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SCHOOL OF ENGINEERING

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Effect of Carbon Fibre Reinforced Polymer Strengthening on the Axial Capacity and Ductility of Non-slender Square Concrete Columns

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A thesis submitted in partial fulfilment for the Degree of Master of Science in Civil Engineering (Structural Engineering) at the Department of Civil & Construction Engineering, University of Nairobi

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UNIVERSITY OF NAIROBI

DECLARATION OF ORIGINALITY

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DEDICATION

To my parents: Mr. Munyua Mutuaruchiu & Dr. Jennifer K. Munyua and my siblings: Mugambi and Naitore, for their endless support and encouragement.

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ABSTRACT

This research investigated the effect of Carbon Fibre Reinforced Polymer (CFRP) strengthening on the axial capacity and ductility of non-slender square concrete columns. There is a problem of buildings collapsing in Kenya. Retrofitting of the buildings vulnerable to collapse is of great importance to ensure the safety of the occupants and to address the housing deficit in the country. An experimental research programme was conducted on 95 non-slender square concrete columns to find out the gain in axial capacity and ductility of the columns strengthened by CFRP. The specimens (150mm x 150mm x 350mm) were made of plain and reinforced concrete. Three different concrete grades: C8/10, C12/15 and C16/20 were used. The specimen had varying configurations of CFRP wrap: partial and full confinement in one and two layers. Four parameters are investigated in this study: concrete grade, steel reinforcement, degree of confinement and the number of layers of CFRP wrap. The specimens were subjected to uniaxial compression up to failure, and the stress-strain curves were plotted. This study found that the weakest concrete grade experiences the highest effect due to CFRP strengthening. Concrete C8/10 was 5.42 times more affected than Concrete C16/20 in terms of axial capacity (542%) but the effect was comparable in terms of ductility. Plain concrete specimens experienced higher effect in both axial capacity (114%) and ductility (145%) than reinforced concrete specimens. Full confinement was more effective in both the axial capacity (383%) and ductility (275%). Similarly, two layers was more effective in both axial capacity (435%) and ductility (292%). Experimental design optimisation showed that, partial CFRP confinement offers better material efficiency as compared to full CFRP confinement. These findings are instrumental in developing a rationale and a design method for retrofitting existing columns with CFRP wrap.

Keywords: Carbon Fibre Reinforced Polymer Wrap, Non-slender columns, retrofitting, axial capacity, ductility, CFRP, confinement, square concrete columns

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ABBREVIATIONS

Acronym	**Meaning**
BS	British Standard
BSI	British Standards Institute
CEN	European Committee for Standardisation
CFRP	Carbon Fibre Reinforced Polymer
EAS	East African Standard
EC	Eurocode
EN	European Standard
FRP	Fibre Reinforced Polymer
KEBS	Kenya Bureau of Standards
KS	Kenya Standards
LVDT	Linear variable differential transformers
NCA	National Construction Authority
NCAPD	National Coordinating Agency for Population & Development
NDT	Non-Destructive Test
RC	Reinforced Concrete
SLS	Serviceability Limit State
ULS	Ultimate Limit State

LIST OF SYMBOLS

Greek Symbols

Symbol	Meaning
ε_{c2}	Strain in concrete
$\varepsilon_{c2,c}$	Strain in confined concrete.
ε_{cu2}	Ultimate strain in concrete
$\varepsilon_{cu2,c}$	Ultimate strain in confined concrete.
ε_{fu}	Design ultimate tensile strain of CFRP.
σ	Working stress.
σ_1	Maximum principal stress in complex system.
σ_2	Stress in the lateral direction.
σ_3	Stress in the lateral direction (typically $=\sigma_2$)
σ_{ec}	Maximum stress at elastic limit in compression.
σ_{et}	Maximum stress at elastic limit in tension.
τ_{max}	Maximum shear stress.
ν_c	Poisson's ratio for concrete

Latin Symbols

Symbol	Meaning
ANOVA	Analysis of Variance
D	Diameter
df	Degrees of freedom
E_0	Secant modulus of plain concrete
E_i	Initial tangent modulus of concrete strain
E_p	Post-crushing tangent modulus
f_0	The intercept of post-crushing tangent modulus with the stress axis
f_{ck}	Characteristic strength of concrete
$f_{ck,c}$	Characteristic strength of confined concrete
f_{td}	Ultimate design tensile strength of CFRP
MPa	Megapascals – N/mm^2
n	Empirical factor
R	Radius of column

CHAPTER 1: INTRODUCTION

1.1 Background of the Study

Reinforced concrete (RC) columns are composed of concrete and steel reinforcement. Concrete is the material that bears most of the compressive stresses while steel reinforcement bears most of the tensile stresses in the column (McKenzie, 2013). The manufacture of concrete is influenced by numerous factors that lead to significant variability in its strength. RC columns may not attain the target strength when manufactured on site. Failure to attain the target strength may lead to difficulties such as higher deflections than estimated, cracking or even complete failure by crushing or buckling.

To determine the strength of the cast-in-place concrete, sample cubes are usually made and tested by the cube crushing method according to BS EN 12390-2:2009 (BSI, 2009). Non-destructive tests may also be used to determine the strength of cast-in-place concrete members. These non-destructive tests depend on the fact that specific physical properties of concrete are related to strength. Such properties include hardness, resistance to penetration by projectiles, rebound capacity and ability to transmit ultrasonic pulses, X-rays, and Y-rays. If columns do not attain the target strength, then the column may be demolished and recast. Demolitions are usually the last-ditch solution as they are usually quite expensive: financially, politically, time-wise and to the environment (The Constructor, 2019).

Alternative options to avoid demolition of the condemned columns is the use of steel angles or the use of Carbon Fibre Reinforced Polymer (CFRP) wrap to strengthen the weak columns. CFRP is a composite material which relies on the carbon fibre to offer strength and stiffness while the polymer provides a cohesive matrix to protect and hold the fibres together. CFRPs are frequently used wherever high strength-to-weight ratio and rigidity are required, such as in aerospace engineering, ships, automotive industry, civil engineering structures, sports equipment, and several consumer and technical applications (Fekete & Hall, 2017). The use of steel angles is the method commonly adopted in Kenya. Steel angles usually have the additional difficulty of having to damage the existing weak RC column by hacking in order to interface the concrete with the additional steel angles for composite action to take place. CFRP strengthening does not have this difficulty (The Concrete Society, 2000). However, CFRP wrap is not extensively used in Kenya. Its use is considered cutting-edge locally. This study shed light on the use of CFRP in strengthening columns.

Globally, the use of CFRP is not new and has been implemented in several other countries. In the United States of America, CFRP has been used to strengthen old infrastructure such as bridges. In Japan and other earthquake-prone countries, CFRP is used in seismic retrofitting (Bank, 2006).

Some of the advantages of using CFRP are its high strength, light weight, corrosion resistance, ease of use and speed of application. Its disadvantages include susceptibility to mechanical damage and poor response to fire (Khaled et al., 2018). Despite its advantages and relatively few disadvantages, retrofitting of columns with CFRP is still rare in Kenya.

The construction industry has experienced an upsurge in the requirement to reinstate, rejuvenate, strengthen and upgrade existing concrete structures. These requirements are in place to mitigate problems such as poor construction practices, design inadequacies, irregular maintenance, increase in loads and seismic conditions (Horse Construction, 2019; Jaya & Mathai, 2012). Since 1996, Kenya has experienced approximately two building collapses per year. Between 2006 and 2014, 17 buildings collapsed in Kenya, causing 84 deaths and more than 290 injuries (Kibet, 2016). In 2018, an audit report by the Nairobi City County revealed the following: out of the 1,572 buildings inspected, 884 were considered safe for human occupation, 471 were deemed unfit for occupancy, and 217 needed to be demolished as they were dangerous for human occupation (Omullo, 2018).

Source: Omullo, 2018

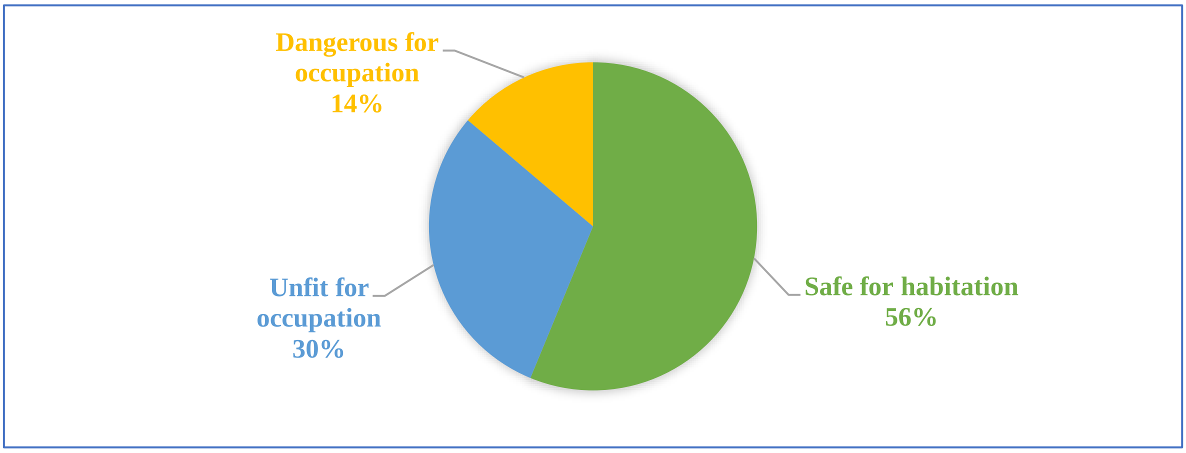


Figure 1.1: Graphic summary of Nairobi City Building Audit Report on buildings.

This audit showed that approximately 44% of buildings were unsafe. There is a need to strengthen some of these existing buildings to make them safe. Retrofitting existing columns to make them stronger may be one method of improving the structural soundness of these buildings.

In addition to that, the National Coordinating Agency for Population & Development (NCAPD) recommended that there was a need to plan land use more effectively in response to the increasing rural-urban migration (NCAPD, 2009). One of the methods of planning land use more effectively is to increase the capacity of existing buildings. This additional capacity would cater to the increasing population. Densifying the population by using taller buildings is an effective way of reducing the housing demand. This research intended that the use of CFRP to increase the axial capacity of columns would enable engineers to add the amount of occupiable space of existing buildings by the **safe** addition of new floors.

In this study, CFRP was used as a wrap in different configurations to strengthen concrete columns. Columns are structural members which are designed to carry predominantly compressive stresses in structures. The primary purpose of columns is to transfer loads to the foundation. Depending on the mode of failure, columns can either be classified as either slender or non-slender. Slender columns primarily fail by buckling, while non-slender columns fail by crushing (McKenzie, 2013). This study conducted tests on non-slender columns only. This is due to the fact that non-slender columns fail by crushing after the maximum axial capacity has been attained; unlike slender columns which fail by buckling before attaining the maximum axial load. The axial capacity of the columns was one of the variables to be investigated. Aesthetic, functional and structural requirements dictate the geometry of columns. Based on these requirements, columns can have different shapes: circular, triangular, square or rectangular. Square columns were selected for this study because they are widely used in construction for their aesthetic value since they are easily concealed within walls (Guo & Zeng, 2019). Moreover, square columns are also preferred over other shapes because of their symmetry along both axes.

Kenya has experienced several building collapses (Associated Press, 2019). In 2015, the National Construction Authority (NCA) found that 58% of buildings in Nairobi were unfit for habitation. That NCA building audit reported that eight buildings collapsed and killed 15 people in that year (NCA, 2015). This statistic is comparable to an audit report three years later by Nairobi City County that showed that 44% of buildings were unsafe (Omullo, 2018). Following such audits and reports, NCA and the various local governments restricted access to these unsafe buildings or condemned them for demolition. Evacuation and demolitions are expensive measures that contribute to the existing problem of high demand for housing (Obuya, 2012). Retrofitting some of these unsafe buildings is a viable solution to ensuring the safety of the buildings' users. Retrofitting is a better alternative than evacuation and demolition.

Nevertheless, there is limited knowledge and guidance for conducting such retrofits with regards to CFRP (The Concrete Society, 2000). The lack of guidance or design codes has led to a reluctance to adopt CFRP strengthening by engineers and owners. The absence of these design codes is due to the fact that CFRP is fairly novel and is only used in the maintenance of old buildings and infrastructure. CFRP has therefore not found wide application in several countries. This research addressed this problem by investigating four variables to improve the understanding of CFRP strengthening in columns.

Literature implies that change in behaviour of CFRP strengthened columns is caused due to full confinement. Most research conducted (Jaya & Mathai, 2012; Khaled et al., 2018; Mirmiran & Shahawy, 1997; Mohamed Saafi, 1999; Nanni & Bradford, 1995; Parvin & Wang, 2001; Rahai et al., 2008; Rochette & Labossière, 2000; Shehata et al., 2002; Shrive et al., 2003; The Concrete Society, 2000; Toutanji et al., 2007) studied the effect of full confinement of columns. However, some CFRP manufacturers claim that partial confinement offers similar benefits with less material being used (Horse Construction, 2019). A study by Guo et al. (2018) states that the confinement mechanism of columns partially wrapped by CFRP is less understood compared to partial confinement. That study supported the manufacturer's claim as it concluded that partial CFRP confinement was a promising and economical alternative to the fully CFRP strengthening technique. This research aimed at investigating this manufacturer's claim.

The behaviour of square columns strengthened by CFRP is not widely studied as CFRP wraps are recommended for circular columns (The Concrete Society, 2000). Square columns usually lead to stress concentrations at the edges resulting in premature failure before the maximum capacity of the retrofitted columns is attained. Nonetheless, square columns are preferred in construction because of architectural and constructability considerations (Guo & Zeng, 2019). This research provided additional data and knowledge on square columns strengthened by CFRP.

The NCA Building Report shows that building collapses have become more frequent, and the trend is likely to progress unless remedial measures are undertaken (NCA, 2015). Use of CFRP strengthening is one such remedial measure that may help curb this problem (Abadi et al., 2019).

It is against this background that the experimental study was conducted to investigate the effect of CFRP strengthening on the axial capacity and ductility of non-slender square concrete columns.

1.2 Problem Statement

Kenya experiences several building collapses. There is an urgent need to solve the problem of collapsing buildings by retrofitting condemned buildings with CFRP. The influence of the parameters required in retrofitting is not well understood and requires further research. Those parameters are the grade of concrete, the presence of steel reinforcement and the number of the confining layers. Secondly, there is a claim that partial CFRP confinement is as effective as full CFRP confinement. This claim has not yet proven to be true. The lack of design codes for CFRP retrofitting necessitates the need to investigate this claim.

Due to the absence of design codes for CFRP retrofitting, this study set out to find out how four parameters affected CFRP strengthening using an evidence-based approach. In this approach, samples would be tested to a failure and the data recorded. The data was then analysed and interpreted. The interpretation from the data would be used to justify any engineering decisions to be made with regards to the procedure of retrofitting RC columns.

1.3 Research Objectives

1.3.1 Overall Objective

The overall objective of this study was to investigate the effect of CFRP strengthening on the axial capacity and ductility of non-slender square concrete columns.

1.3.2 Specific objectives

To achieve the overall objective, the study pursued the following specific objectives:

1. To compare the increase in axial capacity and ductility due to CFRP strengthening on three different grades of concrete.
2. To compare the increase in axial capacity and ductility due to CFRP strengthening on plain and reinforced concrete.
3. To compare the increase in axial capacity and ductility between partial CFRP confinement and full CFRP confinement.
4. To find out the increase in axial capacity and ductility due to additional layers of CFRP strengthening.

1.4 Research Questions

These research questions guided the study:

1. What is the contribution of CFRP strengthening on three different concrete grades to the axial capacity and ductility of non-slender square columns?
2. What is the difference of CFRP strengthening on plain and reinforced non-slender square concrete columns?
3. What is the difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns?
4. What is the effect of additional layers of CFRP wrap on axial capacity and ductility of non-slender square concrete columns?

1.5 Hypotheses

This study tested four hypotheses. These hypotheses were stated in the null and alternative forms:

H_0: There is no relationship between concrete grades and the change in axial capacity and ductility of CFRP strengthened columns.

$\mu_{C8/10} = \mu_{C12/15} = \mu_{C16/20}$

H_A: There is a relationship between concrete grades and the change in axial capacity and ductility of CFRP strengthened columns.

$\mu_{C8/10} \neq \mu_{C12/15} \neq \mu_{C16/20}$

H_0: There is no significant difference between CFRP strengthening on plain and reinforced concrete in terms of axial capacity and ductility of non-slender square columns.

$\mu_{absent} = \mu_{present}$

H_A: There is a significant difference between CFRP strengthening on plain and reinforced concrete in terms of axial capacity and ductility of non-slender square columns.

$\mu_{absent} \neq \mu_{present}$

H_0: There is no significant difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns.

$\mu_{bare} = \mu_{partial} = \mu_{full}$

H_A: There is a significant difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns.

$\mu_{bare} \neq \mu_{partial} \neq \mu_{full}$

H_0: There is a significant difference between single layer CFRP confinement and double layer CFRP confinement on axial capacity and ductility of non-slender square concrete columns.

$\mu_{none} = \mu_{single} = \mu_{double}$

H_A: Additional layers of CFRP wrap have a relationship with the increment on axial capacity and ductility of non-slender square concrete columns.

$\mu_{none} \neq \mu_{single} \neq \mu_{double}$

1.6 Justification of the Study

This study was a multi-variable investigation of the parameters required when determining strengthening procedures of columns using CFRP wrap. The results and findings of this study shed light on four parameters when developing a design method for such procedures. Understanding these parameters would bridge the gap that exists due to the absence of design codes for retrofitting columns with CFRP.

Construction projects encounter challenges during the execution phase. Some problems that are of great significance are quality assurance difficulties such as when material provided does not meet the specifications. RC buildings have the additional complication in that tests on concrete take seven to 28 days to establish whether the concrete delivered to the site was up to standard. This lengthy testing procedure affects the project schedule in case the tests indicate that the material delivered was not satisfactory. Demolitions are typically the next course of action but are quite expensive. It is crucial to provide alternative solutions when such problems occur. The use of CFRP wrap may be viable when such issues arise since it is a solution that can be implemented rapidly on site ultimately resulting in the reduction of the project duration.

The high population pressure and the high cost of land in urban areas may necessitate the need for increasing buildings' capacity to cater for the increased housing demand. Use of CFRP strengthening may be adopted to increase the amount of occupiable space of existing buildings safely thus reducing the housing demand of existing housing.

1.7 Significance of the Study

This study is essential as it may provide the necessary guidance to structural engineers when using CFRP strengthening:

1. To increase the load-bearing capacity of a structure, for example, the addition of new floors or change of use from residential to commercial.

2. To add reinforcement to a structure that is already under-designed or wrongly constructed.
3. To develop a better understanding and consequently to leverage this understanding to fine-tune the parameters in the structural design process.

1.8 Assumptions

It was assumed that CFRP wraps increase the axial capacity and ductility of columns because of the confining effect CFRP will have on the concrete columns (Rahai et al., 2008).

Concrete mix design was not performed; the study assumed that the prescribed ratios of cement, fine and coarse aggregate for the manufacture of concrete would result in the target strength of concrete required (IQSK, 2019).

1.9 Scope and Delimitations of the Study

The axial capacity and ductility of columns were the only dependent variables to be investigated. The scope of the study was restricted to non-slender concrete columns under concentric loading. The loading had no eccentricity; therefore, failure would only occur once the maximum axial capacity of the columns was attained. The specimen height was also limited by the maximum clearance of the compression test machine, which consequently governed the cross-section dimensions to ensure the column would be categorised as non-slender.

The maximum number of layers to be used was two to minimise the cost of the experimental programme. The grades of concrete chosen: C8/10, C12/15 and C16/20 were the grades below the concrete commonly used for structural purposes. That typical concrete grade used for structural purposes is C20/25 (IQSK, 2019).

This experimental study is relevant to the construction industry. The findings of this study are relevant in retrofitting buildings wrongly constructed or designed. The results of the study present an opportunity to incorporate these findings in the repair and strengthening of existing columns. Repair, retrofitting and strengthening of columns is an area that needs to be explored further.

1.10 Definition of terms

Axial capacity refers to the maximum stress that the column can withstand (McKenzie, 2013). In this study, axial capacity was one of the dependent variables and was measured by subjecting the different specimen to axial compression until failure by using a compression test machine. The failure load was divided by the specimen cross-sectional area to calculate the axial capacity of the column.

Carbon fibres are fibres produced by the pyrolysis of organic precursor fibres such as rayon, polyacrylonitrile or pitch in an inert atmosphere. Carbon fibres are similar to graphite fibres but differ in the carbon content and the temperature at which the fibres are made and heat-treated (Fekete & Hall, 2017).

Carbon Fibre Reinforced Polymer (CFRP) are composite materials which rely on the carbon fibre to provide strength and stiffness. The polymer or plastic provides a cohesive matrix to protect and hold the fibres together (Fekete & Hall, 2017). In this study, CFRP was in the form of a wrap provided by a CFRP manufacturer.

Concrete grade refers to the different strength classes of concrete based on the failure stress when tested in a compression machine, and the mix ratio of cement, fine aggregate and coarse aggregate among other factors.

Concrete nomenclature is based on the designation adopted by the Eurocode to indicate a specific strength class $\mathrm{C}f_{ck}/f_{ck,cube}$. For instance, C20/25 represents standard concrete 'C' with a 28-day compressive cylinder strength of 20MPa and a corresponding cube strength of 25MPa. C20/25 would be designated as Class 25 concrete according to British Standards (McKenzie, 2013).

Ductility is a measure of the ability of a material to undergo significant plastic deformation before failure. This ability may be expressed as percent elongation. In this study, the ductility was quantified as the final deformation in a structural member as it was progressively loaded divided by its original length. The deformation was measured using a dial gauge.

Full confinement refers to external confinement by CFRP wrap by completely wrapping the specimen.

Non-slender columns refer to structural members which sustain predominantly compressive stresses. The failure mode of non-slender columns is by crushing. This terminology was previously known as short columns according to the BS 8110 designation (McKenzie, 2013).

Partial confinement refers to external confinement by CFRP wrap in the form of 50mm bands with 50mm spacing between the bands.

Polymer is a typically organic material, composed of molecules characterised by the repetition of one or more monomeric units (The Concrete Society, 2000).

Resin is an epoxy adhesive commonly used by the application on concrete. It is typically a solvent-free, two-pack material that cures at ambient temperature (The Concrete Society, 2000).

Slender columns refer to structural members which sustain predominantly compressive stresses and considerable flexural stresses. Because of the significant flexural stresses, slender columns experience first-order and second-order moments. The failure mode of slender columns is by buckling (CEN, 2004).

Steel reinforcement are the steel bars that are installed when placing fresh concrete to make it reinforced concrete. It is also known as rebar.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Fibre composites such as CFRP have been applied in other disciplines of engineering such as automotive and aerospace. In structural engineering, fibre composites have been used in aggressive environments such as factories to offer protection against chemical attack (The Concrete Society, 2000; Wei et al., 2020). This study did not look into such applications; it focused on the strengthening of concrete structures using CFRP bonded to the surface of the columns.

There are scenarios in which the axial capacity of a structure may need to be increased. Such scenarios include: the change of use of a building, where substandard materials were used, or where the structure was damaged. Steel plates or CFRP may be used to achieve this. According to The Concrete Society (2000), the techniques developed about that time for CFRP strengthening used similar principles as for steel plate bonding. CFRP is advantageous to steel plates in this application; in that they can be used in circumstances where it would be impractical to use steel. For example, CFRP is very flexible and can be formed into complicated shapes, unlike steel. CFRP is very light as compared to steel, and last of all is that CFRP is easily cut to length on site (Abadi et al., 2019).

2.2 Reinforced Concrete Columns

CFRP wrap is used in the strengthening of columns and takes advantage of the Poisson's effect in improving concrete strength by confinement. RC columns have steel reinforcement embedded. Steel reinforcement broadly falls into two categories: longitudinal and transverse. Longitudinal reinforcement caters for the tension in the columns due to flexure caused by eccentricity in the resultant load, and part of the compression. The concrete bears most of the compression. Transverse reinforcement, which is commonly known as stirrups, provides confinement of the concrete core as columns typically experience compression (McKenzie, 2013).

2.3 Theory of Confinement

When a vertical concrete column is subjected to uniaxial compression, the concrete deforms by a contraction in the longitudinal direction as it expands in the transverse direction. This phenomenon is known as Poisson's effect.

Adapted from: Par, 2010

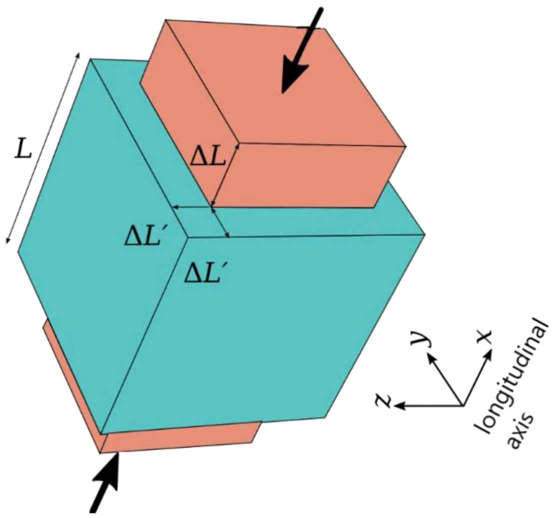


Figure 2.1: Illustration of Poisson's effect

The lateral expansion results in tension in the column. Concrete has low tensile strength capacity; hence the tension developed leads to failure of the column. When CFRP resists this lateral expansion, the concrete is changed to a three-dimensional compressive stress state. In this state, the performance of the concrete column is significantly influenced by the confinement pressure (Rahai et al., 2008). CFRP strengthening works by providing external confinement to the column, thus limiting the lateral expansion. The column is now in a three-dimensional state with no tension being experienced in the column and thus achieving higher stress before failure occurs.

Confinement, therefore, affects the stress-strain behaviour of standard concrete. Other factors influencing the stress-strain curve of confined concrete are concrete column characteristics which are attributed to the modulus of elasticity, strength and Poisson's ratio. The modulus of elasticity is the slope of the stress-strain curve within the range of elasticity. It is represented as change in stress over the change in strain as shown in Equation 2.1.

$$Modulus\ of\ elasticity = \frac{Change\ in\ stress}{Change\ in\ strain} \qquad \text{Equation 2.1}$$

CFRP confinement works by reducing the amount of strain at a particular stress and therefore alters the modulus of elasticity of the material confined. The strength of the material is correlated with the stress while the Poison's ratio determines how much the material deforms under stress and is related to the strain. These three parameters: modulus of elasticity, strength and Poisson's ratio, affect the confinement pressure generated in the confined column under axial compression. Secondly, the CFRP characteristics such as the CFRP modulus of elasticity and strength of the composite also affect the stress-strain behaviour. Finally, the cross-section characteristics, for instance, the geometry and shape, that is, circular, square or rectangular; play a significant role.

2.3.1 Strengthening Procedures

A Concrete Bridge Development Group (2000) reviewed four forms of strengthening RC structures. The first technique was to increase the reinforced concrete cross-section. This solution was readily acceptable by engineers because of its proven track record and is the most popular rehabilitation technique (Mohammed et al., 2020). It was reported that its most considerable disadvantage was that loading restrictions have to be imposed as the concrete cured to the required strength. This technique had negative implications on project schedules or disruptions, for example, in bridge closures.

The second option reviewed was the use of prestressing to relieve the dead load. Since prestressing had been used successfully and extensively in the past, engineers were very likely to recommend this option as well. However, because of the expensive equipment required in applying prestressing forces, this solution may not be technically viable where such equipment is not readily available.

The third option highlighted was to use steel plate and steel angle bonding to enhance the tensile reinforcement of the structural elements to be retrofitted. The disadvantages of this procedure were that: steel has difficult handling and fabrication; steel has the requirement of corrosion protection, and anchoring of the steel to the concrete section while avoiding significant damage to the existing member.

The final solution outlined was to add material around the existing structural element to provide confinement in compression members. Confinement could be achieved by installing in situ reinforced concrete. Jacketing also had the disadvantage of having to impose loading restrictions as the concrete cured and attained the desired strength. This solution significantly increases the overall structural self-weight that affects the foundation and attracts more forces in seismic events (Mohammed et al., 2020). In situations where space requirements would not allow a significant increase in cross-section, such as in parking facilities, or where installation time was critical, the use of CFRP was a viable solution (The Concrete Society, 2000).

2.3.2 Advantages of CFRP

CFRPs have higher ultimate strength and lower density than steel. These two properties, in combination, lead to fibre composites having the strength to weight ratio higher than the steel plate (Shrive et al., 2003). CFRPs have a yield stress and a tensile modulus of 4840MPa and 230GPa, respectively; while steel has 275MPa and 210GPa respectively. The density of CFRP is 2000kg/m^3 as compared to steel with 7850kg/m^3 (Gulgunje et al., 2015). The lower weight makes handling and installation considerably easier than steel especially when installing the material in cramped locations.

Works on soffits to repair slabs or beams may be carried out from human-access platforms rather than full scaffolding. Steel plates require heavy lifting gear and must be held in place while the adhesive gains strength. On the other hand, once CFRP has been rolled on carefully to remove entrapped air and excess adhesive, it may be left unsupported. Moreover, since there is no requirement to drill into the column to fix bolts, there is no risk of damaging the existing reinforcement.

CFRPs are available in very long lengths while steel plates are generally limited to six metres. CFRP is durable if correctly specified, and requires little maintenance. If damage occurs in service, it is relatively simple to repair them by adding another CFRP layer (Khaled et al., 2018).

The use of CFRP does not significantly increase the weight of the structure or the dimensions of the member. This property of CFRP is essential in the repair of bridges and other structures with limited headroom. CFRP has high economic benefits by minimising the time the structure is off-service (Mohammed et al., 2020). CFRPs are used in the maintenance of infrastructure located in marine environments not only to offer structural strengthening but also to offer cathodic protection (Wei et al., 2020).

In terms of environmental impact and sustainability, studies have shown that the amount of energy required to produce CFRP is less than that for conventional materials. CFRP has a minimal environmental impact (The Concrete Society, 2000).

2.3.3 Disadvantages of CFRP

The main disadvantage of using CFRP is the risk of fire or mechanical damage. Mechanical damage can be caused by vandalism or accidental damage unless the strengthening is protected. Studies and experience of the long-term durability of CFRP are scarce (The Concrete Society, 2000).

The lack of adequate knowledge is a disadvantage for structures for which a very long design life is required, but this can be overcome by appropriate monitoring. A perceived disadvantage of using CFRP is the relatively high cost of the materials.

The last significant disadvantage of using CFRP is the lack of accepted design standards on CFRP strengthening. This study attempted to address that challenge by developing a rationale in which the axial strength and ductility of strengthened columns can be determined based on scientific principles.

2.4 CFRP Properties

Some mechanical properties of the carbon fibre that are essential for the design of CFRP strengthening are the tensile strength, tensile modulus and elongation at break.

2.4.1 Performance of CFRP

Carbon fibre is resistant to most forms of chemical attack, which increases its utility in structural engineering applications. Nonetheless, there is evidence to suggest that in the presence of salts, fracture of the carbon fibres can occur due to the formation of angular crystals (The Concrete Society, 2000).

Carbon fibres can be used for exterior retrofits as the fibres are not affected by ultraviolet light. However, direct exposure to sunlight usually embrittles the resins, and protective paint is recommended if direct exposure to the elements is likely (The Concrete Society, 2000).

Carbon fibres have some electrical conductivity, and therefore, in Japan, CFRP strengthening material in railway application close to power lines must be electrically isolated from any steel reinforcement. With regards to stiffness, the elastic modulus value of carbon fibre varies from that of steel (210GPa) to significantly higher than that. (230GPa – 500GPa) (Gulgunje et al., 2015). When exposed to fire, carbon fibres oxidise in air above 650°C, but in this case, the resin polymer behaviour usually dominates performance and usually generates toxic smoke. All fibres, carbon fibre included, present a risk to human health in regular use. Proper care should be taken when cutting and machining all CFRP materials because fine fibre particles may irritate skin, eyes and mucous membrane. In addition to that, protective attire must be worn when handling the resin polymer. Carbon fibres cause an insignificant effect on the environment. Carbon fibres are non-toxic, inert and are not considered as hazardous waste. Furthermore, approaches have been developed to recycle composites (The Concrete Society, 2000).

2.5 Fibre Type

The fibre type determines the amount of confinement pressure that can be generated in the column. There are three main types of fibres utilised in FRP strengthening: glass, aramid and carbon fibre. Glass fibres are more cost-effective than carbon fibres. However, carbon fibres have superior characteristics in terms of strength. Aramid fibres have lower tensile load capacities compared to the other fibre types (Mohammed et al., 2020).

2.6 Stress-Strain Relationships for Confined Concrete

The design of CFRP strengthening systems is based on limit state principles. The aim of the limit state design is the achievement of an acceptable probability that the structure being strengthened will perform satisfactorily during its design life. Design involves checking that the structure does not reach a limit state during its intended life which may render it unfit for use. Limit states can either be ultimate or serviceability. Ultimate limit states typically encompass mechanisms that cause the partial or complete collapse of the structure. In contrast, serviceability limit states correspond to states which principally affect the appearance or proper performance of the structure. This study investigated compression strength at the ultimate limit state and the deflection at the serviceability limit state.

Several researchers (Saafi et al., 1999; Toutanji et al., 2007) proposed models that attempt to predict the compressive stress-strain behaviour of confined concrete. A large body of work relates to steel rather than CFRP confined concrete. The first studies to be conducted with regards to confined concrete were performed by Richart et al. (1928) and established Equation 2.2 (The Concrete Society, 2000).

$$f_{ck,c} = f_{ck} + 4.1\frac{\sigma_2}{f_{ck}} \qquad \text{Equation 2.2}$$

where f_{ck} = unconfined compressive strength

$f_{ck,c}$ = confined compressive strength

σ_2 = confinement pressure

Other researchers interested in the effect of steel confined concrete confirmed the validity of Equation 2.2. The confirmation was surprising considering the difference between the two modes of confinement applied. In the experimental programme conducted by Richart et al. (1928), the test specimens were subjected to active confinement due to hydrostatic pressure; hence the confinement pressure remained constant throughout the test. The other studies used steel reinforcement which offers passive confinement since the confining stress is neither uniform nor constant. Further research done on CFRP confirmed the applicability of Equation 2.2 to confined concrete. Comparison of the predicted and actual experimental results showed that, whereas the proposed models were sufficiently accurate concerning strength prediction, they grossly underestimated the ultimate strain. Consequently, significant discrepancies in the actual and predicted stress-strain responses existed (The Concrete Society, 2000). Following that realisation, Lilliston and Jolly (2000) conducted further research and showed that the strain response of FRP confined concrete could be predicted using Equation 2.3.

$$f_{ck,c} = \frac{0.67}{\gamma_{mc}} \times \frac{\left(E_i - E_p\right)\varepsilon_{c2,c}}{1 + \left[\varepsilon_{c2,c}\left(E_i - E_p\right)f_0\right]} \qquad \text{Equation 2.3}$$

where $\varepsilon_{c2,c}$ = axial confined concrete strain

E_i = initial tangent modulus of concrete strain given as:

$$E_i = 21500\left[\frac{(0.8f_{ck} + 8)}{10}\right]^{\frac{1}{3}} \qquad \text{Equation 2.4}$$

E_p = post-crushing tangent modulus

$$E_p = 1.282\left(\frac{2t_f}{D}\right)E_{fd} \qquad \text{Equation 2.5}$$

D = diameter of the column

f_0 = intercept of post-crushing tangent modulus with the stress axis given by:

$$f_0 = \frac{0.8f_{ck}(E_i - E_p)}{(E_t - E_i)} \qquad \text{Equation 2.6}$$

E_0 = secant modulus of plain concrete

$$E_0 = (0.8f_{ck} + 8)\varepsilon_{cu2} \qquad \text{Equation 2.7}$$

ε_{cu2} = ultimate strain on plain unconfined concrete.

Equation 2.3 is similar to Equation 2.8 for CFRP confined concrete by Arduini et al. (1999). Equation 2.8 was based on the experimental work of Miyauchi et al. (1997).

$$f_{ck,c} = \frac{0.67}{\gamma_{mc}} \times \frac{(E_i - E_p)\varepsilon_{c2,c}}{1 + [\varepsilon_{c2,c}(E_i - E_p)f_0]} \qquad \text{Equation 2.8}$$

where E_0 = secant modulus of concrete strain given by:

$$E_0 = 9.5\,(f_{ck} + 8)^{\frac{1}{3}} \;\; [E_0 \text{ in kN/mm}^2;\ f_{ck} \text{in MPa}] \qquad \text{Equation 2.9}$$

$\varepsilon_{c2,c}$ = axial confined concrete strain

n = empirical factor = 8

E_p = post crushing tangent modulus

$$E_p = \frac{(f_{ck,c} - f_{ck})}{\varepsilon_{c2,c}} \qquad \text{Equation 2.10}$$

$f_{ck,c}$ = characteristic strength of confined concrete

$$f_{ck,c} = f_{ck} + \frac{4 \times 0.85 f_{td} t_f}{R} \qquad \text{Equation 2.11}$$

$\varepsilon_{cu2,c}$ = ultimate axial strain of confined concrete

$$\varepsilon_{cu2,c} = \frac{\varepsilon_{fu}}{\nu_c}\left(1 + \sqrt{\frac{f_{ck}R}{f_{td}t_f}}\right) \qquad \text{Equation 2.12}$$

f_{td} = ultimate design tensile strength of CFRP

ε_{fu} = design ultimate tensile strain of CFRP

R = radius of column

ν_c = Poisson's ratio for concrete = 0.2

Comparisons of predicted results as proposed by Arduini et al. (1999) and that by Lilliston and Jolly (2000) exhibited similarities and accuracy with regards to confined strain. Likewise, further comparison of predicted values and experiment results from other studies (Saafi et al., 1999; Toutanji et al., 2007), showed good agreement between the empirical and the calculated values (The Concrete Society, 2000).

2.6.1 Anomalies in the Equations

A check of the equations in the publication by The Concrete Society (2000), reproduced here as Equation 2.2 all the way to Equation 2.12 shows that the equations may not be entirely accurate. Looking at Equation 2.2, the first term f_{ck}, would have the units in Megapascals (MPa) while the second term $\frac{\sigma_2}{f_{ck}}$, would be dimensionless because the units of the numerator (MPa) would cancel out with the units of the denominator. This error is replicated in all the equations from Equation 2.2 to Equation 2.12. The anomalies in these expression underpin the importance of conducting confirmatory studies on these equations.

2.6.2 Local Design Process

Kenya adopted the Eurocodes which provides some guidance with regards to confined concrete (Republic of Kenya, 2014). In the specific Eurocode standard EN 1992-1-1:2004; clause 3.1.9 (1) states that confining concrete results in an adjustment of the effective stress-strain relationship whereby higher strength and higher critical strains may be achieved. For design purposes, other essential material characteristics may be considered as unaffected (CEN, 2004).

EN 1992-1-1:2004 gives a design procedure where the characteristic strength of the concrete may be increased according to Equation (3.24) and Equation (3.25) reproduced as Equation 2.13 and Equation 2.14 respectively.

EN 1992-1 (3.24) $$f_{ck,c} = f_{ck}\left(1.000 + 5.0\frac{\sigma_2}{f_{ck}}\right)\ for\ \sigma_2 \leq 0.05 f_{ck}$$ Equation 2.13

EN 1992-1 (3.25) $$f_{ck,c} = f_{ck}\left(1.125 + 2.5\frac{\sigma_2}{f_{ck}}\right)\ for\ \sigma_2 > 0.05 f_{ck}$$ Equation 2.14

According to EN 1992-1-1:2004, we may also get additional strains as given in equation (3.26) and equation (3.27):

EN 1992-1 (3.26) $$\varepsilon_{c2,c} = \varepsilon_{c2}\left(\frac{f_{ck,c}}{f_{ck}}\right)^2$$ Equation 2.15

EN 1992-1 (3.27) $$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2\frac{\sigma_2}{f_{ck}}$$ Equation 2.16

Where σ_2 $(= \sigma_3)$ is the effective lateral compressive stress at the ULS due to confinement and $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$. Figure 3.6 of EN 1992-1-1, reproduced here as Figure 2.2 shows the stress-strain curve for confined concrete (CEN, 2004). The units of the expressions given in EN 1992-1-1 by CEN (2004) are consistent and do not have the challenges of Equation 2.2 to Equation 2.12 given by The Concrete Society (2000).

Source: CEN, 2004

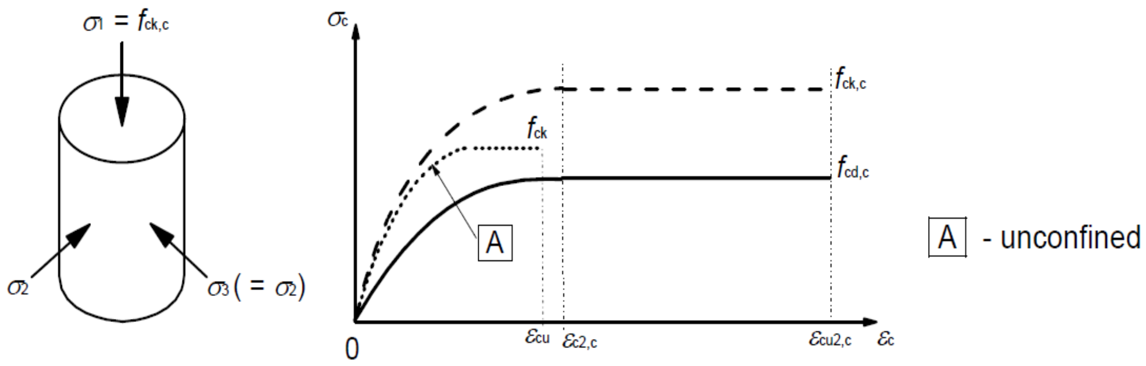


Figure 2.2: Stress-strain relationship for confined concrete.

It is, however, beneficial to note that since CFRP is typically applied after loading has been applied, there is little confinement pressure caused by the CFRP wrap. The confinement pressure only comes into play once deformation has occurred in the column as it is loaded. This kind of confinement is passive (Arduini et al., 1999). The effective lateral compressive stress σ_2 and σ_3, is therefore unknown and requires further research to establish its value. As the confinement pressure increases, the more the deformation in the concrete column (Shrive et al., 2003).

Several studies have been conducted investigating CFRP strengthening on columns. Some of the findings are presented in the following sections categorised by the independent variables of this study.

2.7 Concrete Grade

Shrive et al. (2003) used a simple analysis of circular columns, performed finite element analysis of a chamfered column and concluded that the confining effect of the wrap was not engaged until the concrete started failing and dilating. The strain readings analysed confirmed the conclusion arrived at; columns composed of weaker strength grades fail at lower axial stress and dilate more than columns of stronger concrete grades. This higher dilation will cause higher lateral pressures and more confining effect on the weaker columns resulting in an increased axial capacity gain. It was found that strength gains can be expected to reduce with increasing strength of the concrete. The finding showed that there is an inverse relationship between concrete strength grade and an increase in gain in strength.

2.8 Reinforcement

Shrive et al. (2003) investigated reinforced and prestressed concrete columns with one or two layers of CFRP wrap to failure in axial compression. This study found the predictions of two proposed design methods consistently underestimated actual failure loads and that the design procedure was thus conservative. That study did not investigate strengthening on plain concrete.

2.9 Degree of CFRP Confinement

The work of Richart et al. (1928) on triaxially confined concrete based on tests on cylindrical specimens subjected to constant hydrostatic pressure showed that both axial strength and ductility of concrete increases with increasing confinement pressure. CFRP strengthening utilises this principle.

The studies reviewed utilised full confinement (Arduini et al., 1999; Jaya & Mathai, 2012; Khaled et al., 2018; Masia et al., 2004; Nanni & Bradford, 1995; Parvin & Jamwal, 2004; Parvin & Wang, 2001; Rochette & Labossière, 2000; Shrive et al., 2003). This study found few studies where partial CFRP confinement was recommended (Guo et al., 2018; Guo & Zeng, 2019). Further investigation was made by this research programme to find out the effect of reducing the CFRP wrap for economic purposes based on the manufacturer's claims, as shown in Figure 2.3.

Source: Horse Construction, 2019

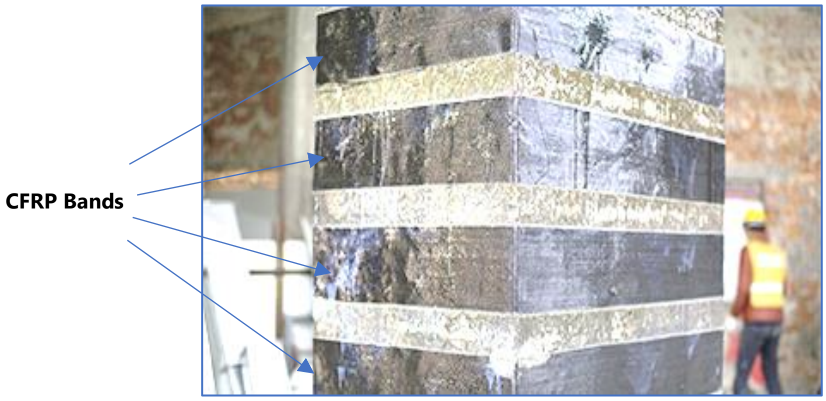


Figure 2.3: Partial CFRP confinement of a column.

A study of partial CFRP confinement by Guo and Zeng (2019) only looked at axial behaviour and did not look at the ductile behaviour of partially confined columns. This study claims that another reason why partial CFRP confinement may be considered is because partial CFRP confinement has the advantage of easier and faster application than utilising full CFRP confinement (Guo & Zeng, 2019).

2.10 CFRP Thickness

CFRP thickness has substantial effects on the strength and ductility of repaired columns (Mohammed et al., 2020). Thickness was directly related to the exerted confinement pressure of the FRP jacket as the confinement effectiveness increases with the higher thickness (Ozbakkaloglu, 2013).

Saafi et al. (1999) tested non-slender concrete columns confined with FRP tubes under uniaxial compressive load to investigate the effects of FRP jacket thickness in addition to fibre type and concrete strength. It was reported that FRP jacket thickness has a direct relationship with the increase in axial capacity and ductility of the columns. Rochette and Labossière (2000) axially tested the effect of wrap thickness and cross-section shape of non-slender columns on its strength and also came to similar conclusions; CFRP thickness increases axial capacity and ductility.

Parvin and Wang (2001) performed an experimental and numerical analysis of FRP jacketed square concrete columns under eccentric loading. Later, Parvin and Jamwal (2004) investigated small-scale FRP wrapped concrete cylinders under uniaxial compressive loading through nonlinear finite element analysis. This subsequent investigation found that an increase in wrap thickness had a positive change on axial strength and ductility of the concrete cylinders. The results from the two separate studies thus showed that the strength and ductility of concrete columns could significantly increase due to CFRP strengthening (Parvin & Jamwal, 2004; Parvin & Wang, 2001).

Shehata et al. (2002) conducted an experimental research programme that included tests on 54 non-slender columns to find out the gain in axial capacity and ductility of concrete columns externally confined by CFRP wrap. The variables studied in the programme were column cross-section shape and the amount of confinement expressed in the number of CFRP sheets applied to the specimen. The number of layers was limited to a maximum of two. Shrive et al.

(2003) investigated the effect of wrap thickness and showed that whereas strength gain is directly related to CFRP wrap thickness, the relationship is not linear.

2.11 Column shape

Most of the available studies on the behaviour of FRP confined concrete columns have focused on circular columns, while relatively few studies have looked into rectangular columns (Toutanji et al., 2007). The column given in Figure 3.6 of the Eurocode, EN 1992-1-1:2004, is that of a cylindrical specimen with a circular cross-section. The bias on circular columns is somewhat because a square section is not uniformly confined, and the compressive pressure is unevenly distributed.

Nanni and Bradford (1995) test specimens consisted of 150 x 300 mm high cylinders to verify existing analytical models. Their experimental results indicated that the FRP jackets increased the ductility and strength significantly. Mirmiran et al. (1997) also used cylinders which had FRP jackets and found that there was a significant increase in axial capacity and ductility. Their findings indicated that, as the jacket thickness increased, the strength and ductility increased as well.

In square and rectangular cross-sections, the stress-strain curve is affected by the radius to which the corners of the sections are rounded off in order to avoid the breakage of fibres. Rounding of corners leads to a reduction in cross-section area leads to a corresponding decrease in axial capacity. This reduction in strength may lead to catastrophe for buildings already vulnerable to collapse. This research programme did not round the corners of the square columns. Failure to round the corners of square columns would have the effect of reduced confined area (Masia et al., 2004).

Source: Masia et al., 2004

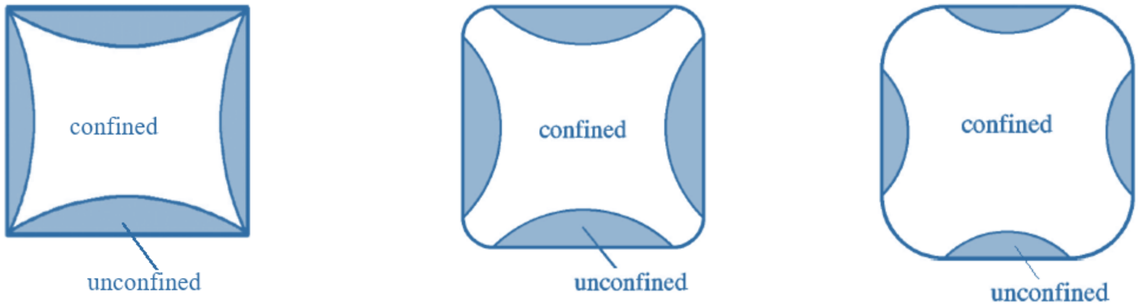


Figure 2.4: Increased confined areas for increased rounding radiuses.

Despite the effectiveness of CFRP confinement in circular columns, square columns are usually preferred because of ease in the fabrication of formworks, constructability and architectural reasons (Guo et al., 2018; Guo & Zeng, 2019).

2.12 Size of the Columns

Masia et al. (2004) investigated the size effects of square columns. That study found significant effects in axial capacity and ductility were achieved by wrapping. The effectiveness of the wrap, as measured by the percentage increases in strength and peak axial strain, reduced with increasing cross-sectional size. The bigger the column cross-section, the less the effect of CFRP strengthening.

2.13 Theories of Failure

Theories of failure are the criteria use to predict the failure of a material under multi-axial stress. Failure is defined as the point at which the material can no longer perform its design function. Failure occurs due to complete failure characterised by brittle fracture or by excessive deformation characterised by ductile failure. Under axial loading, the stress-strain curve can be used to represent the response until failure. Under multi-axial stress, failure theories are needed for representing the material behaviour based on plasticity or yielding and fracture. Certain theories have been advanced to explain the cause of failure and many of theories have received considerable experimental investigation. No great uniformity of opinion has been reached, and there is still room for a great deal of further experimental investigation.

The principal theories are:

1. Maximum principal stress theory
2. Maximum shear stress or stress difference theory
3. Strain energy theory
4. Shear strain energy theory
5. Maximum principal strain theory
6. Mohr-Coulomb theory

2.13.1 Maximum Principal Stress Theory

The first theory to be reviewed is the maximum principal stress theory. This theory is associated with Rankine, but also received considerable support from other writers. The maximum principal stress theory is the simplest and oldest theory of failure.

According to this theory, failure will occur when the maximum principal tensile stress σ_1 in the complex system reached the value of the maximum stress at the elastic limit σ_{et} in simple tension or the minimum principal stress reached the elastic limit stress σ_{ec} in simple compression. If the maximum principal stress is the design criterion, the working stress σ must not exceed the maximum principal stress for the material. Hence, $\sigma \leq \sigma_1$.

This theory disregards the effect of other principal stresses the shearing stresses of other planes through the element. For brittle materials which do not fail by yielding but by brittle fracture, the maximum principal stress theory is considered to be reasonably satisfactory. The maximum principal stress theory appears to be approximately correct for ordinary cast-irons and brittle metals. Concrete being a brittle material is most likely to follow this theory (Jianxia, 2012). One of the contentions of this theory is that on a mild steel specimen when simple tension is carried out sliding occurs and this shows that the failure is due to maximum shear stress rather than direct tensile stress. Secondly, it has been found that a material which is even weak in simple compression yet can sustain hydrostatic pressure far in excess of the elastic limit in simple compression.

2.13.2 Maximum Shear Stress or Stress Difference Theory

The second theory is the maximum shear stress theory and is also known as stress difference theory. This theory is also called Guest's or Tresca's theory. This theory implies that failure will occur when the maximum shear stress τ_{max} in the complex system reaches the value of the maximum shear stress in simple tension at the elastic limit. This theory gives good correlation with results of experiments on ductile materials. This theory has been found to give quite satisfactory results for ductile materials. The demerits of this theory are that: it does not give accurate results for the state of stress or pure shear in which the maximum amount of shear is developed such as in a torsion test. The theory is not applicable in the case where the state of stress consists of triaxial tensile stresses of nearly equal magnitude reducing the shearing stress to a small magnitude so that failure would be by brittle fracture than by yielding. Lastly, the theory does not give as close results as found by experiment on ductile materials. However, it gives safe results.

2.13.3 Strain Energy Theory

This theory which has a thermodynamic analogy and a logical basis is due to Haigh. This theory states that the failure of a material occurs when the total strain energy in the material reaches the total strain energy of the material at the elastic limit in simple tension. However, the results of this theory are not similar to experimental results for ductile materials. This theory does not apply to materials for which maximum elastic tensile stress is quite different from elastic compressive stress. This theory does not give results exactly equal to the experimental results even for ductile materials, even though the results are close to the experimental.

2.13.4 Shear Strain Energy Theory

This theory is also called 'Distortion energy theory' or 'Mises-Henky theory'. According to this theory, the elastic failure occurs where the shear strain energy per unit volume in the stressed material reaches a value equal to the shear strain energy per unit volume at the elastic limit point in the simple tension test.

The contentions of this theory are that it does not agree with experimental results for the material for which maximum elastic tensile stress is quite different from maximum elastic compressive stress. Secondly, the theory gives $\sigma_{et} = 0$ for hydrostatic pressure or tension, which means the material will never fail under any hydrostatic pressure or tension and this is obviously not correct. Actually, when three equal tensions are applied in three principal directions, brittle fracture occurs and as such maximum principal stress theory will give reliable results in this case. Lastly, this theory is regarded as one to which conform most of the ductile materials under the action of various types of loading.

2.13.5 Maximum Principal Strain Theory

This theory is associated with Saint Venant. The theory states that the failure of a material occurs when the principal tensile strain in the material reaches the strain at the elastic limit in simple tension or when the minimum principal strain, that is, the maximum principal compressive strain reaches the elastic limit in simple compression (CEN, 2004).

The disadvantage of this theory is that it overestimates the behaviour of ductile materials and the theory does not fit well with the experimental results except for brittle materials for biaxial tension, for which it is sometimes recommended, and is not much used in practice.

2.13.6 Mohr-Coulomb Theory

Mohr–Coulomb theory is a mathematical model describing the response of brittle materials such as concrete, soils, or rubble piles, to shear stress as well as normal stress. Most of the classical engineering materials somehow follow this rule in at least a portion of their shear failure envelope. Generally, the theory applies to materials for which the compressive strength far exceeds the tensile strength (Juvinall & Marshek, 2019).

2.14 Expressions

Some of the expressions, found in the literature, for estimating the confined concrete strength and the axial strain at failure are summarised in Table 2.1.

Table 2.1: Summary of expressions for confined concrete properties in literature.

Author	Type of Confinement	Strength $f_{ck,c}$	Ultimate Axial Strain $\varepsilon_{cu,c}$
Saafi et al. (1999)	CFRP and CFRP encased concrete	$f_{ck}\left[1+2.2\left(\frac{f_t}{f_{ck}}\right)^{0.84}\right]$	$\varepsilon_{cu}\left[1+(537\varepsilon_f+2.6)\left(\frac{f_{cc}}{f_{ck}}-1\right)\right]$
Toutanji et al. (2007)	CFRP and GFRP wrapped concrete	$f_{ck}\left[1+3.5\left(\frac{f_t}{f_{ck}}\right)^{0.85}\right]$	$\varepsilon_{cu}\left[1+(310.57\varepsilon_f+1.9)\left(\frac{f_{cc}}{f_{ck}}-1\right)\right]$
Spoelstra & Monti (1999)	CFRP and GFRP wrapped encased concrete	$f_{ck}\left[0.2+3\left(\frac{f_t}{f_{ck}}\right)^{0.5}\right]$	$\varepsilon_{cu}\left[2+1.25\frac{E_c}{f_{ck}}\varepsilon_f\left(\frac{f_f}{f_{ck}}\right)^{0.5}\right]$
Eurocode EC2 (2004)	Any $for\ \sigma_2 \le 0.05f_{ck}$	$f_{ck}\left[1.000+5.0\frac{\sigma_2}{f_{ck}}\right]$	$\varepsilon_{c2,c} = \varepsilon_{c2}\left[\frac{f_{ck,c}}{f_{ck}}\right]^2$
	Any $for\ \sigma_2 > 0.05f_{ck}$	$f_{ck}\left[1.125+2.5\frac{\sigma_2}{f_{ck}}\right]$	$\varepsilon_{cu2,c} = \varepsilon_{cu2}+0.2\frac{\sigma_2}{f_{ck}}$

The main challenge of using these expressions is that they have parameters which are not known. For example, the expressions from Eurocode 2 have the parameter σ_2, which denotes the confinement pressure. The confinement pressure σ_2, is however not known and varies with the pressure applied (Shrive et al., 2003). The lack of knowledge on these parameters necessitates the need to conduct further research on confined concrete.

2.15 Contentions with Previous Studies on CFRP Confined Concrete

Several studies (Khaled et al., 2018; Mirmiran & Shahawy, 1997; Nanni & Bradford, 1995) used circular cylinders, whereas this research used square columns. Previous research has a bias to circular columns which have a higher degree of confinement than in square columns as evidenced in the study by Masia et al. (2004).

The design procedure advocated by Eurocode 2 (EN 1992-1-1) assumes that the lateral pressure is known and constant however other literature shows that the confining effects of the wrap are not engaged until the concrete fails and dilates causing confinement pressure that varies with the deformation experienced in the concrete (Shrive et al., 2003).

The equations highlighted in Table 2.1 do not have a variable to cater for the effect of thickness of the CFRP wrap or the confining material and the design methods use the simple extension of the models developed for conventional reinforced concrete columns (Mirmiran & Shahawy, 1997).

2.16 Failure Modes of Columns Under Compression

The degree of platen restraint on the concrete section depends on the friction developed at the interface of the concrete specimen and steel platen, and on the distance from the end surfaces of the concrete. Consequently, in addition to the imposed uniaxial compression, there is a lateral shearing stress, the effect of which is to increase the apparent compressive strength of the concrete. The influence of platen restraint can be seen from the typical failure modes of test cubes, shown in Figure 2.5. The effect of shear is always present. That effect decreases towards the centre of the cube, so that the sides of the cube have near-vertical cracks, or completely disintegrate to leave a relatively undamaged central core (a) and (b) of Figure 2.5. This type of failure happens when testing in a rigid testing machine, but a less rigid machine can store more energy so that an explosive failure is possible as shown in Figure 2.5 (c); one face touching the platen cracks and disintegrates to leave a pyramid or a cone. Types of failure, for instance, those shown in Figure 2.6, are regarded as unsatisfactory and indicate a probable fault in the testing apparatus (Neville & Brooks, 2010).

Source: BSI, 2019

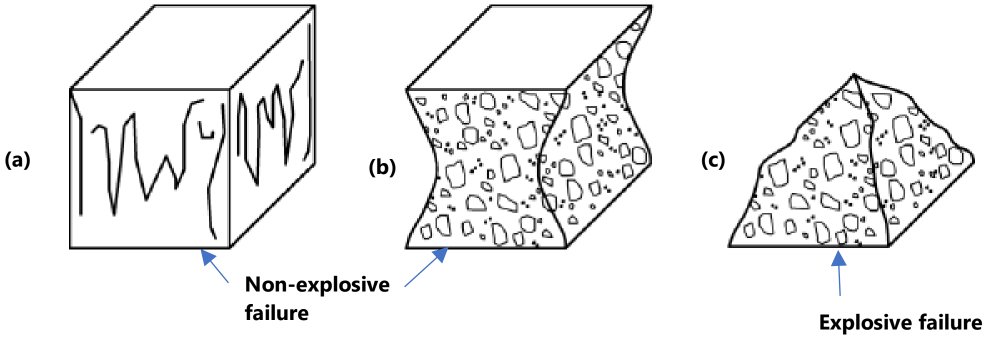


Figure 2.5: Satisfactory failure of specimens.

Figure 2.6 shows the unsatisfactory failure of specimens characterised by the formation of tensile cracks.

Source: BSI, 2019

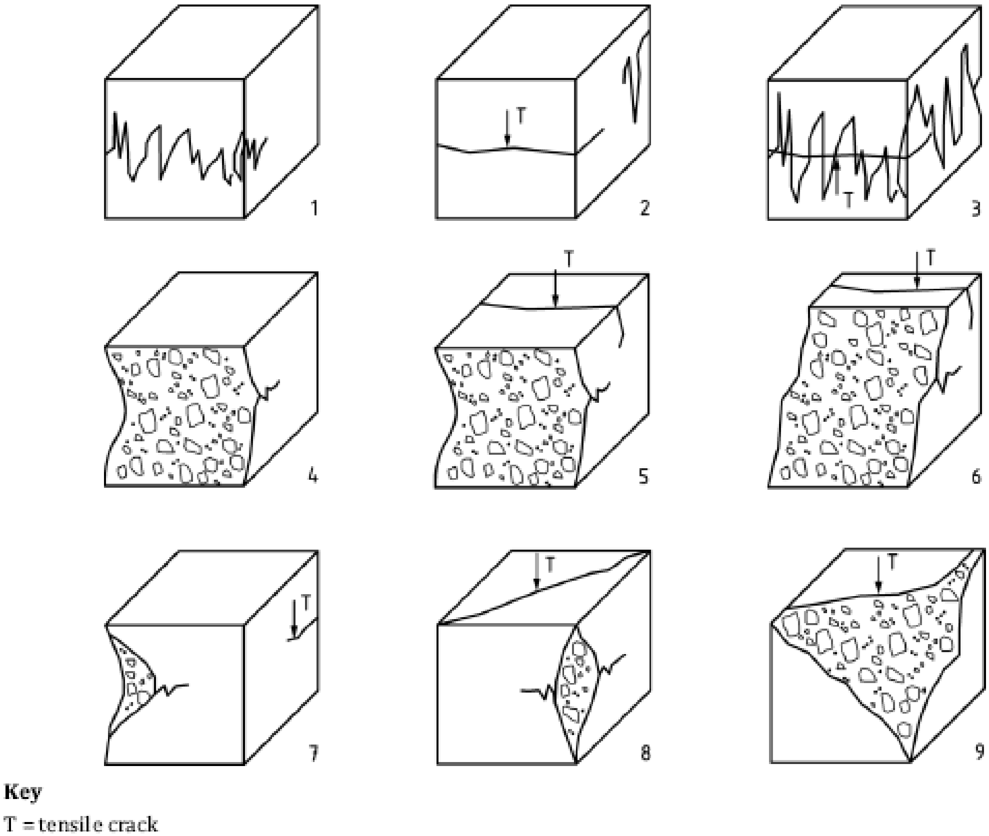


Figure 2.6: Unsatisfactory failure of specimens.

2.17 Literature Gap

An accurate prediction of the stress-strain curve of composite confined concrete is rather complex to obtain due to the high number of variables that affect the model (Shehata et al., 2002). With this realisation and despite extensive work on CFRP on concrete columns, there is little research with regards to the effect of CFRP on non-slender columns when comparing:

1. Concrete strength grades.
2. Plain and reinforced concrete.
3. Partial and full confinement.

An experimental programme is necessary to find out the effect of these parameters.

2.18 Experimental Design

An experimental design is helpful when carrying out an investigation where there are several variables to be investigated so that a conclusion could be drawn regarding a hypothesis statement. The intention is to establish the effect that a factor or an independent variable has on a dependent variable. The experimental design is used to test and validate the relationship between and among experimental variables (Bell, 2009). Experimental design is a procedure in the creation of a detailed experimental plan that allows the maximum amount of information specific to the objectives to be obtained (McIntosh & Pontius, 2017). This procedure enables the research to maintain control over other factors that affect the result of an experiment.

Optimisation of an experimental design refers to the process whereby maximum amount of useful information is obtained with the minimum number of experiments (Poole & Poole, 2012). One factor can be varied at a time to find out the effect of that factor. The procedure of varying one factor at a time requires that several controls with different treatment to be set up. To find out the combined effect of all the factors, the tested samples can be compared against the different controls. To make this comparisons, and to draw conclusions from the data; statistical tests are usually conducted (Bell, 2009).

2.19 Statistical Tests

At times random variations can make it difficult to know if there are true differences or whether it is just random. Test statistics are calculated from sample data and allow us to quantify how close; things are to our expectation or theories. Test statistics allow us to add mathematical rigour to the test process to facilitate decision-making. The amount of variance in a group is really important in judging a difference (Brangard, 2018b). In order to test the significance and the trends in the data, three main statistical analysis methods are considered. The tests are the z-score, t-test and the Analysis of Variance (ANOVA).

2.19.1 Z-Score

The z-score also called the z-statistic or z-test tells us how many standard errors away from the sampling distribution is from our group mean is. The z-statistic of around 1 or -1 tells us that the typical distance we would expect a typical sample mean to be from the mean of the null hypothesis is one standard deviation (Molugaram & Rao, 2017). We can use z-tests to do hypothesis tests about means, differences between means, proportions, or even differences between proportions. Critical value is a value of our test statistic that marks the limits of our extreme values. A test statistic that is more extreme than these critical values that is towards the tails causes the rejection of the null hypothesis. The critical value is calculated by finding out which test-statistic value corresponds to the top 0.5%, 1% or 5% most extreme values. For a z-test with $\alpha = 0.05$, the critical values are 1.96 and -1.96. If the z-statistic is more extreme than the critical value, it is statistically significant. The z-test is used where there is the population standard deviation (Brangard, 2018a). In some situations, a z-test does not apply, and when that happens, we use the t-distribution and corresponding t-statistic to conduct a hypothesis test.

2.19.2 T-Test

The t-test is similar to the z-test. It uses the same general formula for its t-statistic. We use a t-test if we do not know the true population standard deviation (Brangard, 2018c). The t-statistic is similar to the z-statistic, except that we are using our sample standard deviation instead of the population standard deviation in the denominator. The t-distribution looks like the z-distribution, but with thicker tails. Estimation adds a little more uncertainty which means thicker tails, since extreme values are a little more common. But as we get more and more data, the t-distribution converges to the z-distribution. This implies that, with really large samples, the z-test and t-test should give us similar p-values. The tails are thicker because we are estimating the true population standard deviation. In situation where the population standard deviation is available, a z-test is most favourable. A t-test is favourable when the population standard deviation is not available. T-tests are good for testing the difference between two groups.

For the z-test and the t-test, two methods are used to decide whether something was significant: Critical values and p-values. These two methods are equivalent. Large test statistics and small p-values both refer to samples that are extreme. A test statistic that is bigger than the critical value allows us to reject the null hypothesis. A test statistic that is larger than the critical value will have p-value less than 0.05. So, the two methods lead us to the same conclusion. But we often use the p-values instead of critical values because each test-statistic, like the z-statistic or t-statistic, have different critical values, but a p-value of less than 0.05 means that your sample is in the top 5% of extreme samples regardless of the test-statistic method such as the f-test or chi-square. Test statistics form the basis of how we can test if things are actually different or what we are seeing is just normal variation. They help us know how likely it is that our results are normal, or if something interesting is going on (Brangard, 2018b). T-distributions are used when the number of samples is less than 30.

Alternatively, another statistical method called ANOVA could be used to compare the three groups. This study had more than the 30 samples and thus ANOVA would be used.

2.19.3 Analysis of Variance Test (ANOVA)

The analysis of variance test is also known as ANOVA. Unlike the t-test which measures the variations between two groups, ANOVA is used to test the difference between multiple groups (Brangard, 2018d). It is used whenever there is a measurement of more than two groups. It uses the general linear model framework.

2.19.4 Post hoc tests after ANOVA

The ANOVA tests if there is a difference between multiple groups. The ANOVA test will however not tell which group is different from which. Post hoc tests are necessary to do comparisons between multiple groups that have already been done using ANOVA. Post hoc tests are pairwise comparisons that are often made after conducting a variety of inferential statistical procedures such as ANOVA. Some post hoc tests and their characteristics are given a brief highlight. Post hoc tests do not run if there are only two levels of independent variables (Grande, 2015). Post hoc tests are omnibus tests. Omnibus tests are tests that contain many items or groups (Brangard, 2018d).

There are several post hoc tests. The researcher selects the best possible method for analysis. Types of post hoc tests in the software used for analysis are discussed (Grande, 2015). Least significant difference (LSD) test is not ideal as it does not control for type I error inflation so the LSD is considered a liberal post hoc test since the probability of a type I error is high as compared to a conservative test where the probability of a type I error is low.

The Bonferonni method is a good post hoc test for controlling for type I error and it has good statistical power when the number of comparisons is low (IBM Corp., 2021). The term statistical power means the ability to detect a difference that is really there. If you want a control for the type I error but you have a large number of comparisons the Tukey post hoc test is a popular method. The REGWQ is good for controlling Type I errors and has good statistical power (IBM Corp., 2021). The REGWQ is not good when group sizes are different.

Two post hoc test that are good when comparing group sizes are different are Hotchberg's GT2 and Gabriel. If the group sizes are slightly different, the Gabriel method is the better. When the group sizes are greater, the Hotchberg's GT2 is a better choice (Grande, 2015).

Regardless of the post hoc test method used, the conclusions that could be drawn from using these tests are identical. Only the significance values may be different depending on the post hoc test used. Usually one post hoc method is selected based on the characteristics of the research design, the design data and group sizes (Grande, 2015).

2.19.5 Level of Confidence

The level of confidence shows how confident the study is in its decision (Brangard, 2018b). This has the abbreviation of **c**.

2.19.6 Level of Significance

The level of significance is denoted by α and is the complement of the level of confidence. The α-level is arbitrary but 0.05 is typically used in several other studies. The value of 0.05 means that only 5% of tests done on groups with no real difference will incorrectly reject the null hypothesis resulting in a type I error (Brangard, 2018c).

$$\alpha = 1 - c \qquad \text{Equation 2.17}$$

where α = level of significance.

c = level of confidence.

2.19.7 Type I and Type II errors

There are two possible outcomes when performing a hypothesis test. A correct decision or an error may be made. An error is made when a true hypothesis is rejected or when a false hypothesis is accepted. A Type I error is made when a true hypothesis is rejected while a Type II error is made when a false hypothesis is accepted. Increasing the level of significance α, increases the likelihood of committing a Type I error. Increasing the mean, increases the likelihood of committing a Type II error. To decrease the likelihood of committing Type I or Type II errors, the sample size is usually increased.

2.20 Theoretical Framework

The theory of Poisson's effect guided this research. It is assumed that most of the increase in axial capacity and ductility of the concrete column was provided due to the confining effect of the CFRP wrap. EN 1992-1-1 states that the confinement may be generated by adequately closed links or cross-ties, which reach the plastic condition due to the lateral extension of the concrete (CEN, 2004).

First, the study established the influence of concrete grade on CFRP strengthened columns. Secondly, it determined the contribution of each of the confining materials, the steel reinforcement bars and CFRP wrap. The third objective was to differentiate the effect of degree of confinement on the axial capacity and ductility. Lastly, it investigated the effect of the number of layers on axial capacity and ductility of CFRP strengthened concrete columns.

2.21 Conceptual Framework

The independent variables for this study were:

1. Concrete Strength Grade – The study performed tests on the specimen manufactured with the three concrete grades: Concrete C8/10, Concrete C12/15 and Concrete C16/20. The three different concrete grades had 30 specimens each.
2. Presence or absence of reinforcement – Specimen were either plain concrete or reinforced concrete. Half of the specimens manufactured had no steel reinforcement, and the remaining half had steel reinforcement.
3. Degree of CFRP confinement – Eighteen specimens had no CFRP confinement, CFRP fabric bands partially wrapped 36 specimens, and 36 specimens were fully covered by the CFRP fabric wrap.
4. The number of CFRP wrap layers – The specimen to be confined by CFRP wrap either had one or two layers. Thirty-six specimens had one layer of CFRP wrap, and 36 specimens had two layers of CFRP wrap.

The summary of the specimens is represented graphically in Figure 3.12.

The dependent variables that were measured were:

1. Axial capacity – This was the value at which the concrete columns failed by crushing when subjected to increasing axial load in the compression test machine.
2. Ductility – This was measured as longitudinal deflection as the column was subjected to increasing axial load. Ductility was calculated as the total strain at failure as a percentage.

An illustration of the conceptual framework is presented in Figure 2.7.

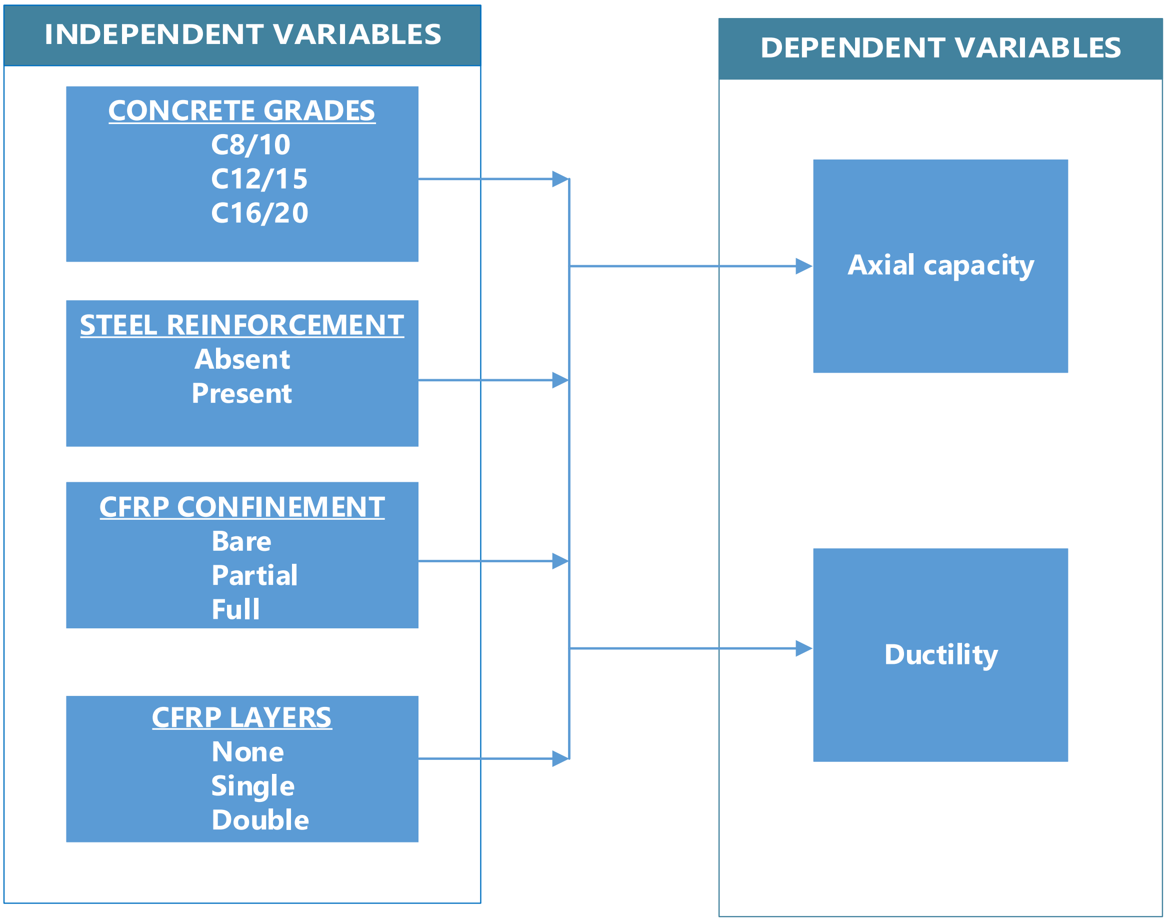


Figure 2.7: Conceptual model of the investigation.

CHAPTER 3: MATERIALS AND METHODS

3.1 Introduction

The study took an empirical based approach to generate the data necessary to help answer the research questions. Samples of non-slender concrete columns were manufactured and subjected to uniaxial compression until failure occurred.

3.2 Research Design

The plan for the research began with determining the number of samples to be manufactured. The research plan was guided by the conceptual framework diagram in Figure 2.7. The study was a comparative investigation of the effect of CFRP strengthening on columns and therefore the specimens manufactured were either control specimen or test specimen. For the following explanation of the number of samples cast, the term ‘specimen’ refers to a non-slender column with a particular treatment while the term ‘sample’ refers to one instance of the specimen with the same treatment.

Three concrete grades were to be used. Therefore, the study needed three specimens to act as control specimens and three other test specimens that resulted in a total of six specimens to be manufactured. To answer the second research question, plain concrete specimens had to be compared with reinforced concrete specimens resulting in doubling of all the concrete specimens required from six to 12; six control specimens and six test specimens. To compare partial and full CFRP confinement, the test specimens were doubled from six to 12 since six specimens had partial CFRP confinement and six specimens had full CFRP confinement. The result of this addition was a total of 18 specimens. Finally, for the last research question, the test specimens were also doubled from 12 to 24 since some had one layer of CFRP while others had two layers of CFRP resulting in a total of 30 specimens; six control specimens and 24 test specimens. Three samples were made for each specimen according to the testing procedure of BS 12390-2:2019 that requires a minimum number of three samples of any concrete property being tested. As a result, 90 samples were required in this experimental investigation. Samples that had a high deviation from the mean were recast to confirm if they were outliers. In the end, a total of 95 samples were cast.

The research plan adopted ensured that the data generated from this research would answer all the four research questions validly, objectively and economically.

3.3 Experimental Investigation

Ninety-five column samples were manufactured and tested to investigate the effects of concrete grade, steel reinforcement, degree of confinement and number of CFRP layers. Thirty unique column specimens were required to investigate the four variables stated. Three samples of each unique specimen were then manufactured to result in a total of 90 column samples. Five additional samples were manufactured and tested as a quality assurance procedure. The number of samples for each specimen was chosen in line with BS 12390-3:2019 (BSI, 2019). Comprehensive details of the specimen are discussed in the subsections that follow and summarised in Figure 3.12. These subsections have been arranged as per the specific objectives of this investigation.

3.3.1 Independent variables of the study

3.3.1.1 Concrete grades used in the study

This experimental programme used three strength grades of concrete C8/10, C12/15 and C16/20 for the specimens as per the first research question. The concrete grade was an intrinsic variable. There were 30 specimens for each grade of concrete. Three samples were made for each specimen. The concrete strength grades chosen for these tests were as shown in Table 3.1. These concrete grades were selected because of their lower characteristic strength than Concrete C20/25. Concrete C20/25 is usually the target strength for cast-in-place concrete for many construction projects in Kenya (IQSK, 2019). Any retrofits in this experiment attempted to attain the characteristic strength of concrete C20/25.

Table 3.1: Characteristic strength of grades of concrete.

Concrete Grade	f_{ck} (MPa)	$f_{ck,cube}$ (MPa)	BS Designation
Concrete C8/10	8	10	Class 10
Concrete C12/15	12	15	Class 15
Concrete C16/20	16	20	Class 20
Concrete C20/25	20	25	Class 25

3.3.1.2 Presence of Steel Reinforcement

Of the 90 specimens, 45 had no steel reinforcement while the remaining 45 had steel reinforcement. The presence or absence of steel reinforcement in the column was an intrinsic variable of the column. The criteria in the design of the specimen would assist in achieving the second specific objective of this study that attempted to quantify the effect of steel reinforcement on CFRP strengthened samples.

3.3.1.3 Degree of confinement

With regards to the various configurations of CFRP confinement, 36 specimens with partial CFRP confinement and 36 specimens with full CFRP confinement were manufactured. Eighteen specimens did not receive any application of CFRP. The degree of confinement was an extrinsic variable. The 18 specimens without CFRP treatment were used as the control for this comparative study. The choice of these configurations was as per the third specific objective, which was to compare partial and full confinement.

3.3.1.4 Number of layers

To find out the effect of the number of layers, 18 of the 36 specimens with partial CFRP confinement had one layer of the CFRP wrap, and the remaining 18 had two layers of CFRP wrap. Eighteen of 36 specimens with full confinement had one layer of CFRP wrap, and the remaining had two layers of CFRP wrap. The 18 specimens without CFRP wrap were used as the control. Comparisons were drawn from testing these samples. The number of layers was an extrinsic variable.

3.3.2 Dependent variables of the Study

A compression test machine axially loaded the specimen to failure. The failure load and deflections were recorded and were used to calculate the axial capacity and ductility of the column specimens.

3.3.2.1 Determination of the axial capacity of the columns

The axial capacity was measured as the maximum load the column could withstand before the failure occurred. The maximum load was divided by the cross-section area to determine the axial capacity. The calculation is shown in Equation 3.1.

$$Axial\ capacity = \frac{Load\ at\ failure}{Area\ of\ colum\ cross-section} \qquad \text{Equation 3.1}$$

3.3.2.2 Determination of the ductility of the columns

As the specimen was loaded, the longitudinal deflections were measured using a dial gauge. The deflections with the corresponding loads that caused those deflections were recorded. The values of the deflections were utilised in determining the ductility of the column specimen. Ductility was calculated as the final longitudinal deformation expressed as a percentage, as expressed by Equation 3.2.

$$Ductility = \frac{Deflection\ at\ failure}{Initial\ length\ of\ column} \times 100\% \qquad \text{Equation 3.2}$$

The lateral deformation at the middle position such as the hoop strain is a better measure of ductility since it is correlated to the passive confinement pressure experienced by the column being loaded (Arduini et al., 1999). However, the equipment necessary to measure the lateral deformation was not available. Therefore, there was no data for lateral or horizontal deformation that was recorded.

3.4 Specimen Dimensions

3.4.1 Non-dimensional Slenderness

Columns can either be slender or non-slender as per the Eurocode classification (CEN, 2004). Non-dimensional slenderness is a factor of the length and cross-section dimensions of the specimen. Slender columns fail by buckling while non-slender columns fail by crushing. This study investigated non-slender columns. The choice of non-slender columns necessitates the need for limiting the column height. The Euler critical load is affected by the following factors: material properties, the effective length of the columns, cross-section geometry, and end conditions. The nature of the compression test machine was such that the end conditions of the column while being loaded had pinned supports at both the top and bottom. This setup did not affect the effective length; the actual column length was the same as the effective column length.

The column was loaded concentrically to ensure that there was no eccentricity. Eccentricity leads to the development of significant flexural stresses that could affect the determination of the axial capacity of the columns. The Euler buckling load is the compressive axial force required to cause lateral instability of a vertical, weightless column (Patillo, 2018). The formula for the Euler buckling load is expressed in Equation 3.3.

$$N_{crit} = -k\frac{\pi^2 EI}{L^2} \qquad \text{Equation 3.3}$$

where N_{crit} = Euler buckling load

E = Young's modulus

I = Moment of inertia of the column cross-section

L = Column length

k =The value of k varies with the end conditions

Table 3.2: End conditions and k values of columns under compression.

Top constraint	Bottom constraint	k
Fixed	Fixed	4
Fixed	Free	0.25
Pinned	Fixed	2.046
Pinned	Pinned	1

Source: Patillo, 2018

In an ideal scenario, the column could either buckle or crush at the Euler buckling load. However, because of geometrical imperfections of the column and eccentricities in load application, slender members under compression will fail by buckling before the Euler buckling load is attained. For non-slender columns, the parameters of the column should be below the Euler buckling curve, as shown in Figure 3.1. Non-slender columns do not have the additional requirement of calculating second-order effects (CEN, 2004).

Adapted from: Osofero, 2012

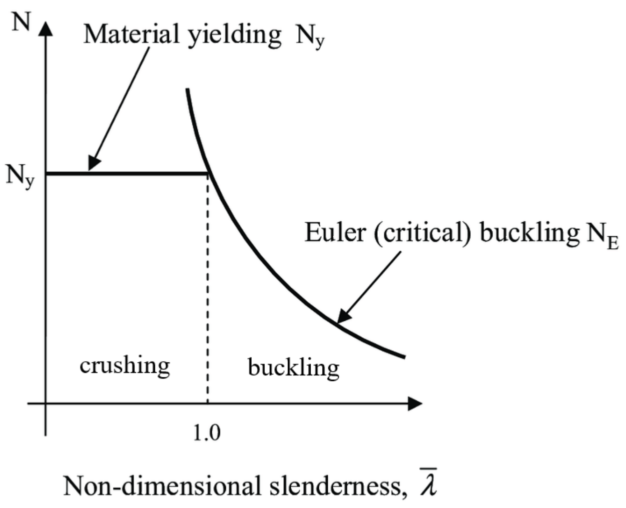


Figure 3.1: The Euler Buckling curve.

There should be no restraints from the platens of the testing machine. In practice; however, some lateral compression is introduced because of the friction generated between the steel platen and the concrete. In an ordinary testing machine, it is difficult to eliminate this friction. However, its effect can be minimised by using a specimen whose length to width ratio is greater than two so that the central position of the specimen is free from platen restraint (Neville & Brooks, 2010).

3.4.2 Dimensions of the Sample Manufactured

The specimen dimensions were based on the conditions of Euler critical load and the clearance height of the compression test machine at the Materials Testing Laboratory, University of Nairobi. The clearance height was 380mm, as shown in Figure 3.2. This study adopted 350mm as the height of the concrete column specimen. The 30mm clearance was necessary for moving the columns to the appropriate position before clamping down on the sample.

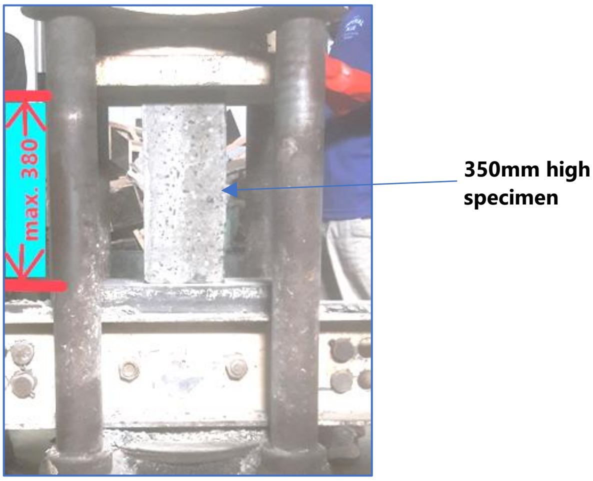


Figure 3.2: Maximum clearance of the compression machine.

The cross-sectional dimensions were 150mm x 150mm. A smaller cross-section size would pose challenges when fabricating and installing the rebar cage while a bigger cross-section would make the sample heavier and difficult to handle. Reducing the cross-section size has the effect of increasing the steel to concrete ratio of the cross-sectional area, and thus, the contribution of rebar would be higher. Bigger cross-sections would also mean higher costs. Furthermore, bigger cross-sections would lead to higher axial capacity that would exceed the maximum capacity of the compression test machine available. The dimensions chosen ensured that the specimen was non-slender as per the standard EN 1992-1-1 (CEN, 2004).

3.4.3 Slenderness Check of the Specimen

The slenderness λ, is a key component in the design of vertical elements that support compression loads (The Structural Engineer, 2013). Slenderness was calculated as shown in Equation 3.4.

$$\lambda = \frac{l_0}{i} \qquad \text{Equation 3.4}$$

where l_0 = Effective length of the column

i = Radius of gyration

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{Moment\ of\ Inertia}{Area\ of\ cross-section}}$$

where

$$I = \frac{bd^3}{12} = \frac{150mm \times (150mm)^3}{12} = 42.1875 \times 10^6 mm^4$$

$$A = 150mm \times 150mm = 2500mm^2$$

Therefore

$$\lambda = \frac{l_0}{i} = \frac{350\text{mm}}{\sqrt{\frac{42.1875 \times 10^6\text{mm}^4}{2500mm^2}}} = 2.7$$

The non-dimensional slenderness of the specimen chosen was found to be 2.7. The standard, EN 1992-1-1, gives the criterion against which a column can be determined as slender or non-slender. The slenderness limit λ_{lim} for a column, is given by Expression 5.13N of EN 1992-1-1 and is reproduced here as Equation 3.5 (CEN, 2004).

$$\lambda_{lim} = 20 \times A \times B \times C/\sqrt{n} \qquad \text{Equation 3.5}$$

where:

A $= 1/(1 + 0.2\emptyset_{ef})$ and $\emptyset_{ef}$ is the effective creep ratio. If $\emptyset_{ef}$ is not known, A can be taken as 0.7.

B $= \sqrt{1 + 2\omega}$ and ω is the mechanical reinforcement ratio. If ω is not known, B can be taken as 1.1.

C $= 1.7 - r_m$ and r_m is the first order end moments ratio. If r_m is not known, C can be taken as 0.7.

n = relative normal force and is defined as the ratio of the design ultimate axial load to the area of uncracked concrete section multiplied by the compression strength of the concrete.

Since the columns were tested to failure:

$$n = \frac{Design\ ultimate\ axial\ load}{area\ of\ cross-section \times compression\ strength\ of\ the\ concrete}$$

$$= 1$$

Therefore, $\lambda_{lim} = 20 \times A \times B \times C/\sqrt{n} = 20 \times 0.7 \times 1.1 \times 0.7/\sqrt{1} = 10.78$

So, $2.7 < 10.78 \Rightarrow \lambda < \lambda_{lim}$ $\therefore$ The column is **non-slender**.

Alternatively, the critical load from the Euler Critical load N_{crit}, was given by Equation 3.3.

$$N_{crit} = k\frac{\pi^2 EI}{L^2} \qquad \text{Equation 3.3}$$

$$N_{crit} = 1 \times \frac{\pi^2 \times 25 \times 10^3 N/mm^2 \times 42.1875 \times 10^6 mm^4}{(350mm)^2}$$

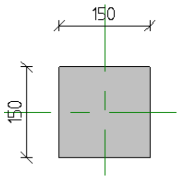
$$N_{crit} = 84974.24kN$$

The value of Young's modulus E and characteristic strength of concrete f_{ck}, was determined by interpolation of values from Table 3.1 of EN 1992-1-1 that contains the strength and deformation characteristics for concrete. The load required to cause crushing was:

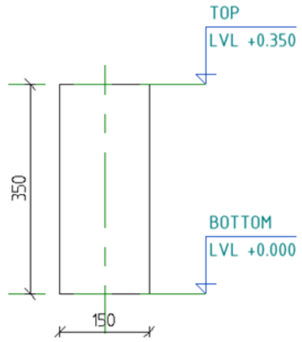
$$N_{max} = A_{conc} \times f_{ck} = (150mm \times 150mm) \times 8N/mm^2 = 180kN$$

The load required to cause the specimen to buckle N_{crit} was higher than N_{max}. The calculations confirmed that the columns were non-slender and would fail by crushing and not buckling. With this confirmation, the manufacture of the specimens with the dimensions 150mm x 150mm x 350mm high proceeded with the guarantee that failure would be as predicted; by crushing.

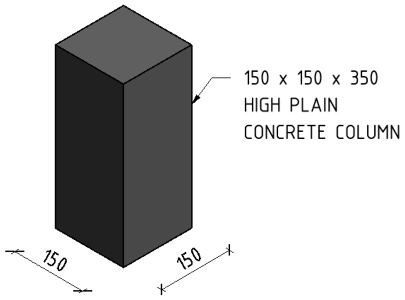
The plain concrete specimen details were as shown in Figure 3.3.



Section dimensions of the specimen.



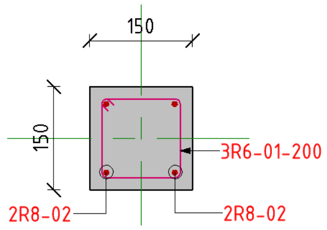
Elevation dimensions of the specimen.



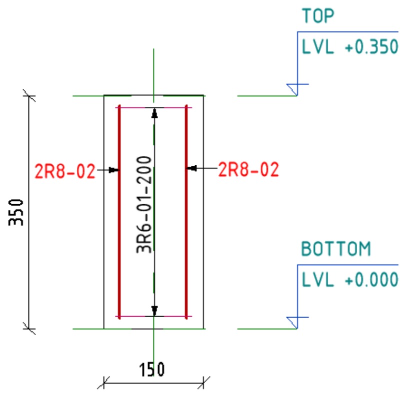
Three-dimensional representation of the specimen.

Figure 3.3: Plain concrete specimen details.

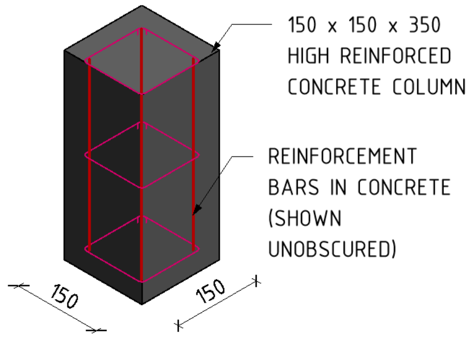
The details for the reinforced specimen were as shown in Figure 3.4.



Section dimensions of the specimen.



Elevation dimensions of the specimen.



Three-dimensional representation of the specimen.

Figure 3.4: Reinforced concrete specimen details

Steel reinforcement used was mild round steel bars with 6mm and 8mm diameters with a yield strength of 250MPa. Mild steel reinforcement with the diameters selected, 6mm and 8mm, is easier to bend and place in moulds. There was a need to ensure that the length to width ratio was greater than two to reduce the effect of platen restraint at the centre of the specimen (Neville & Brooks, 2010). The ratio of the length to the width of the specimen used was 2.33.

Nine concrete cubes were cast to determine the characteristic strength of concrete manufactured as per BS EN 12390-2:2009 (BSI, 2009). Timber moulds were fabricated with the specifications made in BS EN 12390-1:2012 (BSI, 2012). The fabrication drawings of these standard moulds are in Figure A.1. Fabrication drawings of the moulds for casting the 90 column specimens are in Figure A.2. Used engine oil was applied as on the inner side of the mould. The oil acted as the shutter release agent for the concrete specimen and the timber moulds when demoulding.

Figure 3.5: Setup for the manufacture of plain concrete specimen.

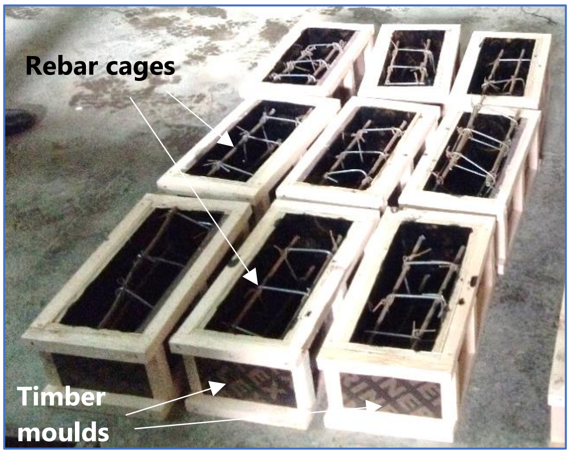


Figure 3.6: Setup for the manufacture of reinforced concrete specimen.

For the CFRP wrap, either one or two layers of CFRP were used as recommended by suppliers. Carbon fibre is an expensive material, and use of the material to more than two layers would make CFRP retrofitting uneconomical. Apart from the fiscal constraints, prior studies also underpinned this limitation of two layers (Shehata et al., 2002; Shrive et al., 2003).

3.5 CFRP Configuration

To provide partial confinement, 50mm bands were used. CFRP wrap is manufactured in standard widths which are in multiples of 50mm, that is 50mm, 100mm, 150mm and 200mm bands. Only one brand of CFRP wrap was used. The mechanical properties of the CFRP wrap that are typically available are shown in Table 3.3. The tests for the CFRP wrap were obtained from the manufacturer's datasheet (Horse Construction, 2019). The tests were reported to be in accordance with the standard ASTM D3039 / D3039M-17 (ASTM, 2017).

Table 3.3: Physical properties of CFRP wrap.

Physical Property	Value
Tensile Strength	4 840 MPa
Tensile Modulus	230 GPa
Elongation at Break	1.95%

3.6 Concrete preparation

The test used three grades of concrete: C8/10, C12/15 and C16/20, with target strengths, as indicated in Table 3.1. Batching was done by mass and manufactured in a lab electric pan concrete mixer in batches. The concrete was based on prescribed ratios (IQSK, 2019). The materials used in the manufacture of concrete were: 20mm crushed granites for the coarse aggregate, river sand for the fine aggregate and potable water. These prescribed ratios and the corresponding water-cement ratios are as indicated in Table 3.4.

Table 3.4: Prescribed ratios for various grades of concrete.

Concrete Grade	Prescribed ratios	Cement	Fine aggregate	Coarse aggregate
C8/10	1:4:8	1	4	8
C12/15	1:3:6	1	3	6
C16/20	1:2:4	1	2	4

Nine 150mm x 150mm x 150mm test cubes (three test cubes for each concrete grade) were cast and tested to establish the strength of the concrete at 28 days to determine the structural properties of the concrete used to manufacture the test specimen as per BS EN 12390-1 (BSI, 2012). Even though grading of the aggregate assists in the workability of concrete, grading was not performed as it would not affect the effectiveness of the CFRP strengthening. Grading would not affect the accuracy of the results since the comparative analysis was based on control samples that had undergone the same process of manufacture as the test samples.

3.7 Casting

The concrete was cast using the prescribed ratios described in Table 3.4. Concrete was manufactured in batches to reduce the formwork requirements. The material was sourced from one location to reduce variability. Other quality control measures such as quartering and using a riffle sample divider were employed. A riffle sample divider is a mechanical device made up of a metal box that has a series of vertical slats through which granular material is poured and randomly divided into two samples; this process was performed severally to obtain a small representative sample of the bulk material.

Source: Indiamart, 2020

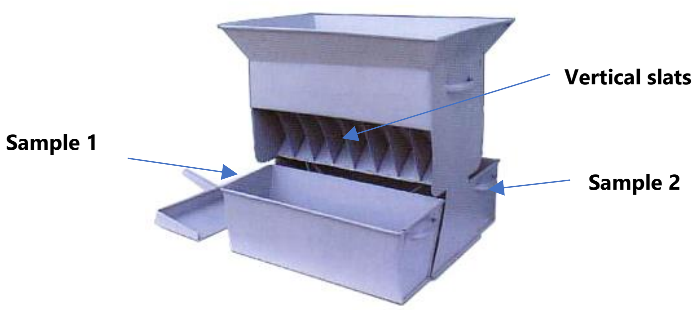


Figure 3.7: A riffle sample divider.

Figure 3.8: A quartered batch of fresh concrete.

3.8 Curing

The samples were labelled and wholly submerged in a water bath and cured for 21 days. The samples were removed from the water bath and were washed off with a pressurised hose pipe to get rid of sediment following which the specimens were air-dried for 24 hours and labelled at the top surface.

Figure 3.9: Samples after removal from the water bath.

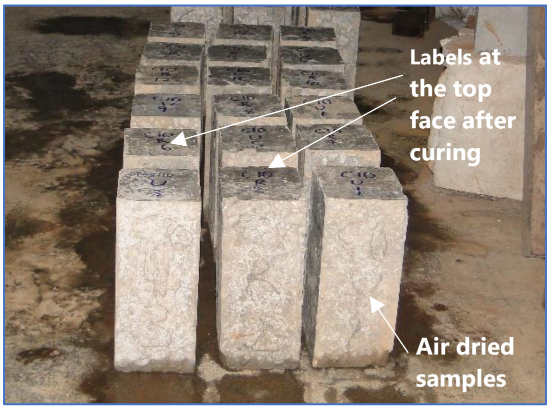


Figure 3.10: Samples after air drying for 24 hours.

Labelling was done on the top face since some of the labels on the sides would be obscured once the CFRP wrap was applied. The specimens were visually inspected for any dirt. Any mounds of dust or mud were cleaned off with a wire-brush before CFRP wrap was applied. The CFRP wrap was then cured for seven more days before testing. At the time of testing, the concrete specimen had cured for 28 days; 21 days in a water bath and seven days in the open. Figure 3.11 shows how the curing durations for both the concrete and CFRP was achieved by curing the epoxy and the concrete simultaneously.

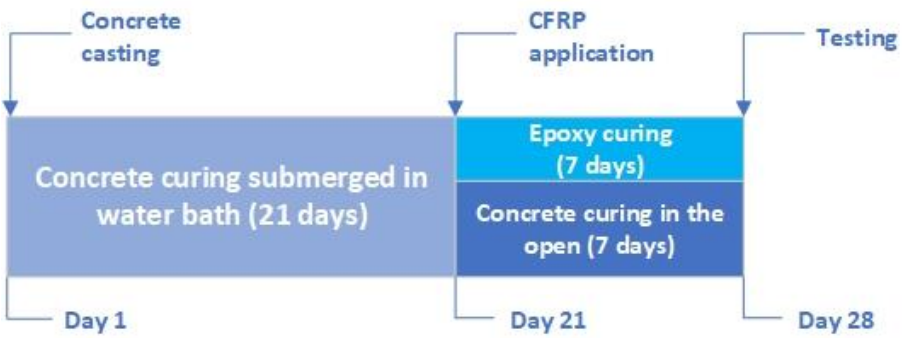


Figure 3.11: Curing process for the concrete columns and epoxy done simultaneously.

3.9 Specimen Details

Ten types of specimen were chosen to test the effect of various parameters required in CFRP strengthening. Each specimen was manufactured by the three concrete grades in Table 3.4. For this qualitative experiment, three samples were manufactured for each configuration to demonstrate a definite pattern of the effect of the four parameters. The three identical components were manufactured to produce credible data. Details of the different types of specimens are shown Figure B.1 and Figure B.2.

The specimens are listed in order of increasing confinement by rebar and CFRP wrap then by the number of CFRP layers. All the specimens with CFRP wrap had a 150mm overlap to prevent lap joint failure due to debonding (The Concrete Society, 2000).

Table 3.5 presents the specimen composition in a matrix format.

Table 3.5: Composition matrix of column specimens to manufactured.

Specimen Type	No of samples	Rebar	Partial CPRF	Full CFRP	CFRP Layers
A	3	✗	✗	✗	0
B	3	✓	✗	✗	0
C	3	✗	✓	✗	1
D	3	✓	✓	✗	1
E	3	✗	✗	✓	1
F	3	✓	✗	✓	1
G	3	✗	✓	✗	2
H	3	✓	✓	✗	2
I	3	✗	✗	✓	2
J	3	✓	✗	✓	2

This composition matrix of manufactured specimens is presented graphically in Figure 3.12.

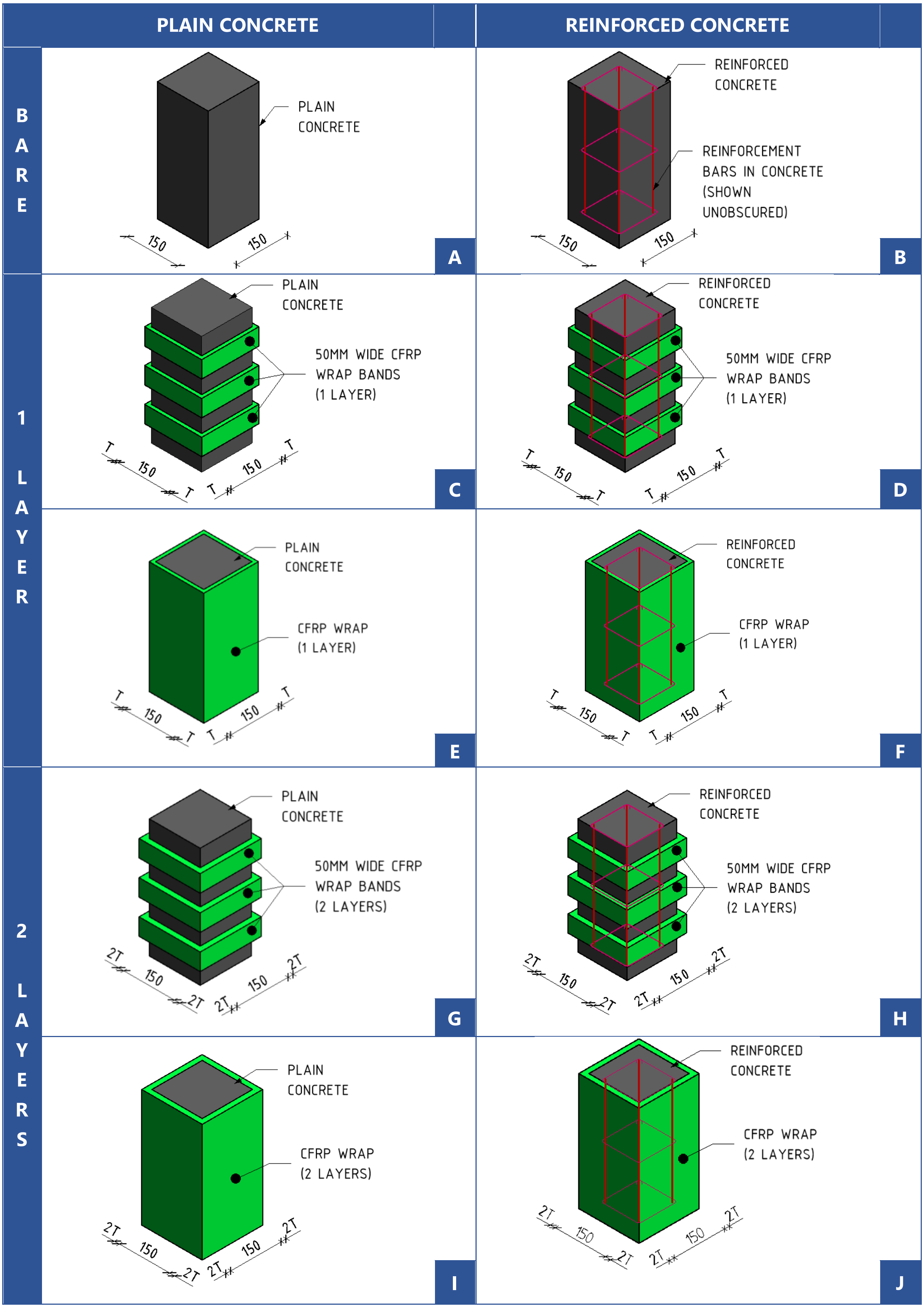


Figure 3.12: Graphical representation of manufactured specimens.

Specimen A was used to determine the axial capacity of the concrete column without steel rebar. It had no CFRP strengthening. The axial capacity was determined purely from the strength of concrete. Specimen A was the control for the other specimens that did not have rebar.

Specimen B had steel rebar and no CFRP wrap. It replicated how most square columns are constructed on site. Results from this test specimen were used to determine the axial capacity of the column when steel rebar was present. The data was instrumental in determining the contribution of steel rebar in concrete columns as per the second objective of this study. Comparing the results with Specimen A established the contribution of steel reinforcement to the axial capacity and ductility of the column without CFRP strengthening. This specimen was the control for the other specimens with both rebar and CFRP strengthening.

Specimen C had no steel rebar and had partial CFRP wrap. Specimen C was used to determine the axial capacity of the column when partially wrapped with one layer of CFRP wrap. Partial confinement was achieved by wrapping 50mm bands of CFRP at 50mm intervals between each strip. Comparing the results with that from Specimen A established the increase in axial capacity and ductility of the column due to partial confinement of one layer of CFRP wrap. This method was typically proposed as a cost-cutting measure by suppliers (Horse Construction, 2019).

Specimen D had steel rebar present and 50mm CFRP bands in one layer to offer partial confinement. It was used to determine the increase in axial capacity and ductility of Specimen B when CFRP strengthening was applied in 50mm bands. Partial confinement was achieved by using bands as opposed to completely covering the column. Testing Specimen D was justified by claims made by some CFRP manufacturers that partial confinement was an effective method of rehabilitating columns in austerity. Specimen B mirrors the construction of columns on site. Comparison of test results with Specimen B determined the individual strength contribution of the CFRP bands. Comparison of the test results with Specimen C determined the individual contribution of the CFRP rebar. Comparison of Specimen D with Specimen A would determine the combined effect of both CFRP and rebar on the columns.

Specimen E had no steel rebar but had 50mm full CFRP wrap in one layer. The test data was used to determine the axial capacity of the column when fully wrapped with one layer of CFRP wrap. Comparing the results with that from Specimen A established the increase in axial capacity and ductility of the column due to full confinement of CFRP wrap. Care was taken to ensure that there was a band at the middle section of all the specimens with partial CFRP confinement. It is as the middle where the effect of platen restraint is least (Neville & Brooks, 2010).

Specimen F had steel rebar present and one full layer of CFRP wrap. It was used to determine the increase in axial capacity and ductility of Specimen B. Specimen F mirrored prescribed methods of rehabilitating columns in engineering practice (The Concrete Society, 2000). The comparison of test results from Specimen B and Specimen E determined the individual strength contribution of the CFRP wrap and the individual contribution of the rebar, respectively. When compared with Specimen A, the combined effect of rebar and CFRP wrap was determined. This specimen facilitated the realisation of the second objective, which was to find out the individual contribution of steel reinforcement and CFRP strengthening.

Specimen G had no steel rebar and had two layers of 50mm CFRP bands offering partial confinement. Data from Specimens A, E and G, was used to determine the effect of an additional CFRP layer on plain concrete.

Specimen H had steel rebar and two layers of 50mm CFRP bands. Data from conducting tests on this specimen was used to find out the effect of an additional CFRP layer on the reinforced concrete specimen when compared against Specimen F. Comparison of the data with Specimen G would determine the effect of rebar on axial capacity and ductility of strengthened columns.

Specimen I had no steel rebar but had two layers of CFRP offering full confinement. Comparing data of this specimen against Specimens A and E were used to determine the effect of additional layers of CFRP wrap. This specimen facilitated the realisation of the fourth objective.

Specimen J had steel rebar and two layers of CFRP offering full confinement. This specimen was used to determine the effect of an additional CFRP layer acting in combination with steel reinforcement when compared with Specimen F. Comparing the data of this specimen with Specimen I would calculate the contribution of rebar to the axial capacity and ductility of CFRP strengthened columns.

3.10 CFRP wrap preparation

The CFRP wrap was supplied in 100-metre rolls of 200mm width with the epoxy adhesive set, as shown in Figure 3.13.

Figure 3.13: Carbon fibre wrap and epoxy adhesive.

A pair of ordinary scissors was used to cut the CFRP to the required sizes to attain the configurations illustrated in Figure 3.12. CFRP was relatively easy to cut (Abadi et al., 2019). For the full CFRP confinement, the cut width sizes made were of 200mm and 150mm widths.

Combining the two widths ensured the 350mm high concrete column specimens manufactured were entirely covered. The cut lengths were 750mm for one-layer confinement and 1350mm for the two-layer confinement. For partial confinement, the cut width sizes made were 50mm widths. The cut lengths were similar to full CFRP confinement, that is, 750mm to achieve one layer of CFRP, and 1350 mm to achieve two layers of CFRP round the manufactured column specimens. In all the samples, there was an overlap of 150mm to prevent a joint failure at the lap.

Figure 3.14: A pair of scissors was used to cut the CFRP to the required sizes.

3.11 Epoxy

3.11.1 Description of Epoxy

The epoxy typically comes in a set of two viscous fluids. The epoxy is an organic compound. The smell of one is like cobbler’s glue while the other has the distinct smell of ammonia. The two were mixed in the ratio of 2:1. The two materials react exothermically releasing heat becoming more viscous and then eventually hardening. This epoxy is the polymer that laminated the carbon fibres and caused the wrap to bind to the column surface. This binding allows the carbon fibre and the column to act together.

3.11.2 Mixing

The epoxy set was weighed and mixed in the ratio of 2:1 by mass. The epoxy was weighed using a mass balance, as shown in Figure 3.15. Each batch made was 3kg. Mixing of the epoxy set was done for five minutes until the mix attained a uniform colour. A smooth wooden rod was used for stirring to prevent entraining air while mixing. It was essential to ensure that no bubbles were introduced in the mixture. The setting time once the epoxy set had been mixed and prepared was one hour. Safety measures such as the use of gloves to prevent direct contact with the epoxy were implemented. This mixing procedure was as recommended by the supplier.

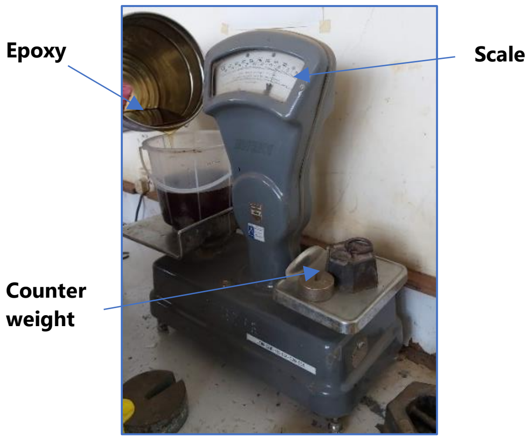


Figure 3.15: Weighing the epoxy adhesive on a mass balance.

Figure 3.16: Mixing of the epoxy set with the ratio of 2:1.

3.12 CFRP Application

The prepared epoxy set was used to externally bond CFRP to the column in the various configurations, as indicated in Figure 3.12. The carbon fibre wraps were installed using the dry application process with an epoxy-based impregnating resin. The CFRP strengthened specimens were left undisturbed for seven days as the epoxy adhesive cured and gained strength.

3.13 Prepared specimen

The specimen without CFRP strengthening is shown in Figure 3.17. Specimen with partial CFRP confinement is shown in Figure 3.18, while full CFRP confinement is shown in Figure 3.19.

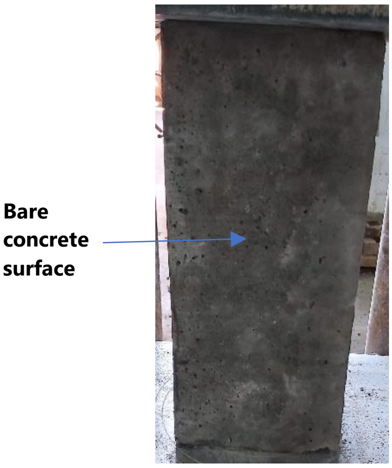


Figure 3.17: Specimen without CFRP strengthening.

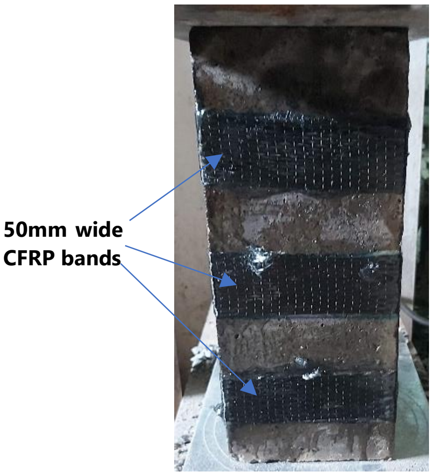


Figure 3.18: Specimen with partial CFRP strengthening.

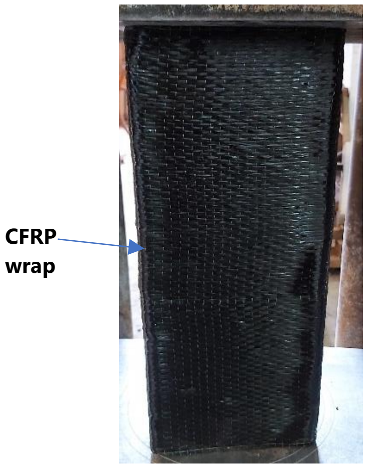


Figure 3.19: Specimen with full CFRP strengthening.

3.14 Testing

Each specimen was centred on the loading platform, and a compressive force was applied by the compression test machine. The axial loading was applied by employing a hydraulic actuator with a maximum capacity of 2,000kN. The loading rate was manually controlled. Calibration of the compression test machine had been done by the Kenya Bureau of Standards (KEBS) 11 months before testing in September 2018. Testing was done from August 2019. Longitudinal deflections were measured using a dial gauge. Each deflection was measured at a corresponding load.

The compression test machine is powered by electricity. A hydraulic actuator pumps hydraulic fluid into an upstroke hydraulic piston that pushes the specimen upwards against a permanently fixed clamp. This upward movement generates a compressive force in the sample being tested. Valves on the control board control the loading rate. Calibrated scales show the force exerted.

The dial gauge in contact with the bottom platen of the testing machine measures the vertical displacement of the hydraulic piston. Figure 3.20 and Figure 3.21 shows the setup of the testing equipment.

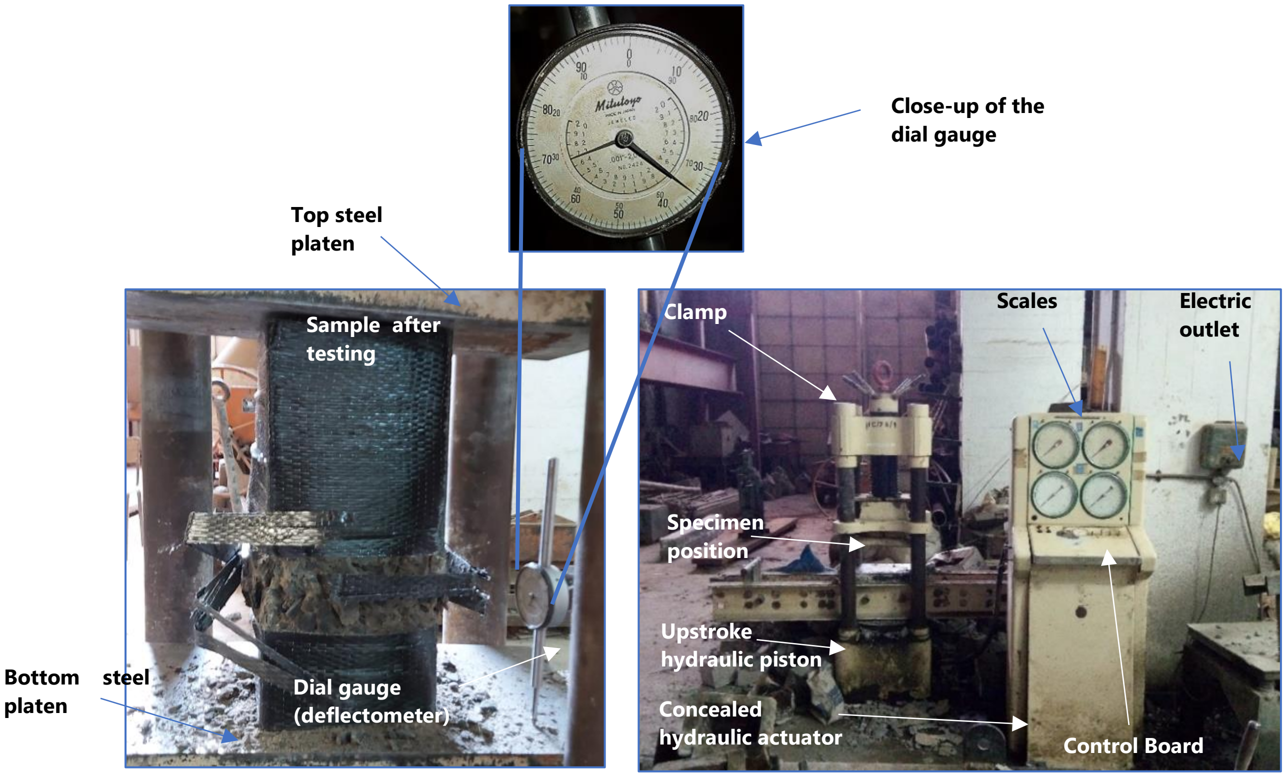


Figure 3.20: Dial gauge in position to measure the deflection.

Figure 3.21: Set up of the compression test machine.

Instrumentation such as linear variable differential transformers and displacement transducers to measure deflection would have been ideal; however, these sensors were unavailable and thus a procedure to get the required values was improvised for this study. The loads and longitudinal deflections values were acquired by observers reading off the scales of the compression test machine and the dial gauge. One person read the axial load from the scale on the compression test machine while another person read the corresponding value of the deflection from the dial gauge. A third individual tabled the values on a pre-printed sheet of paper. The loading rate was not constant but was manually controlled as per the procedure described in BS EN 12390-3: 2019 (BSI, 2019). The speed of loading was adjusted accordingly to allow the convenient reading of the values from the dial gauges of the compression test machine and the dial gauge measuring deflection. From the recorded loads and deflections, the respective stresses and strains were calculated. The stress-strain curves were then plotted. The curves were presented in Appendix G. The experimental stress-strain curves were terminated at the point where:

1. The CFRP wrap rupture occurred in specimens with full CFRP confinement.
2. The concrete core failed in specimens without CFRP wrap and those with partial CFRP confinement.

The results obtained were then subjected to statistical tests to find out if the difference of the data between and among groups was statistically significant at the 95% level of confidence.

3.15 Test Statistics

In test statistics, there were controls which received no CFRP strengthening. Controls played a huge role in this experimental programme. Controls assisted in dividing up the changes we observed due to CFRP strengthening and changes that are due to other factors such as variability in the concrete mix. The test statistics were calculated from sample data and was used for hypothesis testing. The level of confidence for the experimental investigation was chosen as 95% which implies that $c = 0.95$ and $\alpha = 1 - 0.95 = 0.05$. The α-value of 0.05 is commonly used in other scientific studies and was adopted in this experimental programme as well (Brangard, 2018b). For this study, since the hypothesis was testing the equality of two groups then two-tailed tests were performed.

3.15.1 T-test

T-tests are good for testing the difference between two groups (Brangard, 2018b). A t-test can be employed to test the difference between plain concrete specimens and reinforced concrete specimens. It can also be used to test the difference between partial and full CFRP confinement and finally between the one layer and two layers of CFRP wrap. For the first research question, the t-test could be applied thrice between Concrete C8/10 and Concrete C12/15, between Concrete C8/10 and C16/20 and finally between Concrete C12/15 and Concrete C16/20. However, multiple t-tests are not ideal because of they have the tendency of increasing the type I error. A type I error occurs when a true hypothesis is rejected. Doing the test sequentially would result in introduction of more errors in the analysis.

T-tests were performed between only two groups. In this study, the t-test was done only to test the second null hypothesis which compares samples with rebar present and those without. Specifically, a two-sample t-test was conducted. The two-sample t-test is also known as the independent t-test or unpaired t-test (Brangard, 2018c). The remaining three hypotheses were tested using ANOVA.

3.15.2 ANOVA

ANOVA tells us if the means of the various groups are different. ANOVA is suitable when testing among three or more groups. Post hoc testing after the ANOVA is performed can further reveal the difference between two difference groups. Post hoc testing is less likely to introduce Type I errors compared to performing multiple t-tests. ANOVA was used to test the three hypotheses relating to concrete grade, degree of confinement and number of layers. Testing these three hypotheses involved testing three sets of data as highlighted in the conceptual framework in Figure 2.7. The Gabriel method in the SPSS statistical analysis package was preferred for post hoc testing because of the difference in sample sizes (Grande, 2015).

The results obtained were analysed, presented and discussed in the following chapter.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Stress-strain curves

The plotted charts for the stress-strain curves for all the specimens are presented in Appendix G. For quality control and to verify the experimental data, some specimens were manufactured again and retested. An additional five samples were retested hence the total number of specimens cast increased from 90 as proposed earlier, to 95. The results for all the 95 specimens are presented in Table C.1 for axial capacity and Table D.1 for ductility. The results were recorded and the analysis was performed using a software package called Microsoft Office Excel Version 2019. Microsoft Office Excel 2019 is a data visualisation and analysis tool (Microsoft Excel, 2019). The stresses were plotted with their corresponding strains to come up with stress-strain curves for the samples. The stress-strain graphs for all the samples tested in this experimental study are in Appendix G. The typical stress-strain graphs for a specimen with three samples is presented in Figure 4.1.

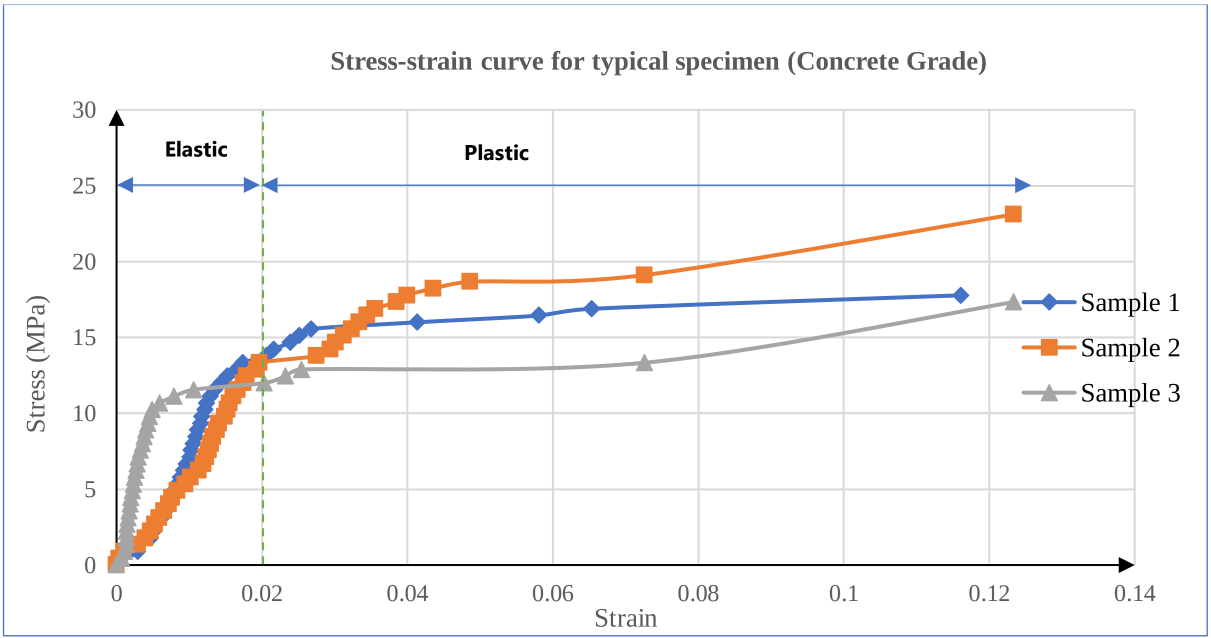


Figure 4.1: Typical stress-strain graphs three samples of the same CFRP configuration.

Figure 4.1 is similar to Figure 3.6 of EN 1992-1-1, reproduced here as Figure 2.2. The graph illustrates the stress-strain relationship of confined concrete.

The curves can be described in two main parts; the elastic phase and the plastic phase. The elastic phase occurs before cracks developed, and the stress varies almost linearly with the strain up to the point where cracks start to develop. The point where hairline cracks appeared indicate the beginning of the plastic phase. Deflections happened rapidly. In the plastic phase, the concrete had yielded, and failure occurred immediately after large visible cracks develop on the concrete specimens. By inspection of the graphs in Appendix G, the area under the stress-strain curves was significantly increased by CFRP strengthening. The area under the stress-strain curve is a measure of energy absorption (Masia et al., 2004). This energy absorption is important and is the reason why CFRP strengthening is used for seismic retrofitting (Bank, 2006).

4.2 Concrete Mix Design

The mix design was not performed as it was one of the assumptions of the investigation. Table 4.1 shows the target strengths against the actual strengths of the cubes tested.

Table 4.1: Compressive stresses of test cubes for the grades of concrete manufactured.

Designation	Prescribed ratios	Water: cement ratio	Target cube Strength	Actual cube Strength	Percentage difference
C8/10	1:4:8	0.7	10.0MPa	10.2MPa	+2%
C12/15	1:3:6	0.65	15.0MPa	14.7MPa	-2%
C16/20	1:2:4	0.6	20.0MPa	15.6MPa	-22%

This study found the prescribed ratios used in the Building Construction Handbook 2018/2019 (2019) did not attain the required design strength for Concrete C16/20 when the compressive test was done as per BS EN 12390-2:2009 (BSI, 2009). The failure of this ratio to attain the concrete design strength may be one reason that buildings may be considered as unsafe as had been revealed by building audits in Nairobi City County (NCA, 2015; Omullo, 2018). The failure of the prescribed ratios to attain the design strengths in this particular study is attributed to the fact that a pozzolanic cement IV/B was used. The cement was manufactured as per standard KS EAS 18-1:2017 (KEBS, 2017).

Pozzolanic types of cement attain full strength slowly and may attain strength after 28 days (Okumu et al., 2017). The use of prescribed ratios may only be applicable when using Ordinary Portland cement CEM I (Joel & Mbapuun, 2016). The tests in this study were done after 28 days of curing concrete. This finding implies that engineers should pay particular attention to the mix design, if not, only ordinary Portland cement should be used when using the prescribed ratios. The finding that the prescribed ratios would not meet the target design strength was anticipated based on a study in Nigeria that found the ratios did not attain the required strength (Adewole et al., 2015).

Nonetheless, several studies have proven that the prescribed ratios work. A thorough analysis of studies done shows that the failure to attain the design strengths might have been because of the high water to cement ratio used.

This anticipation that concrete manufactured using the prescribed ratios would fail necessitated the requirement to do a comparative test, not based on the target strengths, but on control specimens. This finding justifies the need for having Specimen A and Specimen B, which were used as the control specimens in this research. Therefore, the failure to attain the desired strength does not affect the validity of this experimental investigation.

4.3 Mechanical Properties of the Columns

The mechanical properties of the columns are the physical properties that the columns exhibited after the testing. The physical properties of the column analysed were the axial capacity and ductility. The data from the testing process was recorded and analysed. Statistical analysis and tests were done using IBM SPSS Statistics Software (IBM Corp., 2021). The results were presented in the form of bar graphs using Microsoft Excel (Microsoft Excel, 2019). The results were discussed in subsections of axial capacity and ductility. In the respective subsections, the first bar graphs show the average value of the axial capacity and ductility for each concrete grade from Specimen A to Specimen J. After that, Specimens C, D, E, F, G, H, I and J were compared with the control specimens A and B to determine the effect of CFRP strengthening. The effect of CFRP strengthening was quantified as a percentage change. The comparisons were then reorganised to find out the impact of the four variables under investigation. The analysis is first organised by concrete grade, then by rebar, followed by the degree of confinement and finally by the number of layers. This reorganisation of the results facilitated the identification and visualisation of trends.

The average values of the various configurations were presented in radar charts and treemaps to find out the impact of the variables on axial capacity and ductility. The calculations for the determination of these effects are in Appendix D and Appendix E. The typical failure modes are then presented before the final discussion, and a summary of findings is made.

4.3.1 Axial Capacity of the Specimens

The average axial capacity of the columns tested is presented in Figure 4.2 for Concrete C8/10.

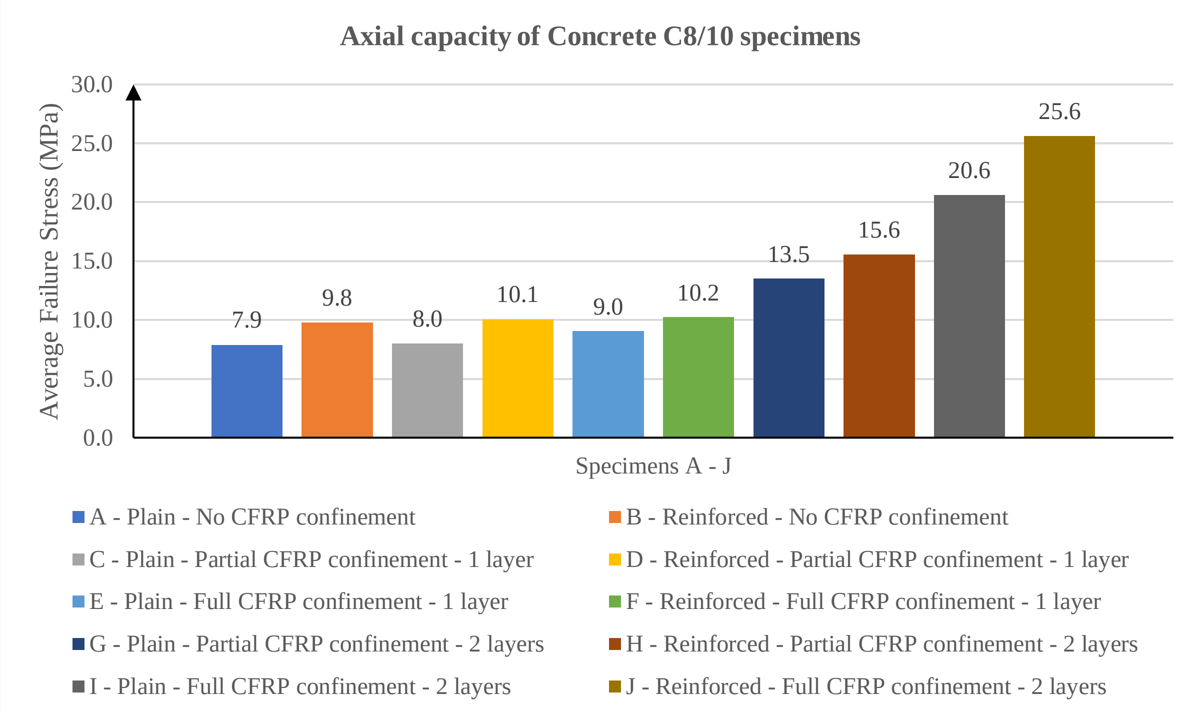


Figure 4.2: Average axial capacity of Concrete C8/10 specimens.

There were approximately three samples for each specimen. Figure 4.2 shows the results from testing 32 specimens. The average axial capacity of specimens without CFRP strengthening (Specimens A and B) was 8.85MPa while that of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) was 14.08MPa. CFRP strengthening caused a 59.6% change in the axial capacity of the columns. By inspection of the chart in Figure 4.2, it is evident that the axial capacity increased with increasing confinement. The target strength of 25MPa was only achieved when two CFRP layers were used on the reinforced column specimens. This finding confirms that CFRP may be used as a viable solution in retrofitting existing columns safely.

Results from testing the Concrete C12/15 and Concrete C16/20 are presented in Figure 4.3 and Figure 4.4, respectively. Further analysis incorporating those results from the other grades of concrete is made in order to draw comparisons with the data presented in Figure 4.2.

Figure 4.3 shows the average axial failure stress of the specimen cast for Concrete C12/15.

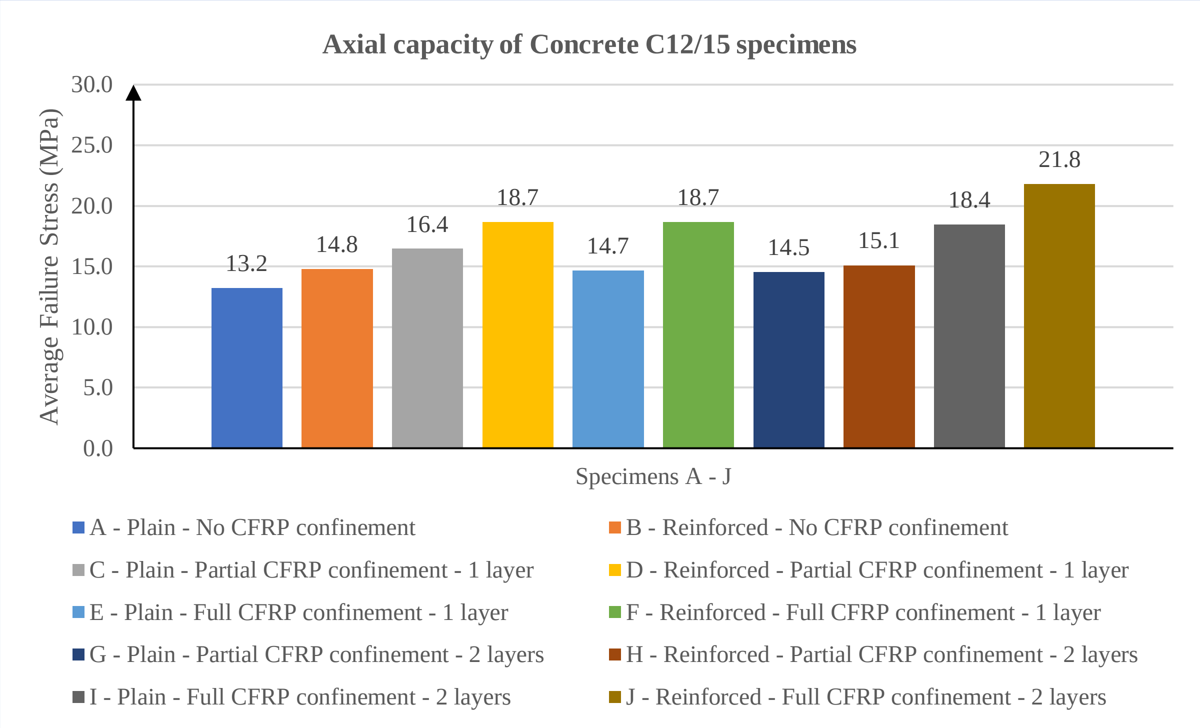


Figure 4.3: Average axial capacity of Concrete C12/15 specimens.

Figure 4.3 is a bar graph of the average values of 32 specimens. The average axial capacity of specimens without CFRP strengthening (Specimens A and B) was 14MPa. In contrast, that of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) was 17.3MPa. On average, CFRP confinement caused a positive change in the axial capacity of 23.5%. There was less effect of CFRP confinement on Concrete C12/15 as compared to Concrete C8/10. By observation, there was a slight change in the axial capacity as the level of confinement increased from Specimen A towards Specimen J. Results in this chart seem to indicate that using two layers of full CFRP confinement had a higher effect on the axial capacity of the concrete columns. Further inspection reveals reinforced concrete Specimens B, D, F, H and J had higher axial capacity than plain concrete Specimens A, C, E, G and I. A more in-depth analysis incorporating data from Concrete C16/20 specimens would reveal trends identified here only by mere observation.

Results for the axial capacity of Concrete C16/20 specimens showed a slight change in the axial capacity of the concrete columns, as shown in Figure 4.4.

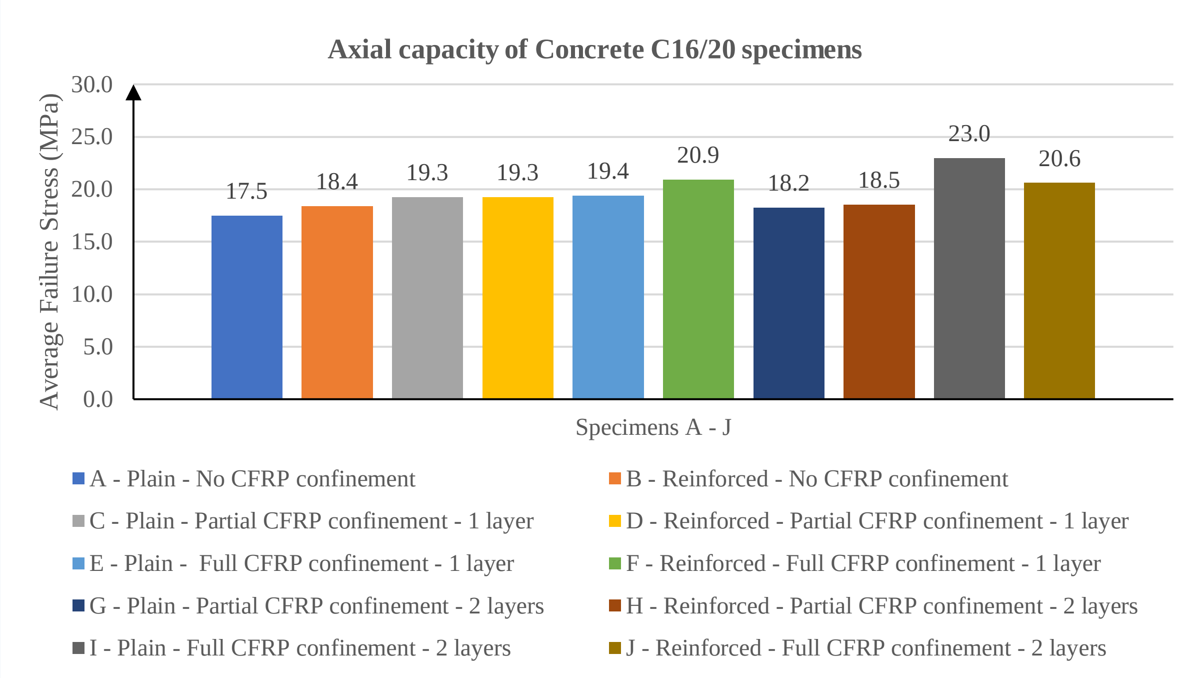


Figure 4.4: Average axial capacity of Concrete C16/20 specimens.

The average axial capacity of specimens without CFRP strengthening (Specimens A and B) was 17.95MPa while that of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) was 19.9MPa. CFRP strengthening caused an 11% increase in strength. Concrete C16/20 did not seem to have been affected quite notably by CFRP strengthening as much as Concrete C8/10 and Concrete C12/15, which experienced 59.6% and 23.4% increases in axial capacity, respectively. This trend follows literature in that stronger concretes do not dilate as much as weaker concretes (Shehata et al., 2002). This is because the passive confinement pressure is not generated as much and thus, CFRP strengthening has minimal effect on the axial capacity of the stronger concrete grade. By observation, reinforced concrete specimens B, D, F and H had marginally higher axial capacity than plain concrete specimens A, C, E and G. This was not repeated between Specimens I and J. Rebar seemed to have a reducing effect on the axial capacity of the non-slender concrete columns where two layers of full CFRP confinement was used. This finding was analysed further in the following subsections to determine whether it was a valid trend or a sample error.

Effect of Dependent Variables on Axial Capacity of CFRP Strengthened Columns

4.3.1.1 Effect of Concrete Grade on Axial Capacity of CFRP Strengthened Columns

Using the sample data on axial capacity in Table C.1, the following means and standard deviations were calculated and presented in Table 4.2.

Table 4.2: Descriptive statistics of axial capacity of samples of different concrete grades.

Concrete Grade	Mean	Std. Deviation	N
C8/10	12.7634	5.90594	32
C12/15	16.3053	2.99852	32
C16/20	19.0100	2.92129	31
Total	**15.9948**	**4.87621**	**95**

ANOVA test was carried out at the 95% confidence interval and a level of significance of 5%. The results are as indicated in Table 4.3. The analysis of the sample data shows that the mean difference between the samples of the different concrete grades is significant with a 95% level of confidence.

Table 4.3: ANOVA table of axial capacity between groups of different concrete grades.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	619.054	2	309.527	17.621	0.000
Within Groups	1616.025	92	17.565		
Total	**2235.079**	**94**			

The ANOVA test, $F(2,92) = 17.621, p < 0.001$, shows that treatments between the groups of different grades were significantly different at the 95% confidence interval. Since the ANOVA test shows the axial capacities of the samples of the three concrete grades were significantly different, post hoc testing was done to find out which group was different from the others. The group sizes were different; Concrete C8/10 had 32 samples; Concrete C12/15 had 32 samples and Concrete C16/20 had 31 samples. The Gabriel method was used for post hoc testing for multiple comparisons within and between the groups because of the different sample sizes. The results of the post hoc tests are as shown in Table 4.4.

Table 4.4: Post hoc testing of axial capacity of groups of different concrete grades.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
C8/10	C12/15	-3.5419*	0.53700	0.000	-4.8567	-2.2271
	C16/20	-6.2466*	0.54131	0.000	-7.5719	-4.9212
C12/15	C8/10	3.5419*	0.53700	0.000	2.2271	4.8567
	C16/20	-2.7047*	0.54131	0.000	-4.0300	-1.3794
C16/20	C8/10	6.2466*	0.54131	0.000	4.9212	7.5719
	C12/15	2.7047*	0.54131	0.000	1.3794	4.0300
* The mean difference is significant at the 0.05 level.						

If a group has a significance value p, of less than 0.05 then the group difference is statistically significant. The post hoc testing shows that each group is significantly different from the other two groups. Since the difference between the groups was significant the means between the groups could then be compared.

4.3.1.1.1 Findings of Statistical Significance on Concrete Grade on Axial Capacity

There was a significant difference in axial capacity among the three concrete grades $F(2,92) = 17.621, p < 0.001$. Post hoc testing revealed significant differences between Concrete C8/10 $(M = 12.76, SD = 5.91)$ which had the least axial capacity, Concrete C12/15 $(M = 16.31, SD = 3.0)$ and C16/20 $(M = 19.01, SD = 2.92)$ which had the highest axial capacity. A further comparison of the test samples was then made with control samples to find out the effect of concrete grade on the axial capacity of the strengthened columns.

4.3.1.1.2 Comparison of Concrete Grades on Axial Capacity

Following the finding that there is a significant difference among the concrete grades, the results presented in Section 4.3.1 were compared against the control specimens and organised by concrete grade to come up with the bar graph in Figure 4.5.

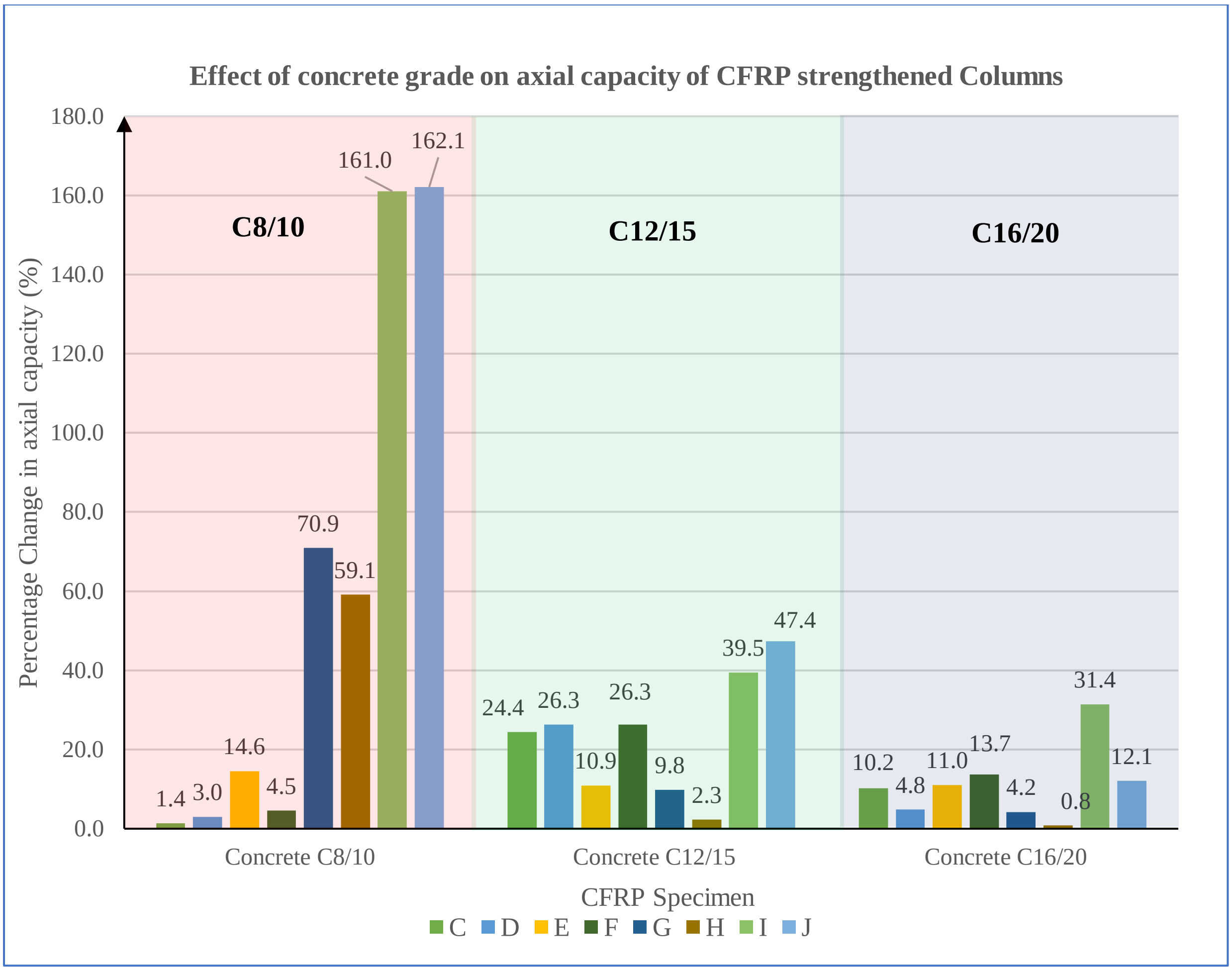


Figure 4.5: Change in axial capacity against specimens of different concrete grades.

The illustration and descriptions of the specimen in Figure 4.5 are as indicated in Figure 3.12. Specimens D, F, H and J were compared against Specimen B to determine the percentage change due to CFRP strengthening while Specimens C, E, G and I were compared against Specimen A. The results prove that the gain in axial capacity decreases with an increase in the strength of concrete as summarised in Figure 4.6.

4.3.1.1.3 Summary of the Effect of Concrete Grade on Axial Capacity of CFRP strengthened columns

Figure 4.6 shows a bar graph that illustrates the change in means of the axial capacity of the three concrete grades.

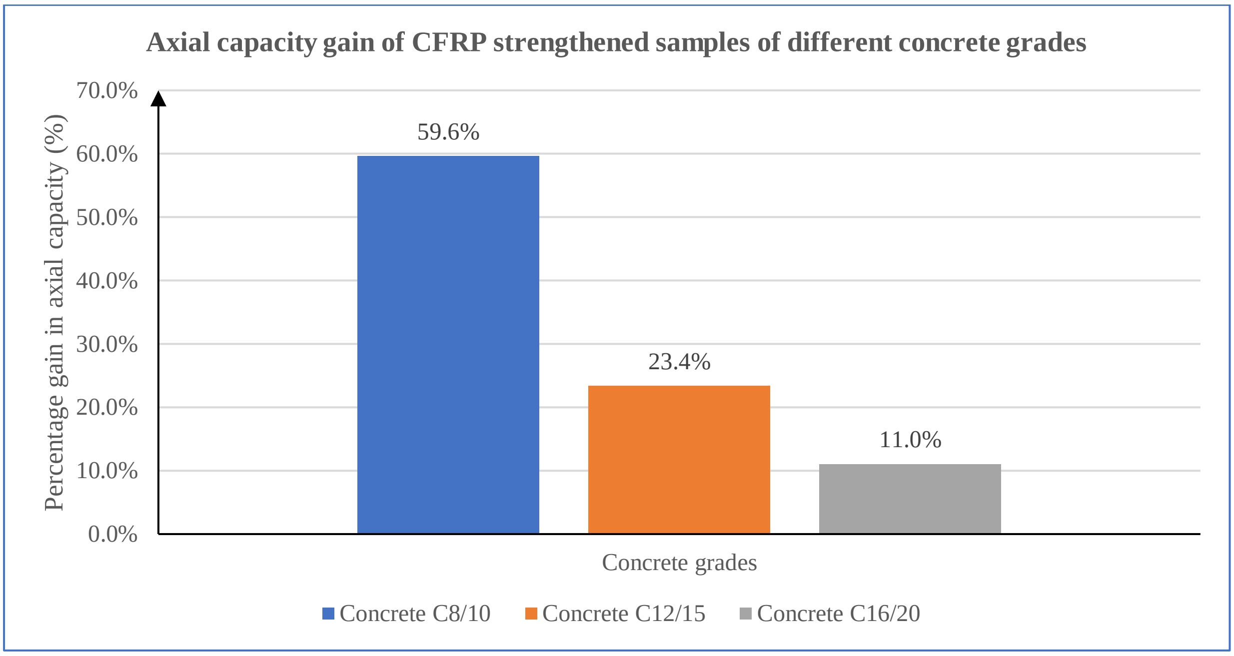


Figure 4.6: Summary of the effect of concrete grade on the axial capacity of CFRP strengthened samples.

The chart shows that with all the other variables not taken into consideration, Concrete C8/10 experienced the highest effect when CFRP wrap was installed. The experimental investigation found that concrete grade affected the gain in axial capacity. This analysis, therefore, confirms the findings by Shehata et al. (2002) that weaker grades of concrete develop the most tremendous change in axial capacity when strengthened by CFRP. The higher change experienced in Concrete C8/10 is because when the weak concrete is subjected to axial compression, it dilates more and results in higher confinement pressures and thus higher increments in axial capacity in the weaker concrete grades.

4.3.1.2 Effect of Rebar on Axial Capacity of CFRP Strengthened Columns

The means and standard deviation for ductility of groups with rebar present and with rebar absent were calculated and presented in Table 4.5.

Table 4.5: Descriptive statistics of axial capacity of samples with rebar absent or present.

Rebar	Mean	Std. Deviation	N
Absent	15.0273	4.84211	48
Present	16.9830	4.76009	47

A Levene's test for equality of variances was performed for the t-test for equality of means and is indicated in Table 4.6.

Table 4.6: T-test for axial capacity of groups with rebar absent and present.

Levene's test for equality of variances	F	Sig. (p)	t	df	Sig. (2-tld)	Mean Difference	Std. Error Difference	95% Confidence Interval of the Difference	
								Lower	Upper
Equal variances assumed	0.077	0.782	-1.985	93	0.05	-1.9556	0.9853	-3.912	0.0010
Equal variances not assumed			-1.985	92.9	0.05	-1.9556	0.9851	-3.912	0.0006

A t-test was conducted to find out if the effect of rebar was significant. The number of samples was 95 and the critical value of the t-test was calculated as 1.986. A Microsoft Excel workbook template was used to perform this calculation (Daniel & Kostic, 2019). Levene's test checks whether variances are equal. If Levene's test is not significant, then equal variances are assumed. In this case, the f-value is 0.077 and the p-value is 0.782. The p-value is greater than 0.05 and therefore the Levene's test is significant and therefore equal variances are assumed. The t-value of -1.985 does not exceed the critical value of 1.986 and the significance value is 0.782 which is greater than the significance value p, of 0.05 and therefore the difference between the two groups is not statistically significant. Also, the 95% confidence interval for the difference between sample means, shows there is a lower bound value of -0.3.912 and an upper bound value of 0.0010 and therefore crosses zero; this means that the difference between the two groups is not statistically significant. This finding was contrary to expectation since it was expected that the steel reinforcement would cause a statistically significant increase in axial capacity (Sayed & Diab, 2019). Further analysis was therefore performed to determine whether the other various components of the study affected the effect of CFRP strengthening.

4.3.1.2.1 Individual Effect of Rebar on Axial Capacity of Columns without CFRP Strengthening

The three grades of plain concrete specimens were compared with the reinforced concrete specimens. The effect of rebar of the three grades of concrete was calculated, as shown by Equation 4.1.

$$Percentage\ gain\ in\ strength \quad = \frac{Specimen\ B - Specimen\ A}{Specimen\ A} \times 100\% \qquad \text{Equation 4.1}$$

Figure 4.7 highlights this difference which shows the trend that the weaker the concrete is, the higher the effect of steel rebar on the axial capacity of the column.

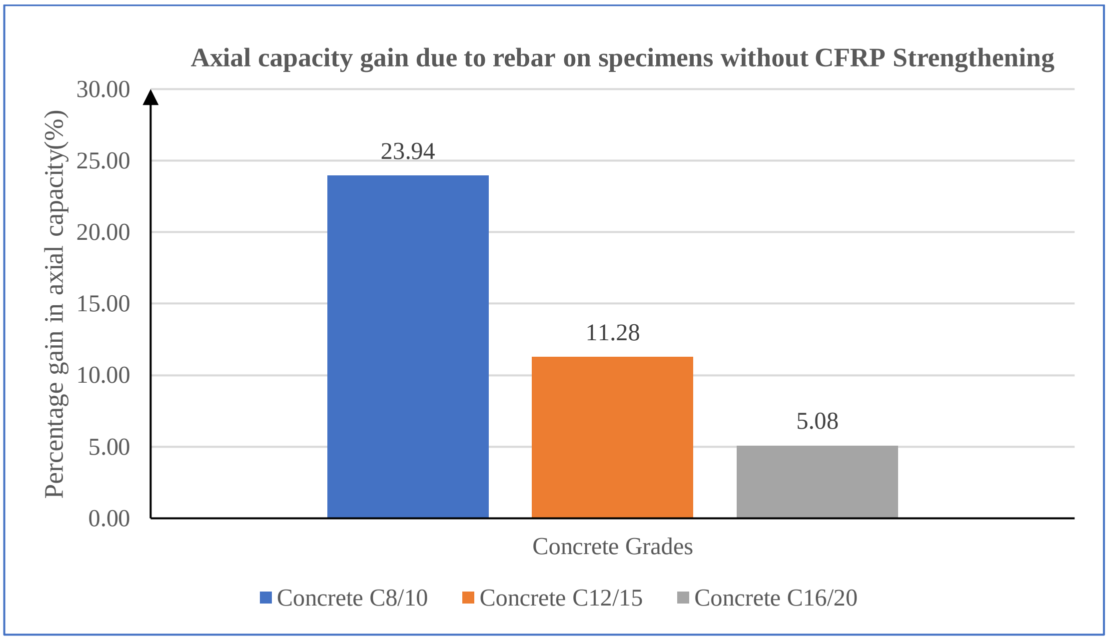


Figure 4.7: Change in axial capacity due to rebar on specimens without CFRP strengthening.

The investigation conducted indicated that specimens with steel reinforcement had higher failure stresses on average than specimens without steel reinforcement of the concrete grades used in this study. This difference may be attributed to the higher yield stresses of steel compared to that of concrete. In addition to that, the steel reinforcement offered some confinement of the concrete core resulting in the higher yield stress as had been found by Rahai et al. (2008). This finding also ascertains Clause 3.1.9 (2) of the Eurocode standard EN 1992 in the design of concrete columns that states that confinement can be generated by closely spaced links or cross-ties, which reach the plastic condition due to lateral extension of the concrete (CEN, 2004). Following this realisation, the study investigated the effect of rebar in CFRP strengthened samples in line with the second objective, which was to find out the effect of rebar on CFRP strengthening in non-slender square concrete columns.

4.3.1.2.2 Individual Effect of Rebar on Axial Capacity of CFRP Strengthened Columns

To determine the effect of rebar contribution on the axial capacity, specimens with the same characteristics save for steel rebar were compared. The comparison between the specimens is, as shown in Table 4.7.

Table 4.7: Comparison of samples to determine the effect of rebar on axial capacity.

Reference Sample	Test Sample
Specimen C	Specimen D
Specimen E	Specimen F
Specimen G	Specimen H
Specimen I	Specimen J

The change in axial capacity between the reference sample and test sample was calculated by Equation 4.2.

$$Percentage\ gain\ in\ strength \quad = \frac{Test - Reference}{Reference} \times 100\% \qquad \text{Equation 4.2}$$

The results are presented in Figure 4.8.

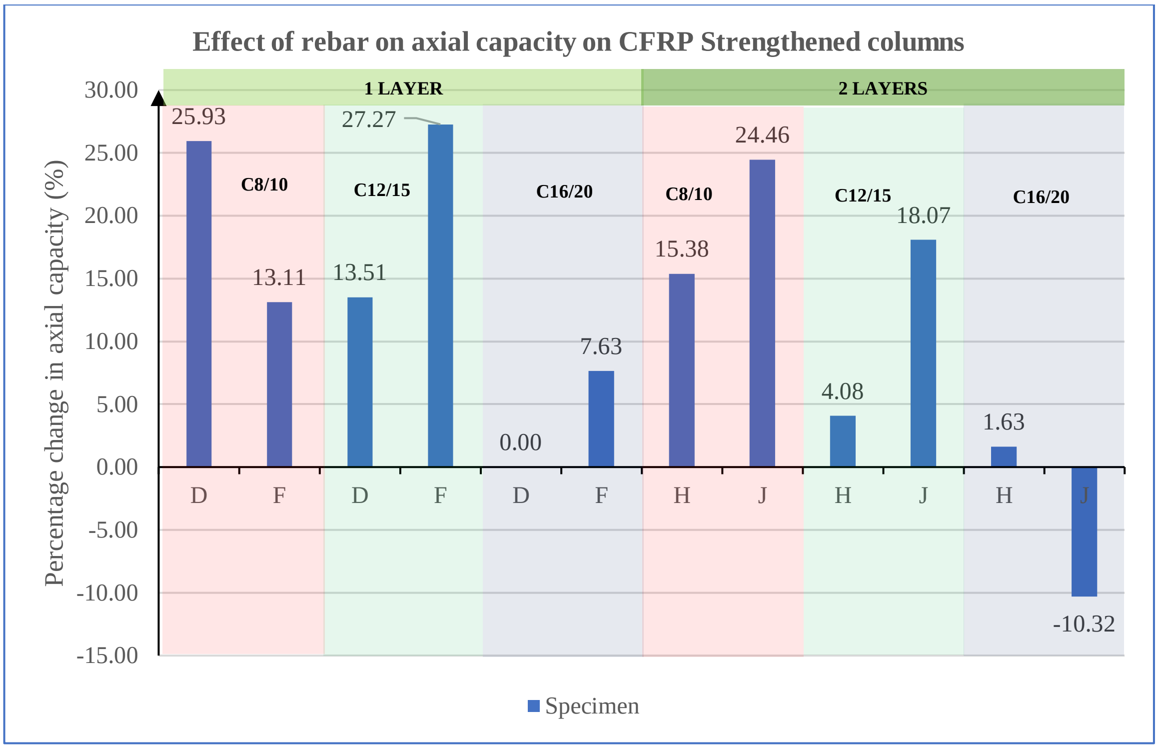


Figure 4.8: Change in axial capacity due to rebar in CFRP strengthened columns.

Figure 4.8 shows the change in the axial capacity of CFRP strengthened samples with rebar present compared with similar corresponding specimen with rebar absent. In the chart, specimens were classified based on the concrete grade, which was highlighted by the different colours in the background and by the number of layers. Samples D and F have one layer of CFRP confinement, while Samples H and J have two layers of CFRP confinement. On average, specimens with rebar (Specimens D, F, H and J) had an 11.73% higher axial capacity than specimens without rebar (Specimens C, E, G and I). Based on this data, we can state that rebar caused an increase in the axial capacity of the column specimens. There was a decrease in axial capacity of concrete columns of Concrete C16/20 when two layers of CFRP were used with rebar present. This decrement may be attributed to the fact that rebar may have reduced the lateral expansion of the concrete column resulting in less passive confinement generated hence the lower axial capacity of the reinforced specimen. In addition to that, Concrete C16/20 is more brittle than the other grades of concrete used in this experiment, and when coupled with rebar, it did not dilate as much as the other specimens tested.

Whereas the analysis presented shows that rebar increased the axial capacity of CFRP strengthened samples, it does not show how it affects the CFRP strengthening in itself. To find the effect of rebar on CFRP strengthening and not on the CFRP strengthened columns; the individual effect of CFRP strengthening on axial capacity of plain and reinforced concrete was compared.

4.3.1.2.3 Individual Effect of CFRP Strengthening on Axial Capacity of Plain Concrete

The plain concrete specimens C, E, G and I were compared with Specimen A of the corresponding concrete grades. The effect of CFRP strengthening on plain concrete was thus determined. The data was separated based on concrete grade and the number of CFRP layers to end up with the chart shown in Figure 4.9.

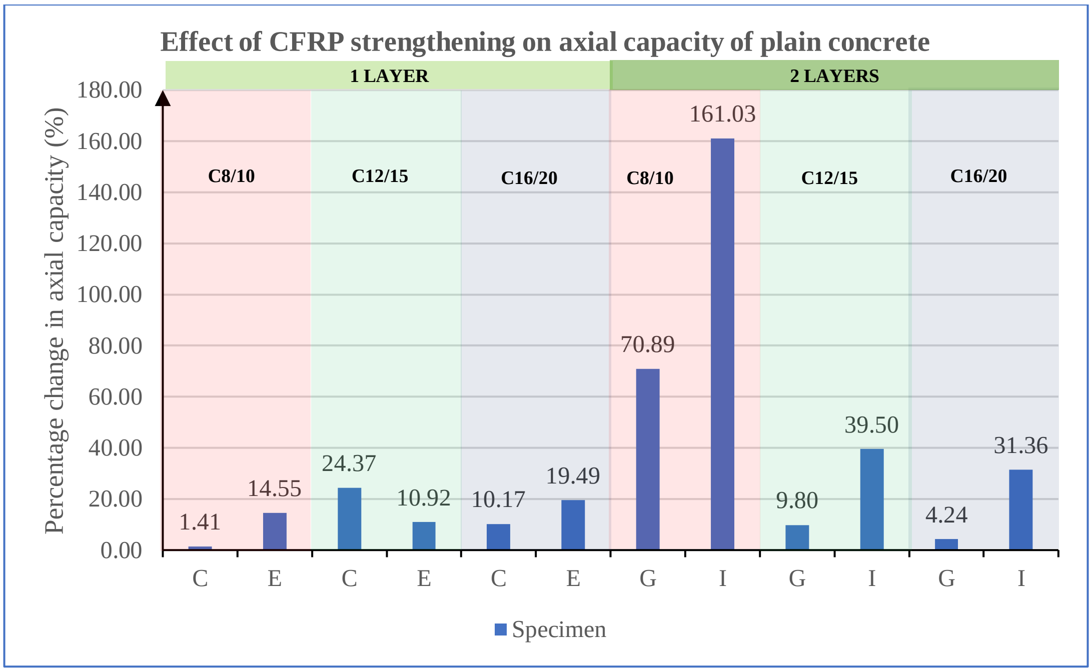


Figure 4.9: Change in axial capacity due to CFRP strengthening on plain concrete.

Figure 4.9 shows that CFRP strengthening caused an increase in axial capacity of the plain concrete specimen regardless of the different configurations of CFRP used. The study found that CFRP increased the axial capacity of the plain concrete specimens on average by 33.13%. This increase is anticipated because of the confinement mechanism of the CFRP wrap externally bonded to the specimen that causes a modification in the stress-strain behaviour of concrete (CEN, 2004).

4.3.1.2.4 Individual Effect of CFRP Strengthening on Axial Capacity of Reinforced Concrete

In the attempt to find out how rebar affects CFRP strengthening, the axial capacities of reinforced concrete specimens D, F, H and J were compared with Specimen B of the corresponding concrete grades. Specimen B was the control sample with rebar present. The effect of CFRP strengthening on reinforced concrete was thus determined. The effect was presented as the percentage change, as shown in Figure 4.10.

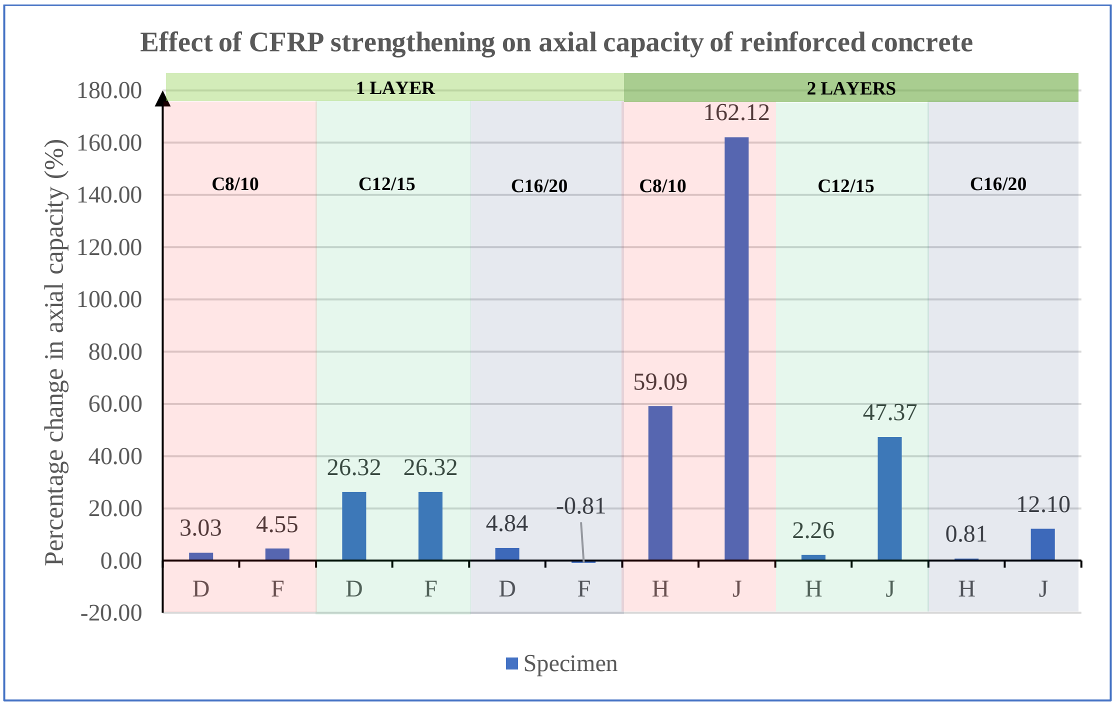


Figure 4.10: Change in axial capacity due to CFRP strengthening of reinforced concrete.

Averaging the values indicated in the chart, the study found that CFRP increased the axial capacity of the reinforced concrete specimens on average by 29%. This change is different from the effect of CFRP strengthening on plain concrete, as shown previously in Figure 4.10. The increase in axial capacity can be explained by the confinement mechanism of CFRP wrap (Abadi et al., 2019; CEN, 2004; Guo et al., 2018). However, the slight changes in the effect of

CFRP on axial capacity between plain concrete specimens, and reinforced concrete specimens would be investigated further.

Comparison of the effect of CFRP strengthening on axial capacity between plain concrete and reinforced concrete

To find out the difference between CFRP strengthening between plain concrete and reinforced concrete, the values in Figure 4.9 and Figure 4.10 were compared side to side resulting in Figure 4.11.

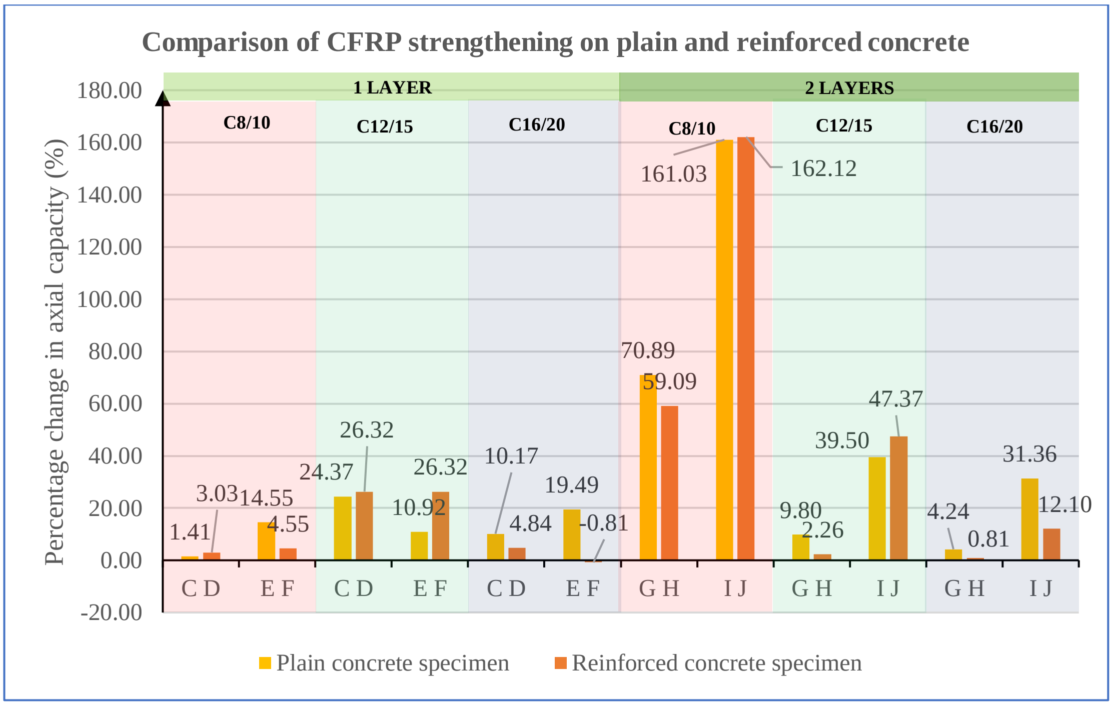


Figure 4.11: Comparison of change in axial capacity between plain and reinforced CFRP strengthened concrete specimens.

The comparison in Figure 4.11 shows that the various CFRP configurations cause a similar change in axial capacity regardless of the initial conditions of the concrete columns prior to the addition of CFRP strengthening. It confirms that CFRP strengthening causes a similar increment in the axial capacity, despite the presence or absence of steel reinforcement. This finding negates the second research hypothesis in Section 1.5 that stated there is a significant difference between CFRP strengthening on the plain and reinforced concrete specimen on the axial capacity of non-slender concrete columns. Figure 4.11 shows that the presence of rebar does not seem to affect the effect of CFRP strengthening on axial capacity. This apparent behaviour was inspected further when comparing the combined effect of both CFRP and rebar.

4.3.1.2.5 Combined Effect of Rebar and CFRP Strengthening on Axial Capacity of Columns

To find out the combined effect of rebar and CFRP strengthening on axial capacity, Specimen D, F, H and J were compared with Specimen A. Specimens D, F, H and J had both CFRP strengthening and rebar. At the same time, Specimen A was plain concrete specimen without CFRP strengthening. The combined effect was compared with the **sum** of the individual effects of both rebar and CFRP strengthening.

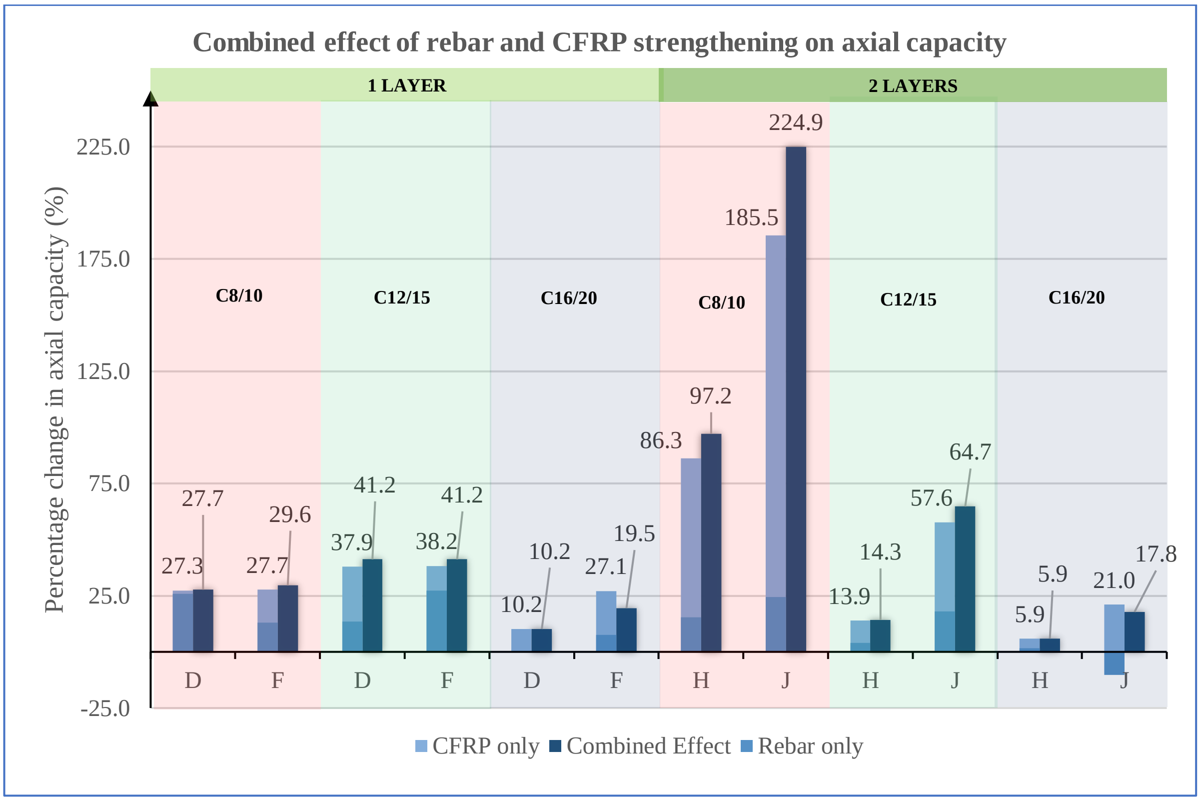


Figure 4.12: Combined effect of rebar and CFRP strengthening on the axial capacity.

Averaging the values indicated in Figure 4.12, the **sum** of the gain in axial capacity due to the individual effect of rebar and the individual effect of CFRP was 44.88%. From the chart again, the average gain in axial capacity due to the combined effect of both rebar and CFRP strengthening was 49.51%. The calculations of the averages were as shown:

Individual effect of rebar + Individual effect of CFRP

$$Average = \frac{27.3 + 27.7 + 37.9 + 38.2 + 10.2 + 27.1 + 86.3 + 185.5 + 13.9 + 57.6 + 5.9 + 21.0}{12}$$

$$= 44.88\%$$

Combined effect of rebar and CFRP

$$Average = \frac{27.7 + 29.6 + 41.2 + 41.2 + 10.2 + 19.5 + 97.2 + 224.9 + 14.3 + 64.7 + 5.9 + 17.8}{12}$$

$$=49.51\%$$

Comparison of the final calculated values and from the inspection of Figure 4.12, it is evident that the combined effect of CFRP strengthening and rebar on axial capacity is comparable to the **sum** of the individual effect of rebar and the individual effect of CFRP strengthening. This realisation is vital as it provides a rationale to which engineers can determine the combined effect of CFRP strengthening and rebar on the axial capacity of RC columns.

4.3.1.2.6 Summary of the Effect of Rebar on Axial Capacity of CFRP Strengthened Columns

Figure 4.13 provides a summary of the individual and combined effects of both rebar and CFRP strengthening on the axial capacity of CFRP strengthened columns.

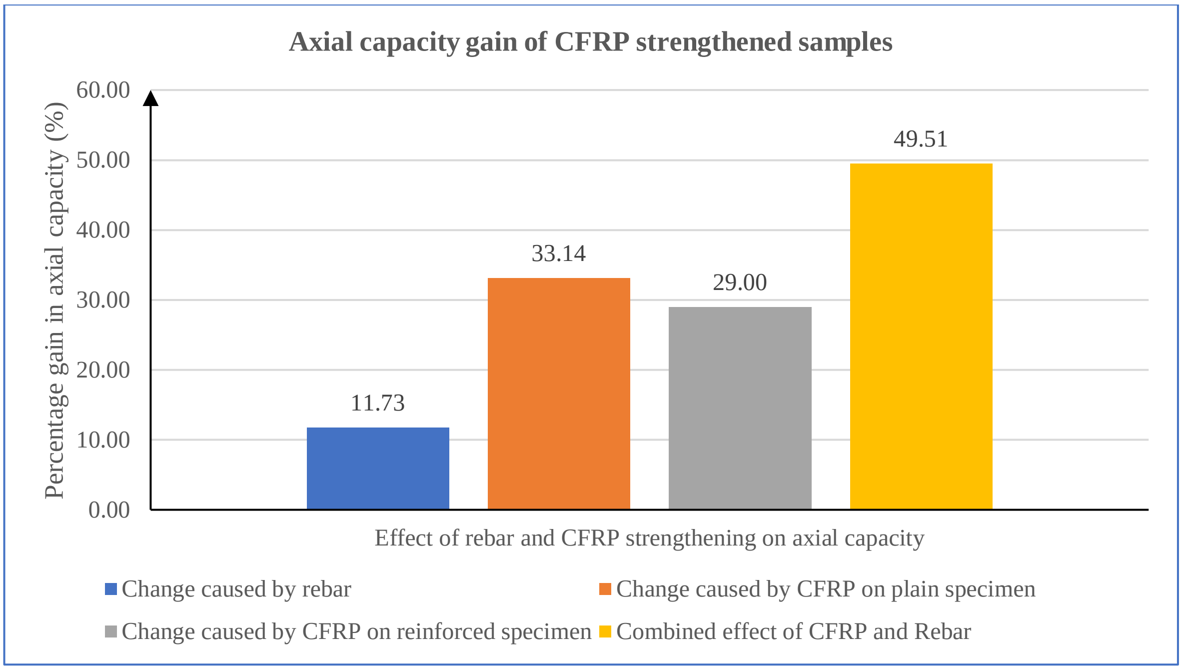


Figure 4.13: Summary of the effect of rebar and CFRP on the axial capacity of CFRP strengthened samples.

This chart generally illustrates the phenomenon discussed; that there is a slight difference between the effect of CFRP strengthening on the axial capacity of plain and reinforced concrete and the combined effect of both CFRP and rebar is roughly the sum of the individual effects as shown in Table 4.8.

Table 4.8: Comparison of the combined effect of rebar and CFRP on axial capacity.

Contribution	Change in axial capacity
Effect of rebar on CFRP strengthened columns + Effect of CFRP on plain concrete specimen	=11.73%+33.14% = 44.87%
Effect of rebar on CFRP strengthened columns + Effect of CFRP on reinforced concrete specimen	=11.73%+29.00 = 40.73%
Combined Effect of rebar and CFRP	=49.51%

4.3.1.3 Effect of Degree of CFRP confinement on Axial Capacity of CFRP Strengthened Columns

Using the sample data on axial capacity in Table C.1, the following means and standard deviations were calculated and presented in Table 4.9.

Table 4.9: Descriptive statistics of axial capacity of samples with different degrees of confinement.

Concrete Grade	Mean	Std. Deviation	N
Bare	13.0235	3.99110	23
Partial	15.4564	3.87065	36
Full	18.4317	5.16305	36
Total	**15.9948**	**4.87621**	**95**

The ANOVA test was conducted at the 95% confidence interval and a level of significance of 5%. The results are indicated in Table 4.10. The analysis of the sample data shows that the mean difference between the samples of partial CFRP and full CFRP confinement is significant with a 95% level of confidence.

Table 4.10: ANOVA table of axial capacity between groups with different degrees of confinement.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	427.277	2	213.638	10.872	0.000
Within Groups	1807.802	92	19.650		
Total	**2235.079**	**94**			

The ANOVA test, $F(2,92) = 10.872, p < 0.001$, shows that treatments between the groups with different degrees of confinement are significantly different at the 95% confidence interval. Since the ANOVA test shows that treatments between the groups with varying degrees of confinement were significantly different, post hoc testing was done to find out which group was different from the others. The groups were of different sizes; 23 samples were bare; 36 samples had partial CFRP confinement and 36 samples had full CFRP confinement. The Gabriel method was used for post hoc testing for multiple comparisons within and between the groups as shown in Table 4.11.

Table 4.11: Post hoc testing of axial capacity of groups with different degrees of confinement.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
Bare	Partial	-2.43291	1.18329	0.118	-5.2910	.4252
	Full	-5.40819*	1.18329	0.000	-8.2663	-2.5501
Partial	Bare	2.43291	1.18329	0.118	-0.4252	5.2910

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
	Full	-2.97528*	1.04483	0.016	-5.5146	-0.4360
Full	Bare	5.40819*	1.18329	0.000	2.5501	8.2663
	Partial	2.97528*	1.04483	0.016	0.4360	5.5146
* The mean difference is significant at the 0.05 level.						

If a group has a significance value p, of less than 0.05 then the group difference is statistically significant. The post hoc testing shows that each group is different from each other significantly except for the comparison between no confinement and partial CFRP confinement. Post hoc testing reveals that at the 5% level of significance, the axial capacity of columns with partial CFRP confinement is not statistically significantly different from columns without any CFRP strengthening. Post hoc testing also reveals that at the 5% level of significance, the axial capacity of columns with full CFRP confinement is statistically significantly different from both the columns with partial CFRP confinement and columns with no CFRP strengthening. Since the difference between the groups was statistically significant, the means between the groups could then be compared.

4.3.1.3.1 Findings of Statistical Significance on Degree of Confinement on Axial Capacity

There was a significant difference among the three concrete grades $F(2,92) = 10.872, p < 0.001$. Post hoc testing revealed significant differences between the groups with different treatment: bare columns ($M = 13.02, SD = 3.99$) which had the least axial capacity, Partial CFRP confinement ($M = 15.46, SD = 3.87$) and Full CFRP confinement ($M = 18.43, SD = 5.16$) which had the highest axial capacity.

4.3.1.3.2 Comparison of Partial CFRP and Full CFRP Confinement

The results were rearranged and categorised to compare the effect of partial CFRP confinement versus full CFRP confinement, as shown in Figure 4.14.

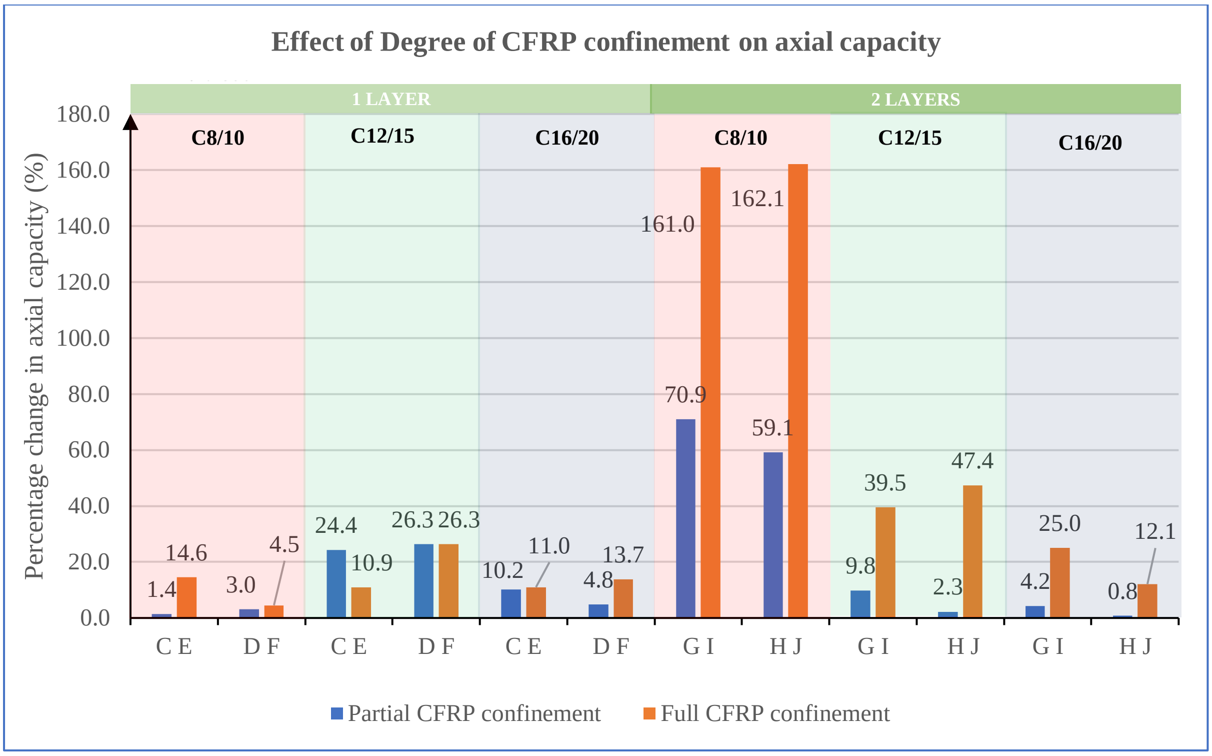


Figure 4.14: Effect of CFRP confinement on axial capacity.

The analysis in Figure 4.14 thus shows that full CFRP confinement causes a higher increment in axial capacity than partial CFRP confinement **except** for the comparison of Specimens C and E of Concrete C12/15. There seems to be no rational explanation for this occurrence apart from assuming that it was a sample variance. Whereas Figure 4.14 confirms that partial CFRP confinement increases the axial capacity of square columns, it disproves claims made by some CFRP manufacturers that partial and full CFRP confinement have the same effect in terms of the axial capacity of concrete columns (Horse Construction, 2019).

4.3.1.3.3 Summary of the effect of degree of CFRP confinement on axial capacity of CFRP strengthened samples.

Figure 4.15 shows the comparison between partial CFRP confinement and full CFRP confinement.

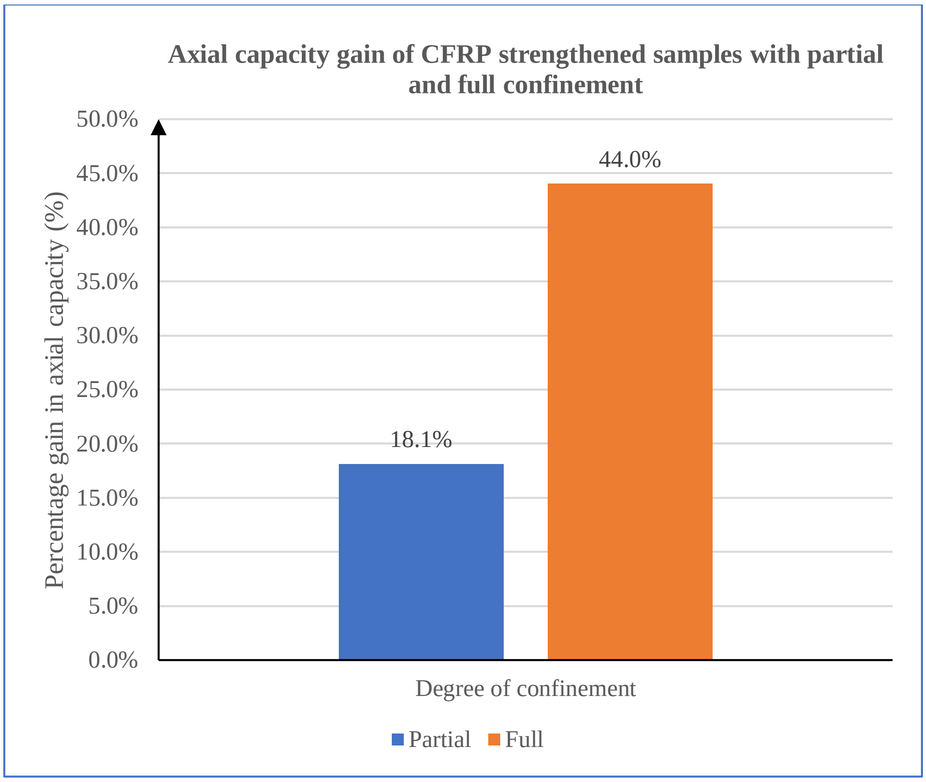


Figure 4.15: Summary of the effect of degree of CFRP confinement on axial capacity of CFRP strengthened samples.

On average, partial CFRP confinement (Specimens C, D, G and H) caused an 18.1% gain in axial capacity while full CFRP confinement (Specimens E, F, I and J) caused an increase of 44% in axial capacity. Full CFRP confinement offers more passive pressure as the column dilates hence a higher axial capacity than partial CFRP confinement (Guo et al., 2018).

Full CFRP confinement should therefore be used whenever higher increases in strength are required. In situations where small increases in axial capacity are required, partial confinement serves as a viable solution.

4.3.1.4 Effect of Number of Layers on Axial Capacity of CFRP Strengthened Columns

Using the sample data on axial capacity in Table C.1, the following means and standard deviations were calculated and are presented in Table 4.12.

Table 4.12: Descriptive statistics of axial capacity of samples with different number of layers.

Number of Layers	Mean	Std. Deviation	N
None	13.0235	3.99110	23
Single	15.3822	4.75020	36

Number of Layers	Mean	Std. Deviation	N
Double	18.5058	4.31386	36
Total	**15.9948**	**4.87621**	**95**

ANOVA was carried out at the 95% confidence interval and the results are as indicated in Table 4.13. The analysis of the sample data shows that the mean difference between the samples of partial CFRP and full CFRP confinement is significant with a 95% level of confidence.

Table 4.13: ANOVA table of axial capacity between groups with different number of layers.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	443.561	2	221.780	11.389	0.000
Within Groups	1791.518	92	19.473		
Total	**2235.079**	**94**			

The ANOVA test, $F(2,92) = 11.389, p < 0.001$, shows that treatments between the groups with the different number of layers are significantly different at the 95% confidence interval. Since the ANOVA test shows that treatments between the groups were statistically significantly different, post hoc testing was done to find out which group was different from the others. The groups were of different sizes; no layer: 23 samples, single layer: 36 samples and double layers: 36 samples; the Gabriel method was used for post hoc testing for multiple comparisons within and between the groups as shown in Table 4.14.

Table 4.14: Post hoc testing of axial capacity of groups with different number of layers.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
None	Single	-2.35874	1.17795	0.133	-5.2040	0.4865
	Double	-5.48236*	1.17795	0.000	-8.3276	-2.6371
Single	None	2.35874	1.17795	0.133	-0.4865	5.2040
	Double	-3.12361*	1.04011	0.010	-5.6515	-0.5958
Double	None	5.48236*	1.17795	0.000	2.6371	8.3276
	Single	3.12361*	1.04011	0.010	.5958	5.6515
* The mean difference is significant at the 0.05 level.						

If a group has a significance value p, of less than 0.05 then the group difference is statistically significant. The post hoc testing shows that double layer CFRP confinement is statistically significantly different from the columns with single layer confinement and no confinement.

4.3.1.4.1 Findings of Statistical Significance on Number of Layers on Axial Capacity

There was a significant difference among the CFRP layers $F(2,92) = 11.389, p < 0.001$. Post hoc testing revealed significant differences between the groups with different treatment: bare columns ($M = 13.02, SD = 3.99$) which had the least axial capacity, followed by single CFRP layer confinement ($M = 15.38, SD = 4.75$) and finally full CFRP confinement ($M = 18.51, SD = 4.31$) which had the highest axial capacity. Since the difference between the single and double layers was significant, the means between the groups could then be compared.

4.3.1.4.2 Comparison of Single and Double CFRP Layers on Axial Capacity

The results were sorted and organised to find the effect of the number of CFRP layers on the axial capacity of the concrete columns. This analysis resulted in Figure 4.16 that shows the change caused by one layer versus two layers side by side.

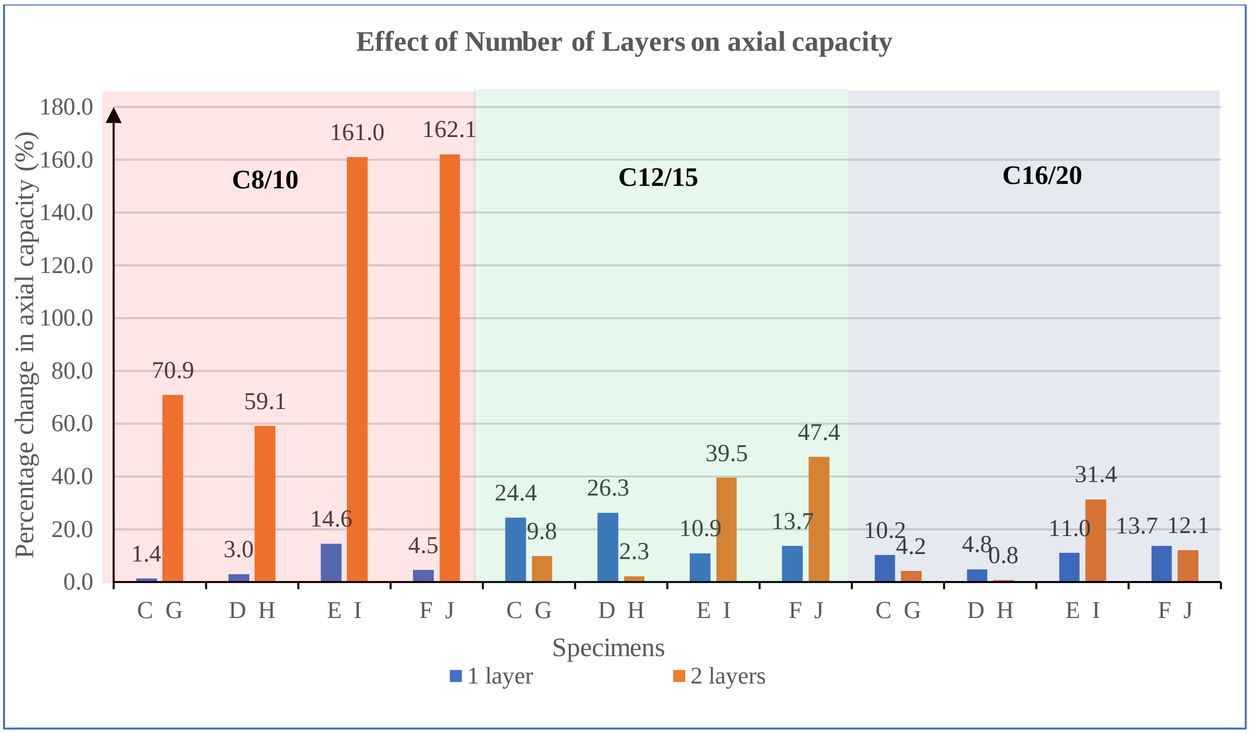


Figure 4.16: Effect of the number of CFRP layers on axial capacity.

By observation of the chart, two layers of CFRP had the highest effect in the axial capacity of the weakest grade of concrete. In Concrete C8/10, two layers of CFRP strengthening was consistently higher than one layer of CFRP strengthening. This trend; however, does not apply to Concrete C12/15, as shown in Figure 4.16. Surprisingly, one layer of partial CFRP confinement had a higher effect than two layers of partial CFRP confinement in two grades of concrete: C12/15 and C16/20. Concrete C16/20 samples show a slight change between the number of layers and the gain in axial capacity. This trend goes against the findings in literature (Shehata et al., 2002; Shrive et al., 2003) and is attributed to sample variance.

There was no trend visualised in this chart for Concrete C12/15 and Concrete C16/20. The lack of a definite pattern found no scientific explanation. This necessitated the need to calculate the average effect to bring out the differences.

4.3.1.4.3 Summary of the Effect of the Number of Layers on Axial Capacity of CFRP Strengthened Columns

The comparison of the number of layers is shown in Figure 4.17.

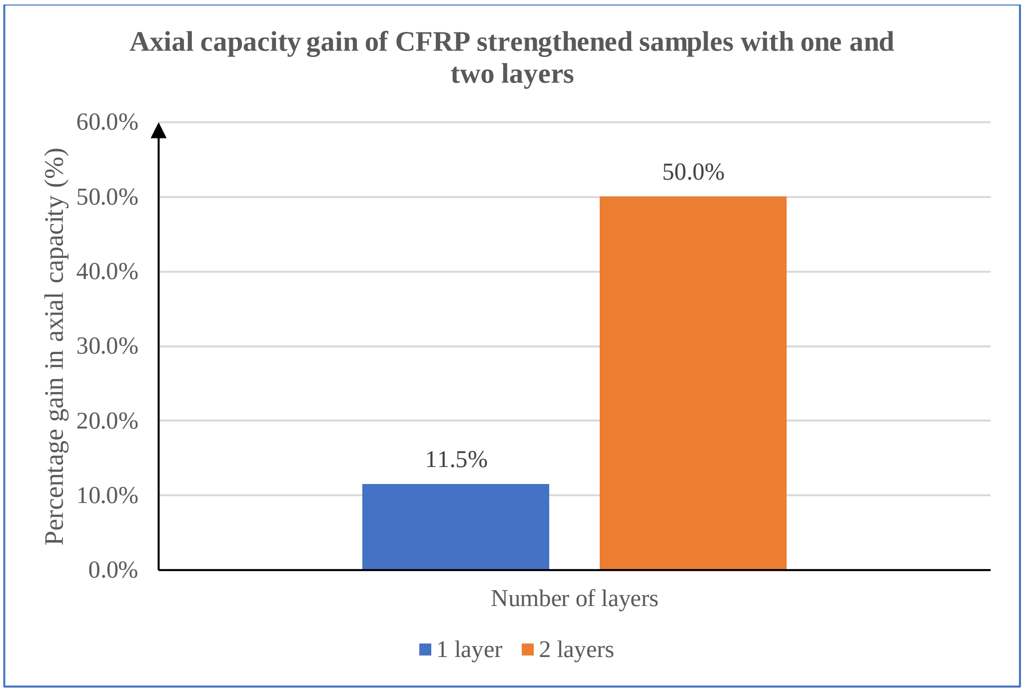


Figure 4.17: Summary of the effect of the number of CFRP layers on the axial capacity of CFRP strengthened samples.

The difference between one layer and two layers of CFRP confinement was evident. On average, one layer of CFRP confinement (Specimens C, D, E and F) caused an 11.5% gain in axial capacity while two layers of CFRP confinement (Specimens G, H, I and J) caused a 50% gain in axial capacity. The number of layers increases the thickness of the CFRP confinement and seems to have the highest effect on the axial capacity of the four parameters investigated (Mohammed et al., 2020).

Two layers of CFRP confinement should be used when greater changes in axial capacity are required. For example, in a building being retrofitted, lower storeys that bear higher loads may have two layers of CFRP installed. In the upper storeys, where the loads are less, one layer of CFRP may be used. This optimisation of resources ensures maximum benefit at the least cost.

4.3.2 Ductility of Specimens

The organisation of this section is as that presented in the analysis of axial capacity of specimens. To analyse the effect of CFRP strengthening, the results of ductility of the specimens are first presented. After that, comparisons with the control samples are made in order to find out the effect of concrete grade, rebar, degree of confinement and number of CFRP layers on ductility.

The ductility of the column specimens was measured and calculated as percentage deformation at failure. The deformation was measured along the longitudinal axis. There was difficulty in obtaining the equipment necessary to measure the transverse deformation. Transverse deformations, that is, hoop strains have been left out of this study.

The average values of ductility for specimens with Concrete C8/10 are presented in Figure 4.18.

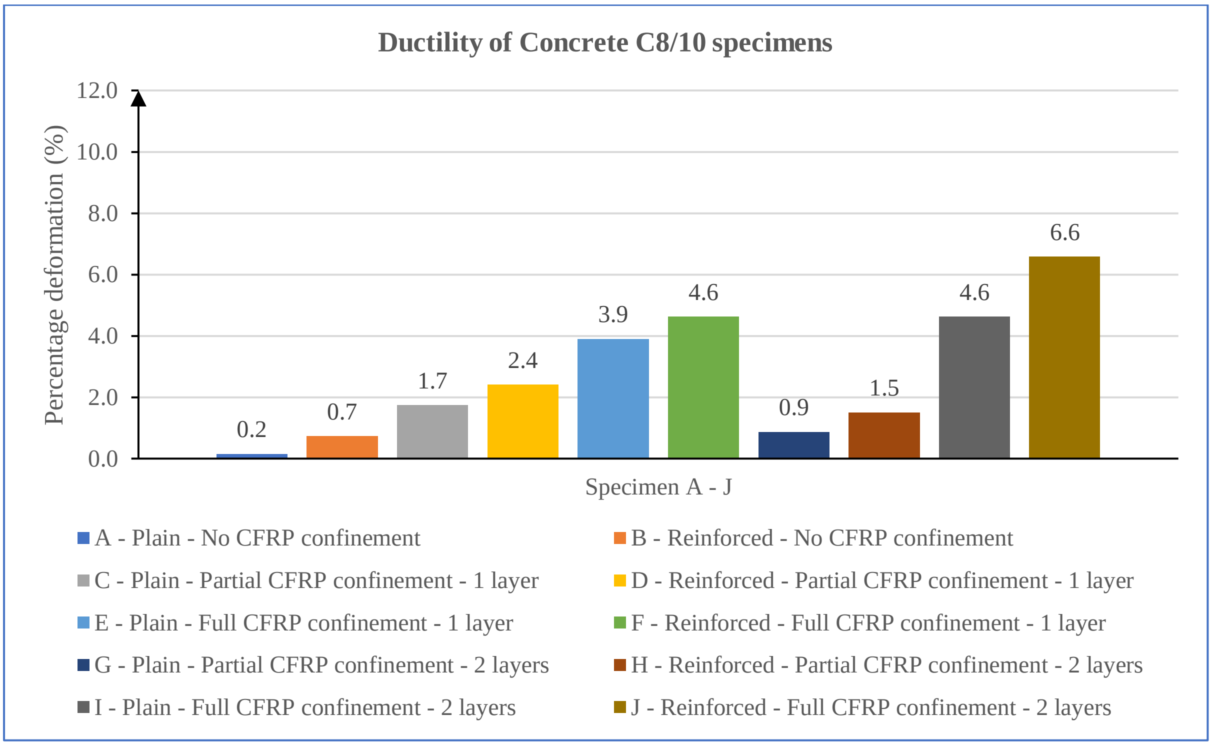


Figure 4.18: Average ductility of Concrete C8/10 specimens.

Figure 4.18 shows the results of 32 concrete specimens. The average ductility of specimens without CFRP strengthening (Specimens A and B) was 0.45% whereas the average ductility of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) was 3.3%. CFRP strengthening caused an average change in ductility of 633.3%. The plain concrete specimen with no CFRP strengthening had the least ductility while the reinforced concrete specimen with two layers of full CFRP confinement had the highest ductility. These results show that CFRP strengthening increases the ductility of columns. Further analysis and discussions will be made after the results of ductility of Concrete C12/15 and C16/20 are presented.

For Concrete C12/15, the trend for ductility evident in Concrete C8/10 is not as clear.

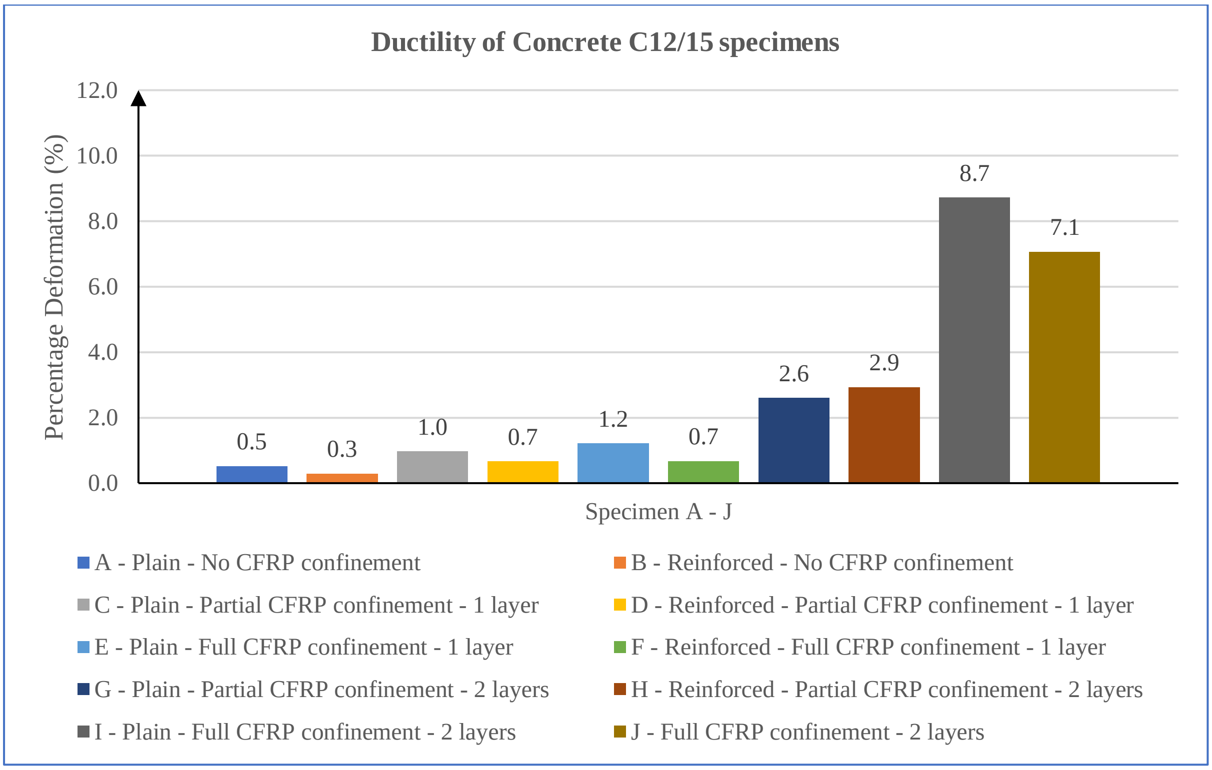


Figure 4.19: Average ductility of Concrete C12/15 specimens.

The average ductility of specimens without CFRP strengthening (Specimens A and B) is 0.4%. In contrast, the average ductility of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) is 3.1%. The percentage change in ductility was 718%. The chart in Figure 4.19 shows that there is an effect on the increase in ductility caused by the different configurations of CFRP. It is worth noting that there is a conspicuous change in ductility when two layers of CFRP are used as evidenced by the values of specimens G, H, I and J which had two layers of CFRP wrap. By inspection of the chart, specimens with rebar (Specimens B, D, F, H and J) seemed to have less ductility than specimens without rebar (Specimens A, C, E, G and I). This behaviour seems to go against findings in the literature reviewed since rebar should improve the ductility of columns (Hou et al., 2019). Further analysis in the subsequent sections was done to identify the trend.

This trend of ductility in Concrete C12/15 specimens was repeated in columns made of Concrete C16/20 specimens, as shown in Figure 4.20.

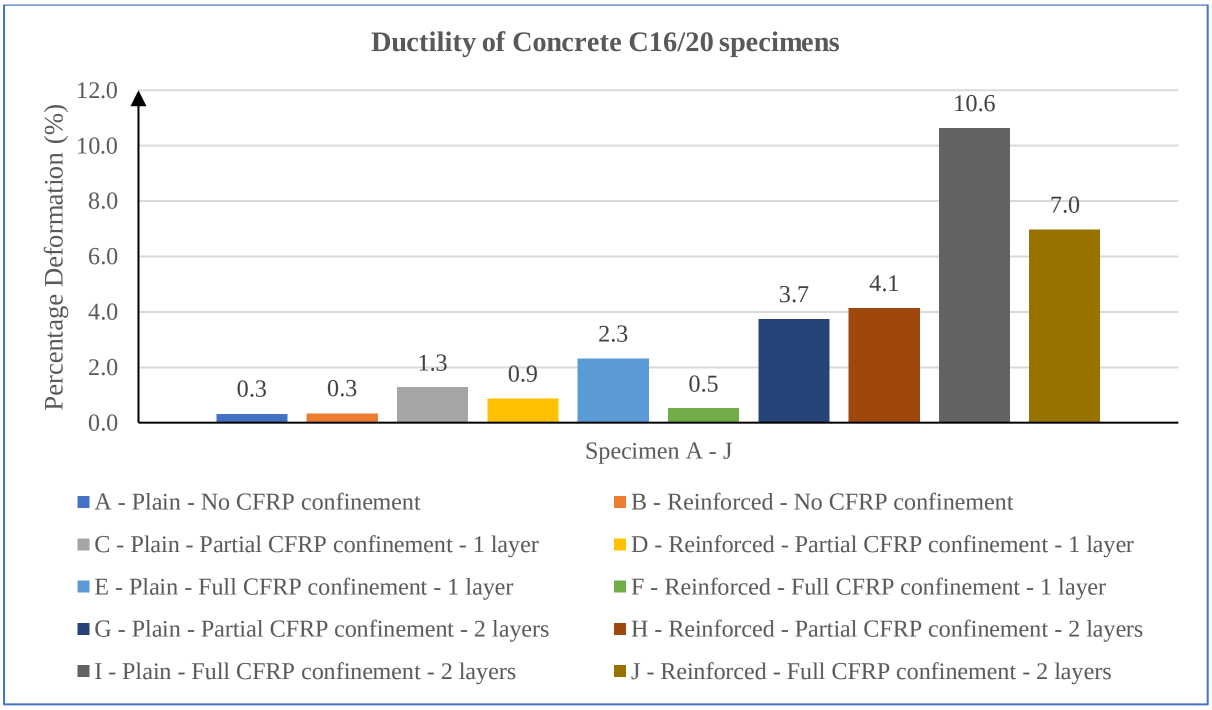


Figure 4.20: Average ductility of Concrete C16/20 specimens.

The ductility of specimens without CFRP strengthening (Specimens A and B) was 0.3% while that of specimens with CFRP strengthening (Specimens C, D, E, F, G, H, I and J) was 3.8%. The percentage change in ductility from 0.3% to 3.8% was 1166.7%. CFRP strengthening increases the ductility of the column specimens. There is a slight increase in ductility when one layer of CFRP is used as shown by results of specimens C, D, E and F. A higher change in ductility was reported when two layers of CFRP were used as evidenced by the results of specimens G, H, I and J. This observed trend was analysed further to determine how the concrete grade, steel rebar, degree of confinement and number of CFRP layers affected the ductility.

Effect of Dependent Variables on Ductility of CFRP Strengthened Columns

4.3.2.1 Effect of Concrete Grade on Ductility of CFRP Strengthened Columns

Using the sample data on axial capacity in Table D.1, the following means and standard deviations were calculated and presented in Table 4.15.

Table 4.15: Descriptive statistics of ductility of samples of different concrete grades.

Concrete Grade	Mean	Std. Deviation	N
C8/10	0.025688018	0.0237435394	32
C12/15	0.024234321	0.0305740525	32
C16/20	0.030284525	0.0344117035	31
Total	**0.026698265**	**0.0296441609**	**95**

ANOVA was carried out at the 95% confidence interval and a level of significance of 5%. The results are as indicated in Table 4.16. The analysis of the sample data shows that the mean difference of ductility between the samples of the different concrete grade is not significant at the 95% level of confidence.

Table 4.16: ANOVA table of ductility between groups of different concrete grades.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	0.001	2	0.000	0.351	0.705
Within Groups	0.082	92	0.001		
Total	**0.083**	**94**			

The ANOVA test, $F(2,92) = 0.351, p < 0.705$, shows that treatments between the groups of different grades were not significantly different at the 95% confidence interval. Post hoc testing was done to find out if there was an individual group that was different from the others. The group sizes were different; Concrete C8/10 had 32 samples; Concrete C12/15 had 32 samples and Concrete C16/20 had 31 samples; therefore, the Gabriel method was used for post hoc testing for multiple comparisons within and between the groups as shown in Table 4.4.

Table 4.17: Post hoc testing of ductility of groups of different concrete grades.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
C8/10	C12/15	0.00145	0.00746	0.996	-0.01668	0.01959
	C16/20	-0.00459	0.00752	0.903	-0.02287	0.01368
C12/15	C8/10	-0.00145	0.00746	0.996	-0.01959	0.01668
	C16/20	-0.00605	0.00752	0.806	-0.02433	0.01223
C16/20	C8/10	0.00459	0.00752	0.903	-0.01368	0.02287
	C12/15	0.00605	0.00752	0.806	-0.01223	0.02433

If a group has a significance value p, of greater than 0.05 then the group difference is not statistically significant. The post hoc testing shows that each group is not different from each other significantly.

4.3.2.1.1 Findings of Statistical Significance on Concrete Grade on Ductility

There was no significant difference in ductility among the three concrete grades $F(2,92) = 0.351, p < 0.705$. Post hoc testing revealed no significant differences between Concrete C8/10 ($M = 0.0257, SD = 0.0237$), Concrete C12/15 ($M = 0.0242, SD = 0.0306$) which had the least ductility and C16/20 ($M = 0.0303, SD = 0.0344$) which had the highest ductility. Even though the difference between the groups was not significant, further investigation was done to discover any trend that may be evident.

4.3.2.1.2 Comparison of Concrete Grades on Ductility

The percentage change in ductility of the three grades of concrete was compared against the control specimens, and the result is presented in Figure 4.21.

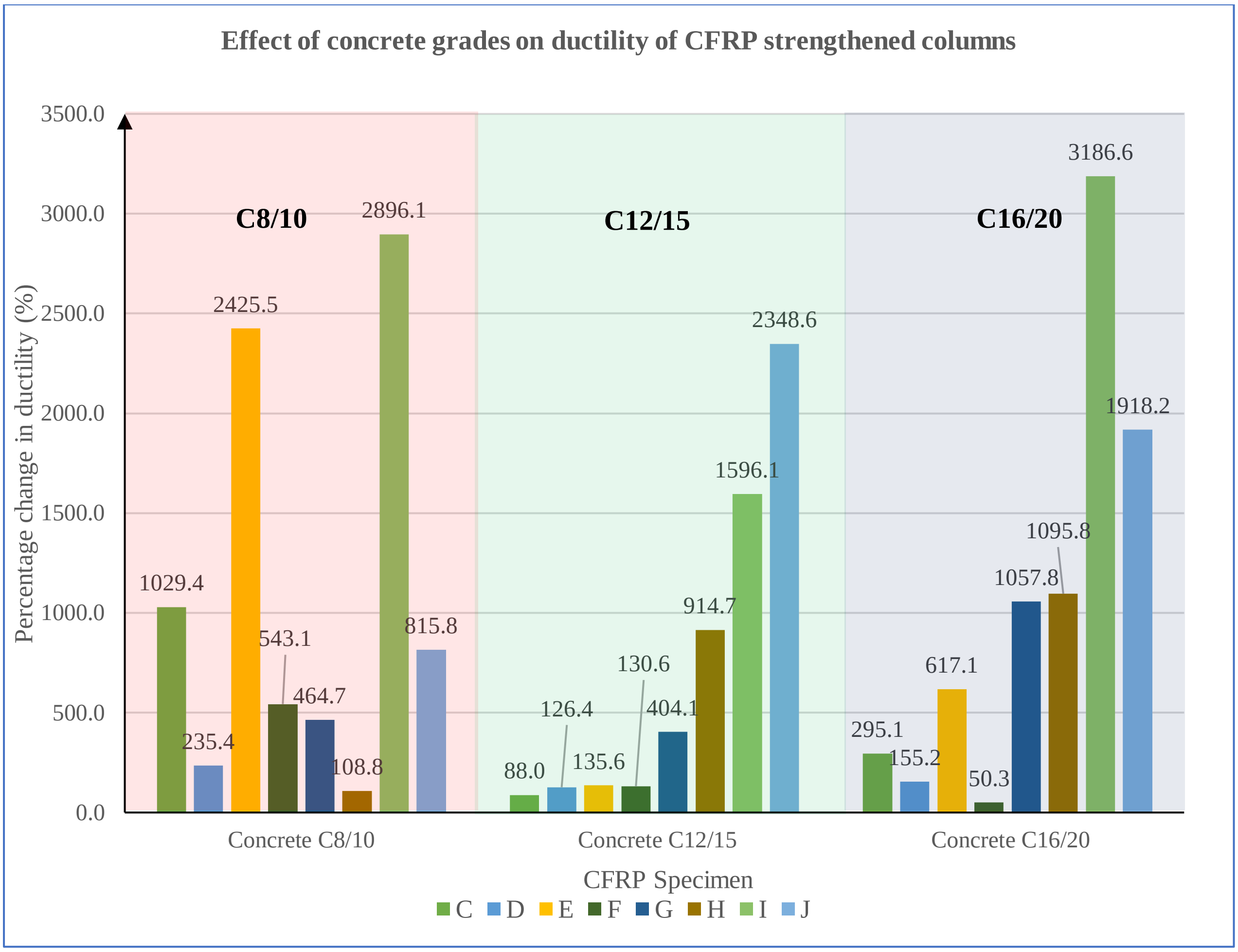


Figure 4.21: Change in ductility against specimens of different concrete grades.

From the changes between the values of ductility of the concrete, it is evident that concrete grade affects the ductility of non-slender square concrete columns. This finding confirms the first hypothesis that stated concrete grade affects the ductility of non-slender square concrete columns; however, there was no apparent pattern observed from one concrete grade to another in the chart. To visualise any trend, the average effect of CFRP strengthening for all concrete grades was performed, as presented in Figure 4.22.

4.3.2.1.3 Summary of the Effect of Concrete Grade on the Ductility of CFRP Strengthened Columns

Figure 4.22 shows a bar graph illustrating the change in means of the ductility of the three concrete grades.

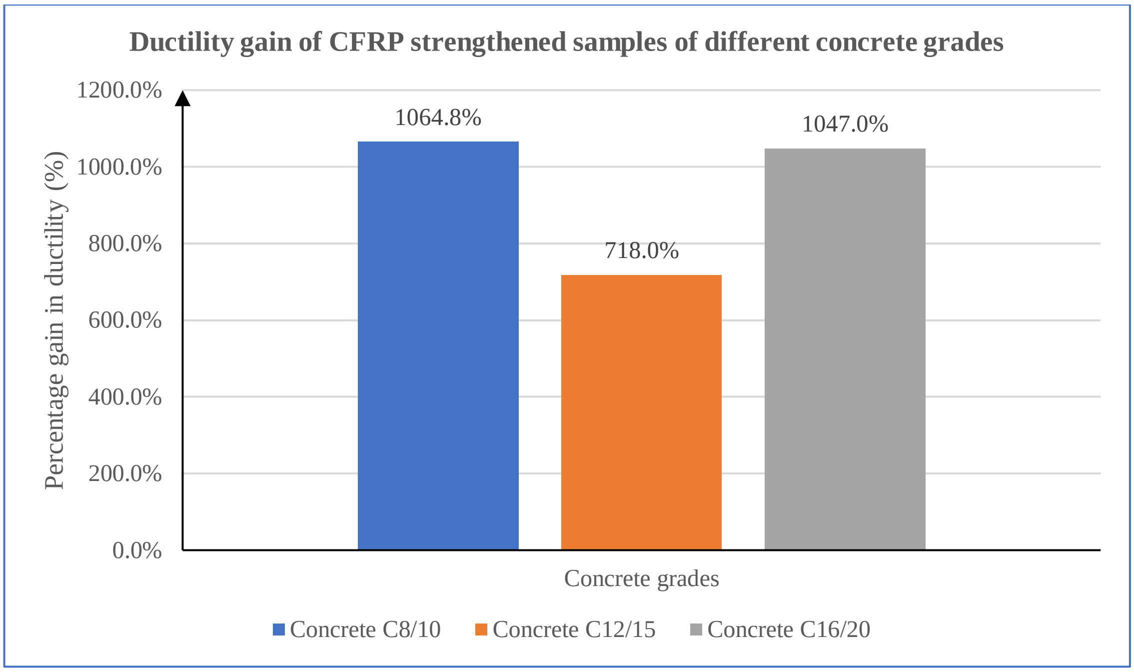


Figure 4.22: Summary of the effect of concrete grade on the ductility of CFRP strengthened samples.

There seems to be no direct relationship between concrete grades and the ductility of the reinforced concrete columns; however, on average; Concrete C8/10 experienced a 1064.8% change in ductility, Concrete C12/15 experienced 718% change while Concrete C16/20 had 1047% change. Whereas these changes seem relatively high, these changes are consistent with a study by Wang et al. (2019) that had a 500% increase in ductility for Concrete C25/30. That study had sophisticated sensors to measure deflections and was verified using Finite Element Analysis (Wang et al., 2019). Masia et al. (2004) also found increases in ductility of 522% for specimens of similar dimensions (125mm x 125mm x 375mm) to the one used in this study (150mm x 150mm x 350mm). Concrete C8/10 had the most considerable change in ductility caused by CFRP strengthening. This finding confirms the observations of studies by Shehata et al. (2002) and Shrive et al. (2003).

4.3.2.2 Effect of Rebar on Ductility of CFRP Strengthened Columns

The means and standard deviation for ductility of groups with rebar present and with rebar absent were calculated and presented in Table 4.18.

Table 4.18: Descriptive statistics of ductility of samples with rebar absent or present.

Rebar	Mean	Std. Deviation	N
Absent	0.027447875	0.0318040305	48
Present	0.025932705	0.0275878531	47

A Levene's test for equality of variances was performed for the T-test for equality of means and is indicated in Table 4.19.

Table 4.19: T-test for ductility of groups with rebar absent and present.

	Levene's Test for Equality of Variances		T-test for Equality of Means						
	F	Sig. (p)	t	df	Sig. (2-tld)	Mean Diff.	Std. Error Diff.	95% Confidence Interval Lower	95% Confidence Interval Upper
Equal variances assumed	0.423	0.517	0.248	93	0.805	0.00151	0.00611	-0.0106	0.01365
Equal variances not assumed			0.248	91.7	0.805	0.00151	0.00610	-0.0106	0.01364

A t-test was conducted to find out if the effect of rebar was significant. The number of samples was 95 and the critical value of the t-test was calculated as 1.986 using a Microsoft Excel template (Daniel & Kostic, 2019). The Levene's test checks whether variances are equal. If Levene's test is not significant, then equal variances are assumed. In this case, the f-value is 0.423 and the p-value is 0.517. The p-value is greater than 0.5 for the Levene's test and therefore the Levene's test is significant and therefore equal variances are not assumed. The t-value of 0.248 does not exceed the critical value of 1.986 and the significance value is 0.805 which is greater than the significance value of the p-value of 0.05. Also, at the 95% confidence interval for the difference between sample means, shows there is a lower bound of -0.01061 and an upper bound of 0.01364 and therefore crosses zero, it means that the difference of the two groups is not statistically significant. This finding was contrary to expectation since it was expected that the steel reinforcement would cause a statistically significant increase in ductility (Sayed & Diab, 2019) . The study went ahead to find out how other parameters affected the change in ductility in relation to the presence of steel reinforcement.

4.3.2.2.1 Individual effect of rebar on Ductility of columns without CFRP Strengthening

This experimental investigation sought to find out the effect of steel reinforcement on the ductility of reinforced concrete specimen. The difference in ductility of the plain concrete specimens was calculated by Equation 4.3.

$$\text{Percentage gain in ductility} = \frac{\text{Specimen } B - \text{Specimen } A}{\text{Specimen } A} \times 100\% \qquad \text{Equation 4.3}$$

Figure 4.23 shows the difference.

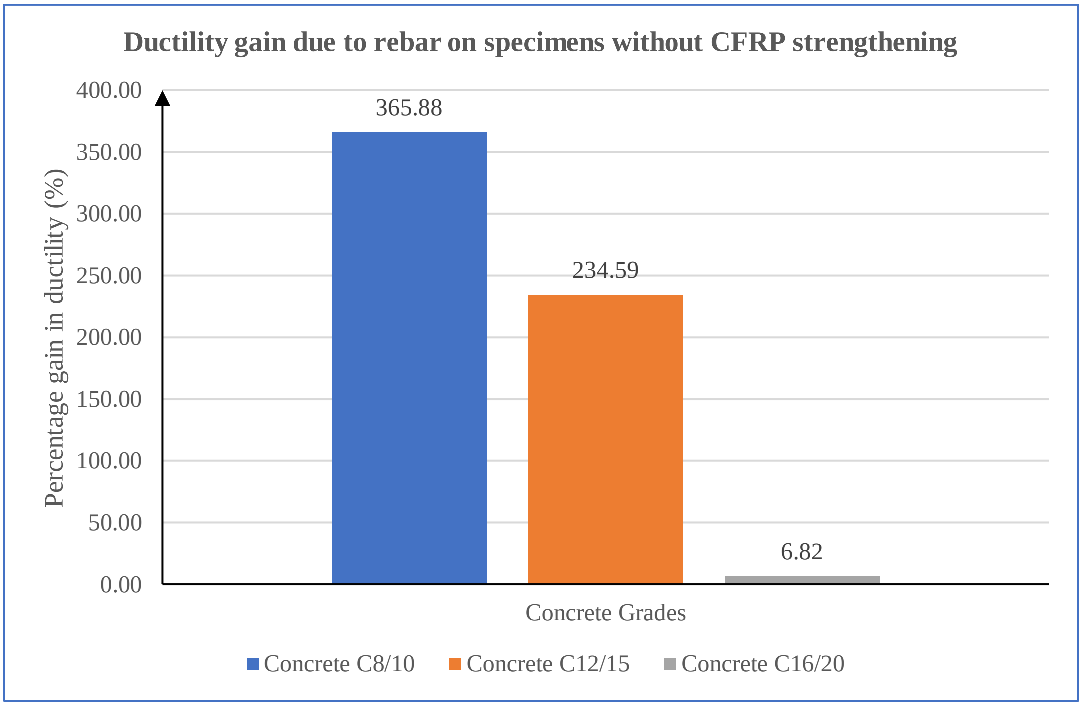


Figure 4.23: Ductility gain due to rebar.

The results show that rebar causes quite a notable increase in the ductility of concrete columns. There is an inverse relationship between rebar contribution on ductility and the concrete grade. The weaker the concrete, the more the contribution of rebar on ductility. The stronger the concrete, the less the steel reinforcement contributes to the ductility of the column. This finding was consistent with the findings of axial capacity discussed in the preceding sections. Rebar plays a significant role in the ductility of columns.

4.3.2.2.2 Individual Effect of Rebar on Ductility of CFRP Strengthened Columns

To determine the effect of rebar on the ductility of samples with the same CFRP configuration and the same concrete characteristics except for rebar, the comparison in Table 4.20 was made.

Table 4.20: Comparison of the specimens to determine the effect of rebar on ductility.

Control	Test
Specimen C	Specimen D
Specimen E	Specimen F
Specimen G	Specimen H
Specimen I	Specimen J

The change in ductility between the reference sample and test sample, as indicated in Table 4.20 was calculated by Equation 4.4.

$$Percentage\ change\ in\ ductility = \frac{Test - Reference}{Reference} \times 100\% \qquad \text{Equation 4.4}$$

The difference in ductility of the reinforced concrete specimens compared with the plain concrete specimens is presented in a bar graph, as shown in Figure 4.24.

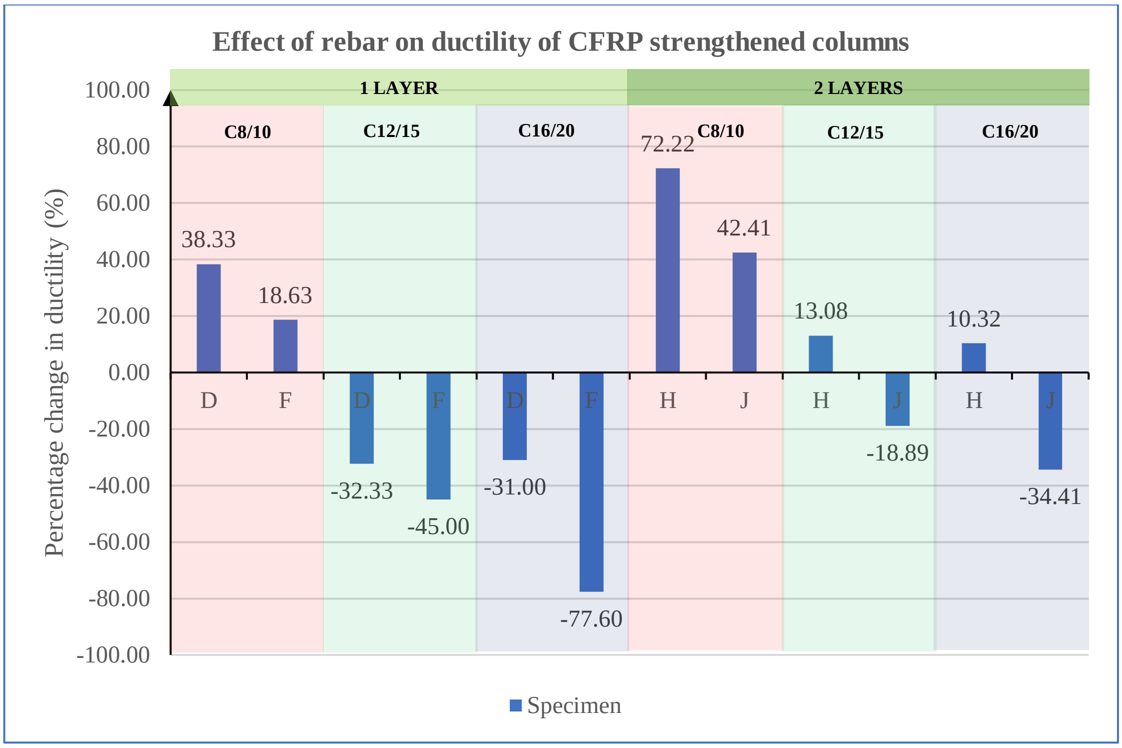


Figure 4.24: Effect of rebar on the ductility of specimens strengthened with CFRP.

This chart shows the effect of rebar on the ductility of specimens strengthened with CFRP. Reinforced samples of concrete grade C8/10 experienced gain in ductility compared to plain concrete specimens. Reinforced concrete samples of Concrete grade C12/15 and Concrete C16/20 experienced loss in ductility compared to the plain concrete specimen. That trend was repeated when either one layer or two layers of CFRP were used. This pattern was investigated further in the following subsection, where the effect of CFRP strengthening on plain concrete samples was determined.

On average, CFRP strengthened specimens with reinforcement had 3.6% less ductility than specimens without reinforcement. This finding was contrary to the literature reviewed in this study (Hou et al., 2019; The Concrete Society, 2000). It was assumed that the confinement offered by both rebar and CFRP would be higher than confinement provided by CFRP alone. The higher confinement would thus cause an increase in the ductility of the column. The results, however, indicate different behaviour of the CFRP strengthened samples. This contrary finding may be attributed to the fact that rebar contributes to the confinement of the deforming concrete as it is loaded; therefore, there is less lateral expansion of the concrete. Less dilation of the concrete core results in less passive confinement pressure generated on the CFRP wrap. The less confinement pressure results in less ductility in reinforced concrete samples while compared to plain concrete samples that have similar CFRP configuration. In addition to that, the CFRP strengthening would make the column stiffer and therefore, less ductile.

4.3.2.2.3 Individual Effect of CFRP Strengthening on Ductility of Plain Concrete

To find out the individual effect of CFRP strengthening on ductility, Specimen A was used as the control and compared with plain concrete specimens (Specimens C, E, G and I).

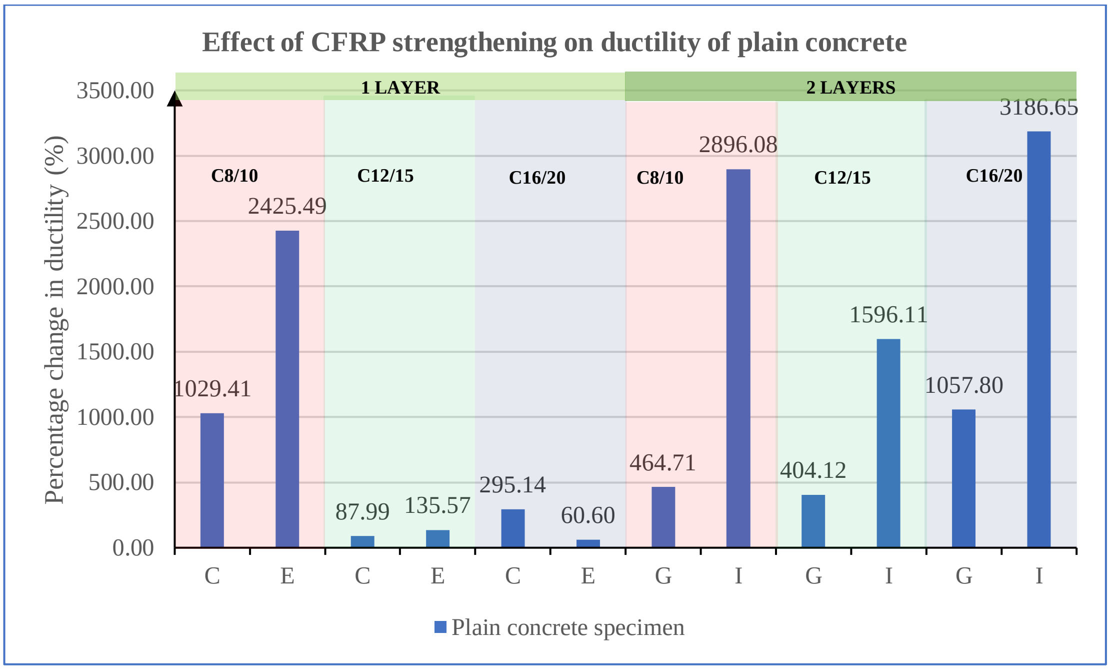


Figure 4.25: Effect of CFRP strengthening on the ductility of plain concrete columns.

All plain concrete specimens had an increase in ductility. On average, plain concrete specimens with CFRP strengthening had an increase of 1136.64% in ductility. This increase in ductility is attributed to the confining properties of CFRP (CEN, 2004).

4.3.2.2.4 Individual Effect of CFRP Strengthening on Ductility of Reinforced Concrete

Similarly, as in the earlier section, Specimen B was the control specimen compared to the reinforced concrete specimen (Specimens D, F, H and J) to find out the individual effect of CFRP strengthening on the ductility of reinforced concrete.

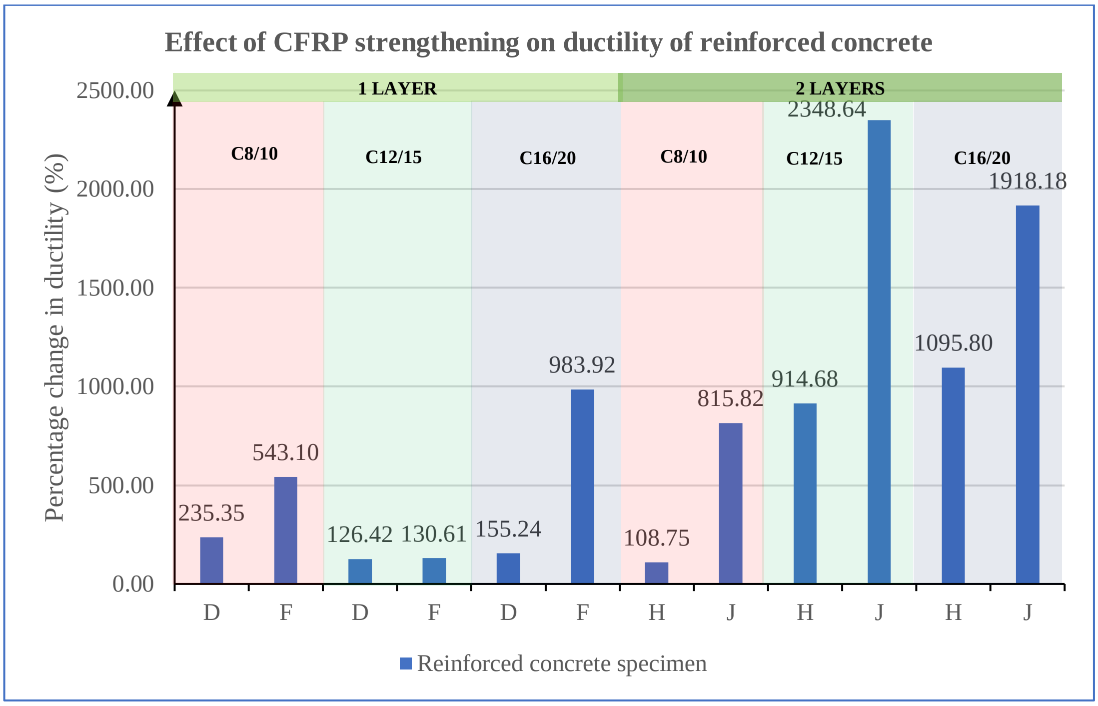


Figure 4.26: Effect of CFRP strengthening on the ductility of reinforced concrete columns.

It is evident again that CFRP caused an increase in ductility of the reinforced column specimens. On average, reinforced concrete samples had an increase of 718% in ductility. This change may be attributed to the fact that CFRP increased the confinement in concrete columns. Higher strengths and higher critical strains are obtained in confined concrete (CEN, 2004).

Comparison of the Effect of CFRP Strengthening on ductility between Plain Concrete and Reinforced Concrete

Figure 4.25 and Figure 4.26 were combined to visualise the effect of CFRP strengthening on plain concrete specimens and reinforced concrete as per the second research question that investigated the difference in CFRP strengthening between plain concrete specimens and reinforced concrete specimens. The outcome of the combination of the two charts was the chart shown in Figure 4.27.

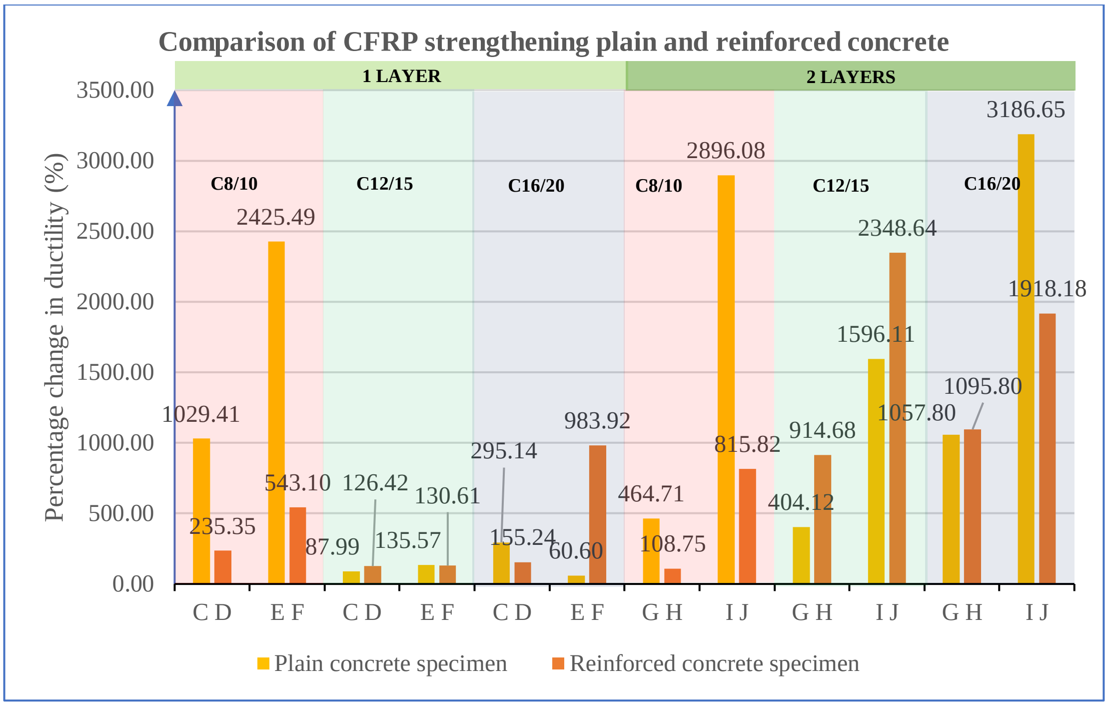


Figure 4.27: Comparison of change in ductility between plain and reinforced CFRP strengthened concrete specimens.

Mostly, the plain concrete specimens had higher ductility than the reinforced concrete samples. The less ductility in reinforced concrete samples can be attributed to the fact that rebar made the columns stiffer and therefore less ductile. Figure 4.27 shows that plain concrete specimens had higher increases in ductility; however, the gain in ductility between some plain and reinforced concrete samples with similar CFRP configuration, such as Specimens C, D, E and F of Concrete C12/15 with one CFRP layer was negligible. These differences may be attributed to material variability and human factors. Further analysis was done in the following subsection by comparing the reinforced specimens with CFRP strengthening against plain concrete specimens.

On site, it would be essential to find out the ductility of a column with both rebar and CFRP strengthening. This necessitates finding out how the two materials act together to affect the ductility of the columns.

4.3.2.2.5 *Combined Effect of Rebar and CFRP Strengthening on Ductility of Columns*

To determine the combined effect of both CFRP and rebar specimens, reinforced specimens with CFRP strengthening were compared with plain concrete specimen with no CFRP strengthening and no rebar, Specimen A. The comparison was made with Specimen A of the respective concrete grades and shown in Figure 4.28. The results were rearranged to analyse the trends of the column ductility based on rebar.

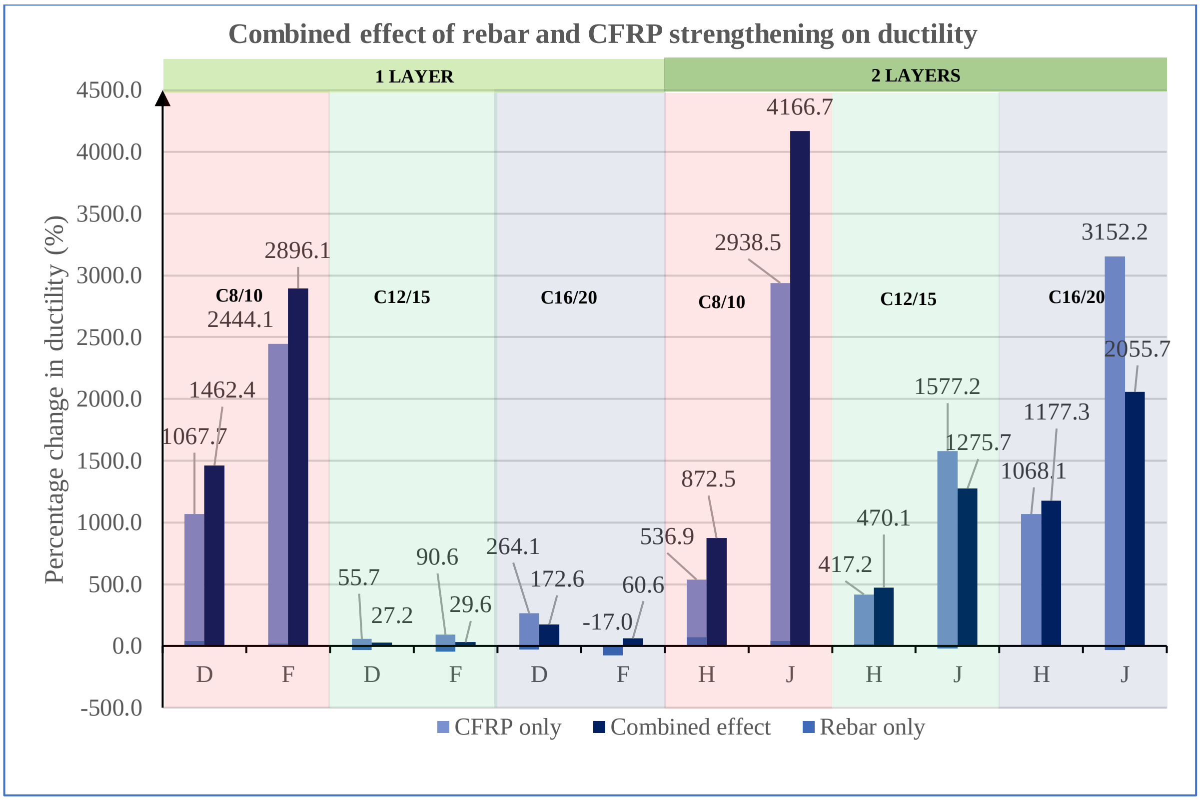


Figure 4.28: Combined effect of rebar and CFRP strengthening on ductility.

The side-by-side comparison shows that the two bar graphs follow a similar pattern. Averaging the values of the sum of gain in ductility due to individual effect of CFRP and the individual effect of rebar was 1132.95% while the average gain in ductility due to the combined effect of both rebar and CFRP strengthening was 1222.21%. The calculation of the values was as shown:

Individual effect of rebar on ductility + Individual effect of CFRP on ductility

$$Average = \frac{1067.7 + 2441.1 + 55.7 + 90.6 + 264.1 - 17.0 + 536.9 + 2938.5 + 417.2 + 1577.2 + 1068.1 + 3152.2}{12}$$

$$= 1132.95\%$$

Combined effect of rebar and CFRP on ductility

$$Average = \frac{1462.4 + 2896.1 + 27.2 + 29.6 + 172.6 + 60.6 + 872.5 + 4166.7 + 470.1 + 1275.7 + 1177.3 + 2055.7}{12}$$

$$=1222.21\%$$

From Figure 4.28, it is evident that the contribution of rebar to ductility is considerably less than the contribution of CFRP to ductility.

4.3.2.2.6 Summary of the Effect of Rebar on Ductility of CFRP Strengthened Columns

To further illustrate the effect of rebar on ductility in comparison with CFRP strengthening, the average gains in ductility are presented in Figure 4.29. Rebar causes a reduction in the effect of CFRP strengthening to increase ductility since it increases the stiffness of columns and therefore makes the columns less ductile.

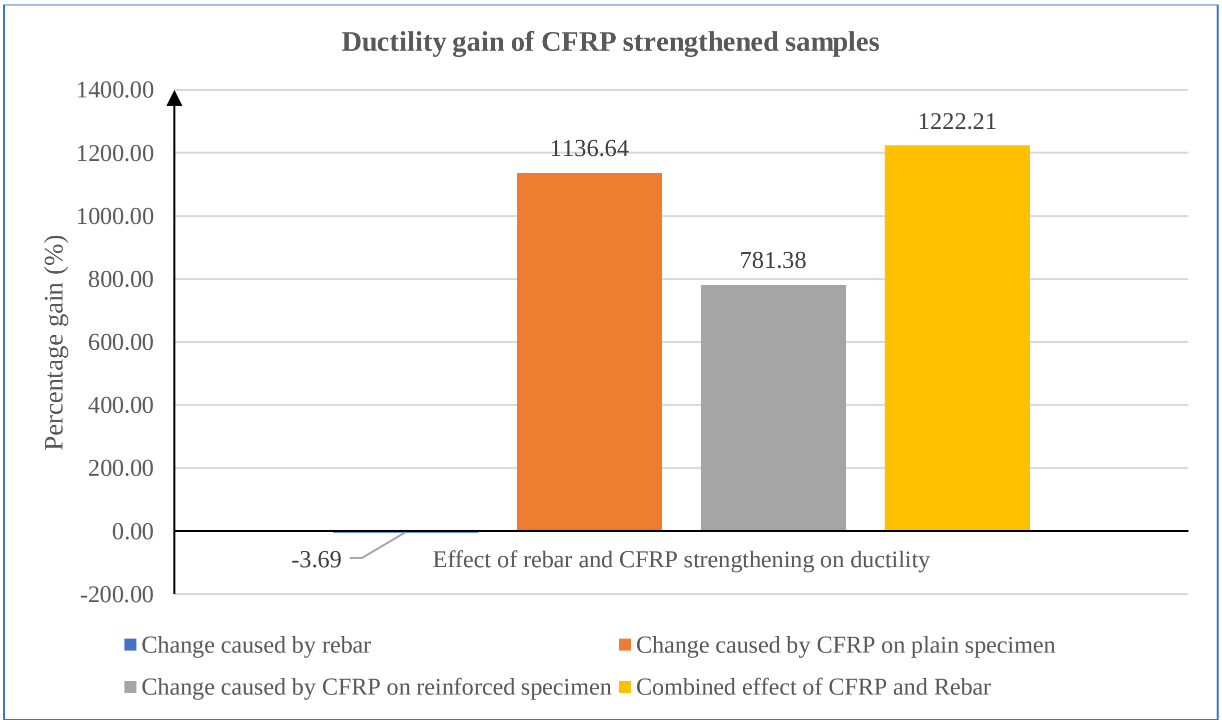


Figure 4.29: Summary of the effect of rebar and CFRP on the ductility of CFRP strengthened samples.

Basing our analysis on the results of the ductility shown in Figure 4.29, this study assumes that there is a difference between the effect of CFRP strengthening on plain concrete and reinforced concrete specimens. This finding is crucial as it is a rationale to determine the ductility of an existing reinforced column that has been retrofitted with CFRP. Finding out the amount of existing reinforcement would also be important.

The combined effect of CFRP and rebar was roughly comparable to the sum of the individual effect of rebar and the individual effect of CFRP strengthening.

Table 4.21: Comparison of the combined effect of rebar and CFRP on ductility.

Contribution	Change in ductility
Effect of rebar on CFRP strengthened columns + Effect of CFRP on plain concrete specimen	=-3.69+1136.64 = 1132.95%
Effect of rebar on CFRP strengthened columns + Effect of CFRP on reinforced concrete specimen	=-3.69+781.38 = 727.69%
Combined Effect of rebar and CFRP	=1222.21%

4.3.2.3 *Effect of Degree of CFRP confinement on Ductility of CFRP Strengthened Columns*

Using the sample data on ductility in Table D.1, the following means and standard deviations were calculated and are presented in Table 4.22.

Table 4.22: Descriptive statistics of ductility of sample of with different degrees of confinement.

Concrete Grade	Mean	Std. Deviation	N
Bare	0.003926745	0.0032238820	23
Partial	0.019753540	0.0129044841	36
Full	0.048191460	0.0363533166	36
Total	**0.026698265**	**0.0296441609**	**95**

ANOVA was carried out at the 95% confidence interval and a level of significance of 5%. The results are indicated in Table 4.23. The analysis of the sample data shows that the mean difference between the samples of partial CFRP and full CFRP confinement is significant at the 95% level of confidence.

Table 4.23: ANOVA table of ductility between groups with different degrees of confinement.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	0.030	2	0.015	26.638	0.000
Within Groups	0.052	92	0.001		
Total	**0.083**	**94**			

The ANOVA test $F(2,92) = 26.638, p < 0.001$, shows that treatments between the groups with different degrees of confinement are significantly different at the 95% confidence interval. Since the ANOVA test shows that treatments between the groups with varying degrees of confinement were significantly different, post hoc testing was done to find out which group was different from the others. The groups were of different sizes; 23 samples were bare; 36 samples had partial CFRP confinement and 36 samples had full CFRP confinement. The Gabriel method was used for post hoc testing for multiple comparisons within and between the groups as shown in Table 4.24.

Table 4.24: Post hoc testing of ductility of groups with different degrees of confinement.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig. (p)	95% Confidence Interval	
					Lower Bound	Upper Bound
Bare	Partial	-0.01582*	0.00636	0.042	-0.03120	-0.00045
	Full	-0.04426*	0.00636	0.000	-0.05963	-0.02889
Partial	Bare	0.01582*	0.00636	0.042	0.00045	0.03120
	Full	-0.02843*	0.00562	0.000	-0.04209	-0.01477
Full	Bare	0.04426*	0.00636	.0000	0.02889	0.05963
	Partial	0.02843*	0.00562	0.000	0.01477	0.04209
* The mean difference is significant at the 0.05 level.						

If a group has a significance value of p, of less than 0.05 then the group difference is statically significant. The post hoc testing shows that each group is different from each other significantly.

4.3.2.3.1 *Findings of Statistical Significance on Number of Layers on Ductility*

There was a statistically significant difference among the CFRP layers $F(2,92) = 26.638, p < 0.001$. Post hoc testing revealed significant differences between the groups with different treatment: bare columns $(M = 0.00393, SD = 0.00322)$ which had the least ductility, followed by partial CFRP confinement ($M = 0.01975, SD = 0.0129$) and finally full CFRP confinement ($M = 0.04819, SD = 0.03635$) which had the highest ductility. Since the difference between the groups was significant the means between the groups could then be compared.

4.3.2.3.2 Comparison of Single and Double CFRP Layers on Ductility

The results were rearranged and categorised to compare the effect of degree of CFRP confinement on the ductility of specimens as presented in Figure 4.30.

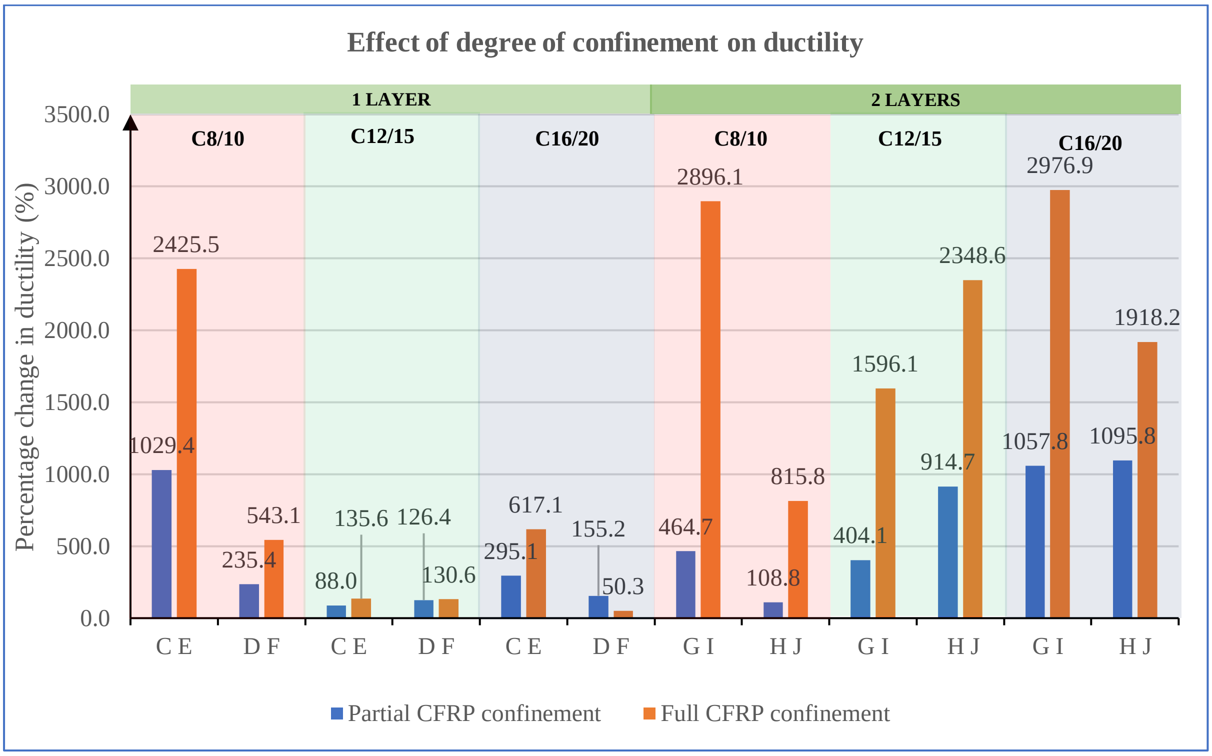


Figure 4.30: Effect of degree of CFRP confinement on ductility.

The results show that full CFRP confinement has a significant incremental change in the ductility of columns than partial CFRP confinement. It is only in Specimen F of Concrete C16/20 where partial CFRP confinement had a higher effect on ductility than full CFRP confinement. This result was attributed to a sample error. Since the sample size of this experimental study was small, taking the average of the various samples would assist in the visualisation of any trend.

4.3.2.3.3 Summary of the Effect of Degree of Confinement on Ductility of CFRP Strengthened Columns

The average was calculated and the results presented in the chart shown in Figure 4.31.

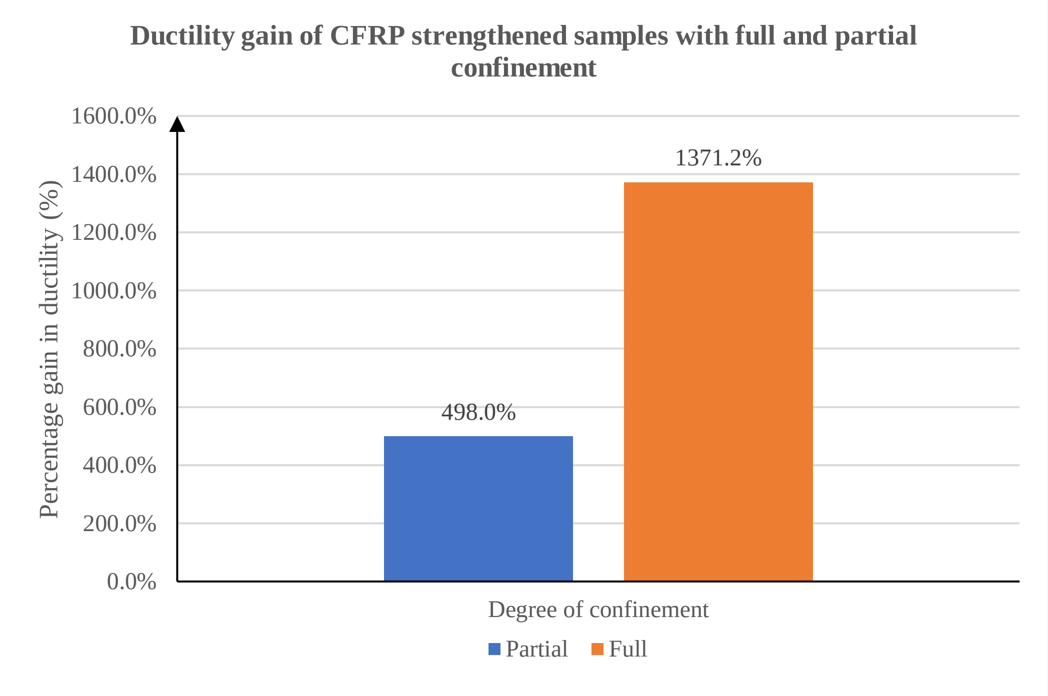


Figure 4.31: Summary of the effect of degree of CFRP confinement on the ductility of CFRP strengthened samples.

Specimens with partial CFRP confinement (Specimens C, D, G and H) had an average increase of 498% in ductility. In contrast, specimens with full CFRP confinement (Specimens E, F, I and J) had an average increase of 1371.2% in ductility. This finding disproves the claim that partial CFRP confinement has the same effect as full CFRP confinement in terms of ductility.

4.3.2.4 Effect of Number of layers on Ductility of CFRP Strengthened Columns

Using the sample data on ductility in Table D.1, the following means and standard deviations were calculated and presented in Table 4.25.

Table 4.25: Descriptive statistics of ductility of samples with different number of layers.

Number of Layers	Mean	Std. Deviation	N
None	0.003926745	0.0032238820	23
Single	0.017636873	0.0173092602	36
Double	0.050308127	0.0324874660	36
Total	**0.026698265**	**0.0296441609**	**95**

ANOVA was carried out at the 95% confidence interval and the results are as indicated in Table 4.26. The analysis of the sample data shows that the mean difference among the samples of no layer, a single layer and double layers is significant with a 95% level of confidence.

Table 4.26: ANOVA table of ductility of samples with different number of layers.

ANOVA	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	0.035	2	0.017	33.736	0.000
Within Groups	0.048	92	0.001		
Total	**0.083**	**94**			

The ANOVA test, $F(2,92) = 33.736, p < 0.001$, shows that treatments between the groups with different CFRP layers are significantly different at the 95% confidence interval. Since the ANOVA test shows that treatments between the groups with different number of layers were significantly different, post hoc testing was done to find out which group was different from the others. The groups were of different sizes; no layer: 23 samples, single layer: 36 samples and double layers: 36 samples; the Gabriel method was used for post hoc testing for multiple comparisons within and between the groups as shown in Table 4.27.

Table 4.27: Post hoc testing of ductility of groups with different number of layers.

Concrete Grade (I)	Concrete Grade (J)	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
					Lower Bound	Upper Bound
None	Single	-0.01371	0.00607	0.074	-0.02838	0.00096
	Double	-0.04638*	0.00607	0.000	-0.06105	-0.03170
Single	None	0.01371	0.00607	0.074	-0.00096	0.02838
	Double	-0.03267*	0.00536	0.000	-0.04570	-0.01963
Double	None	0.04638*	0.00607	0.000	0.03170	0.06105
	Single	0.03267*	0.00536	0.000	0.01963	0.04570

* The mean difference is significant at the 0.05 level.

If a group has a significance value p, of less than 0.05 then the group difference is statistically significant. The post hoc testing shows that each group is different from each other significantly except the comparison between group with no layer and the group with a single CFRP layer. Post hoc testing reveals that at the 5% level of significance, the ductility of columns with a single layer is not statistically significantly different from the ductility of columns with no

CFRP layer. Post hoc testing also reveals that at the 5% level of significance, the ductility of columns with two layers of CFRP is statistically significantly different from the ductility of columns with a single layer of CFRP and the ductility of columns with no CFRP strengthening.

4.3.2.4.1 Findings of Statistical Significance on Number of Layers on Ductility

There was a significant difference among the CFRP layers $F(2,92) = 33.736, p < 0.001$. Post hoc testing revealed statistically significant differences between the groups with different treatment: bare columns $(M = 0.003927, SD = 0.003224)$ which had the least ductility, followed by single CFRP layer $(M = 0.17637, SD = 0.017309)$ and finally full CFRP confinement $(M = 0.050308, SD = 0.032487)$ which had the highest ductility. Since the difference between the groups was significant the means between the groups could then be compared.

4.3.2.4.2 Comparison of Single and Double CFRP Layers on Ductility

The ductility of Specimen C, E, G and I were compared with Specimen A while that of Specimen D, F, H and J were compared with Specimen B. The results are thus presented in Figure 4.32.

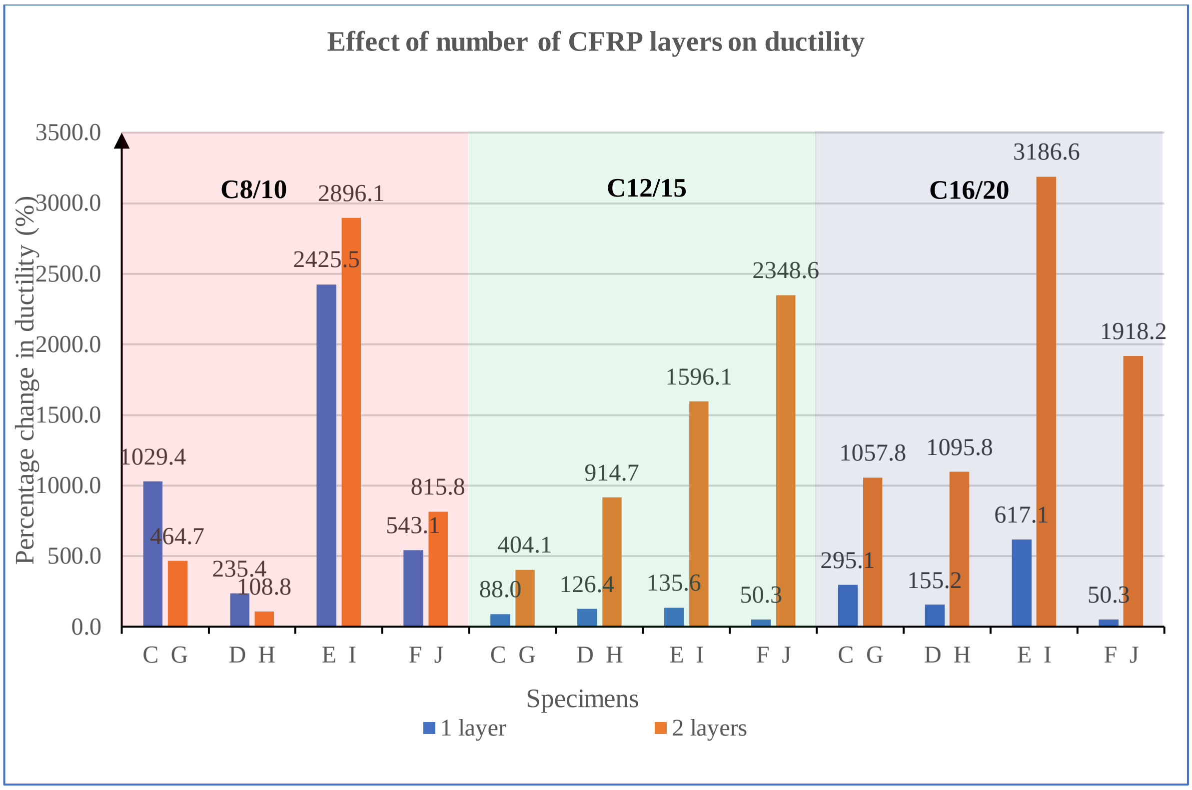


Figure 4.32: Effect of CFRP layers on ductility.

Figure 4.32 shows that the number of layers is directly proportional to the increase in ductility. The number of layers has a significant effect on ductility. The higher the number of layers increases the confining properties of the wraps and thus a higher modification of the stress-strain behaviour of the confined concrete (CEN, 2004).

4.3.2.4.3 Summary of the Effect of the Number of CFRP layers on the Ductility of CFRP strengthened Columns.

Figure 4.33 shows the comparison of single CFRP confinement and Double CFRP confinement.

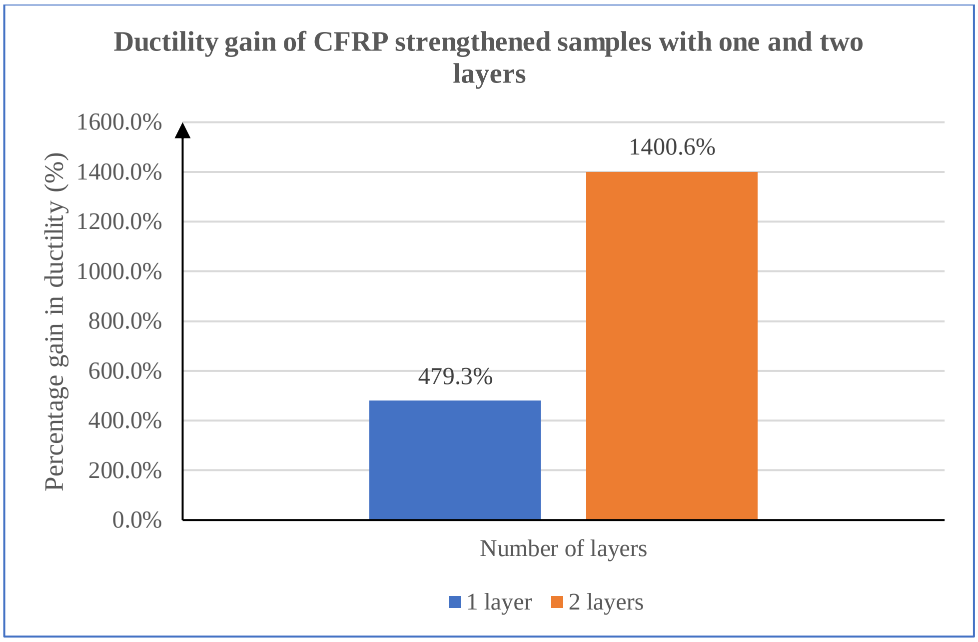


Figure 4.33: Summary of the effect of the number of CFRP layers on the ductility of CFRP strengthened samples.

On average, Specimens G, H, I and J which had two layers of CFRP confinement had a higher change in ductility (1400.6%) than Specimens C, D, E and F which had one layer of CFRP confinement (479.3%). This finding proves our fourth research hypothesis that stated that additional layers of CFRP wrap have a direct relationship with increment in the ductility of non-slender square concrete columns.

4.4 Summary of Experimental Results

The experimental results were summarised in the form of radar charts and treemaps. A radar chart is a graphical technique of visualising multivariate data in the form of a two-dimensional chart of three or more variables represented on axes starting from a single point in a radial manner. The angle and relative position of the axes is uninformative (Porter, 2018). In this case, for instance, the four independent variables are on the same radar chart, the line connecting the values in each axis does not mean that the effect varies linearly between the two. The radar chart has only been employed to reveal distinct correlations and differences. A treemap represents several data values as the area of rectangles. It is useful for visualising and representing hierarchical information (Microsoft Excel, 2019).

4.4.1 Impact of Concrete Grade, Rebar, Degree of Confinement and Number of Layers on Axial Capacity of Non-slender Square Concrete Columns

The impact of these four variables is presented as a radar chart in Figure 4.34.

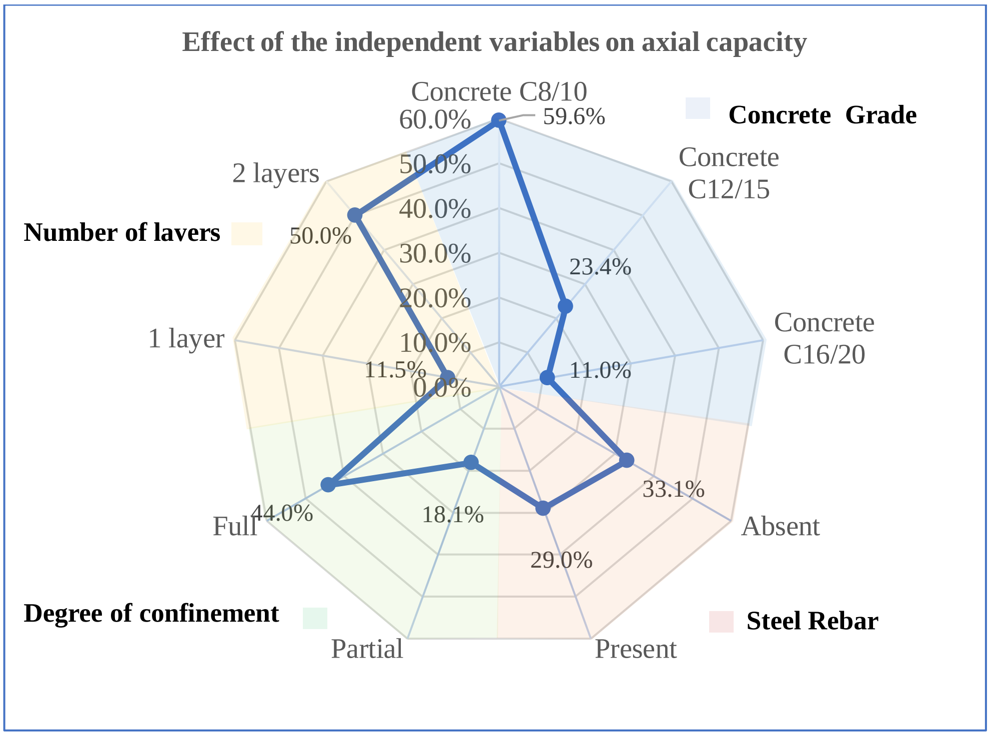


Figure 4.34: Radar chart showing average percentage effect of the variables on axial capacity.

For further clarification, the information has been presented by a treemap in Figure 4.35.

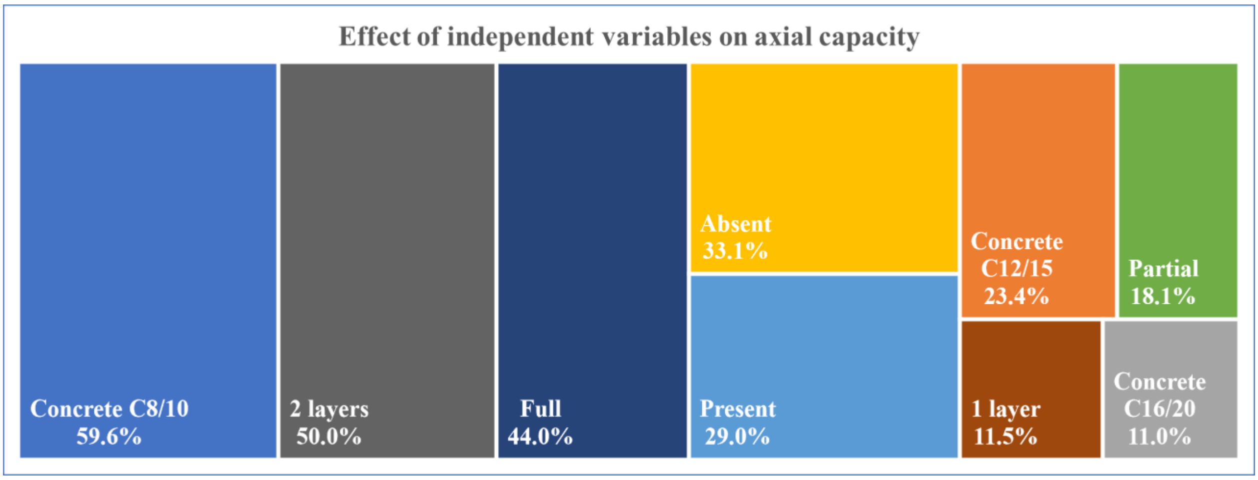


Figure 4.35: A treemap showing the effect of variables on axial capacity.

Variables with the greatest effect are placed top left while the variables with the least effect are placed bottom right of the treemap. Concrete C8/10 experienced the highest effect of CFRP strengthening, while Concrete C16/20 experienced the least effect.

4.4.2 Impact of Concrete Grade, Rebar, Degree of Confinement and Number of Layers on Ductility of Non-slender Square Concrete Columns

A radar chart was also presented to determine each of the variable's contribution to change in ductility.

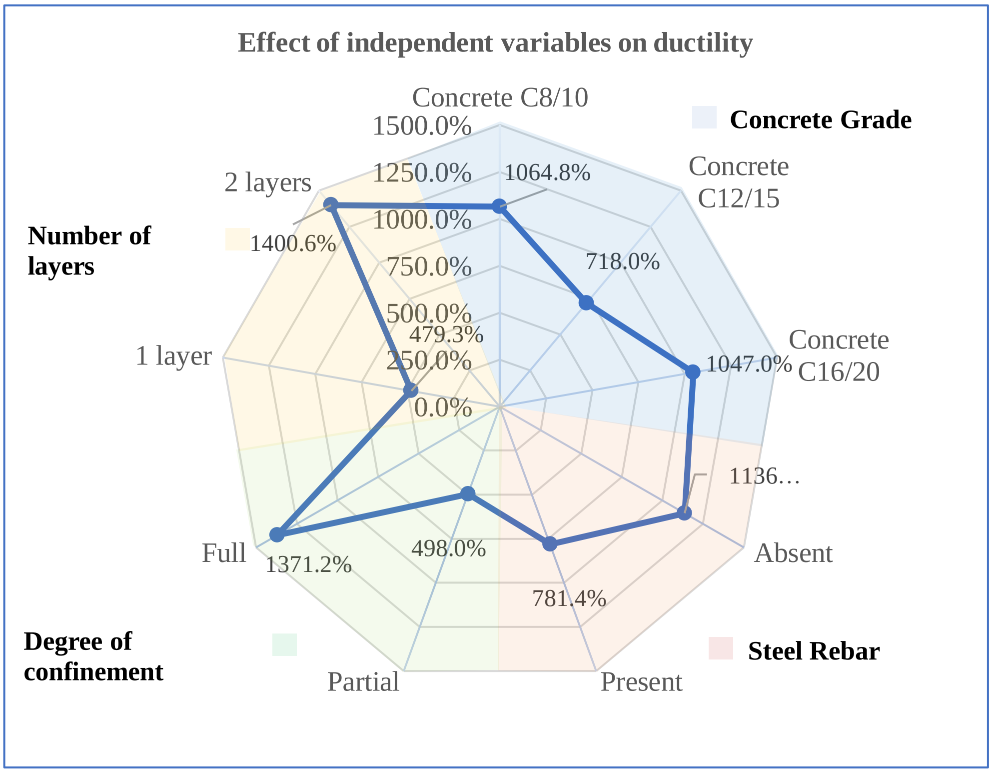


Figure 4.36: Radar chart showing average percentage effect of the variables on ductility.

Once again, the results are presented in a treemap for a more straightforward interpretation.

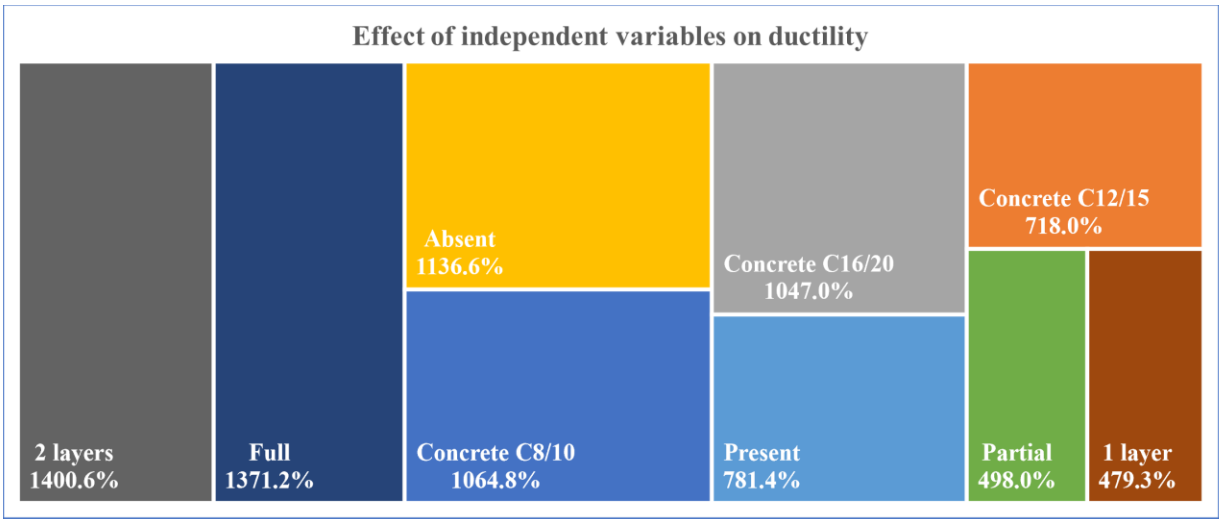


Figure 4.37: Treemap showing the effect of variables studied on ductility.

Having two layers of CFRP confinement had the greatest effect while having one layer of CFRP strengthening had the least effect of CFRP strengthening on average.

4.4.3 Comparison of Effectiveness within the Variables

There was a need to compare the effectiveness of the parameters of the four independent variables. The columns that had the least change in axial capacity and ductility were used as the benchmarks. For instance, to find out the difference in effectiveness between one layer (11.5%) and two layers (50%) in axial capacity, the following calculation was made:

$$Change\ in\ effectiveness = \frac{Value\ of\ parameter}{Least\ value\ of\ parameter} \quad \text{Equation 4.5}$$

$$Change\ in\ effectiveness = \frac{50\%}{11.5\%} = 4.35$$

Following this calculation, it can be stated that using two layers of CFRP is 4.35 times more effective than using one layer. Equation 4.5 was then used to compute the change in effectiveness and the results are presented in Table 4.28.

Table 4.28: Comparison of effectiveness within the variables.

Independent variables		Axial capacity	Effectiveness	Ductility	Effectiveness
Grade of concrete	C8/10	59.6%	5.42	1064.8%	1.48
	C12/15	23.4%	2.13	718.0%	**1.00**
	C16/20	11.0%	**1.00**	1047.0%	1.46
Steel Rebar	Absent	33.1%	1.14	1136.6%	1.45
	Present	29.0%	**1.00**	781.4%	**1.00**
Degree of confinement	Partial	18.1%	**1.00**	498.0%	**1.00**
	Fully	44.0%	3.83	1371.2%	2.75
Number of layers	1 layer	11.5%	**1.00**	479.3%	**1.00**
	2 layers	50.0%	4.35	1400.6%	2.92

Following the calculation in Table 4.28, the following comparisons could be drawn: Concrete C8/10 was 5.42 times more affected than Concrete C16/20 in terms of axial capacity (542%) but the effect was comparable in terms of ductility. Plain concrete specimens experienced a higher effect in both axial capacity (114%) and ductility (145%) than reinforced concrete specimens. Full confinement was more effective in both the axial capacity (383%) and ductility (275%). Similarly, using two layers was more effective in both axial capacity (435%) and ductility (292%) than one layer.

4.5 Failure of the Specimen Under Axial Loading

4.5.1 Theory of Failure

The use of different failure criterion is usually appropriate for different materials. For example, the maximum principal stress/strain theory is appropriate for brittle materials like cast iron or glass. On the other hand, the failure of ductile materials such as steel is best represented by the Von-Mises criteria. The Mohr-Coulomb criterion would work well for material such as concrete or soil. The CFRP wrap facilitated the concrete column to be subjected to a three-dimensional state of stress and therefore affected the mode of failure and seemed to follow the Mohr-Coulomb criterion of failure.

4.5.2 Modes of failure

All the columns failed by crushing as predicted by the calculations. The failure occurred slowly with the development of vertical cracks before the column gave way. The failure of the columns strengthened by CFRP was characterised by sudden and explosive fracture as the CFRP wrap failed. The more the number of layers, the more explosive the failure. The concrete grade had little effect on the type of failure mode. All specimens with similar rebar and CFRP configuration but of different concrete grades exhibited similar failure modes. Damage on the CFRP strengthening was typically localised at the corners.

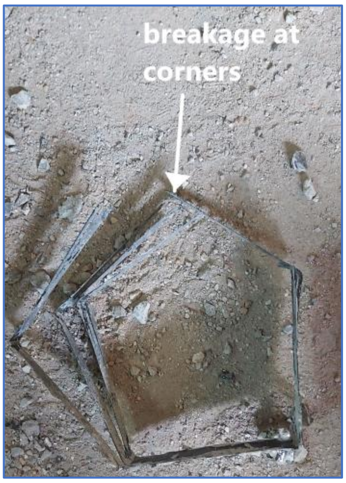


Figure 4.38: Failure was characterised by rupture at the corners.

Stress-raisers are sharp corners, grooves, notches or acute changes of the section that cause stress concentrations under typical loadings (Matthew, 1998). They can be found on both rotating and static components. The stress concentration factors of sharp corners and grooves are high and difficult to determine accurately. Components that have failed predominantly by a fatigue mechanism are nearly always found to exhibit a crack initiation point – a sharp feature at which the crack has started and then progressed to failure. It was expected that failure would occur at this point due to stress concentrations (Rochette & Labossière, 2000). This informs the decision to reduce stress raisers by use of rounding the edges and smoothening the surface (Campbell, 2015). Engineers should strive to ensure that sharp edges are rounded before the application of CFRP wrap.

Specimens with CFRP strengthening tended to have cracks develop later than columns without CFRP wrap. Fewer cracks are attributed to the fact that the CFRP wrap held the concrete core together, thus reducing the lateral expansion due to Poisson's effect. This is an important observation as visible cracks imply a serviceability limit state has been reached. In addition to that, cracks reduce the effective cross-sectional area of a column. Limiting the development of cracks, thus conserves the effective cross-sectional area.

The failure was mostly located at the centre as expected. The literature reviewed (BSI, 2019; Neville & Brooks, 2010) showed that the centre of the specimen had the least effect of platen restraint of the compression test machine. The platen restraint occurs when there is some lateral compression caused by friction between the steel platen and the specimen being tested. This has the effect of increasing the compression capacity of the material in contact with the steel platen. Figure 4.39 shows typical failure patterns of the specimens tested.

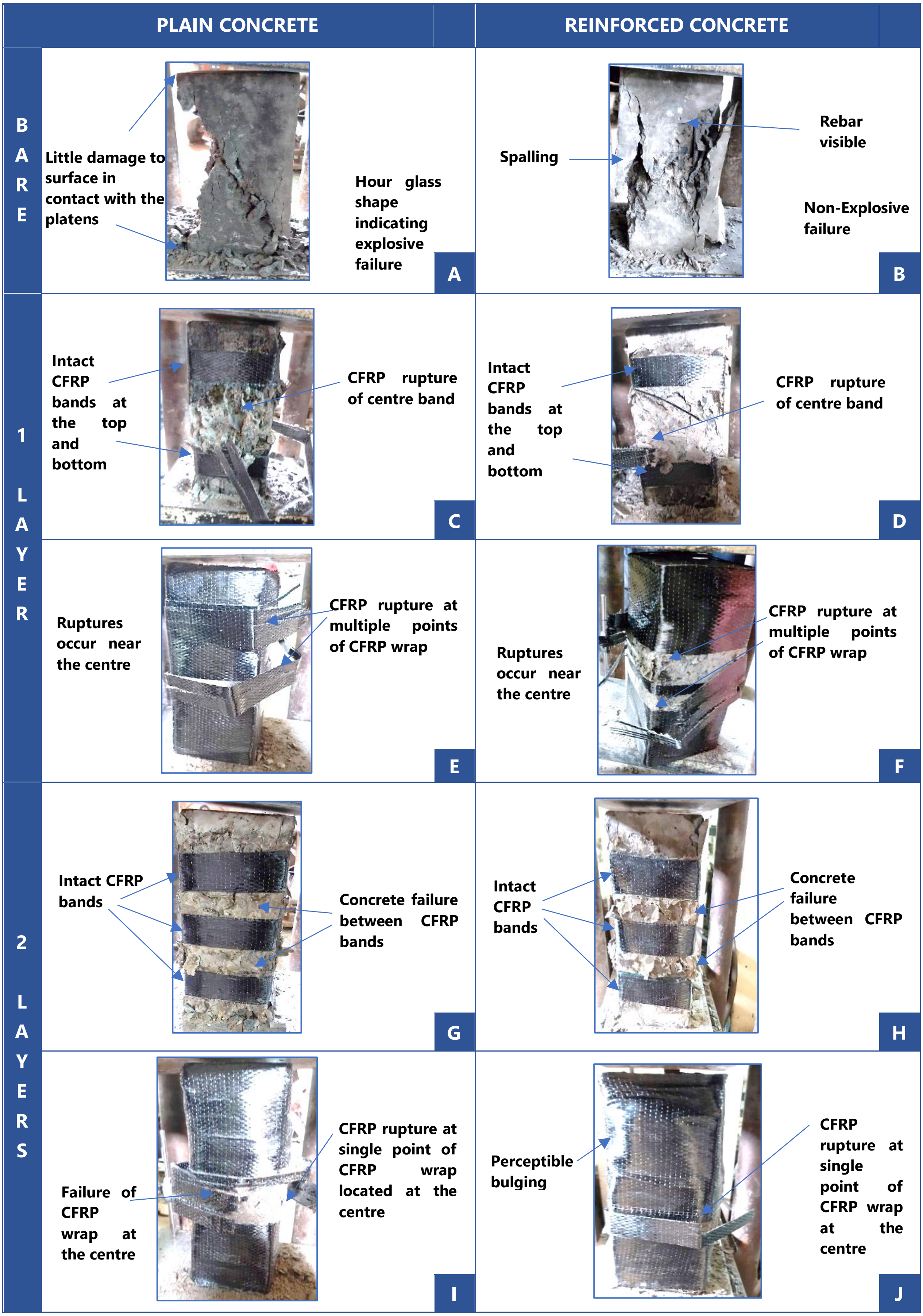


Figure 4.39: Typical failure modes of the specimens.

Specimen A had no rebar and failure occurred immediately after visible cracks appeared on the surface of the column. The failure of this specimen was explosive. It was characterised by large chunks of concrete falling off the column core. There was little damage on the surface in contact with the platens. This was because of the platen restrain (Neville & Brooks, 2010).

Specimen B had rebar, and thus as the failure occurred, the rebar retained the shape of the column, unlike Specimen A where rebar was absent. The failure was non-explosive. The rebar provided some confinement that prevented the column from crumbling. The failure of this specimen was characterised by the concrete cover surrounding the rebar cage spalling off. The effect of rebar on plain concrete is that failure occurs more gradually. This phenomenon is one of the reasons nominal reinforcement is provided in all engineered concrete buildings (McKenzie, 2013).

Specimen C with partial confinement and one layer of CFRP configuration. The failure occurred as visible vertical cracks appeared between the bands followed by the sudden rupture of the middle CFRP band. The vertical cracks indicate the failure of the concrete core once the axial capacity of the concrete was reached. However, because the CFRP bands held the concrete core in place, the column retained shape as the uniaxial load was increased until the middle band gave way. The failure of the middle band is because that is the position that experienced the largest lateral expansion, coupled with the least platen restraint (Neville & Brooks, 2010). This finding is important in that engineers should inspect the centre of retrofitted columns to ensure it is correctly installed since it is the most likely position where failure occurs.

Specimen D had rebar and one layer of partial CFRP confinement. Effect of CFRP strengthening on this sample was compared with Specimen B to determine the effect of the CFRP. Failure was characterised by the sudden rupture of the middle CFRP bands. The cracks visible between the bands where the concrete surface was still visible were smaller than in

Specimen C. The smaller cracks can be attributed to the steel reinforcement resisting the tensile forces developing in the column as the columns expand laterally. This finding is crucial; engineers should find out the existing rebar in columns when retrofitting columns with CFRP.

Specimen E had one layer of full CFRP confinement and no rebar. Failure was characterised by rupture of the CFRP wrap. Failures occurred at multiple points along the wrap near the centre of the column. The location of the failure is explained by the fact that the middle part of the specimen experiences the least platen restraint hence most susceptible to failure (Neville & Brooks, 2010).

Specimen F has one layer of full CFRP confinement and rebar. Failure was characterised by large ruptures or tears of CFRP wrap similar to Specimen E or small ruptures at multiple points on the corners of the specimens. Since Specimens E and F were entirely obscured by the CFRP wrap, the development of cracks as the specimens were loaded to failure could not be observed. The failure of the specimen at the corners was due to stress concentrations at the corners of the samples (Campbell, 2015).

Specimen G had two layers of partial CFRP confinement with two layers and without rebar. Failure was characterised by the failure of the concrete between the CFRP bands. Continuous loading to achieve the failure of the CFRP bands was not possible as it was clear that the concrete had already failed; chunks of concrete fell off from the space between the bands. Reason being that the lateral expansion of the concrete between the bands was unconfined, resulting in the development of tension in the concrete and the eventual failure of the specimen. In addition to that, the weak grades of concrete: Concrete C8/10, Concrete C12/15 and Concrete C16/20 used in this experimental programme was another reason the concrete gave way before the CFRP. The CFRP bands remained intact unlike Specimens C and D; the intactness of the bands is attributed to the additional layer of CFRP.

Specimen H was reinforced and had two layers of partial CFRP confinement. Failure characterisation was similar to that of Specimen G. The CFRP bands remained intact after the test. This was attributed to the additional layer of CFRP when compared with Specimens C and D, where the CFRP bands failed. The concrete core developed cracks and failed. This failure mode where the concrete between the bands falls off might be another disadvantage of using partial CFRP confinement that had not been discussed in previous studies (Abadi et al., 2019; Guo et al., 2018; Guo & Zeng, 2019; The Concrete Society, 2000). Engineers should leverage this knowledge when retrofitting columns to ensure the serviceability limit is not exceeded.

Specimen I consisted of plain concrete with two layers of full CFRP confinement. Failure was characterised by explosive rupture of the CFRP at a localised point, typically near the middle region. The failure occurred at the middle because it experiences the least platen restraint (Neville & Brooks, 2010). There was visible bulging of the specimen, indicating a serviceability failure before the ultimate failure occurred. The bulging also indicates that the concrete is no longer a continuous body; it has disintegrated, and it is only held by the CFRP wrap.

Specimen J had steel reinforcement and two layers of full CFRP confinement. The failure occurred explosively with a distinct pop sound. Comparison of failure with Specimen F that had rebar and similar CFRP configuration apart from the number of layers was that failure was localised at a point, unlike Specimen F that had ruptured at multiple points. The localised failure occurred at the corners of the specimen and is attributed to the formation of a stress raiser (Campbell, 2015). There was visible bulging similar to Specimen I indicating a serviceability failure preceding the rupture of the CFRP wrap.

4.6 Summary of Hypothesis Testing

The research hypotheses were tested before further comparisons between the groups were drawn in the preceding sections. The summary of the hypothesis tests for both the axial capacity and ductility performed earlier are summarised in Table 4.29.

Table 4.29: Summary of decisions for the hypotheses tested.

Null Hypothesis	Decision
There is no relationship between concrete grades and the change in axial capacity and ductility of CFRP strengthened columns. $\mathbf{H_0}: \boldsymbol{\mu_{C8/10} = \mu_{C12/15} = \mu_{C16/20}}$	Reject H_0 for axial capacity. Failed to reject for ductility.
There is no significant difference between CFRP strengthening on plain and reinforced concrete in terms of axial capacity and ductility of non-slender square columns. $\mathbf{H_0}: \boldsymbol{\mu_{absent} = \mu_{present}}$	Failed to reject H_0 for axial capacity. Failed to reject H_0 for ductility.
There is no significant difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns. $\mathbf{H_0}: \boldsymbol{\mu_{bare} = \mu_{partial} = \mu_{full}}$	Reject H_0 for axial capacity. Reject H_0 for ductility.
There is no significant difference between single layer CFRP confinement and double layer CFRP confinement on axial capacity and ductility of non-slender square concrete columns. $\mathbf{H_0}: \boldsymbol{\mu_{none} = \mu_{single} = \mu_{double}}$	Reject H_0 for axial capacity. Reject H_0 for ductility.

From the hypothesis tests and the experimental investigations, the study found:

1. There is an inverse relationship between concrete grades and the change in axial capacity of CFRP strengthened columns.
2. There is no significant difference between CFRP strengthening on plain and reinforced concrete in terms of axial capacity and ductility of non-slender square columns.
3. There is a significant difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns.

4. Additional layers of CFRP wrap have a direct relationship with the increment on axial capacity and ductility of non-slender square concrete columns.

4.7 Findings of the Effect of Concrete Grade, Rebar, Degree of Confinement and Number of Layers on Axial Capacity and Ductility

Concrete C16/20 experienced the least effect. Stronger concretes are brittle; hence they do not dilate as much as weaker concretes. Less dilation contributes to less confinement pressure that increases the axial capacity and ductility of the square concrete columns. The weaker the concrete grade, the greater the lateral expansion of the concrete. This lateral expansion increases the confinement pressure. The higher the confinement pressure, the higher the effect of CFRP strengthening on axial capacity and ductility. The first research hypothesis that stated that there is an inverse relationship between concrete grades and the change in axial capacity and ductility of the columns strengthened by CFRP was accepted.

The effect of the different configurations of CFRP strengthening was approximately equal in both plain concrete specimen and reinforced concrete specimens. Steel reinforcement adds to the confining effects of the column by making the column more robust. It was found that the presence of steel reinforcement contributes to an increase in axial capacity and ductility of columns, especially in weaker concretes. However, steel reinforcement reduces the effectiveness of CFRP strengthening. When reinforced concrete specimens were compared with plain concrete specimens, the presence of steel reinforcement had a negative outcome on the effect of CFRP strengthening on the axial capacity and ductility. The study attributes the confining effect of rebar to the loss of CFRP effectiveness in increasing the axial capacity. In terms of ductility, steel rebar increases the stiffness of columns, thus causing making the columns less ductile than columns without rebar. The paradox in this is that steel increases the axial capacity and ductility of columns, whereas steel negatively affects the performance of CFRP in increasing the axial capacity and ductility of columns. The effect of the similar configurations of CFRP strengthening was approximately equal in both plain concrete specimen and reinforced concrete specimens hence the detrimental effect of steel rebar to CFRP strengthening is not large. The second hypothesis that stated there is a significant difference between CFRP strengthening on plain and reinforced concrete in terms of axial capacity and ductility of non-slender square concrete columns failed to be accepted.

Partial CFRP confinement offers an increase in axial capacity and ductility, although not as much as full CFRP confinement. Partial confinement and full confinement all offered increases in both axial capacity and ductility. The implied claim by the manufacturer that partial confinement is equivalent to full confinement has thus been disproved by the findings of this study (Horse Construction, 2019). The third hypothesis that stated there is a significant difference between partial CFRP confinement and full CFRP confinement on axial capacity and ductility of non-slender square concrete columns was accepted.

The number of CFRP layers has the effect of increasing the change in axial capacity and ductility of non-slender concrete columns. This phenomenon was consistent throughout the study. This is due to the fact that an increase in the number of layers increases the amount of confining properties of CFRP. More CFRP material can withstand the deformation of the columns as the columns are loaded. The fourth hypothesis that stated, additional layers of CFRP wrap have a direct relationship with the increment on axial capacity and ductility of non-slender square concrete columns was accepted.

4.8 Material Efficiency of Partial CFRP Confinement

Based on observation of the treemaps in Figure 4.35 and Figure 4.37, the number of layers was the extrinsic variable that had the greatest impact on both the axial capacity and ductility while the degree of confinement was the extrinsic variable with the least impact. With this finding, the need to compare single layer full confinement and double layer partial layer confinement arose. The experimental design was thereby optimised in order to find more information from the same number of tests already conducted (Poole & Poole, 2012). It was interesting to note that when the same number of layers were used, partial CFRP confinement configuration used in this experimental programme used 57.2% less material than full CFRP confinement. From the summary of the experimental results, it has been shown that the number of layers has more impact than the degree of confinement. With this realisation, the study compared the specimens with two layers of partial CFRP confinement against specimens with one layer of full CFRP confinement, as shown in Figure 4.40.

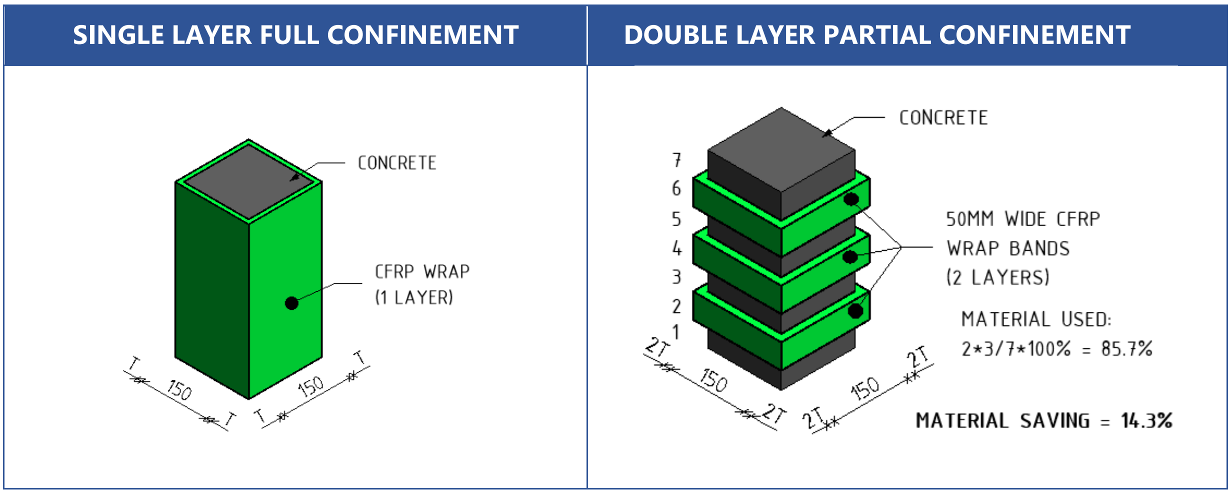


Figure 4.40: Material saving of double confinement over single full confinement.

When two layers of partial CFRP confinement is compared with one layer of full CFRP confinement, there is a material saving of about 14.3%. The axial capacity and ductility were compared in Figure 4.41 and Figure 4.42 respectively.

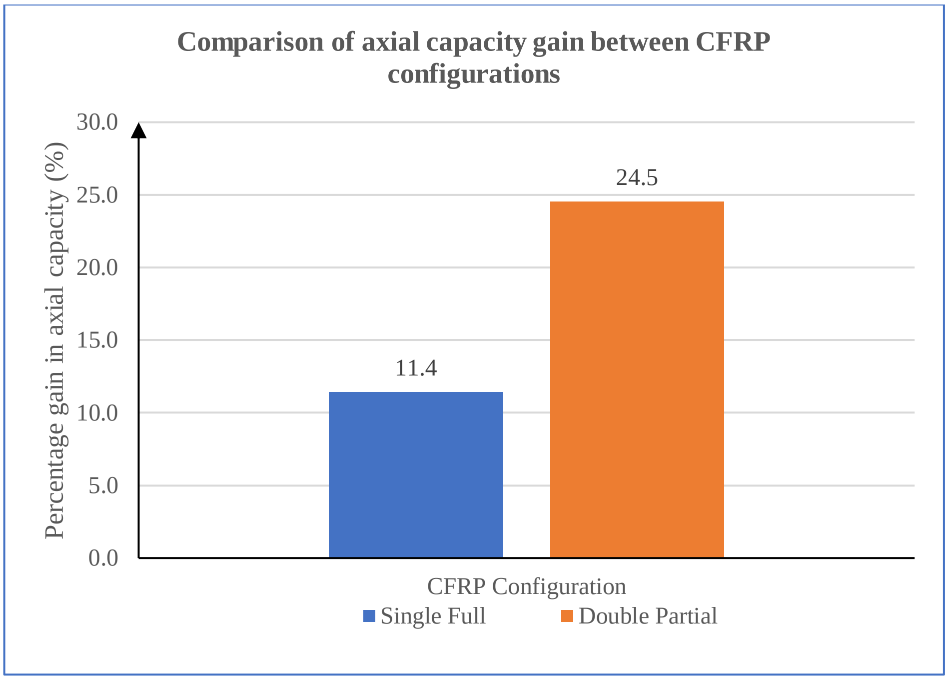


Figure 4.41: Comparison of axial capacity gain between one layer of full CFRP confinement and two layers of partial CFRP confinement.

Specimens with two layers of partial CFRP confinement had an average increase in axial capacity of 24.5% while specimens with one layer of full CFRP confinement had an average increase of 11.4%. This represents a 114.9% difference in effectiveness. The increase in effectiveness is attributed to the fact that two layers of CFRP wrap could resist more lateral deformation resulting in a higher axial capacity.

A similar comparison for ductility was performed and the results presented in Figure 4.42.

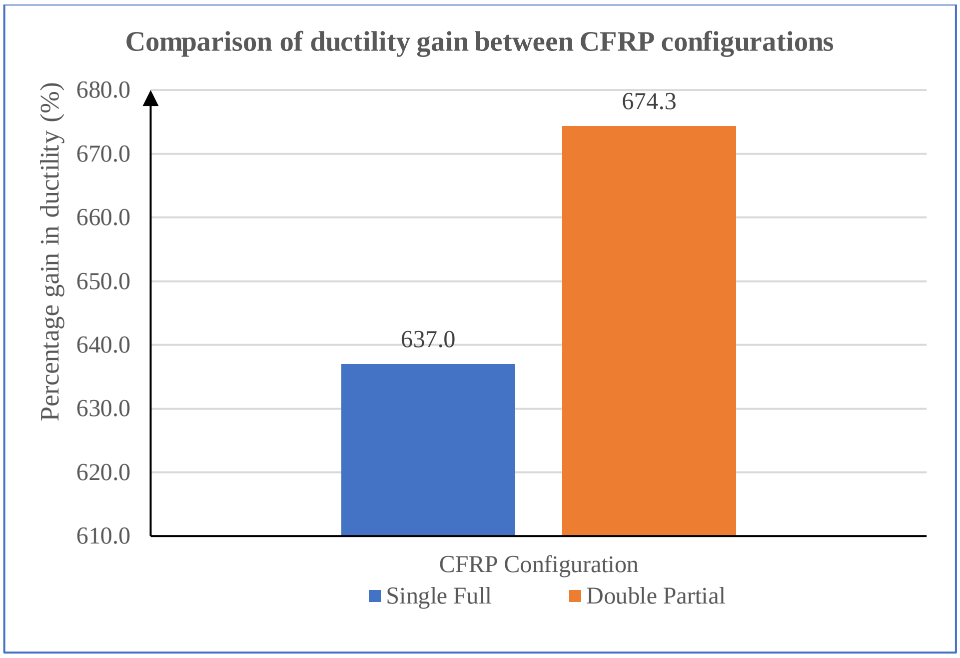


Figure 4.42: Comparison of ductility gain between one layer of full CFRP confinement and two layers of partial CFRP confinement.

Single full CFRP confinement had a 637% increase in ductility on average while double partial CFRP configuration had 674.3% increase in ductility. This represented a 5.8% change. This finding is important as engineers know that it is more effective to use two layers of CFRP in partial confinement than one layer of full confinement. With this argument, the claim by the manufacturer, as shown in Figure 2.3, that partial confinement offers similar benefits to full confinement can thus be said to have some basis (Horse Construction, 2019).

4.9 Contribution to Knowledge

The behaviour of composite confined concrete is difficult to predict because of the high number of variables affecting confined concrete (Shehata et al., 2002). This study had found that there was little research with regards to the concrete strength grades, presence of steel reinforcement and the degree of confinement. This study has contributed to new findings that assist in determining the relationship of four parameters that affect confined concrete.

The study has confirmed the effect of concrete grade on the performance of CFRP strengthening. CFRP strengthening is very effective in weaker concrete grades both in axial capacity and ductility. However, the effectiveness of CFRP strengthening in terms of axial capacity alone reduces with higher strength concrete grades. This study has found that even when dealing with high strength concretes the gains in ductility are still very high. CFRP could thus be used in seismic retrofitting even for stronger concrete grades since it is the ductility that is necessary in the energy dissipation that occurs during a seismic event such as an earthquake. The study does not recommend the use of CFRP wrap when dealing with high strength concrete grades and where significant increases in axial capacity are required.

Previous research had established as a fact that CFRP thickness is correlated to the increase in the effectiveness of CFRP strengthening (Guo et al., 2018; Parvin & Jamwal, 2004; Parvin & Wang, 2001; Saafi et al., 1999; Shrive et al., 2003). The study has also found that of the four variables investigated, the number of layers of CFRP has the highest impact on the strength and ductility of the strengthened columns. The degree of confinement did not have such a high impact on the performance of CFRP strengthening when compared with the number of layers. Combining this new knowledge with known facts and thus using partial CFRP confinement with two layers offers better material efficiency and economy as compared to full CFRP confinement with one layer.

4.10 Challenges Experienced in the Study

Some of the challenges experienced in this investigation that could have affected the reliability of the study are:

1. The hoop strain would be a better measure of ductility as it would have been more accurate in determining the passive confinement pressure experienced by the column being loaded. Unfortunately, the instrumentation necessary to measure hoop strain were unavailable.
2. CFRP in this investigation was applied before the columns were loaded. However, if CFRP strengthening were to be used to retrofit buildings, then the column would

already have been loaded and undergone some deformation before CFRP was installed. Applying the CFRP to columns after this deformation has occurred results in less confinement pressure generated on the CFRP and thus less effectiveness of the CFRP strengthening as compared to this study (Arduini et al., 1999).

3. Human factors such as fatigue when manufacturing the specimen could have affected the uniformity of the samples. For example, when wrapping the specimen with CFRP bands, the orientation of the fibres may not be completely horizontal in some bands. The skewed orientation of the fibres in some specimen might have affected the accuracy of the results.

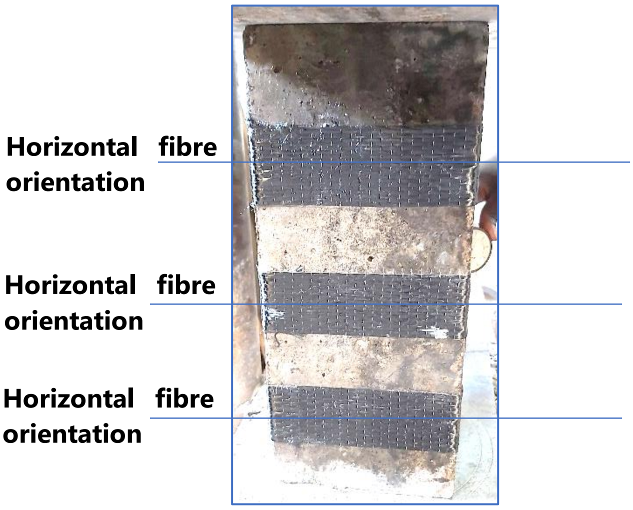


Figure 4.43: Specimen with correct fibre orientation.

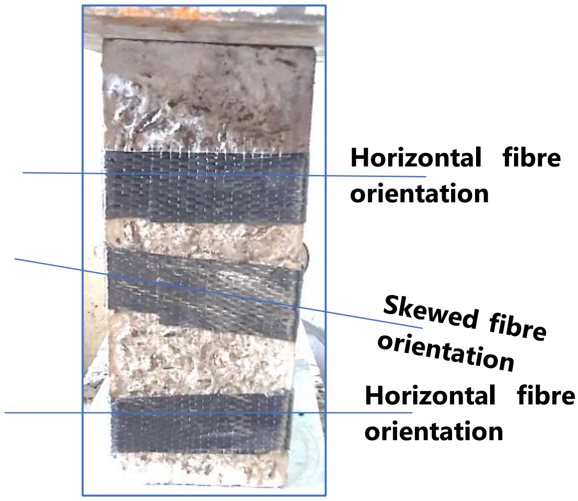


Figure 4.44: Specimen with skewed fibre orientation.

4. It was impossible to monitor the development of cracks in the specimens with full CFRP confinement since the CFRP wrap obscured the surface of the concrete.

5. Due to the large number of samples and the size of the specimens, the manufacture of concrete had to be done in different batches and on different days. Quality assurance and control measures utilised were: sourcing material from one location; using a riffle sample divider to select coarse aggregate, and quartering to select the concrete. Even though these quality control procedures were employed, it is assumed that some batches had differences that affected the homogeneity of the different samples cast.

6. The manual method of collecting data that relied on three individuals affected the accuracy of the data since instruments such as linear variable differential transformers would have made plotting the stress-strain graphs more accurate. This challenge was solved by conducting test statistics to ensure the difference is not due to random variation between the means of the groups. The level of significance α was set at 0.05. The precise data from this equipment would also facilitate generation of models that predict the failure mechanisms.

4.11 Verification of the Experimental Methodology

To verify the results of this experimental programme, the stress-strain graphs obtained in this study were compared with other stress-strain curves in literature. It was found that the graphs of the curves of with full CFRP confinement were similar to other stress-strain curves in literature reviewed (CEN, 2004). The results were also compared with the results of other studies. The findings made in this investigation are consistent with similar studies (Guo et al., 2018; Wang et al., 2019).

In addition to that, random samples were manufactured at a later date, tested, and the values plotted on charts. The stress-strain graphs of the random specimens tested had a high similarity with the initial specimens tested shown in Appendix G.

The data used in this study is reliable since the equipment used had been calibrated within a year of doing the tests. Furthermore, the use of control samples improved the validity of this experimental procedure as it was a comparative study. The failure modes of the specimens, shown in Figure 4.39, indicate satisfactory failure as described in BS EN 12390-3:2019. This phenomenon eliminates the existence of a probable fault in the testing equipment (BSI, 2019; Neville & Brooks, 2010).

From the analysis and discussions; and with the experimental methodology verified, this investigation drew several conclusions presented in the next chapter.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Ninety-five column samples with varying configurations were tested to failure, and it was found that CFRP strengthening increased the axial capacity and ductility of non-slender square concrete columns. Based on the experimental evidence, the findings of the effect of the variables investigated on CFRP strengthened columns are as follows:

1. The grade of concrete has an inverse effect on CFRP strengthening in terms of axial capacity and ductility.
2. Steel reinforcement decreases the effect of CFRP strengthening on the axial capacity and ductility of non-slender concrete columns.
3. Partial CFRP confinement offers an increase in axial capacity and ductility, although to a lesser extent than full CFRP confinement.
4. Additional number of layers increases the effect of CFRP strengthening on axial capacity and ductility.

5.2 Recommendations

This study makes the following recommendations:

5.2.1 Recommendations from this study

1. The combined effect of rebar and CFRP on the axial capacity and ductility can be approximated as the sum of the individual effect of rebar and the individual effect of CFRP strengthening, as shown by Equation 5.1.

$$\begin{matrix} \textit{The combined effect of} \\ \textit{rebar and CFRP} \end{matrix} = \begin{matrix} (\textit{Individual effect of Rebar} + \\ \textit{Individual effect of CFRP}) \end{matrix} \qquad \text{Equation 5.1}$$

2. Partial CFRP confinement has greater material efficiency than full CFRP confinement. Using two layers of partial CFRP confinement is more effective than using one layer

of full CFRP confinement in terms of material and the gain in axial capacity and ductility.

3. A design proposal is recommended in Appendix F.

5.2.2 Recommendations for further research

1. Further research should be performed to determine the effect of different widths of the bands and at different spacings.
2. This study recommends that further studies should adopt the use of smaller specimens to reduce the cost of the experimental programme.
3. Tests should be performed on multiple specimens, approximately 1000 specimen to increase the accuracy of models developed.
4. A numerical analysis should be performed to determine the effect of CFRP accurately. Using computers will minimise human factors that affect the performance of CFRP. Such human factors include the incorrect placement of the orientation of the CFRP fibres.

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APPENDICES

Appendix A: Mould Fabrication Drawings

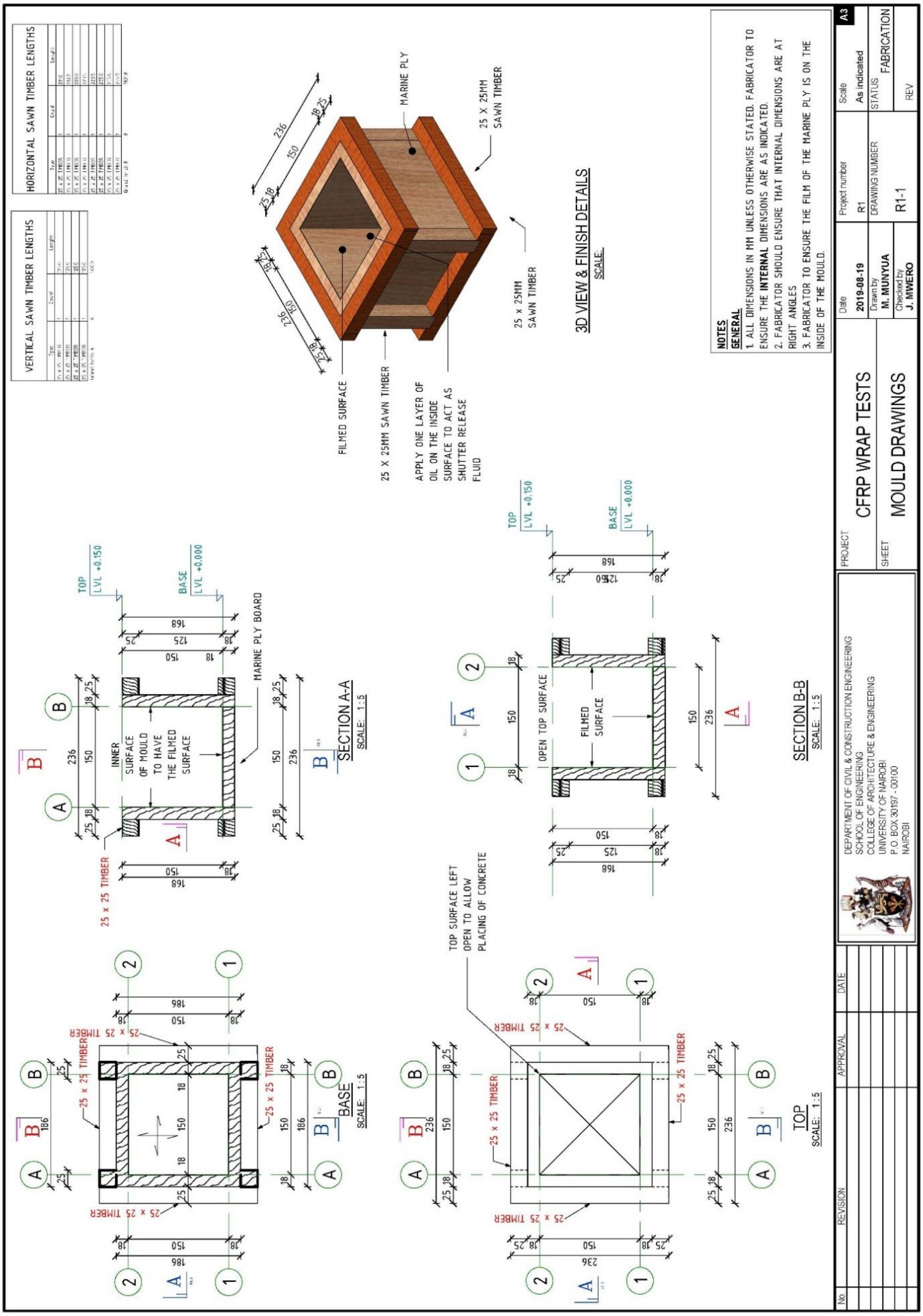


Figure A.1: A drawing of 150mm x 150mm x 150mm timber mould.

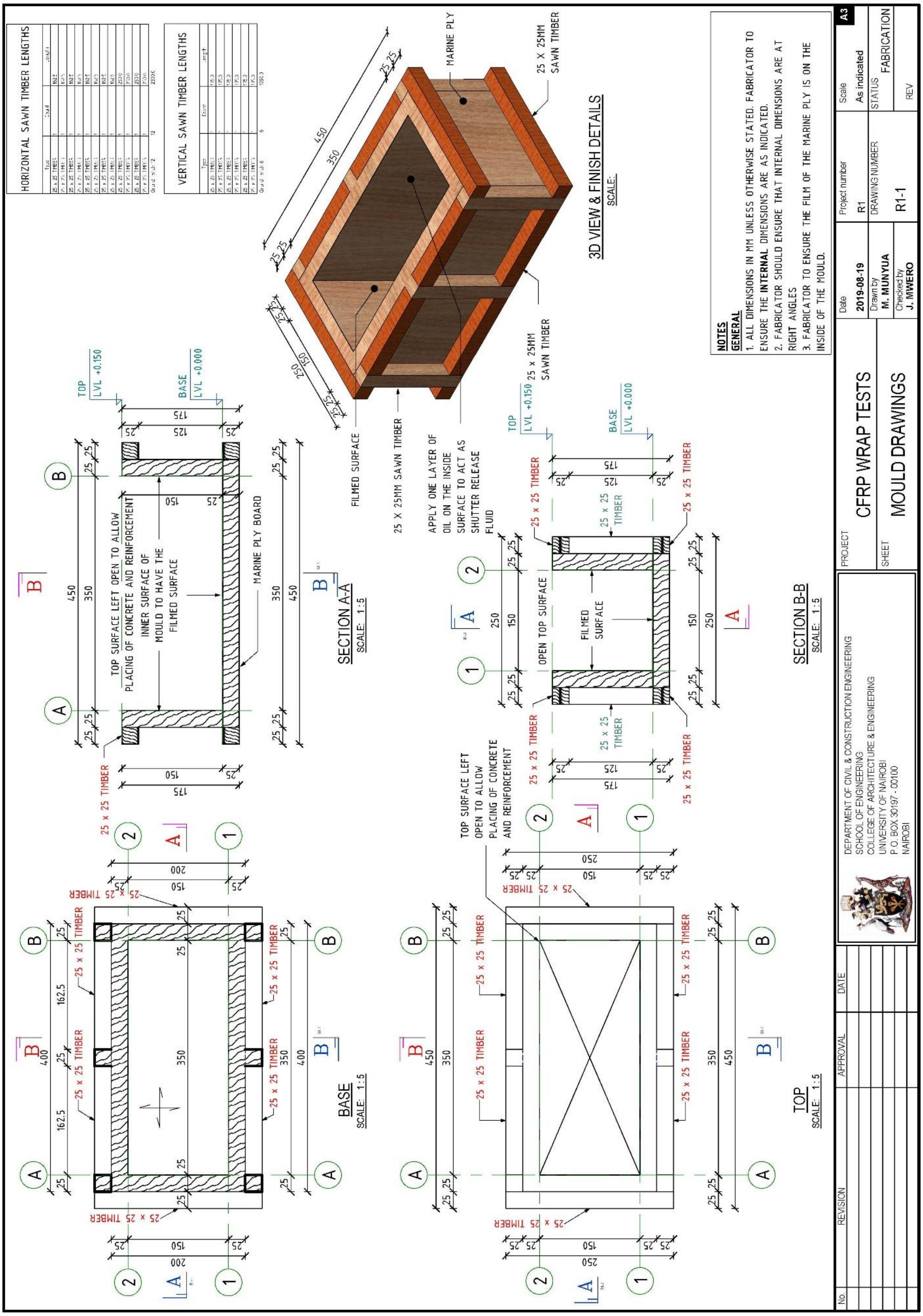


Figure A.2: Mould drawing for the specimen.

Appendix B: Specimen Drawings and Quantities

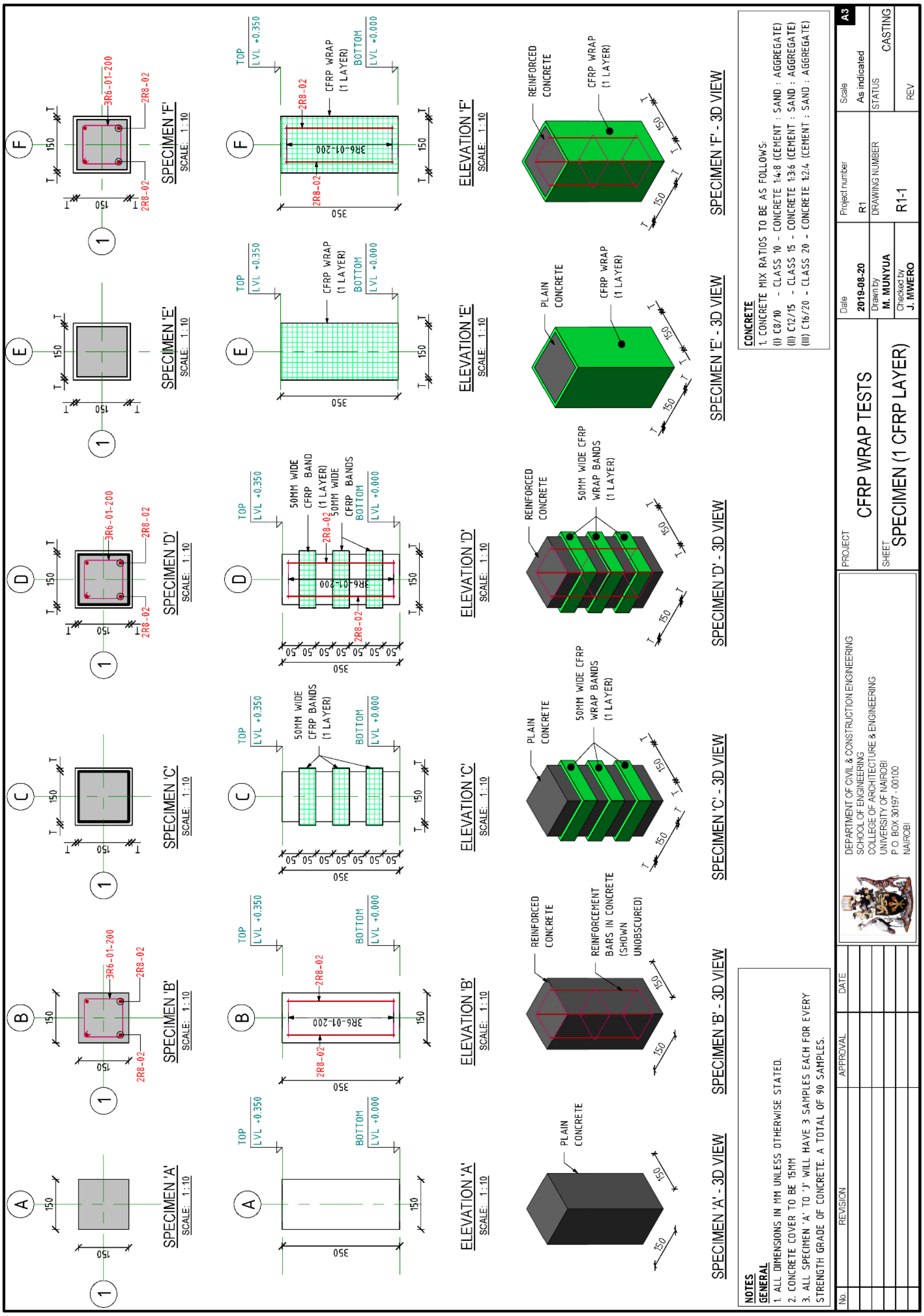


Figure B.1: Drawings for specimen specifications – 1 layer.

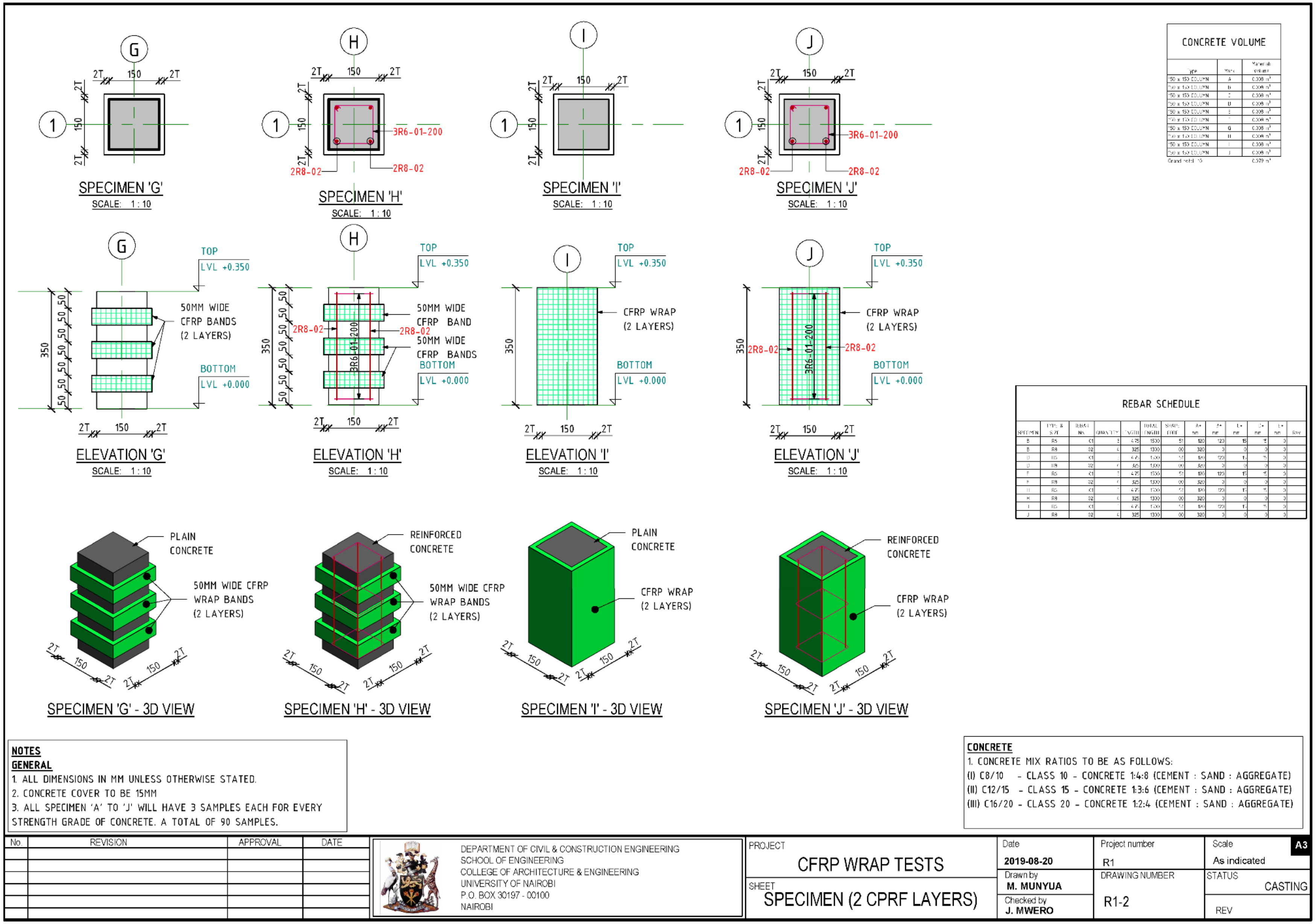


Figure B.2: Drawings for specimen specifications - 2 layers.

Appendix C: Axial Capacity Results

C.1 Failure Stresses of Samples

Table C.1: Recorded failure stresses of the samples.

Serial	Failure stress (MPa)
A.10.1	7.11
A.10.2	9.78
A.10.3	8.00
A.10.4	6.67
A.15.1	12.44
A.15.2	12.89
A.15.3	16.00
A.15.4	11.56
A.20.1	17.33
A.20.2	16.89
A.20.3	18.22
A.20.4	9.33
B.10.1	10.22
B.10.2	9.78
B.10.3	10.67
B.10.4	8.44
B.15.1	15.11
B.15.2	15.56
B.15.3	15.11
B.15.4	13.33
B.20.1	20.44
B.20.2	18.22
B.20.3	16.44
C.10.1	8.00
C.10.2	8.00
C.10.3	8.00
C.15.1	16.00
C.15.2	17.33
C.15.3	16.00
C.20.1	18.22
C.20.2	20.44
C.20.3	19.11
D.10.1	9.33
D.10.2	10.67
D.10.3	10.22
D.15.1	16.00
D.15.2	20.89
D.15.3	19.11
D.20.1	17.33
D.20.2	19.56
D.20.3	20.89
E.10.1	9.33
E.10.2	8.89
E.10.3	8.89
E.15.1	15.56
E.15.2	13.33
E.15.3	15.11
E.20.1	18.22
E.20.2	18.67
E.20.3	21.33
F.10.1	9.33
F.10.2	10.67
F.10.3	10.67
F.15.1	17.33
F.15.2	17.78
F.15.3	20.89
F.20.1	20.44
F.20.2	21.78
F.20.3	20.44
G.10.1	13.33
G.10.2	18.22
G.10.3	8.89
G.15.1	15.56
G.15.2	13.78
G.15.3	14.22
G.20.1	13.33
G.20.2	18.67
G.20.3	17.78
H.10.1	12.44
H.10.2	16.44
H.10.3	17.78
H.15.1	16.44
H.15.2	14.22
H.15.3	14.67
H.20.1	18.67
H.20.2	18.67
H.20.3	18.22
I.10.1	20.44
I.10.2	24.00
I.10.3	17.33
I.15.1	17.78
I.15.2	19.11
I.15.3	13.33
I.20.1	20.00
I.20.2	22.22
I.20.3	26.67
J.10.1	26.67
J.10.2	25.78
J.10.3	24.44
J.15.1	25.78
J.15.2	20.44
J .15.3	19.11
J.20.1	18.67
J.20.2	21.33
J.20.3	21.78

C.2 Average Failure Stresses

Table C.2: Average failure stresses of the specimen.

				Failure Stress (MPa)		
Specimen	**Rebar**	**CFRP**	**Layers**	**C8/10**	**C12/15**	**C16/20**
A.	Unreinforced	No wrap	None	7.9	13.2	17.5
B.	Reinforced	No wrap	None	9.8	14.8	18.4
C.	Unreinforced	Partially	1 layer	8.0	16.4	19.3
D.	Reinforced	Partially	1 layer	10.1	18.7	19.3
E.	Unreinforced	Fully	1 layer	9.0	14.7	19.4
F.	Reinforced	Fully	1 layer	10.2	18.7	20.9
G.	Unreinforced	Partially	2 layers	13.5	14.5	18.2
H.	Reinforced	Partially	2 layers	15.6	15.1	18.5
I.	Unreinforced	Fully	2 layers	20.6	18.4	23.0
J.	Reinforced	Fully	2 layers	25.6	21.8	20.6

C.3 Effect of concrete grade on axial capacity

Table C.3: Average percentage gain of axial capacity classified by concrete grade.

	1 LAYER				2 LAYERS			
	Partial CFRP		**Full CFRP**		**Partial CFRP**		**Full CFRP**	
	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present
C8/10	1.4	3.0	14.6	4.5	70.9	59.1	161.0	162.1
C12/15	24.4	26.3	10.9	26.3	9.8	2.3	39.5	47.4
C16/20	10.2	4.8	11.0	13.7	4.2	0.8	31.4	12.1

C.4 Effect of rebar on axial capacity

Comparison with axial capacity of plain concrete and reinforced concrete.

Table C.4: Average percentage gain in axial capacity due to rebar.

Grade of concrete	Percentage gain
C8/10	23.94
C12/15	11.28
C16/20	5.08

Table C.5: Average axial capacity gain classified by the presence of rebar.

	1 LAYER						2 LAYERS					
	C8/10		C12/15		C16/20		C8/10		C12/15		C16/20	
	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP
Change caused by rebar	25.93	13.11	13.51	27.27	0.00	7.63	15.38	24.46	4.08	18.07	1.63	-10.32
Change caused by CFRP on plain specimen	1.41	14.55	24.37	10.92	10.17	19.49	70.89	161.03	9.80	39.50	4.24	31.36
Change caused by CFRP on reinforced specimen	3.03	4.55	26.32	26.32	4.84	-0.81	59.09	162.12	2.26	47.37	0.81	12.10
Effect of CFRP and Rebar compositely	27.70	29.58	41.18	41.18	10.17	19.49	97.18	224.88	14.29	64.71	5.93	17.80

C.5 Effect of partial and CFRP confinement on axial capacity

Table C.6: Average percentage gain in axial capacity classified by the degree of confinement.

	1 LAYER						2 LAYERS					
	C8/10		C12/15		C16/20		C8/10		C12/15		C16/20	
	Rebar absent	Rebar present	Rebar absent	Rebar Present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present
Partial	1.4	3.0	24.4	26.3	10.2	4.8	70.9	59.1	9.8	2.3	4.2	0.8
Full	14.6	4.5	10.9	26.3	11.0	13.7	161.0	162.1	39.5	47.4	25.0	12.1

C.6 Effect of layers on axial capacity

Table C.7: Average percentage gain in axial capacity classified by number of layers.

	C8/10				C12/15				C16/20			
	Partial CFRP		Full CFRP		Partial CFRP		Full CFRP		Partial CFRP		Full CFRP	
	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present
Single	1.4	3.0	14.6	4.5	24.4	26.3	10.9	13.7	10.2	4.8	11.0	13.7
Double	70.9	59.1	161.0	162.1	9.8	2.3	39.5	47.4	4.2	0.8	31.4	12.1

Appendix D: Ductility Results

D.1 Failure Strains of Samples

Table D.1: Recorded failure stresses of the samples.

Serial	Failure Strains
A.10.1	0.000362857
A.10.2	0.000943429
A.10.3	0.002322286
A.10.4	0.00254
A.15.1	0.001088571
A.15.2	0.007257143
A.15.3	0.002032
A.15.4	0.01016
A.20.1	0.004535714
A.20.2	0.003265714
A.20.3	0.002685143
A.20.4	0.002467429
B.10.1	0.009071429
B.10.2	0.002249714
B.10.3	0.004281714
B.10.4	0.013135429
B.15.1	0.001451429
B.15.2	0.003265714
B.15.3	0.002902857
B.15.4	0.003918857
B.20.1	0.006458857
B.20.2	0.001451429
B.20.3	0.002467429
C.10.1	0.018868571
C.10.2	0.024674286
C.10.3	0.008708571
C.15.1	0.013788571
C.15.2	0.007257143
C.15.3	0.007910286
C.20.1	0.006531429
C.20.2	0.002830286
C.20.3	0.029028571
D.10.1	0.019594286
D.10.2	0.032657143
D.10.3	0.020029714
D.15.1	0.006531429
D.15.2	0.008708571
D.15.3	0.004354286
D.20.1	0.00508
D.20.2	0.01778
D.20.3	0.003628571
E.10.1	0.039188571
E.10.2	0.036285714
E.10.3	0.041365714
E.15.1	0.007257143
E.15.2	0.0254
E.15.3	0.003628571
E.20.1	0.006531429
E.20.2	0.056605714
E.20.3	0.006531429
F.10.1	0.079828571
F.10.2	0.019594286
F.10.3	0.039188571
F.15.1	0.007982857
F.15.2	0.00254
F.15.3	0.009434286
F.20.1	0.004354286
F.20.2	0.006894286
F.20.3	0.004354286
G.10.1	0.007257143
G.10.2	0.014514286
G.10.3	0.004354286
G.15.1	0.0254
G.15.2	0.015965714
G.15.3	0.036285714
G.20.1	0.038462857
G.20.2	0.043542857
G.20.3	0.03048
H.10.1	0.014514286
H.10.2	0.02032
H.10.3	0.01016
H.15.1	0.034108571
H.15.2	0.029028571
H.15.3	0.024674286
H.20.1	0.042091429
H.20.2	0.043542857
H.20.3	0.038462857
I.10.1	0.0508
I.10.2	0.070394286
I.10.3	0.017417143
I.15.1	0.116114286
I.15.2	0.072571429
I.15.3	0.072571429
I.20.1	0.108857143
I.20.2	0.1016
I.20.3	0.108857143
J.10.1	0.073297143
J.10.2	0.079828571
J.10.3	0.044268571
J.15.1	0.087085714
J.15.2	0.094342857
J .15.3	0.03048
J.20.1	0.087085714
J.20.2	0.079828571
J.20.3	0.042526857

D.2 Average Ductility

Table D.2: Average ductility of the samples.

				Ductility (%)		
Specimen	**Rebar**	**CFRP**	**Layers**	**C8/10**	**C12/15**	**C16/20**
A.	Unreinforced	No wrap	None	0.2	0.5	0.3
B.	Reinforced	No wrap	None	0.7	0.3	0.3
C.	Unreinforced	Partially	1 layer	1.7	1.0	1.3
D.	Reinforced	Partially	1 layer	2.4	0.7	0.9
E.	Unreinforced	Fully	1 layer	3.9	1.2	2.3
F.	Reinforced	Fully	1 layer	4.6	0.7	0.5
G.	Unreinforced	Partially	2 layers	0.9	2.6	3.7
H.	Reinforced	Partially	2 layers	1.5	2.9	4.1
I.	Unreinforced	Fully	2 layers	4.6	8.7	10.6
J.	Reinforced	Fully	2 layers	6.6	7.1	7.0

D.3 Effect of concrete grade on ductility

Table D.3: Average percentage gain in ductility classified by concrete grade.

	1 LAYER				2 LAYERS			
	Partial CFRP		**Full CFRP**		**Partial CFRP**		**Full CFRP**	
	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present	Rebar Absent	Rebar Present
C8/10	1029.4	235.4	2425.5	543.1	464.7	108.8	2896.1	815.8
C12/15	88.0	126.4	135.6	130.6	404.1	914.7	1596.1	2348.6
C16/20	295.1	155.2	617.1	50.3	1057.8	1095.8	3186.6	1918.2

D.4 Effect of rebar on ductility

Comparison with the ductility of plain concrete and reinforced concrete.

Table D.4: Average percentage gain in ductility due to rebar.

Grade of concrete	Percentage gain
C8/10	365.88
C12/15	234.59
C16/20	6.82

Table D.5: Average percentage gain in ductility classified by the presence of rebar.

	1 LAYER						2 LAYERS					
	C8/10		C12/15		C16/20		C8/10		C12/15		C16/20	
	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP	Partial CFRP	Full CFRP
Change caused by rebar	38.33	18.63	-32.33	-45.00	-31.00	-77.60	72.22	42.41	13.08	-18.89	10.32	-34.41
Change caused by CFRP on plain specimen	1029.41	2425.49	87.99	135.57	295.14	60.60	464.71	2896.08	404.12	1596.11	1057.80	3186.65
Change caused by CFRP on reinforced specimen	235.35	543.10	126.42	130.61	155.24	983.92	108.75	815.82	914.68	2348.64	1095.80	1918.18
Effect of CFRP and Rebar compositely	1462.35	2896.08	27.21	29.56	172.64	60.60	872.55	4166.67	470.08	1275.74	1177.31	2055.74

D.5 Effect of partial and full CFRP confinement on ductility

Table D.6: Average percentage gain in ductility classified by the degree of confinement.

	1 LAYER						2 LAYERS					
	C8/10		C12/15		C16/20		C8/10		C12/15		C16/20	
	Rebar absent	Rebar present	Rebar absent	Rebar Present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present
Partial	1029.4	235.4	88.0	126.4	295.1	155.2	464.7	108.8	404.1	914.7	1057.8	1095.8
Full	2425.5	543.1	135.6	130.6	617.1	50.3	2896.1	815.8	1596.1	2348.6	2976.9	1918.2

D.6 Effect of layers on ductility

Table D.7: Average percentage gain in ductility classified by the number of layers.

	C8/10				C12/15				C16/20			
	Partial CFRP		Full CFRP		Partial CFRP		Full CFRP		Partial CFRP		Full CFRP	
	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present	Rebar absent	Rebar present
Single	1029.4	235.4	2425.5	543.1	88.0	126.4	135.6	50.3	295.1	155.2	617.1	50.3
Double	464.7	108.8	2896.1	815.8	404.1	914.7	1596.1	2348.6	1057.8	1095.8	3186.6	1918.2

Appendix E: Average Effect of the Variables

Table E.1: Effect of the independent variables on axial capacity and ductility.

Independent variables		Axial capacity	Ductility
Grade of concrete	C8/10	59.6%	1064.8%
	C12/15	23.4%	718.0%
	C16/20	11.0%	1047.0%
Steel Rebar	Absent	33.1%	1136.6%
	Present	29.0%	781.4%
Degree of confinement	Partial	18.1%	498.0%
	Fully	44.0%	1371.2%
Number of layers	1 layer	11.5%	479.3.6%
	2 layers	50.0%	1400.6%

Treemaps categorised by the independent variables

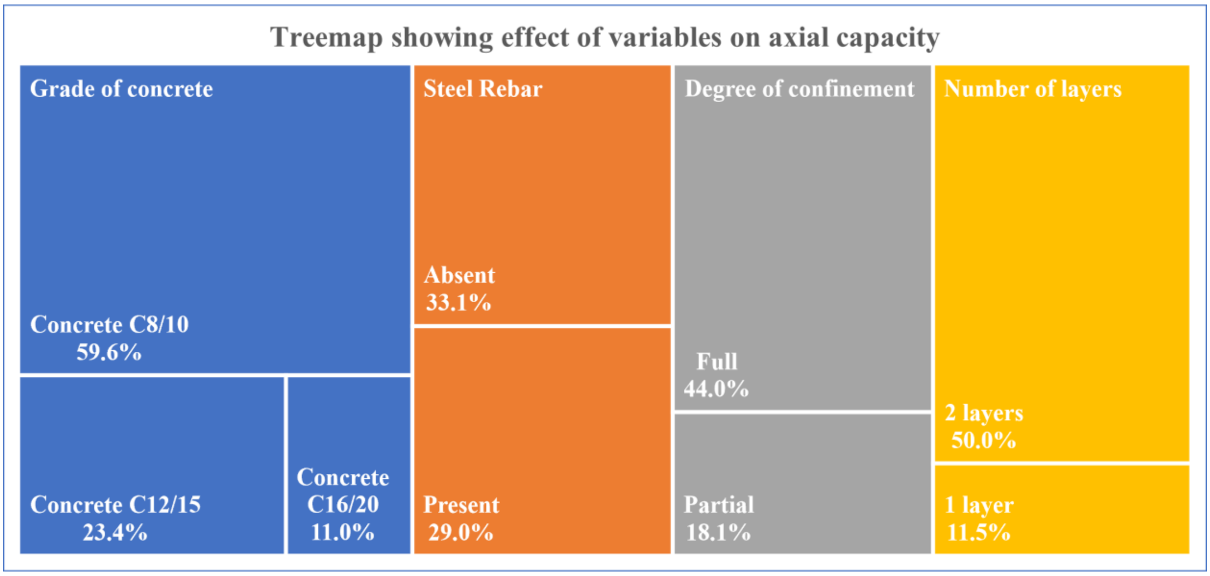


Figure E.1: A treemap showing the effect of the variables on the axial capacity.

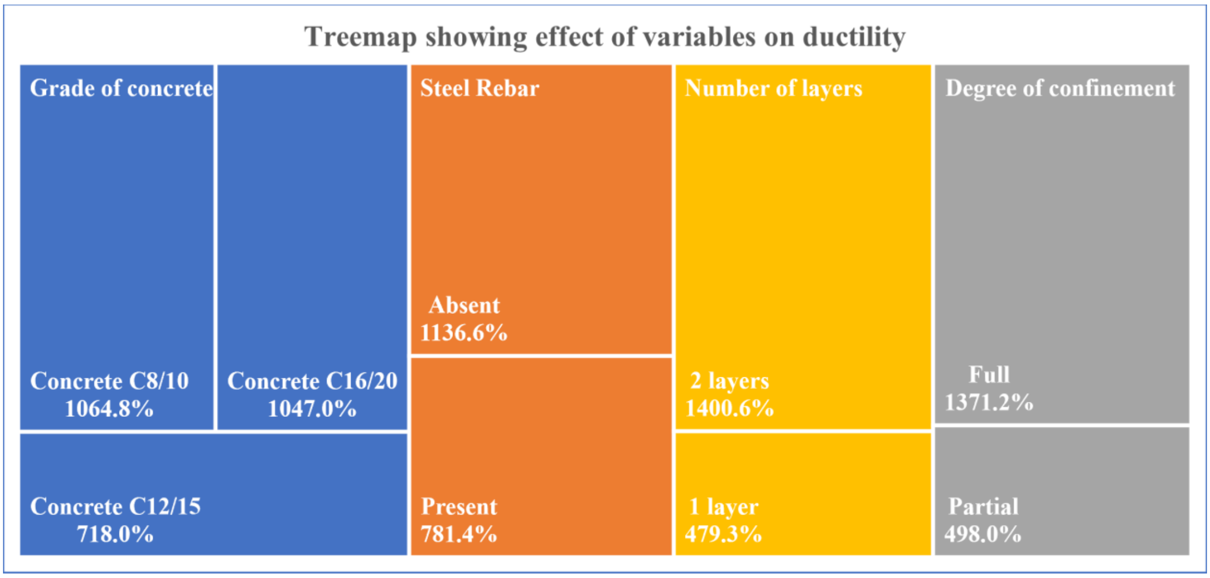


Figure E.2: A treemap show effect of the variables on the ductility.

Appendix F: Design Proposal Procedure

1. Determine the columns to be strengthened. Test the columns with a Schmidt Hammer. The recommended procedure is per EN 12504-2:2012 (CEN, 2012). Other Non-destructive tests may be used to determine the strength of the columns. The Rebound Values from the Schmidt Hammer will be used in determining the strength of concrete.

2. Determine the tensile strength of the CFRP wrap as per the standard ASTM D3039 / D3039M-17 (ASTM, 2017).

3. Find out the target strength as per the requirements of the design engineer.

4. Compare the actual strength against the target strength as a percentage.

5. Use the table to determine the number of layers required.

Table F.1: Recommended CFRP confinement required.

Percentage Change	Number of Layers
<10%	One layer partial confinement
10-30%	One layer full confinement
30-50%	Two layers partial confinement
>50%	Two layers full confinement

After the determination of the percentage change required, the procedure to apply the CFRP can follow the process outlined by the manufacturer (Horse Construction, 2019).

F.1 Surface preparation

The corners of the column should be rounded to a minimum radius of 25mm. Care should be exercised to prevent exposing the steel reinforcement to the elements.

All rough edges and protrusions should be smoothened. Cavities should also be filled with putty or any other structural filler materials such as epoxy fillers.

About 24hrs to the application, the column surfaces should be washed to get rid of dust and other fine particles that can necessitate the need to use a higher amount of the epoxy. (Fine particles have a high surface area to volume ratio.

F.2 Setting Out

The concrete surface should be left clean and kept dry.

F.3 Apply primer

Apply a primer adhesive onto the surface to aid bonding between the existing concrete and putty that will smoothen the surface.

F.4 Levelling of the concrete

Putty should be applied for repairing and levelling if need be.

F.5 Fabric Cutting

The carbon fibre should be cut to the sizes as designed.

F.6 Preparing the impregnation adhesive.

Weigh and mix the adhesive according to ratio and stir until the colour mixture is even. Care should be taken to avoid introducing air bubbles.

F.7 Applying Impregnation Adhesive

Apply impregnation adhesive when primer adhesive is touch dry.

F.8 Apply carbon fibre fabric

Apply carbon fibre fabric onto the concrete surface as designed. Levelling the surface from one end to end

F.9 Removal of Air Bubbles

Apply impregnation carbon fibre adhesive again. Make sure the adhesive impregnates fully into the fabric: the surface should be flat and with no air bubbles. The process should be repeated when applying two or more layers.

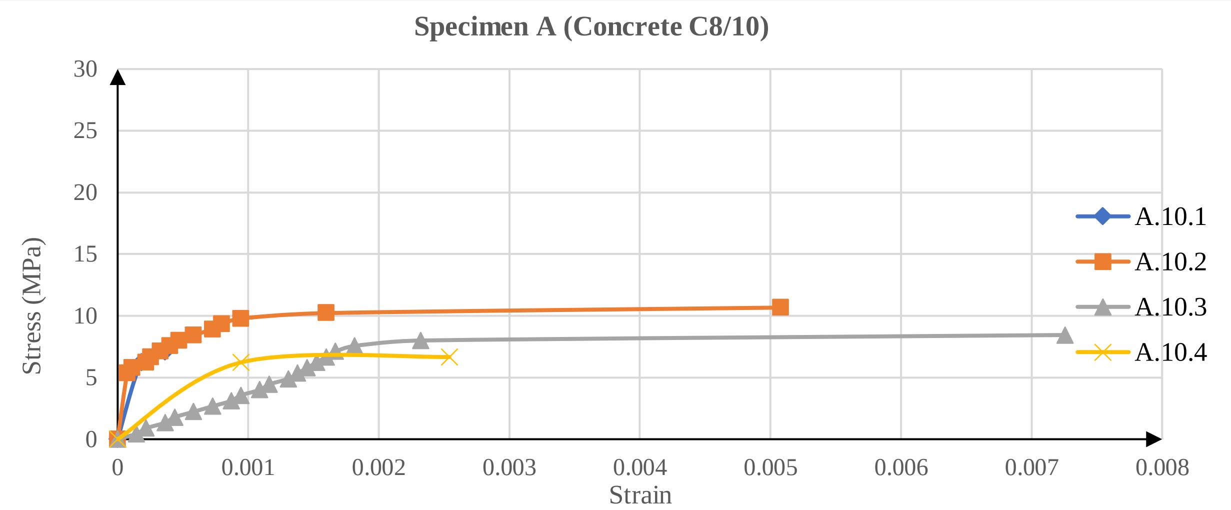
F.10 Worker Safety

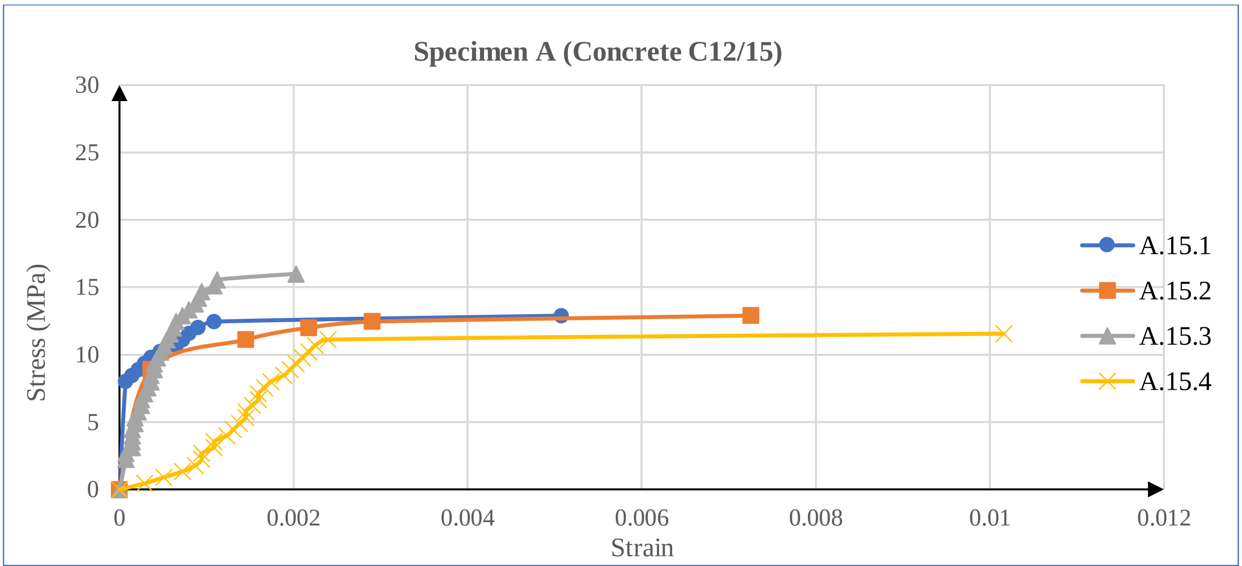
1. The construction workers should take protective measures such as wearing masks, gloves and goggles, among other protective equipment.

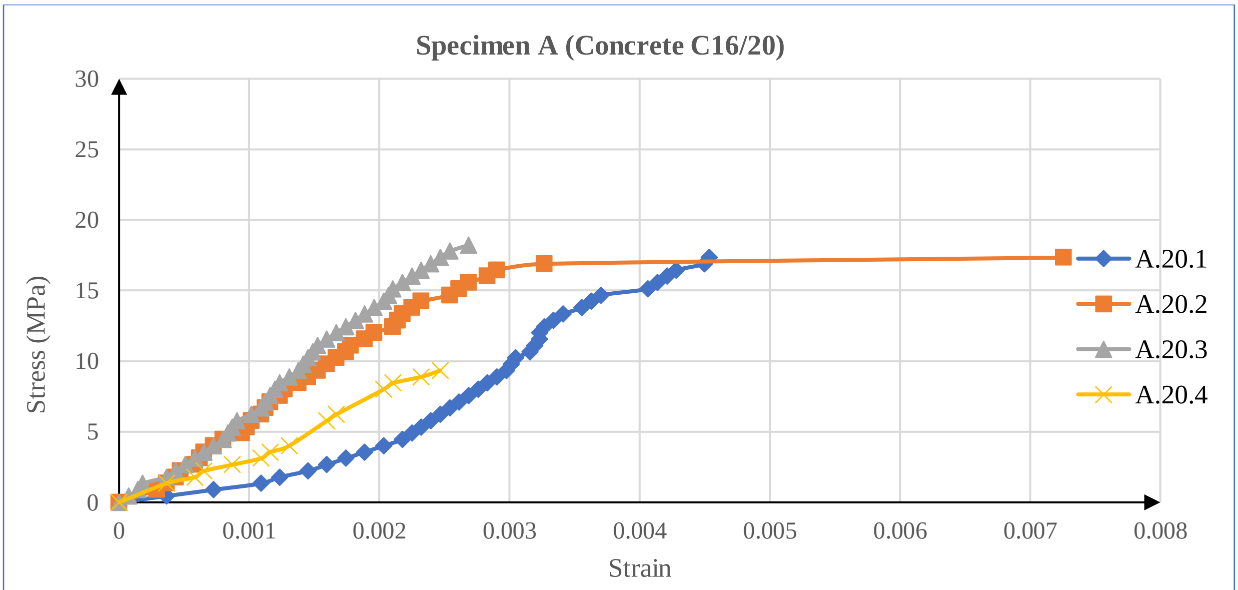
2. Fire prevention procedures should be undertaken and maintain proper ventilation on site.

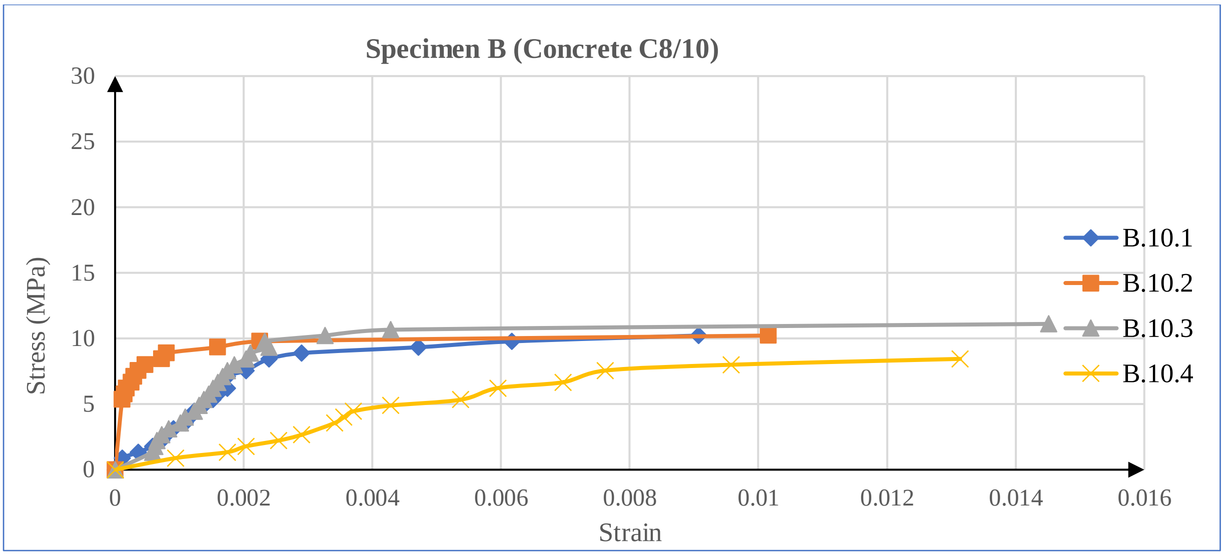
3. Carbon fibres have electrical conductivity properties, and care should be taken when handling electrical equipment with carbon fibre.

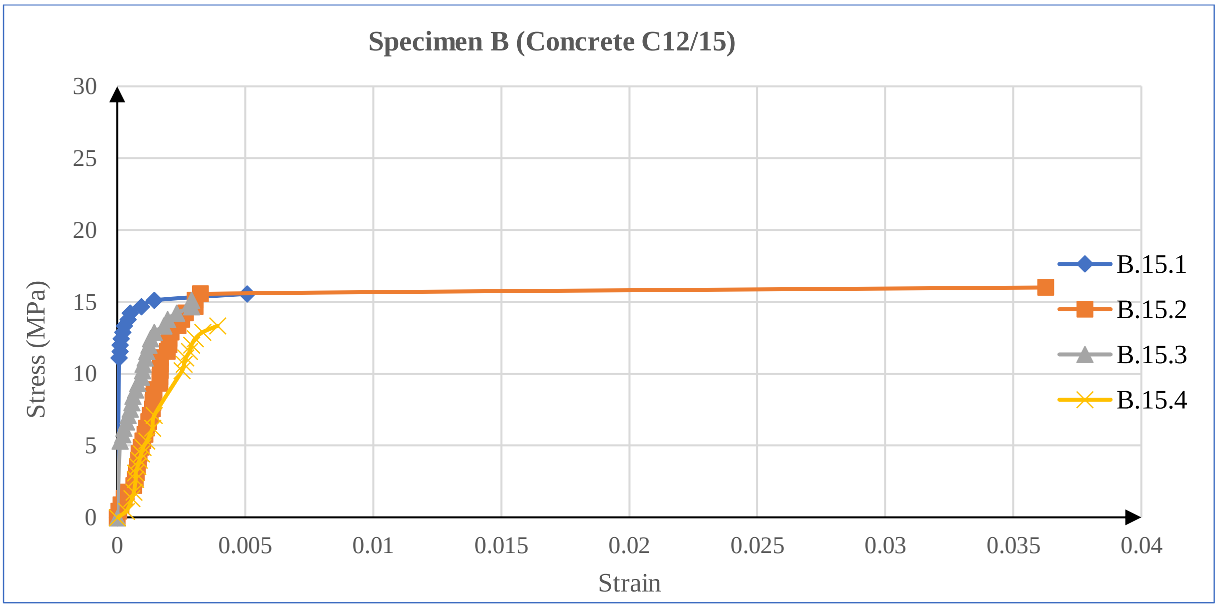
Appendix G: Stress-Strain Graphs








Specimen B (Concrete C8/10)
Stress (MPa)
Strain
30
25
20
15
10
5
0
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
B.10.1
B.10.2
B.10.3
B.10.4


Specimen B (Concrete C12/15)
Stress (MPa)
Strain
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25
20
15
10
5
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0
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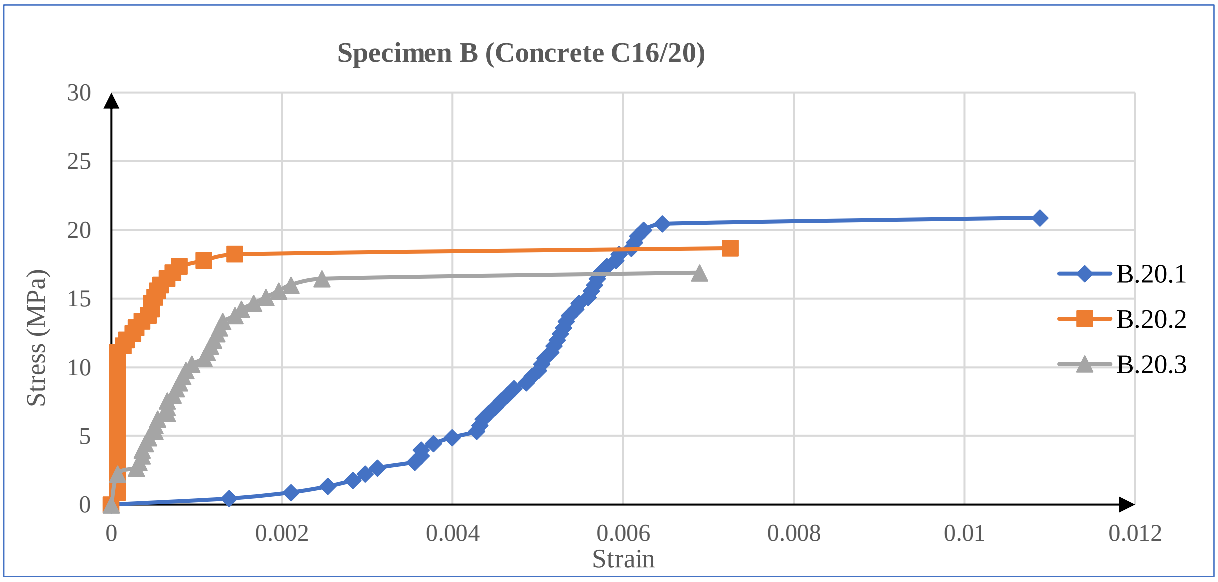
0.04
B.15.1
B.15.2
B.15.3
B.15.4


Specimen B (Concrete C16/20)
Stress (MPa)
Strain
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25
20
15
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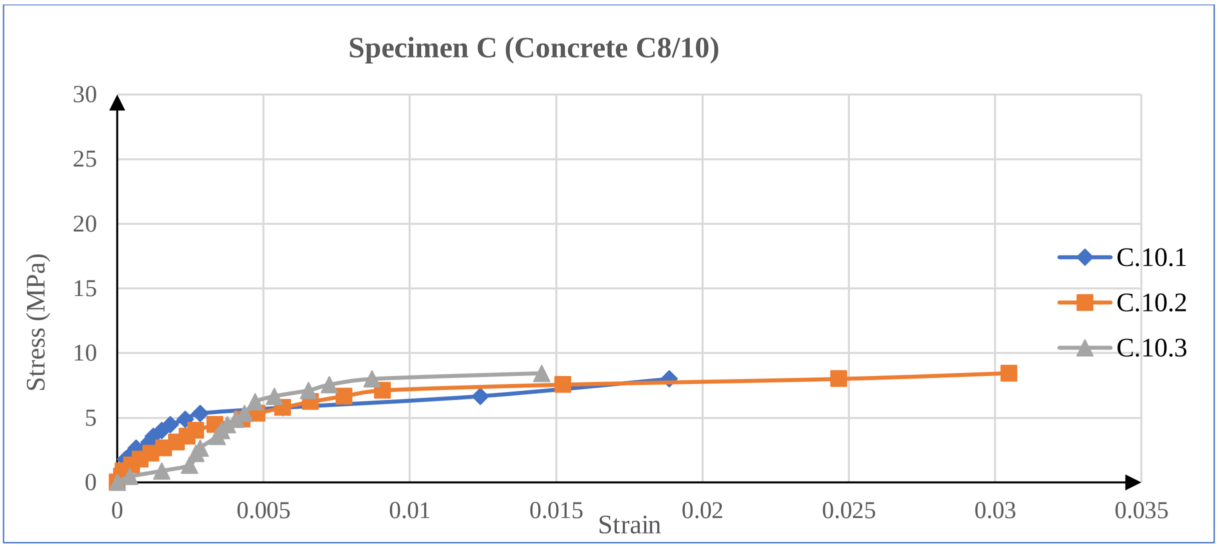
0.008
0.01
0.012
B.20.1
B.20.2
B.20.3


Specimen C (Concrete C8/10)
Stress (MPa)
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25
20
15
10
5
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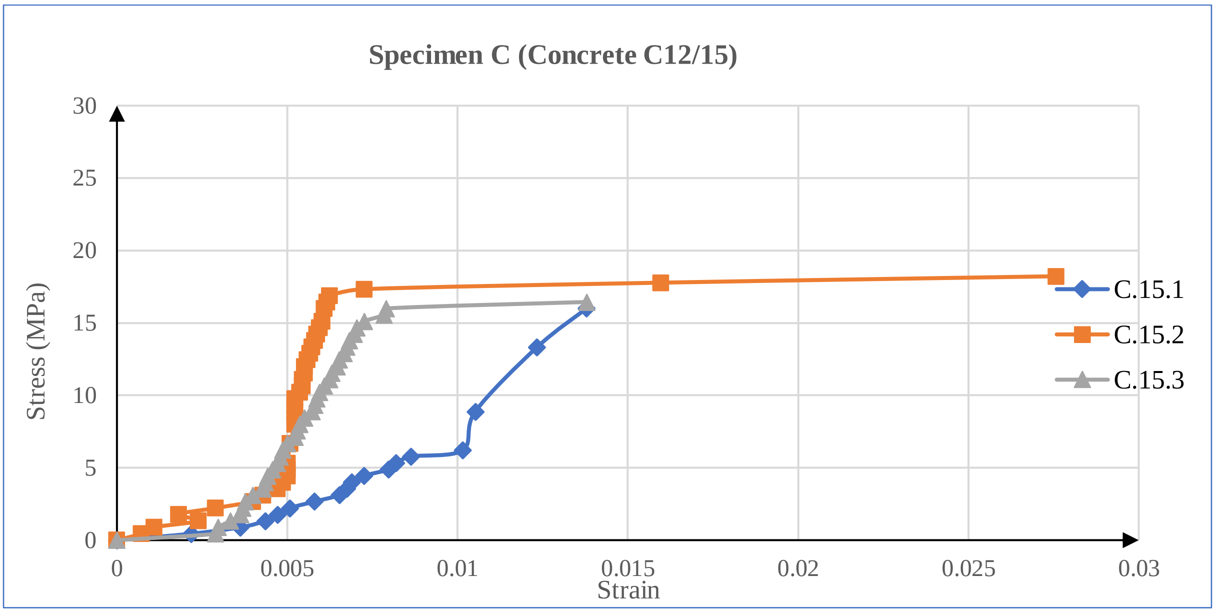
0.025
0.03
0.035
Strain
C.10.1
C.10.2
C.10.3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Stress (MPa)
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25
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15
10
5
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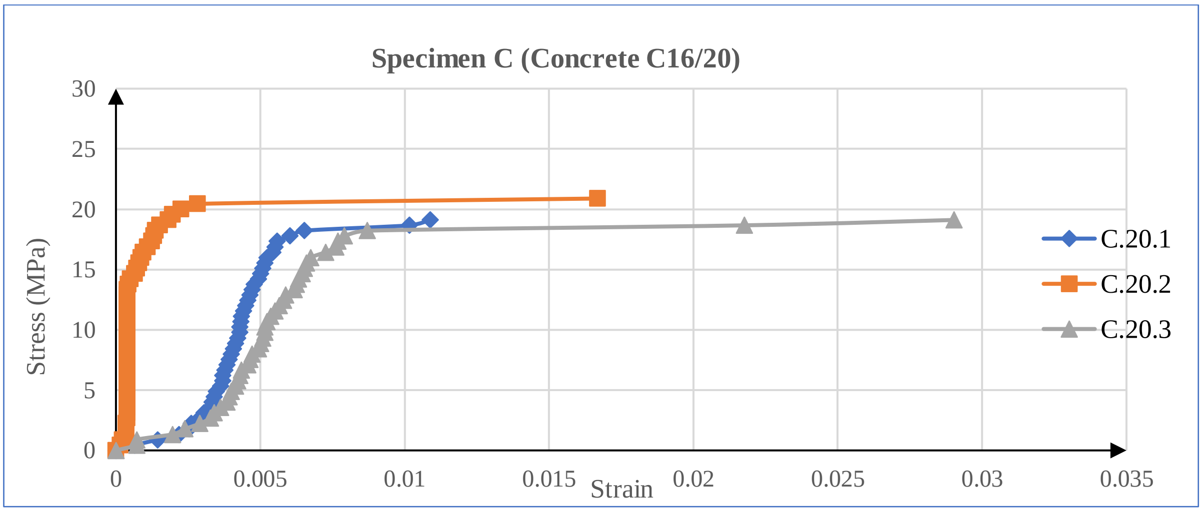
0.025
0.03
Strain
C.15.1
C.15.2
C.15.3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Stress (MPa)
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25
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10
5
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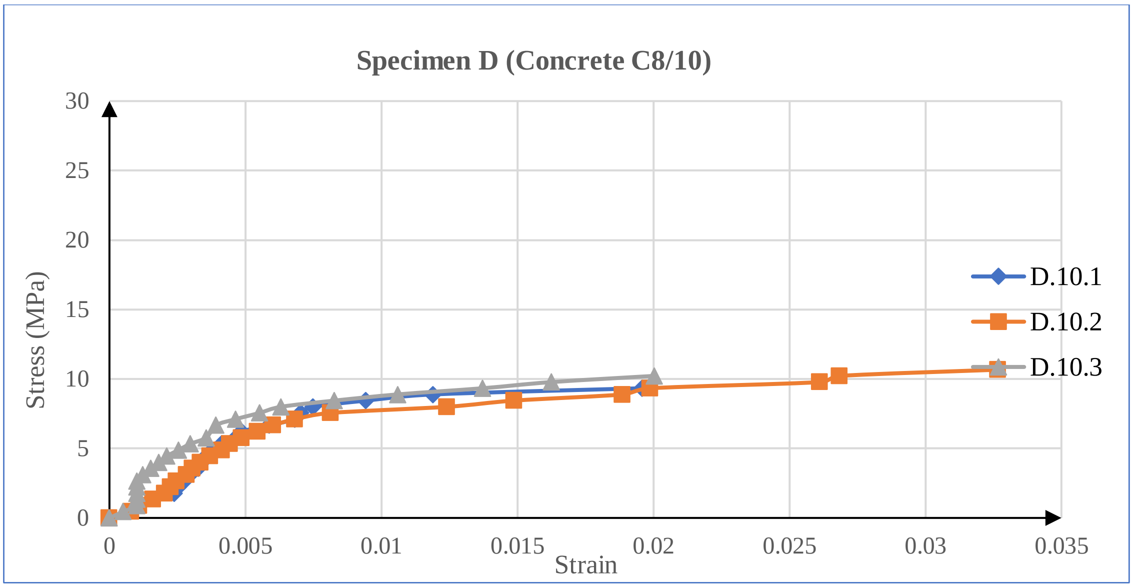
0.03
0.035
Strain
C.20.1
C.20.2
C.20.3


Specimen D (Concrete C8/10)
Stress (MPa)
Strain
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25
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15
10
5
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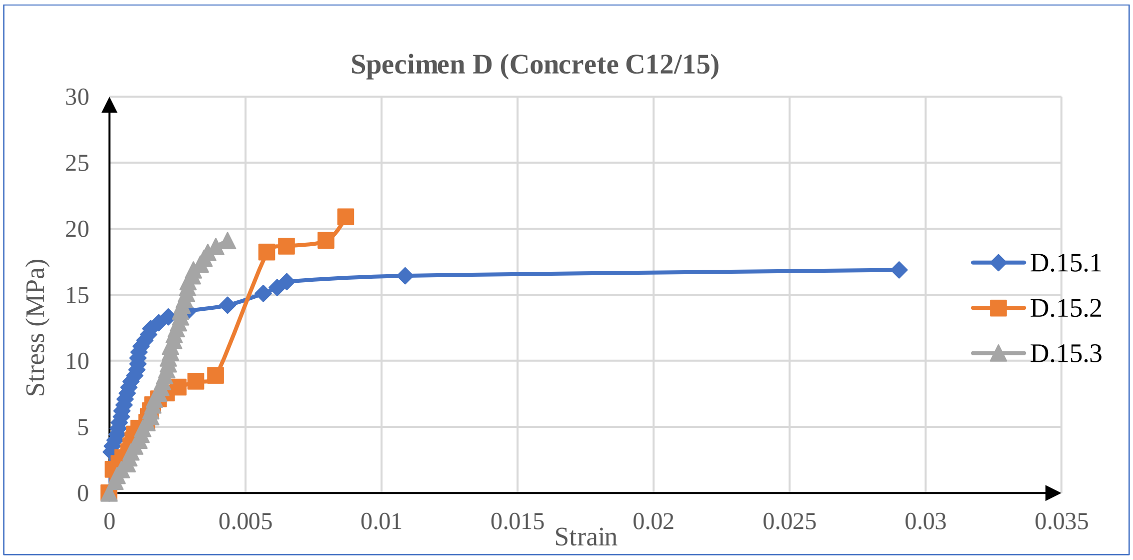
0.02
0.025
0.03
0.035
D.10.1
D.10.2
D.10.3


Specimen D (Concrete C12/15)
Stress (MPa)
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25
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15
10
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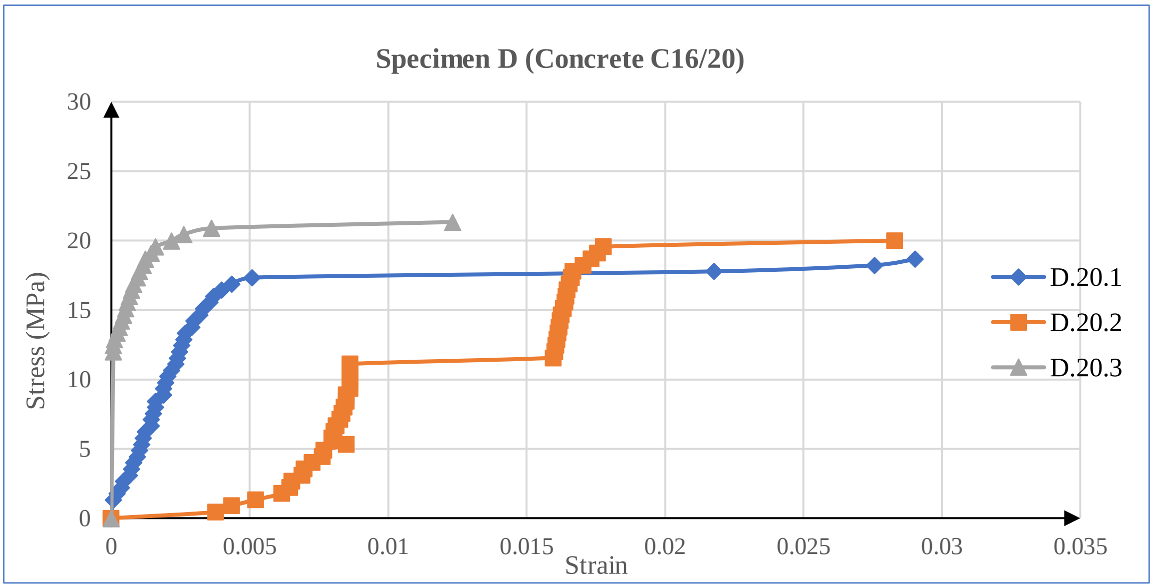
0.02
0.025
0.03
0.035
D.15.1
D.15.2
D.15.3


Specimen D (Concrete C16/20)
Stress (MPa)
Strain
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25
20
15
10
5
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0
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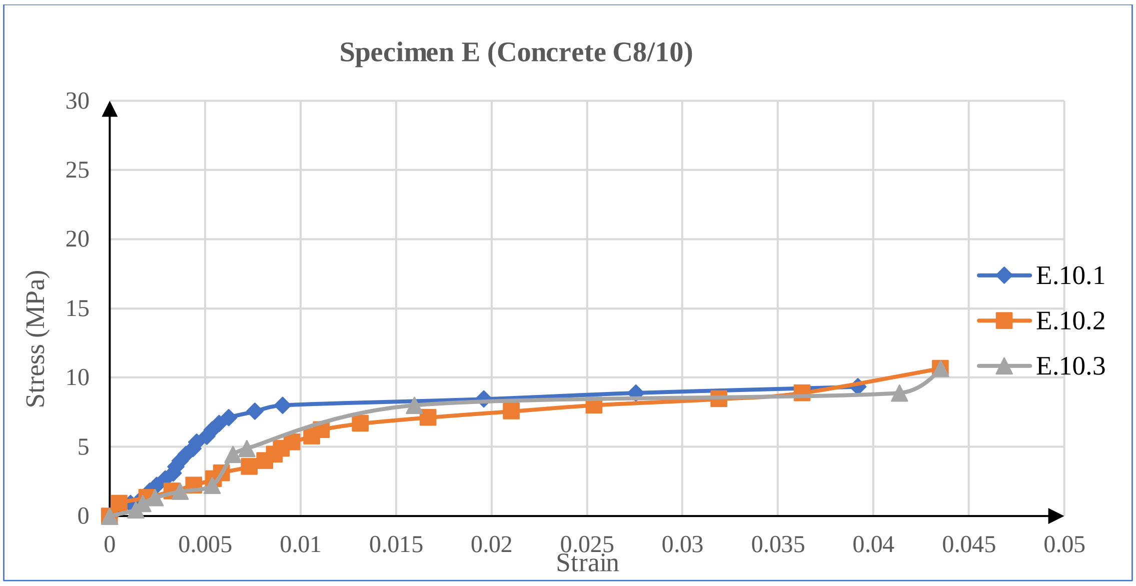
0.015
0.02
0.025
0.03
0.035
D.20.1
D.20.2
D.20.3



Specimen E (Concrete C8/10)
Stress (MPa)
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25
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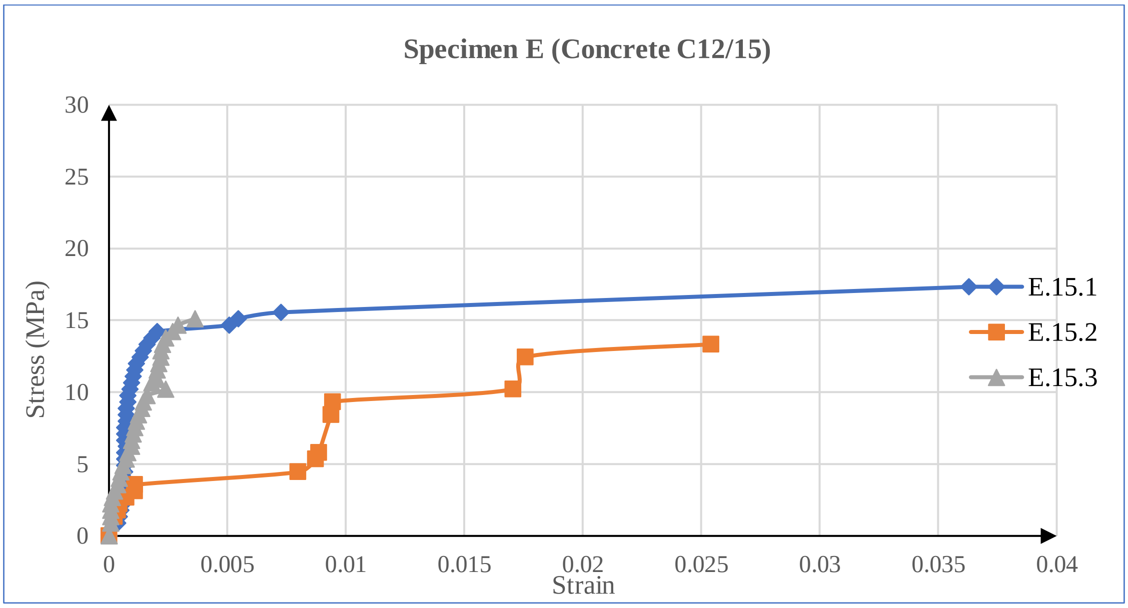
15
10
5
0
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05
Strain
E.10.1
E.10.2
E.10.3



Specimen E (Concrete C12/15)
Stress (MPa)
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25
20
15
10
5
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0
0.005
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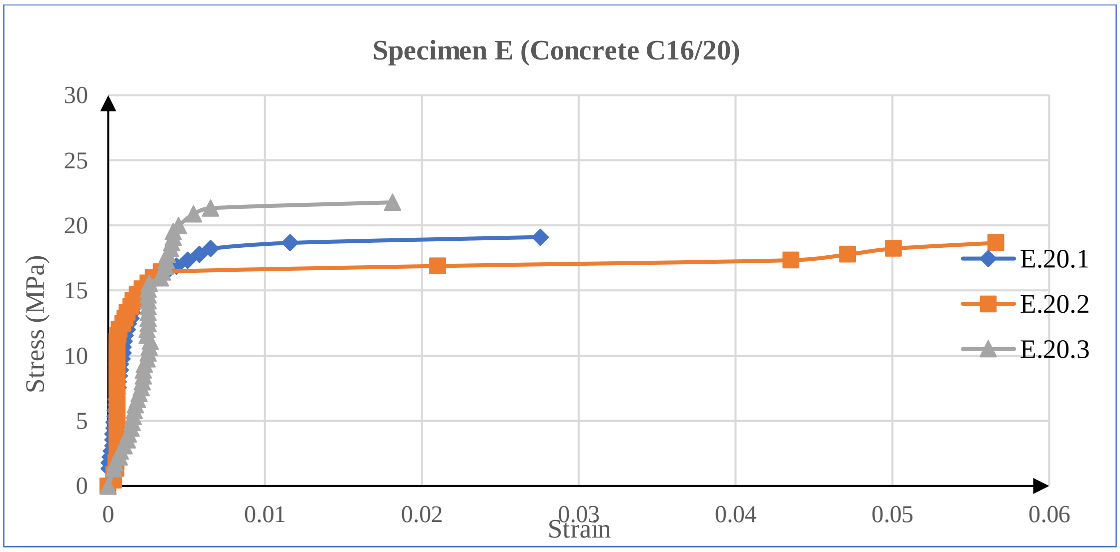
0.03
0.035
0.04
Strain
E.15.1
E.15.2
E.15.3



Specimen E (Concrete C16/20)
Stress (MPa)
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25
20
15
10
5
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0
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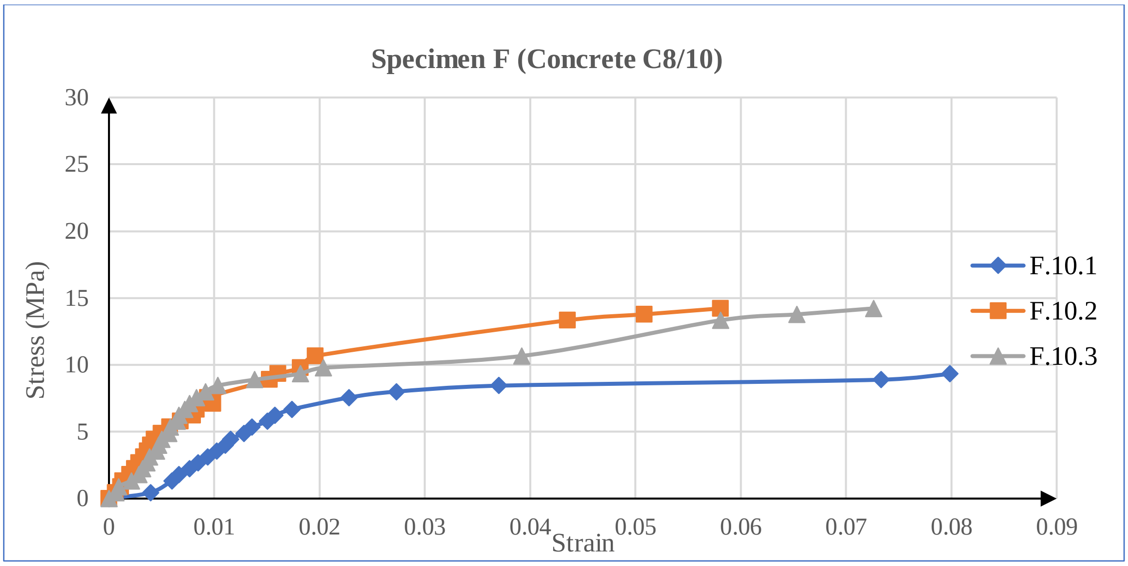
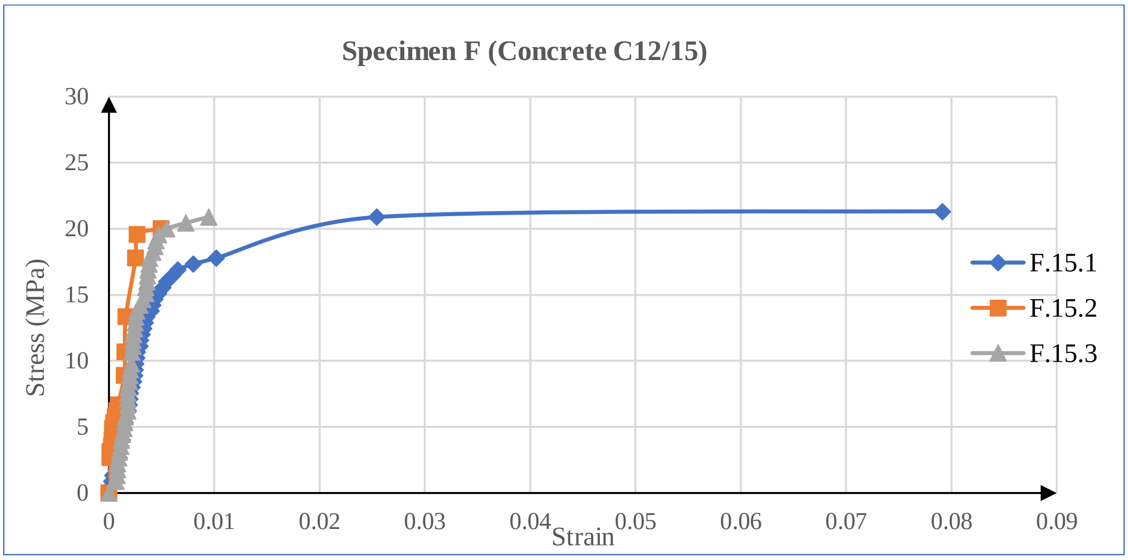
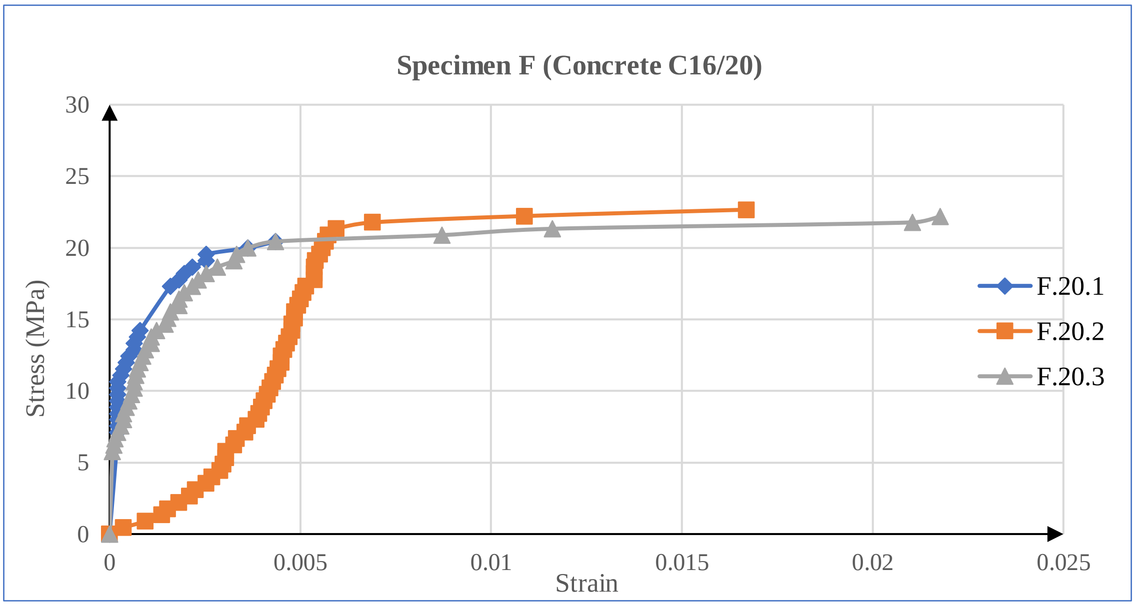
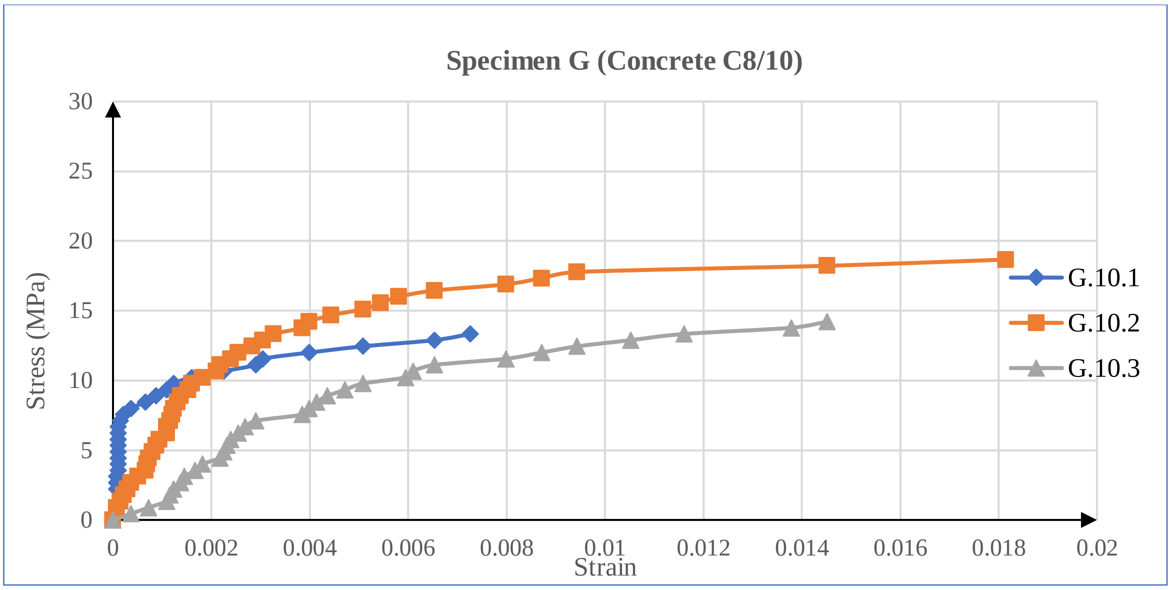
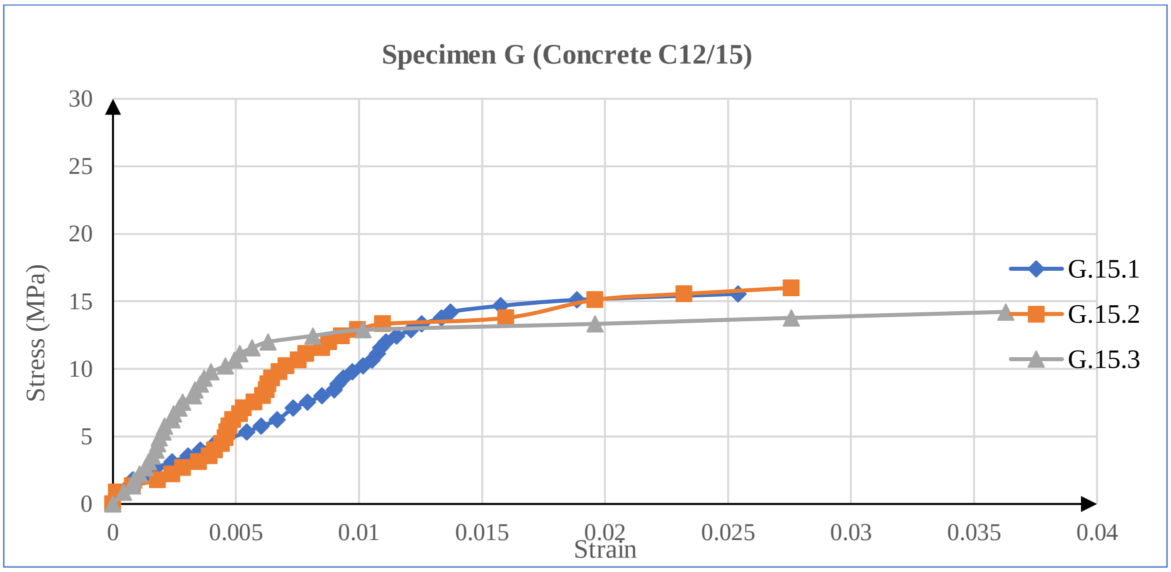
0.02
0.03
0.04
0.05
0.06
Strain
E.20.1
E.20.2
E.20.3

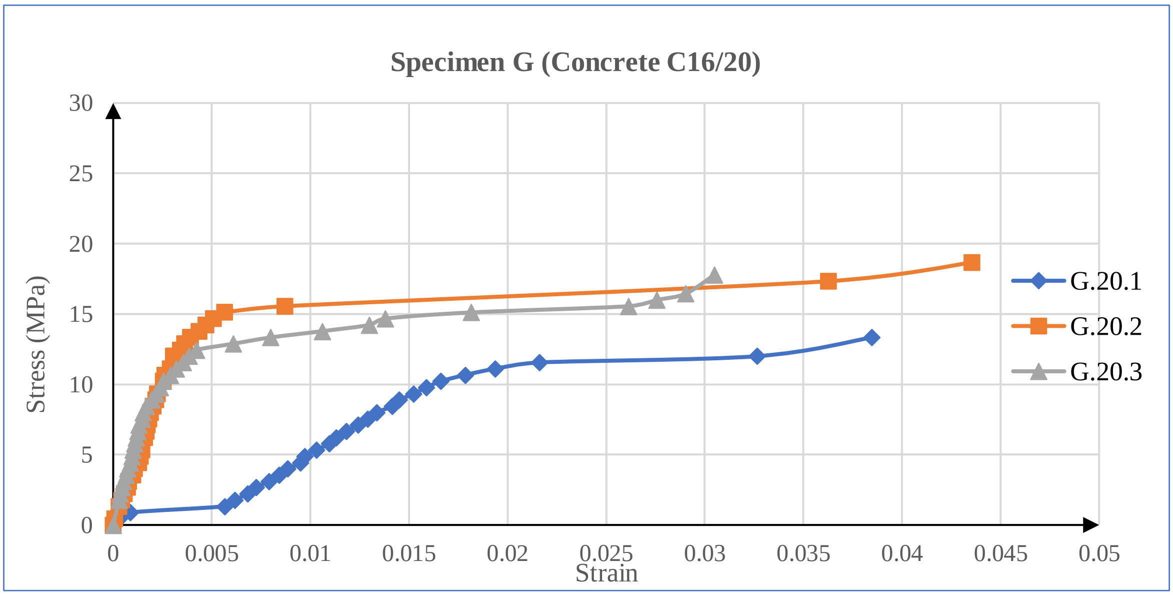

Specimen F (Concrete C8/10)
Stress (MPa)
30
25
20
15
10
5
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
Strain
F.10.1
F.10.2
F.10.3

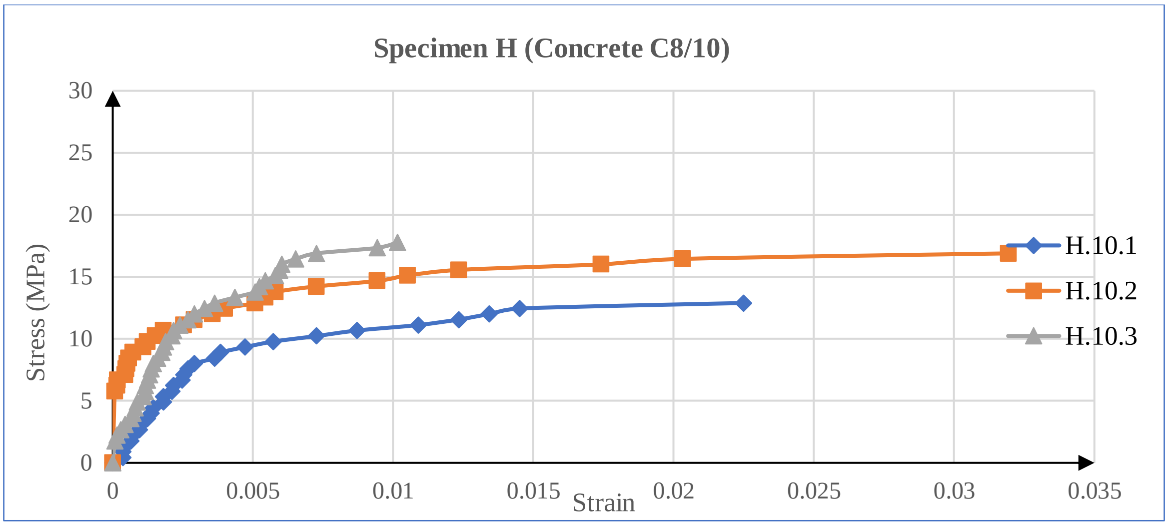

Specimen F (Concrete C12/15)
Stress (MPa)
30
25
20
15
10
5
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
Strain
F.15.1
F.15.2
F.15.3

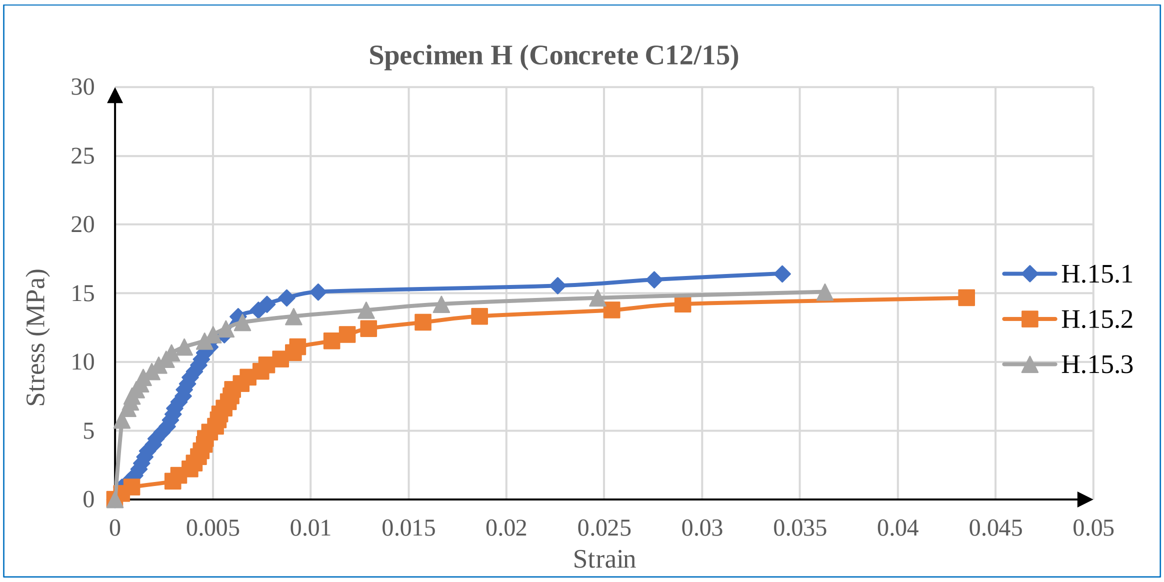

Specimen F (Concrete C16/20)
Stress (MPa)
30
25
20
15
10
5
0
0
0.005
0.01
0.015
0.02
0.025
Strain
F.20.1
F.20.2
F.20.3

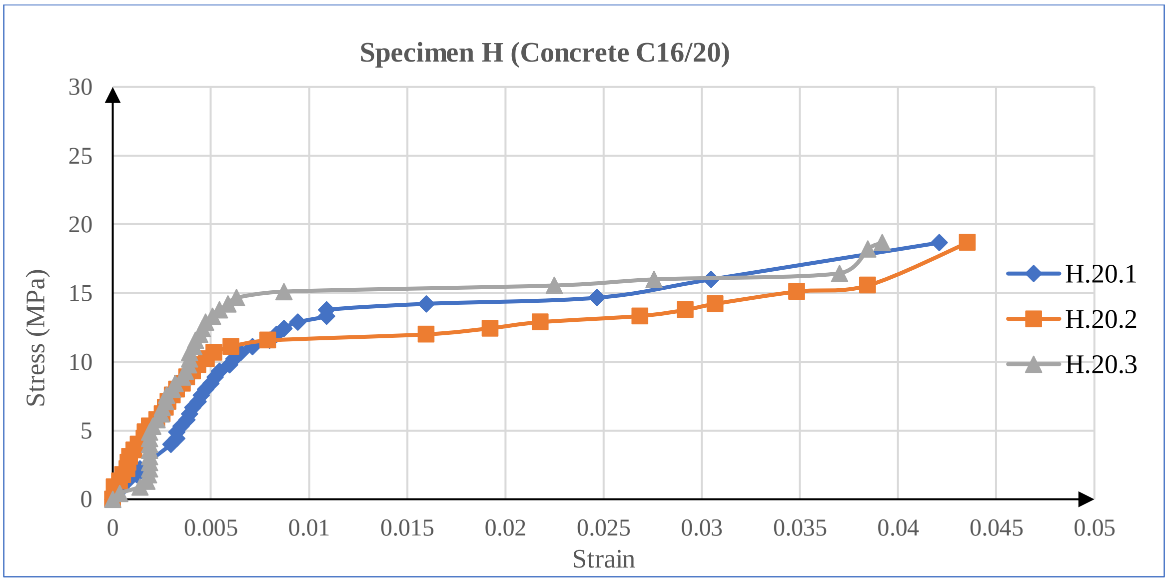

Specimen G (Concrete C8/10)
30
25
20
15
10
5
0
Stress (MPa)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
Strain
G.10.1
G.10.2
G.10.3

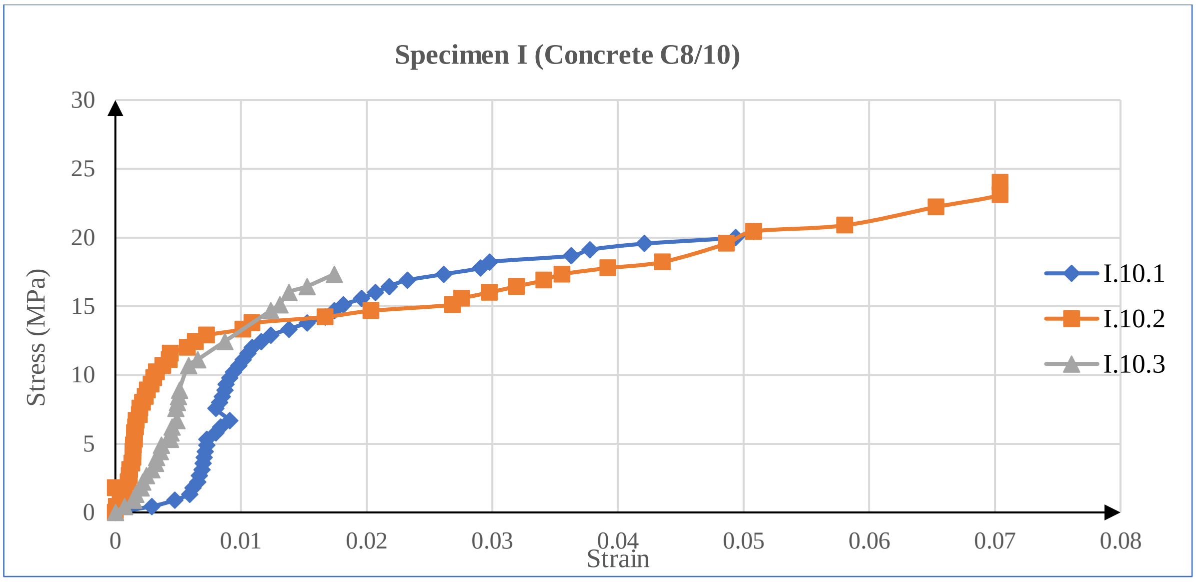

Specimen G (Concrete C12/15)
30
25
20
15
10
5
0
Stress (MPa)
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
Strain
G.15.1
G.15.2
G.15.3

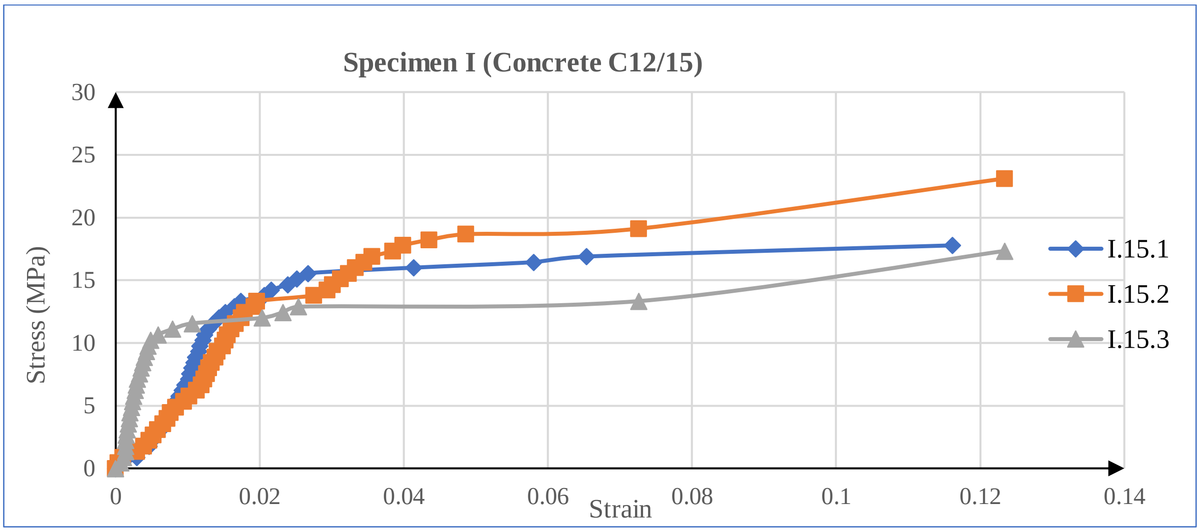

Specimen G (Concrete C16/20)
30
25
20
15
10
5
0
Stress (MPa)
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05
Strain
G.20.1
G.20.2
G.20.3

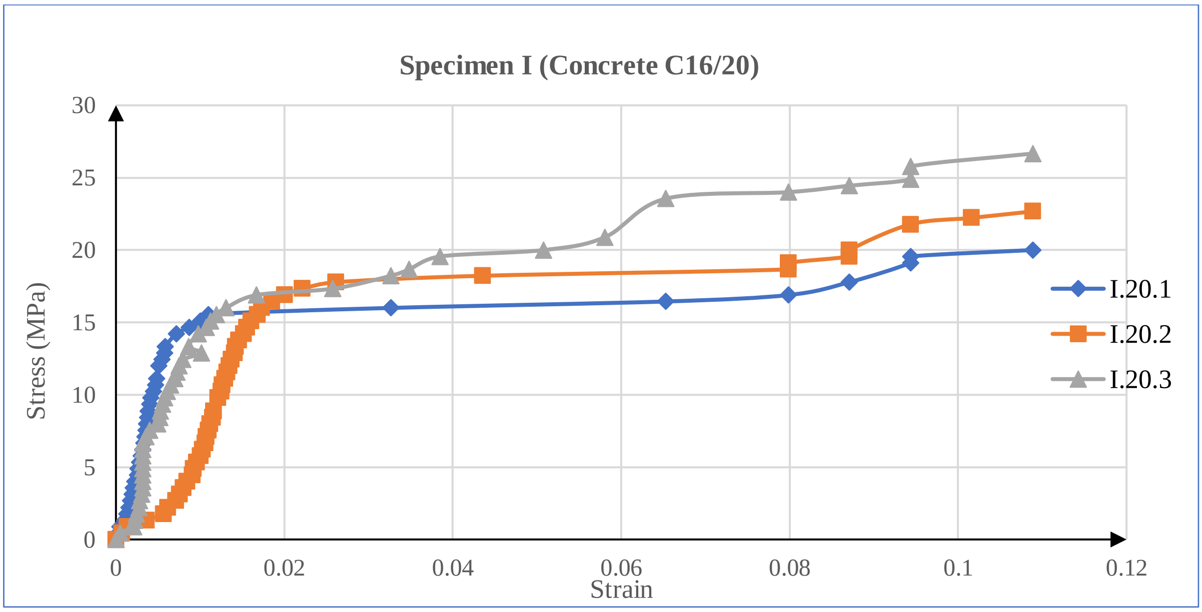

Specimen H (Concrete C8/10)
Stress (MPa)
Strain
H.10.1
H.10.2
H.10.3
30
25
20
15
10
5
0
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035

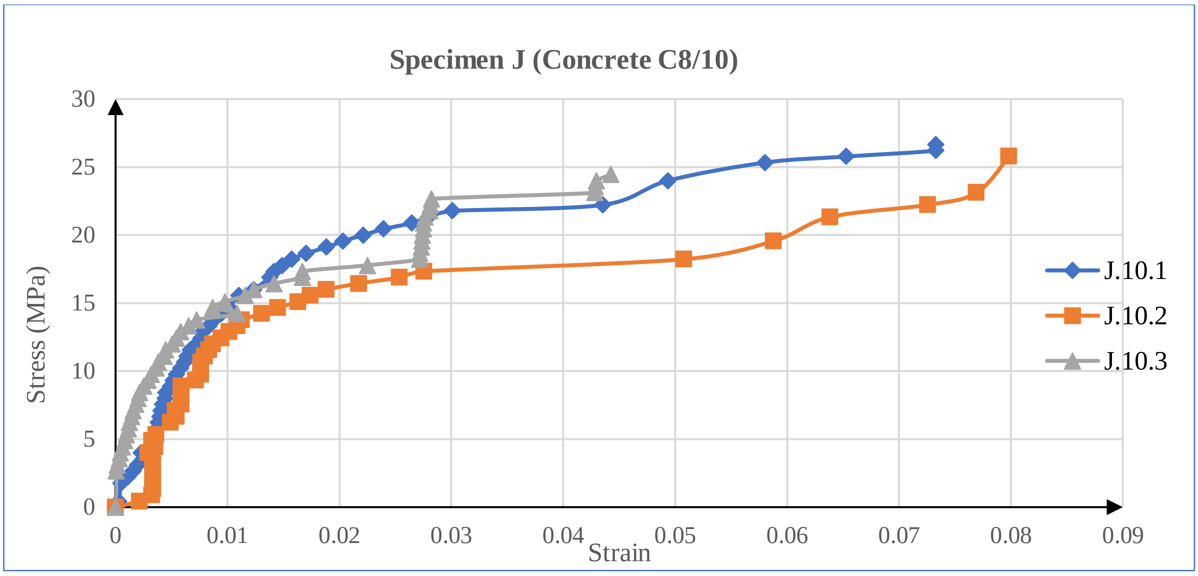

Specimen H (Concrete C12/15)
Stress (MPa)
Strain
H.15.1
H.15.2
H.15.3
30
25
20
15
10
5
0
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05

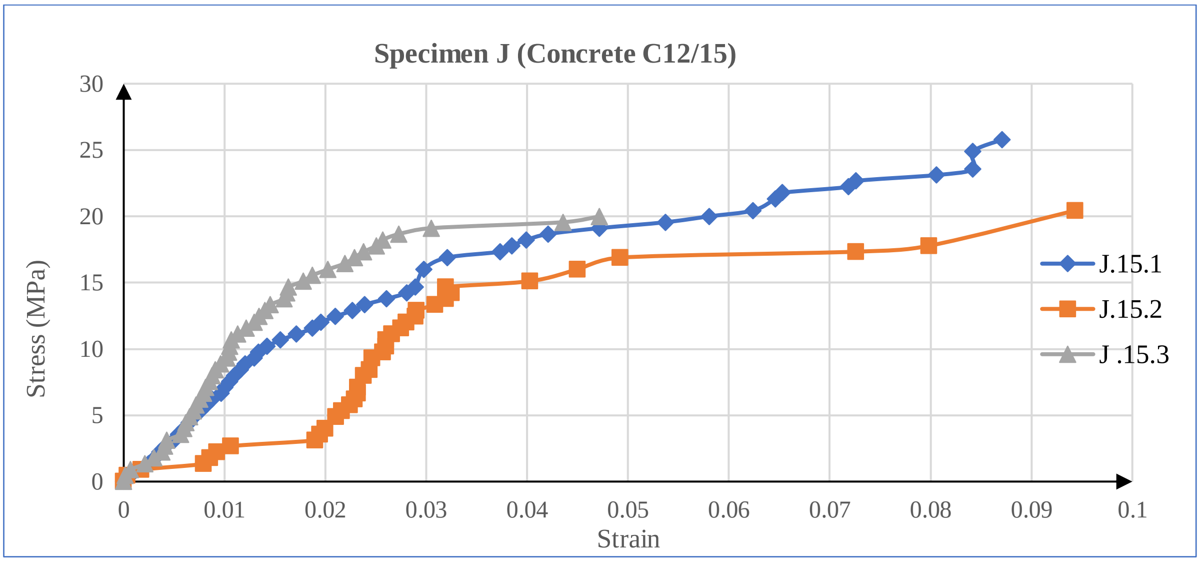

Specimen H (Concrete C16/20)
Stress (MPa)
Strain
H.20.1
H.20.2
H.20.3
30
25
20
15
10
5
0
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05

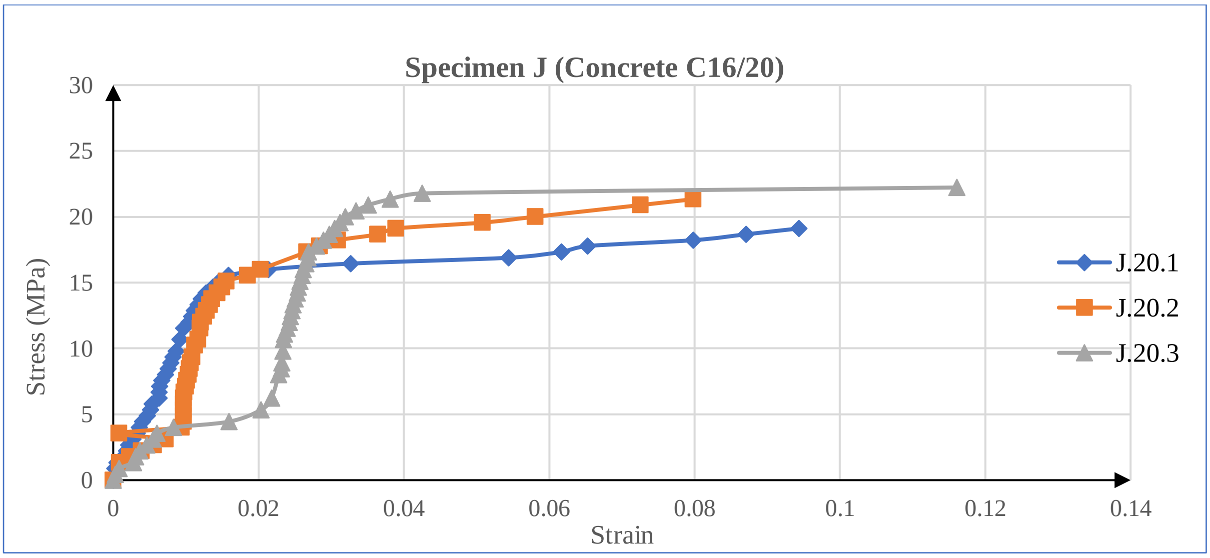

Specimen I (Concrete C8/10)
Stress (MPa)
30
25
20
15
10
5
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
Strain
I.10.1
I.10.2
I.10.3


Specimen I (Concrete C12/15)
Stress (MPa)
30
25
20
15
10
5
0
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
Strain
I.15.1
I.15.2
I.15.3


Specimen I (Concrete C16/20)
Stress (MPa)
30
25
20
15
10
5
0
0
0.02
0.04
0.06
0.08
0.1
0.12
Strain
I.20.1
I.20.2
I.20.3


Specimen J (Concrete C8/10)
Stress (MPa)
Strain
0
5
10
15
20
25
30
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
J.10.1
J.10.2
J.10.3


Specimen J (Concrete C12/15)
Stress (MPa)
Strain
0
5
10
15
20
25
30
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.1
J.15.1
J.15.2
J .15.3


Specimen J (Concrete C16/20)
Stress (MPa)
Strain
0
5
10
15
20
25
30
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
J.20.1
J.20.2
J.20.3

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