OPTIMUM DESIGN OF THE THICKNESS OF CARBON FIBRE REINFORCED POLYMER MATERIAL (CFRP) REQUIRED FOR STRENGTHENING OF DEFICIENT REINFORCED CONCRETE BEAM

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A DISSERTATION SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES, AHMADU BELLO UNIVERSITY, ZARIA IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF MASTER DEGREE IN CIVIL ENGINEERING

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DECEMBER, 2019

Declaration

I declare that the work in this thesis titled "Optimum design of the thickness of Carbon fibre reinforced polymer material required for strengthening of deficient reinforced concrete beam" has been carried out by me in the Department of Civil Engineering under the supervision of Dr, Amana Ocholi and Professor Y.D Amartey.The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree or diploma at this or any other institution.

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Nuruddeen Muhammad

Certification

This thesis titled **"Optimum Design of the thickness of Carbon Fibre Reinforced Polymer Material Required for Strengthening of Deficient Reinforced Concrete Beam"**by NuruddeenMUHAMMAD meets the regulations governing the award of the degree of Masters of Civil Engineering of the Ahmadu Bello University, and is approved for its contribution to knowledge and literacy presentation.

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Dedication

I dedicate this research to the memory of my late parents, Engr.Muhammad MuazuAugie and MalamaZainabIbrahim.May the Almighty forgivethem,Ameen.

Acknowledgements

I thank Almighty Allah for guiding me and making this research a success .With great pleasure,I wish to express my sincere gratitude to Dr. Amana Ocholi and Professor Y.D.Amartey who supervised and guide me throughout the research,iam immensely grateful and cannot appreciate them enough for the sacrifices made in terms of the time invested in ensuring the study became a success. My sincere appreciation goes to my late parents Engr Muhammad MuazuAugie and MalamaZainabIbrahim for training and supporting me both financially and morally throughout my education, may the Almighty reward them abundantly. My profound gratitude also goes to Alh.Abubakar Ibrahim Bubuche and HajiyaRakkiya for their immense contribution to the success of this research. I wish to acknowledge all my family members for their prayers and support during the course of this research. My sincere appreciation also goes to my immediate younger brother Dr, Yunusa Muhammad for supporting me both morally and financially throughout the period of this reaseach.My profound gratitude also goes to Engr Ishaq D. Muhammad and Architect Nasir Bello for their immense support and contribution towards the success of this research. I thank all my MSc course mates and the entire staffs of the Department of Civil Engineering Ahmadu Bello University Zaria who in one way or the other contributed towards the success of this research.

Abstract

A Carbon fiber-reinforced polymer (CFRP) has emerged as promising material for rehabilitation of existing reinforced concrete structures. However, one of the major concerns of using this material is its high cost. In this research, a classical optimization technique was employed to optimize the thickness of Carbon Fibre Reinforced Polymer material used in strengthening of shear and flexuraldeficient reinforced concrete beam. A simply supported reinforced concrete beam was designed in accordance with Eurocode 2 design criteria;the beam was subjected to shear deficiency and subsequently flexural deficiency at 25% capacity reduction level. Strengthening of the beam was achieved using externally bonded system with CFRP material of 1.2mm design thickness and in accordance with Intelligent sensing for imaging structures(ISIS2004) strengthening guidelines. Results indicate that the application of CFRP material increased the overall shear and flexural resistance of the beam by 95% and 79% respectively.Generalised Reduced Gradient (GRG) method was employed for the optimization. An optimum CFRP thickness of 0.06mm and 0.41mm was obtained from the result of the GRG program implemented for shear and flexural strengthening of the deficient Reinforced Concretebeam. Sensitivity analysis was carried out to determine the influence of parameters such as load ratio, steel reinforcement ratio and different CFRP elastic modulus on the optimum thickness of the material at 10, 20, 30, 40, 50 60, 70, 80, and 90% capacity levels. CFRP modulus of 25GPa, 50GPa, 75GPa 100GPa125GPa and 150GPawas considered in the analysis. The results indicate that the optimum thickness of the FRP material depends largely on the magnitude of load ratio, the elastic modulus of the CFRP material and the steel reinforcement ratio.

Table of Contents

List of Figures

List of Tables

List of Appendices **83**

Notations

ACI American Concrete Institute

A_{frp} Crossectional area of the FRP material

A_s Area of steel reinforcement required

A_{smin} Minimum area of steel required

b_w Width of the FRP material

CFRP Carbon fibre reinforced polymer

d Effective depth

d_{frp} Effective depth of the externally bonded FRP material

E_{frp} Modulus of elasticity of the FRP material

E_s Modulus of elasticity for steel

f_{Ck} Characteristic cylindrical strength for concrete

FRP Fibre reinforced polymer

f_{yk} Characteristic yield strength for steel reinforcement

Gk Magnitude of dead load

L Span of the beam

L_e The effective anchorage length

MA Applied bending moment due to loadings

Qk Magnitude of live load

R Reduction factor

RC Reinforced concrete beam

S_{frp} Spacing of the FRP material placed externally

S_{max} Maximum spacing for vertical shear

t_{frp} Thickness of FRP material

V_A Applied shear due to dead load and self-weight of the beam

V_c Contribution of shear resistance due to concrete

v_{Ed} Design shear stress

V_{FRP} Contribution of shear resistance due to FRP

V_r Factored shear resistance for the strengthened beam

V_S Contribution of shear resistance due to steel

Z Section modulus

ϕRn Design strength of the existing member without the FRP material

$\emptyset_c$ Material resistance factor for concrete

$\emptyset_{frp}$ Material resistance factor for the FRP material

$\emptyset_S$ Material resistance factor for steel

α Reduction coefficient for effective strain

λ Reduction factor depending on the density of concrete

ε_{frpe} Effective strain in the FRP material

ε_{frpu} Ultimate strain of FRP material at failure

β Angle of shear reinforcement to the longitudinal axis of the beam

ρ_{frp} Fibre reinforced polymer Shear reinforcement ratio

CHAPTER ONE

INTRODUCTION

1.1 General

Deterioration of reinforced concrete bridge decks, beams, girders, columns, buildings, parking structures and others may be attributed to ageing, environmentally induced degradation, poor initial designor construction, lack of maintenance, and accidental events such as earthquakes(Task Group 9.3 2001). Fullstructure replacement might have determinate disadvantages such as high cost for materialsand labour, a stronger environmental impact and inconvenience due to the interruption of thefunction of thestructure. When possible, it is often better to upgrade the structure by retrofitting (yasmeen 2010).The two most common failure modes resulting from the above phenomenon in reinforced concrete members are shear and flexural failure, which constitute a major challenge in safety and economy.

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Composite materials have been used in construction for centuries. One of the first was the use of straw as reinforcement in mud and clay bricks by the ancient Egyptians (khabari,2007).Carbon Fibre reinforced polymer (CFRP) composites consist of two main constituents, a load bearing constituent, mainly fibers, and a polymeric matrix that serves as a binder and protector of the fibers. The matrix facilitates load transfer among fibers, and ensures that embedded fibers maintain their orientation and directional stability(Ansley 2009). Properties of CFRP composites have some variation depending on the manufacturing and fabrication processes employed (Elarbi, 2011). Three types of reinforcing fibers are available commercially; glass, aramid, and carbon fibers. Each type has different grades with varying properties. Carbon fibers have the highest modulus while glass fibers have the lowest modulus and largest elongation. Early fiber composite applications have mainly been limited to aerospace, chemical and shipbuilding due to cost and research limitations (Eamon 2012). The use of FRP was first extended to civil engineering in the 1980's and are increasingly been used in strengthening and retrofitting of reinforced concrete structures due to ease of application, cost effectiveness and efficient performance (karbhari 2007).

For structural applications, CFRP is mainly used in two areas. The first area involves the use of fibre reinforced polymer bars instead of steel reinforcing bars or pre-stressing strands in concrete structures. The other application, which is the focus of this thesis, is to strengthen structurally deficient structural members with external application of CFRP material (ISIS Canada 2004). The existence of optimization methods can be traced to the days of Newton, Bernoulli, Euler, Lagrange and Weirstrass. Inspite of their early contributions, very little progress was made until the middle of the twentieth century, when high-speed digital computers made implementation of the optimization procedures possible and stimulated further research on new methods. An optimal solution normally implies the most economic structure without impairing the functional purposes the structure is supposed to serve (Rafiq 1995).

In recent years, few studies have been conducted on shear and flexural strengthening of Reinforced concrete beams using Carbon Fibre Reinforced Polymer (CFRP) material, However, such studies does not include the optimum thickness of the material required in relation to the level of deterioration of the member in shear or flexure, hence the need for this research.

1.2 Statement of the Problem

The high cost of CFRP composite material remains an issue of major concern by users globally. This development has made the market less competitive and less attractive for decades. Although concrete structures are reinforced with steel for strength, corrosion of steel reinforcement embedded in concrete structures has been the main problem of steel reinforced concrete structures(Abejide and Okoro 2013).

Numerous deterioration mechanisms can lead to shear or flexural deficiency in reinforced concrete members which may result in serious durability problems and ultimately reduce the load carrying capacity of the structure. Several investigations confirmed that the rate of deterioration slowed down significantly when FRP composite materials were used externally in reinforced concrete members (Mufti *et al.,* 2005, Sheikh *et al.,* 1997).

Although the use of steel plates providedan insight into the increase in flexural, shear strength and stiffness capabilities of externally applied reinforcement, steel plates had some disadvantages. Some of the most notable ones are' difficulty in handling during installation, possibility of corrosion affecting the bond surface at the steel/adhesive interface; and the problem of forming clean butt joints with relatively short plates, (Meier and Kaiser 1991).

Hence all over the world there are availability of large number of deteriorated reinforced concrete structures whose repair is constrained by lack of suitable local material and poor economic consideration in design and construction of their strengthening system.

1.3 Justification for the Study

It is a challenge for engineers to designan efficient and cost-effective systems without compromising the integrity of the system. Over the past two decades, there have been significant developments in the use of fibre reinforced polymer material in civil engineering applications, for example, maintaining and upgrading of existing infrastructural facilities, such as buildings, bridges, transmission lines, water supply, and sewer lines. FRPs are also used in aerospace and automotive industries,(Karbhari 2007).

Fiber-reinforced polymer composites (FRPs) have played a significant role in structural engineering applications (Karbhari and Zhao 2000, Mufti *et al.* 2005).For safety and economic consideration, structural rehabilitation of reinforced concrete members using CFRP material is considered as a viable option rather than demolition and reconstruction (Yasmeen 2010).

Moreover,for economy in design and construction, there is availability of modern optimization techniques (both Deterministic and probabilistic) with high speed and accuracy.

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1.4 Aim

The Aim of this research is to evaluate an optimum thickness of Carbon Fibre Reinforced polymer material required for strengthening of deficient reinforced concrete beam.

1.5 Objectives

The specific objectives of this research includes:-

i. To design and strengthen a simply supported reinforced concrete Beam in accordance with Eurocode 2 and ISIS Canada (2004) strengthening criterion

ii. To formulate and determine the Objective function for the strengthened Reinforced concrete beam

iii. To perform an optimization of the thickness of CFRP material using Generalized Reduced Gradient (GRG)Method.

iv. To carryout sensitivity analysis to examine the influence of changes in design variables on the optimum thickness of the FRP material

1.6 Scope and Limitation

The scope of this research covers optimization of the thickness of carbon fibre reinforced polymer (CFRP) material required for shear and flexural strengthening of a reinforced concrete beam using generalized reduced gradient method. It also includesa sensitivity analysis to examine the changes in design variables on the optimum thickness of the material. The research is limited to simply supported reinforced concrete beam in shear and flexural deficiency.

CHAPTER TWO

LITERATURE REVIEW

2.1 Preamble

Due to a growing need for innovative and alternative methods for the rehabilitation of existing concrete structures, recent studies have focused on the use of Fiber Reinforced Plastics (FRPs) as strengthening materials for these structures. Retrofitting with adhesive bonded FRP has been established around the world as an effective method applicable to many types of concrete structural elements such as columns, beams, slabs and walls, (Ashour*et al.*,2004). Early post-strengthening techniques of reinforced concrete beams, which have been studied and used in field applications, were directed toward the use of epoxy bonded steel plates. Although the use of steel plates provided an insight into the increase in shear and flexural stiffnessof reinforced concrete members, steel plates had some disadvantages, Some of the most notable ones are' difficulty in handling during installation, possibility of corrosion affecting the bond surface at the steel/adhesive interface; and the problem of forming clean butt joints with relatively short plates, (Meier and Kaiser 1991).

2.2 Fibre Reinforced Polymer materials

Fibre reinforced polymer consist of mainly a Fibre and a polymeric matrix. The fibreis the load bearing component while the matrix serves as a binder and provide orientation and dimensional stability to the fibre. The FRP essentially acts as additional Reinforcement. The major difference as compared to traditional reinforcing bars is that the strength of FRP is much higher; the behavior of most FRP is linear elastic to failure as opposed to mechanical bond between bars and concrete. While the reinforcement fiber is largely responsible for determining key structural properties, such as tensile strength and stiffness in the fiber direction, the successful performance of a composite relies greatly on both constituent phases. High-level composite performance can be achieved only through correct selection of both the fiber reinforcements and the polymer matrix (resin) that binds them into a cohesive structural unit. The primary purpose of the reinforcement in a polymer composite system is to improve the strength and stiffness of the system. Fiber reinforcements possess strength and stiffness properties two or three orders of magnitude above that of the neat polymer resin. However, fiber reinforcement is essentially a cable-type element, in that it possesses excellent structural characteristics under tensile loading but little or no compressive or shear capacity when acting in isolation.

The purpose of the resin is to bind the reinforcement fibers into a single cohesive structural system. In doing so, the resin must hold the reinforcement in place and act as a path for load transfer between the fibers through a combination of adhesive and cohesive characteristics, the resin enables the development of a single material system. The new system provides not only tensile capacity but also compressive and shear capacity. The polymer matrix in addition serves to protect the reinforcement fiber from adverse environments. The reinforcement component of FRPs consists of a number of individual long micro-fibers which constitute about 40 to 65% of the total volume of the material. Depending upon their orientation; the fibre materials are responsible for strength and dimensional stability of the FRP material.Fibre reinforced polymer material has the desired mechanical characteristics of providing both shear and tensile strength when used in structural application such as structural retrofitting.

2.2.1 Types of fibre materials

Several types of fibre materials are available commercially with each having different mechanical properties and areas of application, some of the most commonly available ones used in structural engineering applications includes: glass, carbon and aramid fibers, carbon fibre however has the highest tensile modulus, elastic modulus accompanied with light weight and resistance to corrosion.

1) Carbon fibers:-Carbon fibreare used in a variety of structural engineering applications in different forms and shapes, such as strengthening strips, sheets and fabrics, and in prestressing tendons. Carbon fibers, also known as graphite fibers, consist of more than 90% by weight of carbon. Carbon fibers have the highest elastic modulus of all reinforcing fibers, and have an advantage of an exceptionally high tensile strength-to-weight ratio along with a high tensile-modulus to weight ratio,In addition; carbon fibre has fatigue strength and very low coefficient of linear thermal expansion and in some cases even negative thermal expansion. These features provide dimensional stability which allows the composite to achieve near zero expansion to temperature as high as 300^0C in critical structures such as space craft antenna. If protected from oxidation, carbon Fibre can withstand temperatures as high as 2000^0C.Above this temperature, they will thermally decompose. Carbon fibres are chemically inert and not susceptible to corrosion or oxidation at a temperature below 400^0C (Amateau2003).Carbon fibre possesses high electrical conductivity, which is quite advantageous to the air craft designer who must be concerned with the ability of an aircraft to tolerate lightning strikes. One of their key disadvantages is their low impact resistance and high cost, (Mallick1993).

2) Glass Fibers: -Glass fibres has roughly comparable mechanical properties with other fibres such as carbon fibre.Althouh not as rigid as carbon fibre,it is much cheaper and significantly less brittle when used in composite. Theyare used in the majority of large or low-cost composite applications. The main characteristics of glass fibers are their moderate strength, moderate elastic modulus, density, and low thermal conductivity. Silica (SiO_2) is the principal ingredient in all types of glass formulations, and it is about 50% to 70% by weight of the glass(Amateau2003).

3) Aramid Fibers:-According to Deb Kim (1995), Aramid fibers consist of aromatic poly-amide molecular chains. Theybelong toa class of heat-resistance and strong synthetic fibres. The chain molecules in the fibres are highly oriented along the fibre axis. As a result, a higher proportion of the chemical bond contribute more to the fibre strength than in many other synthetic fibres.Aramide fibres are not ascommonly used as the other two types of fibers, because of their difficulty in processing, high moisture absorption (up to 10% of the fiber weight), low melting temperatures, poor compression properties and sensitivity to ultraviolet rays, which indicate low durability over time as compared with other types of fibers(Deb Kim 1995). Some of the notable advantages of Aramide fibres includes: extremely high toughness, and lightness in weight, with a density of about 1.4 g/cm^3.According to Hillermeier and Karlheinz (1984),Aramide fibres are mainly used in aerospace and military applications, in bicycle tires, marinecordage, marine hull reinforcement and as substitute for asbestos .The principal properties of the various types of fibers are summarized in Table 2.1.

Table 2.1: Typical Properties of Fibre Materials

	Material	Elastic modulus (GPa)	Tensile Strength (MPa)	Ultimate Tensile Strength (MPa)
CARBON				
	High strength	215-235	3500-4800	1.4-2.0
	Ultra High Strength	215-235	3500-6000	1.5-2.3
	High modulus	350-700	2100-2400	0.5-0.9
	Ultra high modulus	500-700	2100-2400	0.2-0.4
GLASS				
	E	70	1900-3000	3.0-4.5
	S	85-90	3500-4800	4.5-5.5
ARAMID				
	Low modulus	70-80	3500-4100	4.3-5.0
	High modulus	115-130	3500-4000	2.5-3.5

Source: (Foulds 1981)

From Table 2.1, Carbon fibers have the highest elastic modulus of all reinforcing fibers with ultra-high modulus ranging between 500 to 700 GPa .Carbon fibre also has an additional advantage of an exceptionally high tensile strength-to-weight ratio along with a high tensile-modulus to weight ratio, high fatigue strength and resistance to damaging effect of weather which is essential in prevention againstcorrosion and deterioration.

2.2.2 Polymeric Matrix

In an FRP composite, fibers are encased in a tough polymer matrix, which serves to hold the fibers together in a structural unit and distributes the imposed loads to many neighboring fibers within the composite, (karbhari 2007). The matrix also protects the fibers from environmental degradation due to moisture, ultraviolet radiation, corrosive chemicals, and to some extent from susceptibility to fire. Selection of the appropriate matrix material for environmental durability is critical in ensuring the longer term viability of a composite system, particularly in harsh service environments such as off-shore and shoreline applications, chemical plants, and cold climates where products such as de-icing salts are used,(GangaRao*et al.*,2007).

The polymer matrix provides all the inter-laminar shear strength of the composite, as well as resistance to crack propagation and damage. It can also be used to contribute properties such as ductility, toughness, or electrical insulation. The resin also affects the temperature performance of the material, typically determining properties such as the maximum service temperature. Most ofthe matrix materials are polymeric resins, which are used to cure and form the matrix. The curing process is essential to concrete FRP bond strength and to the overall durability of the laminate system. The properties of the matrix materials depend on the type of resin used. Several types of resins exist with different properties, manufacturers' specifications and recommended areas of application. These resins can be classified in several ways, based on their molecular structure, or their reactions to heating and cooling processes.Diffrent types of resins exist commercially,the most common types of matrix materials and their properties are summarized in Table 2.2

Table 2.2 Typical properties of common resin matrices.

Properties	TYPES				
	Polyester	Epoxy	Vinyl ester	Phenolic	Poly-Propylene
Tensile strength (MPa)	50-75	60-85	75-95	60-80	30-37
Modulus of elasticity (GPa)	3.0	3.0	3.2	-	1.1-1.6
Strain at failure (%)	5	3-6	2-6.7	-	2-7
Density (kg/m3)	1200	1200	1120	1240	900-9100

Source: Bank (2006).

The main types of resins for FRPs in structural engineering applications are unsaturated polyesters and epoxies. The selection of a resin system for a matrix for an FRP depends on a number of characteristics; the most important among them are the adhesive properties, mechanical properties, and durability characteristics (micro-cracking resistance, fatigue resistance, moisture degradation and damage due to ultraviolet radiation.

2.3.0 Choice of Adhesives Material

The purpose of the adhesive is to provide a shear load path between the concrete surface and the composite material, so that full composite action may develop. The science of adhesion is a multi-disciplinary one, demanding a consideration of concepts from such topics as surface chemistry, polymer chemistry, rheology, stress analysis and fracture mechanics. Epoxy adhesive is the result of mixing an epoxy resin (polymer) with a hardener, (GangaRao*et al.*,2007). Depending on the application demands, the adhesive may contain fillers, softening inclusions, toughening additives and others. The successful application of an epoxy adhesive system requires the preparation of an adequate specification, which must include such provisions as adherent materials, mixing/application temperatures and techniques, curing temperatures, surface preparation techniques, thermal expansion, creep properties, abrasion and chemical resistance.

When using epoxy adhesives there are two different time concepts that need to be taken into consideration. The first is the pot lifeand the second is the open time. Pot life representsthe time one can work with the adhesive after mixing the resin and the hardener before it starts to harden in the mixture vessel, for an epoxy adhesive, it may vary between a few seconds up to several years (Keller 2003). Open time is the time that one can have at his/her disposal after the adhesive has been applied to the adherents and before they are joined together. Another important parameter to consider is the glasstransition temperature, most synthetic adhesives are based on polymeric materials, and as such they exhibit properties that are characteristic for polymers. Polymers change from relatively hard, elastic, glass-like to relatively rubbery materials at a certain temperature. This temperature level is defined as glass transition temperature, and is different for different polymers. Epoxy adhesives have several advantages over other polymers as adhesive agents for civil engineering use (Hollaway and Leeming1999).Some of the most notable advantages of epoxy adhesives includes the following:-

1. High surface activity and good wetting properties for a variety of substrates
2. May be formulated to have a long open time
3. High cured cohesive strength; joint failure may be dictated by adherent strength
4. May be toughened by the inclusion of dispersed rubbery phase
5. Lack of by-products from curing reaction minimizes shrinkage and allows the bonding of large areas with only contact pressure
6. Low shrinkage compared with polyesters, acrylics and vinyl types
7. Low creep and superior strength retention under sustained load
8. can be made thixotropic for application to vertical surfaces
9. Able to accommodate irregular or thick bond line

Careful preparation such as removing the cement paste grinding the surface by using a sander, removing the dust generated by surface grinding using an air blower and careful curing are critical to bond performance. Epoxy resins are a broad family of material that provides better performance as compared to other organic resins. Epoxies generally outperform most other resins types interms of mechanical properties and resistance to environmental degradation (mallick1993).

2.3.1 Adhesive properties of resin

Adhesive properties of a resin system are important in achieving the required mechanical properties of an FRP composite. Adhesive properties of a resin in FRPs, involving adhesion of the resin matrix to the fibers in an individual ply and within the subsequent plies of a lamina, are important. The matrix adhesive compatibility with other bonding agents is also very important to develop interfacial shear stresses between the FRP and other substrates (GangaRao*et al.*,2007). A polyester resin is normally used to manufacture pultruded FRP profiles/sections used in structural engineering applications. The strength and modulus of polyester resins are lower than those of epoxy resins. Vinyl-ester resins are unsaturated resins, and are produced by reactions between the epoxy resin and acrylic or meth acrylic acids, which produce an unsaturated phase and render themvery reactive. Vinyl-ester resins offer excellent corrosion resistance and higher fracture toughness than epoxy resins (GangaRao*et al.*,2007). Epoxies normally consist of two separate parts - an epoxy resin and a hardener. These epoxy-based resins have excellent resistance to aggressive chemicals, and exhibit a low shrinkage of about 2 to 3% Partial details are summarized in Table 2.2.Keller (2003).

Epoxies are used in high performance composites to achieve superior mechanical properties and resistance to corrosive environments (GangaRao*et al.*,2007), and are usually used for strengthening and repair applications, to attach FRP laminates to the existing under-strength concrete members, and for manufacturing of FRP tendons for prestressed concrete. FRP materials have been used for compressive, flexural, and shear strengthening of reinforced concrete structures since the late 1970s. FRP strengthening has since been established as a potentially efficient and economical technique for the repair and rehabilitation of deteriorating concrete structures, including bridges.

2.4 Durability of CFRP Materials

The ability of FRPs to resist the surrounding environmental factors such as, moisture, Temperature, wetting and drying cycles, freezing and thawing cycles, ultra violet radiation and other factors, such as chemical attack, alkaline environment of concrete, creep, stress relief, fatigue and fire along with the age of the structure" (Karbhari2007). In case of an FRP subjectedto any of the above factors separately, or in combination with one factor(s), the mechanical,Physical or chemical properties will be negatively influenced with the passage of time (ECDC2005).FRPs can be affected by a variety of factors, including factors related to the natural andsurrounding environment, and the effect of individual factors, or their combinations. The literatureshows that durability of FRPs is a function of both the resin and the fiber. It should be re-emphasized that durability of a material or structure should be defined as its "ability to resistcracking, oxidation, chemical degradation, delamination, wear and/or the effects of foreign objectdamage over a specified period of time, under the appropriate load conditions, under some specifiedenvironmental conditions,(karbhari*et al*.,2000).

However, Application of FRPs for strengthening, rehabilitating and retrofitting of deteriorated, structurally deficient concrete structures, or replacing of deteriorated members have become an efficient and a recommended solution for upgrading concrete structures. The strength properties of FRPs are one of the primary reasons for their choice in such applications; these uniqueproperties include the high tensile strength, the light weight, resistance to the electrochemical corrosion and formability, (Mufti *et al.,* 2005).

2.4.1 Durability against water

According to Bank *et al.*, (1995), When FRPs are exposed directly to rain, humidity, moisture, and are partly or fully immersed, some moisture can diffuse into the composite which leads to changes in its thermo-physical, mechanical and chemical characteristics. This can occur in the case of external column jackets used around bridge columns, or structures used in harbour, quality and service-life issues for composite decks that could be subjected to ponding or overflow in times of heavy rain(Wu *et al.,*2006). The Absorption of moisture depends on the type of fiber and their orientation, the type of polymeric matrix, the matrix-fiber interface, the quality and composition of FRP sheets and the method of manufacturing(Karbhari and Zhang 2003). FRPs swell when they absorb moisture; the deterioration of a polymer matrix can occur when water molecules act as resin plasticizers and disrupt the Vander Waals bonds in polymer chains, causing de-polymerization of the matrix.

The primary effect of absorption of moisture in the resin results from hydrolysis, plasticization andother mechanisms that cause both reversible and irreversible changes in the resin polymer structure. The effects of moisture absorption are plasticization, polymer matrix cracking due toswelling stresses, resulting in loss of integrity, reduction of glass transition temperature, strength and stiffness. Moisture and chemicals can cause degradation in aramid and glass fibers. In glass fibers, degradation is initiated by moisture extracting ions from the fiber, thereby alteringits structure. The moisture contents and diffusion coefficients of composite materials can bedetermined by a variety of tests which have been reported in the literature (Bank*et al.*,1995; Karbhari2004; Saadatmanesh*et al.*,2010).However, it is possible to protect composite materials from rapid attack to a significant degree through the selection of appropriate resin systems, processing conditions, and the application of gel coats and protective coatings (Karbhari 2007). Since FRPs used in bridges come in contact with moisture and some solutions, either due to natural sources (rain and snow) or its location in rivers or streams, it is essential that both the short-term and long-term effects of these preventive measures are well understood.

2.4.2 Durability against freezing and thawing cycles

Fibre reinforced polymer material (FRP)can be subjected to a wide range of temperatures and moisture changes alongwith the effects resulting from applied loads(Keller 2003). Structures exposed to cold environments must be assessed for the effects of freezing and thawing cycles, which can be more severe when FRPs are exposed directly to the environment. Low freezing and thawing cycles with the low variation in temperature appears to cause greater deterioration than constant immersion at below freezing temperatures (Karbhari 2004).FRP materials are used in structural application as protectionagainst aggressive environment.

Exposure to freezing and thawing cycles can cause damage to FRPs as a result of cyclic expansion and contraction of the entrapped water, which can also significantly affect the bond between the lamina, and the bond between the FRPs and the concrete substrate. The thermal characteristics of the constituents used in FRPs are different; both fibers and polymeric matrix have different thermal expansion characteristics due to notable difference in their coefficients of thermal expansion. Most of the epoxy resins, used as matrices in FRPs, have coefficients of thermal expansion ranging from 45×10^{-6}°C to 65×10^{-6}°C, while the coefficient of thermal expansion of glass fibers is 5×10^{-6}°C and carbon fibers is in the range of - 0.2×10^{-6}°C to $+0.6 \times 10^{-6}$°C. This large difference in expansion coefficient produces residual stresses. As a result of the difference in thermal expansion, FRPs can undergo a tensile strain, resulting in micro-cracking of the polymeric matrix. These micro-cracks in the polymeric matrix propagate when subjected to freezing and thawing cycles.

An experimental investigation was conducted by El-Hacha and Omran (2012) to study the effects of freezing and thawing cycles on large-scale reinforced concrete beams strengthened in flexure with prestressed and non-prestressed FRP reinforcement. Five reinforced concrete beams, 5.15 m long, 200 mm wide and 400 mm deep, were tested under four-point transverse monotonically increasing loads at a rate of 1 mm/min rate of displacement. Before strengthening, these beams were cracked at a load of about 20 % above the analytical cracking load.

Two out of four pre-cracked beams were strengthened with one 9.5 mm diameter sand coated CFRP rebar, and other two beams with two 2x16 mm rough textured CFRP strips, which were glued together and installed in a groove which was made on the tension face of the beams. These CFRP strips were prestressed to a level of 40% of their ultimate tensile strength. These beams were exposed to 500 freezing and thawing cycles at the rate of three cycles per day with temperature variation of +34°C to -34°C. The results indicated that the freeze-thaw exposure caused 13.4% decrease in the energy absorption characteristics in comparison with that in unexposed beams.

The thermal incompatibility of the two materials caused development of residual stresses which led to the deterioration of concrete and the concrete-epoxy interface.

Brant (2008), studied the combined effects of freezing and thawing cycles on flexural performance of beams. Forty-five small-scale beams, 1220 mm long, with cross-sectional dimensions of 152 mm x 102 mm, longitudinally reinforced with 4-10M top and bottom bars andwith external strengthening with glass and carbon FRP laminates were prepared in the laboratory. The beams were designed and strengthened to ensure that FRPs failed before the overall failure of beams. These beams were subjected to a sustained loading of 72% of design loads, along with freezing and thawing cycles according to ASTM Standard C 666-97 procedure (from +5°C to -18°C and back to +5°C within a period of 5 hours to complete a single freeze-thaw cycle).

After 300 freezing and thawing cycles and about 3 months of sustained loading, the beams were tested under flexural loading. It was found that the beams subjected to combined loads did not suffer a significant loss in their average ultimate strength; there was a strength reduction of 1.4% for thebeams reinforced with GFRP sheets, 11.3% for the beams strengthened with CFRP sheets, and 7.6% for the beams strengthened with CFRP plates (Brant 2008). However, the greater inconsistency of the results for these beams relative to the control beams implies that lower guaranteed strengths should be used for situations with cyclic humidity and freezing/thawing conditions.

2.4.3Durability against ultraviolet radiation

According to Saadatmanesh (2010), FRPs can undergo photochemical damages near the exposed surface, because of ultra violet radiation when exposed to the natural sunlight. The effects of ultraviolet exposure, or photo-degradation, are usually confined to the top few microns of the surface; however, this degradation at the component surface affects mechanical properties of the FRPs disproportionately. The effect of ultraviolet radiation is also compounded by the action of temperature, moisture, wind-borne abrasives, freezing and thawing cycles and other environmental phenomena.Homan (2000) noted that photochemical damage led to discoloration and reduction in molecular weight; however, there was no significant damage to the FRPs. This is due to its ability to withstand strongly the harmful corrosion agents as they come in contact due to environmental effect.

2.4.4 Durability against alkalis

Alkalinity of the concrete protects embedded steel reinforcing bars in conventional reinforced concrete elements. FRPs can be adversely affected by this alkalinity, when exposed to the concrete. FRPs can come into contact with alkaline media through interaction with a variety of sources – including alkaline chemicals, soil and concrete. The main concern stems from the potential effects of degradation due to concrete pore water solution, which contains high hydrogen ion concentrations (pH of about 13.5). Over the last two decades, significant research has beenconducted on degradation of bare glass fibers in contact with, or immersed in alkaline solutions, especially those derived from the concrete (Wu *et al*.,20016) . It is well established that bare glass fibers in alkaline environment can get severely degraded due to a combination of mechanisms, ranging from pitting, hydroxylation, hydrolysis and leaching. The presence of resin as a matrix around an individual fiber in FRP can protect fibers from any such attack. The alkaline solutions can accelerate the degradation of the interface bond and of some resins themselves, especially if they are not fully cured (Karbhari *et al.*,2004).

Saadatmanesh*et al.*,(2010) carried out experimental investigation of the long-term behavior and environmental effects on mechanical properties of different types of FRP laminates produced using a wet lay-up technique. A total 525 coupons, 25 mm wide and 400 mm long were tested under uniaxial tension. These tension test specimens were obtained from seven different types of FRPs laminates made of unidirectional loose glass, unidirectional glass, unidirectional carbon, unidirectional hybrid glass-carbon, bidirectional glass, bidirectional hybridglass-carbon, and bidirectional hybrid glass aramid fibers. Nine different environments were Implemented as exposure conditions, including fresh water, saturated Ca(OH)2 solution with pH =12.5, saturated $Ca(OH)_2$ solution with pH = 10.0, HCl solution with pH = 2.5, seawater, moist alkaline soil with micro-organisms, dry air at a temperature of 60°C, air temperature of 50°C and RH of 95% and ultraviolet radiation. The study revealed a significant loss (about 40% to 65%) in the strength and ultimate strain for GFRP in environments with high pH values (pH = 12.5), while carbon and hybrid glass-carbon laminates showed very little loss (< 10%) in their mechanical properties. The study also suggested that GFRP laminates are not recommended for application in environments with high pH values.

2.4.5 Durability against fatigue

According to Karbhari(2007), Structures such as Bridges are always subjected to a variety of loadings, including dynamic cyclic loading due to moving traffic. The fatigue phenomenon is important for durability and safety of bridges. Every cycle of loading, due to passing of trucks or other vehicles over the bridge results in a very small amount of permanent or residual strain in the constituent materials (Bank 2006).Repeated loadings lead to gradual accumulation of small amounts of damage with each cycle of loading.

FRPs can experience micro cracking, delamination, fiber fracture, fiber/matrix decoupling and micro-buckling undercompressive loads. These damage modes can accumulate fatigue damage, and FRPs loses itsresistance, which eventually results in fracture. The fatigue performance of FRPs are influenced by properties of the constituents materials, bothfibers and matrix, the fiber/matrix interface or the inter-phase region, the manufacturing process, loading parameters (such as frequency, stress range, and stress ratio of permissible stress to thestress at ultimate load), and in-service environmental conditions, such as hydrothermal, chemical land thermal-mechanical effects. The fatigue performance of FRP laminate isaffected significantly by the fiber type, its orientation and the lay-up scheme. The matrix affects thetransverse directional and shears properties, and axially loaded unidirectional fatigue properties. For unidirectional laminates, the effect of the matrix is reflected in the failure mode. Epoxy-baselaminates tend to show very good fatigue resistance when compared with both polyester and vinyl ester-based laminates (GangaRao*et al.,* 2007; Karbhari 2007).

2.4.6 Resistance against creep and long-term sustained loading

For FRPs, creep or stress relaxation properties are governed by the resin matrix-dependent Characteristics, rather than fiber or fiber-matrix interfacial properties. Under-cured resins are susceptible to significant creep and possible micro-crack initiation during the early stages of any service environmental exposure. The absorbed moisture and higher service environmental exposure temperatures both enhance the creep damage susceptibility; however, proper curing of FRPs enhances durability against creep and long term sustained loading (Bank 2006; Karbhari 2007)

2.5 Effect of Corrosion on Reinforced Concrete Beams

According to Abejide and Okoro(2013), Reinforced concrete is recognized to be durable and capable of withstanding a variety of environmental condition, never the less failure of structures still do occur as a result of premature steel reinforcement corrosion. Althoughconcrete structures are reinforced with steel for strength, corrosion of steel reinforcement embedded in concrete structures has been the main problem of steel reinforced concrete structures. Reinforced concrete Structures are designed on the principle that steel and concrete act together to withstand induced forces,butcorrosion of steel reinforcement does not permit this action. Porous concrete will allow water and corrosive agents such as salt to penetrate and reach reinforcing steel. Once exposed to those corrosive agents, steel will start corroding and when rusting, it expands and cracks the concrete surrounding it. Reinforcement of concrete with steel is done to strengthen thestructural element in tension as concrete is weak in it, but structures do fail as a result of corrosion attack on steel (Mohammed*et al.*, 2003).According to Fontana (1986), Corrosion is a phenomenon which results in the deterioration or destruction of a material when they are exposed to different Environmental condition. Corrosion of concrete involves an electrochemical process in which both flow of electrical currents and chemical reactions occur. The steel in reinforced concrete structures is in passive conditions and are protected by a thin layer of oxide which is due to the alkalinity of concrete pH between 12 to 13,(Ormellese*et al.*, 2006, Mohammed*et al*., 2003, Vaysburd and Emmons 2004).

2.5.1 Corrosion mechanism in reinforcedconcrete

Corrosion in concrete is induced by the generation of theelectrochemical potentials in following ways:

1. When two different metals are present in concrete, such as steel rebars, aluminum conduit pipes, or when significant variation exists in surface characteristics of the steel, formation of composition cell can occur.

2. Concentration cells may be formed near reinforcing steel because of the differences in the concentration of dissolved ions, such as alkalis and chlorides, (Quraishi*et al*., 2011).

A according to Mehta and Monteiro, (2006), the following reactions occur at anode and cathode.

Anode: $Fe \rightarrow Fe^{2+} + 2e^-$ (Metallic iron) (2.1)

Cathode: $.O_2 + H_2O + 2e^- \rightarrow 2OH$ (2.2)

Some Parameters are essential to initiate corrosion. Presences of oxygen, humidity (electrolyte) are the two important parameters without which corrosion is not possible (Mullick 2004, Gaidis 2004).The rate of corrosion is slow if the amount of water or oxygen is limited. Presence of humidity, moisture and oxygen acts as catalyst for corrosion to occur, forming more OH- thereby producing more rust component (OH) (Quraishi*et al.*,2011).Equation 1 to 3 represent the formation of the rust after the iron dissolution occurs at the anodic sites in the reinforcement,(Isgor and Razaqpur2006).

$Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2$ (Ferrous Hydroxide) (2.3)

$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3$ (Ferric Hydroxide)(2.4)

$2Fe(OH)_3 \rightarrow 2H_2O + Fe_2O_3 \cdot H_2O$ (Rust) (2.5)

Following reactions (Equation 1 to 3) represent the formation of the rustafter the iron dissolution occurs at the anodic sites in the reinforcement (Isgor and Razaqpur2006).

2.5.2 Chloride induced corrosion

Chloride attack is one of the main reasons behind the corrosion of steel in reinforced concrete. Major source of Chloride ions (Cl^-) are de-icing salts or seawater (Mohammed *et al.*,2003). Cement, water, aggregate and sometimes admixtures can also facilitate chloride in concrete. Chlorides penetrates into the concrete through the pore network and micro cracks, forming the oxide film over the reinforcing steel and hence, accelerates the reaction of corrosion and concrete deterioration,(skoglund*et al.*,2008). The passivity of steel is broken when a sufficient quantity of Chlorides is present in the pore solution. The passivity of steel also depends upon the OH-concentration of the pore solution. Some author shows that the passivity is broken when the ratio of Cl-concentration to OH^- concentration exceeds a particular value (hussain*et al.*,1996). The mechanism of reinforcement corrosion in concrete due to chloride attack is basicallyan electrochemical process by which the passivation layer of steel is lost by means of formation of micro cells on the surface of steel by chloride ions. The moisture present in the pores of concrete acts as an electrolyte and the area adjacent to the concentration of chloride ions becomes cathode, thus starting the electrochemical process,(Ramesh and Reddy 2001).

Chlorides present in concrete in two forms, namely bound chlorideand free chloride. Among bound chloride, chemically bound chlorides are utilized in the hydration product of cement and physically bound chlorides are absorbed on the surface of the gel pores. This is important as only the free chlorides are relevant to the corrosion of reinforcement.

Quraishi*et al.*, (2011), defined the chloride threshold level as the ion concentration of steel bars in concrete provided that there are no damages at the rebar concrete interface, According to Ahmad (2003) Corrosion of steel reinforcement significantly depends on the ion concentration on the steel reinforcement embedded in concrete. The thin passive layer however developed due to the high alkalinity of concrete serves as a protection mechanism against corrosion in reinforced concrete members

2.5.3 Sulphate induced corrosion

Most soil contains somesulphate in the form of gypsum ($CaSO_4.2H_2O$) other sulphate in the form of calcium, magnesium, sodium and potassium(Mehta and Monteiro2006). Sulphate in the form of solid does not attack the concrete severely but in the solution form it can penetrate through concrete pores and react with hydrated products of cement. Due to Sulphate attack cement paste expand in concrete or mortar. Sulphate salt reacts with aluminates present in concrete in the form of calcium aluminate hydrate gel (C-A-H) and form calcium sulphoaluminate (also known as ettringite) within the hydrated cementpaste. As a result of this subsequent expansion of the solid phase concrete deterioration takes place. Cement contains some amount of alumina in the form of C_3A and little amount in the form of C_4AF.C_4AF shows higher resistance to sulphate attack compared to hydrate of calcium aluminates(Miller and Tang 1996; Mehta and Monteiro 2006.)

2.5.4 Chemical reaction in sulphate attack

When C_3A is more than 5% in Portland cement, hydration product contain alumina in the form of monosulphate hydrate, $C_3A.CS.H_{18}$. And if C_3A content of the cement is more than 8 percent, the hydration product will also contain $C_3A.CH.H_{18}$. When hydration takes place in the Portland cement calcium hydroxide is generated. This calcium hydroxide reacts with sulphateand both the alumina-containing hydrates form high sulphate named ettringite ($C_3A.3CS.H_{32}$) as shown by equation (2.6) and (2.7)(Mehta and Monteiro 2006).

$C_3A.CH.H_{18}$+2CH+ 3S+11H → $C_3A.3CS.H_{32}$(2.6)

$C_3A.CS.H_{18}$+2CH+2S+12H → $C_3A.3CS.H_{32}$ (2.7)

Calcium sulphate attack only calcium aluminates hydrate producing CalciumSulphuraluminates (3CaO. Al_2O_3. $3CaSO_4$. $32H_2O$) known as ettringite. Sulphate attack can be controlled through the following:-

- Using sulphate resisting cement
- Quality concrete
- Use of air-entrainment
- Use of pozzolana
- High pressure steam curing
- Using high alumina

2.6 Flexuralstrengthening of reinforced concrete beams

In the 1980's, research began at the Swiss Federal Laboratories for Material Testing and Research (EMPA) to investigate the feasibility of replacing steel plates with

CFRP plates for flexural strengthening of concrete beams, Meier1991, 1992). Meier and his fellow associates conducted tests on twenty-six beams with span lengths of 2 m and one beam with a span length of 7 m. The strengthening of beams was achieved by bonding a continuous unidirectional CFRP (0.3 mm to 1.0 mm thick) plate to the extreme tension face of the beams .The results of the tests indicated ultimate strength increase in the 2 m span beams of approximately twice the original strength. He stated that the beam having a 7m span had a more realistic steel reinforcement ratio, and the increase in strength of that beam was about 22 percent. The failure mode for all beam specimens was tension failure of the CFRP plates which occurred at beam deflections 50 percent lower than those of the strengthened beams.

The research by Tedesco*et al.*,(1996) was based on optimizing the length and orientation of the CFRP to increase beam strength and ductility. Eight beams (200x300x3000mm) were minimally reinforced with steel (two 10mm diameter bars) and overdesigned for shear to cause a flexural failure. One beam was used as a control while the others were bonded with three layers of CFRP (0.3x160mm). The results of the experiment showed an increase in strength and stiffness and a decrease in deflection as compared to the control beam. All failures occurred at a load at least 57% higher than the control beam.

Ritchie *et al.,* (1991) presented results from tests conducted on sixteen under reinforced concrete beams. The beams were tested under a two point loading with span length of 2.74 m. The research was designed to test the feasibility of reinforcing concrete beams with a variety externally applied FRP plates. In addition, a theoretical analysis was developed to see whether or not reinforced beam behavior could be accurately predicted for further development of design procedures. Various types of

FRP plates and fiber orientations were used by Ritchie *et al*, for flexural strengthening. The three types of fibers selected were glass, carbon, and aramid with orientations ranging from 0 to 90 degrees. The authors suggested that continuing the FRP plates to the supports would be more effective for beams with lower shear to moment ratios (longer spans). The results of the tests indicated that the load capacity and stiffness of the beams was moderately increased, and in some cases greatly increased by the application of the FRP plates. The increase in stiffness above that of the control specimens ranged from 17 to 99 percent over the service load range. Similar strength increases, from 19 to 99 percent, in the service load range were also seen. At ultimate, the increase in strength ranged from 28 to 97 percent despite premature brittle concrete shear failure occurring in many beams. Results from a theoretical analysis program developed to determine load deflection relationships using strain compatibility, proved fairly successful in the prediction of beam behaviors with bonded FRP plates.

Meier *et al.*,(1992) described results from two bridges in Switzerland reinforced with CFRP plates. The Ibach bridge, built in 1969 in the County ofLaurence, was damaged during installation of new traffic lights, when pre-stressingtendons in the outer web of a box girder were severed with an oxygen lance. Due to traffic considerations through the underpass, repairs had to be made atnight. Two 150 x 5000 x 1.75 mm and one 150 x 5000 x 2.00 mm CFRP sheetswere used in the rehabilitation of the damaged section. After load testing, it wasdetermined that the application of the CFRP sheets yielded satisfactory rehabilitation of the bridge girder.

Tedesco *et al.*, (1996) presented findings from a study conducted on a damaged multi-girder reinforced concrete bridge in Bullock County, Alabama. The study was conducted for the Alabama Department of Transportation (ALDOT). Results from static and dynamicload tests conducted before and after the application of the reinforcementindicated favorable service load strength and stiffness results.

Chajes*et al*., (1994) performed tests on fourteen under-reinforced concrete beams with externally bonded composite fabrics. Comparisons were made between beams having additional internal steel reinforcement and those having additional external fabric reinforcement. The beams were tested under two point loading with span lengths of 1.12 m. The results of the study indicated average increases in flexural capacity of 53, 46, and 45 percent for the beams strengthened with aramid, E-glass, and graphite fabrics, respectively. An average increase in flexural stiffness of 40 percent was seen for all externally reinforced beam specimens. Comparison of results of the additional internal steel reinforced beams with those of the external reinforced beams showed similarities in terms of strengthening. From comparison of the experimental results and theoretical calculations, the authors concluded that with the use of the analytical method, the ultimate flexural capacity, mode of failure and surface strains of the fabrics could accurately be predicted.

2.7Shearstrengthening of reinforced concretebeams

Uji (1992) carried out the tests of eight simply supported RC beams strengthened for shear with CFRP sheets using two different wrapping schemes; total wrap or two sides of the beam. He concluded that the application of CFRP substantially improves the shear capacity of RC members. He also found that the strains in the stirrups and the CFRP are different even at the same location. This is because a stirrup stretches evenly over its length, while only a limited area of CFRP stretches at the crack. Thus, the strain in CFRP is greater than in stirrups at the crack location.

Umezu*etal*., (1997)carried out an extensive experimental program in order to determine the effects of carbon and aramid FRP sheets on the shear capacity of simply supported RC beams. They used total wrap as strengthening scheme for all of their test beams. The application of FRP sheets was found to enhance shear capacity and deformation characteristics. In their analysis, they stated that the contribution of AFRP to shear capacity could be evaluated by the truss theory, based on an average stress of AFRP equal to the tensile strength of the sheet multiplied by a reduction coefficient, determined from the test results.

Khalifa and Nanni (2000) investigated the performance of RC beams strengthened in shear with CFRP sheets. they found that by using various configurations of CFRP sheets the shear capacity of the beam could be increased by 35 to 135% .it was also found that an optimum quantity of FRP exist beyond which strengthening effectiveness is uncertain. Strips of FRP applied only to the beam sides provided less strength enhancement than those bonded in U-shaped configuration.

Zheng *et al.*,(2005) tested three controls beams and eight RC beams without shear reinforcement strengthened externally by using CFRP strips and fabrics.Eleven RC beams having cross sectional dimension of 154x228.6mm2.The five beams had 1.22mm along and the other six beams had 1.83mm long, all the beams were designed to fail in shear. Five batches of concrete were used in the study, externally bonded CFRP were applied on both sides of the beam at various orientation with respect to the axis of the beam. The results show the feasibility of using an externally bonded

CFRP laminate system to strengthenor increase the ultimate shear capacity and ductility of the beam.Reseachers concluded that the beams with CFRP strips and those with CFRP fabrics have completely different failure mechanisms. The strips failed by debonding while the fabric failed by rapture of the fibre.The research shows that indicated that used strips increased the shear strength more than fabrics.

YasmeenTalebObaidat, (2010) studied the Retrofitting of reinforced concrete beams using composite laminates and the main variables considered are the internal reinforcement ratio, position of retrofitting and the length of CFRP. The experimental tests were performed to investigate the behavior of beams designed in such a way that either flexural or shear failure will be expected. The beams were loaded in four-point bending until cracks developed. The beams were then unloaded and retrofitted with CFRP. Finally the beams were loaded until failure. The ABAQUS program was used to develop finite element models for simulation of the behavior of beams. The concrete was modeled using a plastic damage model and two models, a perfect bond model and a cohesive model, were evaluated for the concrete-CFRP interface. From the analyses the load-deflection relationships until failure, failure modes and crack patterns were obtained and compared to the experimental results. The FEM resultsagreed well with the experiments when using the cohesive model regarding failure mode and load capacity while the perfect bond model was not able to represent the debonding failure mode. The results showed that when the length of CFRP increases the load capacity of the beam increases both for shear and flexural retrofitting. FEM results also showed that the width and stiffness of CFRP affect the failure mode of retrofitted beams. The maximum load increases with increased width.

Increased CFRP stiffness increases the maximum load only up to a certain value of the stiffness, and thereafter it decreases the maximum load.

Kaura (2017) carried out cost optimization of FRP strengthening of a simply supported reinforced concrete beam that lost part of its flexural strength due to factors such as corrosion. The cost was expressed interms of the required FRP material thickness in mm in conjunction with the material Elastic modulus, dead to live load ratio and the steel reinforcement ratio.Generalised reduced gradient method was employed for the optimization. The results shows that the lower the thickness of CFRP with particular value of Elastic modulus, the lower the flexural strengthening cost of the reinforced concrete beam.

2.8 Classical Search and Optimization Techniques Concept

Traditional search and optimization methods can be classified into two distinct groups: Direct and Gradient-based Methods (Deb, 1995; Reklaitis*et al*., 1983). In direct methods, only objective function and constraints are used to guide the search strategy, whereas gradient-based methods use the first and/or second-order derivatives of the objective function and/or constraints to guide the search process. Since derivative information is not used, the direct search methods are usually slow, requiring many function evaluations for convergence. For the same reason, they can be applied to many problems without a major change of the algorithm. On the other hand, gradient-based methods quickly converge to an optimal solution, but are not efficient in non-differentiable or discontinuous problems. The method is not also efficient in handling problems having discrete variables.

Because of the nonlinearities and complex interactions among problem variables often exist in complex search and optimization problems, the search space may have many optimal solutions, of which most are locally optimal solutions having inferior objective function values. When solving these problems, if traditional methods get attracted to any of these locally optimal solutions, there is no escape from it. Depending on the specific choice of design variables, objective functions, and constraints, various types of optimization problems may exist. Table 2.3 presents a classification of optimization problem which has been collected from various researchers, (Foulds, 1981, Rao, 1984, Arora, 1989, Haftka and Gurdal, 1992, Kirsch, 1993, Sarma and Adeli, 1998,Vanderplaats, 1999, Sarma and Adeli, 2000).

Table 2.3 Classification of Optimization Problem(Sarma and Adeli 2000)

Base for Classification	Category	Specification
Number of Design variables	Single variable	The vector of design variable includeOnly one variable
	Multi –variable	The vector of design variable includes more than one variable
Number of objective	Single objective	There is one criterion express as an

function		objective function
	Multiple objectives	There are many criterion which are considered to determine optimum solution
Presence of constraints	Unconstrained	A minimum or maximum of objective function without any limitation is expected
	Constrained	Some constraints define the set of feasible solution
Features of constraints and objective functions	Linear programming	Objective functions and constraints are linear
	Nonlinear programming	Some of the objective functions and constraints can be nonlinear.
Nature of design variables	Static	Design variables are independent. They are not functions of other parameters
	Dynamic	Design variables are functions of other parameters eg.time

2.8.1 Generalized reduced gradient method

In 1960, Wolfe developed the reduced gradient method based on a simple variable elimination technique for equality constrained problems (Abadieand Carpenter 1969). The generalized reduced gradient (GRG) method is an extension of the reduced gradient method to accommodate nonlinear inequality constraints. In this method, a search direction is found such that for any small move, the current active constraints remain precisely active. If some active constraints are not precisely satisfied because of nonlinearity of constraint functions, the Newton-Raphson method is used to return to the constraint boundary. Thus, the GRG method can be considered somewhat similar to the gradient projection method. Since inequality constraints can always be converted to equalities

2.8.2 The concept of GRG program

According to Leon *et al.,* (1973), the basic concept of GRG method involves linearization of non-linear objective and constraint functions at a local solution with Taylor series expansion. The nonlinear program to be solved is assumed to have the form:-

Minimize f(x) (2.8)

Subject to $g_i(x) = 0, i = 1, \dots, . m$ (2.9)

And $\not{C}_i \leq x_i \leq U_i$, i =1… n (2.10)

Wherexis n-vector and i, are given lower and upper bounds$U_i > \rho_i$ .It is assumed that $m < n$ since, in most cases, $m \geq n$ implies an infeasible problem or one with a unique solution. Equation 6-8 is completely general, since inequality constraints may always be transformed to equalities as in equation two, by the addition of slack variables. The vector x contains as components boththe "natural" variables of the problem and these slacks. The fundamental idea of GRGis to use the equalities in equation two to expressm of the variables, called basic variables, in terms of the remaining n-m non basic Variables. The methods use a successive search within the solution space which can be based on the objective function, its gradient information or both. They focus on optimization of continuous variables.

2.9FailuremodeforCFRPstrengthened RCmembers

To account for the complex behavior and various possible failure mechanisms of FRP bonded structures, extensive experimental investigations were carried out by numerous researchers,(Seible 1997, Mo *et al*., 2004, Nanni2004, Ludovico*et al*., 2005, Walker and Karbhari2006). For FRP bonded flexural beams, several failure modes were generally observed:

1. Crushing of the concrete in the compression zone before rupture of the FRP sheet or yielding of the reinforcing steel (Brittle failure).
2. Yielding of the tension steel before concrete crushing or rupture of FRP sheet (ductile failure).
3. Yielding of the compression steel reinforcement of a doubly reinforced section (relatively ductile failure).
4. Rupture of the FRP sheet before steel yield and the compressive strain in the concrete is below its ultimate strain (the most brittle failure).
5. Anchorage failure (delamination) in the bond zone of the FRP sheet (often a ductile failure).
6. Peeling or shear/tension failure of the concrete substrate near the FRP sheet's cut off zone (brittle failure).

The six failure modes were classified into two types by Thomsen (2004). Type one includes modes exhibiting composite action up to failure, either due to concrete crushing, FRP rupture, or lack of shear resistance. Type two consists of failures by loss of composite action due to debonding of the FRP sheet, or by end peeling, where the concrete cover near the support regions peels off. To avoid detachment failure at the FRP/concrete interface, ACI 440.2R (2008) introduces a bond reduction factor (km) to limit the strain permitted in the FRP system. However, due to the high cost of experimental research, values for the bond reduction factor were based on a limited number of experimental investigations.

Correspondingly, a critical factor potentially not adequately accounted for is the potential deterioration of the bond strength over time. According to ACI 440.2R, kmis taken as a value not greater than 0.9, which reduces the usable strength of the FRP below its ultimate rupture strain. Even so, FRP rupture or delamination might occur if the FRP or bond strength later deteriorates. Thus for the flexural design of externally strengthened reinforced concrete beams, four failure modes are assumed possible, two corresponding to failure of the concrete in compression and two corresponding to failure of the FRP sheet (Choi *et al.*,2008).

Concrete crushing after steel yields, concrete crushing before steel yields, steel yield followed by FRP rupture, and debonding of the FRP at the FRP/concrete interface.

The additional shear strength that can be provided by the FRP is based on several factors, including the geometry of the beam or column, the wrapping technique, and the quality of the existing surface of the concrete. There are three typical types of FRP wrapping schemes, depending on the number of sides wrapped (4, 3, or 2). The four-sided wrapping scheme is most efficient. However, this is often impossible for bridge members, such as in the case of a T-beam integral with a deck slab above. In such situations, shear strength can also be improved by wrapping three sides (U-wrap) or bonding to two sides of the member, though the latter is the least effective (ACI 440R 2007).

2.9.1 Delamination

A critical concern of externally bonded FRP reinforced structures exposed to severe weather environments is delamination, or debonding. Although detailed weather-induced bond deterioration is not fully understood, the results of debonding and its related failure phenomenon are well known. The quality of interfacial bonding has a strong influence on structural performance, as this significantly affects the composite action required for many applications, and ultimate failure of the strengthened component is often caused by debonding of the FRP sheet from the concrete substrate (Meier 1995, Buyukozturk and Hearing 1998).

The FRP-concrete bond line is the critical component to the effectiveness of most FRP structural strengthening applications, as this is the location where the transfer of stresses occurs. An exception to this would be a structural element that is confined with FRP, such as column wrapping. Field experience has shown that the bond between the composite and concrete cannot always be assured. The bond can degrade over time, eventually causing the system to become ineffective. Bond quality is influenced by the condition of the existing concrete, surface preparation of the concrete substrate, quality of the composite system application, quality of the composite and the durability of the epoxy primer and resin. A large number of parameters affect bond strength, including exposure to ultra-violet radiation, chemical activity, temperature, moisture, and stress level, as well as other factors (Karbhari1997).

From the review of the literatures, most of the experimental studies focused on theCapability of the externally bonded FRP composites to enhance the shear or flexural capacity of reinforced concrete beams and the investigation of the possible failure modes. Although some studies on shear and flexural strengthening of reinforced concrete beams exist, the optimum thickness of such FRP materials in relation to the level of deterioration of the members in shear or flexure was not evaluated.

CHAPTER THREE

METHODOLOGY

3.1 Reinforced Concrete Beam Design Criteria

The Simply Supported Reinforced Concrete beam was designed as singly reinforced using BSEN1992 design criteria, and the section was tested adequate against, Bending, Shear and Deflection failure. The reinforcement concrete cover was taken as 27mm while a cylindrical concrete characteristic strength of 25MPa and characteristic yield strength for steel of 450MPa was used for the design.

The section of the beam considered and loading combination for both the permanent and variable action used for the design is as shown in Figure 3.1.

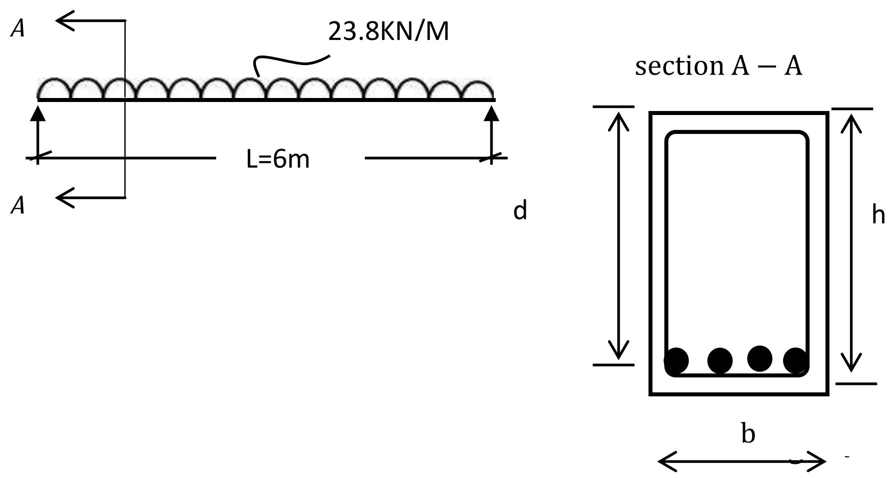


Fig 3.1 Structural formation of the beam under consideration.

Where

L=Total span of the beam

h=Overall height of the beam

b=width of the beam

d=Effective depth of the beam

QK&Gk =variable and permanent action on the beam

3.1.1 Flexural failure determination

In order to subject the Reinforced Concrete beam to flexural deficiency, the flexural resistance of the beam was Reduced by 5, 10, 15, 20, 25, up to 50% and the various flexural resistance corresponding to each percentage reduction in flexural capacity of the beam was compared with the applied flexural strength from which the level of deterioration that causes flexural failure of the beam was determined. This condition occurs when the applied Bending moment exceed the flexural resistance of the beam.

For the purpose of this research, the RC beam was subjected to flexural deficiency at a flexural deterioration level of 25%. FRP material thickness of 1.2mm was used in the design which enhances the flexural capacity of the beam by 66%.

3.1.2 Minimum existing flexural strength limit

The Minimum Flexural strength requirement for the RC beam to qualify for strengthening was determined in accordance with ACI 440.2R as follows:

$$\phi Rn \ 1.1DL+0.75LL \tag{3.1}$$

Where:-

ϕRn is Design strength of the existing member without the FRP material

DL is the new design dead load

LL is new design live load

3.2 Flexural Strengthening Analysis

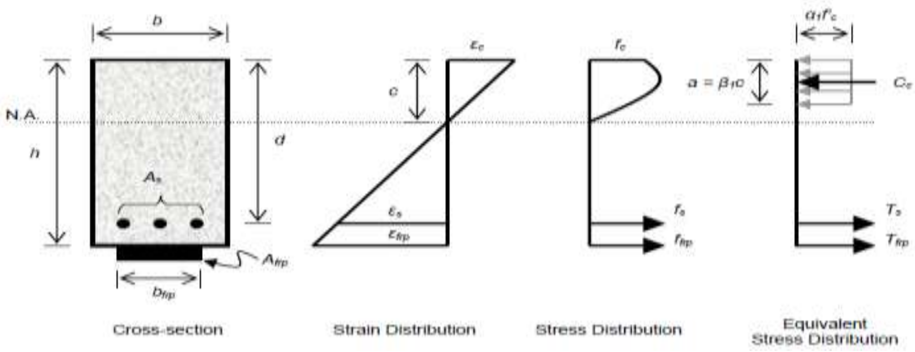


Fig.3.2 Stress diagram for RC beam strengthened in flexure using FRP

Assumed stress strain relationship for a singly reinforced concrete beam strengthened in flexure with externally bonded material (with initial strain neglected)

From the above figure, the equilibrium of internal forces the stress resultant (concrete in compression C_C, steel in tensionT_S, and the FRP in tensionT_{frp}, will sum to zero. Hence

$C_C = T_s + T_{frp}$ Where (3.2)

$C_C = \emptyset_C \alpha_1 f_c' \beta_1 bc$ (3.3)

$T_S = \emptyset_S A_S f_s$ with$f_s \leq f_y$ (3.4)

$T_{frp} = \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frp}$ where$\varepsilon_{frp} \leq \varepsilon_{frpu}$ And thus, (3.5)

$C_C = T_s + T_{frp}$

$\emptyset_C \alpha_1 f_c' \beta_1 bc = \emptyset_S A_S f_s + \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frp}$ (3.6)

Equation (3.6) was used to determine the depth of neutral axis C

And the moment of resistance was determine by taking moment about the compressive stress resultant neglecting the thickness of the FRP which is usually comparatively small thus,

$M_r = \left(d - \frac{a}{2}\right) + T_{frp} \left(h - \frac{a}{2}\right)$ Where $a = \beta_1 C$ (3.7)

It is assumed that compression failure occurs at the compression Fibre of the section and that the longitudinal steel has yielded which implies that the strain in the compression zone $\varepsilon_c = 0.0035$ which represent the failure strain for concrete in compression. From strain compatibility equation,

$$\varepsilon_{frp} = \varepsilon_{cu} \frac{h - c}{c}, \text{and } \varepsilon_{cu} = \varepsilon_{cu} \frac{d - c}{c}$$

Since the steel is assumed to have yielded, $f_s = f_y$ and the concrete stress block factors are

$\alpha_1 = 0.85 - 0.0015f_c' \geq 0.67$

$\beta_1 = 0.97 - 0.0025f_c' \geq 0.67$

Check if the tensile strain in the FRP is less than ultimate strain$\varepsilon_{frp} = \varepsilon_{cu}\frac{h-c}{c} \leq \varepsilon_{frpu}$, then the factored moment of resistance for the steel is given by

$M_r = \emptyset_s f_y A_S \left(d - \frac{a}{2}\right)$ And for the frp (3.8)

$M_{frp} = \emptyset_{frp} E_{frp} A_{frp} \varepsilon_{frp} \left(h - \frac{a}{2}\right)$ (3.9)

And therefore the total moment of resistance for the steel and the FRP is given as by:-

$M_R = \emptyset_s f_y A_S \left(d - \frac{a}{2}\right) + \emptyset_{frp} E_{frp} A_{frp} \varepsilon_{frp} \left(h - \frac{a}{2}\right)$, Where $a = \beta_1 c$ (3.10)

The internal steel reinforcement is assumed to have yielded to avoid brittle failure of the externally strengthened member i.e. $\varepsilon_s = \varepsilon_{cu}\frac{d-c}{c} > \varepsilon_y$.

Where $a = \beta_1 c$ and, $\emptyset_s, \emptyset_{frp}, \emptyset_c$,are the material resistance factors for steel, for FRP material and for concrete.$E_{frp}, t_{frp}, w_{frp}$ Are modulus of elasticity of FRP in N/mm^2, thickness in mm and width of FRP respectively.β and cAre known constants depending on the compressive strength of concrete and d is the effective depth of the beam, (ISIS Education module 2004).

3.3 Optimization of the FRP Material Thickness in Flexural Strengthening

To optimize the thickness of the FRP material used for flexural strengthening of the beam, the problem was formulated as constrained nonlinear programming problem. A deterministic method of constrain optimization using Generalized Reduced gradient method (GRG) was employed.

From Fig 3.3,it is clear from the flowchart that the program terminates immediately the values of the model input and output variables converge.

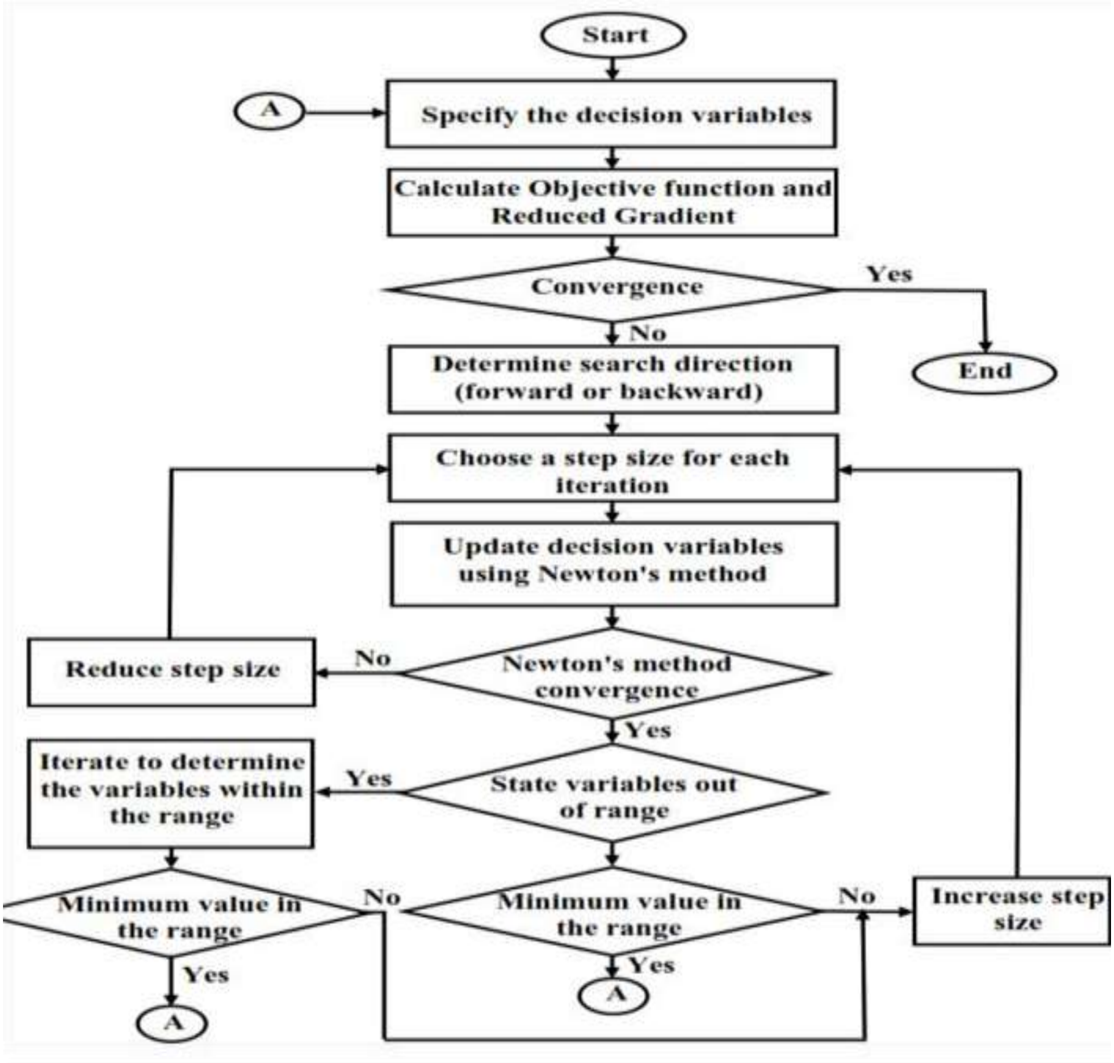


Fig 3.3: Program Flowchart for GRG Implementation

3.3.1 Determination of objective function

The objective function f(x) for the problem is determined from the relation

$$f(x) = (M_r + M_{FRP}) - M_A \qquad (3.11)$$

M_r, M_{FRP}, and M_A Are moment of resistance due to steel, moment of resistance due to FRP and the applied moment due to dead load and self-weight of the beam.

The objective function represents the output variable that will be minimized by correctly adjusting the input variables or the decision variables through a set of constrain functions. Equation (3.11) is a nonlinear programming problem.

3.3.2 Determination of constrain functions

A set of constrain functions were considered for the optimization as follows:

The thickness of the FRP must be non-negative; the width of the FRP must not be greater than the overall width of the beam and must be greater than zero also. The thickness of the FRP material must be less than the width of the beam. This constrains are given by the mathematical relation as follows:-

$$t_{frp} \geq 0, w_{frp} \leq b_w, w_{frp} \geq t_{frp}, w_{frp} \geq 0. \qquad (3.12)$$

3.3.3 Determination of decision variables

In order to achieve the goal of the optimization program, a number of input or decision variables were considered in the program. They include:-The effective depth of the strengthened beam d(mm), the overall height of the strengthened beam h(mm), the thickness of the FPR material t_{frp} (mm), the width of the beam b_w (mm)and the area of steel reinforcement provided A_S (mm^2).

3.4 Sensitivity Analysis

A sensitivity analysis was carried out to determine the effect of various parameters including dead to live load ratio, steel reinforcement ratio, and the different modulus of elasticity of the FRP material on the optimum thickness of the material. For this purpose, FRP modulus of 50GPa, 75Gpa,100GPa, 125GPa, 150GPa and steel reinforcement ratios of 0.2, 0.4, 0.6, 0.8,1 were considered in the analysis. Furthermore, load ratios of 0.4, 0.8, 1.2, and 1.6 were also considered at flexural capacity reduction of10%, 20%, 30%, 40, upto 90%.The influence of these parameters on the optimum thickness of the FRP material at different level of deterioration in flexure was analyzed.

3.5 Shear Strengthening

For the purpose of shear strengthening of the beam, the CFRP material is assumed to have been placed externally at the tensile zone and the two side faces of the beam in form of U wrap to enhanceits tensile strength and increase the overall shear strength of the Reinforced concrete beam.

3.5.1Shear failure determination

In order to subject the Reinforced Concrete beam to shear deficiency, the overall shear resistance of the beam comprising of the resistance due to concrete and steel was reduced by 5, 10, 15, 20, 25, up to 50%. The various shear resistance corresponding to each percentage reduction in the overall shear capacity of the beam was compared with the applied shear force from which the level of reduction that causes shear failure of the beam was determined.

For the purpose of this research, the Reinforced concrete beam was subjected to shear deficiency at 25%. At this level of deterioration, the existing shear strength of the beam was compared with the ACI 440.2R minimum requirement before strengthening.

This condition ensures that, should the FRP material be compromised, the structure should be able to maintain sufficient capacity to carry the existing service load without collapse before any other alternative repair measure is taken

3.5.2 Shear Strengthening of the beam

For the purpose of designing the externally bonded shear reinforcement for the shear deficient RC Beam using CFRP material, The Canadian ISIS (2004) design criterion was employed from which the contribution of the CFRP material, the Steel reinforcement and the concrete to the overall Shear resistance of the beam was determined.

Table 3.1: Material Resistance factors, (ISIS Canada 2004)

S/N	**Material**	**Resistance factor (Ø)**
1	Steel	0.85
2	Concrete	0.6
3	FRP	0.75

The materials resistance factors used for the design were based on ISIS Canada (2004).The resistance factors increase safety and economy in

Retrofitting/strengthening of the structural element(ISIS Canada 2004). An FRP design thickness of 1.2mm was used for strengthening and the factored shear resistance for the strengthened beam was determined from the relations as follows:-

$$V_r = (V_c + V_s + V_{frp}) \quad \text{Where} \tag{3.13}$$

$$V_c = \left(\frac{260}{1000+d}\right)\lambda\phi_c\sqrt{f_c'}b_w d \tag{3.14}$$

And $V_c > 0.1\lambda\phi_c\sqrt{f_c'}b_w d$ and

$$V_s = \frac{\phi_s f_y A_v d}{s} \tag{3.15}$$

And for the FRP material the contribution to the shear resistance was determine from:-

$$V_{frp} = \left(\frac{\phi_{frp} A_{frp} E_{frp} \varepsilon_{frpe} d_{frp} (\sin\beta + \cos\beta)}{s_{frp}}\right) \tag{3.16}$$

Where $A_{frp} = 2t_{frp} w_{frp}$ and the effective strain in the FRP is determined by applying a reduction factor R to the ultimate strain of the composite thus:

$\varepsilon_{frpe} = R.\varepsilon_{frpu} \leq 0.004$. The effective strain limited to $\varepsilon_{frp} \leq 0.004$to ensure aggregate interlock in the concrete by preventing shear cracks from widening beyond acceptable limit, ISIS Canada (2004).The reduction factor R is given by the relation

$$R = \alpha\lambda_1\left(\frac{f_c'^{2/3}}{\rho_{frp} E_{frp}}\right) \tag{3.17}$$

Where $\alpha = 0.8,\ \lambda_1 = 1.35, \lambda_2 = 0.30$ For Carbon Fibre reinforced polymer (FRP).

The FRP shear reinforcement ratio ρ_{frp} is given by

$$\rho_{frp} = \left(\frac{2t_{frp}}{b_w}\right)\left(\frac{W_{frp}}{S_{frp}}\right) \tag{3.18}$$

And the maximum allowable shear strengthening for buildings is given by:-

$V_r \leq V_c + 0.8\lambda\phi_c\sqrt{f_c'}b_w d$ Where $\lambda = 1.0$(3.19)

Where Vr is the overall shear resistance due to Steel, Concrete and CFRP

3.5.3 Optimization of the FRP material thickness in shear strengthening

To optimize the thickness of the FRP material used for strengthening of the shear deficient beam, the problem was formulated as constrained nonlinear programming problem. A deterministic method of constrain optimization using Generalized Reduced gradient method (GRG) was employed.

3.5.4 Determination of objective function

The objective function f(x) for the problem was determined from the relation:-

$$f(x) = (V_c + V_s + V_{FRP}) - V_A \qquad (3.20)$$

Where V_s and V_A are shear resistance due to steel, concrete, FRP, and the applied shear due to dead load and self-weight of the beam respectively.

This function represents the output variable that will be minimized by correctly adjusting the input variable or the decision variable after satisfying a set of conditions or constrain.

3.5.5 Determination of constrain functions

A number of constrains or conditions were considered for the optimization as follows:-

The thickness of the FRP must be non-negative; the width of the FRP must not be greater than the overall width of the beam and must be greater than zero also. The thickness of the FRP material must be less than the width of the beam. This constrains are given by the mathematical relation as follows:-

$$t_{frp} \geq 0, w_{frp} \leq b_w, w_{frp} \geq t_{frp}, w_{frp} \geq 0, S \leq 300 \qquad (3.21)$$

3.5.6 Determination of decision variables

In order to achieve the goal of the optimization problem, a number of input or decision variables were considered in the program. They include:-The effective depth of the strengthened beam d (mm), the overall height of the beam h (mm), the thickness of the FPR material t_{frp} (mm), the width of the beam b_w (mm) and the area of steel reinforcement provided A_S (mm^2).

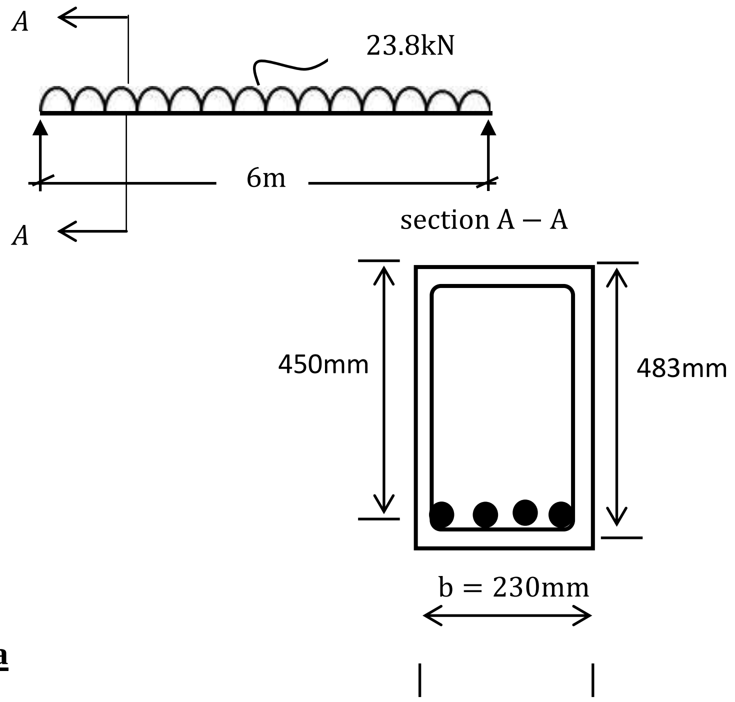
3.6Sensitivity Analysis

A sensitivity analysis was carried out to determine the effect of various parameters including dead to live load ratio, steel reinforcement ratio, and the different modulus of elasticity of the FRP material on the optimum thickness of the material. For this purpose, FRP modulus of25GPa, 50GPa, 75Gpa,100GPa and steel reinforcement ratio of 0.2, 0.4, 0.6, 0.8 were considered in the analysis. Furthermore, load ratios of 0.5, 1, 1.5, 2 were also considered at shear capacity reduction of10%, 20%, 30%, 40, upto 90%.The influence of this parameters on the optimum thickness of the FRP material at different level of deterioration in shear was analyzed.

3.7 Reinforced Concrete Beam Design

member	calculation	output

BEAM



<u>Data</u>

Use

$f_{ck} = 25\text{MPa}$

$f_{yk} = 450\text{MPa}$

$E_s = 200\text{GPa}$

$t_{frp} = 1.2\text{mm}$

$A_{frp} = 1.2 \times 230 = 276\text{mm}^2$

$b = 230\text{mm}$ and $h = 483\text{mm}$

cover = 27mm

<u>LOADINGS</u>

Use

Unit weight of concrete $= 2500\text{kg/m}^3$

$= 2500 \times 9.81\text{N/mm}^3 \times 10^{-3} = 25\text{kN/m}^3$

Beam self weight $= 0.230 \times 0.483 \times 25 = 2.8\text{kNm}^{-1}$

$q_s = 2.8\text{KNm}^{-1}$

member ref	calculation	output
BEAM	$q_{ll} = 1.5\mathrm{kN/m^2} = 1.5 \times 6 = 9\mathrm{kNm^{-1}}$ $q_{dl} = 12\mathrm{KNm^{-1}}$ The applied momment due to loadings is determined From:- $M_A = \frac{1}{8}(q_s m_s + q_{dl} m_{dl} + q_{ll} m_{ll})L^2$ $M_A = 0.125(2.8 \times 1.0 + 12 \times 1.0 + 9 \times 1.0) \times 6^2$ $M_A = 107\mathrm{kNm}.$ The ultimate momment of resistance is given by $M_R = 0.167F_{ck}bd^2$ $= 0.167 \times 25 \times 230 \times 450^2 \times 10^{-6}$ $M_R = 194\mathrm{kNM}.$ $M_A < M_R (107\mathrm{kNm} < 194\mathrm{kNm}),$ $K_o = \frac{M}{F_{ck}bd^2} = \frac{107 \times 10^6}{25 \times 230 \times 450^2} = 0.09$ $Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K_0}\right]$ $= \frac{450}{2}\left[1 + \sqrt{1 - 3.53 \times 0.09}\right]$ $Z = 411\mathrm{mm}.$ $A_s = \frac{M}{0.87f_{yk}Z} = \frac{107 \times 10^6}{0.87 \times 450 \times 411} = 665\mathrm{mm^2}$	Singly Reinforced section use section modulus $Z = 411\mathrm{mm}$ provide $4y12(A_s = 804\mathrm{mm^2})$

member ref	calculation	output
	CHECKS $A_{smin} = \frac{0.26f_{ctm}\,b_t d}{f_{yk}} = \frac{0.26 \times 2.6 \times 230 \times 450}{450}$ $= 155mm^2 < 804mm^2$ **3.7.1 check for Shear** Design shear stress $v_{Ed} = \frac{V_{Ed}}{b_w z}$ $where Z = 0.9d$ shear force V_{Ed} , $w = 23.8kN/m$ $\Sigma M_A = 0, -R_B \times 6 + 23.8 \times 6 \times 3 = 0$ $R_B = \frac{428.4}{6} = 71.4kN$ $\Sigma V = 0$, $R_A - 23.8 \times 6 + R_B = 0$ $R_A = 142.8 - 71.4 = 71.4kN$ $V_{Ed} = 71.4kN$ Therefore $v_{Ed} = \frac{71.4 \times 10^3}{0.9 \times 230 \times 450}$ $= 0.77N/mm^2$ from table 7 the value for v_{Rd}, max cotθ for f_{ck} class 25 $= 3.1N/mm^2$. $v_{Ed} < v_{Rd}$, max $\cot\theta = 2.5$ Therefore $$\frac{A_{sw}}{S} = \frac{v_{Ed}\,b_w}{f_{ywd}\cot\theta} = \frac{0.77 \times 230}{\left(\frac{450}{1.15}\right) \times 2.5} = 0.18$$ maximum spacing for vertical shear S_{max} $S_{max} = 0.75d = 0.75 \times 450 = 337.7mm$	section is ok provie Y8 @300mm c /c section is ok

member ref	calculation	output
BEAM	**3.7.2 check for Deflection** determine basic $l/_d$ from figure 7 $\frac{A_{srqd}}{bd} = \frac{665}{230 \times 450} = 0.0064 \times 10^2 = 0.64$ basic $l/_d$ for $0.64\% = 17$ for a simply supported span $K = 1.0$ and $F1 = F2 = F3 = 1.0$ Basic $l/_d \times K \times F1 \times F2 \times F3$ $= 17 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 17$ Actual $l/_d = 6/_{0.450} = 13$ Hence $17 > 13$	Section is o.k

3.8Design of FRP Shear Strengthening

Member	Calculation	output

CFRP	**<u>Data</u>** Span of the beam $l = 6.0m$ Concrete strength $f_{ck} = 25MPa$ $f_{yk} = 450MPa$ $t_{frp} = 1.2mm$ $b_w = 230mm$ $A_{frp} = 2 \times 1.2 \times 100 = 240mm^2$ $E_{frp} = 25GPa$ $\varepsilon_{frp} = 1.55\%$ $\emptyset_s = 0.85, \emptyset_c = 0.60, \emptyset_{frp} = 0.75$ **3.8.1 Shear resistance due to concrete** $V_C = \left(\frac{260}{1000 + d}\right) \lambda\emptyset_c\sqrt{f_c'}b_w d$ $V_C = 0.2 \times 1.0 \times 0.60\sqrt{25} \times 230 \times 450 = 62100N$ Area of steel $A_v = \pi\frac{d^2}{4} = 3.142 \times \frac{8^2}{4} = 50.34mm^2$ Area of FRP material $A_{frp} = 2 \times 1.2 \times 100 = 240mm^2$ **3.8.2Shear resistance due to steel V_S** $V_s = \frac{\emptyset_s f_y A_v d}{s} = \frac{0.85 \times 450 \times 50.34 \times 450}{300} = 28882N$ FRP shear reinforcement ratio ρ_{frp} is determine from:-	

$$\rho_{frp} = \left(\frac{2t_{frp}}{b_w}\right)\left(\frac{W_{frp}}{S_{frp}}\right) = \left(\frac{2 \times 1.2}{230}\right)\left(\frac{100}{250}\right) = 0.4\%$$

Effective anchorage length l_e

$$l_e = \frac{25350}{\left(t_{frp}\, E_{frp}\right)^{0.58}} = \frac{25350}{(1.2 \times 25000)^{0.58}} = 64\text{mm}$$

Effective strain due to ultimate strain ratio

$$R = 0.8 \times 1.35\left(\frac{25^{2/3}}{0.004 \times 25000}\right)^{0.30} = 0.51$$

Member	Calculation	Output

-	Effective strain in FRP $\varepsilon_{frpe} = 0.51 \times 0.0155 = 0.008$ Total shear resistance due to concrete and steel $V_r = (V_C + V_S) = 62100+28882 = 90982N$ Applied shear force $V_A = 71400N$ $71400N < 90982N$	Section is ok
	3.8.3 Shear Capacity Reduction The beam was subjected to shear deficiency at a capacity reduction of 25%. $V_r \times 25\% = 90982 \times 0.25 = 22745.5N$ Shear resistance at failure $V_r = 90982 - 22745.5 = 68236.5N$ $71400N > 68236.5N$ i.e $V_A > V_r$	Beam deficient in shear
	The minimum existing shear strength is given by $\phi Rn = 1.0DL + 0.5LL$ $\phi Rn = 1.0 \times 14.8 + 0.5 \times 9 = 19.3kN/m$ $\phi Rn = 19.3 \times 3 = 57.9kN < 68.236kN$	Beam Qualified for strengthening
	3.8.4Shear strengthening Application Shear resistance due to FRP material placed externally $V_{frp} = \left(\frac{\emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frpe} d_{frp} (\sin\beta + \cos\beta)}{s_{frp}}\right) =$ $\left(\frac{0.75 \times 240 \times 0.008 \times 25000 \times 450(\sin 0 + \cos 0)}{250}\right)$ $= 64800N.$ Overall shear resistance due to steel, concrete, and FRP $V_r = V_C + V_S + V_{frp} = 68236.5 + 64800 = 133036.5N$ $71400N < 133036.5N$	1.2mm CFRP thickness adequate

	The objective function for the problem is given by $V_r - V_A = 133036.5 - 71400 = 61636.5\text{N}$ **3.8.5Check for maximum shear strengthening** Applied shear stress $v_{Ed} = \frac{71.4 \times 10^3}{0.9 \times 230 \times 450} = 0.76\text{N/mm}^2$ Resistance shear stress $= \frac{133.036 \times 10^3}{1.0 \times 230 \times 450} = 1.28\text{N/mm}^2$ $0.76\text{N/mm}^2 < 1.28\text{N/mm}^2$ Net shear stress $= 1.28 - 0.76 = 0.52\text{N/mm}^2$	Section is ok
	Maximum shear strengthening $V_r \leq 62100 + 0.8 \times 1 \times 0.60 \times \sqrt{25} \times 230 \times 450 =$ $310500\text{N}.$ $133036.5\text{N} < 310500\text{N}$	1.2mm CFRP thickness adequate

member	***calculation***	***output***

	3.9 Design of FRP flexural Strengthening $t_{frp} = 1.2mm$ $A_{frp} = 1.2 \times 230 = 276mm^2$ $E_{frp} = 150GPa$ $\varepsilon_{frp} = 1.55\%$ $\emptyset_s = 0.85, \emptyset_c = 0.60, \emptyset_{frp} = 0.75$ factored momment of resistance for steel $M_r = \emptyset_s f_y A_s \left(d - {}^{a}/_{2}\right) \text{where } a = \beta_1 c$ $\beta_1 = 0.97 - 0.0025f_c^{'} \geq 0.67$ $\alpha_1 = 0.85 - 0.0015F_C^{'} \geq 0.67$ $\beta_1 = 0.97 - 0.0025 \times 25 = 0.91$ $\alpha_1 = 0.85 - 0.0015 \times 25 = 0.81$ Determineation of depth of neutral axis C, For equillibriumof forces: – $\emptyset_c \alpha_1 f_c^{'} \beta_1 bc = \emptyset_s f_y A_s + \emptyset_{frp} E_{frp} A_{frp} \varepsilon_{frp}$ $0.6 \times 0.81 \times 25 \times 0.91 \times 230 \times C = 0.85 \times 450 \times$ $804 + 0.75 \times 150000 \times 276 \times 0.0035 \left(\frac{483 - C}{C}\right)$ $2542.995C = 307530 + 108675 \left(\frac{483 - C}{C}\right)$ $2542.995C^2 = 307530C + 52490025 - 108675C$ $2542.995C^2 - 198855C - 52490025 = 0,$ Solvingthe quadractic equation using formular	Satisfied use depth Of neutral axis C= 188mm

member	calculation	output

	momment of resistance due to steel M_r $M_r = \emptyset_s f_y A_s \left(d - \frac{a}{2}\right)$ where $a = \beta_1 C$ $M_r = 0.85 \times 450 \times 804 \times \left(450 - \frac{0.91 \times 188}{2}\right)$ $M_r = 112\text{kNm}.$ Moment applied due to dead and live loads $M_A = 107\text{kNm}$ $107\text{KNM} < 112KNM$	Section is ok
	3.9.1 Flexural Capacity Reduction The beam was subjected to flexural deficiency at a capacity reduction of 25%. $M_r \times 25\% = 112 \times 0.25 = 28\text{kNm}$ Flexural resistance at failure $M_r = 112 - 28 = 84\text{kNm}$ $84\text{KNM} < 107\text{KNM}$ i.e $M_A > M_r$	Beam deficient in flexure
	3.9.2 flexural strengthening Flexural resistance due to FRP material placed externally at the tension zone of the beam $M_{frp} = \emptyset_{frp} E_{frp} A_{frp} \varepsilon_{frp} \left(h - \frac{a}{2}\right)$ $M_{frp} = 0.75 \times 150000 \times 276 \times 0.0054 \times$ $\left(483 - \frac{0.91 \times 188}{2}\right) = 67\text{kNm}.$ Overall flexural resistance due to the beam and FRP material $M_R = M_r + M_{frp} = 84 + 67 = 151\text{kNm} > 107.$ $151\text{kNm} > 107\text{kNm}$	1.2mm CFRP Thickness adequate

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Beam Design Results and Discussion

The Reinforced concrete beam with cross section $483 \times 230\text{mm}$ was design as singly reinforced to withstand a total applied load including self-weight of 23.8kN/m.The total area of steel reinforcement provided is 804mm^2 and the beam was tested adequate against bending, shear and deflection failure.The result presented in Table 4.1

Table 4.1 Effect of load ratio on Bending Moment, shear, and deflection

Load Ratio	**B.M ($\times 10^6$Nmm)**	**Shear Stress ($\times 10^3$N/mm^2)**	**Deflection($\times 10^{-3}$mm)**
0.5	0.6	0.4	0.7
1	0.8	0.57	0.9
1.5	1	0.75	1.1
2	1.2	0.92	1.3
2.5	1.4	1.1	1.5

From the Table above, the result of the plot shows an increase in the magnitude of bending moment, shear, and deflection as the load ratio increases from 0.5 to 2.5. A bending moment of $1.40\ \text{kNm}$ was recorded at a peak load ratio of 2.5 and a maximum shear stress of $1.10\ \text{kN/m}^2$ was recorded at a peak load ratio of 2.5. Similarly, a minimum bending moment of $0.60\ \text{kNm}$ was recorded for a lower load ratio of 0.5. The magnitude of deflection also increases with increase in the load ratio. The variation in load ratio therefore has a direct influence on the flexural, shear and deflection characteristics of the beam.

4. 2 Flexural Strengthening Results

The Application of the FRP material has increased the overall flexural capacity of the deficient Beam by 80%.The results of the flexural resistance for the Reinforced Concrete Beam at different level of deterioration in flexure is presented in table 4.2

Table 4.2 flexural resistance for RC beam at different capacity levels

S/N	Flexural Capacity Reduction %	flexural Resistance (kN)
1	0	112.09
2	5	106.48
3	10	100.88
4	15	95.28
5	20	89.67
6	25	84.07
7	30	78.46
8	35	72.86
9	40	67.25
10	45	61.65
11	50	56.04

From the results of flexural resistance presented in the table 4.2, the value of flexural strength at 25% deterioration level was determined as 84.07kN, this has satisfied the

ACI 440. 2R requirement for minimum existing strength required for a beam to qualify for strengthening which was determined at 63.6% flexural capacity level. The choice of 25% flexural deficiency level for strengthening of the deficient reinforced concrete beam is therefore considered adequate having satisfied the requirement of the ACI 440. 2R design code.

4.2.1 Optimization of CFRP laminate thickness

The 1.2mm CFRP material thickness with modulus of 150GPa used for the flexural strengthening of the deficient Reinforced Concrete beam was optimized using Generalized Reduced Gradient method (GRG), and the result presented in table 4.3

Table 4.3: Optimum thickness of FRP material for RC beam at different capacity level

Flexural Capacity Reduction (%)	Optimized FRP thickness (mm)
50	0.902
45	0.803
40	0.704
35	0.605
30	0.506
25	0.407
20	0.308
15	0.209
10	0.109
5	0.010

From table 4.3, the optimum thickness of the FRP material required for flexural strengthening of the beam at 25% flexural capacity reduction was determine as 0.40712mm, and the total difference between the design and optimized CFRP thickness is 0.79288mm.The difference in the thickness represent 66% saving in the design thickness of the material (Minimization of the objective function).

Based on the result presented above, an optimum FRP thickness of 0.40712mm is considered as the minimum thickness of the material required to restore back the flexural capacity of the deficient Reinforced concrete beam.

4.3 Sensitivity Analysis

Sensitivity analysis for the strengthened Reinforced concrete beam was carried out and the influence of design parameters such as various CFRP stiffness, dead to live load ratios, and the steel reinforcement ratio on the optimum thickness of the CFRP material was analyzed.

$4.3.1$ Influence of different CFRP Modulus

For the purpose of this analysis, CFRP Elastic modulus of$50\text{GPa}, 75\text{GPa}$, 100GPa 125GPa and 150GPa were considered. The different levels of reduction in flexural capacity of the reinforced concrete beam used in the analysis are 10%, 20%, 30%, 40, upto 90% and the results presented in figure 4.1

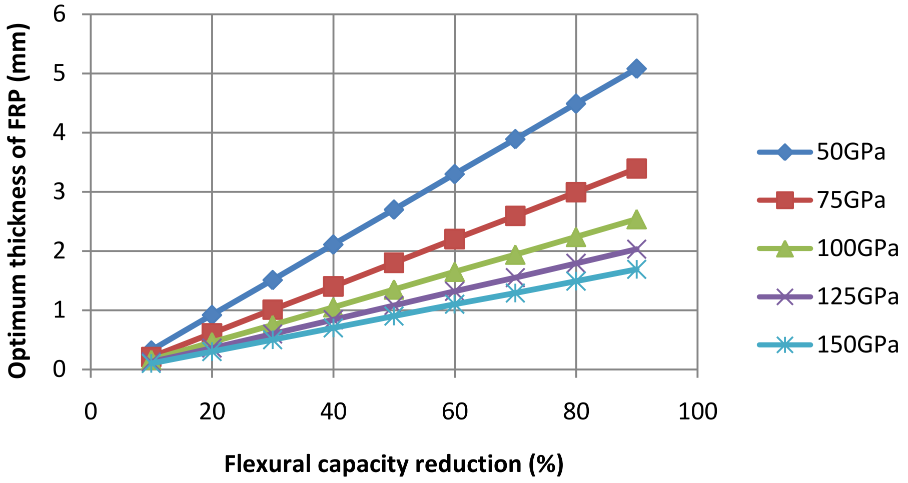


Fig 4.1: Influence of CFRP Elastic modulus on the optimum thickness of the material

From figure 4.1 it can be observed that the optimum thickness of the CFRP material increases as the flexural capacity of the beam reduces.

The result of the plot also indicates a gradual decrease in the optimum thickness of the material with increase in the CFRP stiffness. The percentage decrease in the optimum thickness of the material at 25% level of deterioration for the different CFRP modulus is presented in table 4.4.

Table 4.4: Shows the % decrease in optimum thickness for different CFRP stiffness

FRP Modulus	**50GPa**	**75GPa**	**100GPa**	**125GPa**	**150GPa**
Optimized FRP thickness	1.221	0.814	0.610	0.488	0.407
% Decrease in FRP thickness	0%	33%	25%	20%	16.5%

From the table above, the optimum thickness of the FRP material decreases by 33% as the stiffness increase from 50 to 75GPa, it also increases by 16.5% as the stiffness decreases from 150 to 125GPa.The optimum thickness of the material increases disproportionately with decrease in CFRP stiffness.

4. 3. 2 Influence of Dead to live load ratio

For the purpose of this analysis, Dead to live load ratios of 0.4, 0.8,1.2 and 1.6 were considered and their effect on the optimum thickness of the FRP material was investigated. A total dead load of 14.8 kN/m and live load of 9 kN/m were used for the design and the result of the analysis presented in figure 4.2.

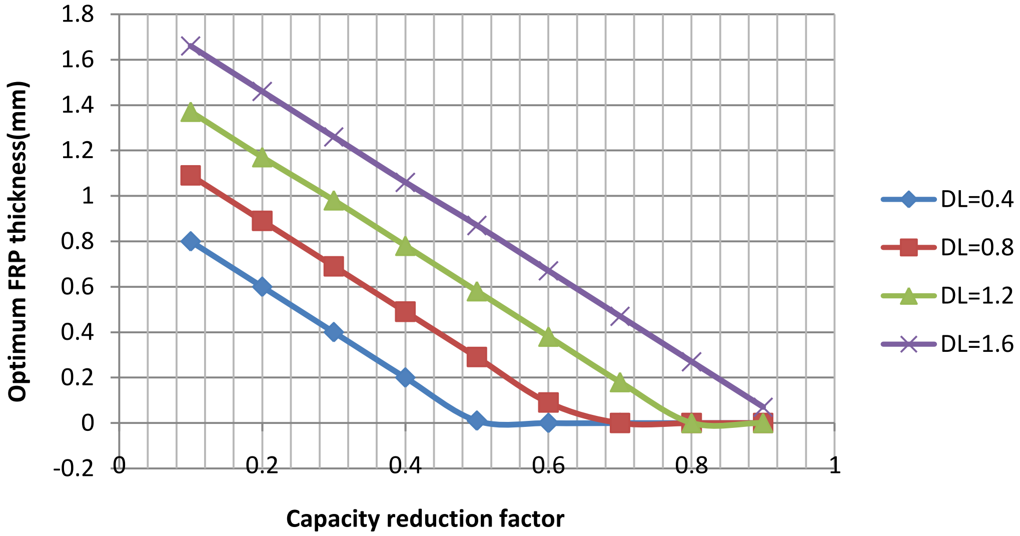


Fig4.2: Influence of dead to live load ratio on the optimum thickness of FRP material.

Considering the figure 4.2, there is a gradual increase in the optimum thickness of the CFRP material with reduction in the flexural strength of the beam. It is also clearly observed that the optimum thickness of the material decrease with decrease in load ratio and also increases with increase in load ratio. Furthermore zero thickness of CFRP material or no strengthening is required at 40%, 30%, 20% and 10% flexural capacity reduction for a load ratio of 0.4,also not required at 30%, 20% and 10% flexural capacity reduction for a load ratio of 0.8 Additionally, Zero or no FRP material is required at 20% and 10% shear capacity reduction for dead to live load ratio of 1.2 . The load ratio therefore has direct influence on the optimum thickness of the CFRP material

4.3.3 Steel Reinforcement Ratio

The steel reinforcement ratio for the Reinforced concrete Beam was determine and varied, and the effect of such variation on the optimum thickness of the CFRP material was observed and the result presented in Figure 4.3.

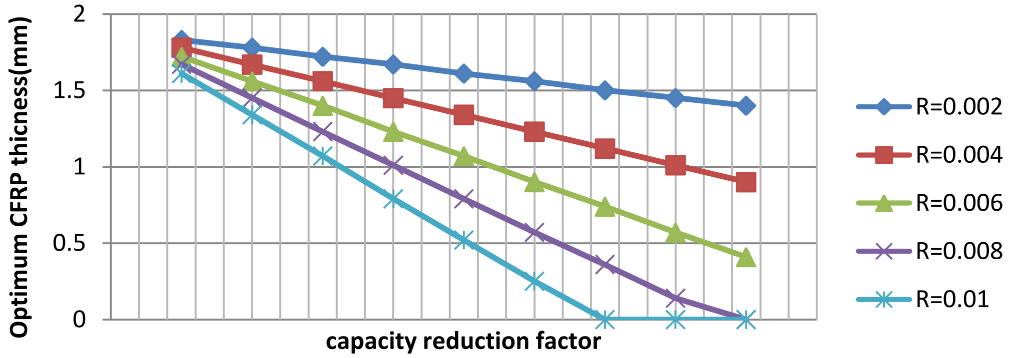


Fig 4.3: Effect of Steel Reinforcement Ratio on the Optimum CFRP Thickness

From the result of the sensitivity analysis presented in figure 4.3, it can be seen that the optimum thickness of the CFRP material decline as the steel reinforcement ratio increases from 0.2, 0.4, 0.6, 0.8, upto 1.0 Indicating an increase in the flexural capacity of the beam. The thickness of the material also decline as the flexural capacity of the beam increases. It can be observed also that zero thickness or no strengthening is required at 10% 20% and 30% flexural capacity reduction for steel reinforcement ratio of 0.01 the CFRP material is also not required at 10% capacity reduction for steel ratio of 0.08.

Table4.5: PercentageDecrease inFRP thickness for various steel reinforcement ratio

Steel reinforcement Ratio ($X10^{-2}$)	$\mathbf{0.2}$	$\mathbf{0.4}$	$\mathbf{0.6}$	$\mathbf{0.8}$	$\mathbf{1.0}$
Optimum FRP thickness(mm)	1.48	1.07	0.66	0.25	0
% **Decrease in thickness**	0%	28%	38%	62%	100%

From the Table 4.5, the optimum thickness of the material decrease by 28% as the steel reinforcement ratio increases from 0.02 to 0.04 and also decreases by 38% as the steel reinforcement ratio increase from 0.4 to 0.6.The optimum thickness of the FRP material increases disproportionately with decrease in steel reinforcement ratio.

4. 4Shear Strengthening Results

The Application of the CFRP material has increased the overall shear capacity of the deficient Beam by 95%.The results of the shear resistance for the Reinforced Concrete Beam at different level of deterioration in shear is presented in Table 4.6

Table 4.6: Shear resistance of RC beam at various deterioration levels

S/N	Capacity Reduction %	Shear Resistance (N)
1	0	90982.58
2	5	86433.45
3	10	81884.45
4	15	77335.19
5	20	72786.06
6	25	68236.93
7	30	63687.80

8	35	59138.67
9	40	54589.55
10	50	45491.29

From the results of shear resistance presented in Table 4.6, the value of shear strength at 25% shear deterioration level was determined as 68236.93N. This has satisfied the ACI 440. 2R requirement for minimum existing strength required for a beam to qualify for strengthening which was determined at 36.4% shear deficiency level and equivalent to 57900N.The choice of 25% shear deficiency level for strengthening of the shear deficient reinforced concrete beam is therefore considered adequate having satisfied the requirement of the code.

4.4.1 Optimization of CFRP laminate thickness

The thickness of the CFRP material with modulus of 25GPa used in strengthening of shear deficient Reinforced Concrete beam was optimized using Generalized Reduced Gradient method (GRG) and the result presented in Table 4.7.

Table 4.7: Optimized FRP thicknesses for RC beam at different shear capacity levels

Shear Capacity Reduction (%)	Optimized FRP thickness (mm)
50	0.47
45	0.39
40	0.31
35	0.22

30	0.14
25	0.06
20	0
15	0
10	0
5	0

From the result of the GRG program presented in Table 4.7, the optimum thickness of the CFRP material required for shear strengthening of the beam at 75% shear capacity level was determine as 0.06mm.The total difference between the design and optimized thickness of the CFRP material used for shear strengthening of the deficient beam was determined as 1.141319mm. The difference in the thickness represent 95% saving in the design thickness of the material which implies Minimization of the objective function.

Based on the result presented above, an optimum FRP thickness of 0.06mm is considered as the minimum thickness of the material required to restore back the shear capacity of the deficient Reinforced concrete beam.

4.5 Sensitivity Analysis

Sensitivity analysis for the shear strengthened Reinforced concrete beam was carried out and the influence of design parameters such as various CFRP stiffness, dead to live load ratios, and the steel reinforcement ratio on the variability of the optimum thickness of the CFRP material was analyzed.

4.5.1 Influence of different CFRP modulus

Various FRP modulus of 25GPa, 50GPa, 75GPa and 100GPa were considered in the analysis. The levels of reduction in shear capacity of the reinforced concrete beam used in the analysis are 10%, 20%, 30%, 40, upto 90% and the results presented in figure 4.4.

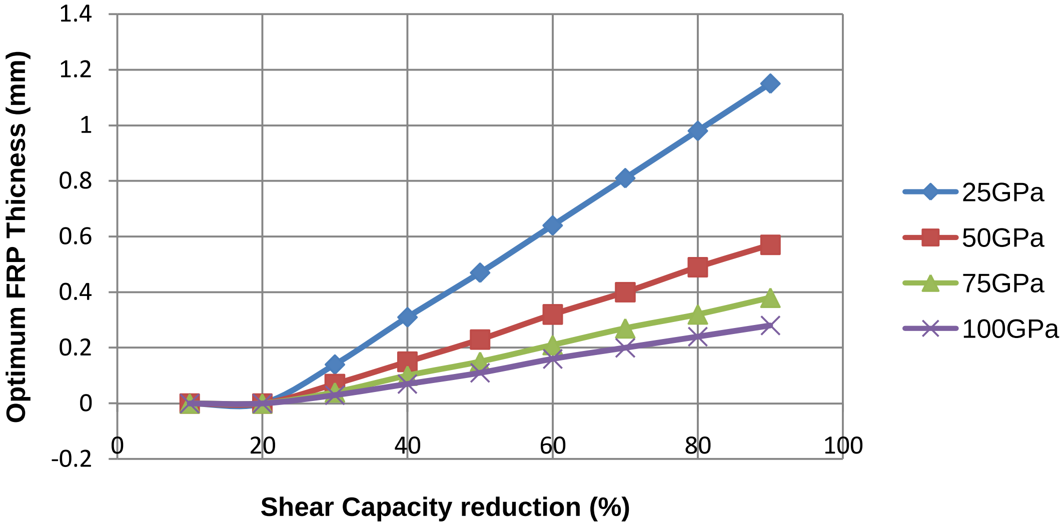


Fig 4.4: Influence of CFRP Elastic modulus on the optimum thickness of the material

From Figure 4.4, it can be observed that the optimum thickness of the FRP material increases as the shear capacity of the beam deteriorates. The increase in the optimized FRP thickness is essential in restoring the lost shear strength of the deficient beam.

The result of the plot also indicates a gradual decrease in the optimum thickness of the material with increase in the CFRP stiffness. Furthermore, Zero or no strengthening is required at 20% and 10% shear capacity reduction for CFRP modulus of 25GPa, 50GPa, 75GPa, and 100GPa respectively. The percentage decrease in the optimum thickness of the material at 25% level of deterioration for the different CFRP modulus is presented in table 4.8.

Table 4.8 Effect of CFRP Modulus on the optimum thickness with their % decrease

CFRP Modulus	25GPa	50GPa	75GPa	100GPa
Optimized FRP thickness (mm)	0.06	0.03	0.02	0.01
Decrease in optimum thickness (%)	0	50	33	50

From the results shown in table 4.8, the optimum thickness of the CFRP material decreases by 50% as CFRP modulus increases from 25 to 50GPa, the optimum thickness also decreases by 35% as the modulus increases from 50 to 75GPa.the thickness finally decreases by 50% with increase in FRP stiffness from 75 to 100GPa.

4.5.2 Dead to live load ratio

For the purpose of this analysis, Dead to live load ratios of 0.5,1.0,1.5 and 2.0 were considered and their effect on the optimum thickness of the CFRP material was investigated. A total dead load of 14.8 kN/m and live load of 9 kN/m were used for the design and the result of the analysis presented in figure 4.5.

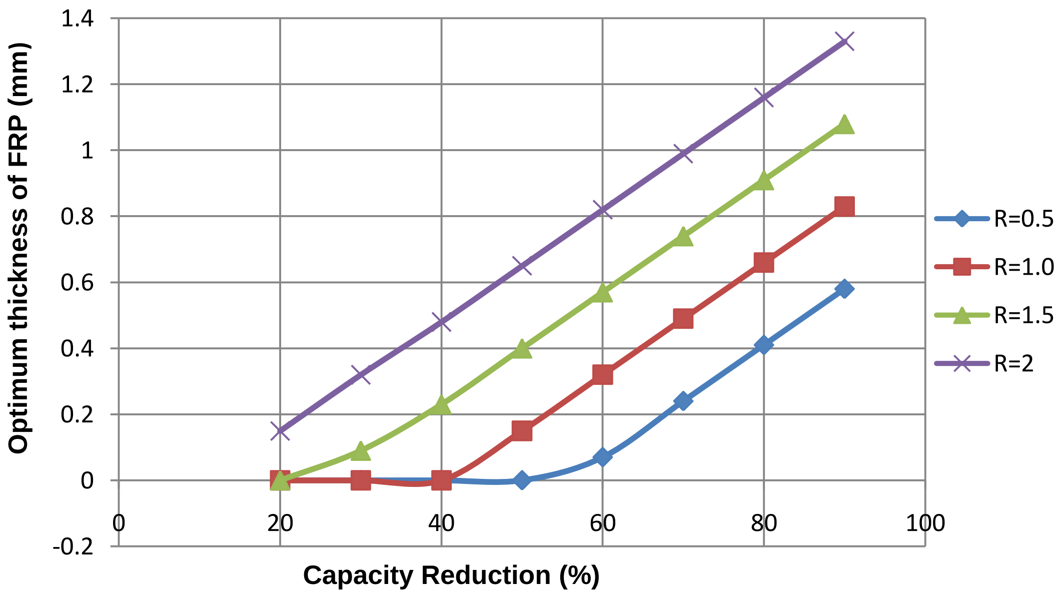


Fig4.5: Influence of dead to live load ratio on the optimum thickness of CFRP.

From the result of the load ratio shown in figure 4.5, a shear capacity reduction of 10%, 20% 30% 40% 50% 60% up to 90% were considered for the analysis. It can be observed from the result that zero thickness of CFRP material or no strengthening is required at 50%, 40%, 30%, 20% and 10% shear capacity reduction for a dead to live load ratio of 0.5.Furthermore shear strengthening is not required at 40 30 20 and 10% shear deterioration level for load ratio of 1.0. Additionally, Zero or no CFRP material is required at 20% shear capacity reduction for dead to live load ratio of 1.5.

The result also shows an increase in the optimum thickness of the FRP material with decrease in the shear capacity of the beam. Finally, the optimum thickness of the CFRP material increases with increase in load ratio and decreases with increase in the shear capacity of the Reinforced concrete beam. The load ratio therefore has direct influence on the optimum thickness of the CFRP material

4.5.3 Steel reinforcement ratio

The influence of various steel reinforcement ratio on the optimum thickness of the FRP material was investigated and the resulted presented in table 4.9Below.

Table 4.9:Influence of different steel ratio on the optimum thickness of CFRP

S/N	**Capacity Reduction factor**	**SR=0.002**	**SR=0.004**	**SR=0.006**	**SR=0.008**

1	0.1	1.18	1.16	1.13	1.11
2	0.2	1.04	0.99	0.95	0.90
3	0.3	0.90	0.83	0.76	0.69
4	0.4	0.76	0.67	0.57	0.48
5	0.5	0.62	0.51	0.39	0.27
6	0.6	0.52	0.34	0.20	0.06
7	0.7	0.35	0.18	0.02	0
8	0.8	0.21	0.02	0	0
9	0.9	0.07	0	0	0

From Table 4.9, it can be observed that no strengthening is required at 10% capacity reduction for steel reinforcement ratio of 0.04, at10 and 20% for steel reinforcement ratio of 0.006, and finally at 10, 20 and 30% shear capacity reduction for steel reinforcement ratio of 0.008.The optimum thickness of the CFRP material increases with decrease in steel reinforcement ratio and also decreases as the shear capacity of the beam increases.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the result of the design and the optimization program implemented, the following conclusion are made:

i. The choice of GRG nonlinear optimization technique is considered adequate since the objective function for the problem was determined as constrained nonlinear programming problem.
ii. The optimumCFRP thickness of 0.06mm and 0.41mm obtained from the result of the GRG program for both shear and flexural strengthening of the beam is lower than the material design thickness of 1.2mm by 95% for shear and 79% for flexure which implies minimization of cost.
iii. The optimum thickness of the CFRP material required for shear or flexural strengthening of the reinforced concrete beamdefends to a large extent on the magnitude of load ratio, steel reinforcement ratio and the stiffness of theCFRPmaterial used in the design.
iv. The overall shear and flexural capacity of the Reinforced concrete Beam at 25% shear deterioration level was enhanced significantly on the application of the CFRP material.
v. The result also indicates the feasibility of using CFRP material in strengthening of Reinforced concrete Beam deficient in shear or flexure.

5.2 Recommendation

Based on the findings of this study, the following recommendations are made:

1. There is need to study the relationship between the optimum thickness of the FRP material obtained and the concrete laminate bond strength

2. The optimum thickness of 0.06mm and 0.41mm for shear and flexural strengthening should be used tocarry out a reliability based assessment of the strengthened beam to accommodate any uncertainty
3. The methodology used in this research is recommended for strengthening of continues reinforced and prestressed concrete beams.
4. There is need to study the relationship between the objective function and the numberof decision variables used for the optimization.

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Capacity reduction factor	SR=0.002	SR=0.004	SR=0.006	SR=0.006	SR=0.01

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APPENDICES

0.1	1.83	1.78	1.72	1.67	1.61
0.2	1.78	1.67	1.56	1.45	1.34
0.3	1.72	1.56	1.4	1.23	1.07
0.4	1.67	1.45	1.23	1.01	0.79
0.5	1.61	1.34	1.07	0.79	0.52
0.6	1.56	1.23	0.9	0.57	0.25
0.7	1.50	1.12	0.74	0.36	0
0.8	1.45	1.01	0.57	0.14	0
0.9	1.40	0.90	0.41	0	0

Appendix A: Results of sensitivity analysis for RC beam strengthened in Flexure

Table A1: Optimum thickness of CFRP for various steel reinforcement ratios

Table A2: Optimum thickness of CFRP material for different CFRP Modulus

Capacity reduction factor	50GPa	75GPa	100GPa	125GPa	150GPa

0.1	5.08	3.39	2.54	2.03	1.69
0.2	4.49	2.99	2.24	1.79	1.49
0.3	3.89	2.59	1.94	1.55	1.29
0.4	3.30	2.20	1.65	1.32	1.1
0.5	2.70	1.80	1.35	1.08	0.90
0.6	2.11	1.40	1.05	0.84	0.70
0.7	1.51	1.01	0.75	0.60	0.50
0.8	0.92	0.61`	0.46	0.36	0.30
0.9	0.32	0.21	0.16	0.13	0.10

APPENDIX B: Results of sensitivity analysis for beam strengthened in Shear

TABLE B1: Optimum thickness of CFRP material for different CFRP Modulus

CAPACITY REDUCTION FACTOR	DL=0.5	DL=1.0	DL=1.5	DL=2.0
CAPACITY RDUCTION FACTOR	25GPa	50GPa	75GPa	100GPa
0.1	1.15	0.57	0.38	0.28
0.2	0.98	0.49	0.32	0.24
0.3	0.81	0.40	0.27	0.20
0.4	0.64	0.32	0.21	0.16
0.5	0.47	0.23	0.15	0.11
0.6	0.31	0.15	0.10	0.07
0.7	0.14	0.07	0.04	0.03
0.8	0	0	0	0
0.9	0	0	0	0

TABLE B2: Optimum thickness of CFRP material for various load ratio

0.1	0.58	0.83	1.08	1.33
0.2	0.41	0.66	0.91	1.16
0.3	0.24	0.49	0.74	0.99
0.4	0.07	0.32	0.57	0.82
0.5	0	0.15	0.40	0.65
0.6	0	0	0.23	0.48
0.7	0	0	0.09	0.32
0.8	0	0	0	0.15