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Upgraded the Performance of the Fortifying R.C. beam by CFRP In Flexural

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Abstract: *The use of carbon fiber fortified polymer (CFRP) has been comprehensively utilized for the reinforcing of concrete structures. Some development organizations utilize CFRP in flexural fortifying of reinforced concrete beams, with bonded fully covering surface, without taken into account the concrete cover thickness and the low tensile strength of the concrete. Consequently, the point of this investigation is examine, through an exploratory program. Twelve reinforced concrete (RC) beams strengthened with different scheme bonded surface CFRP sheet and two un-strengthened beam (CB) were prepared and tested. The basic parameters of fortified concrete beam flexural fortified by CFRP are the concrete cover thickness, replacement the concrete cover by high strength material (epoxy adhesives material) and the contribution externally bonded U-jacket CFRP sheet around the flexural strengthening longitudinal strips. The results showed that the RC beams strengthened by CFRP strips with replacing the concrete cover by high strength material greater stiffness and load capacity than the fortified beams by CFRP strips at the concrete cover. The use of transverse U-Shaped CFRP sheet for the fortifying of RC beams increased their load carrying capacity and stiffness. The flexural strength of the tested beams was compared with strengths according to the practice ACI 440.2R-08 and fib analytical formulations.*

Keywords: *Flexural behavior, Cover, Bond, Wrapping, Load capacity, CFRP.*

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I. Introduction

A great part of the concrete structures are indicating huge crumbling and pain. The utilization of remotely fortified fiber-strengthened polymers (CFRP) is turning into a prevalent methods for repair and restoration to broaden the administration life of such structures [1 to 24]. In any case, existing exploratory research demonstrates that the hypothetical high extreme limit of CFRP fortified strengthened concrete (RC) beams frequently can't be accomplished as a result of CFRP plate debonding or even breaking and consequent loss of concrete cover beneath the reinforced steel. This fundamentally lessens the quality improvement gave by the CFRP and can make sudden failure. Subsequently, there is noteworthy worry in the design of externally bonded CFRP strengthening of RC structures. Debonding disappointment of CFRP strengthened RC beams as a rule happens by means of zones of high stress focuses. These are normally connected with CFRP material end and the nearness/form of cracks in the concrete substrate (Fig. 1). The way of debonding spread (see Fig. 2) relies upon properties of the substrate concrete, CFRP, and interface (bonding agent) and takes after the smallest resistance. CFRP debonding through the concrete substrate was recognized as a critical disappointment mode since it happens at lighter load ranks and the failure is inelastic failure. Early research showed uncalled for determination of glues progresses the probability of failure, and that the failure conduct was intensely subject to the current steel reinforced proportion and the sort of FRP. It has additionally been demonstrated that expanding the CFRP reinforcement however much as could reasonably be expected toward the supports diminishes the capability of debonding, yet does not dispose of it. Buyukozturk et al's. [1] explore uncovered that that both failure load and malleability of pre-cracked (in service) RC beams can be altogether expanded through the expansion of CFRP shear reinforcing which helps the anchorage of CFRP sheets utilized for flexural fortifying; in any case, one of the ranges without a general comprehension is as yet the interface and bond between the CFRP and the concrete substrate.

II. Preceding Research On The Interface Bond Of Frp Attached To Concret

Arduini and Nanni [2] studied a parametric to research the impacts of FRP strengthening on serviceability, forte, and disappointment systems for FRP repaired RC beams. The four disappointment modes they distinguished were (1) FRP rupture when its strain surpasses the maximum strain; (2) Concrete smashing;

(3) FRP debonding from the concrete surface; (4) the disappointment of concrete cover due to the Shear tension at FRP end. Disappointments (1) and (2) are characteristic of alluring basic execution and occur after extensive deflections. Sorts (3) and (4) are brittle and happen at loads much lower than expected utilizing regular design methods. The lessening of beam deflection under service loads is emphatically affected by the FRP thickness and its stiffness. Other shifting variables were irrelevant for decreasing the maximum deflection.

For strengthening beams, the result of notification was the proportion between extreme loads of repaired beams contrasted with unreinforced beams. For the beams strengthened with the greatest stiffness FRP, the ultimate strength proportion is unequivocally affected by the bonded length to shear span proportion. At the point when this proportion was under 0.65, there was for all intents and purposes no advantage to repairing the beam for strength. At the point when the FRP was 0.1 mm thick the disappointment mode was dependably FRP rupture free of the bonded length to shear span proportion. With a thickness of 0.5 mm, crack occurred for the longest bonded length. Every single other case brought about bothersome shear strain disappointment mechanisms (concrete cover division). This disappointment additionally shows that it is never again conceivable to expand the flexural capability by increasing the FRP thickness. Arduini and Nanni [2] noticed that more slight adhesive layers bring down the odds of a concrete adhesive interface disappointment.

Studies have demonstrated that an extension in the bonded length is not effective in increasing the maximum transferrable load to the bonded FRP or a void deboning [4]. Subsequently, different procedures are expected to build the viability of the FRP and member strength. It ought to be noticed that an exhaustive comprehension of the debonding procedure and other FRP disappointment modes is required to assess the need for anchorage in every circumstance. The debonding disappointment modes appeared in Fig. 1, particularly concrete cover separation, have been as often as possible recorded. The present way to deal with preventing debonding disappointment is to constrain the plan strain in the FRP to levels considerably less than the failure strain [7], which consequently, limits the efficiency of the fortifying system. It should likewise be noticed that expanding the number of layers of FRP can lessen the strengthened member ductility.

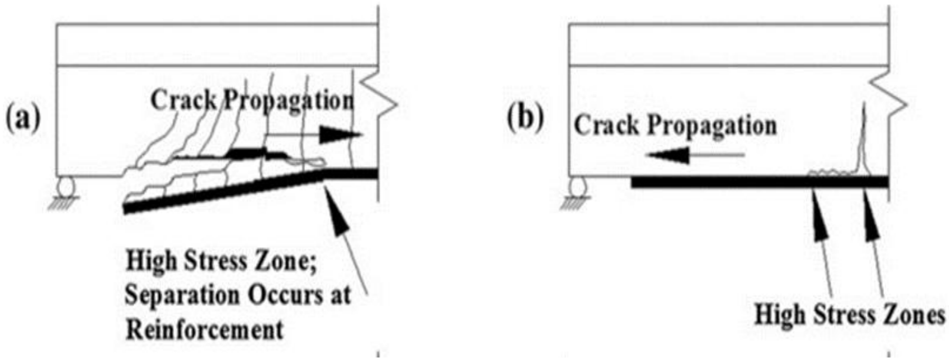


Fig. 1 FRP debonding failure modes of fortified RC beams

Harmon et al. [5] directed four-point bending tests on five strengthening beams with bonded surface CFRPs with various fiber and gum structures. These researchers achieved a few conclusions based on their experiment. The bond layer's thickness and shear modulus are basic in bond effecting. Controlling the thickness is imperative to avoid premature disappointment. The bond strength is restricted by the concrete and is relative to square root of it.

Mohammed [6] investigated flexural behavior of strengthening concrete beams using bonded CFRP strips with or without outside anchorages. The test comes about demonstrated that ultimate loads were increased up to 111.76% the strengthening beams with external anchorage bonded CFRP sheet over the un- strengthened beam. Likewise, these fortified beams demonstrated a lower deflection at comparing loads as for the un-fortified strengthened beam. The debonding or failure of the concrete cover occurs, because of the existent tensile normal forces that related to certain debonding failure modes. The anchorage system at the end of FRP sheet is used for preventing ‘plate-end’ interfacial debonding and concrete cover separation (Fig. 2).

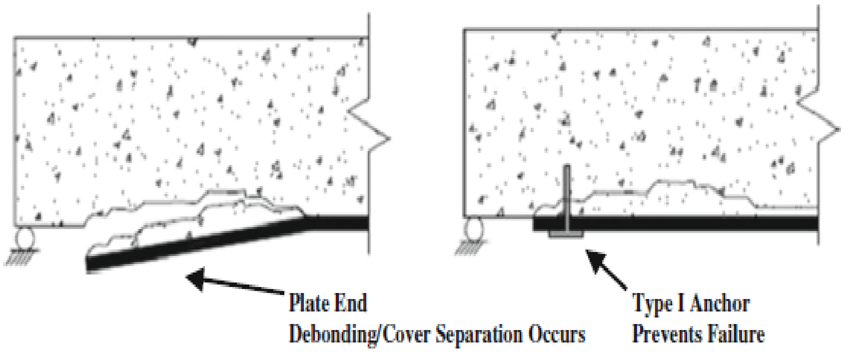


Fig. 2 The anchorage FRP to prevent debonding failure modes of fortified RC beams

Duthinh and Starnes [3] tested seven cracks concrete beams fortified with CFRP laminate. They concluded that the additional strength by CFRP laminate is inverse proportion to the increase of internal reinforced steel. Using wrapping at the ends of FRP laminate with adhesive bonding is effective in preventing the debonding failure of the laminate [3].

In a few circumstances, wrapping reinforced FRP transversely with another FRP sheet will give a bracing impact prove by strains estimated in the wrapped FRP [8], consequently giving a type of anchorage. Transverse wrapping can be as discrete strips situated at the overlay end or along its length or as ceaseless along the length. Fiber direction might be vertical to the longitudinal axis of the beam or might be inclined. A case of transverse wrapping is appeared in Fig. 3. It is essential to take note of that transverse wrapping anchorage haven isn't effective until the point when a specific level of tensile stress is come to in the wrap. Accordingly, it might be attractive to prestress the transverse wraps so as to produce a higher bracing control. While prestressing of surface-reinforced FRP has been somewhat unsuccessful practically, substitution ideas have been researched [9; 10]. Like anchor spikes, the material utilized as a part of a transverse wrap can be the same as the strengthening material, which disposes of potential erosion risks that can come about because of different materials. Establishment of the wrap, be that as it may, might challenge because of part geometry and access to its neighboring sides. Transverse wrapping anchorage has been looked into broadly, incorporating into thinks about by [11 to 21].

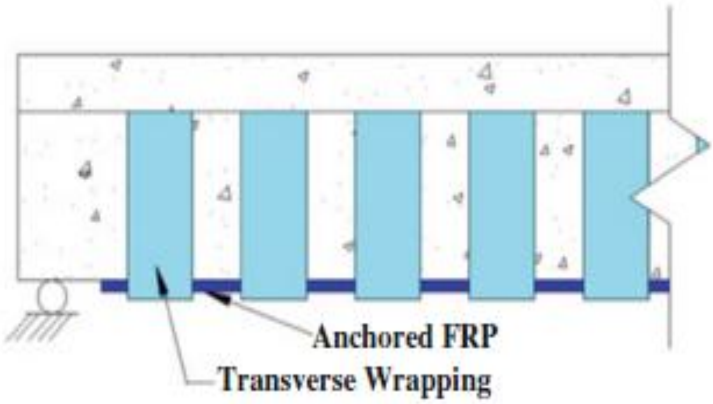


Fig. 3 Transverse wrapping of fortified beam with FRP

III. Research Significance

This paper studies experimentally how Upgraded the performance of RC beams external fortified in flexural with CFRP sheet. The first objective of research is to avoid or delay the process of debonding, which occurs when externally CFRP sheet separates from the RC substrate because of the low tensile strength of concrete by studying the effect of bonding for the CFRP sheet with epoxy material that replacing the concrete cover. The second objective is to study effect of increase in concrete cover in the performance of CFRP strengthening in flexural. The results are compared with ACI 440.2R-08 [7] and fib [24].

IV. Experimental Details

Fourteen specimens were tested in this program, a control specimen, and thirteen specimens strengthened using two different bonding and strengthening techniques (the conventional epoxy bonding CFRP with concrete cover, the conventional epoxy bonding CFRP with replacing concrete cover with epoxy and the wrapping anchorage technique).

1. Description of the beams

Fourteen reinforced concrete beams having total length of 1.70 m were divided into two Groups (I and II) in this study. The groups I and II had a cross section of 100mm x 200 mm with effective depth (d) 185 mm (concrete cover 15 mm), and 100mm x 250 mm with effective depth (d) 185 mm (concrete cover 65 mm) respectively. For flexure reinforcement two 10 mm deformed bars with yielding stress 360 MP and 8 mm with yielding stress 240 MP were used as top and bottom reinforcement respectively. The shear reinforcement consisted of 6 mm stirrups with yielding stress 240 MP (Fig. 4).

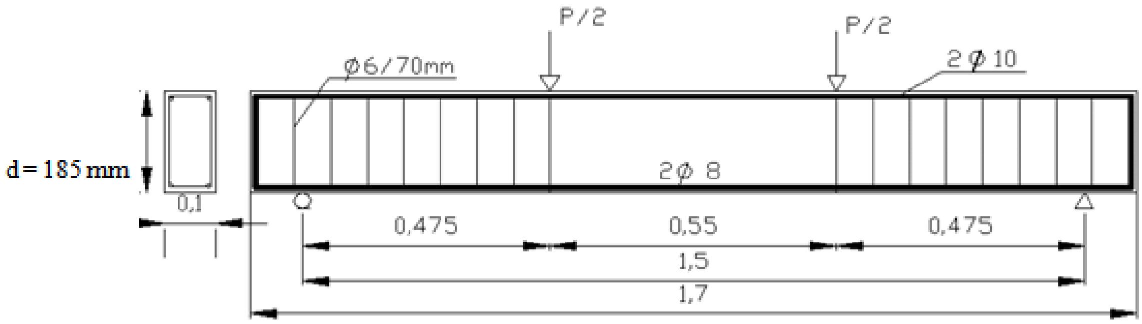


Fig. 4 The reinforcement details of the fortified beams.

Fourteen reinforced concrete beams were tested. The first group I consist of six specimens with effective depth 185 mm and cover 15mm; ‘‘Control B1’’ was a control specimen without any strengthening. The second specimen ‘‘B1-1’ was strengthened in flexural using CFRP sheet (three layers) of width 100 mm and externally bonded to the bottom of the beam using epoxy resin only (tradition scheme). Fig. 5 shows the operation of strengthening. The third specimen ‘‘B1-2’’ was strengthened in flexural using CFRP sheet (three layers) and bonded using epoxy to two groove filling with kemapoxy 165 with dimensions 100x250 mm and thickness 15 mm (concrete cover) to enhance the bonding technique. The fourth specimen ‘‘B1-3’’ was strengthened in flexural using CFRP sheet(three layers) and bonded using epoxy to three groove filling with kemapoxy 165 with dimensions 100x250 mm and thickness 15 mm (concrete cover) to enhance the bonding technique. The five specimen ‘‘B1-4’’ was strengthened in flexural in same technique with specimen ‘‘B1-1’ with addition four wrapping bonded FRP of 50 mm width transversely U shape around the tension face and web after the CFRP sheets were attached to the bottom side of the tested beam. The six specimen ‘‘B1-5’’ was strengthened in flexural using CFRP sheet (three layers) externally bonded to each side of beam with 50 mm height.

The second group II consist of six specimens with effective depth 185 mm and cover 65 mm to test the effect of increasing cover in strengthening; ‘‘Control B2’’ was a control specimen for group II without strengthening. The second specimen ‘‘B2-1’ was strengthened in flexural using CFRP sheet (three layers) of width 100 mm and externally bonded to the bottom of the beam using epoxy resin only. The third specimen ‘‘B2-2’’ was strengthened in flexural using CFRP sheet (three layers) and bonded using epoxy to two groove filling with kemapoxy 165 with dimensions 100x250 mm and thickness 65 mm (concrete cover) to enhance the bonding technique. The fourth specimen ‘‘B2-3’’ was strengthened in flexural using CFRP sheet(three layers) and bonded using epoxy to three groove filling with kemapoxy 165 with dimensions 100x250 mm and thickness 65 mm (concrete cover) to enhance the bonding technique. The five specimen ‘‘B2-4’’ was strengthened in flexural similar to specimen ‘‘B1-4’. The six specimen ‘‘B2-5’’ was strengthened in flexural similar to specimen ‘‘B1-5”. The seven specimen ‘‘B2-6’’ was strengthened in flexural using CFRP sheet (three layers) externally bonded to the bottom of the beam with U shape around the two side of web with height 100 mm using epoxy resin. The eight specimen ‘‘B2-7’’ is similar to specimen ‘‘B2-2’’ and the difference in the removing cover (100x400 mm and thickness 65 mm). Table 1 provides a summary of the details of the tested beams.

Table 1. Strengthening schemes of tested beams

GROUP	Spec.	Effective depth mm	Cover Thickness mm	Strengthening schemes	Cross section Strengthening
I	B1	185	15		Control "without any strengthening"
	B1-1	185	15		Strengthening three layers at concrete surface (tradition scheme)
	B1-2	185	15	250 250	Strengthening three layers at partial kemapoxy 165 (2x (100x250mm))
	B1-3	185	15	250 250 250	Strengthening three layers at full kemapoxy 165 (3x (100x250mm))
	B1-4	185	15		Strengthening three layers with U CFRP strip around the CFRP flexural layers
	B1-5	185	15		Strengthening three layers at each side With 50 mm height.
II	B2	185	65		Control "without any strengthening"
	B2-1	185	65		Strengthening three layers at concrete surface
	B2-2	185	65	250 250	Strengthening three layers at partial kemapoxy 165 (2x (100x250mm))
	B2-3	185	65	250 250 250	Strengthening three layers at full kemapoxy 165 (3x (100x250mm))
	B2-4	185	65		Strengthening three layers with U CFRP strip around the CFRP flexural layers
	B2-5	185	65		Strengthening three layers at each side With 50 mm height.
	B2-6	185	65		Strengthening three layers with U CFRP around the bottom surface With 100 mm height.
	B2-7	185	65	400 400	Strengthening three layers at partial kemapoxy 165 (2x (400x100mm))

2. Material

The cylindrical compressive strength of the concrete that used in the study was 25 Mpa. The thickness of CFRP fabrics (SikaWrap-230C) is 0.131 mm per layer were utilized. The elastic modulus, tensile strength, and elongation for CFRP equal 238 Gpa, 4300 Mpa, and 1.8 %, respectively. The saturation resin for the applied fiber is a two unit; thixotropic epoxy based saturating sap/cement (Sikadur-330) with thickness 1.3 kg/l, with elastic modulus and tensile strength equal 4500 Mpa and 30 Mpa respectively. Compressive strength (ASTM D 695) and Flexural strength (ASTM D 790) of kemapoxy equal 80 and 40 Mpa respectively.

3. Test setup and instrumentation

All specimens were tested under four point bending. The clear span of the beams was 1.50 m and the distance between the loads was 0.55 m. The deflection and the strain for CFRP sheet at mid-span were measured using one dial gauge and strain gauge respectively. Loading was applied manually by a hydraulic jack at increments of 2.5 Kn, Fig. 6 shows the test setup for the specimen.

Fig. 5 the application of epoxy to the concrete surface for CFRP sheet bonding.

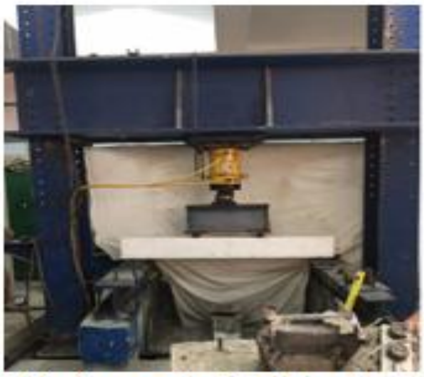


Fig. 6 Set-up for four-point bending test in laboratory.

V. Results And Discussion

1. Crack pattern and failures modes

Fig. 7 shows the failure form of the tested beams. For the control specimens(B1, B2) ‘‘without strengthening’’, the first visible crack appeared at a load of about 2.5 kN, the cracks extended from the beam bottom and start to stretched out from beam bottom to top of the beam with widening as the loading increased at the whole span of the beam. As loading progressed the crack widened. The specimens failed at a load of 14 and 15.9 kN for B1 and B2 respectively. For the Specimens (B1-1, B2-1) flexural strengthened with CFRP sheet and bonded with epoxy resin only, The first visible crack appeared at a load of 5 and 7.5 kN at mid span respectively, A horizontal crack at one of the ends of the glued CFRP sheet beneath the interior steel level and spreads to mid-span, leading with final complete separation of the CFRP sheet with the nearby concrete cover at 30 kN (cover delamination of concrete) For the specimens (B1-2, B2-2), which are flexural strengthened with CFRP sheet and bonded with epoxy with replacing the concrete cover with kemapoxy 165 only, at a load of 2.5 kN the first noticeable crack performed at the mid span of the beams. As loading progressed cracks widened then the specimen B1-2 finally failed by wide shear crack at a load of 35 kN, the specimen B2-2 failed at load 38.7 kN with separation of the CFRP sheet with the adjacent kemapoxy cover layer and concrete cover.

For the Specimens (B1-3, B2-3), the first visible crack appeared at a load of 2.5 kN at the mid span of the beams. As loading progressed cracks widened then the specimen B1-3 finally failed by wide shear crack at a load of 46 kN, the specimen B2-3 failed at load 56 kN with separation of the CFRP sheet with the adjacent kemapoxy cover layer. The first visible crack for the specimens (B1-4, B2-4) appeared at a load of 5 kN at the mid span of the beams. The cracks extended from the beam bottom and start to spread from beam bottom to the beam top under the point load. The specimens failed at a load of 67.5 and 80 kN for B1-4 and B2-4 respectively with rupture of FRP sheet.

For the Specimens (B1-5, B2-5), there is no visible crack appeared and the failure occurred in such a way that the CFRP plate split apart from the concrete along a horizontal side surface with several hair cracks at mid span starting from a wide flexural -shear crack. As loading progressed cracks widened then the specimens finally failed at loads 55 and 60 kN for specimens B1-5 and B2-5 respectively. The specimens B2-6 failed at load 80 kN, the failure of B2-6 is sudden failure with shear cracks near the support and separation of CFRP from the side face of beam near the support due to high shear stresses between the concrete and the CFRP sheet. The specimen B2-7 failed at load 70 kN with separation of the CFRP sheet with the adjacent kemapoxy cover layer and concrete cover.

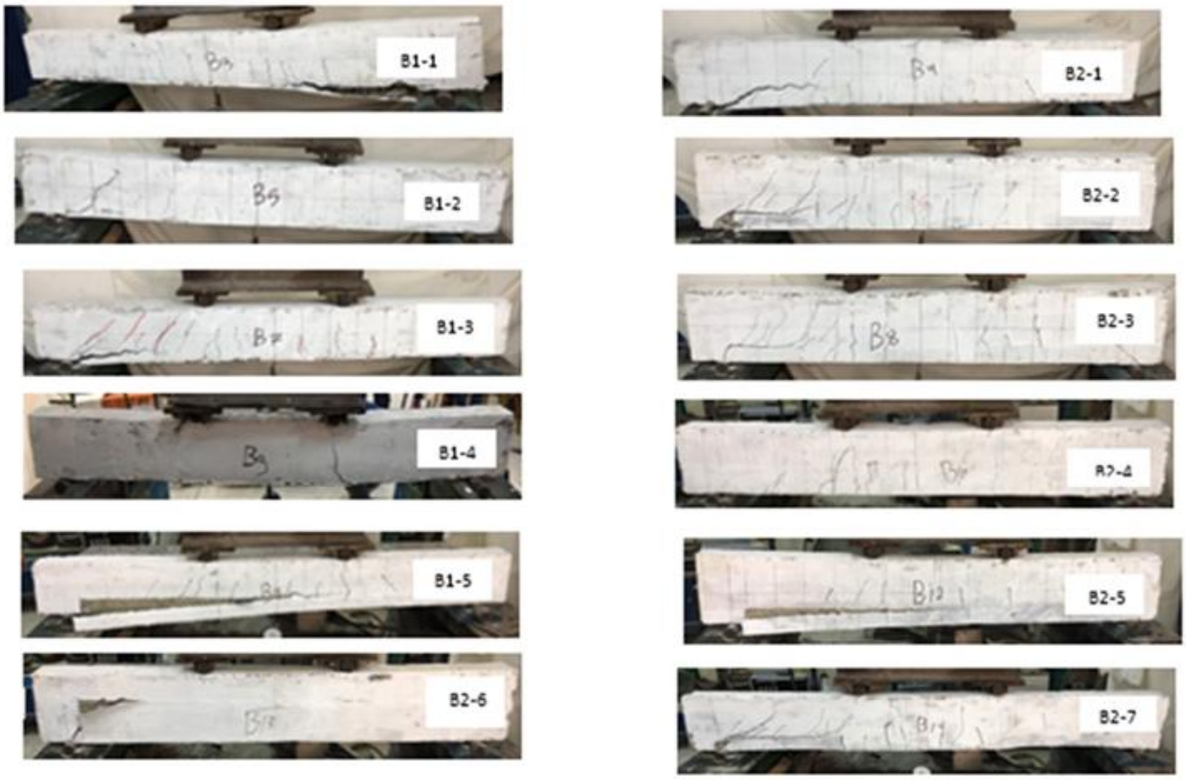


Fig. 7 Failure pattern of tested beams

2. Load–deflection, and load–strain relationships

The beams were tested under four point bending (4PB) up to failure. Figures 8, 9, and 10 show the load-mid-span deflection responses obtained from the tests. Tables 2 and 3 summarize the experimental results in terms of the ultimate load (Pu), mid-span deflection at ultimate load (δu), the percent of increase in ultimate loads (% Pu) compared to that of the control beams and the percent decrease of deflection at ultimate loads (% δu) compared to that of the control beams . The ultimate load and the maximum deflection of the control (B1, B2) and the strengthening specimens (B1-1, B2-1) are used as benchmarks for measuring the performance of other beams.

It is clear from Figures 8, 9 and Tables 2 and 3 that the ultimate load of the control beam (B2) increased slightly about 1.9 % of the control beam (B1), the strengthened beams for group I have larger post cracking stiffness and load capacity than those of the control beam (B1). It is observed that the increase in the ultimate (peak) load of the strengthened beams ranged from 92.3% to 252.5 % of the un-strengthened control RC beam (B1), The strengthened beams failed at 30, 35, 46, 67.5 and 55 kN, for beams B1-1, B1-2, B1-3, B1-4 and B1-5 respectively. The failure load of beams B1-1 and B1-2 increased 92.3 and 124 % over the control beam B1 respectively. The beam B1-2 that strengthened by bonding CFRP on kemapoxy surface had 16% increase in load failure over the strengthening specimen B1-1 (tradition scheme) but did not prevent the delamination failure. The ultimate load of the strengthening beam B1-3 improved due to the increase in contact length between the CFRP and the kemapoxy surface. The increasing in failure load for beams B1-1, B1-2 and B1-3 due to the external strengthening is accompaniment by catastrophic brittle failure due to surprise delamination of kemapoxy with CFRP sheet. The load failure of the strengthening B1-3 increased to 194.8 and 53 % of the control beams B1 and B1-1 respectively. The beam B1-4 that strengthened with addition U strip CFRP had 332.6 and 125 % increase in failure load over the B1 and B1-1 respectively. For strengthening beam B1-5, the failure load increased by 252.5 and 83% with respect to B1 and B1-1 respectively.

The deflections of the strengthened beams corresponding to the failure loads were less than the corresponding of the control beam. The reduction in deflection at ultimate load for B1-1, B1-2, B1-3, B1-4 and B1-5 reached to 53.6 %, 64.2%, 60.7%, 28.6 and 32.9% respectively, with respect to the un-strengthened control beam (B1). The deflection at ultimate load for, B1-2, B1-3, reduced 23% and 15.3% with respect to the strengthening beam B1-1, whereas for beams B1-4 and B1-5 increased 53.8%, 44.6% respectively. Fig. 9 shows that the control beam was stiffened substantially by the strengthening. The different schemes had marginal effect on the beam stiffness. Beam B1-5 exhibited lower stiffness than all strengthening beams, while Beam B1-4 exhibited higher stiffness. The load–deflection behavior of the control beam and beam strengthened with CFRP sheet with different bonding techniques for Group II are shown in Fig. 10 and table 3. It is observed that the increase in the ultimate (peak) load of the strengthened beams ranged from 88.6 % to 403 % of the un-strengthened control RC beam (B2), The strengthened beams failed at 30, 38.7, 56.0, 80.0, 60.0, 80.0 and 70.0 kN, for beams B2-1, B2-2, B2-3, B2-4, B2-5, B2-6 and B2-7 respectively.

The failure load of beams B2-1 and B2-2 increased 88.6 and 143 % over the control beam B2 respectively, the percentage of increase for beam B2-1with compared to the similar strengthening specimen in group I (B1-1) decreased due to the increase in concrete cover. The specimen B2-2 that strengthened by bonding CFRP on kemapoxy surface had 29% increase in load failure over the B2-1 bonding of CFRP on concrete surface (tradition method) due to the enhanced mechanical properties of glued surface, the delamination failure occurred for both the specimens B2-1 and B2-2. It is observed that there is an increment in the ultimate experimental load once the length between the CFRP and the kemapoxy surface is increased, the percentage of increase in the ultimate load for beams B2-3 and B2-7 is 86% and 133% of the strengthening beam B2-1 respectively. It can be inferred from the test results that the debonding load was dependent on strengthening materials. It is observed that the strengthened beams B2-7 increases load even more significantly relative to the strengthened beams B2-3 due to the location of replacing the concrete cover with kemapoxy near the end of CFRP sheet.

It can further be seen that for the strengthening beams B2-1, B2-2, B2-3 and B2-7, the final pattern failure occurred by delamination of kemapoxy with CFRP sheet near the end of CFRP sheet, the ductility of all strengthened beams was reduced in comparison with their respective control beam B2. the specimen B2-4 that strengthened with addition U strip CFRP had 403 and 166 % increase in failure load over the B2 and B2-1 respectively, the increase in cover is more effective in upgrading strengthening technique, because of the increasing in the lever arm of the moment capacity, it can be noticed that the arrangement of the U-shape strip CFRP results in an increase in the ultimate capacity and ductility of the RC beams. For strengthening beam B2-5, B2-6 the failure loads increased by 100 and 166 % with respect to B2-1 respectively, the strengthening technique with full CFRP rounded U form (B2-6) is more effective similar to U strip CFRP (B2-6).

According to table 3, the deflections of the strengthened beams corresponding to the failure loads were less than the corresponding of the control beam. The reduction in deflection at ultimate load for B2-1, B2-2, B2-3, B2-4, B2-5, B2-6 and B2-7 reached to 58.1%, 77.2%, 63.7%, 45.6%, 58.1%, 71.2% and 74.3% respectively, with respect to the un-strengthened control beam (B2). The deflection at ultimate load for, B2-2, B2-3, reduced 45.6 % and 13.4 % with respect to the strengthening beam B2-1, whereas for beam B2-4 increased 29.8%. Fig. 10 shows that the control beam was stiffened substantially by the strengthening. The different schemes had marginal effect on the beam stiffness. Beam B2-1 exhibited lower stiffness than all strengthening beams, while Beam B2-4 and B2-6 exhibited higher stiffness.

Table 2. Test results for Group I

Spec.	Pu kN	δu mm	% Pu Increase Over B1	% Pu Increase Over B1-1	% δu decrease Over B1	% δu Decrease or (+) increase Over B1-1	Strain of CFRP at failure load	Failure Mode
B1	15.6	14	0.0	-	0.0	-	-	failure by bending
B1-1	30.0	6.5	+92.3	0.0	-53.6	0.0	0.00231	failure by cover delamination of concrete
B1-2	35.0	5.0	+124	+16	-64.3	-23	0.00303	failure by delamination of kemapoxy with CFRP sheet
B1-3	46.0	5.5	+194.8	+53	-60.7	-15.3	0.00461	failure by delamination of kemapoxy with CFRP sheet
B1-4	67.5	10.0	+332.6	+125	-28.6	+53.8	0.01412	failure by Rupture of FRP sheet
B1-5	55	9.4	+252.5	+83	-32.9	+44.6	0.00592	failure by debonding of CFRP sheet

Table 3. Test results for Group II

Spec.	Pu kN	δu mm	%Pu Increase Over B2	%Pu Increase Over B2-1	% δu decrease or Over B2	% δu Decrease (+) increase Over B2-1	Strain of CFRP at failure load	Failure Mode
B2	15.9	16.0	0.0	-53	0.0	-	-	failure by bending
B2-1	30.0	6.7	+88.6	0.0	-58.1	0.0	0.00150	failure by cover delamination of concrete
B2-2	38.7	3.64	+143	+29	-77.2	-45.6	0.00242	failure by delamination of kemapoxy with CFRP sheet
B2-3	56.0	5.8	+252	+86	-63.7	-13.4	0.00426	failure by delamination of kemapoxy with CFRP sheet
B2-4	80.0	8.7	+403	+166	-45.6	+29.8	0.01315	failure by Rupture of FRP sheet
B2-5	60.0	6.7	+277	+100	-58.1	0.0	0.00469	failure by debonding of CFRP sheet
B2-6	80.0	4.6	+403	+166	-71.2	-31.3	0.00345	shear failure with separation of CFRP from the side face of beam
B2-7	70.0	4.1	+340	+133	-74.3	-38.8	0.00575	failure by delamination of kemapoxy with CFRP sheet

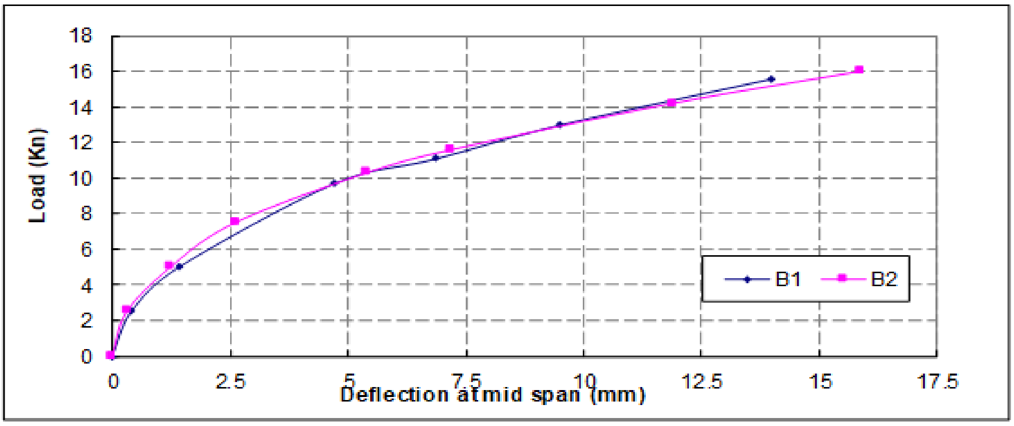


Fig. 8 Load-mid-span deflection for control specimen

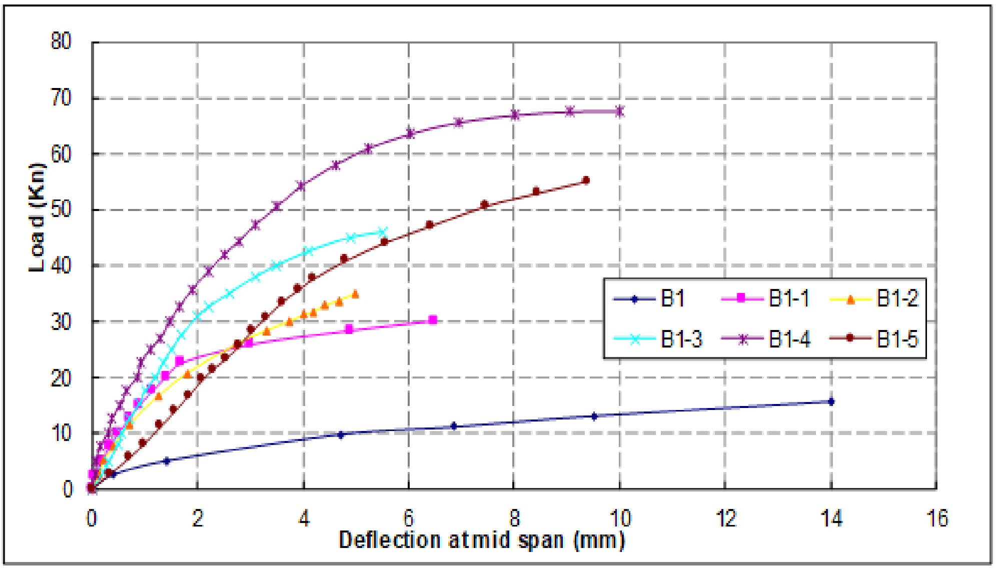


Fig. 9 Load-mid-span deflection for Gr

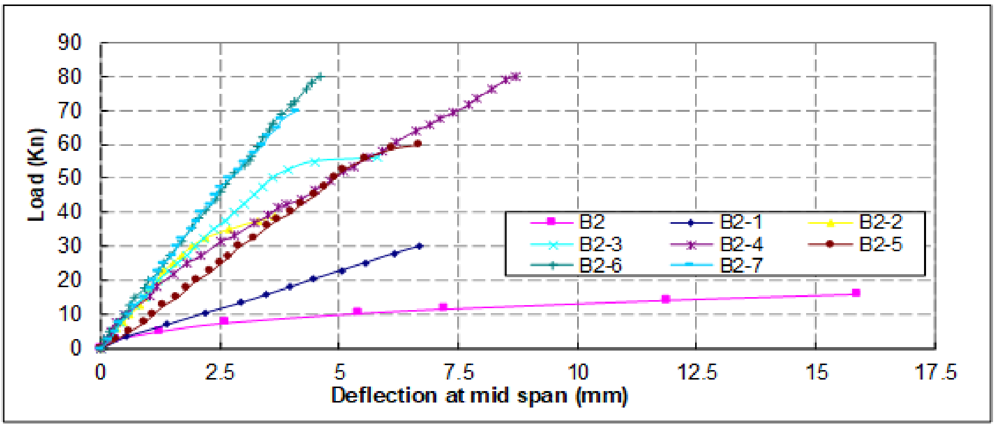
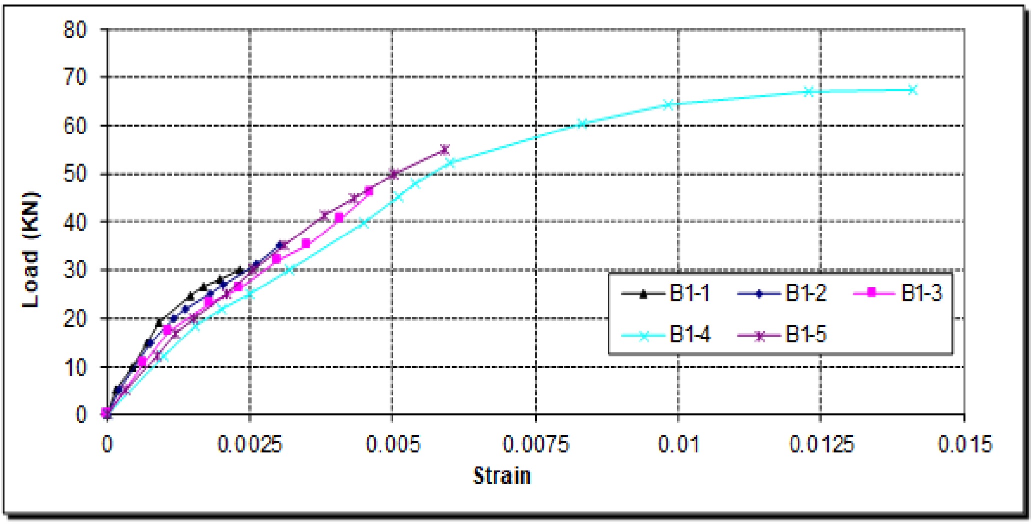


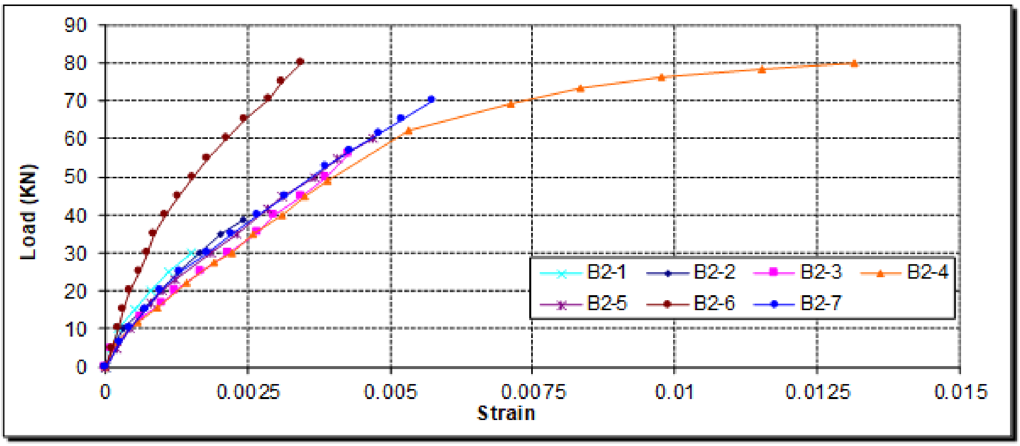
Fig. 10 Load-mid-span deflection for Group II

3. *Load-FRP Strain Relationships*

Strain readings in the FRP at mid-span were recorded from the attached strain gages during testing using a data gaining system. Figs.11, 12 and tables 2, 3 show the load versus FRP strain for the strengthened specimens for groups I and II respectively. It can further be seen that for the strengthening beams (Group I) B1-1, B1-2, B1-3, B1-4 and B1-5, the FRP strain at failure is 0.232%, 0.303%, 0.461%, 1.412% and 0.592%, respectively, this indicates that 12.88%, 16.82%, 25.59%, 78.44% and 32.86% of the capacity of the CFRP sheet was utilized. Whereas the strengthening beams (Group II) B2-1, B2-2, B2-3, B2-4, B2-5, B2-6 and B2-7, the FRP strain at failure is 0.150%, 0.242%, 0.426%, 1.315%, 0.469%, 0.345% and 0.575%, respectively, this indicates that 8.33%, 13.44%, 23.66%, 73.05%, 26.05%, 19.16 and 31.94% of the capacity of the CFRP sheet was utilized. The strain of the strengthening specimens B1-1 and B2-1(tradition method) at failure was lower than that of the other strengthening specimens due to separation of CFRP with cover from the concrete substrate. To acquire higher extreme strain on flexural fortifying of beams, one solution would be to embrace a suitable anchor system for this fortifying to prevent the concrete cover pull out, as referred by Beber [23], the replacing the concrete cover with epoxy delayed the pull out cover and contributed to use the CFRP capacity.



Figs.11 Load-strengthening strain at mid-span for Group I



Figs.12 Load-strengthening strain at mid-span for Group II

VI. The Anticipations Of Ultimate Load-Carrying Capacity Based On Aci 440.2r-08 And Fib Analytical Formulation

The actual strain of the strengthening material (CFRP), εfe, according to ACI [7] is gauged conferring to the following equation:

$$\varepsilon_{fe} = 0.003\left(\frac{h-c}{c}\right) - \varepsilon_{ci} \ll k_m\, \varepsilon_{fu} \qquad (1)$$

Where εci the initial is strain of the concrete substrate and is assumed to be zero during the installation of the CFRP system, because no load was applied to the beam specimens, εfu is the CFRP design maximum strain and km is bond coefficient and can be calculated from the following equations:

$$k_m = \begin{cases} \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{nE_f t_f}{360000}\right) \ll 0.90 & for\;\; nE_f t_f \ll 180000 \\ \frac{1}{60\varepsilon_{fu}}\left(\frac{90000}{nE_f t_f}\right) \ll 0.90 & for\;\; nE_f t_f > 180000 \end{cases} \qquad (2)$$

Where n is the number CFRP flexural plies, and tf is the CFRP thickness.
The nominal flexural capacity of the strengthened section can be determined from the following equation:

$$\emptyset M_n = \emptyset\,[A_s\, f_s\left(d - \frac{a}{2}\right) + \varphi\, A_f\,(E_f\, \varepsilon_{fe})\left(h - \frac{a}{2}\right)] \qquad (3)$$

Where the As and Af are the area of the steel and CFRP reinforcement respectively, fs is the stress in the steel reinforcement, d is the distance from the compression zone to the tensile reinforcement, a is the depth of the rectangular stress block, Ef is the tensile modulus of elasticity of the CFRP reinforcement, h is the overall thickness of the member. For failure control, the reduction factor for both steel yield and FRP rupture is $\emptyset$= 0.9 and reduction factor φ = 0.85, is applied for CFRP system.

In the fib approach (24) the extreme CFRP force may be calculated using the following equation:

$$N = \propto c_1 k_c k_b \sqrt{f'_c\, nE_f t_f} \qquad (4)$$

Where ∝ is a reduction factor for bond strength, equal to 0.9, c1 is an experimental factor supposed to be 0.64 for CFRP, f'c is the concrete compressive strength, kc is a factor for concrete compaction (kc = 0.67), and kb is a factor depend on the width of the CFRP (bf) and the width of the beam section (b):

$$k_b = 1.06\sqrt{\left(2-\frac{b_f}{b}\right)}/(1+b_f) \gg 1.0 \qquad (5)$$

Tables 4 and 5 compare the measured ultimate load (P_{um}) and expected ultimate load [(P_{uea}) by ACI 440.2R-08 and (P_{uef}) by fib analytical formulations] capacity of the tested beam specimens and its ratio to the measured ultimate load capacity for group I and group II respectively. It is clear from Tables 4 and 5 that the predicted results using the fib provisions for the strengthened schemes B1-4 and B2-4 are very close to the measured experimental values with a maximum deviation of 7% and 6% respectively. The ACI 440 provisions give slightly conservative estimates of the ultimate load capacity (Pu) for the specimens B1-4 and B2-4 by margins of 18% and 22% respectively.

Table 4. Test results for Group I

Spec	Measured P_{um} kN	Expected (P_{uea}) by ACI kN	P_{um} / P_{uea}	Expected (P_{uef}) by fib kN	P_{um} / P_{uef}
B1	15.6	15.58	1.00	-	1.11
B1-1	30.0	80	2.66	62.84	2.09
B1-2	35.0	80	2.28	62.84	1.79
B1-3	46.0	80	1.73	62.84	1.36
B1-4	67.5	80	1.18	62.84	0.93
B1-5	55	80	1.45	62.84	1.14

Table 5. Test results for Group II

Spec	Measured P_{um} kN	Expected (P_{uea}) by ACI kN	P_{um} / P_{uea}	Expected (P_{uef}) by fib kN	P_{um} / P_{uef}
B2	15.9	15.58	0.98	-	0.98
B2-1	30.0	97.67	3.25	75.38	2.51
B2-2	38.7	97.67	2.52	75.38	1.94
B2-3	56.0	97.67	1.74	75.38	1.34
B2-4	80.0	97.67	1.22	75.38	0.94
B2-5	60.0	97.67	1.62	75.38	1.25

VII. Conclusion

Experimental and analytical results to explore the conduct of RC beams fortified in flexure by CFRP sheets are displayed. twelve beams externally fortified in flexure with CFRP sheets, and an un fortified two reference beam were tested under four point bending and the flexural effectiveness of the proposed fortifying method with CFRP sheets were explored. It is watched that the ultimate load carrying capacity using ACI440.2R-08 is marginally near to the experimentally measured values while the fib analytical formulations is very near the exploratory outcomes. The accompanying conclusions are gotten:

1) The increase in the load capacity of the strengthened beams (new schemes) ranged from 16% to 166% of the strengthened control RC beam (tradition scheme (B1-1, B2-1)) depending on the concrete cover thickness and the contact surface area of kemapoxy and CFRP area.
2) The efficacy of strengthening increased with the increase of the replacing the cover thickness with kemapoxy and the longitudinal surface area of kemapoxy, the percentage of increase in the ultimate load for beams B2-3 and B2-7 is 86% and 133% of the strengthening beam B2-1 respectively.
3) The replacing of the concrete cover with kemapoxy is more effective near the end of CFRP sheet in increasing the load capacity of the strengthened beam.
4) The fib provisions gave very accurate prediction of the ultimate load capacity for the beams with strengthening scheme CFRP sheet (B1-4 and B2-4) with a maximum deviation of 7% and 6% respectively, but was not conservative estimates of the ultimate load capacity for both the tradition strengthening (B1-1, B2-1) and the other schemes.

5) The ACI 440 provisions give slightly conservative estimates of the ultimate load capacity (Pu) for the specimens B1-4 and B2-4 by margins of 18% and 22% respectively but was not reasonable for the other schemes.
6) As the concrete cover increase, the load capacity of the strengthened beams (new schemes) increases depending on the scheme strengthening method, while the concrete cover has no effect in the load capacity of the strengthened beams with tradition scheme strengthening (B1-1, B2-1).
7) The maximum reading of the strain in the CFRP is about 38% for the scheme strengthening B2-4 and 42% for the scheme strengthening B1-4 of the maximum CFRP strain.
8) The CFRP fortifying straightforwardly added to the rigidity increase of reinforced beams, diminishing the deflection of the same for s given load.

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