INVESTIGATION ON PERFORMANCE OF HEAVILY CRACKED CONCRETE BEAM STRENGTHENED WITH CARBON FIBER REINFORCED POLYMER (CFRP) SHEETS

Prasad Dhammika Dharmaratne

(138731 H)

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Department of Civil Engineering

University of Moratuwa
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DECLARATION

I declare that, this is my own work and this dissertation does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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The above candidate has carried out research for the Masters Dissertation under my supervision.

Dr.(Mrs). G.C.P.H. Gamage **Date**

Dr.(Mrs). D. Nanayakkara **Date**

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ABSTRACT

Repair and retrofit of existing structures especially buildings, bridges, water tanks etc., have been amongst the most significant challenges in Civil Engineering. In the past construction was evolved from thousands of years back with various construction materials such as rocks, clay bricks and timber etc. There after concrete was introduced as a sustainable construction material which is most suitable than that of previously used materials.

Although concrete has high compressive strength, it is very weak in tension and become brittle under tensile loads. Because of these reasons, Engineers moved to reinforced concrete structures. Since concrete structures are long lasting structures, carrying out the rehabilitation work of existing structures becomes more vital.

Nowadays, there are different kind of problems were encountered in construction field due to original design, construction errors or poor construction supervision, damages of earthquakes etc.. That needs to be retrofitted to meet the demand usage in a more economic and effective ways. The techniques based on the externally bonded Fiber Reinforced Polymer (FRP) materials is one of the most widely application for retrofitting existing damaged structures.

The use of Carbon Fiber Reinforced Polymer (CFRP) in strengthening reinforced concrete structures has become popular retrofit technique. The technique of strengthening reinforced concrete structures by externally bonded CFRP fabric was started in 1980s and has attracted researchers around the world wide.

The aim of this research is to investigate the flexural behavior of pre cracked and non-cracked reinforced concrete beams going to be strengthened with different configurations of Carbon Fiber Reinforcement Polymer layers.

12 Nos. of Reinforced concrete beams of the width 125mm, depth 200mm and length of 1900mm were prepared and tested for this investigation. Beams were tested in accordance with ASTM C78 guidelines

Beams consist of different CFRP arrangements such as non-anchored CFRP sheet, CFRP sheet with end anchors and CFRP sheet with end and intermediate anchors at cracked locations.

FRP can be bonded to reinforced concrete elements using different methods such as external bonding, wrapping and near surface mounting. FRP sheets can be sticked to the tension face of a structural element to provide flexural strength or sticked along the web of a beam to provide shear strength.

Observation shows that increment of flexural capacity is in between 81% to 110% in beams those strengthened with CFRP sheets with respect to non-strengthened beams. Highest strength gained was observed in cracked beams strengthened with CFRP with end anchors and intermediate anchors. Similar behavior was observed in non-anchored CFRP strengthened cracked and non-cracked beams. However the flexural capacity was high in CFRP strengthened cracked beams. All the cracked beams failed in debonding. But some non-cracked beams failed by rupture of CFRP.

At the end of this dissertation, presents the experimental procedure, results, analysis and conclusion.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

CFRP	Carbon Fiber Reinforced Polymer
EB	Externally Bonded
ETS	Embedded Through Section
FRP	Fiber Reinforced Polymer
NSM	Near Surface Mounted
As	Area of tension reinforcement
Asv	Area of links at neutral axis level
b	Effective breadth of section
b_v	Breadth of section for shear
d	Effective depth of the tension reinforcement
f_{cu}	Characteristic strength of concrete
f_{yv}	Characteristic strength of links
f_y	Characteristic strength of links
M	Design ultimate resistance moment
S_v	Spacing of link along the member
v_c	design shear strength of concrete

LIST OF APPENDIX

1. INTRODUCTION

1.1. Background

Renovation of deteriorated structures and making alterations to existing buildings are main challenges faced by Civil Engineers in the construction industry. The deteriorated, load-damaged and cracked structural elements are to be strengthened to avoid further damage and ensure safety. Some structures are strengthened when the owners intend to make alterations to structure or change the purpose of building and higher loads are expected in future. Therefore strengthening of structural elements plays a main role in present construction industry.

There are number of methods of strengthening structures. Among them utilization of Fibre-Reinforced Polymer (FRP) laminates, sheets and rods for strengthening structural elements is a vital solution. FRP can be used for shear strengthening, flexural strengthening, and torsional strengthening and increase confinement of structural elements. This strengthening method is quite popular as it’s less time consuming, create minimal disturbances and easy to apply when compared with other methods. FRP composites have favorable characteristics of high strength-to-weight ratio and good corrosion resistance. That enhances the durability of structure.

CFRP materials are relatively expensive when compared with traditional materials. However it contains superior properties. This strengthening method is considered as an expensive method due to the high material cost. Usually CFRP materials are wrapped all around the structural elements due to lack of knowledge. That creates an adverse impact on project cost. But CFRP material amount can be minimized in a proper strengthening design.

The CFRP may help to enhance flexural strengthening, improving the ductility of compression members and shear strengthening. It is very popular that reinforced concrete beams strengthened with externally bonded fiber reinforced polymer to the tension face can exhibit ultimate flexural strength greater than their original flexural strength. However, these FRP and CFRP strengthened beams could lose some of their ductility due to the brittleness of FRP and CFRP plates.

Strengthening methods such as post tensioning and reinforced overlay were used widely during early ages and later introduced externally bonded methods for strengthening of structures. Among other strengthening methods FRP system become very popular recently. Fiber material could be carbon, glass or aramid depending on the required physical and mechanical properties and economical consideration. Matrix which is a polymer resin is used to bind the material together.

When considering the strength CFRP become the prominent material. There are many advantages of CFRP such as high strength to weight ratio, High stiffness, High corrosion resistance, Low thermal expansion, Corrosion resistance, Good fatigue performance, Easy of fabrication and bonding.

Major disadvantage of CFRP system is the higher cost when compared to other traditional methods. However when comparing with overall performance, CFRP becomes most popular retrofitting method to rehabilitate the structures.

1.2. Objective

The main objective of this study was to investigate the flexural performance of heavily cracked [cracked width is greater than 0.3 mm, BS 8110, Part 2, 1997, Cl 3.2.4.1] concrete beams, by using different arrangements of CFRP sheets.

To achieve the main objective, this research was divided into sub methodologies. They are

1. To conduct literature review to understand the current status of knowledge.
2. To collect the previous research data related to flexural performance of CFRP/concrete composite.
3. To conduct a detail test program and collect the data.
4. To analyse data and compare with previous research carried out.
5. To provide recommendations in strengthening heavily cracked beam using CFRP.

1.3. Methodology

The literature review was conducted and data was collected related to the topic and hence research gap was identified. Then planning and designing of test program was completed.

12 Nos. of beams [125 mm in width, 200 mm in depth and 1900 mm in length] were casted and 6 Nos. of beams were loaded until 0.3 mm cracks appeared.

CFRP sheets were sticked at the bottom of the beams. “U” wraps were pasted at the selected locations to find out the effective arrangement. All the beams were tested under 4 points bending testing machine until 0.3 mm cracks appear. The crack patterns were observed. Deflections at the mid points and loaded points were recorded. Failure loads and modes of failure were noted down. Compare the results with literatures carried out under similar situations.

1.4. Outline

Chapter 1- gives the outline of the ways to transfer historical construction methods to the retrofitted techniques. The basic introduction about FRP materials, their advantages, the CFRP systems with their properties and benefits to the construction industry. In addition to that, usages of CFRP with more prominent way during the strengthen purposes. This chapter also contemplates on the outline of the literature review, objectives of the research program, methodology and summary of test program.

Chapter 2- consists of literature review. It includes different failure modes of beams such as flexural and shear failures. Then CFRP material, adhesive and different bonding types of CFRP to beams such as externally bonding method and near surface mounted method were included. Summary of related research was also included. Finally, the research gap was identified.

Chapter 3-consists of relevant tests, importance of each test parameter and the procedures. Design and preparation of test samples and the usage of CFRP materials in different patterns were also discussed.

Chapter 4- presents all the test data and all the results obtained from experiments. Analysis and results are also compared with related literature was also included.

Chapter 5- presents the conclusions and recommendation including summary of this research. Finally expresses the recommendation for the Civil Engineering Society.

2. LITERATURE REVIEW

2.1. General

CFRP retrofitting method has been implemented in the construction industry in the past's centuries. In the structural applications, CFRP is used in two ways. The first area includes the use of FRP bars instead of steel bars or pre-stressing strands in concrete elements. The Second area includes the retrofit the CFRP to the structural members as an external application.

Usage of FRP can be in different such as external bonding, wrapping and near surface mounting. In the flexural structural elements, FRP sheets are sticked along the tension face and to enhance the shear strength of the beams, those are sticked along the web.

2.2. Externally Bonded Reinforcement (EBR)

EBR is an effective and frequently applied method for repair and strengthening of structural elements.

The same method was also used to strengthen RC floor slabs and supporting RC beam in several old buildings in Zurich, Switzerland in order to withhold additional live load (Alfarabi et al, 1994)

Quite naturally, steel plates have been widely used for such rehabilitation works, but more recently non-metallic CFRP are being considered as a competitor to replace steel. CFRP materials have many favorable engineering properties that can be used as external plate reinforcement, combined with its lightweight nature and freedom from electrochemical corrosion that occurs to steel. It was recognized early in the work that, CFRP is not economically feasible when viewed on the basis of material cost. However, it becomes more attractive because of the reduced time required on site and the considerable reduction in false work compared with the use of steel plates.

In Europe, the first application was on the Ibach Bridge in Lucerne, Switzerland, where steel tendons had been severed when the bridge was drilled to support a sign gantry (Burgoyne,1999).

Commercial use of CFRP as externally bonding reinforcement was started around 1993 and the amount of CFRP material used for strengthening increased every year thereafter (Taerwe, 2001).

2.3. Flexural strengthening of beams

Different kind of CFRP materials such as laminate, plate and rods have been used to strengthen the concrete beams in flexure throughout the history. Therefore, many researches have been studied flexural performance with different CFRP parameters, to increase flexural capacity, the FRP should be glued to the member in the way that fibers are parallel to the direction of the principal stress since unidirectional material.

The use of carbon fiber for structural application which has proved to be commercially successful was the use of CFRP plates as externally bonded reinforcement. The CFRP plates are much thinner than its steel counterpart, which allow lap joints can be made successfully between different elements. The reduced eccentricity of the plate also reduces the tendency for peeling failure (Burgoyne,1999). Researchers published test data showed that, flexural strengthening with CFRP plates behave very similarly to steel plates (Swamy and Mukhopadhyaya,1995). Besides the classical failure modes, such as steel fracture, concrete crushing or shear failure, bond failures could occur in the interface between the externally bonded CFRP plates and the concrete body. This undesirable bond failure resulted in sudden drop in loads and a brittle type of failure due to high concentration of interface shear and normal peeling stress at the cutoff point of the plates. Although substantial increases in load capacity can be achieved with CFRP plates, the possibility of unexpected brittle failure mechanisms needs to be included in design considerations.

Swamy studied the plate separation and anchorage of RC beams strengthened by epoxy bonded steel plates (Jones et al, 1988). They concluded that plate separation is due to high local interface bond stresses and peeling forces at the ends of the plates. The magnitude of these concentrated shear and normal stresses at the end of the plates depend on the geometry of the plate reinforcement, the engineering properties of the adhesive and the shear strength of the original concrete beam. With careful selection of the geometry of the plate, it is possible to develop ductile failures with the composite beam reaching its full flexural capacity.

Taljsten used fracture mechanics approach to derive closed-form formula for the case of linear elastic shear slip relationship (Taljsten, 1997). The results from both theory and finite-element analysis showed that the stresses are very large at the end of the plate, but they quickly diminish when move nearer to the center of the beam. The magnitude of the stresses is influenced not only by the geometrical and material parameters of the beam, but also by the adhesive and the strengthening material. To minimize these stresses, the distance from the support to the cutoff point of the plate should be kept as short as possible. Furthermore, the parametric study indicates that with increasing stiffness of the adhesive that the shear and peeling stresses also increase. This is also the fact with decreasing thickness of the adhesive layer or if the thickness of the plate is increased. In addition, if the Young's modulus of the plate is increased, then the shear and peeling stresses increase as well.

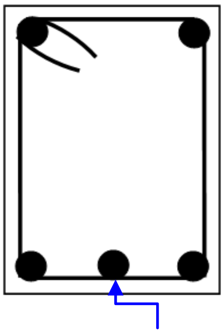
Blaschko studied the bond failure modes of flexural member strengthened with CFRP plates (Blaschko et al, 1998). He found that bond failures can occur in the interface between the externally bonded reinforcement and the concrete body caused by flexural cracks along the tension face of the concrete, shear cracks and unevenness of the concrete surface. Each of these failures can cause the total high interface shear and normal stresses to concentrate at the end of the plates have been proved to increase the stiffness of the member and the load capacity and reduce the cracking.

Meier (1995) carried out analysis about how the carbon fiber/epoxy composite strengthens structures. Initially Meier evaluated alternative materials and emphasize that carbon fiber polymer is the most suitable for post strengthen structures. Further emphasized that CFRP will increase the flexure capacity as the same way of conventional reinforcement increases.

The attention was paid to factors affecting the strength in order to find most economical solutions. One of the parameter was length of the CFRP used at the tension face of the member. Li et al (2006) investigated experimentally and numerically about the thickness and length of CFRP on repaired reinforced concrete beams and identified that there is a limit and beyond that CFRP will not increase the strength and stiffness of the member.

Major problem faced by the users of CFRP was failure of member due to de-bonding between FRP and concrete surface and due to failure of concrete cover. This scenario is common for both EBR and NSM methods. Therefore to find a solution Morsy et al (2015) conducted a research on increasing flexural capacity and strengthen the member using embedded CFRP rods instead of conventional methods.

Beams were repaired in such a way that concrete cover was removed without damaging to the existing steel reinforcement and CFRP rod was embedded through bottom reinforcement. Epoxy bonding material used to have better bondage between old and new concrete. Figure 2.1 shows the arrangement.



Additional CFRP rod for strengthened the beam

Figure 2.1: CFRP external bonding on beams to enhance flexural strength

Results emphasized that this method is effective in increasing flexural capacity as all beams failed under flexure without occurring brittle failure. This method can be used for corroded or damaged beams. However the drawback is non-simplicity of this method when comparing with other methods of strengthening.

Ali et al (2013), conducted a parametric study about the finite element modeling of strengthened simple beams using FRP techniques. At the end, they stated that, flexural strengthening with only more than two layers significantly increases load carrying capacity and beam ductility. Cracks and failure mode in flexural strengthening can be controlled by attaching legs of CFRP layer to beam sides.

Balamuralikrishnan and Jeyasehar (2009) tested flexural strength on 10 beams of effective span 3000 mm. CFRP fabric was bonded to tension face of beams. Monotonic and cyclic loads were applied until failure. Various conclusions were obtained after analysis of test results. Increase in flexural strength was 18 % under static loading and 20 % under compression cyclic loading when a single CFRP layer was applied. Increase in flexural strength was 40 % under static loading and 45 % under compression cyclic loading when two CFRP layers were applied. Flexural strengthening drastically reduced the deflection of strengthened beam and increased the stiffness of beam. Beams failed in flexural failure. CFRP strengthened beams gained enough ductility as well. Test results showed that, at least two layers are needed for flexural strengthening. A flexible epoxy preventted the adhesive bond break before failure.

According to above experiments, it has been found out that the load carrying capacity or the flexural capacity of a CFRP strengthened reinforced concrete element depends on the shape of CFRP reinforcement used, the texture of the CFRP material, the strength of the adhesive used, the method of mounting and many other factors.

Carbon Fiber sheets have been developed primarily in Japan and used for seismic retrofitting of existing RC structures. More than 1000 strengthening projects have been performed there in a manner similar to bonded steel plates (Thomas,1996). Currently, many grades of carbon fiber sheets are available, offering elastic modulus values from 230 kN/mm^2 to 640 kN/mm^2. Moreover, the carbon fiber can be aligned and woven in uni-directional and bi-directional ways to produce a fine mesh sheet of fiber. Thus, the designers have freedom of design when considering structural strengthening with CFRP sheets. CFRP sheets are easy to handle and install and also can be used in a variety of applications. They are ideal for complex shapes where strengthening is required.

Yoshizawa carried out a series of experiments to clarify the influence of bonding conditions between CFRP sheet and concrete members (Yoshizawa et al, 1996). By changing the method of concrete surface preparation (water jet and sander), type of carbon fiber and debonding area rate, bonding strength tests were performed through a four point bending test. It revealed that when epoxy was used, surface treatment with a water jet was more effective for increasing bonding strength compared to an ordinary sander. The bonding strength of high modulus CFRP sheets was higher than low modulus, high tensile strength CFRP sheet. As for artificial debonding of CFRP sheet up to 10% in area ratio, no significant influence on bonding strength was found.

Tumialan studied the concrete cover delamination in reinforced concrete beam strengthened with CFRP sheets (Tumialan, 1998). Six RC beams strengthened in flexure with varied plies of CFRP sheets were tested. Two types of failure modes were observed, concrete cover delamination starting at the cutoff point of the sheets and cover delamination starting at the intermediate flexural cracks and developed towards the beam mid span. The former failure might have occurred when more than one ply of CFRP sheet was attached to the concrete surface. During the test, it was observed that the application of CFRP sheets delayed the presence of the first visible cracks. Similarly, the flexural crack spacing was reduced when the number of plies of CFRP sheets was increased. In addition, important increases in flexural stiffness and ultimate capacity were achieved despite the ductility losses.

Alkhrdaji and Nanni investigated the behavior of an existing bridge strengthened with CFRP systems in order to provide the necessary field verification of design method, structural performance, and failure mode (Alkhrdaji et al 1999; Nanni, 2000). The bridge consisted of three simply supported solid RC decks having a thickness of 460 mm (18 in.) and a roadway width of 7.6 m (25 ft). Each of the three decks spanned 7.9 m (26 ft). Two of the three decks were strengthened with externally bonded CFRP sheets and NSM CFRP bars respectively, while the third deck was left as a benchmark. The failure mode of the deck strengthened with CFRP sheets was a combination of rupture and peeling of the sheets. The CFRP sheets strengthened decks had smaller deflection and 17% higher moment capacity at their ultimate point than the un-strengthened deck. The contribution of the strengthening system to the nominal capacity was less than originally predicted due to the significantly higher strength of the benchmark.

2.4. Shear strengthening of beams

Shear capacity is an important parameter in concrete members. Therefore many researches have been conducted on strengthening structures for shear using various methods.

- Remove concrete locally. Then place extra stirrups and filled with new concrete
- Strengthen the cross section with steel tendons.
- Strengthen the cross section with steel reinforced shot create.
- Strengthen the cross section with epoxy bonded steel plates.
- Cross section strengthen with pre-stressed steel tendons.
- Strengthen the cross section using pre-stressed steel straps.

Although they are acceptable methods, some drawbacks are there such as, no significant increase in strength in some methods, difficulty in construction, risk of damaging to existing reinforcement. Therefore advanced composite methods become more popular over traditional methods.

In the past decade, beams retrofitted for increased shear strength has attracted scientific attention; e.g Khalifa et al (1999), Daiaud and Chang (1998) and Sundarraja and Rajamohan (2009). Shear strengthening is usually provided by bonding external FRP reinforcement on the sides of the beam in perpendicular direction to the shear or with an angle corresponding to the principal stress directions.

The modes of failure and gain in the ultimate strength depend on the orientation of the FRP (Norris et al, 1997). The inclined FRP strips can result in increased shear strength and stiffness with substantial reduction in the shear cracking (Sundarraja and Rajamohan, 2009).

U-wrap, with FRP also in the flexural region, is the most effective configuration with respect to load capacity. Using U-wrap with FRP in the flexural region, both the shear and the flexural capacity increase and this may also prevent brittle failure (Sundarraja and Rajamohan, 2009). The shear capacity is also dependent upon steel stirrup spacing and amount and distribution of FRP (Khalifa and Nanni, 2002).

Jayaprakash et al (2009), investigated the shear strength of six pre-cracked beams, six non-cracked beams strengthened with bi-directional CFRP intermediate U wraps and four control beams. All the beams were cast without any internal shear reinforcement. It was concluded that CFRP intermediate “U” wraps increase the shear strength and controls de-boning of “U” wrap. They pointed out that this method is more economical with low material quantity. The increase in longitudinal tensile reinforcement ratio by 56% resulted in the maximum increase of 76% in shear strength. The beams were not provided with shear reinforcement. The ends of longitudinal tensile bars were curtailed with any provision for an anchorage. This resulted in bond failure along the tensile reinforcement. The less spacing of CFRP intermediate “U” wraps increased shear capacity and minimized the crack distribution. Comparison between the crack distribution of pre-cracked repaired specimens and non-cracked specimens showed that CFRP intermediate U wraps help to prevent the widening and propagation of early-developed cracks. At the end they have come up with two recommendations to apply bi-directional CFRP intermediate “U” wrap application technique. The recommendations are given below.

1. Recommended, not to use low width CFRP intermediate “U” wraps (<80mm) considering the strength gain and difficulties in applying.

2. For the shear strengthening of beams, it's recommended to apply bi-directional CFRP intermediate U wraps with $0^0/90^0$ orientation and L-wraps inclined at $45^0/135^0$ to the longitudinal axis of beam.

Ali and Noorwirdawati (2013) investigated about the shear behavior of pre-cracked continuous beams repaired using externally bonded CFRP strips. Five beams were tested. After analysis of experimental results, they arrived at number of conclusions. CFRP strip orientation $45^0/135^0$ to the longitudinal axis of beam resulted in fewer or less cracks and gained higher stiffness than the beams with CFRP strips at $0^0/90^0$ orientation. CFRP strips with higher modulus of elasticity enhanced stiffness of beam. The shear strengthening with CFRP strips couldn't reduce deflection of beams. The elongation of CFRP strips restrained the shear crack propagation and increased the shear resistance. The beams wrapped at four sides failed with rupture of CFRP strips and the beams wrapped at three sides failed due to both CFRP rupture and debonding of CFRP strips from the concrete surface.

Fathelbab conducted a parametric study about the finite element modeling of strengthened simple beams using FRP techniques (Fathelbab et al, 2011). Shear strengthening of beams didn't affect load carrying capacity, even with increasing of number of plies. But that has significantly enhanced beam ductility.

With the introduction of CFRP to increase the strength, researchers have narrow down their path to finding ways to get full utilization of this material.

Norris, Saadatmanesh and Eshani studied experimentally and analytically about shear and flexural strengthening of reinforced concrete beams with carbon fiber sheets (Ehsani et al, 1997). In this study, three types of FRP systems were used. System 1 was made of continuous fiber sheets and commercially available epoxy. System 2 was composed of unidirectional fabric and a rubber toughened epoxy. System 3 consisted of a cross ply fabric and rubber toughen epoxy. System 1 and System 2 were tested under three fiber orientations (0^0, 90^0, and 45^0) with respect to the axis of the beam. . Ultimate strength and mode of failure were observed under ultimate load. Finally they concluded that greater strength and stiffness gained to the existing concrete beam in terms of shear and flexure when CFRP sheets are bonded to the web and tension face and further emphasize that large increase in stiffness and strength when the CFRP were placed perpendicular to cracks in the beam and a brittle failure occurred as a result of stress concentration near the end of the CFRP.

Taljsten carried out a research varying the weight of fabric which is directly proportional to the thickness of the fiber and angle of the fiber (Täljsten, 2003). Fiber angles were taken as 0^0, $45^0$and $90^{0.}$ Failure load, deflection and angle of shear were measured. According to the test results Taljisten concluded that, concrete beams can be strengthened for shear by CFRP sheets and it is better to place that fabrics or laminates perpendicular to the shear crack which tends to over-strengthen structures making the shear strength limit as concrete compressive strength. Strain measurement shows that when the situation of thinner fiber was used, it shown the better utilization of the fabric.

Normal way of strengthening of reinforced concrete beams in shear is by bonding CFRP externally to their vertical faces using epoxy adhesive with different spacing and orientations to control diagonal cracks. Another area that researchers had paid their attention is influence of flexural CFRP sheets on shear resistance as in some practical situations it may be difficult to address vertical sides of beams.

Hawileh et al (2015) investigated on this stream and identified that there is relationship between flexural CFRP sheets and shear capacity of reinforced concrete beams. CFRP sheets were externally bonded to the beam soffit and three group of beams were used with different steel reinforcement ratios. Finally conclusions were made that shear strength will increase by 10 – 70% by external longitudinal flexural CFRP and if the steel reinforcement is low higher percentage of strength will increase.

Apart from the traditional methods, new methods were introduced by some researchers. One such method is Embedded Through Section (ETS) technique. This method involves in some construction works. First holes are drilled through the beam section. Then steel or FRP material are entered into these holes and adhesive materials are used to make proper bonding. Breveglieri et al (2015) have studied on effectiveness of different ETS strengthening configurations on shear. ETS technique provides more effective shear strengthening, when compare to traditional methods such as EBR and NSM. Behavior of both vertical and inclined (45^0) ETS CFRP bars were considered in the experiment and identified inclined ETS bars provide higher strengthening effectiveness. Further identified that this method is more effective in strengthening for shear when the transverse reinforcement is low.

2.5. Pre- Cracked beams

Most of the times, structures are not in good conditions whenever they need to be strengthened. Therefore researchers tend to focus on behavior of pre-cracked beams and effectiveness of strengthening pre-cracked structures. Since shear failure is sudden and more critical many studies are focus on impact on shear strength of pre-cracked beams. Ali and Noorwirdawati (2013) studied the behavior of pre-cracked continuous beams with externally bonded CFRP strips and observed 18% to 40% of increase of shear capacity of beams.

2.6. Failure modes

There are three main categories of failure in concrete structures retrofitted with FRP that have been observed experimentally (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007). The first and second type consist of where the composite action between concrete and FRP is maintained.

In the first failure mode, the reinforcement yields followed by rupture of FRP as shown in Figure 2.2.

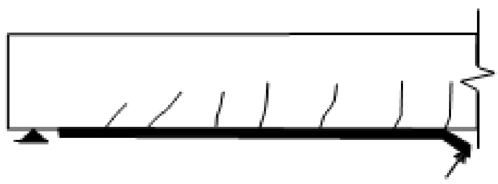


Figure 2.2: FRP rupture (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

In the second type, there is failure in the concrete. This type occurs either due to crushing of concrete before or after yielding of tensile steel without any damage to the FRP laminates. Figure 2.3 shows the inclined shear crack at the end of the plate.

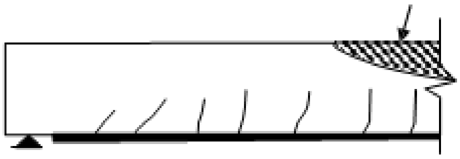


Figure 2.3: Concrete compression failure (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

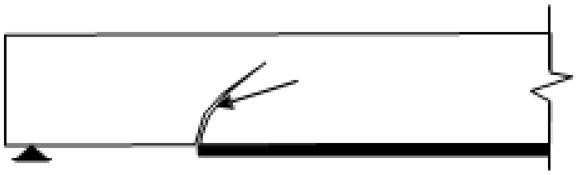


Figure 2.4: Diagonal shear crack at CFRP sheet end (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

Figure 2.4 shows the failure modes involving loss of composite action.

The most recognized failure modes within this group are de-bonding modes. In such a case, the external reinforcement plates no longer contribute to the beam strength, leading to a brittle failure, if no stress redistribution from the laminate to the interior steel reinforcement occurs.

Figure 2.5: CFRP sheet interfacial de-bonding (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

Figure 2.5 shows failure modes of the third type for RC beams retrofitted with FRP.

The failure starts at the end of the plate due to the stress concentration and ends up with de-bonding propagation inwards. Stresses at this location are essentially shear stress due to small but non-zero bending stiffness of the laminate, however normal stresses can arise.

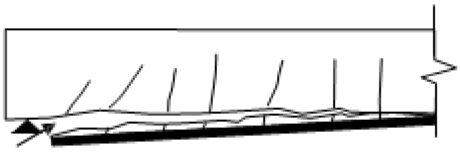


Figure 2.6: CFRP sheet with concrete cover separation (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

For the case in Figure 2.6, the entire concrete cover is separated.

This failure mode usually results from the formation of a crack at or at the end of the plate, due to the interfacial shear and normal stress concentrations. Once a crack occurs in the concrete near the plate end, the crack will propagate to the level of tensile reinforcement and extend horizontally along the bottom of the tension steel reinforcement. With increasing external load, the horizontal crack may propagate to cause the concrete cover to separate with the FRP plate.

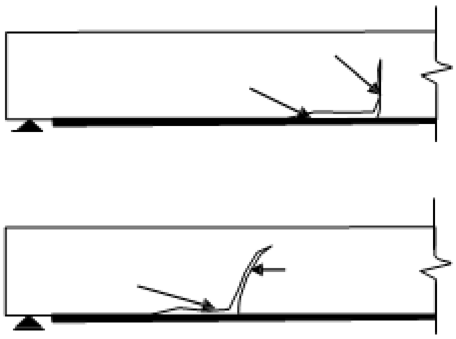


Figure 2.7: Crack propagation parallel to bonded plate (Garden and Hollaway, 1998; Smith and Teng, 2002; Ashour et al, 2004; Esfahani et al, 2007).

In Figures 2.7, the failure is caused by crack propagation in the concrete parallel to the bonded plate and adjacent to the adhesive to concrete interface, starting from the critically stressed portions towards one of the ends of the plate. It is believed to be the result of high interfacial shear and normal stresses concentrated at a crack along the beam. Also mid span de-bonding may take concrete cover with it.

One method to reduce the stress concentration at the plate end and avoid brittle failure is taper end FRP design (Gao et al, 2006). Taper end FRP design means length difference between the adjacent FRP layers at the end.

2.7. Summary of strengthening techniques

Setunge has produced a summary of suitable techniques of strengthning using externally bonded fiber reinforcement polymer (Sujeeva, 2002). This summary is based on number of research carried out in the past. Table 2.1 lists down a summary of strengthening techniques.

Table 2.1: Summary of strengthening techniques (Sujeeva, 2002).

Strengthening method	Design action	Type of FRP	Special considerations
Wet lay up of FRP sheets to the tension zone of the soffit of a beam or slab	Flexural strengthening	Sheets or laminates	De-bonding
Attaching prefabricated FRP sheets to the tension zone of the soffit of a beam or slab	Flexural strengthening	Sheets or laminates	De-bonding
The different types of wrapping schemes to increase the shear strength of a beam or column	Shear strengthening	Sheets	Direction of fibres
Automated winding of wet fibres under a slight angle around columns or other structures	Shear and axial compression strengthening	Sheets	Equipment availability
Attaching prestressed FRP laminates to the tension zone of the soffit of a beam or slab	Flexural strengthening	Laminates	Anchorage
Fusion-bonded pin-loaded straps	Flexural and shear strengthening	Pin-loaded straps	Equipment availability
In-situ fast curing using heating device	Flexural strengthening	Laminates	-
Prefabricated U or L shape strips for shear strengthening	Shear strengthening	Laminates	Direction of fibres
Bonding FRP strips inside	Flexural	Laminates	Crack initiation

concrete slits	strengthening		
FRP impregnation by vacuum to the tension zone of the soffit of a beam or slab	Flexural strengthening	Laminates	Equipment availability
Prefabricated FRP shells or jackets for the confinement of circular or rectangular columns	Axial compression strengthening and ductility enhancement	Sheets	Confining pressure will be different to that of steel
FRP wrapping for axial compression strengthening and ductility enhancement	Axial compression strengthening and ductility enhancement	Sheets	Confining pressure will be different
FRP wrapping for torsional strengthening	torsional strengthening	Sheets	Direction of fibres

2.8. Significance of shear span to depth ratio on failure mode

Khan & Fareed tested six beams with external bonded CFRP wraps at the bottom with and without end anchorages with different shear span to depth (a/d) ratios (Khan et al, 2014). Ultimate load carrying capacity and first crack appeared load was high in beams with low a/d value.   Beams tested at high a/d ratios failed in flexure. Strengthened beams with low a/d ratios failed in two ways depending on availabilty of end anchorages. They failed in shear, although they had adequate shear reinforcement. The beams with end anchorages failed in shear-compression due to diagonal cracks in the web shear zone. The beams without end anchorages failed in shear-tension due to rupture of concrete in web shear zone.

2.9. Effect of end anchors installation

Six beams were flexural strengthened with externally bonded CFRP sheet applied along the tension face of beam and end anchorages were provided in few(Khan et al, 2014). The ductility and stiffness of beams and the load carrying capacity significantly increased with the application of end anchors.

2.10. Effect of length of CFRP on failure mode

Ding investigated the effect of CFRP length on failure mode, failure process and crack propagation of strengthend concrete beams with an initial notch of size 20 mm (Ding, 2014). The dimensions of beam was 100 mmx100 mmx 400 mm. CFRP was bonded on to the bottom of beam. Length of CFRP had an effect on the mode of failure. When the CFRP length was 100 mm (25% of span), the critcal stress region at the end of CFRP plate widenend and resulted in an inclined crack at the end of CFRP plate, crack at the initial notch. When the CFRP length increased from 100 mm (25% of span) to 300 mm (75% of span) overstressed region at the end of plate reduced and resulted in a combination of inclined crack at end of plate, crack at initial notch and smeared crack at the root of notch along the interface. Further increase of CFRP length caused the stress concentration at the end of CFRP to disappear. But due to the tensile strength of the CFRP, stress at the root of notch increased. This resulted in the propergation of smeared crack at the notch and crack at notch. The author recommended 300 mm (75% of span) as the optimum length of CFRP, at which CFRP is fully functioning. The trend of disappearing stress concentraion at the end of CFRP started at the 300 mm length of CFRP. Therefore author has concluded that beam strengthening reached its optimum status at this 300 mm (75% of span) length of CFRP.

2.11. Durability of adhesive used for bonding between concrete and CFRP

Houssam (1997) investigated the the area of long-term durability of externally bonded concrete beams with FRP sheets. It was aimed to study the effect of different environmental conditions such as wet and dry cycling using salt water on the performance of FRP-bonded concrete beams and on the interfacial bond between the fiber and the concrete.

Concrete beams were strengthened with 4 different method. 2 were carbon fiber and 2 were glass fiber. 3 different types of two-part epoxy were used. Test variables included the type of fiber, the type of epoxy system, and the environmental exposure condition. The specimens were arranged in two different environments. One is room temperature (+20 °C), and other one is 300 wet/dry cycles (salt water was used for the wet cycles and hot air at 35 °C and 90% humidity for the dry).

At the end of each exposure, load-deflection curves of the specimens were obtained in order to evaluate their maximum capacity, stiffness, and ductility. The performance of the wet and dry exposed specimens was compared with those kept at room temperature.

Results showed that specimens subjected to wet and dry environmental conditions and those kept at room temperature exhibited significant improvement in flexural strength when FRP sheets were bonded to the tension face of the concrete beams. However, the specimens subjected to wet and dry conditions showed less improvement than those kept at room temperature. None of the specimens failed due to FRP rupture but rather due to the debonding between the FRP sheet and the concrete interface. The selection of epoxy was shown to be very important for using the FRP strengthening technique, especially in a marine environment.

2.12. Summary of literature review

Literature survey was conducted to study the past research woks and technics about performance of heavily cracked R/F cconcrete beams strengthened with Carbon Fiber Reinforced Polymer (CFRP) Sheets. During that survey, it was observed that different researches were carried out to different methods to understand the behavior of CFRP in the retrofitting work.

In this survey, the way to use CFRP in effectively and efficiency way for the strengthening purposes of pre-cracked and non-cracked concrete beams and identify the failure modes, material properties that they used, different kinds of failure mechanisms have been discussed.

Mainly two bonding techniques were used called Externally Bonded Reinforced (EBR) and Near Surface Mounted (NSM) method to fix the CFRP in to the concrete beams. According to the experimental investigations were carried out so far, it can be concluded that NSM provides better shear and flexural capacities compared to externally bonded reinforcement apart from this traditional methods. However the comparing with cost of the retrofitting work and practical difficulties, adopting of NSM methods are not much interest among the society.

So adopting this method to existing site, it causes some additional issues and site people has to utilize the additional precaution for the site.

Many Investigations have been carried out to find the relationships between the CFRP parameters and flexural strength of the beams. It can be concluded that CFRP increases the flexural capacity in a larger percentage and this can be used to strengthen the both pre-cracked and non-cracked beams.

The bond lengths, thicknesses, widths and number of layers of the CFRP sheets, were considered in several investigations. Thereafter, identified that these factors have some impact on the flexural capacity of the structural elements. CFRP will not fully utilized due to de-bonding issues, however the embedded CFRP rods overcome this problem.

Shear failure was a major disaster in beams and therefore shear strength became more popular among many researchers. It was identified that CFRP helps to increase the shear in large extent. Researches have been conducted on optimizing the use of CFRP reinforcement and identified that fiber orientation has greater impact on shear strength. Finally concluded that fiber orientation should be perpendicular to the crack in order to have higher strength. Further identified that, CFRP also has impact on shear strength.

2.13. Research gap identification and needs

Many research gaps were identified during the literature survey. Most of the research were carried out regarding strengthening of non-cracked concrete beams in shear and flexure. Very few have addressed about strengthening of pre-cracked concrete beams with CFRP in flexure.

In most practical situations, we have to strengthen the both pre-cracked and non-cracked beams in structures to prevent structural failures.

Therefore, proper investigation should be carried out to find out the strengthening of pre-cracked concrete beams in flexure by economical and effective way. Although the CFRP strengthening method has undergone a lot of research, the effects of length variation of CFRP sheets and the effects of applying ‘U’ wrap as the end anchorages and intermediate anchorages have not been addressed.

CFRP is an expensive material. Therefore CFRP strengthening is considered as an expensive strengthening method. However the used material amount can be optimized using correct design and arrangement.

So followings were identified as research gap of this project.

- It is important to investigate the effect of length of CFRP sheets on strengthening non-cracked rectangular beams which are commonly used in most of the structures specially in buildings and bridges.

  So the research is based on to investigate the effect of length of CFRP and installation of end anchors on flexural strengthening in non-cracked rectangular beams with CFRP and find the optimum solution.

- And also, it is important to investigate the effect of the end and intermediate anchors in some selected locations on strengthening pre-cracked rectangular beams which are commonly met in most of the structures such as in buildings, bridges etc.

3. EXPERIMENTAL PROCEDURE

3.1. Introduction

This chapter provides details of the experimental program. The experimental program was carried out to study following streams.

- Flexural performance of heavily cracked concrete beams strengthened with CFRP.
- Compare strength gain in heavily cracked and non-cracked beams strengthened with CFRP.
- Performance and strength gain variation with length of the CFRP sheet in tension face.

12 beams were used for the experimental program and six of these beams were cracked during the experiment before strengthening. Four point bending load test was used to test the beams in accordance with ASTM C78.

3.2. Specimen Details

The twelve beams were arranged and 6 of them were non-cracked and other six beams were cracked specimens. The length of each beam was 1900 mm and the span of beam between supports was 1800 mm. The depth of beam was 200 mm and the width was 125 mm. [Theoritical calculations were done in accordance with BS 8110 Part-1 1985. The design calculations are shown in Appendix 1].

The reinforcement arrangement is shown in Figure 3.1. Theoretical calculations show flexural capacity of 12.7 kNm and shear capacity of 79 kN. Since the shear capacity is too high value than flexural, author expects flexural failure happens in all of the beams.
The theoretical calculations of expected beam capacity are shown in Appendix 2.

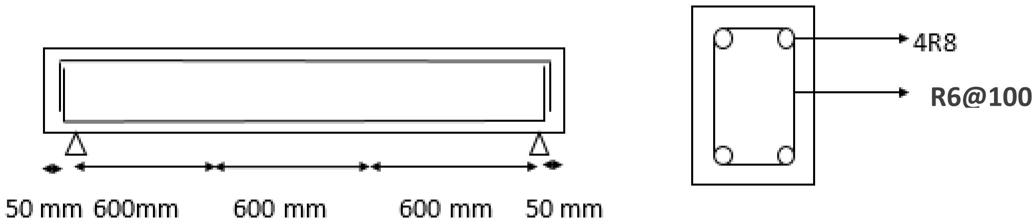


Figure 3.1: Reinforcement arrangement of beam

3.3. Properties of Materials

3.3.1. Carbon Fiber Reinforcement Polymer Sheet

Carbon Fiber Reinforced Polymer (CFRP) is a material with high tensile strength. The manufacturer of CFRP materials used was Simpson Strong-Tie. C-FRP-2014/15 ASIA specifications document provided by the manufacturer contained the following details. The Table 3.1 shows the material properties of CFRP.

Table 3.1: Material properties of CFRP sheets
[www.strongtie.asia, Technical data sheet]

Parameter	**Property**
Technical data	CFRP sheet
Elastic modulus (GPa)	240
Tensile strength (MPa)	4300
Elongation at rupture (%)	1.7
Bond strength (MPa)	≥1.0
Design thickness (mm)	1.68
Weight per unit area of sheet (g/m^2)	330
Width of sheet (mm)	500
Carbon content (g/m^2)	300

3.3.2. Epoxy

Two types of epoxies were used for the experiment.

I. Epoxy grout was used for crack filling purposes to have proper bondage between separated concrete particles.

II. Resin Epoxy was used as the adhesive for CFRP and concrete surface.

The epoxy grout was used for the crack filling purpose. Two paste components (Part A, Part B) are provided for the epoxy. Add the hardener "Part B" to the base "Part A" and mix using a slow speed drill (500rpm) with a coating mixer paddle for 2 to 3 minutes or until both components have fully dispersed and are uniform in colour**.**

The properties of epoxy grout are listed in Table 3.2.

Table 3.2: Material properties of epoxy grout

[www.strongtie.asia, Technical data sheet]

Property	**Typical results**		
Compressive strength	24 hours 60MPa	3 days 65MPa	7 days 85MPa
Tensile strength	-	-	50Mpa
Flexural strength	-	-	60MPa
Application temperature	5 to 40C		
Minimum thickness	0.2 mm		
Maximum thickness	15mm		
Pot life	20C 60 min	30C 30 min	40C 15 min
Mixed viscosity	< 200 cps		

Resin Epoxy was used as the adhesive for CFRP and concrete surface. Two components are provided for the epoxy and the mixture is obtained by mixing those two components with 2:1 proportion on weight. Under high tension, there is a possibility that, the epoxy could fail being ruptured far earlier than the CFRP strip soon after the concrete is being ruptured.

Table 3.3 shows the technical data of epoxy resin and Table 3.4 shows the physical parameters of epoxy resin.

Table 3.3: Technical data of epoxy resin
[www.strongtie.asia, Technical data sheet]

Parameter	**Property**
Technical data	SST resin epoxy 55
Density	1.1 kg/l
Mix ratio by weight	2:1 (resin to hardener)
Tensile strength after 14 days	38.5 MPa
Elongation at break after 14 days	2.3%
Pull off strength on concrete	Failure in concrete
Pull off strength on steel	>15 N/mm^2

Table 3.4: Physical parameters of epoxy resin

[www.strongtie.asia, Technical data sheet]

Parameter	**Value**	
Application temperature	+8^{0}C to +35^{0}C	
Application time	+10^{0}C	180 min
	+20^{0}C	≤ 45 min
	+30^{0}C	≤ 30 min
Dust-dry after		4 hr
Storage	+5^{0}C to +25^{0}C	24 months (from date of manufacture)
Full cure time		7 days

3.4. Methodology

The experimental program was conducted in series of steps as mentioned below.

- Casting of reinforced concrete beams.
- Loading until 0.3mm cracks occur.
- Filling of the cracks.
- Prepare the concrete surface.
- Strengthening beams with CFRP.
- Apply loads until R/F concrete beams were failed.

3.4.1. Casting of Reinforced Concrete Beams

Concrete beams were casted in the building material laboratory in Civil Engineering Department of University of Moratuwa. The beams were casted using grade 30 concrete.

Mix proportion for the Grade 30 concrete is shown in Table 3.5.

Table 3.5: Mix proportion of concrete

Material	**Cement**	**Coarse Aggregate**	**Fine Aggregate**	**Water**
Weight per concrete metre cubic (kg/m^3)	340	1385	515	160

10 concrete cubes were casted to do the compressive strength test. The 28 days average concrete cube strength was 35.29 N/mm^2 with a standard deviation of 2.414.

The test results of the concrete compressive strength are given in Appendix 3.

The concrete compressive strength test is shown in Figure 3.2

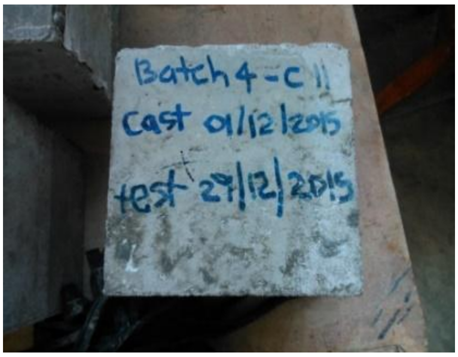


Figure 3.2: Concrete compressive strength test

The reinforcement cage was placed inside the formwork arrangement. Beams before casting and casted concrete beams are shown in Figure 3.3. Electrical compactor (Poker Vibrator) was used for compaction work. The prepared 12 beams were kept to cure for 28 days.

Figure 3.3: Prepared formwork arrangement with R/F cage and casted beam

3.4.2. Initial loading of the beams

6 beams were used for the initial cracking. Those are called the control beams. Cracking of reinforced concrete beams were done using Amsler testing machine according to ASTM C78 [Standard Strength of Concrete simple beam with 4-Point loading]. According to the ASTM guidelines, support blocks should be placed minimum of 1 inch away from the each edge. Therefore support blocks were placed

2 inches away from each beams with the effective span length 1800 mm length. Two dial gauges were fixed at the center of the beam. Then the load was applied accordingly until 0.3mm cracks occur. After that the crack patterns and the deflection of the beam were measured. Figure 3.4 shows the loading arrangement of the beams.

Figure 3.4: Beam testing using Amsler machine

3.4.3. Filling of the cracks

Filling of cracks have to be done before the surface preparation. This task was carried out using an epoxy grout according to ASTM C881 Type I and IV, Grade I, Class C. Initially, drilled through the cracks using drilling machine on the three sides of each beam on crack surface. Injection packers were fixed to pre drilled holes using an epoxy. Surface of the crack was sealed with an epoxy and allowed to cure at least 12 hours. Resin was injected and allowed to cure for 24 hours. The external spilled epoxy was removed using a grinder.

Figure 3.5 shows the sample of drilled beams.

Figure 3.5: Crack filling

3.4.4. Surface preparation

Surface preparation is a prerequisite for strengthening with FRP. That enhances the proper load transferring from FRP to concrete. Sand blasting or grinding can be applied to expose base substrate by removing cement laitance. In the guidelines,[www.strongtie.com; www.strongtie.asia, installation guide line S.1 Substrate] manufacturers state that surface roughness should be maintained in the range of 0.5 mm to 1.00 mm and over a 2 m length of the beam, the plane of the beam shouldn’t vary more than 5 mm.

Figure 3.6: Air compressor

Figure 3.7: Sand blasting

The air compressor machine in Figure 3.6, which was used to generate high pressure air to inject silica particles at high speed is shown in the left side of Figure 3.7. The surface was prepared using sand blasting process. The concrete surface after sand blasting is shown in Figure 3.8.

Figure 3.8: Concrete surface after surface preparation

This concrete surface should be properly cleaned without any substances unless sticking of CFRP cannot be properly done. Sand blasting process was adopted to achieve this target.

3.4.5. Summary of non-cracked beam specimens

The beams are categorized on the proportion of span covered with CFRP. Non-cracked beams were strengthened with CFRP and length of the CFRP was changed in these specimens.

Mainly there are four types of beams. They are “F” Beams [full length CFRP sheet], “IN1” beam [Intermediate CFRP sheet with end anchors], “IN2” beam without end anchors and “M” beams [middle CFRP sheet with end anchors].

The width of strip of CFRP sheet pasted along the tension face of beam was 100 mm. The width of “U” wrap end anchors were 150 mm and they were wrapped around three sides of beams only. The pieces of CFRP sheet were cut along the lines of fiber. A thin layer of epoxy was applied over the concrete surface. After five minutes, the CFRP sheet was placed over applied epoxy layer. A ribbed roller was used to press the CFRP sheet on concrete and remove air bubbles entrapped in the bond line. After that another epoxy layer was carefully applied over the CFRP sheet in fiber direction.

Table 3.6 shows the summary of non-cracked beam specimens.

Table 3.6: Summary of non-cracked beam specimens

Beam Notation	Details of CFRP application
F1	1600 mm long (88% of span length) CFRP sheet pasted along the tension face of beam
F2	1600 mm long (88% of span length) CFRP sheet pasted along the tension face of beam
IN1	1400 mm long (77% of span length) CFRP sheet pasted along the tension face of beam and with U wrap end-anchorage
IN2	1400 mm long (77% of span length) CFRP sheet pasted along the tension face of beam and **without** U wrap end-anchorage
M1	1200 mm long (66% of span length)CFRP sheet pasted along the tension face of beam with 150mm wide, 200mm deep U wrap end-anchorage
M2	1200 mm long (66% of span length)CFRP sheet pasted along the tension face of beam with U wrap end-anchorage

These beams are shown in Figures 3.9 to 3.12.

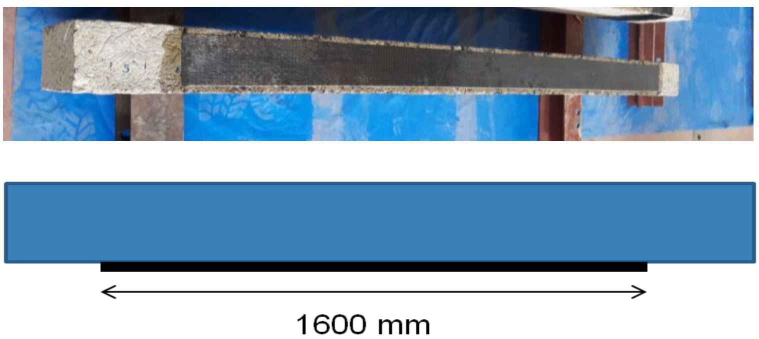


Figure 3.9: ‘F1’ and ‘F2’ beams

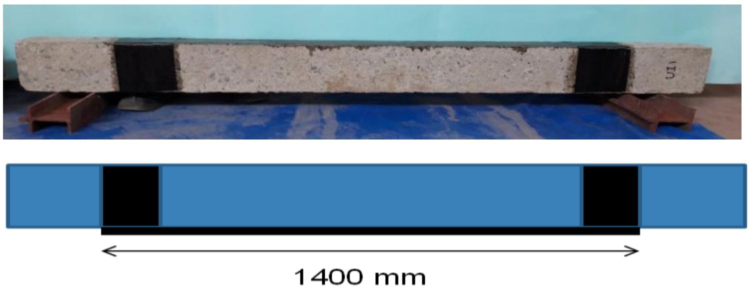


Figure 3.10: ‘IN1’ beam with U wrap end anchorage

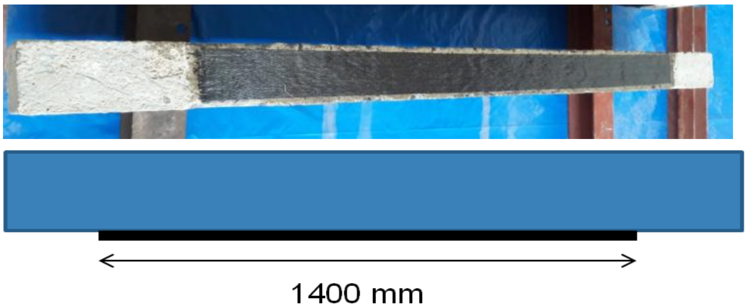


Figure 3.11: ‘IN2’ beam without U wrap end anchorage

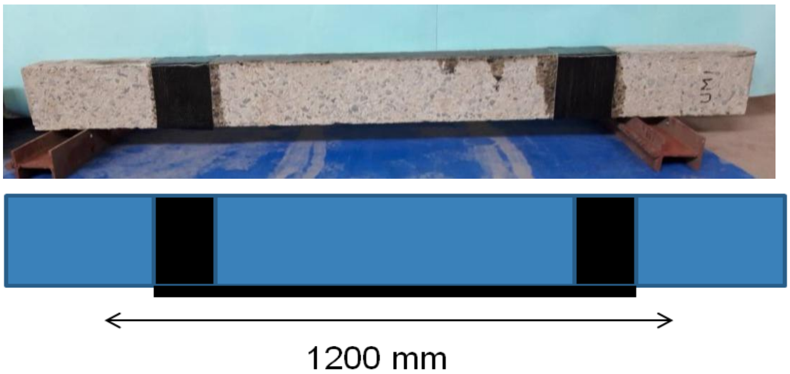


Figure 3.12: ‘M1’ and ‘M2’ beams with U wrap anchorage

3.4.6.Summary of Heavily-cracked beam specimens

Heavily cracked beams were strengthened with different CFRP arrangements. Mainly there are three types of cracked beams and each type has two beams. They are “CF” type, “CFE” type and “CFI” type beams.

Summary of test specimens are shown in Table 3.7.

Table 3.7: CFRP arrangements in heavily cracked beams

Beam Notation	**Details of CFRP application**
CF (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam
CFE (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam with 150mm wide, 200 mm deep U wrap as end-anchorages
CFI (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam with 150mm wide, 200mm deep U wrap as end-anchorage and U wrap at cracked places

They are shown in Figures 3.13, 3.14, and 3.15.

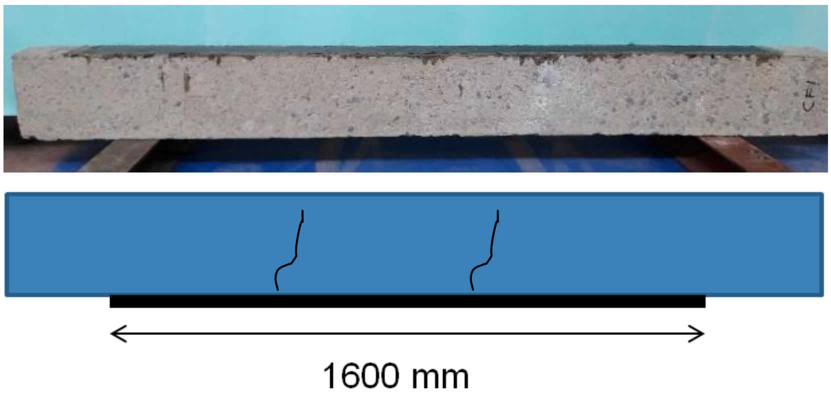


Figure 3.13: "CF" beams

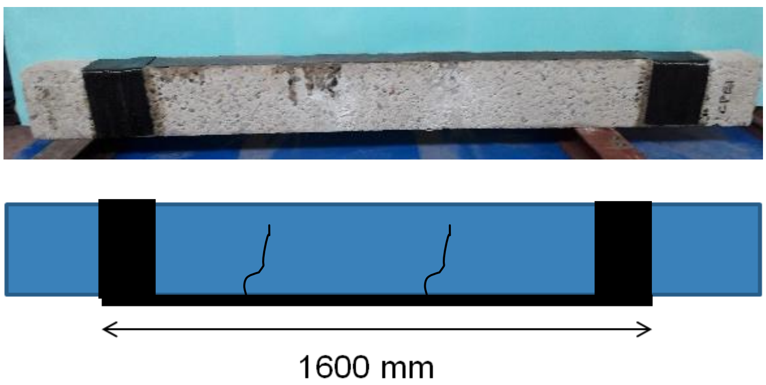


Figure 3.14: "CFE" beams

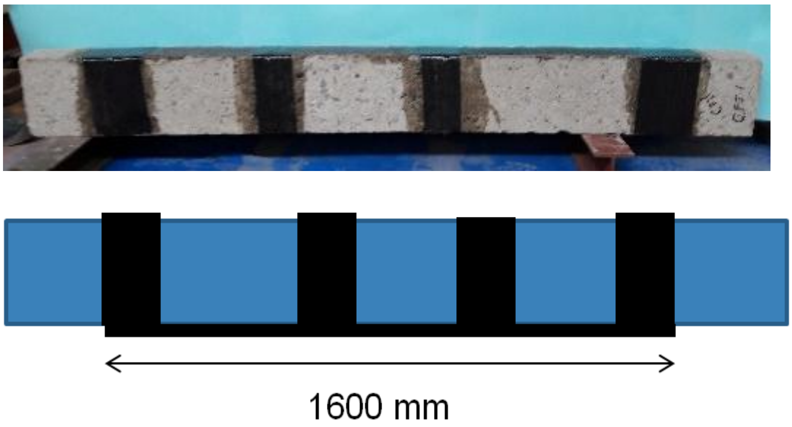


Figure 3.15: "CFI" beams

3.4.7. Testing of beams

The load was gradually applied on reinforced concrete beams using Amsler machine in accordance with ASTM C78. The supports were placed 50 mm away from each end of the beam. Therefore the span of the beam was 1800 mm.

One dial gauge was fixed at the center of the beam. Other two dial gauges were fixed at 300 mm away from the center of beam. The locations at which dial gauges were fixed are shown in Figure 3.16.

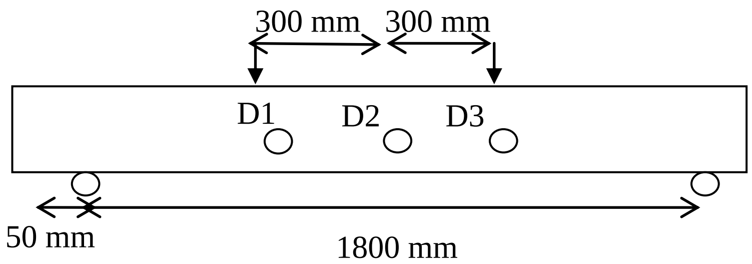


Figure 3.16: Installation of dial gauges

The dial gauges readings and applied load were recorded at 0.1 metric ton intervals. Arrangement is shown in Figure 3.17. The cracks were measured and the load at which 0.3 mm cracks appeared was recorded. The each beam was loaded until failure. The failure load, mode of failure and crack pattern were observed in each failure.

Figure 3.17 : Fixed dial gauge to beam

3.5. Summary

This chapter provides the details of the experimental program. It includes the sample details and properties and technical data of used materials. Further, this presents the way to prepare concrete beams, initial loading of beams, crack filling methods, way to prepare the surface of the concrete beams, and ultimately strength of beams with CFRP.

4. RESULTS AND ANALYSIS

4.1. General

12 Nos. of pre-cracked and non-cracked concrete beams were tested. Four point bending load test in accordance with ASTM C47 was used for this testing work. The parameters such as deflection at which 0.3 mm cracks appear and ultimate failure loads were monitored. Finally the crack patterns and failure mode were observed.

4.2. Test results of control beams

Ultimate failure load at each control beam specimen, load and deflection at which 0.3 mm wide cracks appeared were summarized in the Table 4.1.

Table 4.1: Results of control beams

Control Beam Notation	The load [at which 0.3 mm wide crack appears (kN)]	The ultimate failure load (kN)	Deflection [at which 0.3 mm wide cracks appear (mm)]
C1	14.715	16.090	3.5
C2	14.715	15.500	5.0
C3	10.790	13.730	12.0
C4	7.850	14.510	8.0
C5	14.720	16.100	9.0
C6	12.950	14.130	10.0

When considering the average load for 0.3mm cracks appeared of beams was 12.62 kN, However the ultimate load vary from 13.73kN to 16.10 kN. Maximum deflection show in beam samples of C3 and C6. Mean average load for 0.3mm cracks appeared was 12.295 kN and relevant Standard deviation was 2.547.

Figure 4.1 shows the load Vs. Deflection graph of control beams. When the load increases gradually up to 8 kN, deflection is below the 1 mm range. This increment of deflection can be negligible. However the load increment from 8 kN to 14 kN range, mostly 3 mm increment of deflection can be appeared. ultimately up to failure stage, rapid deflection can be noted down.

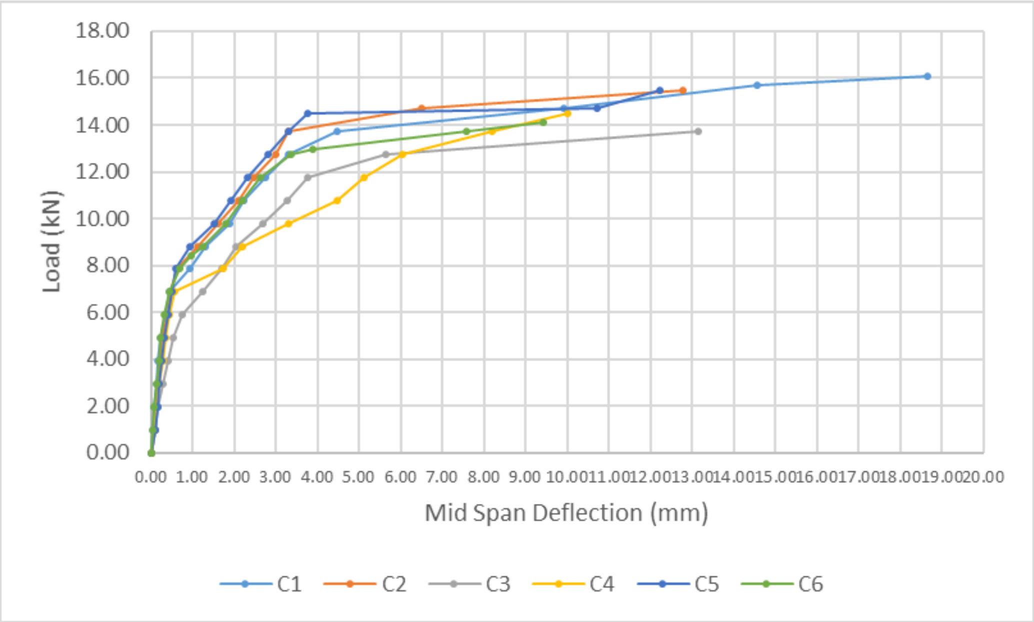


Figure 4.1: Load Vs. Deflection of control beams

Figure 4.2 shows the crack pattern of control beams. Flexural cracks appear at the side face of the all beams. Around 2-3 Nos. of cracks could be noted after initial loading.

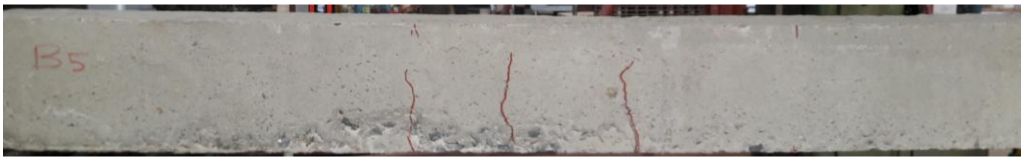
Beam C1

Beam C2

Beam C3

Beam C4

Beam C5



Beam C6

Figure 4.2: Crack patterns in control beams

4.3. Non-cracked concrete beams strengthened with CFRP

6 Nos. of non-cracked beams were strengthened in 4 ways as shown in Table 3.6. They are “F” type [1.6m length CFRP sheet], “IN” beam [Intermediate length CFRP sheet(1.4m) with end anchors], “IN” beam [Intermediate length CFRP sheet(1.4m)] without end anchors and “M” beams [Middle length CFRP sheet(1.2m)].

4.3.1. Beam Type F1

Figure 4.3: Failure mechanism of beam Type F1

A flexural crack appeared towards the left side of beam. Then the crack widened to 0.3 mm when the load was increased up to 16.7 kN. At the left side of beam, i.e end of the CFRP sheet started delaminating with the propagation of flexural crack. The delaminated end is circled in the Figure 4.3. The deflection in the middle of beam was about 3.5mm at failure.

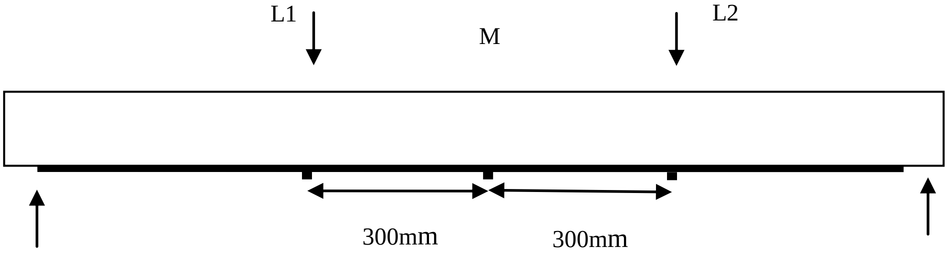


Figure 4.4 : Schematic diagram of loading arrangement

The load vs. Deflection graph of F1 beam is shown in Figure 4.5. The curve shows that the beam is approaching a failure load of 17 kN. The percentage increment of strength with respect to control beam was 2%. Delamination of CFRP would be the main reasons to less strength gain.

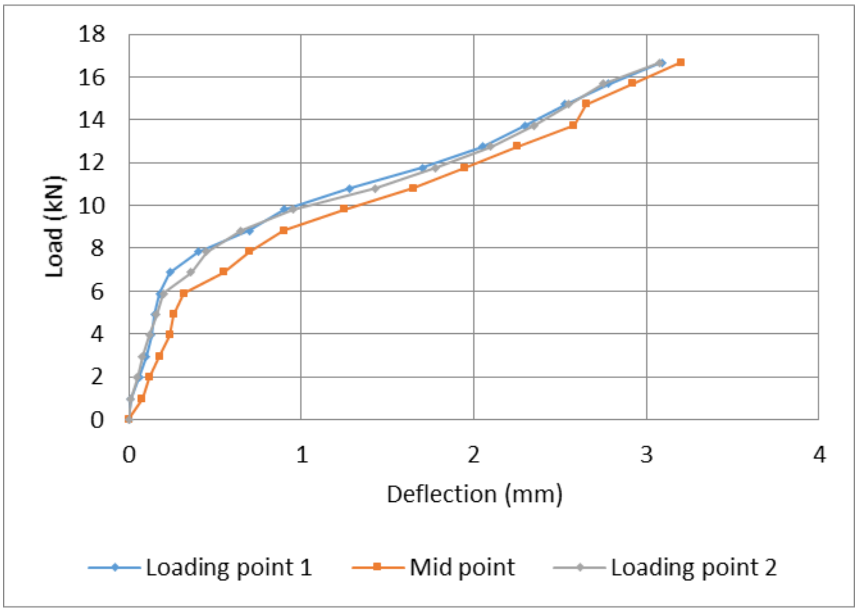


Figure 4.5 : Load Vs. Deflection of beam type F1 beam type F2

Figure 4.6: Failure mechanism of beam Type F2

After the loading of beam “F2”, two flexural cracks appeared in the middle of beam as marked in the Figure.4.6. When the load was increased to 19.6 kN 0.3mm cracks appeared.

The ends of CFRP sheet was delaminated with the propagation of flexural cracks. The deflection at the middle of beam was about 5 mm at failure. The load vs. Deflection graph of beam “F2” is shown in Figure 4.7. The curve shows that the beam is approaching a failure load of 20 kN. The percentage increment of strength with respect to control beam was 20%.

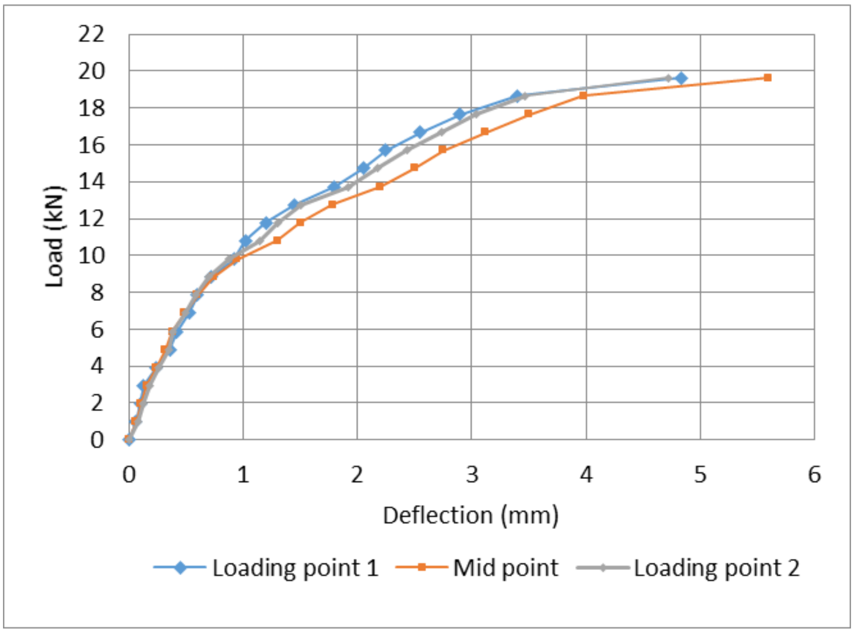


Figure 4.7: Load Vs. Deflection of beam Type F2

4.3.2. Beam type IN

The two beams were tested under this category.

Figure 4.8: Failure Mechanism of beam Type “IN 1”

In IN 1 beam, very small 6 Nos. of uniformly distributed flexural cracks appeared along the beam as shown in Figure 4.8. The cracks widened to 0.3 mm, when the load was increased up to 30.4 kN. The CFRP started to rupture in the middle of beam further increase in loadings. The load vs. Deflection graph of IN 1 beam is shown in Figure 4.9. The curve shows that the beam is approaching a failure load of 35 kN. This is about 110% strength gain when compare with control specimen. The deflection at the middle of beam was about 12.0 mm at failure.

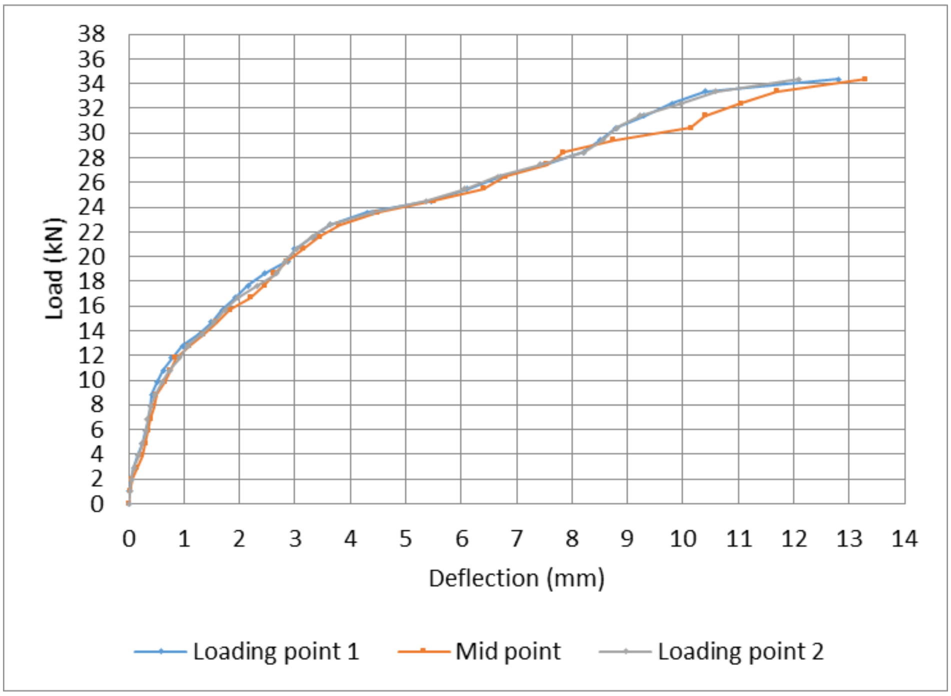


Figure 4.9: Load Vs. Deflection of beam type “IN1”

Figure 4.10: Failure Mechanism of beam type “IN 2”

In the IN 2 beam, about 7 large flexural cracks appeared along the beam as shown in Figure 4.10. The cracks widened to 0.3 mm, when the load was increased up to 26.5 kN. This is about 62% strength gain with respect to control beams. The CFRP sheet started to delaminate at the two ends of beam with further increment of loadings. Delamination of CFRP sheet at one end is shown in Figure 4.11.

Figure 4.11: Delamination end of CFRP sheet

The load deflection relationship is shown in Figure 4.12. The maximum mid span deflection at failure was 8.0 mm. 10.791mm deflection was appeared in control beam. This showed that the reduction of deflection after strengthening. The curve shows that the beam is approaching a failure load of 27 kN.

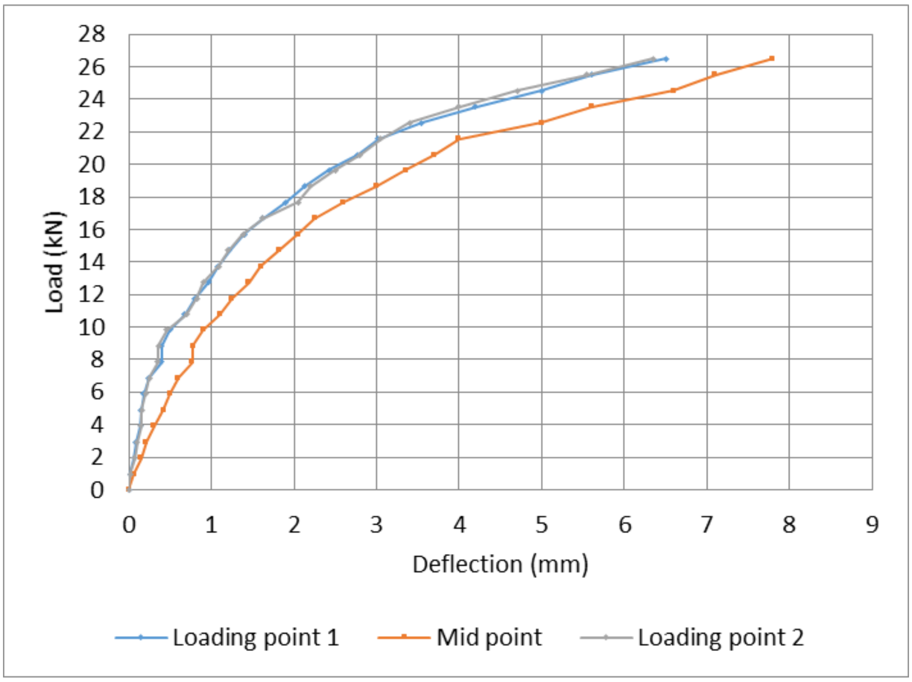


Figure 4.12: Load Vs. Deflection of beam type "IN2"

4.3.3.Beam Type M

In this beam, 1,200mm long CFRP sheet was pasted along the tension face of the beam with 100mm wide and 150mm height with end "U" wraps.

Figure 4.13: Failure mechanism of beam type M1

In M1 beam, three flexural cracks appeared in the middle of beam at failure. The Figure 4.13 shows the failure mechanism. The cracks widened to 0.3 mm when the load was increased to 26.5 kN. This is about 62% strength gain with respect to control specimen. The CFRP started to rupture in the middle of beam with the load increment. At the failure, flexural cracks propagated towards the top of beam and the “U” wrap end anchor at one end delaminated.

Delamination of CFRP “U” wrap end anchor at one end is shown in Figure 4.14.

Figure 4.14: Delamination of U wrap end anchors

The load vs. Deflection graph of M1 beam is shown in Figure 4.15. The curve shows that the beam is approaching a failure load of 27 kN. The maximum mid span deflection noted at failure was 9 mm. This is about 4.4% decrement when comparing with control specimen.

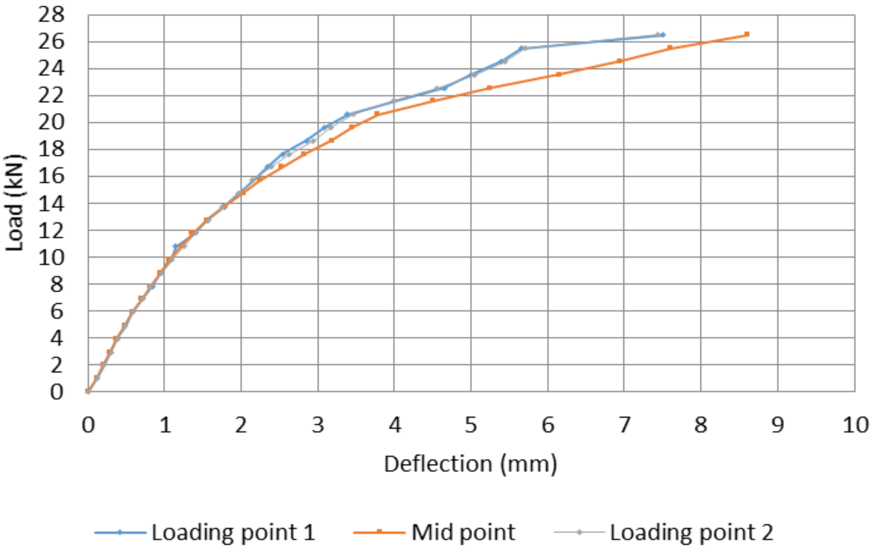


Figure 4.15: Load Vs. Deflection of M1

M2 beam

Figure 4.16: M2 beam after loading

In M2 beam, two major flexural cracks appeared in the middle of beam. The cracks widened to 0.3 mm when the load was increased up to 23.5 kN. The CFRP started to rupture in the middle of beam with the load increment. At the failure, flexural cracks propagated towards the top of beam and CFRP ruptured. The propagation of flexural crack is shown in Figure 4.16.The deflection in the middle of beam was about 10 mm at failure. Figure 4.17 shows that the flextural cracks appeared at the middle of the beam.

Figure 4.17: Large flexural cracks appeared at the middle of beam

The load vs. Deflection graph of M2 beam is shown in Figure 4.18. The curve shows that the beam is approaching a failure load of 28 kN.

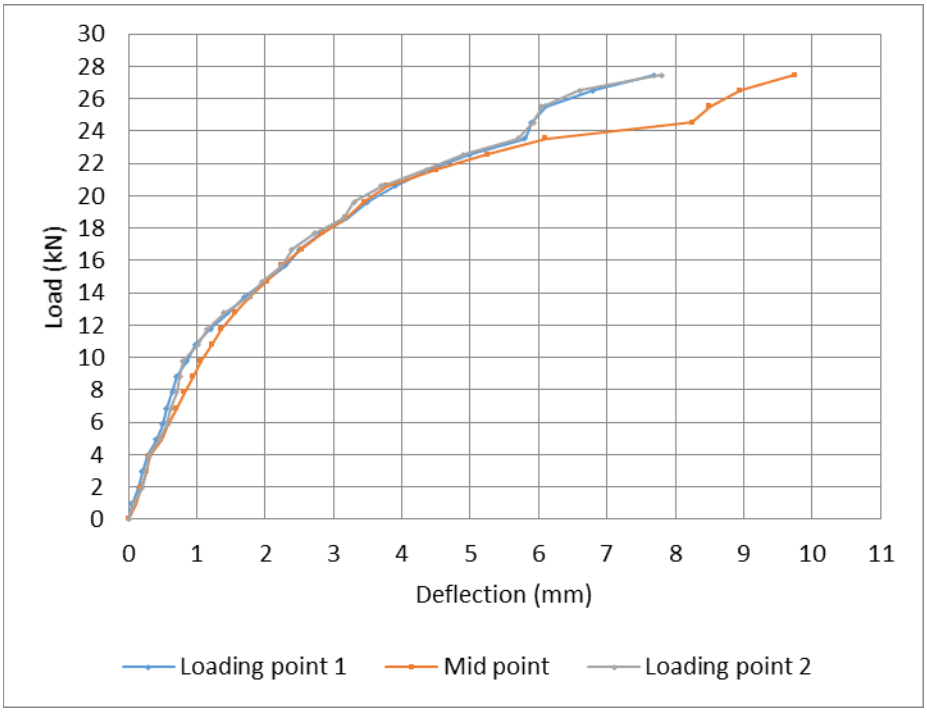


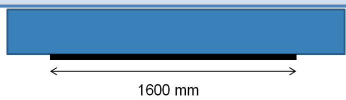
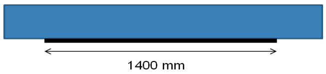
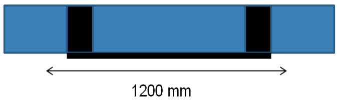
Figure 4.18: Load vs. Deflection of M2

4.3.4.Comparison

Different failure mechanisms were observed in F, IN and M beams during the testing stage. The most dominant failure mode in beams without end anchors was delamination of CFRP sheet and the beams with end anchors, CFRP rupture was dominant.

They are listed in Table 4.2

Table 4.2: Summary of failure modes of beam specimens

Beam Notation	**Mode of failure**	**Beam arrangement before failure**
F1	De bonding end of CFRP	
F2	De bonding end of CFRP	
IN1	CFRP rupture	 
IN2	De bonding end of CFRP	 
M1	CFRP rupture and de bond U wrap	
M2	CFRP rupture at mid span	

The failure load of each beam specimen and the load at which 0.3 mm wide cracks appeared are summarized in the Table 4.3.

Table 4.3: Percentage strength gained in non-cracked beams relative to control beams

Beam Notation	**The load at which 0.3 mm wide cracks appears (kN)**		**The Ultimate Failure Load (kN)**		**Percentage of strength gain relative to control beams**		**Remarks**
		Average		**Average**		**Average**	
F1	16.7	18.15	17	18.5	2%	10.78%	CFRP at bottom surface
F2	19.6		20		20%		
IN1	30.4	28.45	35	31	110%	85.63%	CFRP at bottom surface with end and intermediate anchors
IN2	26.5		27		62%		
M1	26.5	25	27	27.5	62%	64.67%	CFRP at bottom surface with end anchors
M2	23.5		28		68%		

When comparing average ultimate failure load with F, IN and M beams with respect to control beam, noted values are higher. However IN type beam shows the highest Ultimate Failure load. This is about 85.63% strength gain relative to control beam.

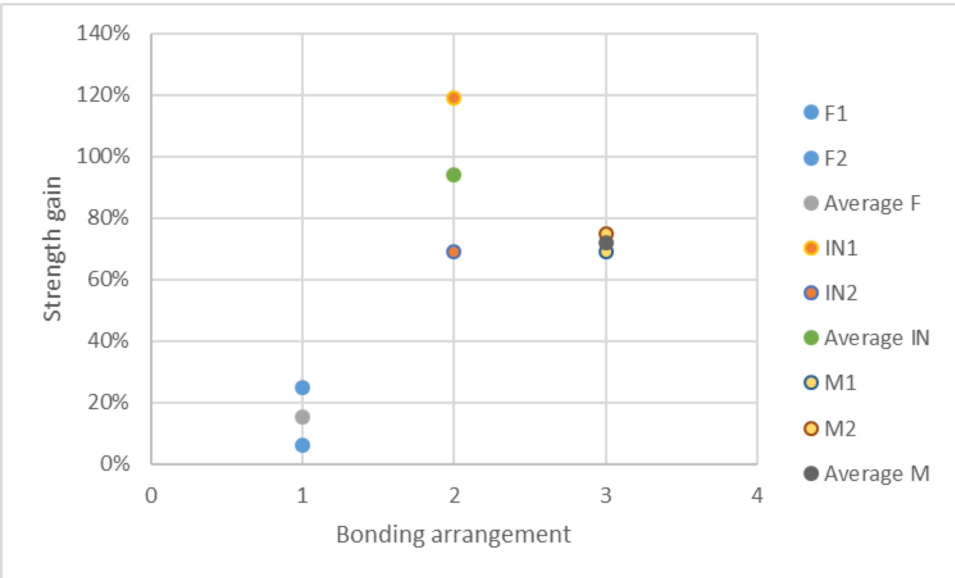


Figure 4.19: Percentage Strength gain Vs. Bond arrangement

The highest strength gain was indicated in the IN type beams arrangements and the least strength gain was noted in the F type beams arrangements. This highest strength gain includes the 1400 mm length CFRP sheets with end anchorage in IN1 type beam. IN2 beam shows less strength gain comparing with IN1 beam. The % strength gain Vs. bond arrangement is shown in Figure 4.19.

Graph shows the highest strength gain in IN type beams.

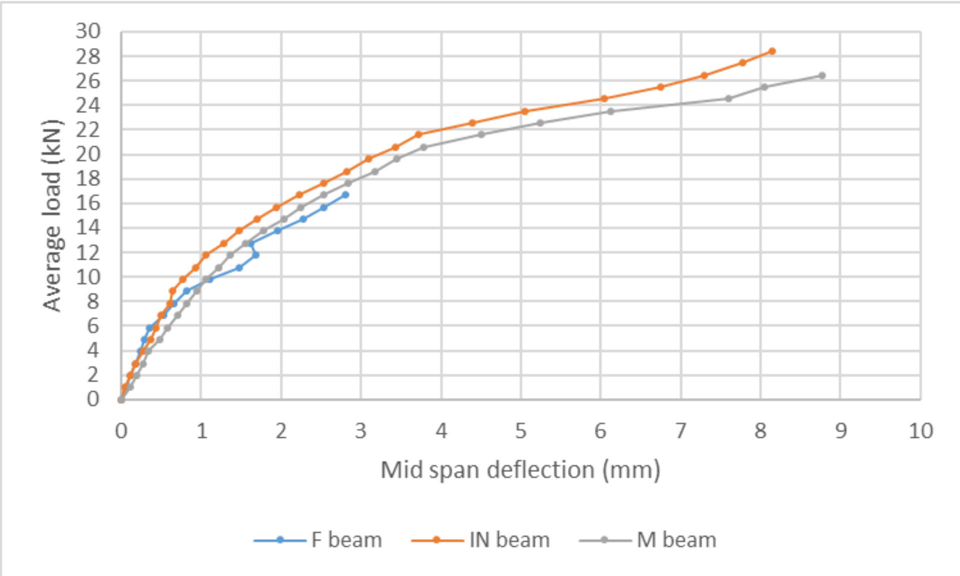


Figure 4.20: Average load Vs. Average mid span deflection

Average load Vs. mid span deflection is shown in Figure 4.20. According to this figure, the maximum deflection shows in ‘F’ type beams and less deflection shows in ‘IN’ type beams for some particular load.

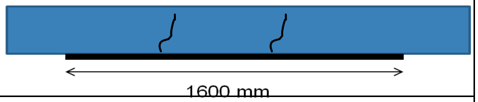
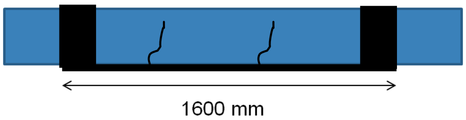
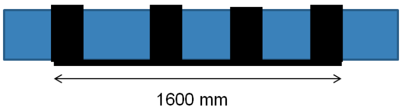
4.4. Pre-cracked reinforced concrete beams strengthened with CFRP

4.4.1. Arrangement of CFRP sheets

3 patterns of CFRP systems were considered. First pattern is 100mm wide 1600mm long CFRP sheet on the bottom side of the beam. Second pattern is 100mm wide and 1600mm long CFRP sheet on bottom face and 150mm wide “U” wraps at the end of CFRP sheet. Third pattern is 100mm wide 1600mm long CFRP sheet on the bottom side of the beam with 150mm wide U wraps at end of CFRP sheet and 100mm wide U wraps at each crack of the beam.

The Summary of CFRP arrangement in heavily cracked beams is given in Table 4.4.

Table 4.4: Summary of CFRP arrangement in heavily cracked beams

Beam Notation	Details of CFRP application	CFRP arrangement
CF (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam	 
CFE (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam with 150mm wide, 200 mm deep U wrap as end-anchorages	 
CFI (2 Nos)	1600 mm long CFRP sheet pasted along the tension face of heavily cracked beam with 150mm wide, 200mm deep U wrap as end-anchorage and U wrap at cracked places	 

CF1 Beam

In CF1 beam, 2 flexural cracks appeared in the middle of beam. The cracks widened to 0.3 mm, when the load was increased to 18.64 kN. The ends of CFRP sheet was delaminated with the propagation of flexural cracks as shown in Figure 4.21. The deflection in the middle of beam was about 5.2 mm at failure. The percentage increment of strength with respect to control beam was 40%.

Figure 4.21: Delamination of end CFRP

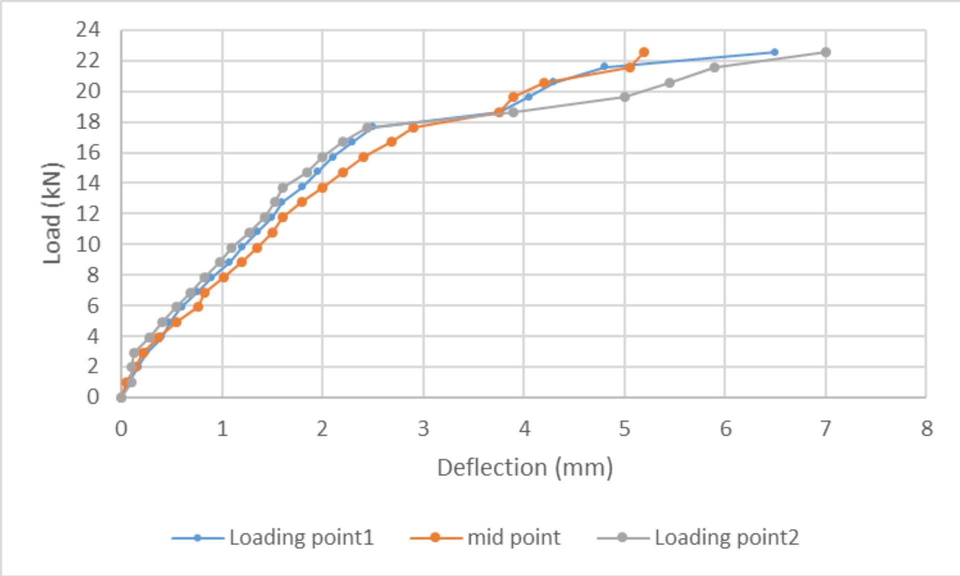


Figure 4.22: Load Vs Deflection of CF1

The load vs. Deflection graph of CF1 beam is shown in Figure 4.22 above. The curve shows that the beam is approaching a failure load of 23 kN.

CF2 Beam

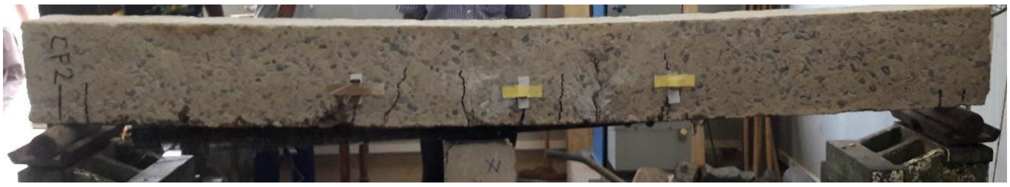


Figure 4.23: CF2 beam after loading

In CF2 beam, uniformly distributed five flexural cracks appeared along the beam as shown in Figure 4.23. The cracks widened to 0.3 mm when the load was increased to 27.5 kN. The CFRP started to delaminate at one end of the beam with load increment. The deflection in the middle of beam was about 6 mm at failure.

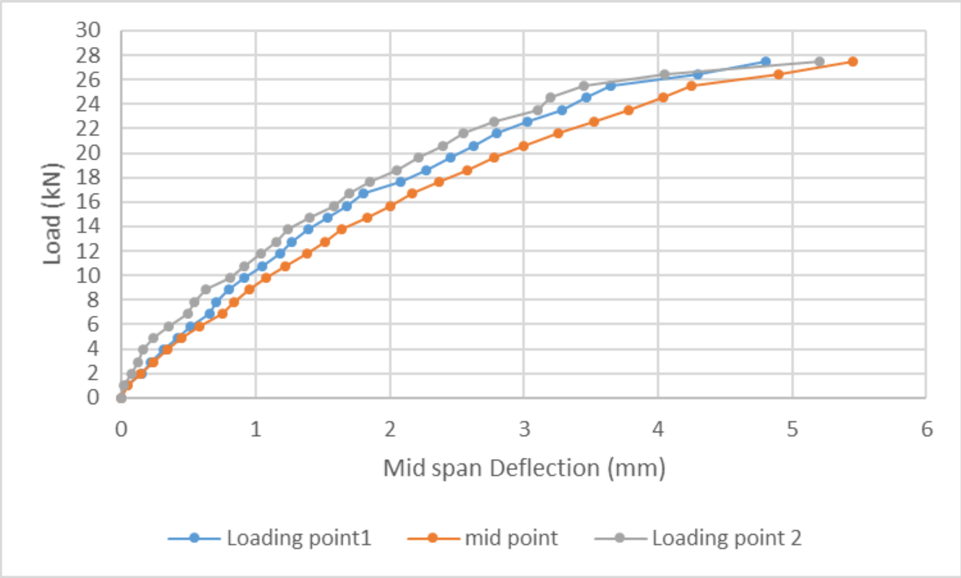


Figure 4.24: Load Vs. Deflection of CF2 beam

The load vs. deflection graph of CF2 beam is shown in Figure 4.24. The curve shows that the beam is approaching a failure load of 27.5 kN. The percentage increment of strength with respect to control beam was 122%.

CFE1 Beam

Figure 4.25: CFE1 beam after loading

In CFF1 beam, two flexural cracks appeared in the middle of beam as shown in Figure 4.25. Then the cracks widened to 0.3 mm when the load was increased to 18.7kN. The end anchorage was delaminated with the propagation of flexural cracks. The deflection in the middle of beam was about 6 mm at failure.

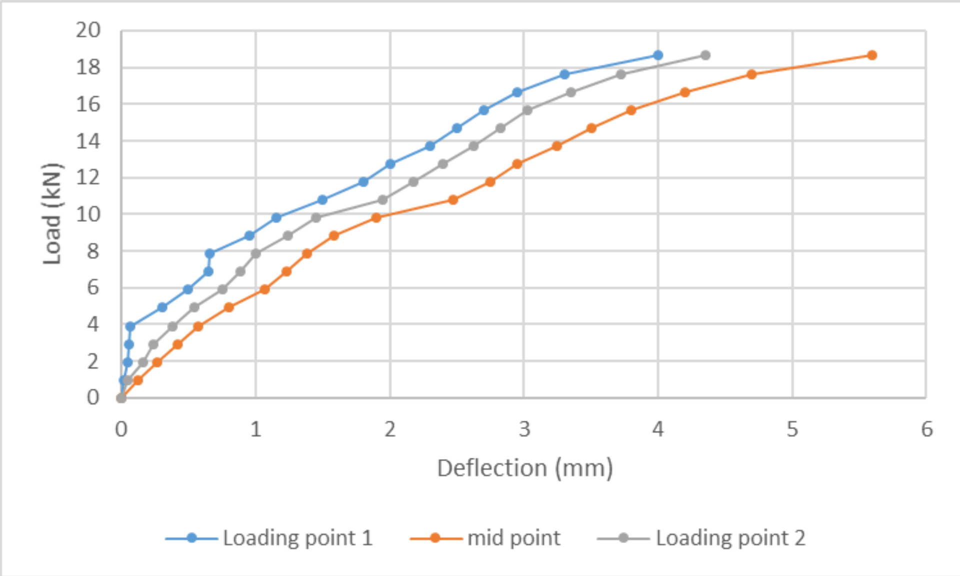


Figure 4.26: Load Vs. Deflection of CFE1

The load vs. deflection graph of CFE1 beam is shown in Figure 4.26 above. The curve shows that the beam is approaching a failure load of 18.7kN. The percentage increment of strength with respect to control beam was 73%.

CFE2 Beam

Figure 4.27: CFE2 Beam after loading

In CFE2 beam, 8 flexural cracks appeared in the middle of beam as shown in Figure 4.27. Then the cracks widened to 0.3 mm when the load was increased to 27.5kN. The end anchorage was delaminated with the propagation of flexural cracks. The deflection in the middle of beam was about 6 mm at failure.

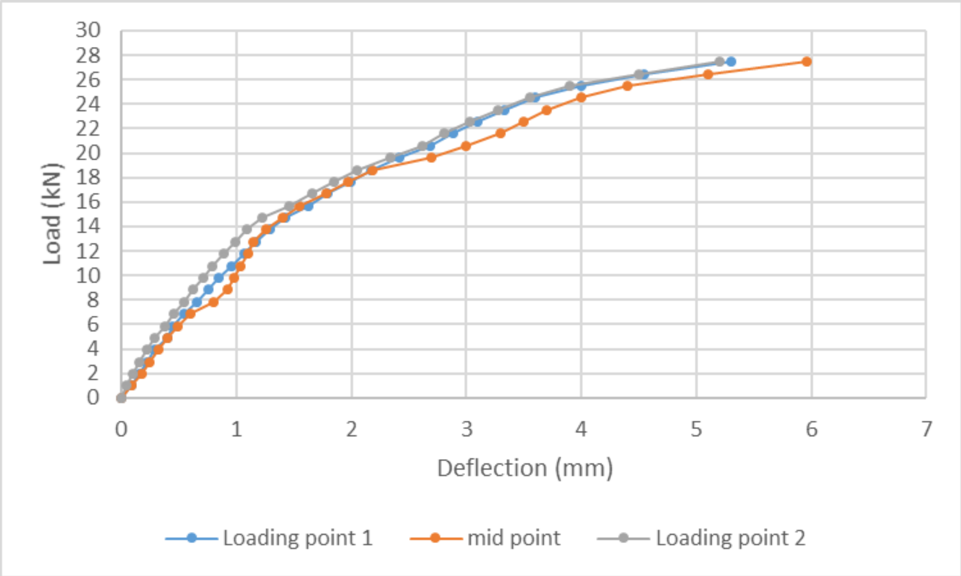


Figure 4.28: Load Vs. Deflection of CFE2 beam

The load vs. Deflection graph of CFE2 beam is shown in Figure 4.28 above. The curve shows that the beam is approaching a failure load of 27.8 kN. The percentage increment of strength with respect to control beam was 250%.

CFI1 Beam

Figure 4.29: CFI1 beam after loading

In CFI1 beam, three flexural cracks appeared in the middle of beam as shown in Figure 4.29. Then the cracks widened to 0.3 mm, when the load was increased to 25.5 kN. CFRP sheet was delaminated with the propagation of flexural cracks. The deflection in the middle of beam was about 5 mm at failure.

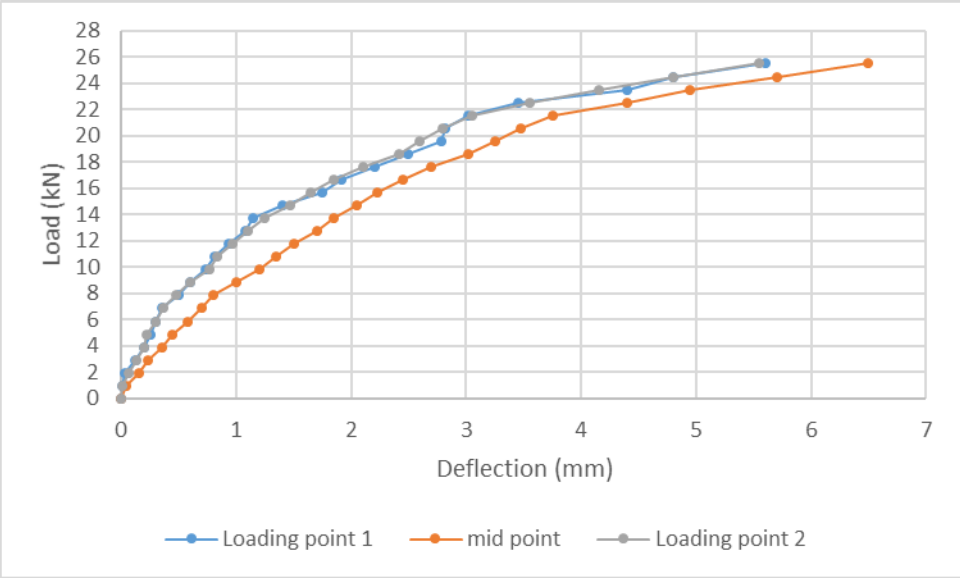


Figure 4.30: Load Vs. Deflection of CFI1

The load vs. Deflection graph of CFI1 beam is shown in Figure 4.30 above. The curve shows that the beam is approaching a failure load of 28.0 kN. The percentage increment of strength with respect to control beam was 73%.

CFI2 Beam

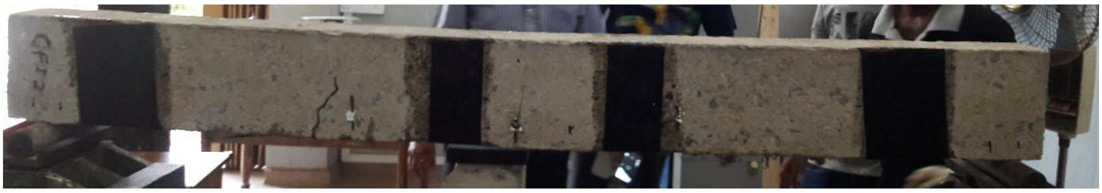


Figure 4.31: CFI2 beam after loading

In CFI1 beam, 3 flexural cracks appeared in the middle of beam as shown in Figure 4.31. Then the cracks widened to 0.3 mm, when the load was increased to 25 kN. CFRP sheet was delaminated with the propagation of flexural cracks. The deflection in the middle of beam was about 3.2 mm at failure.

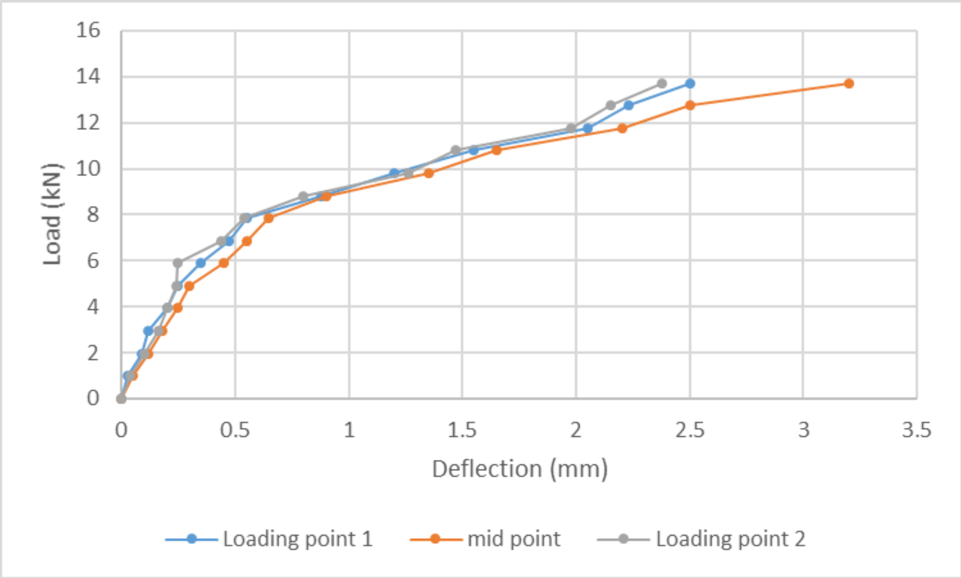


Figure 4.32: Load Vs. Deflection of CFI2 beam

The load vs. Deflection graph of CFI2 beam is shown in Figure 4.32 above. The curve shows that the beam is approaching a failure load of 13.8 kN. The percentage increment of strength with respect to control beam was 93%.

The summary of failure modes of CFRP strengthened heavily cracked beam specimens are summarized in the Table 4.5.

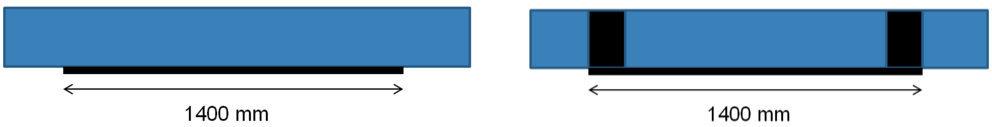
Table 4.5: Summary of failure modes of beam specimens

Beam notation	Mode of failure
CF1	End CFRP de-bonding
CF2	End CFRP de-bonding
CFE1	CFRP de-bonding (initially u wraps)
CFE2	CFRP de-bonding (initially u wraps)
CFI1	CFRP de-bonding
CFI2	CFRP de-bonding

Table 4.6: Percentage strength gain in cracked beams relative to control beams

Beam Notation	The load at which 0.3 mm wide cracks appears (kN)		The Ultimate Failure Load (kN)		Percentage of strength gain Relative to control beams	
CF1	18.64	23.07	23	25.25	27%	56.78%
CF2	27.5		28		87%	
CFE1	18.7	23.1	21.5	24.75	73%	161.81%
CFE2	27.5		28		250%	
CFI1	25.5	19.65	28	21	73%	83.14%
CFI2	13.8		14		93%	

4.5. Test results analysis of non-cracked (Control) strengthened beams



(a) IN2 Type Beam **(b)** IN1 Type Beam

Figure 4.33: IN2 type beam with IN1 type beam

Figure 4.33 shows the schematic diagram of IN2 Type beam with IN1 Type beam. The test results obtained from IN 2 beam without end anchors and IN 1 beam with end anchors were compared with each other. Installation of end anchors has affected the mode of failure and failure load of beams significantly. There has been a 48% [110%-62%] increment in strength of beams with "U" wrap end anchorage. And also the beams without end anchors failed with CFRP sheet delamination. When the end anchors were applied, the delamination was prevented and rupture in the middle of the beam. The CFRP was allowed to reach its ultimate strength and caused to enhance the beam strength. The comparison of deflections of the IN1 beam 12 mm and IN2 beam 8 mm were observed at the failure.

(a) IN2 Type Beam **(b)** F Type Beam

Figure 4.34: IN2 type beam with F type beam

Figure 4.34 shows the schematic diagram of IN2 Type beam and F Type beam.

The test results obtained from IN 2 beam and F1 and F2 beams were compared with each other. The average of failure loads of F1 and F2 beams is 18.5 kN. The average failure load of IN2 beam was 27 kN. That shows a 46% increase in strength of beam with reduction of length of CFRP.

77% of span is covered with CFRP in IN 2 beam and 88% of span is covered with CFRP in F beams. The deflection at failure in IN2 beam was 12 mm and average deflection of Type F beam was 4.25mm. It was observed that 14% CFRP sheet length increment would course to considerable changes of deflection from 12 mm to 4.25mm.

Therefore, when the percentage of span covered with CFRP was increased from 77% to 88%, a strength reduction of 46% was observed. So the beams with 77% of span covered with CFRP without end anchors showed better performance.

(a) M Type Beam **(b)** IN1 Type Beam

Figure 4.35: M type beam and IN1 type beam.

Figure 4.35 shows the schematic diagram of M Type beam and IN1 Type beam

The M beams with 66% of span covered with CFRP and IN 1 beam with end anchors and 77% of span covered with CFRP show the effect of length of CFRP on beams with end anchors. The average of failure loads of M1 and M2 beams is 27.5

kN. Therefore the failure load of M beams could be taken as 27.5 kN. The failure load of IN 1 beam was 35 kN.

When the length of CFRP was increased from 1200 mm to 1400 mm with end anchorages, the strength of beam was increased by 27%. This shows the beams with end anchors with 77% of span covered with CFRP showed better performance when compared with the beam with 66% of span covered with CFRP including end anchorages.

It was observed the 17% CFRP length increment would cause to considerable reduce of average deflection from 12.5 mm to 8.0 mm. In these situations, the common failure mechanism of CFRP was rupture and U wrap de bonding.

4.6. Test results analysis of pre-cracked strengthened beams with control beams

4.6.1. Flexural capacity

In the experiment, it has been shown that, CFRP caused to increase the flexural capacity of both non-cracked and pre-cracked beams. Figure 4.36 shows the Failure loads of cracked beams, before and after strengthening.

When the ‘CF’ beam pattern was used in the cracked beams, it showed the increment of flexural capacity by 56.78% and when ‘CFE’ beam Pattern showed the increment of flexural capacity by 161.81%. When the ‘CFI” beam Pattern showed the increment of flexural capacity by 83.15%.

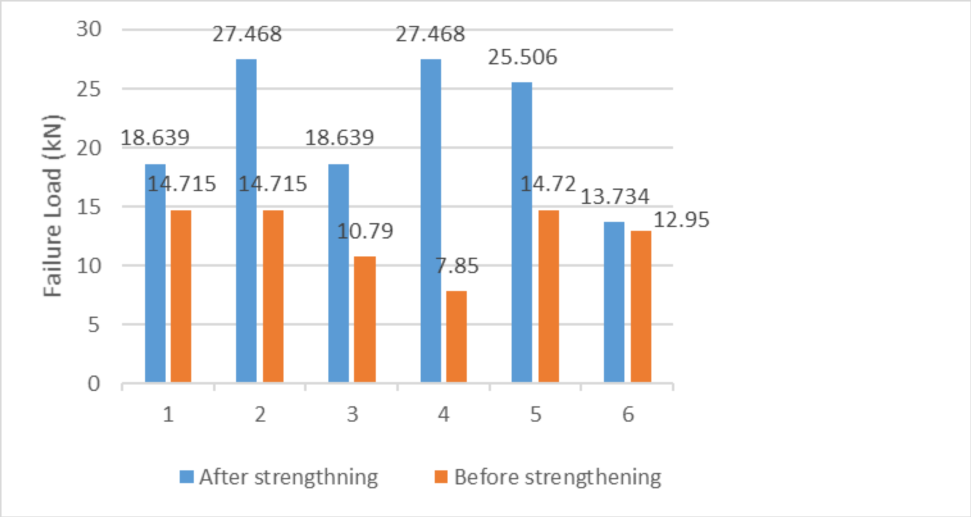


Figure 4.36 : Failure loads of cracked beams before and after strengthening

It was observed that providing CFRP in tension face (CF) and providing CFRP in tension face with end U wraps (CFE) gave much higher results. Generally 'U' wrap are provided for shear strengthening purposes. However here it was used to confine the bottom CFRP sheet. According to the results, it has increased the flexural capacity drastically. Therefore we can conclude that even though confinement provide at ends, it will help to increase flexural capacity in larger scale in beams. When provide more confinement in cracked places, it has shown that, there is some increment in flexural capacity.

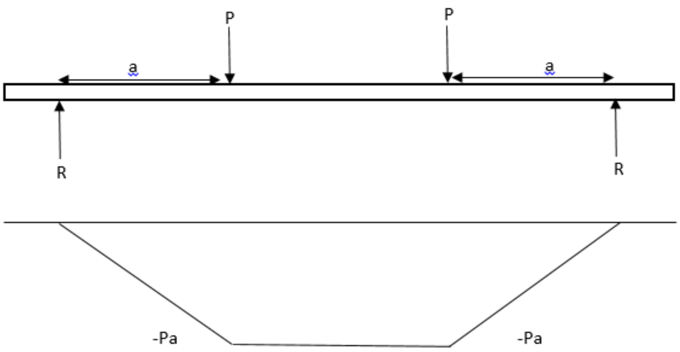


Figure 4.37: Bending moment diagram for 4 points load test

According to the bending moment diagram, the highest moment occurs between 2 loading points. It was observed that, initial flexural cracks were propagated in this zone. When "U" wraps were provided within this zone, it has shown much better results. Finally it can be concluded that it is more effective to use confinements (U wraps) in the high bending moment zone.

Same pattern of CFRP was used for both Control beams and pre-cracked beams. It observed that high flexural capacity shows in pre-cracked beams with compare to Control beams.

Table 4.7: Failure load of pre-cracked beams with control beams

Beam	Failure load (0.3mm crack) (kN)	Average failure load (kN)
CF1	18.64	23.05
CF2	27.45	
C1	14.715	14.715
C2	14.715	

4.7. Deflection

According to the dial gauge readings, there is a clear relationship between the way of arrangement of CFRP sheets and deflections. Results further emphasize that under the same load, strengthened beam has lower deflection compare to un-strengthened beam even though they were heavily-cracked before strengthening. Figure 4.48 shows the Deflection vs. Load graph for control beams and Figure 4.49 shows the Deflection vs. Load graph for strengthened pre-cracked beams.

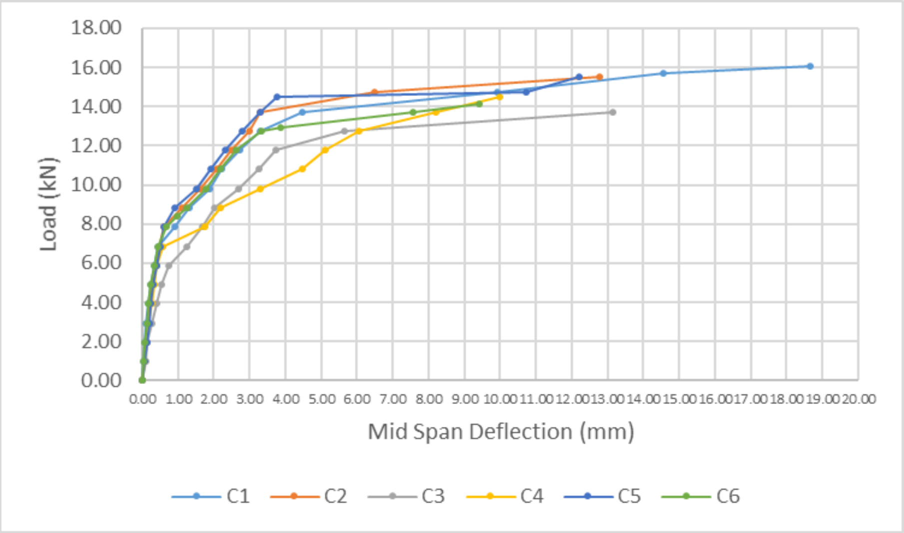


Figure 4.38 : Deflection vs. Load graph for control beams

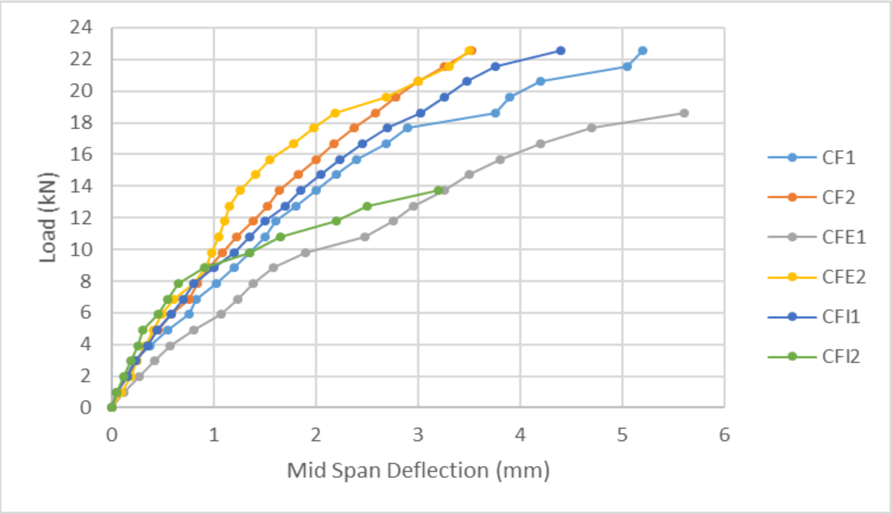


Figure 4.39 : Deflection vs. Load graph for strengthened pre-cracked beams

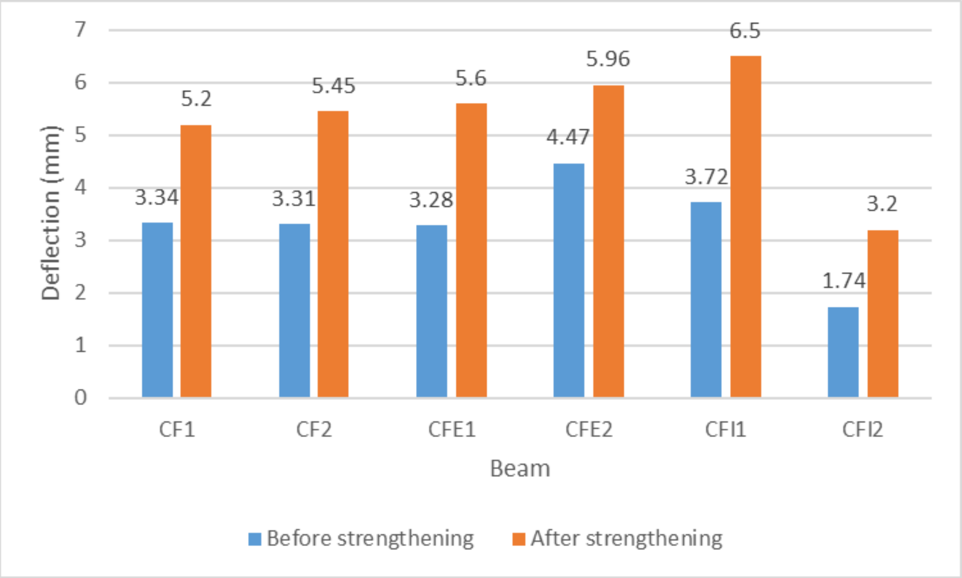


Figure 4.40: Maximum deflection at failure, before and after strengthening

In the experiment, it has also considered the deflection of control beams and pre-cracked beams under the same CFRP arrangement. According to the results, maximum deflections in control beams are higher than that of pre-cracked strengthened beams as shown in Figure 4.40. And also under the same load condition of control beams have shown the lower deflection than cracked beams, when strengthened with CFRP. This will show in Figure 4.41.

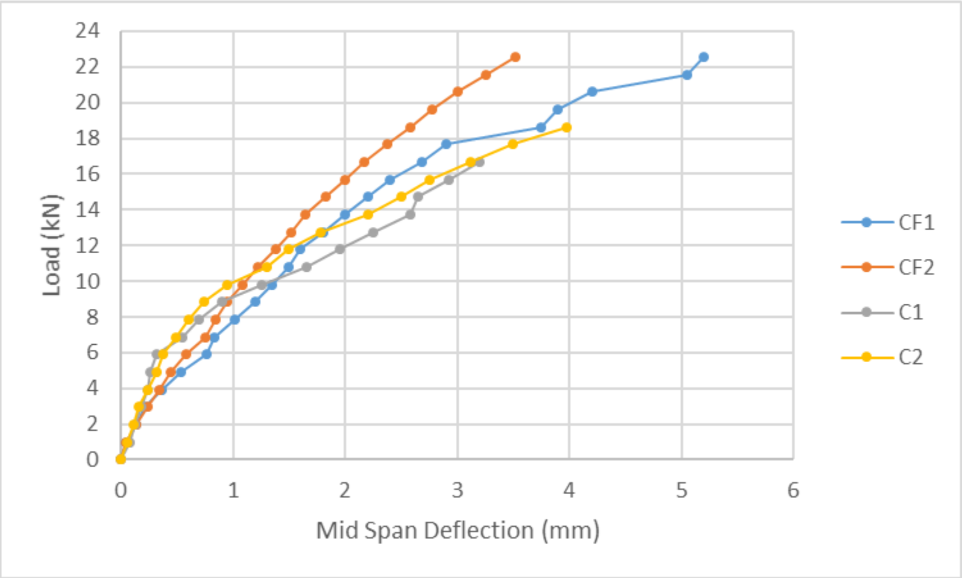


Figure 4.41: Deflection vs. Load graph for pre-cracked and control beams

In the testing, three dial gauges were used to measure the deflection at mid-span and two loading points. According to the bending moment diagram in 4-point bending load test, it shows that the constant bending moment occurs between two loading points. However the deflection varied within this region and the highest deflection has been recorded at the mid-point. Following 4 Figures [Figure 4.42, Figure 4.43, Figure 4.44, Figure 4.45] show the deflection vs. load graphs in CFE1,CFE2, CFI1 and CFI2 beams.

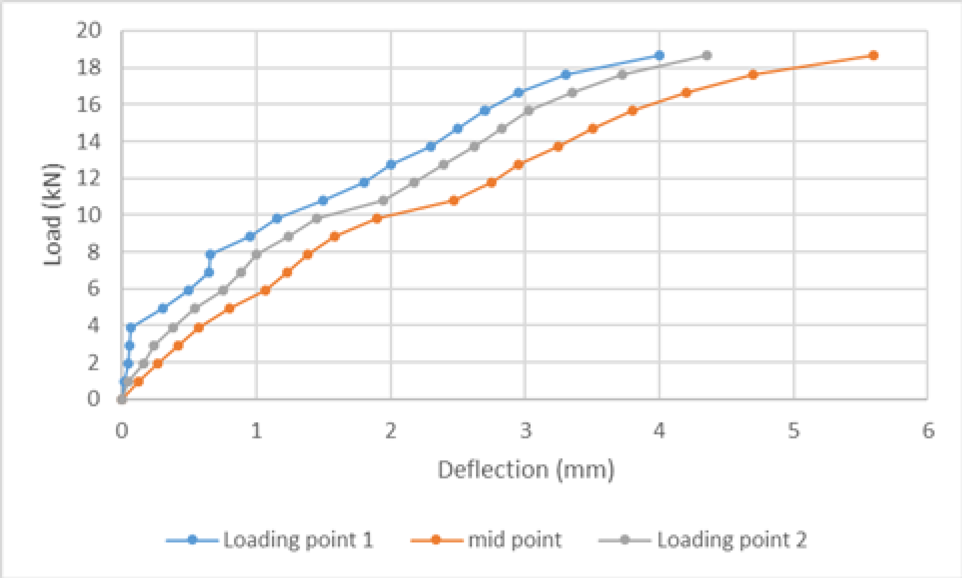


Figure 4.42: Deflection vs. Load graph for CFE1

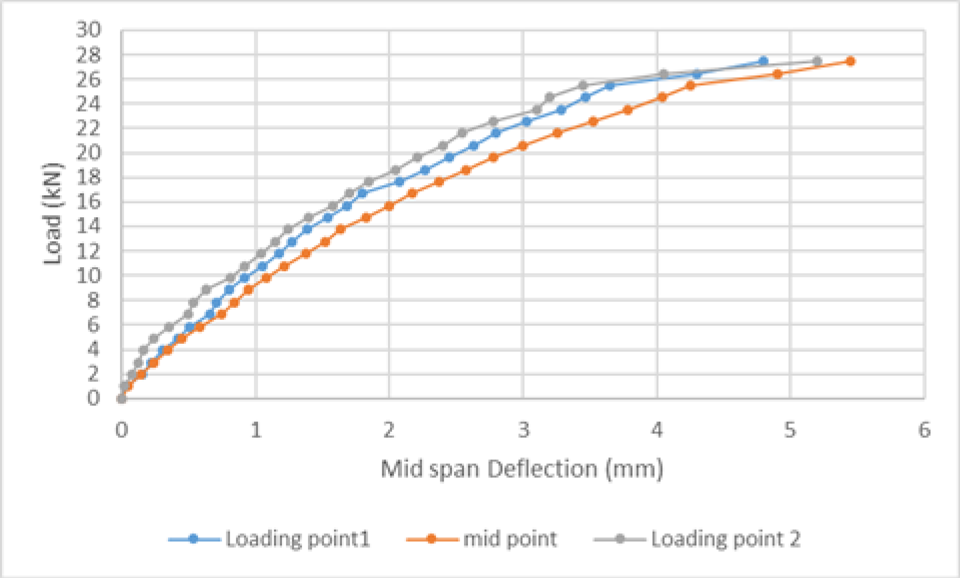


Figure 4.43: Deflection vs. Load graph for CFE2

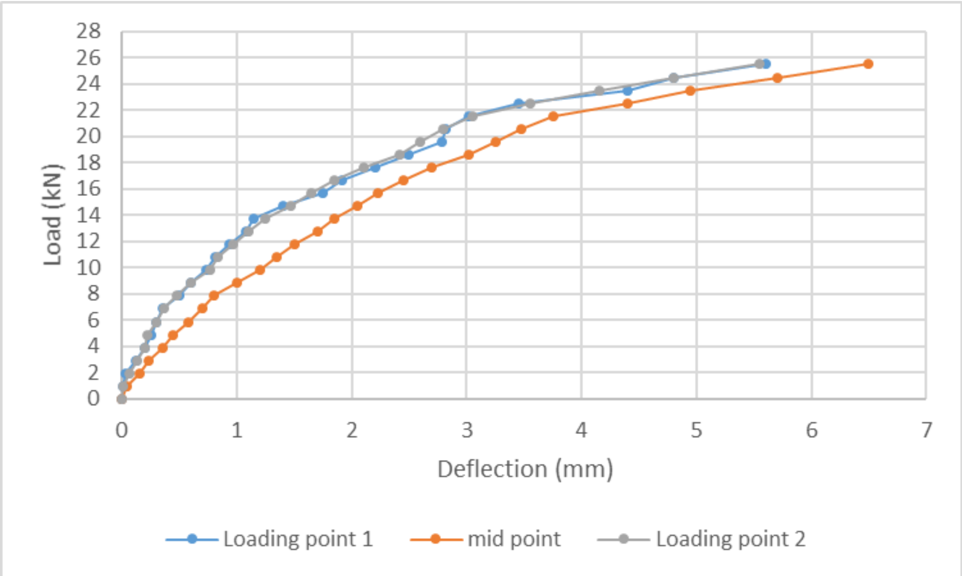


Figure 4.44: Deflection vs. Load graph for CFI1

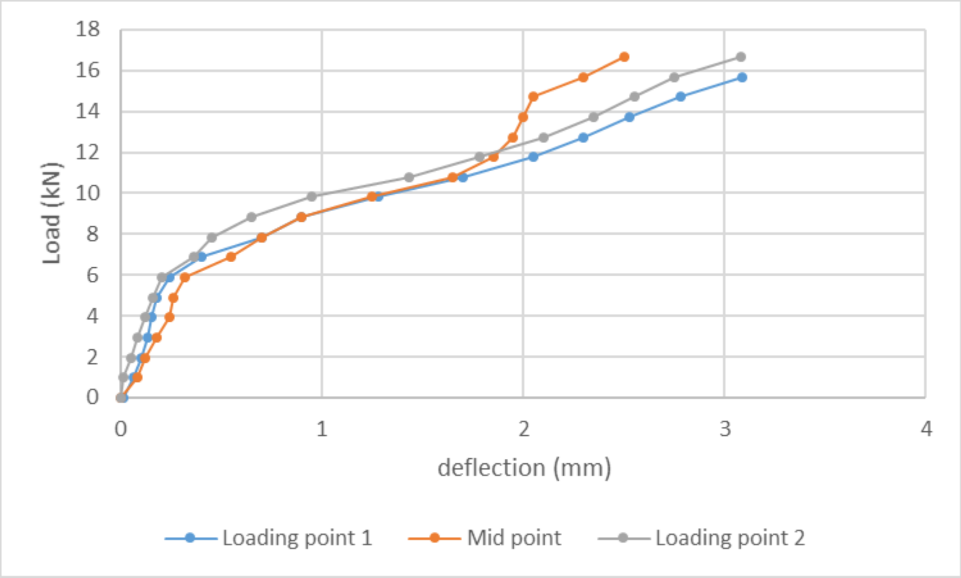


Figure 4.45: Deflection vs. Load graph for CFI2

4.8. Failure Pattern

All the beams were strengthened using externally bonded method. Therefore expected failure mechanism was de-bonding of CFRP sheet. This type occurs with a loss in the composite action between the bonded CFRP and the concrete beam. Failure load was taken as the load at which 0.3mm crack propagate. However in order to see failure mechanism, beams were loaded until considerable de-bonding occur. This load is much higher than that of 0.3mm crack propagate load. Therefore it can be seen higher safety with strengthened beams. Figure 4.46 shows the Deflection of Beams with 0.3mm crack propagate and ultimate failure loads.

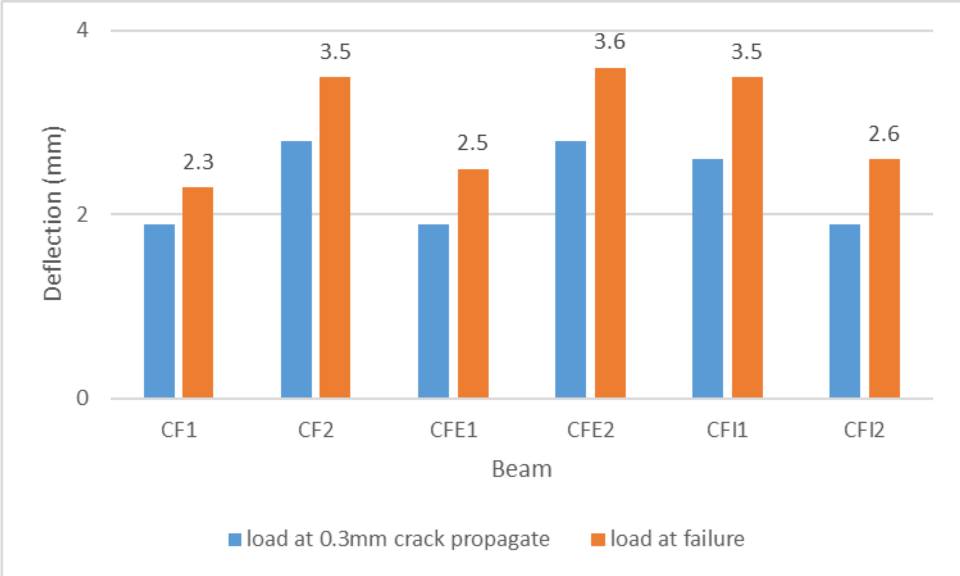


Figure 4.46: Deflection of beams with 0.3mm crack propagate and ultimate failure loads

Another observation that has seen was, new cracks propagated in different places in pre-cracked beams. This might due to the high bondage between concrete and epoxy grout which was used for crack filling purpose. According to the laboratory test data the used epoxy has high compressive strength (85MPa), Tensile strength (50MPa) and Flexural (60MPa) strength. This also helps to increase of ultimate failure load of pre-cracked beams. It was observed that evenly distributed cracks in strengthened beams as shown in Figure 4.47.

Figure 4.47: Crack distribution of CFRP strengthened pre-cracked beams

5. THEORITICAL ANALYSIS

5.1. Introduction

In this theoretical study, the design procedures used to CFRP strengthen non-cracked and cracked concrete beams with and without U-wraps were evaluated in accordance with the ACI 440 Committee report. In this experimental program, strength gain variation with different length of the CFRP sheets and different U wraps arrangements were observed for heavily cracked and non-cracked beams. Ultimate capacity of the beams were determined using the four point bending test method in accordance with ASTM C78.

The beams were categorized on the proportion of span covered with CFRP sheets. Non-cracked beams were strengthened with varying length of CFRP with and without end anchorages. Summary of CFRP arrangement of non -cracked beams specimens are presented in Table 3.6 and the summary of heavily cracked beams are presented in Table 3.7 in this report.

The theoretical calculations were carried out to find out the moment capacities of strengthened of non-cracked and cracked beams with and without U wraps anchored situations. Thereafter, relevant values were used to compare the experimental values. This indicates the deviations between experimental and theoretical predictions based on the existing guidelines.

First, the moment capacities of non-anchored CFRP concrete beam were calculated, based on the explanation in Section 15.3 of ACI 440. Thereafter a comparison was done considering the theoretical moment capacity with Experimental moment capacity.

5.2. ACI 440 design guidelines

In the ACI 440 Committee report[8], stepped design procedure is presented to calculate the moment capacity of non-anchored CFRP-strengthened concrete beams. Further, under the design examples in guideline chapter 15, it highlights that flexural strengthening of NSM FRP bars, flexural strengthening of prestressed concrete beams with FRP laminates etc…. The importance of different arrangements of end U-wraps were discussed under the chapter 11 specially describing the increment of de-bonding capacity of longitudinal CFRP sheet at failure and methods are presented to calculate the area of transverse fixing of CFRP U-wraps in order to minimize the concrete cover separation failure. However, none of the design recommendations describes the corresponding strength gains with respect to the area of the U wraps since very less number of researches were carried out for this fields.

5.3. Procedure to calculate the moment capacities of non-anchored CFRP strengthened beams

A total of 12 beams were assessed in accordance with the ACI guidelines and the computed theoretical moment capacities. A comparison was made between the values for theoretical moment capacities and the experimental moment capacities. The calculation procedure of moment capacity in according to ACI 440 was summarized in Table 5.1. Beam F1,F2 and IN2 were arranged without U wraps in non-crack beams and CF1 and CF2 beams were arranged in heavily cracked stage.

Table 5.1: The summarized calculation procedure of moment capacity in accordance with ACI guidelines

Steps	1	Calculate the FRP system design material properties
	2	Preliminary calculations
	3	Determine the existing state of strain on the soffit
	4	Determine the bond-dependent coefficient of the FRP system
	5	Estimate c, the depth to the neutral axis
	6	Determine the effective level of strain in the CFRP reinforcement
	7	Calculate the strain in the existing reinforcing steel
	8	calculate the stress level in the reinforcing steel and FRP
	9	Calculate the internal force resultants and check equilibrium
	10	Adjust “c” until force equilibrium is satisfied.
	11	Calculate flexural strength components
	12	Calculate design flexural strength of the section

5.4. Procedure to calculate the moment capacity of anchored CFRP concrete beams with end U-wraps

When non-anchored CFRP concrete beams fail in de-bonding or cover separation failure mode, then the arranging U- wraps at both ends of the anchoring the CFRP laminate can effectively delay end de-bonding failure during the loading. This causes to enhance flexural capacity and shear capacity also. The generated vertical stress down ward at the level of concrete cover can be minimized by the anchoring force applied by U -wraps. Design calculation and calculation procedure is shown under Item No 5.5. U-wraps were arranged in beams of “IN1,M1,and M2” for non-cracked beams and CFE1, CFE2, CFI1 and CFI2 in heavily cracked beams. Accordingly, the anchorage forces were calculated for the above beams.

5.5. Theoretical calculation

Flexural Strengthening of an Interior Reinforced Concrete beam with FRP laminates.

Length of beam (l)		=	1900	mm			
Width of beam (b)		=	125	mm			
Span between supports		=	1800	mm			
h	=	200	mm			[height of beam]	
d	=	200-25-6-4 =		165	mm	[Effective depth of beam]	
fc'	=	30	N/mm^2				
fy	=	250	N/mm^2				
ϕ Mn without FRP		=	4.04	kNm			
bars ϕ		=	8	mm			

FRP System for non cracked beams

F1 , F2	=	1600mm long CFRP sheet pasted along tension face
IN1	=	1400mm long CFRP sheet with Uwarp end-anchorage
IN2	=	1400mm long CFRP sheets
M1	=	1200mm long CFRP sheet with 150mm wide 200mm deep Uwarp
M2	=	1200mm long CFRP sheet with Uwarp

FRP System for heavily cracked beams

CF1,CF2	=	1600mm long CFRP sheets pasted along the tension face
CFE1,CFE2	=	1600mm long CFRP sheets pasted along the tension face with U wraps end anchores
CFI1, CFI2	=	1600mm long CFRP sheets pasted along the tension face with U wraps end anchores and U wraps at cracks locations

Thickness per Ply (t_f)	=	0.168	mm
Ultimate tensile strength (f_{fu})	=	4300	N/mm^2
Rupture strain ε $f_{u)}$	=	0.017	mm/mm
Modulus of elastisity (E_f)	=	2E+05	N/mm^2

	1. Calculation of moment capacity of non-anchored CFRP/concrete beams					
	STEP 1	**Calculate the FRP system design material properties**				
	The beam is located in an interior space and a CFRP material will be used.					
Table 9.1	**Environmental reduction f**			=	0.95	
			f_{fu}	=	0.95 x 4300	
				=	4085	N/mm^2
			ε_{fu}	=	0.95 x 0.017 mm/mm	
				=	0.016	mm/mm
	STEP 2	**Preliminary Calculations**				
	β_1	=	1.05 - 0.05 f_{c1}/6.9			
		=	0.832609			
	E_c	=	4700 √(30)			
		=	25742.96	N/mm^2		
	Steel: As		=	2 (π x 8^2 /4)		
			=	100.53	mm^2	
	Properties of the externally bonded FRP R/F					
	A_f	=	(1 pile)(0.168 mm/ply)(125 mm)			
		=	21	mm^2		
	STEP 3	**Determine the existing state of strain on the soffit.**				
	The existing state of strain is calculated assuming the beam is cracked and the only loads acting on the beam at the time of the FRP insulation are dead loads.					
	A cracked section analysis of existing beam.					
	P '	=	A s'	=	2 x π x 8^2	
			bd		2 x 125 x 165	
		=	0.004874			
	P	=	A s	=	2 x π x 8^2	
			bd		2 x 125 x 165	
		=	0.004874			
	d '	=	25 + 6 + 4	=	35	mm
	n	=	E s	=	2E+05	
			E c		4700 √(30)	
		=	7.769114			

k	**=**	**-**	**[(n - 1) p' + pn]**		**±**	**1/2√(2(n-1)p'+2pn) 2 + 4(2(n-1)p'd'/d+2pn)**			
	=	-	[(7.77 - 1) 0.0049+ (0.0049x0.769)]						
±	1/2√(2(7.769-1)0.0049+2x0.0049x7.769) 2 + 4(2(7.769-1)0.0049x35/165+2x0.0049x7.769)								
	=	-	0.0709	±	0.308				
	=	0.237	or	-0.379					
	=	0.237							
I_{cr}	**=**	**1/3 b kb^3**	**+**	**(n-1)p'bd(kd-d')^2 + npbd(kd-d)^2**					
	=	14882187.64			mm^4				
ε_{bi}	=	<u>0.24 kNm x (200- 0.237 x 165)</u>							
		2.515 x 10^6 x 25.7 kN/mm2							
	=	0.0000001		mm					
STEP 4 -		**Design strain of FRP System**							
ε_{fd}	=	0.41	√[30/(2 x 240000 x 1.687)]						
	=	0.007908	<	0.9 x 0.01615					
	=	0.007908	<	0.015					
STEP 5 -		**Estimate e , depth to the neutral axis**							
assumed									
C	=	0.5d							
	=	73	mm						
STEP 6 -		**Estimate e , depth to the neutral axis**							
ε_{fe}	=	0.003[(200-73)/73]-0.0000001							
	=	0.005219	>	ε_{fd}	=	0.00790805		mm	
	=	0.007908	mm						
ε_{c}	=	(0.002501 + 0.00000001)(73/(200-73)							
	=	0.004546	mm						
STEP 7 -		**Calculation the strain in the existing reinforcing steel**							
ε_{s}	=	<u>(Ɛ fe + Ɛ bi)x (d-c)/(d_f-c)</u>							
	=	(0.001437 + 0.00000001)(165-73)/(200-73)							
	=	0.005729	mm						
STEP 8 -		**Calculation the stress level in the reinforcing steel and FRP**							
f s	=	E s . Ɛ s	=	200 x 0.001812					
			=	1.146	<	0.25			

STEP- 9		**Calculaition the internal force resultants and check equilibrium**				
f_{fe}	=	240 x 0.0025		=	1.898	kN/mm2
$\varepsilon_{c'}$	=	$\underline{1.7\ (30)}$	=	0.002		
		25742.96				
β 1	=	$\underline{4\ \varepsilon c' - \varepsilon c}$	=	1.209		
		$6\ \varepsilon c' - 2\ \varepsilon c$				
α 1	=	$\underline{3 \times 0.002 \times 0.001437 - 0.001437^{2}}$				
		3 x0.72x0.002				
	=	0.44644				
STEP-10		**Adjust the "C" until force equilibrium is satisfied.**				
c	=	$\underline{As\ fs + Af\ f_{fe}}$				
		$\alpha\ 1\ fc'\ \ \beta\ 1\ b$				
	=	$\underline{100.53 \times 250 + (210 \times 2160)}$				
		(0.764198x 30x 125 x 0.72)				
	=	32.12		mm		
STEP 11-		**Calculate the Flextural Strength Component[Steel contribution to bending]**				
Mns	**=**	**As.fs(d-B1c/2)**				
	=	**100.5 x 250 x (165- 0.72 x 73/2)**				
	=	3.04	KNm			
		FRP Contribution to bending				
Mnf	=	$A_f.f_{fe}(df\text{-}\beta\ 1.c/2)$				
	=	210 x 600 x (200-0.72x73/2)				
	=	6.212958	kNm			
STEP 12-		**Calculate design Flextural strength of the section**				
ϕ mn	=	ϕ [Mns + ¥f Mnf]				
	=	0.9 [3.04 + 0.85 x6.212958]				
	=	2.736+4.75291287				
	=	**7.49**	**KNm**			

2. Calculation of moment capacity of anchored CFRP-concrete beams with end U-wraps (Ariyachandral et al, 2014)

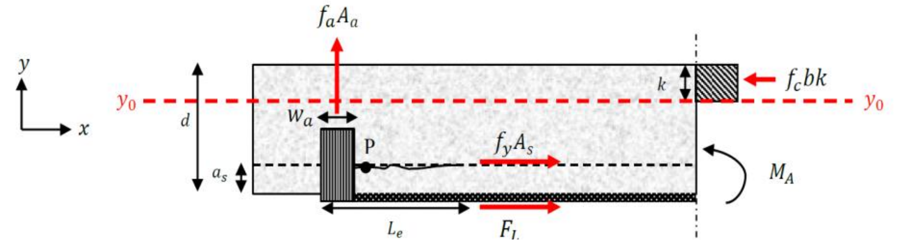


Figure 5.1: Schematic diagram of forces and dimensions of anchored CFRP - concrete beam with end U wraps

Effective Length, Le

$$L_e = \sqrt{\frac{nE_f t_f}{\sqrt{f_c}}}$$ = ?[(1x240000x0.168)/?30]
= 85.8 mm

where ??is number of CFRP layers and ????and ????denote elastic modulus and thickness of CFRP sheet respectively ????

Therefore when cover separation failure occurs can be computed as

$$F_L = \frac{f_t b_f L_e^2}{6a_s}$$ = 150x3x85.8^2 / 6x25
= 22084.92 kN

Depth to the neutral axis. K

$$k = \frac{f_y A_s + F_L}{f_c b}$$ = 250x100.5 + 22084.92 / 30x125
???? = 12.6 mm

Therefore increased can be computed as

$$F_L = \frac{b_f L_e^2}{6F_L a_s}\left(\frac{2f_a A_a}{bL_e} + f_t\right)$$ = 1.03

f_a

Now, according to Hook's law, can be written as follows

$$f_a = \varepsilon_a E_a$$

where εa is the strain developed in the U-wrap and $Ea$is the elastic modulus of the U-wrap

The area of U-wrap, Aa can be represented using the thickness of U-wrap, ta and the width of the U-wrap, wa

$$A_a = t_a w_a$$

The theoretical moment capacity, M_A can be obtained with modified for anchored CFRP-concrete beams with end U-wraps MA

$$\boldsymbol{M_A = f_y A_s \left(d - a_s - \frac{k}{2}\right) + \frac{b_f L_e^2}{6F_L a_s}\left(\frac{2\varepsilon_a E_a t_a w_a}{bL_e} + f_t\right)\left(d - \frac{k}{2}\right)}$$

M_A	=	2.69 + 4.94	
	=	**7.63**	**kNm**

5.6. Summary of theoretical analysis

A very few research studies have focused on enhancing the flexural capacity of CFRP-concrete composite beams using transverse U-wraps placed at both ends of the longitudinal CFRP laminates within the effective bond length. To minimize the previous de-bonding failure modes, the allowable strain levels in CFRP laminates are significantly controlled in compliance with existing guidelines. The provision of anchorages for CFRP laminates provides an excellent method of preventing or delaying early de-bonding failure or de-lamination failure. So abnormally increasing the flexural capacity of CFRP-concrete composite system.

Firstly, the detailed design procedure presented in the ACI 440 Committee report was used to calculate the moment capacity of CFRP-strengthened concrete beams. This process was done for both 12 Nos of non- anchored and anchored beams.

5.7. Comparison of experimental values with theoritical values.

The ACI 440 Committee report describes the importance of transverse end U-wraps in terms of increased de-bonding strain of longitudinal CFRP sheet at failure and guidelines are presented to obtain the area of CFRP U-wraps in order to diminish concrete cover separation failure.

In the present study, an experimental investigation was carried out to determine the flexural performance of CFRP- strengthened concrete beams anchored with different arrangement of CFRP with transverse end and intermediate U-wraps.

From the comparison of theoretical values with experimental values in non-cracked beams, it could be summarized that, the calculated theoretical moment capacities of IN1 type beam occurs 76% from experimental moment capacity value and moment capacity of average of M type beams 123% from the experimental moment capacities. These 2 sets are arranged with U wrap. However the average moment capacity of F type beam shows the 30% when comparing with experimental values and IN2 type beams shows the 10% decrement of moment capacity those without U wraps.[Refer Appendix-4 for comparison table]

When comparing theoretical moment capacities with experimental moment capacity of beams, theoretical capacities shows the higher values.

The comparison of theoretical values with experimental values in cracked beams, it could be summarized that, the calculated theoretical moment capacities of average of CF1 and CF21 type beams occur 1.6 % from experimental moment capacity values. The moment capacity of average of CFE type beams 105% from the experimental moment capacities and also CFI type beams 165% from the experimental moment capacities. These 2 sets are arranged with U wrap. Comparison tables are in Appendex-6

So it could be concluded that, increasing the U anchors at the ends or particular cracked locations, it causes to enhance the flexural capacity of each and every beams.

When non-anchored CFRP-concrete beams fail in cover separation failure, the provision of transverse U-wraps at both ends of the CFRP laminate can effectively delay end-de-bonding failure, resulting in enhanced flexural capacities. Due to the reduced stress levels within the concrete cover zone, anchored CFRP-concrete beams can withstand much higher failure loads, which eventually increase the stress level in longitudinal CFRP laminates. Hence, the contribution of CFRP laminate is increased and can effectively delay the failure mode of premature de-bonding.

6. CONCLUSION AND RECOMMENDATION

6.1. Introduction

The main objective of this project was to investigate the flexural performance of heavily cracked concrete beams, by using different arrangements of CFRP sheets. To achieve the main objective, this research was divided into sub topics.

Initially, conduct the literature survey and collect the previous research data related to flexural performance of CFRP. Thereafter carried out the detail test program and results were noted down. At the end, analysed the test data and issued the recommendation for the Civil Engineering society.

6.2. Conclusion

Under this research program, researcher tried to explain basic introduction about the CFRP materials and their properties and benefits to the construction industry. Report included the different failure modes of beams such as flexural and shear failures etc. Under conclusion, special consideration would be taken out in flexural capacity, deflection amount and failure patterns etc.

Different CFRP arrangements were considered during the experiment program. It was observed that, flexural capacity of different kinds of CFRP arraignment beams could be increased by providing when the CFRP “U” wraps at high bending moment zone. This will increase the flexural capacity of structural elements efficiency and effectively.

Further investigation should be carried out to identify the most economical way of using “U” wraps in the critical zone to enhance the flexural performance of heavily cracked beams. The installation of CFRP “U” wrap as end anchors increase the strength further by 83%.

Mode of failure changed from delamination to rupture by the installation of CFRP End anchorages.

The beams with 77% of span covered with CFRP has shown better results in the both categories of IN1 and IN2 beams with end anchors and without end anchors.

Therefore, 77% of span covered with CFRP sheet applied along the tension face of beams caused to optimize the strengthening of long rectangular concrete beams.

The most of the times in practical situations, retrofitting structures are cracked elements. In this research program, it has addressed with long beams to simulate actual conditions and showed that CFRP materials are effective in strengthening both newly constructed and damaged elements.

Pre-cracked beams can undergo similar or higher load than non-cracked beams when strengthened with same CFRP arrangement. This depends on the amount of damage and parameters of the epoxy used to fill the crack.

When U wraps were provided in the zone of high bending moment, then the higher failure load was recorded. This was due to the confinement provided to the bottom CFRP sheets by U wraps in the critical zone. Therefor high bending moment zone should be confined to have better strength.

When beams are strengthened with CFRP material, it can undergo higher deflection, since CFRP will increase the stiffness of heavily cracked beams.

Load was recorded when 0.3mm cracks initiated and when beam failed. Failure load is much higher than 0.3mm cracks occur load. Therefore, it has high safety margin when structures are strengthened with CFRP systems.

6.3. Recommendations

According to the results obtained, it is concluded that CFRP material can be used to strengthen cracked and non-cracked beams in flexure effectively. Different CFRP arrangements were considered during the experiment program and identified that flexural capacity can be increased by providing CFRP U wraps at high bending moment zone. It is suggested that CFRP is effective in increasing flexural performance of heavily cracked beams and the most appropriate method of strengthening is the use of “U” wraps in the critical zone (high bending moment zone) along with the CFRP in tension face. This will increase efficiency and effectiveness of using CFRP material in construction industry.

Installation of end anchors has affected the mode of failure and failure load of beams significantly. There has been a 48% [110%-62%] increase in strength of beams by providing the “U” wrap end anchorage. And also the beams without end anchors failed with CFRP sheet delamination. When the end anchors were applied, the delamination was prevented and rupture in the middle of the beam. When the CFRP was allowed to reach its ultimate strength and caused to enhance the beam flexural capacity.

The beams with 77% of span covered with CFRP has shown better performance in the both catogories of beams with end anchors and beams without end anchors. Therefore the author recommends to optimize the strengthening of long rectangular concrete beams with only 77% of span covered with CFRP sheet applied along the tension face of beams.

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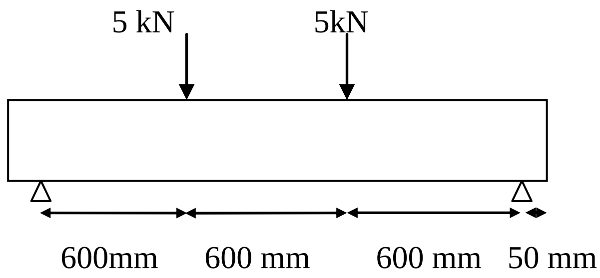
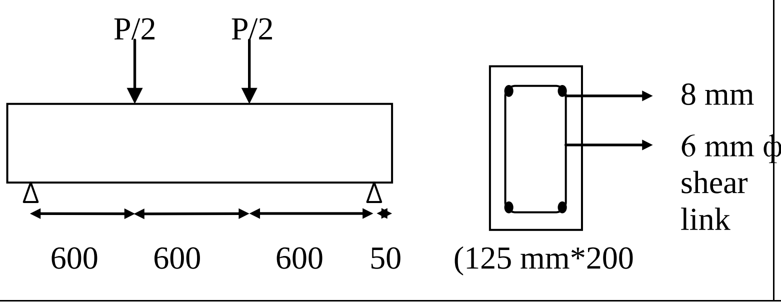
APPENDIX

APPENDIX - 1

Design calculation for the flexural failure.

Reference	Calculations	Output
BS 8110: Part 1 1985 3.4.4.4	Uniformly distributed self weight = 1*.125*.2*2400 kg = 0.589 kN/m  M= bending moment due to self weight + bending moment due to point loads $= (0.589*1.8^2)/8 + 10*(1.8/6)$ = 3.24 kNm Assume main reinforcement bar diameter= 8 mm Assume shear link bar diameter= 6 mm Effective depth = d = 200-25-6-4 = 165 mm $K=M/bd^2f_{cu}$ $=3.24*10^6/125*165^2*30$ $=0.0317$	Effective depth= 165 mm

	K'=0.156, K<0.156; compression reinforcement not required $z=d(0.5+\sqrt{(0.25-0.0317/0.9)})$ $=165(0.5+\sqrt{(0.25-0.0317/0.9)})$ $= 159$ mm < 0.95 d Since $f_{cu} = 30$ N/mm^2 and $f_y = 250$ N/mm^2 $As=M/(0.87 f_y z)$ $=3.24*10^6/(0.87*250*159)$ $=94$ mm^2 Main Reinforcement = 2R8 [2 mild steel bars] $As =2*\pi*r^2$ mm^2 $=100.5$ mm^2	Main R/F = 2R8

Reference	Calculations	Output
BS 8110: Part- 1, 1985, Cl : 3.4.5.2	Take failure load under shear = 60 kN V= 60/2 = 30 kN	Failure load under shear = 60 kN
	Diameter of shear links = 6 mm Effective depth = 165 mm $v = V/b_v d$ $= 30*10^3/125*165$ $= \underline{1.4\ N/mm^2}$ <lesser of ($0.8\sqrt{f_{cu}}$, or 5 N/mm^2)	
Table 3.9	$v_c = 0.79(100\ As/(b_v d))^{1/3}(400/d)^{1/4}/\Upsilon_m$ $= 0.79(100*100.5/(125*165))^{1/3}(400/165)^{1/4}/1.25$ $= \underline{0.62\ N/mm^2}$	
Table 3.8	$v_c+0.4 = 1.02\ N/mm^2 < v$ $A_{sv} >= b_v s_v(v - v_c)/0.87f_{yv}$ $A_{sv} = 2*\pi*3^2\ mm^2$ $= \underline{57\ mm^2}$ $s_v <= 127$ mm	
Cl : 3.4.5.5	Minimum spacing of links = 0.75d = 124 mm Spacing of links = 100 mm  	Spacing of links = 100 mm

APPENDIX - 2

Expected Theoretical Calculations

Reference	Calculations	Output
	<u>Beam Parameters</u> Span of the beam = 1900 mm Overall depth = 200 mm Breadth = 125 mm Grade of concrete = 30 N/mm^2 Top Reinforcement = 8mm ø mild steel Bottom Reinforcement= 8mm ø mild steel Stirrups = 6mm ø mild steel Grade of mild steel = 250 N/mm^2 Cover = 25 mm Effective depth = 200 – 25 – 6 – 4 = 165 mm	
BS 8110: Part 1: 1985	<u>Flexural Capacity</u> Flexural capacity was calculated as follows As = Tension Reinforcement f_y = Yield stress of Reinforcement x = Depth to the neutral axis b = Breadth of the beam f_{cu} = Compressive strength of concrete = The average 28 days concrete compressive strength from Appendix- 2 = 35.3 N/mm^2	

Compressive force in concrete = 0.9 ×0.67 ×X xb× fcu

Tensile force in steel = $0.87As\ f_y$

Considering beam to be singly reinforcement,

Tensile force of the beam = Compressive strength of the beam

$$0.87As \times fy = 0.9 \times 0.67 \times x\ \mathbf{x}b \times fcu$$

$$0.87 \times \pi \times 4^2 \times 2 \times 250 = 0.9 \times 0.67 \times 125 \times 35.3 \times x$$

$$x = 8.21\ mm$$

Lever arm (Z) = d – (0.45x)

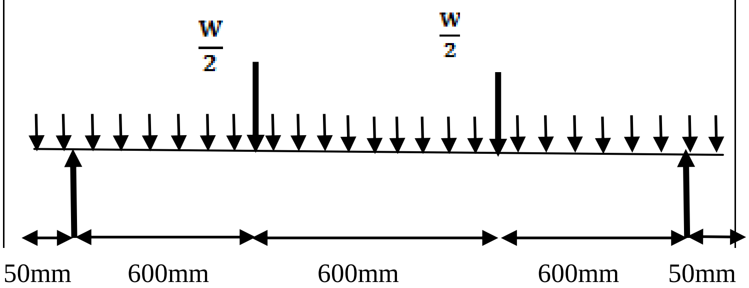
= 165 – (0.45 × 8.21)

= 161.3 mm

Flexural capacity = F_t x Z

= $\pi \times 4^2 \times 2 \times 250 \times 161.3$

= <u>4.04 kNm</u>



<u>Expected flexural capacity of beam</u>

1. Bending moment due to W/2 point loads

Maximum moment will be occurred between point loads,

Moment, M $= \frac{W}{2}\ (x - 0.05) - \frac{W}{2}\ (x - 0.65)$

<table>
<tr>
<td>BS8110: Part 1: 1985
Table 3.9</td>
<td>
2. Bending moment due to beam self-weight
[0.050 m < x < 1.850 m]
Moment, M = $\frac{wl}{2}(x-0.05)$ - $\frac{wx^2}{2}$
For maximum moment,
$\frac{dM}{dx}=0$
Therefore x = l/2 = 0.95 m

By principal of super position,
Moment for total load = $\frac{wl}{2}(x-0.05)$ - $\frac{wx^2}{2}$ + $\frac{W}{2}(x-0.05)$ - $\frac{W}{2}(x-0.65)$
Therefore maximum moment
M_{max} = 0.3W + 0.40375w

According to the previous calculation, Flexural capacity of the beam = 4.02 kNm
At failure, 0.3W + 0.40375w
= 4.04
0.3W + 0.40375 × 0.125 × 0.2 × 2.4 × 9.81 =4.04

W = <u>12.7 kN</u>
Hence expected failure load under flexure =12.7 kN

<u>Expected shear capacity of beam</u>

$v_c = 0.79 \times \left(\frac{100As}{bv \times d}\right)^{1/3} \times \left(\frac{400}{d}\right)^{1/4} \times \left(\frac{fcu}{25}\right)^{1/3} \times \frac{1}{rm}$
v_c = Design shear stress of concrete
As = Req. Tension Reinforcement
f_{cu} = Compressive strength of concrete
b_v = breadth of section
γ_m = safety factor for materials (taken as 1.15)
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	$A_s = \pi \times 16 \times 2$ $= 100.53\ mm^2$ $b_v = 125\ mm$ $v_c = 0.659\ N/mm^2$ Shear taken by stirrups (v_{fv}) $= \frac{Asv \times fy}{bv \times sv}$ s_v = space between shear link reinforcement A_{sv} = Total cross section of links at neutral axis $A_s = \pi \times 9 \times 2$ $= 56.5\ mm^2$ $S_v = 120mm$ $v_{fv} = 0.942\ N/mm^2$ Expected total shear capacity $= 0.659 + 0.942$ $= 1.601\ N/mm^2$ Maximum shear force is at the support, Shear force due to applied load $= \frac{W}{2}$ Shear force due to self-weight $= \frac{wl}{2} - 0.05 \times w$ Therefore, $S_{max} = \frac{W}{2} + 0.125 \times 0.2 \times 2.4 \times 9.81 \times (1.9/2 - 0.05)$ $= \frac{W}{2} + 0.52974$ $S_{max} =$ Design shear stress of the concrete beam $= 1.601 \text{ x } 125 \text{ x } 200 \text{ x } 10^{-3}$ $= 40.05\ kN$ $\frac{W}{2} + 0.52974 = 40.05$ $W = \underline{79kN}$ Hence expected failure load under shear =79 kN	

APPENDIX – 3

Concrete compressive strength test results

	Date of cast	**Date of test**	**Compressive strength (N/mm^2)**
C1	28/11/2015	28 days after cast	35.48
C2		28 days after cast	37.54
C3		28 days after cast	34.35
C4		28 days after cast	34.3
C5	30/11/2015	28 days after cast	36.67
C6		28 days after cast	34.45
C7	1/12/2015	28 days after cast	37.82
C8		28 days after cast	29.48
C9	3/12/2015	28 days after cast	36.04
C10		28 days after cast	36.76
		Average compressive strength	**35.29**
		Standard deviation	**2.414**

APPENDIX - 4

Comparison between experimental moment and theoretical moment

Non Cracked Beam

	Experimental Moments		Theoritical Moments					
Beam Notation	Ultimate Failure Load(kN)	Max Moment (0.3W+.40375w) /(kNm)	Flexural component from steel /Mns (kNm)	CFRP component for bending /Mnf (kNm)	U wrap component for bending / FL(d-k/2) / (kNm)	Theoritical Values / (kNm)	% increment of Moment capacity w.r.t Experimenta moments	Average Increment %
F1	17.00	5.34	2.736	4.753	0.000	7.489	40.30	30%
F2	20.00	6.24	2.736	4.753	0.000	7.489	20.06	
IN1	35.00	10.74	2.736	4.753	11.400	18.889	75.91	76
IN2	27.00	8.34	2.736	4.753	0.000	7.489	-10.18	-10
M1	27.00	8.34	2.736	4.753	11.400	18.889	126.55	123%
M2	28.00	8.64	2.736	4.753	11.400	18.889	118.68	

Cracked Beam

	Experimental Moments		Theoretical Moments					
Beam Notation	Ultimate Failure Load(kN)	Max Moment (0.3W+.40375w) /(kNm)	Flexural component from steel /Mns (kNm)	CFRP component for bending /Mnf (kNm)	U wrap component for bending / FL(d-k/2) / (kNm)	Theoritical Values / (kNm)	% increment of Moment capacity w.r.t Experimenta moments	Average Increment %
CF1	22.75	7.063	3.141	4.753	0.000	7.894	11.77	2%
CF2	28.00	8.638	3.141	4.753	0.000	7.894	-8.61	
CFE1	21.50	6.688	3.141	4.753	7.630	15.524	132.13	106%
CFE2	28.00	8.638	3.141	4.753	7.630	15.524	79.72	
CFI1	28.00	8.638	3.141	4.753	7.630	15.524	79.72	165%
CFI2	14.00	4.438	3.141	4.753	7.630	15.524	249.82	

W-Failure load under flexture/(kN)
w-Beam self weight/(kN/m)