PROPERTIES OF STEEL FIBER MORTAR AND CONCRETE

BY

Amartey, Yusuf Dada
PH.D. (CIVIL ENGINEERING) A.B.U.
(PhD/Eng./25591/2000-2001)

A thesis submitted to the Postgraduate School, Ahmadu Bello University, Zaria, in fulfillment of the requirement for the award of a degree of Doctor of Philosophy in Civil Engineering.

Department of Civil Engineering
Ahmadu Bello University,
Samaru - Zaria.

August, 2008

DECLARATION

I, Yusuf Dada Amartey, hereby declare that this thesis contains the report of my research works and has not been presented in this form in any previous application for the award of a higher degree. All relevant information from other sources has been duly acknowledged by means of references.

---------- ----------

Yusuf Dada AMARTEY Date

CERTIFICATION

This thesis titled “Properties of Steel Fibre Mortar and Concrete” by Amartey, Yusuf Dada, meets the regulations governing the award of the Degree of the Doctor of Philosophy(Civil Engineering) of Ahmadu Bello University, Zaria, Nigeria; and is approved for its contribution to knowledge and literary presentation.

---------- ----------
Engr. (Dr) S.P. Ejeh Date
Chairman-Supervisory Committee.

---------- ----------
Prof. K.J. Osinubi Date
Member Supervisory Committee

---------- ----------
Dr. I. Abubakar Date
Member Supervisory Committee

---------- ----------
Engr. (Dr) S.P. Ejeh Date
Head of Civil Engineering

---------- ----------
Prof. S. K. Nkom Date
Dean, Postgraduate School

DEDICATION

This Thesis is dedicated to my wife, Maryam Amartey,

And

My children; Muhammad Amartey, Hajara Amartey, Rukkayya Amartey, And

Musa Amartey,

For their love, patience, and understanding.

ACKNOWLEDGEMENT

Praise be to ALLAH, the Elevated, for the success of this thesis and ask for bountifully reward all those who have undertaken or participated in it no matter how subtle a way.

My heartfelt gratitude goes to my supervisor, Dr. Stephen Pinder Ejeh, for his patience, concern, encouragement, advice, and guidance throughout this research period, his Fatherly role has been of great inspiration to me.

Special thanks to Prof. K.J Osinubi and Dr. I. Abubakar, members of my supervisory committee. I sincerely appreciate all the advice and support you provided.

Many thanks to my loving and wonderful Dad (BABA), for the encouragement and the never wavering support through this life.

Brother; Bimbo, I appreciate your support and encouragement.

To all my friends (so numerous to mention) and members of Department of Civil Engineering , Ahmadu Bello University, Zaria, for the various roles they played in this work, I thank you all.

Most of all, to all my teachers, I sincerely appreciate you all for showing me the excitement of learning.

I shall like to acknowledge the ABU MacArthur Foundation Project Grant for the Dissertaion Completio Grant.

Finally,

May we all share in ALLAH'S blessings. Ameen.

ABSTRACT

Steel fibre mortar and concrete are composite materials made with introduction of steel fibres into cement-based materials within certain percentage of fibre. Steel fibre mortar and concrete had improved properties when compared to plain mortar or concrete. In this work; three types of fibres namely - Circular Steel fibres (CSF), Rectangular Steel Fibres (RSF) and Steel Shaving Steel Fibres (CHSF) were investigated as composite materials. The following percentage were used in the mix - one – half percent, one percent, one and half percent and two percentage volume dosage rate of each steel fibre with a control mix (without fibre). Various tests like slump, compacting factor, flexural strength, compressive strength and beam deflections were performed on the samples produced to determine the mechanical properties of these composites.

It was observed that, one and half percentage of fibre in concrete is a critical percentage, the compressive strength, flexural strength were improved in mortar specimens and the compressive, tensile and flexural strength were also improved in concrete for Circular Steel fibres (CSF) and Rectangular Steel Fibres (RSF) while Steel Shaving Steel Fibres (CHSF) had a decrease as the fibre volume increased above one and half percentage. Workability, (slump and compacting factor) decrease with an increase in steel fibre percentage.

Relationships were also established between compressive strength of mortar and concrete and spit tensile and flexural strength of steel fibre mortar and concrete specimens.

TABLE OF CONTENTS

	Contents	Pages
	Title page	i
	Declaration	ii
	Certification	iii
	Dedication	iv
	Acknowledgement	v
	Abstract	vi
	Table of Contents	vii
	List of Figures	xi
	List of Tables	xiii
	List of Plates	
	Chapter One: Introduction	1
1.1	Preamble	1
1.2	Research Aim and Objectives	2
1.3	Scope and Methodology	2
1.4	Research Limitation	3
1.5	Research Outcomes	3

Chapter Two: Literature Review 4

2.1 Types of Fibres 5

2.1.1 Steel Fibres 8

2.1.2 Shape and Geometry of Steel Fibres 8

2.1.3 Durability of Steel Fibres 9

2.2 Glass Fibres 10

2.3 Synthetic Fibres 10

2.4 Other Types of Fibres 11

2.4.1 Asbestos Fibres 11

2.4.2 Natural Fibres 12

2.4.3 Carbon Fibres 12

2.5 Advantages and Limitations of Fibre Reinforced Concrete (FRC) 13

2.6 Field Performance of Fibre Reinforced Concrete (FRC) 16

2.7 Historical Development in Fibre Reinforced Concrete (FRC) 17

2.8 Previous Investigation in to Fibre Reinforcement 19

2.8.1 Fibre Effects and Parameters on the Behavious of FRC 19

2.8.2 Different types of Fibres in Fibre Reinforced Concrete 23

2.8.3 Usage of Fibres with Conventional Steel Reinforcement 24

2.8.4 Other Applications and Test Methods on FRC 26

2.8.5 Guides and Practice of Fibre Reinforced Concrete 29

2.9 Shape, Geometry and Distribution of Fibre Reinforced Concrete 31

2.10 Interaction between Fibres and Concrete Matrix 33

2.11 Critical Fibre Volume Dosage 37

2.12 Efficiency of Fibre Reinforcement 39

2.12.1	Length Efficiency	40
2.12.1	Fibre Orientation	43
2.13	Prediction of the Behaviours and Properties of FRC	45
	Chapter Three: Experimentations	51
3.1	Preamble	51
3.2	Materials	51
3.2.1	Fine Aggregate (Sand)	52
3.2.2	Coarse Aggregate/ Stones	52
3.2.3	Cement	53
3.2.4	Water	54
3.2.5	Fibres	54
3.3	Steel Fibre Mortar/Concrete Tests	56
3.3.1	Steel Fibre Mortar Cube Tests	57
3.3.2	Steel Fibre Mortar Beam Flexural Tests	59
3.3.3	Workability Test	62
3.4	Steel Fibre Concrete Cube Test	65
3.5	Steel Fibre Concrete Tensile Strength	67
3.6	Steel Fibre Concrete Flexural Test	69
3.7	Steel Fibre Concrete Flexural Deflection	71
3.8	Chips Steel Fibre Concrete Cubes Confirmation Test	73
	Chapter Four	75
4.0	Analysis and Discussion of Results	75
4.1	Sand	75
4.2	Coarse Aggregates	76

4.3	Cement	77
4.4	Water	78
4.5	Fibres	78
4.6	Steel Fibre Mortar	78
4.6.1	Steel Fibre Mortar Cube	79
4.6.2	Steel Fibre Mortar Beam Flexural Strength	84
4.7	Workability of Steel Fibre Concrete	88
4.7.1	Slump Test	88
4.7.2	Compacting Factor Test	91
4.8	Steel Fibre Concrete Cube	95
4.9	Steel Fibre Concrete Cylinder Split Tests [Tensile]	98
4.10	Steel Fibre Concrete Beam Flexural Strength	102
4.11	Load/Deflection Response	106
4.12	Chips Steel Fibre Concrete Cubes Tests	113
4.13	Prediction Model for Strengths of Steel Fibre Composites.	116
4.14	Relationships between Compressive, Tensile and Flexural Steel Fibre Mortar and Concrete	125
4.15	Toughness of Steel Fibre Concrete	132
	Chapter Five: Conclusions and Recommendations	134
5.1	Preambles	134
5.2	Conclusions	134
5.3	Recommendations	137
	References	138
	Appendices	147

LIST OF FIGURES

Figure 2.1: Classification of Fibres 7

Figure 2.2: Different types and Geometry of Steel Fibres. 9

Figure 2.3: Comparison of cracks with and without Fibre reinforced. 15

Figure 3.4: Classification of fibre arrangement in 1, 2 and 3 dimensional. 33

Figure 2.5: Pullout geometry to simulate the interaction between Fibres and cement. 34

Figure 2.6: Interaction of fibre-uncracked matrix. 36

Figure 2.7: Interaction of fibre-cracked matrix. 37

Figure 2.8: Definition of critical length: 41

Figure 2.9: The intersection of an oriented fibre across a crack with an angle. 44

Figure 2.10: The multiple cracking process and the stages of stress/strain Curve related to the multiple cracking processes. 46

Figure 2.11: Final schematic description of stress/strain curve. 47

Figure 2.12: The parallel model of 'rule of mixture'. 49

Figure 4.1: Particle Size Distribution (Fine Aggregate) 75

Figure 4.2: Particle size distribution (Coarse Aggregate) 76

Figure 4.3: Compressive Strength of Mortar Cubes Vs Fibre Volume Dosage 80

Figure 4.4: Percentage Increase of compressive strength over to control 82

Figure 4.5: Compressive Strength Increase over Control 83

Figure 4.6: Flexural Strength against Fibre volume Dosage 85

Figure 4.7: Flexural Strength Increase over Control 87

Figure 4.8: Percentage Increase of Flexural strength over to control mix 87

Figure 4.9: Average Slump Height vs. Fibre Dosage for all Fibres 89

Figure 4.10: Percentage Difference of Slump Height vs. Fibre Dosage 91

Figure 4.11: Average compacting factor Vs. Fibre dosage for all fibres 93

Figure 4.12: Percentage Difference of Compacting Factor vs. Fibre Volume. 95

Figure 4.13: Average Compressive Strength vs. Fibre Volume Dosage 96

Figure 4.14: Average Tensile Strength vs. Fibre Volume Dosage 99

Figure 4.15: Percentage difference of tensile strength vs. fibre volume 100

Figure 4.16: Tensile Strength Increase over Control 101

Figure 4.17: Flexural Strength of concrete against Fibre volume Dosage 103

Figure 4.18: Percentage Increase of Flexural strength over control mix 105

Figure 4.19: Flexural Strength Increase over Control 105

Figure 4.20: Load/Deflection Curve for CSF at Different Dosage 108

Figure 4.21: Load/Deflection Curve for RSF at Different Dosage 108

Figure 4.22: Load/Deflection Curve for CHSF at Different Dosage 109

Figure 4.23: Load vs. displacement graph at 0.5% volume dosage 110

Figure 4.24: Load vs. displacement graph at 1.0% volume dosage 111

Figure 4.25: Load vs. displacement graph at 1.5% volume dosages 111

Figure 4.26: Load vs. displacement graph at 2.0%volume dosage 112

Figure 4.27: Compressive strength of chips steel fibre 115

Figure 4.28: Experimental and Predicted – 28 days Compressive Strength 121

Figure 4.29: Experimental and Predicted – 28 days Flexural Strength 122

Figure 4.30: Experimental and Predicted – 28 days Compressive Strength 123

Figure 4.31: Experimental and Predicted – 28 days Tensile Strength 124

Figure 4.32: Experimental and Predicted – 28 days Compressive Strength 125

LIST OF TABLES

Table 2.1: Typical Properties of Some Fibres 6

Table 2.2: Orientation Efficiency Factor for Unconstrained and Constrained FRC. 45

Table 3.1: Sieve Analysis Result for Fine Aggregate 52

Table 3.2: Sieve Analysis Result for Coarse Aggregate 53

Table 3.3: Consistency Test on Dangote (OPC) 54

Table 3.4: Compressive Strengths Test Results – CSF (Mortar Cubes) 58

Table 3.5: Compressive Strengths Test Results – RSF (Mortar Cubes) 58

Table 3.6: Compressive Strengths Test Results – CHSF (Mortar Cubes) 59

Table 3.7: Flexural Strengths Test Results - CSF (Mortar Beams) 61

Table 3.8: Flexural Strengths Test Results- RSF (Mortar Beams) 61

Table 3.9: Flexural Strengths Test Results - CHSF (Mortar Beams) 62

Table 3.10: Slump Test Results – CSF (Concrete Mix) 63

Table 3.11: Slump Test Results - RSF (Concrete Mix) 63

Table 3.12: Slump Test Result – CHSF (Concrete Mix) 64

Table 3.13: Compacting Factor Results – CSF (Concrete Mix) 64

Table 3.14: Compacting Factor Results – RSF (Concrete Mix) 65

Table 3.15: Compacting Factor Results – CHSF (Concrete Mix) 65

Table 3.16: Compressive Strengths Test Results– CSF (Mortar Cubes) 66

Table 3.17: Compressive Strengths Test Results – RSF (Concrete Cubes) 66

Table 3.18 Compressive Strengths Test Results – CHSF (Concrete Cubes) 67

Table 3.19: Tensile Strengths Test Results – CSF (Mortar Cubes) 68

Table 3.20: Tensile Strengths Test Results -RSF (Concrete Cubes) 68

Table 3.21: Tensile Strengths Test Results – CHSF (Concrete Cubes) 69

Table 3.22: Flexural Strengths Test Results -CSF (Concrete Beams) 70

Table 3.23: Flexural Strengths Test Results- RSF (Concrete Beams) 70

Table 3.24: Flexural Strengths Test Results - CHSF (Concrete Beams) 71

Table 3.25: Flexural Deflection Results -CSF (Concrete Mix) 72

Table 3.26: Flexural Deflection Results RSF (Concrete Mix) 72

Table 3.27: Flexural Deflection Results CHSF (Concrete Mix) 73

Table 3.28: Confirmation of Chips Steel Fibre Compressive Strength 74

Table 4.1: Comparison of the Properties of Dangote (OPC) with the Requirements of BS 12 (1986) 77

Table 4.2: Compressive Strength of Mortar Cubes at Different Fibre Dosage 79

Table 4.3: Fibre Mortar Cubes Compressive Strengths Increase over Control 81

Table 4.4: Comparism of Flexural Strength of Fibre Mortar Beam 84

Table 4.5: Increase in Flexural Strength at Different Fibre Volume Dosage 86

Table 4.6: Comparism of Slump Test Results for Fibre Concrete 88

Table 4.7: Percentage Decrease in Slump of Steel Fibre Concrete 90

Table 4.8: Comparism of Compacting Factor Test Results for Fibre Concrete 92

Table 4.9: Compacting Factor Decrease from Control Value 94

Table 4.10: Compressive Strength of Concrete Cubes at Different Fibre Dosage 96

Table 4.11: Compressive Strengths Increase over Control- (Concrete Cubes) 97

Table 4.12: Tensile Strength of Concrete Cylinder at Different Fibre Dosage 98

Table 4.13: Increase in Tensile Strength at Different Fibre Volume Dosage 100

Table 4.14: Comparism of Flexural Strength of Fibre Concrete Beam 103

Table 4.15: Increase in Flexural Strength at Different Fibre Volume Dosage 104

Table 4.16: Load / Deflection Results for the Three Fibres 107

Table 4.17: Compressive Strength of Chips Steel Fibre Concrete Cubes 114

Table 4.18: Coefficients for Mortar Cubes 117

Table 4.19: Coefficients for Mortar Beams 117

Table 4.20: Coefficients for Concrete Cubes 117

Table 4.21: Coefficients for Concrete Cylinders 118

Table 4.22: Coefficients for Concrete Beams 118

Table 4.23: Compressive Strengths of Mortar Cubes 120

Table 4.24: Flexural Strength of Mortar Beams 121

Table 4.25: Compressive Strength of Concrete Cubes 122

Table 4.26: Tensile Strength of Concrete Cylinder at Different Fibre Dosage 123

Table 4.27: Flexural Strength of Concrete Beams at Different Fibre Dosage 124

Table 4.28: Experimental and Estimated Values of Mortar Beam Flexural Strength – CSF 127

Table 4.29: Experimental and Estimated Values of Mortar Beam Flexural Strength – RSF 127

Table 4.30: Experimental and Estimated Values of Mortar Beam Flexural Strength – CHSF 128

Table 4.31: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – CSF 128

Table 4.32: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – RSF 129

Table 4.33: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – CHSF 129

Table 4.34: Experimental and Estimated Values of Concrete Beam Flexural Strength – CSF 130

Table 4.35: Experimental and Estimated Values of Concrete Beam Flexural Strength – RSF 130
Table 4.36: Experimental and Estimated Values of Concrete Beam Flexural Strength – CHSF 131
Table 4.37: Toughness Index for Steel Fibre Concrete Beam – CSF 132
Table 4.38: Toughness Index for Steel Fibre Concrete Beam – RSF 132
Table 4.39: Toughness Index for Steel Fibre Concrete Beam – CHSF 133

List of Plates

Plate 3.1: The three types of waste fibres used in this work 55
Plate 3.2: Test Rig with Beam specimen positioned for test 60
Plate 4.1: Photograph of concrete specimens at failure showing specimens with and without fibres 102

0

CHAPTER ONE

INTRODUCTION

1.1 Preamble

Concrete is acknowledged to be a relatively brittle material when subjected to normal stresses and impact loads, where tensile strength is only approximately one tenth of its compressive strength Neville, (1997). As a result of these characteristics, plain concrete members cannot support such loads and stresses that are usually imposed on structural elements. Historically, concrete members are reinforced with continuous reinforcing bars to withstand tensile stresses and compensate for the lack of ductility and tensile strength. Steel reinforcement was adopted to overcome high potentially tensile stresses and shear stresses at critical location in concrete members.

Steel fibre mortar or concrete is either mortar or concrete where some percentages of steel fibres are introduced into the mortar or concrete. Steel fibre mortar or concrete in general has specialized properties that enhances resistance to impact, abrasives, improves brittleness, good resistance to vibration loads and has high durability Lees (2001) and Ghagal (2003).

In the early days of fibre concrete (FC), it was only used for pavement and industrial slabs. But recently, applications of fibre-reinforced concrete have wide variety of usage in structures such as heavy-duty pavement, airplane runways, industrial slabs, etc.

In this work, the properties of steel fibre mortar and concrete are the major point of investigation.

1.2 Research Aim and Objectives:

The aim of this work is to carry out a study on steel fibres mortar and concrete and that would provide the needed improvement in the mechanical properties of steel fibre mortar and concrete. This will aid in a better understanding of the properties of steel fibres mortar or concrete and would enable one make use of the steel fibre mortar or concrete in structures.

The objectives and scope will include:

(a) Conduct a comprehensive literature review in order to determine the current state of the art regarding steel fibre reinforced concrete.

(b) Sourcing and processing of steel fibre.

(c) Evaluation of the strength properties of cubes compressive strength, beams flexural strength and cylinders split tensile strength.

(d) Development of model equations for the prediction of the strength properties of steel fibre mortar and concrete composite.

1.3 Scope and Methodology

The work in this thesis covers the following properties of steel fibre mortar and concrete

- Workability
- Compressive strength of mortar and concrete
- Flexural strength of mortar and concrete
- Flexural deflection of concrete beams
- Tensile strength of concrete

The methodology involves intensive literature review followed by an experimental set up in accordance with codes and standards to determine the above properties, analysis of the obtained results to arrive at a reasonable conclusion.

1.4 Research Limitation

The work covers only three types of fibres like Circular steel fibre (CSF), Rectangular steel fibre (RSF) obtained from burnt tyres and chips steel fibres (CHSF) which is a waste from Armaco Steel Company, Kaduna, in Kaduna State. It does not touch the aspect of polymer fibres, natural fibres and any other synthetics fibres.

1.5 Research Outcomes

The results shows that there is increase in compressive strength as the fibre percentage increases up to a critical percentage of one and half percentage. There is a decrease in workability of concrete as the steel fibre increases and good improvement on flexural and tensile strengths of steel fibre mortar and concrete.

CHAPTER TWO

LITERATURE REVIEW

Historically, fibres have been used to reinforce brittle materials since ancient times; straws were used to reinforce plaster and asbestos fibres have also been used to reinforce Portland cement, Shah and Rangan (1970). Patents was granted since the turn of the 19th century for the various methods of incorporating wire segments or metal chips into concrete, Romualdi and Batson (1963). The low tensile strength and brittle character of concrete have been by-passed by the use of reinforcing rods in the tensile zone of the concrete since the middle of the nineteenth century. The reinforcing rods would carry the tension, and the metal's ductile nature could be utilised in making the composite more ductile.

Around 1920, Griffith's observations on rupture and flow in solids gave insight to the understanding of brittle matrix fracture. Griffith observed that the actual load-carrying capacity of an isotropic material was many times less than the theoretical strength it should have, based on its molecules' bond strengths. Griffith postulated this weakness was due to the presence of inherent discontinuities or flaws in the material.

Romualdi and Batson (1963) applied Griffith's postulation on brittle fracture of concrete. They hypothesized that the addition of closely spaced wire reinforcement would increase the strength of concrete by "arresting" any crack growth, thereby preventing several cracks from coalescing into a failure plane.

2.1 Types of Fibres

The pioneering work of Romualdi and Mandel (1964) on random fibres has been followed with considerable enthusiasm in exploring the potentials of various types of fibres in a low-modulus matrix, and these has resulted in a good understanding of the fibre composite behaviour. Figure 2.1 shows the broad classification of fibres. Most of the fibres used for construction purposes are found under the natural fibres. However, because of the cheap and availability of some vinyl polymers and waste like fibres, many of these are also been used in civil engineering applications, Table 2.1.

Fibres vary in types, geometry, properties and availability in construction industry. Most common types of fibres are steel fibres, glass fibres, and polypropylene fibres. Their usages may alter in concrete for different applications and the applications mostly depend and are adopted on properties, effectiveness, cost and availability. Special types of fibres such as carbon, and kevlar, natural fibres, mineral fibres, and asbestos fibres may be used in harsh environment. These differences and usage of fibres depend on the requirements, behaviour and properties for a concrete, allowing the increase in the explicit effects and mechanical properties. Fibre geometry varies from hooked end fibres, deformed fibres, deformed wires, fibre mesh, wave-cut fibres, large end fibres with different types and geometries.

Table 2.1: Typical Properties of Some Fibres (ACI , 1993).

Type of fibre Specific	Tensile Strength N/mm^2	Young's Modulus 10^3 N/mm^2	Elongation Percent	Ultimate Gravity
Acrylic	207-414	2.07	25-45	1.10
Asbestos	552-966	82.8-138	≈0.6	3.20
Cotton	414-690	4.83	3-10	1.50
Glass	1035-3795	69	1.5-3.5	2.50
Nylon (high tenacity)	759-828	4.14	16-20	1.10
Polyester (high tenacity)	724.5-862.5	8.28	11.13	1.40
Polyethylene	690	0.14-0.41	10	0.95
Polypropylene	552-759	3.45	25	0.90
Rayon (high tenacity)	414-621	6.90	10-25	1.50
Rock Wool (Scandinavian)	483-759	69-117.3	0.6	2.70
Steel	276-2760	200.10	0.5-35	7.80

2.1.1 Steel Fibres

Steel fibres are widely used in civil engineering applications and concrete reinforcement, due to its relative availability, reasonable cost and better experience in its application with conventional steel reinforcement. Bentur and Mindness (1990) stated that the early research and studies on fibre reinforced concrete in 1950's to 1960's mainly were on the behaviour of steel fibre reinforced concrete. Steel fibres greatly increase toughness of concrete, which primarily is used for crack and shrinkage controls, to serve as secondary reinforcement for pavements, slabs, pipes, channel and tunnels, Elvery and Samarai, (1975). Its potential improvements are to increase toughness, minimize cracking due to temperature changes and increase resistance due to extreme loading and environmental effects such as impact, abrasion, blasting and fatigue. Furthermore, steel fibre reinforced concrete greatly reduces the potential for fractures and spalling.

2.1.2 Shape and Geometry of Steel Fibres

Cross sectional dimensions of typical steel fibre of range from 0.5mm to 1mm thick, 0.25mm to 0.90mm wide, with diameter range of 0.25mm to 0.75mm, where created in various form of geometry. Steel fibres were produced in steel sheet form, through the process of cutting steel sheets. Depending on the geometry desired, steel fibres are crimped and construct to deformed, end flat and enlarged end shapes. Using similar process, chopped drawn wire shape of steel fibres has been produced. Steel fibres with hooked and wave shapes have been produced and well-known in use for construction industry currently. These different geometries and shapes of steel fibres are widely used in industry to fulfill the desirable behaviour and properties requirement of concrete. Figure 2.2 below shows some deformed fibres that are available.

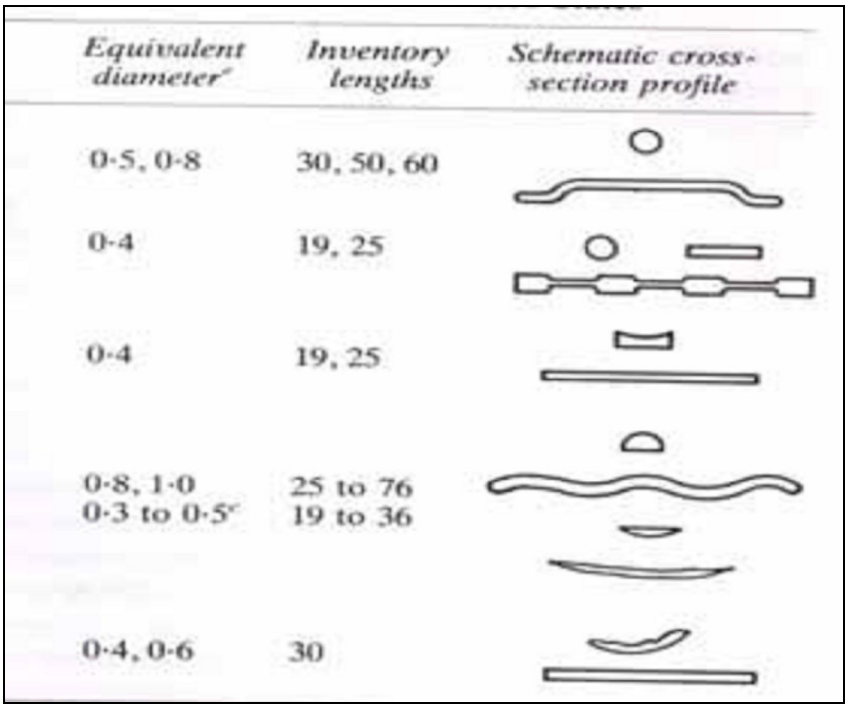


Figure 2.2: Different types and geometry of steel fibres.
(Source: Bentur & Mindess, 1990)

2.1.3 Durability of Steel Fibres

Steel fibre corrosion may be a major concern of durability of fibre reinforced mortar and concrete. Guidelines set in BS 8100 state that corrosion in conventional steel reinforcement could be avoided if suitable cover is provided. However, these guidelines are only applicable at particular position for conventional steel reinforcement. On fibre reinforced concrete, the steel fibres are randomly distributed throughout the matrix, as some corrosion can happen at the surface of the concrete, where it is very difficult for each fibre to be cover with the cement. However, with cement cover of more than 1mm, the fibres are safe from corrosion. Thus, corrosion of steel fibre is consider a minor problem, as it does not affect the mechanical properties of the fibre reinforced concrete.

2.2 Glass Fibres

Soviet research in late 1950's explored the low alkali properties of glass fibres in cement system, which had low value of pH. Until 1960's, glass fibres were classified as possible reinforcement to high pH value cement systems only, James, (1990). Glass fibre is a strong, lightweight material, which has tremendous fracture toughness, posses high tensile (280 to 3500N/mm^2) and modulus of elasticity (3.1 to 3.5 kN/m^2) in high alkaline cement systems.

2.3 Synthetic Fibres

In recent years, synthetic fibres have become more attractive for reinforcement of cement and concrete material. According to James (1990), Shell Chemical Company started the investigation on the use of polypropylene fibres in concrete around 1965. The developments of synthetic fibres were successfully utilized in bonding and reinforcement in cement matrix, James, (1990) and Zaher and Bayomy, (1999). Synthetic fibres have very high tensile strength, but these fibres can be differentiated into two categories, either by high or low modulus of elasticity, Synthetic industries concrete company, (2000). Most of the synthetic fibres in use fall in the categories of low modulus of elasticity, such as polypropylene, polyester, polyethylene, and nylon. The main advantages of these fibres are alkali resistance, high melting point (up to 165^{o}C) and low cost of the raw material. Disadvantages are poor fire resistance, poor bond with cement matrix and sensitive to sunlight and oxygen. Low modulus of elasticity synthetic fibres shows its usefulness in increasing the toughness and shrinkage cracking in concrete. However, they seem less applicable in increasing the flexural strength and ductility of concrete, Bentur et. al, (1990) and Lees and Burgoyne, (2000).

2.4 Other types of Fibres

The above three types of fibres are the most commonly used fibres in the industry and constructions today. There are more types of fibres in used, but their applications are limited. Such limitation of these fibres may be due to non availability in the current market, high costs of material or less effectiveness in the fibre reinforced composite. However, these fibres may have some advantages over steel, glass and synthetic fibres. These other types of fibres include: Asbestos fibres, Natural fibres, and Carbon fibres, etc.

2.4.1 Asbestos Fibres

Asbestos fibres are made of natural crystalline fibrous minerals. Asbestos/cement was the first fibre-reinforced composite in modern times, and are still in use more than any other fibre-reinforced materials.

Asbestos fibres relatively have high modulus of elasticity and strength, which permits effective dispersion of large fibre volume and enhance the bond with cement matrixes. These fibres are utilized with fibre-reinforced materials and are suitable in low cost housing and infrastructure.

2.4.2 Natural Fibres

These are the oldest form of fibre-reinforced composites, using fibres such as straw and horse hair in structures. Recently, with modern technology, natural fibres have been extracted economically from various vegetable and animal, such as jute, bamboo and wool. These fibres requires low amount of energy to extract. Relatively, they have limited use due to high water absorption and low tensile strength compared to steel and synthetic fibres.

Primary problem with these fibres are their tendency to fragment in an alkaline environment. Special treatments for this problem is by using admixtures to improve their durability and making concrete less alkaline, and allowing these fibres to increase its strength. These natural fibre-reinforced composites are commonly uses for thin sheet and cement products, as well as the application for cement cladding.

2.4.3 Carbon Fibres

Carbon fibres have limited use in cementitious material, because of its high cost in mid 1980's, but recently, low cost carbon fibres have been manufactured using petroleum and coal pitch. The two processes of making carbon fibres involve heat treatments of various grade of carbon in chemicals and these fibres find their applications as substitute to cement-based pipe and wood in structures. Carbon fibres have specialized applications in improving tensile and flexural strength. Typically, they have an elastic modulus as high as steel, yet they are very light. Its common uses are applications in sheeting and wrap as externally reinforced degrading concrete structures. Properties of carbon fibres composite which are greatly increased are strengths, chemical stability and stiffness. There are several precursors for production of carbon fibres. Carbon fibres are produced through controlled oxidation, orientation of graphitic crystallites, carbonization and stretching from carbon precursors. These precursors include polyacrylonitrile (PAN), cellulose fibres, pitch precursors, non-heterocyclic aromatic polymers, aromatic heterocyclic polymers, linear polymers and coal James, (1990).

2.5 Advantages and Limitations of Fibre Reinforced Concrete (FRC)

Fibres, which are randomly distributed throughout the concrete, can overcome cracks and control shrinkage more effectively. These materials have outstanding combinations of strength and energy absorption capacity. In general, the fibre reinforcement is not a substitution for conventional steel reinforcement. The fibres and steel reinforcement have their own role in concrete technology. Therefore, in many applications both fibres and continuous reinforcing steel bars can be use together.

However, fibres are not efficient in withstanding the tensile stresses compare to conventional steel reinforcement. But, fibres are more closely spaced than steel reinforcement, which is better in controlling crack and shrinkage. Consequently, conventional steel reinforcement are used to increase the load bearing capacity of concrete member; while fibres are more effective in crack control.

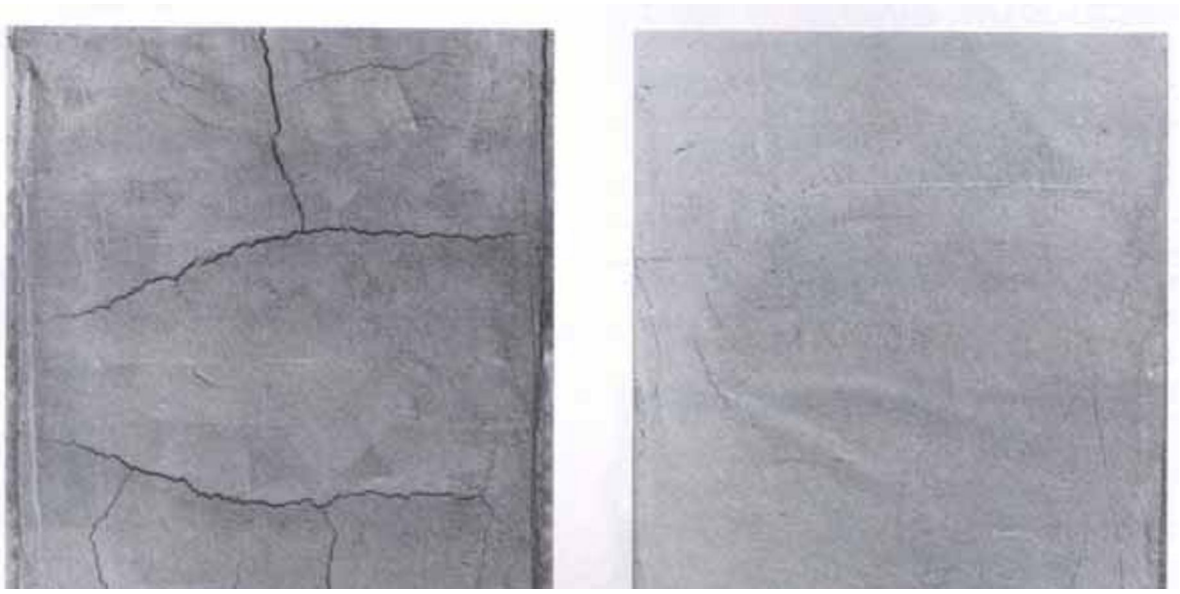
Due to these differences, there are particular applications that fibres reinforcement are advantageous than conventional steel reinforcement. These include:

- Fibres comprise as ‘primary reinforcement’, in which the conventional steel reinforcement cannot be utilized. The fibre concentrations are comparatively high in thin sheet materials, normally exceeding 5% by volume, acts to increase toughness and strength of mortar or concrete.
- Fibres can be components to withstand locally high loads or deformations, which are applies to structures like precast piles, precast walls, blast resistant structures or sewer tunnel and linings.
- Applications that control cracks persuaded by temperature and humidity, such as pavements and slabs, where fibres acts as ‘secondary reinforcement’.

The use of steel bars and wire mesh require unnecessary labor and material costs for structural concrete. With replacement of randomly distributed short fibres as an alternative reinforcement, this will significant reduce both labour and material costs, greatly reduce construction and project time.

Fibres substantially reduce formation of plastic shrinkage and settlement; enable the concrete to develop its full potential long-term strength in application to structural and providing solution to exceed and meet their performance and economical prospect.

Additionally, fibres provide an effective secondary reinforcement for shrinkage and crack width control. Macro-cracks and potential problems are prevented and blocked when micro-cracks intersect fibres as concrete hardens and shrink. Effects of crack control reinforcement by additional of fibres in concrete is shown in figure 2.3 below.



Without fibre-reinforced With fibre-reinforced

Figure 2.3: Comparison of cracks with and without Fibre reinforced.
(Source: Fibremesh, 1989)

Benefits of using fibre-reinforced concrete are:

- Increase impact and shatter resistance, fatigue endurance and shear strength of concrete.
- Requires no special equipments to install reinforcement.
- Increase crack resistance, long-term ductility, energy absorption capacity and toughness of concrete.
- Reduce labor and material costs in concrete applications.
- Provides multi-directional concrete reinforcement.
- Compatible with admixtures, all types of cement and concrete mixtures.
- Reduce plastic shrinkage and crack width formation.

Restrictions and limitations of using fibre-reinforced concrete are:

- Control crack as result of external stresses.
- Reduction in curling and creep.
- Justification for a reduction in the size of support columns.
- Higher structural strength development.
- Replacement of any moment for structural steel reinforcement.
- Decreasing the thickness of slab on grade.

Although short fibres cannot completely replace conventional steel reinforcement, they create supplementary reinforcement use to achieve increase in strength, higher ductility, greater shrinkage, crack control, fatigue, impact and abrasion resistance. However, development and advances in technologies has led to the discovery of more effects for fibres behaviour and mechanical properties of concrete.

2.6 Field Performance of Fibre Reinforced Concrete

Concrete is the most common material used in construction field Hoff, (1975). Fibre reinforced concrete used in slab and pavement applications, general performed well than plain concrete that has the same thickness, concrete flexural strength and foundation subgrade condition. The performance of fibre reinforced concrete in the construction industry is wide, which includes industrial development, light commercial structures, residential, precast, shotcrete and transportation field.

2.7 Historical Development in Fibre Reinforced Concrete

The concept of using fibres in a brittle matrix was first recorded with the ancient Egyptians who used hair from animals and straw from grass as reinforcement for mud bricks and walls in housing. This dates back to 1500 B.C. Balaguru and Shah, (1992). At the same time period, straws were used to reinforce sun-baked bricks for a 57 m high hill of 'Aqar Quf', which is located near Baghdad. It is not until the 1900's that asbestos fibres were developed, manufactured and widely used to augment mechanical properties of cement matrix as described by Bentur and Mindess (1990).

Balaguru and Shah (1992) reported that the modern developments of using only straight steel fibres began in the early 1960's. But now, wide ranges of other types of fibres are used in cement matrices. Construction industries in many countries have led the development of conventional fibres such as steel, stainless steel and glass; where new types of fibres such as kevlar and carbon; and several low modulus fibres, such as man made fibres (polypropylene, nylon) or natural fibres (jute, sisal, bamboo and wood pulp) are also coming up, they are varies in their properties, cost and effectiveness Table 2.1. They may be produce as bundled of filaments or fibrillated films, or may be used as mats or woven

fabrics Bentur et. al, (1990). Primarily, the fibres used in modern industries are discontinuous fibres. Development of concrete with modified polymer fibres systems increases the explicit effects and mechanical properties of concrete.

In the early stage of fibre development, steel and glass fibres with geometry of straight and smooth were used, as these fibres improve the ductility, flexural strength and fracture toughness of concrete matrix. The primary factors that controlled this composition were fibre volume fraction and length/diameter (aspect ratio). However, the problems faced were difficulty in mixing and workability. Balaguru and Shah (1992) reported that fibres that are long and at higher volume fractions were found to ball up during the mixing process. The process called 'balling' occurs, causes the concrete to become stiff and there is a reduction in workability with increase in volume dosage of fibres. This has a tendency to influence the quality of concrete and strength. For more than 40 years now, discovery and acceptance of reinforcement and fibres for enhancement of concrete properties has rapidly increased for use in concrete industries, research and development. Numerous types of fibres have successfully been adapted in different applications of concrete. Technological advancement has bought forward the development of fibres with different geometric shapes and properties to expand the benefits of fibres in concrete structures. New manufacturing techniques and applications on fibres for concrete have been developed. These introduced various aspects of fibre reinforced concrete and introduced them into the market worldwide.

All these fibres with more complicated geometric, shape and sizes have developed, mainly to modify each of their mechanical bonding with cement matrix. When fibre is

added to a concrete mix, each and every individual fibre receives a coating of cement paste. Modification of fibre geometry includes hooked end fibres, deformed fibres, deformed wires, fibre mesh, wave cut fibres, large end fibres. This increases bonding without increasing the length and minimize chemical interaction between fibres and the cement matrices. This also modifies and enhances the mechanical properties and behaviour of concrete in its applications.

Fibres can be use with admixtures such as superplasticizer, air entraining agents, set retarding, set-accelerating admixtures and all types of cement and concrete mixture, these produce special types of concrete with desired characteristics in fresh and hardened concrete, Newman, (1965). They increase workability, accelerate and retard rate of hydration of cements, and resistance to freeze and thaw conditions. They provided a significant improvement to the fibre-reinforced concrete used in the fields.

2.8 Previous Investigation in to Fibre Reinforcement

It has been known that fibre-reinforced concrete had been used in the early years of structural building. Between the years from 1960 to 2000, many researches have been carried out and quite a number of investigations have been performed on fibre-reinforced concrete. The use of randomly distributed fibre reinforcement can be considered to be a lucrative method of providing higher structural strengths to concrete structures, pavements and etc. However, stresses caused by shrinkage to concrete itself historically has been a problem to control because of their unpredictable and irregular occurrence.

2.8.1 Fibre Effects and Parameters on Behaviour of Fibre Reinforced Concrete

Fibre reinforced concrete was successfully used in a variety of engineering applications, because of its satisfactory and outstanding performance in the industry and construction field. However, most engineers and researchers do not fully understand how and why the fibres perform so successfully. So, to recognize the usage of fibres in concrete, in these last four decades, most of the researches were done on mechanical behaviour of fibre reinforced concrete and the fibres itself.

The fibre reinforced concrete in many applications is subject primarily to bending rather than axial loading, as this indicates the performance in flexure. Johnson (1982) and Troxell el al (1968) conducted tests by determining the factors influencing the flexural strength measurement of fibre reinforced concrete. They proposed that such parameters that affect the performance of the flexural strength were the loading mode in flexure, specimen's size, shape and span, fibre length, dimension of fibres and fibre volume fraction.

After 10 years, Johnson and SkarendahI (1992) and Stang (1992) conducted similar tests by examining beams (150 x 100 x 750 mm) under a three point loading with different types of steel fibres in amounts from 30 to 100 kg/m^3. They concluded that the first-crack strength primarily depends on the matrix characteristic, while secondary crack depends on fibre parameters such as type, size and amount. At the post cracking state, the toughness of concrete depends on the fibre type, amount and fibre aspect ratio. However, Tat et al (1998) reported that the higher fibre concentration and longer fibres lead to better performance while bond stress between the matrix and fibres is a major influence to the flexural strength of fibre reinforced concrete.

Banthia and Dubey (2000) used residual strength test method (RSTM) to measure the flexural toughness of fibre-reinforced concrete in terms of its post peak residual

strength, which was investigated. This method has the ability to identify the influence of different fibre characteristics such as type, length configuration, volume fraction, geometry, and the modulus of elasticity. The results were based on two sets of testing. Test of set I clearly stated that fibrillated polypropylene fibres provided a better toughness than monofilament polypropylene fibres. Test of set 2 noted that hooked-end steel fibres had a better toughening strength than crimped steel fibres in fibre-reinforced concrete.

Some investigations were based on the effect of fibre content and damaging load on fibre reinforced concrete stiffness. Patton and Whittaker (1983) investigated steel fibre content for dependence of modulus of elasticity and correlation changes on damage due to load. They found out that there was approximately 3.3 percent increase over the modulus of elasticity of plain concrete for every 1.0% increase in fibre content by volume. Furthermore, the investigation showed that degeneration of stiffness started at approximately 30 percent of the ultimate load before the first visible crack appeared.

Rossi et al (1987) and Wafa and Nick (2004), analysed the effects of steel fibres on cracking at both local level (behaviour of steel fibres) and global level (behaviour of the fibre/cement composite) and showed that they were dependant on each other. The results of this analysis showed that 1.0% volume content of steel fibres could replace approximately 0.15% of flexural steel reinforcement. With the same fibre material, there was difference in behaviour of fibre reinforced concrete if the geometry of the fibres were different.

Barros and Figueiras (1999) used two types of steel fibres in fibre reinforced concrete for their research. These two fibres had similar tensile strength; however, their aspect ratio was different. Two tests were conducted: uniaxial compression tests and three-point loading flexural tests. They noted that increase in fibre percentage will significantly improve the load carrying capacity and decreased the crack opening and crack spacing.

Furthermore, the higher fibre aspect ratio of steel fibres exhibited an ultimate load twice the ultimate load of the other steel fibres.

There are relation between the flexural strength with the compressive strength and tensile strength of the concrete. Dwarakanath and Nagaraj (1991) predicted flexural strength of steel fibre concrete by these parameters such as direct tensile strength, split cylinder strength and cube compressive strength. The experimental test results and the determination of direct tensile strength for the composite from the results was reflected by the combined effects of fibre volume and ratio of length and diameter parameters in steel fibre reinforced concrete.

Investigation showed that the toughness of fibre reinforced concrete increase rapidly than plain concrete. Trottier et al (1994) investigated the toughness of fibre reinforced concrete by using different geometry of steel fibres, which included hooked end, crimped circular, crimped crescent and twin cone end steel fibres. One fibre volume fraction ($40kg/m^3$) was used throughout the research. The test included compressive strength test and flexural strength test, with measurement of deformation of specimen as the load is applied. They found out that fibres brought significant improvement in the toughness and energy absorption capacity of concrete. Based on four fibre geometries, fibres with deformations only at end appeared more effective than those with deformations over the entire length.

Within the same period Chen et al (1994) conducted similar test (toughness concept) to Trottier et al (1994), by determining the first crack and flexural toughness of steel fibre reinforced concrete using steel fibres with different dimensions. The research used hooked end steel fibre with 30 mm long and 0.5 mm in diameter. The investigation showed that all toughness parameter were affected by the width of the beam, even the depth

and span were unchanged. Furthermore, the specimen size not only influenced toughness, but also affects stress and deflection at first crack and ultimate flexural strength.

2.8.2 Different types of Fibres in Fibre Reinforced Concrete

There has been discussion on some usage of the different geometries of fibres in concrete, but the researches in this section have been based on the behaviour and mechanical properties of other types of fibres (material) that are used in concrete. Nanni et. al (1992) conducted an investigation on the use of newly developed aramid fibres for the reinforcement of Portland cement based concrete. The aramid fibres were produced in chopping a bundle made of epoxy-impregnated braided into aramid filaments. In this investigation, the behaviour of reinforced concrete of aramid fibres was compared to steel fibres and polypropylene fibres. Beams of 100 x 100 x 350 mm were tested under four point flexural loading. It was found that aramid fibres acted similar to steel fibres and is superior to polypropylene fibres. They concluded that aramid fibres were lacking in corrosion problems while having a higher performance than polypropylene fibres. However, the use of aramid fibres was not very economical.

Wang et. al (2000) applied recycled fibres as reinforcement in concrete. The recycled fibres included tire cords/wires, carpet fibres, feather fibres, wood fibres from paper waste and high-density polyethylene. The research conducted was based on shrinkage, durability and toughness characteristics test. The results of each test showed that recycled fibres can effectively improve the toughness shrinkage and durability characteristics of concrete. Wang et. al (2000) recommended and encouraged the use of low cost fibre for reinforcement which could lead to improved infrastructure with better

durability and reliability, as these applications will reduced solid waste from industrials and the environment.

Perry (2003) used large and small synthetic fibres to reinforced external pavements. He reported that the abrasion of pavement surface when in use, do exposed the steel fibres, creating health and safety hazards. Two tests were done. First test method conducted in a smaller area of external concrete pavement and compares the evaluation of steel fibre (hooked end, 60mm long) at a dosage of 30kg/m^3 and synthetic fibres (50mm long) at a dosage of 6.9 kg/m^3. Flexural strength and flexural toughness test were conducted as second test under three-point loading. The results of flexural test demonstrated that the steel fibre reinforced concrete has an equivalent flexural strength ratio of 53%, while synthetic fibre reinforced concrete was recorded as 78%. On the external concrete pavement, steel fibre has an equivalent flexural strength ratio of 20% and synthetic fibre was 41%. Perry (2003) concluded that synthetic fibre could provide concrete with the same level and even more of post-crack performance than steel fibres.

2.8.3 Usage of Fibres with Conventional Steel Reinforcement

The use of fibres also can be applied with the conventional steel reinforcement. Swamy and Sa'ad (1981) had done an investigation on deformation and ultimate strength of flexural in the reinforced concrete beams under four point loading with the usage of steel fibres, which consists of 15 beams (dimensions of 130 x 203 x 2500 mm) with same steel reinforcement (2Y-10 top bar and 2Y- 12 bottom bar) and variables of fibres volume fraction (0%, 0.5% and 1.0%). They concluded that fibres were effective in resisting deformation at all stage of loading from first crack to failure and also very effective in increasing the flexural stiffness at the failure stage of the beams. Furthermore, this

investigation showed that steel fibres prevented any advancing cracks and increased the ductility and post-cracking stiffness of the beam right till to failure.

Similar crack behaviour investigations, which were based on combination of 5 full scales reinforced concrete beams (350 x 200 x 3600 mm) with steel fibres (volume fraction of 0.38% and 0.56%) were done by Vandewalle (2000). In this investigation, the experimental results and theoretical predictions on the crack widths were compared. Vandewalle (2000) also concluded that the addition of steel fibres decreases the crack spacing and crack width. However, he reported that prediction of crack widths stated in Eurocode 2 on the combination of fibres with conventional steel reinforcement overestimated measured values. Thus, he established a simple empirical expression on the final crack spacing of steel fibre reinforced member.

Sener et. al (2002) calibrated the size effect of the 18 concrete beams under four-point loading. The beams thickness were uniform at 40 mm and length of 800 mm, but the height of the beams were varied at 40 mm, 80 mm and 160 mm. The results show that as height of the beam increased, the ultimate flexural strength increased. Also, the bending failure in fibre reinforced concrete exhibits a greater size effect and higher brittleness than concrete containing no fibres.

Most of the investigation of steel fibre reinforced concrete was based on flexural strength and crack width. In Singapore, Tan et, al (1993) conducted some investigation on the shear behaviour of steel fibre reinforced concrete. Six simply supported I-beams were tested under two-point loading with hooked steel fibres of 30mm long and 0.5mm diameter, as the fibre volume fraction increased every 0.25% from 0% to 1.0%. This investigation confirmed that the shear strength increased as much as 70 percent by adding small quantities of steel fibres (1.0%) into ordinary reinforced concrete.

2.8.4 Other Applications and Test Methods on Fibre Reinforced Concrete

Most of the investigations on fibre reinforced concrete have been base on the basic mechanical properties and behaviour. However, the investigations and researches of fibre reinforced concrete can be extented further to other types of structures and applications.

Sanjuan et. al (1998) investigated the effect of polypropylene fibre reinforced mortars on steel reinforcement corrosion induced by carbonation. In this investigation, crack control by fibres in plastic state mortars and crack evolution with time was studied. Furthermore, the influence of crack width on steel bar corrosion induced by carbonation was also monitored. The objective of the investigation is to assess the effectiveness of polypropylene fibre as secondary reinforcement to delay the initiation of reinforcement corrosion induced by carbonation. The fresh polypropylene fibre reinforced mortar was cast into a cylindrical ring and a solid cube of 70 mm (containing 5 steel reinforcement bars) located inside the mortar. They found that polypropylene fibres were able to control crack width in inadequately cured mortars and the addition of fibres reduced the corrosion rate on the steel reinforcement. However, there is no relationship between the corrosion rate and crack width.

Gupta et. al (2000) conducted impact test on fibre reinforced wet mix. It is known that shotcrete is often subjected to impact and dynamic load. Ten different commercially available shotcrete fibres were investigated in wet-mix shotcrete. The ten fibres included: four deformed steel fibres, two straight polypropylene fibres, one crimped polypropylene fibre, the straight carbon microfibres and one deformed polyvinyl alcohol (PVA) fibre. The mixes were shot onto wooden forms (600 x 500 x 100 mm) with fibre volume fraction of 10 to 60 kg/m^3. The result showed that fibre reinforcement in wet-mix shotcrete improved the energy absorption and toughness under impact loading. However, the improvement did

not happen under static conditions. Furthermore, Gupta et. al (2000) concluded that wet-mix shotcrete is highly sensitive to the rate at which load is applied.

Luo et. al (2001) conducted test on the mechanical properties and resistance against impact on steel fibre reinforced high-performance concrete. Five different geometry of fibres were included; steel-sheet-cut fibres and steel-ingot-milled fibres with four fibre volume fractions (4%, 6%, 8% and 10%) were applied into the mix. Beams (100 x 100 x 400 mm) and cubes (100 x 100 mm) were cast. The investigation showed that increase in fibre percentage improved the mechanical properties, and the peak compressive strength and flexural strength reached 140 N/mm^2 and 80 N/mm^2, respectively. This showed an increase of 61% and 774% compared to specimens containing no fibres. The impact test, showed that the specimens containing no fibres were smashed up and steel fibre reinforced high-performance concrete were kept intact with some radial cracks developed in front faces and minor cracks in side faces.

Fatigue is an important consideration with regard to the durability of thin concrete repairs. Repeated loading and restrained shrinkage can cause damages and debonding of repair layer. Mailhot et. al (2001) and Kesse and Lees (2007), all studied the flexural fatigue behaviour of steel fibre reinforced concrete by conducting series of flexural fatigue test (under three point-loading) with volume dosage of 40 kg/m^3. Three different types of steel fibres (hooked, nail-anchored and crimped) and two-water/cement ratios (0.35 and 0.45) were applied into the mix design. Six slabs (125 x 425 x 500 mm) were made with each batch. The tests were carried out at three different repeated stress levels: 85, 75 and 70% of the first crack strength. The survival life under repeated loadings was defined as the difference between the number of cycles at failure and number of cycles at onset of the first crack. The investigations found that the specimen with fibres exceeded 80% of the overall

life cycle, while survival life of specimen containing no fibre were extremely short, and the parameters affecting this were water/cement ratio and type of fibres used.

In the last two decades, steel fibres have replaced the conventional reinforcement in industrial ground floors Chen (2004), Vandewalle (2000) and Wafa and Nick (2004), although, in Nigeria, wire meshes are used. Research and practice have shown that steel fibre reinforcement is more efficient and economical for industrial floors. Experimental comparative done on ground slabs by Chen (2004), investigated the strength of 15 steel fibre reinforced and plain concrete ground slabs. The slabs were 2 x 2 x 0.12 m, reinforced with hooked end steel fibres and mill cut steel fibres. All slabs were centrally loaded using hydraulic and electric pump through 100 x 100 mm steel plate. He concluded that the load bearing capacity of concrete could be effectively increased when the slabs are reinforced with steel fibres. In addition, he also indicated that the energy absorption capacity of steel fibre reinforced concrete specimens can be used in assessing the effect on the load carrying capacity of steel fibre reinforced concrete ground slabs.

2.8.5 Guides and Practice of Fibre Reinforced Concrete

As discuss above, the fibre reinforced concrete have been so successfully used in the construction industries, in developed countries. However, there is no standards for the practices and a few engineers generally accepted the practice of fibre reinforced concrete and in developing countries it is still a new idea. Thus, this obstructs the understanding of the fibres and probably tends to discourage potential users from specifying on fibres. To overcome this problem, guide, good practice and awareness must be provided and applied to fibre reinforced concrete.

A report prepared by ACI Committee 544 (1993), gave guidance on specifying, mixing, placing and finishing of fibre reinforced concrete. The guide emphasized the difference between conventional concrete and fibre reinforced concrete and methods to deal with them. The report warned that calcium chloride should not be added with fibre reinforced concrete, but recommended the usage of water reducing and air-entraining admixtures with fibres. Furthermore, ACI Committee 544 (1993) suggested that fibres must be stored properly in other to prevent deterioration. The fibres have a tendency to protrude sharp corners, as this can be hazardous to personnel. The guide suggested the sharp corners should be chamfered.

The guide by ACI Committee (1984) suggested methods of adding the fibres into the fresh concrete mix as these methods provide good dispersion of fibres and prevent clumping (balling). The first method is that fibre can be added last into the fresh concrete mix, while second method is that fibres were mix with the aggregates before the addition of water into the mixer. All fibres must be clumping free (as rain of individual fibres) during the addition of fibres into the mixer. Furthermore, the guide stated that balling may occurs if the fibre volume fraction is more than 2% or even 1% with high aspect ratio and the other reason was the clumping of fibres before and during adding the fibres. On placing consideration, the fibre tends to be stiff and not workable. The recommendation is that vibration must be done to improve the placability. Again, the guide specified water/cement ratio must be in the range of 0.40 to 0.65.

Furthermore, the guide by ACI Committee (1993) specified the transporting and placing of fibre reinforced concrete with conventional equipment must be properly designed, maintained and clean. If pumping were used on transporting fibre reinforced concrete, some important point were suggested by the guide, 1) the pump must be capable

of handling the volume and pressure required, 2) the diameter of pump hose must be at least 150 mm wide and 3) avoid flexible hose if possible. However, the guide did not suggest any special attention on the finishing, but it indicated that overwork on the surface could result in bringing excessive fines and bleeding. The guide also indicated that curing of fibre reinforced concrete is same as conventional concrete.

Dunstan et. al (1986) recommended that the key to good practice dealing with fibre reinforced concrete and fibres are emphasis on the manufacture, design and constructional guides, as all materials used for engineering or building purpose, quality and design are interdependent. Failure in performing adequately in practice will results customer dissatisfaction, inadequately quality control and potential of defect appear on structure.

2.9 Shape, Geometry and Distribution of the Fibres in Concrete Matrix

The influences of fibres on fibre reinforced concrete are the shape, geometry and mechanical properties of fibres and the dispersion of fibres in the cementitious matrix. The knowledge of the fibre properties is important for design purpose. James (1990) stated that the high ratios of fibre modulus of elasticity have direct influences to the matrix modulus of elasticity where this facilitates the stress transfer from the matrix to the fibre. Fibre with a higher tensile strength is essential for the reinforcing action. Furthermore, fibres that have large values of failure strain will tend to have high extended or prolongation in the composites. The most common types of fibres are steel fibres and polymers fibres, due to low cost and their availability. However, other types of fibres may be used in the concrete composites depending on the needs. The properties and types of fibres are shown in Table 1. Properties of cement matrix are also included in the table.

James (1990) stated that having a lower Poisson's ratio prevented such problems on fibre-matrix interface associated with the fibre debonding. Furthermore, Riley and Reddaway (1968) stated that most fibres have surface flaws, due to handling, processing and manufacturing, as these surface defects can affect the strength properties of the composite. Such presence of flaws was varies by fibre length and diameter, which acts to strength reduction of fibre reinforced concrete. Additionally, the tensile strength of the fibres decreases when the fibre length increases James, (1990).

Each type of fibre can be categorized into two groups:

• Discrete monofilaments, which fibres are separated one from another (e.g. steel)

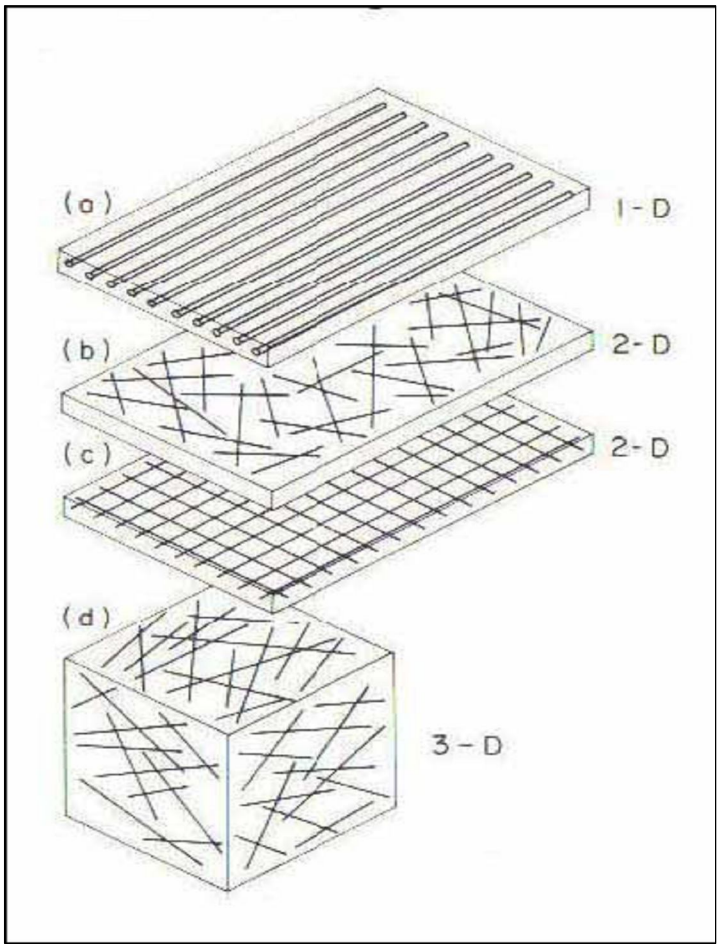
• Bundles of filaments, which all the fibres assemblies together, as each with a diameter of 10μm or less. Majority of man made fibres, such as inorganic fibres (e.g. glass), organic fibres (e.g. carbon, kevlar) and natural fibres (e.g. asbestos) all belong to this categories.

The monofilaments fibres due to their uniform improvement were commonly used in structural concrete to enhance the fibre-matrix interaction through mechanical anchoring

Bundled fibres usually do not break up into separate filaments, as they maintain their bundled nature in the cement matrix.

The reinforcing arrays of fibres are in two different ways: Continuous reinforcement and Discrete short fibres. The continuous reinforcements are usually in the form of long fibres, which are incorporated into the matrix in the methods of filament winding or layers of fibre mats. However, discrete short fibres with a length approximately 50mm or less are incorporated into the matrix by the methods of spraying and mixing. The reinforcing arrays are classified accounting to the distribution of fibres in the matrix as 1-, 2- or 3-

dimensional which have large effect on the mechanical properties of fibre reinforced concrete. The classification of fibre arrangement is shown in Figure 2.4.



Descriptions:

(a)1-dimensional arrangement.

b),(c)2-dimensional arrangement.

(d) 3-dimensional arrangement.

(a), (c) continuous fibres.

(b), (d) short discrete fibres.

Figure 2.4: Classification of fibre arrangement in 1, 2 and 3 dimensional.
(Source: Bentur and Mindness, 1990)

2.10 Interaction between Fibres and Concrete Matrix

Many detailed analytical predictions and models have been developed in the interaction of fibre-matrix stress transfer and crack bridging, as well as analysing the shear stresses that develop across the fibre-matrix interface. Many of the models were done by simulating analytical solution on fibre-matrix interaction, which are based on simple pullout geometry shown in Figure 2.5. These analytical models involved the shear stress

and frictional stress which were developed between the fibre and cement matrix, offering predictions on the efficiency of short, randomly oriented fibres in the concrete matrix. The effectiveness of fibres in the mechanical properties of the fibre reinforced concrete is influenced in two ways:

• Processes where load is transferred from the cement matrix to the fibres, and

• The bridging effect of the fibres in the concrete when the concrete cracks.

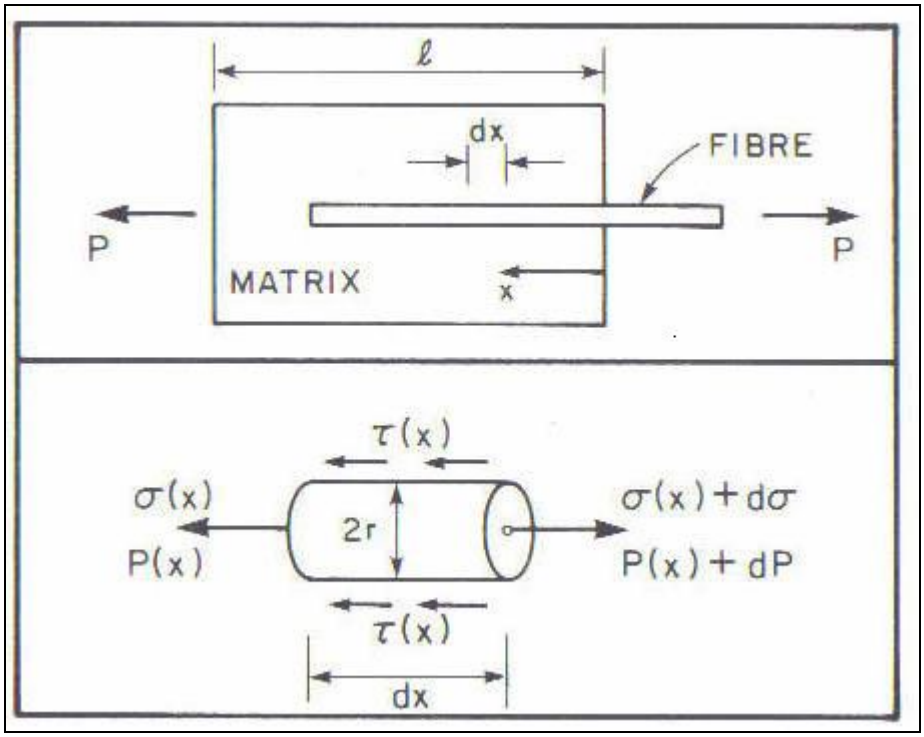


Figure 2.5: Pullout geometry to simulate the interaction between fibres and cement matrix. (Source: Bentur and Mindness, 1990)

The stress transfer effects must be considered in both pre-cracking case and post cracking case for the brittle fibre reinforced concrete, as the processes of stress transfer are different in these two cases. Such understanding of mechanisms for the stress transfer permits the prediction of stress/strain curve on the fibre reinforced composite, the mode of fracture and

a basis for developing performance on the composite with the modification of the interaction of fibre-cement matrix.

In uncracked state of the fibre-cement matrix, the major mechanism is that the load is transferred from the matrix to the fibre in the elastic stress stage. This means that the strain (longitudinal displacement) of the fibre and the matrix at the interface are almost the same. The stress that developed at the interface, which need to distribute the external load between the fibres and the matrix is shear stress. This is required in order for these two strains to remain same, where the elastic moduli of these two components are different. The elastic shear transfer was used in the prediction of limit of proportionality, modulus of elasticity, elastic stress/strain behaviour and determination of the first crack stress of the fibre-matrix composite. However, the elastic shear stress distribution and deformation along the interaction of fibre and matrix was not consistent (Figure 2.6).

A simple fibre-matrix system containing one single fibre is shown in Figure 2.6. Under unloaded stage, stresses in the fibre and the matrix were assumed to zero. The stress and deformation of the fibre and matrix was remaining same. When a load was applied, either by tension or compression, some of the load was transferred to the fibre along its surface. This means that the stiffness of the fibre and the matrix are different as shear stress develops on the surface of the fibre. The deformation and interaction of fibre-matrix when tension and compression are exerted, is also shown in Figure 2.6.

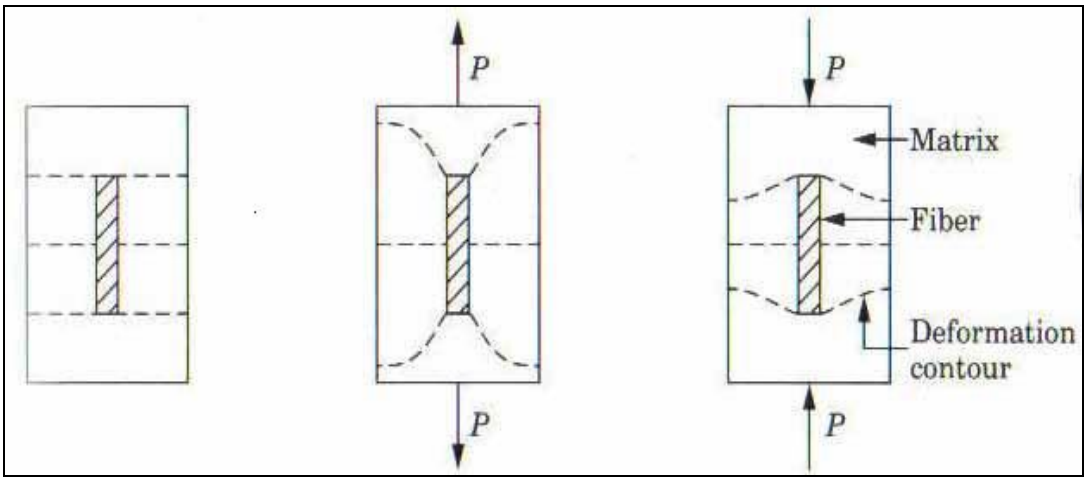


Figure 2.6: Interaction of fibre-uncracked matrix.
Left to right: unloaded, tension and compression.
(Source: Balaguru and Shah, 1992)

As the load increased, debonding around the surrounding of the interface takes place and such frictional slip occurs as a process controlling stress transfer at that area. Once this situation happens, some deformation between the fibre and the matrix will develop and the frictional stress will be assumed uniformly distributed at the interface of fibre and matrix. The controlling process of stress transfer is important where such properties like ultimate strength and strain can be determined, while this process is fundamental in the post-cracking case as the fibres bridge across the cracks.

In the cracked state, adhesional shear bond strength and frictional shear strength are the two major mechanisms for the stress transfer between the interaction of the fibre and matrix. The shear stress at the interface from elastic state is transfered to frictional stress and adhesional shear bond stress as the loading exceeds the fibre and matrix shear strength. When this stress is exceeded, the debonding of fibre and matrix occurrs, while frictional shear stress is developed on the interface at the debonded surrounding. However, the post-cracking behaviour is very difficult to predict, as the fibre orientation and the fibre length

efficiency start to participate in the behaviour of the concrete. A further description of the stress transfer between the fibre and matrix composite is shown in Figure 2.7.

Once the matrix containing the fibres cracks at a certain stage when it is loaded with tension force, the load is carried on to the fibres across the cracks and spread from one side of the matrix to the other. This interaction of fibre-matrix on cracked condition based on tension is shown in Figure 2.7.

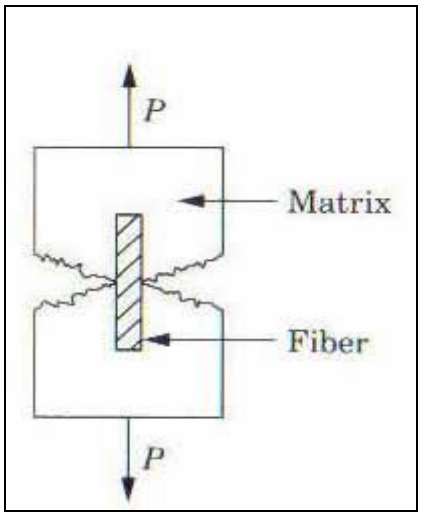


Figure 2.7: Interaction of fibre-cracked matrix.
(Source: Balaguru and Shah, 1992)

2.11 Critical Fibre Volume Dosage

The load bearing capacity of a fibre reinforced concrete depends on the volume dosage rate applied into the concrete matrix. In fibre cement composite, the failure strain of fibre is normally greater than the failure strain of the concrete. To prevent the failure of fibre, the load bearing capacity of the fibre must be greater than the load applied on the concrete when the first crack appears. This assumes that the concrete does not contribute any further strength beyond the point of first crack, as the load is fully transferred to the fibre. Furthermore, the fibres are able to carry more load, resulting in the ultimate strength of the fibre cement composite been higher than the matrix strength itself. An equation for

minimum fibre volume dosage rate, V_{cr}, has been developed which is to equal the load bearing capacity of the fibre/cement composite and the fibre load bearing capacity.

The minimum or critical fibre volume dosage rate, V_{cr}, that needs to be added into concrete for its loading bearing capacity or to sustain the load after the concrete occurs is given as (James, 1990):

$$V_{cr} > \frac{\sigma_{mu}}{\sigma_{mu} + (\sigma_{fu} - \sigma'_{fu})} \quad \text{..... 2.1}$$

where V_{cr}, = critical/minimum fibre volume dosage

σ_{mu} = ultimate tensile strength of the concrete

σ_{fu} = ultimate tensile strength of the fibre

σ'_{fu} = stress on the fibres when concrete fails at its first crack

BS8110 (1997) stated that the strain of the concrete (ultimate concrete strain) at the point of first crack is 0.003. If the strain on the concrete and the fibre is assumed to be same.

The stress in the fibre at the point of first crack can be taken as the product of the ultimate strain of the concrete and the modulus of elasticity of fibre. The above equation 2.1 can be rearranged as:

$$V_{cr} > \frac{\sigma_{mu}}{\sigma_{mu} + (\sigma_{fu} - E_{st}\varepsilon_{cu})} \quad \text{..... 2.2}$$

where E_{st} = modulus of elasticity of the steel

ε_{cu} = ultimate strain of concrete = 0.003

Equation 2.3 below taken from BS8100 was used to predict the ultimate tensile strength of the concrete, as the tensile strength of the concrete was required to obtain the minimum fibre volume dosage rate.

$$\sigma_{mu} = f_{ct} = 0.4x(f'_c)^{0.5} \quad \text{..... 2.3}$$

where f'c = characteristic compressive strength of concrete

f'_{ct} = characteristic tensile strength of concrete

James (1990) stated that the minimum fibre volume dosage rate for steel, glass and polypropylene fibres in concrete matrix is calculated to approximately 0.31%, 0.40% and 0.75% respectively. For chopped and randomly oriented fibre composites, the minimum fibre volume dosage rate is higher than the value stated as the efficiency factor such as fibre length and orientation effects can influence the volume dosage rate. The load of the concrete at the point of first crack is enough to distribute on to the fibres when the minimum fibre volume dosage rate has been reached. It is important that equation 2 gives an indication of the volume of fibres required to be added into the concrete, where it will increase the ductility and strength of concrete.

2.12 Efficiency of Fibre Reinforcement

The fibre reinforced concrete consists of distribution of short fibres in the cement matrix. The contribution of short, inclined fibres on the mechanical properties of fibre reinforced concrete is usually less than long fibres placed parallel to the load. This means that the efficiency of the short and inclined fibres is less. However, the efficiency of the fibres in the cement matrix to enhance the mechanical properties of concrete can be judged in two ways:

i) The property enhancement in the strength of the concrete, and

ii) The property enhancement in the toughness of the concrete.

These effects on the properties of concrete are depending on the fibre length, the orientation of fibres distributed in the concrete and the shear bond strength of the fibre/cement composite. All of these three factors are not independent as the effects on the fibre length and orientation are largely extended to the bond between the fibre and cement matrix.

In most of the engineering applications, the fibre efficiencies are expressed in terms of efficiency factor, which values are from 0 to 1, Bentur et. al, (1990). The efficiency factor was used to express the load applied on the ratio between the reinforcing effect on the short inclined fibres and the continuous fibres aligned parallel. Determination of efficiency can be obtained by empirical or analytical calculations on the factors for length efficiency η_l, and orientation efficiency η ,.

2.12.1 Length Efficiency

The effects of length of the fibre can be analysed by the mechanisms of stress transfer on the performance of the concrete, which has been explained in Section 2.5. The critical length parameter, l_c, can be defined as the minimum fibre length which is needed to build up stress or load in the fibre from the frictional and shear stress transfer to it; which is equal to its failure strength (load). The definition of the critical length of fibre is shown in Figure 2.8 Bentur et. al, (1990). In Figure 2.8, curve 1 represents frictional stress transfer mechanism and curve 2 represents an elastic stress transfer mechanism. For curve 1, the fibre length is less than the critical length, where there is not sufficient embedded length to produce a stress equal to the fibre strength. If the length of the fibre exceeds lc, the stress on most of the fibres will reach its yield strength, as this is shown on the curve 2. The critical length of a fibre can be calculated as James, (1990,):

$$l_c = \frac{df\sigma f}{2\tau} \quad \text{..... 2. 4}$$

where d_f = fibre diameter

σ_f = ultimate strength of the fibre

τ = interfacial bond strength

Although the interfacial bond strength depends on the strength of the concrete and the bonding type of the fibre, but Balaguru and Mindness (1992) stated that it can be taken approximately as $1N/mm^2$.

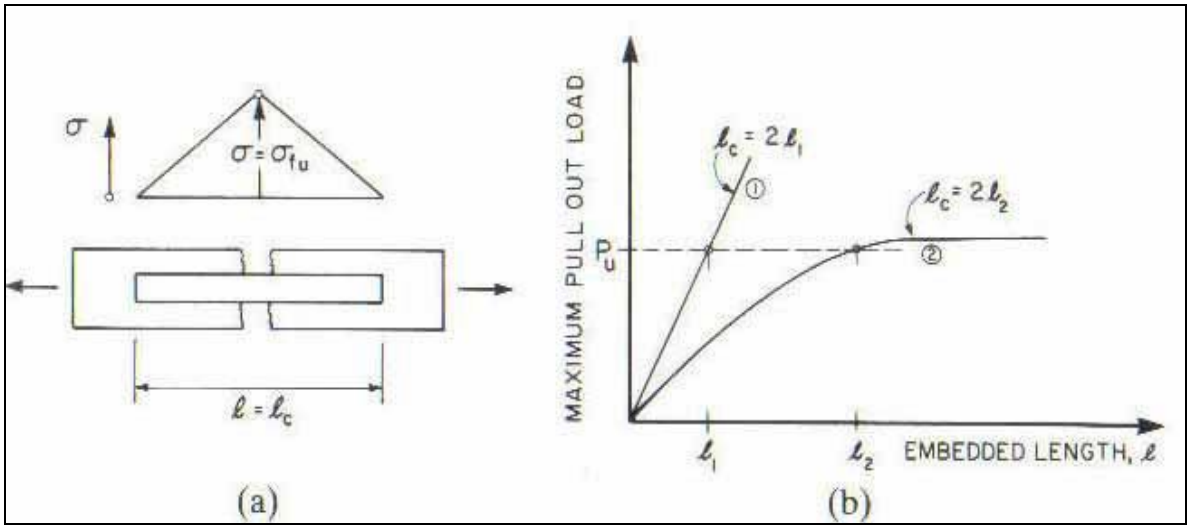


Figure 2.8: Definition of critical length: (a) Frictional stress distribution on fibres. (b) Intersection of fibre breaking load P_u, with pullout load versus embedded length. (Source: Bentur and Mindness, 1990)

The stress in the fibre is not constant along the entire length for discontinuous fibres. However, the stress developed linearly at the end of the fibre with a distance half of the fibre length, which is shown on Figure 2.8. But in most of the fibre reinforced concrete, the fibres are not placed and aligned parallel to the direction of applied stress. This shows

that the fibres are not fully effective in the strengthening for fibre reinforced concrete. Furthermore, fibre placed perpendicular to the applied stress tends to have less or even no effect in the increasing strength of fibre reinforced concrete.

Equation 2.4 is the accurate method to describe the required length for the fibre to transfer load, but there are other indications of efficiency factor on the load applied to the fibre, which is the length to diameter ratio or fibre aspect ratio of the fibre. The length to diameter ratio is a simplified way to estimate the effectiveness of fibre to transfer the load. As the diameter of the fibre is larger, more loads can be transferred on to the fibres. Similar to length to diameter ratio, the fibre aspect ratio shows that the more surface of the fibre is in contact with the concrete matrix, the greater the load can be transferred to the fibre.

The length efficiency factors are used for the prediction of the properties of fibre reinforced concrete in pre-cracking and post-cracking state. The length efficiency factors, which take account of the critical fibre length on both pre-crack and post crack state, are shown by the following equation (Bentur et. al, 1990):

Pre-cracking state:

$$\eta_1 = 1 - \frac{l_c \varepsilon_{mu}}{2l\varepsilon_{fu}} \quad \text{.....2.5}$$

Post-cracking state:

$$\eta_1 = 1 - \frac{l_c}{l} \qquad \text{for } l >> 2l_c \text{ .....2.6}$$

$$\eta_1 = \frac{1}{4} - \frac{l_c}{l} \qquad \text{for } l << 2l_c \text{ .....2.7}$$

where η_1 = length efficiency factor,

l_c = critical fibre length (obtained from Equation 2.4),

ε_{mu} = strain of the fibre (the point of first crack),

l = embedded length of fibre in the cement matrix,

ε_{fu} = ultimate strain of the fibre.

2.12.2 Fibre Orientation

If all fibres were placed parallel to the direction of applied stress, the orientation efficiency is unity James, (1990) and Murdock et al (1968). However, fibres in the concrete matrix is randomly distributed, where the orientation of the fibre is unpredictable within the concrete with either in one, two or three-dimensional arrays. In such distributions, some of the fibres are placed at an angle (θ) to the load orientation or applied stress, this is shown on Figure 2.9. It shows that fibre at an angle can carried less load to those fibre placed parallel to the load direction by using vector analysis in the components of x, y and z. Furthermore, fibres at an inclined angle to the load direction carry more bending stress during the bridging a crack and decreases the fibre efficiency in carrying load applied to the concrete.

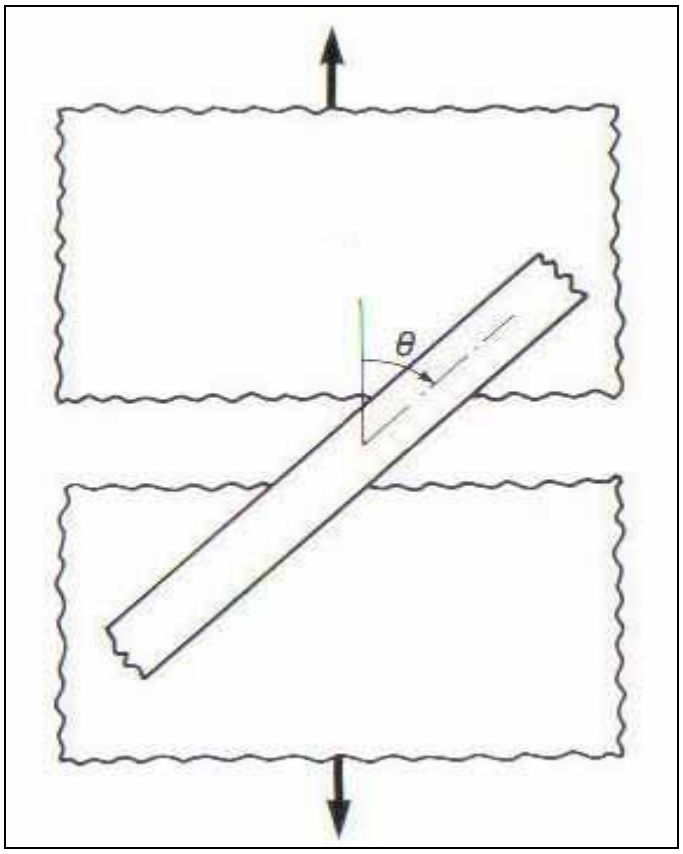


Figure 2.9: The intersection of an oriented fibre across a crack with an angle (θ).
(Source: Bentur and Mindness, 1990)

The orientation efficiency of the fibre in the concrete matrix is classified into two approaches. The first approach assumed that the fibre-reinforced composite is constrained, and the deformation of fibre is subject only in one direction of applied stress. The second approach (unconstrained) assumed that the deformation occurs in other directions of applied stress. Such example of constrained and unconstrained can be show in hardened property test like flexural strength test, where the concrete beams is subject to deformation in one direction only, and direct compressive strength test, concrete cylinders were subject to deformation in different planes. However, the vibration and compaction of the concrete rearranges the fibres, so most of the fibre reinforced concrete are assumed in randomly two-

dimensional orientation. The orientation efficiency factor for unconstrained and constrained with different fibre orientation is shown in Table 2.2.

Table 2.2: Orientation Efficiency Factor for Unconstrained and Constrained Fibre Reinforced Concrete. (Source: Bentur and Mindness, (1990))

Fibre Orientation	$\eta_{\theta,}$ Orientation efficiency factor	
	Unconstrained	Constrained
Aligned, 1-D	1	1
Random, 2-D	1/3	3/8
Random, 3-D	1/6	1/5

2.13 Prediction of the Behaviour and Properties of FRC

From Section 2.10, the major roles of fibres occur in the post-cracking state and the fibres act as bridge across the cracks on fibre reinforced concrete. However, the first crack on the composite will not lead to shattering failure. Eventually, this will results in the redistribution of the load between the fibres and the concrete, as discussed previously from Section 2.11. As additional load is applied on the composite, more cracks are developed until the composite is separated into few numbers of segments (a to g). The separation of the composite into segments by cracks is known as 'multiple cracking'. Figure 2.9 shows the stress and the strain of failure of the fibre which occured as more of the cracks developed. The range of initial constant stress (the first crack stress, $E_c\varepsilon_{mu}$) is eventually known as the modulus of elasticity of the FRC composites (E_c). When the multiple cracks stopped and faded out, the additional load will cause the pullout of the fibres (z), as shown in Figure 2.9. At this region, the slope is 'E_fV_f' where the aligned and continuous fibre will stretch and fail when the fibres reach their maximum load bearing capacity ($\sigma_{fu}V_f$). The

final schematic description of the stress/strain curve for the FRC composite is shown in Figure 2.10.

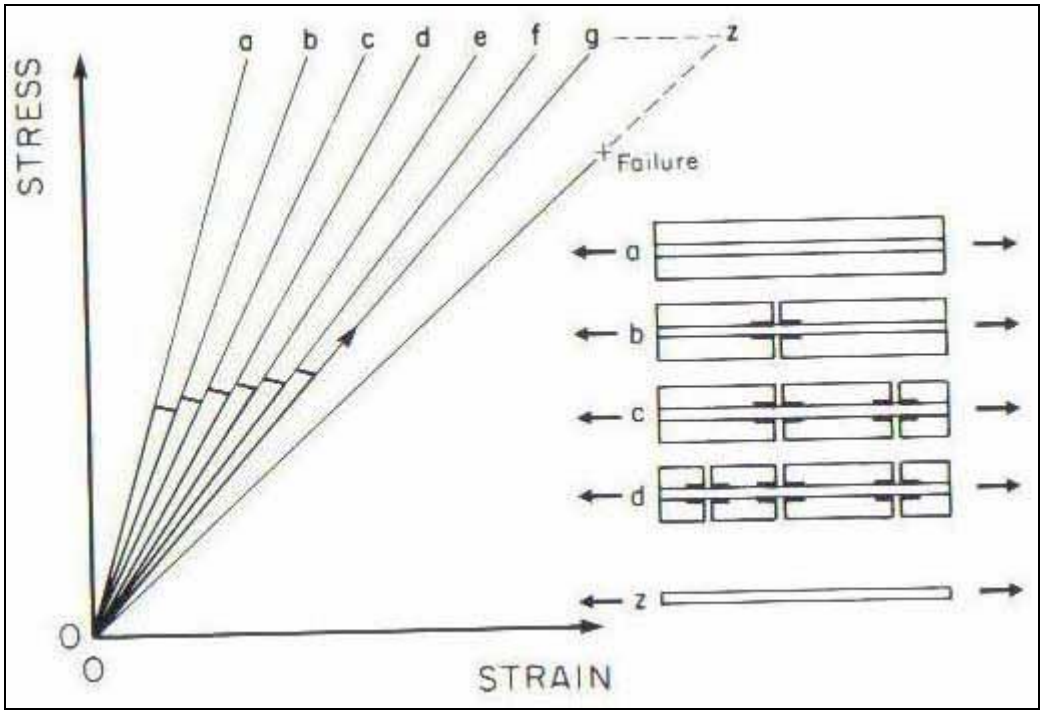


Figure 2.10: The multiple cracking process and the stages of stress/strain curve related to the multiple cracking processes. (Source: Bentur and Mindness, 1990)

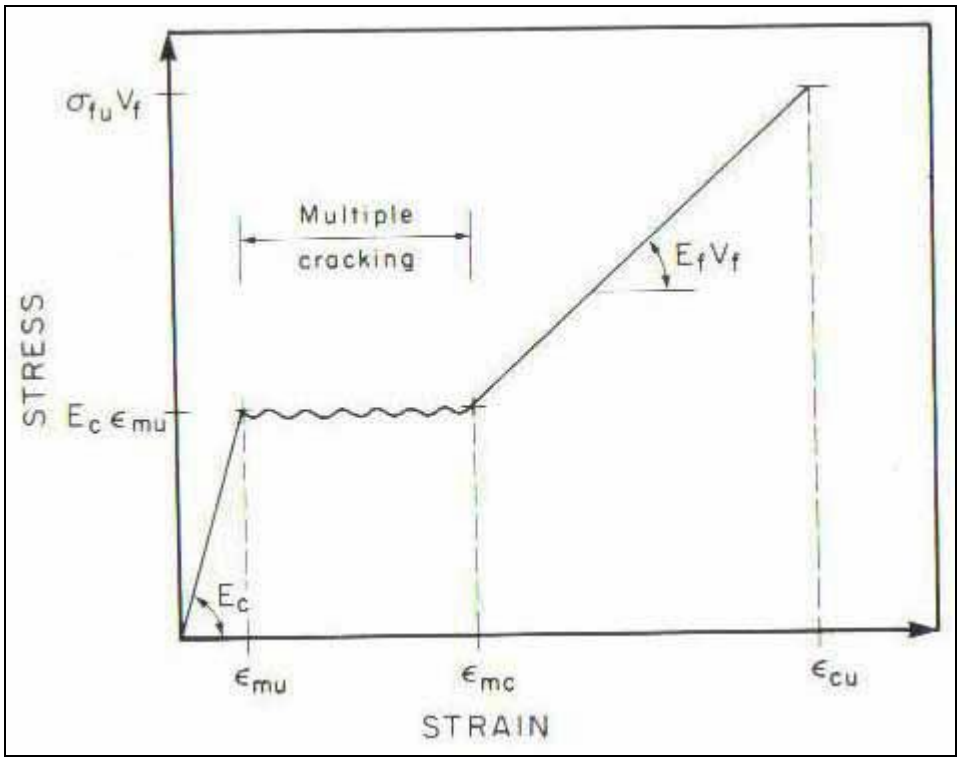


Figure 2.11: Final schematic description of stress/strain curve.
(Source: Bentur and Mindness, 1990)

Consequently from Figure 2.11, the mechanical behaviour of fibre reinforced concrete can be illustrated by three stages of the tensile stress versus strain curve:

i) Elastic stage.

In this stage, the load is carried by both the fibres and matrix. The stress is transferred to the fibres when the deformation in the matrix occurs, while the stress will transfer back to the matrix when the deformation stopped. This stage continued up till the point of first crack, where the concrete strain arrived at a value is 0.003.

ii) 'Multiple cracking' stage.

The concrete strain has exceeded the ultimate strain of its composite, which is above the stain value of 0.003, as the cracking and energy absorption takes place in this

stage. When the stress continues to increase between the fibres and matrix, formation of fine cracks are developed.

iii) Post-multiple cracking stage.

In this stage, the matrix no longer carries the load, and the stress is transferred to the bridging fibres, as the pullout and stretch occurred in fibres.

Many models and analytical predictions are used to predict the modulus of elasticity, the first crack stress and strain from the shape of the tensile stress/strain curve. In such models and predictions, attention was given to the energy involved in the failure fracture of the fibre reinforced concrete composite, and this attention comprised of composite materials approach, facture mechanics and multiple cracking.

The 'rule of mixtures' based on the composite material approach, was shown by models of the composite in Figure 2.11. Bentur and Mindness (1990) stated that the 'rule of mixtures' for the properties of the composite are equal to the weight average of the properties of each individual components. The components such as modulus of elasticity and strength are valid when these two components are in the elastic stage.

Hence, the 'rule of mixtures' can only applied at the pre-cracking stage of the fibre reinforced concrete composite. However, the prediction of the components by 'rule of mixtures' takes the effect of the fibre length efficiency and fibre orientation efficiency. The prediction of modulus of elasticity, (Ec) and first crack tensile stress of the composite, (σ_{mu}) were developed by Bentur and Mindness (1990), while the first crack flexural stress, σ_f was developed by Namy (2001). Through these predictions, these rules were able to apply into the concrete design and have a better utilisation and advantages of fibre reinforced concrete.

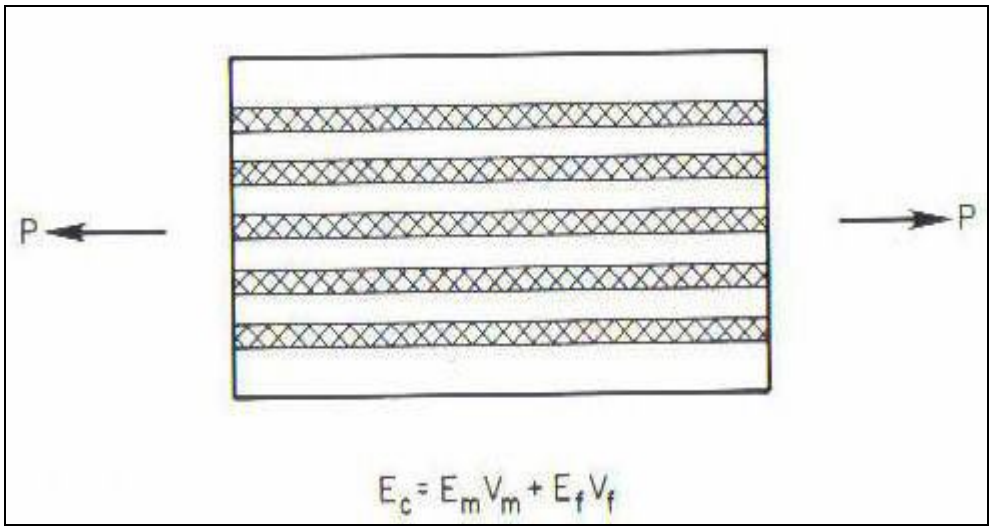


Figure 2.12: The parallel model of 'rule of mixture'.
(Source: Bentur and Mindness, 1990)

Modulus of Elasticity:

E_c = '$E_m V_m$' (matrix) + '$\eta_1 \eta_\theta E_f V_f$' (fibre) ……2.8

where E_c = modulus of elasticity of the fibre reinforced composite

E_m = modulus of elasticity of the matrix

V_m = volume fraction of the matrix

η_l = fibre length efficiency factor

η_θ = fibre orientation efficiency factor

E_f = modulus of elasticity of fibre

V_f = volume fraction of the fibre

First crack tensile stress:

σmu = '$\sigma'_{mu} V_m$' (matrix) + '$\eta_l \eta_\theta \sigma'_f V_f$' (fibre) ……2.9

where σ_{mu} = first crack tensile strength of the fibre reinforced composite

σ'_{mu} = tensile strength of the matrix at point of first crack

σ'_f = tensile strength of the fibre at point of first crack

First crack flexural stress (modulus of rupture):

σ_f = '0.843 $f_r V_m$' (matrix) + '425 $V_f (l/d_f)$' (fibre) ……..2.10

where σ_f = first crack flexural strength of the fibre reinforced composite

f_r = stress in the matrix (modulus of rupture of the plane concrete)

l/d_f = fibre aspect ratio (ratio of length to diameter)

CHAPTER THREE

EXPERIMENTATION

3.2 Preamble

This chapter gives the details of the experiments performed starting with materials used in the experiments. About two hundred and sixty specimens including mortar cubes, concrete cubes, mortar beams, concrete cylinders and concrete beams were tested to determine the properties of steel fibre mortar and concrete.

All the experiments were performed in accordance with the respective Codes and Standards in the Heavy Structural Laboratory of the Department of Civil Engineering, Ahmadu Bello University, Samaru, Zaria. The results are as presented below in this chapter.

3.2 Materials

The materials used in this experiment include the following: sand, coarse aggregates, cements and fibres. The fibres are of three types namely circular steel fibre (CSF), rectangular steel fibre (RSF) and chips shaving steel fibre (CHSF).

3.2.1 Fine Aggregate (Sand)

The sand used in this experiment was clean river sand obtained locally from Samaru, Zaria. It was subjected to particle size distribution tests in accordance with BS 812 (1985) : Part 103.1, Clause 5b The results are as presented in Table 3.1 below.

Table 3.1: Sieve Analysis Result for Fine Aggregate

B S Size (mm)	Weight Retained (g)	Cummulative Percentage Retained (%)	Cummulative Weight Passing (%)
4.75	40	4	96
2.36	150	16	84
1.18	124	32	68
600µm	175	49	51
300µm	387	88	12
150µm	90	97	3

Each value is an average of three test values. See the Appendix 2 for the raw data.

3.3.2 Coarse Aggregate/ Stones

The coarse aggregate is also subjected to particle size distribution test in accordance with BS 812 (1985). Part 103.1 and the results are presented in Table 3.2.

Table 3.2: Sieve Analysis Result for Coarse Aggregate

B S Size (mm)	Weight Retained (g)	Cummulative Percentage Retained (%)	Cummulative Weight Passing (%)
38.1	30	1	99
25.4	130	6	94
20.0	430	6	94
20.0	430	20	80
14.0	900	50	80
10.0	900	80	20
6.35	400	93	07
5	200	100	0

Each value is an average of three measured values. See Appendix 3, for the raw data.

3.3.3 Cement

Dangote Ordinary Portland Cement was used in this experiment. This cement was subjected to consistency tests in accordance with BS12 (1996) Part 2. And the results are as shown in Table 3.3. Raw data can be seen in Appendices 4 and 5.

Table 3.3: Consistency Test on Dangote (OPC)

Property	Value
Normal Consistency (%)	34.5
Initial Setting Time (min)	89.0
Final Setting Time (min)	148.0
Soundness (mm)	2
Specific Gravity	3.14
Cube Compressive Strength (N/mm^2)	
3 – day	14.5
7 – day	24.2
28 – day	27.5

3.3.4 Water

The water used in this experimental test was obtained from the water tap inside the Laboratory and it is portable.

3.2.5 Fibres

At the first stage of the experiments, three of the fibres were used. These fibres are shown in Plate 1, indicating chipping steel fibre, rectangular steel fibre and circular steel fibre.



Plate 3.1: The three types of waste fibres used in this work

To be able to study effectively the properties of waste steel fibre reinforced concrete. Three types of locally available industrial waste fibres from two main sources were used. See Plate 3.1. They are:

1. Circular Section Steel Fibres (CSF): These fibres are gotten from used cars tires after burning off the rubber fabric and cutting or chopping the wire into lengths of fibres. These fibres are smooth straight fibres with an average diameter of 1.1mm, and a tensile strength of 700-800 N/mm^2.
2. Rectangular Section Steel Fibres (RSF): These fibres are gotten from used tires after burning off the rubber fabric and cutting or chopping the wire into lengths of fibres like CSF. These fibres are smooth straight fibres with typical cross section

ranging from 0.85mm to 1mm thickness by 0.85 to 1.14mm width.

3. Steel Shavings (Chips): These are waste steel materials from metal machining, generally having a rough surface, toothed edges, and twisted sections. These fibres are obtained by cutting the shavings into proper lengths. It was found that the shavings could be stronger than the original steel, with a typical tensile strength of 1,200 N/mm^2 whereas the tensile strength of steel was 700 N/mm^2.The steel shaving fibers used in this work has the following dimensions: 40-50 mm in length and 0.8-1.0 mm in equivalent diameter (Plate 3.1).

3.4 Steel Fibre Mortar/Concrete Tests

The tests involved mixing the three types of fibres in mortar and concrete to determine their effect on the mortar and concrete materials. There are control mixes where no fibres are added to both mortar and concrete. The mortar mix is one part cement to two part sand (1:2) and the concrete mix is one part cement to two part sand and four part coarse aggregate (1:2:4) with percentages by volume of each fibre ranging from 0.5, 1.0, 1.5 to 2.0 See Appendix 1 for sample calculations. Confirmation tests were also carried out to confirm the literature as well as the obtained results in the case of chip steel fibre.

3.3.2 Steel Fibre Mortar Cube Tests

The mix ratio is as earlier stated with various percentages of the three fibres as indicated above.

The test on mortar cubes were performed in accordance with BS 1881 Part 4 (1988) and the result are presented below in Tables 3.4 to 3.6. In addition to control mortar cubes, sixteen mortar cubes were produced for each type fibre and cured for 28 days inside water prior to test.

The compressive strength of concrete can be calculated using the following formula Neville, (1996) below:

$$f'c = \frac{PX1000}{A} \quad \text{.....3.1}$$

Where: f'c = Compressive strength of concrete (N/mm^2).

P = Maximum load applied to the specimen in kN.

A = Cross sectional area of the specimen (mm^2).

The test results are as shown in Tables 3.4 to 3.6:

Table 3.4: Compressive Strengths Test Results – CSF (Mortar Cubes)

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)
0	27.5
0.5	28.2
1.0	29.4
1.5	32.7
2.0	30.0
Each value is an average of four tests, see Appendix 6 for the raw data	

Table 3.5: Compressive Strengths Test Results – RSF (Mortar Cubes)

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)
0	27.5
0.5	28.4
1.0	29.7
1.5	30.9
2.0	34.2
Each value is an average of four tests, see Appendix 7 for the raw data	

Table 3.6: Compressive Strengths Test Results – CHSF (Mortar Cubes)

Fibre Percentage (%)	Average Compressive Strength(N/mm^2)
0	27.5
0.5	28.8
1.0	29.8
1.5	33.2
2.0	33.6
Each value is an average of four tests, see Appendix 8 for the raw data	

3.3.2 Steel Fibre Mortar Beam Flexural Tests

Using same ratio as mortar cubes above, a mortar beam of length of 750 millimeters with a cross-section of 150 x150 millimeters were produced and cured for 28 days in water. For each type of fibre, twelve number beams were produced apart from the control beams which are of zero fibre. The curing was under normal laboratory temperature. Considering the three fibres and the number of percentage, a total of 39 mortar beams including control were produced. All beam tests were performed in accordance with BS1881 Part 116 (1996). The results are presented in Tables 3.7 to 3.9 below. The flexural test was three-point load test and the arrangement is as shown in Plate 3.2.

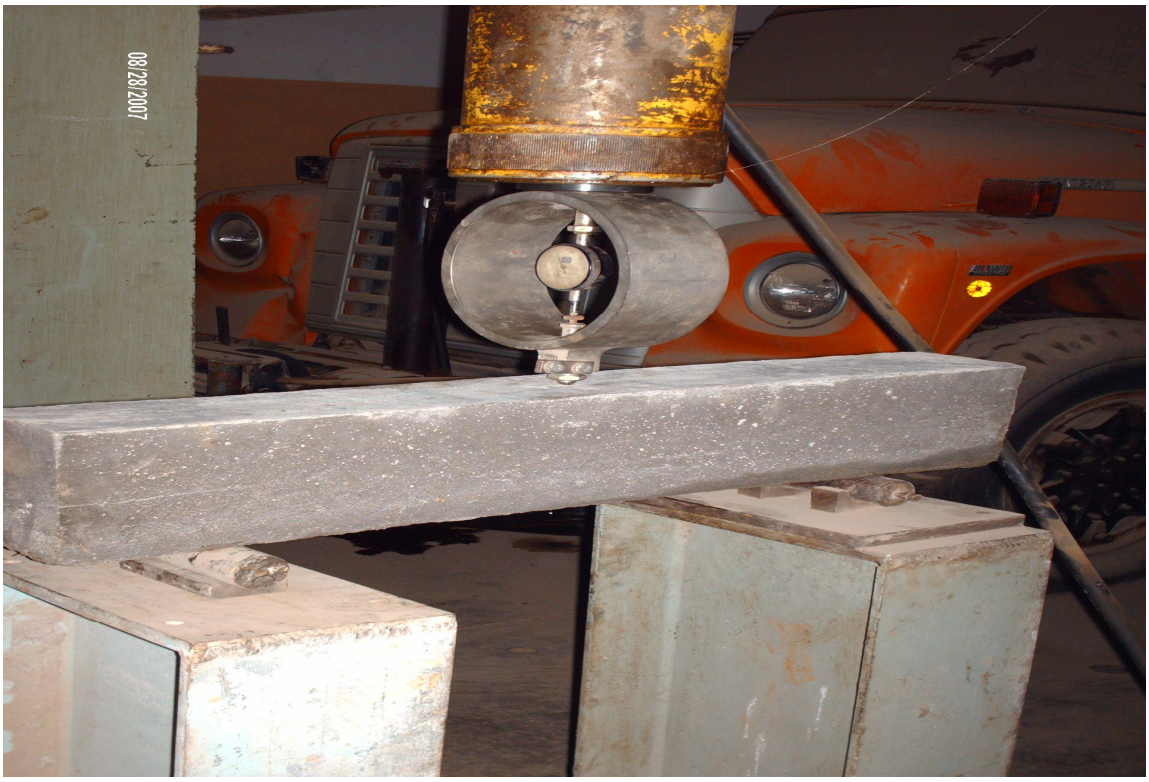


Plate 3.2: Test Rig with Beam specimen positioned for test

The attached central dial gauge was used to measures the load and deflection of concrete beam specimens

The flexural strength of concrete can be calculated using the following formula:

$$f_{cf} = \frac{PXLX1000}{BXD^2} \qquad \text{.....3.2}$$

Where: f_{cf} = Flexural strength of concrete (N/mm^2).

P = Maximum load applied to the specimen in kN.

L = Length of the specimen in mm.

B = Width of the specimen in mm

D = Diameter of the specimen in mm.

Table 3.7: Flexural Strengths Test Results

- CSF (Mortar Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.55
0.5	4.60
1.0	4.75
1.5	4.95
2.0	5.25
Each value is an average of three tests, see Appendix 9 for the raw data	

Table 3.8: Flexural Strengths Test Results

- RSF (Mortar Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.55
0.5	4.55
1.0	4.90
1.5	5.05
2.0	5.20
Each value is an average of three tests, see Appendix 10 for the raw data	

Table 3.9: Flexural Strengths Test Results

- CHSF (Mortar Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.55
0.5	4.92
1.0	5.10
1.5	5.30
2.0	5.30
Each value is an average of three tests, see Appendix 11 for the raw data	

3.5.3 Workability Test

One of the ways of measuring the workability of concrete is by the use of slumps and compacting factor tests. These two measures are used indirectly to assess workability. The mix ratio used are one part of cement to two part of sand and four parts of coarse aggregate (1:2:4) with water-cement ratio of 0.6. In addition to the control mix without fibre, various percentages of fibre ranging from 0.5, 1.0, 1.5, and 2.0 were used to produce a concrete of mix ratio as indicated above. After mixing the fresh concrete the slump tests as well as the compacting factor tests were performed in accordance with BS1881 Part 102 and Part 103 (1996) for slump and compacting factors test respectively. These results are as presented in Tables 3.10 to 3.12 for slump tests and 3.13 to 3.15 for compacting factor test.

Table 3.10: Slump Test Result – CSF (Concrete Mix)

Fibre Percentage (%)	Slump Height (mm)
0	166
0.5	160
1.0	149
1.5	140
2.0	126
Each slump value is an average of three tests values, see Appendix 12 for the raw data	

Table 3.11: Slump Test Result - RSF (Concrete Mix)

Fibre Percentage (%)	Slump Height (mm)
0	166
0.5	159
1.0	157
1.5	130
2.0	120
Each slump value is an average of three tests values, see Appendix 13 for the raw data	

Table 3.12: Slump Test Results – CHSF- (Concrete Mix)

Fibre Percentage (%)	Slump Height (mm)
0	166
0.5	120
1.0	102
1.5	84
2.0	60
Each slump value is an average of three tests values, see Appendix 14 for the raw data	

Table 3.13: Compacting Factor Results – CSF (Concrete Mix)

Fibre Percentage (%)	Compacting Factor (Ratio)
0	0.995
0.5	0.969
1.0	0.940
1.5	0.910
2.0	0.891
Each slump value is an average of three tests values, see Appendix 15 for the raw data	

Table 3.14: Compacting Factor Results – RSF (Concrete Mix)

Fibre Percentage (%)	Compacting Factor index
0	0.995
0.5	0.984
1.0	0.980
1.5	0.940
2.0	0.892
Each slump value is an average of three tests values, see Appendix 16 for the raw data	

Table 3.15: Compacting Factor Results – CHSF (Concrete Mix)

Fibre Percentage (%)	Compacting Factor index
0	0.995
0.5	0.901
1.0	0.894
1.5	0.829
2.0	0.801
Each slump value is an average of three tests values, see Appendix 17 for the raw data	

3.6 Steel Fibre Concrete Cube Test

Concrete cubes were cast using the same mix used for the slump test as well as the compacting factor test and were vibrated using laboratory vibrating table for one minute. This test was performed to find the increase and differences of strength according for the increasing percentage of fibre in the concrete. For each type of fibre, sixteen cubes each

were produced in addition to control cubes, and were all cured in water for 28 days in the laboratory atmosphere prior to crushing. The results are presented below in Tables 3.16 to 3.18.

Table 3.16: Compressive Strengths Test Results – CSF (Concrete Cubes)

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)
0	33.7
0.5	34.4
1.0	40.4
1.5	42.7
2.0	48.0
Each value is an average of four tests values, see Appendix 18 for the raw data	

Table 3.17: Compressive Strengths Test Results – RSF (Concrete Cubes)

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)
0	33.7
0.5	34.2
1.0	39.7
1.5	41.9
2.0	44.2
Each value is an average of four tests values, see Appendix 19 for the raw data	

Table 3.18: Compressive Strengths Test Results – CHSF (Concrete Cubes)

Fibre Percentage (%)	Average Compressive Strength(N/mm^2)
0	33.7
0.5	35.3
1.0	36.3
1.5	31.4
2.0	20.4
Each value is an average of four tests values, see Appendix 20 for the raw data	

3.7 Steel Fibre Concrete Tensile Strength

An indirect way of testing for concrete tensile strength was by applying tension in the form of splitting on a cylinder suggested by Fernando Carneiro, a Brazilian. About thirty nine concrete cylinders were produced using the various percentage range of fibres as earlier indicated and were subjected to splitting tests in accordance with BS 1881 Part 116, (1996). The results are as presented in Tables 3.19 to 3.21. Each value in the Table below is an average of three cylinder tests.

Table 3.19: Tensile Strengths Test Results – CSF (Concrete Cylinders)

Fibre Percentage (%)	Average Tensile Strength (N/mm^2)
0	2.94
0.5	3.90
1.0	4.18
1.5	4.41
2.0	4.63
Each value is an average of three tests and the raw data are as presented in Appendix 21	

Table 3.20: Tensile Strengths Test Results – RSF (Concrete Cylinders)

Fibre Percentage (%)	Average Tensile Strength (N/mm^2)
0	2.94
0.5	3.89
1.0	4.11
1.5	4.39
2.0	4.56
Each value is an average of three tests and the raw data are as presented in Appendix 22	

Table 3.21: Tensile Strengths Test Results – CHSF (Concrete Cylinders)

Fibre Percentage (%)	Average Tensile Strength (N/mm^2)
0	2.94
0.5	4.06
1.0	4.58
1.5	4.83
2.0	4.88
Each value is an average of three tests and the raw data are as presented in Appendix 23	

3.6 Steel Fibre Concrete Flexural Test

Similarly as outlined in Section 3.3.2 these tests were carried out in accordance with BS 1881, Part 116, (1996). The mix is one part of cement to two parts of sand to four parts of coarse aggregates (1:2:4) with a water–cement ratio of 0.6. The curing is the same as outlined for mortar beams, immersing in water after demoulding for 28 days prior to test. It is a three-point load test and the flexural strength is given as in Equation 3.3. The results are presented in Tables 3.22 to 3.24.

Table 3.22: Flexural Strengths Test Results

- CSF (Concrete Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.94
0.5	5.10
1.0	4.93
1.5	4.47
2.0	4.27
Each value is an average of three tests and the raw data are as presented in Appendix 24	

Table 3.23: Flexural Strengths Test Results

- RSF (Concrete Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.94
0.5	5.13
1.0	4.97
1.5	5.40
2.0	6.33
Each value is an average of three tests and the raw data are as presented in Appendix 25	

Table 3.24: Flexural Strengths Test Results

- CHSF (Concrete Beams)

Fibre Percentage (%)	Average Flexural Strength (N/mm^2)
0	3.94
0.5	5.47
1.0	5.07
1.5	4.87
2.0	4.50
Each value is an average of three tests and the raw data are as presented in Appendix 26	

3.7 Steel Fibre Concrete Flexural Deflection

The experimental set up for the flexural measurement is shown in Plate 3.2 above. A total number of 39 beams (ie 3 beams for each fibre volume content) measuring 750 mm X 150 mm X 150 mm were cast using wooden moulds. The moulds were oiled before casting to allow for easy removal of specimens. Vibration and compaction was done using a vibrating table. The beams were demoulded within 24hours and cured in water for 28 days before allowing for surface drying. The specimens were tested using a test frame jacking system at the Civil Engineering Department laboratory of Ahmadu Bello University, Zaria.

The deflections were measured using dial gauges. The loads were applied in increments of 5 kN. Throughout the testing, at each interval the formation of crack was observed. The results are as shown in Tables 3.25 to 3.27. The values are average.

Table 3.25: Flexural Deflection Results -CSF (Concrete Mix)

Fibre Percentage (%)	Load (kN) – with Corresponding Deflections												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.053	0.105	0.168	0.237	0.272	0.316	0.368	0.377	-	-	-	-
0.5	0	0.053	0.105	0.237	0.316	0.340	0.352	0.362	-	-	-	-	-
1.0	0	0.053	0.105	0.210	0.263	0.301	0.368	0.400	0.474	-	-	-	-
1.5	0	0.053	0.105	0.210	0.263	0.301	0.368	0.400	0.474	0.053	0.105	0.210	
2.0	0	0.053 1	0.104	0.147	0.210	0.287	0.311	0.342	0.368	0.401	0.502	0.610	0.74 2
Each value is an average of three results and the raw data are as presented in Appendix 27													

Table 3.26: Flexural Deflection Results -RSF (Concrete Mix)

Fibre Percentage (%)	Load (kN) – with Corresponding Deflections												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.053	0.105	0.168	0.237	0.272	0.316	0.368	0.377	-	-	-	-
0.5	0	0.054	0.110	0.236	0.316	0.334	0.355	0.366	-	-	-	-	-
1.0	0	0.055	0.108	0.210	0.252	0.306	0.308	0.413	0.415	-	-	-	-
1.5	0	0.054	0.108	0.238	0.269	0.290	0.329	0.350	0.369	0.500	0.528	0.667	-
2.0	0	0.054	0.112	0.129	0.200	0.291	0.330	0.343	0.389	0.411	0.499	0.580	0.734
Each value is an average of three tests and the raw data are as presented in Appendix 28													

Table 3.27: Flexural Deflection Results -CHSF (Concrete Mix)

Fibre Percentage (%)	Load (kN) – with corresponding deflections												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.053	0.105	0.168	0.237	0.270	0.316	0.368	0.377	-	-	-	-
0.5	0	0.062	0.111	0.178	0.191	0.249	0.309	0.321	0.342	0.423	-	-	-
1.0	0	0.062	0.111	0.178	0.247	0.249	0.326	0.401	0.420	-	-	-	-
1.5	0	0.058	0.060	0.074	0.111	0.154	0.185	0.269	0.278	-	-	-	-
2.0	0	0.052	0.063	0.070	0.099	0.149	0.178	0.259	0.260	-	-	-	-
Each value is an average of three tests and the raw data are as presented in Appendix 29													

3.8 Chips Steel Fibre Concrete Cubes Confirmation Test

In literature, Ghugal (2003) and Stang, Reinhardt and Naaman (2003), it is known that short steel fibre concrete decreases in compressive strength after one percent addition of fibre by volume. From tests done, it only shows significant decreased at one and half percent, 36.3 N/mm^2 at one percent volume dosage and 21.4 N/mm^2 at two percent volume dosage. To confirm these results on chips steel fibre, further tests were carried out with chips steel fibre in accordance with BS 1881 Part 116 (1996). The results are as shown in Table 3.28.

Table 3.28: Confirmation of Chips Steel Fibre Compressive Strength

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)
0	33.7
0.5	35.5
1.0	35.9
1.5	36.2
2.0	30.2
2.5	20.1
3.0	18.5
3.5	Zero

Each average compressive strength is an average of eight cubes. The raw data are shown in Appendix 30.

CHAPTER FOUR

ANALYSIS AND DISCUSSION

The results as presented in chapter three are analysed in this chapter to arrive at reasonable conclusions.

4.1 Sand

The results of the particle size distribution was presented in chapter three, as Table 3.1 and the particle size distribution curve is as shown in Figure 4.1.

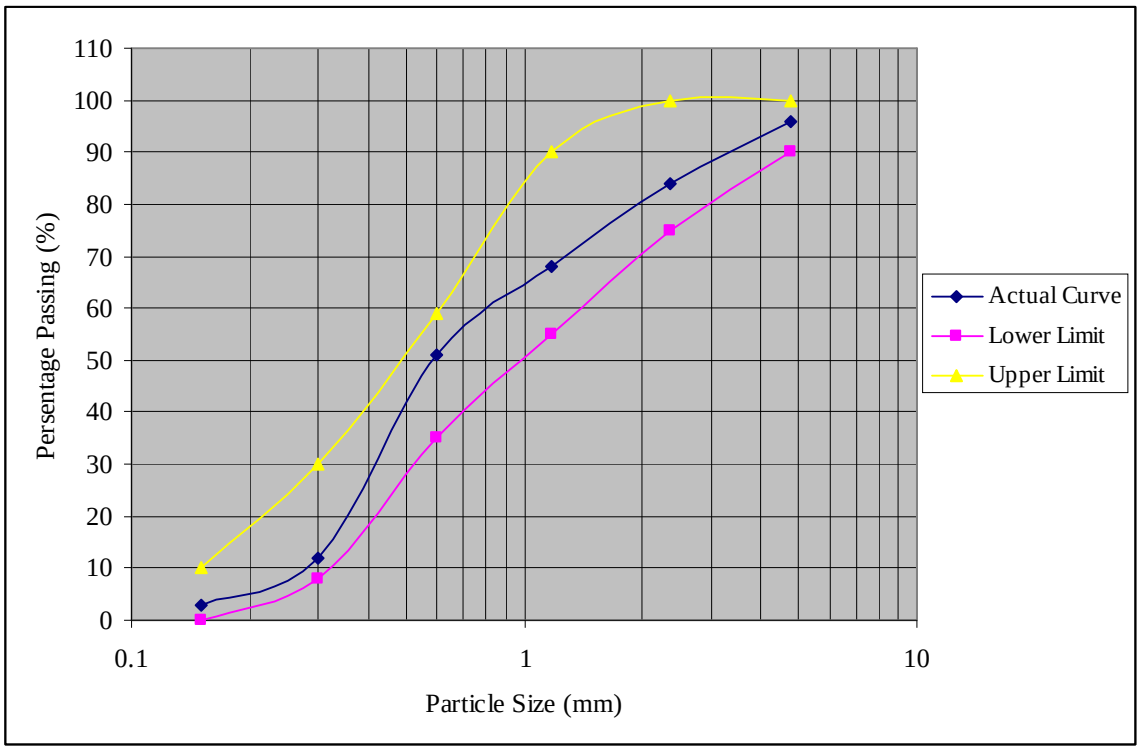


Figure 4.1: Particle Size Distribution (Fine Aggregate)

From the graph the upper limits as well as the lower limits for zone two are indicated with the keys in the graph. This shows that the sand is in zone two (2). Zone one (1) is the coarsest while zone four (4) is the finest. Thus zone two (2) is finer than zone one and can be used for lower water-cement ratio; and for rich mixes. Thus, it is more workable. From the shape of the graph, the fine aggregate is of well graded sand. Thus the sand used in the experiment is satisfactory and could be used for a workable mix.

4.2 Coarse Aggregates

The particle size distribution of the coarse aggregates was presented in Table 3.2 of chapter three and the particle size distribution curve is as shown in Figure 4.2. The grading is in accordance with BS 882: clause 5a (1996).

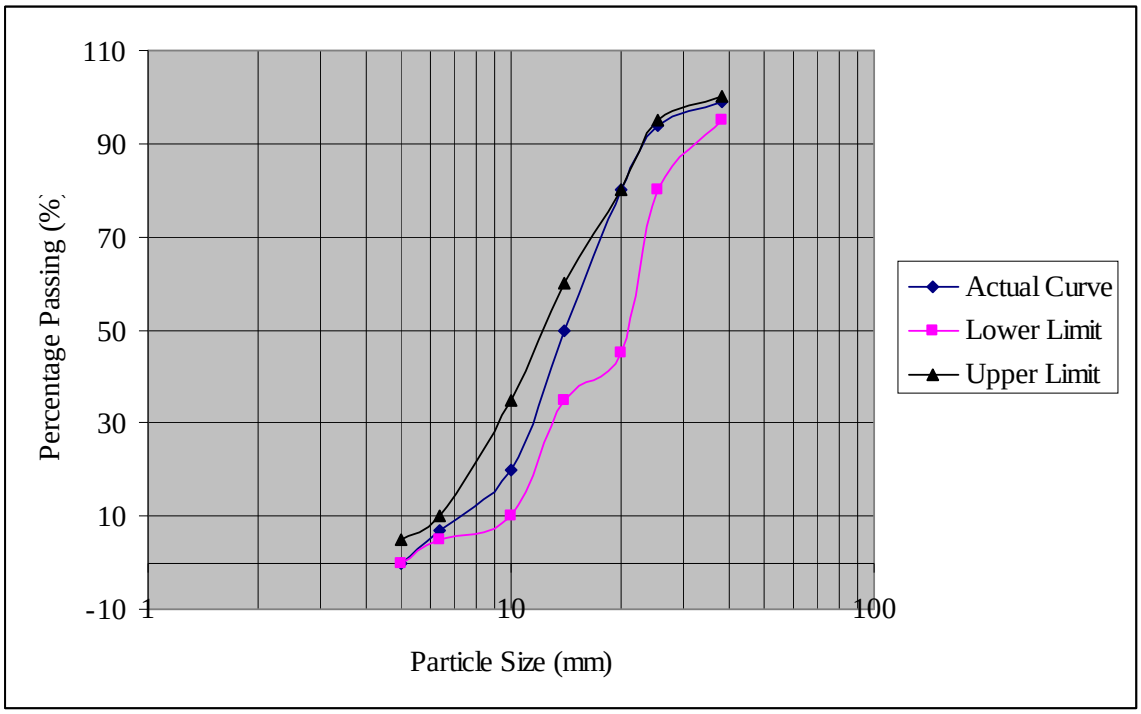


Figure 4.2: Particle size distribution (Coarse Aggregate)

The results show that the size is of nominal size of 40 to 50 mm. The graph shows the upper and lower limits of this size. The maximum nominal size is however 38.5 millimeters. The graph indicates a well graded aggregate. Thus the coarse aggregate is satisfactory.

4.3 Cement

The physical and consistency tests results on Dangote Portland cement used in this experiment are given in Table 3.3 in chapter three. Table 4.1 gives the same results and it is compared with the requirements of BS 12, (1984) and NIS II (1974).

Table 4.1: Comparison of the Properties of Dangote Ordinary Portland Cement with the Requirements of BS 12 (1996)

Property	Value	BS 12 (1996) Requirements and NIS II (1974).
Normal Consistency (%)	34.5	25 – 35
Initial Setting Time (min)	89.0	> 45
Final Setting Time (min)	148.0	< 600
Soundness (mm)	2	< 10
Specific Gravity	3.14	
Cube Compressive Strength (N/mm^2)		
3 – day	14.5	> 14
7 – day	24.2	> 25
28 – day	27.5	> 25

The consistency test met the requirement of the standard. The 3-days and 7-days barely met the standard requirement, while the 28-days was vary okay.

However the results presented when compared with the requirements of BS 12: (1996), Dangote Ordinary Portland Cement can be said to have met the requirements of BS 12 for Ordinary Portland Cement and therefore it is good for concrete making.

4.7 Water

Although, no test was carried out on water specially, it is generally known that the water is portable and everybody drink it in the laboratory, it is pumped from the A.B.U water works and it is well treated for the community for consumption. It is therefore portable water.

4.8 Fibres

The fibres as shown in plate one of chapter three are mainly chipping steel fibre, rectangular steel fibre and circular steel fibre. They are obtained from metal machining; and used tires after burning off the rubber fabric and were well cleaned prior to use in the experiments. They are therefore clean fibre materials.

4.9 Steel Fibre Mortar

The steel fibre mortar test is of two types; that of steel fibre compressive mortar cube tests and that of steel fibre mortar beam flexural tests. The analysis and discussion are presented below.

4.6.1 Steel Fibre Mortar Cube

The results of the steel fibre mortar cubes are as presented in Tables 3.4 to 3.6 of chapter three and the summary of the results are presented in Table 4.2.

Table 4.2: Compressive Strength of Mortar Cubes at Different Fibre Dosage

Fibre Percentage (%)	CSF N/mm^2	RSF N/mm^2	CHSF N/mm^2	Average Strength N/mm^2
0	27.5	27.5	27.5	27.5
0.5	28.2	28.4	28.8	28.5
1.0	29.4	29.7	29.8	29.6
1.5	32.7	30.9	31.2	32.3
2.0	30.0	31.2	30.6	30.6

Observing the results in Table 4.2, it can be seen that there is a gradual increase of strength with respect to increase in fibre percentages in all types of fibres. The strength is the same in all types of fibres except as from one and half percentage of fibre dosage; when different fibres began to manifest different strengths. As from this dosage, the highest strength was 32.7 N/mm^2 for circular steel fibre (CSF); followed by chips steel fibre (CHSF) with strength of 31.2 N/mm^2 and rectangular steel fibre (RSF) with 30.9 N/mm^2. At two percentage fibre dosage, the highest strength was that of rectangular steel fibre, followed by chips steel fibre (CHSF) and circular steel fibre (CSF) having the least strength of the three. The results in Table 4.2 are represented in Figure 4.3 below showing the strength of fibre mortar cubes against the fibre percentage by volume.

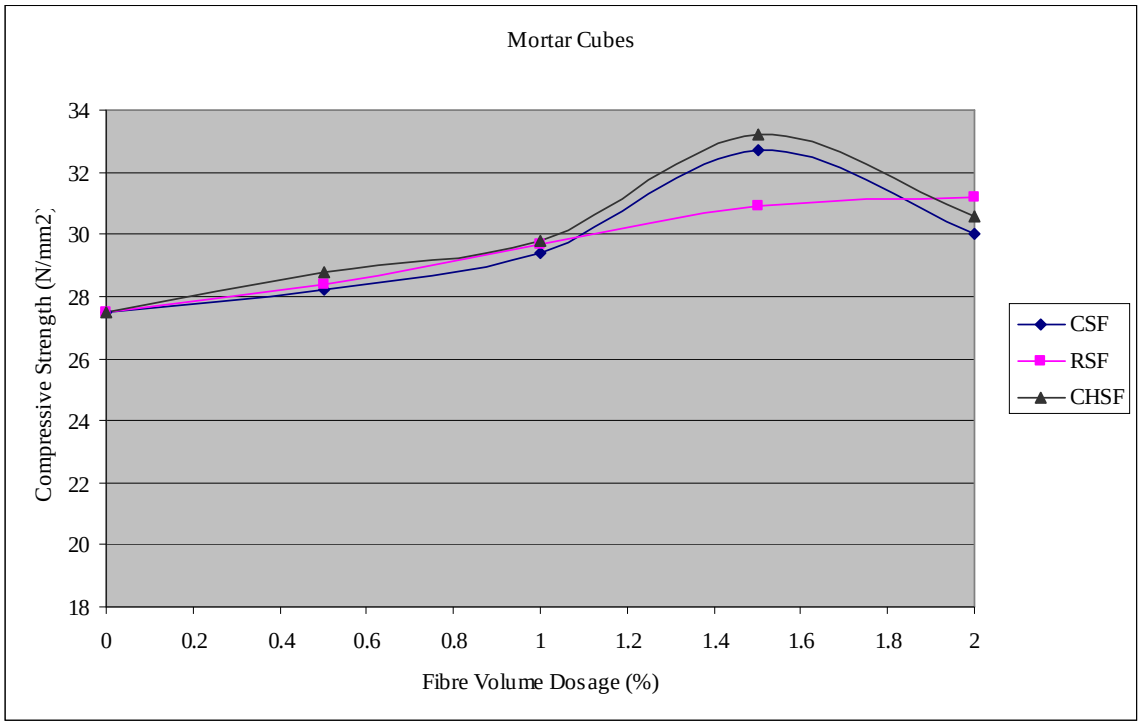


Figure 4.3: Average Compressive Strength of steel Fibre Mortar Cubes Vs Fibre Volume Dosage

Again it can be observed that circular steel fibre cubes showed the highest at one and half percentage of fibre followed by chips steel fibre. It can be observed also that while Circular steel fibre dropped sharply at one and half percentage dosage of fibre; chipps steel fibre also dropped slightly and rectangular continued to increase beyond one and half percentage dosage.

On observing both Table 4.2 and Figure 4.3, the strengths of the fibre mortar cubes can be compared with the control strength (zero fibre percentage), and the percentage increase over the control strength and over the range of fibre percentage dosage are obtained. This is shown in Table 4.3.

Table 4.3: Fibre Mortar Cubes Compressive Strengths Increase over Control- Mortar cubes

Fibre Percentage (%)	Compressive Strength Increase (N/mm^2)/ Percentage Increase (%)					
	CSF		RSF		CHSF	
	Strength Increase (N/mm^2)	Percentage Increase (%)	Strength Increase (N/mm^2)	Percentage Increase (%)	Strength Increase (N/mm^2)	Percentage Increase (%)
0	0	-	0	-	0	-
0.5	0.7	2.5	0.9	3.3	1.3	4.7
1.0	1.9	6.9	2.2	8.0	2.3	8.4
1.5	5.5	18.9	3.4	12.4	5.7	13.5
2.0	2.5	9.1	3.7	13.5	3.1	11.3

The Table above is also represented in a graph as shown in Figure 4.4 below with respect to percentage increase.

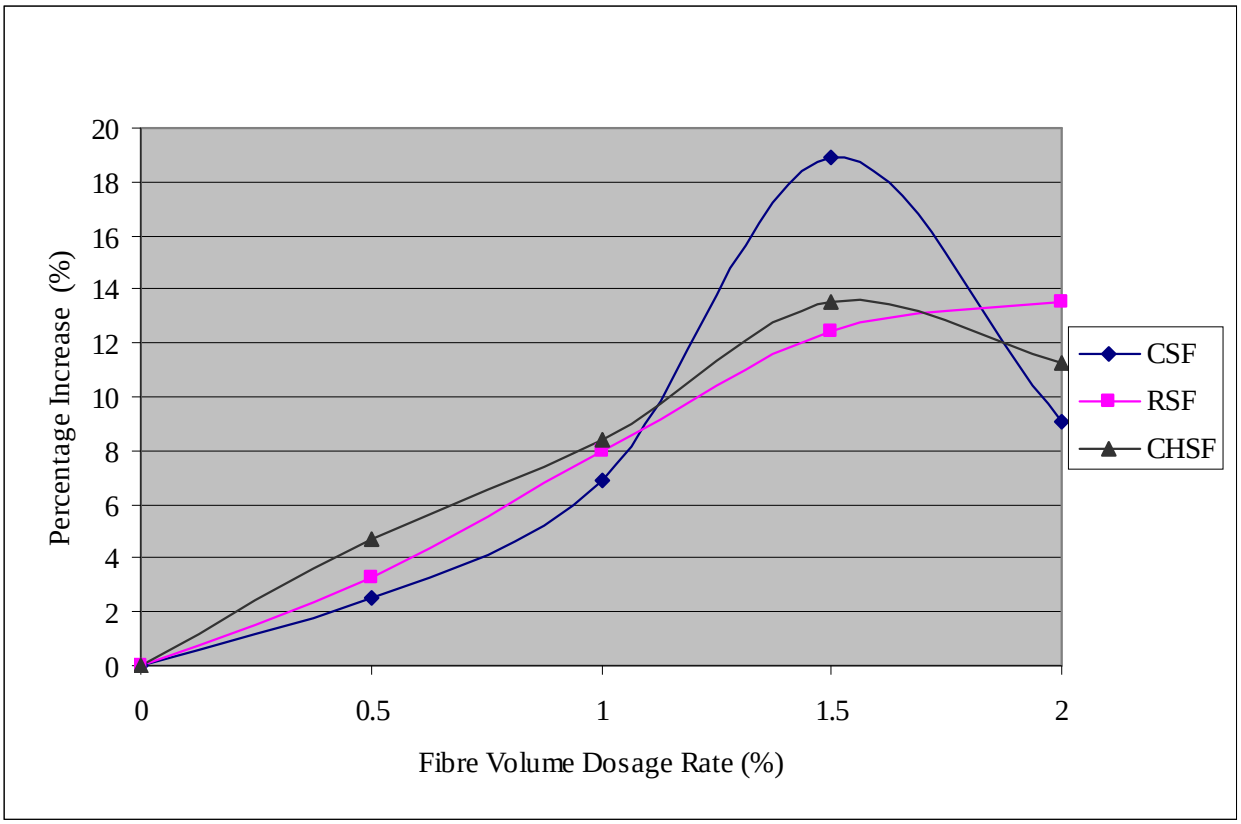


Figure 4.4: Percentage Increase of compressive strength over control mix Fibre Mortar

On observing the figure there is again a sharp drop at less than one and half percentage dosage for circular steel fibre and chips steel fibre. That of circular steel fibre drop is sharper than that of chips steel fibre. That of rectangular steel fibre is seen to continue to increase in this case. It is worth noting that change occurred at one and half percentage dosage for circular steel fibre and chips steel fibre. This could be due to the apparent reduction in workability of the fibre reinforced mortar after one and half percentage dosage for all the fibres. This case is normal with short steel fibre reinforced mortar, Balaguru and Shah (1992) and Luo et. al (2001)

The above discussions are represented in the form of bar chart shown in Figure 4.5.

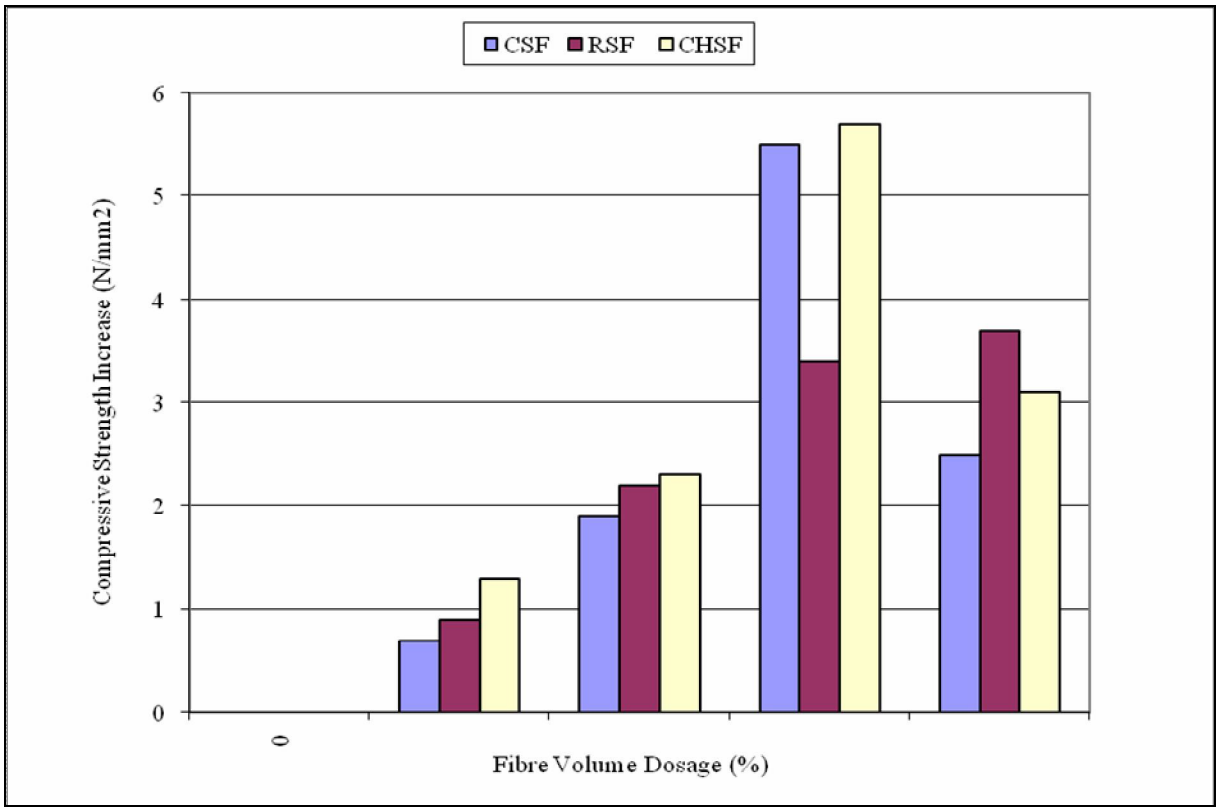


Figure 4.5: Compressive Strength Increase over Control at Different Fibre Volume Dosage

The highest increase in compressive strength occurred at one and half percentage dosage for CSF and CHSF.

The decrease in strength by chips steel fibre may be due to the decrease in workability as the fibre content increases and according to work by Luo et al (2001). The compressive strength of high-performance cement base materials only increased by about forty percent over control at a steel fibre content of between one-half percentage to one and half percentage of fibre volume dosages. This is shown by the results of circular, rectangular and chips steel fibre mortar cube strengths.

4.6.2 Steel Fibre Mortar Beam Flexural Strength

The results of the tests performed for the flexural strength on mortar beams for the three types of fibres were shown in Table 3.7 to 3.9 in chapter three. The summary of the results is presented in Table 4.4 below for proper comparism.

Table 4.4: Comparism of Flexural Strength of Fibre Mortar Beam

Fibre Percentage (%)	Flexural Strength of Fibre Mortar Beams (N/mm^2)			
	CSF	RSF	CHSF	Average
0	3.55	3.55	3.55	3.55
0.5	4.60	4.55	4.92	4.69
1.0	4.75	4.90	5.10	4.92
1.5	4.95	5.05	5.30	5.10
2.0	5.25	5.20	5.30	5.25

Table 4.4 shows that, there is a gradual increase in flexural strength as the fibre content increases in all the three types of fibres. However, the flexural strength tends to stabilize as from one and half percentage dosage of fibre as the value is almost at 5 N/mm^2 in all the fibres. Table 4.4 above is shown in Figure 4.6 below for visual comparism and analysis.

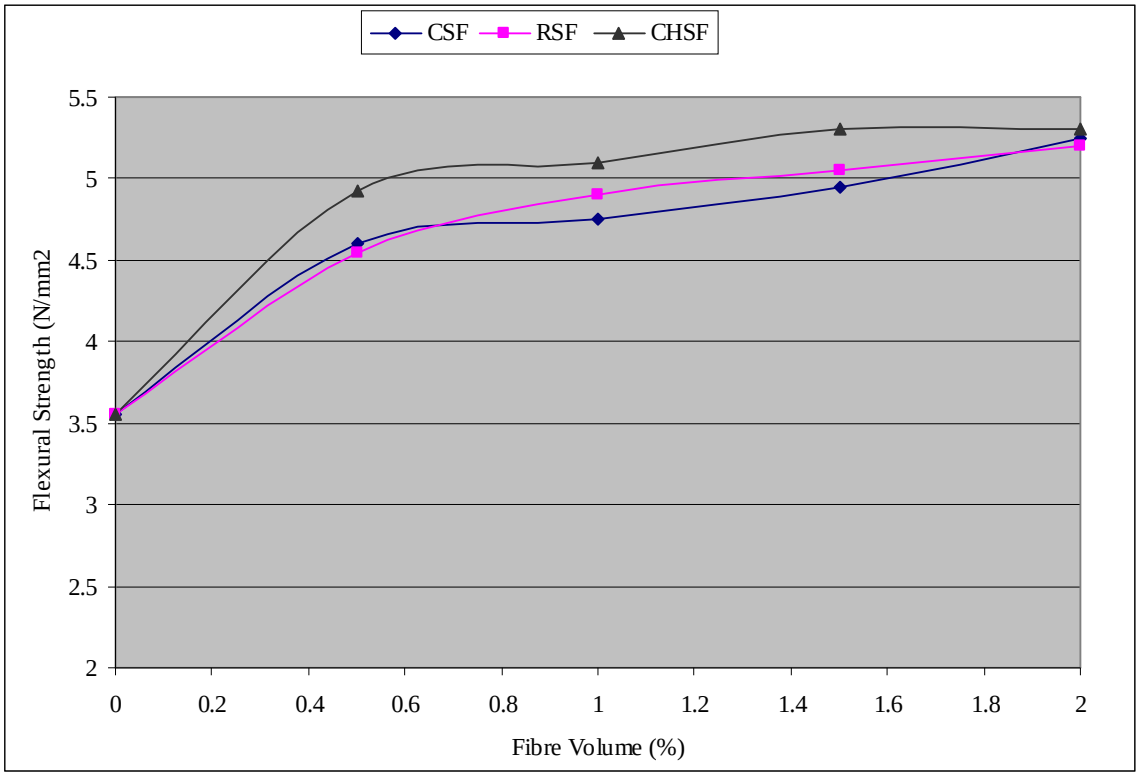


Figure 4.6: Flexural Strength against Fibre volume Dosage (Mortar Beams)

Similarly, as outlined in Section 4.6.1 above, the values of flexural strength are compared with control beam (zero fibre content beam) and the differences are observed as in Table 4.5.

Table 4.5: Increase in Flexural Strength at Different Fibre volume Dosage

Fibre Percentage (%)	Flexural Strength Difference (N/mm^2)/ Percentage Difference (%)					
	CSF		RSF		CHSF	
	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase
0	0	-	0	-	0	-
0.5	1.05	29.6	1.00	28.2	1.37	38.6
1.0	1.20	33.8	1.35	38.0	1.55	43.7
1.5	1.40	39.4	1.50	42.2	1.75	49.3
2.0	1.70	48.0	1.65	46.5	1.75	49.3

The table above is also represented in bar chart and graphs as shown in Figures 4.7 and 4.8 below with respect to flexural strength increase and percentage increase

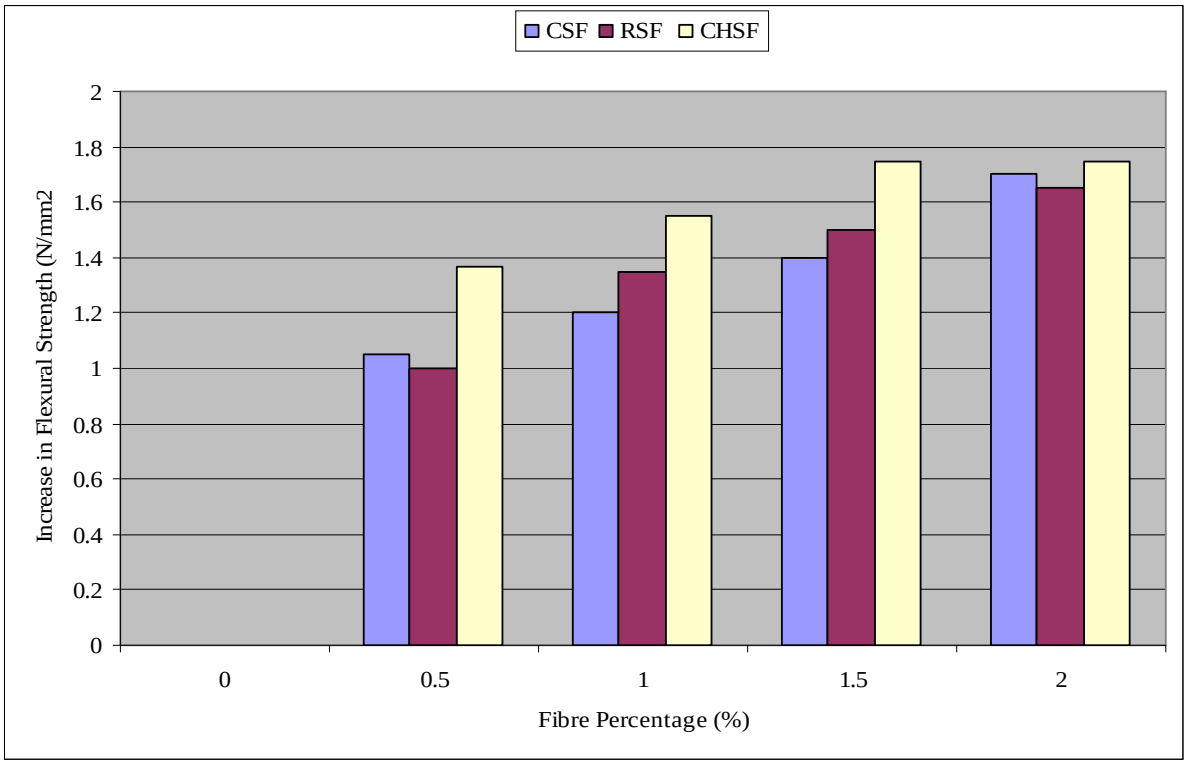


Figure 4.7: Flexural Strength Increase over Control at Different Fibre Volume Dosage

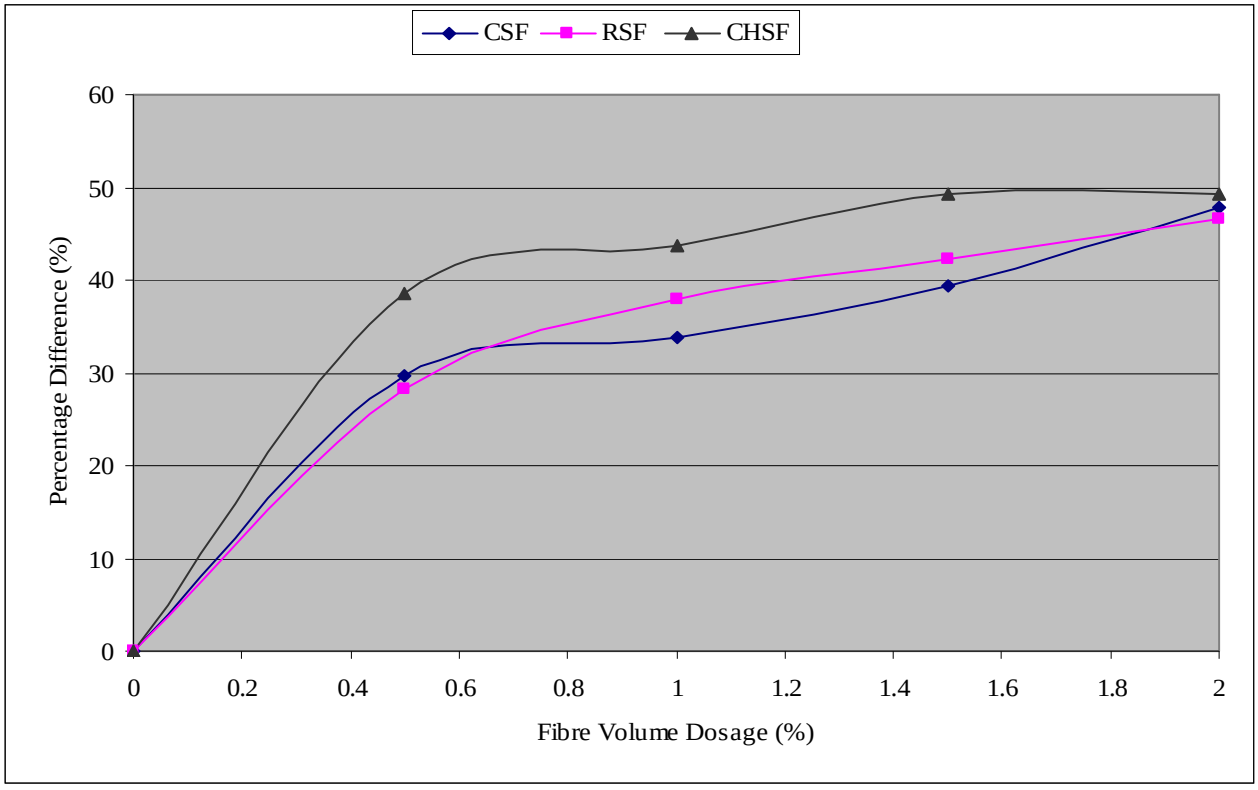


Figure 4.8: Percentage Increase of Flexural strength over to control mix Fibre Mortar

A study of Table 4.5 and Figures 4.7 and 4.8 show that chips steel fibre has the highest increase in flexural strength when compared with the other two fibre reinforced mortar beams at all the fibre volume dosage investigated. There is an appreciable increase of the flexural strength of chips steel fibre at one-half percentage of fibre dosage. This increase decreases with an increase in the fibre volume dosage when Figure 4.7 is considered. They tend to be equal at high fibre dosage.

4.7 Workability of Steel Fibre Concrete

4.7.1 Slump Test

The results of the slump test performed on concrete mixes for the three fibres are as presented in Tables 3.10 to 3.12 in chapter three. A summary of the results are presented in Table 4.6 for comparism.

Table 4.6: Comparism of Slump Test Results for Fibre Concrete

Fibre Percentage (%)	Slump test Results for all Fibre Concrete (mm)			
	CSF	RSF	CHSF	Average
0	166	166	166	166
0.5	160	159	120	146
1.0	149	157	102	136
1.5	140	130	84	118
2.0	126	120	60	102

From Table 4.6, it can be observed that there is a continuous decrease in slump with an increase in percentage of fibre. The decrease is more pronounced in the case of chips steel fibre where at two percent fibre dosage, the value of slump for chips steel fibre is one-half

of that of circular and rectangular steel fibres of the same dosage. The decrease in slump as the percentages increases can be attributed to the mix being stiff as more fibre is added. Secondly the sharp decrease in chips steel fibre can be said to be as a result of geometry (wave cut geometry and tough-like edges).

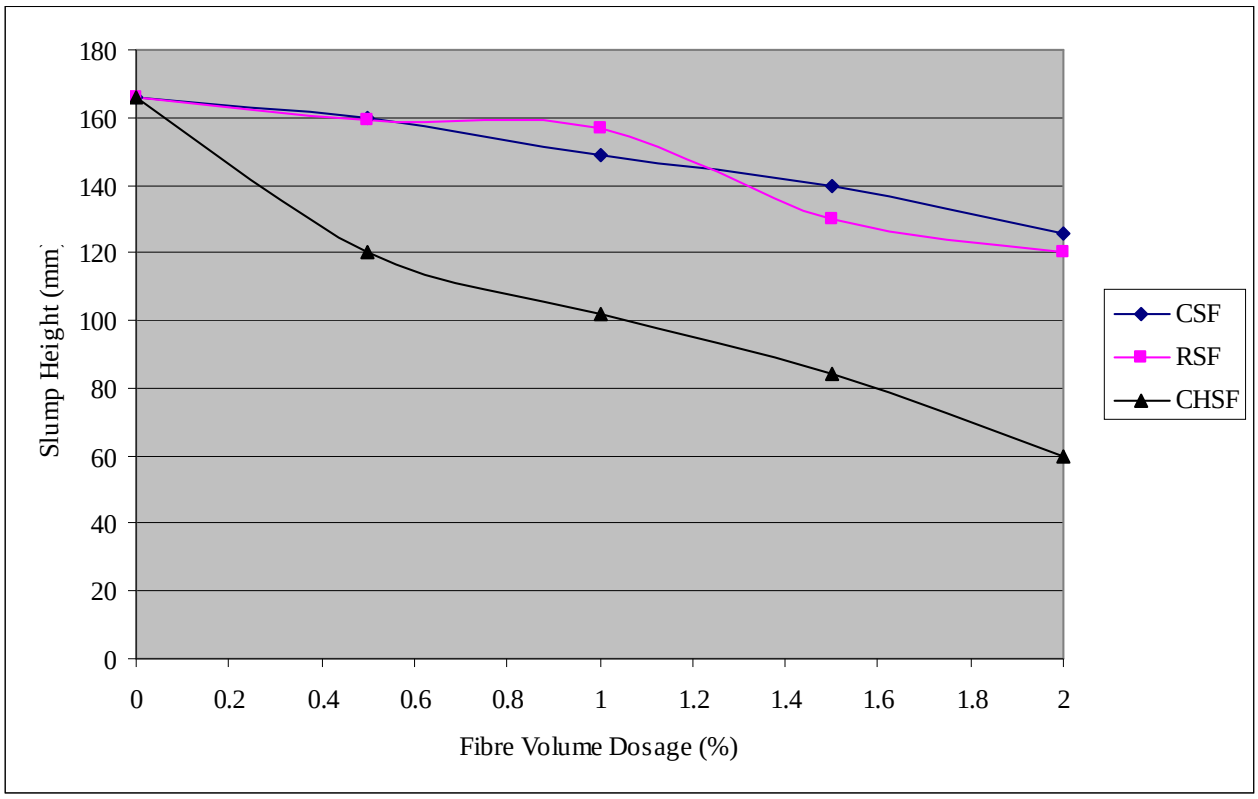


Figure 4.9: Average Slump Height vs. Fibre Volume Dosage for all Fibres

Table 4.6 is represented in graph as shown in Figure 4.9 above. The same trend as discussed above is displayed in the figure.

Considering the slump at zero fibre percentage as a base, the difference in slumps for the various fibres are obtained as shown in Table 4.7.

Table 4.7: Percentage decrease in slump of Steel Fibre Concrete

Fibre Percentage (%)	CSF		RSF		CHSF	
	Decrease in Slump (mm)	Percentage Decrease (%)	Decrease in Slump (mm)	Percentage Decrease (%)	Decrease in Slump (mm)	Percentage Decrease (%)
0.5	-6	4	-7	4	-46	28
1.0	-17	10	-9	5	-64	39
1.5	-26	16	-36	22	-82	49
2.0	-40	24	-46	28	-106	64

Observing carefully Table 4.7, the highest percentage decrease in slump is 24 percent for circular steel fibre, 28 percent for rectangular steel fibre, 64 percent for chips steel fibre. Obviously, it is easier for a concrete to be more workable with circular cross-section, followed by rectangular cross-section and certainly more difficult with wave-cut, tough-like end section. Thus it is an issue of geometry in the above case. Thus, the issue of decrease in slump is as a result of stiff mix of concrete and geometry of the steel fibre. Table 4.7 is represented in a graph as Figure 4.10.

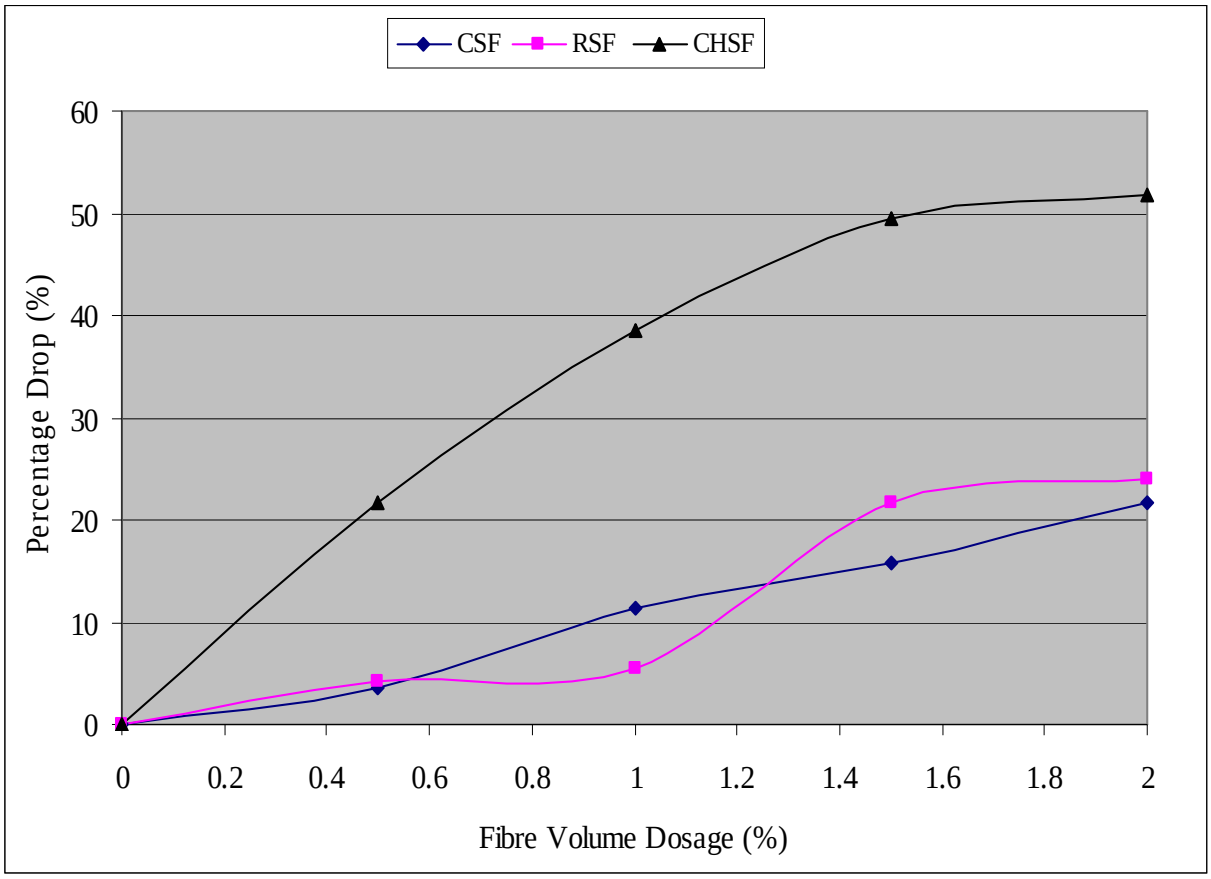


Figure 4.10: Percentage Difference of Slump Height vs. Fibre Volume Dosage

The slumps obtained from all the fibre concrete mix were considered satisfactory; since the values fall within medium to high degree of workability (50 mm to 175 mm) of Neville, (1997). Judging from the difference in slump; it is clear that workability of concrete with fibre is less than that without fibres and if fibre volume dosage increased, a lower workability of concrete would occur.

4.7.2 Compacting Factor Test

The results of the Compacting factor test performed on concrete mixes for the three fibres are as presented in Tables 3.13 to 3.15 in chapter three. A summary of the results is presented in Table 4.8 below for comparism.

Table 4.8: Comparism of Compacting Factor Test Results for Fibre Concrete

Fibre Percentage (%)	Compacting factor test results for all Fibre Concrete		
	CSF	RSF	CHSF
0	0.995	0.995	0.995
0.5	0.969	0.984	0.901
1.0	0.940	0.980	0.894
1.5	0.910	0.940	0.829
2.0	0.891	0.892	0.801

Table 4.8 shows that there is a general decrease in compacting factor value when fibre dosage increased. The general decrease with increase in fibre percentage tends to be uniform with all the type of fibres. To confirm this, the zero fibre – control result is used as a base and the general decrease in percentage are compared as shown in table 38. Table 4.8 is shown in Figure 4.11.

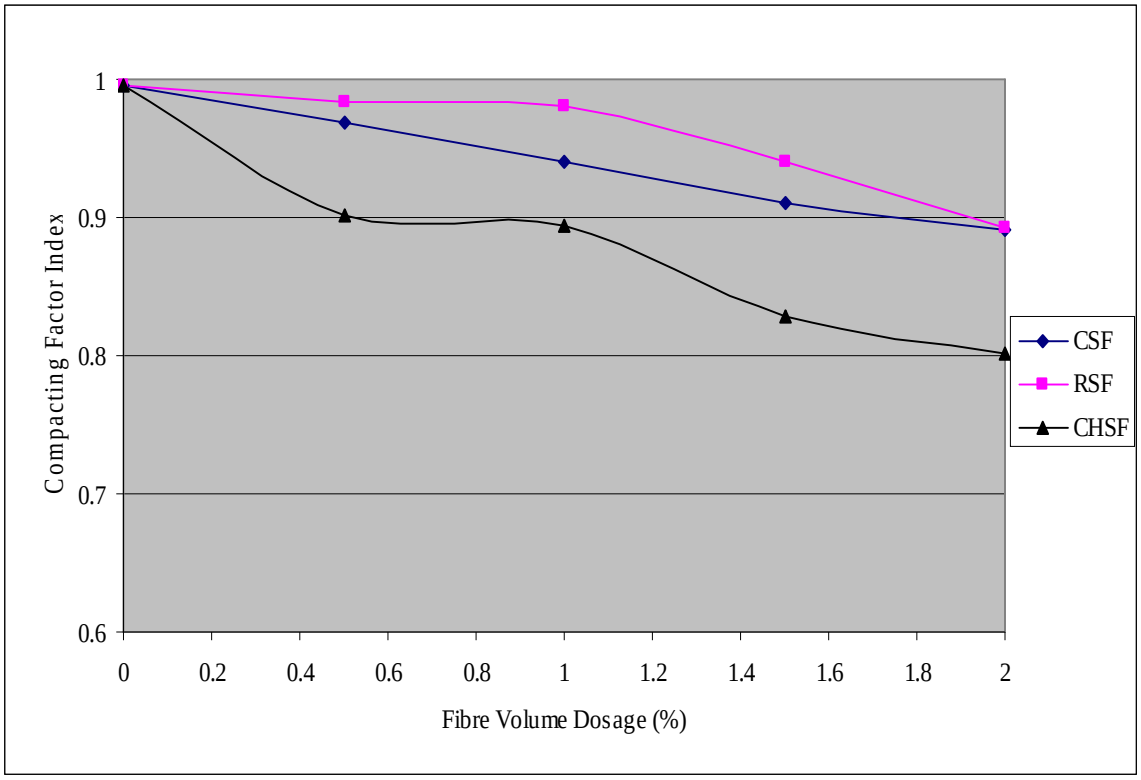


Figure 4.11: Average compacting factor Vs. Fibre volume dosage for all fibres

From Figure 4.11, it can be seen that the decrease in compacting factor is more pronounced in chips steel fibre. Rectangular steel fibre even though less produced than chips steel fibre, it is more than circular steel fibre. This is following the slump workability trend and the same reason can be given for the values of compacting factor.

Table 4.9: Compacting Factor Decrease from Control Value

Fibre Percentage (%)	CSF		RSF		CHSF	
	Decrease from Control Value	Percentage Decrease (%)	Decrease from Control Value	Percentage Decrease (%)	Decrease from Control Value	Percentage Decrease (%)
0.5	-0.026	3	-0.011	1	-0.094	9
1.0	-0.055	6	-0.015	2	-0.101	10
1.5	-0.085	9	-0.055	6	-0.166	12
2.0	-0.104	10	-0.103	10	-0.194	19

From observation of Table 4.9, the circular and rectangular steel fibre concretes have decrease in compacting factor to a maximum of ten percent. This can be said to be almost negligible. However, the maximum for chips steel fibre concrete is in the tune of 20 percent. The same reason give earlier can be attributed to this behaviour. Table 38 is represented in a graph as Figure 4.12.

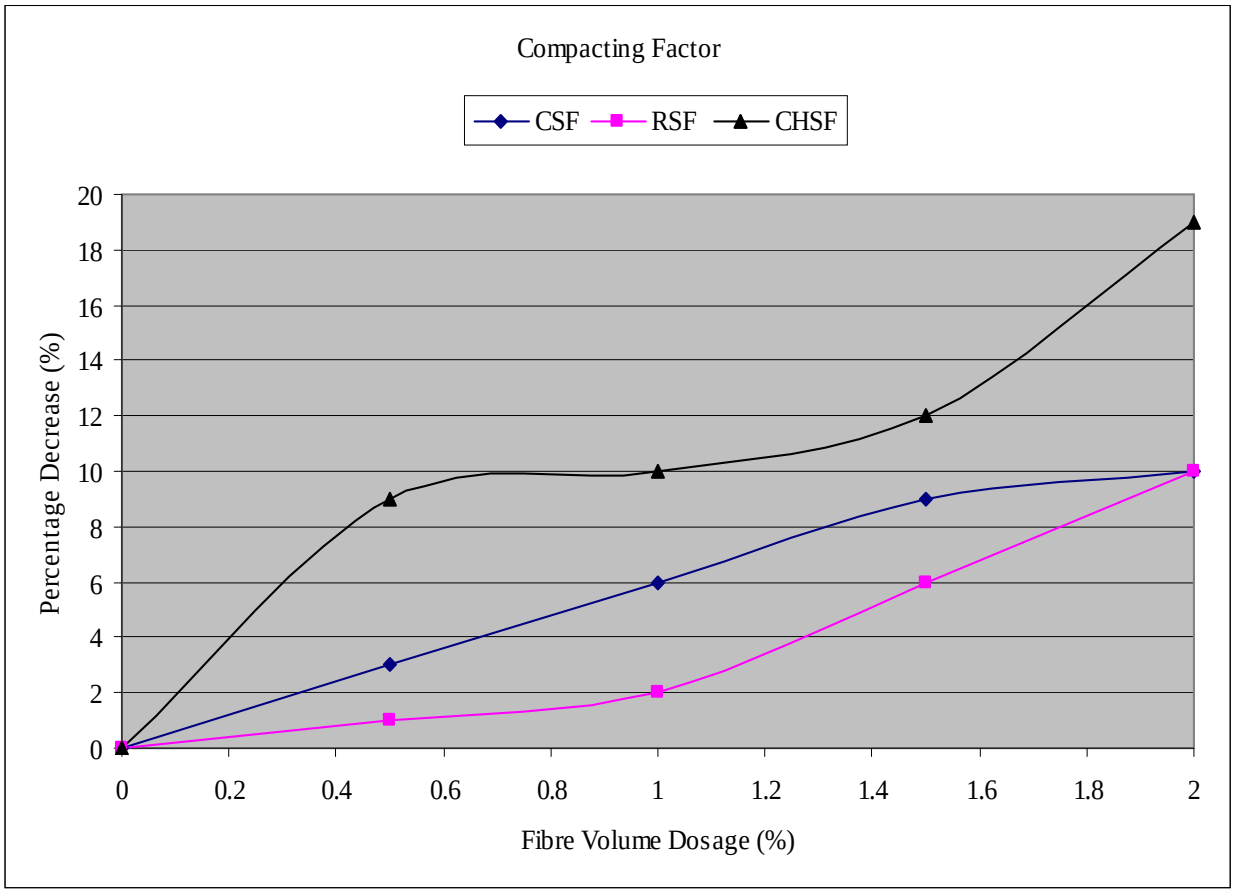


Figure 4.12: Percentage Difference of Compacting Factor vs. Fibre Volume Dosage.

4.8 Steel Fibre Concrete Cube

The results of the steel fibre concrete cubes are as presented in Tables 3.16 to 3.18 of chapter three and the summary of the results are presented in Table 4.10.

Table 4.10: Compressive Strength of Concrete Cubes at Different Fibre Dosage

Fibre Percentage (%)	CSF N/mm^2	RSF N/mm^2	CHSF N/mm^2	Average Strength N/mm^2
0	33.7	33.7	33.7	33.7
0.5	34.4	34.2	35.3	34.6
1.0	40.4	39.7	36.3	38.8
1.5	42.7	41.9	31.4	38.7
2.0	48.0	44.2	20.4	37.5

Observing Table 4.10 carefully, it shows that inclusion of fiber into concrete has influence on the compressive strength of concrete. Figure 4.13 gives the shape of Table 4.10.

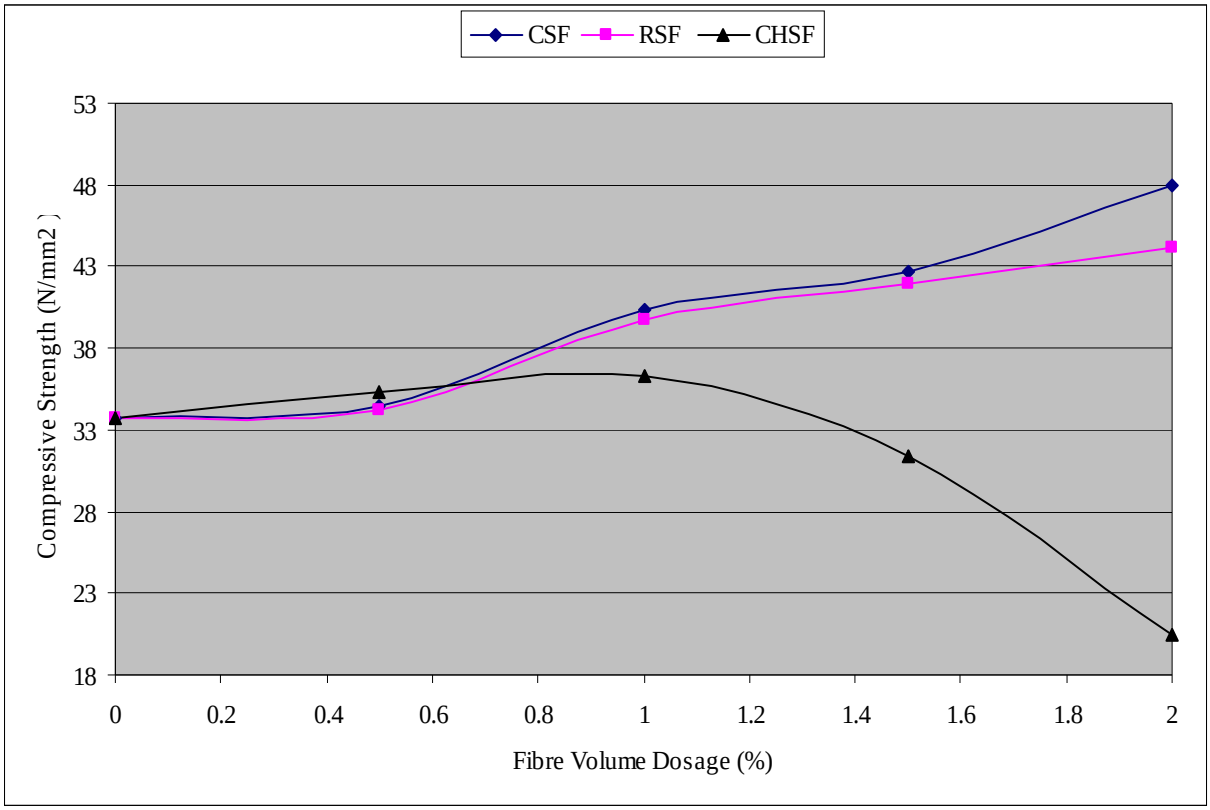


Figure 4.13: Average Compressive Strength vs. Fibre Volume Dosage for all Fibres

A careful observation of the table and figure shows that there is no significant increase in strength at one-half percent volume dosage. All the three fibres have the same strength. At one percent fiber dosage there was increase in strength by circular and rectangular fiber which continued up to two percent increase but chips steel fiber started to decrease at one percent fiber volume dosage. The decrease by chips steel fiber is very sharp and this may be due to the apparent decrease in workability because of the geometry and shape of chips steel fiber.

Table 4.11 shows the strength increase with the coresponding percentage increase over control at different fiber percentage investigated.

Table 4.11: Compressive Strengths Increase over Control- (Concrete cubes)

Fibre Percentage (%)	Compressive Strength Increase (N/mm^2)/ Percentage Increase (%)					
	CSF		RSF		CHSF	
	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase	Strength Difference	Percentage Increase
0	0	-	0	-	0	-
0.5	0.7	2.1	0.5	1.5	1.6	4.8
1.0	6.7	19.9	6.0	17.8	2.6	7.7
1.5	9.0	26.7	8.2	24.3	-2.3	-6.8
2.0	14.3	42.4	10.5	31.2	-13.3	-39.5

For the three fibres the one-half percent and one percent volume dosage are good for structural members in compression.

Again the increase in compressive strength of fibre concrete cube over control cubes are uniform between one-half to one percent fibre dosage of values ranging between 3 – 4 percent for all the three types of fibres. As from one and half to two percentage fibre dosage, each fibre start manifesting different behaviour with circular steel fibre and chips steel fibre having their percentage dropped at two percentage fibre dosage, same reason can be attributed to this.

4.9 Steel Fibre Concrete Cylinder Split Tests [Tensile]

The results of the steel fibre concrete cylinder split tests are as presented in Tables 3.25 to 3.27 of chapter three and the summary of the results are presented in Table 4.12 for comparison purposes.

Table 4.12: Tensile Strength of Concrete Cylinder at Different Fibre Dosage

Fibre Percentage (%)	CSF N/mm^2	RSF N/mm^2	CHSF N/mm^2	Average Strength N/mm^2
0	2.94	2.94	2.94	2.94
0.5	3.90	3.89	4.06	3.95
1.0	4.18	4.11	4.58	4.29
1.5	4.41	4.39	4.83	4.54
2.0	4.63	4.56	4.88	4.69

Table 4.12 above is represented in Figure 4.14 below for visual comparism and analysis.

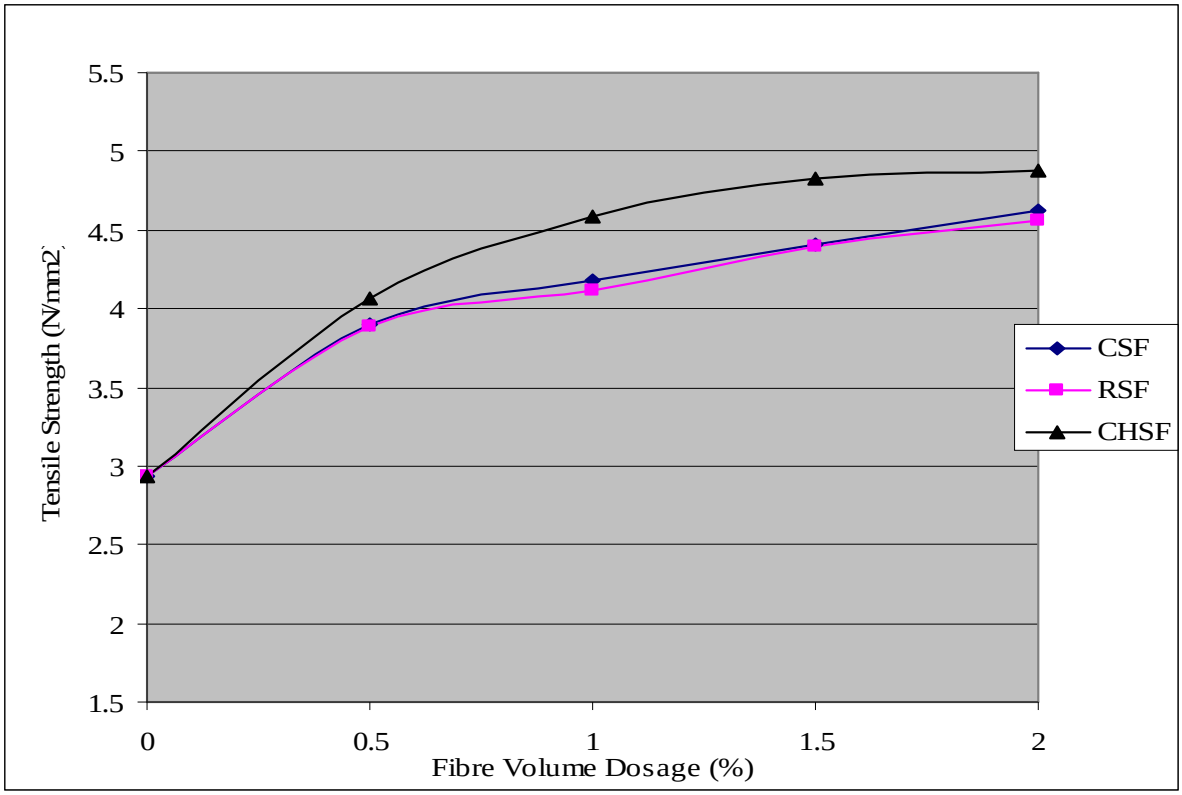


Figure 4.14: Average Tensile Strength vs. Fibre Volume Dosage for all Fibres

From the Table 4.12 and Figure 4.14 above, there is a gradual increase and improvement in the tensile strength of steel fibre concrete with increase in steel fibre dosage. However, the increase between one-half to one and half fibre percentage dosage is gradual and not much, though at two percent there is a slight increase or improvement on the former gradual increase between one-half and one and half dosage. Thus, steel fibre in concrete improves the tensile strength of concrete.

Using the control tensile strength as a base, the difference between the various dosage of steel fibre with their corresponding tensile strength are compared and is as shown in Table 4.13.

Table 4.13: Increase in Tensile Strength at Different Fibre Volume Dosage

Fibre Percentage (%)	Tensile Strength Increase (N/mm^2)/ Percentage Difference (%)					
	CSF		RSF		CHSF	
	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase
0	0	-	0	-	0	-
0.5	0.96	32.7	0.95	32.3	1.12	38.1
1.0	1.24	42.2	1.17	39.8	1.64	55.8
1.5	1.47	50.0	1.45	49.3	1.89	64.3
2.0	1.69	57.5	1.62	55.1	1.94	65.9

The table above (Table 4.13) is also represented in graph as shown in Figure 4.15 and a bar chat-graph as presented in Figure 4.16 for the same table.

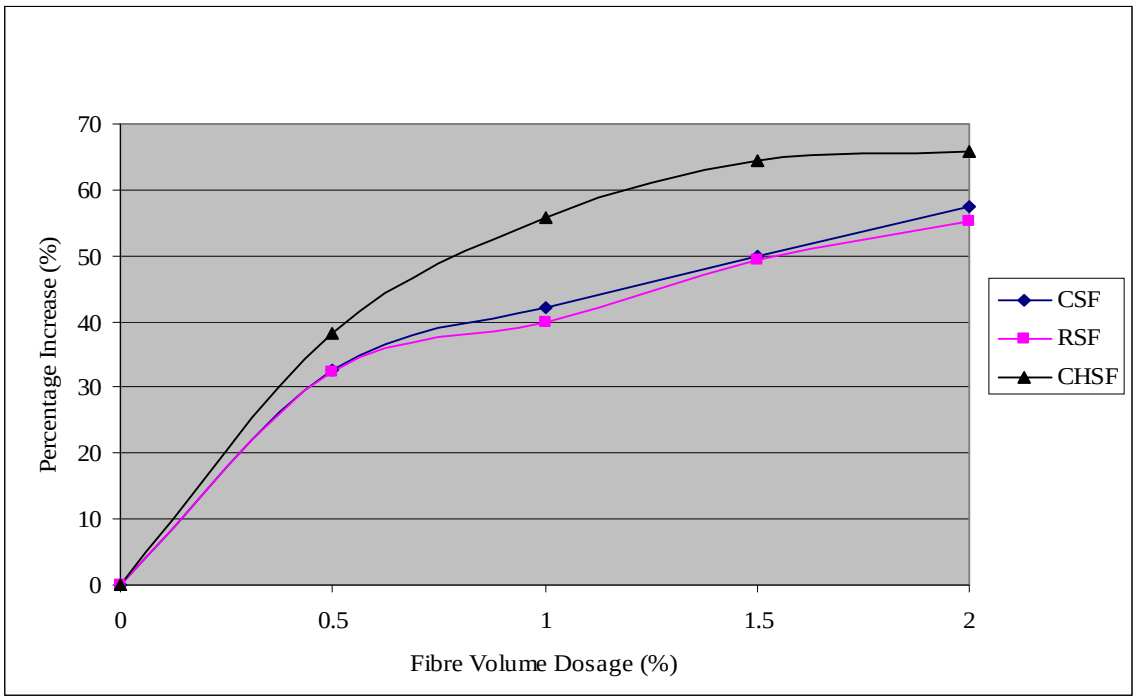


Figure 4.15: Percentage Difference of tensile strength vs. fibre volume dosage

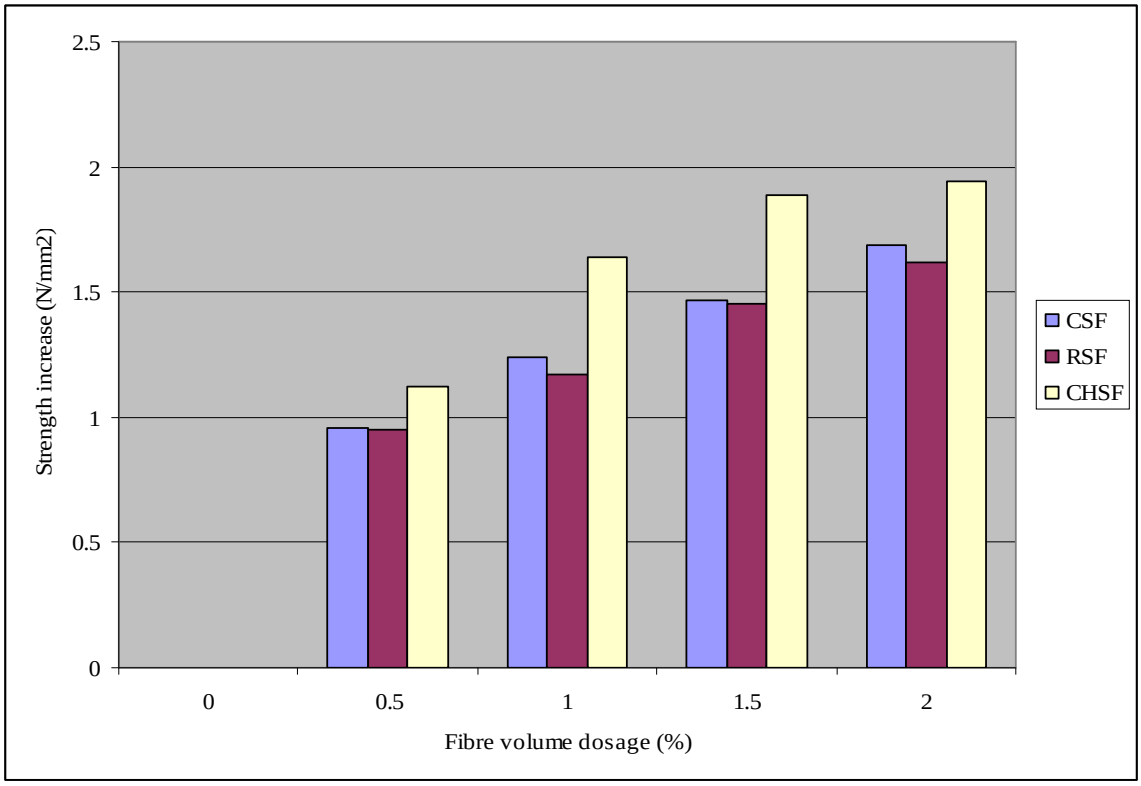


Figure 4.16: Tensile Strength Increase over Control at Different Fibre Volume Dosage

From Table 4.13, Figure 4.15 and the bar chart of Figure 4.16, the chips steel fibre has higher tensile strength when compared to the circular and rectangular steel fibres which are almost the same value in tensile strength. The chips steel fibres are twisted and are more flexural in nature. Thus, it acts as reinforcement more than the circular/rectangular steel fibres which may be more brittle.

At failure, the none fibre cylinder split without warning while that of chips steel fibre cylinders even though they have failed, are still together as one unit and only showed line crack at the point of failure. This is shown in Plate 4.1. Thus, failure of steel fibre concrete gives warning prior to failure but that of non-fibre cylinder gives no warning.



Plate 4.1: Photograph of Concrete specimens at failure showing Specimens with and without Fibres

4.10 Steel Fibre Concrete Beam Flexural Strength

The results of the tests performed for the flexural strength on concrete beams for the three types of fibres are presented in Tables 3.22 to 3.24 in chapter three. The summary of the results is presented in Table 4.14.

Table 4.14: Comparism of Flexural Strength of Fibre Concrete Beam

Fibre Percentage (%)	Flexural Strength of Fibre Concrete Beams (N/mm^2)			
	CSF	RSF	CHSF	Average
0	3.94	3.94	3.94	3.94
0.5	5.10	5.13	5.47	5.23
1.0	4.93	4.97	5.07	4.99
1.5	4.47	5.40	4.87	4.91
2.0	4.27	6.33	4.50	5.03

Table 4.14 shows that, there is a general increase in flexural strength when fibres were added to concrete. Table 4.14 above is also presented as a graph as shown in Figure 4.17.

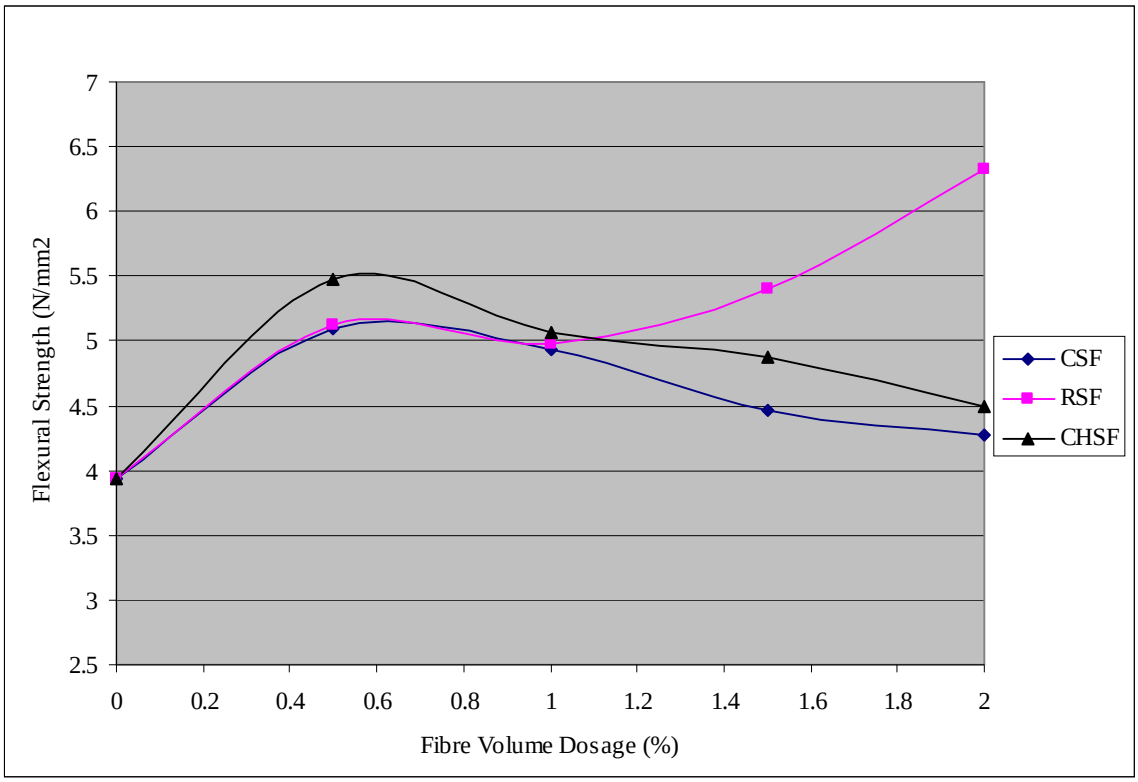


Figure 4.17: Flexural Strength of concrete against Fibre volume Dosage

From Table 4.14 and Figure 4.17, there is flexural strength increase at one - half percent fibre volume by all the fibres investigated, after one - half percentage volume addition, circular and chip steel fibres reinforced concretes recorded decreasing trends in the flexural strength as the fibre volume dosage increases through to two percent fibre volume dosage, and the values are very close to each other. Rectangular steel fibre recorded increasing trend. These observed trends may be due to the fact that workability of concrete decreases more rapidly as the fibre content increases which may lead to inadequate compaction and possible decrease in strength or may be due to the geometry of the sections involved.

The values of flexural strength are compared with control beam (zero fibre content beams) and the differences are observed as in Table 4.15.

Table 4.15: Increase in Flexural Strength at Different Fibre volume Dosage

Fibre Percentage (%)	Flexural Strength Increase (N/mm^2)/ Percentage Increase (%)					
	CSF		RSF		CHSF	
	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase	Strength Increase	Percentage Increase
0.5	1.16	29.4	1.19	30.2	1.53	38.8
1.0	0.99	25.1	1.03	26.1	1.13	28.7
1.5	0.53	13.5	1.46	37.1	0.93	23.6
2.0	0.33	8.3	2.39	60.7	0.56	14.2

Table above is also represented in graph and bar chart as shown in Figures 4.18 and 4.19 below with respect to flexural strength increase and percentage increase.

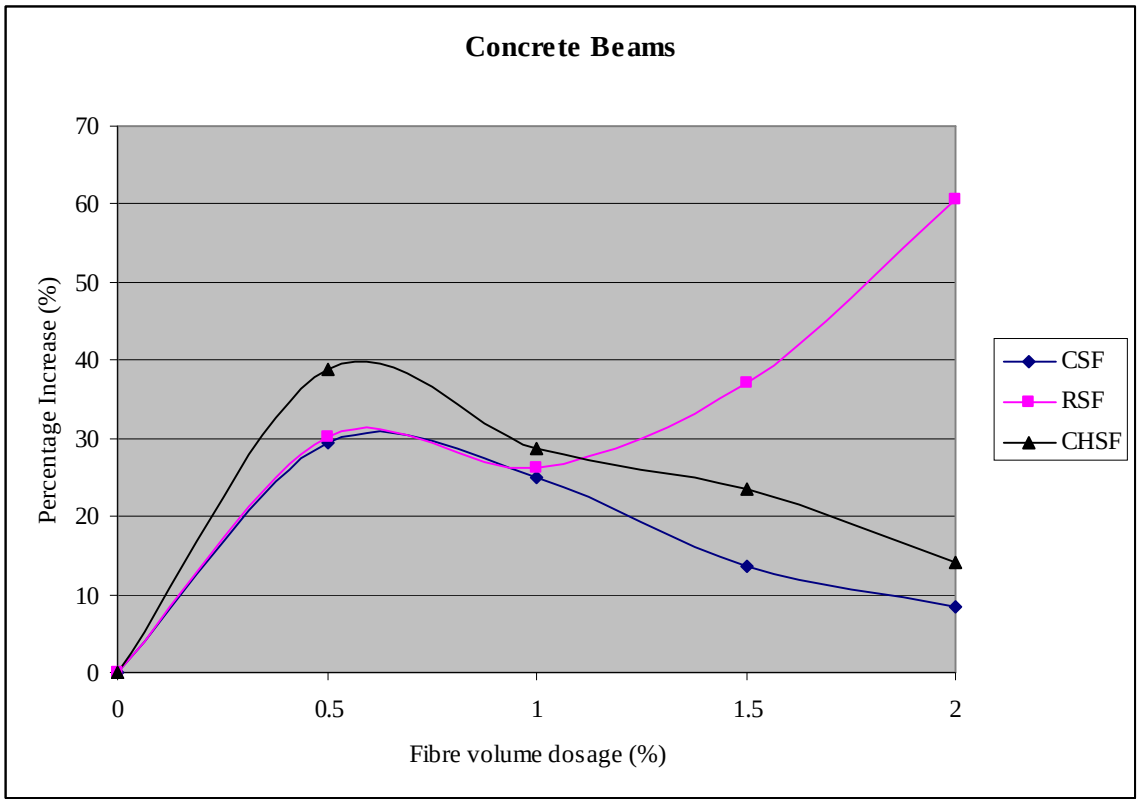


Figure 4.18: Percentage Increase of Flexural strength over to control mix Fibre concrete

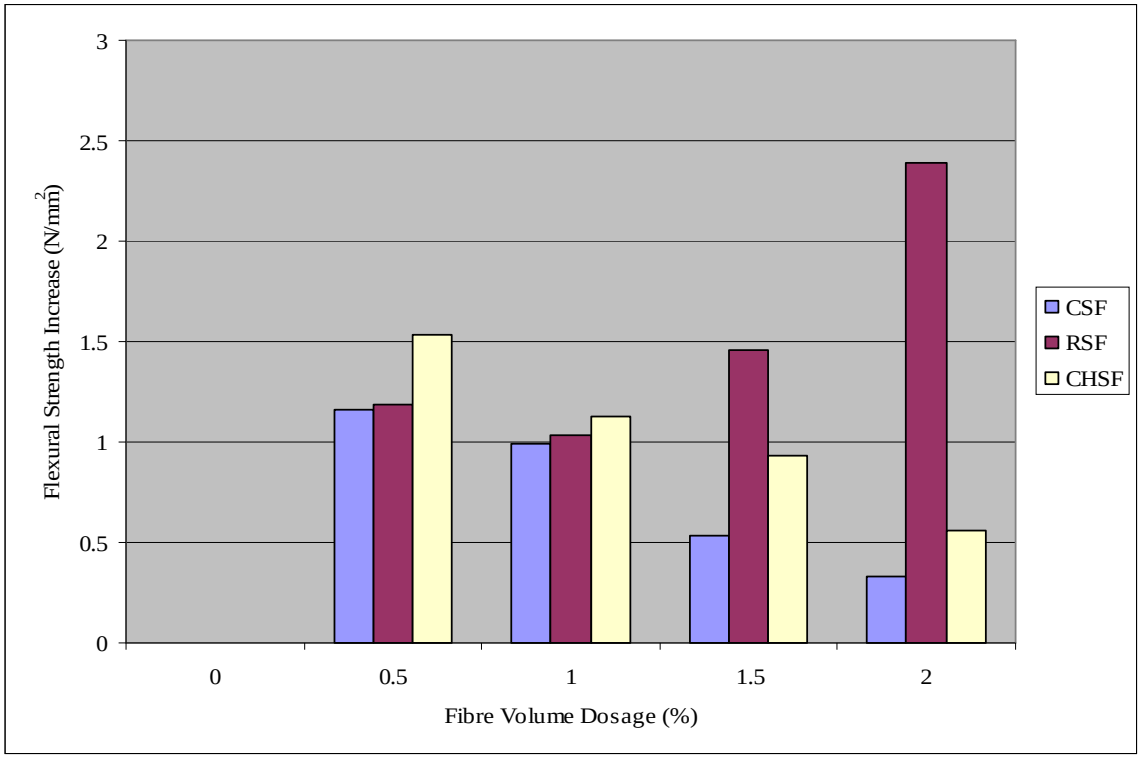


Figure 4.19: Flexural Strength Increase over Control at Different Fibre Volume Dosage

Observing the results in Table 4.14 and Figure 4.18 through to Figure 4.19, it can be seen that there is increase in flexural strength of concrete beams when different types of fibres are added to concrete. The control (zero percent fibre) had a flexural strength of 3.55 N/mm^2. Rectangular steel fibre recorded the highest increase in flexural strength of 61 percent at two percent fibre volume; which is 2.39 N/mm^2 over control.

A study of Table 4.14, Figures 4.18 and 4.19 shows that all the steel fibres have increase in flexural strength when compared with the control.

4.11 Load/deflection Response

The results of the load – deflection response from the test performed on all fibre reinforced concrete beams under one point load were presented in Tables 3.21 to 3.23 of chapter three, the summary of the results are presented in Table 4.16 below for comparison purposes.

Table 4.16: Load / deflection Results for the three Fibres at Different Fibre Percentage

Fibre Percentage (%)	Load (kN)												
	0	5	10	15	20	25	30	35	40	45	50	55	60
	Deflection (mm)												
0	0	0.053	0.105	0.168	0.237	0.272	0.316	0.368	0.377	-	-	-	-
0.5 (CSF)	0	0.053	0.105	0.237	0.316	0.340	0.352	0.362	-	-	-	-	-
0.5 (RSF)	0	0.054	0.110	0.236	0.316	0.334	0.355	0.366	-	-	-	-	-
0.5 (CHSF)	0	0.062	0.111	0.178	0.191	0.249	0.309	0.321	0.342	0.423	-	-	-
1.0 (CSF)	0	0.053	0.105	0.210	0.263	0.301	0.368	0.400	0.474	-	-	-	-
1.0 (RSF)	0	0.055	0.108	0.210	0.252	0.306	0.308	0.413	0.415	-	-	-	-
1.0 (CHSF)	0	0.062	0.111	0.178	0.247	0.249	0.326	0.401	0.420	-	-	-	-
1.5 (CSF)	0	0.053	0.105	0.210	0.263	0.301	0.368	0.400	0.474	0.053	0.105	0.210	-
1.5 (RSF)	0	0.054	0.108	0.238	0.269	0.290	0.329	0.350	0.369	0.500	0.528	0.667	-
1.5 (CHSF)	0	0.058	0.060	0.074	0.111	0.154	0.185	0.269	0.278	-	-	-	-
2.0 (CSF)	0	0.0531	0.104	0.147	0.210	0.287	0.311	0.342	0.368	0.401	0.502	0.610	0.742
2.0 (RSF)	0	0.054	0.112	0.129	0.200	0.291	0.330	0.343	0.389	0.411	0.499	0.580	0.734
2.0 (CHSF)	0	0.052	0.063	0.070	0.099	0.149	0.178	0.259	0.260	-	-	-	-

Table 4.16 is represented in Figures 4.20 to 4.22 for visual comparism and analysis.

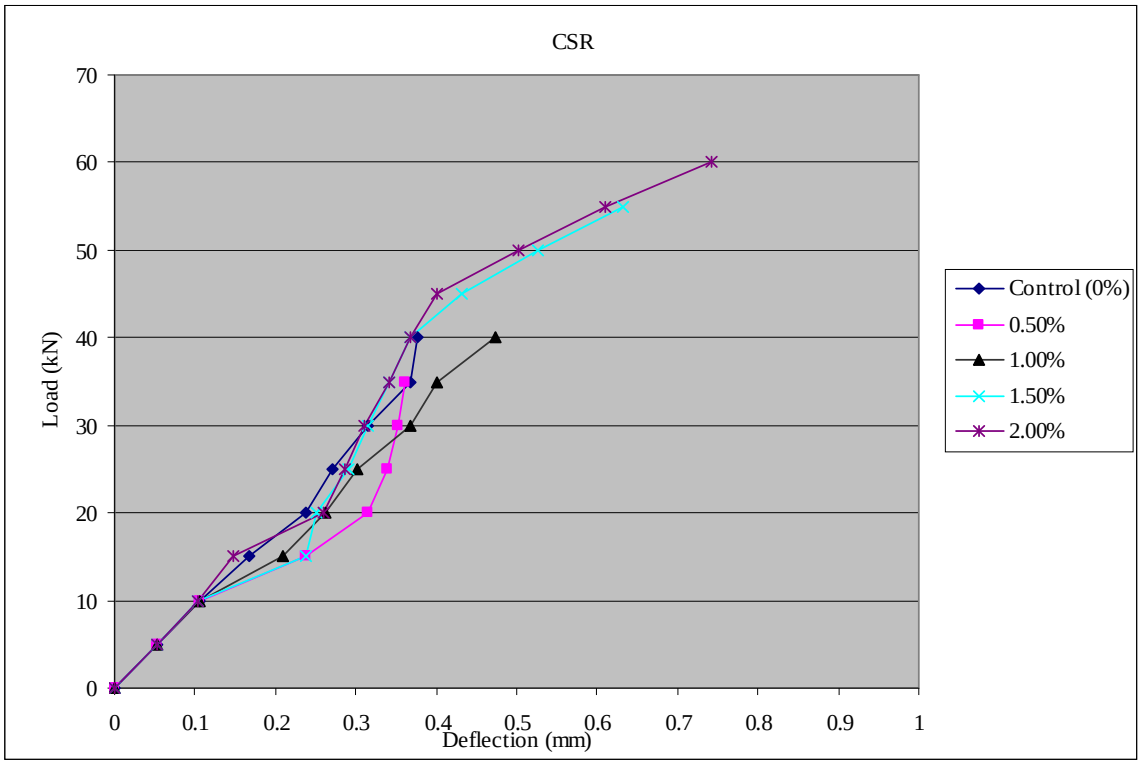


Figure 4.20: Load/Deflection Curve for CSF at Different Volume Dosage

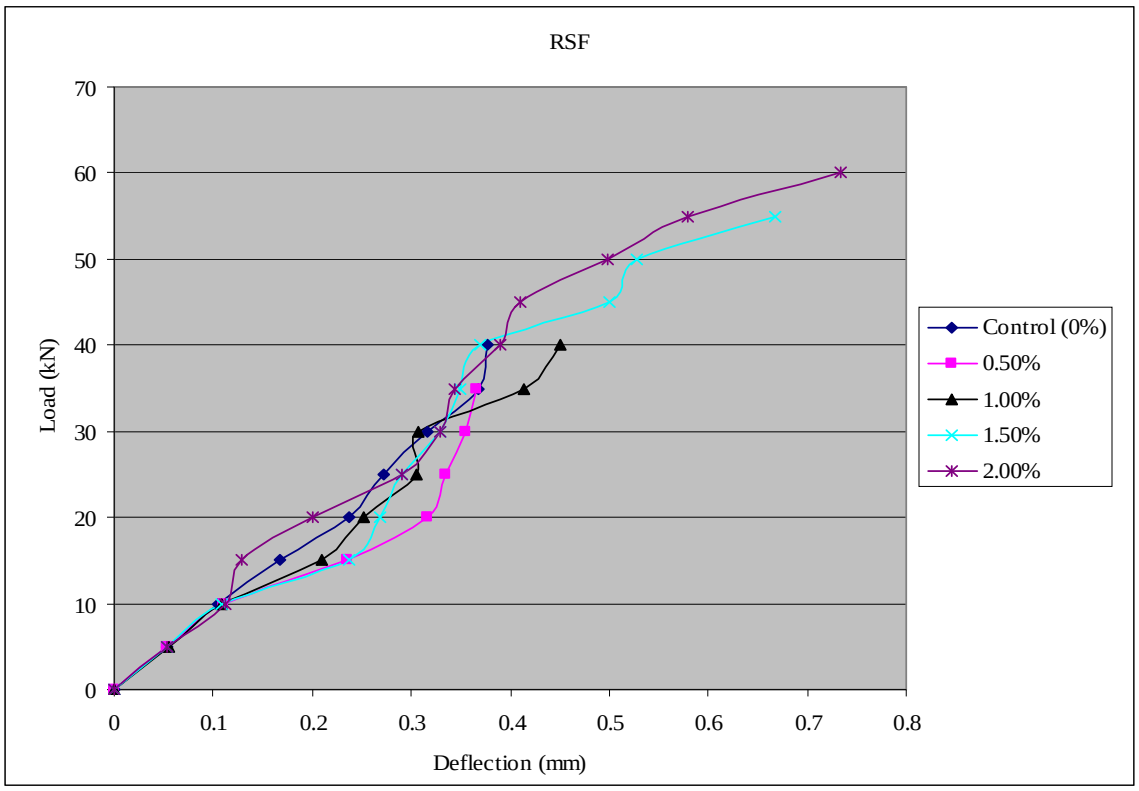


Figure 4.21: Load/Deflection Curve for RSF at Different Volume Dosage

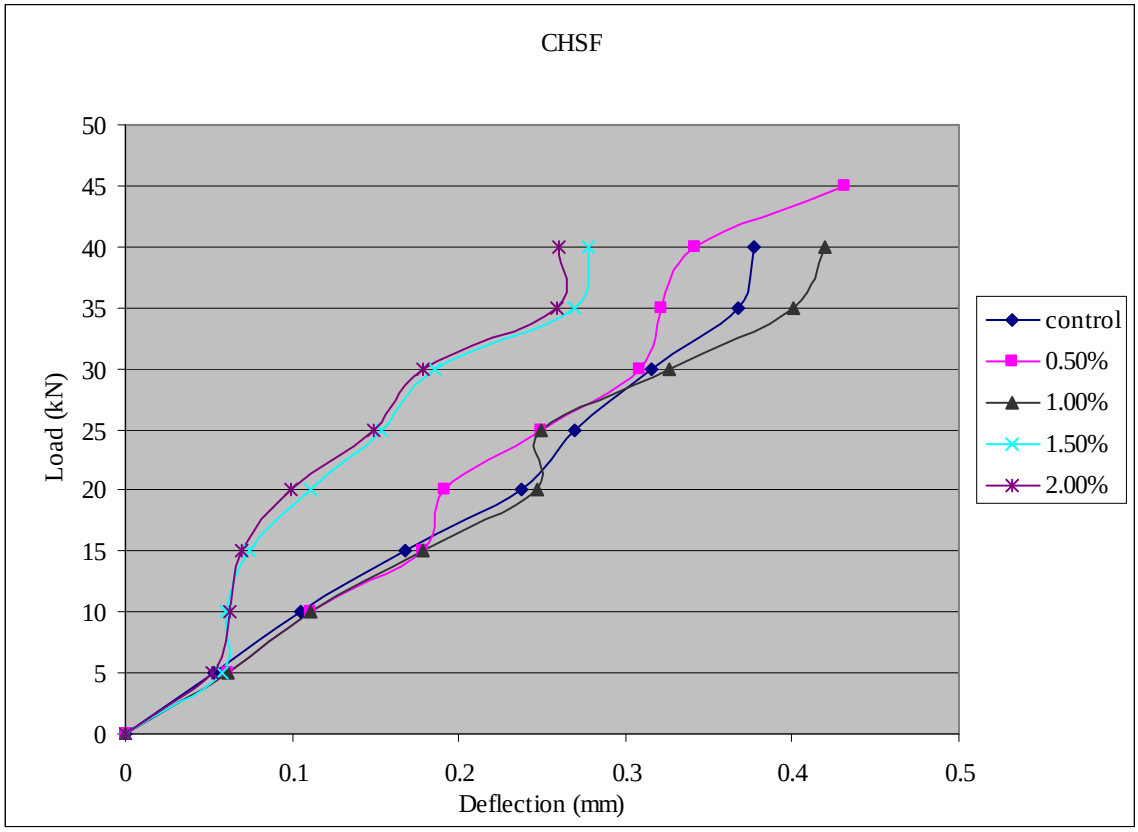


Figure 4.22: Load/Deflection Curve for CHSF at Different Volume Dosage

Table 4.16 and Figures 4.20 to 4.22 show that the results for the zero percent fibre concrete displays typical brittle fracture behavior, with a maximum load of 40 kN at a deflection of 0.377 mm. The failure was sudden without warning and was virtually perpendicular to the longitudinal direction of the specimen. The fracture surfaces of the zero percent fibre concrete specimens are flatter and smoother than those of fibre reinforced concrete.

A careful examination of Figures 4.20 to 4.22 also reveal that addition of circular steel fibre improved the load carrying capacity over that of the unreinfored (zero percent fibre) concrete and shows a reduction in the rate of deflection as the load is increased. These two observations hold for the three fibres used in this work. Circular and rectangular steel fibres sustained the highest load of 60 kN and deflections of 0.742 mm and 0.734 mm

respectively before failure begins; this was at two percent fibre volume dosage. This shows an increase in maximum load of about 50% over the maximum load carried by the control specimen. The improved load carrying capacity of all the fibre reinforced concrete (circular, rectangular and chips steel fibres) as shown in Figures 4.20 to 4.22 over the unreinforced concrete are basically due to the bridging effect of the fibres in the concrete and the interaction between fibres and concrete.

Table 4.16 also shows and compared the load-deflection characteristics of the three types of steel fibres at the same fibre percentage. This is also represented in graphs below as Figures 4.23 to 4.26.

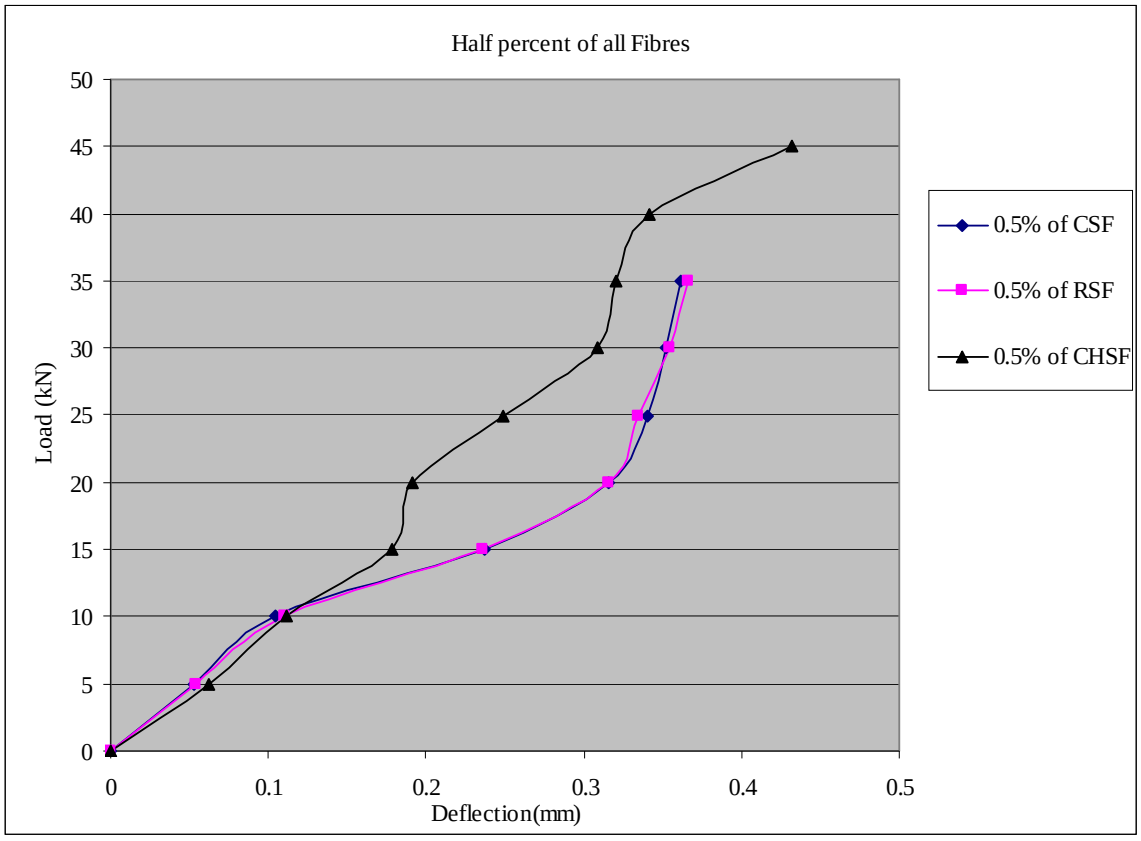


Figure 4.23: Load vs. displacement graph for all the fibre at 0.5%volume dosage

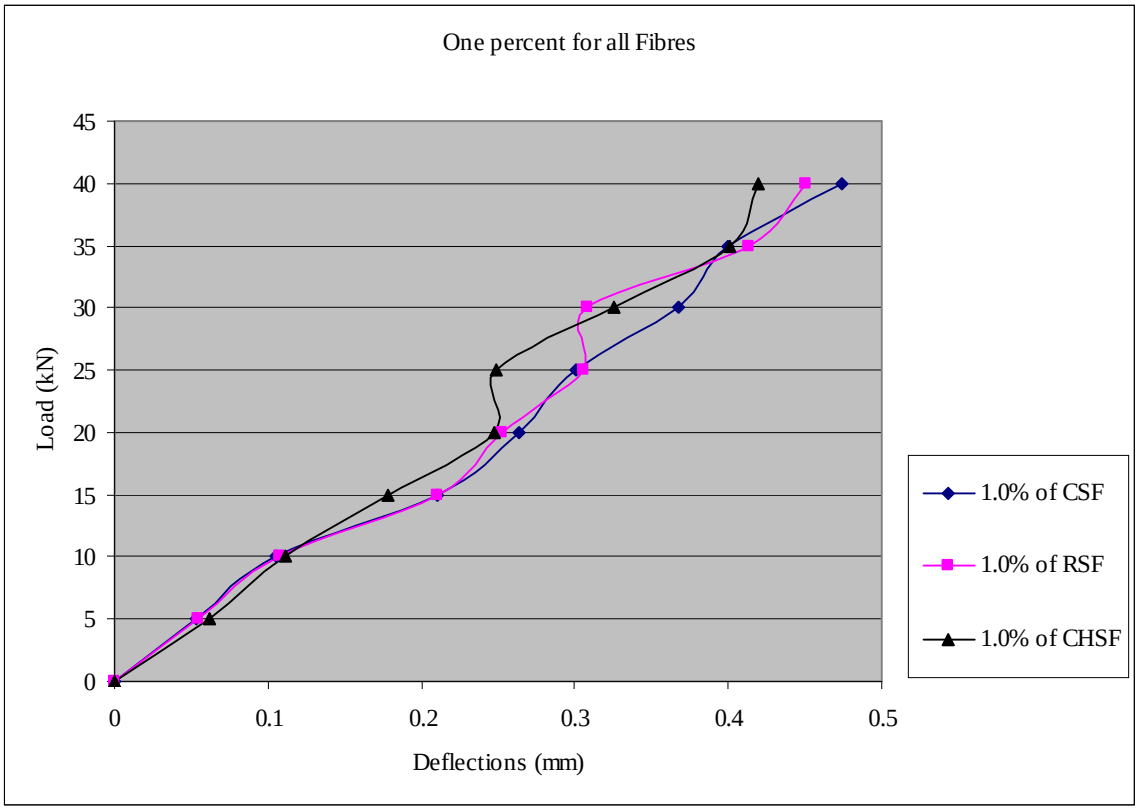


Figure 4.24: Load vs. displacement graph for all the fibre at 1.0%volume dosage

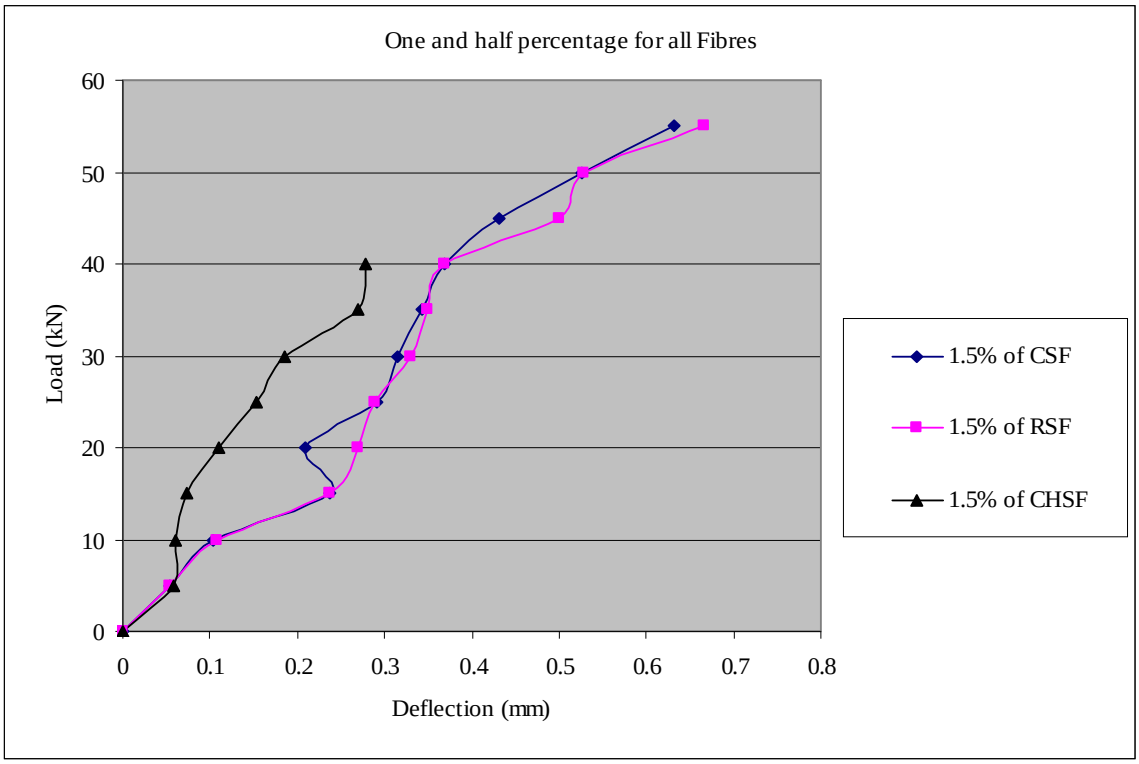


Figure 4.25: Load vs. displacement graph for all the fibre at 1.5%volume dosage

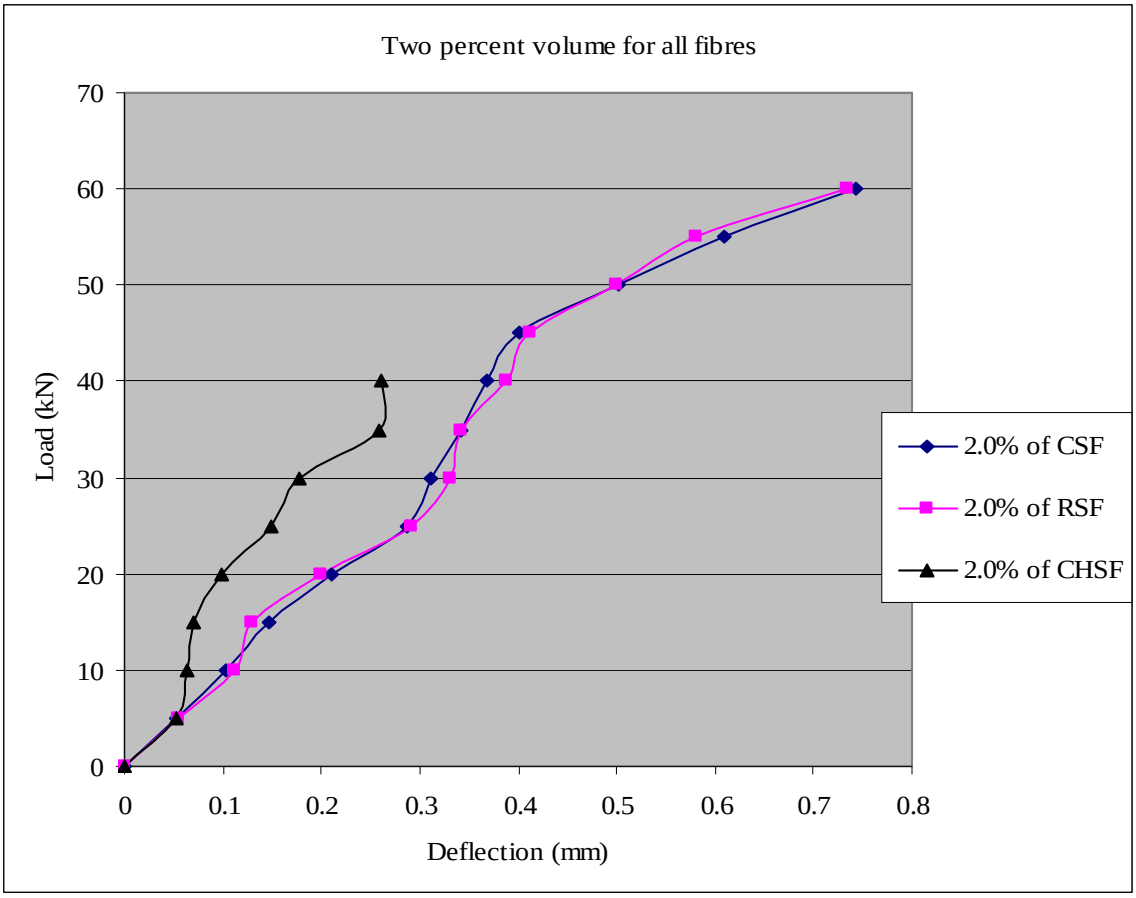


Figure 4.26: Load vs. displacement graph for all the fibre at 2.0%volume dosage

Figures 4.23 to 4.26 show the load/deflection curves for fibre reinforced concrete, the figures compare the variations of the load carrying capacity of fibres as the fibre volume dosage remain the same for all the fibres used in this work. At one-half percent fibre volume dosage chip fibre reinforced concrete shows less deflections as the load increases as compared to the other two fibres reinforced concrete at a load of 35 kN, the deflection was 0.321 mm for chip steel fibre, while the deflection was the same (0.362 mm) for circular and rectangular steel fibres at the same 35 kN. Chips steel fibre carried more load (maximum of 45 kN) before failure as compare to a load of 35 kN for circular and rectangular steel fibres reinforced concrete. A carefully observation of Figures 4.23 to 4.26

shows that circular and rectangular steel fibres reinforced concrete carried more loads at higher fibre volume dosages than chip steel fibre concrete.

From the results of flexural strength test, it is clear that the three different fibres used for this work shows improved performances over control specimen through their influence on load sharing with the concrete matrix and the bridging of cracks. It is also clear that the ranges of fibre volume dosage used are quite suitable dosage to apply to concrete and will give a result of more than 50% increase in flexural strength. As a general overview, waste steel fibres do increase the flexural strength; increase the energy absorption of concrete at the pre-cracking and post-cracking stages.

4.12 Chips Steel Fibre Concrete Cubes Tests

The results of the confirmation chips steel fibre concrete cubes test are as presented in Tables 3.28 of chapter three. These results are compared with the results of chips steel fibre concrete of the first stage of this work and are as presented Table 4.17:

Table 4.17: Compressive Strength of Chips Steel Fibre Concrete Cubes

Fibre Percentage (%)	First CHSF N/mm^2	Confirmation CHSF N/mm^2	Average Strength N/mm^2
0	33.7	33.7	33.7
0.5	35.3	35.5	35.4
1.0	36.3	35.9	36.1
1.5	31.4	36.2	33.8
2.0	20.4	30.2	25.3
2.5	-	20.1	20.1
3.0	-	18.5	18.5
3.5	-	0	Zero

Table 4.17 is presented in Figure 4.27.

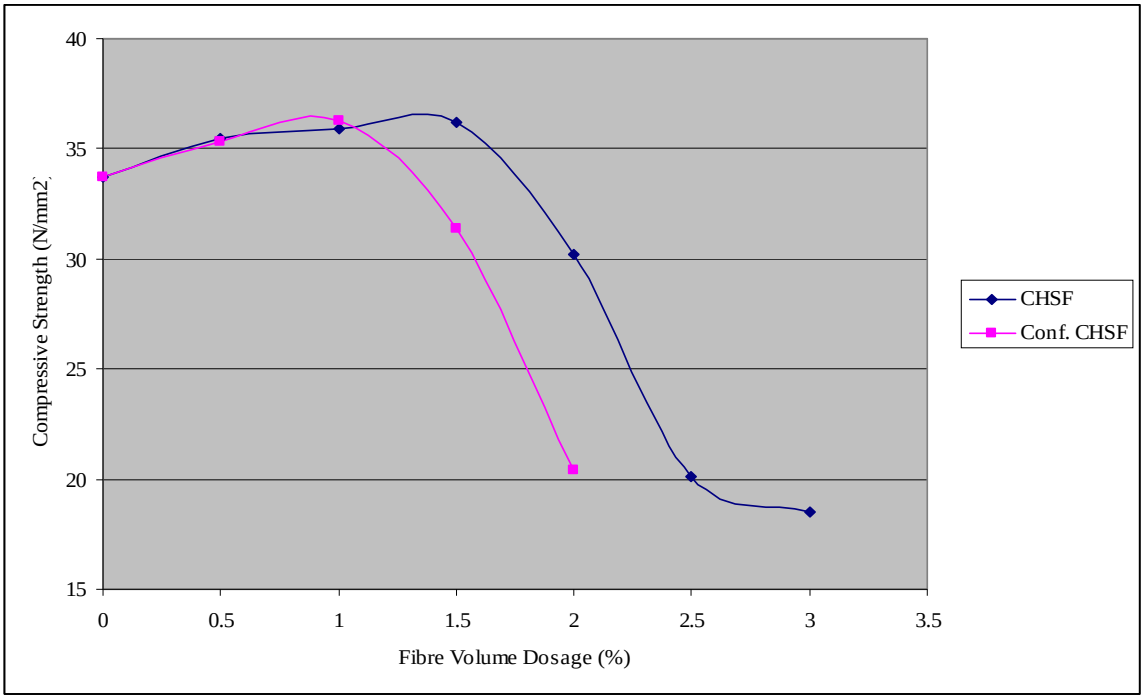


Figure 4.27: Compressive strength of chips steel fibre cubes

A careful observation of Table 4.17 and Figure 4.27 show that there was increase in compressive strength of chips steel fibre from zero percent to one and half percent for the confirmation test and there was increase up to only one percent fibre volume for the first test. The highest compressive strength 36.3 N/mm^2 occurred at one percent fibre volume for the first test and dropped to the lowest of 20.4 N/mm^2 at two percent fibre dosage. For the confirmation chips fibre test, the highest compressive strength is 36.2 N/mm^2 at one percent fibre dosage and dropped to zero at three and half percent fibre dosage. The confirmation results confirm the earlier test results of compressive strength of chips steel fibre concrete. This drop in compressive strength of chips steel fibre concrete as the fibre volume dosage increases is as a result of drop in workability as the fibre volume dosage increases. For example, as the fibre volume dosage increased above two percent, the

workability decreased to a level that full compaction could not be achieved and this lead to a further drop in strength as shown in Table 45 and Figure 4.27.

4.13 Prediction Model for Strengths of Steel Fibre Composites.

In developing the 28-day strengths prediction model of the fibre reinforced mortar and concrete specimens, the influence of the fibre volume dosage on the strength properties are considered.

The statistical package in EXCEL MICROSOFT OFFICE 2005 was used for developing the prediction model. This package was used to predict the models by generating curves and equations that would best fit the experimental data. A comparison is also made between the experiment data and data generated by the prediction model.

The results obtained from the test performed on mortar and concrete specimens and the graphs of these results are shown earlier in chapter three and four. The generation of fitting curves and equations using Microsoft Excel Package can be represented by a general polynomial equation of the form:

$$F_{(x)} = ax^3 + bx^2 + cx + d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.1$$

Where $F_{(x)}$ is the strength at a curing age of 28 days and a, b, c and d are coefficients and x is the fibre percentage.

Using the statistical package in EXCEL MICROSOFT OFFICE 2005, the coefficients a, b, c and d; the coefficient of determinations (R^2) are obtained for all the strength data in chapter three. The coefficients a, b, c and d and the coefficient of determinations (R^2) are shown in tables below:

Table 4.18: Coefficients for Mortar Cubes

Property	Fibre Type	Coefficients				
		A	b	c	d	R^2
Compressive strength of Mortar Cubes	CSF	-4.3333	11.657	-4.7310	27.639	0.9169
	RSF	-0.8667	2.2286	0.8595	27.504	0.9999
	CHSF	-3.8000	9.8571	-2.9643	27.659	0.9040

Table 4.19: Coefficients for Mortar Beams

Property	Fibre Type	Coefficients				
		A	b	c	d	R^2
Flexural strength of Mortar Beams	CSF	0.6667	-2.4143	3.0119	3.5629	0.9931
	RSF	0.4333	-1.8429	2.7774	3.5536	0.9995
	CHSF	0.6600	-2.7571	3.7493	3.5704	0.9865

Table 4.20: Coefficients for Concrete Cubes

Property	Fibre Type	Coefficients				
		A	b	c	d	R^2
Compressive strength of Concrete Cubes	CSF	-1.5333	6.1714	0.9405	33.476	0.9753
	RSF	-3.2667	9.8857	-1.4548	33.533	0.9775
	CHSF	-3.667	2.1143	3.7881	33.627	0.9978

Table 4.21: Coefficients for Concrete Cylinders

Property	Fibre Type	Coefficients				
Tensile strength of concrete cylinders		A	b	c	d	R^2
	CSF	0.4467	-1.7771	2.6126	2.9484	0.9971
	RSF	0.4133	-1.6686	2.4938	2.9537	0.9918
	CHSF	0.2667	-1.4886	2.8805	2.9437	0.9996

Table 4.22: Coefficients for Concrete Beams

Property	Fibre Type	Coefficients				
Flexural strength of Concrete Beams		A	b	c	d	R^2
	CSF	1.0600	-4.0400	4.0050	3.9450	0.9962
	RSF	1.2333	-3.6800	3.6217	3.9690	0.9801
	CHSF	1.1733	-4.5486	4.6838	3.9757	0.9340

The coefficients above are use to generate predicting equations that will best fit the experimental curves. The equations are shown below:

Using the coefficients in Table 4.18, the equations for the compressive strength of mortar cubes are:

$F_{(CSF)} = -4.3333x^3 + 11.657x^2 – 4.731x + 27.639$……..4.2

$F_{(RSF)} = -0.8667x^3 + 2.2286x^2 + 0.8595x + 27.504$…… ……...4.3

$F_{(CHSF)} = -3.8x^3 + 9.8571x^2 – 2.9643x + 27.659$…….……...4.4

Using the coefficients in Table 4.19, the equations for the flexural strength of mortar beams are:

$F_{(CSF)} = 0.6667x^3 – 2.4143x^2 + 3.0119x + 3.5629$……4.5

$F_{(RSF)} = 0.4333x^3 – 1.8429x^2 + 2.7774x + 3.5536$…….. …….4.6

$F_{(CHSF)} = 0.66x^3 – 2.7571x^2 + 3.7493x + 3.5704$……. ……4.7

Using the coefficients in Table 4.20, the equations for the compressive strength of concrete cubes are:

$F_{(CSF)} = -1.5333x^3 + 6.1714x^2 – 0.9405x + 33.476$…… ….4.8

$F_{(RSF)} = -3.2667x^3 + 9.8857x^2 – 1.4548x + 33.533$…… ….4.9

$F_{(CHSF)} = -3.6667x^3 + 2.1143x^2 + 3.7881x + 33.627$……4.10

Using the coefficients in Table 4.21, the equations for the tensile strength of concrete cylinders are:

$F_{(CSF)} = 0.4467x^3 – 1.7771x^2 + 2.6126x + 2.9484$……4.11

$F_{(RSF)} = 0.4133x^3 – 1.6686x^2 + 2.4938x + 2.9537$……4.12

$F_{(CHSF)} = 0.2667x^3 – 1.4886x^2 + 2.8805x + 2.9437$…… ….4.13

Using the coefficients in Table 4.22, the equations for the flexural strength of concrete beams are:

$F_{(CSF)} = 1.06x^3 – 4.04x^2 + 4.005x + 3.945$……..……...4.14

$F_{(RSF)} = 1.2333x^3 – 3.68x^2 + 3.6217x + 3.969$……..…… …….4.15

$F_{(CHSF)} = 1.1733x^3 – 4.5486x^2 + 4.6838x + 3.9757$……..… …….4.16

R-squares obtained for these equations were above 0.99, in each of the predicted equations, in other words, we could explain 99% of the variability for the data around the regression line and 1% of the residual data could not be explained by these equations.

Using the equations above, the predicted values of the strengths and the experimental values are presented in Tables 4.23 to 4.27 below. These Tables are also represented by graphs showing the obtained and predicted curves.

Table 4.23: Compressive Strengths of Mortar Cubes

Fibre Percentage (%)	CSF		RSF		CHSF	
	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)
0	27.5	27.6	27.5	27.5	27.5	27.7
0.5	28.2	27.7	28.4	28.4	28.8	28.2
1.0	29.4	30.2	29.7	29.7	29.8	30.8
1.5	32.7	32.2	30.9	30.9	31.2	32.5
2.0	30.0	30.1	31.2	31.2	30.6	30.8
2.5	-	21.0	-	17.5	-	22.4

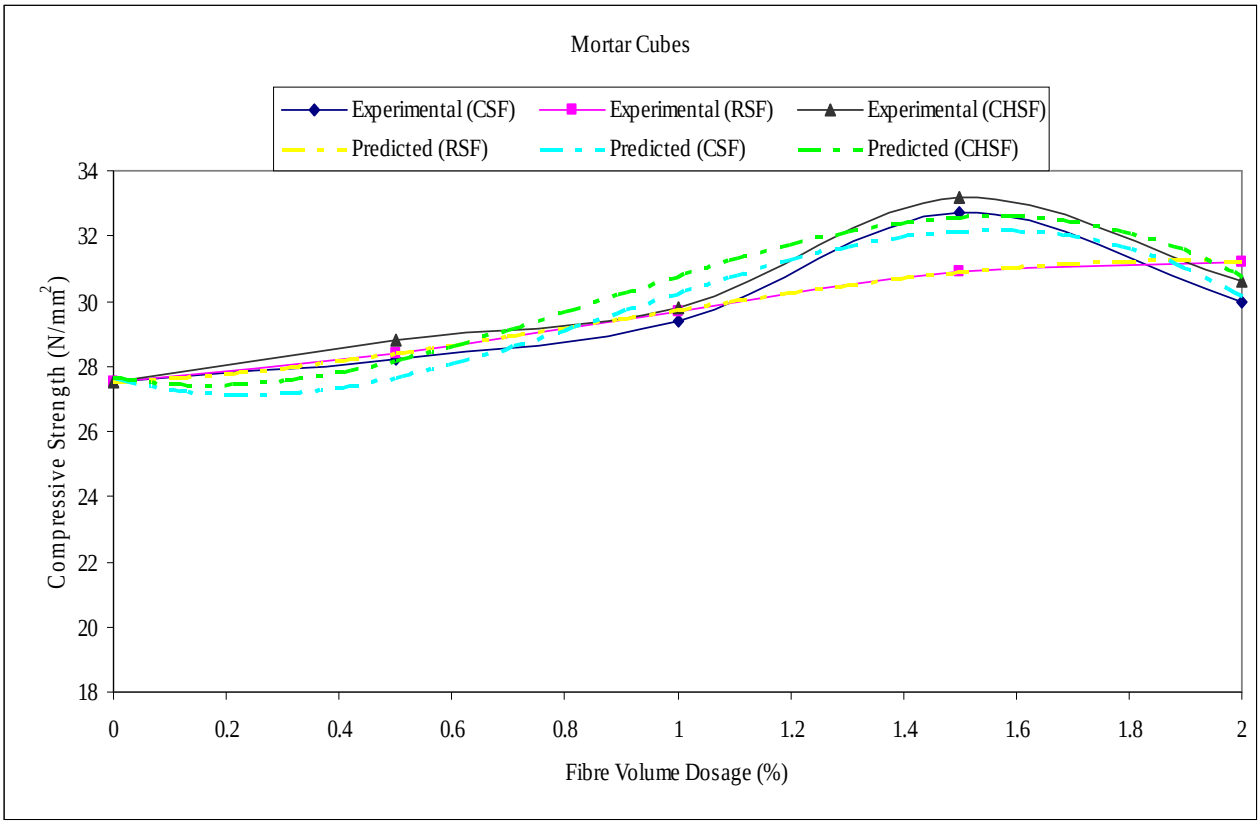


Figure 4.28: Experimental and Predicted – 28 days Compressive Strength

Table 4.24: Flexural Strength of Mortar Beams

Fibre Percentage (%)	CSF		RSF		CHSF	
	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)
0	3.55	3.56	3.55	3.55	3.55	3.57
0.5	4.60	4.55	4.55	4.54	4.92	4.76
1.0	4.75	4.83	4.90	4.92	5.10	5.22
1.5	4.95	4.90	5.05	5.04	5.30	5.22
2.0	5.25	5.26	5.20	5.20	5.30	5.32
2.5	-	6.42	-	5.75	-	6.02

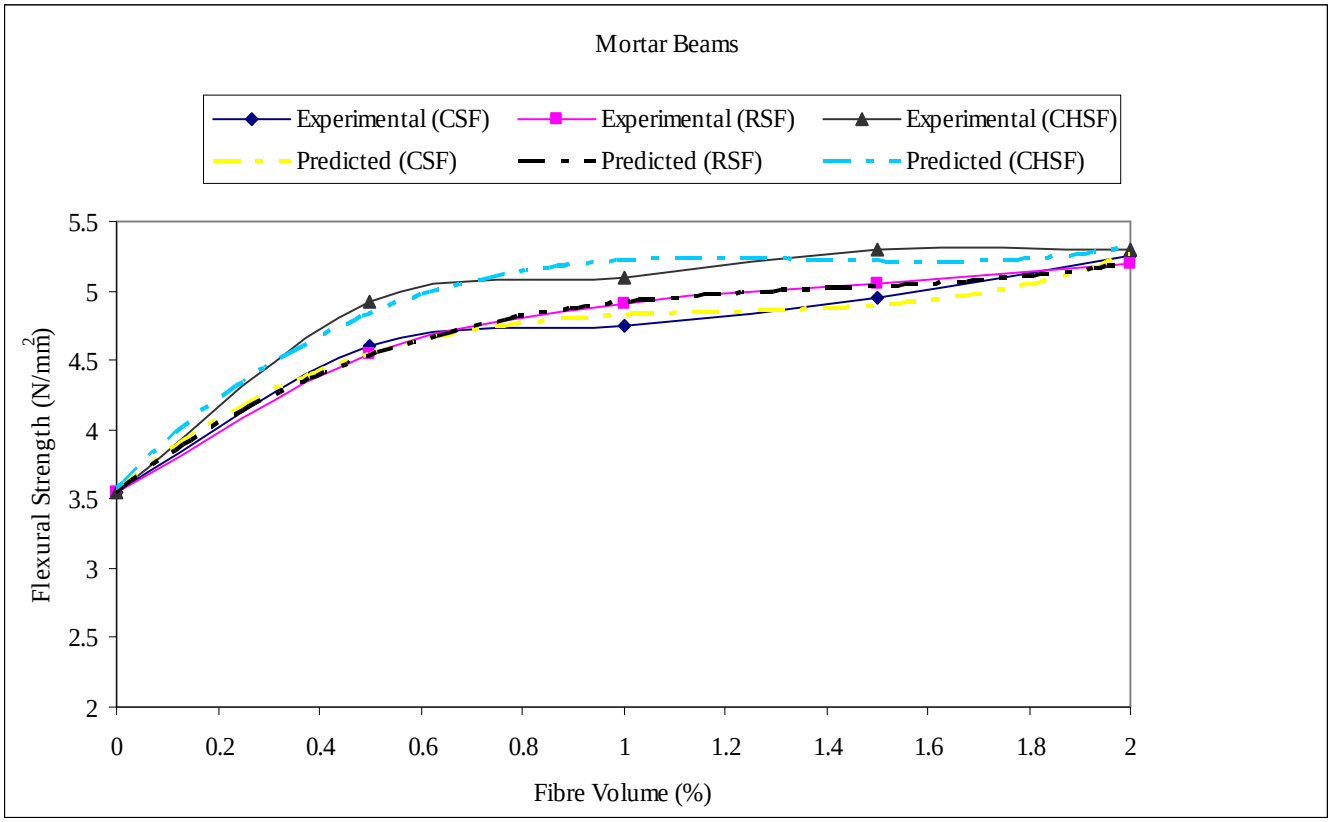


Figure 4.29: Experimental and Predicted – 28 days Flexural Strength

Table 4.25: Compressive Strength of Concrete Cubes

Fibre Percentage (%)	CSF		RSF		CHSF	
	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)
0	33.7	33.48	33.7	33.53	33.7	33.63
0.5	34.4	34.36	34.2	34.87	35.3	35.59
1.0	40.4	37.17	39.7	38.70	36.3	35.86
1.5	42.7	40.78	41.9	42.57	31.4	31.69
2.0	48.0	44.01	44.2	44.03	20.4	20.33
2.5	-	45.73	-	40.64	-	-0.98

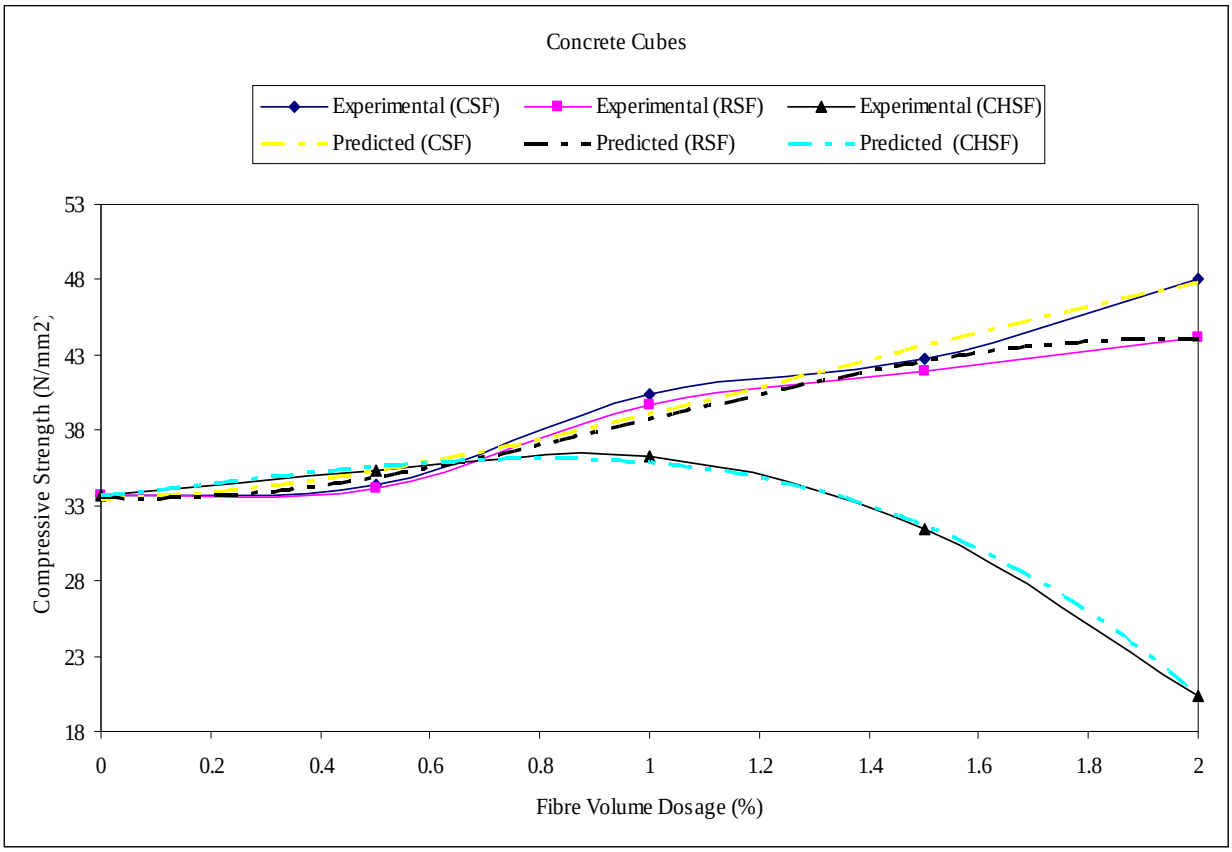


Figure 4.30: Experimental and Predicted – 28 days Compressive Strength

Table 4.26: Tensile Strength of Concrete Cylinder at Different Fibre Dosage

Fibre Percentage (%)	CSF		RSF		CHSF	
	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)
0	2.94	2.95	2.94	2.95	2.94	2.94
0.5	3.90	3.87	3.89	3.84	4.06	4.05
1.0	4.18	4.23	4.11	4.19	4.58	4.60
1.5	4.41	4.38	4.39	4.34	4.83	4.82
2.0	4.63	4.64	4.56	4.57	4.88	4.88
2.5	-	5.35	-	5.22	-	5.01

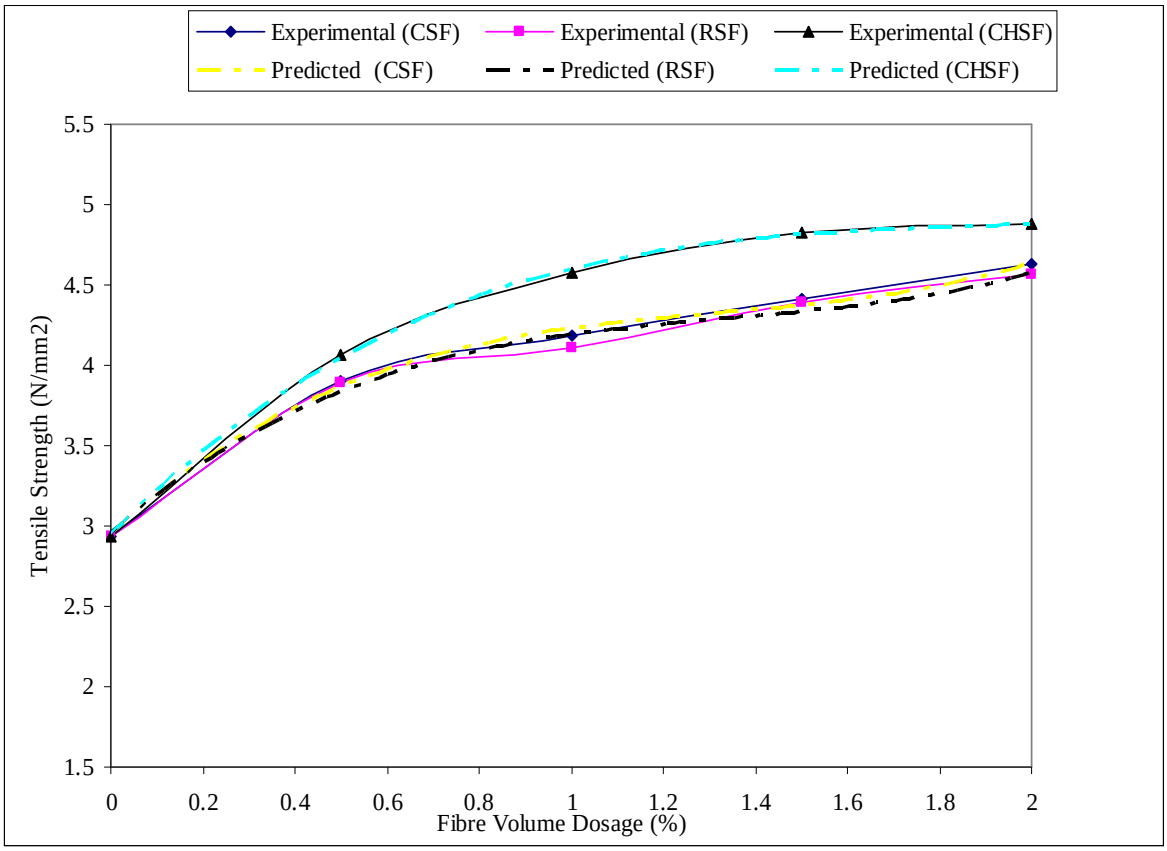


Figure 4.31: Experimental and Predicted – 28 days Tensile Strength

Table 4.27: Flexural Strength of Concrete Beams at Different Fibre Dosage

Fibre Percentage (%)	CSF		RSF		CHSF	
	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)	Obtained Strength (N/mm^2)	Predicted Strength (N/mm^2)
0	3.94	3.95	3.94	3.97	3.94	3.98
0.5	5.10	5.07	5.13	5.01	5.47	5.33
1.0	4.93	4.97	4.97	5.14	5.07	5.28
1.5	4.47	4.44	5.40	5.28	4.87	4.73
2.0	4.27	4.28	6.33	6.36	4.50	4.54
2.5	-	5.27	-	9.29	-	5.59

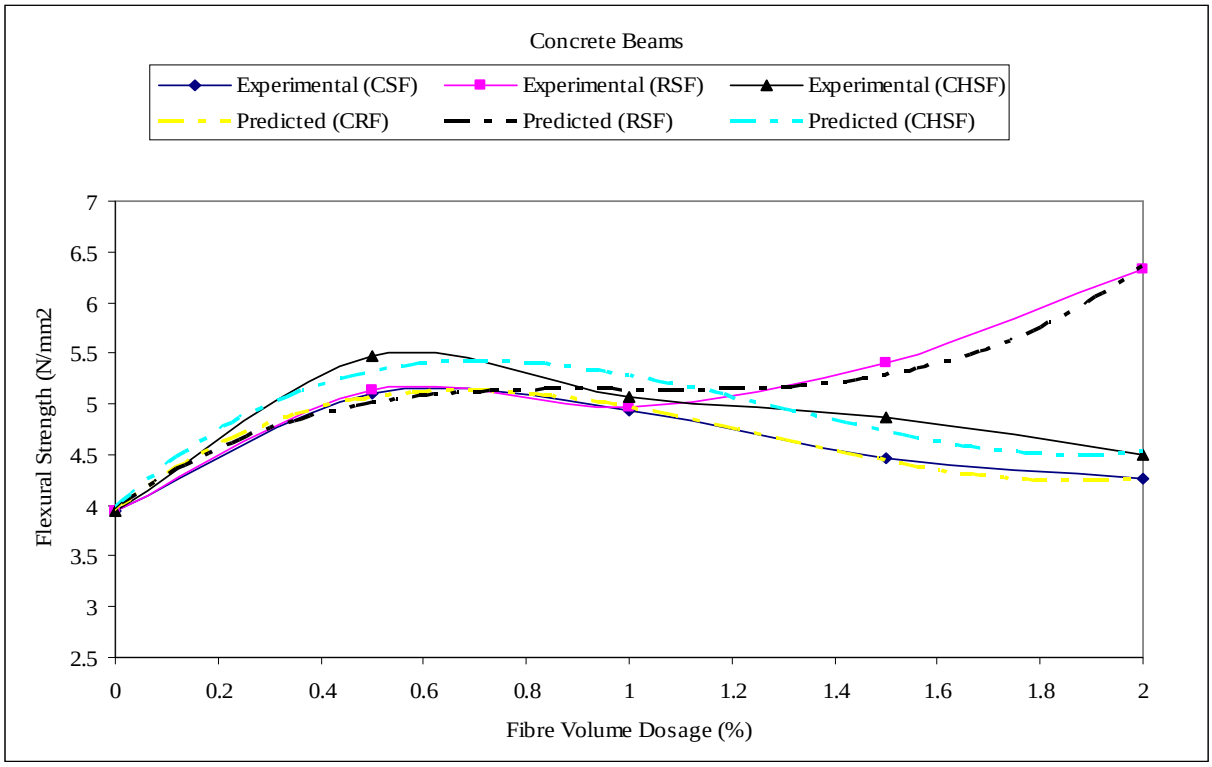


Figure 4.32: Experimental and Predicted – 28 days Compressive Strength

Plots of samples of the matching curves for the prediction and experimental results are shown in Figures 4.28 to 4.32. The smooth lines are the experimental, while the dotted lines are the predicted.

4.14 Relationships between Compressive, Tensile and Flexural Strengths of Steel Fibre Mortar and Concrete.

There is no direct proportionality (relationship) between compressive strength and tensile and flexural strengths of mortar or concrete, but generally, as the compressive strength increases, the flexural and tensile strengths also increases at a decreasing rate for plain concrete, Neville (1997), Mehta and Monteiro (1993), and Sullivan (2001).

The tensile strength of plain concrete is usually taken to be about one-tenth of its

compressive strength, Neville (1997) and Murdock and Blackledge, (1968). In general, as a guide, the indirect tensile strength of concrete may be taken as $0.45\sqrt{f_{cu}}$ Nmm^2, where f_{cu} is the cube compressive strength and the flexural strength may be taken as $0.70\sqrt{f_{cu}}$ Nmm^2. In this work, the following relationship could be established between compressive strength and tensile and flexural strengths of steel fibre mortar and concrete and these relationships are as stated below:

1. The relationship between compressive strength of mortar cubes and flexural strength of mortar beam could be taken as $0.89\sqrt{f_{cu}}$ N/mm^2, (ie $f_{cr} = 0.89\sqrt{f_{cu}}$) where f_{cu} is the cube compressive strength and f_{cr} is flexural strength. Using this established relationship, the experimental and estimated values of mortar beam flexural strength are shown in Table 4.28 to 4.30.
2. The relationship between compressive strength of concrete cube and tensile strength of concrete cylinder could be taken as $0.66\sqrt{f_{cu}}$ N/mm^2, (ie $f_{ct} = 0.66\sqrt{f_{cu}}$), where f_{cu} is the cube compressive strength and f_{cr} is the tensile strength. Using this established relationship, the experimental and estimated values of concrete tensile strength are shown in Table 4.31 to 4.33
3. The relationship between compressive strength of concrete cube and flexural strength of concrete beam could be taken as $0.85\sqrt{f_{cu}}$ N/mm^2, (ie $f_{ct} = 0.85\sqrt{f_{cu}}$), where f_{cu} is the cube compressive strength and f_{cr} is flexural strength. Using this established relationship, the experimental and estimated values of concrete tensile strength are shown in Table 4.34 to 4.36.

Table 4.28: Experimental and Estimated Values of Mortar Beam Flexural Strength – CSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	27.5	3.55	3.67
0.5	28.2	4.60	4.73
1.0	29.4	4.75	4.82
1.5	32.7	4.95	5.09
2.0	30.0	5.25	4.87

Table 4.29: Experimental and Estimated Values of Mortar Beam Flexural Strength – RSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	27.5	3.55	3.67
0.5	28.4	4.55	4.74
1.0	29.7	4.90	4.85
1.5	30.9	5.05	4.95
2.0	34.2	5.20	5.21

Table 4.30: Experimental and Estimated Values of Mortar Beam Flexural Strength – CHSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	27.5	3.55	3.67
0.5	28.8	4.92	4.78
1.0	29.8	5.10	4.86
1.5	33.2	5.30	5.12
2.0	33.6	5.30	5.16

Table 4.31: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – CSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Tensile Strength (N/mm^2)	Estimated Tensile Strength (N/mm^2)
0	33.7	2.94	2.61
0.5	34.4	3.90	3.87
1.0	40.4	4.18	4.19
1.5	42.7	4.41	4.31
2.0	48.0	4.63	4.57

Table 4.32: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – RSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Tensile Strength (N/mm^2)	Estimated Tensile Strength (N/mm^2)
0	33.7	2.94	2.61
0.5	34.2	3.89	3.86
1.0	39.7	4.11	4.16
1.5	41.9	4.39	4.27
2.0	44.2	4.56	4.39

Table 4.33: Experimental and Estimated Values of Concrete Cylinder Tensile Strength – CHSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Tensile Strength (N/mm^2)	Estimated Tensile Strength (N/mm^2)
0	33.7	2.94	2.61
0.5	35.3	4.06	3.92
1.0	36.3	4.58	3.97
1.5	31.4	4.83	3.70
2.0	20.4	4.88	2.98

Table 4.34: Experimental and Estimated Values of Concrete Beam Flexural Strength – CSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	33.7	3.94	4.06
0.5	34.4	5.10	4.99
1.0	40.4	4.93	5.40
1.5	42.7	4.47	5.55
2.0	48.0	4.27	5.89

Table 4.35: Experimental and Estimated Values of Concrete Beam Flexural Strength – RSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	33.7	3.94	4.06
0.5	34.2	5.13	4.97
1.0	39.7	4.97	5.36
1.5	41.9	5.40	5.50
2.0	44.2	6.33	5.65

Table 4.36: Experimental and Estimated Values of Concrete Beam Flexural Strength – CHSF

Fibre Percentage (%)	Average Compressive Strength (N/mm^2)	Average Experimental Flexural Strength (N/mm^2)	Estimated Flexural Strength (N/mm^2)
0	33.7	3.94	4.06
0.5	35.3	5.47	5.05
1.0	36.3	5.07	5.12
1.5	31.4	4.87	4.76
2.0	20.4	4.50	3.84

Observing Tables 4.28 to 4.36 above, it could be said that the established relationships gave very close estimate to the experimental values.

4.15 Toughness of Steel Fibre Concrete

The change in flexural toughness was quantified using the toughness index according to ASTM C 1018-97, (1998). The index is the area under the flexural test curve and could be calculated using the expression I_{10} / I_5, where I_{10} is the deflection at 10 kN and I_5 is the deflection at 5 kN. Using the above expression, the toughness indies for the different types of fibres and at the different percentages investigated are presented in Table 4.37 to 4.39:

Table 4.37: Toughness Index for Steel Fibre Concrete Beam – CSF

Fibre Percentage (%)	I_{10}	I_5	Toughness Index
0	0.105	0.053	1.981
0.5	0.105	0.053	1.981
1.0	0.105	0.053	1.981
1.5	0.105	0.053	1.981
2.0	0.104	0.0531	1.977

Table 4.38: Toughness Index for Steel Fibre Concrete Beam – RSF

Fibre Percentage (%)	I_{10}	I_5	Toughness Index
0	0.105	0.053	1.981
0.5	0.110	0.054	2.037
1.0	0.108	0.055	1.964
1.5	0.108	0.054	2.00
2.0	0.112	0.054	2.074

Table 4.39: Toughness Index for Steel Fibre Concrete Beam – CHSF

Fibre Percentage (%)	I_{10}	I_5	Toughness Index (J)
0	0.105	0.053	1.981
0.5	0.111	0.062	1.790
1.0	0.111	0.062	1.790
1.5	0.060	0.058	1.034
2.0	0.063	0.052	1.212

The toughness index values as shown above for the three fibres were insensitive to the fibre dosage rate for circular steel fibres (CSF) and rectangular steel fibres (RSF), but chips steel fibres (CHSF) was sensitive to increase in fibre dosage rate and at higher fibre dosage rate the toughness decreases as shown in Table 4.39. The toughness index for CSF and RSF can be said to be approximately 2.0 J. Where as that of chips steel fibre beam decreases with increase in fibre dosage as earlier said from 1.981 to 1.212 with an average value of 1.56. Thus, that of chip steel fibre is about 78 percent of circular and rectangular steel fibres. From mere observation and feeling of the fibres, these values are true to some extent.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Preambles

From the analyses and discussions in chapter four resulting from test results of chapter three, the following conclusions and recommendations can be made.

5.2 Conclusions

1. The fine aggregate (sand) used in the research work is of well graded sand and in zone two of the four zones of fine aggregate zones.
2. The coarse aggregate used is also well graded aggregates with nominal size of 38.5 millimeters.
3. The Dangote cement used satisfies all the standard requirements for Ordinary Portland Cement and is good for concrete making.
4. Three types of fibres namely; Circular steel fibres, rectangular steel fibres and chipping steel fibres were used for this research work. They were properly cleaned and can be said to be a clean fibre materials.
5. The compressive strength of the mortar cubes increased uniformly with increase in fibre dosage up to one and half percentage dosage, when different fibres began to manifest different compressive strengths.
6. As from one and half percentage fibre dosage the compressive strength of mortar cubes dropped except that of rectangular steel fibre which increased slightly as the fibre dosage increased.
7. The dosage of one and half percentage of steel fibre is a critical dosage for mortar compressive strength.

8. The value of the steel fibre mortar flexural strength tends to stabilize towards a value around 5 N/mm^2 at one and half percentage dosage – the critical dosage.
9. The chips steel fibre (CHSF) showed the highest increase in flexural strength and all strengths tend to converge at higher values above one and half percentage dosage.
10. The three fibres used showed a decrease in slump value with an increase in steel fibre dosage.
11. The chips steel fibre (CHSF) has lower values of slump when compared to the rest two types of fibres.
12. The decrease in slump in the steel fibre concrete for all the three types of fibres are as a result of stiff mixes and geometry of the various fibres.
13. Steel fibre concrete has lower workability than ordinary concrete.
14. There is generally a uniform decrease in compacting factor of steel fibre concrete with an increase in fibre dosage.
15. The percentage decrease in compacting factor of steel fibre concrete can be up to 20 percent with respect to chips steel fibre concrete as the dosage of fibre increases.
16. One and half percentage fibre dosage was also a critical fibre dosage for steel fibre concrete compressive strength.
17. All the three types of fibres used improved the tensile strength of concrete as the fibre volume dosage increased.
18. The chips steel fibre (CHSF) concrete has higher tensile strength than rectangular (RSF) and circular (CSF) steel fibre concrete.

19. In tensile strength tests, the steel fibre concrete (all three types) gave warning prior to failure but non-steel fibre concrete failed without warning.
20. One-half percentage fibre dosage is a critical point for steel fibre concrete flexural strength.
21. There was a reduction in deflection of concrete beams (prism) as the fibre dosage increased for in all the three types of fibre used.
22. The circular steel fibre concrete and the rectangular steel fibre concrete carried more loads at higher fibre volume dosage than the chips steel fibre concrete at the same beam deflection.
23. Fifteen equations estimating the 28- day strength of steel fibre mortar and concrete specimens were generated. This accurately predicted the 28 – days strength. It also predicted the deflection very well.
24. There are established relationships between the compressive strength of steel fibre mortar and flexural strength of steel fibre mortar and relationships between the compressive strength of concrete and tensile and flexural strengths of concrete.
25. All the established relationships gave good estimate of the experimental results.
26. The toughness index was insensitive to the increase in fibre volume dosage rate in circular and rectangular steel fibres, but toughness index decreased as the fibre dosage increased in chips steel fibre (CHSF).

5.3 Recommendations:

Based on the scope and the results of this study the following are recommended for further research.

i. There is need to investigate the effect of chemical admixtures on the workability of the fresh fibre reinforced concrete, which may reduce the problem of workability at high fibre volume dosage.

ii. The combination of short steel fibres may tend to provide more efficient mechanical properties of concrete. Further investigation should be carried out by combination of different types of short steel fibres into mortar and concrete mixes.

iii. To widen the use of fibre reinforced concrete, different or more complicated geometry of fibres can be used to investigate the effects of the fibres on the fresh and hardened concrete.

iv. The properties of fibre reinforced mortar and concrete may be different at various temperatures. Test on freeze – thawing conditions are recommended.

v. Further confirmatory tests on the areas already investigated in this work should be studied.

REFERENCES

1. ACI Committee 544, (1993): "Guide for Specifying, Mixing, Placing, and Finishing Steel Fibre Reinforced Concrete", ACI Journal, Volume 81, Number 2, March-April 1984, p.140-147.
2. Agarwal, B.D., Broutman, L.J., (1980) "Analysis and Performance of Fiber Composites", John Wiley & Sons, New York, 1980
3. Allen, H.G., "Glass-Fiber Reinforced Cement, Strength and Stiffness," CIRIA Report 55, September, 1975.
4. American Society for Testing and Materials, (1986) "Standard Method of Test for Splitting Tensile Strength of Cylindrical Concrete Specimens" (ASTM C496-86), Philadelphia.
5. ASTM C1018-97 (1998) "Standard Test Method for Flexural Toughness and First Crack Strength of Fibre-Reinforced Concrete", West Conshohocken, PA.
6. ASTM C78-94 (1998) "Standard Test Method for Flexural Strength of Concrete", West Conshohocken, PA.
7. ASTM C39-96 (1998) "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens", West Conshohocken, PA.
8. ASTM C192 (1998) "Standard Practice for Making and Curing Test Specimens in the Laboratory", West Conshohocken, PA.
9. Balaguru P.N. and Shah S.P., (1992). Fiber-Reinforced Cement Composites, McGraw-Hill Inc., New York, United State of America.

10. Banthia N. and Dubey A.,(2000). “Measurement of Flexural Toughness of Fibre-Reinforced Concrete Using Technique – Part 2: Performance of Various Composites”, ACI Materials Journal, Volume 97, Number 1.

11. Barros J.A.O. and Figueiras J.A., (1999). “Flexural Behaviour of SFRC: Testing and Modeling”, Journal of Materials in Civil Engineering, Volume 11, Number 4, November 1999, p.331-338.

12. Bentur A. and Mindess S., (1990). “Fibre Reinforced Cementitious Composites”, Elsevier Science Publishing Ltd., New York, United State of America.

13. British Standard Institution, BS 812: Part 101, (1984). “Guide to Sampling and Testing Aggregates” British Standard online at bsonlinetechindex.co.uk

14. British Standard Institution, BS 812: 103, (1985). “Methods for determination of Particle Size Distribution”, British Standard online at bsonlinetechindex.co.uk

15 British Standard Institution, BS 12:, (1971). “Ordinary and Rapid Hardening Portland Cement”, British Standard online at bsonlinetechindex.co.uk

16. British Standard Institution, BS 1881: 102, (1996). “Testing Concrete. Method for Determination of Slump Test”, British Standard online at bsonlinetechindex.co.uk

17. British Standard Institution, BS 1881: 103, (1996). “Testing Concrete. Method for Determination of Compacting Factor Test”, British Standard online at bsonlinetechindex.co.uk

18. British Standard Institution, BS 1881: 108, (1996). “Testing Concrete. Method for making Test Cubes from Fresh Concrete”, British Standard online at bsonlinetechindex.co.uk

19. British Standard Institution, BS 1881: 116, (1996). “Testing Concrete. Method for Determination of Compressive Strength of Concrete Cubes”, British Standard online at bsonlinetechindex.co.uk

20. British Standard Institution, BS 8110: 1, (1997). “Structural Use of Concrete Part 1: Code of Practice for Design and Construction.” British Standard online at bsonlinetechindex.co.uk

21. Chen L., Mindness S. and Morgan D. R., (1994). “Specimen Geometry and Toughness of Steel-Fiber-Reinforced Concrete”, Journal of Material in Civil Engineering, Volume 6, Number 4, February 1994, p.529-541.

22. Chen S., (2004). “Strength of Steel Fibre Reinforced Concrete Ground Slabs”, Structures and Buildings, Issue SB2, April 2004, p.157-163.

23. Dunstan I. and Swamy R. N., (1986). “Fibre Reinforced Cement and Concrete: Research into Practice”, Rilem Symposia FRC-86 Symposium.

24. Dwarakanath H.V. and Nagaraj T.S., (1991). “Comparative Study of Predictions of Flexural Strength of Steel Fiber Concrete”, ACI Materials Journal, Volume 88, Number 73.

25. Eldin, N. N., and Senouci, A. B., (1993) ‘‘Rubber-Tire Particles as Concrete Aggregate.’’ Journal of Material in Civil Engineering, ASCE, 5(4), 478–496.

26. Elvery, R. H., and Samarai, M.A., “Reduction of Shrinkage Cracking in Reinforced Concrete due to the Inclusion of Steel Fibers,” Fiber-reinforced Cement and Concrete, RILEM Symposium, 1975, pp. 149-159.

27. Epps, J. A., (1994). ‘‘Uses of Recycled Rubber Tires in Highways’’ Synthesis of Highway Practice 198, Transportation Research Board, National, 1994.

28. Fibremesh, (1989). “Fibremesh Micro-Reinforcement System, Synthetic Industries”, Fibremesh Division, TN, United State of America.

29. Ghugal Y.M. (2003). “Effects of Steel Fibres on various Strength of Concrete”, Indian Concrete Institute Journal 4, 23 – 29.

30. Ghugal Y.M (2006). “Performance of Alkali – Resistant Glass Fibre Reinforced Concrete”, Journal of Reinforced Plastics and Composites, 25, 617 – 630, SAGE Publications, USA

31. Gupta P., Banthia N., and Yan C., (2000). “Fibre Reinforced Wet-Mix Shotcrete under Impact”, Journal of Materials in Civil Engineering, Volume 12, Number 1, February 2000, p.81-90.

32. Hoff, G. C., (1975). “The use of Fiber Reinforced Concrete in Hydraulic Structures and Marine Environments,” Fiber reinforced Cement and Concrete, RILEM Symposium, pp. 395-407, Construction Press Ltd.

33. James J. Beaudoin, (1990). “Handbook of Fibre-Reinforced Concrete: Principles, Properties, Development and Applications”, Noyes Publications, New Jersey, United State of America.

34. Johnston C. D., (1982). “Steel Fibre Reinforced and Plain Concrete: Factors influencing Flexural Strength Measurement”, ACI Journal, Volume 79, Number 14, March-April 1982, p.131-138.

35. Johnston C. D. and Skarendahl A., (1992). “Comparative Flexural Performance Evaluation of Steel Fibre-Reinforced Concretes According to ASTM C1018 shows Importance of Fibre Parameters”, Materials and Structures, Volume 25, Number 148, May 1992, p.191-200

36. Kesse, G.K. and Lees, J.M. (2007) "Experimental Behaviour of Reinforced Concrete Beams Strengthened with Prestressed CFRP Shear Straps", *ASCE Journal of Composites for Construction*

37. Lees J.M. (2001). "Fibre-reinforced Polymers in Reinforced and Prestressed Concrete Applications: Moving Forward", *Progress in Structural Engineering and Materials,* **v. 3**, no. 2, April/June, pp. 122-131. ISSN 1365-0556

38. Lees, J.M. and Burgoyne C.J. (2000). "Analysis of Concrete Beams with Partially-Bonded Composite Reinforcement", *ACI Structural Journal,* **v. 97,** No. 2, March-April, pp. 252-258. ISSN 0889-3241

39. Luo X., Sun W. and Chan S. Y. N., (2001). "Steel fibre Reinforced High Performance Concrete: a Study on the Mechanical Properties and Resistance against Impact", Materials and Structures, Volume 34, Number 237, April 2001, p.144-149.

40. Mailhot T., Bissonnette B., Saucier F. and Pigeon M., (2001). "Flexural Fatigue Behaviour of Steel Fibre Reinforced Concrete before and after Cracking", Materials and Structures, Volume 34, Number 240, July 2001, p.351-359.

41. Mehta, P. K., and Monteiro, P. J. M.,(1993): "Concrete Structure, Properties, and Materials", 2ndEd., Prentice-Hall, Englewood Cliffs, N.J., 1993

42. Murdock L.J. and Blackledge G.F., (1968). "Concrete Materials and Practice", London EDWARD ARNOLD (Publishers) Ltd., N. Ireland.

43. Nanni A., and ACSE, (1992). “Properties of Aramid-Fibre Reinforced Concrete and SIFCON”, Journal of Materials in Civil Engineering, Volume 4, Number 1, February 1992, p.1-13.

44. Nawy, E.G., (2001). “Fundamental of High Strength Performance Concrete”, Second Eedition, John Wiley and Sons Inc, Canada.

45. Newman K., (1965). “Properties of concrete, Structural Concrete”, 2, No.11, Sept/Oct, 1965, Reinforced Concrete Association.

46. Neville A.M., (1997). “Properties of Concrete”, John Wiley & sons Inc., New York, United State of America.

47. NIS II (1974). “Specification for Cement”, Nigeria Industrial Standard, Federal Ministry of Industries, Victoria Island, Lagos.

48. Novocon, (2000). “Synthetic Industries”, SI Concrete System, viewed 30 August 2005, <http://www.novocon.com>

49. Patton M. E. and Whittaker W. L., (1983). “Effects of Fibre Content and Damaging Load on Steel Fibre Reinforced Concrete Stiffness”, ACI Journal, Volume 80, Number 1, January-February 1983, p.13-16.

50. Perry B., (2003), “Reinforcing External Pavements with both Large and Small Synthetic fibres”, Concrete online, September 2003, , p.46-47.

51. Rapoport, J, Aldea, C, Shah, S., Ankenman, B. and Karr, A., (2001): “Permeability of cracked steel fibre-reinforced concrete”, Technical Report Number 115, National Institute of Statistical Sciences, USA

52. Riley, V.R. and Reddaway, J.L., (1968). “Tensile Strength and Failure Mechanics of Fibre Composites”, Journal of Materials Science.

53. Romualdi J.P and Mandel J.A (1964). “Tensile Strength of Concrete Affected by Uniformly Distributed Closely Spaced Short Lengths of Wire Reinforcement” ACI Journal, Jun, Vol. 61, No. 6, pp. 657-671

54. Rossi P., Acker P. and Malier Y., (1987): “Effect of Steel Fibres at two different stages: the Material and the Structure”, Materials and Structures, Volume 20, p.436-439.

55. Sanjuan M.A., Andrade C, and Bentur A., (1998), “Effect of Polypropylene Fibre Reinforced Mortars on Steel Reinforcement Corrosion Induced by Carbonation”, Materials and Structures, Volume 31, Number 209, June 1998, p.343-349.

56. Sener S., Begimgil M. and Belgin C., (2002). “Size Effect on Failure of Concrete Beams with and without Steel Fibers”, Journal of Materials in Civil Engineering, Volume 14, Number 5, September-October 2002, p436-440.

57. Shah S. P,and Rangan B. V. (1970). “Effect of Reinforcement on Ductility of Concrete”. Journal of Structural Engineering, Jun, Vol. 96, No. ST6, pp. 1167-1184.

58. Stang, H. (1992) “Evaluation of Properties of Cementitious Fiber Composite Materials”, High Performance Fiber Reinforced Cement Composites, Vol. 1. (eds) H.W. Reinhardt and A.E. Naaman. E & FN Spon, London, 1992. pp. 388–406.

59. Sullivan B.W., (2001)., “Part C: Concrete Technology”, CIV2605 Construction Engineering, DEC, Toowoomba, Australia.

60. Swamy R.N. and Sa’ad A. Al-Ta’an, (1981). “Deformation and Ultimate Strength in Flexure of Reinforced Concrete Beams Made with Steel Fibre Concrete”, ACI Material, Volume 78, Number 5, September-October 1981, p.395-405.

61. Synthetic Industries Concrete Company, (2000). “Synthetic Industries”, SI Concrete System, viewed 30 August 2005, <http://www.novocon.com>

62. Tan K.H., Murugappan K., and Paramasivam P., (1993). “Shear Behavior of Steel Fibre Reinforced Concrete Beams”, ACI Structural Journal, Volume 90, Number 1, January- February 1993, p.3-11.

63. Tat-Seng Lok, ASCE, and Jin-Song Pei, (1998). “Flexural Behaviour of Steel Fibre Reinforced Concrete”, Journal of Materials, Volume 10, Number 2, May 1998, p.86- 97.

64. Trottier J. and Banthia N., (1994). “Toughness Characterization of Steel-Fiber Reinforced Concrete”, Journal of Materials in Civil Engineering, Volume 6, Number 2, May 1994, p.264-289.

65. Troxell G.E., Davis H.E. and Kelly J.W., (1968). “Composition and Properties Of Concrete”, McGraw-Hill Inc., New York, United State of America.

66. U.S. Environmental Protection Agency et al,(1993). “Scrap Tire Technology and Markets” Noyes Data Corporation, NJ 1993.

67. Wafa L and Nick E. (2004). “An Investigation into the Use of Fibres in Concrete Industrial Ground-Floor Slabs”. School of the Build Environment, Liverpool John Moores University, Liverpool,L3 3AF. W.A.Mohammad@2004.ljmu.ac.uk

68. Wang Y., Wu H. C. and Victor C. Li, (2000). “Concrete Reinforcement with Fibres”, Journal of Materials in Civil Engineering, Volume 12, Number 4, November 2000, p.314-319.

69. Vandewalle L., (2000). Cracking Behaviour of Concrete Beams Reinforced with Combination of Ordinary Reinforcement and Steel Fibres, Materials and Structures, Volume 33, Number 227, April 2000, p.164-170.

70. Zaher, K. K., Bayomy, F. M., (1999). “Rubberized Portland Cement Concrete” Journal of Materials in Civil Engineering, Vol. 11, No. 3, 1999, pp. 206-213.

APPENDICES

APPENDIX 1: SAMPLE DETAILED CALCULATION OF QUANTITIES OF MATERIALS FOR ONE MORTAR BEAM AND ONE CONCRETE BEAM

i) DETAILED CALCULATION OF QUANTITIES OF MATERIALS FOR ONE MORTAR BEAM

Sample detailed calculation of quantities of materials for one mortar beam, at a prescribed mix of one part cement to two parts sand (1:2) and a water cement ratio of 0.56.

The Dimensions of one beam are length = 700mm Width = 150mm and Depth = 150mm

Volume of beam = (700 X 150 X 150) X $10^{-9}m^3$

= 15.75 X $10^{-3}m^3$

Assuming a density of mortar = 2100 Kg m^{-3}

Mass = Volume X Density.

Mass of one mortar beam = 15.75 X $10^{-3}m^3$ X 2100 Kg m^{-3}

= 33.075 Kg

Considering 10% shrinkage and waste

ie mass of one mortar beam = 33.075 + 33.075 x10/100

= 33.075 + 3.3075 = 36.45kg

Taking a mortar mix of 1:2 and a water cement ratio of 0.56 ie Total ratio = 3

Mass of cement = 1/3 X 36.45kg = 12.13kg

Mass of sand = 2/3 X 36.45kg = 24.26kg

Mass of water = 0.56 X 12.13kg = 6.063kg

Mass of steel fibre at 0.5% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X $10^{-3}m^3$

0.5% of total volume of beam = 15.75 X $10^{-3}m^3$ X 0.5/100

Mass of fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 0.5/100 X7.8 kg

= 0.61g

Mass of steel fibre at 1.0% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X $10^{-3}m^3$

1.0% of total volume of beam = 15.75 X $10^{-3}m^3$ X 1.0/100

Mass of fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 1.0/100 X7.8 kg

= 1.23g

Mass of steel fibre at 1.50% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X 10^{-3} m^3

1.50% of total volume of beam = 15.75 X $10^{-3}m^3$ X 1.5/100

Mass of fibre at 0.5% = 15.75 X 10^{-3} m^3 X 1.5/100 X7.8 kg

= 1.84 g

Mass of steel fibre at 2.0% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X 10^{-3} m^3

2.0% of total volume of beam = 15.75 X $10^{-3}m^3$ X 2.0/100

Mass of fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 2.0/100 X7.8 kg

= 2.46g

ii) DETAILED CALCULATION OF QUANTITIES OF MATERIALS FOR ONE CONCRETE BEAM

Detailed calculation of quantities of materials for one concrete beam, at a prescribed mix of one part cement to two parts sand to four parts coarse (1:2:4) and a water cement ratio of 0.60.

The Dimensions of one beam are length = 700mm Width = 150mm and Depth = 150mm

Volume of beam = (700 X 150 X 150) X $10^{-9}m^3$

= 15.75 X $10^{-3}m^3$

Assuming a density of concrete = 2400 Kg m^{-3}

Mass = Volume X Density.

Mass of one concrete beam = 15.75 X $10^{-3}m^3$ X 2400 Kg m^{-3}

= 37.80Kg

Considering 10% shrinkage and waste

ie mass of one concrete beam = 37.80 + 37.80 x10/100

= 37.80 + 3.78 = 41.58kg

Taking a concrete mix of 1:2:4 and water cement ratio of 0.60. Total ratio = 7

Mass of cement = 1/7 X 41.58kg = 5.94kg

Mass of sand = 2/7 X 41.58kg = 11.88kg

Mass of coarse = 4/7 X 41.58kg = 23.76kg

Mass of water = 0.60 X 5.94kg = 3.56kg

Mass of steel fibre at 0.5% fibre volume dosage for one concrete beam

Total volume of beam = 15.75 X $10^{-3}m^3$

0.5% of total volume of beam = 15.75 X $10^{-3}m^3$ X 0.5/100

Mass of steel fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 0.5/100 X 7.8 kg

= 0.61g

Mass of steel fibre at 1.0% by fibre volume dosage for one concrete beam

Total volume of beam = 15.75 X $10^{-3}m^3$

1.0% of total volume of beam = 15.75 X $10^{-3}m^3$ X 1.0/100

Mass of steel fibre at 1.0% = 15.75 X $10^{-3}m^3$ X 1.0/100 X 7.8 kg

= 1.23g

Mass of steel fibre at 1.5% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X $10^{-3}m^3$

1.50% of total volume of beam = 15.75 X $10^{-3}m^3$ X 1.5/100

Mass of fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 1.5/100 X 7.8 kg

= 1.84g

Mass of steel fibre at 2.0% by fibre volume dosage for one mortar beam

Total volume of beam = 15.75 X $10^{-3}m^3$

0.5% of total volume of beam = 15.75 X $10^{-3}m^3$ X 2.0/100

Mass of fibre at 0.5% = 15.75 X $10^{-3}m^3$ X 2.0/100 X 7.8 kg

= 2.46g

APPENDIX 2: - SIEVE ANALYSIS FOR FINE AGGREGATE SAMPLES: TOTAL WEIGHT OF AGGREGATE: 1KG

BS eve size	Weight Retained (g)			Percentage Retained			Cumulative Percentage Passing			Cumulative Percentage Retained		
	S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	
.75mm	20	40	60	2.0	4.0	6.0	98.0	96.0	94.0	2.0	4.0	
.36mm	95	120	250	9.5	12.0	25.0	88.5	84.0	69.0	11.5	6.0	
.18mm	210	60	100	21.0	6.0	10.0	67.5	78.0	59.0	32.5	22.0	
600 µm	270	140	115	27.0	14.0	11.5	40.5	64.0	47.5	59.5	36.0	

300 µm	360	450	350	36.0	45.0	35.0	4.5	19.0	12.5	95.5	81.0	
150µm	20	160	90	2.0	16.0	9.0	2.5	3.0	3.5	97.5	97.0	

SAMPLE 1 (S1), SAMPLE 2 (S2) AND SAMPLE 3 (S3)

APPENDIX 3: SIEVE ANALYSIS FOR COARSE AGGREGATE SAMPLES
TOTAL WEIGHT OF AGGREGATE: 3KG

BS ve size mm)	Weight Retained (Kg)			Percentage Retained			Cumulative Percentage Passing			Cumulative Perce Retained		
	S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	
8.1	-	0.1	-	-	3.3	-	100	96.7	100	-	3.3	
5.4	0.1	0.3	-	3.3	10.0	-	96.7	86.7	100	3.3	13.3	
0.0	0.5	0.5	0.3	16.7	16.7	10.0	80.0	70.0	90.0	20.0	30.0	
4.0	1.1	0.9	0.7	36.7	30.0	23.3	43.4	40.0	66.7	56.6	60.0	
0.0	0.8	0.7	1.2	26.7	23.3	40.0	16.7	16.7	26.7	83.3	83.0	
.35	0.4	0.3	0.5	13.3	10.0	16.7	3.37	6.7	10.0	96.3	93.0	
.00	0.1	0.2	0.3	2.7	6.7	10.0	0.0	0.0	0.0	99.3	99.7	

SAMPLE 1 (S1) SAMPLE2 (S2) AND SAMPLE3 (S3)

APPENDIX 4: CONSISTENCY TEST ON DANGOTE (OPC)

Property	Value			
	Sample I	Sample II	Sample III	Average
Normal Consistency (%)	33.5	35.0	35.0	34.5
Initial Setting Time (min)	99.0	94.0	74.0	89.0

Final Setting Time (min)	149.0	139.0	156.0	148.0
Soundness (mm)	1.80	1.95	2.25	2
Specific Gravity	3.12	3.24	3.06	3.14

APPENDIX 5: Compressive Strengths Test Results for Dangote cement

- (Mortar cubes)

Fibre Percentage (%)	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)
3 – days	7.19	2130.4	303.8	13.5	7.08	2097.8	348.8	15.5	7.04	2086.0	326.3	14.5
7- days	7.11	2106.7	477.0	21.2	7.16	2121.5	562.5	25.0	7.02	2080.0	596.3	26.5
28 – days	7.02	2080.0	585.0	26.0	7.13	2112.6	697.5	31.0	6.94	2056.3	573.8	25.5

APPENDIX 6: Compressive Strengths Test Results for the Four Specimens

- CSF (Mortar cubes)

Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
Specimen I				Specimen II				Specimen III				Specime			
Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	
7.67	2272.6	618.8	27.5	7.33	2171.9	614.3	27.3	7.26	2151.1	623.30	27.7	7.23	2142.3	6	
7.50	2222.3	639.0	28.4	7.20	2133.4	639.0	28.4	7.13	2112.6	636.75	28.3	7.21	2136.3	6	
7.61	2254.8	657.0	29.2	7.31	2165.9	659.3	29.3	7.42	2198.5	659.25	29.3	7.42	2198.5	6	
7.51	2225.2	733.50	32.6	7.24	2145.2	735.8	32.7	7.51	2225.2	740.25	32.9	7.60	2251.9	7	
7.32	2168.9	670.5	29.8	7.62	2257.8	675	30.0	7.80	2311.1	672.75	29.9	7.91	2343.7	6	

APPENDIX 7: Compressive Strengths Test Results for the Four Specimens

- RSF (Mortar cubes)

Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
Specimen I				Specimen II				Specimen III				Specime			
Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	

	7.67	2272.6	618.80	27.5	7.33	2171.9	614.3	27.3	7.26	2151.1	623.30	27.7	7.23	2142.3	6
	7.51	2225.2	636.75	28.3	7.46	2210.4	636.65	28.3	7.46	2210.4	643.50	28.6	7.49	2219.3	6
	7.51	2225.2	666.00	29.6	7.41	2195.6	663.75	29.5	7.71	2284.5	672.75	29.9	7.48	2216.3	6
	7.59	2248.9	715.50	31.8	7.57	2242.9	681.75	30.3	7.69	2278.5	699.75	31.1	7.91	2343.7	6
	8.02	2376.3	681.75	30.3	7.67	2272.6	708.75	31.5	7.92	2346.7	704.25	31.3	7.88	2334.8	7

APPENDIX 8: Compressive Strengths Test Results for the Four Specimens - CHSF (Mortar cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)														
	Specimen I				Specimen II				Specimen III				Specime		
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F
	7.67	2272.6	618.80	27.5	7.33	2171.9	614.3	27.3	7.26	2151.1	623.30	27.7	7.23	2142.3	6
	7.26	2151.1	632.25	28.1	6.95	2059.3	641.25	28.5	7.39	2189.7	650.25	28.9	7.04	2085.9	6
	7.54	2234.1	657.00	29.2	7.21	2136.3	675.00	30.0	7.36	2180.8	679.50	30.2	7.14	2115.6	6
	7.65	2266.7	744.75	33.1	7.36	2180.8	717.75	31.9	7.11	2106.7	751.50	33.4	7.02	2080.0	7

	7.74	2293.4	681.75	30.3	7.19	2130.4	681.75	30.3	7.30	2162.9	693.00	30.8	7.09	2100.8	6

APPENDIX 9: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - CSF (Mortar Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	
0	35.6	2260.2	30.75	4.10	35.2	2234.9	24.75	3.30	35.8	2272.9	18.75	
0.5	34.7	2203.1	33.75	4.50	35.6	2260.2	36.75	4.90	35.3	2241.2	31.50	
1.0	35.7	2266.6	35.25	4.70	35.6	2260.2	34.50	4.60	35.7	2266.6	36.00	
1.5	35.6	2260.2	35.18	4.69	35.8	2272.9	38.33	5.11	36.0	2285.6	37.58	
2.0	35.7	2266.6	46.05	6.14	35.9	2279.3	35.40	4.72	36.4	2311.0	35.93	

APPENDIX 10: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - RSF (Mortar Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	
0	35.6	2260.2	30.75	4.10	35.2	2234.9	24.75	3.30	35.8	2272.9	18.75	
0.5	35.3	2241.2	30.45	4.06	35.2	2234.9	31.05	4.14	35.0	2222.2	40.95	
1.0	35.6	2260.2	34.73	4.63	35.5	2253.9	38.40	5.12	35.1	2228.5	30.75	
1.5	35.9	2279.3	37.50	5.00	36.1	2292.0	37.65	5.02	35.6	2260.2	38.40	
2.0	36.1	2292.0	37.43	4.99	36.5	2317.4	31.20	4.16	36.2	2298.3	50.25	

APPENDIX 11: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - CHSF (Mortar Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	
0	35.6	2260.2	30.75	4.10	35.2	2234.9	24.75	3.30	35.8	2272.9	18.75	
0.5	35.1	2228.5	31.92	4.56	35.5	2253.9	33.98	4.53	35.8	2272.9	44.03	
1.0	34.8	2209.5	36.0	4.80	34.9	2215.8	54.30	7.24	35.4	2247.6	27.15	
1.5	34.8	2209.5	39.15	5.22	34.6	2196.8	41.33	5.51	34.7	2203.1	37.58	
2.0	34.2	2171.4	36.6	4.88	34.5	2190.4	44.55	5.94	34.3	2177.7	36.83	

APPENDIX 12: Slump Test Results for Three Samples - (CSF)

Fibre Percentage (%)	Sample I			Sample II			Sample III		
	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)
0	300	139	161	300	133	167	300	130	170
0.5	300	138	162	300	138	162	300	144	156
1.0	300	155	145	300	148	152	300	150	150
1.5	300	156	144	300	154	146	300	170	130
2.0	300	181	119	300	185	115	300	156	144

APPENDIX 13: Slump Test Results for Three Samples - (RSF)

Fibre Percentage (%)	Sample I			Sample II			Sample III		
	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)
0	300	139	161	300	133	167	300	130	170
0.5	300	139	161	300	140	160	300	144	156
1.0	300	131	169	300	146	154	300	152	148
1.5	300	180	120	300	167	133	300	163	137
2.0	300	181	139	300	180	120	300	179	121

APPENDIX 14: Slump Test Results for the Samples - (CHSF)

Fibre Percentage (%)	Sample I			Sample II			Sample III		
	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)	Height of Cone (mm)	Height of Concrete (mm)	Slump (mm)
0	300	139	161	300	133	167	300	130	170
0.5	300	182	118	300	169	131	300	189	111
1.0	300	200	100	300	191	109	300	203	97
1.5	300	210	90	300	217	83	300	221	79
2.0	300	244	66	300	219	81	300	257	43

APPENDIX 15: Compacting Factor Test Results for Three Samples - CSF.

Fibre Percentage (%)	Sample I		Sample II		Sample III	
	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass
0	21.00	21.24	21.39	21.43	20.88	20.92
0.5	19.28	19.92	19.31	19.93	19.42	20.61
1.0	18.99	20.33	20.04	21.23	19.34	20.53
1.5	19.19	21.04	19.28	21.54	19.88	21.54
2.0	18.54	20.85	17.21	19.98	18.31	20.59

APPENDIX 16: Compacting Factor Test Results for Three Samples - RSF.

Fibre Percentage (%)	Sample I		Sample II		Sample III	
	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass
0	21.00	21.24	21.39	21.43	20.88	20.92
0.5	20.53	20.91	19.38	19.77	19.63	19.83
1.0	20.82	21.22	20.75	21.19	21.60	22.04
1.5	19.08	20.34	18.93	20.14	19.62	20.83
2.0	17.96	20.25	18.45	20.71	18.64	20.76

APPENDIX 17: Compacting Factor Test Results for Three Samples - CHSF.

Fibre Percentage (%)	Sample I		Sample II		Sample III	
	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass	Partially Compacted Mass	Fully Compacted Mass
0	21.00	21.24	21.39	21.43	20.88	20.92
0.5	19.60	21.44	18.56	20.83	18.70	20.82
1.0	18.93	21.13	18.65	20.84	18.17	20.93
1.5	17.40	20.81	17.19	20.71	16.92	20.61
2.0	16.16	20.23	16.50	20.68	16.48	20.44

APPENDIX 18: Compressive Strengths Test Results for the Four Specimens - CSF (Concrete cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
	Specimen I				Specimen II				Specimen III				Specime			
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	
	8.00	2370.4	756.00	33.6	7.68	2275.6	765.00	34.0	7.91	2343.7	760.5	33.8	8.02	2376.3	7	
	8.13	2408.9	774.00	34.4	8.21	2432.6	774.00	34.4	8.03	2379.3	774.00	34.4	8.17	2420.8	7	
	8.22	2435.6	904.50	40.2	8.37	2480.0	906.75	40.3	8.29	2456.3	909.00	40.4	8.10	2400.0	9	
	8.50	2518.6	972.00	43.2	8.53	2527.4	981.00	43.6	8.63	2557.1	929.25	41.3	8.18	2423.7	9	
	8.66	2565.9	1077.8	47.9	8.64	2560.0	1068.8	47.5	9.01	2669.7	1082.3	48.1	8.50	2518.6	1	

APPENDIX 19: Compressive Strengths Test Results for the Four Specimens - RSF (Concrete cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
	Specimen I				Specimen II				Specimen III				Specime			
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	
	8.00	2370.4	756.00	33.6	7.68	2275.6	765.00	34.0	7.91	2343.7	760.5	33.8	8.02	2376.3	7	
	7.99	2367.4	778.50	34.6	8.22	2435.6	769.50	34.2	8.25	2444.5	762.75	33.9	8.04	2382.3	7	
	8.03	2379.3	868.5	38.6	8.57	2539.3	873.00	38.8	8.45	2503.7	915.75	40.7	8.22	2435.6	9	
	8.63	2557.1	956.25	42.5	8.89	2634.1	924.75	41.1	8.65	2562.9	938.25	41.7	8.34	2471.1	9	
	8.95	2651.9	1012.5	45.0	8.59	2545.2	994.50	44.2	8.53	2527.4	992.25	44.1	8.39	2485.9	9	

APPENDIX 20: Compressive Strengths Test Results for Four Specimens at each Fibre Percentage - CHSF (Concrete cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
	Specimen I				Specimen II				Specimen III				Specime			
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	
	8.00	2370.4	756.00	33.6	7.68	2275.6	765.00	34.0	7.91	2343.7	760.50	33.8	8.02	2376.3	7	
	7.74	2293.4	774.00	34.4	7.87	2331.9	756.00	33.6	8.23	2438.5	785.25	34.9	7.88	2334.8	8	
	8.11	2402.9	785.25	34.9	8.09	2397.1	801.00	35.6	8.34	2471.1	798.75	35.5	8.20	2429.7	8	
	7.89	2337.80	726.75	32.3	7.90	2340.77	729.00	32.4	8.03	2379.89	697.50	31.0	8.04	2382.25	6	
	8.04	2382.3	454.5	20.2	7.88	2334.84	479.25	21.3	7.92	2346.70	474.75	21.1	7.96	2358.55	4	

APPENDIX 21: Tensile Strengths Test Results for Three Specimens at each Fibre Percentage - CSF (Concrete Cylinders).

Fibre rcentage (%)	Tensile Strengths of Specimen (N/mm^2) at 28 days (Diameter of Cylinder =150mm, Height of Cylinder = 300mm)										
	Specimen I				Specimen II				Specimen III		
	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt.of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)
0	12.2	2302.1	178.7	2.53	12.3	2321.0	213.3	3.02	12.2	2302.1	200.7
0.5	11.9	2245.5	218.3	3.09	12.4	2339.9	358.9	5.08	12.6	2377.6	264.2
1.0	12.4	2339.9	348.3	4.93	12.4	2339.9	244.5	3.46	12.3	2321.0	344.1
1.5	12.5	2264.4	320.1	4.53	12.3	22321.0	299.6	4.24	12.8	2415.4	308.7
2.0	12.4	2339.9	299.6	4.24	12.5	2264.4	320.8	4.54	13.1	2471.9	253.6

APPENDIX 22: Tensile Strengths Test Results for Three Specimens at each Fibre Percentage - RSF (Concrete Cylinders).

Fibre rcentage (%)	Tensile Strengths of Specimen (N/mm^2) at 28 days (Diameter of Cylinder =150mm, Height of Cylinder = 300mm)										
	Specimen I				Specimen II				Specimen III		
	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt.of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)

0	12.2	2302.1	178.7	2.53	12.3	2321.0	213.4	3.02	12.2	2302.1	200.7
0.5	12.4	2339.9	229.6	3.25	12.1	2283.3	315.8	4.47	12.4	2339.9	185.1
1.0	12.2	2302.1	315.8	4.47	12.4	2339.9	316.5	4.48	12.4	2339.9	304.5
1.5	12.4	2339.9	228.2	3.23	12.9	2434.2	295.3	4.18	12.5	2264.4	298.1
2.0	12.8	2415.4	286.1	4.05	13.2	2490.8	285.4	4.04	13.0	2453.1	335.6

APPENDIX 23: Tensile Strengths Test Results for Three Specimens at each Fibre Percentage - CHSF (Concrete Cylinders).

Fibre rcentage (%)	Tensile Strengths of Specimen (N/mm^2) at 28 days (Diameter of Cylinder =150mm, Height of Cylinder = 300mm)										
	Specimen I				Specimen II				Specimen III		
	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt.of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)	Tensile Strength (N/mm^2)	Wt. of Cylinder (kg)	Density (kg/m^3)	Failure Load (kN)
0	12.2	2302.1	178.7	2.53	12.3	2321.0	213.4	3.02	12.2	2302.1	200.7
0.5	12.2	2302.1	286.1	4.05	12.4	2339.9	276.2	3.91	12.3	2321.0	240.2
1.0	12.5	2264.4	340.5	4.82	12.6	2377.6	303.1	4.29	12.7	2396.5	287.6
1.5	12.1	2283.3	325.0	4.60	12.3	2321.0	262.1	3.71	12.6	2377.6	347.6
2.0	12.2	2302.1	356.1	5.04	12.1	2283.3	343.4	4.86	12.2	2302.1	371.6

APPENDIX 24: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - CSF (Concrete Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)										
	Specimen I				Specimen II				Specimen III		
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)
0	37.2	2361.8	27.15	3.62	37.1	2355.5	30.00	4.00	37.0	2349.1	31.50
0.5	37.3	2368.2	36.30	4.84	37.3	2368.2	48.60	6.48	37.4	2374.5	29.85
1.0	37.7	2393.6	44.55	5.94	37.5	2380.9	33.68	4.49	37.0	2349.1	41.70
1.5	37.4	2374.5	38.33	5.11	37.7	2393.6	39.45	5.26	36.9	2342.8	45.30
2.0	37.9	2406.3	43.65	5.82	38.2	2425.3	41.93	5.59	37.6	2387.2	42.00

APPENDIX 25: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - RSF (Concrete Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	
0	37.2	2361.8	27.15	3.62	37.1	2355.5	30.00	4.00	37.0	2349.1	31.50	
0.5	36.9	2342.8	37.05	4.94	37.4	2374.5	39.60	5.28	37.1	2355.5	38.78	
1.0	37.4	2374.5	36.90	4.92	37.2	2361.8	39.00	5.20	37.5	2380.9	35.93	
1.5	37.8	2399.9	43.95	5.86	37.5	2380.9	40.58	5.41	38.0	2412.6	36.98	
2.0	37.9	2406.3	47.85	6.38	38.0	2412.6	46.20	6.16	38.1	2418.9	48.38	

APPENDIX 26: Flexural Strengths Test Results for Three Specimens at each Fibre Percentage - CHSF (Concrete Beams).

Fibre ercentage (%)	Flexural Strength of Specimen (N/mm^2) at 28 days (Length of Beam =700mm, Width = 150mm and Depth = 150mm)											
	Specimen I				Specimen II				Specimen III			
	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt.of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	Flexural. Strength (N/mm^2)	Wt. of Beam (kg)	Density (kg/m^3)	Failure Load (kN)	
0	37.2	2361.8	27.15	3.62	37.1	2355.5	30.00	4.00	37.0	2349.1	31.50	

0.5	36.9	2342.8	40.65	5.42	36.9	2342.8	45.75	6.10	37.0	2349.1	36.68	
1.0	37.3	2368.2	36.08	4.81	37.2	2361.8	41.18	5.49	37,5	2380.9	36.83	
1.5	36.8	2336.4	42.08	5.61	36.6	2323.7	51.45	6.86	36.9	2342.8	38.55	
2.0	37.0	2349.1	40.13	5.35	36.6	2323.7	45.68	6.09	36.8	2336.4	37.95	

APPENDIX 27: Load (kN)/ Deflection (mm) for Circular Steel Fibre (CSF)

Fibre Percentage (%)	Load (kN)/ Deflection (mm) CSF												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.0532	0.098	0.170	0.237	0.251	0.313	0.362	0.374	-	-	-	-
		0.0528	0.112	0.167	0.242	0.266	0.320	0.372	0.379	-	-	-	-
		0.0530	0.105	0.167	0.232	0.293	0.315	0.370	0.378	-	-	-	-
0.5	0	0.0552	0.106	0.236	0.319	0.340	0.348	0.354	-	-	-	-	-
		0.0540	0.104	0.238	0.313	0.339	0.352	0.372	-	-	-	-	-
		0.0498	0.105	0.237	0.318	0.343	0.356	0.360	-	-	-	-	-
1.0	0	0.0536	0.106	0.212	0.268	0.297	0.371	0.404	0.474	-	-	-	-
		0.053	0.103	0.211	0.262	0.305	0.367	0.401	0.476	-	-	-	-
		0.0524	0.106	0.207	0.259	0.301	0.366	0.395	0.472	-	-	-	-
1.5	0	0.0525	0.101	0.239	0.271	0.284	0.320	0.343	0.371	0.436	0.525	0.631	-
		0.0540	0.099	0.237	0.267	0.294	0.316	0.339	0.370	0.428	0.523	0.627	-
		0.0528	0.112	0.235	0.272	0.295	0.309	0.344	0.363	0.432	0.530	0.638	-
2.0	0	0.0529	0.101	0.150	0.213	0.289	0.318	0.339	0.369	0.397	0.502	0.616	0.745
		0.0534	0.105	0.142	0.216	0.287	0.310	0.336	0.368	0.402	0.499	0.614	0.734
		0.0530	0.106	0.149	0.201	0.285	0.305	0.351	0.367	0.404	0.505	0.600	0.747

APPENDIX 28: Load (kN)/ Deflection (mm) for Rectangular Steel Fibre (RSF)

Fibre Percentage (%)	Load (kN)/ Deflection (mm)RSF												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.0532	0.098	0.170	0.237	0.251	0.313	0.362	0.374	-	-	-	-
		0.0528	0.112	0.167	0.242	0.266	0.320	0.372	0.379	-	-	-	-
		0.0530	0.105	0.167	0.232	0.293	0.315	0.370	0.378	-	-	-	-
0.5	0	0.056	0.106	0.235	0.321	0.340	0.359	0.370	-	-	-	-	-
		0.057	0.104	0.235	0.315	0.329	0.353	0.367	-	-	-	-	-

		0.049	0.120	0.238	0.312	0.333	0.353	0.361	-	-	-	-	-
1.0	0	0.056	0.108	0.208	0.253	0.304	0.310	0.416	0.462	-	-	-	-
		0.049	0.107	0.211	0.252	0.302	0.304	0.403	0.431	-	-	-	-
		0.060	0.109	0.211	0.251	0.312	0.310	0.420	0.460	-	-	-	-
1.5	0	0.052	0.110	0.234	0.268	0.294	0.329	0.342	0.365	0.498	0.532	0.674	-
		0.056	0.106	0.238	0.264	0.269	0.328	0.358	0.376	0.504	0.528	0.659	-
		0.054	0.108	0.242	0.275	0.307	0.330	0.350	0.366	0.498	0.524	0.668	-
2.0	0	0.053	0.109	0.131	0.192	0.291	0.330	0.343	0.389	0.411	0.499	0.580	0.734
		0.049	0.116	0.133	0.208	0.300	0.341	0.338	0.389	0.411	0.494	0.581	0.734
		0.060	0.111	0.123	0.200	0.282	0.320	0.348	0.388	0.408	0.502	0.579	0.722

APPENDIX 29: Load (kN)/ Deflection (mm) for Chips Steel Fibre (CHSF)

Fibre Percentage (%)	**Load (kN)/ Deflection (mm) CHSF**												
	0	5	10	15	20	25	30	35	40	45	50	55	60
0	0	0.0532	0.098	0.170	0.237	0.251	0.313	0.362	0.374		-	-	-
		0.0528	0.112	0.167	0.242	0.266	0.320	0.372	0.379		-	-	-
		0.0530	0.105	0.167	0.232	0.293	0.315	0.370	0.378		-	-	-
0.5	0	0.062	0.109	0.181	0.189	0.247	0.312	0.324	0.335	0.436	-	-	-
		0.067	0.110	0.172	0.194	0.248	0.307	0.319	0.350	0.430	-	-	-
		0.057	0.114	0.181	0.190	0.252	0.308	0.320	0.341	0.430	-	-	-
1.0	0	0.064	0.113	0.174	0.249	0.250	0.325	0.402	0.418	-	-	-	-
		0.064	0.109	0.181	0.246	0.249	0.327	0.391	0.422	-	-	-	-
		0.058	0.111	0.179	0.246	0.248	0.326	0.410	0.420	-	-	-	-
1.5	0	0.059	0.060	0.078	0.114	0.156	0.179	0.269	0.274	-	-	-	-
		0.054	0.061	0.075	0.109	0.151	0.187	0.271	0.279	-	-	-	-
		0.061	0.059	0.069	0.110	0.155	0.189	0.267	0.281	-	-	-	-
2.0	0	0.053	0.060	0.071	0.096	0.148	0.181	0.258	0.261	-	-	-	-
		0.057	0.065	0.071	0.109	0.149	0.178	0.259	0.260	-	-	-	-

		0.049	0.064	0.068	0.092	0.150	0.175	0.266	0.259	-	-	-	-

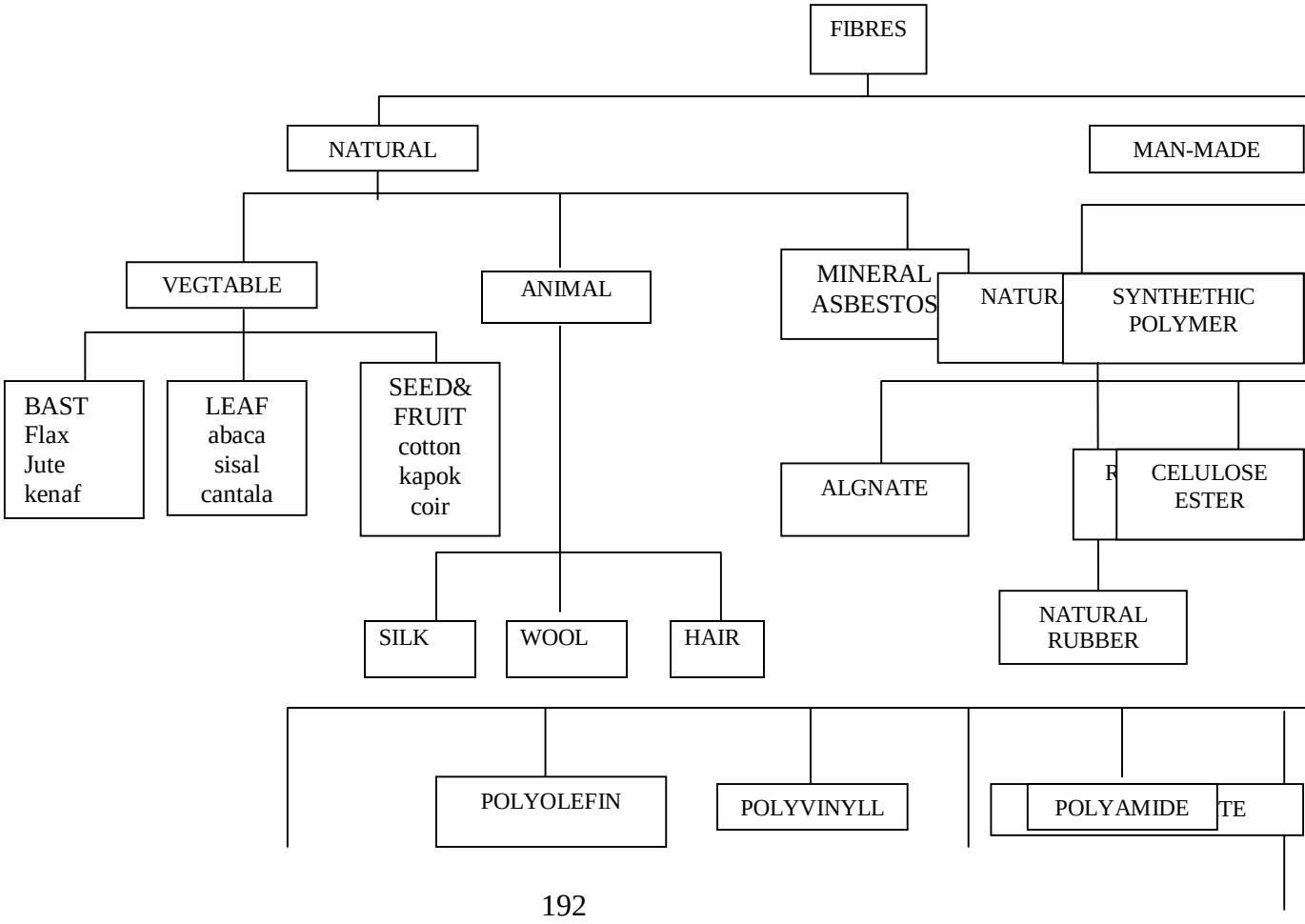
APPENDIX 30: Compressive Strengths Test Results for Confirmation Specimens at each Fibre Percentage - CHSF (Concrete Cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)															
	Specimen I				Specimen II				Specimen III				Specime			
	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F	
	7.42	2198.6	753.8	33.5	8.11	2402.9	742.5	33.0	7.76	2299.3	760.5	33.8	7.95	2355.6	7	
	7.32	2168.9	792.0	35.2	8.04	2382.3	805.5	35.8	7.81	2314.1	774.0	34.4	7.23	2142.3	7	
	8.15	2414.9	821.3	36.5	7.94	2352.6	794.3	35.3	7.32	2168.9	805.5	35.8	7.43	2201.5	7	
	7.80	2311.1	857.3	38.1	7.58	2245.9	810.0	36.0	7.35	2177.8	828.0	36.8	7.90	2340.8	8	
	7.50	2222.3	675.0	30.0	7.15	2118.6	690.8	30.7	7.05	2088.9	672.8	29.9	6.90	2044.5	6	
	7.12	2109.7	436.5	19.4	6.87	2035.6	533.3	23.7	-	-	-	-	-	-		
	-	-	-	-	6.89	2041.5	463.50	20.6	-	-	-	-	-	-		
	-	-	-	-	-	-	-	-	-	-	-	-	-	-		

Compressive Strengths Test Results for Confirmation Specimens at each Fibre Percentage - CHSF (Concrete Cubes)

e	Compressive Strength of Specimen (N/mm^2) at 28 days (Volume of Cube = 3.375 X 10^{-3} m^3 and Area of Cube = 22.5 X 10^{-3} m^2)			
	Specimen V	Specimen VI	Specimen VII	Specimen

	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt.of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	Failure Load (kN)	Compr. Strength (N/mm^2)	Wt. of Cube (kg)	Density (kg/m^3)	F
	8.01	2373.7	546.8	34.3	7.65	2266.7	778.5	34.6	7.64	2263.7	733.5	32.6	7.90	2340.8	7
	7.54	2234.4	828.0	36.8	7.90	2340.8	814.5	36.2	7.73	2290.4	807.8	35.9	7.34	2174.8	7
	7.42	2198.6	828.0	36.8	7.85	2326.0	807.8	35.9	7.41	2195.6	850.5	37.8	7.20	2133.4	7
	7.21	2136.3	796.5	35.4	7.23	2142.3	821.3	36.5	7.62	2257.8	801.0	35.6	7.42	2198.6	7
	7.40	2192.6	693.0	30.8	6.98	2068.2	697.5	31.0	7.95	2355.6	675.0	30.0	7.05	2088.9	6
	-	-	-	-	-	-	-	-	6.45	1911.1	387.0	17.2	6.55	1940.8	4
	-	-	-	-	6.05	1792.6	369.0	16.4	-	-	-	-	6.05	1792.6	4
	-	-	-	-	-	-	-	-	-	-	-	-	-	-	



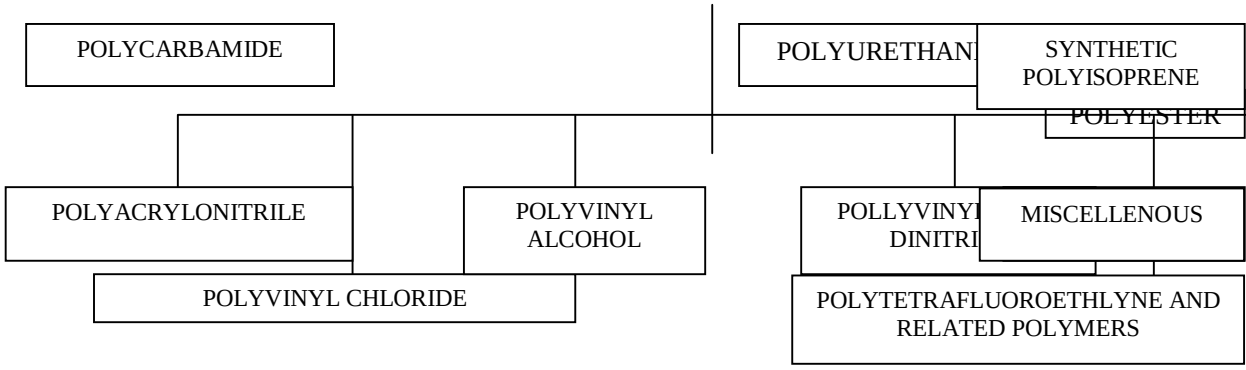


FIGURE 2,1 Classification of Fibres