# STRENGTH AND DEFORMATION CHARACTERISTICS OF RECYCLED POLYETHYLENE FIBRE REINFORCED CONCRETE



By

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## DECLARATION

I hereby declare that this submission is my own work towards the MPhil and that, to the best of my knowledge, it contains no material previously published by another person nor material which has been accepted for the award of any other degree of the University, except where due acknowledgement has been made in the text.

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# ABSTRACT

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σ = theoretical tensile strength of concrete c

= size of crack

$\gamma$ = unit weight of concrete

$\gamma_m$ = material factor $E_s$ =

elastic modulus of steel n =

modular ratio

$E_c$ = elastic modulus of concrete $\sigma_{fl}$ =

flexural strength (modulus of rupture)

$l_c$ = critical length of fibre below which fibre pull-out will occur

$l$ = length of fibre $\sigma_{fu}$ =ultimate tensile strength of fibre $V_f$ =

volume fraction of fibre in the composite r = radius of fibre τ =

interfacial shear bond strength between fibre and matrix X=

minimum crack spacing x= neutral axis depth

$V_m$ = matrix volume fractions in the composite

$A_f$ = cross sectional area of fibre

$P_f$ = perimeter of fibre

$\sigma_{mu}$ = ultimate tensile strength of matrix
R = modulus of rupture, ($N/mm^2$) l = span

between centers of lower supports L = span

of beam between supports $b$ = average

width of prism $d$ = average depth of prism

$f_{cu}$ = cube compressive strength P = point

load

$P_{ult}$ = ultimate load at collapse

$P'_{ult}$ = theoretical ultimate load $\nu_c$ = shear

capacity of concrete V = shear resistance of

concrete $f_{yv}$ = tensile strength of shear

reinforcement $S_v$ = shear link spacing

$A_{sv}$ = area of two legs of shear link $M_{ult}$ = ultimate

moment of resistance ω = load per unit length due

to self-weight of beam z = distance from point

load to the nearest support b = average width of

beam or cube

$\rho_b$ = balanced reinforcement ratio

$_{cu}$ = ultimate strain of concrete = 0.0035

$f_y$ = yield strength of steel (MPa)

$_y$ = yield strain of steel

$\beta_1$ = the ratio of the depth of equivalent rectangular stress block to the neutral axis depth $\rho_a$ =

actual reinforcement ratio

$A_s$ = total cross sectional area of tension steel rebars

$d_s$ = effective depth of the steel rebar level

$\Delta_c$ = mid-span deflection $\delta''_{ult}$ =

theoretical deflection at failure $\delta_{ult}$ =

experimental deflection at failure $\delta_{cr}$ =

deflection at first crack h = depth of

rectangular section

$I_g$ = second moment of area, gross

$I_{cr}$ = second moment of area of a cracked section

$\Delta V$ = loss in volume after 16 cycles, $mm^3$

$\Delta m$ = loss in mass after 16 cycles, in g; and

PR = density of the specimen, in $g/mm^3$

MOR = modulus of rupture

PET = polyethylene terephthalate

PFRC = polyethylene fibre reinforced concrete

FRC = fibre reinforced concrete

BS = British Standards International

ASTM = American Society for Testing and Materials

C-S-H = Calcium Silicate Hydrate
LM = low modulus HM = high modulus ur =

under-reinforced without fibre dose $ur\text{-}f^1$ =

under-reinforced with low dose fibre $ur\text{-}f^2$ =

under-reinforced with medium dose fibre $ur\text{-}f^3$ =

under-reinforced with high dose fibre

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# CHAPTER 1 - INTRODUCTION

## 1.1 Background

This study is an investigation into the strength and deformation characteristics notably compressive strength, flexural strength, post crack toughness, cracking pattern and crack propagation, durability and water absorption, and surface abrasion resistance of polyethylene fibre-reinforced concrete.

Concrete is a brittle material with low tensile strength and strain capacity (Hamoush et al, 2010). The low tensile strain capacity makes concrete susceptible to cracking and shrinkage resulting in the deterioration of the concrete and eventual loss of durability. According to a study by Barris et al (2009), the role of fibres in concrete is to bridge these cracks when the ultimate strain of the concrete has been exceeded. Further studies have shown that the fibres have the contribution in improving the deflection, ultimate loading and ductility of concrete structures. Kim et al (2010) and Fraternalli et al (2011) have both conducted some studies on the use of polyethylene terephthalate (PET) as fibres in reinforced concrete. Both studies however used factory reproduced PET in strip form. The use of shredded polyethylene fibres recycled from waste sachet bags in concrete is yet to be studied and this research is therefore structured to investigate that.

The ability of fibre reinforced concrete to absorb energy (toughness) has long been recognized as one of the most important benefits of the incorporation of fibres into concrete (Golpalaratham and Gettu, 1995). It is also evident in „the Concrete Society report of 2007" that steel and synthetic fibres have been used in concrete floor slabs to great effect in providing crack control since the 1970"s and 1980"s. Kobayashi and Cho (1981) in their work titled „Flexural behavior of polyethylene fibre reinforced concrete" alluded to some advantages of synthetic fibres such as nylon and polypropylene in cementitious composites as:

1. Excellent resistance of the fibres to deterioration in a cement matrix, or in aggressive environments;
2. Improved post-cracking ductility;
3. Improved impact resistance.

## 1.2 Problem Statement

Concrete as mentioned earlier is a brittle material with low tensile strength and strain capacity. This makes the concrete susceptible to cracking and shrinkage resulting in the deterioration of the concrete and eventual loss of durability. To ensure durability of concrete, it is very essential to control the crack propagation process and be able to predict the cracking pattern (crack width, crack length, and crack spacing). Conventional reinforcement has used steel bars in tension zones but this has been proven to be inadequate as it provides only one-dimensional reinforcement and not the three-dimensional reinforcement effect required in most structural members. Secondly, to achieve adequate crack control in structural elements, large amounts of conventional reinforcement are needed, especially in structures where only very small crack widths ($w \leq 0.1$mm) are allowed. The use of polyethylene products is widespread in the Ghanaian society. The country is estimated to generate about 2000 to 4000 tonnes of plastic waste every month (WACEE, 2013). Few of the uses of polyethylene in Ghana include the wrapping of both raw food stuffs and cooked foods, packaging of mineral water, and packaging of consumables such as toiletries, confectionaries and a wide range of other products. Most of the polyethylene products are however non-biodegradable, non-compactible, and non-destroyable hence their disposal presents a huge challenge to the Waste

Management Authorities and the nation as a whole. Some of the dangers posed by the indiscriminate disposal of the polyethylene products include flooding as a result of choking of drainage ways, pollution of water bodies including ground water resources, upsetting the food chain, and causing land and air pollution. They are poisonous and therefore kill animals and cause ill-health in humans and impairment of soil nutrients for agricultural purposes as a result of over centuries of non-decay of buried plastics. Studies by Integrated

Waste Management Authority (2016) indicate that 86% of ocean debris is plastic and over

1,000,000 seabirds and marine mammals die each year from plastic ingestion of entanglement. There is therefore the need for re-cycling of polyethylene products in Ghana if the current level of usage of plastics in the country is to be sustained. In Kumasi and other parts of the country, polyethylene products are collected by human scavengers, melted and recycled to produce tyres, bottles and sachet water bags but this alone is not adequate. There is the need for more recycling options.

This research is therefore geared towards addressing the strength and performance problems of conventionally reinforced concrete structural elements and at the same time providing an avenue for recycling of the used polyethylene sachet water bags thereby solving the menace of polyethylene waste in the country.

## 1.3 Aims and Objectives

This research is aimed at exploring the use of waste sachet bags (polyethylene product) as a fibre in concrete. The general objective of the research is to determine the effect of different volume fractions of the fibre on the strength and deformation characteristics of concrete. The specific objectives of the research are to determine the effect of the fibre on the following;

- Compressive strength of concrete
- Flexural strength of concrete
- Post crack toughness of concrete
- Crack propagation, cracking pattern and crack distribution in concrete
- Water absorption of concrete and
- Surface abrasion resistance of concrete

## 1.4 Scope

Compressive strength of plain and polyethylene fibre-reinforced concrete specimens were determined using 150mm cubes to BS 1881- part 116: 1983. The modulus of rupture (flexural tensile strength) was determined from flexural test on concrete prisms of dimensions 100mm × 100mm × 300mm. Reinforced concrete beams containing different volume fractions of polyethylene fibres and measuring 150mm × 200mm × 2500mm were tested to study the loaddeflection characteristics and assess their crack propagation and post crack toughness indices.

Water absorption and by extension the durability of the polyethylene fibre reinforced concretes and plain concrete were examined using 150mm cubes to BS 1881-part122:1983. Finally resistance of the polyethylene fibre-reinforced concrete (PFRC) to surface abrasion was also investigated using 100mm cubes.

## 1.5 Overview of Thesis

The introduction of the thesis is presented in Chapter 1 which explains the Background, Problem Statement, Aims and Objectives, and Scope of the study. Review of existing literature on the topic is presented in chapter two. Chapter three explains the research methodology. It describes the specimen and test details for the Modulus of Rupture (MOR), Compressive strength, Post Crack Toughness, Surface Abrasion Resistance and Water absorption developed for this study, including specimen design, materials used, test parameters, test procedure, and results computation. Chapter four contains the test results and the analysis, including comparison of the measured results against theoretically computed values and discussions of observed phenomena against documented phenomena. Chapter five contains the summary, conclusions and recommendations.

# CHAPTER 2 - LITERATURE REVIEW

## 2.1 Introduction

The idea of creating a new and more ductile concrete using short, discrete, and randomly distributed fibres was first patented by the American A. Berard in 1874 (Jansson, 2008). Fibres have been used as reinforcement in concrete since the ancient times. Historically, horse hair was used in mortar and straw in mud bricks. In the 1900‟s asbestos was used in concrete products such as pipes and for roofing materials but was later stopped due to the health risks associated with the use of asbestos. In the 1950‟s fibre use in concrete was one of the topics of interest and since the 1960‟s, steel, glass and synthetic fibres such as polyethylene fibre have been used for both structural and non-structural purposes (Lofgren, 2005). Early works by Goldfein (1963) on synthetic fibres in Potland cement led to a patent covering polyolefin fibres (polypropylene, polyethylene, nylon, polyvinyl chloride etc). Research into new fibre reinforcement for concrete continues today and available literature on the topic is therefore varied and extensive.

Fibre reinforced concrete (FRC) is a cement-based composite material reinforced with short, discrete, and usually random distributed fibres within a concrete matrix (Jansson, 2008). The fibres are added to bridge discrete microcracks and thereby provide for increased control of the fracture process and also to increase the fracture energy thereby yielding in a more ductile behavior.

In order to ensure durability of concrete, it is very essential to control the crack propagation process and be able to predict the crack pattern (crack width, crack length, and crack spacing).

To achieve crack control, large amounts of conventional reinforcement are needed, especially in structures where only very small crack widths ($w \leq 0.1$mm) are allowed such as water retaining structures. The bad side to this technique is that: structural dimensions often need to be larger than what is needed for load carrying capacity in order to fit all the steel; heavy labour is required in placing the steel; and also difficulties in pouring the concrete past the tightly packed reinforcement bars in the formwork. By using fibres in combination with or instead of conventional reinforcement to control cracking, however, these drawbacks may be reduced or even eliminated completely.

## 2.2 Concept of Fibre Reinforcement in Structural Concrete

Structural concrete is a strong material in compression but very weak in tension ( Richardson 2005). There is the need therefore, to provide some sort of reinforcement to mitigate the rather low tensile strength. The function of the reinforcement has been grouped into two by Richardson (2005) as: first, the control of cracks which will result in improved durability; and second, to resist tensile forces resulting from applied loads (ie increase load bearing capacity).

Theoretical works covering the principles of fibre reinforcement were first developed and summarized by Hannant (1978). Current understanding of the behavior of the fibre-matrix interface is based on studies by Bentur et al. (1985), Golpalaratham and Shah (1987), Namur and Naaman (1989), Bentur and Mindess (1990), Stang and Shah (1990), Wang et al. (1990a), Wang et al. (1990b), Li et al. (1993), Leung and Li (1991), Chanvillard and Aitcin (1996), Kullaa (1994), Li and Stang (1997) and Grunewald (2004).

Theoretical studies into flexural behaviour of fibre reinforced concrete (FRC) can be obtained from the works of Lok and Pei (1998) and Lok and Xiao (1999). Analytical models developed for depicting the flexural behavior of FRC can be found in Ezeldin and Shah (1995) and Zhang and Stang (1998). Cracking of FRC under flexure has also been investigated by Stang et al (1995) and Rossi (1999).

Potential benefits of fibres in concrete include improved crack control and the possibility of designing more slender structures, three dimensional reinforcement of structural elements, reduced plastic cracking, controlling shrinkage and thermal cracking, increased impact resistance, reduced permeability, increased toughness and ductility, increased abrasion resistance, increased freeze-thaw resistance, enhanced post-crack residual strength, and enhanced fatigue résistance. Structural applications of FRC include tunnel linings (Nanakorn and Horii, 1996**;** Kooiman, 2000) and suspended flat slabs with reduced conventional reinforcement (Gossla, 2006).

### 2.2.1 Micro-cracking and Crack Propagation Process

Concrete is a composite material consisting of two phases: hydrated cement paste and aggregates (Neville and Brooks, 2010). Properties of concrete are therefore governed by the properties of the two phases and also by the presence of interfaces between them. Very fine bond cracks exist at the interface between the coarse aggregate and the hydrated-cement paste prior to load application. These cracks result from the differential volume changes between the cement paste and the aggregates due to differences in their stress-strain behavior and also as a result of thermal and moisture movement. These cracks are stable and remain stable at stress levels below 30 percent of the ultimate strength of the concrete. The cracks at this stage do not grow and the stress remains proportional to the strain.

At higher stresses (above 30 percent of ultimate strength of concrete), the stress-strain relation for concrete becomes curvilinear even though the stress-strain relations for the aggregate alone and for the cement paste alone remain linear (Figure 2.1). This behavior of concrete can be explained as a result of the development of micro-cracks. At such higher stress values, the bond cracks at the interface between the two phases begin to increase in length, width, and number. As a consequence, the strain increases at a faster rate than the stress. At this stage, there is a generally slow propagation of micro-cracks although stable under sustained loading.

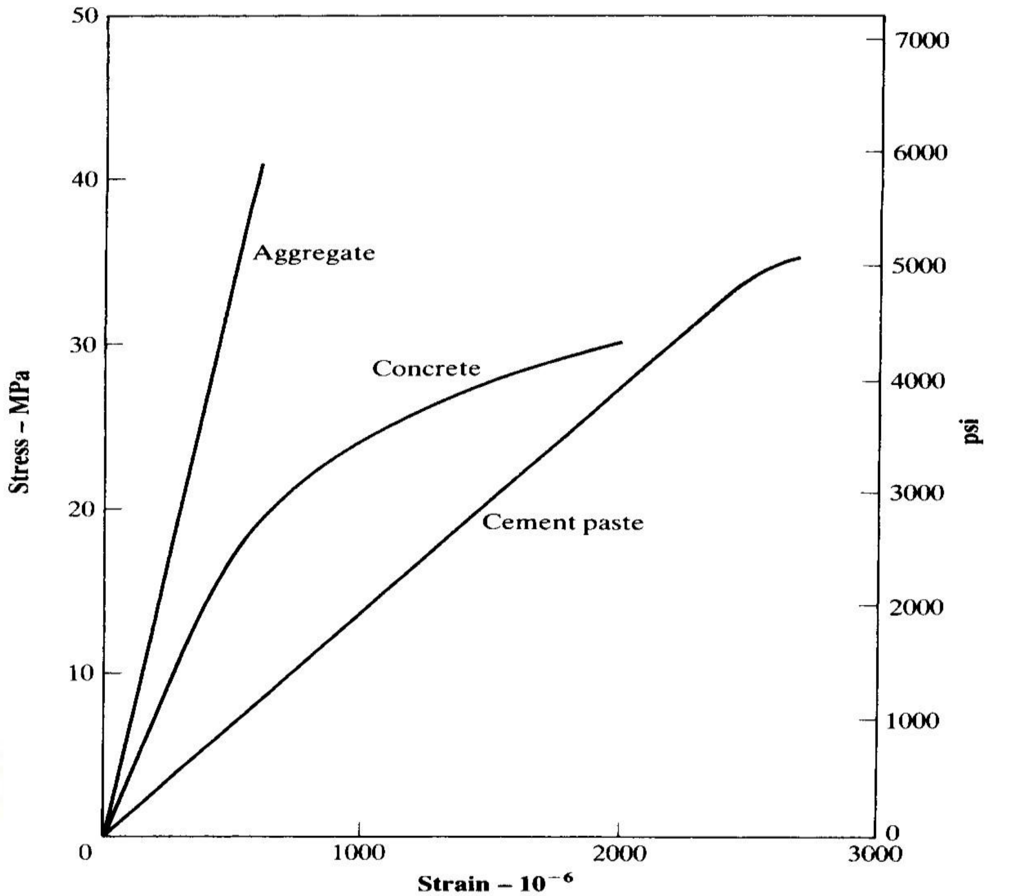


**Figure 2.1: Stress-strain relations for cement paste, aggregate, and concrete (Neville and Brook, 2010).**

At stress levels of 70 to 90 percent of the ultimate strength of concrete, cracks open through the cement paste and the aggregates bridging the bond cracks so that a continuous crack pattern is formed. This stage marks the beginning of faster or rapid propagation of cracks which eventually result in the collapse or failure of the concrete. It is therefore evident that voids and microcracks in concrete serve as crack initiators.

### 2.2.2 Theory of Strength of Brittle Materials

Concrete is brittle as a result of the brittle nature of the cement paste and the aggregates. The strength of such brittle materials can be determined theoretically using Griffith theory as follows (Griffith 1921):

$$\sigma = \sqrt{\frac{2E\gamma}{\pi c}} \qquad \textbf{(2.1)}$$

where $\sigma$ = tensile strength of the concrete, (N/mm$^2$)

c = size of crack, (mm)

$\gamma$ = unit weight of concrete, (kN/mm$^3$)

E = elastic modulus of concrete, (kN/mm$^2$)

It can be seen that, the tensile strength depends on the size of the crack. Concrete has a lower tensile strength due to the existence of large cracks, c in the matrix. If the size of the cracks within the matrix can be reduced, the tensile strength of the concrete can be increased.

Fibres are therefore used to bridge the cracks thereby preventing the elongation of the cracks and increase of crack width. This is the philosophy behind the design and use of FRC. The purpose of reinforcing the cement-based matrix with fibres is to increase the tensile strength of the matrix by delaying the growth of cracks and also to increase the toughness by transmitting stress across the cracked section so that much larger deformation is possible beyond the peak stress. Crack control is therefore not targeted at eliminating cracks caused by intrinsic stresses nor is it to increase the load-carrying capacity of the concrete. Crack control is rather targeted at replacing the random pattern of relatively large cracks with a more deliberately structured pattern of closely spaced fine cracks. This in turn makes concrete less water permeable and hence more durable.

## 2.3 Fibres for Concrete

Fibres for use in concrete have been classified into three (3) by the American Society for Testing and Materials (ASTM C116 - 2003) as follows:

Type I: Steel fibres

Type II: Glass fibres, and

Type III: Synthetic fibres

Steel fibres are categorized into five types as follows:

- ✓ Cold drawn wire
- ✓ Cut sheet
- ✓ Melt extracted
- ✓ Mill cut and
- ✓ Modified cold drawn wire

Steel fibres have ultimate strength of 700MPa, elastic modulus of 200GPa and percentage elongation of 7.0% (Table 2.1). Their length ranges between 20mm and 60mm, diameter between 0.25mm and 0.9mm, and aspect ratio between 30 and 250. Typical volume fractions of steel fibres used in concrete range from 0.5% to 2.0%, which is equivalent to 40kg/$m^3$ to 160kg/$m^3$ by weight. Steel fibres are heavier than glass and synthetic fibres and are applied for both structural and non-structural purposes. Their shortfall is however that, they are susceptible to corrosion just like the conventional steel reinforcement bars.

Glass fibres are chemically inorganic fibres obtained from molten glass. They have high tensile strength, high modulus of elasticity, high impact resistance, high shear strength, high water resistance, high thermal conductivity, less creep, light weight, low density, high resistance to corrosion, and high fire endurance. They are mainly used for non-structural applications such as exterior building façade panels and architectural ornaments. Glass fibres come in the form of ravings, chapped strands, and yarns. Glass fibres however require a highly technological process to produce as compared to steel and synthetic fibres.

Synthetic fibres are broadly classified as polymer fibres and divided into micro fibres and macro fibres. Class 1 micro fibres are for non-structural applications and have diameter less than 0.3mm. They are employed for plastic shrinkage control, impact resistance, anti-spalling and passive fire control. Class 2 macro fibres are for structural applications and have diameter greater than 0.3mm. They can be used to replace crack control mesh and also conventional
reinforcement in concrete.

### 2.3.1 Properties of Synthetic Fibres

***Mechanical Properties***

Synthetic fibres (except for polyaramid also known as Kevlar) generally, are characterized by low modulus of elasticity and high elongation (Table 2.1). The modulus of elasticity for most cementitious materials is in the range of 15 to 30 MPa. It is therefore highly unachievable to obtain a higher strength composite with the relatively low modulus fibres without having to engineer the fibres. The advantage of the fibres lies in increasing properties such as strain capacity, toughness, and crack control.

**Table 2.1: Properties of Concrete Fibres (Ludirdja and Young, 1992)**

| Fiber | Density | Tensile Properties |
|---|---|---|

| | (g/cm$^3$) | Ultimate Strength MPa | Elastic Modulus GPa | Ultimate Elongation (%) |
|---|---|---|---|---|
| Polyethylene LM | 0.95 | 500 | 5 | - |
| HM | 0.95 | 500 | 15-30 | - |
| Polypropylene LM | 0.91 | 600 | 7 | 21 |
| HM | 0.91 | 600 | 15 | - |
| Polyvinyl alcohol | 1.26 | 1500 | 30 | - |
| Polyacrylic | 1.20 | 600 | 10 | 13 |
| Nylon (Polyamide) | 1.14 | 1000 | 6 | 10 |
| Kevlar (Polyaramid) LM | 1.40 | 3000 | 70 | 4 |
| HM | 1.40 | 3000 | 130 | 2.5 |
| Polyester | 1.35 | 1100 | 10 | 24 |
| Steel | 7.90 | 700 | 200 | 7 |
| Glass | 2.70 | 1100 | 70 | 2.5 |

LM = low modulus HM = high modulus

***Physical Properties***

Synthetic fibres are characterized by low density (Table 2.1) so that a relatively low mass of fibres yields a high volume of fibre in concrete. Fibres are mostly very flexible hence fibre breakage during mixing is not a problem.

***Chemical Durability***

The minimum pH in a cement paste is about 12.3. This highly alkaline environment is detrimental to many organic materials and polymers. Polyesters, polyacrylics, and polyamides can undergo alkaline hydrolysis under such conditions. The rate of alkaline hydrolysis is very much affected by temperature change. Hydrolysis may be slow at room temperature but significantly accelerated at higher temperature. A study by Wang et al. (1987) on polyacrylic fibres showed only slight loss of strength at $20^0$C after 2 months but a significant loss at $50^0$C in the same period. Hydrophobic polymers such as polyethylene and polypropylene fibres are highly resistant to such alkaline conditions.

### 2.3.2 Primary Reinforcement using Synthetic Fibre

Synthetic fibres are usually used as primary reinforcement in thin sheet elements where conventional steel reinforcement is not feasible (Ludirdja and Young, 1992). Volume fractions of fibre above 5 percent are required to ensure there is adequate enhancement of the flexural properties. Development of high modulus fibres such as Kevlar allows the replacement of steel and glass as reinforcement but at high cost that makes current use of synthetic fibres as primary reinforcement uneconomical.

### 2.3.3 Secondary Reinforcement using Synthetic Fibre

Secondary reinforcement is required for control of cracking caused by intrinsic tensile stresses such as plastic shrinkage, drying shrinkage and temperature changes. For this purpose, lower volume fractions in the range 0.1 to 0.3 percent can be used.

## 2.4 Tensile and Compressive Strengths

There appears to be varied opinion when it comes to the effect of synthetic fibres on the tensile and compressive strengths of concrete. Some studies (Hasaba et al., 1984; Zollo, 1984; Ramakrishnan et al. 1987) have reported no significant improvements in tensile or compressive strength when low volumes of polypropylene fibres were added to concrete. Other studies (Wiss et al. 1985) have reported slight decrease (less than 10 percent) in compressive strength of polypropylene fibre concrete over control specimens without the fibre.

## 2.5 Modulus of Rupture (MOR)

Theoretically, the flexural tensile strength or modulus of rupture of a random three dimensional fibre reinforced brittle matrix composite can be calculated using the relation developed by Hannant (1978) as follows:

$$\sigma_{fl} = 2.44\left(1 - \frac{l_c}{2l}\right)\sigma_{fu} \times V_f \qquad \textbf{(2.2)}$$

where $\sigma_{fl}$ =flexural tensile strength (modulus of rupture), (N/mm$^2$)

$l_c$ = critical length of the fibre below which fibre pull-out will occur, (mm)
$l$ = length of fibre, (mm) $\sigma_{fu}$ =ultimate tensile strength of the fibre, (N/mm$^2$) $V_f$ = volume fraction of the fibre in the composite, (%)

And

**(2.3)**

where

r = radius of the fibre, ($^0$) τ = interfacial shear bond strength between fibre and matrix, (N/mm$^2$)

Typical values of τ are given in Table 2.2 for selected synthetic fibres in commercial use.

**Table 2.2: Interfacial Bond Strengths for Synthetic Fibres (Ludirdja and Young, 1992)**

| Fibre type | Fibre Geometry | Shear Bond Strength, τ (MPa) |
|---|---|---|
| Polypropylene | Monofilaments | 0.7 – 1.2 |
| | Rovings | 0.1 |
| | Fibrillated | 0.2 – 0.4 |
| Polyacrylic | Monofilaments | 3 – 4 |
| Polyester | Monofilaments | 0.1 – 0.2 |
| Polyamide | Monofilaments | 0.1 – 0.2 |
| Polyaramid | Rovings | 3.8 |

Flexural tensile strength of FRC has been investigated by different researchers. Zollo (1984), Hasaba et al. (1984), and Wiss et al. (1985) all reported greater flexural tensile strength values in concrete containing polypropylene fibres compared to unreinforced concrete, especially when the specimens were cured in air. Their investigation also showed that low volume fraction fibre concretes (below 0.5%) yielded small increments in flexural tensile strength compared to concretes containing higher volume fraction of fibres (1% and above).

## 2.6 Crack Spacing and Pattern

Fibre reinforced concrete is expected to have finer crack widths and short-spaced crack pattern than plain concrete. This is due to the bridging action of the fibres at the onset of cracking. The formula proposed by Hannant et al. (1978) for computing theoretical crack spacing for a fibre reinforced concrete is given as follows:

$$X = \frac{V_m}{V_f} \times \frac{\sigma_{mu} A_f}{\tau P_f} \qquad \textbf{(2.4)}$$

where X= minimum crack spacing, (mm)

$V_m$,$V_f$ = matrix and fibre volume fractions, (%) in the composite respectively

$\tau$ = interfacial shear bond strength, (N/mm$^2$)

$\sigma_{mu}$ = ultimate tensile strength of the matrix, (N/mm$^2$)

$A_f$, $P_f$ = cross sectional area, (mm$^2$) and perimeter, (mm) of the fibre respectively,

The crack spacing is directly related to the radius of the fibre for circular cross sections as shown in equation 2-5. It can be seen therefore that the finer the fibre, the smaller the crack spacing.

$$\frac{A_f}{P_f} = \frac{r}{2} \qquad \textbf{(2.5)}$$

## 2.7 Plastic Shrinkage Cracking

Plastic shrinkage cracking is caused by excessive loss of water from fresh concrete due to evaporation (Ludirdja and Young, 1992). High temperatures such as those experienced during summer months increase the rate of evaporation of moisture to the extent that it exceeds the rate of bleeding of water to the surface of concrete. Local drying at the surface will therefore result in plastic shrinkage cracking. The magnitude of tensile stresses required to cause this type of cracking is quite low but exceeds the tensile strength of the concrete in the plastic stage, hence the cracking of the concrete. Because the magnitude of the tensile stresses is low, synthetic fibres have the potential to prevent plastic shrinkage cracks.

Reduction in plastic shrinkage cracking of up to 25 percent has been recorded using 0.3 percent volume fractions of polypropylene fibres in concrete (Zollo and Itler, 1986). The report also pointed out a general stabilization of the matrix, preventing segregation and minimizing bleeding. Kraai (1985) also reported similar reduction in the extent of cracking but reiterated that different fibres may yield different improvements in plastic shrinkage cracking of concrete. Studies sponsored by the Fibermesh Company (1987) revealed that cracking in a control slab without any fibre started earlier (about 2.5 hours after end of casting) and the cracking intensified up to about 4 hours after end of casting. However, slabs that contained fibrillated polypropylene fibres had virtually no cracking 24 hours after end of casting.

## 2.8 Drying Shrinkage Cracking

Drying shrinkage cracking is caused by restraining the volume reduction that accompanies loss of water from hardened concrete (Ludirdja and Young, 1992). Goeb (1989) reported improvement in drying shrinkage cracking as a result of fibre addition though not very appreciable. Drying shrinkage usually results in unequal volume changes in green/fresh concrete, creating restraints in it. These restraints to volumetric changes develop tensile stresses that normally exceed the tensile strength of concrete, and hence cracking results. Inclusion of fibres in the concrete helps to resist the tensile stresses in the concrete.

## 2.9 Post Crack Toughness

Fibre reinforced concrete is expected to be more ductile than unreinforced concrete. ASTM C1018-89 provides for measurement of ductility by computing the area under the load-deflection curve up to a certain deflection and dividing it by the area under the load-deflection curve up to the first crack deflection. This procedure is well illustrated in figure 2.2.

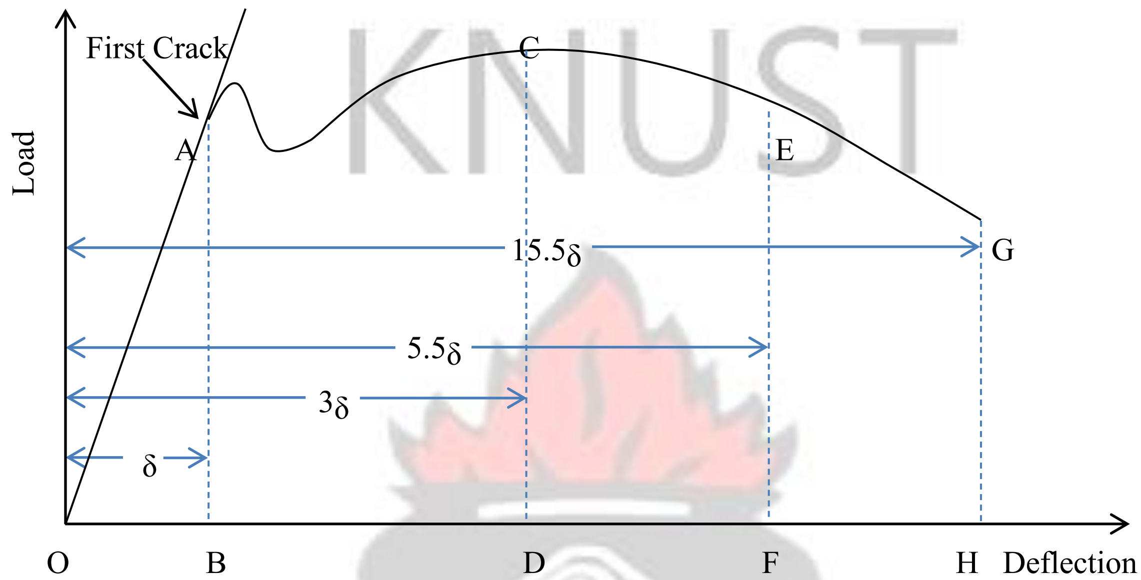


**Figure 2.2: Analysis of Load-Deflection Curve (ASTM C1018)**

Relative values of toughness (toughness index) are then computed as follows:

$$I_5 = \frac{Area\ OACD}{Area\ OAB} \quad \textbf{(2.6)}$$

$$I_{10} = \frac{Area\ OAEF}{Area\ OAB} \quad \textbf{(2.7)}$$

$$I_{30} = \frac{Area\ OAGH}{Area\ OAB} \quad \textbf{(2.8)}$$

For purely elastic-plastic material, $I_5 = 5$, $I_{10} = 10$, and $I_{30} = 30$.

For purely brittle material, $I_5 = I_{10} = I_{30} = 1$.

Table 2.3 shows the toughness test results for concretes reinforced with polypropylene fibres (Ludirdja and Young, 1992) . It can be observed from the table that, the plain concrete with 0% fibre content exhibited a purely brittle behavior with toughness indices at $I_5 = I_{10} = I_{30} = 1.0$. The inclusion of the polypropylene fibres at a volume fraction of 0.1% increased the toughness to $I_5$ = 3.519. This is a significant improvement in the ductility of the concrete when compared to a purely elastic-plastic material with toughness index $I_5$ = 5.0. At toughness index $I_{10}$, the 0.1% polypropylene fibre reinforced concrete recorded a toughness value of 5.11 compared to 10.0 for a purely elasti-plastic material and at toughness index $I_{30}$, it recorded a toughness value of 9.0 compared to 30.0 for a purely elastic-plastic material. The polypropylene fibre reinforced concrete therefore shows an intermediate behavior. This goes to show that fibre reinforced concrete is more ductile than concrete without fibres further buttressing the need for fibres in concrete.

**Table 2.3: Toughness Indices of Polypropylene Fibre Reinforced Beams (Ludirdja and Young, 1992)**

| Series | Fibre Content (%) | First Crack Toughness (In/lb) | $I_5$ | $I_{10}$ | $I_{30}$ | $I_{10}$ / $I_5$ | $I_{30}$ / $I_{10}$ |
|---|---|---|---|---|---|---|---|
| I | 0 | - | 1.000 | 1.00 | 1.00 | 1.00 | 1.00 |
| | 0.1 | 17.28 | 3.519 | 5.11 | 9.00 | 1.47 | 1.76 |
| | 0.5 | 24.65 | 3.330 | 5.32 | 11.99 | 1.60 | 2.30 |
| | 1.0 | 19.79 | 3.510 | 6.53 | 16.49 | 1.87 | 2.53 |

## 2.10 Concrete Reinforcement Practices in Ghana

The construction industry in Ghana is dominated largely by reinforced concrete structures which are predominantly reinforced with steel bars, steel tendons and steel meshes. The raw material for producing the bars, tendons and meshes is imported into the country at a great cost in foreign exchange to the nation and the steel industries.

There is therefore sufficient justification for a study into Fibre Reinforced Concrete made from recycled waste polyethylene bags to complement steel bars for reinforcement of concrete in the country. This will result in a reduction in the importation of steel for concrete reinforcement at the same time making efficient use of the polyethylene waste.

# CHAPTER 3 - METHODOLOGY

## 3.1 Introduction

### 3.1.1 Modulus of Rupture

Modulus of Rupture is one measure of the tensile strength of concrete. The two commonly used test methods for modulus of rupture test are the 3 point loading test (centre-point loading) and the 4 point loading test (third-point loading). Both tests have however been proven (Department of Transport California, test 523: 2012) to give equivalent answers. The test essentially entails loading a beam with span length at least three times the depth until it fails by crushing. In this study the methods and procedures outlined in the American Society for Testing and Materials (ASTM C78-2009) were followed. The modulus of rupture (MOR) of concrete is expressed in $N/mm^2$.

### 3.1.2 Compressive Strength

The compressive strength of concrete is the ultimate load the concrete can withstand per its cross sectional area before it collapses. The compressive strength of concrete is usually determined by crushing 150×300mm cylinders or 150mm cubes in a compression machine. The compressive strength of the specimens in this study was determined using the procedures outlined in the British Standards Institution BS 1881-part 116:1983. The compressive strength of concrete is expressed in $N/mm^2$.

### 3.1.3 Water Absorption

The purpose of this test is to determine the relative absorption of water and other fluids by concrete when the concrete surface is subjected to moist conditions. The permeability of plain and polyethylene fibre reinforced concretes in this research was examined per the British Standards Institution‟s BS 1881-part 122:1983 guidelines to see if polyethylene fibres interfere with the flow of water through the concrete matrix. It has been said that "Excellent long term performance in concrete pavements is associated with both concrete strength and durability properties like permeability and chloride ion resistance" (Morh et al, 2000). It goes to suggest therefore that, low permeability in concrete equates to durability of the concrete all other things being equal.

### 3.1.4 Post Crack Toughness

Toughness is an indication of the energy absorption capability of a concrete specimen. The toughness or post-crack ductility was examined using a Four-Point Loading Test per the American Society of Testing and Materials ASTM C1018(1997) guidelines to provide the loaddeflection data using 150mm×200mm×2500mm beam specimens.

### 3.1.5 Surface Abrasion Resistance

Surface abrasion may be defined as surface wear that causes progressive loss of materials from concrete surface. It results from dynamic forces and displacements such as rubbing, scraping, skidding, or sliding of objects on the concrete surface. Surface abrasion resistance is a measure of the resistance of concrete pavements and water retaining structures to abrasive forces and therefore an indication of the durability of such concrete structures.

## 3.2 Materials and Specimen Design

### 3.2.1 Materials

***Concrete***

The concrete mix used in this study was a medium strength concrete with nominal strength of $30N/mm^2$ at 28 days. The mix reflected the potential use of fibre concrete in a structural situation and the component parts per $m^3$ were 340kg of Ordinary Portland Cement (Diamond brand) obtained from local suppliers, 720kg of river sand, 370kg of 10mm crushed granitic rock, and 720kg of 20mm crushed granitic rock with an optimum water cement ratio of 0.55 and variable fibre dosage. The aggregates were obtained from local quarries and potable water was sourced from GWCL taps. Refer to Table 3.1 and Appendices C.1- C.5 for details of the mix design.

***Reinforcement***

The mild steel bars sold on the local market are milled from scrap metals and are available in commercial lengths of 9.14m (30feet) with mean yield strengths of 490, 370, and $340N/mm^2$ (Kankam and Adom-Asamoah**,** 2002). The chemical and physical properties of the steel bars have also been reported in the above mentioned research by Kankam and Adom-Asamoah

(2002). 12mm diameter and 6mm diameter of such mild steel bars of nominal yield strength of $370N/mm^2$ were obtained from local suppliers and used as longitudinal reinforcement and shear reinforcement respectively for the beams.

***Fiber***

Used polyethylene bags were obtained from plastic waste collectors contracted by recycling companies to collect and sort plastic waste for cash. The Low Density Polyethylene (LDPE) has a melt flow index of 7gm/10min, density of $0.922g/cm^3$, and low crystallinity (50-60% crystalline), Jimoh and Kolo (2011). The bags were then cut into pieces measuring 5mm wide by 40mm long by 0.095mm thick and added to the concrete in piece meal during mixing. The maximum length of the polyethylene was limited to 40mm so that workability will not be affected. The fibre application per $m^3$ of concrete was 0kg, 2.25kg, 4.49kg, and 8.99kg representing 0%, 0.25%, 0.5%, and 1.0% volume fractions respectively. Successful use of fibre in concrete depends to a large extent on the bond developed between the concrete matrix and the fibre (Richardson et al. 2010). Results of pull-out test conducted by researchers on various fibres have therefore served as the basis or have helped informed on the dosage of fibre in concrete. The use of 0%, 0.25%, 0.5%, and 1.0% volume fractions in this study was informed by the work of Richardson et al. (2010). They investigated pull-out and toughness characteristics of type 2 synthetic and steel fibres and concluded that typical fibre contents vary up to a maximum of about $12kg/m^3$ for polymeric fibres, which is the equivalence of approximately 1.35% by volume.

***Mould***

Adjustable steel moulds with internal dimensions of 150mm×150mm×150mm were used for casting the concrete cubes used for the compressive strength tests and the water absorption tests and 100mm×100mm×100mm were used for casting the concrete cubes used for the surface abrasion resistance test (Figure 3.1a). The following precautions were however observed during the selection and use of the steel moulds.

a) The moulds were readily available, nonabsorbent and nonreactive with concrete.
b) The moulds maintained their dimensions and shape under conditions of severe use.
c) The joints of the moulds were mortar tight to avoid loss of cement grout.
d) Internal faces of the moulds were lightly coated with oil before each use to aid the release of moulds.
e) All surfaces of the moulds were smooth and free from blemishes.
f) The sides, bottom, and ends were at right angles to each other and straight.

***Formwork***

Wooden formworks with internal dimensions of 100mm×100mm×300mm were used for casting the prisms used for the modulus of rupture tests and 150mm×200mm×2500mm were used for casting the beams used for the post crack flexural toughness tests (Figure 3.1a). The following precautions were observed during the selection and use of the formworks.

a) Plywood was selected and used as the formwork because it is nonreactive with the concrete.
b) The plywood was adequately thick (3/8inch) and sufficiently braced to help maintain its dimensions and shape under conditions of severe use.
c) The joints were firmly fastened together using nails to avoid loss of cement grout.
d) Internal faces of the formwork were lightly coated with oil before each use to reduce moisture absorption and also aid the release of formwork during shuttering.
e) All surfaces of the formwork were made as smooth and free from blemishes as possible.
f) It was ensured that the sides, bottom, and ends were at right angles to each other and

straight.

**Table 3.1: Concrete mix proportions**

| Material | Quantity per $m^3$ of concrete (kg) |
|---|---|
| Cement | 340 |
| 20mm aggregates | 720 |
| 10mm aggregates | 370 |
| Sand | 720 |
| Water | 210 |
| Design strength (Nominal), $N/mm^2$ | 30 |

### 3.2.2 Specimen Design

***Casting of specimens***

Batching of materials was done by weight using an electronic balance in accordance with the designed mix indicated in Table 3.1. A suitably sized electric concrete mixer (Figure 3.2a) was used for the mixing to ensure an even, consistent concrete mixture. The materials were added to the mixer in accordance with the sequence indicated on the concrete mixer as follows: inner part of the drum of the mixer was wiped clean; the water was weighed and poured into the drum; the mixer was then started and left to run at low speed; cement was then weighed and added; fine and coarse aggregates were subsequently weighed and added to the mix followed by the fibres which were added in bits and pieces to avoid balling of the fibres. The mixer was then run at high speed for another ten minutes to facilitate the complete mixing of the constituents. All of the concrete was batched simultaneously, and the plain and fibre reinforced concretes were then re-mixed at high speed for 4 minutes to ensure the same mixing time was given to each batch. Before casting the specimen, the workability of each batch of concrete mix was determined using the slump test

(Figure 3.1b) and inside faces of the moulds and formworks were oiled to facilitate stripping. The concrete was placed in layers and compacted by extensive rodding and tamping with a mallet to close the voids created by the tamping rod. Surplus concrete was then struck off and the concrete surface finished with a wood float.

***Curing of cubes and prisms***

The concrete specimens were removed from the moulds/formworks after 24 hours of casting. The specimens were immediately transferred into metal tanks containing water at room temperature for the curing to start. Curing then continued with the specimens fully submerged under water at room temperature for 28 days. The test specimens were removed from the curing tank 24 hours before testing. The cubes were kept submerged under water throughout the curing period to ensure there was no reduction in the strength of the specimens (Figure 3.2b).

***Curing of beams***

The formworks were carefully removed after 24 hours of casting and the beams were cured under damp hessian sacks at 100% relative humidity and 22°C room temperature for 28 days. The hessian sacks were kept thoroughly damp throughout the 28 days period to prevent excessive loss of the heat generated by cement hydration in the concrete (Figure 3.3a).

***Slump test***

Water was sprinkled slightly on the inside face of the slump test cone and the base plate to make them slightly moist at the beginning of each test. The base plate was then placed on a fairly flat ground devoid of vibration and the cone placed on it with the smaller opening at the top. The cone was then filled with concrete in three layers with each layer being tamped 25 times with a standard steel tamping rod. The cone was firmly held on the base plate throughout the process by means of the foot rest brazed to the cone. The top surface of the concrete was then struck off by screeding and rolling of the tamping rod across the smaller opening. The area immediately around the base of the cone was cleaned of all surplus concrete and the cone was slowly lifted from the base plate and the decrease in the height at the center of the slumped (unsupported) concrete was measured to the nearest 5mm.



**(a)** **(b)**

**Figure 3.1: a) Steel and wooden moulds/ formwork for casting concrete cubes and prisms and b) Slump test for checking consistency and workability of concrete.**

**(a)** **(b)**

**Figure 3.2: a) Portable electric concrete mixer in operation during concrete mixing and b) Curing of test cubes and prisms in a steel curing tank.**

a) b)

**Figure 3.3: a) Beams curing under damp hessian sacks covered with tarpaulin, and b) Fresh beams just after stripping from formwork**

## 3.3 Compressive Strength Test

### 3.3.1 Test Parameters

Twelve (12) 150mm concrete cubes were manufactured in accordance with the British Standards BS 1881-part 116: 1983 and as outlined in section 3.2.1 above. Details of the test specimen are shown in Table 3.2.

**Table 3.2: Compressive strength test specimen**

| Cube designation | Cube dimension (mm) | Volume fraction of fibre (%) | Cube type |
|---|---|---|---|
| CA1 | 150×150×150 | Nil | Plain concrete |
| CA2 | 150×150×150 | Nil | Plain concrete |
| CA3 | 150×150×150 | Nil | Plain concrete |
| CB1 | 150×150×150 | 0.25 | Low content fibre concrete |

| | | | |
|---|---|---|---|
| CB2 | 150×150×150 | 0.25 | Low content fibre concrete |
| CB3 | 150×150×150 | 0.25 | Low content fibre concrete |
| CC1 | 150×150×150 | 0.50 | Medium content fibre concrete |
| CC2 | 150×150×150 | 0.50 | Medium content fibre concrete |
| CC3 | 150×150×150 | 0.50 | Medium content fibre concrete |
| CD1 | 150×150×150 | 1.00 | High content fibre concrete |
| CD2 | 150×150×150 | 1.00 | High content fibre concrete |
| CD3 | 150×150×150 | 1.00 | High content fibre concrete |

### 3.3.2 Test Procedure and Computation of Results

A universal compression testing machine was used to provide a constant applied load at the rate of 0.2MPa/sec – 1.0MPa/sec in accordance with BS 1881-part 116: 1983. The test procedure was as follows: the testing machine was positioned on a firm foundation well removed from the influence of jars and vibration caused by passing traffic; the cubes were then brushed clean, turned on their sides with respect to the position in the mould and placed centrally in the testing machine; the plates of the testing machine were brought into contact with the cast faces of the cube; loading of the cube then continued at the specified rate of loading until the cube crushed.

The cube compressive strength was then calculated using the formula:

$$f_{cu} = \left(\frac{P}{b^2}\right) \quad \textbf{(3.1)}$$

$f_{cu}$ = cube compressive strength, (N/mm$^2$)

P = ultimate load at collapse, (N)

b = average height/ width/ breadth of cube, (mm)

## 3.4 Modulus of Rupture Test

### 3.4.1 Test Parameters

The modulus of rupture was obtained by conducting a three-point loading test system on 100mm×100mm×300mm concrete prisms. A total of twelve (12) concrete prisms comprising plain and varying volume fractions of polyethylene fibres were cast in accordance with the American Society for Testing and Materials ASTM C78-09 (2009). The test specimens are presented in Table 3.3.

**Table 3.3: Modulus of rupture test specimen**

| Prism designation | Prism dimension (mm) | Volume fraction of fibre (%) | Type of prism |
|---|---|---|---|
| PA1 | 100×100×300 | Nil | Plain concrete |
| PA2 | 100×100×300 | Nil | Plain concrete |
| PA3 | 100×100×300 | Nil | Plain concrete |
| PB1 | 100×100×300 | 0.25 | Low content fibre concrete |
| PB2 | 100×100×300 | 0.25 | Low content fibre concrete |
| PB3 | 100×100×300 | 0.25 | Low content fibre concrete |
| PC1 | 100×100×300 | 0.50 | Medium content fibre concrete |
| PC2 | 100×100×300 | 0.50 | Medium content fibre concrete |
| PC3 | 100×100×300 | 0.50 | Medium content fibre concrete |
| PD1 | 100×100×300 | 1.00 | High content fibre concrete |
| PD2 | 100×100×300 | 1.00 | High content fibre concrete |
| PD3 | 100×100×300 | 1.00 | High content fibre concrete |

### 3.4.2 Test Procedure and Computation of Results

A universal test frame (UTF) was used to provide a constant applied loading in increments of 2kN and the test setup is as illustrated in Figure 3.4. The test procedure was as follows: the testing machine was positioned on a firm foundation well removed from the influence of jars and vibration caused by passing traffic; the specimens were brushed clean, turned on their side, with respect to the position in the formwork and placed in the testing machine; the plunger of the jack was then brought into contact with the bearing bar positioned centrally on the specimen; a slight force was applied with the lever pump to seat the specimen firmly in its supports; loading of the specimen continued via the lever pump at the specified rate of loading until the first crack appeared. The maximum load at first crack was then recorded as P. The modulus of rupture was calculated using the formula:

$$R = 1.5\left(\frac{Pl}{bd^2}\right) \quad \textbf{(3.2)}$$

where

R = modulus of rupture ($N/mm^2$) P = maximum load on prism (N) l = span between centers of lower supports (mm) b = average width of prism (mm) *d* = average depth of prism (mm)

P

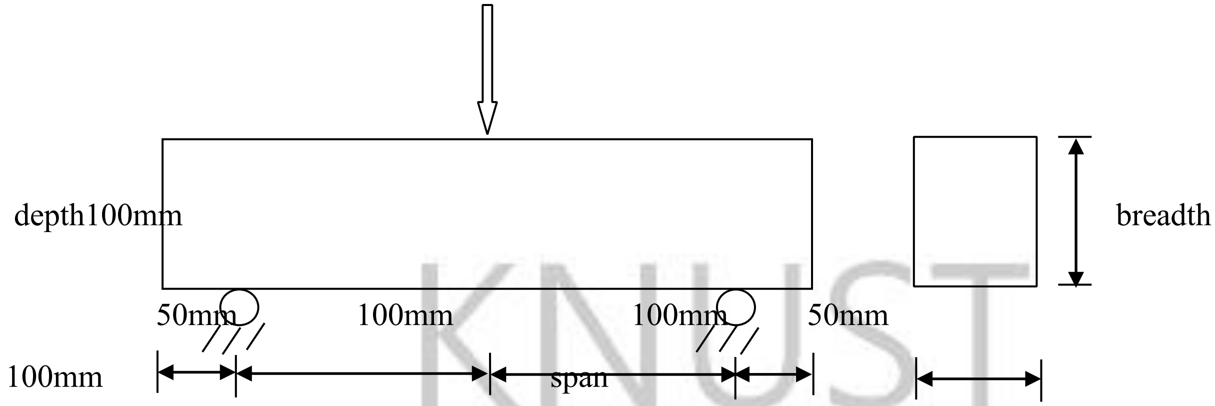


**Figure 3.4: Modulus of rupture test setup**

## 3.5 Water Absorption Test

### 3.5.1 Test Parameters

Three cubes of dimension 150mm were cast for each concrete type per the British Standards BS 1881-part 122: 1983 using the same mix design as in Table 3.1. The test specimens are presented in Table 3.4.

**Table 3.4: Specimens for water absorption test**

| Cube designation | Cube type | Number of samples |
|---|---|---|
| A1,A2,A3 | Plain concrete | 3 |
| B1,B2,B3 | Low dose polyethylene fibre concrete | 3 |
| C1,C2,C3 | Medium dose polyethylene fibre concrete | 3 |
| D1,D2,D3 | High dose polyethylene fibre concrete | 3 |

### 3.5.2 Test Procedure and Computation of Results

The water absorption test was done in accordance with BS 1881-part122 as follows: each concrete cube was measured using a vernier scale to determine the initial dimensions. The specimens were then transferred into a drying oven for a period of 72 hours at a temperature of 105ºC ensuring that there was a 25mm minimum space between specimens and the walls of the oven. The specimens were removed after 72 hours and each specimen cooled for 24 hours in an airtight container containing silica gel crystals. Upon removal from the airtight container, the specimens were weighed immediately using an electronic balance complying with the criteria as set out in the British Standard. The specimens were then immersed in a tank of water at 20ºC, with the longitudinal axis of the specimen horizontal and parallel to the base of the tank. Removal of the specimens was done after 24 hours of immersion in water. Upon removal the specimens were wiped with a dry cloth and weighed. The percentage water absorption was then computed using the following relationship:

$$\boldsymbol{water\ absorption}\ (\%) = \frac{\textbf{weight of water absorbed specimen} - \textbf{weight of oven dry specimen}}{\textbf{weight of oven dry specimen}} \qquad \textbf{(3.3)}$$

## 3.6 Post Crack Toughness Test

### 3.6.1 Test Parameters

A total of 12 beams measuring 150mm×200mm×2500mm were cast with steel rebars and varying volume fraction of polyethylene fibres as contained in Table 3.5. Two (2) no. 12mm diameter mild steel bars were provided in each beam with a clear concrete cover of 25 mm to the shear reinforcement (links) from the bottom face as longitudinal reinforcement. Shear reinforcing stirrups/ links were provided using 6mm diameter mild steel ribbed bars and placed at 100mm intervals at the two ends of each beam (ie shear spans) but no stirrups were used in the middle portion (ie constant bending moment region). A clear concrete cover of at least 25mm was provided at each side of the stirrups. Figure 3.5 shows the reinforcement details of the test beams. Apart from test beams BA1, BA2, and BA3 which had zero polyethylene fibres in them, the remaining beams had varying volume fraction of fibres. Table 3.5 presents the details of the beam. Also to be noted from Table 3.5 is that each beam has been designed as under-reinforced section with reinforcement ratio of 0.97 percent. Refer to appendix A for the computation of the reinforcement ratios. Testing of the beams were done per the American Society for Testing and Materials ASTM C1018 (1997).

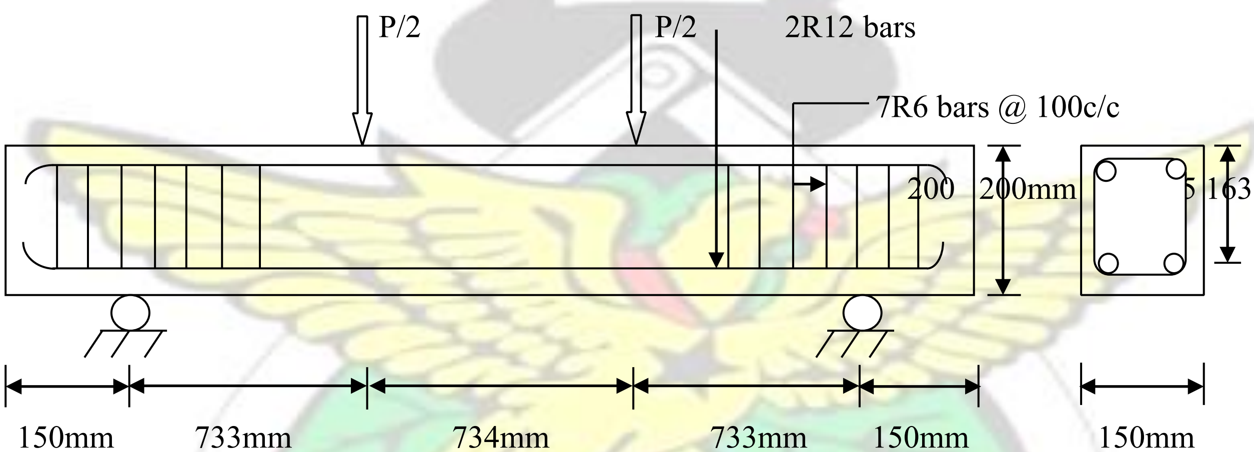


**Figure 3.5: Test beam setup and reinforcement details**

**Table 3.5: Post-crack toughness test specimens**

| Beam designation | Number of steel bars | Volume fraction of fibre (%) | Balanced reinforcement ratio (%) | Actual reinforcement ratio (%) | Beam classification |
|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| BA1 | 2 | Nil | 4.69 | 0.97 | ur |
| BA2 | 2 | Nil | 3.96 | 0.97 | ur |
| BA3 | 2 | Nil | 4.26 | 0.97 | ur |
| BB1 | 2 | 0.25 | 3.41 | 0.97 | ur-f$^1$ ur- |
| BB2 | 2 | 0.25 | 3.16 | 0.97 | f$^1$ ur-f$^1$ |
| BB3 | 2 | 0.25 | 3.82 | 0.97 | |
| BC1 | 2 | 0.50 | 2.83 | 0.97 | ur-f$^2$ ur- |
| BC2 | 2 | 0.50 | 2.91 | 0.97 | f$^2$ ur-f$^2$ |
| BC3 | 2 | 0.50 | 2.73 | 0.97 | |
| BD1 | 2 | 1.00 | 2.59 | 0.97 | ur-f$^3$ ur- |
| BD2 | 2 | 1.00 | 2.23 | 0.97 | f$^3$ ur-f$^3$ |
| BD3 | 2 | 1.00 | 2.31 | 0.97 | |

ur under-reinforced without fibre dose ur-f$^1$ under reinforced with low dose fibre ur-f$^2$ under reinforced with medium dose fibre ur-f$^3$ under reinforced with high dose fibre

### 3.6.2 Test Procedure and Computation of Results

A universal testing frame (UTF) was used to provide a constant applied loading at load increments of 2kN. The beams were brushed clean, painted white to make cracks appear visible and their identification easier and then placed on the bench of the testing machine. The plunger of the jack was then brought into contact with the bearing plate on the beam to firmly seat the beam in its supports. After contact was made and when only firm pressure had been applied, the needle on the dial gauge was adjusted to "0." Loading of the beam then continued via the lever pump at load increments of 2kN until the first crack appeared. At each load increment, all required measurements were taken. Deflections at mid-span were measured using dial gauge reading to 0.01mm mounted beneath the beam, crack widths were measured on the concrete surface using a crack microscope reading to 0.02mm and records of load at first crack were noted from the load gauge attached to the lever pump. Loading of the cracked beam was then continued at the specified rate of loading until the beam could no longer sustain any load increase. Readings of deflection at mid-span, crack widths, spacing, length as well as records of load for subsequent cracks were all noted. Relative values of toughness (toughness index) were then computed using equations 2-6, 2-7, and 2-8.

### 3.6.3 Theoretical Analysis

The theoretical failure load $P'_{ult}$ for the beams was calculated by considering the three failure scenarios in a reinforced concrete beam. These scenarios are summarized as follows:

i. Yielding of the steel in tension; ii. Crushing of the concrete in compression; iii. Shear failure.

***Ultimate flexural load***

For a simply supported beam subjected to two-point symmetrical loading system with a constant moment in the central region, the ultimate flexural load $P_{ult}$ for the beams was computed using:

$$\boldsymbol{P'_{ult} = \frac{2(M_{ult} - \frac{\omega}{8}L^2)}{z}} \tag{3.4}$$

where

$P'_{ult}$ = ultimate flexural load (N)

$M_{ult}$ = ultimate moment of resistance (N-mm)
ω = load per unit length due to self-weight of beam (N/mm)

L = span of beam between supports (mm) z = distance from point load to the nearest support (mm).

The ultimate moment of resistance of the concrete was calculated from the equation (BS 8110: Part 1 (1997)):

$$M_{ult} = 0.156 f_{cu} b d^2 \tag{3.5}$$

where $f_{cu}$ = compressive strength of the concrete ($N/mm^2$) b = width of concrete section (mm) d = effective depth of the section (mm).

For a singly reinforced rectangular section in flexure, the ultimate moment of resistance of the tension steel reinforcement is obtained using the equation (BS 8110: Part 1(1997)):

$$M_{ult} = 0.87 f_y A_s (d - 0.45x) \tag{3.6}$$

where $f_y$ = tensile strength of the steel bar ($N/mm^2$) $A_s$ = area of tension steel (mm)

x = neutral axis depth (mm) and is given by the equation (BS 8110: Part 1(1997)):

$$x = 2.164\left(\frac{f_y A_s}{f_{cu} b}\right) \tag{3.7}$$

***Shear strength***

The theoretical shear strength of the beams was calculated in accordance with the British Standard BS 8110: Part 1(1997)) as follows:

$$V = \left[0.87 \frac{A_{sv} f_{yv}}{S_v} + b v_c\right] d \tag{3.8}$$

where

$A_{sv}$ = area of shear links for two legs ($mm^2$) $S_v$ = shear link spacing (mm) $f_{yv}$ = tensile strength of shear reinforcement bars ($N/mm^2$)

V = shear resistance (N)

$\nu_c$ = shear capacity of the concrete (N/mm$^2$)

Shear capacity of the concrete, $V_c$, can be calculated from the equation:

$$\boldsymbol{\nu_c = 0.79\frac{\left[\frac{100A_s}{bd}\right]^{1/3}\left[\frac{400}{d}\right]^{1/4}}{\gamma_m}} \qquad \textbf{(3.9)}$$

where $\gamma_m$ = material factor of safety for concrete in shear (=

1.25)

For a one-span simply supported beam, the ultimate shear load, $P'_{ult}$, is given by the equation:

$$\boldsymbol{P'_{ult} = 2V = 2\left[0.87\frac{A_{sv}f_{yv}}{S_v} + b\nu_c\right]d} \qquad \textbf{(3.10)}$$

***Theoretical deflection***

The theoretical deflection of the beams at failure $\delta''_{ult}$, was computed by considering the three distinct stages of loading, viz:

i. Stage 1: Beam deflection under self-weight ii. Stage 2: Beam deflection under imposed load + self-weight iii. Stage 3: Combined deflection due to stage 1 and stage 2.

The deflection of the beams under self-weight was computed from the equation:

$$\boldsymbol{\Delta_c = \frac{5}{384}\frac{\omega L^4}{E_c I_g}} \qquad \textbf{(3.11)}$$

where

$\Delta_c$ = mid-span deflection of beam, (mm)

$E_c$ = modulus of elasticity of concrete, (kN/mm$^2$) $I_g$ = second

moment of gross area of section (mm$^4$) ω = load per unit length

due to self-weight of the beam (kN/mm)

L = span of beam between supports (mm)

The second moment of area, $I_g$, is given by;

$$I_g = \frac{bh^3}{12} \tag{3.12}$$

and the modulus of elasticity of the concrete was calculated using the equation;

$$E_c = 5.5\left(\frac{f_{cu}}{\gamma_m}\right)^{1/2} \tag{3.13}$$

where h = overall depth of the section, (mm) $\gamma_m$ = material

factor of safety for concrete in flexure, (= 1.5) $f_{cu}$ =

compressive strength of concrete (N/mm$^2$)

For a simply supported beam that is subjected to a two-point symmetrical loading system, the deflection of the beam can be obtained from the equation:

$$\Delta_c = \frac{23}{1296}\frac{PL^3}{E_c I_{cr}} \tag{3.14}$$

where

$I_{cr}$ = second moment of area for a cracked section (mm$^4$)

P = ultimate load (kN)

The second moment of area, $I_{cr}$, for a cracked section is given by (BS 8110: Part 1(1997)):

$$I_{cr} = \frac{bk^3d^3}{3} + nA_s(d - kd)^2 \quad \textbf{(3.15)}$$

where

$I_{cr}$ = second moment of area of a cracked section, ($mm^4$) n

= modular ratio which is given by;

$$n = \frac{E_s}{E_c} \quad \textbf{(3.16)}$$

where

$E_s$ = modulus of elasticity of steel (200 $kN/mm^2$ for all steel grades)

The quantity kd, can be obtained from the equation;

$$kd = \frac{\left(\sqrt{2dB+1}-1\right)}{B} \quad \textbf{(3.17)}$$ where $$B = \frac{b}{nA_s} \quad \textbf{(3.18)}$$

where b = width of concrete section (mm)

$A_s$= area of tension steel ($mm^2$)

## 3.7 Surface Abrasion Resistance Test

### 3.7.1 Test Parameters

Twelve (12) 100mm concrete cubes consisting of plain and polyethylene fibre were subjected to the surface abrasion resistance test in accordance with the Bureau of Indian Standards IS 15658 (2006). The average of each set was used as the surface abrasion resistance of the concrete type.

Refer to Table 3.6 for the details of the surface abrasion resistance test specimen. **Table 3.6: Surface abrasion resistance test specimen**

| Cube designation | Cube type | Number of cube samples |
|---|---|---|
| A4,A5,A6 | Plain concrete | 3 |
| B4,B5,B6 | Low dose polyethylene fibre concrete | 3 |
| C4,C5,C6 | Medium dose polyethylene fibre concrete | 3 |
| D4,D5,D6 | High dose polyethylene fibre concrete | 3 |

### 3.7.2 Test Procedure and Computation of Results

The cubes were air-dried for 24 hours prior to the surface abrasion test. The test then proceeded as follows: The contact and opposite faces of the specimen were made as parallel and flat as possible; the weight of the specimen was taken to the nearest 0.1g using electronic scale; the volume was determined from the actual dimensions of the specimen and density of the specimen was then calculated; the grinding path or disc of the abrasion testing machine was evenly strewed with 20g of abrasion powder; the specimen was placed on the disc of the abrasion testing machine and centrally loaded with a 200N load; the grinding disc was run at a speed of 45rpm and stopped after one cycle of 15 revolutions (20seconds); the specimen was turned 90 degrees in the clockwise direction and 20g of abrasive powder was evenly strewed on the testing track before the next cycle was started; the test cycle was repeated 8 times and the specimen was then wiped clean to remove any dust particles; the final weight of the specimen was recorded to the nearest 0.1g. The abrasive wear of the specimen after the 8 cycles was calculated as the mean loss in specimen volume, $\Delta V$**,** from the equation:

$$\Delta V = \frac{\Delta m}{PR} \tag{3.19}$$

where $\Delta V$ = loss in volume after 8 cycles ($mm^3$)

$\Delta m$ = loss in mass after 8 cycles (g)

PR = density of the specimen ($g/mm^3$)

From $\Delta V$ therefore, the abrasion wear as the mean loss in specimen thickness $\Delta t$, in millimeters was determined.

# CHAPTER 4 - RESULTS, ANALYSIS AND DISCUSSION

## 4.1 Introduction

Results of the experimental tests conducted on the various concrete types are presented in this chapter followed by analysis and discussion of the results with much emphasis on the influence of the fibres on the strength and performance characteristics of concrete. It is however instructive to note that, the observed behavior of the plain and fibre reinforced concretes was essentially the same as reported in previous literature reviewed by the researcher in chapter two, notwithstanding, the current research is reporting some interesting percentage changes that are worth considering.

## 4.2 Compressive Strength

### 4.2.1 Results

Results of the 28 days compressive strength test conducted on plain and polyethylene fibre reinforced concrete (PFRC) specimens are presented in Table 4.1, summarized in Table 4.2, and

Figure 4.1. It can be observed from Table 4.2 that there is a significant reduction in average compressive strength from 33.07N/mm$^2$ for the plain concrete to 25.84N/mm$^2$ for the 0.25 percent fibre reinforced concrete. This represents a reduction in compressive strength of 22 percent. There is however a drop in compressive strength from 33.07N/mm$^2$ to 21.09N/mm$^2$ and 17.74N/mm$^2$ for 0.5 percent fibre concrete and 1.0 percent fibre concrete respectively. This represents a 36 percent and 46 percent reduction respectively.

**Table4.1: 28 days compressive strength test results**

| s/n | specimen id | Specimen type | load (KN) | actual dimensions | | | compressive strength (N/mm2) |
|---|---|---|---|---|---|---|---|
| 1 | CD1 | 1.0% fibre concrete | 438 | 151 | mm) 150 | 150 | 19.34 |
| 2 | CD2 | 1.0% fibre concrete | 374 | 150 | 150 | 150 | 16.62 |
| 3 | CD3 | 1.0% fibre concrete | 391 | 151 | 150 | 150 | 17.26 |
| 4 | CC1 | 0.50% fibre concrete | 472 | 149 | 150 | 150 | 21.12 |
| 5 | CC2 | 0.50% fibre concrete | 486 | 149 | 150 | 150 | 21.74 |
| 6 | CC3 | 0.50% fibre concrete | 456 | 149 | 150 | 150 | 20.40 |
| 7 | CB1 | 0.25% fibre concrete | 572 | 150 | 150 | 150 | 25.42 |
| 8 | CB2 | 0.25% fibre concrete | 534 | 151 | 150 | 150 | 23.58 |
| 9 | CB3 | 0.25% fibre concrete | 646 | 151 | 150 | 150 | 28.52 |
| 10 | CA1 | control-plain concrete | 838 | 150 | 150 | 150 | 37.24 |
| 11 | CA2 | control-plain concrete | 660 | 149 | 150 | 150 | 29.53 |
| 12 | CA3 | control-plain concrete | 730 | 150 | 150 | 150 | 32.44 |

**Table 4.2: Average 28 days compressive strength of plain and fibre reinforced concrete**

| specimen type | Volume fraction of fibre (%) | Ave. density (kg/m3) | Ave. compressive strength (N/mm2) | Reduction in compressive strength due to fibre addition (%) |
|---|---|---|---|---|

| | | | | |
|---|---|---|---|---|
| control-plain concrete | 0.00 | 2425.31 | 33.07 | 0.00 |
| 0.25% fibre concrete | 0.25 | 2336.27 | 25.84 | 21.87 |
| 0.50% fibre concrete | 0.50 | 2300.77 | 21.09 | 36.24 |
| 1.00% fibre concrete | 1.00 | 2279.30 | 17.74 | 46.36 |

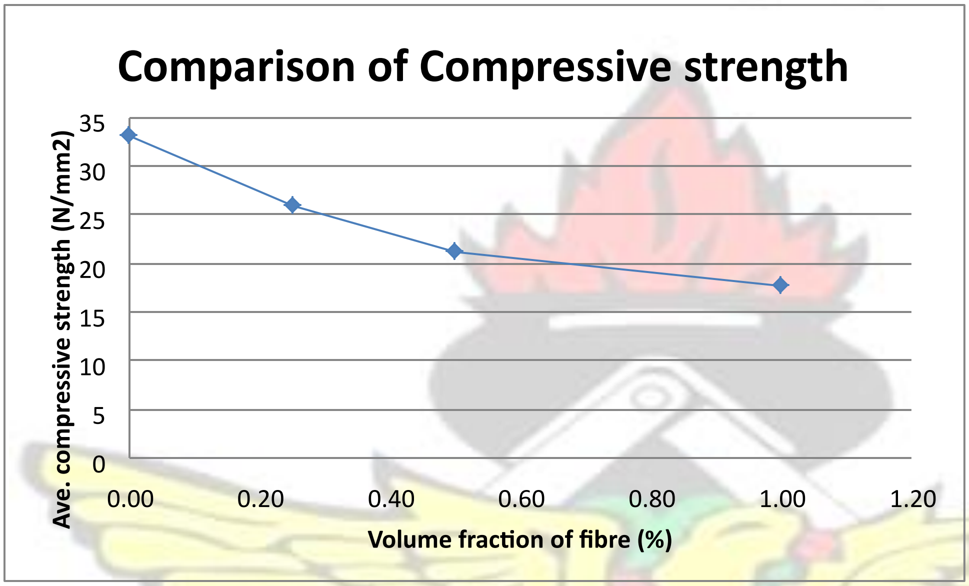


**Figure 4.1: Comparison of compressive strength of plain and fibre reinforced concrete**

### 4.2.2 Analysis and Discussion

Wiss et al. (1985), reported slight decrease (less than 10%) in compressive strength of monofilament polypropylene fibre concrete over control specimens when low volumes (less than 0.5% volume fraction) of the monofilament polypropylene fibres measuring 30mm long by 50µm in diameter were added to concrete. Singh et al. (2010) also reported a 13% reduction in compressive strength as a result of a 1% volume fraction of fibrillated polypropylene fibre addition to concrete. The thickness of the fibrils film was 30µm and each fibril measured 100µm in width. An average reduction in compressive strength of 20.4% was discovered when monofilament polypropylene fibres measuring 12mm by 6.5denier at fibre content of 0.9kg per $m^3$ of concrete was used (Richardson, 2006). The Building Research Establishment (2000) stated that "strength tests on specimens found the cube compressive strength of concrete containing polypropylene fibres to be significantly reduced because of the resulting lower density".

It is evident in the foregoing that polymer fibres when added to concrete result in a reduction in the compressive strength of the concrete compared to a plain concrete. The magnitude of the reduction in compressive strength of the polymer fibre concrete depends on the fibre properties such as type of fibre, geometry of fibre and dimensions of the fibre (Ludirdja and Young, 1992).

Polymer fibre types include polypropylene, polyethylene, polyacrylic, polyester, polyamide (nylon), polyaramid (Kevlar). Polymer fibre geometries include monofilaments, rovings, and fibrillated; polymer fibre dimensions are varied and cover aspects such as length, breadth, diameter, and aspect ratios of the fibres.

The results shown in Table 4.2 suggest a reduction in compressive strength of 7.23N/mm$^2$, which equates to 21.87% between the plain and 0.25% (2.25kg of fibre per m$^3$ of concrete) fibre concrete. The loss in compressive strength thus equates to 3.21N/mm$^2$ per kilogram of polyethylene fibre. Doubling the fibre content (2.25kg to 4.5kg per m$^3$ of concrete) resulted in compressive strength loss of 11.98N/mm$^2$ in the 0.5% fibre concrete which equates to 2.67N/mm$^2$ loss in compressive strength per kilogram of polyethylene fibre. Quadrupling the fibre content (2.25kg to 9.0kg per m$^3$ of concrete) in the 1.0% fibre concrete resulted in compressive strength loss of 15.33N/mm$^2$ which also equates to 1.71N/mm$^2$ loss in compressive strength per kilogram of polyethylene fibre.

The density of normal weight concrete when compared with polyethylene shows that, the concrete has a nominal bulk density of 2400kg/m$^3$ whereas polyethylene has a nominal bulk density of 900kg/m$^3$, this represent a density ratio of 2.7 to 1. The simple fact in this case is that polyethylene is of a lower density and compressive strength than the surrounding concrete. It therefore follows that in the fibre concrete specimens, a certain volume fraction of the concrete is taken up by the polyethylene fibre which is a low-density material. This has in turn affected the density and compressive strength of the fibre concrete by lowering them. Several other reasons have been adduced for the significant reduction in density and compressive strength of fibre concrete apart from the difference in material densities alluded to above.

Aulia (2002) and Richardson (2005) gave possible reasons for the lower compressive strength as that the fibres function as the initiators of microcracking because of their low modulus of elasticity compared to the cement matrix. Low bond strength of the fibres results in breaks in the Calcium Silicate Hydrate (C-S-H) bond between the cement and the surrounding aggregates, hence the lower compressive strength. Gold (2000) attributed the reduction in compressive strength to the air entraining action of the fibres in concrete. Clark (2006) also advanced a reason for the lower density of fibre concrete as that, fibres contribute to the problems of compacting in fibre reinforced concrete hence the reduced density.

## 4.3 Modulus of Rupture (MOR)

### 4.3.1 Results

The results of the modulus of rupture test are presented in Table 4.3 and summarized in Table 4.4 and Figure 4.2 below. The results show an increase in flexural tensile strength (modulus of rupture) as a result of fibre addition to concrete. MOR increased from 6.56MPa to 6.72MPa as a result of the addition of 2.25kg of polyethylene fibre to $1m^3$ of concrete in the 0.25% fibre concrete. This represents a 2.5 percentage increase in flexural strength. Doubling the fibre content from 2.25kg to 4.5kg in the 0.5% fibre concrete resulted in a 14 percent increase in flexural strength. This represents a significant increase compared to the 2.5 percent considering the fact that the fibre content was only doubled. Doubling the fibre content from 4.5kg to 9.0kg in the 1.0% fibre concrete increased the flexural strength further by 26.89 percent from 7.48MPa to 8.32MPa (Table 4.4).

**Table 4.3: Modulus of Rupture test results**

| s/n | specimen id | Specimen type | load (kN) | actual dimensions (mm) | effective span (mm) | modulus of rupture, R (Mpa) |
|---|---|---|---|---|---|---|
| 1 | PD1 | 1.0% fibre concrete | 28 | 100 101 300 | 200 | 8.23 |
| 2 | PD2 | 1.0% fibre concrete | 28 | 100 102 300 | 200 | 8.07 |
| 3 | PD3 | 1.0% fibre concrete | 30 | 100 102 300 | 200 | 8.65 |
| 4 | PC1 | 0.50% fibre concrete | 25 | 101 101 300 | 200 | 7.28 |
| 5 | PC2 | 0.50% fibre concrete | 26 | 100 100 300 | 200 | 7.80 |
| 6 | PC3 | 0.50% fibre concrete | 25 | 100 101 300 | 200 | 7.35 |
| 7 | PB1 | 0.25% fibre concrete | 23 | 100 102 300 | 200 | 6.63 |
| 8 | PB2 | 0.25% fibre concrete | 23 | 100 102 300 | 200 | 6.63 |
| 9 | PB3 | 0.25% fibre concrete | 23 | 100 100 298 | 200 | 6.90 |
| 10 | PA1 | control-plain concrete | 22 | 100 100 300 | 200 | 6.60 |
| 11 | PA2 | control-plain concrete | 22 | 100 100 298 | 200 | 6.60 |
| 12 | PA3 | control-plain concrete | 22 | 100 101 298 | 200 | 6.47 |

**Table 4.4: Average modulus of rupture for test specimens**

| s/n | specimen type | volume fraction of fibre (%) | ave. modulus of rupture (Mpa) | increase in flexural strength due to fibre addition (%) |
|---|---|---|---|---|

| | | | | |
|---|---|---|---|---|
| 1 | control-plain concrete | 0.00 | 6.56 | 0.00 |
| 2 | 0.25% fibre concrete | 0.25 | 6.72 | 2.51 |
| 3 | 0.50% fibre concrete | 0.50 | 7.48 | 14.04 |
| 4 | 1.00% fibre concrete | 1.00 | 8.32 | 26.89 |

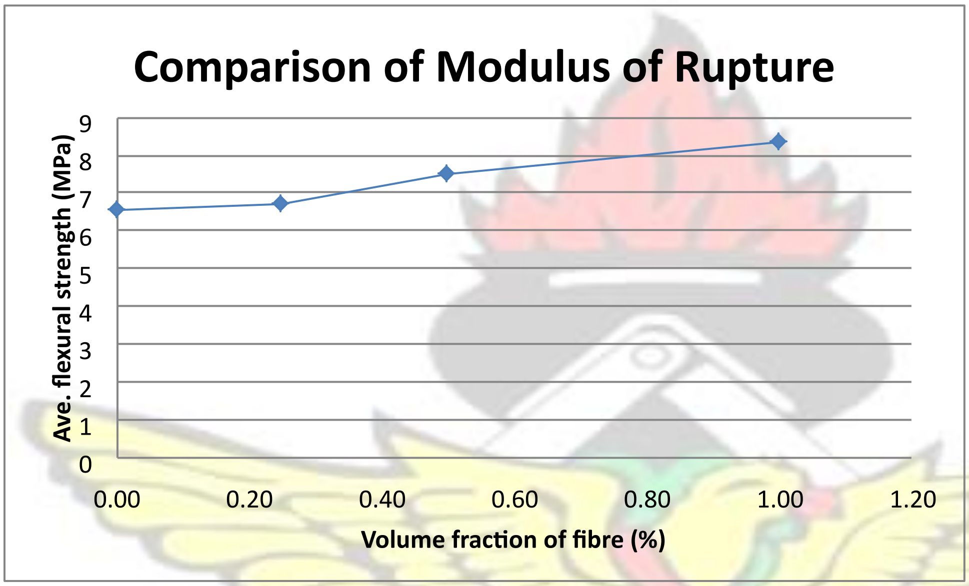


**Figure 4.2: Comparison of modulus of rupture of plain and fibre concrete**

### 4.3.2 Analysis and Discussion

Kandasamy and Murugesan (2011) recorded an increase in tensile strength of 1.63 percent due to addition of 0.5 percent polyethylene fibre to concrete. The tensile strength of concrete in this case was determined using the split tensile test on cylinders. Pelisser et al. (2012) also recorded an increase in flexural tensile strength of concrete due to addition of polyethylene terephthalate (PET) fibres to concrete. Flexural strength values of 3.75, 4.26, 4.30, and 4.47MPa were recorded for 0, 0.05, 0.18, and 0.30 percent volume fraction of the PET fibres. This represents an increase of 13.60, 14.67, and 19.20 percent for volume fractions of 0.05, 0.18, and 0.30 percent PET fibres over plain concrete. Several studies (Hasaba et al., 1984; Zollo and Itler, 1986; Wiss et al., 1985; Hannant, 1995; Hughes and Fattuhi, 1976; and Beddar, 2004) all reported greater static flexural strength in concrete with addition of polypropylene fibres than in unreinforced concrete.

The current study recorded increase of 2.51, 14.04, and 26.89 percent due to the addition of 2.25kg, 4.5kg, and 9.0kg of polyethylene fibres to $1m^3$ of concrete respectively. It is clear from the results that the inclusion of the 2.25kg of the fibres did not change the flexural strength of the plain concrete significantly. This could be attributed to fewer quantities of fibres trying to bridge and slow the propagation of the microcracks during load application. As the fibre content increased from 2.25kg to 4.5kg, there were enough fibres to bridge the voids and micro-cracks in the concrete thereby resulting in more significant increase in the flexural capacity of the concrete. The significant improvement in the flexural capacity can therefore be attributed to the fibres slowing crack propagation during loading through progressive bridging of micro-cracks in the concrete. The fibres are known to possess much higher ultimate tensile strength (500MPa) compared to the more brittle concrete (4 - 8MPa); hence their inclusion is expected to improve the tensile strength of the concrete. The enhanced flexural strength of the fibre reinforced concrete is also evident in the mode of failure of the test prisms. Figure 4.3 shows a complete separation of the plain concrete immediately after the first crack and therefore represents a typical brittle failure.


**Figure 4.3: Plain concrete prisms after failure**

The fibre reinforced concrete specimens, however, showed a more ductile behavior at failure with fibres visibly seen spanning the cracked section and therefore preventing the complete separation of the member. Figures 4.4, 4.5, and 4.6 show the test specimens for 0.25, 0.5, and 1.0 percent fibre concretes at failure.


**Figure 4.4: 0.25 percent fibre concrete prisms after failure**


**Figure 4.5: 0.50 percent fibre concrete prisms after failure**


**Figure 4.6: 1.0 percent fibre concrete prisms after failure**

## 4.4 Post Crack Toughness, Crack Propagation and Cracking Pattern

### 4.4.1 Results

Results of the toughness test are presented in Tables 4.5 to 4.12 and Figures 4.7 to 4.10. All twelve beams exhibited similar behavior with the load increasing linearly with the deflection until first crack beyond which there was disproportional increase in deflection compared to the load as cracks propagated. All beams also reported first crack formation within the middle third of the beam with majority of the cracks having been recorded in this region. All the beams failed in pure compression after considerable deflection and extensive cracking deep into the compressive zone; none of the beams failed in shear. The detailed load-deflection curves of all specimens are presented in Appendix B.

**Table 4.5: Results of load vs. deflection for control specimens**

| Load (kN) | Deflection (mm) | | | ave. deflection (mm) |
|---|---|---|---|---|
| | BA1 | BA2 | BA3 | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 2 | 0.2100 | 0.1800 | 0.2500 | 0.2133 |
| 4 | 0.4500 | 0.3600 | 0.5500 | 0.4533 |
| 6 | 0.7000 | 0.7500 | 1.1000 | 0.8500 |
| 8 | 1.0000 | 1.6800 | 2.0500 | 1.5767 |
| 10 | 1.7500 | 4.9000 | 4.5000 | 3.7167 |
| 12 | 3.1500 | 7.9000 | 7.2000 | 6.0833 |
| 14 | 5.8300 | 10.9000 | 10.3000 | 9.0100 |
| 16 | 8.7000 | 14.0000 | 13.7000 | 12.1333 |
| 18 | 11.1000 | 16.3800 | 17.1000 | 14.8600 |
| 20 | 13.3000 | 19.0000 | 21.1500 | 17.8167 |
| 22 | 15.7000 | 21.8000 | 24.8500 | 20.7833 |
| 24 | 18.2300 | 24.3000 | 29.3800 | 23.9700 |
| 26 | 21.1000 | 27.3000 | 35.0000 | 27.8000 |
| 28 | 23.7500 | 30.3800 | 41.1000 | 31.7433 |

**Table 4.6: Results of loads, deflections, and cracking properties for control specimens**

| beam characteristics | specimen id. | | | ave. values |
|---|---|---|---|---|
| | BA1 | BA2 | BA3 | |
| First crack load (kN) | 8.00 | 7.00 | 8.00 | 7.67 |
| First crack deflection (mm) | 1.00 | 1.22 | 2.05 | 1.42 |
| Ultimate load (kN) | 28.00 | 28.00 | 28.00 | 28.00 |
| Ultimate deflection (mm) | 23.75 | 30.38 | 41.10 | 31.74 |
| First crack width (mm) | 0.06 | 0.10 | 0.08 | 0.08 |
| Biggest crack width at failure (mm) | 1.00 | 3.20 | 1.20 | 1.80 |
| Total number of cracks | 12.00 | 10.00 | 14.00 | 12.00 |

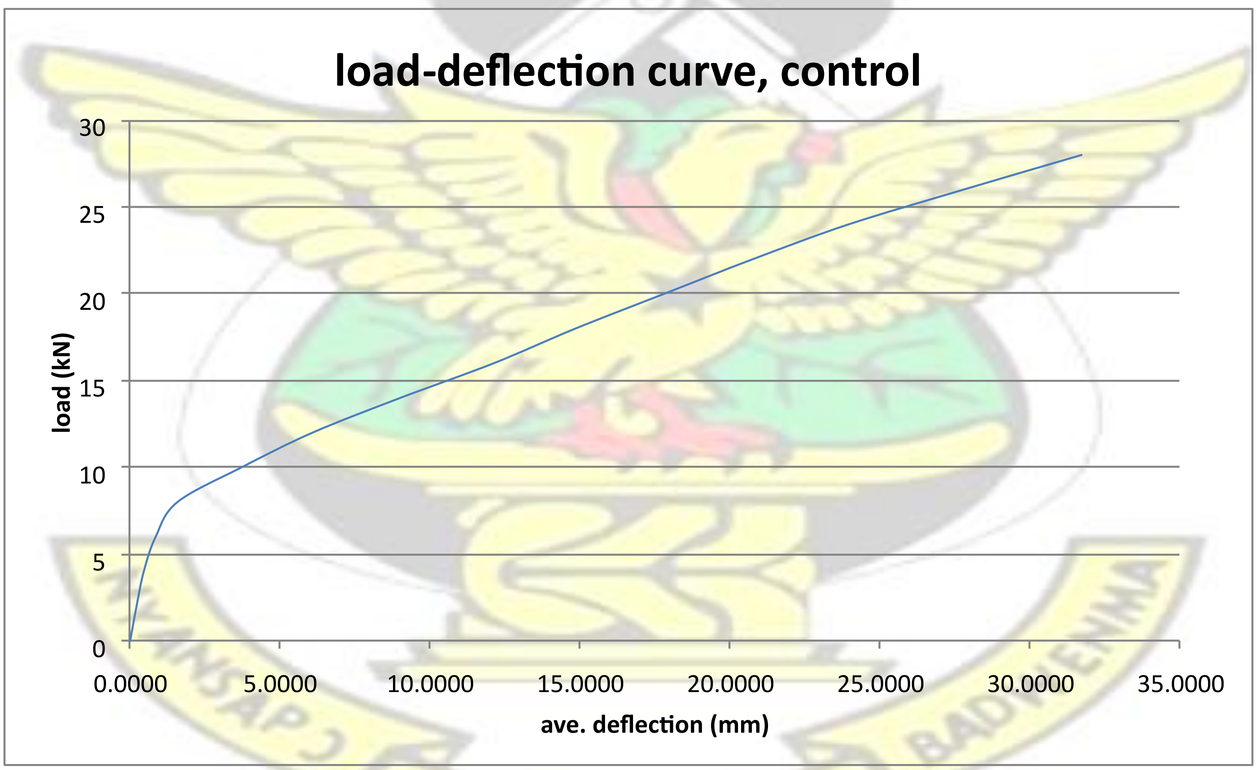


**Figure 4.7: Load deflection curve for control specimens**

Table 4.7: Results of load vs. deflection for 0.25% fibre concrete specimens

| Load (kN) | Deflection (mm) | | | ave. deflection (mm) |
|---|---|---|---|---|
| | BB1 | BB2 | BB3 | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 2 | 0.1000 | 0.1500 | 0.2000 | 0.1500 |
| 4 | 0.2200 | 0.2600 | 0.6100 | 0.3633 |
| 6 | 0.6000 | 0.5000 | 0.9100 | 0.6700 |
| 8 | 0.9800 | 0.9000 | 3.6000 | 1.8267 |
| 10 | 3.8000 | 2.2000 | 6.8000 | 4.2667 |
| 12 | 8.7000 | 4.5000 | 9.7000 | 7.6333 |
| 14 | 12.6000 | 6.5100 | 12.5000 | 10.5367 |
| 16 | 17.2000 | 8.6000 | 16.2000 | 14.0000 |
| 18 | 20.8400 | 10.1000 | 19.8000 | 16.9133 |
| 20 | 24.8000 | 12.2000 | 24.1000 | 20.3667 |
| 22 | 28.6000 | 14.1000 | 28.9000 | 23.8667 |
| 24 | 33.1000 | 15.6000 | 33.4000 | 27.3667 |
| 26 | 37.0000 | 17.7000 | 40.0000 | 31.5667 |
| 28 | 40.2000 | 19.7000 | 45.8000 | 35.2333 |
| 30 | 44.0000 | 21.8000 | 54.5400 | 40.1133 |

Table 4.8: Results of loads, deflections, and cracking properties for 0.25% fibre concrete specimens

| beam characteristics | specimen id. | | | ave. values |
|---|---|---|---|---|
| | BB1 | BB2 | BB3 | |
| First crack load (kN) | 9.00 | 8.00 | 6.00 | 7.67 |
| First crack deflection (mm) | 2.39 | 0.90 | 0.91 | 1.40 |
| Ultimate load (kN) | 30.00 | 30.00 | 30.00 | 30.00 |
| Ultimate deflection (mm) | 44.00 | 21.80 | 54.54 | 40.11 |
| First crack width (mm) | 0.10 | 0.06 | 0.06 | 0.07 |
| Biggest crack width at failure (mm) | 1.80 | 2.20 | 1.20 | 1.73 |

| Total number of cracks | 14.00 | 12.00 | 11.00 | 12.33 |
|---|---|---|---|---|

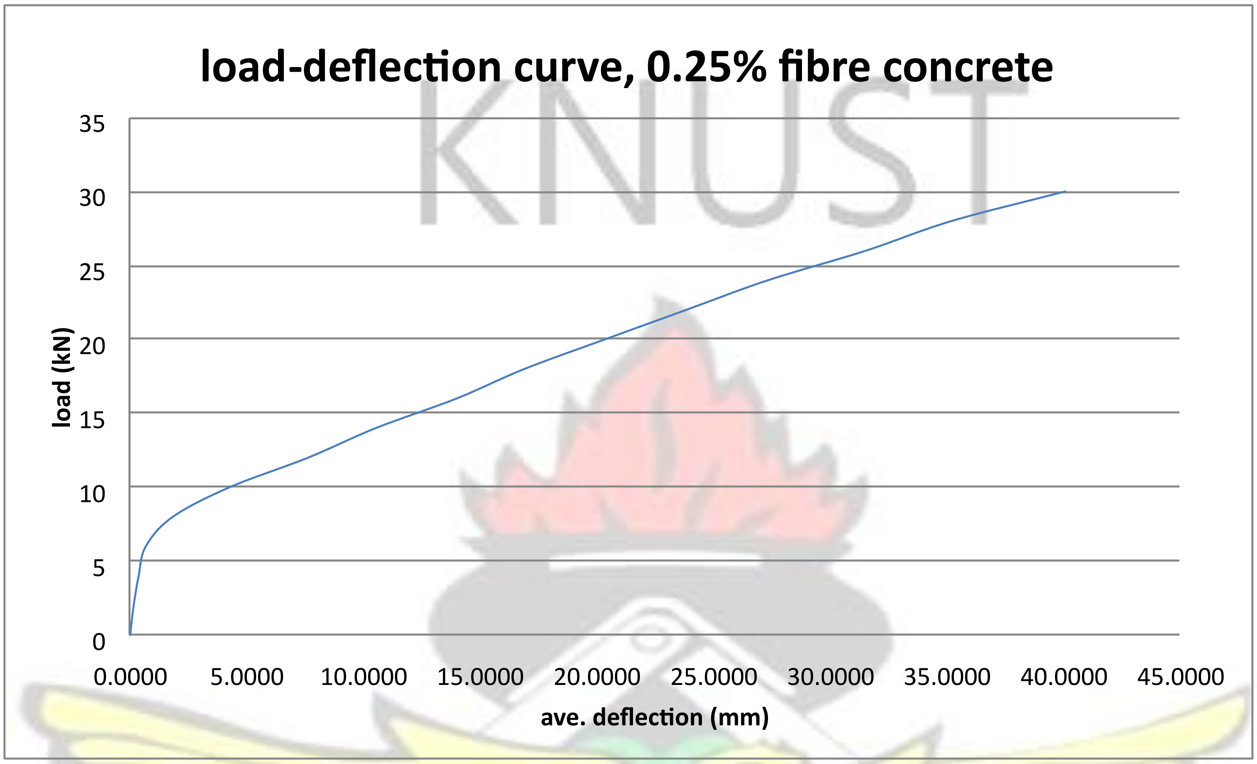


**Figure 4.8: load deflection curve for 0.25% fibre concrete specimens**

**Table 4.9: Results of load vs. deflection for 0.5% fibre concrete specimens**

| Load (kN) | Deflection (mm) | | | ave. deflection (mm) |
|---|---|---|---|---|
| | BC1 | BC2 | BC3 | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 2 | 0.2000 | 0.1000 | 0.0000 | 0.1000 |
| 4 | 0.4200 | 0.2400 | 0.0400 | 0.2333 |
| 6 | 0.6300 | 0.5200 | 0.4200 | 0.5233 |
| 8 | 1.1400 | 0.8000 | 0.6200 | 0.8533 |
| 10 | 4.1000 | 1.7000 | 1.6100 | 2.4700 |
| 12 | 6.6800 | 4.5000 | 2.8400 | 4.6733 |
| 14 | 10.2000 | 7.6000 | 7.8600 | 8.5533 |
| 16 | 13.7500 | 10.3000 | 13.8000 | 12.6167 |
| 18 | 16.4300 | 13.1000 | 20.7100 | 16.7467 |
| 20 | 19.6000 | 16.2000 | 26.2200 | 20.6733 |

| | | | | |
|---|---|---|---|---|
| 22 | 22.2000 | 19.3000 | 31.4500 | 24.3167 |
| 24 | 25.4000 | 22.9000 | 36.7900 | 28.3633 |
| 26 | 28.1400 | 27.4000 | 42.1800 | 32.5733 |
| 28 | 31.8000 | 32.7000 | 46.8000 | 37.1000 |
| 30 | 35.1000 | 37.5200 | 51.8000 | 41.4733 |

**Table 4.10: Results of loads, deflections, and cracking properties for 0.5% fibre concrete specimens**

| beam characteristics | specimen i ł. | | | ave. values |
|---|---|---|---|---|
| | **BC1** | **BC2** | **BC3** | |
| **First crack load (kN)** | 8.00 | 9.00 | 9.00 | 8.67 |
| **First crack deflection (mm)** | 1.14 | 1.25 | 1.12 | 1.17 |
| **Ultimate load (kN)** | 30.00 | 30.00 | 30.00 | 30.00 |
| **Ultimate deflection (mm)** | 35.10 | 37.52 | 51.80 | 41.47 |
| **First crack width (mm)** | 0.10 | 0.06 | 0.30 | 0.15 |
| **Biggest crack width at failure (mm)** | 1.20 | 2.40 | 0.70 | 1.43 |
| **Total number of cracks** | 13.00 | 12.00 | 13.00 | 12.67 |

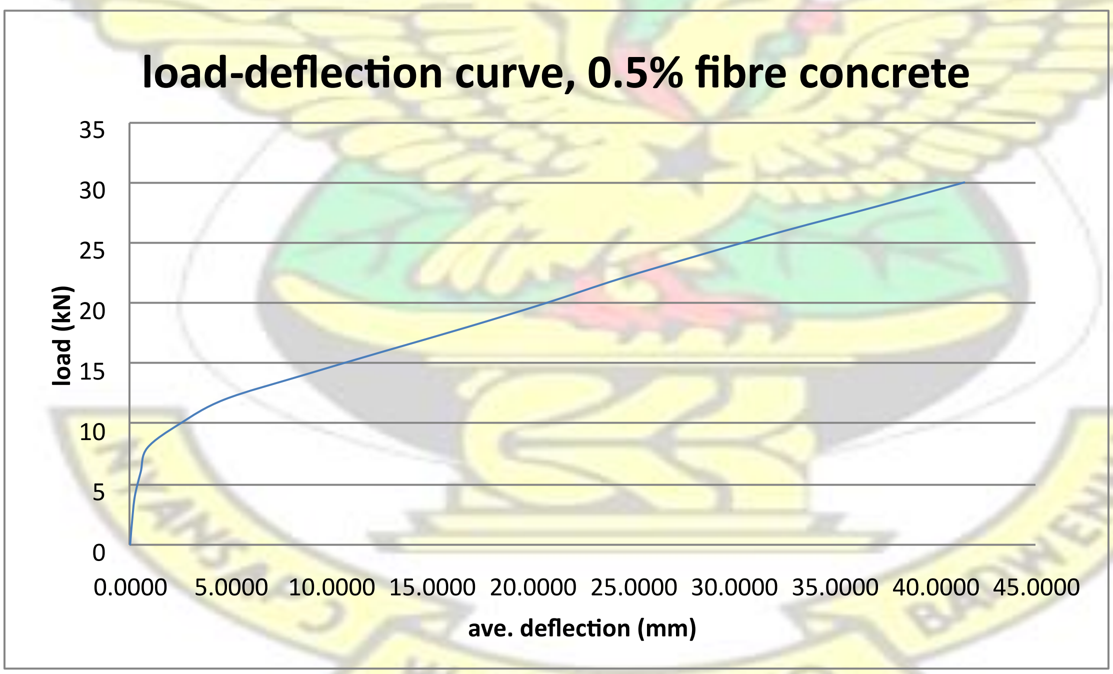


**Figure 4.9: load deflection curve for 0.5% fibre concrete specimens**

**Table 4.11: Results of load vs. deflection for 1.0% fibre concrete specimens**

| Load (kN) | Deflection (mm) | | | ave. deflection (mm) |
|---|---|---|---|---|
| | BD1 | BD2 | BD3 | |
| 0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| 2 | 0.5000 | 0.4900 | 0.6100 | 0.5333 |
| 4 | 0.9000 | 0.9400 | 1.2300 | 1.0233 |
| 6 | 1.3000 | 1.3500 | 1.9500 | 1.5333 |
| 8 | 2.0000 | 2.0000 | 2.6900 | 2.2300 |
| 10 | 2.6000 | 2.6200 | 3.6000 | 2.9400 |
| 12 | 3.6000 | 3.2800 | 4.6000 | 3.8267 |
| 14 | 5.6800 | 6.6400 | 8.2000 | 6.8400 |
| 16 | 9.2000 | 11.2300 | 11.6000 | 10.6767 |
| 18 | 12.7000 | 16.1000 | 15.2000 | 14.6667 |
| 20 | 17.8000 | 20.4800 | 18.8000 | 19.0267 |
| 22 | 23.5600 | 24.3200 | 22.3000 | 23.3933 |
| 24 | 30.2500 | 28.2000 | 26.1300 | 28.1933 |
| 26 | 35.8000 | 32.6000 | 29.5000 | 32.6333 |
| 28 | 40.6000 | 36.4000 | 32.8500 | 36.6167 |
| 30 | 45.9000 | 40.1000 | 37.2000 | 41.0667 |
| 32 | 50.3900 | 43.6000 | 41.6000 | 45.1967 |
| 34 | 53.9200 | | | 53.9200 |

**Table 4.12: Results of loads, deflections, and cracking properties for 1.0% fibre concrete specimens**

| beam characteristics | specimen id. | | | ave. values |
|---|---|---|---|---|
| | BD1 | BD2 | BD3 | |

| | | | | |
|---|---|---|---|---|
| **First crack load (kN)** | 14.00 | 12.00 | 12.00 | 12.67 |
| **First crack deflection (mm)** | 5.68 | 3.28 | 4.60 | 4.52 |
| **Ultimate load (kN)** | 34.00 | 32.00 | 32.00 | 32.67 |
| **Ultimate deflection (mm)** | 53.92 | 43.60 | 41.60 | 46.37 |
| **First crack width (mm)** | 0.90 | 0.08 | 0.10 | 0.36 |
| **Biggest crack width at failure (mm)** | 1.10 | 0.18 | 0.20 | 0.49 |
| **Total number of cracks** | 12.00 | 13.00 | 15.00 | 13.33 |

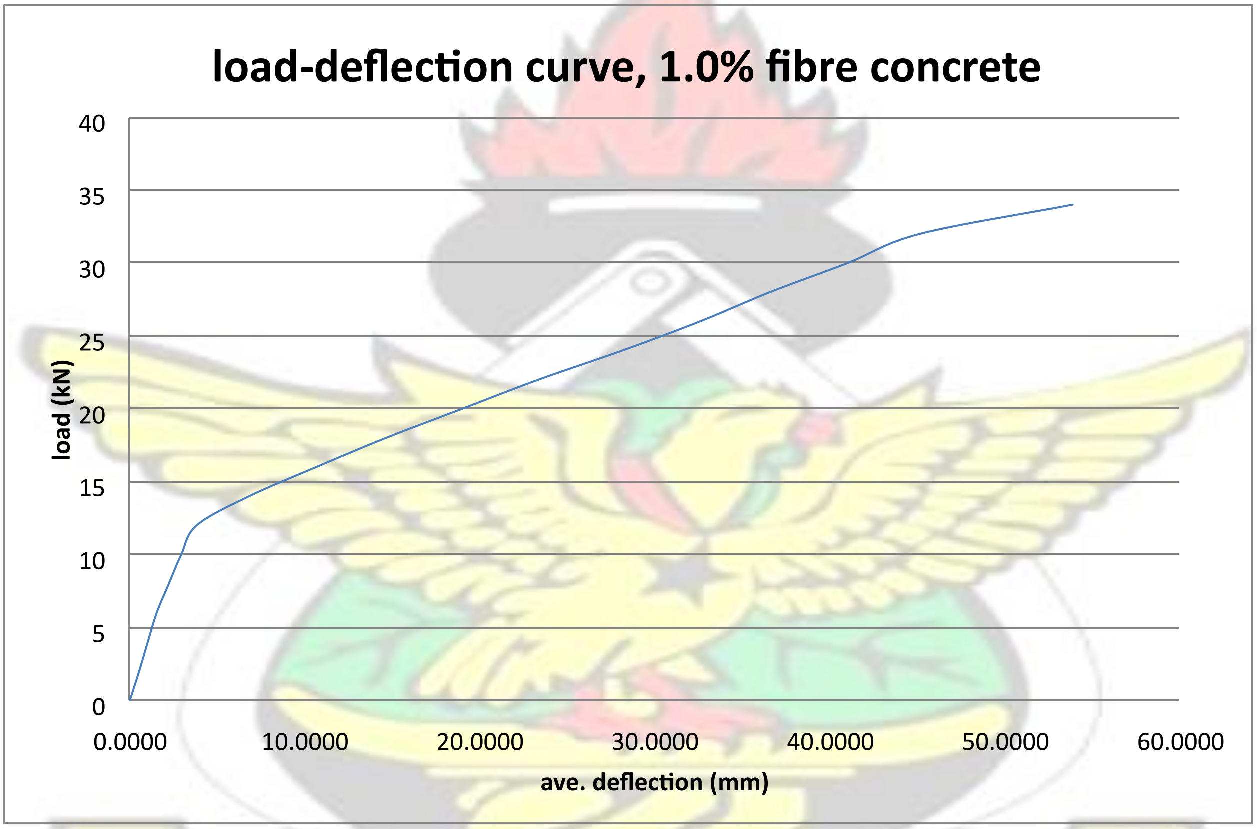


**Figure 4.10: load deflection curve for 1.0% fibre concrete specimens**

The toughness indices were computed using the area under load deflection curve as follows;

a) Each graph was composed of a linear portion and a curved portion.

b) For the linear portion of the graph, a best fit line was established by adding a trend line in EXCEL.

c) For the curved portion of the graph, the data points were fitted with appropriate polynomial curve and the parameters of the curve were determined using the EXCEL solver tool pack.

d) Areas under the linear and polynomial best-fit curves were then determined by direct integration of the curves.

e) Total areas from zero to Δ (first crack deflection), 3Δ, 5.5Δ, and 15.5Δ were obtained by adding the individual areas corresponding to the specific range of data points.

f) The respective toughness indices $I_5$, $I_{10}$, and $I_{30}$ values corresponding to 3Δ, 5.5Δ, and 15.5Δ were calculated.

### 4.4.2 Analysis and Discussion

***Post crack toughness***

The toughness test results are presented in Table 4.13 and load-deflection curves of the four concrete types is also presented in Figure 4.11 for comparison. The mean value of $I_5$ for the control specimens is approximately 3.9; this value is slightly higher than the approximate values of 3.2 and 2.8 for the 0.25 percent and 0.5 percent fibre concrete specimens, respectively. However, the value of $I_5$ for the 1.0 percent fibre concrete which is approximately 4.8 is much higher than that of the control. In percentage terms, the control specimens outperformed the 0.25 percent and 0.5 percent fibre concretes by 18 percent and 28 percent, respectively. On the other hand, however, it underperformed by 23 percent compared to the 1.0 percent fibre concretes.

| type of concrete | first crack load (kN) | first crack deflection (mm) | flexural strength (Mpa) | toughness indices | | |
|---|---|---|---|---|---|---|
| | | | | $I_5$ | $I_{10}$ | $I_{30}$ |

| | | | | | | |
|---|---|---|---|---|---|---|
| control | 7.67 | 1.42 | 6.56 | 3.9192 | | |
| 0.25% fibre concrete | 7.67 | 1.4 | 6.72 | 3.1743 | 6.6373 | 28.0769 |
| 0.5% fibre concrete | 8.67 | 1.17 | 7.48 | 2.8222 | 5.6402 | 21.8119 |
| 1.0% fibre concrete | 12.67 | 4.52 | 8.32 | 4.7901 | 11.1556 | 51.2661 |
| | | | | | 8.5663 | 37.2377 |

**Table 4.13: Post crack toughness indices of control and polyethylene fibre reinforced concrete**

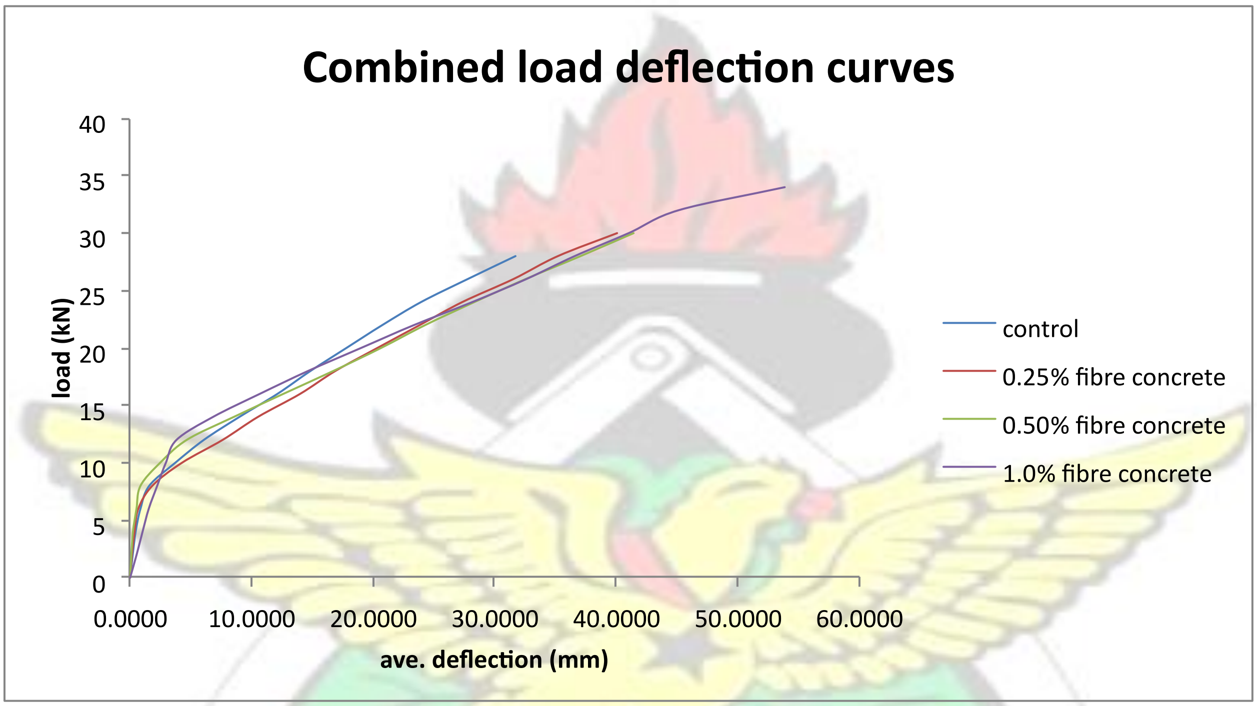


**Figure 4.11: load deflection curves for control and fibre reinforced concretes.**

In the case of the toughness index $I_{10}$, the control specimens out-performed the 0.25 percent fibre concretes by 23 percent and the 0.5 percent fibre concretes by 35 percent, but under-performed by 30 percent compared to the 1.0 percent fibre concrete. At $I_{30}$ the control specimens outperformed the 0.25 percent fibre concrete by 24 percent and the 0.5 percent fibre concrete by 41 percent but again under-performed by 38 percent compared to the 1.0 percent fibre concrete.

It is evident therefore that, the control specimens (beams BA"s) possess a higher energy absorption capacity at $I_5$, $I_{10}$, and $I_{30}$ compared to the 0.25% (beams BB"s) and 0.5% (beams BC"s) fibre concretes. A critical look at Tables 4.6, 4.8 and 4.10 reveals that, the first crack deflection of the 0.25% and 0.5% fibre concretes is lower than that of the control concrete. Also the first crack loads of the three specimens (control, 0.25%, 0.5%) are approximately equal. The possible reason for this is that initially, the entire load was borne by the concrete alone.

Immediately after first crack, the fibre and main steel reinforcement which were initially idle began to bear the full load. The ability of the steel and polyethylene fibres to take up the load depends on the concrete-steel and concrete-fibre bond strengths. The concrete-fibre bond strength in turn depends on the quantity of the fibres present in the concrete mix. Pilakoutas et al.

(2009) suggested that "bond between concrete and FRP is the most important factor when FRP is used as reinforcement". They went further to say that "sufficient bond must be mobilized between reinforcement and concrete for the successful transfer of load from one to the other". The low volume fraction of polyethylene fibres and by extension the low bond weakened the concrete and compromised the strength and toughness of the 0.25% and the 0.5% fibre concretes at $I_5$, $I_{10}$, and $I_{30}$ compared to the control specimens but the ultimate deflection was unaffected.

***Crack propagation and Cracking pattern***

Cracks originated from the bottom face of the beam and propagated upwards towards the neutral axis. This resulted in the neutral axis which was originally around the mid-section of the beam to start moving upward into the compression zone. As loading continued, the neutral axis continued to move deep into the compression zone towards the upper fibres until the concrete section became insufficient to carry any more load. It is quite evident from Figures 4.12, 4.13, 4.14, and 4.15 that the cracks went deep into the compression zone and very close to the upper fibres. The researcher therefore concluded that the beams failed in compression due to the inadequacy of the concrete to support any further load after excessive cracking and deflection due to yielding of the reinforcing steel bars in conformity with expected behavior of the beams in line with the underreinforced design philosophy.


**Figure 4.12: Crack propagation and characteristics of the control specimen**

**Figure 4.13: Crack propagation and characteristics of the 0.25% fibre concrete**


**Figure 4.14: Crack propagation and characteristics of the 0.5% fibre concrete**


**Figure 4.15: Crack propagation and characteristics of the 1.0% fibre concrete**

A critical look at figures 4.12 to 4.15 shows that the fibre reinforced concrete beams (Figures 4.13, 4.14, and 4.15) had a closer crack distribution than the control beams although they contain the same number of tensile steel reinforcement as the control specimens. The addition of fibres to beams shown in Figures 4.13, 4.14, and 4.15 influenced the crack distribution ability of the beams. Measurements of crack widths using crack microscope showed that the crack openings in the fibre reinforced beams were visibly smaller than the crack openings in the beams without fibres. Also the crack spacing in the fibre reinforced concretes beams was closer than the more widely spaced crack pattern of the concrete beams without fibres. Furthermore, it was observed that the fibre reinforced beams provided additional cracks compared to the beams without fibre (Table 4.14). Refer to Appendix E for details of the beam crack spacing. **Table 4.14: Summary of beam loads, deflections and cracking properties**

| beam characteristics | plain concrete | 0.25% fibre concrete | 0.50% fibre concrete | 1.0% fibre concrete |
|---|---|---|---|---|
| First crack load (kN) | 7.67 | 7.67 | 8.67 | 12.67 |
| First crack deflection (mm) | 1.42 | 1.40 | 1.17 | 4.52 |
| Ultimate load (kN) | 28.00 | 30.00 | 30.00 | 32.67 |
| Ultimate deflection (mm) | 31.74 | 40.11 | 41.47 | 46.37 |
| First crack width (mm) | 0.08 | 0.07 | 0.15 | 0.36 |
| Biggest crack width at failure (mm) | 1.80 | 1.73 | 1.43 | 0.49 |
| Total number of cracks | (10-14) | (11-14) | (12-13) | (12-15) |

It can also be seen from Table 4.14 that, the 1% fibre reinforced concrete beams withstood a higher load before the appearance of first crack compared to the concrete beams without fibre. This is due to the action of the fibres which were evenly distributed within the matrix interfering with the propagation of micro-cracks. Again, it can be noted from Table 4.14 that the entire fibre reinforced concrete specimens had a higher ultimate deflection values at higher ultimate loads compared to the control specimens. This is a clear attestation of the superior ductility of the fibre reinforced concrete over the non-fibre reinforced concrete.

### 4.4.3 Theoretical vs. Experimental Loads

The theoretical and experimental loads at first crack and at failure are presented in Table 4.15. The experimental failure loads $P_{ult}$ for the 0% fibre concrete specimens averaged 100% of their predicted failure loads $P'_{ult}$ which were governed solely by the yielding of the tension steel. This is in complete agreement with the actual failure of the beams which was characterized by the yielding of the tensile steel bars followed by the crushing of the concrete in compression after flexural cracks had extended deep into the compression zone. In the case of the 0.25% fibre concrete specimens, the experimental failure loads $P_{ult}$ averaged 111% over their predicted failure loads $P'_{ult}$. Once again the theoretical failure of the beams which was governed by the yielding of the tension steel also agreed with the actual failure which was characterized by the yielding of the tension steel bars followed by crushing of the concrete in compression. The difference of 11% in the average values of the 0% and the 0.25% fibre concrete is as a result of the inclusion of the polyethylene fibre which is the only variable in the two concretes. It has been proven in this study and also in previous studies such as Barris et al (2009), Kim et al (2010), and Fraternalli et al (2011) that fibres interfere with the propagation of micro-cracks by bridging the cracks when the ultimate strain of the concrete is exceeded thereby resulting in concrete with a higher ultimate load carrying capacity. The experimental failure loads $P_{ult}$ for the 0.5% fibre concrete specimens averaged 115% over their predicted failure loads $P'_{ult}$ and were governed by the yielding of the tension steel. This again concurs with the actual failure of the beams which was characterized by the yielding of the tensile steel bars followed by the crushing of the concrete in compression. The difference in the average values is 15% and 4% compared to the 0% and the 0.25% fibre concrete specimens respectively and represents the increase in fibre content. With regard to the 1.0% fibre concrete, the experimental failure loads $P_{ult}$ averaged 130% over their predicted failure loads $P'_{ult}$ and were also governed by the yielding of the tension steel. This again is consistent with the actual failure of the beams which was characterized by the yielding of the tensile steel bars followed by the crushing of the concrete in compression. The difference in the average values is 30%, 15%, and

4% compared to the 0%, 0.25%, and the 0.5% fibre concrete specimens respectively and represents the increase in fibre content.

The cracking loads $P_{cr}$ averaged 26%, 27%, 29%, and 39% of the experimental failure loads $P_{ult}$ for the 0%, 0.25%, 0.5%, and the 1.0% fibre concrete specimens respectively. This is an indication of the load carrying capacity of the beams after cracking. It is evident that, the 1.0% fibre concrete specimens could carry up to 39% of the first crack load in addition to the first crack load before collapse compared to 26% for the 0% fibre concrete specimens. The analogy here is that if the steel reinforcement alone withstood 26% of the first crack load in addition to the first crack load after the appearance of the first crack in the case of the 0% fibre concrete specimens, then the polyethylene fibres in the fibre reinforced concrete specimens contributed to increase the load carrying capacity to 39% of first crack load in addition to first crack load in the case of the 1.0% fibre concrete specimens.

The shear strength of the beams were 1.8, 2.3, 2.8, and 3.4 times greater than their strength in compression and 3.5, 3.6, 3.7, and 3.9 times greater than their strength in tension for the 0%, 0.25%, 0.5%, and 1.0% fibre concrete specimens respectively. It was therefore expected that the beams will fail in tension through yielding of the tension steel bars.

**Table 4.15: Theoretical and experimental loads**

| Beam no. | First crack load, $P_{cr}$ (kN) | Experimental failure load, $P_{ult}$ (kN) | Theoretical failure load $P'_{ult}$ (kN) | | | $P_{cr}$ / $P_{ult}$ | $P_{ult}$ / $P'_{ult}$ |
|---|---|---|---|---|---|---|---|
| | | | Steel yielding | Concrete crushing | Shear failure | | |
| BA1 | 8.00 | 28.00 | 28.34[a] | 62.03 | 97.03 | 0.29 | 0.99 |
| BA2 | 7.00 | 28.00 | 27.51[a] | 48.88 | 97.03 | 0.25 | 1.02 |
| BA3 | 8.00 | 28.00 | 27.88[a] | 53.87 | 97.03 | 0.29 | 1.00 |
| BB1 | 9.00 | 30.00 | 27.01[a] | 41.99 | 97.03 | 0.30 | 1.11 |
| BB2 | 8.00 | 30.00 | 26.67[a] | 38.86 | 97.03 | 0.27 | 1.12 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| BB3 | 6.00 | 30.00 | 27.46[a] | 47.24 | 97.03 | 0.20 | 1.09 |
| BC1 | 8.00 | 30.00 | 26.14[a] | 34.69 | 97.03 | 0.27 | 1.15 |
| BC2 | 9.00 | 30.00 | 26.29[a] | 35.76 | 97.03 | 0.30 | 1.14 |
| BC3 | 9.00 | 30.00 | 26.00[a] | 33.52 | 97.03 | 0.30 | 1.15 |
| BD1 | 14.00 | 34.00 | 25.71[a] | 31.71 | 97.03 | 0.41 | 1.32 |
| BD2 | 12.00 | 32.00 | 24.77[a] | 27.08 | 97.03 | 0.38 | 1.29 |
| BD3 | 12.00 | 32.00 | 25.02[a] | 28.17 | 97.03 | 0.38 | 1.28 |

[a] Governing theoretical failure load.



### 4.4.4 Theoretical vs. Experimental Deflections

The theoretical and experimental deflections of the beams are presented in Table 4.16. The ultimate deflection for the 0% fibre concrete specimens exceeded the predicted deflection on the average by approximately 434%. For the 0.25%, 0.5%, and the 1.0% fibre concrete specimens, the ultimate deflections exceeded the predicted deflections averagely by 549%, 571%, and 646% respectively. The difference between the theoretical deflection and the experimental deflection in the case of the 0% fibre concrete specimens can be attributed to the properties of the steel bars and the aggregates used for the concrete. The steel bars which are produced from scrap metals have been shown (Kankam and Adom-Asamoah, 2002) to possess a higher tensile strength and very little elongation compared to a standard mild steel bar. In the case of the fibre reinforced concrete specimens, the difference between the theoretical deflection and the experimental deflection can be attributed partly to the properties of the steel bars, the aggregates used and the presence of the fibres in the concrete.

The ratio of maximum deflection at collapse to deflection at first crack ranges from 20.05 to 24.90 for the 0% fibre concrete specimens, 18.41 to 59.93 for the 0.25% fibre concrete specimens, 30.02 to 46.25 for the 0.5% fibre concrete specimens, and 19.12 to 32.10 for the

1.0% fibre concrete specimens (Table 4.16). This is an indication of the ductility of the concrete. The control specimens exhibited little deflection, averaging 31.7mm and very low ductility prior to collapse compared to the fibre reinforced concrete specimens who averaged 40.1mm, 41.5mm, and 46.4mm for 0.25%, 0.50%, and 1.0% fibre reinforced concrete specimens respectively. The larger deflections at greater failure loads recorded for the fibre concrete specimens can be attributed to the action of the fibres interfering with the propagation of micro-cracks in the concrete. The fibres possess much higher ultimate tensile strength (500MPa) and are more ductile compared to the more brittle concrete (4-8MPa) hence their inclusion improves the ductility and tensile strength of the concrete.

**Table 4.16: Theoretical and experimental deflections**

| Beam no. | Deflection at first crack $\delta_{cr}$ (mm) | Deflection at failure $\delta_{max.}$ (mm) | Theoretical deflection at failure $\delta'_{max.}$ (mm) | $\delta_{max.}/\delta_{cr}$ | $\delta_{max.}/\delta'_{max.}$ |
|---|---|---|---|---|---|
| BA1 | 1.000 | 23.750 | 7.305 | 23.75 | 3.25 |
| BA2 | 1.220 | 30.380 | 7.317 | 24.90 | 4.15 |
| BA3 | 2.050 | 41.100 | 7.318 | 20.05 | 5.62 |
| BB1 | 2.390 | 44.000 | 7.315 | 18.41 | 6.02 |
| BB2 | 0.900 | 21.800 | 7.300 | 24.22 | 2.99 |
| BB3 | 0.910 | 54.540 | 7.324 | 59.93 | 7.45 |
| BC1 | 1.140 | 35.100 | 7.268 | 30.79 | 4.83 |
| BC2 | 1.250 | 37.520 | 7.278 | 30.02 | 5.15 |
| BC3 | 1.120 | 51.800 | 7.258 | 46.25 | 7.14 |
| BD1 | 1.680 | 53.920 | 7.234 | 32.10 | 7.45 |
| BD2 | 2.280 | 43.600 | 7.137 | 19.12 | 6.11 |
| BD3 | 1.600 | 41.600 | 7.165 | 26.00 | 5.81 |

## 4.5 Water Absorption

### 4.5.1 Results

The results of the water absorption test conducted on plain and fibre reinforced concrete (FRC) specimens are presented in Tables 4.17, and summarized and illustrated in Table 4.18 and Figure

4.16, respectively. It can be seen from Table 4.18 that there is a reduction in water absorption from 4.98 for the plain concrete to 4.58 for the 0.25 percent fibre concrete. This represents a reduction of 8.64 percent. There is, however, a drop in water absorption from 4.98 to 4.23 and 4.14 for 0.5 percent fibre concrete and 1.0 percent fibre concrete respectively. This represents a 17.61 percent and 20.33 percent reduction respectively compared to the plain concrete specimens.

**Table 4.17: Water absorption test results for plain and fibre reinforced concrete specimens**

| s/n | specimen id | Specimen type | actual dimensions mm) | | | oven dry weight, W1 (kg) | saturated weight, W2 (kg) | water absorption (%) |
|---|---|---|---|---|---|---|---|---|
| 1 | D1 | 1.0% fibre concrete | 149 | 149 | 150 | 7.44 | 7.62 | 2.42 |
| 2 | D2 | 1.0% fibre concrete | 149 | 149 | 150 | 7.38 | 7.76 | 5.15 |
| 3 | D3 | 1.0% fibre concrete | 148 | 149 | 152 | 7.44 | 7.80 | 4.84 |
| | | | | | | | | |
| 4 | C1 | 0.50% fibre concrete | 148 | 148 | 152 | 7.46 | 7.80 | 4.56 |
| 5 | C2 | 0.50% fibre concrete | 150 | 150 | 149 | 7.46 | 7.78 | 4.29 |
| 6 | C3 | 0.50% fibre concrete | 148 | 146 | 150 | 7.28 | 7.56 | 3.85 |
| | | | | | | | | |
| 7 | B1 | 0.25% fibre concrete | 150 | 150 | 152 | 7.48 | 7.86 | 5.08 |
| 8 | B2 | 0.25% fibre concrete | 150 | 150 | 152 | 7.62 | 7.94 | 4.20 |
| 9 | B3 | 0.25% fibre concrete | 149 | 150 | 150 | 7.62 | 7.96 | 4.46 |
| | | | | | | | | |
| 10 | A1 | control-plain concrete | 150 | 150 | 149 | 7.6 | 7.98 | 5.00 |
| 11 | A2 | control-plain concrete | 150 | 150 | 150 | 8.02 | 8.38 | 4.49 |
| 12 | A3 | control-plain concrete | 150 | 150 | 149 | 7.72 | 8.14 | 5.44 |

**Table 4.18: Average water absorption of plain and fibre reinforced concrete**

| s/n | specimen type | volume fraction of fibre (%) | ave. water absorption (%) | decrease in water absorption due to fibre addition (%) |
|---|---|---|---|---|
| 1 | control-plain concrete | 0.00 | 4.98 | 0.00 |
| 2 | 0.25% fibre concrete | 0.25 | 4.58 | 8.64 |
| 3 | 0.50% fibre concrete | 0.50 | 4.23 | 17.61 |
| 4 | 1.00% fibre concrete | 1.00 | 4.14 | 20.33 |

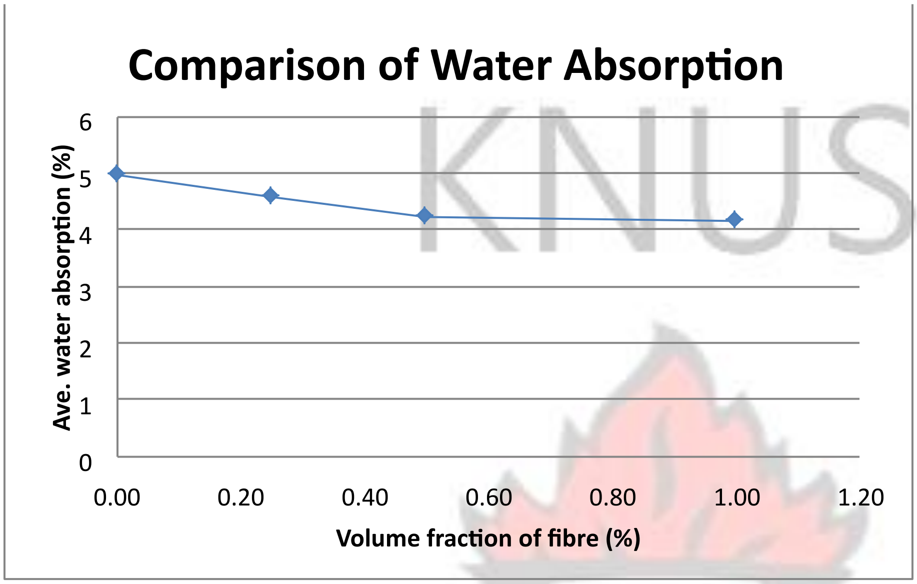


**Figure 4.16: Comparison of water absorption of plain and fibre reinforced concrete**

### 4.5.2 Analysis and Discussion

Richardson (2003) states that concrete durability, to a large extent, is generally governed by concrete‟s resistance to the penetration of aggressive media. Past researchers and specialists in the field of concrete durability who have identified permeability of concrete as being the key factor in providing durability include Basheer et al. (2001), Basheer and Norlan (2001), Long et al. (2001), Figg (1992), Roy et al. (1990), and Whiting and Walitt (1988).

Singh et al. (2014) reported a decrease in water absorption as a result of addition of fibre to concrete. A decrease over plain concrete of up to 31.33 percent was recorded when 0.5 percent volume fraction of 50mm long steel fibre was added. Similarly, for a mix containing 1.0 percent volume fraction of the same steel fibres, permeability was found to decrease over plain concrete mix by 51.94 percent. Mather et al. (1987), Vondran and Webster (1997) all recorded reduction in water permeability of between 34 and 75 percent as a result of addition of various types of fibres to concrete.

The decrease in permeability or water absorption may be attributed to the fact that considerable reduction in plastic and drying shrinkage cracks has been achieved as a result of fibre addition. The fibres act by breaking the continuity or interconnectivity of porous channels present in the concrete, thus resulting in the lower permeability of the polyethylene fibre reinforced concrete.

## 4.6 Surface Abrasion Resistance

### 4.6.1 Results

The results of the surface abrasion test are presented fully in Table 4.19, and summarized in Table 4.20 and Figure 4.17. The results indicate an initial reduction in the mean depth of wear as a result of the inclusion of fibres in the concrete. The depth of wear however went up as the fibre content increased from 0.25% and 0.5% and continue on the upward trajectory up to a fibre content of 1.0%.

**Table 4.19: Results of surface abrasion test for plain and fibre reinforced concrete**

| s/n | Specimen id | Fibre vol. fraction (%) | dimensions before test | | | mass before test (g) | mass after test (g) | depth of wear (mm) |
|---|---|---|---|---|---|---|---|---|
| 1 | A4 | 0 | 100 | mm) 100 | 102 | 2540 | 2522.7 | 0.6811 |
| 2 | A5 | 0 | 100 | 100 | 102 | 2560 | 2532.2 | 1.0859 |
| 3 | A6 | 0 | 100 | 100 | 102 | 2560 | 2530.0 | 1.1719 |
| | B4 | | 100 | 100 | 103 | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 4 | | 0.25 | | | | 2580 | 2566.6 | 0.5194 |
| 5 | B5 | 0.25 | 100 | 100 | 102 | 2540 | 2526.8 | 0.5197 |
| 6 | B6 | 0.25 | 100 | 100 | 102 | 2550 | 2530.2 | 0.7765 |
| 7 | C4 | 0.5 | 100 | 100 | 102 | 2500 | 2483.5 | 0.6600 |
| 8 | C5 | 0.5 | 100 | 100 | 104 | 2560 | 2528.3 | 1.2383 |
| 9 | C6 | 0.5 | 100 | 100 | 102 | 2540 | 2514.0 | 1.0236 |
| 10 | D4 | 1.0 | 100 | 100 | 102 | 2520 | 2496.3 | 0.9405 |
| 11 | D5 | 1.0 | 100 | 100 | 102 | 2540 | 2508.0 | 1.2598 |
| 12 | D6 | 1.0 | 100 | 100 | 102 | 2520 | 2495.0 | 0.9921 |

**Table 4.20: Mean depth of wear for plain and fibre reinforced concrete specimen**

| specimen type | volume fraction of fibre (%) | mean compressive strength (N/mm2) | mean depth of wear (mm) | % reduction in surface wear due to fibre addition |
|---|---|---|---|---|
| control-plain concrete | 0.00 | 33.07 | 0.980 | 0.00 |
| 0.25% fibre concrete | 0.25 | 25.84 | 0.605 | 38.22 |
| 0.50% fibre concrete | 0.50 | 21.09 | 0.974 | 0.58 |
| 1.00% fibre concrete | 1.00 | 17.74 | 1.064 | -8.62 |

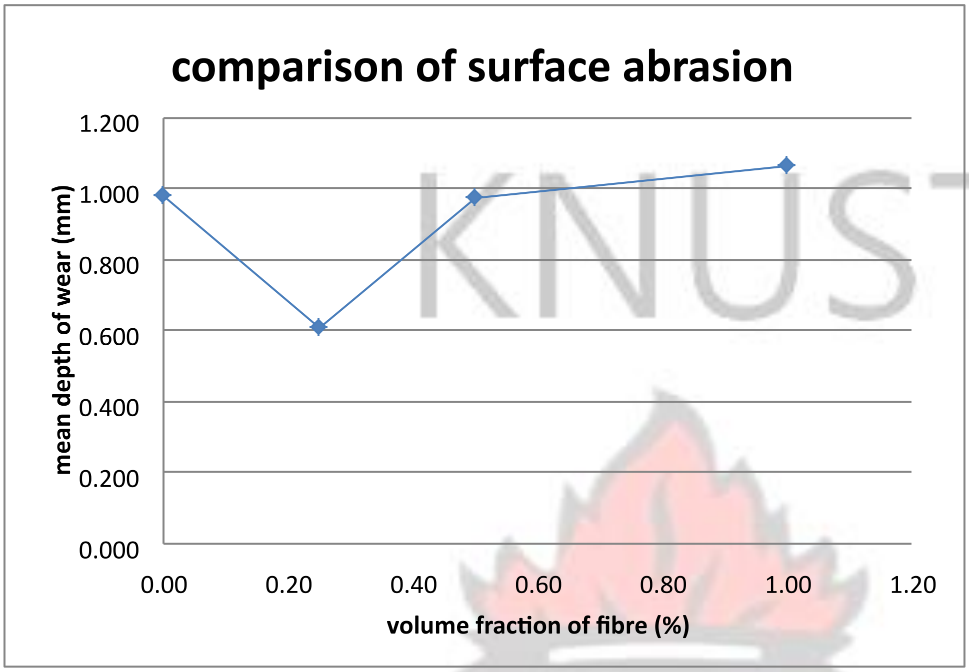


**Figure 4.17: Results of surface abrasion for plain and fibre reinforced concrete.**

### 4.6.1 Analysis and Discussion

Extensive work on the abrasion resistance of fibre concrete can be found in Vassou and Kettle (2005). The study investigated the effects of a wide range of factors including concrete mix constituents and curing regimes and also fibre characteristics such as shape, type, content, and length on the abrasion resistance of fibre reinforced concrete using the Aston abrasion tester. Some of the findings of the study were that; shape of the steel fibres was not a factor that affected the abrasion resistance but the steel fibre content and length of the fibres influence positively the abrasion resistance with the shorter fibres being most effective. Curing regime and type of fibre was also identified as significant factors affecting the abrasion resistance of concrete. Crucial finding of the study was that a relatively low dose, 0.51% of steel fibres significantly improved the abrasion resistance compared to that of the plain concrete and also the higher dose fibre concrete.

The present study recorded a 38% reduction in surface wear due to the addition of 0.25% polyethylene fibres. This value however reduced drastically by a mere 0.6% when the fibre content was doubled. The surface wear then increased marginally by 8.6% when the fibre content was increased to 1.0%. A possible explanation to the decrease in surface wear could be that, the ductile nature of the fibre makes it very difficult for abrasive forces to wear down the exposed surface of the fibre concrete compared to the brittle and rough surface of the plain concrete matrix. As the fibre content increased, the bond between the fibres and the concrete matrix begin to weaken resulting in the easy removal of the fibres from the surface of the concrete matrix and the eventual wearing down of the concrete particles. This phenomenon can be clearly seen from the reduction in compressive strength of the fibre concrete specimen as the fibre content increases.

Research by Liu (1980), Alexanderson (1982), and Nanni (1989) all concluded that "inclusion of fibres into concrete matrix positively affects the abrasion resistance".

# CHAPTER 5 - SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Summary

In this study, concrete properties such as compressive strength, flexural tensile strength, post crack toughness, crack propagation and cracking pattern, water absorption, and surface abrasion resistance of plain concrete and polyethylene fibre reinforced concrete containing different volume fractions of the polyethylene fibre have been investigated.

Twelve (12) concrete cubes measuring 150mm were manufactured, cured for 28 days and then tested in compression to gain insight into the effects of polyethylene fibres on the compressive strength of concrete. The modulus of rupture test was also conducted to investigate the flexural tensile strength of the plain concrete and polyethylene fibre reinforced concrete. Concrete prisms measuring 100×100×300mm were manufactured, cured for 28 days and then subjected to a 3point loading test using a Universal Test Frame at a constant loading in increments of 2kN up to the first crack. The peak load sustained by the prism was used to compute the modulus of rupture for the test specimen.

Post crack toughness test was also conducted on twelve concrete beams measuring 150×200×2500mm to investigate the energy absorption capacity of the polyethylene fibre reinforced concrete as well as to gain more insight into the crack propagation and cracking pattern. The concrete beams were subjected to 4-point loading test using a Universal Test Frame at a constant loading in 2kN increments. Measurements were taken of the load, mid span deflection, crack width and spacing using appropriate measuring instruments. The post crack flexural toughness indices were obtained from the load-deflection curves as per the procedure outlined in ASTM C1018. Areas under the load-deflection curve were obtained by trendline analysis and curve fitting methods using the EXCEL solver tool pack.

Knowledge of abrasion resistance of the polyethylene fibre reinforced concrete was also sought through surface abrasion resistance test. Twelve concrete cubes measuring 100mm were subjected to the surface abrasion per Bureau of Indian standards IS 15658-2006. The concrete cubes were placed on the grinding disc of the abrasion testing machine and the disc was run at 45 rpm for 20 seconds. The abrasive wear of the specimen after 18 cycles was calculated as the mean loss in specimen thickness $\Delta t$ in mm. The water absorption capacity of the polyethylene fibre reinforced concrete was also investigated. Twelve 150mm concrete cubes were oven dried for 72 hours at a temperature of 105°C and then immersed in water for 24 hours. The water absorption was then computed as the difference in weight of the saturated specimen and the oven dried specimen divided by the oven dried weight.

## 5.2 Conclusions

This investigation has evaluated the use of polyethylene fibres as reinforcement in concrete and the major findings are as follows:

- There is a reduction in compressive strength of concrete as a result of addition of polyethylene fibres to concrete. The test results showed a significant reduction in the 28 days compressive strength of polyethylene fibre reinforced concrete from 22% to 46% as the fibre content increased from 0.25% to 1.0% respectively compared to the plain control concrete.
- Flexural tensile strength on the other hand increased as a result of the addition of polyethylene fibres to concrete. Results of the modulus of rupture test showed that the flexural tensile strength increased by 2.5% for 0.25% fibre concrete compared to the reference concrete and by 14% and 27% respectively for 0.50% and 1.0% fibre concrete compared to the reference concrete.
- The post crack toughness of the control beam was higher than that of the 0.25% polyethylene fibre reinforced concrete and the 0.5% polyethylene fibre reinforced concrete. That of the 1.0% polyethylene fibre reinforced concrete was however the highest. The control specimens were found to possess a mean energy absorption capacity of 3.92, 8.57,

and 37.24 at $I_5$, $I_{10}$, and $I_{30}$ respectively compared to the 3.17, 6.64, and 28.08 for the 0.25% polyethylene fibre reinforced concretes and 2.82, 5.64, and 21.81 for the 0.5% polyethylene fibre reinforced concretes. The 1.0% polyethylene fibre reinforced concretes however recorded a mean energy absorption capacity of 4.79, 11.16, and 51.27 at $I_5$, $I_{10}$, and $I_{30}$ respectively.

- The polyethylene fibre reinforced concrete beams had better crack development and cracking pattern compared to the beams without polyethylene fibre. Structural concrete beams are required to have closely spaced cracks of fine crack widths. Measurements of crack widths showed that the crack openings in the fibre reinforced beams were 0.49mm for the 1.0% polyethylene fibre concrete, 1.43mm, and 1.73mm for the 0.5% polyethylene fibre concrete and the 0.25% polyethylene fibre concrete respectively compared to the 1.80mm for the beams without fibres. Crack spacings in the 1.0% fibre reinforced concrete beams were closer, averaging 119.14mm compared to the 128.93mm for the concrete beams without fibres. The 0.25% and the 0.5% polyethylene fibre concretes averaged 127.40mm and 119.65mm respectively. The 1.0% polyethylene fibre concrete produced 12-15 cracks compared to the 10-14 cracks obtained for the control concrete. The 0.25% and the 0.5% polyethylene fibre concretes each produced 11-14 and 12-13 cracks respectively.
- The 100mm concrete cubes containing 0.25% polyethylene fibres showed little surface wear compared to the plain concrete cubes. The 38% reduction in surface wear obtained is significant; therefore the benefit will be tangible and worthwhile. The concrete cubes containing 0.5% and 1.0% polyethylene fibres however showed significant surface wear just like the plain concrete with the 0.5% polyethylene fibre reinforced concrete recording 0.58% reduction and the 1.0% polyethylene fibre reinforced concrete recording -8.62% reductions.

- There is a reduction in water absorption of concrete as a result of addition of polyethylene fibres to concrete. The test results showed a reduction in water absorption of polyethylene fibre reinforced concrete from 9% to 20% as the fibre content increased from 0.25% to 1.0% respectively compared to the plain control concrete.

## 5.3 Recommendations

The strength and performance of polyethylene fibre reinforced concrete was evaluated in this study. The research used fibre of width 5mm, length 40mm and thickness 0.095mm and volume fractions 0.25%, 0.50%, and 1.0%. Based on the findings of the research the following are some recommended applications of the polyethylene fibre concrete.

(i) The improvement in the flexural strength (modulus of rupture) and the surface abrasion resistance of the polyethylene fibre-reinforced concrete suggest that the technology could be more suitable for slab-on-grade construction such as industrial warehouse floors, commercial floor slabs, housing floor slabs, pavements, driveways and sidewalks. Fibre volume fractions of 0.5% - 1.0% can be used in addition to a plasticizer to minimize the reduction in compressive strength of the resulting concrete.

(ii) This research has also shown that polyethylene fibre-reinforced concrete could also be applied in the construction of tunnel linings and water retaining structures such as reservoirs and dams due to the higher surface abrasion resistance, higher resistance to water absorption, higher ductility and post crack flexural toughness and finer cracking pattern of the fibre concrete. In this case, fibre volume fraction of 1.0% can be used together with a plasticizer to minimize the reduction in the compressive strength of the concrete.

Suggestions for future research work on polyethylene fibre reinforced concrete include:

A study that will consider other sizes of the polyethylene fibre, different aspect ratios and also wider volume fractions could be useful for determining the most cost effective fibre size, aspect ratio and fibre content that does not sacrifice strength.

A study on the economics and cost/ benefit aspects of polyethylene fibre reinforcement for concrete is essential. Such a study should consider the cost savings in the areas of construction, installation, and long-term maintenance that could be gained through the use of polyethylene fibre reinforced concrete in structures.

Further studies are also needed using more specimen compared to the few specimen used in this case to help refine the results obtained in the present studies.

Seismicity and dynamic analysis should also be conducted on model structures and structural elements reinforced with polyethylene fibres based on the fact that polyethylene fibre concrete beams have high toughness index.

The fire resistance of polyethylene fibre reinforced concrete elements and structures should also be investigated if polyethylene fibre reinforcement is to be deployed in serious structural application. The results of such a study will provide information on performance of polyethylene fibre reinforced concrete during fire outbreaks.

## REFERENCES

Alexanderson J. (1982). Polymer cement concrete for industrial floors. *Polymers in Concrete; International Congress on Polymers in Concrete. Vol. 1.* pp. 360-373.

American Society for Testing and Materials (2009), ASTM C78-09. Standard Test Method for Flexural Strength of Concrete. *American Society for Testing and Materials.* West Conshohocken, PA.

American Society for Testing and Materials (1997), ASTM C 1018. Standard Test Method for Flexural Toughness and First Crack Strength of Fibre Reinforced Concrete, *American Society for Testing and Materials.* West Conshohocken, PA.

American Society for Testing and Materials (2003), ASTM C116-03. Standard Specification for Fibre Reinforced Concrete and Shortcrete, *American Society for Testing and Materials.* West Conshohocken, PA.

Aulia T.B. (2002). Effects of polypropylene fibres on the properties of high-strength concretes. *LACER. Vol. 7.* pp. 43-59.

Barris C., Torres L.I., Turon A., Baena M. & Catalan A. (2009). An experimental study of the flexural behaviour of GFRP RC beams and comparison with prediction models. *Composite Structures. Vol. 9.* pp. 286-295.

Basheer L., Kropp J. and Cleland D. (2001). Assessment of the durability of concrete from its permeation properties. *Construction and Building Materials. Vol. 15.* pp. 93-113.

Basheer L. and Nolan E. (2001). Near surface moisture gradients and in situ permeation tests. *Construction and Building Materials*. Vol. 15. pp. 105-23.

Beddar M. (2004). Fibre-Reinforced Concrete: Past, Present and Future. *Concrete Society. Vol. 3.*Camberley.

Bentur A. and Mindess S. (1990). Fibre Reinforced Cementitious Composites. *Elsevier Science Publication Ltd.* England, UK.

Bentur A., Mindess S. and Diamond S. (1985). Pull out processes in steel fiber reinforced cement. *International Journal of Cement Composites and Lightweight Concrete. Vol. 7. No. 1.* pp. 29-38.

British Standards Institution (1997), BS 8110. Structural use of concrete. Part 1: code of practice for design and construction. *British Standards Institution, London.*

British Standards Institution (1983), BS 1881: Part 116. Method for Determination of Compressive Strength of Concrete Cubes. *British Standards Institution, London.*
British Standards Institution (1983), BS 1881: Part 122. Method for Determination of Water Absorption. *BSI, London*

Building Research Establishment (2000). Constructing the future. *Cement Association of Canada*. Design Build No. 7.

Bureau of Indian Standards (2006), IS 15658: Annex C. Abrasion Resistance of Concrete Paving Blocks. *Bureau of Indian Standards, New Delhi*

Chanvillard G. and Aïtcin P. C. (1996). Pull-Out Behavior of Corrugated Steel Fibers – Qualitative and Statistical Analysis. *Advanced Cement Based Materials*. *Vol. 8. No. 4.* pp. 2841.

Clark J. (2006). Regarding reduced density in fibre reinforced concrete. *Concrete Society.* United Kingdom.

Concrete Society (2007). Guidance on the Use of Macro-synthetic-fibre-reinforced Concrete. *Technical Report 65, The Concrete Society*. Camberley.

Department of Transportation. (2012). Method of test for flexural strength of concrete. *California test 523.* Sacramento, California.

Ezeldin A. S. and Shah T. W. (1995). Analytical immediate and long-term deflections of fiber reinforced concrete beams. *Journal of Structural Engineering. Vol. 12. No. 4.* pp. 727-738.

Fibermesh Company Job Report (1987). Office/ Warehouse slab protection. *Fibermesh Company Technical Information*.

Figg J. (1992). Early age permeability measurements for prediction of concrete durability. *American Concrete Institute, Detroit.* pp. 289-303.

Fraternali F., Ciancia V., Chechile R., Rizzano G., Feo L. & Incarnato L. (2011). Experimental study of thermo-mechanical properties of recycled PET fibre-reinforced concrete. *Composites Structures. Vol. 93.* pp. 2368-2374.

Goeb E.E. (1989). Do plastic fibers replace wire mesh in slab-on-grade? *Concrete Technology Today. Vol 10. No. 1*.

Gold S.J. (2000). An investigation into the effects of polypropylene fibres on certain properties of reinforced concrete. *Ready Mixed Concrete*. Pp. 2-9.

Goldfein S. (1963). Plastic fibrous reinforcement for portland cement. *Technical Report 1757-TR (U.S. Army Research and Development Laboratories).* Fort Belvoir, VA.

Golpalaratham V.S. and Gettu R. (1995). On the characteristics of flexural toughness in fibre reinforced concrete. *Cement and Concrete Composites. Vol. 17*. pp. 39-254.

Golpalaratham V. S. and Shah S. P. (1987). Tensile Failure of Steel Fiber-Reinforced Concrete. *Journal of Engineering Mechanics*. Vol. 113. No. 5. pp. 635-652.

Gossla U. (2006). Flachdecken aus Stahlfaserbeton. *Beton- und Stahlbetonbau*. Vol. 101. No. 2. pp. 93-102.

Griffith A.A. (1921). The phenomena of rupture and flow in solids. *Philosophical Transactions of the Royal Society of London*. *A 221*. pp. 163-198

Grünewald S. (2004). Performance-based design of self-compacting fibre reinforced concrete. Ph.D. thesis. *Department of Structural and Building Engineering. Delft University of Technology, Netherland*.

Hamoush S., Abu-Lebdeh T. & Cummins T. (2010). Deflection behaviour of concrete beam reinforced with PVA micro-fibers. *Construction and Building Material*. *Vol. 24.* pp. 2285-2293.

Hannant D.J. (1995). Fibre reinforcement in the cement and concrete industry. *Materials Science and Technology. Vol. 11.* pp. 853-61.

Hannant D.J. (1978). Fiber Cements and Fiber Concretes. *Materials Science and Technology. Vol. 2.* pp. 8-17.

Hannant D.J., Zonsveld J.J., and Hughes D.C. (1978). Polypropylene Film in Cement Based Materials. *Engineering Composite. Vol. 9.* pp. 83-88.

Hasaba S., Kamakura M., Koizumi T., and Takemoto K. (1984). Resistibility Against Impact Load and Deformation Characteristics Under Bending Load in Polymer Hybrid (Polymer and Steel) Fiber Reinforced Concrete. *ACI SP-8.* pp. 187-196.

Hughes B.P. and Fattuhi N.I. (1976). Improving the toughness of high strength cement paste. *Concrete. Vol. 7.* pp. 185-8.

Integrated Waste Management Authority (2016). Impact of Plastic. www.iwma.com [accessed 2016 May 21].

Jansson A. (2008). Analysis and design methods for fibre reinforced concrete: a stateof-the-art report. *Department of Civil & Environmental Engineering, Div. of Structural Engineering/ Concrete structures*. Chalmers University of Technology. Göteborg, Sweden.

Jimoh Y.A. and Kolo S.S. (2011). Dissolved pure water sachet as a modifier of optimum binder content in asphalt mixes. *Epistemics in Science, Engineering and Technology. Vol. 1. No. 4. Pp 176-184*

Kandasamy R. and Murugesan R. (2011). Fibre reinforced concrete using domestic waste plastics as fibres. *Journal of Engineering and applied sciences. Vol. 6. No. 3.* pp . 75-82 Kankam C.K. and Adom-Asamoah M. (2002). Strength and ductility characteristics of reinforcement steel bars milled from scrap metals. *Journal of Materials and Design. Vol 23.* pp. 537-545.

Kim S.B., Yi N.H., Kim H.Y., Kim J.J. & Song Y.C. (2010). Material and structural performance evaluation of recycled PET fibre reinforced concrete. *Cement & Concrete Composites*. *Vol. 32*. pp. 232-240.

Kobayashi K. and Cho R. (1981). Flexural behavior of polyethylene fibre reinforced concrete. *The International Journal of Cement Composites and Lightweight Concrete. Vol 3.* pp. 19-25.

Kooiman A. G. (2000). Modelling steel fibre reinforced concrete for structural design. *Doctoral Thesis.* Technical University of Delft, Netherlands. pp.184.

Kraai P.P. (1985). A Proposed Test to Determine the Cracking Potential Due to Drying Shrinkage of Concrete. *Concrete Construction.* pp 775-778.

Kullaa J. (1994). Constitutive modelling of fibre-reinforced concrete under uniaxial tensile loading. *Composites*. *Vol. 25. No. 10*. pp. 935-944.

Leung C. K. Y. and Li V. C. (1991). New strength-based model for debonding of continuous fibres in an elastic matrix. *Journal of Material Science. Vol. 26*. pp. 5996-6010.

Li C. and Stang H. (1997). Interface property characterization and strengthening mechanisms in fibre reinforced cement based composites. *Advanced Cement Based Materials*. *Vol. 6. No. 1*. pp. 1-20.

Li C., Stang H. and Krenchel H. (1993). Micromechanics of crack bridging in fibre reinforced concrete. *Materials and Structures*. Vol. 26. No. 8. pp. 486-494.

Liu T.C. (1980). Maintenance and preservation of concrete structures: EM DASH 3. Abrasionerosion resistance of concrete. *US Army Engineer Waterways Experiment Station Technical Report. No. C-78-4*. p. 129

Löfgren I. (2005). Fibre-reinforced Concrete for Industrial Construction-a fracture mechanics approach to material testing and structural analysis. *PhD Thesis, Department of Civil and Environmental Engineering*. Chalmers University of Technology. Göteborg, Sweden. pp 268.

Lok T. S. and Pei J. S. (1998). Flexural behaviour of steel fiber reinforced concrete. *Journal of Materials in Civil Engineering. Vol. 10. No. 2.* pp. 86- 97.

Lok T. and Xiao J. (1999). Flexural strength Assessment of steel fiber reinforced concrete. *Journal of Materials in Civil Engineering. Vol. 11. No. 3.* pp. 188-196.

Long A.E., Henderson G.D., and Montgomery F.R. (2001). Why assess the properties of nearsurface concrete. *Construction and Building Materials. Vol. 15*. pp. 65-79.
Ludirdja D. and Young J.F. (1992). Synthetic fibre reinforcement for concrete. *USACERL Technical Report FM- 93/02.* Champaign, IL

Mather K., Mather B., Vondran G. & Meyer I.A. (1987). Concrete durability. *American Concrete Institute International Conference. Vol. 1.* USA. pp. 21.

Morh P., Hansen W., Jensen E. & Pane I. (2000). Transport properties of concrete pavements with excellent long-term in-service performance. *Cement and Concrete Research*. *Vol. 30*. pp. 1903-10.

Namur G. G., and Naaman A. E. (1989). A Bond Stress Model for Fiber Reinforced Concrete Based on Bond Stress Slip Relationship. *ACI Materials Journal. Vol. 86. No. 1.* pp. 45-57.

Nanakorn P. and Horii H. (1996). A fracture-mechanics-based design method for SFRC tunnel linings. *Tunnelling and Underground Space Technology*, *Vol. 11. No. 1.* pp. 39-43.

Nanni A. (1989). Abrasion resistance of roller compacted concrete. *American Concrete Institute Material Journal*. pp. 559-565.

Neville A.M. and Brooks J.J. (2010). Concrete technology, 2nd Edition. *Pearson education limited*. Prentice hall. Harlow.

Pelisser F., Montedo O.R., Gleize P.J. & Roman H.R. (2012). Mechanical properties of recycled PET fibres in concrete. *Materials Research. Vol. 15. No. 4.* pp. 679-686.

Pilakoutas K., Guadagninni M., Neocleous K., and Matthys S. (2009). The FIB perspective on FRP reinforcement in RC. *FRPRCS-09. Sydney*.

Ramakrishnan V., Gollapudi S., & Zeller R. (1987). Performance Characteristics and Fatigue Strength of Polypropylene Fiber Reinforced Concrete. *ACI SP-105*. pp. 159-177.

Richardson A.E. (2003). Polypropylene fibres in concrete with regard to durability. *Structural Survey. Vol. 21.* pp. 87-94.

Richardson A.E. (2005). Bond characteristics of structural polypropylene fibres in concrete with regard to post-crack strength and durable design. *Structural survey. Vol. 23*. pp. 210-230.

Richardson A.E. (2006). Compressive strength of concrete with polypropylene fibre additions. *Structural Survey. Vol. 24.* pp. 138-153.

Richardson A.E., Coventry K., & Landless S. (2010). Synthetic and steel fibres in concrete with regard to equal toughness. Structural Survey. Vol. 28. pp. 355-369

Rossi P. (1999). A model for cracking in fibre-reinforced concrete structures. *Materials and Structures*. *Vol. 32.* pp. 125-130.

Roy D.M., Shi D., Scheetz B., & Brown P.W. (1990). Concrete microstructure and its relationships to pore structure, permeability and general durability of Concrete. *Annual American Concrete Institute (ACI) Convention in Toronto, Ontario, Canada.* pp. 139-52.

Singh M.P., Singh S.P., & Singh A.P. (2014). Experimental study on the strength characteristics and water permeability of hybrid steel fibre reinforced concrete. *International Scholarly Research Notices. Vol. 20.* id 518640.

Singh S.P., Singh A.P., & Bajaj V. (2010). Strength and flexural toughness of concrete reinforced with steel-polypropylene hybrid fibres. *Asian Journal of Civil Engineering. Vol. 11. No. 4.* pp. 495-507.

Stang H., Li V. C. & Krenchel H. (1995). Design and structural applications of stress-crack width relations in fibre reinforced concrete. *Materials and Structures*. *Vol. 28.* pp. 210-219.

Stang H. and Shah P. (1990). Pull-out problem: Stress Versus Fracture Mechanical Approach. *ASCE Journal of Engineering Mechanics. Vol. 11. No. 1.,* pp. 2136-2150.

The Concrete Society (2007). Guidance for the Design of Steel-Fibre Reinforced Concrete. *The Concrete Society Technical Report 63*. Camberley.

Vassou V. and Kettle R.J. (2005). The influence of fibre reinforcement on the abrasion resistance of industrial concrete floors. *30$^{th}$ conference on Our World in Concrete and Structures, Singapore Concrete Istitute*. Singapore.

Vondran G.L., and Webster T. The Relationship of Polypropylene FRC to Permeability of concrete. *American Concrete Institute SP-108.* pp 85-98.

Wang *Y.,* Backer S., and Li V.C. (1987). An Experimental Study of Synthetic Fibre Reinforced Cementitious Composites. *Journal of Materials Science. Vol 22.* pp 4281-4291.

Wang Y., Li C. V. & Backer S. (1990a). Tensile Properties of Synthetic Fiber Reinforced Mortar. *Cement and Concrete Composites*. *Vol. 12.* pp. 29-40.

Wang Y., Li C. V. and Backer S. (1990b). Experimental Determination of Tensile Behavior of Fiber Reinforced Concrete. *ACI Materials Journals*. *Vol. 87. No. 5*. pp. 461-468.

West African Clean Energy & Environment. Plastic Recycling Can Earn Ghana Gh ¢1.2m per month (May 2013). www.wacee.net [accessed 2016 May 21].

Wiss, Janney, & Elstner Assoc. (1985). Static Load Tests of Three 10ft by 10ft by 6 in. Concrete Slabs. *Fibermesh Company Report.*

Whiting D. and Walitt A. (1988). Permeability of Concrete. *American Concrete Institute Publication SP 108.* Farmington Hills, MI. pp. 195-222.

Zhang J. and Stang H. (1998). Applications of stress crack width relationship in predicting the flexural behaviour of fibre-reinforced concrete. *Cement and Concrete Research*. *Vol. 28. No. 3.* pp. 439-452.

Zollo R.F. (1984). Collated Fibrillated Polypropylene Fibers in Fiber Reinforced Concrete. *American Concrete Institute SP-8 I.*

Zollo R.F. and Itler J.A. (1986). Plastic and Drying Shrinkage in Concrete Containing Collated Fibrillated Polypropylene Fibre. *Development in Fibre Reinforced Cement and Concrete: Proceedings of the RILEM Symposium.* Sheffield, England. paper 4.5.

# APPENDICES

## Appendix A: Computation of Reinforcement Ratio for Beams

Balanced reinforcement ratio is computed as

$$\rho_b = \frac{0.85 \times \beta 1 \times fc}{fy} \times \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y}$$

$\rho_b$ = balanced reinforcement ratio

= ultimate strain of concrete = 0.0035

$cu$

$f_y$ = yield strength of steel (MPa)

$f_c$ = concrete strength (MPa)

= yield strain of steel (MPa)

$y$

$\beta_1$ = the ratio of the depth of equivalent rectangular stress block to the neutral axis depth

The value of β1 depends on fc. For fc ≤ 30MPa, β1 = 0.85

For fc >30MPa, β1 = 0.85 – (fc – 30)× 0.05/7

Actual reinforcement ratio is computed as

$$\rho_a = \frac{As}{b \times ds}$$

$\rho_a$ =actual reinforcement ratio

$A_s$ =total cross sectional area of steel rebars

$b$ =breadth of beam section $d_s$ =depth of

the steel rebar level **Appendix B: Load-**

**Deflection Curves of Beams**

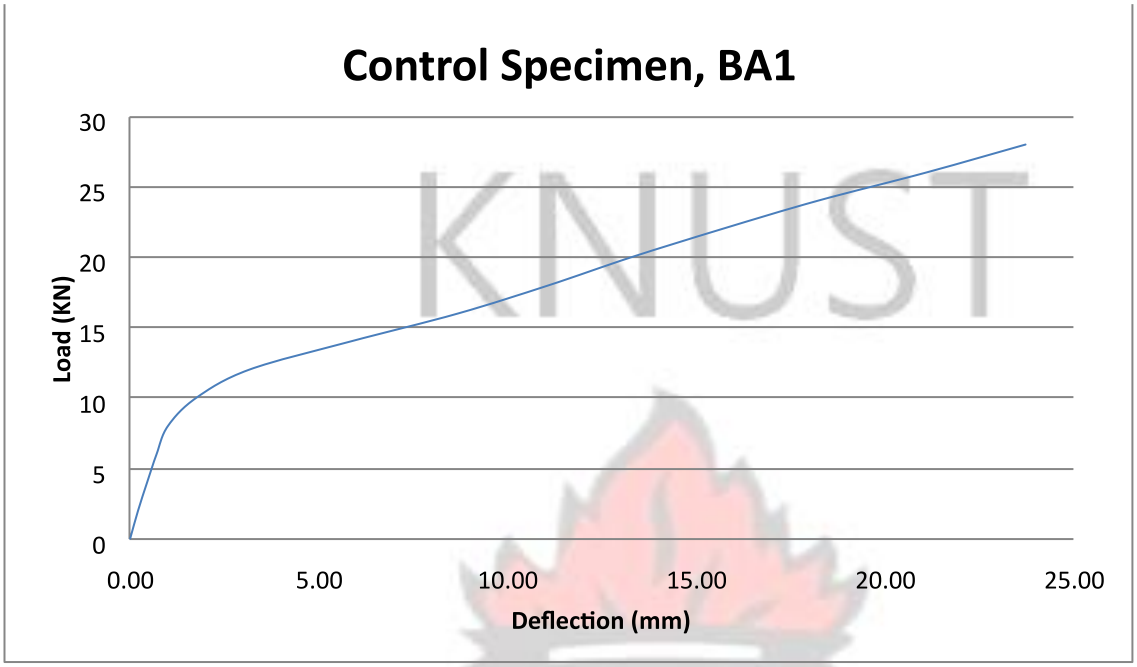


**Figure B.1: Load-Deflection Curve for Specimen BA1**

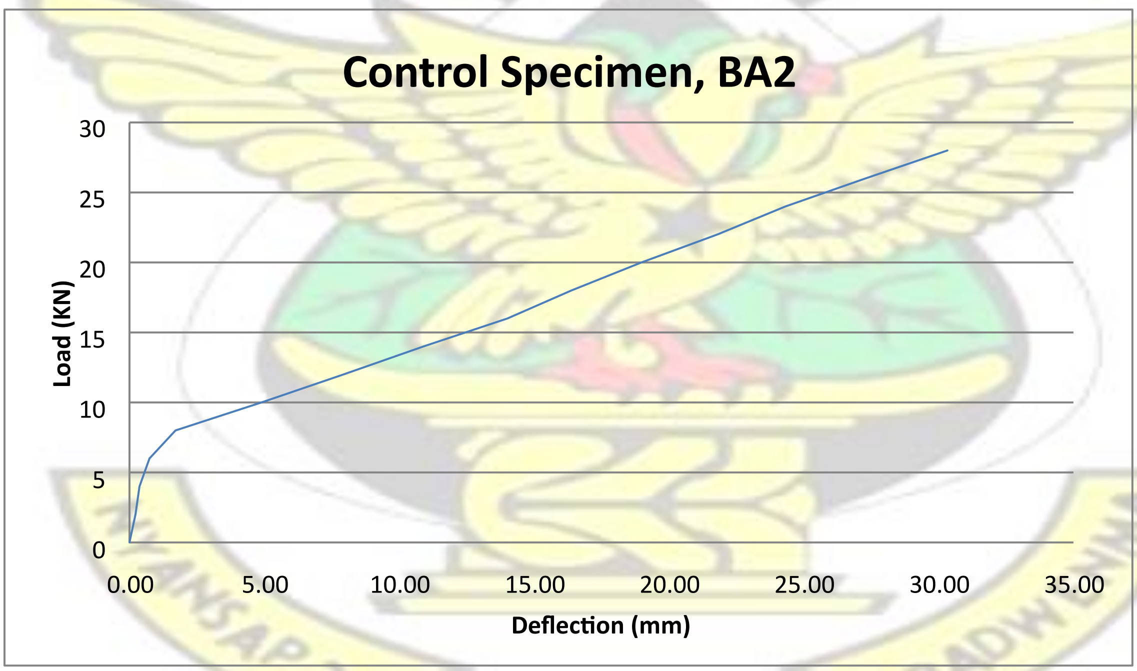

**Figure B.2: Load-Deflection Curve for Specimen BA2**

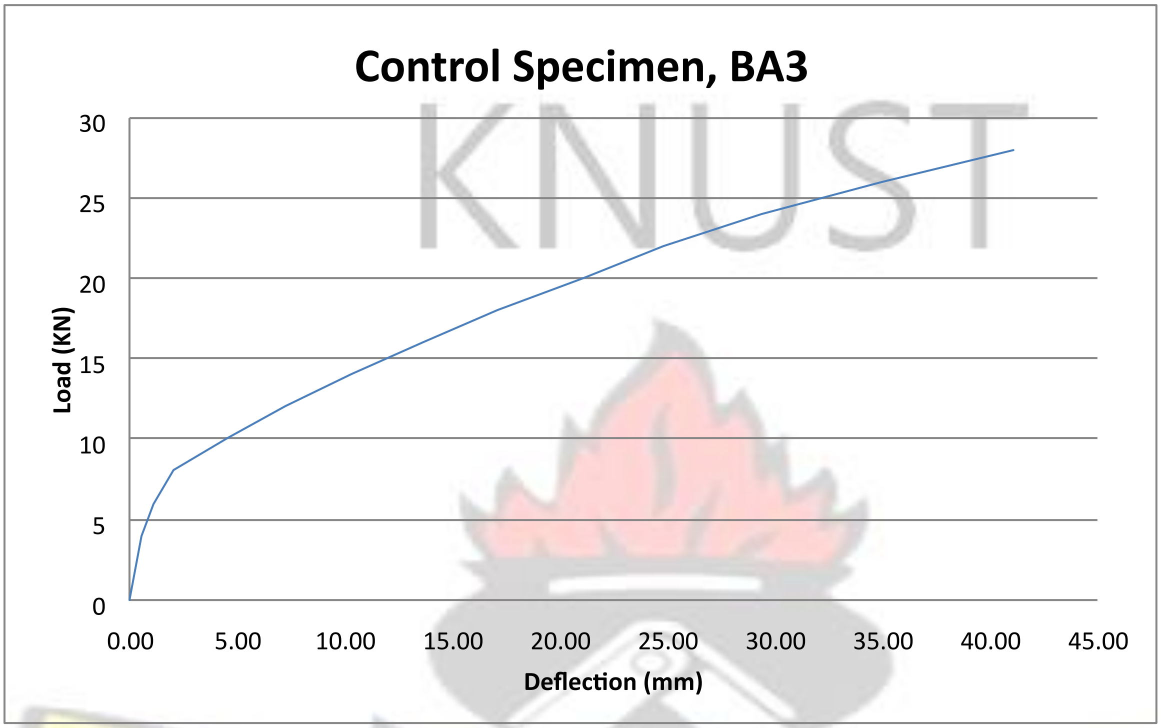


**Figure B.3: Load-Deflection Curve for Specimen BA3**

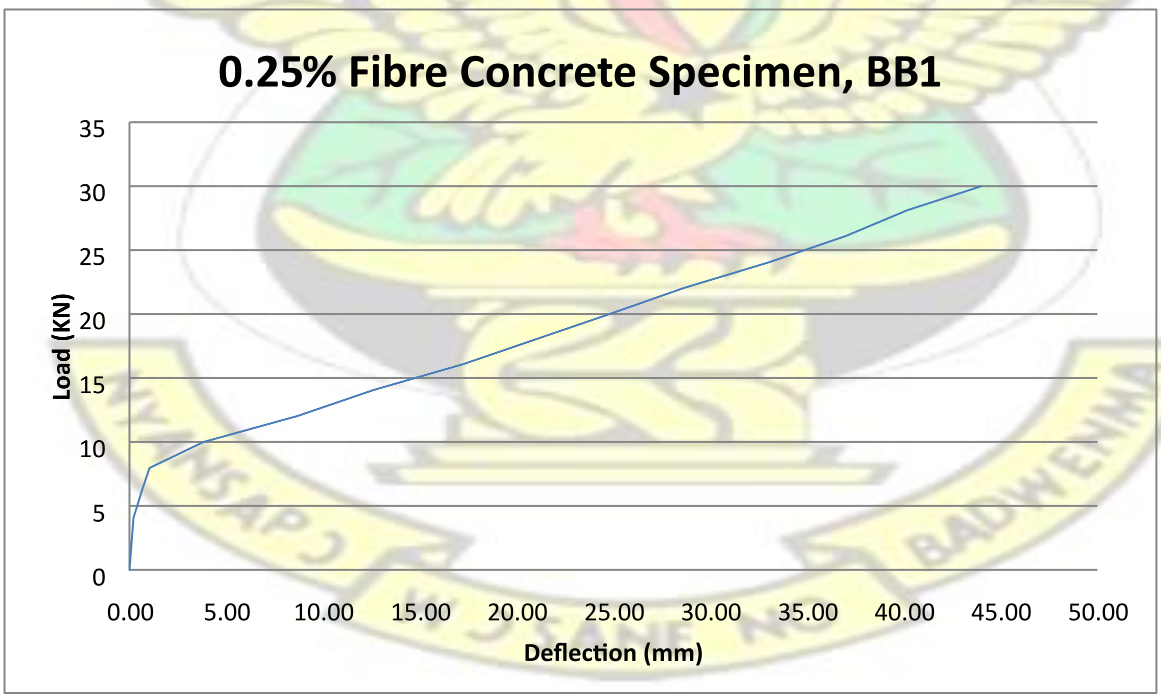

**Figure B.4: Load-Deflection Curve for Specimen BB1**

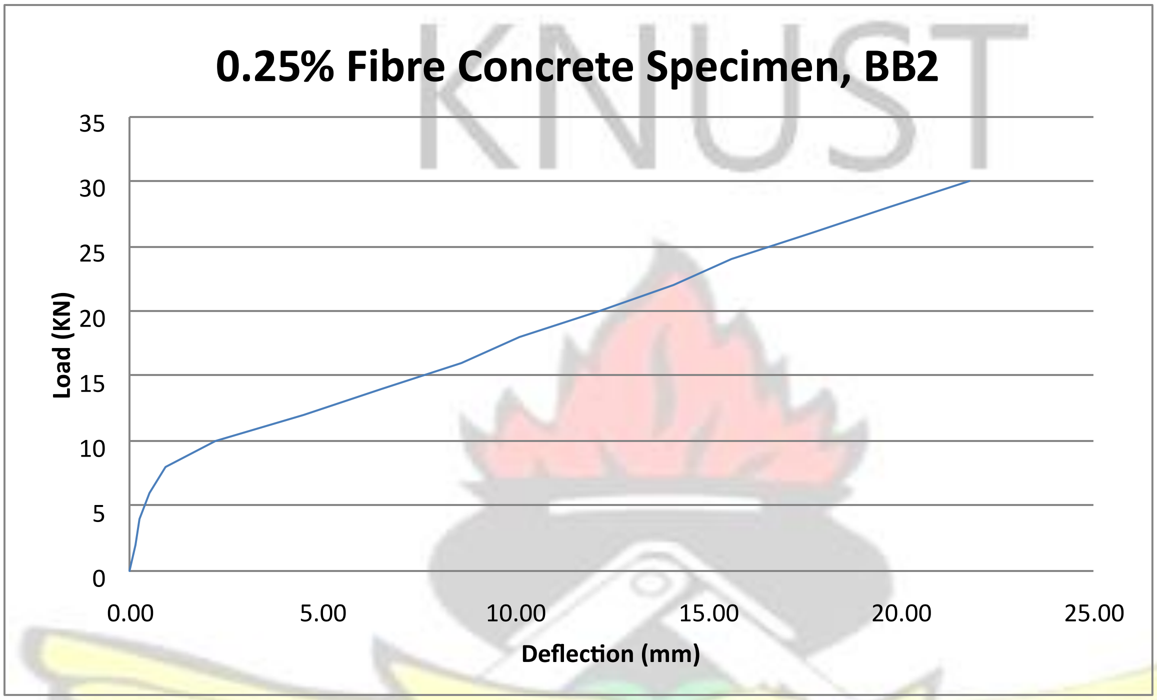


**Figure B.5: Load-Deflection Curve for Specimen BB2**

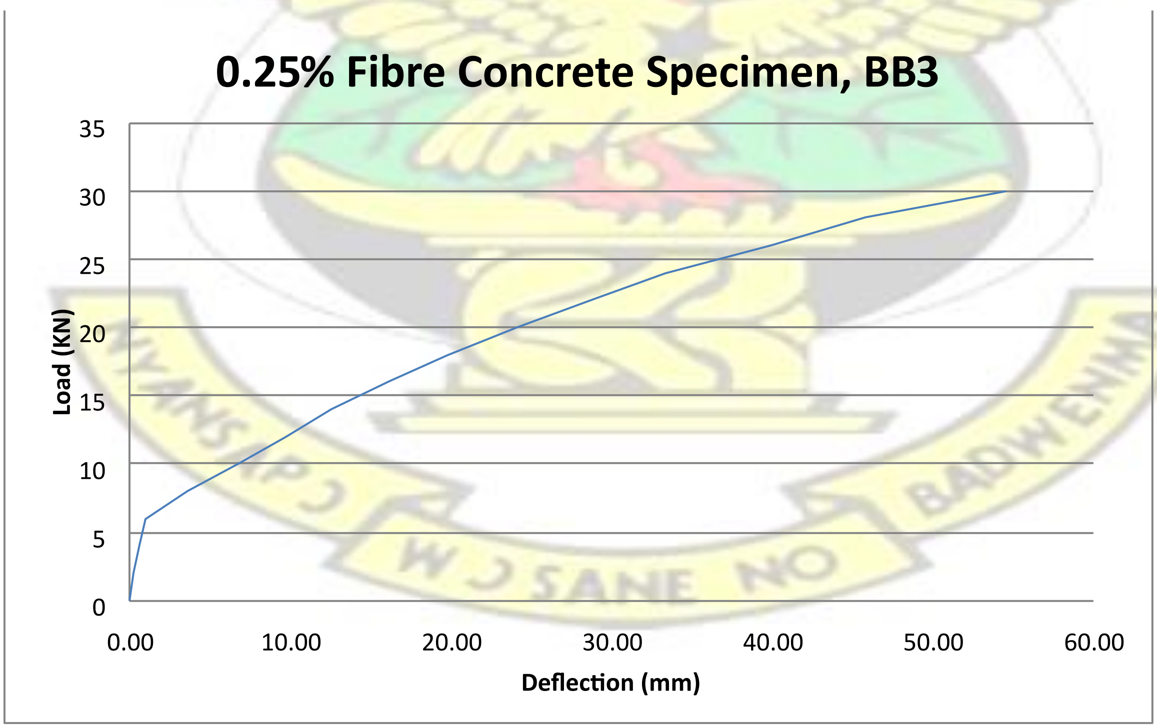

Figure B.6: Load-Deflection Curve for Specimen BB3

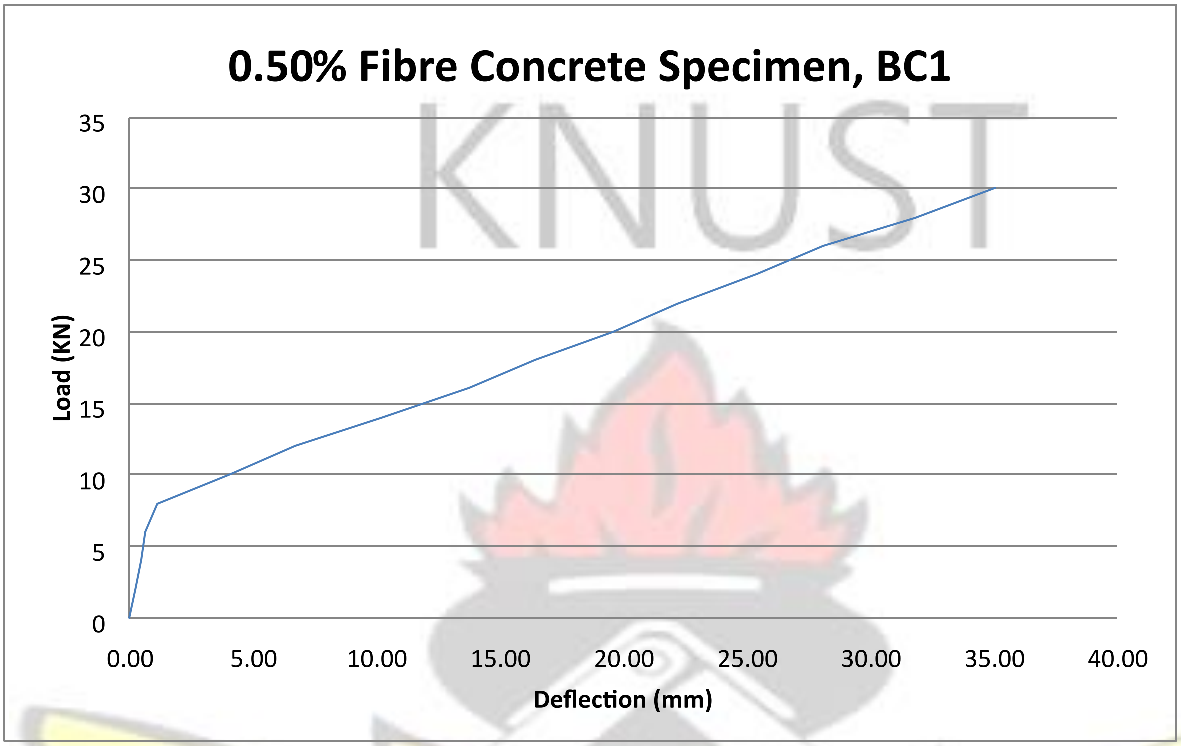


Figure B.7: Load-Deflection Curve for Specimen BC1

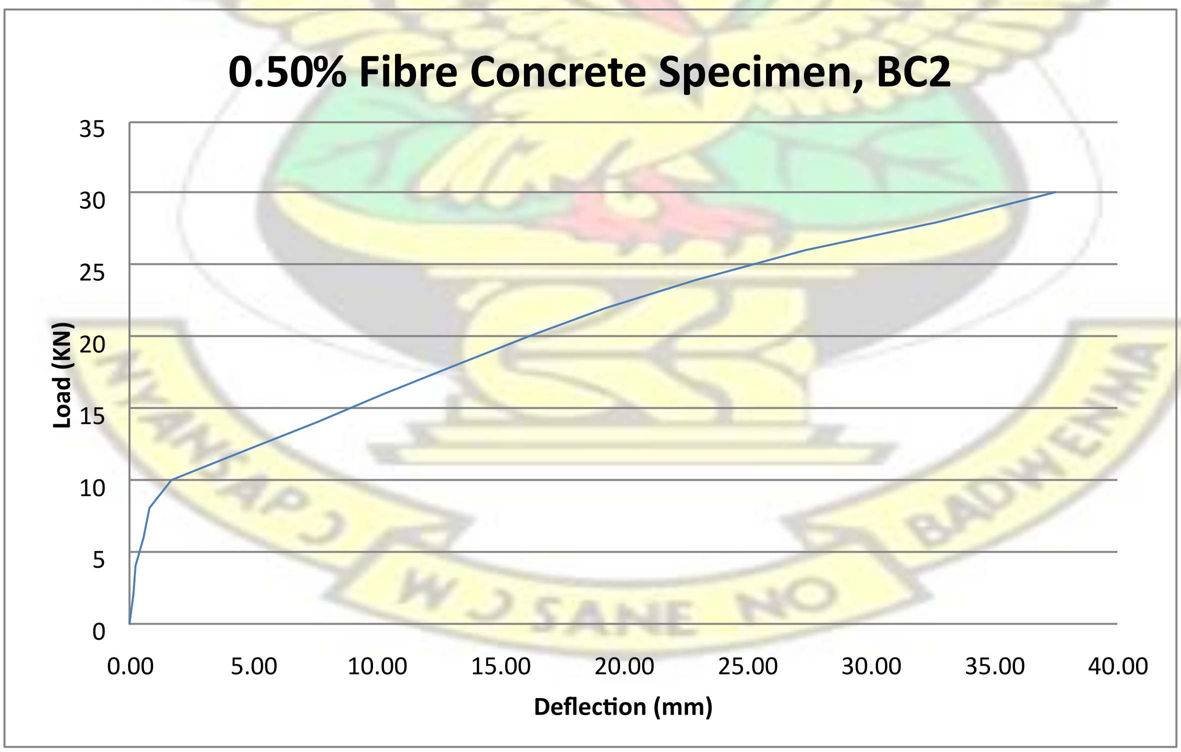

**Figure B.8: Load-Deflection Curve for Specimen BC2**

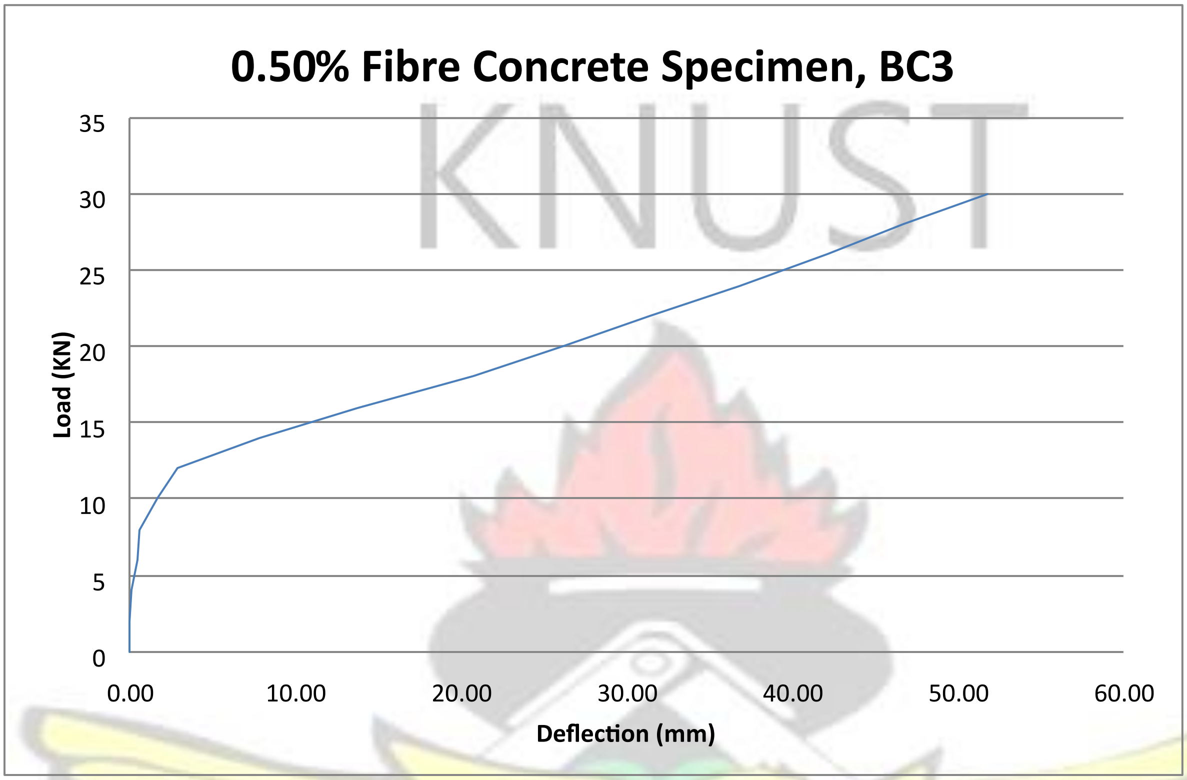


**Figure B.9: Load-Deflection Curve for Specimen BC3**

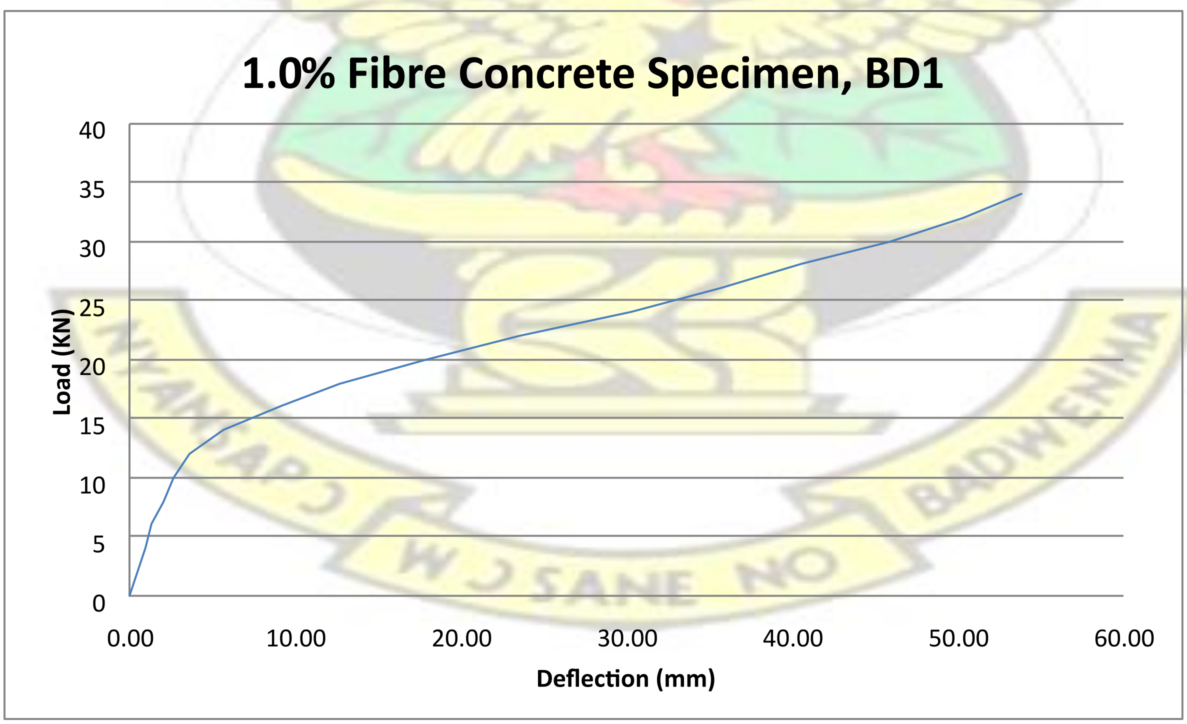

Figure B.10: Load-Deflection Curve for Specimen BD1

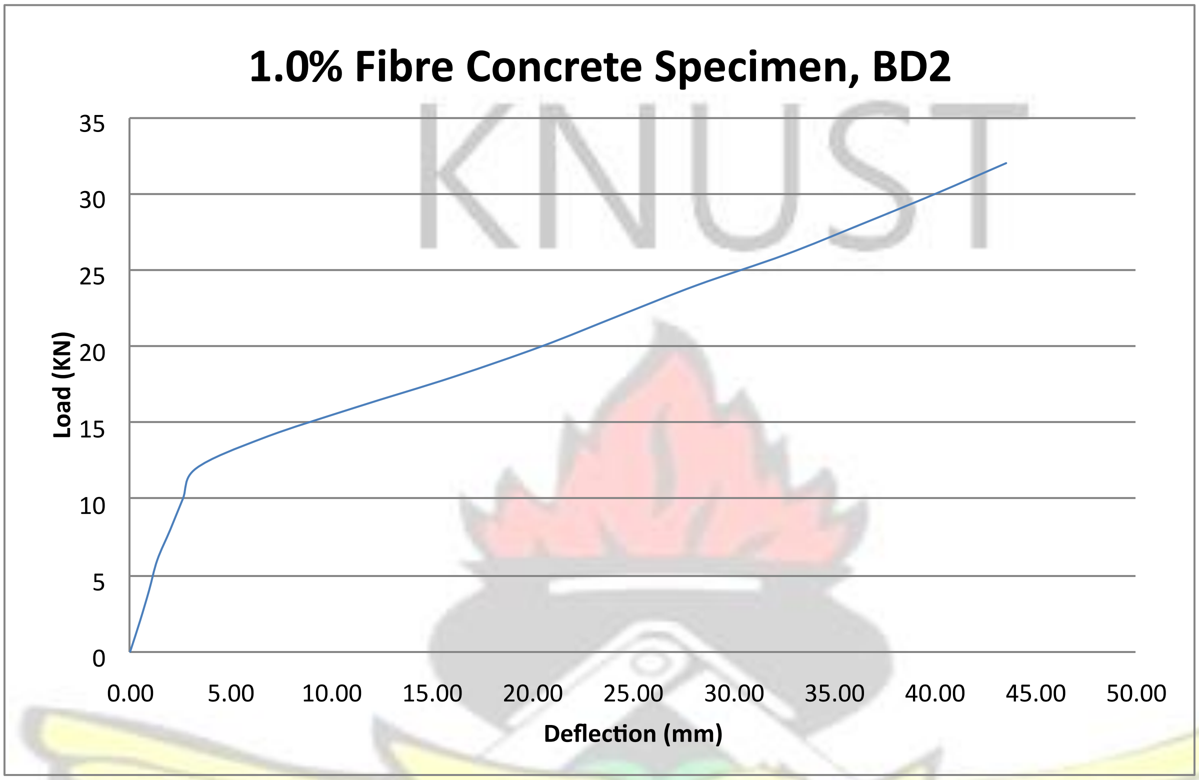


Figure B.11: Load-Deflection Curve for Specimen BD2

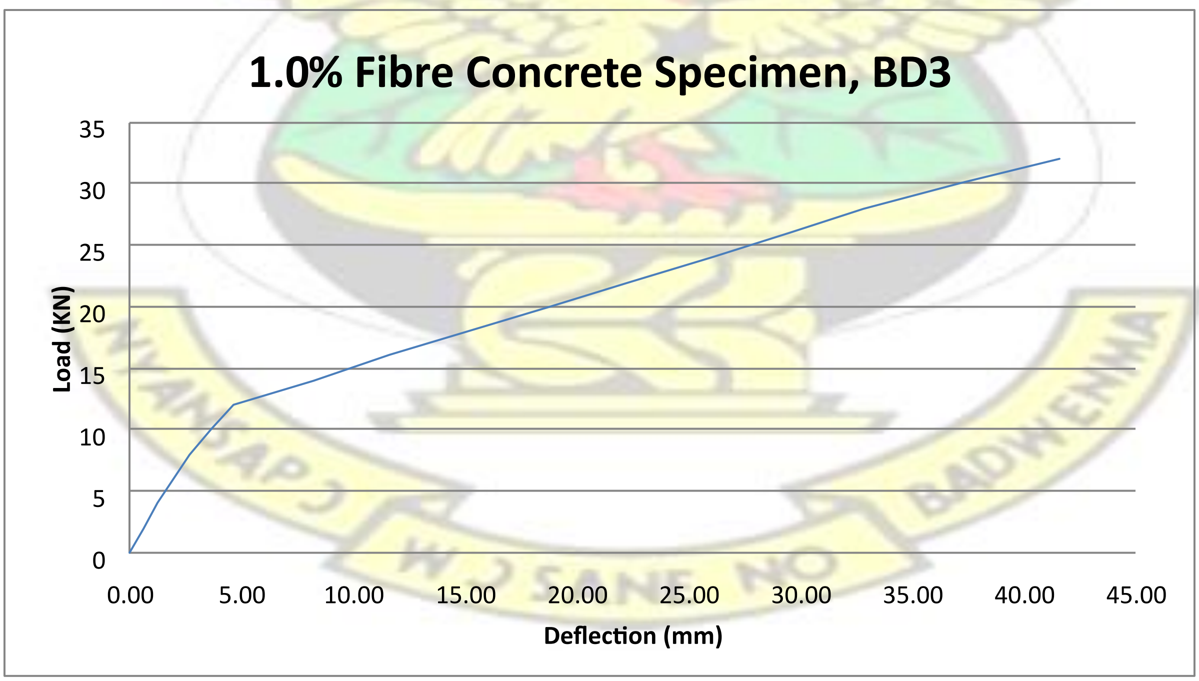

**Figure B.12: Load-Deflection Curve for Specimen BD3 Appendix C: Experimental Test Program**

| Compressive strength test | | | | | | | |
|---|---|---|---|---|---|---|---|
| specimen id | cube dimensions (mm) | specimen type | volume fraction of fibre (%) | volume of concrete (m3) | mass of concrete (kg) | mass of fibre (kg) | volume of fibre (m3) |
| CA1 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CA2 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CA3 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CB1 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CB2 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CB3 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CC1 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CC2 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CC3 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CD1 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| CD2 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| CD3 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| **total** | | | | **0.0405** | **95.580** | **0.159** | **0.0001763** |

**Figure C.1: Test Program for Compressive Strength Test**

| Modulus of rupture test |
|---|

| specimen id | prism dimensions (mm) | specimen type | volume fraction of fibre (%) | volume of concrete (m3) | mass of concrete (kg) | mass of fibre (kg) | volume of fibre (m3) |
|---|---|---|---|---|---|---|---|
| PA1 | 100×100×300 | plain concrete | 0 | 0.003000 | 7.080 | 0.000 | 0 |
| PA2 | 100×100×300 | plain concrete | 0 | 0.003000 | 7.080 | 0.000 | 0 |
| PA3 | 100×100×300 | plain concrete | 0 | 0.003000 | 7.080 | 0.000 | 0 |
| PB1 | 100×100×300 | low content fibre concrete | 0.25 | 0.003000 | 7.080 | 0.007 | 7.489E-06 |
| PB2 | 100×100×300 | low content fibre concrete | 0.25 | 0.003000 | 7.080 | 0.007 | 7.489E-06 |
| PB3 | 100×100×300 | low content fibre concrete | 0.25 | 0.003000 | 7.080 | 0.007 | 7.489E-06 |
| PC1 | 100×100×300 | medium content fibre concrete | 0.5 | 0.003000 | 7.080 | 0.013 | 1.498E-05 |
| PC2 | 100×100×300 | medium content fibre concrete | 0.5 | 0.003000 | 7.080 | 0.013 | 1.498E-05 |
| PC3 | 100×100×300 | medium content fibre concrete | 0.5 | 0.003000 | 7.080 | 0.013 | 1.498E-05 |
| PD1 | 100×100×300 | high content fibre concrete | 1.0 | 0.003000 | 7.080 | 0.027 | 2.996E-05 |
| PD2 | 100×100×300 | high content fibre concrete | 1.0 | 0.003000 | 7.080 | 0.027 | 2.996E-05 |
| PD3 | 100×100×300 | high content fibre concrete | 1.0 | 0.003000 | 7.080 | 0.027 | 2.996E-05 |
| **total** | | | | **0.0360** | **84.960** | **0.142** | **0.0001573** |

**Figure C.2: Test Program for Modulus of Rupture Test**

| **Toughness or post crack ductility test** |
|---|

| specimen id | beam dimensions (mm) | specimen type | volume fraction of fibre (%) | volume of concrete (m3) | mass of concrete (kg) | mass of fibre (kg) | volume of fibre (m3) |
|---|---|---|---|---|---|---|---|
| BA1 | 150×200×2500 | plain concrete | 0 | 0.075000 | 177.000 | 0.000 | 0 |
| BA2 | 150×200×2500 | plain concrete | 0 | 0.075000 | 177.000 | 0.000 | 0 |
| BA3 | 150×200×2500 | plain concrete | 0 | 0.075000 | 177.000 | 0.000 | 0 |
| BB1 | 150×200×2500 | low content fibre concrete | 0.25 | 0.075000 | 177.000 | 0.169 | 0.00019 |
| BB2 | 150×200×2500 | low content fibre concrete | 0.25 | 0.075000 | 177.000 | 0.169 | 0.00019 |
| BB3 | 150×200×2500 | low content fibre concrete | 0.25 | 0.075000 | 177.000 | 0.169 | 0.00019 |
| BC1 | 150×200×2500 | medium content fibre concrete | 0.5 | 0.075000 | 177.000 | 0.337 | 0.00037 |
| BC2 | 150×200×2500 | medium content fibre concrete | 0.5 | 0.075000 | 177.000 | 0.337 | 0.00037 |
| BC3 | 150×200×2500 | medium content fibre concrete | 0.5 | 0.075000 | 177.000 | 0.337 | 0.00037 |
| BD1 | 150×200×2500 | high content fibre concrete | 1.0 | 0.075000 | 177.000 | 0.674 | 0.00075 |
| BD2 | 150×200×2500 | high content fibre concrete | 1.0 | 0.075000 | 177.000 | 0.674 | 0.00075 |
| BD3 | 150×200×2500 | high content fibre concrete | 1.0 | 0.075000 | 177.000 | 0.674 | 0.00075 |
| **total** | | | | **0.9000** | **2124.000** | **3.539** | **0.00393** |

**Figure C.3: Test Program for Post Crack Flexural Toughness Test**

| **Water absorption test** | | | | | | | |
|---|---|---|---|---|---|---|---|
| **specimen id** | **cube dimensions (mm)** | **specimen type** | **volume fraction** | **volume of concrete (m3)** | **mass of concrete (kg)** | **mass of fibre (kg)** | **volume of fibre (m3)** |

| | | | of fibre (%) | | | | |
|---|---|---|---|---|---|---|---|
| CA1 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CA2 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CA3 | 150×150×150 | plain concrete | 0 | 0.003375 | 7.965 | 0.000 | 0 |
| CB1 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CB2 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CB3 | 150×150×150 | low content fibre concrete | 0.25 | 0.003375 | 7.965 | 0.008 | 8.4254E-06 |
| CC1 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CC2 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CC3 | 150×150×150 | medium content fibre concrete | 0.5 | 0.003375 | 7.965 | 0.015 | 1.6851E-05 |
| CD1 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| CD2 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| CD3 | 150×150×150 | high content fibre concrete | 1.0 | 0.003375 | 7.965 | 0.030 | 3.3702E-05 |
| **total** | | | | **0.0405** | **95.580** | **0.159** | **0.0001763** |

**Figure C.4: Test Program for Water Absorption Test**

| **Surface abrasion resistance test** | | | | | | | |
|---|---|---|---|---|---|---|---|
| **specimen id** | **cube dimensions (mm)** | **specimen type** | **volume fraction of fibre (%)** | **volume of concrete (m3)** | **mass of concrete (kg)** | **mass of fibre (kg)** | **volume of fibre (m3)** |
| A4 | 100×100×100 | plain concrete | 0 | 0.001 | 2.36 | 0.000 | 0 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| A5 | 100×100×100 | plain concrete | 0 | 0.001 | 2.36 | 0.000 | 0 |
| A6 | 100×100×100 | plain concrete | 0 | 0.001 | 2.36 | 0.000 | 0 |
| B4 | 100×100×100 | low content fibre concrete | 0.25 | 0.001 | 2.36 | 0.002 | 2.496E-06 |
| B5 | 100×100×100 | low content fibre concrete | 0.25 | 0.001 | 2.36 | 0.002 | 2.496E-06 |
| B6 | 100×100×100 | low content fibre concrete | 0.25 | 0.001 | 2.36 | 0.002 | 2.496E-06 |
| C4 | 100×100×100 | medium content fibre concrete | 0.5 | 0.001 | 2.36 | 0.004 | 4.993E-06 |
| C5 | 100×100×100 | medium content fibre concrete | 0.5 | 0.001 | 2.36 | 0.004 | 4.993E-06 |
| C6 | 100×100×100 | medium content fibre concrete | 0.5 | 0.001 | 2.36 | 0.004 | 4.993E-06 |
| D4 | 100×100×100 | high content fibre concrete | 1.0 | 0.001 | 2.36 | 0.009 | 9.986E-06 |
| D5 | 100×100×100 | high content fibre concrete | 1.0 | 0.001 | 2.36 | 0.009 | 9.986E-06 |
| D6 | 100×100×100 | high content fibre concrete | 1.0 | 0.001 | 2.36 | 0.009 | 9.986E-06 |
| **total** | | | | **0.012** | **28.320** | **0.047** | **5.242E-05** |

**Figure C.5: Test Program for Surface Abrasion Resistance Test Appendix D: Curves and the Corresponding Equations for Computation of Areas under Load-Deflection Curves**

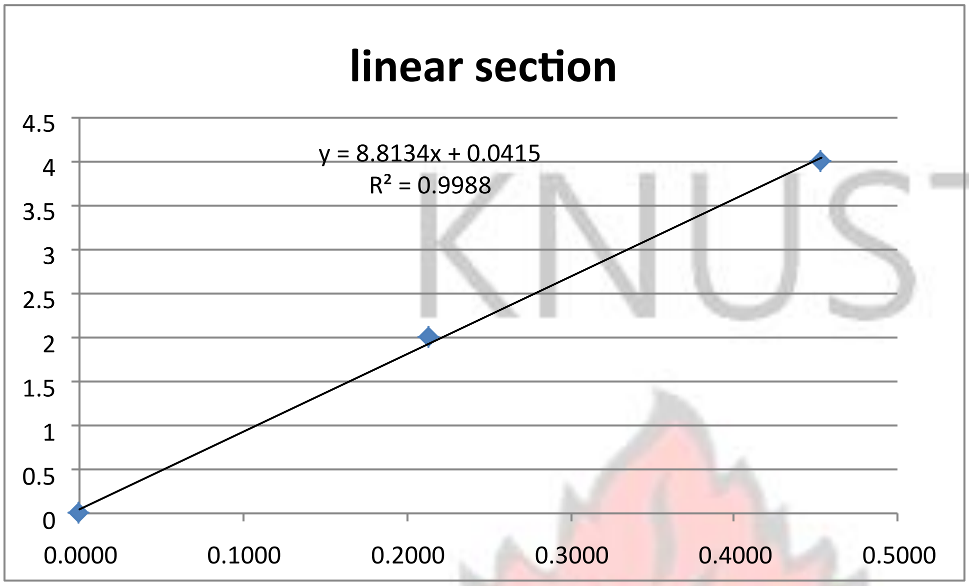


**Figure D.1: Linear Section of Load-Deflection Curve for Control Specimens**

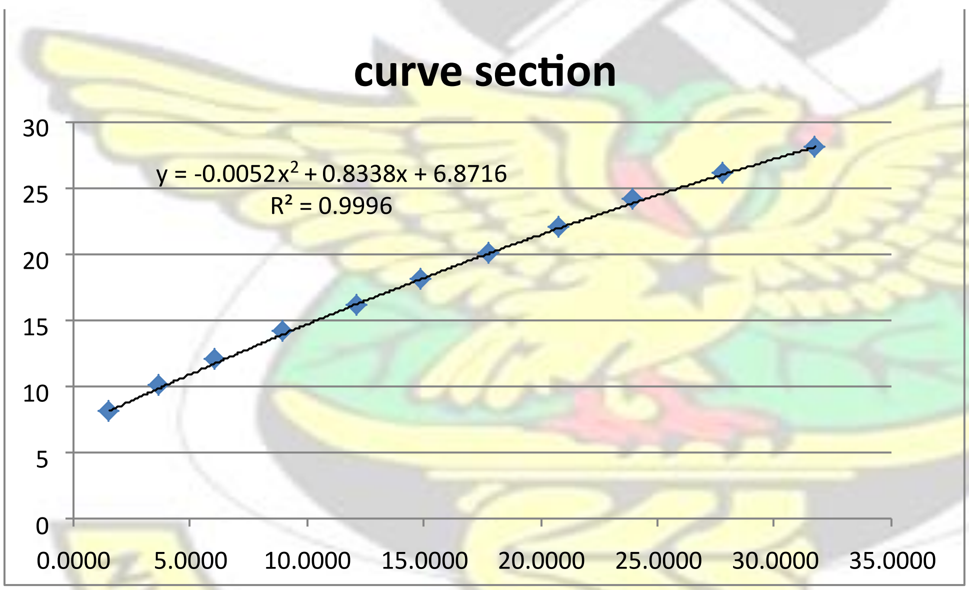


**Figure D.2: Curve Section of Load-Deflection Curve for Control Specimens**

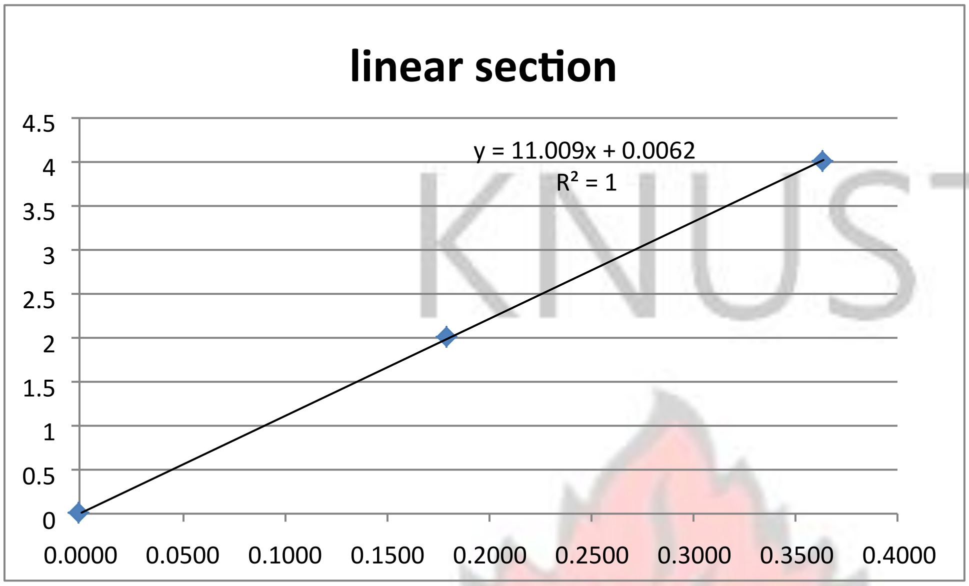


**Figure D.3: Linear Section of Load-Deflection Curve for 0.25% Fibre Concrete Specimens**

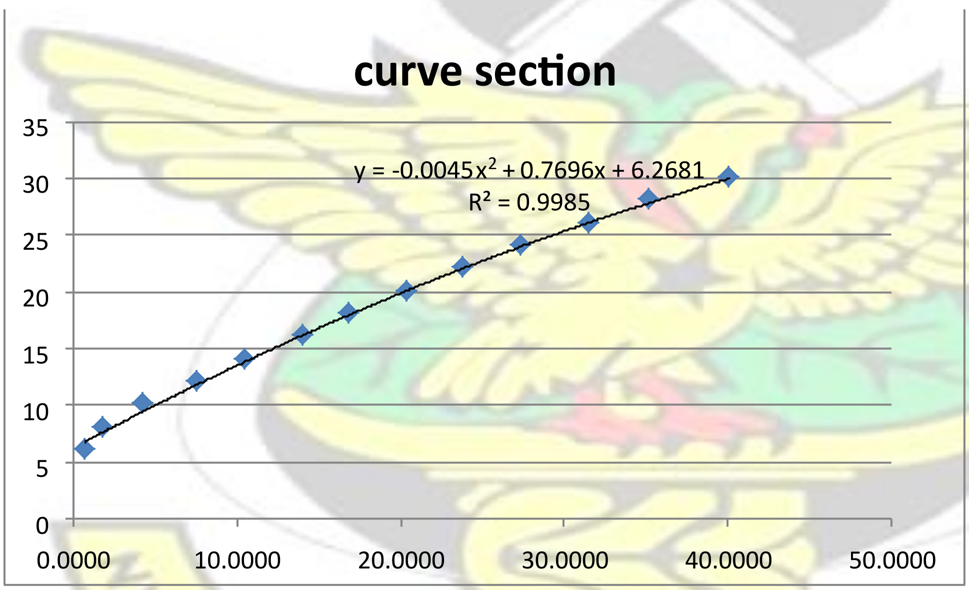


**Figure D.4: Curve Section of Load-Deflection Curve for 0.25% Fibre Concrete Specimen**

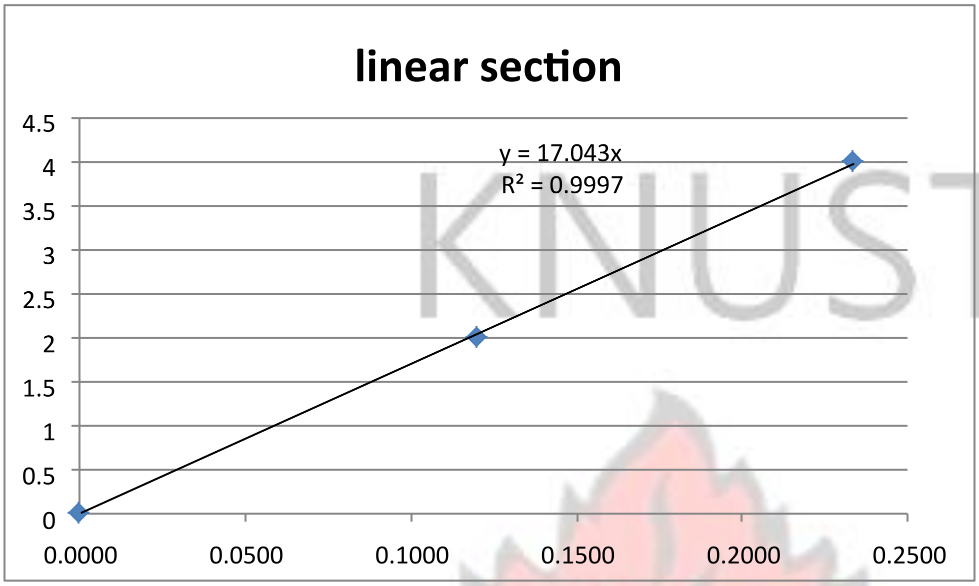


**Figure D.5: Linear Section of Load-Deflection Curve for 0.5% Fibre Concrete Specimens**

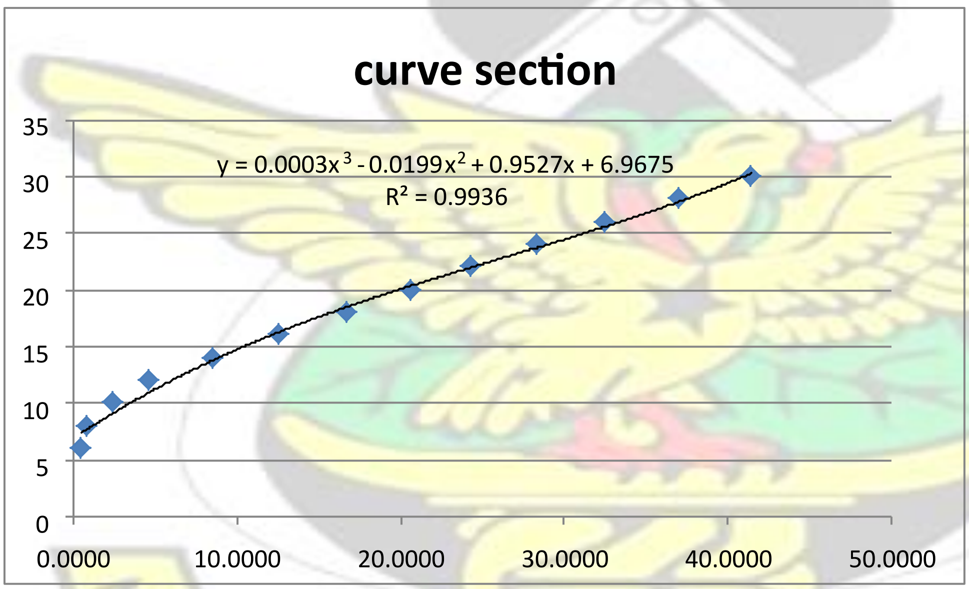


**Figure D.6: Curve Section of Load-Deflection Curve for 0.5% Fibre Concrete Specimens**

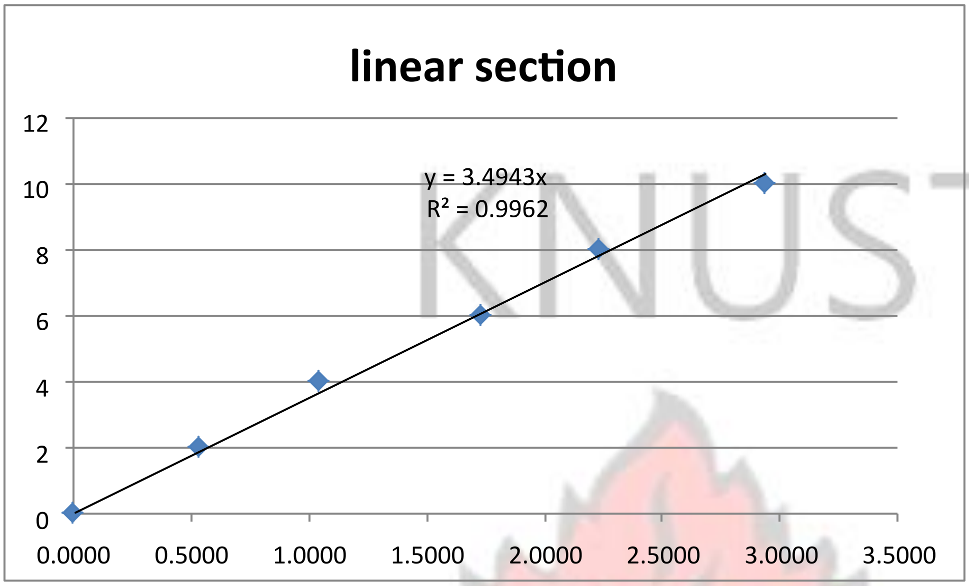


**Figure D.7: Linear Section of Load-Deflection Curve for 1.0% Fibre Concrete Specimens**

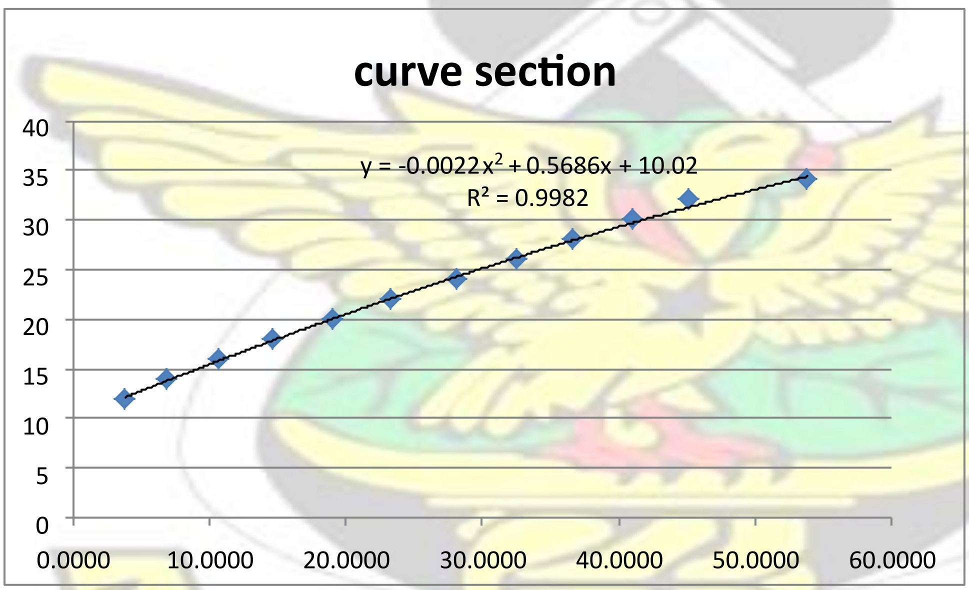


**Figure D.8: Curve Section of Load-Deflection Curve for 1.0% Fibre Concrete Specimens**

**Appendix E: Beam crack spacings**

| **BA1** | | **BA2** | | **BA3** | |
|---|---|---|---|---|---|
| Cracks | Spacing (mm) | Cracks | Spacing (mm) | Cracks | Spacing (mm) |
| 1 | | 1 | | 1 | |
| | 89 | | 196 | | 100 |
| 2 | | 2 | | 2 | |

| | | | | | |
|---|---|---|---|---|---|
| | 170 | | 100 | | 90 |
| 3 | | 3 | | 3 | |
| | 130 | | 130 | | 60 |
| 4 | | 4 | | 4 | |
| | 151 | | 150 | | 90 |
| 5 | | 5 | | 5 | |
| | 145 | | 160 | | 90 |
| 6 | | 6 | | 6 | |
| | 89 | | 180 | | 170 |
| 7 | | 7 | | 7 | |
| | 105 | | 120 | | 186 |
| 8 | | 8 | | 8 | |
| | 183 | | 110 | | 99 |
| 9 | | 9 | | 9 | |
| | 118 | | 110 | | 120 |
| 10 | | 10 | | 10 | |
| | 175 | Total number of cracks = 10 | | | 60 |
| 11 | | | | 11 | |
| | 120 | | | | 120 |
| 12 | | | | 12 | |
| Total number of cracks = 12 | | | | | 136 |
| | | | | 13 | |
| | | | | | 150 |
| | | | | 14 | |
| | | | | Total number of cracks = 14 | |

| | | | |
|---|---|---|---|
| Average | 134.09 | 139.56 | 113.15 |
| Spacing(mm) | | | 128.93 |

**Figure E.1: Crack spacing for control beams**

| BB1 | | BB2 | | BB3 | |
|---|---|---|---|---|---|
| Cracks | Spacing (mm) | Cracks | Spacing (mm) | Cracks | Spacing (mm) |
| 1 | | 1 | | 1 | |
| | 125 | | 99 | | 150 |
| 2 | | 2 | | 2 | |
| | 190 | | 130 | | 140 |

| | | | | | |
|---|---|---|---|---|---|
| 3 | 140 | 3 | 85 | 3 | 100 |
| 4 | 105 | 4 | 100 | 4 | 90 |
| 5 | 85 | 5 | 152 | 5 | 150 |
| 6 | 135 | 6 | 146 | 6 | 110 |
| 7 | 109 | 7 | 155 | 7 | 100 |
| 8 | 102 | 8 | 163 | 8 | 100 |
| 9 | 120 | 9 | 90 | 9 | 190 |
| 10 | 140 | 10 | 110 | 10 | 170 |
| 11 | 60 | 11 | 160 | 11 | |
| | | | | Total number of cracks = 11 | |
| 12 | 155 | 12 | | | |
| | | Total number of cracks = 12 | | | |
| 13 | 170 | | | | |
| 14 | | | | | |
| Total number of cracks = 14 | | | | | |
| Average Spacing(mm) | 125.85 | | 126.36 | | 130.00<br>127.40 |

**Figure E.2: Crack spacing for 0.25% fibre reinforced concretes**

| BC1 | | BC2 | | BC3 | |
|---|---|---|---|---|---|
| Cracks | Spacing (mm) | Cracks | Spacing (mm) | Cracks | Spacing (mm) |
| 1 | 110 | 1 | 175 | 1 | 170 |
| 2 | 130 | 2 | 90 | 2 | 160 |

| | | | | | |
|---|---|---|---|---|---|
| 3 | 90 | 3 | 115 | 3 | 140 |
| 4 | 160 | 4 | 73 | 4 | 100 |
| 5 | 99 | 5 | 160 | 5 | 50 |
| 6 | 130 | 6 | 120 | 6 | 90 |
| 7 | 110 | 7 | 35 | 7 | 136 |
| 8 | 45 | 8 | 130 | 8 | 140 |
| 9 | 135 | 9 | 175 | 9 | 91 |
| 10 | 165 | 10 | 100 | 10 | 112 |
| 11 | 100 | 11 | 140 | 11 | 100 |
| 12 | 157 | 12 | | 12 | 155 |
| 13 | | Total number of cracks = 12 | | 13 | |
| Total number of cracks = 13 | | | | Total number of cracks = 13 | |
| Average Spacing(mm) | 119.25 | | 119.36 | | 120.33 |
| | | | | | 119.65 |

**Figure E.3: Crack spacing for 0.5% fibre reinforced concrete specimen**

| BD1 | | BD2 | | BD3 | |
|---|---|---|---|---|---|
| Cracks | Spacing (mm) | Cracks | Spacing (mm) | Cracks | Spacing (mm) |
| 1 | 139 | 1 | 120 | 1 | 150 |
| 2 | 119 | 2 | 110 | 2 | 135 |
| 3 | 90 | 3 | 110 | 3 | 52 |

| | | | | | |
|---|---|---|---|---|---|
| 4 | 80 | 4 | 120 | 4 | 170 |
| 5 | 70 | 5 | 130 | 5 | 112 |
| 6 | 160 | 6 | 120 | 6 | 90 |
| 7 | 115 | 7 | 30 | 7 | 100 |
| 8 | 165 | 8 | 136 | 8 | 140 |
| 9 | 110 | 9 | 120 | 9 | 130 |
| 10 | 160 | 10 | 170 | 10 | 125 |
| 11 | 152 | 11 | 160 | 11 | 90 |
| 12 | | 12 | 90 | 12 | 115 |
| Total number of cracks = 12 | | 13 | | 13 | 100 |
| | | Total number of cracks = 13 | | 14 | 112 |
| | | | | 15 | |
| | | | | Total number of cracks = 15 | |
| Average Spacing(mm) | 123.64 | | 118.00 | | 115.79 |
| | | | | | 119.14 |

**Figure E.4: Crack spacing for 1.0% fibre reinforced specimen**