INVESTIGATION OF CEMENT BASED ADHESIVES TO REPLACE EPOXY BOND IN CFRP/CONCRETE COMPOSITE

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Degree of Master of Engineering in Structural Engineering Design

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University of Moratuwa
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Dissertation submitted in partial fulfillment of the requirements for the Degree Master of Engineering in Structural Engineering Design

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DECLARATION

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Dr.(Mrs). J.C.P.H.Gamage

Supervisor

ABSTRACT

Fibre Reinforced Polymer (FRP) application is a very effective way to repair and strengthen structures that has become structurally weak over their life span. The strengthening and rehabilitation of existing structures are major issues worldwide. In most situations, strengthening is required when there is an increase in the applied load, human error in the initial construction, a legal requirement to comply with updated versions of existing codes, or as a result of the loss of strength due to deterioration over time. Fibre-Reinforced Polymer (FRP) strengthening systems are enjoying a great deal of popularity as a result of the unique properties of FRPs - namely, their light weight, fatigue resistance non-corrosive characteristics and ease of application. The repair and strengthening technique with epoxy-bonded advanced composites has been applied to a large number of bridges around the world. At elevated temperatures, normally beyond the glass transition temperatures of epoxy adhesive, the mechanical properties of the polymer matrix deteriorate rapidly. It will be very beneficial if they can be replaced by cement grout bonding agents such as modified concrete, in order to produce fire-resistant strengthening systems.

This report includes the investigation of the flexural behavior of FRP-strengthened reinforced concrete beams using epoxy adhesive and cement grout adhesive. A total of ten RC beams were cast. All of them were having the same cross section of 100 mm X 150 mm and the span of 500 mm. Two beams were tested as control beams and another two beams were strengthened with CFRP using epoxy adhesive. Remaining six beams were strengthened with CFRP using cement grout with different bonding arrangements.

Finally, experimental results were compared with theoretical results and previous research data. The results showed that a considerable strength gain can be achieved when beams were strengthened using cement grout as a bonding agent. It was revealed that primer has the ability to increase the ultimate load carrying capacity as well. Furthermore, use of anchoring system to strengthen beams is an effective technique.

KEYWORDS: Carbon Fiber Reinforced Polymers (CFRP), Reinforced Concrete Beams, Failure Load, Anchoring system, Epoxy, Cement grout

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LIST OF ABBREVIATIONS

AFRP	:	Aramid Fibre Reinforced Polymer
ACI	:	American Concrete Institute
A_s	:	Provided area of main tension reinforcement.
A_{sv}	:	Provided area of shear reinforcement
b	:	Width of the section
CFRP	:	Carbon Fibre Reinforced Polymer
d	:	Depth of the section
E	:	Young's Modulus
F_c	:	Force in concrete compression zone.
f_{cu}	:	Characteristic strength of concrete.
FRP	:	Fibre Reinforced Polymer
F_t	:	Force in main tension Reinforcement.
f_y	:	Characteristic tensile strength of main Reinforcement
f_{yv}	:	Characteristic tensile strength of shear Reinforcement
GFRP	:	Glass Fibre Reinforced Polymer
GS	:	Galvanized Steel
IC bonding	:	Intermediate crack bonding
IESL	:	Institution of Engineers of Sri Lanka
l	:	Length of specimen
M	:	Expected maximum moment.
NSM	:	Near Surface Mounted.
RC	:	Reinforced concrete
RF	:	Reinforcement
s	:	Spacing of shear links along the members
UOM	:	University of Moratuwa
w	:	Uniformly distributed self –weight of concrete element.
W	:	Applied point load.
x	:	Depth to the neutral axis
Z	:	lever arm

LIST OF SYMBOLS

A_f	:	$nt_f w_f$, area of FRP external reinforcement (mm^2)
A_g	:	gross area of section (mm^2)
A_s	:	area of nonprestressed steel reinforcement (mm^2)
A_{st}	:	total area of longitudinal reinforcement (mm^2)
b	:	width of rectangular cross section (mm)
c	:	distance from extreme compression fiber to the neutral axis (mm)
CE	:	environmental-reduction factor
D	:	distance from extreme compression fiber to the neutral axis (mm)
d_f	:	depth of FRP shear reinforcement (mm)
E_c	:	modulus of elasticity of concrete (MPa)
E_f	:	tensile modulus of elasticity of FRP (MPa)
E_s	:	modulus of elasticity of steel (MPa)
f_c	:	compressive stress in concrete (MPa)
f'_c	:	specified compressive strength of concrete (MPa)
f_f	:	stress level in the FRP reinforcement(MPa)
$f_{f,s}$	:	stress level in the FRP caused by a moment within the elastic range of the member (MPa)
f_{fe}	:	effective stress in the FRP; stress level attained at section failure (MPa)
f^*_{fu}	:	ultimate tensile strength of the FRP material as reported by the manufacturer (MPa)
f_{fu}	:	design ultimate tensile strength of FRP (MPa)
F_y	:	specified yield strength of nonprestressed steel reinforcement (MPa)
h	:	overall thickness of a member (mm)
M_n	:	nominal moment strength (N-mm)

p^*_{fu} : ultimate tensile strength per unit width of the FRP Reinforcement (N/mm)

T_g : glass-transition temperature, °F (°C)

b1 : ratio of the depth of the equivalent rectangular stress block to the depth of the neutral axis

$€_b$: strain level in the concrete substrate developed by a given bending moment (tension in positive)

$€_{bi}$: strain level in the concrete substrate at the time of the FRP installation (tension is positive)

$€_c$: stain level in the concrete

$€_{cu}$: maximum usable compressive strain of concrete

$€_f$: strain level in the FRP reinforcement

$€_{fe}$: effective strain level in FRP reinforcement; strain level attained at section failure

$€_{fu}$: design rupture strain of FRP reinforcement

$€^*_{fu}$: ultimate rupture strain of the FRP reinforcement

$€_s$ strain level in the nonprestessed steel reinforcement

$€_{sy}$ strain corresponding to the yield strength of nonprestressed steel reinforcement

φ : strength reduction factor

k_a : efficiency factor for FRP reinforcement (based on the section geometry)

k_m : bond-dependent coefficient for flexure

y_f : additional FRP strength-reduction factor

LIST OF APPENDICES

CHAPTER 01: INTRODUCTION

1.1 General

There are many reasons that structures do not perform up to desired standard or do not survive its full design life. Increase of load carrying capacity is needed due to changes in utilization or increased live loads, rapid (early) deterioration due to environmental factors and shortcomings in execution or design.

The two main reasons for repairing and upgrading of deteriorated concrete structures are economic and environmental. It is incentive enough to find effective and economical techniques to maintain the aging infrastructure and buildings despite rebuilding from the scratch. If old structures can be preserved, natural resources will not be used limitless to rebuild the structures which will cause lesser negative impact on environment.

A number of strengthening techniques have been used in the past. In the case of concrete structures, these include section enlargement including additional reinforcement, prestressing either internally or externally and epoxy bonded steel plates. These methods bring additional strengthening more feasible and economical way. Considering all strengthening techniques the fibre reinforcement polymer is a better strengthening material for current use in the world.

Fibre reinforced polymer (FRP) is a composite material made of fibres that have high strength to weight ratio and adhesive that binds the fibres together to fabricate the structural material. Commonly used fibre types are Aramid, Carbon and Glass fibres. The commonly used adhesive is epoxy. FRP was originally developed for aircraft, ships and high-speed trains, because of the beneficial advantages like low weight, high strength and stiffness, and resistance to environmental factors such as corrosion and flexibility. The main disadvantage is the low glass transition temperature, result in low fire resistance.

The use of FRP materials for strengthening concrete structures was developed in Europe and Japan in the 1980s, (Anders, 2003) and since then several thousand projects have utilized FRP systems worldwide. It was discovered that the FRP strengthening technique is suitable for structural repair and retrofitting of existing structures. The FRP system that is non-metallic material is considered to be a beneficial technique due to its higher durability. The most practical solution for repairing and retrofitting structures is to resist higher design loads and other durability problems which can be addressed using FRP. The FRP composites is one of the latest development in the civil engineering industry, there are many other traditional techniques available like externally bonded steel plates, steel or concrete jackets, and external post tensioning.

There are many varying methods exist to strengthening of existing concrete structures. One such commonly used technique utilizes surface epoxy bonded FRPs. The FRP strengthening method is very efficient and has achieved worldwide attention. However, there are some drawbacks with the use of epoxies, e.g. working environment, compatibility and permeability. Substituting the epoxy adherent with a cement based bonding agent will render a strengthening system with improved working environment and better compatibility to the base concrete structure and also it will improve fire performance of the system.

There are some experimental research studies related to strengthening of RC beams with FRP composites with using cement-based material as a bonding agent have been carried out by Siavash and Riadh (2006) and Hashemi and Al-Mahaidi (2011) . All of these studies showed results that led to an increase of load carrying capacity and thereby, using cement based material, is an effective way to increase the load carrying capacity and carrying higher fire resistance compared to epoxy, because the epoxies cannot withstand temperatures above 50°C since their T_g is in the range of 40^0C-60^0C. Use of cement-based bonding material is one way of creating environmentally friendly strengthening system.

1.2 Objectives of the research study

The overall aim of this research is to investigate the RC beams flexurally strength with externally bonded CFRP and the suitability on use of cement-based materials as bonding agent for strengthening of existing strength degraded RC beams.

The objectives of this research are:

1. To understand the background of CFRP strengthening technique

2. To study the structural behavior of RC beam elements strengthened with CFRP sheet
3. To conduct a test program to determine the flexural performance of CFRP strengthened concrete beams using epoxy adhesive
4. To explore the suitability of cement bond adhesives to replace the epoxy bond and conduct test program to identify the performance of cement bond in flexure
5. To analysis experimental results and theoretical results and propose recommendations

1.3 Methodology

The methodology adapted in the research is outlined in this section.

1. Collect experimental data related to flexural strength of concrete beams which are strengthened using CFRP bond with epoxy and cement based adhesives.
2. Experimental study to determine the efficiency of the external strengthening systems for reinforced concrete beams using CFRP bond with epoxy and cement grout adhesive. The dimensions of the specimens will be 100 mm x150 mm and consist of span of 500 mm (The detailed flow chart is given in the Figure 1.1).

3. Analysis of experimental data.
4. Theoretical calculation and predicting the results.
5. Comparison of theoretical and experimental results.
6. Comparison of results with past studies.

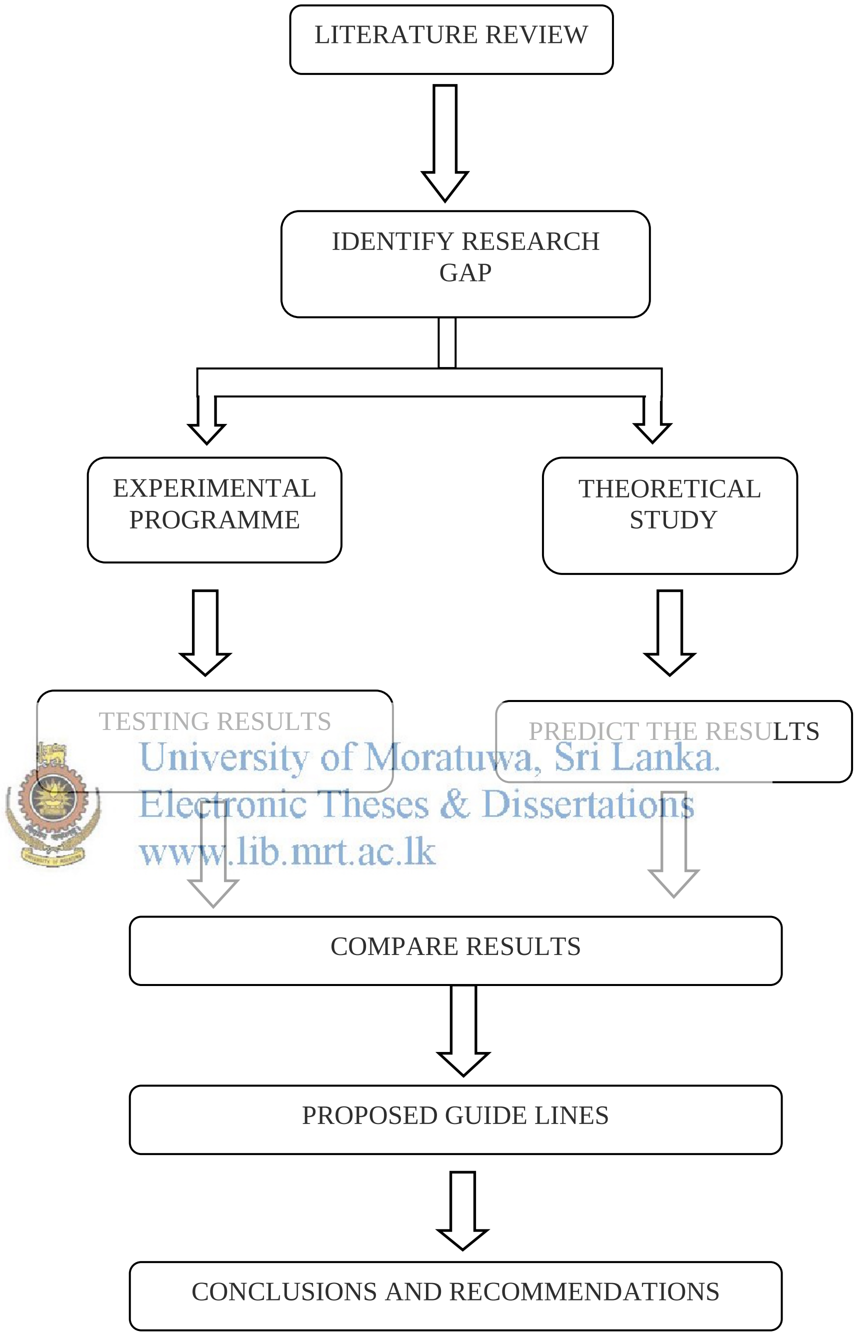


Figure 1.1: Methodology Flow Chart

1.4 Arrangement of the thesis report

Chapter 01 includes the background, the main objectives and the scope of the research work.

Chapter 02 covers literature review of previous research work done by various researchers. In addition to that this Chapter reviews the use of advanced composite materials to strengthen concrete structures through a literature survey and identify properties of different FRP materials, FRP strengthening techniques, shear strengthening, flexural strengthening, failure modes, reasons for failures, use of epoxy and its properties, various types of cement based adhesives. Furthermore, the Chapter reviews the influences of temperature on concrete beams strengthened with CFRP. The content is placed within the framework of the knowledge and the aim of this thesis.

Chapter 03 covers the experimental work carried out during the research study.

Chapter 04 includes theoretical calculation and comparison with experimental result.

Chapter 05 gives simplified specification for cement grout adhesive to strengthening of concrete structures, conclusions drawn from the research work and the recommendations for further research work which should be done to strengthen and support the conclusions made from this study.

1.5 Conclusions

The existing structures can be improved mainly in two ways which are structure repairing and structure strengthening. Repairing means recovering the deficiencies occurred in structures due to long term environmental exposure, fire etc. Strengthening is upgrading structures to withstand against the effect of increased loading, poor material quality, design and construction faults.

Strengthening using FRP composite is a very popular method for structures retrofitting. Since FRP has better material properties such as light weight, high tensile strength, high elastic modulus, corrosion resistance etc. Therefore, FRP strengthening has become a widely used strengthening technique in civil engineering industry.

The most common adhesive used in the industry to create the bond between CFRP and concrete is epoxy resin. The system shows very good short term performance with this resin. However, epoxy adhesive used in civil engineering constructions is very sensitive (Gamage et al, 2006) to temperature. Epoxy contains drawbacks such as relatively high cost, long curing time, low fire resistance and toxicity. Therefore, finding alternative bonding agent to replace the epoxy adhesive is essential.
Therefore, aim of the current study is to investigate cement based bonding agent to replace epoxy resins.

CHAPTER 02: LITERATURE REVIEW

2.1 General information on FRP

FRP is available in many forms like sheets, bars, and mesh. It can be used as a structural reinforcement for concrete elements. The various forms of FRP materials are shown in Figure 2.1(Solrun et al, 2012). The FRP fibres can be placed in multiple directions as shown in Figure 2.2. The Epoxy resin or cement-based bonding material can be used with different types of fibres: AFRP (Aramid Fibre Reinforced Polymers), CFRP (Carbon Fibre Reinforced Polymer), GFRP (Glass Fibre Reinforced Polymer) or BFRP (Basalt Fibre Reinforced Polymer). Therefore, FRP reinforcement forms a group of products where the characteristics are not the same and many reinforcement types can be used in different situations.

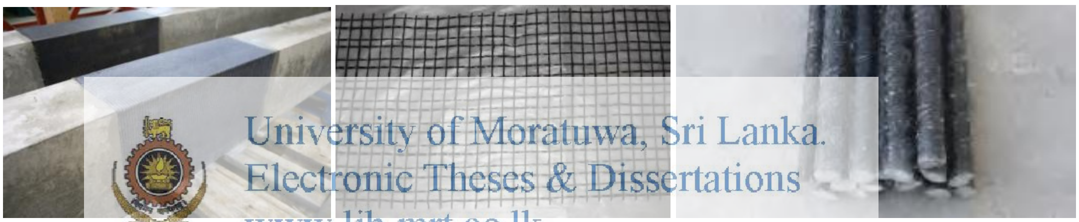


Figure 2.1: various forms of FRP materials.

Source : Solrun et al, 2012

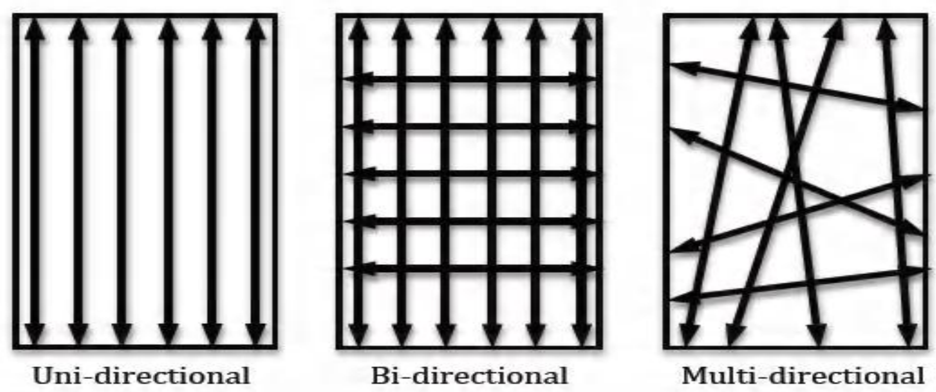


Figure 2.2: Fiber directions in composite materials

Source : Sveinsdottir et al, 2012

Typical stress-strain behavior for FRPs along with reinforcing steel is shown in Figure 2.3. FRP does not experience any yielding during tension; it has a linear elastic behavior from the origin (starting point) up to failure where the ultimate stress is reached. Steel normally has higher modulus of elasticity than FRP element, but FRPs are characterized by high tensile strength in the range of 2400 Mpa to 5400 MPa (ACI Committee 440 (2002)).

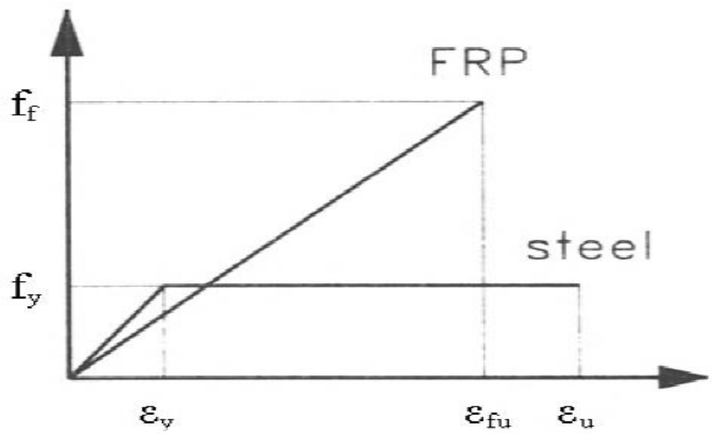


Figure 2.3: Stress-strain behavior of FRP compared to steel

Source : Sveinsdottir et al, 2012

2.2 Material Characteristics

Characteristics of the material used in this study are described in this section.

2.2.1 Fiber Reinforced Polymer

The FRP system is a composite material made of fibers embedded in a polymeric resin matrix where its overall property is governed by the characteristics of fiber, its orientation, the polymer resin, volumetric mix fractions of the constituents as well as the presence of local defects. Attainable strength gains, in addition to the above mentioned, rely on the type of adhesive used and its method of application. The materials used for fibers are generally Glass [E-Glass or S-Glass], Aramid or Carbon [low modulus or high modulus] and are commonly known as GRFP, AFRP and CFRP. In general the FRP systems are non-corrosive, low density, high strength, easy to install and have better long term environmental durability as well as fatigue resistance. As per parameters listed in the Fib bulletin (2001), the following properties can be tabulated as shown in Table 2.1. A general comparison between steel and FRP can also be made, and is tabulated as shown in Table 2.2

Table 2.1: Mechanical Properties of Fibers (Fib-Bulletin et al, 2001)

Criterion	Carbon	Aramid	E-Glass
Tensile Strength (Nmm^{-2})	Very Good (2806)	Very Good (1280)	Very Good (1080)
Compressive Strength (Nmm^{-2})	Very Good (1875)	Inadequate (335)	Good (620)
Modulus of Elasticity (GPa)	Very Good (177)	Good (87)	Adequate (39)
Long term Behavior	Very Good	Good	Adequate
Fatigue Behavior	Excellent	Good	Adequate
Bulk Density (kgm^{-3})	Good (1600)	Excellent (1280)	Adequate (2100)
Alkaline Resistance	Good	Good	Inadequate
Price	Adequate	Adequate	Very Good

Table 2.2: Comparison of Fiber materials with Steel (Fib-Bulletin et al, 2001)

Material	Density (kg/m^3)	Tensile Strength (N/mm^2)	Modulus of Elasticity (N/mm^2)
Carbon	1800-2200	1500-4000	150-420
Aramid	1400-1500	2000-3600	130-160
Glass	2200-2500	1500-3500	70-90
Steel	7850	415	190-210

Typical advantages and disadvantages of FRP systems are summarized as follows.

Advantages of FRP material

- High ultimate strength(2-3 times greater than steel)
- Lower density than steel
- Strength to weight ratio is significantly lesser than steel

- Handling and installation is significantly easier than steel
- Requires little maintenance
- Excellent durability
- Excellent corrosion resistance
- Good flexibility

Disadvantages of FRP material

- High cost
- Long – term durability is not available
- Risk of fire or accidental damage(unless the FRPS are protected)
- The transverse strength is low

2.2.2 Adhesives

The adhesive component does an important role in the strengthening system. By means of an adhesive, two materials are needed; e.g FRP and concrete to connect to each other so that full composite action can be developed. When using the FRP strengthening or repair technique, the adhesive is used to glue the two materials combined together. Also it provides a load path between these two materials. Adhesives are based on the composition to meet certain requirements for the industry, high elastic modulus, high strength, bond quality, workability and durability. Therefore adhesives should exhibit low creep, thermal stability and resistance to moisture and alkaline nature. One of the most widely used and accepted structural adhesives is epoxy based adhessive. Cement-based materials are a good alternative as an adhesive because there are some drawbacks with the use of epoxy adhesives in certain areas, such as where fire resistance is important. In addition to that, the better matrix material should have following properties:

- Sufficient mechanical properties for load transfer.
- Correct consistency, good penetration of the fabrics, and good bond characteristics for embedding of the fabrics.
- Thermal and chemical compatibility of the fibres and the substrate, thermal and fire resistance.

- Good workability (applicability to large vertical surfaces, open-ended time period for application).
- The demand of environmental acceptability.
- Good adhesion to a sand blasted concrete surface.
- A limited shrinkage to prevent tensions between the surface of the strengthened structure and the composite laminate.

2.2.2.1 Epoxy

There are many adhesives in the structural industry like epoxy, vinylester, polyester but most common used and favorable matrix is epoxy. The selection of the matrices type to be used in structural application is governed by various factors including environment and the speed of fabrication. The epoxy resins have a wide range of mechanical and physical properties. Their main advantages for the structural industry are to offer high surface activity, high cohesion and adhesion, low shrinkage and low creep. The major disadvantage of epoxy resin is relatively high cost, long curing time as well as the unavoidable fire resistance. Table 2.3 shows the mechanical properties of different adhesives.

Table 2.3: Mechanical properties of adhesives (Anders et al, 2003)

Materials	**Tensile Strength (Mpa)**	**Tensile modules (Gpa)**	**Failure strain (%)**	**Density (g/cm^{3})**
Polyester	40-90	2-4.5	01-04	1.10-1.46
Vinyylester	70	3	5	1.2
Epoxy	30-100	02-05	03-06	1.11-1.40

2.2.2.2 Cement – based adhesives

The bonding agents used in a cement based strengthening system are often fine grade (1 mm maximum grain size) mortars. To enhance the properties, e.g. workability, flow ability, mechanical properties etc, different mixtures and additives are used. Different additives can be polymers, super plasticizers and reinforcing fibers. The addition of different type of polymers enhances the properties of ordinary Portland cement. There are also a number of chemical admixtures, such as water reducing agents, ashes, aluminosilicate, super plasticizers, etc., that further improve the quality of mortar. All of the above mentioned improvements can enhance strength, shorten setting time, decrease autogenous shrinkage, control the alkali aggregate reaction and improve the durability (Li Z. & Ding Z., 2003). Expectations the cement-based materials have excellent properties, not only the strength bonding but also to achieve good workability. A cement-based bonding system is logically used to bond the fiber composite to the structure and the FRP sheet is there to resist the stresses in the strengthen structure. By replacing the epoxy resin with cement-based material the mentioned drawbacks can be overcome. One of the features of cement-based bonding material is the importance of their chosen constituents. In the following sections a description on the most common constituents for binders can be found. A cement-based strengthening system depends on the use of the binder.

Minerals

The minerals that are used in cement-based materials are the same as the minerals that are used in concrete. Examples of these materials can be ordinary Portland cement, fly ash, silica fume etc. The most common technique is to mix these minerals and add some fine grade aggregates.

Additives

To enhance the properties of cement-based materials' additives can be added in the form of super plasticizers (Sveinsdottir et al, 2012). Adding super plasticizer to the mixture the workability, durability and the strength of the cement-based binder can be improved.

Micro fibre reinforcement

Concrete and cement-based materials are considered to be brittle materials as they have low tensile strength and failure strain. In order to obtain high performance cement-based materials for application such as strengthening, micro fiber reinforcement can effectively improve the mechanical behavior. Different types of fibers can be used are steel, basalt, polypropylene, synthetic micro fibers and natural fibers.

2.3 FRP/Concrete composite

The basis theory behind FRP strengthening is provision of additional tensile reinforcement to structural element. The composite action of existed element and externally added reinforcement will change the element's behavior. There are three main materials which contribute to structural behavior of composite; concrete, steel and FRP. In a strengthened system, both fibre and steel carry tension and concrete carry compression. Stress distribution over the cross section of a flexural strengthen concrete element is shown in Figure 2.4.

In a un- strengthened element, steel reinforcement is the only material which carries tension. Once it is strengthened, fibres too contribute to tension capacity. Since FRP has very high tensile capacity, neutral axis goes down while increasing the contribution of concrete in compression, in order to comply with increased tensile capacity. Therefore, flexural capacity of the composite system is increased significantly. If the composite action is well established in the system the element can be loaded up the limit of shear failure, FRP rupture or concrete crushing. Furthermore, achievable strength gains depend highly on the adhesive substance used. The adhesive helps to bind the composite to the concrete substrate and act as shear load path between the two surfaces.

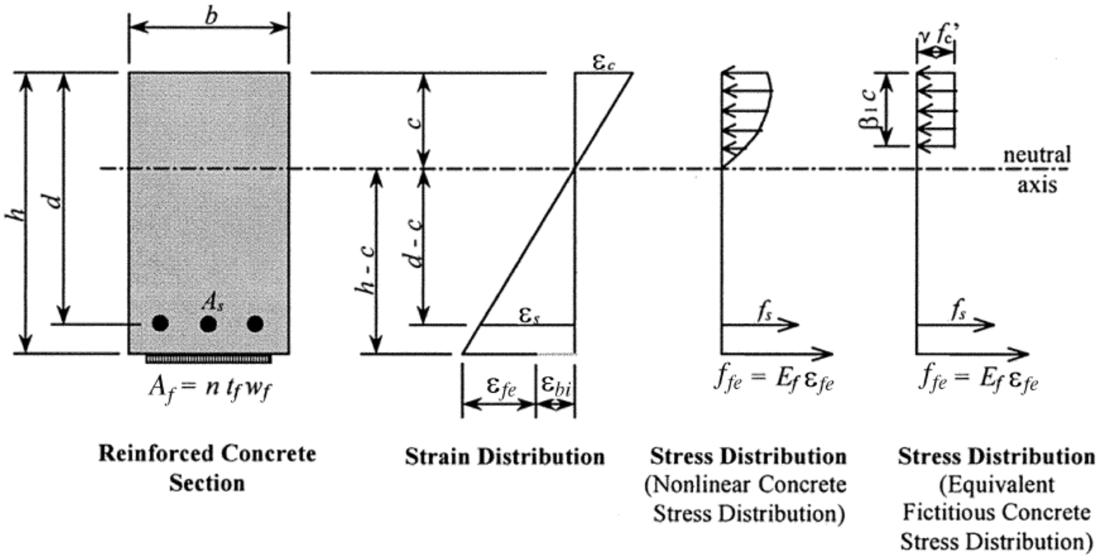


Figure 2.4: Internal strain and stress distribution for a rectangular section under flexure at ultimate stage

Source : ACI 440.2R-02.

2.4 FRP strengthening techniques

Strengthening techniques are concern the application of FRP as structural reinforcement bonded to an existing concrete substrate structure. The techniques can be used under different conditions and different locations of the structural member. ACI 440.2R-08 recognizes four forms of FRP strengthening techniques as follows.

1. Wet layup systems,
2. Prepreg systems,
3. Precured systems
4. Near-surface-mounted systems.

2.4.1 Wet layup systems

In case of the wet layup system dry sheets of fibres are impregnated with resin on-site and then cured in place. The sheets are either saturated with resin and then applied shortly after to the concrete surface, or are applied first and then saturated with resin. After sand blasting and cleaning of the concrete surface, a layer of adhesive is applied, followed by a layer of fabric. This can be continued until sufficient fibres have been applied for the required capacity. One of the main advantages of this technique is the fact that the FRP can be applied in different shapes. However, due to the application method, generally a smaller percentage of fibres per unit area can be achieved, resulting in a thicker layer of FRP compared to the laminates. Moreover, the quality of the application is more sensitive to labour skills, as the composite is made in situ.

Figure 2.5: Wet layup system

Source : Litvinov et al, 2010

2.4.2 Prepreg systems

With prepreg systems the FRP sheets are saturated with resin off-site, and then cured in place. Sometimes additional resin is required to adhere the sheet to the concrete surface, and often additional heating is required for curing.

2.4.3 Pre-cured systems

Precured systems are impregnated with resin and cured offsite, and then typically applied to the concrete with adhesive.

2.4.4 Near-surface-mounted systems

The Near Surface Mounted (NSM) technique has been used in recent years for the strengthening of reinforced concrete beams. It involves the insertion of strips or rods of carbon fibers reinforced polymers (CFRP) in grooves made previously in the external surfaces (top or bottom) of the concrete beams or slabs, filled with epoxy adhesive for fixation as shown in Figure 2.6, compared to traditional use of FRP sheets adhering to the external surfaces, this method causes better transferring of the loads to the surrounding concrete by enhancing the bonding stresses, as well as better protecting the mechanical properties of the FRP strips against any environmental defects and fire. The application of NSM FRP reinforcement does not require surface preparation work as in the case of externally bonded FRP reinforcement.

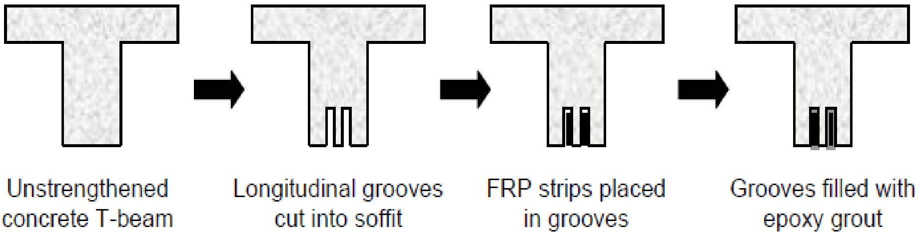


Figure 2.6 : NSM system

Source : Bisby et al, 2004

2.5 Failure modes in flexural strengthened RC beams

Reinforced concrete elements strengthened with FRP systems have several additional failure modes compared with reinforced concrete elements that are not strengthened with FRP systems. Since carbon fibers have very high strength in tension relative to the concrete and to the adhesive that binds them to the concrete, the common failure mode of bonded FRP sheets is de-bonding. De-bonding failures are typically sudden and brittle, and occur before the full strength of the FRP sheet has been reached. Through extensive laboratory testing, researchers have defined several distinct failure modes for flexural strengthened beams. Smith and Teng, (2001) listed six failure modes, which are shown in Figure 2.7.

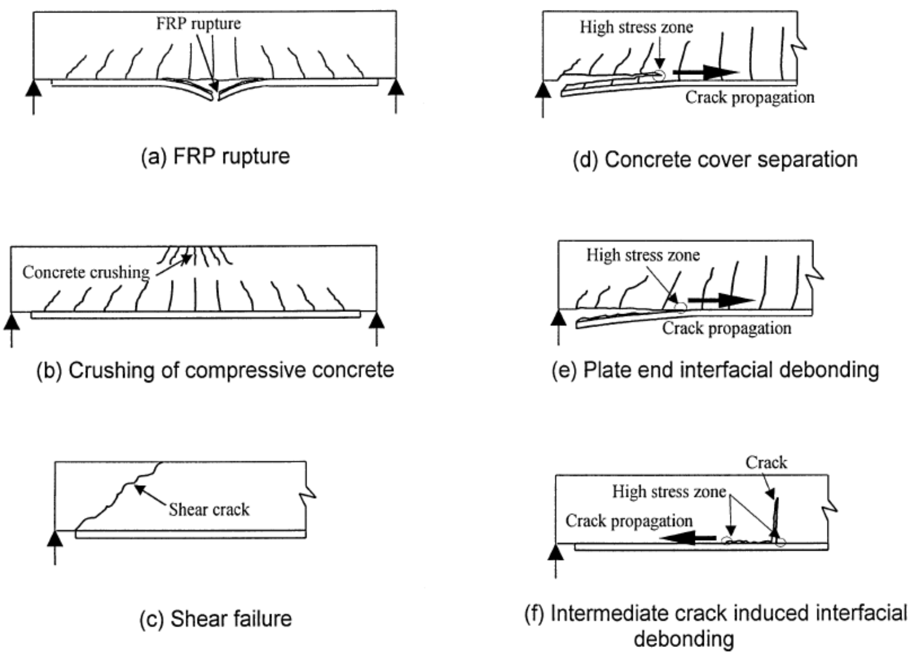


Figure 2.7 : Failure modes of FRP-strengthened RC beams

Source : S.T. Smith and J.G.Teng, 2001

If the ends of the plate are properly anchored, then failure occurs when the ultimate flexural capacity of the beam is reached, by either tensile rupture of the FRP plate (Figure. 2.7a) or crushing of concrete (Figure. 2.7b). For either FRP rupture or concrete crushing, the steel reinforcement generally has already yielded at failure. Due to the brittleness of FRP, when failure occurs by FRP rupture, the concrete has generally not reached failure. This differs from that of conventional RC beams, where due to the ductility of steel reinforcement, the compressive concrete generally has reached failure at the ultimate limit state of the beam. In addition, the brittleness of FRP means that flexural failure of FRP plated RC beams, by either FRP rupture or crushing of concrete, displays limited ductility. As a result, failure by concrete crushing is permissible in FRP plated beams, which contrasts with conventional RC beam design where steel yielding should be ensured to precede concrete crushing. The plated RC beam can also fail brittle in shear if the flexural capacity of the plated beam exceeds the shear capacity of the RC beam along (Figure. 2.7c). In such cases, shear strengthening of the RC beam is required to ensure that the desired strength enhancement can be achieved and that flexural failure precedes shear failure. The latter is necessary because the flexural failure mode of an FRP plated RC beam is still a more ductile mode than a shear failure mode

The three failure modes shown in Figure 2.7d,2.7e,2.7f are generally termed de-bonding failures, and involve the beam failing before the strength of the FRP sheet is reached. De-bonding failures are the most common type of failures, and are particularly troublesome because they are generally non ductile failures, that occur with little warning. De-bonding failures can be further grouped into two categories; plate end Interfacial de-bonding (PE de-bonding) and intermediate crack induced de-bonding (IC de-bonding). In IC de-bonding, de-bonding initiates at the location of an intermediate flexural or flexural-shear crack and then propagates away from the crack towards one of the ends of the beam. In plate end de-bonding, failure initiates near the end of the beam, often at the termination of the FRP sheet, and then propagates towards the middle of the beam. The failure can either travel up to the tensile reinforcement and then along the reinforcement, so that the concrete cover de-bonds, which is termed concrete cover separation, or it can propagate near the FRP-concrete interface, which is termed plate end interfacial de-bonding. What is common among the de-bonding failures is that they initiate at stress concentrations; at the termination of the FRP in plate end failures, and at cracks in interfacial de-bonding failures. Once failure initiates, it usually progresses very quickly, with little or any increase in load capacity of the member Smith and Teng et al (2001). Considering the failure modes of beams flexural strengthened with FRP, de-bonding and plate end de-bonding are the most common failure modes.

2.6 Temperature and humidity considerations

Failure of the FRP /concrete composites is generally initiated by de-bonding of the FRP reinforcement from the concrete surface. It is expected that temperature will affect this de-bonding behavior due to the significant difference in the coefficient of thermal expansion between concrete and FRP and due to the change of the material properties at elevated temperatures, especially of the adhesive. De-bonding of the FRP is generally governed by the failure of concrete adjacent to the concrete-adhesive interface, leaving a small layer of concrete remaining attached to the adhesive after de-bonding.

Not only the surface temperature but also relative humidity of air before and during installation also can affect the FRP strengthening procedure. Primers, saturating resins, and adhesives generally should not be applied to cold or frozen surfaces. If the temperature of concrete surface is below a minimum level as proposed by the manufacturer, an auxiliary heat source must be used to increase the surface and air temperature. If this is not used, improper saturation of the fibers and curing of the resin constituent materials may occur. The heating device should not contaminate uncured FRP system. It is a general practice to apply resins and adhesives to dry and clean concrete surface. If FRP systems are applied to concrete surfaces that are subjected to moisture vapor transmission, it will result in surface bubbles and lead to failure of the bond between the FRP systems. Auxiliary heating is allowed in case of low temperature (below 50^0C) to increase the surface and air temperature. However the method of heating should be approved. Similarly, when temperature exceeds 200^0C, care shall be taken with batch life of epoxies and special precautions may be necessary. Presence of moisture may slow down adhesion of primer and/or resin. FRP should not be applied when rain or condensation is expected. No application shall take place unless the concrete temperature and air temperature are at least 3 degrees higher than the dew-point temperature.

2.7 Thermal properties of materials

This section describes about thermal properties of CFRP, epoxy, concrete, CFRP/epoxy and cement complex.

2.7.1 CFRP

There are number of researches carried out to find the relationship of failure of CRFP- strengthened RC at elevated temperature. According to Klamer et al. (2008), there is no need to take the effect of temperature into account in the design of a FRP-strengthened structure as long as the temperature stays below 50°C. Further he found that temperature of strengthen structures should be avoided above 50°C due to possible failure of bond and reduction of bond strength.

CFRP specimen with normal modulus ((E_f = 170,000 N/mm²) resulted in decreasing failure load from temperature -100°C to 40 °C. Failure due to CFRP happens at temperature -100°C only and failure at other temperatures due to concrete or adhesive material (Di Tommaso et al, 2001).

2.7.2 Epoxy

Especially epoxy adhesives have good mechanical properties and a high resistance against environmental degradation (Morgan et al. 2005) and are therefore preferred in the construction industry, despite the relatively high costs. It can however be seen that the glass transition temperature of commercially available epoxy adhesives is relatively low compared to the glass transition temperature of matrix materials (Table 2-4).

Table 2.4: Mechanical properties of polymer matrix materials (Morgan et al. 2005)

	Young's modulus [N/mm2]	**Tensile strength [N/mm2]**	**Ultimate tensile strain [%]**	**Glass transition temperature [°C]**
Polyester	3200 – 3500	60 – 85	2 – 5	100 – 140
Vinylester	3300	70 – 80	5 – 6	210 – 340
Epoxy	2000 – 4000	80 – 150	1 – 8	50 – 260

Figure 2.8 depicts that failure load decreases with increasing temperature and it is more significant at a certain temperature, as expected.

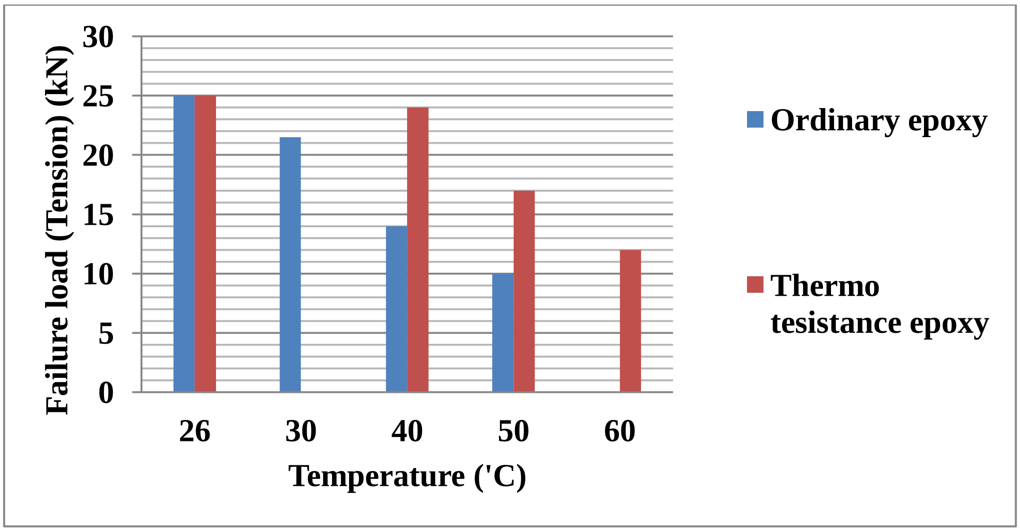


Figure 2.8 : Temperature Vs failure load

Source : Wu et al. 2005

2.7.3 Concrete

Tadeu and Branco [2000] have investigated the effect of temperature on concrete specimens strengthened with externally bonded steel strips, which have, almost the same coefficient of thermal expansion as concrete. In the experiments, a reduction of the failure load with an increase in temperature was found (Figure 2.9), which was expected to be caused by the changed mechanical properties of the adhesive at elevated temperatures.

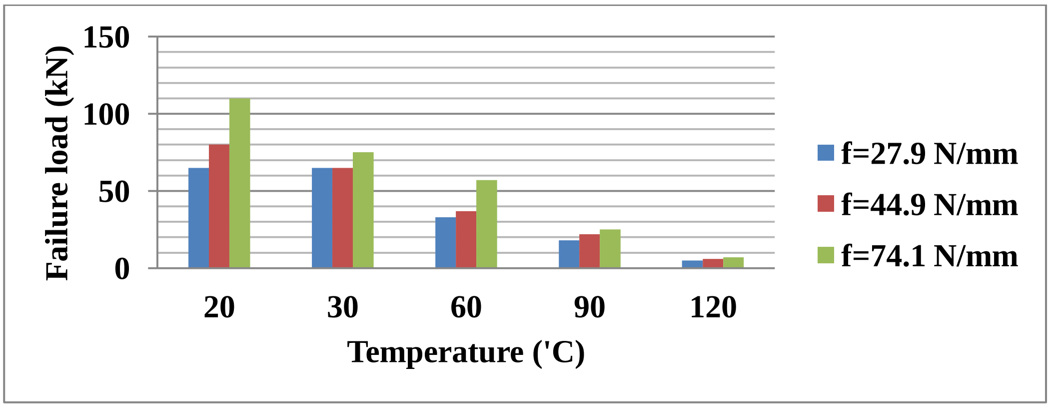


Figure 2.9 : Temperature-failure load relation of double lap shear tests

Source : Tadeu and Branco 2000

2.7.4 CFRP/epoxy/cement complex

The coefficient of thermal expansion of CFRP, epoxy and concrete are different. That causes weaken the strength of the bond at elevated temperature.

2.8 Comparison between epoxy and cement based adhesive

Especially epoxy adhesives have good mechanical properties and a high resistance against environmental degradation (Morgan 2005) and are therefore preferred in the construction industry, despite the relatively high costs. One of the other major advantages of epoxy is the low shrinkage during cure. The major disadvantages of epoxy resins are relatively high cost, long curing time and loosing strength at elevated temperatures. Also, use of epoxy is hazardous for manual worker and it has some environmental problems as well. In addition to that application of epoxy is not possible on a wet or moist surface nor is it possible in temperatures below -10 °C (Anders Wiberg, 2003).

When it comes to cement based materials it can be complex. Cement based adhesive are not costlier than epoxy and it does not show poor behavior above glass transition temperature. To enhance properties of cement based adhesives different mixtures and additives are used. The properties include workability, flowability, and mechanical behaviors (Li Z., & Ding Z., 2003).

Refer Appendix A for details of flexural capacity enhancement of beams with using epoxy adhesive. And Appendix B contains comparison of flexural capacity enhancement of beams using epoxy and cement based adhesives.

2.9 Research done on CFRP and cement based adhesive bond on concrete

Due to poor bond performance of CFRP and concrete using epoxy as adhesive at elevated temperature (Gamage et al, 2006), there has been researches to find alternative bonding material. Cement based adhesive materials are famous alternative bonding material to replace epoxy adhesive. (Hashemi & Al-Mahaidi (2008), Blanksvard & Taljsten (2006)).

As per Sveinsdottir (2012), an increase in flexural load carrying capacity up to 8% using cement-base bonding material and 11% with an epoxy adhesive. In the same experiment, it was found that failure of beams with cement-based bonding material was caused by rupture of longitudinal fibres. For beams strength with epoxy adhesive de-bonding of the fibres was the failure.

Traditionally a FRP strengthening has been considered to reach its full strength after 7 days. However in Wiberg's studies, he found that, if the mortar is based on ultra-fine cement the speed of the strength growth seems to be higher. Also, polymer modified mortar used in this work has proved to harden relatively fast. The composite can be estimated to reach 75-85 % of its final strength after 72 hours depending on the thickness of the composite.

Mineral based composites (MBC) used by Blanksvard & Taljsten (2006) contains basically three materials – a cementitious binder, a CFRP grid and concrete surface primer. In that research they found that to achieve a good bond between the base concrete and the mortar, the surface of the base concrete needs to be roughened. Further, the epoxy based systems have a slightly better performance compared to the cement based system, due to the superior bond between epoxy and the fibers. However, the MBC system for flexural strengthening needed a smaller carbon fiber area, in the tensile direction, to generate a higher bearing capacity compared to the epoxy bonded sheets. Increasing the bond between mortar and fibres by the use of sand, bonded to the surface of the CFRP grid, will cause high stress concentration and premature fibre rupture for flexural strengthening.

Tests conducted by Hashemi & Al-Mahaidi in year 2012 have shown that excellent reinforcement action can be achieved using cement-based adhesives. Refer Appendix C depicts mix ratios of concrete and mortar used in the experiment.

They found following points from the experiment.

- The experimental results showed that the conventionally retrofitted beam demonstrated poor behaviour and the failure occurred at a temperature of 462 °C, since the CFRP delaminated and the beam performed as an ordinary reinforced concrete beam.

- The retrofitted RC beam with cement-based adhesive showed a considerable improvement in flexural performance at high temperature compared to the specimens with epoxy. The failure temperature was 844 °C. This value is close to the failure temperature of reinforced concrete beams. It can be inferred that the beam behaved similarly to an ordinary RC beam, and the desired high temperature resistance was achieved.
- The anchorage did not contribute to an increase in the strength of the section, since the failure occurred at mid-span.
- The performances of beams were predicted with high accuracy by the model employed for FE analysis. Furthermore, the crack pattern and strain distribution correlated closely with the experimental results.

According to Adhikarinayake, S.R et al, 2013 the cement grout can be effectively used as bonding material to replace epoxy adhesive bond in CFRP/concrete composite. As given in the Table 2.5, increased flexural performance was observed with 3 mm thick cement grout bond than 6mm thick bonded using epoxy. It is also important to note that flexural performance was reduced when thickness of cement grout layer increased to 6 mm.

Table 2:5: Test results (Adhikarinayake, S.R et al, 2013)

Type	**Failure load (kN)**	**Deflection at the middle (mm)**	**Increment load carrying capacity**	**Failure mechanism**
Control beam (without CFRP)	11.7	2.7	-	Flexural failure
	12.95	0.65	-	
CFRP bonded using epoxy	15.89	0.86	26	De bonding failure due to initiations of mid span flexural crack
CFRP bonded 3mm cement grout	16.68	2.05	35	CFRP material failure
	17.17	1.84	39	
CFRP bonded 6mm cement grout	15.7	0.85	27	CFRP material failure
	16.67	2.0	35	

A very interesting pioneering work has been presented by Wiberg, 2003. Large-scale tests of ordinary concrete beams strengthened with a cementitious fibre composite were reported. The composite used was made of polymer-modified mortar and a unidirectional sheet of continuous dry carbon fibres were applied by hand. Both flexural and shear strengthening were tested. From the test, it was found that considerable strengthening effects can be achieved. In comparison to epoxy bonded carbon fibre sheets, the amount of carbon fibre needed to reach the same strengthening effect for the cementitious strengthening system was more than double. The reason for this is mainly due to problems with wetting the carbon fibre. This is also emphasized by Badanoiu & Holmgren, 2003, where it was found that the load capacity of the cementitious carbon fibre composite is influenced by the amount of fibres in the tow. If the cementitious matrix can penetrate into the interior of the carbon fibre tow, a higher number of filaments will be active during loading, and this will lead to an increase in load carrying capacity.

Täljsten & Orosz (2006) conducted experiments using five beams with the same geometry, concrete quality (average compressive strength 38 MPa), and steel reinforcement (average tensile strength 517 MPa for the rebars and 530 MPa for the stirrups). Four of them strengthened with CFRP grids, while the first beam served as reference beam without CFRP strengthening. A great amount of longitudinal steel reinforcement ensured that the beams would not fail in bending. All the beams had steel reinforcement for shear only in one side. The CFRP sheets were applied using two types of mortars (Cement I. E1=26.5 GPa, with short glass fibres & Cement II. E2=18.0 GPa) in two layers with a thickness of 10 mm on both sides of the beams. The CFRP sheet was placed between the two layers of the mortar. Before applying the first layer of mortar a primer was applied to the sandblasted concrete surface to optimize the bond between concrete and mortar. All five beams failed in shear. The strengthening effect was significant, the increase in load carrying capacity for the strengthened beams was approximately 40-100 % compared to the reference beam. The theoretical approach gave a reasonable estimation of the shear strengthening effect, however it was difficult to exactly measure the strain in the tows and the scattering was large, therefore the theoretical evaluation is imperfect and further laboratory research together with more detailed analytical and numerical analysis is needed to improve the design model.

Hashemi & Al-Mahaidi (2012) research includes the investigation of the flexural behavior of FRP-strengthened reinforced concrete beams using cement-based adhesives. It is concluded that the use of cement-based bonding materials is a promising technique in FRP applications for structures located in hot regions or in danger of fire. Appendix C depicts mix ratios of concrete and mortar.

The test results in the form of ultimate load and mode of failure are shown in Table 2.6.

Table 2.6: Experimental results (Hashemi & Al-Mahaidi, 2012)

Beam designation	**Failure mode**	**Ultimate load (kN)**	**% Strength gain**
Control	Yielding of steel followed by secondary compression failure	121.2	
ESF	Mid span and end debond	161.7	33.40
MSF	Mid span and end debond	132.1	9.00
MSR	Mid span debond	138.7	14.40
MTF1	Mid span and end debond	151.9	25.33
MTF2	Mid span and end debond	155.2	28.05

ESF beam: A beam retrofitted with two strips of CFRP fabric using normal epoxy adhesive.

MSF beam: A beam which used cementitious mortar adhesive to attach two layers of CFRP fabric strips to the soffit.

MSR beam: A beam which used cementitious mortar adhesive to attach two layers of CFRP fabric strips to the soffit. In addition, the fabric was anchored at both ends in a manner similar to that used in the single-lap shear tests.

MTF beams: Beams which used CFRP textile and cementitious mortar to retrofit the beams.

It can be concluded that a considerable composite action can be achieved using cement-based mortar as an adhesive. Compared to CFRP fabric, CFRP textile is more compatible as well as more efficient with cement-based mortar. The ultimate load achieved by using CFRP textile-cement mortar is around 80% of what was achieved by using CFRP fabric with epoxy adhesive. The FE analysis showed a good consistency with experimental results, and it can be applied to other problems.

As per research by Hashemi & Al-Mahaidi (2008), it has been focused on investigation of flexural behavior of FRP strengthened beams using different type of mortars as bonding agent. The specimens included un-reinforced concrete beams that have been reinforced by FRP sheets and grids. Four types of mortar were used for FRP strengthening of the concrete substrate. The mixing ratios are presented in Appendix C. The results of the experiment are given in Table 2.7.

Table 2.7: Test results (Hashemi & Al-Mahaidi, 2008)

Type	**FRP Type**	**Maximum load (kN)**	**σ_{frp} (Mpa)**	**΄E_{frp} (*us*)**	**FRP efficiency %**	**σc (Mpa)**
OCS	Sheet	3.51	810	4050	20	21
OSS	Sheet	3.83	887	4437	23	24
OLS	Sheet	2.53	585	2926	15	16
OST	textile	4.73	1096	5479	29	29
MSS	Sheet	4.13	956	4782	25	26

The silica fume incorporated mortar (OS) had a better performance compared to ordinary Portland cement mortar (OC) and polymer modified mortar (OL). It shows the higher level of peak load as well as more ductile behavior. Also, it was concluded that replacing 20% of Ordinary Portland cement with microcement in silica-fume incorporated mortar, has improved the flexural performance of FRP strengthened member by increasing peak load and maximum deflection. As it illustrates in Table 2.7, the FRP textile strengthened member (OST) has better performance than FRP sheet strengthened member (OSS) using the same mortar as bonding material. In addition, the failure mode of FRP strengthened members were FRP debonding in all specimens using different types of mortar except silica-fume incorporated one (OSS) in which FRP rupture happened.

It can be concluded that all types of mortars that were used as a bonding material can effectively contribute to increase load carrying capacity and ductility of structural member. Since, higher level of capacity can be achieved by OC mortar without any substantial additives, compared to latex modified mortar (OL), it is not economical to add SBR latex to the mix. Furthermore, using the same strengthening material of FRP sheets, the best flexural performance from the micro-cement added mortar (MS), was obtained. The MSS specimens showed a ductile behaviour and the load carrying capacity has increased up to 2.5 times of un-reinforced concrete. Although such increase has been achieved, the efficiency of FRP material was about 25% of its full capacity. Moreover, the OST specimens, strengthened with FRP textile, presented higher capacity compared to FRP sheets which is due to better mortar penetration through the tows.

An experimental program (Al-Abdwais1 et al, 2013) involved testing sixteen specimens to study the modification of cement paste and identify the best bond properties with different mix designs and the workable ages of cement paste (pot-life). Appendix C presents four different mix designs with different ratios of superplasticizer and primer.

To evaluate efficiency of the tests, pull-out tests using single-lap shear test set-up were used. In all specimens, the bond length was fixed at 50 mm to identify the effect of different mix ratios on bond stress. The test results showed a significant difference of the modified cement-based adhesive as bonding agent between CFRP textile and concrete substrate.

From the experiment it can be concluded that excellent bond properties can be achieved using modified cement-based adhesive and it works efficiently as a bonding agent with the following findings:

- The best results were achieved by M4 with inconsiderable difference of strength within 20 minutes of time. It is therefore recommended to use M4 with 20 minutes of pot-life as a cement bonding agent for strengthening with NSM CFRP textile.
- Strengthening with NSM using modified cement adhesive is about 2.5 times more efficient than the externally-bonded CFRP. The average bond stress was about 10 MPa compared to the externally bonded CFRP textile reported by (Hashimi and Al-Mahaidi, 2012) which achieved only 4 MPa.

- The failure mode was the interfacial zone between fiber and adhesive associated with longitudinal cracks in the adhesive surface for all specimens.

The experiment carried out by Firmo et al. (2015), consisted of double-lap shear tests on concrete blocks strengthened with CFRP strips externally bonded with epoxy adhesive, conducted in both steady state (series S1) and transient (series S2) conditions. In series S1, two types of specimens were tested, with or without mechanical anchorage on the extremities of the CFRP strips. Specimens were first heated up to temperatures of 20 °C, 55 °C, 90 °C and 120 °C, and then loaded up to failure. In series S2, specimens were first loaded up to 25%, 50% or 75% of their ambient temperature strength and then heated up to failure. The obtained results in specimens without mechanical anchorage show that with increased temperature (i) the strain distributions along the bonded length change significantly, becoming closer to linear due to the epoxy adhesive softening; (ii) the effective bond length increases; (iii) the stiffness and the maximum shear stress of the bond–slip relationships suffer considerable reductions; (iv) the failure mode changes from cohesive (at ambient temperature) to adhesive (at elevated temperature); and (v) the overall stiffness and strength of CFRP–concrete interface decrease significantly, with the bond strength reduction in the transient tests being similar to that observed on the steady-state conditions. The incorporation of the mechanical anchorage, for all temperatures tested (i) led to more uniform axial strain distributions; (ii) presented bond–slip relationships with lower maximum shear stresses; and (iii) provided significantly higher bond strength.

Gamage et al. (2006) conducted an experiment to verify the critical temperatures, the temperature distributions and the factors affecting the performance of composite members at elevated temperature. Tests were carried out using CFRP strengthen concrete and adhesive was epoxy. Two series of single shear tests were conducted on the CFRP strengthened concrete blocks. The first series consists of eleven specimens made of non-insulated CFRP strengthened concrete blocks. Two insulated specimens were tested under the second series to determine the effects of insulation on the heat transfer behavior of the CFRP/concrete composites.

Both experimental and finite element results show that the epoxy temperature should not exceed 70 °C in order to maintain the integrity between the CFRP and concrete at high temperature. Also, the bond strength at elevated temperature does not depend on the CFRP bond length. The CFRP–concrete composites reach the failure point within 5.5–6 min under exposure to the standard fire. This indicates the need for a sound insulation system for CFRP strengthened concrete members. Fire resistance levels (FRL) for non-insulated and insulated members (50 mm thick insulation layer with conductivity—0.14 W/s/m/C) under the standard fire are 5.5 min and 1.76 h, respectively. Thermal conductivity and the thickness of the insulation layer affect the fire resistance of the insulated member.

Shehab El – Din & Mohamed (2013) conducted a research to determine the effect of temperature on compressive, tensile, and flexural strength of concrete strengthening with CFRP. In this experimental study, cylinders with 100 mm in diameter and 200 mm in length are cast to investigate compressive and tensile strength. To investigate flexural strength, the experimental program contains prisms 100 x 100 x 500 mm wrapped with CFRP. Epoxy was used as adhesive. There were three different forms of CFRP wrapping as shown in the Figure 2.10 and Figure 2.11.

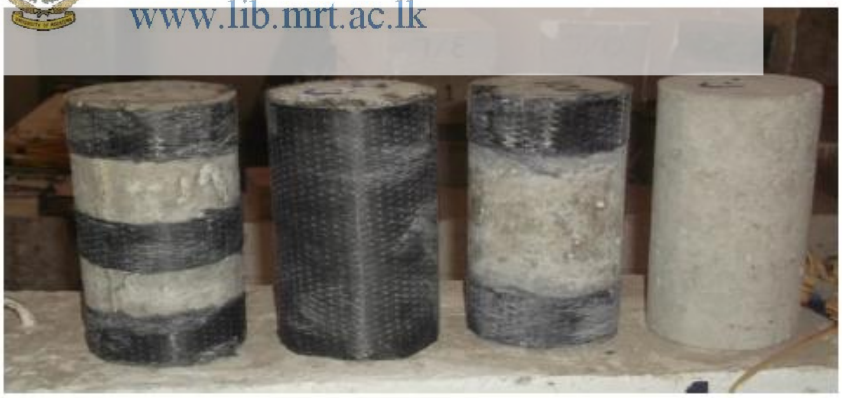


Figure 2.10: Different forms of CFRP wrapped

Source : Shehab El – Din & Mohamed, 2013

Figure 2.11: Different forms of CFRP wrapping on prisms

Source : Shehab El – Din & Mohamed, 2013

The following conclusions could be drawn from the experiment.

The higher the temperature the lower the values of compressive strength, when temperature increased from 0 °C to 200 °C the lowest value of decrease is obtained in case of wrapped with 3 strips and is estimated by about 20.52 %

The values of compressive strength are significantly increased as the specimen area wrapped with CFRP increases. The percentages of increase in cases of totally wrapped and wrapped with 3 strips are estimated by about 248.56 %, and 121.45 % respectively than that without wrapping at 200 °C.

Specimens wrapped with 3 strips have given the lowest percentage of decrease in tensile strength when temperature increased from 0 °C to 200 °C by about 18 %.

The percentages of increase in tensile strength were 204.54 %, 142.32 %, and 100.1 % from without wrapping to totally wrapped, wrapped with 3 strips, wrapped with 2 strips respectively at 200 °C.

The lowest rate of decrease in flexural strength is obtained in case of specimens wrapped with strips of CFRP.

The percentages of increase in flexural strength were 185.7 %, 16.88 %, and 3.9 % from without wrapping to wrap with strips, wrapped with U shape, totally wrapped between two point loads respectively at 200 °C.

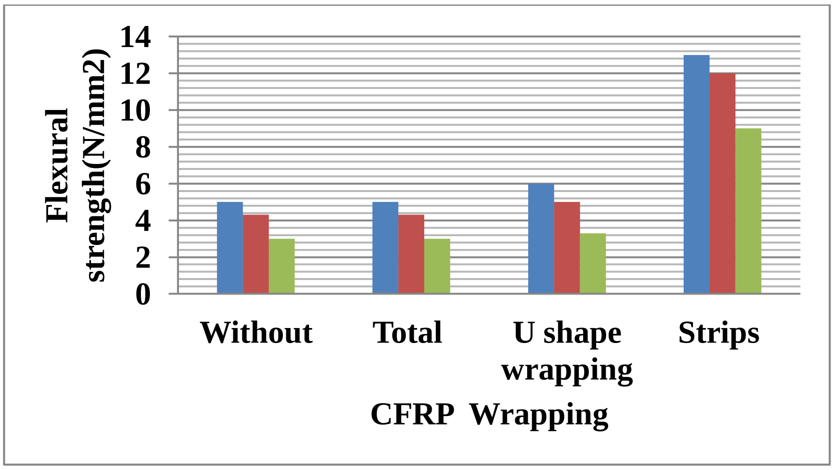


Figure 2.12: Effect of CFRP wrapping on flexural strength

Source : Shehab El – Din & Mohamed, 2013

Table 2.8: Effect of CFRP on flexural strength (Shehab El – Din & Mohamed, 2013)

CFRP Wrapping	Flexural strength (N/mm2)					
	0^0C	% increase	100^0C	% increase	200^0C	% increase
Without total	5.08 5.4	- 6.3	4.28 4.4	- 2.8	3.08 3.2	- 3.9
Without U shape	5.08 6	- 18.11	4.28 4.92	- 14.95	3.08 3.6	- 16.88
Without strips	5.08 12.72	- 150.04	4.28 11.92	- 178.5	3.08 8.8	- 185.7

2.10 Research gap

There are number of studies available about use of CFRP for strengthening concrete beams. From the literature, it was found that most of the researches had used epoxy as adhesive material. Therefore, it is necessary to carried out research studies using different adhesive materials as bonding agent.

Surface treatment on CFRP installation face of the beam and variation of bond performance when mixing different materials with cement based adhesive have not been researched sufficiently. Impact of beam's strength when ends are anchored is another area which has not been researched enough.

In this study it was experimented behavior of flexural performance of CFRP/concrete beams when cement grout was used as bonding agent. Further it was tested when primer coating was applied bonding surface and both ends of beam were anchored.

CHAPTER 03: EXPERIMENTAL STUDY

3.1 Experimental program

The behavior of CFRP strength reinforced concrete beams was examined. The Table 3.1 depicts details of the beams used for the experiment. Two beams were used as control beam specimens. The experimental results observed used to compare gain in flexural strength for different strengthening methods. This chapter provides detailed description of experimental program, procedure and results.

Table 3.1: Details of test beams

Strengthening Mechanism	Name of Beams	Notation	Strengthening Method
Flexural strengthening	Control beam 1	C1	No CFRP
	Control beam 2	C2	No CFRP
Beam Type "A" Flexural strengthening	Strengthened beam 1 - CFRP/Epoxy	A-E1	One layer of CFRP laid on tension face with epoxy adhesive
	Strengthened beam 2 - CFRP/Epoxy	A-E2	One layer of CFRP laid on tension face with epoxy adhesive
Beam Type "B" Flexural strengthening	Strengthened beam 3 - CFRP/Cement grout	B-C1	One layer of CFRP laid on tension face with cement grout adhesive
Beam Type "B" Flexural strengthening	Strengthened beam 4 - CFRP/Cement grout	B-C2	One layer of CFRP laid on tension face with cement grout adhesive

Table 3.1 cont..

Beam Type "C" Flexural strengthening	Strengthened beam 5 - CFRP/Primer/ Cement grout	C-PC1	First, primer was applied on tension face of the beam sample and then One layer of CFRP laid on primer coated tension face with cement grout adhesive
	Strengthened beam 6 - CFRP/Primer/ Cement grout	C-PC2	First, primer was applied on tension face of the beam sample and then One layer of CFRP laid on primer coated tension face with cement grout adhesive
Beam Type "D" Flexural strengthening	Strengthened beam 7 - CFRP/Primer/ Cement grout and anchor	D-PC1	First, primer was applied on tension face of the beam sample and then One layer of CFRP laid on primer coated tension face with cement grout adhesive further anchored both ends of beam with CFRP
	Strengthened beam 8 - CFRP/Primer/ Cement grout and anchor	D-PC2	First, primer was applied on tension face of the beam sample and then One layer of CFRP laid on primer coated tension face with cement grout adhesive further anchored both ends of beam with CFRP

3.2 Material properties

Properties of materials used in the experiment are discussed in this section.

3.2.1 Concrete

Grade 30 concrete was used for specimen casting. The cement strength class 42.5 ordinary Portland cement, 20 mm maximum particle size coarse aggregate and the particle size less than 4mm fine aggregate were used as materials for concrete mix. Density of cement, sand and aggregate are illustrated in the Table 3.2.

Table 3.2: Density of concrete constituents

Item No	Material Form	Value (kg/m^3)
1	Cement loose density	1,440
2	Cement solid density	3,150
3	Sand bulk density	1,600
4	Sand solid density	2,650
5	Coarse aggregate bulk density	1,440
6	Course aggregate solid density	2,800

3.2.1.1 Concrete mix design and requirement of materials

Selected beam specimen dimensions = 150X100X500 mm

Volume of concrete required for the beam = 0.1X0.15X0.5=0.0075 m^3

Characteristic strength = 30 N/mm^2

Mix ratio for grade 30 concrete (0.5 water cement ratio) } = 1:1:2 (20 mm aggregate size)

Table 3.3: Volume of concrete yield per bag of cement

Material	Volume of Loose State (m^3)	Volume of Solid State (m^3)
Cement (1)	$\frac{50}{1,440} = 0.0347$	$\frac{50}{3,150} = 0.0159$
Sand (1)	$\frac{50}{1,440} = 0.0347$	$\frac{0.0347}{2650} x1,600 = 0.021$
Coarse aggregate (2)	$\frac{50}{1,440} x2 = 0.0694$	$\frac{0.0694}{2,800} = 0.0357$
Water (.5)	$\frac{50}{1,440} x0.5 = 0.025$	$\frac{50}{1,440} x0.5 = 0.025$
Total Volume of cement per cement bag		0.098

Table 3.3 shows mix design calculation for grade 30 concrete, the solid state cement, sand and coarse aggregate material.

Table 3.4: Required quantities of material per beam.

Parameters	Values
Required number of cement bags	$\frac{0.0075}{0.098} = 0.076$
Quantity of Cement required per beam (kg)	0.076X50 =3.8
Quantity of Sand required per beam (kg)	0.076X0.0347X1,600=4.22
Quantity of Aggregate required per beam (kg)	0.076X0.0694X1,440=7.6
Quantity of water required per beam (kg)	0.076X0.025X1000 =1.9

Table 3.4 shows the quantity of materials requirement to construct the beam samples.

3.2.2 Reinforcement

Four numbers, 6 mm diameters mild steel bars were used as flexural reinforcement; two bars for top and other two bars as bottom reinforcement. Also, 4 mm diameter galvanized wire (GI) was used as shear links. The properties of reinforcement materials are shown in the Table 3.5. Reinforcements and shear links arrangement are shown in Figure 3.1.

Table 3.5: Properties of reinforcement materials.

Material	Property	Values
Steel reinforcement (Mild steel- 6mm Dia.)	Elastic modulus	200 000 N/mm^2
	Tensile strength	250N/mm^2
Stirrup galvanized wire (4mm dia.)	Elastic modulus	195 000 N/mm^2
	Tensile strength	363 N/mm^2(measured)

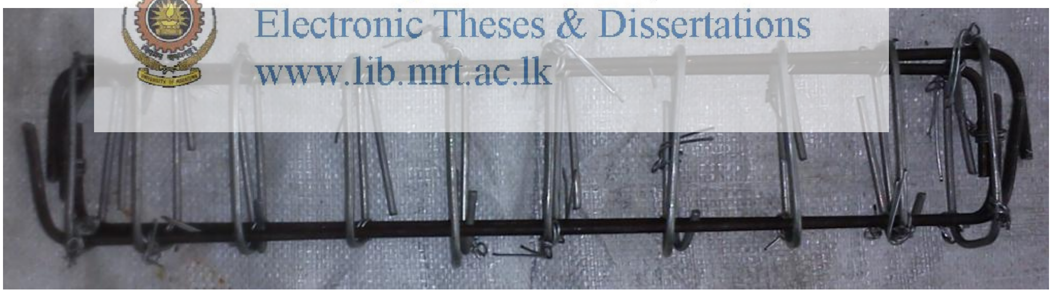


Figure 3.1: Reinforcement cage of specimens.

3.2.3 CFRP specification

Unidirectional CFRP (MBrace fibre) fabric (Figure 3.2) was used as external reinforcement for flexural strengthening. Two type of CFRP sheet were used during the experiment due to unavailability of sufficient material from one type. Properties of CFRP fabrics that were used in experiment according to the manufacture specification are given in the Table 3.6.

Figure 3.2: CFRP sheet

Table 3.6: CFRP manufactures specifications (BSAF,MBrace fabric, May 2009)

	CFRP Sample 1	**CFRP Sample 2**
Ultimate tensile strength	2,650 N/mm^2	4300 N/mm^2
Thickness	0.19 mm	0.168 mm
Sheet Width	0.3 m	0.3 m
Modulus of elasticity	640 kN/mm^2	240 kN/mm^2
Weight per unit area sheet	400 g/m^2	330 g/m^2
Prepared specimens	**AE1/AE2**	**BC1/BC2/C-PC1/C-PC2/D-PC1/D-PC2**

3.2.4 Primer specification

The primer is used to penetrate the surface of the concrete, providing an improved adhesive bond for the saturating resin or adhesive. The primer product used for this study is MBrace primer. The specifications for the primer used for this experimental program is given in Table 3.7.

Table 3.7: MBrace primer manufactures specification (BSAF,MBrace Specifications)

Parameter	**Value**
Mix ratio (Part A:Part B)	5:3 by weight
Tensile properties	
Yield strength	14.5 MPa
Strain at yield	2.0%
Elastic modulus	717 MPa
Ultimate strength	17.2 MPa
Flexural properties	
Yield strength	24.1 MPa
Strain at yeild	4%
Elastic modulus	595 MPa
Ultimate strength	24.1 MPa
Glass transition temperature	77^0C

3.2.5 Cement grout specification

The Cement grout was used as an adhesive material to bond between beam sample and CFRP sheet. The grout was prepared as cement to water ratio 2:1 of their weights.

3.2.6 Saturant specification

The saturating resin is used to impregnate the reinforcing fibers, fix them in place, and provide a shear load path to effectively transfer load between fibres. The specifications for the saturant used in the experiment are given in Table 3.8.

Table 3.8: Manufacturers specifications for saturant (BSAF,MBrace Specifications)

Ratio [Part A : Part B]	5 :2 [weight]
Tensile properties	
Yield strength	54 MPa
Strain at yield	2.5%
Elastic modulus	3,034 MPa
Ultimate strength	55.2 MPa
Flexural properties	
Yield strength	138 MPa
Strain at yeild	3.8%
Elastic modulus	3724 MPa
Ultimate strength	138 MPa
Glass transition temperature	71^{0}C

3.3 Specimen preparation

Preparations of specimens for the experiment are described in this section.

3.3.1 Reinforcement cage

A total of ten beams were cast which have dimensions of 100 mm X 150 mm X 500 mm. Two 6mm diameter mild steel bars were provided to each beam as shown in the Figure 3.3 for flexural reinforcement .And another two 6mm diameter mild steel bars were used at the top surface of each beam. Also as per design, 4 mm diameter galvanized steel bars were placed for shear links at a constant spacing of 50 mm throughout the entire length of the beams.

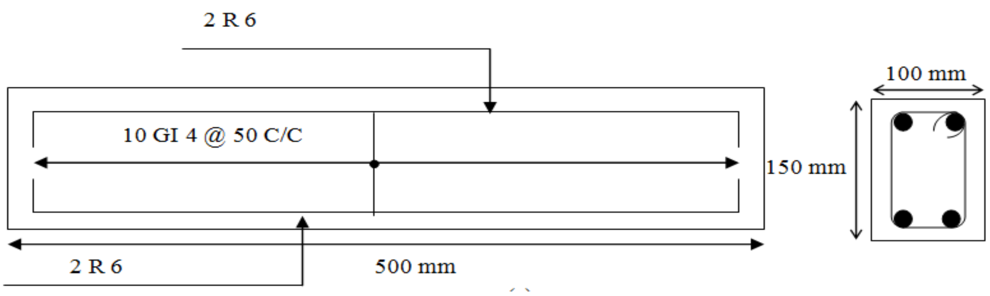


Figure 3.3: Reinforcement arrangement for flexural strengthened beams

3.3.2 Casting of beams

The beams were cast using steel moulds. Firstly, the mould oil was applied in the inner side of the moulds. Secondly, reinforcement cages were placed in moulds by keeping 25mm cover using cover blocks. Then concrete mix was prepared using a mechanical mixer according to the mix propositions mentioned in the section 3.2.1.1. The mixer was poured to the moulds manually and compacted with help of a vibration plate. Three no of concrete cubes (150 mm ×150 mm×150 mm) were cast from each concrete batch to check the compress strength of concrete. The cast beams and cubes were demoulded after 24 hours of casting and then those were dipped into the water tank and kept until 28 days for curing purpose. The Figure 3.4 presents casting of RC beams.

Figure 3.4: Casting of RC beam samples

3.3.3 Surface preparation

Surfaces of all beams were prepared using sand blasting technique. The sand blasting improves the bond strength between adhesive and concrete aggregates by removing weak material, surface laitance, other contaminates. The surface should free from dust, oil, and must be thoroughly dried at the time of application of adhesive. Figure 3.5 illustrates the technique of sand blasting of beams and the Figure 3.6 shows the surface prepared beam after sand blasting.

Figure 3.5: Sand blasting of beams

Figure 3.6: After surface preparation of beam

3.3.4 Application of primer

As shown in Figure 3.7, sand blasted surfaces were cleaned using wire brushes which remove dust particles in the surfaces.

Figure 3.7: Cleaning surface using wire brush

MBrace primer compound was used for this experiment work and it consists of two parts which are part A & part B. The two part of primer were mixed together (5:3 - part A: part B) according to the specification given ratio as shown in Figure 3.8.

Figure 3.8: Two part Primer

Then a layer of primer was applied on prepared cleaned concrete beams surface. Figure 3.9 shows application of primer to form a thin uniform coat on the surface of concrete beams. After application of primer, beams should be kept minimum of 45 minutes to allow for drying. A primed surface beam is given in Figure 3.10.

Figure 3.9: Application of Primer

Figure 3.10: Primed surface of beam

3.3.5 Application of CFRP

Application of CFRP for each beam used in the experiment is described in this section.

3.3.5.1 Application of CFRP on beams type "A"

The CFRP sheets were cut in to 50 mm wide and 450 mm long strips as shown in Figure 3.11. Then, two parts of saturant (base and hardener) were mixed into ratio of 5:2 (Part A: Part B), according to the specification given, as shown in Figure 3.12. After that saturant was mixed thoroughly about 3 minutes until it gets homogeneous. A thin layer of saturant was applied uniformly on the adhering faces of concrete beams and the CFRP sheet was attached to the concrete beam as shown in Figures 3.13 & 3.14. The CFRP sheet was pressed using a hard rib roller so that all entrapped air is removed. Another layer of adhesive was applied on top of the CFRP laminate and again forced with a rib roller. After application of CFRP, the specimens were kept seven days for curing.

Figure 3.11: CFRP sample preparing

Figure 3.12: Saturant sample A &B

Figure 3.13: Application of saturant

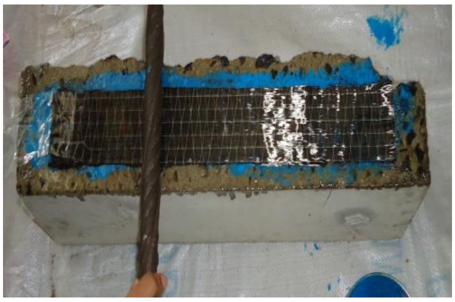


Figure 3.14: Attaching of CFRP and air removing

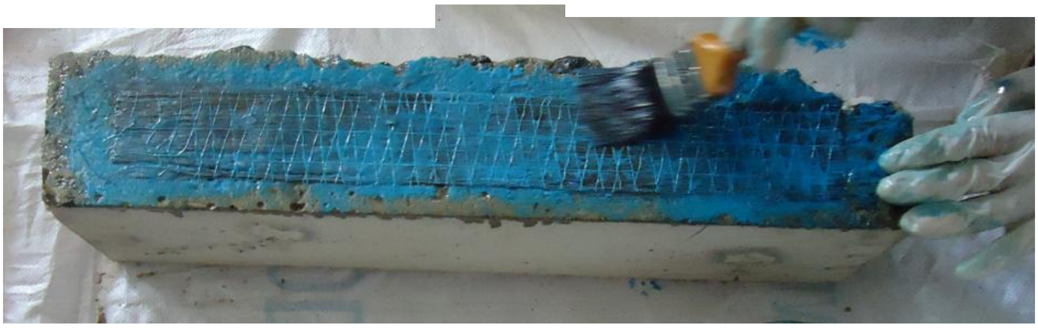


Figure 3.15: Application of saturant on pasted CFRP laminate

3.3.5.2 Application of CFRP on beams type "B"

The CFRP sheets were cut in to 50 mm wide and 450 mm long strips as shown in Figure 3.11. Cement grout was prepared as 2:1 cement to water ratio. Then, cement grout was mixed thoroughly about 3 minutes until it gets homogeneous. The cement grout was applied uniformly on the face of concrete beams and the CFRP sheet was attached to the concrete beams as shown in Figure 3.16. CFRP sheet was pressed using a hard rib roller so that all entrapped air is removed. Then, another layer of cement grout was applied on the face of pasted CFRP beam as shown in the Figure 3.17. Finally, the specimens were kept seven days for curing.

Figure 3.16: CFRP laminate pasted with cement grout

Figure 3.17: Applying of cement grout after pasted CFRP laminate

3.3.5.3 Application of CFRP on beams type "C"

The sand blasted, cleaned and primed coated beams were used for application of CFRP using cement grout as shown in Figure 3.9. The CFRP sheets were cut in to 50 mm wide and 450 mm long strips as shown in Figure 3.11. A Primer layer was applied to the tension face of the beam samples and cement grout that was used in beam type "B" was used to stick CFRP sheet on face of primer coated beam samples. Also, the cement grout was applied uniformly on the primer coated face of concrete beams and the CFRP sheet was attached to the concrete beams as shown in Figure 3.18. CFRP sheet was pressed using a hard rib roller so that all entrapped air is removed. Then, another layer of cement grout was applied on the face of pasted CFRP beams. Finally, the specimens were kept seven days for curing.

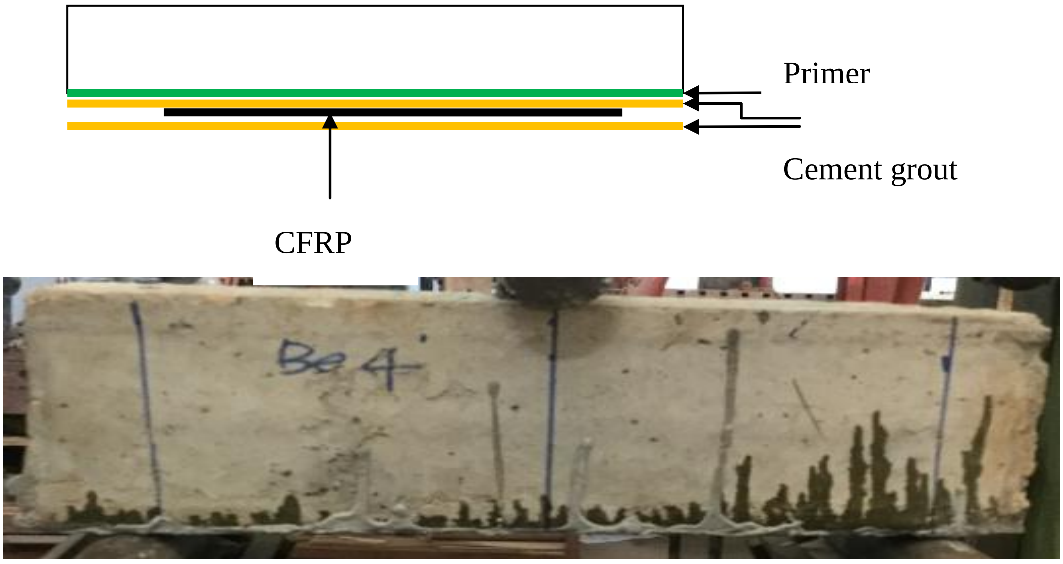


Figure 3.18: pasted CFRP laminate on primer coated beam sample

3.3.5.4 Application of CFRP on beams type "D"

The beam type "D" was prepared by further strengthen at end of the bond length of beam type "C". In this case, 'U' wraps were used to tie the "C" type beam. CFRP size of 250x50mm was used as 'U' wraps. The 'U' wraps were fixed to the beam 75 mm away from edge of the beam as shown in Figure 3.19 using primer and epoxy saturant. Figure 3.20 shows sample type "D" beam.

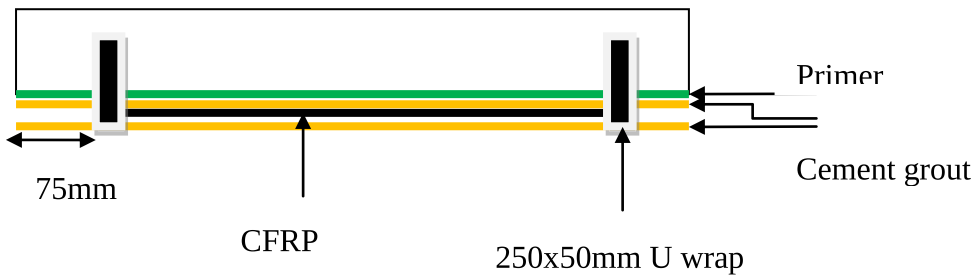


Figure 3.19: Sketch of "D" type beam sample

Figure 3.20: CFRP laminate on primer coated beam sample and strengthen with 'U' wraps at ends

3.4 Testing of materials and specimens

This section presents details of testing procedure and testing data of materials and specimens.

3.4.1 Materials testing

3.4.1.1 Compressive strength of concrete

Three concrete cubes of size 150 mm×150 mm×150 mm were cast from each batch to determine the compressive strength of the concrete. The beams used in this experiment were casted using two different concrete batches. Those cubes were tested in accordance with ASTM C 293.

Figure 3.21: Concrete Cube testing

Test results of each cube are depicted in the Table 3.9. The average compressive strength of the each concrete batches are 41.07 N/mm^2, 45.57 N/mm^2, respectively.

Table 3.9: Cube testing data - cubes at the date of testing beams

Concrete batch	Cube no	Dimensions in mm			Weight of cube kg	Crushing load kN	Compressive strength (N/mm^2)	Average Compressive strength(N/mm^2)	Standerdeviation
		Length	Width	Height					
Batch 01	1	151	153	152	8.1	907.5	39.28	41.07	1.57
	2	150	152	151	8.2	963.1	42.24		
	3	153	152	150	8.2	969.2	41.68		
Batch 02	4	152	152	150	8.1	1,075.3	46.54	45.57	1.09
	5	151	151	151	8.4	1,044.2	45.80		
	6	151	152	152	8.1	1,018.7	44.38		

3.5.1.2 Compressive strength of cement grout

Three cement cubes size of 50 mm×50 mm×50 mm were cast to determine the compressive strength of the cement grout. Those cubes were tested on the testing day of the corresponding specimen to get the actual material strength for the analysis as shown in Figure 3.22 and test results are listed in Table 3.10. The average compressive strength of the concrete is 28.7 N/mm^2.

Figure 3.22: Cement grout testing

Table 3.10: Cube testing data

Cube no	Dimensions in mm			Crushing load kN	Compressive strength (N/mm^2)
	Length	Width	Height		
1	50	50	50	79.5	31.8
2	51	50	51	55.2	21.6
3	52	50	50	84.9	32.7
Average Compressive Strength 28.7 N/mm^2					

3.4.1.3 Tensile strength of steel

Two types of steel which are Mild steel and GI steel were used for this experiment. Mild steel was used for tension reinforcement and GI steel was used for shear links. The ultimate tensile strength of GI steel was measured using tensometer apparatus. The actual tensile strength of GI steel was obtained from the average of two tensile test results. Figures 3.23 and 3.24 displays test results for each specimen separately.

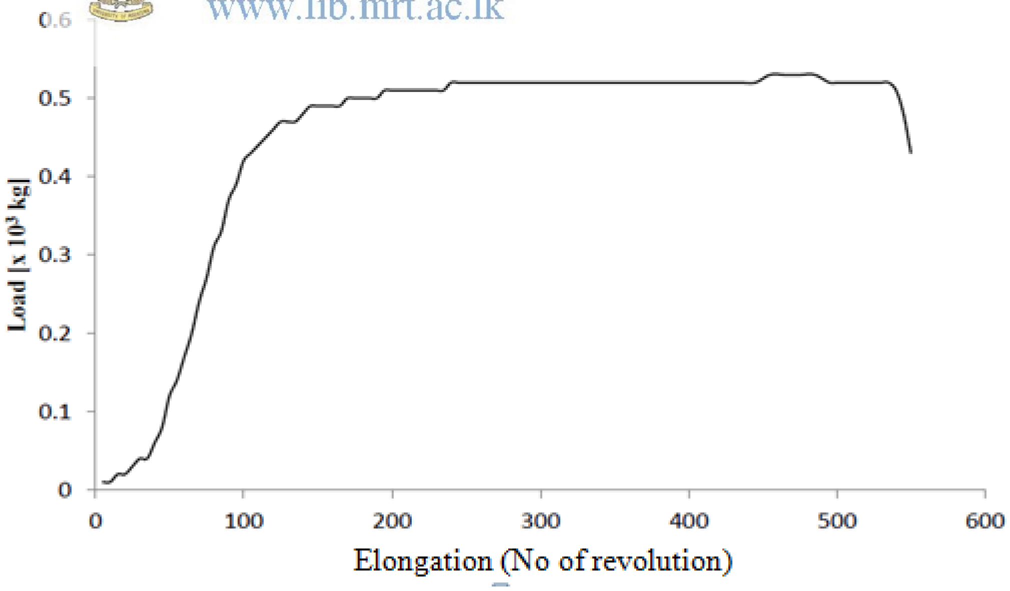


Figure 3.23: Load Vs Elongation for steel specimen no 01

Yield stress = $\frac{\text{Yield load (N)}}{\text{Average area (mm}^2\text{)}}$

For Sample 1, Yield stress is, f_{y_1}

$$f_{y_1} = \frac{0.47X1000X9.81}{\frac{\pi d^2}{4}} = \frac{0.47X1000X9.81}{\frac{\pi\, X4^2}{4}} = 366.76 \text{ N/mm}^2$$

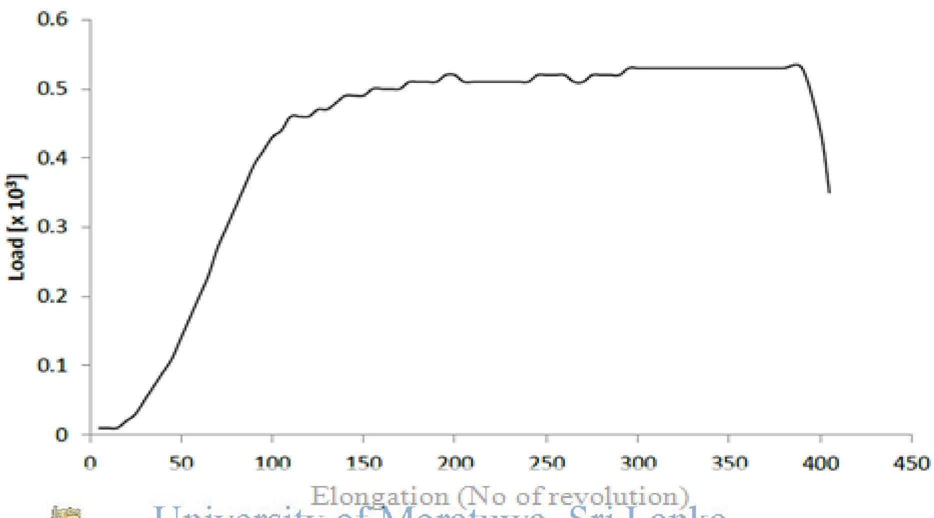


Figure 3.24: Load Vs No of revolutions for steel specimen no 02

For Sample 2, Yield stress is, f_{y_2}

$$f_{y_2} = \frac{0.46\ X\ 1000X9.81}{\frac{\pi d^2}{4}} = \frac{0.46X1000X9.81}{\frac{\pi\, X4^2}{4}} = 358.96 \text{ N/mm}^2$$

$$\text{Hence design } f_y = \frac{366.76+358.96}{2} = \mathbf{363} \text{ N/mm}^2$$

3.4.2 Specimens details

The detail of the tested beam samples are given in the Table 3.11.

Table 3.11: Specimens details

Name of Beams	Notation	Used concrete batch No	Used bonding material	Used CFRP type
Control beam 1	C1	Beam samples were cast using concrete batch 01	-	Beam samples were used Sample 01 CFRP
Control beam 2	C2			
Strengthened beam 1 - CFRP/Epoxy	A-E1		Epoxy	
Strengthened beam 2 - CFRP/Epoxy	A-E2			
Strengthened beam 3 -CFRP/Cement grout	B-C1	Beam samples were cast using concrete batch 02	Cement Grout	Beam samples were used Sample 02 CFRP
Strengthened beam 4 -CFRP/Cement grout	B-C2			
Strengthened beam 5 - CFRP/Primer/ Cement grout	C-PC1			
Strengthened beam 6 -CFRP/Primer/ Cement grout	C-PC2			
Strengthened beam 7 -CFRP/Primer/ Cement grout and anchor	D-PC1			
Strengthened beam 8 -CFRP/Primer/ Cement grout and anchor	D-PC2			

3.4.3 Specimens testing

All the beam specimens were tested over a simply supported span of 400 mm using single-point bending test method using Amsler testing machine. The vertical mid-span deflections were measured using mechanical dial gauges with 0.01 mm accuracy. The beams were loaded in a constant rate and deflection values were recorded 0.2 Mt intervals (Figure 3.25 and 3.26). The crack initiation load which is the load corresponding to 0.3mm crack width and ultimate failure load were also recorded.

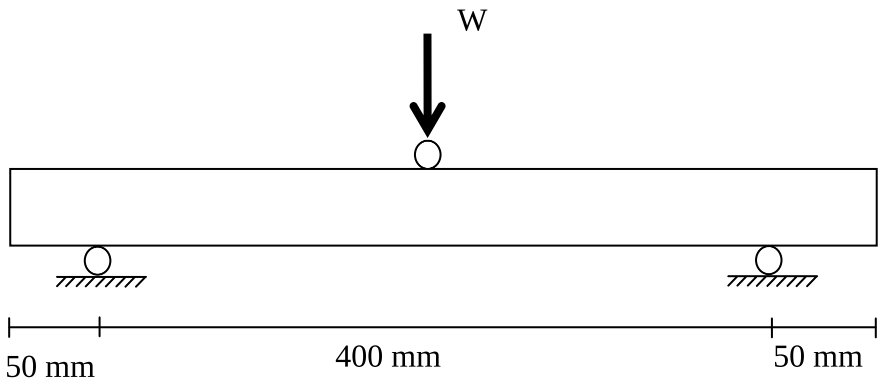


Figure 3.25: Schematic diagram for the test setup.

Figure 3.26: Testing of samples

3.5 Experimental results

This section explains the results of experimental study. It includes failure loads, failure modes and maximum deflection of the beam samples. The ultimate loads and failure modes of the specimens are given in Table 3.12 & Table 3.13 respectively.

Table 3.12: Ultimate failure loads for beams

Specimen identification	Experimental failure load kN	Average Experimental failure load kN	Average % of strength gain with respect to CB(for CFRP sample 01, epoxy used)	Average % of strength gain with respect to CB(for CFRP sample 02, cement grout used)	Average % of strength gain with respect to CFRP sample 2, cement grout used
C1	9.8	10	-	-	-
C2	10.3				
A-E1	14.8	13.9	39%	-	-
A-E2	13.0				
B-C1	31.2	27.9	-	179%	-
B-C2	24.6				
C-PC1	31.6	30.8	-	208%	10%
C-PC2	30.1				
D-PC1	34.6	37.9	-	279%	36%
D-PC2	41.2				

Details of beam specimens referred in Table 3.12 & Table 3.13 are given in Table 3.11. Specimen details of CFRP samples are given in Table 3.6.

Sample 1 CFRP with epoxy as adhesive bond showed 39% strength gain relative to control beam as given in Table 3.12. Its failure was due to de-bonding.

Sample 2 CFRP with cement grout as adhesive bond showed 179% strength gain relative to control beam as given in Table 3.12. Its failure was due to de-bonding. Strength gain was 208 % compared to control beam with priming the substrate. When two ends are anchored strength gain was further improved to 279 % compared to control beam as depicted in Table 3.12. As shown in Table 3.13 failure mode was de-bonding in all specimens except B-C type beams.

Table 3.13: Failure modes of beams

Specimen identification	Description of failure mode
C1	Flexural failure due to steel yielding
C2	Flexural failure due to steel yielding
A-E1	Flexural failure and de-bonding
A-E2	Flexural failure and de-bonding
B-C1	Flexural failure and end delamination
B-C2	Flexural failure and end delamination
C-PC1	Flexural failure and de-bonding
C-PC2	Flexural failure and de-bonding
D-PC1	Flexural failure and de-bonding
D-PC2	Flexural failure and de-bonding

3.6 Analysis of results

Results of this test program are analyzed in this section.

3.6.1 Specimens C1 and C2

Specimens C1 and C2 are the control beams. When the load increases, a single flexural crack was initiated at the middle of the span of both specimens as shown in Figures 3.27 and 3.28. The crack was widened with increased load and finally both control beams were failed at ultimate loads, 9.8kN & 10.3kN. Both control beam failure modes were observed as flexural failure.

Figure 3.27: Failure pattern of C1

Figure 3.28: Failure pattern of C2

3.6.2 Specimens A-E1 and A-E2

These two beams were strengthened with CFRP using epoxy adhesive. Both the specimens were failed due to flexure and concrete cover separation at concrete /epoxy interface. A single flexure crack was initiated at the near mid span of vertical plane and propagated almost in vertical direction. The crack was widened with increasing load and finally two beams failed at the ultimate load level 14.8kN & 13.0kN. The crack patterns are shown in Figures 3.29 and 3.30. The failure modes of the both beam specimens were observed as flexural failure and debonding.

Figure 3.29: Failure pattern of A-E1

Figure 3.30: Failure pattern of A-E2

3.6.3 Specimens B-C1and B-C2

CFRP sheet was attached to the bottom face of the beam using a cement paste. Both specimens failed due to flexure and end delamination between specimens and cement grout interface was observed. When the load is applying, a single flexural crack was initiated at the middle of the span of both specimens as shown in the Figures 3.31 and 3.32. The crack was widened with increasing loads and finally two beams failed the ultimate loads reaching 31.2kN and 24.6kN, respectively.

Figure 3.31: Failure pattern of B-C1

Figure 3.32: Failure pattern of B-C2

3.6.4 Specimens C-PC1and C-PC2

CFRP fibre was pasted on primer coated faces of the beams using cement paste. Both the specimens failed due to flexure and de-bonding between primers coated specimens face and CFRP/cement grout interface. When the load is increasing, a single flexural crack was initiated at the middle of the span of both specimens as shown in the Figure 3.33 and Figure 3.34. The crack was widened with increasing loads and finally two beams failed the ultimate loads reaching 31.6kN and 30.6kN, respectively.

Figure 3.33: Failure pattern of C-PC1

Figure 3.34: Failure pattern of C-PC2

3.6.5 Specimens D-PC1and D-PC2

These two specimens were made further strengthening of "C" type beams which were end anchored with 50mm wide CFRP "U" straps in both side of the CFRP bond length. Both the specimens failed due to flexure. De-bonding happened between primer coated specimens' face and CFRP/cement grout interface at the middle of the span. When the load increases, a single flexural crack was initiated at the middle of the span of both specimens as shown in Figure 3.35 and Figure 3.36. The crack was widened with increasing loads and finally two beams failed the ultimate loads reaching 34.6kN & 41.2kN.

Figure 3.35: Failure pattern of D-PC1

Figure 3.36: Failure pattern of D-PC2

3.7 Load vs. deflection behavior in flexural strengthened beams

Applied load Vs mid span deflection was plotted as shown in Figures 3.37a & 3.37b. The simply supported beams were loaded at the center and deflection was measured.

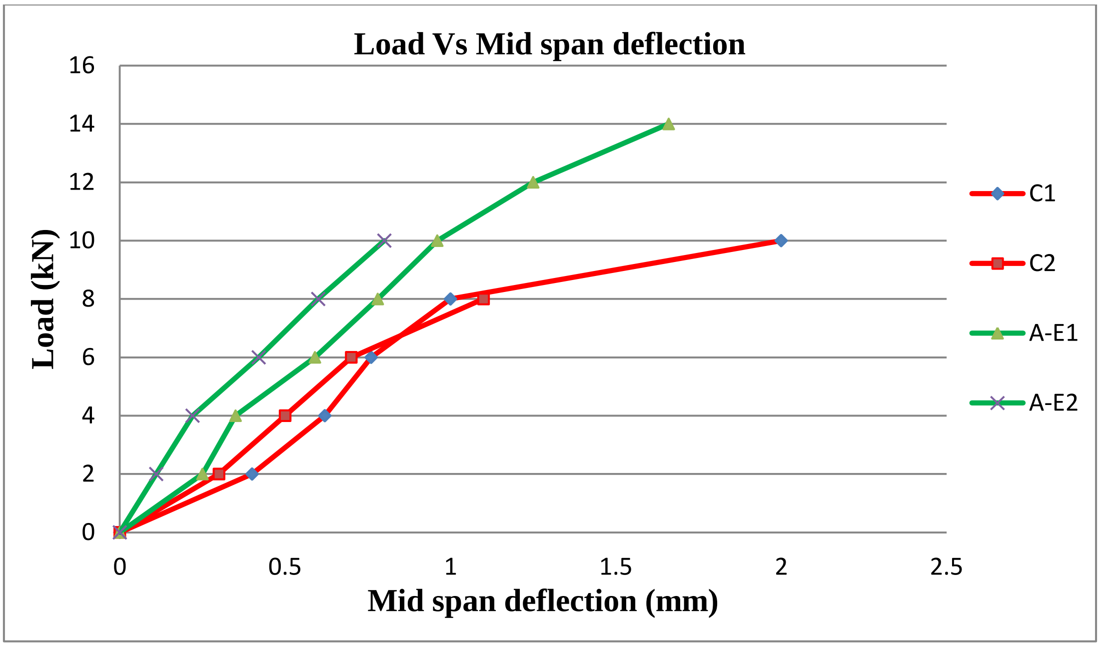


Figure 3.37a: Load Vs Mid span deflection (used CFRP sample 1, epoxy adhesive)

Figure 3.37a illustrates the comparison between strength gains of beams with respect to control beams. It is observed that all the strengthened beams behave as the control beams at the initial stage with the internal reinforcing bars carrying the majority of tensile forces in the section. When the internal steel yields, the additional tensile force is taken by the CFRP system.

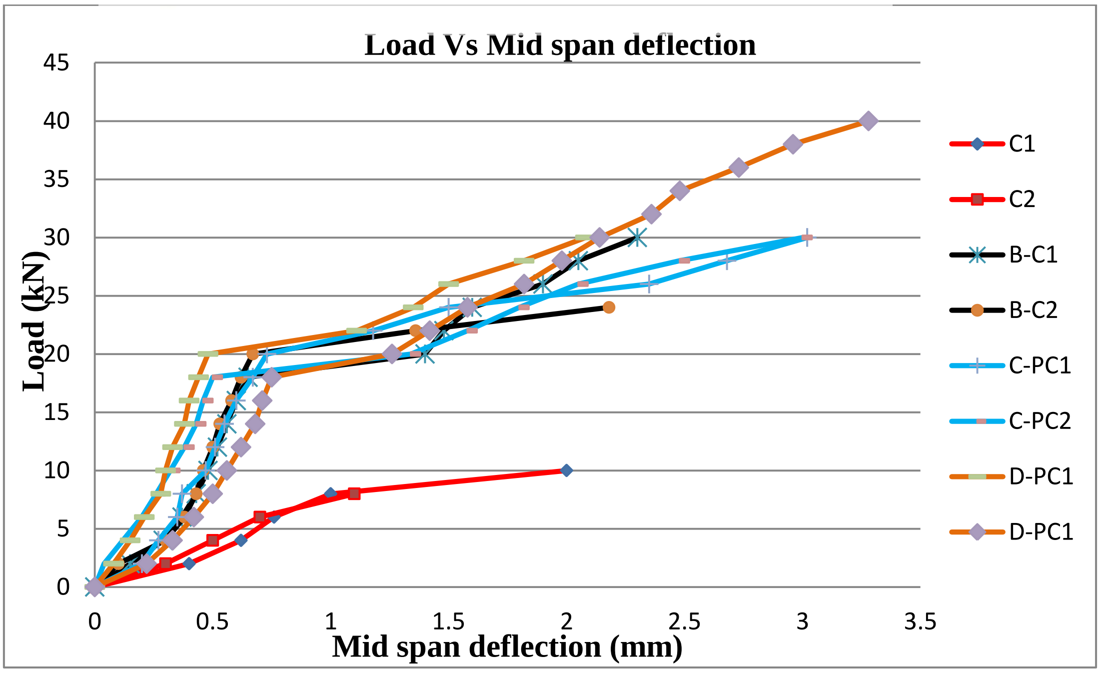


Figure 3.37b: Load Vs Mid span deflection (used CFRP sample 2, Cement grout)

As shown in Figure 3.37b, the beams strengthened using cement grout showed higher load carrying capacity than the beams strengthened using epoxy adhesive. The strength gain by primer coated beams – type C-PC are slightly higher than the beams strengthened using cement grout (type B-C). There is a significant strength increase in the end anchorage beam (type D-PC). Ultimate tensile strength of CFRP sample 2 was higher than CFRP sample 2.

The experimental results illustrate that the deformation capacity of CFRP strengthened beams are significantly higher than control beams, Ductility of a member is defined as its ability to sustain inelastic deformations prior to failure without substantial loss of strength. A ductile system displays sufficient warning before catastrophic failure. It is obvious that the beams retrofitted with CFRP showed the highest deformation before collapse.

Occurrence of 0.3mm wide crack or excessive deflection was considered as the limit for serviceability failure. Load corresponding to 0.3mm crack width was observed and noted during testing. According to the BS 8110 part 1 (1997), Clause 3.2.1., the noticeable deflection can be calculated as span/250.

In this case, span of the test beam samples is 400mm.

Therefore, L/250 = 400/250 = 1.6 mm.

The maximum allowable deflection for the beam in order to satisfy safety requirement is 1.6mm. Figure 3.38 shows serviceability failure loads for the specimens under the two criteria which are 0.3mm crack width (the load observed during beams testing) and 1.6mm deflection.

As shown in Figure 3.38, the majority of these beam samples have reached maximum crack width of 0.3 mm before reaching allowable deflection except beam samples C2 A-E2, C-PC1and C-PC2. The beam samples C2 and C-PC2 have shown other way round. The two beams which are A-E2 and C-PC1, have reached crack width of 0.3mm and maximum allowable defection at same load.

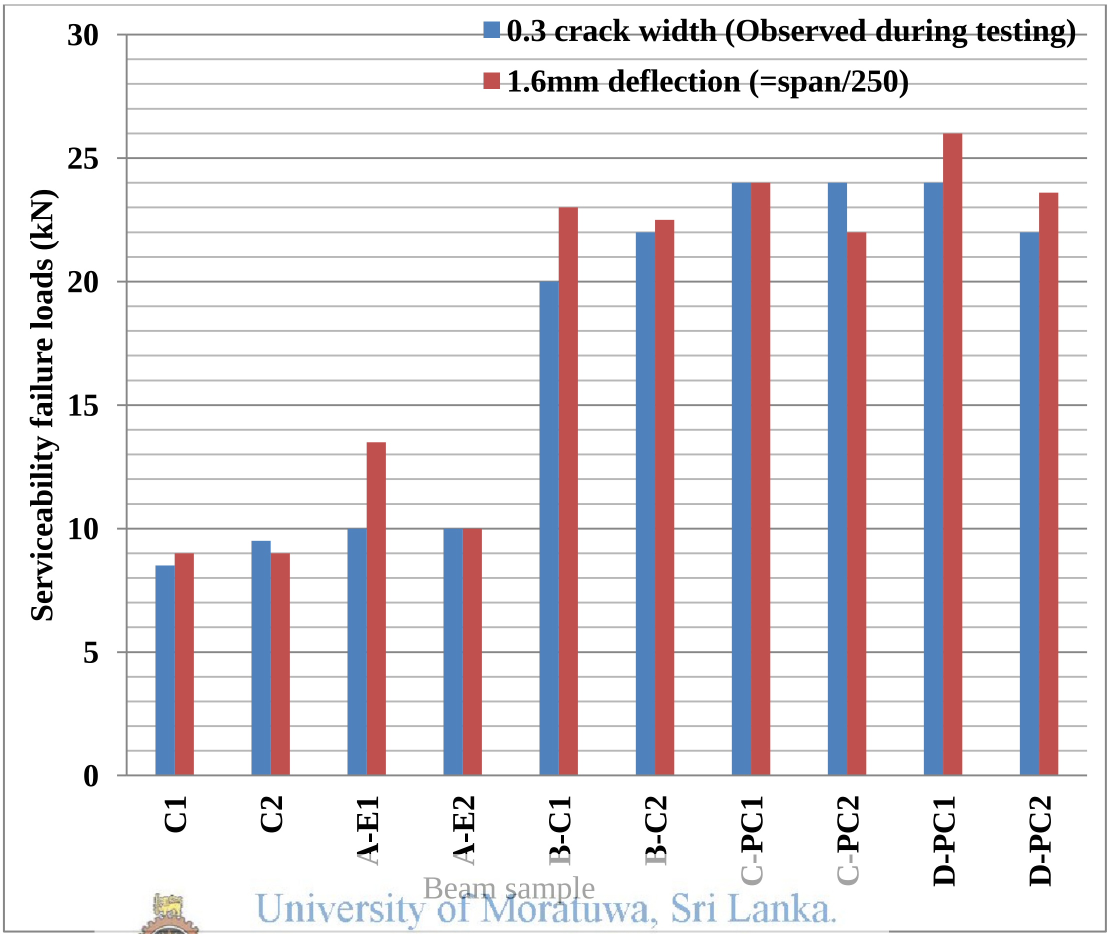


Figure 3.38: Serviceability failure loads Vs beam samples

3.8 Conclusion

The objective of this research was to investigate on RC beam flexural strength with externally bonded CFRP and to investigate the suitability of using cement-based material as bonding agent in strengthening of existing RC beams. There were ten sample beams used for the experiment. The specimen sample types are;

1. Control beams with no external strengthening (Type C beams).
2. External strengthened beams using one layer of CFRP lay on the tension face with epoxy adhesive (Type A-E beams).
3. External strengthened beams using one layer of CFRP lay on the tension face with cement grout adhesive (Type B-C beams).
4. External strengthened beams using one layer of CFRP lay on primer coated tension face with cement grout adhesive (Type C-PC beams).

5. External strengthened beams using one layer of CFRP lay on primer coated tension face and with cement grout adhesive, then anchored both ends of CFRP sheet with in wrapped (Type D-PC beams).

The specific characteristic strength of concrete specimens was $30kN/mm^2$. According to the cube test result, the mean strength of concrete was $41.07kN/mm^2$ for concrete batch 01 and $45.57kN/mm^2$ for concrete batch 02. From preliminary design calculations, it was found that the ultimate flexure strength as 16.16 kN and 16.19 kN for batch 01, and batch 02 respectively. And ultimate shear strength as 113.6 kN in control beams.

The failure mode of control beams were flexural failure due to steel yielding and an ultimate average failure load was 10.0 kN. When compared with design capacity of control beam the experimental value is less than by 6.03Kn.The serviceability loads were observed 8.5 kN and 9.5 kN respectively for both beam samples.

Type A-E beams failed due to flexural failure and debonding with an ultimate average failure load was 13.8 kN. Its ultimate strength gain was 39% higher than control beams. Furthermore, serviceability loads were 10.0 kN for both beam samples.Type B-C beams failed due to flexural failure and end delamination with an ultimate average failure load was 27.9 kN. Its ultimate strength gain was 179% higher than control beams. Furthermore, the observed serviceability loads were 20.0 kN and 22.0 kN respectively for both beam samples.

Considering type C-PC beams, the failure mode of both beams were flexural failure and de-bonding. An ultimate average failure load was 30.8 kN. It is about 208% strength gain compared to control beams and 10% strength gain compared to type B-C beams. Both C-PC beams showed almost same strength increments under serviceability limit failure criteria and it was 24.0 kN. The A-E type beams failed due to flexural failure and debonding with an ultimate average failure load is 37.9 kN. It ultimate strength gain was 279% higher than control beams and 36% strength gain compare with B-C type beams. Furthermore, the observed serviceability loads were 24.0 kN, 22.0 kN for both beam samples respectively.

CHAPTER 04: THEORITICAL ANALYSIS

4.1 Introduction

There is no direct method to design of FRP strengthening systems since it is an evolving method. However, American Concrete Institute (ACI) Committee 440 had developed specifications guide for the design and construction of externally bonded FRP systems (ACI-440-2R-02).The Canadian Design and Construction of Building Composites with Fiber Reinforced Polymers (CAN/CSA S806) is another guide line that provides design rules for externally bonded FRP reinforcement for concrete. European Fib Bulletin 14 (2001), Design and Use of Externally Bonded Fiber Polymer Reinforcements (FRP EBR) for Reinforced Concrete Structures is another informative document available in this area.

4.2 Theoretical calculations for load carrying capacity

This section includes theoretical calculations for load carrying capacity of strengthened and non-strengthened beams.

4.2.1 Prediction of Flexural/ Shear capacities for control beams

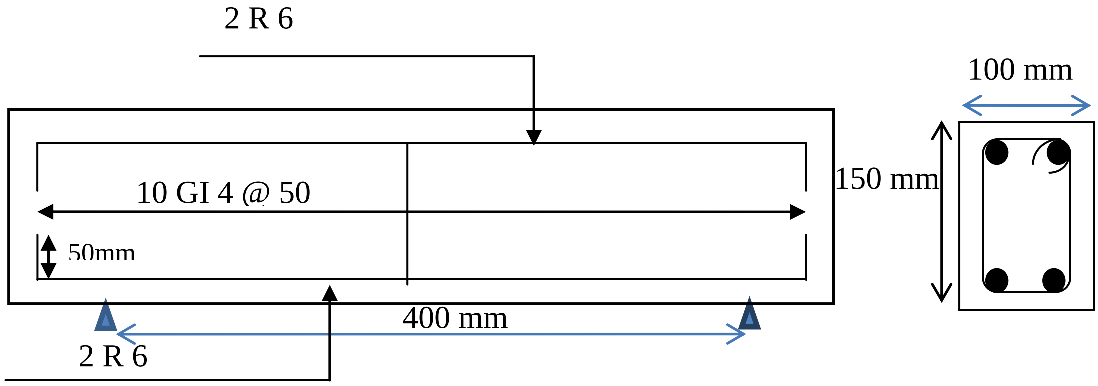


Figure 4.1: Schematic diagram of non strengthened test beam

Theoretical failure load of un-strengthened simply supported beams was calculated in accordance with BS 8110 (1985) code of practices.

Table 4.1: Design parameters for un-strengthened test beam

Parameter	Value
Depth (h)	150 mm
Width (b)	100 mm
Average Concrete Grade (f_{cu})	41.07 N/mm^2 (batch 01 cube test result) – Standard deviation 1.57
	45.57 N/mm^2 (batch 02 cube test result) - Standard deviation 1.09
Tension Reinforcement	Mild steel 6 mm in diameter
Mild steel Grade (f_y)	250 N/mm^2 (Assumed)
Shear Links	Galvanized steel 4 mm in diameter
Galvanized steel Grade (f_{yv})	363 N/mm^2 (Obtained via sample testing)
Spacing of shear links	50 mm
Cover for Reinforcement	25 mm

Expected flexural load capacity (According to BS 8110; part II; 1985)

Effective depth to reinforcement (d) $= 150 - 25 - 4 - \frac{6}{2} = 118$ mm

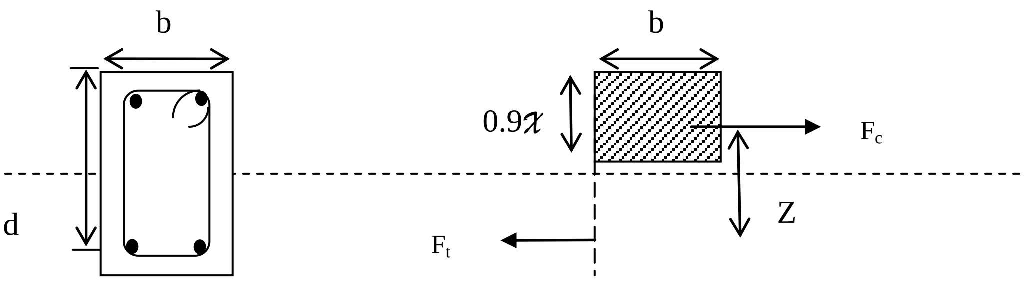
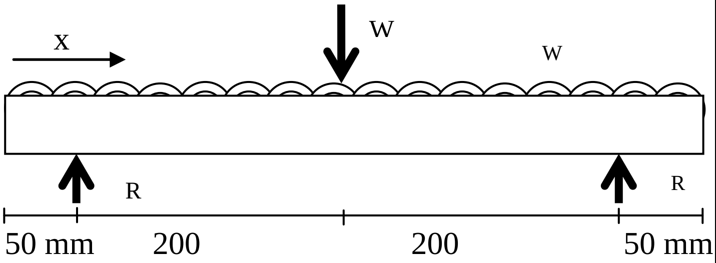


Figure 4.2: Stress distribution in beam

Reference	Calculation	Output
BS 8110: part I- Cl 3.4.4	Assuming that the top reinforcement is under no stress $F_c = F_t$ (in order to balance the force) $F_c = 0.67X\ f_{cu}(0.9\ x\ 100)$ **For $f_{cu} = 41.07\ N/mm^2$** $F_t = A_s f_y$ $F_t = 2X\frac{\pi}{4}X\ 6^2 x f_y$ $0.67x41.07x0.9x100X\ (x) = 2\ X\ \frac{\pi}{4} x\ 6^2 X\ 250$ x = 5.71 mm Flexure capacity (F_c) $= F_t\ x\ Z$ Lever Arm (Z) = d-[(0.9 x)/2) =118-[(0.9x5.71)/2]=115.4 mm	
BS 8110: part I- Cl 3.4.4.4	Ex. Flex. Capacity, M = As*Fy*Z $= \frac{2X\pi X36x250X115.4}{4}$ $= 1.632X10^6$ N mm M = 1.632 kNm **For $f_{cu} = 45.57\ N/mm^2$** M = 1.636 kNm	M =1.632 kNm M =1.636 kNm

Reference	Calculation	Output
	 Maximum moment will occur midway span between supports at x=0.250 m. For 0.05 < x < 0.450 $$M = -\frac{(wl + W)}{2}(x - 0.050) + w\frac{x2}{2}$$ **For M = 1.632 kNm** Therefore for maximum moment at centre M_{max}= 0.10W-0.0456 w Hence expected failure load in flexure $W = \frac{1.632 - 0.0456X0.15X0.1X24}{0.10}$ = 16.16 kN **ForM = 1.636 kNm** W = 16.19kN	W= 16.16 kN W= 16.19 kN

Expected Shear capacity (According to BS 8110; part II; 1985)

Reference	Calculation	Output
BS 8110: part I- Table 3.9	Shear capacity due to concrete $SC_{Conc.} = \dfrac{0.79\left(\frac{100A_s}{b_v d}\right)^{\frac{1}{3}}\left(\frac{400}{d}\right)^{\frac{1}{4}}\left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}}}{\gamma_m}$; $= 0.755\ N\ mm^{-2}$ $where\ \frac{100A_s}{b_v d} \leq 3;\ \frac{400}{d} \geq 1;\ \gamma_m = 1.25$	
BS 8110: part I- Cl 3.4.5.3	Shear capacity of stirrup; **As = $\frac{2x\pi x6x6}{4}$ = 56.5 mm^2** b_v =100 mm d = 118 mm v_c= 0.713 N mm^{-2} Shear carried out by concrete $Sc_{conc} = v_c x\ b_v x\ d$ = 0.755 X100X118 = 8.41 kN Shear carried out by links $Sc_{links} = \frac{A_{sv} f_{yv}\ d}{s}$ $Sc_{links} = \frac{2X22X6^2X363X118}{7x4x50}$ = 48.5 kN Shear capacity = Sc_{conc}+ Sc_{link} Shear capacity = 8.41 +48.5 =56.91 kN	

Reference	Calculation	Output
	The shear force at a distance "*x*" from an end can be written as follows, V= $\left(\frac{W}{2}+\frac{wl}{2}\right)-wx$ Maximum shear force would be at support, ie *x*=0.050 m Self weight of beam =0.1X0.15X 24 kN/m V= $\left(\frac{W}{2}+\frac{01X0.15X24X(0.50-2X0.050)}{2}\right)$ = 56.91 kN Hence expected failure load in shear W =113.6 kN	Shear capacity =113.6 kN

According to design calculations, the ultimate flexural failure loads for control beams are 16.16 kN and 16.19 kN for concrete batch 01 and batch 02 respectively. Hence, there is not much variation of ultimate load carrying capacities for both cases. Therefore, it can be assumed ultimate flexural failure load for both concrete batches was 16.16 kN. The calculated ultimate shear capacity of the control beam is 113.6 kN. Thus, the expected shear failure load of test specimens is about seven times higher than the flexure failure. Therefore, it can be assumed as the control specimens will fail in flexure due to steel yielding.

4.2.2 Prediction of Flexural capacities for CFRP strengthened beams

This consists of design calculations for all strengthened beams used in the experiment.

4.2.2.1 ACI-440-2R design guide

ACI-440-2R is design guide for design and construction of externally bonded FRP systems for strengthening concrete structures, published by ACI committee. It provides guidelines to find the flexural capacity, shear capacity, and axial load carrying capacity of concrete members strengthened using CFRP. In addition to that, it includes design equations, construction requirements, and material quality requirements.

According to the ACI guide lines, following assumptions are made in calculating the flexural resistance of a section strengthened concrete beam using an externally applied FRP system.

1. Design calculations are based on the actual dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened.
2. The strains in the reinforcement and concrete are directly proportional to the distance from the neutral axis.
3. There is no relative slip between external FRP reinforcement and the concrete.
4. The shear deformation within the adhesive layer is neglected since the adhesive layer is very thin with slight variations in its thickness.
5. The maximum usable compressive strain in the concrete is 0.003.
6. The tensile strength of concrete is neglected.
7. The FRP reinforcement has a linear elastic stress-strain relationship to failure.

The nominal flexural strength of a FRP-strengthened concrete member can be determined based on strain compatibility, internal force equilibrium, and controlling mode of failure. The flexural strength of a section depends on the controlling failure mode. The following flexural failure modes should be investigated for FRP strengthened section (GangaRao and Vijay, (1998)).

- Crushing of the concrete in compression before yielding of the reinforcing steel;
- Yielding of the steel in tension followed by rupture of the FRP laminate;
- Yielding of the steel in tension followed by concrete crushing;
- Shear/tension delamination of the concrete cover (cover delamination); and
- Debonding of the FRP from the concrete substrate (FRP debonding).

Concrete crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain (€*c* = €*cu* = 0.003). Rupture of the FRP laminate is assumed to occur if the strain in the FRP reaches its design rupture strain (€*f* = €*fu*) before the concrete reaches its maximum usable strain. Cover delamination or FRP debonding can occur if the force in the FRP cannot be sustained by the substrate. In order to prevent debonding of the FRP laminate, a limitation should be placed on the strain level developed in the laminate.

4.2.2.2 Stress distribution adopted for the design

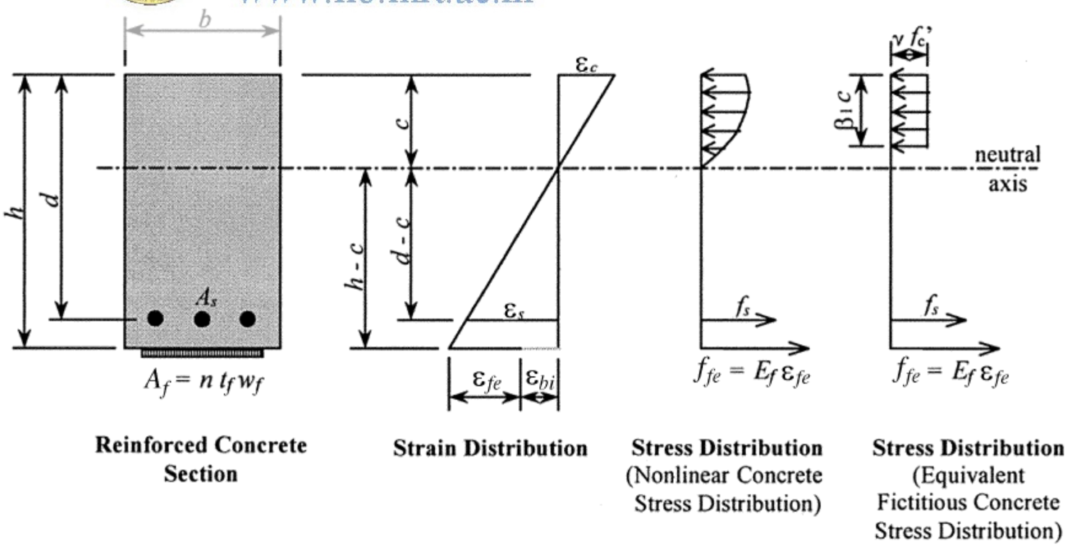


Figure 4.3: Internal strain and stress distribution for a rectangular section under flexure at ultimate stage.

Source : ACI 440.2R-02.

According ACI-440-2R, the assumed stress distribution over the cross section of FRP strengthened member for design purposes is shown in Figure 4.3. The code has idealized concrete stress in the compression zone in to a rectangular area. Compression capacity of top reinforcement and tension capacity of concrete below the neutral axis are neglected.

4.2.2.3 Calculation of flexural capacity according to the ACI-440-2R

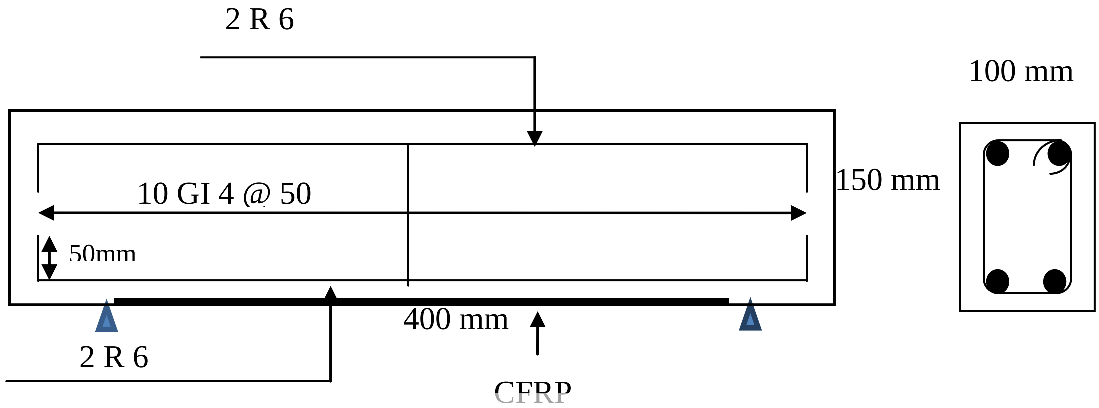


Figure 4.4: Schematic diagram of strengthened test beam

Table 4.2: Design parameters for strengthened test beam

Parameter	Values
Depth (h)	150 mm
Width (b)	100 mm
Tension Reinforcement	Mild steel 6 mm in diameter
Mild steel Grade (f_y)	250 N/mm^2
Shear Links	Galvanized steel 4 mm in diameter
Es	200 GPa
Galvanized steel Grade (f_{yv})	363 N/mm^2 (Obtained via sample testing)
Spacing of shear links	50 mm

Table 4.2 cont..

CFRP	**CFRP Sample 1 (Beam type A-E- Strength using epoxy)**	**CFRP Sample 2 (Beam types B-C,C-PC,D- PC-Strength using Cement based adhessive)**
Ultimate tensile Strength	2,650 N/mm^2	4300 N/mm^2
Thickness	0.19 mm	0.168 mm
Sheet Width	0.3 m	0.3 m
Modulus of elasticity	640 kN/mm^2	240 kN/mm^2
Weight per unit area sheet	400 g/m^2	330 g/m^2
Depth to FRP reinforcement from the extreme compression fibre	150+0.1+0.19	150+0.3+0.168
	=150.29mm	=150.468mm
f'_c	41.07 MPa	45.57 MPa
Length of FRP sample	450mm	
Width of FRP sample	50mm	
Effective depth to reinforcement (d)	150- 25 - 4 -(6/2) = 118 mm	

Reference	Calculation	Output
ACI-440-2R-2 Table 8.1	Environmental reduction factor *CE* , for proper comparison	*CE*=1
ACI-440-2R-2 section 8.4	Design tensile strength of FRP $f_{fu} = CE\, f_{fu}{*}$ for sample 1, f_{fu} =2650x1= 2650 N/mm^2 for sample 2, f_{fu} =4300x1=4300 N/mm^2	f_{fu} = 2650 N/mm^2 sample 1 f_{fu}=4300N/mm^2 sample 2
	Similarly, Design tensile strain of FRP Sample 1 $\varepsilon_{fu} = CE\ E_{fu}\ {*}$ ε_{fu} = 1x2650/6400000 = 0.004141	ε_{fu} = 0.004141 sample 1
	Sample 2 $\varepsilon fu = CE\ E_{fu}\ {*}$ εfu = 1x4300/240000 = 0.017917	εfu = 0.017917 sample 2
ACI-318-99 section 10.2.7.3	Calculation of ß1 $$\beta_1 = 0.85 - 0.05\left(\frac{f_c' - 4000}{1000}\right), \qquad 0.65 \le \beta_1 \le 0.85,$$	
	For batch 01 concrete, $ß_1$ = 0.85 – 0.05 ((41066/6.9)-4000)/1000 = 0.752	$ß_1$ = 0.752 (batch 01)
	For batch 02 concrete, $ß_1$ = 0.719	$ß_1$ = 0.719 (batch 02)
	Exiting strain on the soffit €bi Considering no initial strain when bonding FRP, $€_{bi}$ = 0	$€_{bi}$ = 0

Reference	Calculation	Output
ACI-440-2R-2 section 9.2.1	K_m, bond depended coefficient of the FRP system, **Sample 1,** $nE_f\,t_f$ = 1x640000x0.19 =121600 > 180000 $K_m = \frac{1}{60\varepsilon_{fu}}\left(\frac{90{,}000}{nE_ft_f}\right)$ = (1/60x0.004141)x(90000/121600) = 2.97 $\leq$ 0.9 Therefore, K_m = 0.9 **Sample 2,** $nE_f\,t_f$ = 1x240000x0.168 =40320 < 180000 $K_m = \frac{1}{60\varepsilon_{fu}}\left(1-\frac{nE_ft_f}{360{,}000}\right)$ = (1/60x0.017917)x(1-(40320/360000) = 0.825 K_m = 0.825	K_m = 0.9 sample 1 K_m = 0.825 sample 2
ACI-440-2R-2 section 9.2.2	Assuming depth to neutral axis C = 70mm, <u>The effective strain level in the FRP, ε</u> $\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{h-c}{c}\right) - \varepsilon_{bi} \leq \kappa_m\varepsilon_{fu}$	

Reference	Calculation	Output
	Sample 1,	
	ε_{fe} = 0.003X((150.29 – 70)/70 – 0 ≤ 0.9x0.004141	
	= 0.00344 ≤ 0.0037	ε_{fe} = 0.00344 sample 1
	Sample 2,	
	εfe ε_{fe} = 0.003X((150.47 – 70)/70 – 0 ≤ 0.9x 0.017917	ε_{fe} = 0.00345 sample 2
	= 0.00345 ≤ 0.01611	
	<u>The strain level in the non prestressed tension steel, εs</u>	
ACI-440-2R-2 section 9.2.2	$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d-c}{h-c}\right)$$	
	Sample 1,	
	ε_s = (0.00344+0)x((118-70)/(150.29-70)	ε_s = 0.002056 sample 1
	=0.002056	
	Sample 2,	
	ε_s = (0.00345+0)x((118-70)/(150.47-70)	ε_s = 0.002058 sample 2
	=0.002058	
	<u>The effective stress level in FRP & steel</u>	
ACI-440-2R-2 section 9.2.3	$f_s = \frac{\varepsilon_s E_s}{\gamma_m} \leq \frac{f_y}{\gamma_m}$	
	Sample 1,	
	$= \frac{0.002056 \times 200000}{1.15} \leq \frac{250}{1.15}$	
	f_s = 357.6 ≤ 217.4	f_s = 217.4 N mm^{-2} sample 1

Reference	Calculation	Output
	By iteration of C values,	
	The effective strain level in the FRP, ε	
ACI-440-2R-2 section 9.2.2	$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{h-c}{c}\right) - \varepsilon_{bi} \leq \kappa_m \varepsilon_{fu}$	
	Sample 1,	
	ε_{fe} = 0.003X((150.29 – 16.09)/16.09– 0 ≤0.9x0.004141 = 0.0250 ≤ 0.0037	ε_{fe} = 0.0037 sample 1
	Sample 2,	
	ε_{fe} = 0.003X((150.47 – 10.57)/10.57 – 0 ≤ 0.9x 0.017917 = 0.0397 ≤ 0.01611	ε_{fe} = 0.01611 sample 2
	The strain level in the non prestressed tension steel, ε_s	
ACI-440-2R-2 section 9.2.2	$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d-c}{h-c}\right)$	
	Sample 1,	
	ε_s = (0.0037+0)x((118-16.09)/(150.29-16.09) =0.002056	ε_s = 0.002809 sample 1
	Sample 2,	
	ε_s = (0.01611+0)x((118-10.57)/(150.47-10.57) =0.01237	= 0.01237 sample 2

Reference	Calculation	Output
ACI-440-2R-2 section 9.2.3	<u>The effective stress level in FRP & steel</u> $f_s = \dfrac{\varepsilon_s E_s}{\gamma_m} \leq \dfrac{f_y}{\gamma_m}$	
	Sample 1, $= \dfrac{0.002809 \times 200000}{1.15} \leq \dfrac{250}{1.15}$ $f_s = 488.5 \quad \leq 217.4$	f_s = 217.4 $N\ mm^{-2}$ sample 1
	$f_{fe} = \dfrac{E_f\, \varepsilon_{fe}}{\gamma_m}$ $= \dfrac{640000 \times 0.0037}{1.4}$ $= 1691.4\ N/mm2$	f_{fe} = 1691.4 $N\ mm^{-2}$ sample 1
	Sample 2, $= \dfrac{0.01237 x\ 200000}{1.15} \leq \dfrac{250}{1.15}$ $f_s = 2151 \quad \leq 217.4$	f_s = 217.4 $N\ mm^{-2}$ sample 2
	$f_{fe} = \dfrac{E_f\, \varepsilon_{fe}}{\gamma_m}$ $= \dfrac{240000 \times 0.01611}{1.4}$ $= 2761.7\ N/mm2$	f_{fe} = 2761.7 $N\ mm^{-2}$ sample 2
	Moment carrying capacity M_n,	
ACI-440-2R-2 section 9.6	$M_n = A_s f_s\left(d - \dfrac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(h - \dfrac{\beta_1 c}{2}\right)$ **Sample 1,** = 56.55 x 217.5(118-(0.752 x 16.09/2) + 1x9.5x1691.4x(150.29-(0.736 x 16.09/2) M_n = 56.55 x 217.5x112.08+1x9.5x1691.4x144.37 = 3.7 kNm	M_n = 3.7 kNm sample 1

Reference	Calculation	Output
	Maximum moment, M_{max},	
	M_{max} = 0.15W-0.04219w	
	Ultimate load, W	
	W = 0.10W-0.0456 w	
	W =(3.7 + 0.0456x0.1x0.1x2.4x9.81)/0.10	W = 24.47 kN sample 1
	= 24.47 kN	
	Sample 2,	
	= 56.55 x 217.5(118-(0.719 x 10.57/2) + 1x9.5x2761.7x(150.47-(0.736 x 10.57/2)	M_n = 4.8 kNm sample 2
	M_n = 56.55 x 217.5x114.11+1x8.4x2761.7x146.58	
	= 4.8 kNm	
	Maximum moment, M_{max},	
	M_{max} = 0.10W-0.0456 w	
	Ultimate load, W	
	W = (M_{max} + 0.0456w)/0.10	W = 32.09 kN sample 2
	=(4.8 + 0.04219x0.1x0.15x2.4x9.81)/0.15	
	= 32.09 kN	

In the experiment two types of CFRP materials and two batches of concrete were used. Those were CFRP sample 1, sample 2 and concrete batch 01, batch 02. Bonding agent for sample 1 CFRP and concrete batch 01 was epoxy and bonding agent for sample 2 CFRP, concrete batch 02 was cement grout. The properties of the samples are described in the Table 4.2. Eight beam samples were used for the experiment. The details of the test specimens are given in the Table 3.1.

According to design calculations, the ultimate flexural failure load of sample 1 CFRP and concrete batch 01 with epoxy adhesive is 24.47 kN. Similarly, the ultimate flexural failure load of sample 2 CFRP and concrete batch 02 with cement grout adhesive is 32.09 kN.

4.3 Comparison of theoretical results and experimental results

Figure 4.5 depicts comparison of theoretical and experimental results. In all specimens experimental strength gain is lower than the theoretical figures. However, theoretical and experimental strength gap is narrow in sample 2 CFRP specimens when compared to sample 1 CFRP and control beams. Primer applied sample 2 CFRP specimens showed slightly higher strength than sample 2 CFRP specimens strengthened without priming the substrate. When primer applied sample 2 CFRP specimens are anchored at both ends, there is a noticeable strength gain when compared to all other test scenarios.

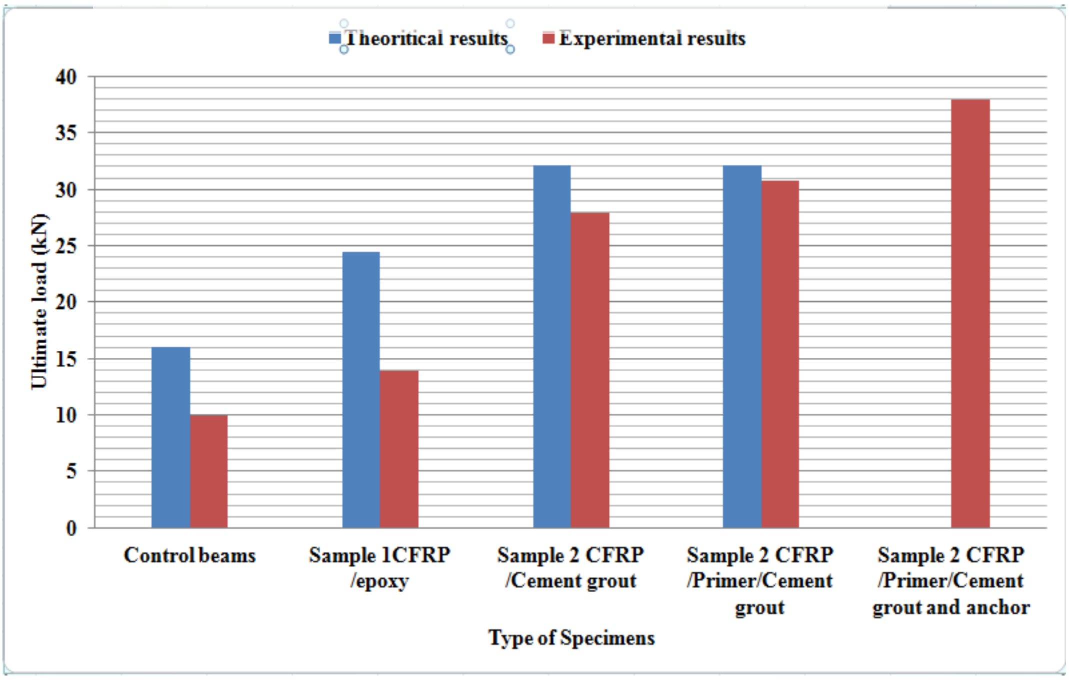


Figure 4.5: Comparison of theoretical results with experimental results

Theoretical ultimate load is calculated using some equations. There are some assumptions made while calculating ultimate load which might not be completely achieved in real world. One such assumption is top reinforcement is under no stress. However, in practice those assumptions will not hold due to various factors like environment conditions, material properties etc. Human errors are significantly contributed to practical results. All these reasons will lead to difference in practical and theoretical results.

Table 4.3: Analysis of Experiment results and theoretical results

Description of specimens	Bonding material type	CFRP type	Theoretical results		Average ultimate load from Experimental results (kN)	% strength gain of theoretical results relative to control beam	% strength gain of experimental results relative to control beam	% strength gain of experimental results relative to control beam(considering all CFRP are sample 2)	% deviation from the theoretical results	% deviation from the strength gain of beams which are used cement grout bonding material
			Ultimate Bending moment (kNm)	Ultimate load (kN)						
C	-	-	1.63	16.16	10.0		-		-37.6%	-
A-E	Epoxy	Sample 1	3.70	24.47	13.9	52.6%	39%	82%	-43.2%	-
B-C	Cement grout	Sample 2	4.80	32.09	27.9	100.1%	179%	179%	-13.1%	-13.1%
C-PC			4.80	32.08	30.8	100.1%	208%	208%	-3.9%	-4.0%
D-PC			-	-	37.9		279%	279%	-	18.1%

4.4 Results and discussion

It was found from the experiment results that load carrying capacity were increased by 39% for beams which used epoxy as bonding agent. It was also found that load carrying capacity was increased in the range of 179%-279% for beams which used cement grout as bonding agent and adding some improvements for the bond. When the surface was treated with primer, the strength improved than specimens without primer while cement grout act as a bonding agent. As per the experimental results when primer coated test specimens are anchored at both ends, the strength gain was significant.

The control beams failed due to flexure and its experimental strength gain was less than the theoretical figures by 37.6 %.Specimen type A-E failed with pattern of deboning and its experimental strength gain was 43.2 % less than the theoretical figure. The results show that although strength is enhanced due to CFRP strengthening, it is much less than the predicted capacity. In practice premature failure occurs in RC beams strength using CFRP with epoxy adhesive due to deboning.

Beam specimen types B-C, C-PC, and D-PC were strengthened using sample type 2 CFRP and cement grout boding agent. According to the theoretical calculation, the ultimate load capacity of the type B-C beam is 32.09 kN. As far as experimental results are concerned, the average ultimate load capacity was 27.9 kN and sample 2 CFRP that is 13.1% lesser than the predicted value. This indicates experimental result is much closer to theoretical predicted result. Even though situation has improved to previous test case, premature failure happened in this case too. The experimental result indicates the average ultimate load capacity of the type C-PC beam was 30.8 kN. When primer was applied on the tension face, load carrying capacity increased by 10.4 % compared to tension face without primer. Therefore, it is indicated that the primer has ability to improve adhesive property. This particular scenario, the ultimate load was increased by 37.9 kN. Considering the experimental results, it is about 35.8% strength gain when compared with beam specimen that is without ends anchored.

As depicted in Table 4.3 ultimate strength gain was significantly higher when cement grout was used as bonding agent. CFRP sample 2 used with cement grout as bonding agent had larger tensile strength than CFRP sample 1. The vast difference of ultimate failure load might be due bonding material and higher tensile strength of CFRP specimen.

4.5 Conclusion

This chapter mainly includes theoretical calculation of ultimate failure loads of un-strengthened concrete beams and strengthened concrete beams. The concrete structures were strengthened using externally bonded CFRP systems. Ultimate failures loads of test specimens were calculated using BS 8110 (1985) code of practices and ACI-440-2R design guide for design and construction.

From the comparison of theoretical and practical results, it was revealed that strength gain in beams which used cement grout as bonding agent is better than beams which used epoxy as bonding agent. When primer was applied on tension face of the beams which used cement grout as bonding agent, there was an increase of strength gain. Also, when both ends of the beams are anchored strength gain was significant high values compared to all other test scenarios.

CHAPTER 05: COMPARISON WITH PREVIOUS STUDIES

5.1 Comparison of strength gains with previous researchers

Results of previous studies collected from the literature survey are given in the Annexure B. The results are plotted in Figure 5.1. Results of the previous studies are plotted from 1 – 11 in Figure 5.1 and those are in order with Annexure B.

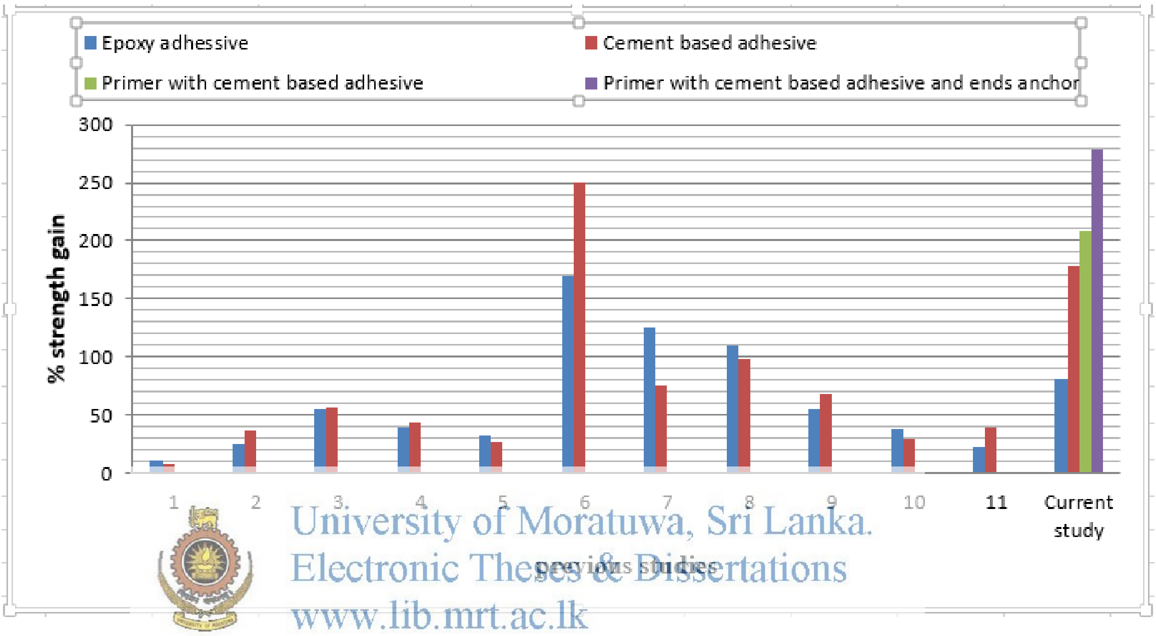


Figure 5.1: Results of the current study and previous studies of strength gains

As depicted in Figure 5.1, some studies show higher strength gain when epoxy used as bonding agent than cement grout. In some studies it is the other way.

5.2 Comparison of failure modes with previous researchers

Results of previous studies collected from the literature survey are given in Table 5.1. Numbers 1 to 11 in Table 5.1 are results of previous studies and those are in order with Annexure B. As depicted in Table 5.1 all beams strengthened using epoxy adhesive were failed due to de-bonding. Same result was observed in the current study too. In the current study when adhesive material was cement grout end delamination and de-bonding failure patterns have been observed.

Table 5.1: Results of the current study and previous studies failure modes

No	Observed failure mode	
	Using Epoxy	Using cement based adhesive
1	de-bonding	Flexural failure
2	de-bonding	Rupture of fibre
3	de-bonding	Flexural failure
4	de-bonding	Flexural failure
5	de-bonding	debonding
6	de-bonding	crushing of concrete
7	de-bonding	Flexural failure
8	de-bonding	Rupture of fibre
9	de-bonding	Rupture of fibre
10	de-bonding	Rupture of fibre
11	de-bonding	Flexural failure
Current study	**de-bonding**	**end delamination/ de-bonding**

5.3 Conclusion

As per previous studies, it cannot be concluded that strength gain was always higher when cement grout was used as a bonding agent. In some studies higher strength gain was observed when epoxy as bonding agent. According to the previous and current study, the common failure pattern is de-bonding for beam strengthened using epoxy adhesive. But, different failure patterns could be observed using cementanious bonding materials.

CHAPTER 06: CONCLUSIONS AND RECOMMENDATIONS

This research is based on flexural behavior of CFRP strength concrete beams. The experimental work was carried out to study on RC beam elements flexural strengthen with externally bonded CFRP and the suitability of using cement grout as bonding agent was investigated. Two other modifications have been carried out for strengthen beams with CFRP bonded with cement grout adhesive. Those were;

1. Strengthening RF concrete (primed) beams with CFRP, and use of cement grout as bonding agent.
2. Strengthening RF concrete (primed) beams with CFRP, and use of cement grout as bonding agent while both ends were anchored with two 'U' wraps.

Two type of CFRP samples were used for the experiment works that was sample 1 (having properties Ultimate tensile strength 2650N/mm^2, Modulus of elasticity 640kN/mm^2) and sample 2 (having properties Ultimate tensile strength 4300N/mm^2, Modulus of elasticity 240kN/mm^2). The sample 1 CFRP was used with beams which were bonded with epoxy and the sample 2 CFRP was used with beams which were bonded with cement grout.

5.1 Conclusions and Recommendations

CFRP fabric properly bonded to the tension face of RC beams can enhance the flexural strength substantially. Within the indicated scope of this investigation, the particular conclusions emerging from this study are summarized as follows:

1. The experimental results show that beams which used CFRP (sample 1) and epoxy adhesive exhibited an increase in flexural strength about 39 %, relative to control beam, for single layer CFRP. According to the theoretical calculation, the predicted strength gain by using sample 1 CFRP was 52.6%.That shows there is a significant effect on enhancement of flexural performance with CFRP. It was observed that failure mode was de-bonding.

2. When observing the experimental results, beams which used CFRP (sample 2) and cement grout adhesive exhibited an increase of flexural strength about 179 %, relative to control beam, for single layer CFRP. According to experimental results, there is a significant effect on flexural performance enhancement with CFRP on ultimate load capacity. It was observed that failure mode was end delamination.

3. When primer coated on tension face of the beams which used cement grout as bonding agent, the ultimate strength was increased by 208%, relative to control beam. This is about 29% increment with respective to non-primed beam strength using cement grout. That clearly implies the primer has ability to increase bond capacity of the cement grout bond. It was observed that failure mode was de-bonding.

4. When primer coated on tension face of the beams which used cement grout as bonding agent and both ends of the beams were anchored using ‘U’ wrapped showed increase of flexural strength about 279%, relative to the control beam. This is about 71 % with respect to end anchored beam with the same substrate condition. Therefore, it can be concluded the ends ‘U’ wrapped can effectively increase the load carrying capacity of the beams. It was observed that failure mode was de-bonding.

5. In this investigation CFRP strengthened beams demonstrated appreciable ductility when compared to the control beam. CFRP specimens used with cement grout as bonding agent had higher tensile strength which demonstrated higher ductility.

Finally, the experimental results have shown that the strengthening with CFRP sheets bonded with cement grout material enhances the flexural stiffness of the beam. Therefore, it can be concluded that when mortar (2:1 cement water ratio) was used as bonding material, it can effectively contribute to increase load capacity and ductility of the structural members. Results show that considerable composite action can be achieved using cement grout as bonding agent. In addition to that, the primer has ability to increase excellent bond properties of the cement grout that will further improve loading capacity of the beams. The proposed ‘U’ wraps at both ends are more effective method to enhance the strength capacity of the beams. It prevented the end deboning failure of CFRP sheet.

5.2 Further studies

1. Better flexural performance was shown when cement grout was used as bonding agent from the current study. Performance was further improved when primer

coated on bonding surface and ends are anchoraged. It is suggested to study shear stress and compression capacity with similar bonding agents and methods.

2. In the current study ends were anchored 75 mm from edge of the beam to test flexural strength gain. It is proposed to study optimum anchoring distance from edge of beam to test flexural strength, shear stress and compression capacity.
3. Finite element modeling of the system should be done for better behavioral understanding and for better predictability of results.
4. It is suggested to compare results of end anchored beams while using bonding agents as epoxy and cement grout.

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Appendix A: Details of flexural capacity enhancement of beams

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure	Observed failure mode
1.	Srisangeerthanan, S. (2013) “Investigation on alternatives to prevent debonding of reinforced concrete members”	Beam size 150 mmX 200 mmX750	Thickness= 0.19 mm, Tensile strength =2,600 N/mm^2 , weight of fabric= 200 g/m^2’uni directional , E = 6.44 X10^5 N/mm^2” G 30 concrete, f_y = 460 N/mm^2	29%	Flexure. debonding
2.	Anthony J. L., Lawrence, C, B. and David, W. S. (2004), “Flexural Strengthening of Reinforced Concrete Beams by Mechanically Attaching Fiber-Reinforced Polymer Strips” Journal of composites for construction volume 8(3), P 203-210	Beam size 304.8mm X304.8mm X 3658.6 mm,	Thickness= 1 mm, Tensile strength =3,600 N/mm^2 , uni directional , E = 1.52 X10^5 N/mm2,, G 30 concrete,	19%	Flexure debonding

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure	Observed failure mode
3.	Imam,M., A. Tahwia,A., Elagamy,A, and Yousef,M.(2013) "Behavior of Reinforced Concrete Beams Strengthened With Carbon Fiber Strips"	Beam size 120 mm X200 mm X 2300 mm,	Thickness= 0.13 mm, Tensile strength =3,500 N/mm^2 , weight of fabric= 220 g/m^2,uni directional , E = 6.44 X105 N/mm^2,, G 30 concrete, f_y = 400 N/mm^2	20%	Flexture. fiber separation
4.	Balamuralikrishnan, R. and Antony, C, J. (2009) "Flexural Behavior of RC Beams Strengthened with Carbon Fiber Reinforced Polymer (CFRP) Fabrics" The Open Civil Engineering Journal. Volume 3 (6). P 102-109	Beam size 150 mm X250 mm X 3000 mm,	Thickness= 0.30 mm, Tensile strength =3500 N/mm^2, weight of fabric= 200 g/m^2 uni directional, E = 1.55X10^5 N/mm^2, G 20 concrete, f_y = 512 N/mm^2	20%	Flexure

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure	Observed failure mode
5.	Davood, M., Seyed , M, S., Ardalan, H. (2012). “Experimental Study on the effectiveness of EBROG method for flexural strengthening of RC beams” Proceedings of the International Conference on FRP Composites in Civil Engineering	Beam size 120 mmX140 mmX1000 mm,	Thickness= 0.12 mm, Tensile strength =4100 N/mm^2 , uni directional E = 2.3X10^5 N/mm^{2,} G 30 concrete, f_y = 530 N/mm^2	52%	CFRP debonding
6	Alaa, M. and Tony, E M. (2012). “Bonding techniques for flexural strengthening of R.C. beams using CFRP” Journal of Ain Shams Engineering Volume 30 (9) P 30-36	Beam size 150 mmX300 mmX2400 mm	Thickness= 0.12 mm, Tensile strength =2600 N/mm^2 , uni directional E = 1.65X10^5 N/mm^2 G 20 concrete, f_y = 360 N/mm^2	12%	FRP debonding with concrete cover separation
7	Siddiqui, N, A. (2009). “Experimental investigation of RC beams strengthened with externally bonded FRP composites” Lathin American journal of solids and structures. Volume 6(10) P 343-362	Beam size 120 mmX140 mmX1000 mm,	Thickness= 1.0 mm, Tensile strength =846 N/mm^2 , uni directional E = 7.7X10^5 N/mm^2) G 35 concrete,f_y = 420 N/mm^2	23%	Debonding

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure	Observed failure mode
8	Riyadh Al-Amery.and Riadh Al-Mahaidi (2006) "Coupled flexural–shear retrofitting of RC beams using CFRP straps" International journal of composite structure, volume 75 (3), P 457–464	Beam size 260 mmX340 mm X2700 mm,	Thickness=1.40 mm, 76 mm wide Tensile strength =1,710 N/mm^2 , uni directional E = 2.15X10^5 N/mm^2) G 30 concrete, f_y = 504 N/mm^2	62%	Debonding,crushing of concrete
9	Dolawatte, N, N, W. (2013) "Study on use of Carbon fiber reinforced polymer (CFRP) for strengthening of reinforced concrete beams (RC)" A Thesis submitted in partial fulfillment of the requirements of IESL Engineering course part III: IESL Sri Lanka	Beam size 200 mmX150 mm X2000 mm,	Thickness= 1 mm, Tensile strength =834 N/mm^2 , uni directional, E = 8.2X10^5 N/mm^2, G 30 concrete, f_y = 490 N/mm^2	78%	Separation of concrete cover
10	Piyong, Y., Silva, P, F. and Antonio, N. (2008) "Flexural Performance of RC beams strengthened with prestressed CFRP sheets"	Beam size 768 mmX305 mmX6096 mm,	Thickness= 1 mm, Tensile strength =760 N/mm^2 , uni directional, E = 2.28X10^5 N/mm^{2,} G 20, concrete ,f_y = 414 N/mm^2	65%	Flexure.

Appendix B: Details of flexural capacity enhancement of beams using Epoxy and Cement based adhesive

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure		Observed failure mode	
				Using Epoxy	Using cement based adhesive	Using Epoxy	Using cement based adhesive
1	S.L. Sveinsdottir, "Experimental research on strengthening of concrete beams by the use of epoxy adhesive and cement-based bonding material"	Beam size 150 mmX 250 mmX2500	Tensile strength =2,500 N/mm^2 , E = 84 Gpa, G 35 concrete, $f_y = 460$ N/mm^2	11%	8%	debonding	Flexural failure
2	"Investigation on allternative bonding agents for CFRP concrete composites",S.R. Adhikarinayake,K.D.J.A. Gayan,N.G.T.T.Thathsarani,J.C.P.H.Gamage,UOM, Sri Lanka	Beam size 100mm X150mm X 600 mm,	Tensile strength =3,800 N/mm^2 , E =230 Gpa, G 30 concrete, $f_y = 460$ N/mm^2	26%	37%	debonding	Rupture of fibre
3	Hashemi S, Al-Mahaidi, "Cement based bonding material for FRP", 11th inorganic-bonded fiber	Beam size 120 mm X200 mm X 2300 mm,	Tensile strength =3,800 N/mm^2 , E = 230 Gpa, G 30 concrete, $f_y = 460$ N/mm^2	56%	57%	debonding	Flexural failure

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure		Observed failure mode	
				Using Epoxy	Using cement based adhesive	Using Epoxy	Using cement based adhesive
	composites conference, November 5-7, 2008 Madrid – Spain.						
4	Siavash Hasmi,Riadh Al Mahandi,May 2011, "Flexural performacne pf CFRP textile-retrofitted RC beam using cement based adhesive at high temperature"	Beam size 120 mm X180 mm X 1300 mm, ;	Tensile strength =3,600 N/mm^2, E= 230 Gpa, G 57 concrete, f_y = 460 N/mm^2	40%	44%	debonding	Flexural failure

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure		Observed failure mode	
				Using Epoxy	Using cement based adhesive	Using Epoxy	Using cement based adhesive
5	Siavash Hasmi,Riadh Al Mahandi,June 2011, "Experiment and finite element analysis of flexure behaviour of FRP-strengthened RC beams uisng cement based adhesive"	Beam size 120 mmX140 mmX1000 mm,	Tensile strength =3,600 N/mm^2 , E = 200 Gpa, G 38 concrete, f_y = 460 N/mm^2	33%	27%	debonding	debonding
6	Al-Abdwais, R. Al-Mahaidi, K. Abdouka, "Modified cement-based adhesive for near-surface mounted CFRP strengthening system", Fourth Asia-Pacific Conference on FRP in Structures, Melbourne, Australia, Melbourne, Australia, 2013	Beam size 75 mmX75 mmX200 mm	Tensile strength =1450 N/mm^2 , E = 135 Gpa, G 41 concrete, f_y = 460 N/mm^2	170%	250%	debonding	crushing of concrete

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure		Observed failure mode	
				Using Epoxy	Using cement based adhesive	Using Epoxy	Using cement based adhesive
7	Heshamdiab,Apri 2015,"Efficiency of cement based bonding agent for FRP sheets vs epoxy"	Beam size 100 mmX100 mmX500 mm,	Tensile strength =3800 N/mm^2 , E = 200 Gpa, G 20 concrete, f_y = 460 N/mm^2	125%	75%	debonding	Flexural failure
8	Thomas Blanksvärd & Björn Täljsten, "Strengthening of concrete structures with cement based bonded composites",	Beam size 180 mmX500 mm X4000 mm,	Tensile strength =3800 N/mm^2 , E = 284 Gpa, G 30 concrete, f_y = 460 N/mm^2	110%	99%	debonding	Rupture of fibre

No	Research and Author	Description of sample	Material Properties	Strength gain in flexure		Observed failure mode	
				Using Epoxy	Using cement based adhesive	Using Epoxy	Using cement based adhesive
9	E,Ferrier,A.Si Labri,J.F. Georging,J.Ambroise,April 2012,"New hybrid cement based composite material externally bonded to control RC beam cracking".	Beam size 150 mmX250 mm X2000 mm,	Tensile strength =2300 N/mm^2 , E = 130 Gpa, G 30 concrete, f_y = 460 N/mm^2	55%	63%	debonding	Rupture of fibre
10	Luciano Ombres,June 2011,"Debonding analysis of RC beams strength with FR cementanious mortar"	Beam size 150 mmX250 mmX2700 mm,	Tensile strength =5800 N/mm^2 , E = 270 Gpa, G 27 concrete, f_y = 460 N/mm^2	38%	30%	debonding	Rupture of fibre
11	Luciano Ombres,June 2011,"Debonding analysis of RC beams strength with FR cementanious mortar"	Beam size 150 mmX250 mmX2700 mm,	Tensile strength =5800 N/mm^2 , E = 270 Gpa, G 23 concrete, f_y = 460 N/mm^2	23%	40%	debonding	Flexural failure

Appendix C: Details of Cement adhesive mix ratios

	Research & Author	Cement bond						Test type
		material	Mix proportion (kg)				bond thickness(mm)	
			1	2	3	4		
1	S.L. Sveinsdottir, "Experimental research on strengthening of concrete beams by the use of epoxy adhesive and cement-based bonding material"	Sand	20250	20250	20250		10	flexure
		Water	3119	3153	3448			
		Cement	11250	11250	11250			
		Silica fume	1125	1125	1125			
		Omnicon(SP)	373	314	112.2			
		Fibers		106				
		Acryl			380			
		SP	1.30%					
2	"Investigation on allternative bonding agents for CFRP concrete composites",S.R. Adhikarinayake,K.D.J.A.Gayan,N.G .T.T.Thathsarani,J.C.P.H.Gamage,U OM,Sri Lanka	cement grout					3	flexure
		cement grout					6	flexure

	Research & Author	Cement bond						Test type
		material	**Mix proportion (kg)**				**bond thickness(mm)**	
			1	**2**	**3**	**4**		
3	Hashemi S, Al-Mahaidi, “Cement based bonding material for FRP”, 11th inorganic-bonded fiber composites conference, November 5-7, 2008 Madrid – Spain.	cement	888	813	776	613		flexure
		micro cement				153		
		water	426	406	310	427		
		Silica fume	754.8	691	659	651.5		
		SBR latex			194			
		Viscocrete5-500 (SP)	8.9	40.6	3.9	42.2		
4	Siavash Hasmi,Riadh Al-Mahandi,May 2011, "Flexural performacne pf CFRP textile-retrofitted RC beam using cement based adhesive at high temperature"	cement	674.3				20	flexure
		micro cement	168.6					
		water	354					
		Silica fume	84.3					
		Filler(Silica200G)	716.6					
		Viscocrete5-500 (SP)	75.9					

	Research & Author	Cement bond						Test type
		material	**Mix proportion (kg)**				**bond thickness(mm)**	
			1	**2**	**3**	**4**		
5	Siavash Hasmi,Riadh Al Mahandi,June 2011, "Experiment and finite element analysis of flexure behaviour of FRP-strengthened RC beams uisng cement based adhesive"	cement	674.3				20	flexure
		micro cement	168.6					
		water	354					
		Silica fume	84.3					
		Filler(Silica200G)	716.6					
		Viscocrete5-500 (SP)	75.9					
6	Al-Abdwais, R. Al-Mahaidi, K. Abdouka, "Modified cement-based adhesive for near-surface mounted CFRP strengthening system", Fourth Asia-Pacific Conference on FRP in Structures, Melbourne, Australia, Melbourne, Australia, 2013.	cement	674.3	674.3	674.3	674.3	4	pull-out
		micro cement	168.6	168.6	168.6	168.6		
		water	354	354	354	354		
		Silica fume	84.3	84.3	84.3	84.3		
		Filler(Silica200G)	716.5	716.5	716.5	716.5		
		Viscocrete5-500 (SP)	42.1	33.7	25,.3	16.9		
		Primer	227.4	151.2	101.1	88.6		

	Research & Author	Cement bond						Test type
		material	**Mix proportion (kg)**				**bond thickness(mm)**	
			1	**2**	**3**	**4**		
7	Heshamdiab,Apri 2015,"Efficiency of cement based bonding agent for FRP sheets vs epoxy"	Cement	888				One layer	flexure
		water	426					
		fine sand	755					
		SP	8.9					
8	Thomas Blanksvärd & Björn Täljsten, " Strengthening of concrete structures with cement based bonded composites",	mortor					One layer	flexure
9	E,Ferrier,A.Si Labri,J.F. Georging,J.Ambroise,April 2012,"New hybrid cement based composite material externally bonded to control RC beam cracking".	Cement	742				35	flexure
		Silica fume	44					
		Basalt sand	698					
		sand	523					
		water	222					
		SP	22					
		Accelataor	10					
		Matalic fiber	131					
		Welam gum	0.262					

	Research & Author	Cement bond						Test type
		material	Mix proportion (kg)				bond thickness(mm)	
			1	2	3	4		
10	Luciano Ombres,June 2011,"Debonding analysis of RC beams strength with FR cementanious mortar"	mortor(compressive strength 30.4 Mpa)					22.5	bending test
11	Luciano Ombres,July 2011, "Flexural analysis of RFC beams strength with the cement based high strength composite materials"	mortor(compressive strength 29 Mpa)						bending test

Appendix D: Details of testing data

Applied loads (MT)	Deflection of beam specimens (Dial gauge readings)									
	C1	C2	A-E1	A-E2	B-C1	B-C2	C-PC1	C-PC2	D-PC1	D-PC2
0	0	0	0	0	0	0	0	0	0	0
0.2	30	40	25	11	15	10	20	4	8	22
0.4	50	62	35	22	29	30	27	12	15	33
0.6	70	76	59	42	37	38	35	20	21	42
0.8	110	100	78	60	43	43	37	26	28	50
0.98	198									
1		200	96	80	48	46	47	32	30	56
1.02		202		94						
1.2			125		52	50	51	38	33	62
1.4			166		56	53	55	43	38	68
1.48			198			55				
1.6					60	58	60	46	40	71
1.8					65	62	67	50	44	75
2					140	67	73	134	48	126
2.2					148	136	118	158	111	142
2.4					160	218	150	180	135	158
2.46						220				
2.6					190		235	205	150	182
2.8					205		268	248	182	198
3					230		302	300	208	214
3.12					232					
3.16							305			
3.2									240	236
3.4									280	248
3.6										273
3.8										296
4										328