load compared to the steel plate strengthened beam. The steel plate strengthened beam recorded a higher cracking load and less deflections, reinforcement bar strains, concrete strains and crack widths compared to the CFRP laminate strengthened beam. Results also showed that the CFRP laminate strengthened beam failed by premature concrete cover separation failure, whereas the steel plate strengthened beam failed by premature plate end interfacial debonding followed by concrete cover separation. The beams were also modelled using a Finite Element Method (FEM) package. The numerical results seemed to agree well with the experimental results.

Key words: Flexure, plate bonding method, un-anchored, finite element method

INTRODUCTION

Strengthening of reinforced concrete beam has been shown to be an important task in the field of structural maintenance. With the development of newer structurally effective adhesives, plate bonding method using steel plate and Carbon Fibre Reinforced Polymer (CFRP) laminate for strengthening of existing concrete structures are on the increase. Steel plate has been widely used because of its availability, cheapness, uniform materials properties, workability, high ductility and high fatigue strength. However, several disadvantages of steel plate including the transportation, handling and installation of heavy plates, corrosion of plates and limited delivery lengths of plates are very apparent. To overcome these problems Carbon Fibre Reinforced Polymer (CFRP) laminate has been used as an alternative material. Usages of CFRP laminate in strengthening r.c. structures has gained popularity compared to the steel plate due to its high strength to weight ratio, corrosion resistance and low maintenance cost. A lot of studies on strengthened r.c. beams both using the steel plate and CFRP laminate had been conducted in the past (Oehlers and Moran, 1990; Garden and Hollaway, 1998; Oh *et al*., 2003; Xiong *et al*., 2007). Studies on the comparison of the two strengthening material of the same capacity has not been reported. Study of their behaviour using numerical model i.e., FEM has also been very limited. The objectives of this study include, (i) to compare the structural behaviour of externally bonded steel plate and CFRP laminate flexurally strengthened r.c. beams with the same design strength and (ii) to come out with a method of using FEM to study the behaviour of these strengthening materials.

MATERIALS AND METHODS

Description of specimens: Three r.c. beams of same geometrical dimensions and material properties were cast and tested in this study and designated as beam A1, B1 and C1. Beam A1 was left un-strengthened to act as the control specimen. Beam B1 was strengthened using steel plate with a geometrical dimension of 2.76×73×1900 mm and beam C1 was strengthened by CFRP laminate with a geometrical dimension of 1.2×80×1900 mm. The dimensions of these strengthening materials were determined using the simplified stress block method of BS 8110 assuming the failure mode of the strengthened beams are due to the failure of the main reinforcing bars. The test variables are summarized and shown in Table 1.

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Table 1: Characteristics of three test specimens

		Strengthening material		
Specimen	Designation	Type	Thickness (mm)	Width (mm)
1	A1			
2	B1	Steel plate	2.76	73
3	C1	FRP	1.2	80

Fabrication of specimens: All the beam specimens were of 2,300 mm long, 125 mm wide and 250 mm deep as shown in Fig. 1. These beams were reinforced with two twelve mm diameter steel bars in the tension zone. Ten millimeter steel bars were used as hanger bars and six mm bars were used for the shear reinforcement which was symmetrically placed as shown in Fig. 1. The spacing of the shear reinforcement was 75 mm.

Strengthening: For all the beams, the length of the strengthening material was maintained at 1900 mm, which covered almost the full-span length of the beams, as shown in Fig. 2. The main reason for the full span-length strengthening with steel plates and CFRP laminates was to maximize the strengthening effects. The thickness and width of the strengthening plates are shown in Table 1.

The surface treatments of both the concrete surface and the strengthening materials prior to plating works were very crucial in ensuring perfect bonding between the concrete and strengthening plates. Concrete was ground with a diamond cutter to expose the coarse aggregates. Dusts were then blown out using compressed air. The surface of the steel plate was sand blasted to eliminate any rust that might be present. Colma cleaner was used to remove the carbon dusts form the bonding face of the strengthening materials. Sikadur adhesive a proprietary of Sika Kimia was then trawled on to the surface of the concrete specimens to form a thin layer. The adhesive was applied using a special dome shaped spatula onto the surfaces of the strengthening materials. The strengthening plates were then placed on the prepared concrete surfaces. Using a rubber roller, the plates were gently pressed on to the outer surfaces until the excessive adhesives were forced out on both sides of the strengthening materials. The surplus adhesives were then removed.

Materials: Ordinary Portland Cement (OPC) was used in casting the beams and the maximum size of coarse aggregate was 20 mm. The concrete mix was designed with a target strength of 30 MPa. The mix proportion adopted is as shown in Table 2. The compressive strengths of the concrete were obtained from three cubes at 28 days in accordance with the British Standard (1881).

Table 2: Mix design

		Contents (kg m^{-3})			
Slump	Water cement ratio	Water	Cement	Coarse aggregate	Fine aggregate
60-180	0.65	208	320	740	1120

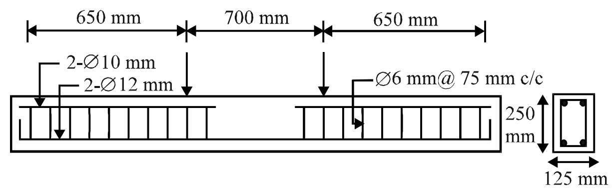

Fig. 1: Beam details

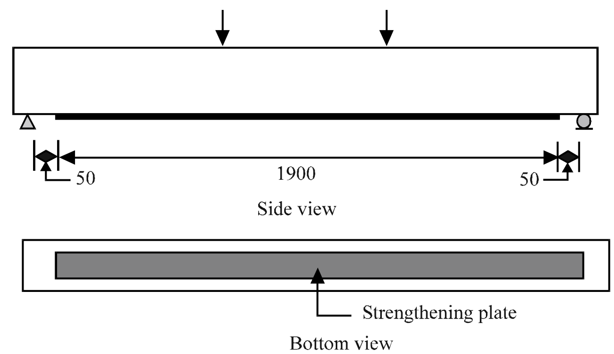

Fig. 2: Strengthening details beams

Two twelve millimeter diameter of high tensile deformed bars were used as the main reinforcement. The measured yield and tensile strength of these bars were 551 and 641 MPa, respectively. Ten millimeter diameter bars were used as hanger bars in the shear span. Six millimeter diameter bars were used for the stirrups. The measured yield and tensile strength of the stirrups were 520 and 572 MPa, respectively. The modulus of elasticity of all the steel bars used was 200 GPa. For the strengthening materials, mild steel plates and CFRP laminates (Sika CarboDur S812) were used. The yield strength, tensile strength and modulus of elasticity of the steel plates were 320, 375 MPa and 200 GPa, respectively. The tensile strength and modulus of elasticity of CFRP laminates were 2800 MPa and 165 GPa, respectively. The design and ultimate strain of CFRP laminates were taken to be 0.0085 and 0.017, respectively as specified by the manufacturer.

Instrumentation and test procedure: Figure 3 shows the location of the different instruments used to record data during testing. Electrical resistance strain gauges were used to measure the strains in the steel plate, CFRP laminate and top face of the r.c. beams. The demac gauges were attached along the height of beam at the mid span region to measure the horizontal strains of the r.c. beams.

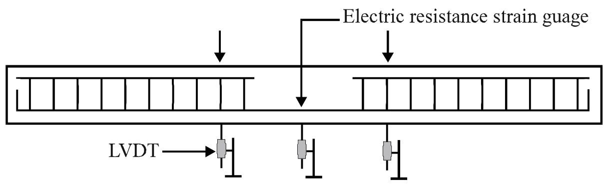

Fig. 3: Location of the measuring instruments on the r.c. beams

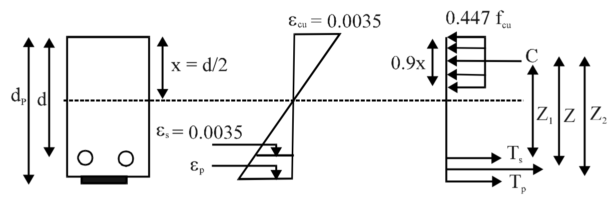

Fig. 4: Strain-stress diagram of strengthened beam

Three linear variable displacement transducers were used to measure the vertical deflections of the beams at mid-span and under the two load points. The beams were tested under four point loading. The load was applied at an increment of 5 kN up to failure using the Instron 8505 servo hydraulic Universal Testing Machine.

Calculation of theoretical ultimate load: The ultimate load of strengthened beams could be obtained using the stress block of BS 8110 (British Standard, 1985). Figure 4 shows the cross section and the stress and strain distribution of strengthened beams. Assuming full composite action of steel plate and beam, the failure load of steel plate strengthened beams by the BS 8110 could be written as follows:

$$P_u = \{A_s f_t\ (d-0.9x/2)+A_p f_{tp}\ (dp-0.9x/2)\}/0.65 \qquad (1)$$

where, A_s, A_p equal to the area of rebar and steel plate, f_y and f_{yp} equal to the yield stress of rebar and steel plate and f_t and f_{tp} equal to the tensile stress of rebar and steel plate, respectively.

For the failure load of CFRP laminate strengthened beam using the same approach would be given by:

$$P_u = \{A_s f_t\ (d-0.9x/2)+A_{frp}\ \sigma_{frp}\ (d_p-0.9x/2)\}/0.65 \qquad (2)$$

where, A_s, A_{frp} equal to the area of rebar and CFRP laminate, f_y equals to the yield stress of rebar, σ_{frp} equals to stress of CFRP laminate and f_t equals to the tensile stress of rebar.

The stress of CFRP laminate, σ_{frp} could be obtained by the following Equation:

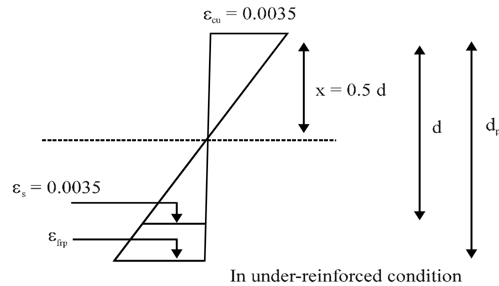

Fig. 5: Strain diagram of CFRP laminate strengthened beam

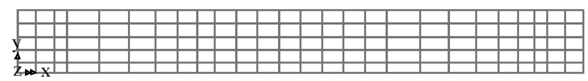

Fig. 6: Modelling of strengthened beam

$$\sigma_{frp} = \{(\varepsilon_{cu} * (d_p - x))/x\} * E_{frp} \qquad (3)$$

where, E_{frp} equals to the modulus of elasticity of CFRP laminate and ε_{cu} could be obtained from Fig. 5.

Finite Element Method (FEM) analysis: In the numerical analysis, a finite element program LUSAS is used to study the structural behaviour of the rectangular reinforced beams strengthened by steel plate and CFRP laminate. 2-D surface elements were used to model the reinforced concrete beams and strengthening plates. The surface element of the steel plate and CFRP laminate are attached to the bottom face of the concrete beam directly as shown in Fig. 6. Perfect bonding between strengthening plate and the concrete was assumed. Plane stress assumption was used for surface meshing. Nonlinear and transient option was selected to run the programme.

RESULTS AND DISCUSSION

Mode of failure: Figure 7 shows the failure modes of control beam (A1), the steel plate strengthened beam (B1) and the CFRP laminate strengthened beam (C1).

It was observed that the control beam failed in a conventional flexural manner by steel yielding followed by crushing of concrete. The steel plate strengthened beam failed by plate end interfacial debonding followed by concrete cover separation failure. The CFRP laminate strengthened beam failed by concrete cover separation failure. Similar modes of failures were reported by earlier such as El-Mihilmy and Tedesco (2001) and Smith and Teng (2002).

The mechanism of debonding failure can be attributed to the discontinuity at the plate end, whereby excessive

Table 3: Experimental results of three specimen

Specimen	Experimental results 1st crack load (kN)	Increase over control beam (%)	Ultimate load (kN)	Increase over control beam (%)	Mode of failure	Theoretical ultimate load, P_u (kN)
A1	14		80.59		Flexure	76.0
B1	35	150	104.3	29.4	Debonding	122.3
C1	27	93	123.9	53.7	Debonding	119.0

Fig. 7: Failure mode of tested specimens

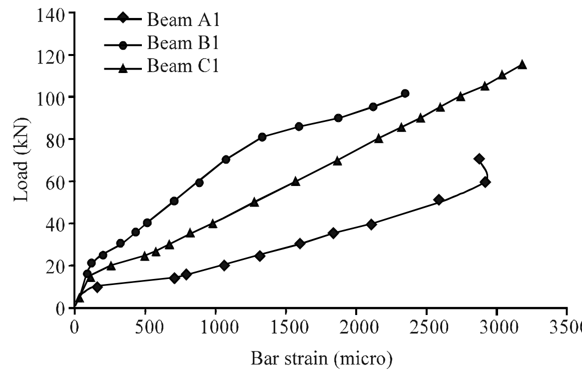

Fig. 8: Load level during steel bar strain test for all beams

shear and normal stress would normally developed. El-Mihilmy and Tedesco (2001) argued that when this shear stress exceeded the shear resisting capacity of the concrete, then shear crack would occur at the end of the plate. This resulted in the plate being debonded either at the level of internal reinforcement or at the level of bonding interface. The intensity of the shear and normal stresses would normally depend on the thickness and stiffness of the plate. The thicker and stiffer plate is the higher are the shear and normal stresses. High shear and normal stresses will normally cause the plates to debond earlier at the level of the bonding interface rather than at the level of internal reinforcement. This type of failure is referred as plate end interfacial debonding.

In this study steel plate had higher thickness and stiffness, thickness is 2.76 mm and EI is 25580 kN mm^2, compared to the CFRP laminate, thickness is 1.2 mm and EI is 1901 kN mm^2. Hence it was expected that the steel plate strengthened beam (B1) failed earlier than CFRP laminate strengthened beam (C1) even though both beams were designed for the same strength. The failure mode of the steel plate strengthened beam B1 was shown to be close to the plate end interfacial debonding rather than concrete cover separation failure. However, due to delay of laminate debonding, the resultant debonding mechanism of CFRP laminate strengthened beam was found to be more explosive compared to the steel plate strengthened beam.

Failure load: Table 3 shows the failure modes, first crack loads and the ultimate loads from the experiment. It can be found that the steel plate strengthened beam recorded the highest first cracking load, while in terms of the ultimate loads the CFRP laminate strengthened beam recorded the highest value.

Table 3 shows the comparisons between the measured failure load and the theoretical failure load. The theoretical failure load was calculated following BS 8110, based on full composite action (Eq. 1, 2). Table 3 indicates that the measured failure load of B1 is less than the theoretical value. This is because the beam had failed by plate debonding before reaching its ultimate load. Whereas, the measured failure load of beam C1 is little bit higher than the theoretical failure load. It might happen due to the delay of laminate debonding. Furthermore, the beam was designed according to BS code in under reinforced design condition. Due to under reinforced design condition, the design strain of steel bar was taken to 0.0035. The strain of CFRP laminate was then calculated from the strain diagram (Fig. 5). This calculated design strain of CFRP laminate was higher than 0.0035 but less than its actual design strain (0.0085). For this, the beam was able to carry more loads after internal steel bar yielding and has shown the larger measured failure load than theoretical value. However, both of the strengthened beams showed higher failure load compared to design load.

Strain characteristics

Bar strain: Figure 8 shows the loads obtained during the test versus steel bar strains for all the beams. It can be seen that, at all load levels, the bar strains of all the

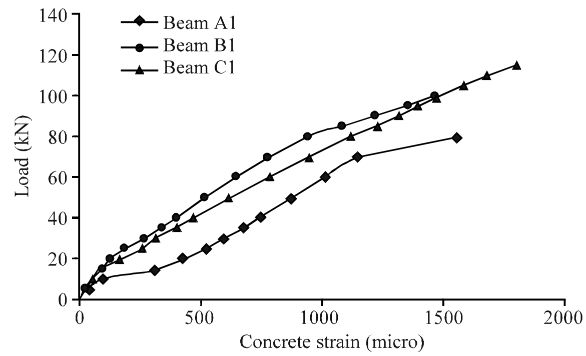

Fig. 9: Load level versus maximum concrete compressive strains for all beams

strengthened beams were less than those in the control beam. Figure 8 also shows that the reinforced bar strain of the steel plate strengthened beam (B1) was less than the reinforced bar strain of CFRP laminate strengthened beam (C1). It could also be seen that strains of the control beam (A1), steel plate strengthened beam (B1) and CFRP laminate strengthened beam (C1) increased rapidly after 10, 20 and 15 kN load, respectively. This could be related to the cracking of concrete section. Where by during cracking the tensile stresses were transferred directly to the reinforcing bar.

Concrete compressive strain: Figure 9 shows the load versus the maximum concrete compressive strains which were measured on the top of the concrete beam. The maximum concrete compressive strains of beams B1 and C1 were both found to be less than the strain of the control beam because of their higher stiffness values. The compressive strain of beam B1 was found to be less than the compressive strain of beam C1 because of the higher value of the stiffness of the steel plate.

Strain variation on beam depth: Figure 10 shows the strain variation over the depth of the beam A1, B1 and C1 at 30 and 70 kN load. This strain was obtained from the demec readings.

Figure 9 shows that the strains of beam B1 and C1 were less than beam A1 due to the higher value of stiffness of the strengthened beams. Since the strains of beam B1 and C1 are less than beam A1, the neutral axis depths of these beams were higher due to the internal force equilibrium requirements.

Deflection: Figure 11 shows the load-mid span deflection curves from the tests for beams A1, B1 and C1. All the beams showed a linear increment of deflection before failure. All strengthened beams showed less deflection than the control beam due to their higher stiffness. The

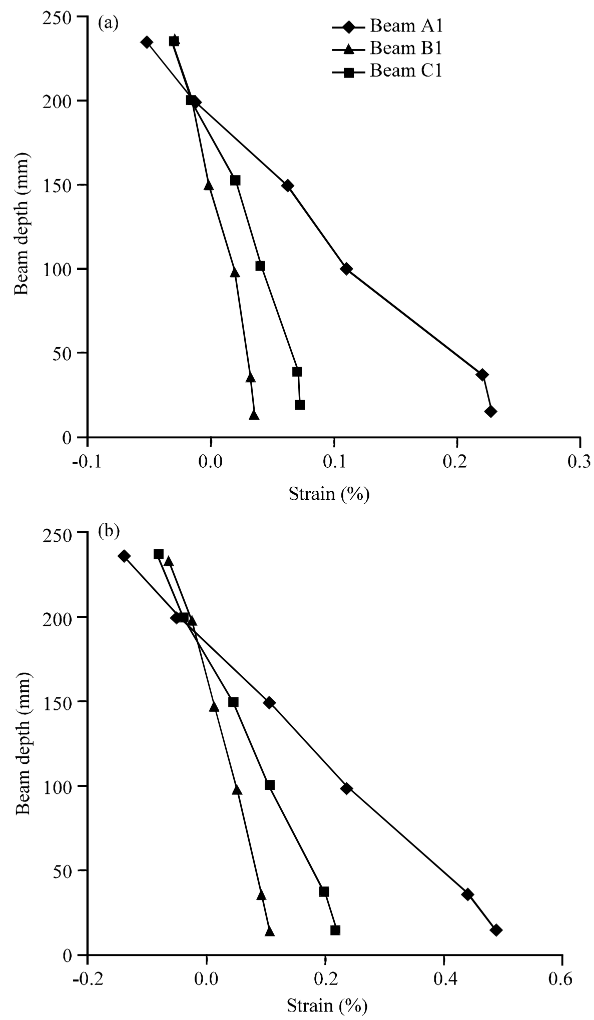

Fig. 10: Strain variation of beams. Strain at 30 kN (a) and 70 kN (b) load

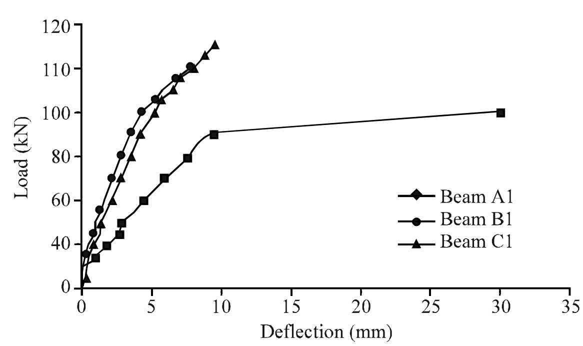

Fig. 11: Load-mid deflection curves during beams test

Fig. 11 also shows that the steel plate strengthened beam (B1) shows a smaller deflection compared to the CFRP laminate strengthened beam (C1).

Table 4: Comparison of failure loads

	Experimental results			Numerical results			Comparison	
Specimen	1st crack load (kN)	Ultimate load (kN)	Mode of failure	1st crack load (kN)	Ultimate load (kN)	Mode of failure	1st crack load (Expt./Num)	Ultimate load (Expt. / Num)
A1	14	80.59	Flexure	12	75	Flexure	1.17	1.07
B1	35	104.3	Debonding	28	120	Flexure	1.25	0.87
C1	27	123.9	Debonding	24	130	Flexure	1.125	0.95

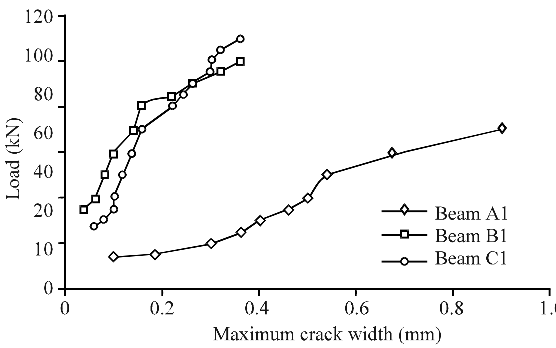

Fig. 12: Load level versus maximum of all the beams

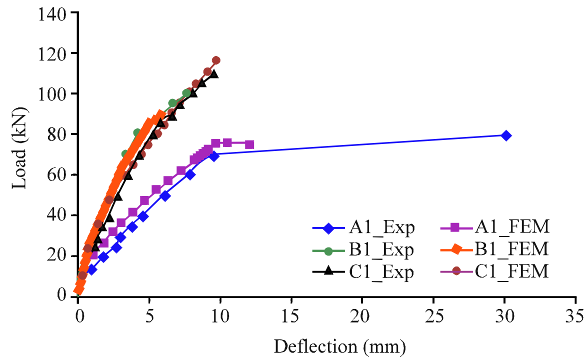

Fig. 13: Comparison of load level versus deflection curve based on experimental and numerical results for all beams

The deflection at failure of beam A1 was found to be bigger than the deflection of beam B1 and beam C1. This could be due to the fact that B1 and C1 failed by plate debonding with brittle failure mode without any warning compared to the flexural failure mode of beam A1.

Cracking patterns

Cracking load: The experimental first crack loads were obtained by observations and are shown in Table 3. It can be seen from the table that the cracking loads of the strengthened beams are higher compared to the control beam. Beam B1 had higher first crack load compared to C1 due to the higher stiffness.

Crack spacing: The total number of cracks in beam A1, B1 and C1 were 11, 15 and 20, respectively. The average cracks spacing of the beams are 182, 133 and 100 mm, respectively. The strengthened beams had smaller crack spacing compared to the control beam. CFRP laminate strengthened beam (C1) had lesser crack spacing compared to the steel plate strengthened beam (B1).

Crack width: Figure 12 shows the load versus maximum crack widths of all the beams. The strengthened beams B1 and C1 had lesser crack widths than the control beam (A1).

The crack width of beam B1 was less than beam C1 which may due to the higher stiffness of beam B1. However, after 90 kN load the crack width of beam C1 became less than the crack width of beam B1 which is could be due to more crack appearing in beam C1 than beam B1.

Comparison of experimental and numerical results

Mode of failure and failure load: Table 4 shows the failure loads and modes of failure of the beams based on experimental and numerical results. The results for the nonlinear finite element analysis shows that the failure modes for all the beams were of flexural in nature rather than premature plate debonding failure as found from the tests. This is because of the assumption of perfect bond between strengthening materials and concrete surface.

The numerical value of failure load of the control beam was almost similar with the experimental result. However, the failure loads of strengthened beams from numerical result are higher when compared to the experimental results. This could be due to the flexural failure assumptions of the strengthened beams rather than premature debonding failure.

Deflection: The load versus deflection curves based on the experimental and numerical results of all the beams are shown in Fig. 13. It can be seen form the Fig. 13 shows that deflections based on numerical analysis are almost identical with the experimental results and all the beams gave linear, elastic portions of the curves at the initial stages. All the strengthened beams showed smaller deflection compared to the control beam due to their higher stiffness.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions that can be drawn from the present study are;

- All strengthened beams gave higher failure loads compared to the control beam. The CFRP laminate strengthened beam had higher failure load compared to the steel plate strengthened beam even though both beams were designed for the same strength
- The failure loads of all the strengthened beams were found to be higher than the design load
- The control beam failed in a ductile flexural manner. The steel plate strengthened beam failed by plate end interfacial debonding followed by concrete cover separation failure. The CFRP laminate strengthened beam failed due to concrete cover separation
- The strains of the reinforcement bars and the concrete of all the strengthened beams were less compared to control beam. Steel plate strengthened beam showed lesser bar and concrete strain compared to the CFRP laminate strengthened beam
- All strengthened beams showed lesser deflections compared to the control beam. Steel plate strengthened beam had lesser deflections compared to CFRP laminate strengthened beam.
- The cracking load of the control beam was found to be less than the strengthened beams. The steel plate strengthened beam gave a higher cracking load and smaller crack widths compared to the CFRP laminate strengthened beam
- The results of all beams based on the nonlinear finite element analysis are almost identical to the experimental results

In this research plate bonding methods were used for strengthening of reinforced concrete beam. However, premature end peeling is the main weakness of this method. End anchors have significant effect to eliminate this debonding failure. Design of end anchors is recommended in future works to overcome the above shortcomings. Further, this research work is limited to testing strengthened beams under static loading condition. For maintenance of elevated highways, bridges and offshore structures which are subjected under repeated loading and fatigue condition, investigation of strengthened beams under these condition is important. Hence, it is recommended that future works look into the behaviour of strengthened beams under repeated loading conditions.

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