INVESTIGATION ON STRUCTURAL BEHAVIOURS OF RCC BEAMS STRENGTHENED WITH CFRP AND FERROCEMENT

A PROJECT REPORT

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IN

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SUBMITTED BY

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CANDIDATE’S DECLARATION

I, Abdisalan Ahmed Abdirahman ,Roll No-2k20/STE/26,student of M.Tech (Structural Engineering), hereby declare that the project Dissertation title “**Investigation on Structural behaviors of RCC beams Strengthened with CFRP and Ferrocement”** which is submitted by me to the Department of Civil Engineering , Delhi technological university, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis of the award of any Degree, Diploma or other similar title.

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CERTIFICATE

I hereby certify that the project Dissertation titled "**INVESTIGATION ON STRUCTURAL BEHAVIOURS OF RCC BEAMS STRENGTHENED WITH CFRP AND FERROCEMENT**" submitted by Abdisalan Ahmed Abdirahman, Roll No - 2K20/STE/26,Delhi technological university, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried To the best of my knowledge, this title has never been used in part or in full for any degree or diploma at this or any other university.

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ABSTRACT

Despite the long service life of RCC structures, external factors can cause deterioration, resulting in a loss of load-bearing capacity and the formation of significant visible cracks. As a result, maintaining the safety and stability of reinforced concrete structures necessitates repair and strengthening. Because retrofitting can be a cost-effective way to replace RCC structural elements, the main purpose of this experiment is to proof that CFRP and Ferrocement strengthening techniques are effective strengthening option.

For this purpose, 12 RCC beams of 700 X 150 X 160 mm, using M20 grade of concrete and Fe415 steel bars, were cast and load tested in both flexural and shear failure modes , with one point loading test for flexural failure mode and two points loading test for shear failure mode. The structural behaviour of reinforced concrete beams strengthened using Carbon Fibre Reinforced Polymer (CFRP) and Ferrocement laminates has been investigated and compared to control beams and also, the load carrying capacity predictions of RCC beams before and after strengthening was carried as per ACI-318M-11,ACI-440-2R-08, ACI 549 for Control, CFRP strengthened and ferrocement strengthened beams respectively.

As a result, it is found that beams strengthened with CFRP for flexure has increased their load-carrying capacity by 20.28% at the final stage comparing to control beams, whereas the deflection and crack width was reduced by 18.75% and 31.5% respectively. However flexural load-carrying capacity was also increased up to 9.5%at the ultimate stage for the beams strengthened with Ferrocement compared to control beams, similarly the deflection and the crack width was also reduced by 6.25% and 16.85% respectively .

Meanwhile, beams reinforced with CFRP for shear has also increased their load-carrying capacity by16.3% at the ultimate stage. but, deflection and crack width was reduced by 20% and 49% respectively .On the other hand, beams strengthened with Ferrocement for shear the load-carrying capacity has increased by 8.2% at ultimate stage comparing to control beams ,as well deflection and crack width was reduced by 13.33% and 38.3% respectively .

The experimental investigation reveals that the strengthened beams with CFRP are better than ferrocement in both and flexural, shear behaviours as well as deflection and crack width control. This investigation also proofs the effectiveness of both CFRP and ferrocement for improving the overall structural behaviours of RC beams.

Keywords: CFRP strengthened beam, cracking load, flexural strength, crack width, ultimate load, Ferro-cement strengthened beam, compressive strength, flexural and shear failure mode.

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I would also like to extend my appreciation to my family for their incessant encouragement and support in all my life and especially at this moment for the completion of this course abroad away from home. These acknowledgments would be incomplete without mentioning my heartfelt appreciation to my beautiful wife **Zahra Abdinasir** for her love, support and understanding .This gratitude also extends to my friends and to the staff of Delhi technological university for their valuable support.

ABDISALAN AHMED ABDIRAHMAN

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Chapter -1

Introduction

1.1 General

It is well known that concrete is the main construction material available in the construction industry mainly due to its durability and flexibility behavior. However, concrete alone may be able to sustain compression force and can resist a small amount of tension force which is neglected mostly in the actual design .for this reason ,concrete needs extra support to resist the tension forces. After the industrial revolutions, steel become major player in construction industry because of its' light weight and higher load carrying capacity compared to concrete.

As a result, concrete and steel have been merged together, so that the structure can withstand the compression and tension as well, so the term reinforced concrete is referred for this process. But RCC structures often exercise distress and damage even before their expected service period because of a number of factors including poor construction, changes in its use and the use of codal material, overloading, earthquakes, explosions, wear and wear, flux and fire, etc.

However, structural maintenance becomes crucial if one of the following scenarios is encountered, but difficulty in strengthening of concrete structures is deciding on a strengthening strategy that will not only improve the structure's strength and serviceability but also taking into account constraints such as constructability, construction operations, and budget.

- If the structure cannot withstand loads that were not foreseen in the initial design, strengthening may be required. As a result of changes in site location, wind and seismic loads may need to be strengthened or blast loading resistance may need to be improved.
- If additional load is to be applied on the structures for instance attaching new mechanical equipment to the member or increasing the load carrying capacity of the member due to adding an addition floor to the structure .
- If the structural member cannot withstand its design load due to deficiency in the design or improperly application of the design due to misinterpretation of the design or working drawings.

Reinforced concrete (RC) structures are prone to severe strength and serviceability loss. RC has remained a popular building material due to ever-improving strengthening and retrofitting procedures. One of the most recent breakthroughs in RC retrofitting is the bonding of Fiber Reinforced Polymers (FRP) to the RC member. FRP composites are made up of a polymer resin matrix embedded in a large number of small, continuous, directed non-metallic fibres with advanced characteristics.

1.2 Need for Strengthening and Rehabilitation

Strengthening is a technical aspect of rehabilitation that relates to the identification of a structure that is deteriorated in appearance and serviceability, either partially or completely.

The term "rehabilitation" refers to the process of returning a structure to its previous state of service at a minimal cost. The rehabilitation plan calls for restoring the structural system as closely as feasible to its original configurations. The damaged structures must be brought back into line, and they must be strong enough to be placed back into service without affecting their safety and utility.

The primary goals of strengthening work would be

- To prevent dangerous development, restore the structure's integrity, and provide adequate protection.
- To increase the functional utility and service life of a structure.
- To relieve discomfort caused by the visible cracks and eliminate faults that poses a threat to life and compromises a structure's durability.
- To improve the visual look.

1.2.1 Damage assessment

Damage is described as a change in structural performance that can be seen as discrete cracks or the emergence of a weak zone, resulting in a drop in stiffness.

Damage can take any physical form, and the rate at which it grows under a particular sort of loading which is depending on a variety of parameters such as the type of material, the position of the loading, and the intensity of the loading.

The research of why structures are harmed is mostly a question of acquiring information through observation, examining previous records, conducting preliminary testing, and then interpreting the results. A systematic investigation of concrete structures is critical in determining the cause of damage, assessing the status of the structures in their damaged state, and formulating restoration suggestions.

However, damage may occur due to over loading and the overloading arises when the load on structural members exceeds the design loads, causing basic cracking signs. Spilling and disintegration follow as a result. A variety of variables commonly lead to degradation of reinforced concrete structures. Physical injury, chemical attack, and material degradation can all cause it when exposed to a harsh environment. Fire damage, explosion damage, impact damage, and damage from natural calamities such as floods, cyclones, and earthquakes in isolation or in combination can cause physical damage to reinforced concrete.

Alkali-silica reaction, alkali-carbonate reaction, carbonation, sulphate attack, and steel corrosion are chemical causes of concrete deterioration. High structural stress, heat stress, shrinkage, and poor material quality are some of the additional elements that contribute to concrete degradation.

1.2.2 Criteria for selection of strengthening materials

Among the most crucial jobs in assuring a long-lasting and reliable repair is choosing the right strengthening material. A deep study of the actual cause of distress is required for a prefect repair system and widely understanding the process of deterioration of materials like concrete and other supplementary materials such as plastics and resins under service conditions is also important. Before settling on a repair material, it should be checked if the selected material for the repair has a chemical component, the availability of relevant materials, equipment, and skilled labor must be checked. In addition, the literature frequently emphasizes the material's composition rather than its performance feature.

A Variety of approaches are available for strengthening and rehabilitating structural damage.

These approaches include

- The use of steel plates or FRP in damaged areas to increase the structure's load carrying capacity.
- The corroded steel bars and damaged concrete are removed and replaced with new materials of the same type.
- Epoxy mortar can be used alone or in conjunction with the two processes indicated above.
- Using Ferro-cement in the damaged area to restore the structure's performance

1.3 Rehabilitation and Retrofitting of an Existing member

1.3.1 GFRP and CFRP strengthening of members

Continuous or non-continuous strong fibres are surrounded by a matrix material in fibre reinforced polymers. The matrix is responsible for distributing the fibres and transmitting the load to them. The bond between the fibres and the matrix is formed during the application phase, and the matrix is hardened in the final stages of fabrication to form a composite material. The quantities of reinforcements and matrix, as well as the form, determine the properties of composite materials .The resin system binds fibres together and transfers loads to the remainder of the structure through the fibres. It protects the composite structure against impact, abrasion, and corrosion in addition to binding it together.

1.3.1.1 GFRP Strengthening

Because of their high surface area to weight ratio, glass fibres are helpful. In comparison to carbon fibre, glass fibre can be stretched further before breaking. Mats, insulation, reinforcement, heat resistant fabrics, corrosion resistant fabrics, and high strength fabrics are the most common applications for ordinary glass fibre. FRP enables the alignment of thermoplastic glass fibres to fit individual design programmes. The direction of reinforcing fibres can improve the polymer's strength and resistance to deformation. When the polymeric fibres are parallel to the force being exerted, they are strongest and most resistant to deforming forces, and when they are perpendicular, they are weakest.

However, high strength, light weight, good resistance to sea water, chemical environment, and durability are all advantages of GFRP. It may be moulded into any complex shape and requires little upkeep. For the building of domes, fountains, columns, beams, balustrade, panels, sculpture, facades, cornice, porticos, and roofs, GFRP can be utilised for both interior and exterior applications with fittings in a range of shapes, styles, and textures. Figure 1.1 and 1.2 shows the applications of GFRP .

Fig 1.1 strengthening of existing beam with GFRP

Fig 1.2 strengthening of existing column with GFRP

1.3.1.2 Carbon Fibre Reinforced Polymer (CFRP)

Carbon Fibre Reinforced Polymer (CFRP) has grown in popularity as a structural engineering material during the last several decades. It's a composite that's extremely strong, light, and pricey material. It has also proven to be cost-effective.

Concrete, masonry, steel, cast iron, and timber are all examples of applications. CFRP can be engaged in this industry for both retrofitting and new construction. Strengthening of an existing building or repairing a damaged structure are two options. Instead of steel reinforcement, it can be utilized as an alternative. CFRP has a major effect on strength and a modest increasing stiffness. This is due to the fact that the material utilized in this application is usually quite robust. However, it is not extremely stiff.

By covering textiles or fibres on the portion to be retrofitted, CFRP can also be used to improve the shear capacity of reinforced concrete. Covering around the structural members with CFRP as shown in fig 1.3,1.4 and 1.5 can also improve ductility by enhancing the section's ability to sustain under seismic loads. Wrapping beams and columns in these FRPs at their respective sensitive zones has resulted in a better flexural and axial capacity respectively.as well the control of the CFRP cover increases the concrete's compressive load resistance as per many practical researches carried out.

Fig 1.3 Strengthening of existing beam with CFRP

Fig 1.4 Strengthening of existing column with CFRP

Fig 1.5 Strengthening of existing slab with CFRP

1.3.1.3 Techniques for FRP application

1.3.1.3.1 Basic techniques

Manual placement of either wet lay-up (also known as hand lay-up) or prefabricated systems utilizing cold cured adhesive bonding is the most often used FRP strengthening method. As much as possible, the external reinforcement is attached to the concrete surface with the fibres roughly parallel to the major tensile stress direction. Figure 1.6 shows how the manual lay-up and prefabricated systems are generally used.

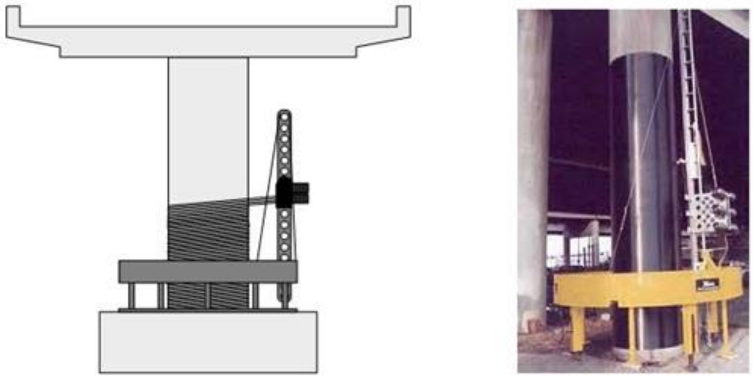
(a) (b)

Fig 1.6 Basic FRP application

(a)Hand lay-up of CFRP sheets on beam. (b) Application of prefabricated strips on slabs

1.3.1.3.2 Automated wrapping

The FRP strengthening technology of automated tow or tape winding was initially developed in Japan in the early 1990s, and then in the United States a few years later. Figure 1.7 shows the procedure, which involves a robot winding wet fibres at a little angle around columns or other structures (such as chimneys, as has been done in Japan). Apart from high quality control, the technique's main advantage is its speed of installation.



(a) (b)

Fig 1.7 Automated RC column wrapping.

(a) Schematic. (b) photograph of robot-

1.3.1.4 Fibers Used in FRP's

Carbon, aramid, and glass fibers are the three most popular forms of fibres used in civil engineering. Carbon and glass fibres are the most common fibres used in structural engineering, with carbon fibres accounting for over 95% of the total (H. Nordin, 2005). Carbon fibers are utilized to strengthen structures in bending utilizing prestressed FRPs because of their relevant features such as high strength and stiffness. Table 1.1 lists the typical features of the various fibers.

Table 1.1 Typical Properties of Different fibers

Material	Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Density (Kg/m^3)	Modulus of elasticity to density ratio(Mm^2/s^2)
Carbon	2200-5600	240-830	1800-2200	130-380
Aramid	2400-3600	130-160	1400-1500	90-110
Glass	3400-4800	70-90	2200-2500	31-33
Steel	280-1900	190-210	7900	24-27

1.3.1.4.1 Carbon Fibers

Carbon fibres have a high strength-to-weight ratio, as well as a high stiffness-to-weight ratio. Their thermal expansion is minimal, but their electrical conductivity is high. Carbon fibres are classified into several categories based on their manufacturing process:

- High-strength (HS).
- High-modulose (HM).
- Ultra-high-modulose (UHM).

Higher stiffness carbon fibres have lower tensile strength, and vice versa. CFRP materials must be handled with attention since UHM carbon fibres are extremely brittle. Carbon fibres are the stiffest and strongest strengthening fibres for polymer composites. They are chemical, UV light, and moisture resistant, they are also creep and fatigue resistant. As a result, carbon fibres are incredibly strong and have excellent mechanical properties.

Galvanic corrosion can occur when electrically conductive fibres come into contact with metals. Surface treatment is generally required because resins have a hard time wetting the fibres. In this case, carbon fibres are typically given an epoxy size treatment, which protect the fibres from abrasion (better handling) and offers a good epoxy matrix interface. (Stijn Matthys,2000).

1.3.1.4.2 Glass fibers

Glass fibres are made by extruding plastic into molten glass and stretching the fibres, which are primarily silicon oxide with trace amounts of other oxides. Glass fibres are typically coated with a sizing agent shortly after manufacture because they are very surface active and hydrophilic. The smaller size helps with polymer matrix coupling and decreases abrasion damage. Glass fibres have a high tensile strength, high electrical resistivity, strong heat resistance, and are very inexpensive. In the presence of moisture, acid, or alkaline solutions, glass fibres are known to deteriorate. They also show substantial creep or stress fracture behaviour, which implies that when repeatedly stressed, tensile strength decreases.

1.3.1.4.3 Aramid fibers

The first prestressing tendons were made from aramid fibres in the 1980s, which were marketed under the names Kevlar and Tarpon. They are now produced by a small number of companies. Aramid fibres have a high toughness and are not as brittle as carbon and glass fibre in their non-composite form, but they have other flaws. The fibres are less appealing for structural engineering because of their comparatively expensive price, limited compression and shear strength, difficulty in manufacturing, and low resistance to ultraviolet (UV) light and moisture (Martin Fornander, 2013).fig 1.8 shows the stress-strain behaviurs of different FRP and reinforcing steel.

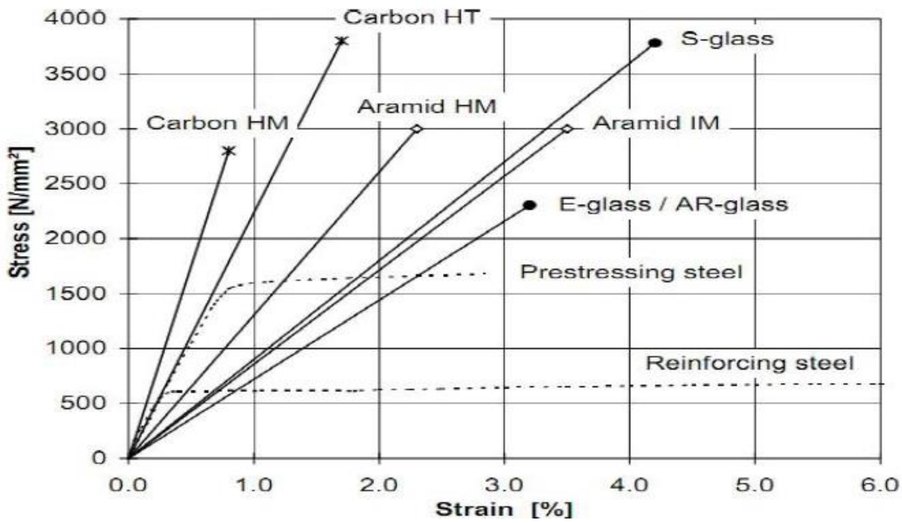


Fig.1.8.Tensile stress-strain behavior of reinforcing materials

1.3.2 Ferrocement for Structural Strengthening

Ferro-cement is defined by the ACI committee 549 (1993) as a type of thin-walled reinforced concrete structure in which cement motor is reinforced throughout the matrix with layers of continuous and relatively small diameter mesh. The important part of ferro-cement that gives it a significant advantage over reinforced concrete in certain situations is that the ferrocement components go through under stress functioning nearly as a relatively homogenous. It has greater crack resistance because of its closely spaced tiny diameter reinforcements. Due to its exceptional crack resistance combined with high strength, durability, ability to cast into any shape, quick constructions without heavy equipment, small extra self-weight they impose, and considering the economic aspects of rehabilitation, ferrocement demonstrate to be a cost effective solution for rehabilitation and applications.

Ferrocement reinforcement can be made into thin bands or sections ranging in thickness from 15 to 25 mm. Over the top layers of reinforcement, only a thin mortar cover was used. Unlike typical concrete, ferrocement can be constructed into the desired shape as shown in figure 1.9.

Ferro-cement has a very high tensile strength to weight ratio and superior cracking behaviour when compared to traditional reinforced concrete.

Fig 1.8 Ferro cement application

1.4 Problem Formulation

At the design stage of any project there shall be forecasting techniques to estimate the lifetime of the structure and doing so, the main target of the design shall be serving for that. However, there can be a lot of obstacles in order to achieve the stipulated life time such as improper design, faulty construction, change in building usage, change in codal provisions, overloading, earthquakes, explosions, corrosion, wear and tear, flood, fire, and so on. So the structural elements should be retrofitted for its ultimate and serviceability requirements. The Serviceability particularly deflection control is difficult to achieve with FRP due to the material's low stiffness. Having enough FRP application to meet the serviceability criterion is also an economic concern. But ferrocement is an appropriate choice, if the economic side is gives the consideration more than the safety. The importance of the structure to be retrofitted has also major role for deciding, if CFRP or ferrocement to be used as material for strengthening.

CFRP and ferrocement is applied to previously deformed structures in flexural and shear for retrofitting of RC beams and bridges. Because no formwork or up-lifting jacks are required, there is a potential that the deflected beams will remain deflected after being refitted for flexure or shear. The strengthened beams has been shown to support a large amount of moment and shear after repair, but the deflection found during the procedure should be calculated ahead of time to ensure compliance with the serviceability limit states.

1.5 Scope of investigation

Many different forms of damage and discomfort can occur in reinforced concrete structures. However, the scope of this research is confined to reinforced concrete beam damage against overloading only. Various methods might be used to rehabilitate the damaged buildings to a desirable level of performance at a low cost. But, this investigation is focusing on using CFRP laminate and ferrocement laminate for strengthening of reinforced concrete beams. Based on the performance of reinforced concrete beams, a comparison of the two approaches has been conducted. An experimental investigation on reinforced concrete beams was intended to attain this goal. Because ferrocement laminate and CFRP laminate are highly ductile materials with a better crack arrest mechanism, the depth of cracks was decreased and deflections were reduced while restoring the load carrying capability of the damaged beams after rehabilitation.

1.6 Objectives of investigation

The main objective of this investigation is to study

- ❖ The structural behaviour of RC beams before and after strengthening
    - flexural and shear behaviour of the beams before and after strengthening
    - to get load –deflection curve before and strengthening
- ❖ The effectiveness of CFRP laminates for improving the load carrying capacity in both flexure and shear.
- ❖ The effectiveness of CFRP laminates for cracks propagation and their width reduction
- ❖ The effectiveness of Ferrocement laminates for improving the load carrying capacity in both flexure and shear.
- ❖ The effectiveness of Ferrocement laminates for crack width reduction
- ❖ The failure mechanisms of the beam before and after strengthening

1.7 Methodology

In order to achieve the main objectives of this investigation, these following tasks have been carried out.

- Evaluating the mechanical properties of the RC beam materials component.
- Testing the control beams before strengthening.
- Proposing adequate techniques for CFRP structural strengthening application.
- Recommending technical applications of Ferrocement in strengthening framework
- Testing of the strengthened beams after supervised strengthening of the beams to increase the flexural and shear capacity of the beams.
- Investigating the failure mechanisms of the beam before and after strengthening.
- Exploring the structural behaviour of beam before and after strengthening
- Preparing a report about the performance of the CFRP and ferrocement strengthened RC beams used and their limitations.
- Demonstrating the effectiveness of CFRP and Ferrocement as a strengthening material and present their ability to improve the beam's load carrying capacity.
- Demonstrating the effectiveness of CFRP and Ferro cement in crack controlling

Chapter-2

Literature review

2.1 Historical Development of FRP’s

FRP reinforcement research and development began in Europe, as well as North America and Japan, in the 1970s. FRP systems were developed in Europe as an alternative to steel plate bonding. The use of adhesive resins to bond steel plates to the tension zones of concrete members has been proved to be a practical technique for enhancing their flexural strength. Many bridges and structures all around the world have been strengthened using this method. Because steel plates can corrode, the link between the steel and the concrete can deteriorate.

Several Scholars have pointed at FRP materials as substitute to steel as the steel is very tough and need the use of heavy machinery.

A commercial application of FRP materials over the last 50 year has advanced at a remarkable pace. Glass fibre goods account for 70% of all glass fibre output. FRP has a higher toughness than steel and is substantially lighter. The weight of a jet can be reduced by using it as a fuel tank and pipe. FRP tanks were used by astronauts who landed on the moon. They also carry little oxygen cylinders made of fiberglass reinforced polymers. FRP processing is simple, and the completed goods are stainless, requiring no painting. China has used FRP extensively in the manufacture of small-scale motorboats, lifeboats, yachts, and automobiles, saving a significant amount of steel.

Due to the substantial disadvantages of steel plates, such as durability and the additional maintenance required for new steel plates, such as surface preparation, painting, and regular inspections, the use of FRPs in strengthening applications was suggested as a replacement for steel plates. This traditional method's costs and hassles are increased by the additional maintenance.

Furthermore, due to the material qualities of steel, there are various practical issues with the process. The high density of steel makes it difficult to handle the plates on the installation site, necessitating the use of large lifting equipment.

The use of FRP materials for retrofitting concrete structures was first documented in Germany in 1978. (Wolf and Miessler, 1989). Switzerland was the first country to apply frp systems on reinforced concrete bridge for flexural strengthening. (Meier 1987; Rostasy 1987). In the 1980s, FRP systems were first used to provide additional confinement to reinforced concrete columns in Japan (Katsumata 1987). Since the 1930s, scientists in the United States have been attracted in fiber-based strengthening for concrete structures.

2.2 Development of CFRP strengthened structures

Fiber reinforced plastic (FRP) is used as an external strengthening in the retrofitting and rehabilitation of reinforced concrete (RC) structural elements.

The latest advancements in structural strengthening materials, technologies, and systems have been incredible. Fiber reinforced polymer (FRP) composites are one of today's cutting-edge approaches, generally considered by structural engineers as "new" and highly promising building materials. Furthermore, current research has concentrated on the use of continuous fibre based textiles in conjunction with mortars (rather than resins, as in FRP), resulting in the development of so-called textile reinforced mortars (TRM). "Continuous fibre composites," "advanced composites," or simply "composites" are terms that can be used to describe both FRP and TRM materials.

2.2.1 Recent Works on CFRP

Sharif et al. (1994) proved the viability of employing externally bonded GFRP plates to strengthen structurally damaged concrete beams. The realization of the strengthened beams' full flexural capability was their primary concern. They devised a theoretical model for estimating flexural strength and plate separation stress, which they compared to experimental results. The flexural capacity of the reinforced beams is determined using a basic flexural theory based on the ACI ultimate strength approach that takes strain hardening into account. They assumed that the flexural strength and strain at the extreme bottom fibre were calculated using compatibility and equilibrium equations and trial and error processes

Kim and Sebastian (2002) evaluated the externally strengthened beams were using CFRP plates. They studied at the plate-to-concrete binding behaviour in FRP plated concrete beams that were loaded for a brief time. The majority of bond failures occurred at a distance of 6 mm from the plate adhesives to the concrete layer. The degree of modelled corrosion of the steel bar rose by 57% to 78%, followed by a loss of rebar area over a short distance around the mid-span and a 50% reduction in the flexural crack-bond failure load.

Hsu et al. (2003) investigated the flexural behaviour of RC beams strengthened with CFRP sheets on the tension face of the beams using anchorage at the ends of CFRP strips. They tested reinforced beams that were constructed as under and over reinforced beams. Strengthened under reinforced beams had appropriate flexural capacity and ductility, while strengthened over reinforced beams had little improvement in comparison to strengthened under reinforced beams.

Kutarba et al. (2004) investigated the flexural strength of corrosion-damaged and CFRP-sheet-strengthened beams. After applying a tiny quantity of flexural loading to create concrete cracking, salt solution was allowed to seep into the beams, corrosion was induced to embedded reinforcement. For 28 weeks, a continuous current of 5 V was used to speed up the corrosion rate in the reinforcement. To restore its original shape, the deteriorated cover concrete was eliminated and replaced with new concrete. Then, for around 22 weeks, 1 mm thick CFRP laminates were bonded and corroded. They discovered that the load carrying capacity of beams was reduced by 9 to 12 percent in the post strengthened specimen. They also discovered that corrosion of reinforcement reduces beam stiffness, while CFRP laminated beam stiffness improves. Since there was less chloride diffusion in CFRP laminated beams, the corrosion rate was reduced.

Hedong Niu and Zhishen Wu (2006) used nonlinear fracture mechanics-based finite element analyses to investigate the impact of interface bond characteristics on the performance of FRP strengthened reinforced concrete (RC) beams in terms of structural crack growth, interface stress transfer, and failure mechanisms. They determined that low stiffness may assist in the spread of more equal stresses in both steel and FRP sheets, reducing local stress concentrations and the danger of debonding in practise.

Tamer EI Maaddawy et al. (2007) presented the findings of an experimental investigation aimed at determining the performance of reinforced concrete beams strengthened with carbon fibre reinforced polymer (CFRP) sheets in corrosive environments. The deflection capability of the beams declined as corrosion continued after retrofitting, according to the authors. The repaired beams had a deflection capacity that was roughly 45 percent lower than the control beam on average.

Yost et al. (2007) studied the properties of steel and FRP reinforcement ratios On twelve full-scale concrete beams strengthened with NSM (Near-Surface- Mounted) carbon FRP (CFRP) strips. When CFRP was used, they discovered a significant increase in yield and ultimate strengths, predictable nominal strengths , failure modes, and effective force transfer between the CFRP, epoxy grout, and surrounding concrete, as well as a decrease in both energy and defection ductility.

Aouicha Bedday, Benjeddou, Omrane, Mongi Ben Ouezdou (2007). This research focus on two types of beams: control beams (no CFRP laminates) and damaged and subsequently repaired beams with differing amounts of CFRP laminates using various criteria (damage degree, CFRP laminate width, concrete strength class).. Four-point bending is used to evaluate all specimens over an 1800 mm span. The experiments took place in a displacement-controlled environment. The degree of damage is the characteristic that has gained the greatest attention in this experimental investigation (ratio between pre-cracked load and load capacity of control beam). Externally bonded CFRP laminates were used to successfully restore damaged RC beams with varying degrees of damage. Peeling off and interfacial debonding were identified as failure modes. The laminate width is the sole determinant of these failure scenarios.

Lijuan Li et al (2008). Investigated the performance of FRP-strengthened RC beams coupled with CFRP and GFRP sheets .They examined at three different kinds of beams: control, polypropylene fibre strengthened concrete, and hybrid fibre reinforced concrete having both polypropylene and steel fibres. A single layer of CFRP sheet, a single layer of GFRP sheet, and a bi-layer of GFRP and CFRP sheet were used to reinforce the beams. The three strengthening processes had different effects on beam stiffness and load carrying capacity, they discovered. The CFRP sheet strengthened beams enhanced ultimate load carrying capacity and rigidity in fibre strengthened concrete beams. Beams reinforced with GFRP sheets had a lower load carrying capacity and were more ductile, but beams reinforced with CFRP and GFRP sheets were stiffer and stronger.

Sarah Orton and James (2009 discovered that forcing hinging to occur at locations with greater rotational ductility allowed them to reach the required load to resist total collapse while using far less CFRP material) In an experiment to investigate the strengthening of the negative moment region . The flexural strengthening technique proved successful in achieving the required load to survive progressive collapse at a reduced level of deformation, but at the cost of significantly greater CFRP. It was discovered that using CFRP to provide continuous strength in a concrete beam can be effective, though it may or may not be sufficient to prevent gradual collapse.

Antonio De Luca and Antonio Nanni (2011) used a single parameter methodology For forecasting the stress-strain behaviour of FRP constrained RC square columns.They looked into it analytically and found that the transverse/ diagonal dilation ratio -axial strain curves are influenced not only by the modulus of elasticity and jacket thickness, but also by the fibre type. The validity of the theoretical framework, on the other hand, is unaffected by fibre type.

Ferrier (2011). Fatigue loading was used to evaluate the damage behaviour of FRP-strengthened reinforced concrete (RC) structures. Two design force-strain relations for hybrid carbon-glass FRP sheets, one with and one without account of hybrid effects, were presented based on the research. The hybrid FRP quantity was around 75% of the maximum in the sample with two plies of hybrid sheets (HF-2ply). The tested hybrid FRP sheet had an effective bond length of around 8 in. (200mm), and the bond shear stress capacity of the hybrid FRP sheet and concrete was around 3 MPa (430 psi), which was similar to the bond shear stress capacity of the carbon FRP sheet and concrete.

Harle, Shrikant, and Ram Meghe (2014) used the hand layup method to investigate the strengthening of RCC Beams using various glass fibres. Externally, E- Glass continuous filament and woven roving mat are used for reinforcement. Finally, it was discovered that GFRP may be utilised to increase the strength and arduousness of beams without causing catastrophic brittle failure when used in conjunction with a strengthening procedure.

Kasimzade and Tuhta (2014) tested reinforced concrete specimens in bending without CFRP and with 1, 2, 3, 4 layers of CFRP (CF-130). The following results are drawn through experimental, analytical, and numerical investigations: Based on the amount of CFRP layers, strengthened protected concrete beams will raise catastrophe load and second around 70%. Specimens without CFRP had higher ductility than specimens supplemented with CFRP for maximizing the benefits of CFRP installation. When attempting out de-bonding, failures were prevalent. According to research, investigative and arithmetical exploration impacts are compatible with analytical assessment outcomes using ACI codes.

2.3 Historical Background of Ferrocement

Ferro cement has a history of around 170-years. The idea of inseminating closely packed wire meshes with rich cement mortar is similar to the traditional Kood walling process. In the Kood method, bamboo and reeds are tightly bonded together, and the matrix is formed of mud and cow dung. In India's rural areas, it is frequently used. As a result, Ferro cement might be considered a modified version of Kood, with regulated raw materials, a methodical construction process, and constant structural properties.

Mesh has been utilized in place of bamboo and reeds, and cement mortar has been used in place of mud. Mr. J. L. Lambot of France used Ferrocement in the form of mesh-reinforced cement mortar in Europe. Nervi of Italy employed Ferrocement for shipbuilding in the early 1940s to overcome a scarcity of steel plates during World War II. In addition, he used Ferro cement techniques in the construction of buildings and warehouses. In Europe, Ferro cement has been used to build domes, stadium roofs, opera buildings, and restaurants. Despite Nervi's demonstration of the material's efficacy, no systematic investigations were conducted until 1960, when it was used as a boat building material in Australia, the United Kingdom, and Southeast Asian countries.

The National Academy of Science of the United States of America organised an Ad-hoc commission to investigate the usage of Ferro cement in developing nations in 1972. In 1973, it released a report titled "Ferrocement Applications in Developing Countries." It sparked a comprehensive investigation of Ferrocement in the United States. The American Concrete Institute followed suit in 1974, forming committee 549 on Ferro cement. Since then, numerous individuals and institutions throughout the world have worked hard to develop ferrocement as a construction material. The "International Ferrocement Information Centre" was founded at the Asian Institute of Technology in Bangkok, Thailand, and it publishes a "Journal of Ferrocement" on a regular basis.

2.3.1 Development of Ferro cement strengthened structures

Ferro cement is made by combining ferro (iron) and cement (cement mortar). Ferro cement is a thin-walled reinforced concrete structure that uses small-diameter wire meshes distributed evenly across the cross section rather than separately placed reinforcing bars and Portland cement mortar rather than concrete. In ferrocement, wire meshes are filled with cement mortar.

Ferrocement is a composite made up of twisted wire mesh tied around skeleton steel and a rich cement mortar. Foundations, walls, floors, roofs, shells, and other structural elements can all be built with ferro cement. They have a thin wall, are light, and have a high moisture resistance. It combines the advantages of thin portions with steel's strength. There is no need for formwork or shuttering during the casting process. Ferro cement is used in a variety of applications, including water and soil retention structures, building components, large-scale space constructions, bridges, domes, dams, boats, conduits, bunkers, silos, and water and sewage treatment plants.

2.3.2 Studies on ferrocement applications

Perumalsamy et al. (1979) conducted fatigue flexure tests on ferrocement beams reinforced with volume fractions ranging from 2% to 6% and a range of square steel meshes. The beams were exposed to three levels of loading equivalent to roughly 40%, 50%, and 60% of static yield load. They suggested linear regression models based on the reported experimental data to predict ferrocement fatigue life as a function of the stress range in the topmost layer of steel mesh. Also, as a function of applied load and number of loading cycles, an exponential relationship with two parameters was created to forecast the rise in deflection, average, and maximum crack widths. They carefully modify the proceeding relations to reinforced concrete that has been subjected to fatigue.

Desayi and Ganesan (1984).proposed a method for determining the maximum fracture width Ferro cement-based flexural components. The experimental findings of eight specimen channel shaped ferrocement flooring elements were used to evaluate the constants in the suggested approach. The width of a crack that forms in a ferrocement member was a significant limit state to consider when evaluating the structural elements' serviceability. Various studies have been conducted on the maximum crack width in reinforced concrete members. For establishing the ideal fracture width in ferrocement flexural elements, there were only a few studies available. Nine channel-shaped ferrocement flooring elements were cast and tested as part of the test programme. The specimens were examined under two equal weights and the loads were placed in a third position. The forces are applied in phases, with each stage noting the resulting surface stresses and crack widths. The calculated predicted crack widths were compared to the experimental crack widths. For all levels of loads, the projected crack width width and average crack width results were comparable to the observed maximum and average crack widths. They concluded that a method for calculating the maximum fracture width in ferrocement flexural components of channel cross sections utilised for flat roofs or floors has been proposed. A statistical analysis was used to find the constants used in this method.

Neelamegam et al. (1984). Studied the flexural behaviour of polymer ferrocement. several polymer mortars were used as a matrix. The flexural behaviour of the polymer ferrocement was investigated using latex cement mortar, resin mortar, and polymer impregnated mortar. Traditional ferrocement was evaluated in the same way as a control. Increasing the specific surface area of the reinforcements, regardless of the kind of polymer matrix, considerably increased the ultimate moment and ductility of the polymer ferrocement, according to the test results. Resin ferrocement and polymer coated ferrocement had much higher flexural loads at the first crack than traditional ferrocement and latex ferrocement.

Naaman and Mc Carthy (1985). has examined flexural behaviour of ferrocement beams reinforced with hexagonal meshes .The compressive strength of the mortar, the number of mesh layers, and the mesh orientation are among the variables. The results were compared to those of similar tests using square meshes, and they were used to calculate reinforcement efficiency factors for hexagonal meshes. When differences in reinforcing yield strength were taken into consideration, hexagonal meshes arranged parallel to the loading direction were shown to be almost as efficient as square meshes. Not only in labor-intensive countries, but even in industrialized countries, the cost of mesh reinforcing is frequently the single highest cost component of ferrocement. Savings in reinforcement costs could have a big impact on the entire cost of ferrocement structures and their application. The average ultimate strength of ferrocement beams reinforced with hexagonal meshes placed transverse to the loading direction was found to be around 69 percent that of identical beams reinforced with meshes placed parallel to the loading direction.

Singh and Xiong (1992). Described the behavior of interactions between the various stages of the Ferrocement composite. Simpler and more dependable models were devised, allowing for the cost-effective construction of individually and evenly reinforced ferrocement with weld mesh. They compared their model to other research models and found that debonding between steel and mortar increases the real moment capacity of the weld mesh reinforced section, which is larger than the estimated value. This was based on the various ultimate moment capacity models' planar deformation assumptions. For a singularly reinforced piece, this seemed to be the most effective and straightforward method. A new model for evenly reinforced sections was proposed, which was demonstrated to be more dependable and simple. For singly reinforced sections, the ultimate moment capacity model based on ultimate steel strength demonstrated the highest agreement with test findings. A model based on ultimate steel strength was developed for a uniformly reinforced section, and it produced the best results.

Mohammed Arif et al. (1999) studied the behaviour of materials reinforced with varied numbers and orientations of mesh layers and established a set of elastic and inelastic material properties. The standard empirical relationship based on mortar crushing strength was found to overestimate the mortar modulus. The elastic module calculated using the law of mixture matched the values obtained from ferrocement specimen tests. The elastic module calculated using the law of mixture matched the values obtained from ferrocement specimen tests. The main goal of the experiment described here was to develop a set of material parameters that could be utilised to predict the mechanical characteristics of ferrocement under a variety of loading circumstances analytically. The experimental programme included tension, compression, and flexure testing of ferrocement specimens. Tension, compression, and flexure tests were performed on similar mortar specimens. Because of the lowest volume fraction of wire mesh in the loading direction, they concluded that 45 degree orientation was the weakest design.

Mansur et al. reported punching shear studies on thirty-one square ferrocement slabs (2001). The slabs were simply held on all four sides and a focused force was applied in the centre. The square loaded area's width, mortar strength, reinforcing volume percentage, slab depth, and effective span were all assessed. Both the cracking and punching shear loads increased as the parameters were raised, with the exception of the effective span. It was determined that the critical perimeter for punching shear failure is 1.5 times the slab depth from the loading plate edge. They developed an equation for calculating the punching shear strengths of ferrocement based on the test results and data from the literature.

Jamal and Tareq (2006) used Ferrocement specimens with steel meshes and fibres in bending test . In thin mortar specimens, they looked at the effects of reinforcing steel meshes and discontinuous fibres. The amount of mesh layers, transverse wire spacing, and fibre type were all looked into. They made 72 ferrocement plates measuring 300 mm x 75 mm x 125 mm, each with two to four layers of woven steel square wire mesh and three instances to achieve this goal. Center point loading bending test conducted on each specimen. They experimented with mesh geometry, including wire spacing, mesh layers, and discontinuous fibres such as glass and brass-coated steel fibres.The researchers discovered that increasing the number of steel mesh layers from two to four greatly boosted flexural strength and energy absorption. They also discovered that adding brass-coated steel fibres to the ferrocement matrix greatly boosted flexural strength, and that adding discontinuous fibres to the matrix successfully prevented the mortar cover from spalling at maximum load.

Mohd Zamin Jumaat and Ashraful Alam (2006) studied the use of ferrocement laminates with skeleton bars attached to the beam's soffit to strengthen reinforced concrete beams. Investigations into ferrocement laminate anchoring in reinforced beams, techniques to enhance the ultimate load of the original beam using ferrocement laminate manage the beams' cracking behaviour, and the effect of damage to the original beams prior to repair. Nine rectangular beams were manufactured and tested in the investigation. They found that beams reinforced with ferrocement laminates performed better in terms of cracking behaviour, midspan deflection reduction, and ultimate load bearing capability

.

Prakash and Patil (2007) examined the influence of prolonged temperatures on the tensile qualities of fibrous ferrocement containing steel fibres . They have used various temperatures such as 200° C, 400° C, and 600° C. Compression, flexural, and impact strength tests had been performed. It was discovered that as the amount of steel fibres in fibrous ferrocement grew, so did the compressive, flexural, and impact strengths. They also discovered that increasing the proportion of welded mesh and chicken mesh boosted all of the fibrous ferrocement's strengths. They determined that increasing the amount of steel fibres or increasing the specific surface areas of welded mesh and chicken mesh might improve the compressive strength, flexural strength, and impact strength of fibrous reinforcement. When fibrous ferrocement was exposed to high temperatures such as 200°C, 400°C, and 600°C, its strength qualities were reduced. At 400° C and 600° C, the percentage drop was quite large. Fibrous ferrocement, on the other hand, outperformed ferrocement in terms of fire resistance.

Kondraivendhan and Balu Pradhan (2009) carried out an study on the influence of ferrocement confinement on concrete behaviour . The usage of fferrocement as an exterior confinement for concrete specimens was studied in this work. Comparing the behaviour of retrofitted and traditional specimens was used to determine the efficacy of confinement. They used planar cement concrete specimens with varying compressive strengths for their study. A total of 42 cylindrical specimens measuring 150 mm in diameter and 900 mm in height were produced. The ferrocement laminates were wrapped around the planar cement concrete examples. On the roughened surface of the specimen, a rich cement mortar was placed, and then chicken mesh of consistent thickness was wrapped around the specimen.

T. Ayub, S. U. Khan, S. F. A. Rafeeqi, and S. F. A. Rafeeqi, 2013.This Experiment was conducted to determine the effectiveness of ferrocement strengthening processes such as cast in situ Ferro-mesh layers and precast ferrocement Laminate. Ten (10) reinforced concrete beams, including one control beam, were designed and specified to fail in flexure to achieve this goal. Beams were tested to the service limit under two-point loads prior to strengthening. Only the flexural dominating zone of beams has been reinforced, and they've all been loaded to failure with the same loading system. The most successful procedure was found to be casting in situ Ferro-mesh layers. Reinforcing beams using precast Ferrocement Laminate B, on the other hand, is not only easy to do at home, but also promising in terms of load capacity.

2.4 Further Need of Investigation

Many litterateurs relating to both FRP and ferro cement materials have been cited under the literature review section, although the researchers have mostly focused on one material at a time. They have, for example, examined the efficiency of different types of FRPs in crack reduction, deflection minimization, and enhancing overall structural load bearing capacity improvements. However, research concentrating on structural strengthening performance comparisons between CFRP and Ferro-cement has been observed to be further investigated based on reviewed literature .Many them have focused either on flexural or shear failure, without considering investigating on flexural and shear failure under one study as this two behaviours are mostly related to each other and sometimes may Couse combined failures .so, this is one of the motivations for this investigation which will focus on not only flexural failure but also shear failure of control and CFRP and ferrocement strengthened beams..

In many parts of developing countries, particularly Ethiopia, most of the people cannot afford FRP as a rehabilitation material and they may prefer ferrocement as a structural rehabilitation option because of its cost effectiveness .So, a detailed investigation of ferrocement is needed to be carried out as its components are locally available material. On the other hand, FRP is favoured for structural strengthening in significant constructions where safety takes priority over cost, necessitating comprehensive study. Furthermore, there is no many large practical investigation around this field in my country so; this study may help to make CFRP laminate application for strengthening as cost effective and adoptable in my home country.

In any case, this study conducts a performance-based comparison of CFRP and ferrocement, taking into account the laminate's performance in flexural capacity, shear capacity, and crack controlling measures in order to improve the load bearing capacity of RC beams in flexure and shear, as well as to satisfy the structural strengthening alternatives of CFRP and ferrocement.

Chapter-3

Theoretical predictions of structural behaviour of RC beam section

3.1 General Introduction

The theoretical prediction of structural behaviour of the beam before and after strengthening with CFRP and ferrocement will be the focus of this chapter. The major goal of this validation is to see if the beam under study meets the ACI code's minimal structural behaviour in both circumstances. However, the main benefit of this validation is that it can be used for design in the event that experiments are not possible, but only if the ACI code-based calculations of structural behaviour of the beam agree to some extents with the actual structural behaviour of the beam observed during experimentation.

3.2 Theoretical Predictions of Structural behaviour of Control Beam Section as per "ACI-318-11"

Control beams are defined in this study as those beams that are utilised as a benchmark since their physical properties are not changed before they are strengthened with CFRP or ferrocement. The purpose of using a control beam as a reference point is to examine the structural improvement after using CFRP and ferrocement as external reinforcement to increase the beams' total structural load capacity.

3.2.1 Control beam parameters

Control beam section properties

- Length of the beam L = 700 mm
- Depth of the section D = 160mm
- Effective depth of the section d = 135mm
- Width of the section = 150 mm
- Bottom bars = 2Φ10 mm
- Top bars = 2 Φ 8 mm
- Shear reinforcement = Φ 8 @ 100mm c/c

Materials used in control beam section

- Grade of concrete used fc = M20
- Grade of steel used fs = fe415

3.2.2 Ultimate moment resistance of control section

The moment resistance of the section will be calculated using the specifications and formulations provided by ACI-318M-11 and the results obtained will later be compared to the resulted ultimate load and flexural resistance of the section from experiments carried out to make sure that ultimate strength of theoretical calculations as per ACI-318-14 is in agreement with the experimental test results.

It is essential to know that the rectangular block's area in Fig.3.1 should be the same as the area of the curved stress block, and the two blocks' centroids should also coincide, in order to identify the structural behaviour of the section, particularly its ultimate moment resistance. According to ACI-318M-11, section 10.2.7.1, concrete stress of 0.85fc′ shall be regarded uniformly distributed over a comparable compression zone surrounded by cross section edges and a straight line placed parallel to the neutral axis at a distance of a = 1C from the fibre of maximum compressive strain.
The values of β1 supplied by the code in section 10.2.7.3, which says that 1 shall be considered as 0.85 for fc′ between 17 and 28 MPa. The values of β1 supplied by the code in section 10.2.7.3, which says that 1 shall be considered as 0.85 for fc′ between 17 and 28 MPa. For fc′ more than 28 MPa, β1 must be lowered linearly at a rate of 0.05 for every 7 MPa of strength greater than 28 MPa, but not less than 0.65. For concretes with a f c > 30 MPa, 1 can be calculated using the formula β1= 0.85 0.008(f c 30 MPa) 0.65.

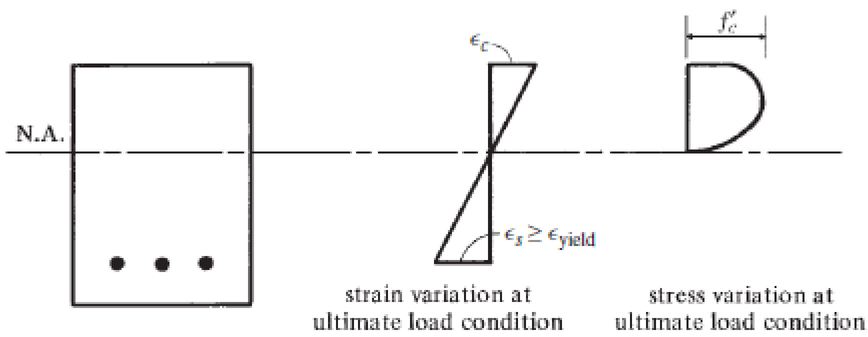


Figure 3.1. Nonlinear stress distribution at ultimate condition.

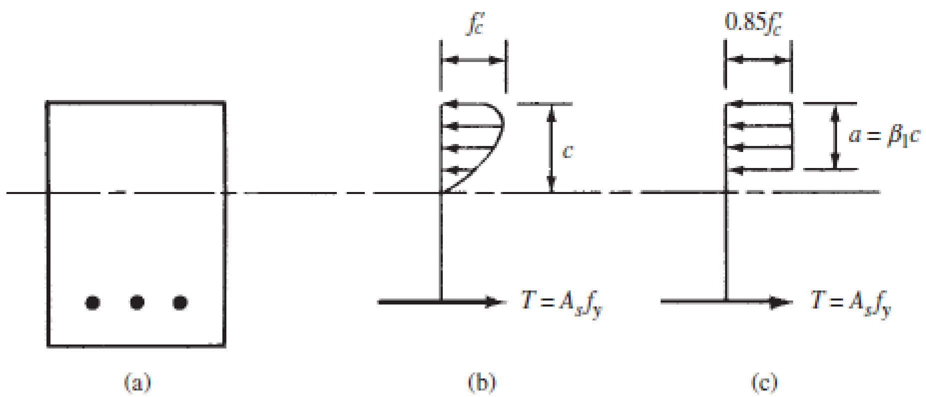


Figure 3.2.Some possible stress distribution shapes

3.2.2.1 Possible Failure mechanisms of the section

According to ACI-318M-11, failures are classified as modes of failure.

• Balanced steel ratio beam: A balanced steel ratio beam is one in which the tensile steel reaches yield at the same time as the extreme compression concrete fibers reach a strain of 0.003.

• Compression controlled section: A section is compression controlled if the compression strain reaches 0.003 before the steel yields, signifying that the member may break abruptly and without warning, which is not ideal in design.

• Tension controlled section: According to Section10.3.4 of the code, tension controlled members have estimated tensile strains of equal to or greater than 0.0050 when the concrete strain is 0.003. The steel will yield before the compression side fails in this situation, generating significant deflections that will warn the users .so this kind of failure is preferred in the designing stage .

Referring to figure 3.3, Statics equations for the sum of horizontal forces and the resistive moment produced by the internal couple may be simply written using these assumptions about the stress block in figure 3.2. These expressions can then be solved for **a** and for the moment M_n. Where Mn is the theoretical or nominal resistance of the section .The usable strength φMn. of a member is equal to its theoretical strength times the strength reduction factor. A member's useable flexural strength, φMn must be at least equal to the calculated factored moment, Mu, caused by the factored load.

$$\varphi Mn \geq Mu \text{……………………………………EQ (3.1).}$$

For writing the beam expressions, reference is made to Figure 4.8. Equating the horizontal forces C and T and solving for a, we obtain.

$$0.85fc' \, a.b = As\, fy \text{………………………… …………………………..EQ(3.2)}$$

$$a = As\, fy/(0.85fc'b) = \rho fyd/(0.85fc), \text{ ………………………………… EQ (3.3)}$$

$$\rho = As/\, bd \text{ ………………………………………………………..…EQ(3.4)}$$

The nominal moment, Mn, can always be expressed as well since the reinforcing steel is limited to an amount that will yield well before the concrete reaches its full strength.

$$M_n = T\,(d - a/2) = As\, fy(\, d - a/2\,), \text{ ……………………………………………EQ(3.5)}$$

And the usable flexural strength is

$$\varphi M_n = \varphi As\, fy(\, d - a/2\,) \text{……………………………………………………….EQ(3.6).}$$

If we insert the value already obtained for "**a**" into this formula and replace As with ρbd, and equate φM_n to M_u we obtain the following expression

$$\varphi M_n = M_u = \varphi bd^2 fy\rho(\, 1 - \rho fy/1.7fc\,) \text{………………… ………………………….EQ(3.7)}$$

where,

ρ = ratio of steel to section area.

F_y = yielding strength of steel

M_n = nominal resistance of the section

M_u = moment applied on the section

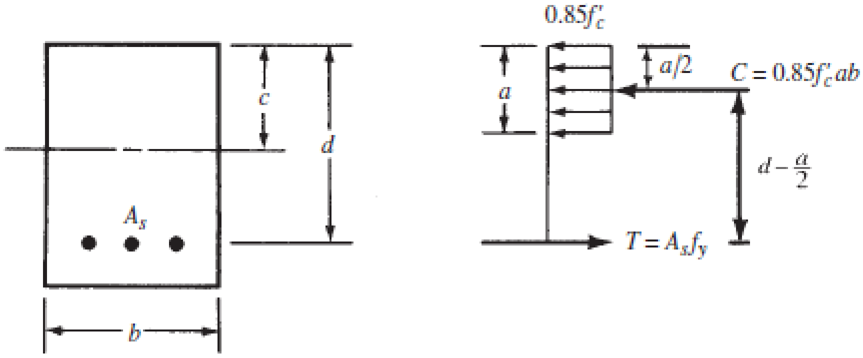


Figure 3.3 Beam internal forces at ultimate conditions.

3.2.3 Ultimate shear resistance of RC control beam section As per ACI-318M-11

The basic shear equations of the **ACI-318M-11,** are expressed in terms of shear forces rather than shear stresses. To put it another way, total shear forces are calculated by multiplying the average shear stresses mentioned in this paragraph by the effective beam area. For the purposes of this discussion, The concrete and the shear reinforcement offer this strength**.**as given bellow.

$$Vn = Vc + Vs \qquad \text{EQ (3.8)}$$

According to ACI-318M-11 . φV_n is a member's design shear strength, which is equal to φV_c plus φV_s, and this must at least be equal the factored shear force to be taken, Vu, the typical shering stress is

$$\tau v = Vu / bd \qquad \text{EQ (3.9)}$$

$$Vu = \varphi Vc + \varphi Vs \qquad \text{EQ (3.10).}$$

The concrete's shear strength, **Vc**, is calculated by multiplying the average shear stress strength by the effective cross-sectional area of the member.

$$Vc = (\lambda \sqrt{f_c'} / 6) \, bw \, d \qquad \text{EQ(3.11)}$$

Where,

Vn= member's theoretical shear strength

Vc = shear strength of concrete

Vs = shear strength steel bars

Ͳv = shear stress of the member

φ= reduction factor

3.2.3.1 Design for Shear

The ultimate shear, Vu, in a beam must not exceed the design shear capacity of the beam cross section, φVn, where φ is 0.75 and Vn is the nominal shear strength of the concrete and the shear reinforcement.

Vu ≤ φVn, where Vn is the combination of design strength of concrete Vc and design shear strength of the shear reinforcement Vs. As indicated in EQ (3.8) of this report.

Vu ≤ φVc + φVs.

Stirrups are used to minimize diagonal tension cracks and transfer tensile stresses from one side to the other, as seen in figure 3.4. Stirrups, on the other hand, can only withstand a minimal amount of pressure before cracking. The strain in the stirrups is equal to the strain in the nearby concrete before inclined cracks appear. The strains in the stirrups are minor at the time because this concrete separates at comparatively low diagonal tensile stresses.

The nominal shear strength of the stirrups crossing the fracture, Vs, can be estimated using the formula below, where n is number of stirrups crossing the crack and Av represents the cross-sectional area of each stirrup crossing the crack. Av is equals to the correctional area of bar used multiplied by two as we are applying two legged stirrups. Figure 3.6 shows the location where shear reduction is not allowed.

$$V_s = A_v\, fyn \text{……………………………….EQ (3.12)}$$

n = d/s, where

d= effective depth of the beam and

s = the spacing of shear reinforcement bars **.so**

$$V_s = A_v\, fyd/s \text{……………………………….EQ(3.13)}$$

$$s = Av\, fyd / V_s \text{……………………………….EQ(3.14)}$$

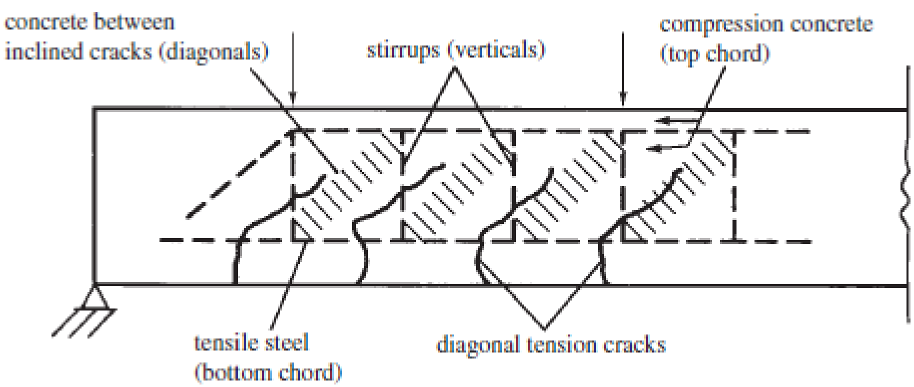


Fig 3.4. Effect of vertical stirrups to control the diagonal tension cracks

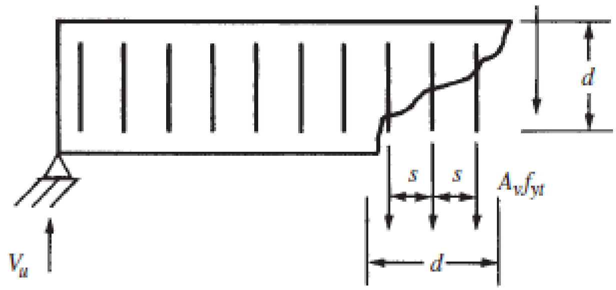


Fig 3.5. Beam with diagonal cracks and vertical stirrups

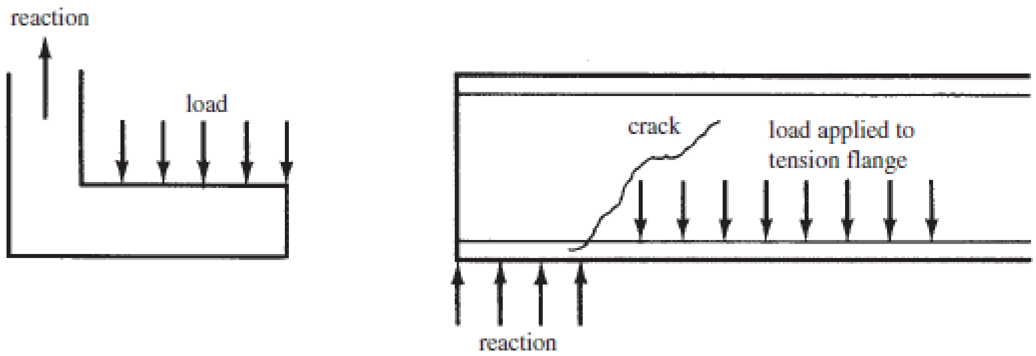


Fig 3.6 .Where end shear reduction is not permitte.

3.3 Deflection control

To get the maximum deflection of simply supported beam shown in figure 3.7. According to **ACI-318M-11 ,** a concentrated load should be applied at the centre of the beam and then EQ 3.15 can be used and the value of deflection determined should not be greater than the values in table (9.3.1.1) of ACI-318-14 and figure 3.8 of this report.

In this case, the applied load is unknown as the experiment is yet to be carried out. But referring the moment resistant of the section in EQ (3.15) , load carrying capacity can be determined .so it is assumed that applied load should be equal or less than the load carrying capacity of the section .

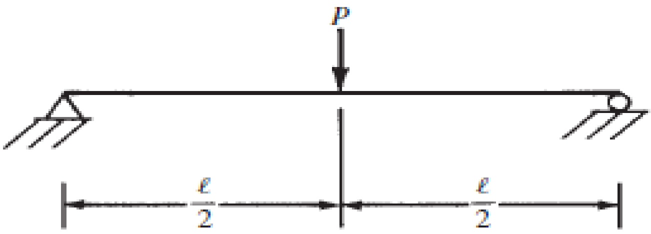
Where,

p = the applied load on the beam,

E= modulus of elasticity,

L= is the length of beam

I= moment of inertia of the member.



$$\delta_{\mathrm{CL}} = \frac{P\ell^3}{48EI}$$ ....**EQ (3.15)**

Fig 3.7. Simply Supported beam using point load

Table 3.1. Minimum depth of nonprestressed beams(ACI-318-14 , table 9.3.1.1)

Support condition	Minimum h[1]
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

3.4 Validation of CFRP Strengthened Beams (as per ACI-440-2R-08)

Under this section, it's planned to focus on the structural improvements of the beam after strengthening it with CFRP. The strengthened been will be evaluated from different structural aspects such change of the control beam after strengthening with respect to its original moment capacity, shear capacity and excessive deflection minimization .It's expected that the beams will have more load carrying capacity after strengthening.

3.4.1 CFRP material used in this investigation

From the manufactures manual, all necessary data required for structural .behaviour of CFRP material used are given and summarized in table 3.2 . From this data, the true stress-strain of this CFRP has been generated as indicated in Figure 3.8 and the necessary calculations concerning the structural behaviour of the beams strengthened with CFRP material as external reinforcement has been carried out in detail in the upcoming sections.

Table 3.2. CFRP properties

Type of CFRP laminate	Sika®CarboDur® S. 1241 of 1.4mm thick (black
Width	50mm
Area	$70mm^2$
Density	$1.60 \ g/cm^3$
mean Tensile strength	$3\,100 \ N/mm^2$
5% Fractile-Value	$2'900 \ N/mm^2$
mean Modulus of elasticity	$170'000 \ N/mm^2$
5% Fractile-Value	$165'000 \ N/mm^2$
Strain at break	>1.8%
Fibre Volume Content	> 68 %
Reference	Manufacturer's Manual

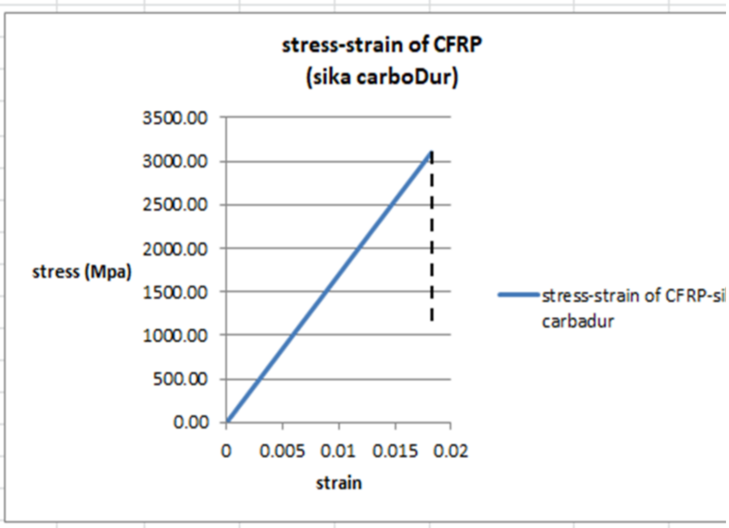


Fig 3.8.stress-strain curve of CFRP

3.4.2 Nominal flexural strength of RC beam section strengthened with CFRP (M_n)

According to the strength design approach, a member's design flexural strength must be higher than its required factored moment, as shown by Eq (3.1). The factored moment Mu is the moment generated from factored loads, while the design flexural strength Mn is the theoretical capacity of the member multiplied by a strength reduction factor.

The factored moment Mu of a section should be determined using load factors as specified by ACI 318M-11, according to this reference. In addition, the flexural strength of the FRP reinforcement alone, M_{nf}, shall be reduced by an extra strength reduction factor for CFRP.

3.4.2.1 Failure modes

The governing failure mode determines a section's **flexural strength**. For a FRP-strengthened section, the following flexural failure modes should be explored (GangaRao and Vijay 1998):

• Tension-induced steel yielding, followed by FRP laminate rupture;

• Steel in tension yielding, followed by concrete crushing

• Concrete cover shear/tension delamination (cover delamination); and

• The FRP is debonded from the concrete base (FRP debonding).

If the compressive strain in the concrete reaches its maximum useable strain ($\varepsilon_c = \varepsilon_{cu} = 0.003$), crushing of concrete is believed to occur. If the strain in the externally bonded FRP reaches its design rupture strain ($\varepsilon_f = \varepsilon_{fu}$) before the concrete reaches its maximum useable strain, the FRP is assumed to rupture. If the force in the FRP cannot be supported by the substrate, cover delamination or FRP debonding can occur as shown in Figure 3.9 .

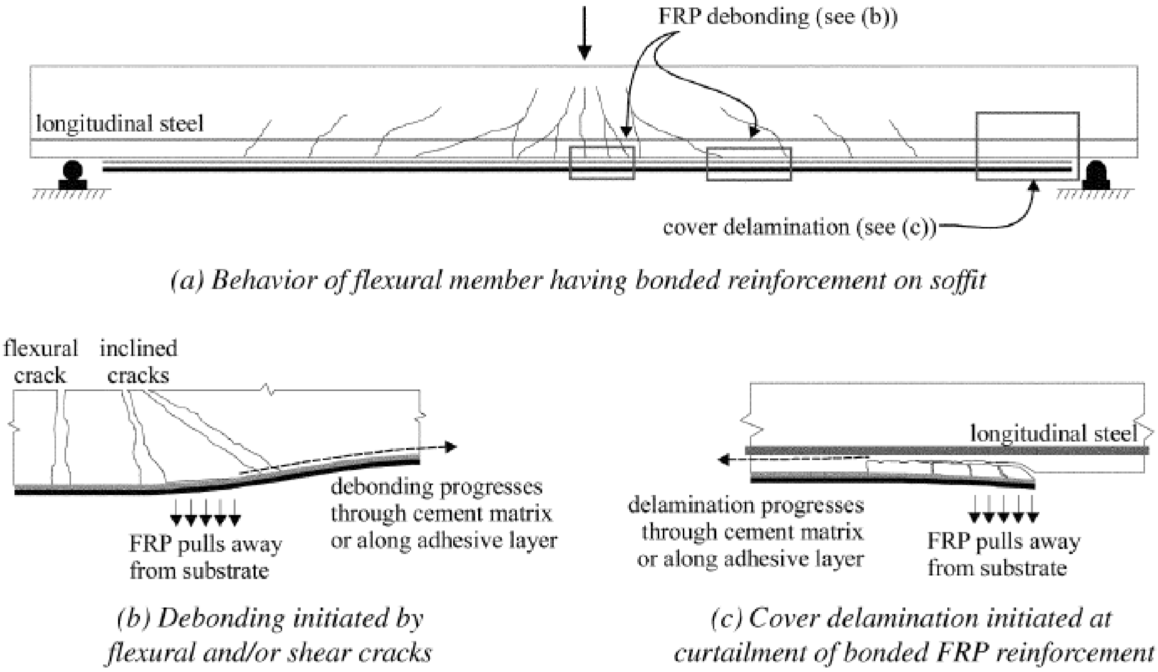


Fig 3.9. Debonding and Delamination of externally bonded FRP systems

To avoid an intermediate crack-induced delamination mode of failure, the functional strain in FRP reinforcement should be confined to the debonding strain level, as defined in Eq (10-2) of ACI-440-2R-08

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu}$$ **EQ (3.16)**

3.4.2.2 Ultimate strength of singly reinforced rectangular section.

For any assumed depth to the neutral axis c, the strain level in the CFRP reinforcement can be computed from eq(10-3) of ACI-440-2R-08 as given bellow.

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$ **EQ (3.17)**

For the given neutral axis depth, equation (3.17) examines the governing failure mode. Concrete crushing controls flexural failure of the section if the inequality's left term is controlled. CFRP failure (rupture or debonding) governs the section's flexural failure if the right term of the inequality controls.

The strain level in the FRP can be used to calculate the effective stress level in FRP assuming completely elastic behaviour.

$$f_{fe} = E_f \varepsilon_{fe}$$ **EQ (3.18)**

The strain level in the nonprestressed steel reinforcement can be calculated using strain compatibility from Eq. (3.19) based on the strain level in the FRP reinforcement.

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d-c}{d_f-c}\right)$$ **EQ (3.19)**

The stress in the steel is determined from the strain level in the steel using its stress-strain curve.

EQ (3.20)

$$f_s = E_s \varepsilon_s \le f_y$$

With the strain and stress values in the FRP and steel reinforcement determined for the predicted neutral axis depth, Eq 3.21 can be used to check internal force equilibrium.
The terms α1 and β1 in Eq. 3.21 are parameters that describe a rectangular concrete stress block that is similar to a nonlinear stress distribution. If concrete crushing is the deciding failure mode before or after steel yielding, the Whitney stress block values (α1= 0.85 and β1=1) from ACI 318M-11 Section 10.2.7.3).as given bellow.

$$c = \frac{A_s f_s + A_f f_{fe}}{\alpha_1 f_c' \beta_1 b}$$ **EQ (3.21)**

Where,

F_{fe} = effective stress of FRP ,

E_f = effective modulus of elasticity of FRP ,

ε_{f} = effective strain of FRP.

εbi = the initial substrate strain

F_S = stress in steel

ε_s = strain level in steel

As = Area of steel

If FRP fracture, cover delamination, or debonding occurs, the Whitney's stress block shown in figure 3.10 delivers reasonably accurate results. For the strain level obtained in the concrete at the ultimate-limit state, a more exact stress block can be used.

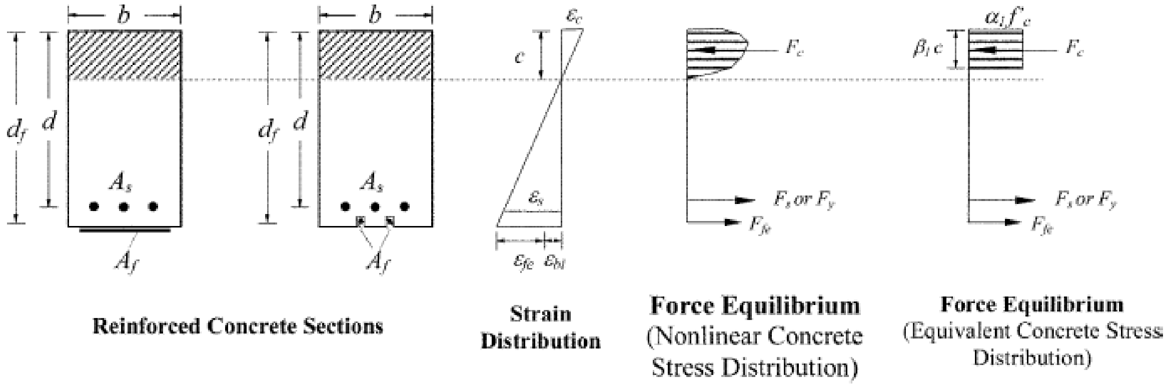


Fig 3.10. Whitney's stress-strain block

Eq3.22 Calculates the nominal flexural strength of the section with FRP external reinforcement The flexural-strength contribution of the FRP reinforcement is reduced by an additional reduction factor, ψf.

The suggested value for ψf is 0.85.as per ACI 318-05 Section 10.2.10 as given bellow.

$$M_n = A_s f_s\left(d\frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(h\frac{\beta_1 c}{2}\right) \qquad \textbf{EQ (3.22)}$$

3.4.3 Nominal shear strength

A concrete member strengthened with a FRP system should have a design shear strength that is more than the necessary shear strength given by Eq. 3.23. The load factors required by ACI 318-05 should be used to calculate the required shear strength of a FRP reinforced concrete member. As stipulated by ACI 318-05, the design shear strength is derived by multiplying the nominal shear strength by the strength reduction factor. **EQ (3.23)**

$$\phi V_n \geq V_u$$

Equation (3.24) can be used to compute the nominal shear strength of a Fiber reinforced concrete element by combining the FRP external shear reinforcement contribution to the reinforcing steel and concrete contributions. The contribution of the FRP system can be reduced by an additional factor ψf.

EQ (3.24)

$$\phi V_n = \phi(V_c + V_s + \psi_f V_f)$$

where Vc shear resisted by the concrete and it is calculated using Eq 3.11 , and

Vs = shear stress resisted by the steel bars and it been determined using Eq3.12 .

3.4.3.1 FRP contribution to shear strength

The dimensions factors utilised in shear-strengthening calculations for FRP laminates are shown in Fig 3.11. The fibre orientation and an expected fracture pattern determine how much the FRP system contributes to a member's shear strength (Khalifa et al. 1998). Calculate the force arising from the tensile stresses in the FRP across the anticipated crack to estimate the shear strength given by the FRP reinforcement. Eq3.25. gives the shear contribution of the FRP shear reinforcement (3.25) as shown bellow.

$$V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_{fv}}{s_f}$$ **EQ (3.25)**

$$A_{fv} = 2 n t_f w_f$$ **EQ (3.26)**

The level of strain that can be created in the FRP shear reinforcement at nominal strength is directly proportional to the tensile stress in the FRP shear reinforcement at nominal strength.

$$f_{fe} = \varepsilon_{fe} E_f$$ **EQ (3.27)**

Where,

V_f = shear strength of FRP

$\mathbf{A_{fv}}$ = Area of FRP laminate

t_f = thickness of FRP

W_f = spacing of FRP laminates

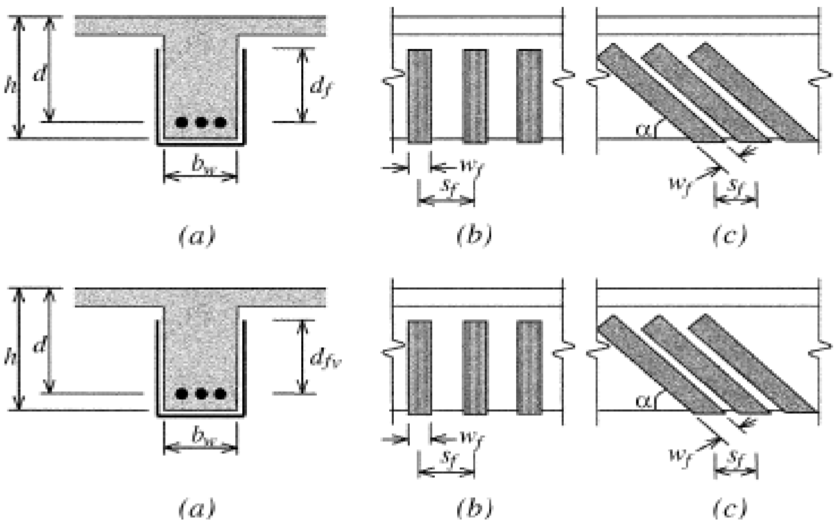


Fig 3.11.The dimensional variables utilised in shear-strengthening calculations for repair, retrofit, or strengthening with FRP laminates

3.4.3.2 Spacing of CFRP laminate for shear strengthening

It is necessary to analyse the contribution of spaced FRP strips used for shear strengthening to determine the shear strength. Spacing should adhere to the limits prescribed by ACI 318M-11 for internal steel shear reinforcement. The distance between the centrelines of FRP strips is known as strip spacing.

3.4.3.3 Reinforcement limits

To compute the overall shear strength produced by reinforcement, add the contributions of the FRP shear reinforcement and the steel shear reinforcement. The sum of the shear strengths produced by the shear reinforcement should be restricted to the requirement specified below, based on the criterion given for steel alone in ACI 318-11, Section 11.5.6.9.

EQ (3.28)

$$V_f \leq 0.66\sqrt{f'_c}\, b_w d$$

3.5 Validation of ferrocement Strengthened Beams

A ferrocement section's flexural strength can be computed using an approach similar to that used for a reinforced concrete column, employing the ACI 318M-11 strength analysis procedure and ACI 549 uses to guide on mesh effectiveness factor, elasticity modulus, and yield strengths.

$$\mathbf{Mn = \sum C_{si} \text{ or } T_{si} \ (d_i - \beta_1 C /2)} \text{……………………….}\mathbf{EQ\ (3.29)}$$

Where

T_{si} = force resisted by the section reinforced with ferrocement .

C = depth of neutral axis

Chapter-4

Experimental Investigation

4.1 Material, specimen preparation and testing procedure

4.2 Materials for control beams

The components of materials used in RC concrete for study includes cement, steel bars, aggregate sand, and water. So to accomplish the necessary properties at the stipulated age, workability of fresh concrete, and durability requirements of these material is tested and presented the results in tabular forms bellow .

4.2.1 Cement

For this study, ordinary Portland cement of grade 43 with specific gravity of 3.15 have been used. It s has been tested and the test results are presented in tabular form .The cement's fines properties ,compressive strength ,and setting time has been summarized in Tables 4.1,4.2 and 4.3 respectively.

Table 4.1. Fineness property of cement

Cement sample	Fineness of cement (% retained on 90micro sieve)		Values required by **IS: 269-2015**	remark
C-1	3	average **3.3%**	**<10%**	Acceptable
C-2	2			
C-3	5			

Table 4.2. Compressive strength of cement

Cube spacemen	Compressive strength(Mpa)	Average strength (Mpa)	Values required by IS: 269-2015(Mpa)	Remarks
SC-1	46	46.33	≥44	Acceptable
SC- 2	48			
SC-3	45			

Table 4.3. Setting time of cement

Sample test	Setting time (minutes)	Requirement IS: 269-2015 (minutes)	remark
Initial setting time	65	≥30	Acceptable
Final setting time	280	≤ 600	

4.2.2 Fine aggregate

The sand used in the experiments collected from a local quarry. The sand was sieved using a 4.75 mm sieve to remove any particles larger than that. It was then subjected to a sieve examination in the laboratory using three samples of one kilogram each. Table 4.4 displays the results of the laboratory tests. Fine aggregate fulfils the requirements of grade I of IS: 383-2016. The specific gravity of sand and water absorption, on the other hand, were found to be 2.65 and 9%, respectively.

Table 4.4. Fine aggregate Properties as per Sieve Analyses performed

Sieve (mm)	Weight (g) retained			Average weight retained	Average % weight retained	Cumulative %Weight retained	% passing	IS: 383-requirmets Zone 1
	Sample(1)	Sample(2)	Sample(3)					
4.75	90	92.5	95	92.52	9.20	9.25	90.75	90-100
2.36	49	20.5	52	20.53	2.05	11.25	88.75	60-95
1.18	350	400	450	400	40.00	51.25	48.75	30-70
0.6	131	115	98	114.71	11.47	62.72	37.28	15-34
0.3	194	167	140	189	18.90	81.62	17.4	5-20
0.15	163	152	141	152	15.20	96.82	3.18	0-10
pan	23	23.5	24	23.54	2.35	∑=313		
Fines modules of sand = ∑% cumulative weight retained / 100 = 313/100 = 3.13								

4.2.3 Course aggregate

Throughout this experimental study, 20 mm down crushed stone aggregate was used as coarse aggregate which has a specific gravity of 2.61 and sieve analysis was conducted in laboratory using three sample of 2kg each. Table 4.5 shows the results of the tests performed in the laboratory, as well as the grade assigned to them.

Table 4.5. Coarse aggregate properties as per sieve analyses performed

Sieve (mm)	Weight (g) retained			Average weight retained	Average %weight retained	Cumulative %Weight retained	% passing	remark
	S-1	S-2	S-3					
20	208	81	158	149	7.45	7.45	92.55	acceptable
16	427	317	267	337	16.85	24.35	75.7	
12.5	753	938	713	801.33	40.07	64.37	35.63	
10	429	442	479	450	22.50	86.87	13.13	
4.75	155	190	348	231	11.55	98.42	1.58	
Fines modules of sand = ∑% cumulative weight retained / 100 = 680/100 = 6.8								

4.2.4 Steel bars

For this experiment study, Fe415 grade of steel bar has been used due to its higher ductility and superiority in strength .Necessary laboratory tests have been carried out on samples of diameters 10mm and 8mm as indicated in figure 4.1.Tthe results gained is acceptable and it has been summarized in table 4.6 below.

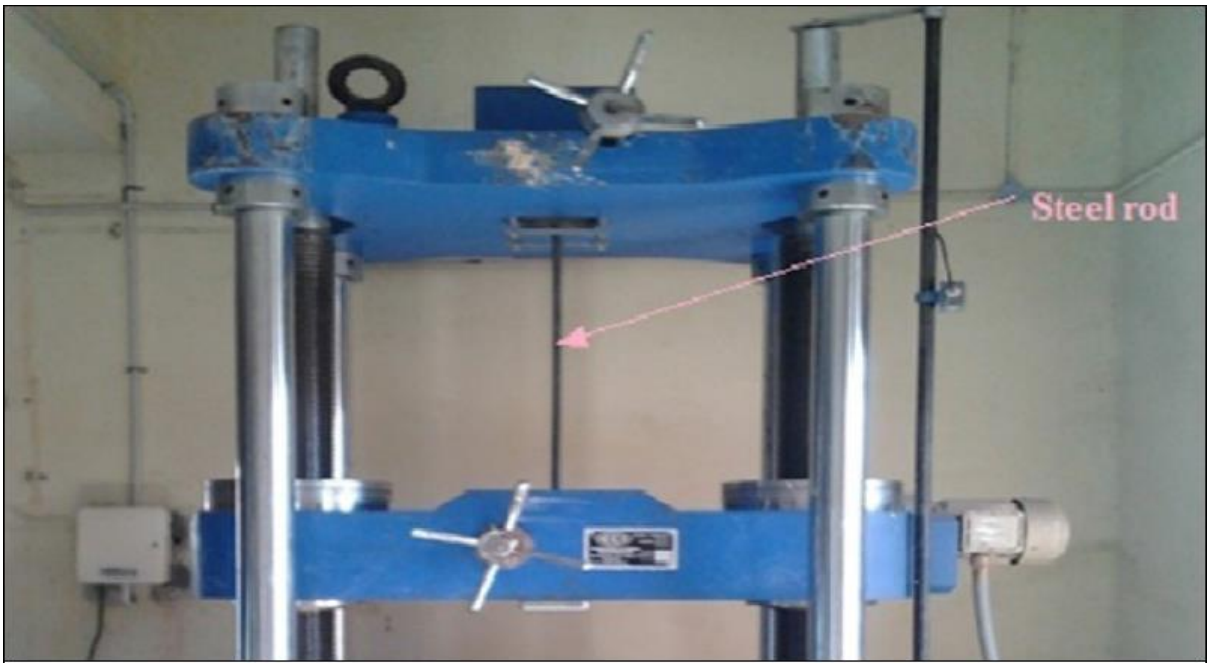


Fig4.1.Tensile Test on Steel Rod

Table 4.6.properties of steel bars

Steel bar spacemen diameter (mm)	Yielding load (KN)	Average Yielding Load (KN)	Yielding strength (Mpa)	Average Yielding strength (Mpa)	Ultimate load (KN)	Average Ultimate load (Kn)	Ultimate strength (Mpa)	Average Ultimate strength (Mpa)	Maximum % of elongation
ST-10	37.31		475		45.53		580		
ST-10	36.27	**36.56**	462	**465.7**	44.27	**44.61**	564	**568**	**18**
ST-10	36.11		460		44.04		561		
ST-8	22.62		450		27.14		540		
ST-8	22.42	**22.8**	446	**447.3**	26.92	**26.94**	535.5	**535.8**	**23**
ST-8	23.39		444		26.74		532		

4.3 Properties of CFRP and its application

After curing the specimens were cleaned and saturated with epoxy, one layer of CFRP laminate was manually put adjacent to the member's surface, applying uniform pressure throughout the whole width of the prepared surface to ensure that all air bubbles/pockets were removed, resulting in a uniform and smooth final surface as indicated in figure 4.2 . The FRP composite materials need 72 hours of air curing. So all specimens were correctly staggered without any contact with each other, the floor, or any other object during air drying to avoid any sticking.

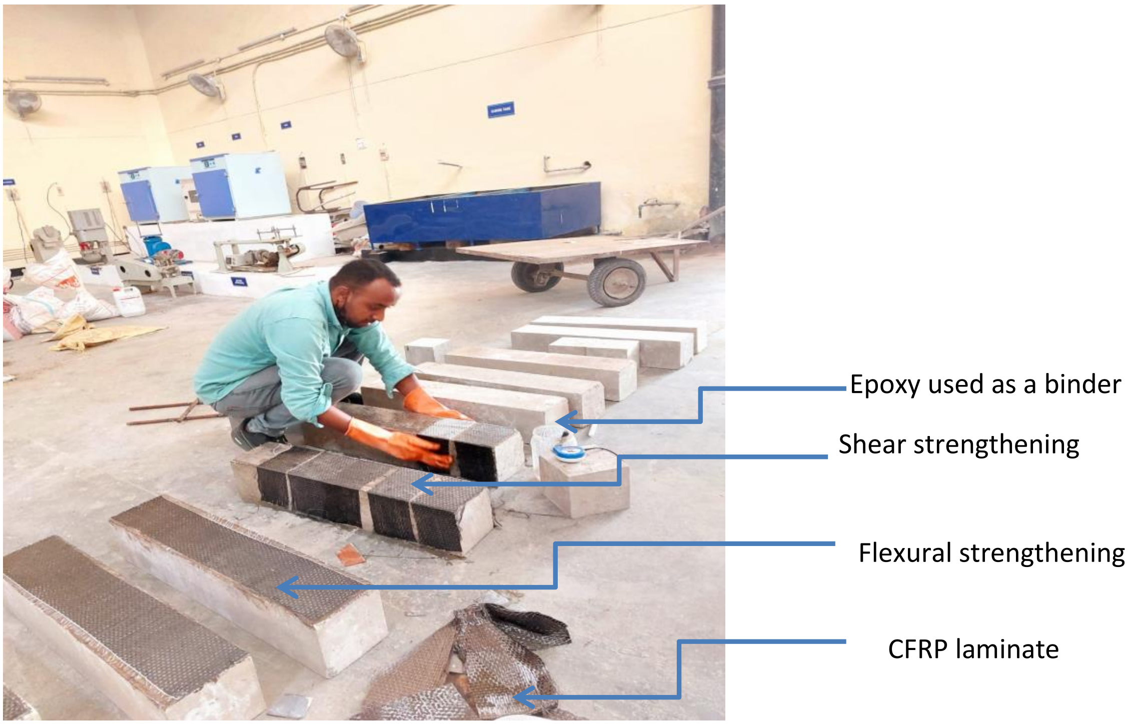


Fig 4.2 . CFRP application.

4.3.1 Type and properties of CFRP used

As shown in fig. 4.3, Sika CarboDur is used as a strengthening material in this study. This CFRP is bonded to the beams as an externally bonded reinforcement adopting Sikadur®-30 IN epoxy resin based adhesive for typical application and/or service temperatures.

4.3.1.1 CFRP advantages

CFRP systems are used to

- improve, increase or repair the performance and resistance of structures
- Increase load carrying capacity of the structural members.
- Repair damaged structural elements.
- Improve the serviceability and ductility of structures
- Reduce the deflection and crack width

Fig 4.3. used CFRP “Sika CarboDur S”

Referring the manufacturer's manual, the properties of the CFRP laminate material and epoxy used as a binder are summarised in Table 4.7 and 4.8 respectively.

Table 4.7 .Structural Properties of CFRP

Type of CFRP laminate	Width	Density	Tensile strength	Modulus of elasticity	elongation	As per
Sika®CarboDur® S of 1.4mm thick (black)	50 mm	1.60 g/cm^3	3 100 N/mm2	170,000 N/mm2	>1.80 %	Manufacturer’s Manual (sika)

Table 4.8. Type of Epoxy

Type of epoxy used	CFRP laminate	Typical consumption of Sikadur®-30	Mixing A and B	As per
Sikadur®-30 A and B	Sika®CarboDur® S1241	0.45 − 0.80 kg/m	1:3 (A:B)	Manufacturer’s Manual (sika)

4.4 Ferrocement Construction techniques and its application

Unlike other complex engineering materials, ferrocement uses easily available materials and requires little skilled labour. Wire mesh, sand, cement, water, and mild steel rod as skeleton reinforcement are the basic components required for ferrocement constructions. Below is a quick summary of the constituent materials and construction process.

4.4.1 Ferrocement material components

4.4.1.1 Wire mesh

One of the most essential components of ferrocement is wire mesh. Metal wire meshes which are square, hexagonal, or expanding are typically 0.5mm to 1.0mm in diameter and spaced 5mm to 25mm apart as shown in Fig.4.5. They must, however, be easy to handle and flexible enough to bend around sharp edges if necessary. The primary function of the wire mesh is to serve as a base material for forming the form and supporting the mortar. It absorbs tensile stresses on the structure that the hardened mortar alone would be unable to bear. In this study, square woven meshes with a wire diameter of 0.8mm and a spacing of 5mm were used.

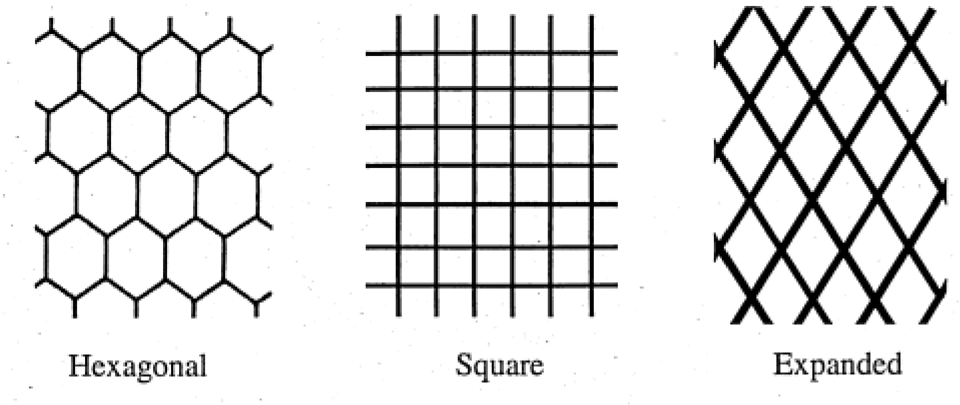


Fig 4.5 .Typical shapes of wire meshes

4.4.1.2 Cement

There are various varieties of cement available commercially, the most common of which is standard or ordinary Portland cement. This type of cement is suitable for applications where there are no specific requirements. Ordinary Portland cement of 43 grade has been used throughout the study, the properties of cement has been already presented in section 4.11 in Table 4.1 to 4.3.

4.4.1.3 Sand

Well-graded coarse sand, such as that used in concrete, is used for preparing mortar for ferrocement construction. There should not be an excessive amount of fine particles, and porous sand particles are not suggested because they impact the mortar's durability and structural effectiveness. The mortar was made with river-borne, well-graded coarse sand with a fineness modulus of 3.13.

4.4.1.4 Water for mortar mixing

The water used to mix the mortar should be free from acids, soluble salts, and other organic matter in which can affect the cement's setting time and, as a result, the structure's strength. Sea water is not ideal for mixing the mortar because it raises the danger of mesh and reinforcement corrosion. The cement mortar in this investigation was made with supplied tap water.

4.4.1.5 Preparation of mortar

It is critical that mortar be mixed in such a way that the necessary strength is consistently achieved. Cement to sand proportions typically range from 1 part cement to 2 parts of sand by weight. The ratio of cement to sand used in the mix was 1:2 by weight. Depending on the dryness of the sand, the water-cement ratio was in the range of 0.4. Sand and cement were first uniformly combined in the mixing procedure. To achieve the desired workability of the mortar mix, water was added in small increments.

4.4.1.6 Plastering

The plastering work determines the strength and durability of a ferrocement building. Before plastering a structure, the wire meshes were checked for appropriate positioning, and any rust, grease, or other pollutants were brushed off the beam surface.

Plastering by hand with a trowel has shown to be the most effective method. The plastering technique used here was a one-stage procedure, which entails a single monolithic application of mortar to fill in the wire mesh and finish both the inner and outside surfaces at the same time before the initial set of cement. Mortar usually stays in place after being placed in hard mixes.

4.4.2 Ferro-cement application on beams

After cleaning the surfaces of four beams, cement slurry was applied to the beams to ensure good bonding between the Ferro-cement laminate and the beam as shown in figure 4.6. These shear-strengthening beams were upgraded with square wire mesh in a 65-degree orientation. After that, one face and three faces of the beams for flexural and shear strengthening respectively were plastered with 25mm cement mortar (1:2 with w/c = 0.4) .

Finally, the beams were allowed to cure for seven days. Following that, these beams were evaluated under one and two point loading for flexural and shear respectively in the same way that the control beams and CFRP strengthened beams to determine maximum load and corresponding deflections.

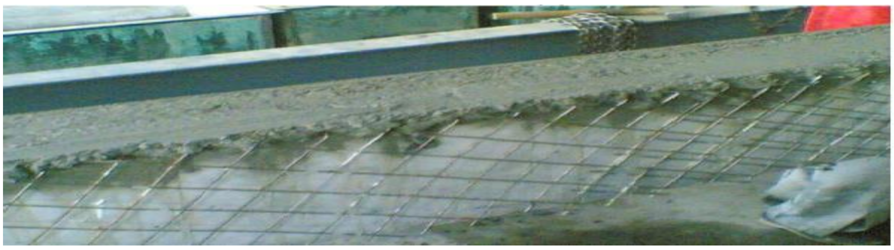


Figure 4.6. Ferro-cement application

4.5 Compressive Strength of Concrete

4.5.1 Concrete mix design

Concrete mix design has the advantage of providing the correct quantities of components, making concrete use more cost-effective in achieving the required structural strength.

The specimens were prepared according to IS 10262: 2019 requirements and M20 grade design mix concrete was used in this study. The water/cement ratio was set at 0.55, and the mix proportions were 1:2.14:3.11. However, based on the mix proportions used, quantities of materials needed for one cube meter of concrete were calculated and reported in table 4.9. Before casting, the workability was checked, and a slump of 65mm was observed.

Slump testing technique and casting of cubes were done after mixing as shown in figure 4.7 and 4.8 respectively.

Fig 4.7. slump test

Fig 4.8 . Cubes after casting

Table 4.9 materials used for M20 concrete mixing

Material	Cement	Fine aggregate	Course aggregate	Water
kg/m3	359	766	1118.4	197

4.5.2 Cube Compression testing

Cube testing was used to assess the concrete's compressive strength, which provided us with a thorough insight. The factors that influence the compressive strength of concrete include the water-cement ratio, concrete strength, concrete material constitutive quality, and quality assurance during the manufacturing process.

A cube or a cylinder is used to assess compressive strength. Various standard codes specify a concrete cylinder or cube as the standard specimen for the test.Six cubes with conventional dimensions of 15X15X15cm were cast and evaluated on a compression test machine as shown fig 4.9 after 28 days of curing for this investigation. The specimen was tested under compression loading as shown in figure 4.9. So, the specimen crashes when the concrete reaches its ultimate compression strength as shown in fig 4.10. Table 4.10 summarises the findings of the tests.

Table 4.10 . Cube compression test results

Cube samples	Actual strength(MPa)
S-1	21.2
S-2	21.6
S-3	21.3
S-4	22.2
S-5	21.5
S-6	22.4
Average	**21.78**

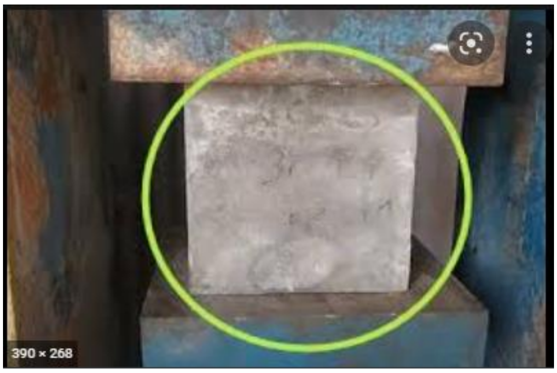


Fig 4.9. M20 Cube load application

Fig 4.10 . Crashed M20 Cube

4.6 RCC beam specifications

M20 grade concrete and Fe415 steel have been used to construct a total of 12 RCC beams with dimensions of 700 x 150 x 160 mm. The beams are designed to be under-reinforced so that the reinforcing bars yield before the concrete crashes. For flexural reinforcement, two bars of 10 mm diameter on the tension side and two bars of 8 mm diameter on the compression side have been provided. Shear reinforcement in the form of 8mm diameter bar with 100mm centre to centre spacing has also been given. The RC section and reinforcement detailing used for both control and CFRP and ferrocement strengthened beams are shown below in figure.4.11 and4.12 2 respectively.

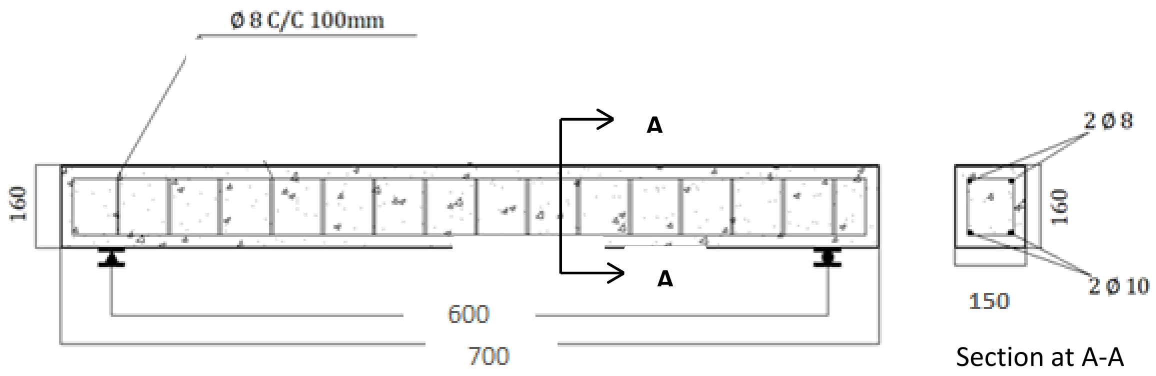


Fig 4.11.Details of control beams

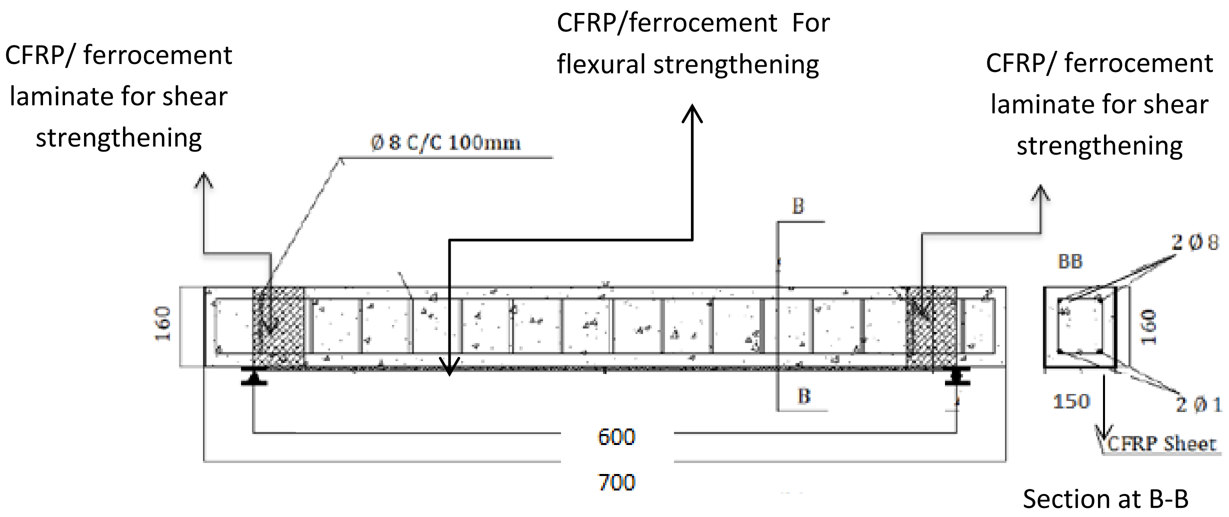


Fig 4.12. Details of strengthened beams

4.7 Experimental setup and test Procedure

Despite the fact that RCC constructions have a long service life, external factors can cause deterioration, leading to the loss of load-bearing capacity and the formation of substantial visible cracks. As a result, repairing and strengthening reinforced concrete structures is critical for maintaining their safety and stability. Retrofitting can be a cost-effective option to replace RCC structural elements; hence the main goal of this experiment is to develop a realistic and economical strengthening alternative. For this research, 12 beams of 700 x 150 x 160 mm, using M20 grade of concrete and Fe415 steel bars, were cast and load tested in both flexural and shear failure modes, with one point loading test for flexural failure mode and two points loading test for shear failure mode. The structural behaviour of reinforced concrete beams strengthened using Carbon Fibre Reinforced Polymer (CFRP) and Ferrocement laminates have been investigated and compared to control beams.

In this experiment, four RCC beams were used as control beams, and their original flexural and shear behaviour was not modified whereas 8 RCC beams were strengthened with CFRP (sika-dur) for both flexural and shear reinforcing purposes, by following the manufacturer's guidelines. The remaining four RCC beams have been reinforced with ferro-cement.

A 1000kN universal testing machine(UTM)was used to apply the load, which was gradually increased at a consistent pace until failure, with the results indicating that load-carrying capacity improvements are likely. The beam specimens were evaluated under one point loading at the centre of the beams for flexural failure testing as shown in figure 4.13. but for shear failure, it’s been evaluated under two point loading with a spacing of 400 mm between them and 600 mm between supports.

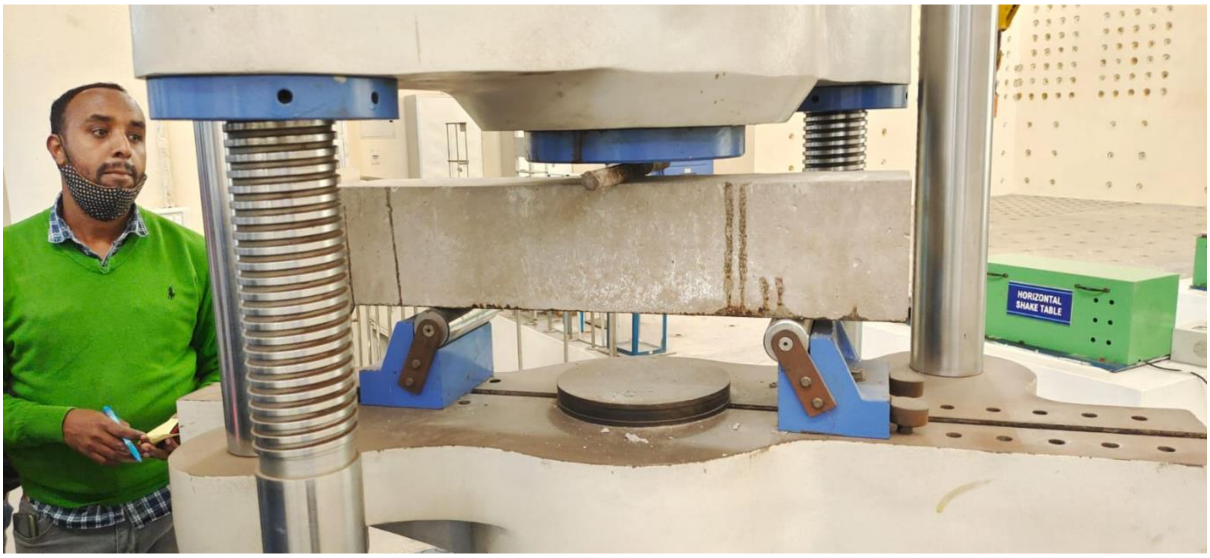


Fig 4.13. Experimental setup

Chapter-5

Discussion of test results

5.1 Structural behaviour of Control Beams

Under monotonically increasing load, the static behaviour of beams was recorded to investigate structural responses of beams such as deflection, crack propagation, and ultimate load bearing capability. As a result, the beams were subjected to monotonous testing with simple supported boundary conditions. The flexure-deficient of control beams were less ductile and gave considerably small deflections before failing.

However, when the load increases the severe bending fibre stresses reach the concrete's tensile strength and also, as the stress on the beam increased, multiple flexural cracks appeared. The maximum amount of bending moment was handled by the tension steel reinforcement, while the rotation of the beams increased, generating an increase in steel stress. The total stiffness of the beam was lowered due to cracking and reduced to minimum when the stress in the steel approached yield value. Flexural cracks also expanded vertically upwards from bottom of the beam at mid span for flexural failure loading whereas shear cracks expended angularly or inclined format for shear failure loading starting from the supports to the loading point and it’s also been observed that the crack width as well as deflection was gradually increasing from the cracking point up to the failure as shown in fig.5.1 and fig.5.2 respectively. As per fig 5. 3 a linear response is observed in the curve up to the first crack appearance when the loading reached at the cracking point as well the load increased and the cracks widen up to some extent and the beam fails at maximum load carrying capacity for both flexural and shear loaded control beams.

Fig 5.1.Flexural failure of control beam

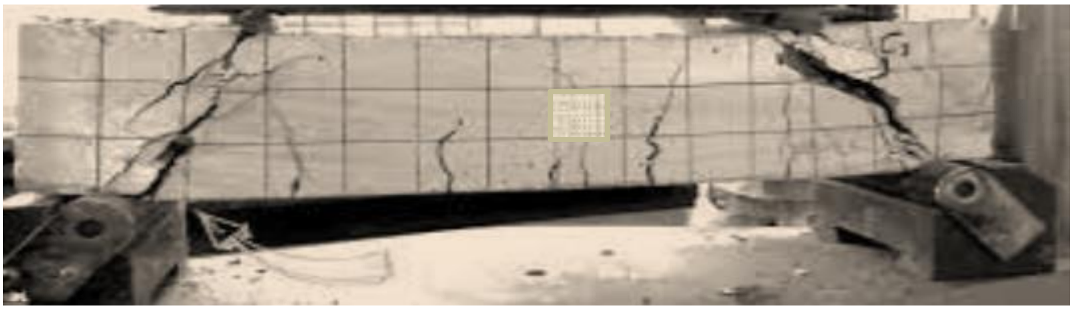


Fig 5.2 .shear failure of control beam

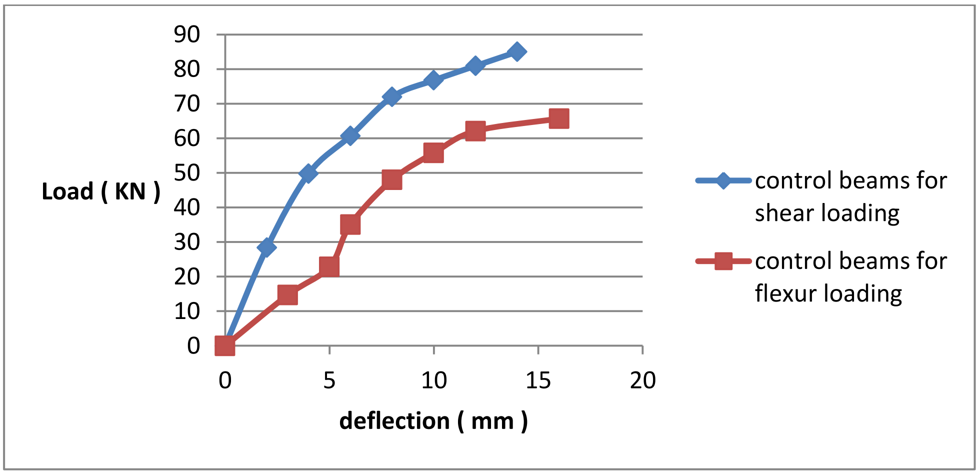


Fig 5.3. load vs mid span deflection curve of control beam under shear and flexural loading.

5.2 Behaviour of Flexural strengthened beams

Retrofitted with CFRP and Ferrocement laminated beams were tested following the same procedure as the control beams. Up to cracking point, the load carrying capacity and deflection of CFRP and ferrocement retrofitted beams show a similar trend in flexural behaviour. The load deflection behaviour of control and retrofitted beams appears to be linear up to first crack load since the entire section is effective and the CFRP sheet influence in moment of inertia is quite minor. The stiffness of strengthened beams increases when compared to control beams this is due to the enhanced flexural stiffness of the strengthened beams. CFRP laminates can resist a considerable amount of bending moment as compared to both ferrocement retrofitting beams and unstiffened beams. Both CFRP and ferrocement strengthened beams showed a reduction in deflection as well in crack width.

5.2.1 Effect of flexural retrofitting on load carrying capacity

The ductile behaviour was observed in beams after reinforcing it with **CFRP** and ferrocement laminates and results are summarized in Table .5.1 which revealed that the R.C. beams' maximum load carrying capacity has increased greatly as a result of the strengthening. The highest load bearing capacity was measured at 82.5 kN , this represents 20% increase over the control beam's maximum load carrying capability which was 65.69KN. When beams were strengthened by **CFRP**, the crack propagation and final crack pattern were markedly different from those of flexure-deficient beams. However, ductility behaviour was also observed in the flexural reinforced beams with **ferro-cement** though the investigation indicating relatively less ductile behaviour than CFRP**.**

The result also indicates that the R.C. beams' maximum load carrying capacity has gently improved as a result of the strengthening. The maximum load bearing capacity was recorded at 72.59KN which represents a 9.5% increase over the control beam's maximum load carrying capability. However, it can be seen that the flexural capacities of the beams obtained by experiments and by the theoretical estimations as per ACI are correlated reasonably well although there is some variations in the values which can be seen in Table 5.1.

Table 5.1. Average Load carrying capacities of control beams, CFRP and Ferrocement flexural strengthened beams

Reference	Average Control beams		Average CFRP strengthened beams		Average Ferrocement strengthened beams	
	CRL (KN)	ULT (KN)	CRL (KN)	ULT (KN)	CRL (KN)	ULT (KN)
Experiment results	**37.15**	**65.69**	**46.66**	**82.51**	**41.05**	**72.59**
predictions as per ACI-318-M-11	**41.26**	**68.74**	**52.63**	**94.24**	**44.62**	**81.73**
% variations	**9.96%**	**4.4%**	**11.3%**	**12.4%**	**8%**	**11%**

Where CRL= cracking load, ULT = ultimate load

5.2.1 Effect of flexural retrofitting on deflection

The span, moment of inertia of the section, loading type, concrete modulus, and CFRP sheet characteristics all have a role in beam deflection. When CFRP or ferrocement laminate is bonded to the tension face of reinforced beams, the moment of inertia and thus stiffness are increased. The reduction in deflection of the strengthened beams under various load increments is influenced by this increase in section.

CFRP-strengthened beams are found to be more ductile than ferrocement-strengthened beams. It means that this kind of beams will be able to withstand the load until failure without excessive deflections. CFRP material has improved the defection behaviour of the beam in this study and it's been observed a reduction in the original deflection by 18.75%. Whereas ferrocement laminates has reduced the original deflection by 6.25% .

However, deflection in retrofitted beams reduced dramatically in the beams strengthened with CFRP while deflection progressively decreased in the beams strengthened with Ferrocement, according to the experiments when compared to control beams. Table 5.2 shows the actual deflection values of the two different strengthening techniques as well as the unstrengthen beam. The load versus mid span deflection values for control and flexural strengthened beams were recorded and presented in fig.5.4. CFRP and ferrocement laminates reduced deflection which proofs their effectiveness in deflection controlling

Table 5.2. Deflection of Control and Strengthened Beams

Designation	Deflection at different loading stage(mm)	
	Cracking stage	Ultimate stage
Control beams	5	16
CFRP strengthened beams	3	13
Ferrocement strengthened beam	4	15

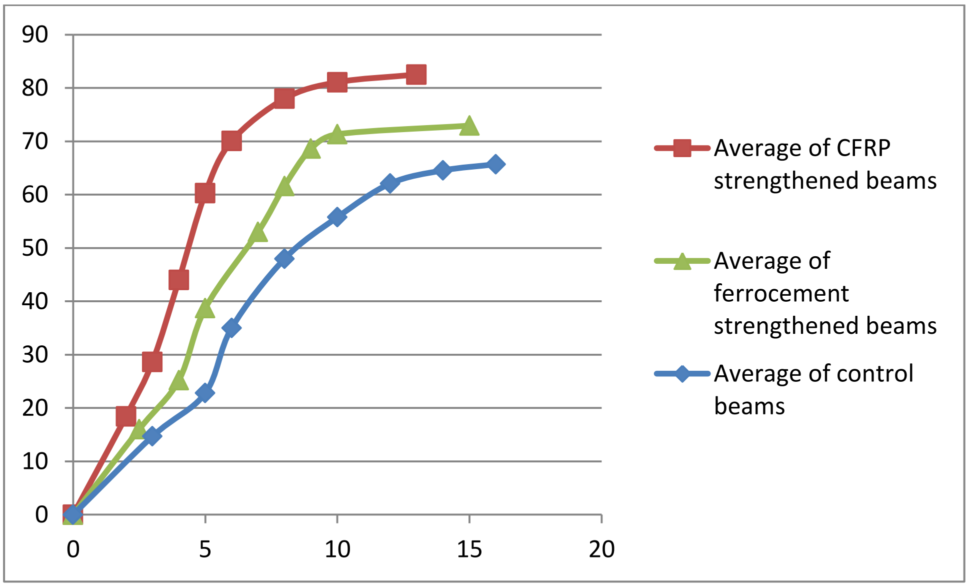


Fig 5.4. Load vs mid span deflection values for control and flexurally strengthened beams

5.2.2 Effect of flexural retrofitting on crack width

At all stages of loading, the control beams have significant flexure cracking, and the strengthened beams have a smaller crack width than control beams. For control and retrofitted beams, the crack width at the cracking stage and final stage were measured experimentally. Table 5.3 shows the crack width at various loading stage for all beams.

From the experiment it was observed that CFRP laminate has reduced the crack width by 31.5%, whereas ferrocement laminates has reduced the crack width by 16.85% . The control beams had a numerous flexural cracks, but the strengthened beam has a small number of flexural cracks with narrow widths. CFRP and ferrocement laminates reduced cracks indicating that they are effective in crack controlling.

Table 5.3.Crack width of control and flexural strengthened beams

Designation	Maximum Cracks width (mm)
Control beams	1.2
CFRP strengthened beams	0.70
Ferrocement strengthened beams	0.75

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5.3 Behaviour of Shear strengthened beams

CFRP and ferrocement shear strengthened beams were tested as per the procedure described earlier for control beams. As a result, flexural cracks developed first, followed by some tiny shear cracks that propagated to the loading points; however, the overall failure was caused by a quickly forming shear crack near the supports that spread to the loading points. Although the cracks generated after the application of CFRP are smaller in number and width, they propagate at a slower rate than those formed in the control beam and ferrocement strengthened beams.

All the beams developed diagonal cracks in the constant shear spans, indicating that they were brittle shear failures. The de-bonding of the fibre wrapping system followed by the diagonal cracking was also observed. When compared to ferrocement laminate, the CFRP system is found to be more effective in improving shear capacity of the beam.

5.3.1 Effect of shear retrofitting on load carrying capacity

Table 5.4 shows the average first crack and ultimate loads values for CFRP , ferrocement strengthened beams, as well as control beams (i.e. un-strengthened beams).The the average first cracks appeared in control beam when the loading reached at 49.75 KN and as the load increased, the cracks widened further and the beam failed when the maximum carrying load reached to 85.5 kN.

Comparing to control beams, the average cracking shear load for CFRP and ferrocement strengthened beams for shear is raised by about 20.28 %, and 10.44 %, respectively. Similarly, the ultimate shear load of CFRP, and ferrocement strengthened beams increased by 16.35% and 8.22% respectively compared to the ultimate load of control beams.

The CFRP strengthened beams deformed more than the ferrocement strengthened beams which indicates that the CFRP system is more successful in improving the beam's ultimate load carrying capacity in shear failure mode as well then ferrocement .

Table 5.4. Load carrying Capacities of Control beams, CFRP and Ferrocement shear strengthened beams.

Reference	Average Control beams		Average CFRP flexural strengthened beams		Average ferrocement flexural strengthened beams	
	CRL (KN)	ULT (KN)	CRL (KN)	ULT (KN)	CRL (KN)	ULT (KN)
Experiment results	**49.75**	**85.5**	**62.41**	**102.22**	**55.55**	**93.16**
predictions as per ACI code	**53**	**96.3**	**85**	**152**	**62.4**	**112.54**
% variations	**6.1%**	**11.2%**	**26.5%**	**32.7%**	**10.9%**	**17.2%**

Where CRL= cracking load, ULT = ultimate load,

5.3.2 Effect of shear retrofitting on deflection

CFRP material has improved the defection behaviour of the beam in this study by reducing the deflection about 20% at the ultimate loading. Whereas ferrocement laminates has reduced the deflection by 13.33% at the ultimate loading. From the experiment, the CFRP-strengthened beams are more ductile than ferrocement-strengthened beams as they withstand the load until failure without sudden collapse.

Table .5.5.shows the actual deflection values for control ,CFRP strengthned beams and ferrocement strengthened beams .The load versus mid span deflection values for control and shear strengthened beams was recorded and presented in fig.5.5.

Table 5.5.Average Deflection of control and shear strengthened beams

Designation	Maximum Deflection at (mm)
Control beams	15
CFRP strengthened beams	12
Ferrocement strengthened beam.	13

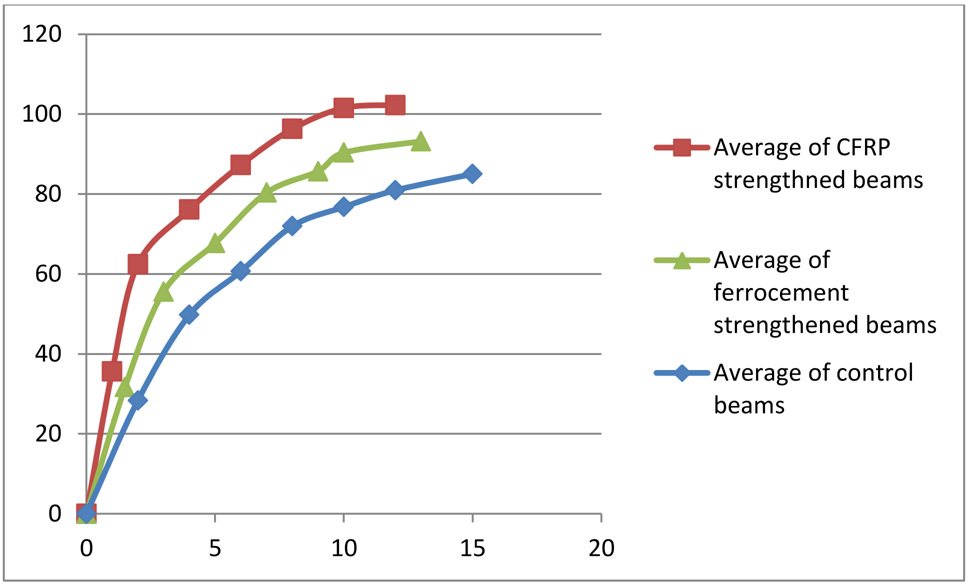

Fig 5.9. shear Load vs mid span deflection values for control and shear strengthened beam

5.3.3 Effect of shear retrofitting on crack width

The crack width at beginning of cracking stage and final stage are measured experimentally for average of control and shear strengthened beams. Table 5.6 shows the crack width for control beams,CFRP strengthned beams and ferrocement strengthened beams .

As the load increased, cracks propagated towards the inside of the section and also towards the supports, eventually causing the beams to break in shear due to concrete crushing in the mid-span region.

The control beams have significant flexure cracking, while the strengthened beams have a lesser crack width, according to the experiment conducted. At the ultimate loading, CFRP sheets decreased the crack width by 49%. At the highest loading, ferrocement laminates also decreased the crack width by 38.3 percent. The ferrocement-refitted beams developed cracks at a higher load than the control beams. When flexural/shear capacity, fracture width reduction ability, and overall ductility of CFRP and ferrocement reinforced beams were compared, the CFRP retrofitted beams surpassed the ferrocement strengthened beams.

Table 5.6. Crack width of control and shear strengthened beams

Designation	Maximum Crack width (mm)
Control beams	0.85
strengthened beams	0.65
Ferrocement strengthened beams	0.76

Chapter-6

Conclusion, recommendations and limitations

6.1 Introduction

The target of this investigation is to test effectiveness of strengthening methods for RC beams specially using CFRP laminates and ferrocement to enhance load carrying capacity of the beams. The flexural and shear behaviour of the reinforced beams are compared to that of the control beam. For RC beam reinforcement, one layer ferrocement and CFRP laminates are used. It has been noticed that application of CFRP and ferrocement is effective in reducing deflection and crack width of reinforced beams as well as improves their load carrying capacity. The following parts provide a summary of the works that has been carried in this study, as well as the conclusions.

6.1.1 Summary of the study

When compared to design requirements, RC structures deteriorate with age due to inherent material limitations related to material availability or environmental conditions. Overloading damage is considered in this work. The goal of this research is to reinforce RC beams with one layer of ferrocement and CFRP sheets to increase ultimate load carrying capacity along with reducing deflection and crack width.

For this investigation, a total of 12 RCC beams with dimensions of 700 x 150 x 160 mm has been caste and tested. The beams are designed to be under-reinforced so that the reinforcing bars yield before the concrete crashes. For flexural reinforcement, two bars of 10 mm diameter on the tension side and two bars of 8 mm diameter on the compression side have been provided. Shear reinforcement in the form of 8mm diameter stirrups at 100mm centre to centre spacing has also been provided. The beams were constructed using M20 grade of concrete and Fe-415 steel grade.

Under static loading, the control beams were examined for flexural and shear behaviours and then the other beams than control beams were reinforced with CFRP and ferrocement to increase the load bearing capacity, reduce the crack width and decrease the deflection of the original beams. The results were reported, and a performance-based comparison of the control and strengthened beams has been conducted.

6.1.2 Conclusions of results

- Compared to control beams, ferrocement and CFRP laminates attached to the soffit of RC beams significantly improved flexural and shear strength capacity while deflection and crack width is reduced for both cracking load and ultimate load levels.

- It has been found that the bending strength has improved. Ultimate strength rose more than 20% and 9.5 %, respectively, when CFRP and ferro-cement reinforced beams were compared to control beams . Ferrocement was shown to breakdown into layers faster than CFRP beams. As a result, all beams experienced flexural failure at the mid-span, as expected.

- The beams strengthened for flexure, CFRP material has improved the defection behaviour of the beam in this study and it’s been observed a reduction in the original deflection by 18.75% at the ultimate stage, whereas ferrocement laminates has reduced the original deflection by 6.25% at ultimate stage. In the case of shear strengthened beams, CFRP material has improved the defection behaviour of the control beams in this study by reducing the original deflection by 20% at the ultimate stage ,whereas ferrocement laminates has reduced the original deflection by 13.33% at the ultimate stage.

- Comparing CFRP and ferrocement strengthened beams to control beams, the ultimate shear capacity is found to be more than 16.3%, and 8.2 % respectively in CFRP, and ferrocement strengthened beams.

- Throughout this investigation, it was discovered that the usage of CFRP in the beam delays the development of early cracks as well as the progression of cracks. Furthermore, the ductile behaviour of FRP provides us with sufficient warning prior to final breakdown. All of the test specimens showed reduction of crack widths, massive deflection at the ultimate load, a significant increase in the ductility ratio, and increase in energy absorption after strengthening, which is indicating that the components were better fitted to withstand to their ultimate capacity for both the cases of CFRP and ferro-cement.

- Failure-flexural stress occurred in the centre of all beams, as expected and the flexural region showed the most cracking for both cases. CFRP reinforced components absorb more energy than ferrocement reinforced beams, according to a comparison of the two materials. Both materials produced positive outcomes. In the case of strengthened beams for flexure, ferrocement laminates has reduced the original crack by 16.85% at the ultimate loading. Whereas CFRP material has reduced the crack width by 31.5% at the ultimate loading as well.

6.2 Recommendations for future study

More research is needed to improve understanding the behaviour of RC members reinforced with ferrocement and Fibre Reinforced Polymer laminates, as well as to follow up on the findings presented in this work under durability effect. Some of the areas for further research are highlighted below.

- Torsion analysis of a CFRP and ferrocement strengthened beam..
- Investigations application Hybrid fibre reinforced polymer for strengthening
- Effect of quality of cement morter on structural performance of ferrocement.
- Investigation of the dynamic behaviour of CFRP and ferrocement reinforced beams.
- Effect of Curtailment of CFRP laminates in various patterns..
- Effect of different thickness CFRP laminates on RC flexure and shear Beams that have been subjected to both static and dynamic loading.

6.3 Limitations

- This research is limited to rectangular RC beams with consistent ferrocement and CFRP laminate thicknesses.
- Despite the fact that there are several factors that can cause a member to los load carrying capacity, only overloading problems has been investigated in this study.
- The effectiveness of ferrocement and CFRP is evaluated for reinforcement of undamaged beams but not for RC beams with various degrees of damage.

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