A Laboratorial Study of Reinforced Concrete Beams Strengthened with CFRP Fiber Reinforcement to Improve the Debonding Mechanisms

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Abstract

Recent studies suggest that most reinforced concrete beams strengthened with fiber reinforced beams (FRPB) fail under peeling effect. The aim of this study is to resolve such problems or to delay them. As a consequence, the study uses anchorage systems not tested thus far. It tests the weld-connecting of horizontal MSN round bars to vertical tension bars to which the end of FRP beams are twisted. This helps prevent the pulling out of anchorage systems. The study required the construction of ten special *armored beams* (150 ×200×2000mm) in two categories, namely 21 and 35 megapascal. The length of the section of the beams were 0.3 ρmax for 21 megapascal beams and 0.2 ρmax for 35 megapascal beams respectively. In each category one beam was selected as the control beam, the rest were reinforced as explained above. The number, the width, and the thickness of all the beams were identical, but the types of anchorage, the length of the layers, which were used to counter the possible weaknesses in the event of sudden ruptures, were different. The results demonstrate the effectiveness of near surface mounted (NSM) systems.

Key words: Flexural resistance, Plate-end debonding failure, CFRP Composite, Anchorage systems, End anchorage,

Introduction

Research on beams reinforced by FRP composites indicate that they are likely to be infected by several failure modes depending on such factors the geometric features of the beams and reinforcements. Since adhesive *failure between the composite and the concrete,* on one hand, and beams reinforced by FRP composites, on the other hand, is quite sudden, the failure mode is brittle. The brittle behavior of concrete along with failures reduces the possibility of using the maximum potential of composite reinforcement. They are also at odds with safety standards.

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The peeling of FRP from the concrete layers occurs as a result of the fact that the reinforcement cannot continue after the bearing of the beam, so that reinforcement must stop at a distance from this point. This will result in insufficient end anchorage of the plates (1). Even in FRP as external reinforcement for concrete, there is an effective bond length, after which the load cannot increase any more, which is one of the main differences between internal reinforcing bars and external plates. That is to say, in internal reinforcing bars, a sufficient bond length for maximal tension strength can be achieved.

1- Failure mechanisms of reinforced beams

Several modes of failure such as shear failure, flexure under load, FRP failure, debonding under peeling effects, and delamination in FRP reinforced beams have been reported by Ascione & Feo (3) and Maalej Bonacci (4).

Such failure modes are undesired and occur suddenly as the FRP plate cannot be fully utilized. Premature failure modes are caused by interfacial shear and normal stress concentration at FRP cut-off points and at flexural cracks along the beam. Comprehensive investigations in the past have resulted in proposing a series of failure modes for RC reinforced beams with FRP plates as follows:

1- Flexural failure by crushing compressive concrete which could occur before or after yielding of tensile steel reinforcement
2- Rupture of FRP after yeiding of steel in tension
3- Cover delamination at the edge of FRP sheets
4- Debonding of FRP from the concrete substrate
a. Plate end interfacial debonding
b. Interfacial debonding induced by flexural cracks
c. Interfacial debonding induced by flexural shear cracks
5- Shear failure

According to Bian (5), Malek and colleagues (6), Feo, Ascione (3), and Lau and colleagues (7) type 3 and 4a failures occur when the ends of the FRP sheets are not properly anchored.

According to Teng and colleagues (5) type 4b and 4c failures depend on bond-slip interaction between FRP sheets and concrete. In addition, the corrosion of bars and the change of the reinforcing bar ratio in the vicinity of large bending moments and shear forces increase the possibility of these types of failures.

2- Experimental program

2.1Machinary used in the experiment

Four point bending flexural tests on specimen beams were carried out in a loading machine with a hydraulic jack. The length of the shear span for all specimens was 50 cm, and the distance between anchorages was 160cm. Picture one depicts a four point loading system.

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Picture 1 a four point flexural loading machine

2.2 Material properties

2.2.1 Concrete

The study used two series of armored beams with 21 and 35 megapascal resistance. Their mixture qualities are depicted in tables one and two.

Table 1 Mixture qualities of 21 megapascal concrete

gravel* (kg/m3)	Sand (kg/m3)	cement (kg/m3)	water (kg/m3)
1041	807	310	160

Table 2 Mixture qualities of 35 megapascal concrete

The concrete was tested in 15 by 15 cube specimens for 28-day cube strength in steam chambers. The compressive strength of specimens will be given below.

2.2.2 Steel

Steel reinforced bars used in the specimens are 8, 10, and 12 mm in diameter of type AIII. Three specimens of each diameter were used in the experiment. They were tested for tension. The yielding strengths obtained were 356, 393 356 megapascal respectively.

2.2.3. CFRP

FRP sheets (type CFRP) with GPa 230 elasticity module and the ultimate yielding of 2.1 percent were designed and used for the purpose of FRP flexural strengthening of beams measuring 125× 0.166. Sheet bonding was used to connect CFRP layers to the elastic surface of the concrete.

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2.2.2. Reclamation of concrete and the preparation stage

The behavior of elements reinforced with FRP composites is related considerably to the reclamation and concrete surface quality, If such precautions are not observed, the reinforced element might deteriorate, before it reaches its designed physical properties. In addition, epoxy hardening, add other fillers were applied *until it reached* consistency and clams were removed afterwards. Accordingly, specimen B3M-F352L12.5 was reinforced twice before the application of epoxy to remove tracks which created as a result of strengthening. All eight reinforced beams and the B3 recycled beam were milled to become as smooth as possible.

Picture 2 Te preparation stage

2.2.3 **Anchorage mechanism**

First, a number of grooves were cut in d/2 distances from the anchorages. Given beam geometry and their length (2 m) and given the fact that the anchorage is located at 20 $_{cm}$ from the end of the beam, the grooves were cut at 27 $_{cm}$ away from the end of the beams. To make the grooves, four wooden chips measuring 15×3.5×3 were used at 27 $_{cm}$ away from the end of the beams to prevent cutting.

Picture 3 depicts the anchorage in two specimens. As can be seen, after removing the chips, the groove location is cleaned with cutter machines to the surface of tension rebars, and all dust particles are removed. Then a cross bar is put in the groove, and is welded to the cross bars. Then a U-wrap is welded to the cross bar. In the end, the NSM cross bar, to which the end of FRP layer is anchored, is welded to the U-wrap.

The reason why this procedure was used is that in previous U shaped anchorages or cross NSM used in groove, pull out debonding has been reported. It is hoped that welding connections or tension bars help prevent this type of debonding. Likewise, hooking the end of NSM bar and and U wraps through welding will add to their strength. It is noteworthy in order to prevent cross shear, FRP was created by the edge of concrete groove, so by concentration tension, the sharpness must be reduced, which we did by cutting machines.

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Picture 3 Bar anchorage with cross NSM bars

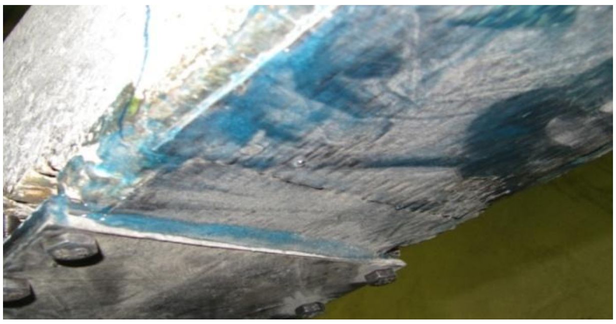
2.2.3 The properties of the beams in the laboratory

Ten reinforced concrete beam specimens with a cross section150×200mm and length of 2000mm were fabricated. Such beams are typical reinforced concrete at the approximate scale of 1:2. They are categorized as thin beams in which the pore *mouth* diameter to pore *depth ratio* is relatively small. They are reinforced with 2Φ12 as tension steel bars, 2Φ10 as stirrups hunger and 8Φ8/m closed stirrups. The geometric percentage (ρ) of the tension bars in all 21 megapascal specimens is equal to 3% ρmax, and is equal to 2%ρmax in all 35 megapiascal specimens. By adding two reinforcing layers measuring 150_{mm} (width) and 0.166_{mm} (thickness), it increases to 0.36ρmax and 0.25ρmax. As such, it is expected that they should be subject to soft modes of failure.

2.3.3.1.

B0-21F-0L: It is a 21 megapascal control beam, whose strain strength was equal to 22.45 at the time of experiment.

B1-21F-2L12.5: This specimen was reinforced by two FRP layers 156 $_{cm}$ long, of which ten centemeters are fixed to the plate. Epoxy is applied on both sides of layers. Then plates are put over the second layer of the FRP and fixed in their place with six screws. As can be seen in picture four, the anchorage screw plate anchorage measuring 15×10 has been welded to the tension beam with angles. The strength of the compressive concrete in this specimen is equal to 23.9MPa.



Picture 4 B1-21F-2L12.5 specimen

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B2-21F-2L12.5: This specimen is also reinforced with two layers, but there are three differences between them.

1- A bar anchorage together cross NSM bars which have been welded to tension bars has been used.
2- Since it is impossible to wrap both ends of FRP layer in MSN bars, and given the location of the grooves at 30 $_{cm}$ distance from the edges, and after the application of epoxy, eight centemeters of the first 156 $_{cm}$ long layer are wrapped around the NSM bar.
3- In accordance with cross section location in Iranian standards regulations and ACI 440-2. R-02, which specifies cross section at 15 $_{cm}$ from the end of two headed simple beams, after the application of epoxy, the 156 $_{cm}$ long second layer is fixed on the first layer. The strength of the concrete in this specimen is equal to 26.5MPa.

B3-21F-2L12.5: It is similar to B1in every respect, except that the length of the second layer is 15 $_{cm}$ less than that of the other layer from both ends. This will help the researchers to compare it with the first specimen in which the FRP failure is likely to take place in the comment area between the plate and FRP. The strength of the concrete in this specimen is equal to 25.4MPa.

2.3.3.2. The beams in the second group

B0-35F-0L: It is a 35 megapascal control beam, whose strain strength was equal to 40.50 at the time of experiment.

B1-35F-2L12.5: This specimen was reinforced by two FRP layers 156 cm long, of which ten centemeters are fixed to the plate. Epoxy is applied on both sides of layers. The difference between this beam and B1-21F is in the strength of the concrete. The difference between this beam and B3-35F is in anchorage type. The difference between this beam and B3M-35F is in flexural pre-cracks. The strength of the compressive concrete in this specimen is equal to 43.5MPa.

B2-35F-2L12.5: It is similar to B2, and was reinforced by two FRP layers. However, there are some differences. Since debonding in B2-F21 was from the anchorage; that is to say, the NSM cross bar was anchored to the back of U wrap. In his beam it was welded to the U-wrap, the end of the bar was hooked as can be seen in picture 5 to give extra strength against pull out The strength of the concrete in this specimen is equal to 45.9MPa.

Picture 5 Anchorage in MSN cross bar in B2-F35-2L12.5 specimen

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B3-35F-2L12.5: This specimen was reinforced by two FRP layers 156 cm and 126 cm long, of which only the first layer is anchored to the end of the beam, but the second layer is 15 cm shorter given the location of cut in the layer. Epoxy is applied on both sides of layers. Then plates are put over the second layer of the FRP and fixed in their place with six screws. The strength of the concrete in this specimen is equal to 44 MPa.

B3M-35F-2L12.5: This is the same as the reinforced B3. The beam was reused after FRP failure because the concrete was intact with some flexural cracks, which were reinforced once more with two FRP 156 $_{cm}$ long layers. The initial rise of the beam was 8$_{mm}$.

B4-35F-2L12.5: This specimen was reinforced by two FRP layers 156 cm long. of which of which ten centemeters are fixed to the plate. Since failure in beams covered on both sides occure from the anchorage age due to high tension, an extra layer, which was 35 cm long was added to the beam. The strength of the concrete in this specimen is equal to 41.5 MPa.

3. Results

The specimen went under four point bending flexural tests till breaking. Table three depicts the results for the ultimate breaking of specimens along with the type of failure.

Table 1 results of loading applied to the first group of beams

Type of debonding	Ultimate loading (Kn)	Specimen
The crushing of compressive concree	60.44	B0-21F-0L
Local debonding of RFP and crushing compressive concrete	118.12	B1-21F-2L12.5
MSN anchorage pul out from the weld and end breaking	100.77	B2-21F-2L12.5
Debonding of FRO from the edge of anchorage and crushing of compressive	113.74	B3-21F-2L12.5

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concrete		

3.1.2. **Comparing group 1 reinforced beams and the control beam**

Diagram six depicts specimens of group 1 reinforced beams. Comparing the load-rise of the reinforced beams with the control beam shows an increase in values related to the yielding load and the ultimate load. The highest increase was observed in B1, which used two plates as anchorage.

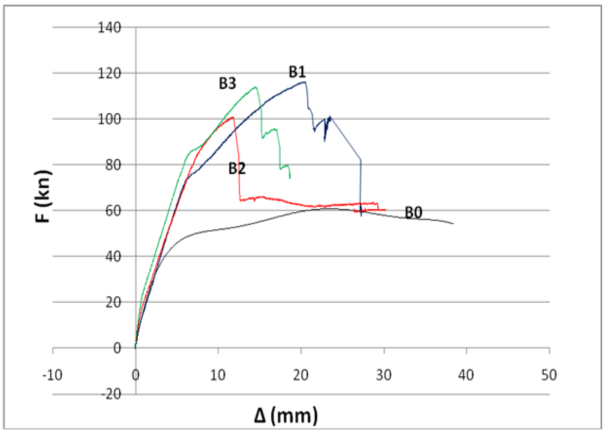

Diagram 6 Load-rise features of group 1

By comparing B1 and B3, which have undergone the same mode of failure in Diagram 6 , it can be concluded that , in which one of the layers were anchored at the end of the beam, the load values for B3 in parallel rises till the ultimate load-rise is higher than B1. On the contrary, B1 registers higher load increase than the control beam and B 3. This is a positive fact indicating a better performance for beams in which both layers are anchored at the end.

3.2.2 Loading of group 2 beams

The specimens in this group went under four point loading till ultimate breaking. Table four depicts the results for the ultimate breaking of specimens along with the type of failure.

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Table 4 Results of the loading of group 2 beams

Type of debonding	Ultimate loading (Kn)	Specimen
The crushing of compressive concrete	73.85	B0-35F-0L
Local debonding of RFP and crushing compressive concrete	127.69	B1-35F-2L12.5
Bebonding of ERP from the end of the anchorage (end debonding)	108.44	B2-35F-2L12.5
Debonding of FRO from the left edge of anchorage	96.62	B3-35F-2L12.5
Local debonding of RFP the edge of anchorage (delamination)	118.97	B3M-35F-2L12.5
Slipping of FRP from the left edge of anchorage- Crussing of the concrete	139.08	B4-35F-2L12.5

2.2.3 Comparing group 2 reinforced beams and the control beam

Diagram six depicts specimens of group 2 reinforced beams. Comparing the load-rise of the reinforced beams with the control beam shows an increase in values related to the yielding load and the ultimate load of .B1 ،B2 ،B3 ، B3M and B4. Most of the beams in group failed because of the depending of FRP layer. This fact is indicative of the positive effect of such layers as anchorage.

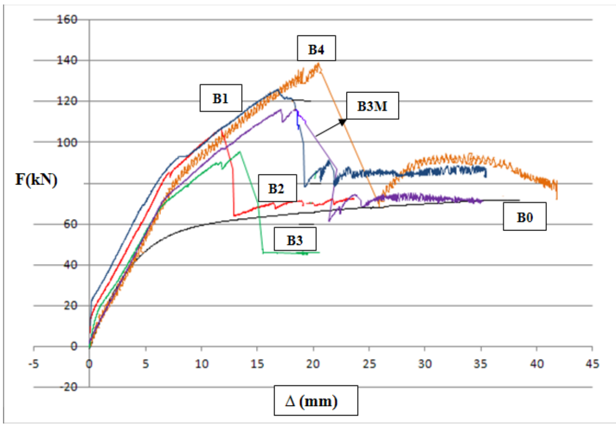

Diagram 6 Load-rise features of group 2

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4. Conclusion

The results obtained are as follows:

1. In all specimens in which plates were used as anchorage, FRP debonding was observed. In two specimens in which cross NMS bars were used as anchorage, FRP debonding was observed in one and anchorage debonding in the other.
2. Plate anchorages outperformed NSM ones. For the 21 megapascal group the resistance ration was 0.95.44 to 0.66.73. For the 35 megapascal group the resistance ration was 0.72.9 to 0.46.8.
3. The cases in which both ends were anchored to the steel plate outperformed to the cases in which only one end was anchored to the steel plate.
4. NSM bars out performed others in cases in which both comparatives were identical in length.
5. The highest performance was observed for plates with two layers and 30 cm additional plate in order to prevent FRP breaking. In this specimen the ultimate rise is even higher than that of the control beam.
6. After the debonding of FRP layers and the reduction of the load to the approximate ultimate load of the control beam, all reinforced specimens in both groups were able to tolerate the deformation. The reason for this phenomenon is that up to that point a big part of deformation was tolerated by the FRP layers, which is transferred to the reinforced concrete after FRP debonding.

5. References

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