

Methods for Reducing Concrete Cover De-bonding in R.C. Beams Strengthened in Flexure with CFRP Strips

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Abstract –

In many cases beams need repair due to steel corrosion. The traditional method for beam repair due to steel reinforcement corrosion is removing the concrete cover to repair the longitudinal steel bars then recasting it again using new concrete cover which considered as a weak point when this member is strengthened with CFRP strips. This technique may lead to de-bonding of the new concrete cover and the strengthened member will fail in a brittle manner without reaching the full effect of strengthening. This research investigates the effect of the compressive strength of the concrete cover compared with that of the old concrete. It also introduces some new techniques for reducing and preventing the concrete cover de-bonding and compares it with a control beam which had no concrete cover removal. The experimental program consists of seven beams with cross section 150*300 mm and effective span 1200 mm. The beams were designed and reinforced to fail in flexure mode. It was noticed that increasing the concrete cover compressive strength enhanced the mode of failure and increased the ultimate load capacity. Also, Using U jacket of CFRP sheets was an effective technique for bonding new concrete cover and old concrete as it enhanced the mode of failure and led to the same ultimate capacity of the beam strengthened by CFRP strips without removing concrete cover. Bonding new concrete cover and old concrete with bonding agent or steel shear connectors was not sufficient to enhance the failure mode or the ultimate load.

Keywords: Concrete cover de-bonding, CFRP strips, Repairing, Strengthening, Flexure, Corrosion

I. INTRODUCTION

The main problem when using CFRP strips as a flexural strengthening technique for R.C. beams is the concrete cover de-bonding which reduces the efficiency of the strengthening process and leads to sudden failure due to horizontal shear stresses between old and new concrete. There are many modes of failure of the R.C. beams strengthened with CFRP strips. Both of concrete crushing in

compression zone and rupture of FRP in tension zone can be predicted with the traditional method of analysis of R.C. members and provide the full strengthening capacity of the FRP strips. On the other hand when the mode of failure deviates to de-bonding of the FRP laminate from concrete surface or the concrete cover delaminates from the old concrete, the failure is sudden and brittle and in addition the member doesn't reach the ultimate capacity of strengthening [1].

It has been observed that the horizontal shear crack found with concrete cover de-bonding occurs at the depth of the longitudinal steel reinforcement. This mode of failure is different from the so-called peeling-off failure mode, where de-bonding is at the interface between concrete and FRP strips. Concrete cover de-bonding failure is caused by shear and local regions of tension (out-of-plane) stresses at the level of the steel reinforcement. The magnitude of these stresses is influenced by geometrical parameters such as thickness of the external cover, reinforcement ratio, adhesive type, distance from the support to the end of the FRP reinforcement, and stiffness of various components [2-5].

The main objective of this research is to study the effect of strengthening beams with FRP strips, introduce new techniques for reducing and preventing the concrete cover de-bonding in strengthened R.C. beams with FRP laminates and also investigate the effect of the compressive strength of the new concrete cover compared with that of the old concrete.

II. EXPERIMENTAL PROGRAM

A. Test Specimens

In this work seven beams were tested, they were designed to resist shear failure mode and fail in flexural. All beams had the same reinforcement, for longitudinal reinforcement two 10 mm deformed bars were used as top and bottom reinforcement. The shear reinforcement consisted of 8 mm stirrups spaced at 8 mm. Since low ratio of flexural

reinforcement was used, the flexural capacity of all test specimens was lower than the diagonal cracking capacity. Therefore, no shear cracks were expected.

All beams were simply supported with three point loading system having an effective span of 1200 mm. The specimens had a cross section of 150 mm x 300 mm with a total length of 1.60 meters. The concrete compressive strength was 35MPa after 28 days for all specimens.

All beams have been loaded till cracking load according the control beam B1 test results before removing the concrete cover and then strengthening the beam. The first specimen B1 was a control specimen without any strengthening. The second specimen B2 was strengthened in flexure using CFRP strip of width 100mm and externally bonded to the bottom of the beam using epoxy resin only without removing the concrete cover and it was considered as control strengthened beam. The third specimen B3 was strengthened by CFRP strip after removing the concrete cover and recasting it using new concrete with compressive strength 20Mpa without any bonding technique between old and new concrete. The fourth beam B4 was similar to B3 but having bonding agent to bond the old and new concrete. In the fifth specimen, B5, steel studs of 6 mm diameter and 100 mm long were used as shear connectors between old and new concrete as shown in Fig.1. Half lengths of the studs were embedded in the old concrete. The studs were drilled in two rows with 8mm spacing between the stirrups.



Fig.1: Studs in the Old Concrete Before Casting the New Concrete Cover Of B5

The sixth beam B6 was similar to B3 but extra CFRP sheets were used as U jacketing till the mid height of the beam cross section to bond the new concrete cover with the old concrete and also to bond the FRP strip. The U jacket is oriented to the mid height of the beam to avoid any effect on the shear capacity of the beam. The CFRP U jacket was of width 100 mm and spacing 100 mm as shown in Fig.2. For the seventh beam B7, a high strength concrete of 50 MPa was used for casting the new cover using silica fume admixture and a water reducing agent. Also, a bonding agent was used between the old concrete and the new concrete cover. The full details for all tested specimens are mentioned in Table 1.

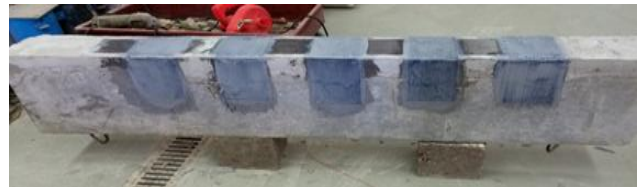


Fig. 2: Strengthening Configuration of B6

B. Materials

A group of laboratory experiments were carried out to determine the physical and mechanical properties of the concrete constituents. The results of these tests were recorded and compared with the Egyptian standard specification (ESS). The CFRP strips have ultimate tensile strength of 3800 MPa, modulus of elasticity 240GPa, thickness 0.176mm, and ultimate strain 1.5% mm/mm according to the manufacturing data sheet. The bonding epoxy used for bonding CFRP strips had ultimate strength of 32 MPa, modulus of elasticity 100 Mpa and compressive strength 60 MPa.

Table 1: Summary of the Details of the Teste Specimens.

	B1	B2	B3	B4	B5	B6	B7
Rein. details	+long. steel: 2T10 *stirrups: 1 R 8 @ 8 cm						
Loading	Until failure	Till the crack load then strengthened and reloaded	Till the crack load then strengthened and reloaded	Till the crack load then strengthened and reloaded	Till the crack load then strengthened and reloaded	Till the crack load then strengthened and reloaded	Till the crack load then strengthened and reloaded
Strengthening method	----	1 CFRP strip	1 CFRP strip	1 CFRP strip	1 CFRP strip	1 CFRP strip	1 CFRP strip
Concrete cover recasting	----	----	✓	✓	✓	✓	✓

Bonding technique between old & new concrete	---	---	----	Bonding agent	Shear connectors	CFRP U-jacketing	Bonding agent
New concrete cover strength	--	--	20 MPa	20 MPa	20 MPa	20 MPa	50 MPa

C. Test Setup and Instrumentation

All specimens were tested under three point bending. The span of the beams was 1200 mm. Dial gauge was used to measure the deflection at mid-span at point of loading. Strain gauges were mounted on lower reinforcement and on the CFRP strips. Loading was applied manually through a hydraulic pump to the hydraulic jacks at increments of 1 ton, at which time readings from the dial gauge and the strain gauges were manually recorded, Fig. 3.



Fig. 3: Test Setup and Instrumentation

III. TEST RESULTS AND ANALYSIS

A. Behavior of Test Specimens

As a result of this experimental work, two modes of failure were generally observed for the strengthened beams. Failure Mode I refers to the curtailment of the CFRP strip adjacent to the support originates a high concentration of stresses at the cut-off point of at the end of the strip. Failure Mode II refers to the concrete cover de-bonding starting from one of the intermediate flexural cracks towards the maximum bending moment zone. The horizontal crack is originated by splitting of concrete at the steel reinforcement level and mainly by shear stresses at that level which are needed to ensure equilibrium. On the other hand, the control beam B1 was loaded until complete failure. This beam started to crack at 50 kN, its ultimate load was 100 kN, the failure was due to flexural cracks followed by shear cracks as shown in Fig.4. The second control beam B2 was loaded till the cracking load. The load was removed then the beam was strengthened with one layer of CFRP strip and reloaded to failure. The crack load was 50 kN and the ultimate load was at 130 kN. Fig.5 clearly shows flexural mode of failure combined with de-bonding of the FRP strip with an increase in the ultimate load = 30% over the un-strengthened beam B1.



Fig. 4 Failure Cracks for Control Beam B1



Fig.5 Failure Cracks for Strengthened Control Beam B2

The Third beam B3 was loaded till the cracking load (50 kN) then the load was removed and a new concrete cover was recast without any bonding technique. The beam was strengthened with 1 layer of CFRP strip and then reloaded up to failure. The ultimate load was at 110 kN. The failure mode showed concrete cover de-bonding followed by de-bonding of the CFRP strip with an increase in the ultimate load = 10% over the un-strengthened beam B1. The ultimate load decreased by 15% compared to the strengthened beam B2 that may be attributed to the early de-bonding of the new concrete cover. The fourth beam B4 having bonding agent between the old and new concrete cover had the same behavior and failure mode of B3. The ultimate load was 110 kN which indicates the using of the bonding agent had no significant effect on the load-deflection behavior or on the mode of failure, Fig. 6.



Fig.6: Failure Cracks for Strengthened Beams B3& B4

The fifth beam B5 was loaded to the crack load (50 kN) and then the load was removed. A new concrete cover was recast using shear connectors (steel studs 100 mm long and 6 mm diameter) then strengthened with 1 layer of CFRP strip and reloaded up to failure. B5 failed due to concrete cover de-bonding followed by de-bonding of the CFRP strip with ultimate load of 108 kN showing a little increase compared to the control beam B1, Fig.7.

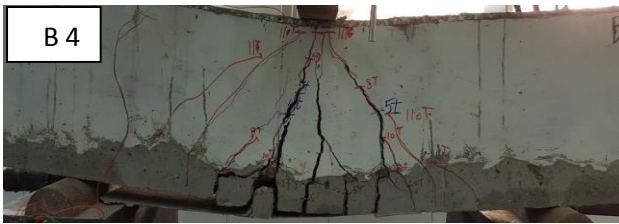


Fig. 7 Failure Cracks for Strengthened Beam B5

On the other hand, B6 which had the same aspects of B3 but U-jackets of CFRP sheets till the half of the depth were used. The failure mode of B6 showed de-bonding of the CFRP strip after rupture of the first CFRP U-jacket which represents a good enhancement in the mode of failure. However, the ultimate load was 130 kN representing a good increase in the ultimate capacity compared to the control beam B1 and also it reached the same carrying capacity of the control strengthened beam B2. The seventh beam B7 had the same aspects of B4 but it had a high strength concrete for the new cover

in addition to the use of bonding agent. The mode of failure showed flexural cracks followed by shear cracks then de-bonding of CFRP strip. The ultimate load was 150 kN representing the highest increase in the ultimate capacity compared to the control beam B1 and it exceeded the carrying capacity for the control strengthened beam B2 by 15%, Fig.8.



Fig. 8 Failure Cracks For Strengthened Beams B6 & B7

Table 2 presents a summary of the test results including cracking load, load corresponding to lower steel reinforcement yielding, and ultimate load. Also shown the strain of the lower steel reinforcement corresponding to crack load and ultimate load, finally present all deflection at the mid span of the beam corresponding to the crack, yield and ultimate loads.

Table 2: Summary of Specimens Test Results

	Crack load (kN)	Yield load (kN)	Ultimate load (kN)	Cracking load strain	Ultimate strain	Deflection at cracking load, mm	Deflection at yielding load, mm	Ultimate deflection, mm
B1	50	48.41	100	0.00027	0.005903	4.95	4.9	15.43
B2	50	79.5	130	0.000093	0.00279	7.88	9	18.14
B3	50	73	110	0.000057	0.000349	11.41	12.7	16.55
B4	50	50	110	0.000183	0.00847	9.55	9.5	18.51
B5	50	73	108	0.00009	0.000298	4.72	6.4	9.86
B6	50	75.6	130	0.000097	0.004633	4.32	6.5	17.92
B7	50	65	150	0.000133	0.002014	4.68	5.5	16.73

IV. EFFECT OF KEY PARAMETERS

A. Effect of Strengthening with CFRP Strips

Comparing the load-deflection relationships of the un-strengthened beam B1 with the strengthened beam B2, it is noticed that both of them have same behavior till the cracking load then the

strengthened beam B2 sustained more carrying loading capacity than the un-strengthened beam with decreasing in the mid-span deflection at the same loading level due to the restraining effect of the CFRP strip. On the other hand the strain in the longitudinal reinforcement exceeded the yielding strain for both beams. Also, the un-strengthened beam B1 sustained more strain for the longitudinal

reinforcement than the strengthened beam B2 at all loading levels, as shown in Fig. 9. Also it is clearly shown that the strengthening technique increased the ultimate loading capacity by 30%.

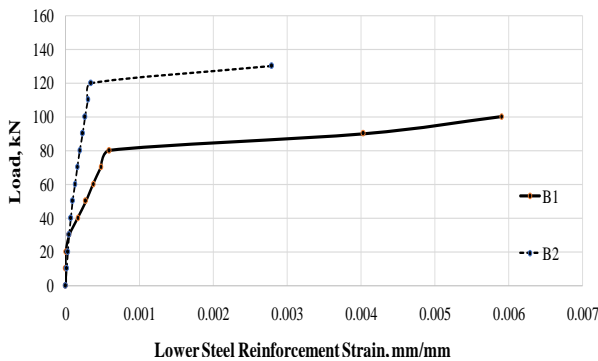
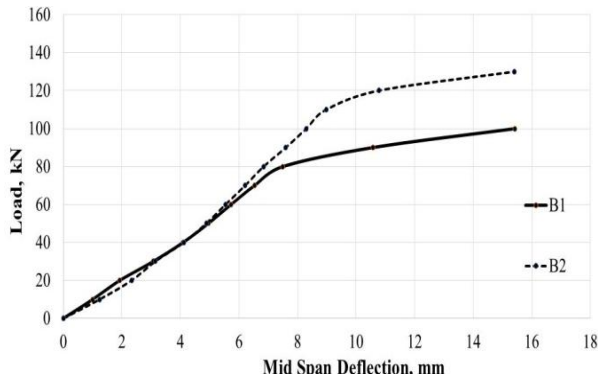


Fig. 9 Load Vs. Mid-Span Deflection and Long. Rft. Strain For B1 & B2

B. Effect of using New Concrete Cover Without any Bonding Technique

Fig.10 compares the behavior of the strengthened beam B2 with beam B3 that had a new concrete cover with no bonding technique. It is clearly noticed the effect of recasting a new concrete cover with lower compressive strength on decreasing the load capacity of the beam by 15% due to the early concrete cover de-bonding which affect the strengthening process. Also the repaired beam B3 did not reach the yielding strain for the lower reinforcement due to the sudden failure which related to the loss of bond between the weak concrete cover and steel reinforcement that led to decrease the beam ductility in spite of getting more load capacity compared with B1, Fig. 10.

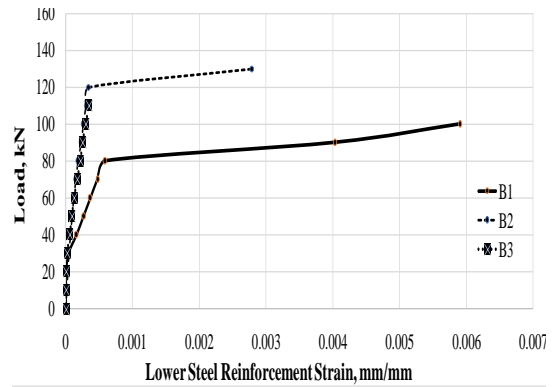
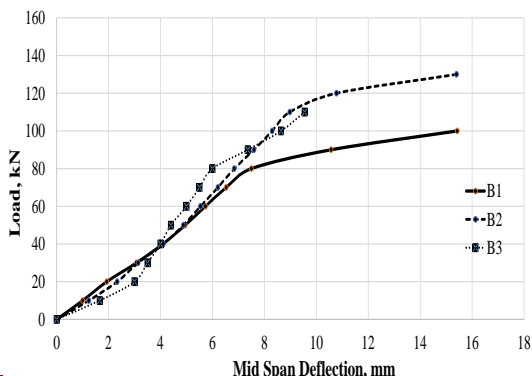


Fig. 10: Load Vs. Mid-Span Deflection and Long. Rft. Strain For B1,B2&B3

C. Effect of Compressive Strength of the New Concrete Cover

Fig. 11 shows the load- deflection relationship of beams B1, B2, B4, and B7. B4 had a new concrete cover with low compressive strength (20 MPa) and a bonding agent was used while B7 had same bonding agent with high compressive strength for the new concrete cover (50MPa). It is noticed that increasing the compressive strength is a very effective parameter as it enhanced the beam carrying capacity by about 40% over the same beam with bonding agent to attach old and new concrete. Among the strengthened specimens, B7 recorded the highest value of ultimate load, 150kN. The ultimate load of the strengthened beams B4 and B7 were higher than that of the control beam B1 by 10% and 50% respectively. All beams reached the yielding strain of the lower reinforcement at the beam failure.

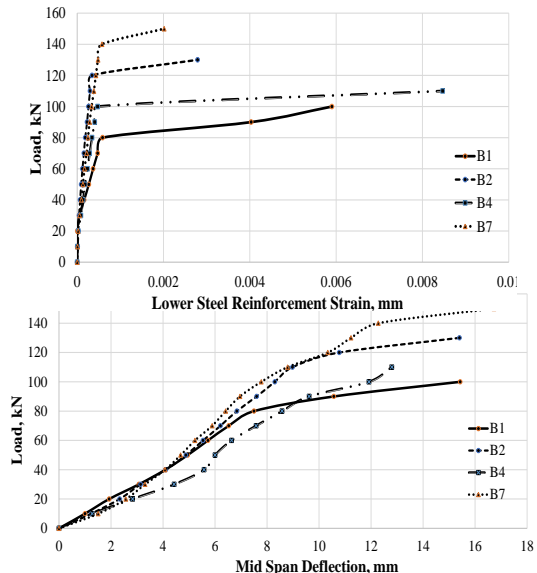


Fig. 11: Load Vs. Mid-Span Deflection and Long. Rft. Strain For B1,B2,B4&B7

D. Effect of Different Methods of Bonding New Concrete Cover

Fig. 12 shows the load deflection relationships for beams B1, B4, B3, B5, and B6. All these beams had different bonding methods to connect old concrete with the new concrete cover. B4 had bonding agent, B5 had steel shear connectors while B6 had CFRP sheets as U- jacket. It was noticed that the U- jacket of CFRP sheets had superior behavior which led to the same ultimate load capacity of B2 (strengthened without removing the concrete cover), Table 2. This indicates that the CFRP U- jackets had dual effect for not only bonding the new concrete cover with old concrete but also bonding the CFRP strips with the old concrete of the repaired beam leading to a more ductile behavior. On the other hand both beams B4 and B5 which strengthened by CFRP strips did not show a significant enhancement in the beam ultimate capacity and failed with a brittle mode of failure. This indicates that bonding new weak concrete cover using shear connectors or bonding agent was insufficient. All beams lower steel reinforcement exceeded yielding point before failure except B3 and B5

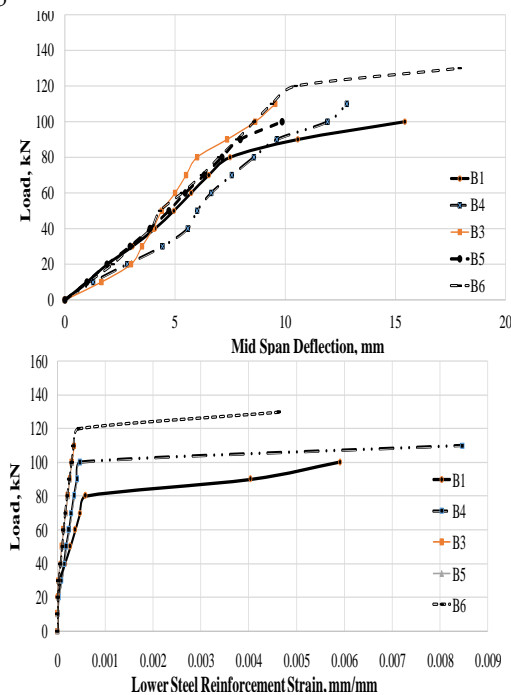


Fig.12 Load vs. Mid-Span Deflection and Long. rft. Strain for B1, B3, B4,B5&B6

V. CONCLUSTIONS

Based on the experimental results and observations, the following conclusions can be stated:

1. Strengthening of RC beams with externally bonded CFRP strips is an effective method for flexural strengthening which increased the ultimate

loading capacity by about 30 % over the un-strengthened beam.

2. Premature failure in beams strengthened with CFRP strips which hadn't any bond technique between new concrete cover and old concrete was observed and it was caused by concrete cover de-bonding.

3. Bonding the new concrete cover with old concrete after repairing beams from steel corrosion is important and highly effective for the FRP strengthening process.

4. Bonding new concrete cover and old concrete with bonding agent or steel shear connectors was not sufficient to enhance the failure mode or the ultimate load.

5. Using U jacket of CFRP sheets was an effective technique for bonding new concrete cover and old concrete as it enhanced the mode of failure and led to the same ultimate capacity of the beam strengthened by CFRP strips without removing concrete cover.

6. Increasing the concrete cover compressive strength is a highly effective method for increasing the bond between new concrete cover and old concrete that enhanced the mode of failure and increased the ultimate load capacity.

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