Geotechnics.
Highway & Transportation.
Water Engineering.

Published by Department of Civil Engineering Nnamdi Azikiwe University Awka, Nigeria

Website: https://naujcve.com/index.php/FINAL-YEAR-PROJECTS/index

Table of Contents

GEOTECHNICS

EVALUATION OF STRENGTH CHARACTERISTICS OF CLAY STABILIZED WITH SUGAR CANE STRAW ASH AND LIME **i-108**

NNADI MODESTUS AFAMEFULE
(NAU/2016224013)

DESICCATION INDUCED SHRINKAGE BEHAVIOUR OF LATERITIC SOILS **i--85**

UMEADI ANTHONY CHUKWUGOZIE
NAU/2015224072

HIGHWAY & TRANSPORTATION

INVESTIGATION ON THE ECONOMIC AND LIFE CYCLE COST BENEFITS OF CONCRETE OVER ASPHALT PAVEMENT (USING ISIEKE EBONYI STATE, AS A CASE STUDY) **i-66**

NWANKWOEKE CHINEDU SHEDRACK
2017224054

INVESTIGATION INTO THE SAFETY CONSIDERATIONS FOR PEDESTRIANS ALONG THE AWKA SEGMENT OF ENUGU┐ ONITSHA EXPRESSWAY **i-57**

OKAFOR FAVOUR KASIEMOBI
(NAU/ 2016224034)

WATER ENGINEERING

EVALUATION OF BANANA PEEL POWDER AS NATURAL COAGULANT FOR TREATMENT OF WASTEWATER **i-52**

ANYAKORA SOMTO VALENTINE
REG. NO: NAU/2016224049

ANALYSIS AND DESIGN OF TREATMENT FACILITY FOR WASTEWATER FROM BOY'S HOSTEL, NNAMDI AZIKIWE UNIVERSITY, AWKA USING WASTE STABILIZATION PONDS SYSTEM **i-76**

OKALIWE DAVID FELIX
2016224007

EVALUATION OF STRENGTH CHARACTERISTICS OF CLAY STABILIZED WITH SUGAR CANE STRAW ASH AND LIME

BY

NNADI MODESTUS AFAMEFULE

(NAU/2016224013)

SUBMITTED TO

THE DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

NNAMDI AZIKIWE UNIVERSITY AWKA.

IN PARTIAL FUFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG) DEGREE IN CIVIL ENGINEERING

FEBUARY, 2022

CERTIFICATION

This is to certify that this project topic titled "Evaluation of Strength Characteristics of Clay Stabilized with Sugar Cane Straw Ash and Lime" was carried out by Nnadi Modestus Afamefule with registration number (NAU/2015224013) in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

______________________________ ______________

Nnadi Modestus Afamefule **Date**

(Student)

APPROVAL PAGE

This research work "Evaluation of Strength Characteristics of Clay Stabilized with Sugar Cane Straw Ash and Lime" has been assessed and approved by department of civil engineering Nnamdi Azikiwe University.

_________________________________ _______________

Engr. Dr. P.D Onodagu **Date**

(Project Supervisor)

_________________________________ _______________

Engr. Dr. C.A. Ezeagu **Date**

(Head of Department)

_________________________________ _______________

Engr. Prof. D.O Onwuka **Date**

(External Examiner)

DEDICATION

This work is dedicated to Almighty God for the gift of life and also for guiding me through the period of my academic pursuit.

ACKNOWLEDGEMENT

Special thanks go to Almighty God for giving me the strength to complete this work and also for His guidance and protection throughout my stay in Nnamdi Azikiwe University.

Also I will like to express my profound gratitude to my Father Chief Mr Augustine Nnadi and my sister Mrs Ogechukwu Nnadi for their moral support, constant prayers throughout my stay in school, special thanks goes also to my siblings for their encouragement during trying times of my academic pursuit.

I want to thank in a very special way, my project supervisor in the person of Engr. Dr. P.D Onodagu for his time and guidance in the accomplishment of this project. May the lord rain blessing from heaven for him and his family and may he also protect your family.

Special thanks go to the Head of Department of Civil Engineering in the person of Engr. Dr. C.A. Ezeagu. He has been more like a father to us all and I appreciate him so much.

I will also like to extend my heartfelt appreciation to all the staff (academic and non-academic) of the Department of Civil Engineering starting from Engr. C. A. Chidolue, Engr. Prof. Ekenta, Engr. Prof. Nwaiwu, Engr. A.I. Nwajuaku, Engr. C.M. Nwakaire, Engr.B. Njotea, Engr. Prof. Aginam, Engr. Dr. B. Adinna, Engr. Ezewamma and the non-academic teams comprising of Engr. Ngozi, Engr. Nnonso, Engr. Samson and Engr. John for their invaluable tutorship and professional guidance.

Finally, I will like to appreciate my friends Dennis, Kelly, Glints, Bugari, Steevo, Shana, Kastro, Vokes, Nelson, Obinna, Clinton and everyone who has in one way or the other contributed to making me a better person, may the Lord Almighty reward you all greatly

ABSTRACT

The study was conducted to evaluate the strength characteristics of clay soil stabilized with lime and sugar cane straw ash. The natural clay was partially admixed with lime and sugar cane straw ash in a stepped increase of 2% by dry weight of natural clay, geotechnical test were carried out to determine the strength properties of natural clay stabilized with lime and sugar cane straw ash. This test includes: Sieve analysis test, moisture content test, Atterberg limit (liquid limit, plastic limit) test, specific gravity, compaction and California bearing ratio test. Sieve analysis test conducted for natural clay classify the sample as A-7-6 according to AASHTO classification system and CH (clay of high plasticity) according to USCS classification system, moisture content result of natural clay to lime mixture ranges from 34.58% to 41.38%, natural clay to sugar cane straw ash ranges from 34.58% to 35.95% at 2%SCSA content beyond 2%SCSA content, moisture content decreased, natural clay to lime to sugar cane straw ash mixture decreased from 34.58% to 31.17% at 6%LM & SCSA content beyond this point, moisture content increased. Specific gravity result of natural clay to lime ranges from 2.66 to 2.91, natural clay to sugar cane straw ash ranges from 2.66 to 2.48 and natural clay to lime to sugar cane straw ash ranges from 2.62 to 2.52 after 10% LM & SCSA content. Atterberg limit result for natural clay to mixture suggest that the liquid limit ranges from 54.46% to 72.14%, plastic limit ranging from 33.66% to 50.2% and plasticity index value ranging from 20.9% to 21.94%, natural clay to sugar cane straw ash suggest a liquid limit ranging from 54.46% to 43.5%, plastic limit ranging from 33.66% to 20.72% and plasticity index ranging from 20.9% to 22.78%. Compaction test result for natural clay to lime mixture suggest that the maximum dry unit weight ranges from 20.29kN/m^3 to 22.19kN/m^3 with optimum moisture content ranging from 17.78% to 12.21%, natural clay to sugar cane straw ash mixture suggest that the maximum dry unit weight ranges from 20.29kN/m^3 to 16.9kN/m^3 with optimum moisture content ranging from 17.78% to 22.57%, natural clay to lime to sugar cane straw ash mixture ranging from 20.29kN/m^3 to 20.75kN/m^3 with optimum moisture content ranging from 17.78% to 16.42%. California bearing ratio result of natural clay to lime to sugar cane straw ash for the soaked sample ranges from 14% to 17% at 4%LM & SCSA content beyond this point, the California bearing ratio decreased, while the California bearing ratio of the unsoaked sample ranges from 14.5% to 22% at 8%LM & SCSA content but decreased at 10%LM & SCSA content. It was observed that addition of lime and sugar cane straw ash recorded an early improvement in strength properties of natural clay with the improved strength satisfying the requirement for use only at the sub-grade level of pavement construction.

TABLE OF CONTENTS

Title page i

Certification ii

Approval Page iii

Dedication iv

Acknowledgement v

Abstract vi

Table of Content vii

List of Tables xi

List of Figures xv

List of Plate xvii

List of Symbols & Abbreviation xix

List of Appendices xxi

1.0 CHAPTER ONE: Introduction 1

1.1 Background of Study 1

1.2 Statement of Problem 3

1.3 Aim and Objectives 3

1.4 Scope of Study 3

1.5 Significance of Study 4

2.0 CHAPTER TWO: Literature Review 6

2.1 Historical Development on Soil Stabilization 6

2.2 Soil Stabilization Process 8

2.2.1 Mechanical stabilization 8

2.2.2 Chemical Stabilization 8

2.3 Importance and Effect of Soil Stabilization 9

2.3.1 Highway Construction 9

2.4 Purposes of Soil Stabilization 9

2.5 Basic Principle of Soil Stabilization 10

2.6 Clay Soil 10

2.6.1 Characteristics of Clay Soil 12

2.6.1.1 Particle Size 12

2.6.1.2 Structure 12

2.6.1.3 Organic Content 13

2.6.1.4 Permeability and Water-Holding Capacity 13

2.6.2 Chemical Properties of Clay Soil 13

2.6.2.1 Ion Exchange 14

2.6.2.2 Clay Mineral-Water Interactions 14

2.6.2.3 Gas Penetrability 16

2.6.2.4 Hydraulic Conductivity 16

2.7 Stabilizing Agents 16

2.7.1 Soil Stabilization Using Lime 16

2.7.2 Sugar Cane Straw Ash 18

2.8 Factors Affecting Strength of Stabilized Soils 19

2.8.1 Organic Matter 19

2.8.2 Sulphates 20

2.8.3 Sulphides 20

2.8.4 Compaction 20

2.8.5 Moisture Content 21

2.8.6 Temperature 22

2.8.7 Freeze-Thaw and Dry-Wet Effect 22

2.9 Review of Previous Works 22

3.0 CHAPTER THREE: Materials and Methods 24

3.1 Collection and Preparation of Materials 24

3.1.1 Clay 24

3.1.2 Lime 24

3.1.3 Sugar Cane Straw Ash (SCSA) 25

3.2 Method of Study 27

3.2.1 Particle Size Distribution (Sieve Analysis) 27

3.2.2 Specific Gravity Test 30

3.2.3 Moisture Content Determination 32

3.3.4 Atterberg limit test 34

3.2.5 Compaction Test 39

3.2.6 California Bearing Ratio Test 43

4.0 CHAPTER FOUR: Results and Discussion 48

4.2 Analysis of Results 52

4.2.2 Moisture Content 52

4.2.3 Specific Gravity 54

4.2.4 Liquid Limit 57

4.2.5 Plastic Limit 58

4.2.6 Plasticity Index 60

4.2.7 Compaction 62

4.2.8 California Bearing Ratio 65

5.0 CHAPTER FIVE: Conclusion and Recommendation 67

5.1 Conclusion 67

5.2 Recommendation 68

REFERENCE 69

APPENDICES 74

LIST OF TABLES

2.0: Chemical Composition of Sugar Cane Straw Ash 19

3.0 Plasticity Ratings for Fine grained Soil 38

3.1 Details of Compaction Mould 40

3.2 Details of Compaction Procedure 40

3.3 Standard load adopted for different penetration on a standard material with CBR value of 100% 44

4.0: Summary of Index Properties of NC, LM and SCSA 48

4.1a: Summary of Index Properties of NC+LM 49

4.1b: Summary of Index Properties of NC+SCSA 49

4.2a: Compaction and CBR Characteristics of NC + LM + SCSA 50

4.2b: Compaction and Characteristics of NC + LM 51

4.2c: Compaction and Characteristics of NC + SCSA 51

A1. Specific Gravity Result for 100%NC + 0%LM + 0%SCA 74

A2: Specific Gravity Result for 96%NC + 2%LM + 2%SCSA. 75

A3: Specific Gravity Result for 92%NC + 4%LM + 4%SCSA 76

A4: Specific Gravity Result for 88%NC + 6%LM + 6%SCSA 77

A5:Specific Gravity Result for 84%NC + 8%LM + 8%SCSA. 78

A6: Specific Gravity Result for 80%NC + 10%LM + 10%SCSA. 79

A7: Specific Gravity Result for SCSA. 80

A8: Specific Gravity Result for LM 81

A9: Specific Gravity Result for NC + 2%LM + 0%SCSA 82

A10: Specific Gravity Result for NC + 4%LM + 0%SCSA 83

A11: Specific Gravity Result for NC + 6%LM + 0%SCSA 84

A12 Specific Gravity Result for NC + 8%LM + 0%SCSA 85

A13: Specific Gravity Result for NC + 10%LM + 0%SCSA 86

A14: Specific Gravity Result for NC + 0%LM + 2%SCSA 87

A15: Specific Gravity Result for NC + 0%LM + 4%SCSA 88

A16: Specific Gravity Result for NC + 0%LM + 6%SCA 89

A17: Specific Gravity Result for NC + 0%LM + 8%SCSA 90

A18: Specific Gravity Result for NC + 0%LM + 10%SCA 91

B1: Sieve Analysis Result for NC 92

C1: Moisture Content Result for NC + 0%LM + 0%SCA 93

C2: Moisture Content Result for NC + 2%LM + 2%SCSA 93

C3: Moisture Content Result for NC + 4%LM + 4%SCSA 93

C4: Moisture Content Result for NC + 6%LM + 6%SCSA 94

C5: Moisture Content Result for NC + 8%LM + 8%SCSA 94

C6: Moisture Content Result for NC + 10%LM + 10%SCSA 94

C7: Moisture Content Result for NC + 0%LM + 0%SCSA 95

C8: Moisture Content Result for NC + 0%LM + 2%SCSA 95

C9: Moisture Content Result for NC + 0%LM + 4%SCSA 95

C10: Moisture Content Result for NC + 0%LM + 6%SCSA 96

C11: Moisture Content Result for NC + 0%LM + 8%SCSA 96

C12: Moisture Content Result for NC + 0%LM + 10%SCSA 97

C13: Moisture Content Result for NC + 0%LM + 0%SCSA 97

C14: Moisture Content Result for NC + 2%LM + 2%SCSA 97

C15: Moisture Content Result for NC + 4%LM + 4%SCSA 98

C16: Moisture Content Result for NC + 6%LM + 6%SCSA 98

C17: Moisture Content Result for NC + 8%LM + 8%SCSA 98

C18: Moisture Content Result for NC + 10%LM + 10%SCSA 99

D1 CBR Result for NC + 0%LM + 0%SCSA (Soaked) 100

D2 CBR Result for NC + 2%LM + 2%SCSA (Soaked) 101

D3 CBR Result for NC + 4%LM + 4%SCSA (Soaked) 102

D4 CBR Result for NC + 6%LM + 6%SCSA (Soaked) 103

D5 CBR Result for NC + 8%LM + 8%SCSA (Soaked) 104

D6 CBR Result for NC + 10%LM + 10%SCSA (Soaked) 105

D7 CBR Result for NC + 0%LM + 0%SCSA (Unsoaked) 106

D8 CBR Result for NC + 2%LM + 2%SCSA (Unsoaked) 107

D9 CBR Result for NC + 4%LM + 4%SCSA (Unsoaked) 108

D10 CBR Result for NC + 6%LM + 6%SCSA (Unsoaked) 109

D11 CBR Result for NC + 8%LM + 8%SCSA (Unsoaked) 110

D12 CBR Result for NC + 10%LM + 10%SCA (Unsoaked) 111

E1: Liquid Limit Result for NC + 0%LM + 0%SCSA 112

E2: Liquid Limit Result for NC + 2%LM + 2%SCSA 113

E3: Liquid Limit Result for NC + 4%LM + 4%SCSA 114

E4: Liquid Limit Result for NC + 6%LM + 6%SCSA 115

E5: Liquid Limit Result for NC + 8%LM + 8%SCSA 116

E6: Liquid Limit Result for NC + 10%LM + 10%SCSA 117

F1: Plastic Limit Result for 100%NC + 0%LM + 0%SCSA 118

F2: Plastic Limit Result for 96%NC + 2%LM + 2%SCSA 118

F3: Plastic Limit Result for 92%NC + 4%LM + 4%SCSA 118

F4: Plastic Limit Result for 88%NC + 6%LM + 6%SCSA 119

F5: Plastic Limit Result for 84%NC + 8%LM + 8%SCSA 119

F6: Plastic Limit Result for 80%NC + 10%LM + 10%SCSA 110

G1: Dry Density Determination for NC + 0%LM + 0%SCSA (BSL) 111

G1a: Moisture Content Determination for NC + 0%LM + 0%SCSA Top (BSL) 111

G1b: Moisture Content Determination for NC + 0%LM + 0%SCSA Bottom (BSL) 111

G2: Dry Density Determination for NC + 2%LM + 2%SCSA (BSL) 112

G2a: Moisture Content Determination for NC + 2%LM + 2%SCSA Top (BSL) 112

G2b: Moisture Content Determination for NC + 2%LM + 2%SCSA Bottom (BSL) 113

G3: Dry Density Determination for NC + 4%LM + 4%SCSA (BSL) 113

G3a: Moisture Content Determination for NC + 4%LM + 4%SCSA Top (BSL) 113

G3b: Moisture Content Determination for NC + 4%LM + 4%SCSA Bottom (BSL) 114

G4: Dry Density Determination for NC + 6%LM + 6%SCSA (BSL) 114

G4a: Moisture Content Determination for NC + 6%LM + 6%SCSA Top (BSL) 115

G4b: Moisture Content Determination for NC + 6%LM + 6%SCSA Bottom (BSL) 115

G5: Dry Density Determination for NC + 8%LM + 8%SCSA (BSL) 115

G5a: Moisture Content Determination for NC + 8%LM + 8%SCSA Top (BSL) 116

G5b: Moisture Content Determination for NC + 8%LM + 8%SCSA Bottom (BSL) 116

G6: Dry Density Determination for NC + 10%LM + 10%SCSA (BSL) 117

G6a: Moisture Content Determination for NC + 10%LM + 10%SCSA Top (BSL) 117

G6a: Moisture Content Determination for NC + 10%LM + 10%SCSA Bottom (BSL) 118

LIST OF FIGURES

2.0: Dry density versus time lapsed since the end of mixing of two materials stabilized with 10% cement 21

4.0: Particle Size Distribution Curve for NC 52

4.1a: Moisture Content Graph for NC + 0%LM + %SCSA 53

4.1b: Moisture Content Graph for NC + %LM + 0%SCSA 53

4.1c: Combined Moisture Content Graph for NC + %LM + %SCSA 54

4.2a: Specific Gravity Graph for NC + 0%LM + %SCSA 55

4.2b: Specific Gravity Graph for NC + 0%LM + %SCSA 55

4.2c: Combined Specific Gravity Graph for Composite Mixture 56

4.2d: Combined Specific Gravity Chart for Composite Mixture 56

4.3a: Liquid Limit Chart for the Composite Mixture 57

4.3b: Liquid Limit Graph for the NC + %LM + 0%SCSA 58

4.3c: Liquid Limit Graph for the NC + 0%LM + %SCSA 58

4.4a: Plastic Limit Graph of NC + 0%LM + %SCSA 59

4.4b: Plastic Limit Graph of NC + %LM + 0%SCSA. 59

4.4c: Chart Showing the Plastic Limit of Composite Mixture 60

4.5a: Graph Showing the Plasticity Index Value of NC + %LM + 0%SCSA 61

4.5b: Graph Showing the Plasticity Index Value of NC + 0%LM + %SCSA 61

4.5c: Graph Showing the Plasticity Index Value of Composite Mixture. 62

4.6a: Compaction Curve for the Composite Mixture 63

4.6b: Compaction Curve for NC + 0%LM + %SCSA. 63

4.6c: Compaction Curve for NC + %LM + 0%SCSA 64

4.6d: Graph of MDUW against Variation in % of LM & SCSA 64

4.6e: Graph of OMC against Variation in % of LM & SCSA 65

4.7a: Graph Showing the CBR Values of NC + %LM + %SCSA (Soaked) 66

4.7b: Graph Showing the CBR Values of NC + %LM + %SCSA (Unsoaked) 66

B1: Particle Size Distribution Curve for NC 92

D1: Graph of Force against Penetration for NC + 0%LM + 0%SCSA 99

D2: Graph of Force against Penetration for NC + 2%LM + 2%SCSA 100

D3: Graph of Force against Penetration for NC + 4%LM + 4%SCSA 101

D4: Graph of Force against Penetration for NC + 6%LM + 6%SCSA 102

D5: Graph of Force against Penetration for NC + 8%LM + 8%SCSA 103

D6: Graph of Force against Penetration for NC + 10%LM + 10%SCSA 104

D7: Graph of Force against Penetration for NC + 0%LM + 0%SCSA 105

D8: Graph of Force against Penetration for NC + 2%LM + 2%SCSA 106

D9: Graph of Force against Penetration for NC + 4%LM + 4%SCSA 107

D10: Graph of Force against Penetration for NC + 6%LM + 6%SCSA 108

D11: Graph of Force against Penetration for NC + 8%LM + 8%SCSA 109

D12: Graph of Force against Penetration for NC + 10%LM + 10%SCSA 110

E1: Liquid Limit Graph for 100%NC + 0%LM + 0%SCSA 111

E2: Liquid Limit Graph for 100%NC + 2%LM + 2%SCSA 112

E3: Liquid Limit Graph for 92%NC + 4%LM + 4%SCSA 113

E4: Liquid Limit Graph for 88%NC + 6%LM + 6%SCSA 114

E5: Liquid Limit Graph for 84%NC + 8%LM + 8%SCSA 115

E6: Liquid Limit Graph for 80%NC + 10%LM + 10%SCSA 116

F1: Plastic Limit Graph for NC + %LM + %SCSA 119

G1: Compaction Curve for NC + 0%LM + 0%SCSA 121

G2: Compaction Curve for NC + 2%LM + 2%SCSA 122

G3: Compaction Curve for NC + 4%LM + 4%SCSA 124

G4: Compaction Curve for NC + 6%LM + 6%SCSA 125

G5: Compaction Curve for NC + 8%LM + 8%SCSA 127

G6: Compaction Curve for NC + 10%LM + 10%SCSA 129

LIST OF PLATE

3.0: Photographs of Sugar Cane Straw 25

3.1: Photograph of Burning Process of Sugar Cane Straw 26

3.3 Apparatus for Particle Size Distribution Test (Sieve Analysis) 28

3.4 Apparatus for Particle Size Distribution Test (Sieve Analysis). 29

3.5 Apparatus used for Specific Gravity Test 31

3.6 Apparatus for Atterberg Limit Test. 36

3.7 Apparatus employed for Compaction Test 41

3.8 California Bearing Ratio testing machine. 45

LIST OF SYMBOL & ABBREVIATION

NC –Natural Clay

LM -- Lime

SP---Sample

SCS --- Sugar Cane Straw

SCSA – Sugar Cane Straw Ash

CBR—California Bearing Ratio

G_S – Specific Gravity

AASHTO – American Association of State Highway and Transportation Officials

USCS – Unified Soil Classification System

ASTM – American Society for Testing and Material

BSL – British Standard Light

BSH – British Standard Heavy

MDUW- Maximum Dry Unit Weight

OMC – Optimum Moisture Content

LL – Liquid Limit

PL – Plastic Limit

SL – Shrinkage Limit

PI – Plasticity Index

D_{10} – Particle Size such that 10% is finer than the Size

D_{30}- Particle Size such that 30% is finer than the Size

D_{60}- Particle Size such that 60% is finer than the Size

C_U – Coefficient of Uniformity

C_C – Coefficient of Curvature

SC – Clayey Sand

SM – Silty Sand

GM – Silty Gravel

GC—Clayey Gravel

GW—Well Graded Gravel

GP—Poorly Graded Gravel

SP—Poorly Graded Sand

SW—Well Graded Sand

CL – Inorganic Clay of Low Plasticity (lean clay)

CH—Inorganic Clay of High Plasticity (fat clay)

ML- Silt of low Plasticity

MH – Silt of High Plasticity

LIST OF APPENDICES

A. Specific Gravity Test 74

B. Sieve Analysis Test 92
C. Moisture Content Test 94
D. California Bearing Ratio Test 99
E. Liquid Limit Test 111
F. Plastic Limit Test 117
G. Compaction Test 118

CHAPTER ONE

INTRODUCTION

1.1 Background of Study

The most suitable approach available to the geotechnical engineer is to improve the existing soil for better use in construction sites. Benefits of sol improvement are high resistance values, reduction in plasticity, lower permeability, low compressibility etc. Amongst various alternatives, reinforcing the soil with some materials is one such successful alternative.

Clayey soils are usually categorized as expansive soils. Other names of these soils are soft soils or fine-grained soils. These types of soils are known lead to critical damage to structures resting on them. Normally in construction industries, the structures that constructed on clay soils are tend to trigger the soil when exposed to additional load as well as external impact. This deformation could potentially cause significant failure to foundation and structures. Besides, the construction of roadway on the soft soils also encounters the same problem. This is because the soils do not have enough physical properties for construction application. It is very risky if the construction is still continuing on these types of soils without any remediation or improvement on the soils. As a general knowledge, the common approach when facing this difficulty is to remove all the soils and replace it with stronger soils or material like crushed rocks. The excessive expenses regarding the soils replacement cause the researcher to explore another method to make the cost become more reasonable.

Soils are stabilized for improvement of their durability and strength to render them more suitable for construction (Khalid *et al.,* 2012). Nowadays, there are many methods to improve the soils. From time to time, researchers make use of soils stabilization technique to enhance the geotechnical characteristics of clay soil to maintain roadways, control foundation settlement, prevent structures from collapsing, as well as avoid any kind of related failure. Various soil stabilization techniques are suitable for stabilization involving expansive clayey soil. These kinds of methods consists of the application of chemicals, soil replacing, rewetting, moisture control, and compaction control, surcharge loading and thermal methods (Chen, 1988; Nelson and Miller, 1992; Yong and Ouhadi, 2007). Instead of using chemical product, recycled or reused materials usually are might offer more economical options for a variety application of soil stabilization. As an example we can use lime and sugar cane straw ash to improve the soil characteristics and also the performance of the soil. Preservation with the aim of getting rid of all environmental considerations is really a series issue (Edil & Craig, 2007). All these methods may have their advantages and disadvantages of being ineffective and also costly. Therefore, new methods are still being research to improve the strength properties in order to reduce the swell potential of expansive soils.

For some decades, lime and cement are the two major additives for the stabilization of soils. The cost of the materials has however increased hugely as a result of the high increase in energy cost since 1970s (Neville, 2000). Much dependence on the use of manufactured soil improving additives like cement, and lime, has made the cost of road construction on stabilized soils huge. Thus, the use of waste materials such as rice husk ash, sawdust ash, sugarcane straw ash etc., which are pozzolanic in nature will considerably reduce the cost of construction and as well help in pollution control which is a burning issue world over (Asiagwu *et al*., 2012).

Among the stabilizing agent that have been identified, lime and sugar cane straw ash have been selected in this research because these types of materials have good characteristics to improve its stability, increase the bearing capacity and reduce settlement and lateral deformation of the clayey soil.

Lime provides an economical as well as powerful way of chemical improvement. The standard utilization of lime stabilization is in the treatment of clay sub grade to create improved road foundation without necessity for large amounts of imported granular aggregates. In United States and Europe, lime stabilization is actually popular regarding improving traffic ability, loading capacity of foundations of road and embankment and also for erosion control (Perry *et al.,* 1977). Contrary to lime modification, lime creates long lasting improvements in soils characteristics offering structural benefits. Other than improving roadways, lime also treat foundations and embankment. Sugar cane straw ash also helps to improves geotechnical properties of the fine-grained soils; for example hydraulic conductivity, swelling behavior, and unconfined compressive strength (Aiticin *et al.,* 1984).

1.2 Statement of Problem

Clay present difficulty to geotechnical engineer due to its complex nature and also contains variable materials. In the preliminary stage, the soils do not have enough physical properties for construction purpose. This is because; marginal soils which include soft clays, loose sand, and organic are not satisfactory materials for construction applications. When clay is moist, it is very pliable, and can easily be moved and manipulated. These extreme changes put a great deal of pressure on foundations, causing them to move up and down, and eventually crack, making clay a poor soil for support.

However, this type of soils are very important in geology, construction, and also for the environmental applications because of their wide consumption as impermeable along with containment barriers inside landfill areas and other environmentally applications.

In civil engineering practice, clay soils are usually modified or stabilized to meet the required specification for a particular project. For example in pavement design and construction on clay soil, regular maintenance and total replacement of existing pavements involve large amount of money and hence put a burden on the budgets of transportation ministries/directorates in many countries. Any approach that can reduce construction cost and at the same time increase life cycle of pavements will be helpful for efficient road maintenance. Necessity to improve soil properties for road or building constructed on clay soil has resulted in the use of various stabilizers. Use of lime, fly ash, Portland cement, saw dust ash and others or in their combined forms usually leads to soil index properties transformation as a result of particles cementation.

Hence, the topic **'evaluation of the strength characteristics of clayey soil stabilized with sugar cane straw ash and lime'** has been chosen. The reason for choosing this is to enable the researcher see how these addictives will affect the engineering properties of clay soils and how these effects will be beneficial to geotechnical studies and project.

1.3 Aims and Objectives

The aim of this research work is to evaluate the strength characteristics of clay soil stabilized with lime and sugar cane straw ash. This study will also examine;

1. To determine the index properties, compaction characteristics, and unconfined compressive strength of natural/clayey soil.

2. To determine the feasibility and efficacy of using lime and sugar cane straw ash as a stabilizer for natural clay.
3. To determine the maximum amount of lime and sugar cane straw ash required for optimum improvement in strength properties of natural clay.

1.4 Scope of Study

In understanding a research of this kind, its scope is normally defined with respect to the number of soil samples used and also the number of laboratory test carried out. The clay soil sample would be partially replaced with lime and sugarcane straw ash in a stepped increase of 2% starting from 0% to 10%.Standard laboratory tests for soil on the samples will be conducted such as sieve analysis, in-situ moisture content , specific gravity, atterberg limits, compaction and CBR. The results will be computed and relevant comparisons will be done and portrayed with graphical plots.

1.5 Significance of Study

The significant of the study includes:

1. Results of the study could contribute to existing knowledge, in particular regarding the behavior of clay soil treated with these selected additives. It can close the gap in understanding the mechanism of the stabilization through the results from macro and micro-structural study, verified by physico-chemical tests.

2. The mineralogical changes, morphological changes, molecular changes and physical changes of the clay soil treated with the selected no additives are new findings and can be used for further and other research on clay soils.

3. Treating of soils using the chosen additives could be an economical alternative method in soil stabilization. This is due to the time taken to obtain increment up to 70% strength of the treated soil could be achieved only after 7 days curing periods. Besides that, stabilizing the soil with

these non-traditional additives is easy and quick at the level of implementation compared to other methods of soil reinforcement.

4. Good performance shown by the treated clay soil as backfill materials could give the confident to the practicing engineers in using these non-traditional additives for treating soils to be used in Geotechnical Engineering project especially in regions with laterite clay as foundation soils. This could also be supported by the results of macro-structural tests from UCT, direct shear tests and consolidation tests which shows the increased resistant of treated soil to settlement and shear failure.

CHAPTER TWO

LITERATURE REVIEW

2.1 Historical Development on Soil Stabilization

Soil stabilization is a method of improving soil properties by blending and mixing other materials. Improvements include increasing the dry unit weight, bearing capabilities, volume changes, the performance of in-situ sub soils, sands, and other waste.

Winter Korn (1946) defined soil stabilization as the collective term for any physical, chemical or biological method or any combination of method employed to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose.

Loughman (1969) defined stabilization as process by which a soil material is improved and made more stable. The goals of stabilization are therefore to improve the soil strength, to improve the bearing capacity and durability under adverse moisture and stress condition, and to improve the volume stability of a soil mass.

Seed (1954) also defined soil stabilization as the process of making a soil support an intended design load without excessive deformation.

O Flaherty (1974) reviewed the definition by the American highway research board (A.H.R.B) in 1983 and concluded that a stabilization fill, sub-grade road surface or road base is one that will stay put. This stabilization is the process by which it has been made that way.

Bowled (1938) defined soil stabilization as a process of improving the undesirable physical properties of a soil to achieve the desired strength and make the soil more stable.

Santosh (1987) Stabilization process includes compaction, pre-consolidation, drainage and many other processes. However, the foremost criteria for stabilization of a soil mass are its composition as pure sands and pure clays behave differently in the field.

Arora (2011) the main purpose of soil stabilization is to improve the natural soil for the construction of highways and air fields.

Regular maintenance and total replacement of existing pavements involve large amount of money and hence put a burden on the budgets of transportation ministries/directorates in many countries. Any approach that can reduce construction cost and at the same time increase life cycle of pavements will be helpful for efficient road maintenance. Modern highways are constructed to provide safety and comfort (Butt et al., 2016). Failures on Nigerian highways are generally due to poor geotechnical properties of the underlying soils which constitute the base or sub grade material for the entire road configuration.

Site feasibility study for geotechnical projects is of far most beneficial before a project can take off. Site survey usually takes place before the design process begins in order to understand the characteristics of subsoil upon which the decision on location of the project can be made. The following geotechnical design criteria have to be considered during site selection.

I. Design load and function of the structure.
II. Type of foundation to be used.
III. Bearing capacity of subsoil.

In the past, the third bullet played a major in decision making on site selection. Once the bearing capacity of the soil was poor, the following were options:

IV. Change the design to suit site condition.
V. Remove and replace the in situ soil.
VI. Abandon the site.

Abandoned sites due to undesirable soil bearing capacities dramatically increased, and the outcome of this was the scarcity of land and increased demand for natural resources. Affected areas include those which were susceptible to liquefaction and those covered with soft clay and organic soils. Other areas were those in a landslide and contaminated land. However, in most geotechnical projects, it is not possible to obtain a construction site that will meet the design requirements without ground modification. The current practice is to modify the engineering properties of the native problematic soils to meet the design specifications. Nowadays, soils such as, soft clays and organic soils can be improved to the civil engineering requirements. This state of the art review focuses on soil stabilization method which is one of the several methods of soil improvement. Soil stabilization aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water proofing the particles or combination of the two (Sherwood, 1993). Usually, the technology provides an alternative provision structural solution to a practical problem. The simplest stabilization processes are compaction and drainage (if water drains out of wet soil it becomes stronger). The other process is by improving gradation of particle size and further improvement can be achieved by adding binders to the weak soils (Rogers et al, 1996).

2.2 Soil Stabilization Process

Soil stabilization can be accomplished by several methods. All these methods fall into two broad categories namely;

2.2.1 Mechanical stabilization: Under this category, soil stabilization can be achieved through physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing. Mechanical stabilization is not the main subject of this review and will not be further discussed.

2.2.2 Chemical Stabilization: The cost of the materials has however increased hugely as a result of the high increase in energy cost since 1970s (Neville, 2000). Much dependence on the use of manufactured soil improving additives like cement, and lime, has made the cost of road construction on stabilized soils huge. Thus, the use of waste materials such as rice husk ash, sugarcane straw ash, sawdust ash which are pozzolanic in nature will considerably reduce the cost of construction and as well help in pollution control which is a burning issue world over (Asiagwu, Newill and Schreiner, 2012). Therefore, under this category, soil stabilization depends mainly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect. A chemical stabilization method is the fundamental of this project and, therefore, throughout the rest of this report, the term soil stabilization will mean chemical stabilization. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller bronchure 32-01E). The method can be achieved in two ways, namely; (1) in situ stabilization and (2) ex-situ stabilization. Note that, stabilization not necessary a magic wand by which every soil properties can be improved for better (Ingles and Metcalf, 1972). The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Ingles and Metcalf, 1972; Sherwood, 1993; Euro Soil Stab, 2002). For a successful stabilization, a laboratory tests followed by field tests may be required in order to determine the engineering and environmental properties. Laboratory tests although may produce higher strength than corresponding material from the field, but will help to assess the effectiveness of stabilized materials in the field. Results from the laboratory tests, will enhance the knowledge on the choice of binders and amounts (Euro Soil Stab, 2002).

2.3 Importance and Effect of Soil Stabilization on Structures.

Stabilized soils provide a strong working platform, the foundation for all other parts of projects. After stabilization techniques, weak soils can be transformed by the formation of permanent pozzolanic reactions Aroja and Nagaraja, (2017). Meaning that soils are not liable to leaching and have drastically reduced permeability resulting in reduced shrink/swell potential and increased freeze thaw resistance. In addition, soils that have been stabilized have also under gone some modification; the soil has physically changed making compaction easier and reducing plasticity Aroja and Nagaraja, (2017). Easier compaction makes achieving maximum dry density easier. Plasticity index is an important geotechnical measure that involves the critical water contents of soils. Any time plasticity in soils is reduced, the soils are more friable and workable.

2.3.1 Highway Construction

When soil is stabilized by use of additives it improves the properties of less-desirable road soils. When used, these stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents.

2.4 Purpose of Soil Stabilization

The purpose of soil stabilization is as follows:

I. To improve the strength of sub-base, base and in the case of low-cost roads, surface course.

II. To reduce the construction cost of roads.

III. To make the locally available soil and other inferior materials usable.

IV. To improve the undesirable properties of soil such as excessive swelling or shrinkage, high plasticity etc.

V. To control dust.

VI. To increase the load-bearing capacity.

VII. To reduce Frost weakness.

VIII. To reduce settlement of soil.

IX. To improve permeability characteristics.

2.5 Basic Principle of Soil Stabilization

The basic principles in soil stabilization may be stated as follows:

I. Evaluation of the properties of a given soil.

II. Deciding the method of supplementing the lacking property.

III. Designing the stabilized soil mix for proposed stability and durability values.

IV. Considering the construction procedure by adequately compacting the stabilized layers.

2.6 Clay Soil

Most of stabilization has to be undertaken in soft soils (silty, clayey peat or organic soils) in order to achieve desirable engineering properties. According to Sherwood (1993) fine-grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter.

Clay**,** are soil particles with the diameters of which are less than 0.005 millimeter; also a rock that is composed essentially of clay particles. Rock in this sense includes soils, ceramic clays, clay shale, mudstones, glacial clays (including great volumes of detrital and transported clays), and deep-sea clays (red clay, blue clay, and blue mud). These are all characterized by the presence of one or more clay minerals, together with varying amounts of organic and detrital materials, among which quartz is predominant. Clay materials are plastic when wet, and coherent when dry. Most clays are the result of weathering.

A clay soil compared to others has a large surface area due to flat and elongated particle shapes. On the other hand, silt materials can be sensitive to small change in moisture and, therefore, may prove difficult during stabilization (Sherwood, 1993). Clayey soils are usually categorized as expansive soils. Other names of these soils are soft soils or fine-grained soils. These types of soils invariably result to critical damage to structures resting on them. Normally, in construction industries, the structures that constructed on clay soils are tend to trigger the soil when exposed to additional load as well as external impact. This deformation could potentially cause significant failure to foundation and structures. Besides, the construction of roadway on the soft soils also encounters the same problem. This is because the soils do not have enough physical properties for construction application.

Naturally occurring expansive soils is found virtually everywhere across the globe Hossain and Mol, (2011). Soils with higher percentage of clay minerals like montmorillonite, expandable illite and vermiculite, are susceptible to swelling and shrinkage. They cause numerous costly damages to the roadways, buildings, bridges and other civil engineering infrastructures. Furthermore, clay soils are generally stiff in dry state but when become saturated, they lose their stiffness.

Soft clays are characterized by low compressive strength and excessive compressibility. The reduction in bearing capacity of soft clays results in compressive failure and excessive settlement, leading to severe damage to buildings and foundations (Chen, 1988: Phanikumar and Nagaraju, 2018: Ikeagwani and Nwonu, 2018). Maintenance and rehabilitation costs for the infrastructure on these soils reach billions of dollars annually. These problems primarily stem from the presence of montmorillonite clay minerals which are derived from basic and ultra basic igneous rocks; essentially the minerals area by product of the decomposition of these rocks.

These minerals swell when moisture is introduced and shrink when the same moisture is retracted. In the case where the soil undergoes excessive heat, i.e. drought, expansive soils tend to contract and shrink excessively. Clay minerals and cations come in various forms and that it is the relative quantities of each type of these minerals that are important factors contributing to the swell/shrink behavior along with the dry density, soil structure, and loading conditions present.

Other researchers added that the arid climate, alkaline environment, and local geology are accountable for the expansive nature of soils.

No other earth material has so wide an importance or such extended uses as do the clays. They are used in a wide variety of industries. As soils, they provide the environment for almost all plant growth and hence for nearly all life on the Earth's surface. They provide porosity, aeration, and water retention and are a reservoir of potassium oxide, calcium oxide, and even nitrogen.

Clay materials have a wide variety of uses in engineering. Earth dams are made impermeable to water by adding suitable clay materials to porous soil; water loss in canals may be reduced by adding clay. The essential raw materials of Portland cement are limestone and clays, commonly impure. After acid treatment, clays have been used as water softeners; the clay removes calcium and magnesium from the solution and substitutes sodium. A major use of clay is as drilling mud, that is; heavy suspension consisting of chemical additives and weighting materials, along with clays, employed in rotary drilling.

2.6.1 Characteristics of Clay Soil

2.6.1.1 Particle Size

Clay has the smallest particle size of any soil type, with individual particles being so small that they can only be viewed by an electron microscope. This allows a large quantity of clay particles to exist in a relatively small space, without the gaps that would normally be present between larger soil particles. This feature plays a large part in clay's smooth texture, because the individual particles are too small to create a rough surface in the clay.

2.6.1.2 Structure

Because of the small particle size of clay soils, the structure of clay-heavy soil tends to be very dense. The particles typically bond together, creating a mass of clay that can be hard for plant roots to penetrate. This density is responsible for clay-heavy soil being thicker and heavier than other soil types, and clay soil takes longer to warm up after periods of cold weather. This density also makes clay soils more resistant to erosion than sand or loam-based soils.

2.6.1.3 Organic Content

Clay contains very little organic material; you often need to add amendments if you wish to grow plants in clay-heavy soil. Without added organic material, clay-heavy soil typically lacks the nutrients and micronutrients essential for plant growth and photosynthesis. Mineral-heavy clay soils may be alkaline in nature, resulting in the need for additional amendments to balance the soil's pH before planting anything that prefers a neutral pH. It's important to test clay-heavy soil before planting to determine both the soil's pH and whether it lacks important nutrients such as nitrogen, phosphorus and potassium.

2.6.1.4 Permeability and Water-Holding Capacity

One of the problems with clay soil is its slow permeability resulting in a very large water-holding capacity. Because the soil particles are small and close together, it takes water much longer to move through clay soil than it does with other soil types. Clay particles then absorb this water, expanding as they do so and further slowing the flow of water through the soil. This not only prevents water from penetrating deep into the soil but can also damage plant roots as the soil particles expand.

2.6.2 Chemical Properties of Clay Soil

Chemical properties of clays are very important to the understanding of their behaviour. The electrical charge and colloidal size of clay mineral particles make them hydrate and interact so that their hydraulic conductivity and stress/ strain properties are quite different from those of sandy soil. Thermodynamics describes how systems change when they interact with each other and also with their surroundings. During the above mentioned interactions, how the energy transfer is controlled by the thermodynamic parameters, is to be studied here.

It is always desirable to define a physico-chemical problem involving only equilibrium states in precise thermodynamic terms. The point of view may be sterile from the standpoint of suggesting fundamental, i.e. atomistic, interpretations of the experiments, but it has the great advantage of organizing the experimental results that immediately clarify at what points theoretical help is needed for further progress. The study of ion exchange on the clay minerals affords an excellent illustration of the advantages of the thermodynamic approach. Certainly the detailed effects observed in the experiments are of great complexity, and no simple interpretations can adequately account for them.

The clay chemical properties to be discussed here are:

2.6.2.1 Ion Exchange

Depending on deficiency in the positive or negative charge balance (locally or overall) of mineral structures, clay minerals are able to absorb certain cations and anions and retain them around the outside of the structural unit in an exchangeable state, generally without affecting the basic silicate structure. These adsorbed ions are easily exchanged by other ions. The exchange reaction differs from simple sorption because it has a quantitative relationship between reacting ions.

Exchange capacities vary with particle size, perfection of crystallinity, and nature of the adsorbed ion; hence, a range of values exists for a given mineral rather than a single specific capacity. With certain clay minerals—such as imogolite, allophane, and to some extent kaolinite—that have hydroxyls at the surfaces of their structures, exchange capacities also vary with the pH (index of acidity or alkalinity) of the medium, which greatly affects dissociation of the hydroxyls.

Under a given set of conditions, the various cations are not equally replaceable and do not have the same replacing power. Calcium, for example, will replace sodium more easily than sodium will replace calcium. Sizes of potassium and ammonium ions are similar, and the ions are fitted in the hexagonal cavities of the silicate layer. Vermiculite and vermiculitic minerals preferably and irreversibly adsorb these cations and fix them between the layers. Heavy metal ions such as copper, zinc, and lead are strongly attracted to the negatively charged sites on the surfaces of the 1:1 layer minerals, allophane and imogolite, which are caused by the dissociation of surface hydroxyls of these minerals.

The ion-exchange properties of the clay minerals are extremely important because they determine the physical characteristics and economic use of the minerals.

2.6.2.2 Clay Mineral-Water Interactions

Clay materials contain water in several forms. The water may be held in pores and may be removed by drying under ambient conditions. Water also may be adsorbed on the surface of clay mineral structures and in smectites, vermiculites, hydrated halloysite, sepiolite, and palygorskite;

this water may occur in interlayer positions or within structural channels. Finally, the clay mineral structures contain hydroxyls that are lost as water at elevated temperatures.

The water adsorbed between layers or in structural channels may further be divided into zeolitic and bound waters. The latter is bound to exchangeable cations or directly to the clay mineral surfaces. Both forms of water may be removed by heating to temperatures on the order of 100°–200° C and in most cases, except for hydrated halloysite, are regained readily at ordinary temperatures. It is generally agreed that the bound water has a structure other than that of liquid water; its structure is most likely that of ice. As the thickness of the adsorbed water increases outward from the surface and extends beyond the bound water, the nature of the water changes either abruptly or gradually to that of liquid water. Ions and molecules adsorbed on the clay mineral surface exert a major influence on the thickness of the adsorbed water layers and on the nature of this water. The nonliquid water may extend out from the clay mineral surfaces as much as 60–100 Å.

Hydroxyl ions are driven off by heating clay minerals to temperatures of 400°–700° C. The rate of loss of the hydroxyls and the energy required for their removal are specific properties characteristic of the various clay minerals. This dehydroxylation process results in the oxidation of Fe^{2+} to Fe^{3+} in ferrous-iron-bearing clay minerals.

The water-retention capacity of clay minerals is generally proportional to their surface area (see the Table). As the water content increases, clays become plastic and then change to a near-liquid state. The amounts of water required for the two states are defined by the plastic and liquid limits, which vary with the kind of exchangeable cations and the salt concentration in the adsorbed water. The plasticity index (PI), the difference between the two limits, gives a measure for the rheological (flowage) properties of clays. A good example is a comparison of the PI of montmorillonite with that of allophane or palygorskite. The former is considerably greater than either of the latter, indicating that montmorillonite has a prominent plastic nature. Such rheological properties of clay minerals have great impact on building foundations, highway construction, chemical engineering, and soil structure in agricultural practices.

2.6.2.3 Gas Penetrability

Gas In practice one can assume gas conductivity to be about a thousand times higher than that of water. Once gas has made its way through buffer clay and further out through even more permeable geological units, its rate of flow is more dependent on the availability of pressurized gas than on the gas conductivity. The factor “critical gas pressure” is the pressure which yields penetration through the buffer clay. According to current hypotheses micro-structural heterogeneity has a decisive influence on the critical gas pressure.

2.6.2.4 Hydraulic Conductivity

Hydraulic conductivity is directly affected by bulk density and swelling pressure and swelling pressure. High density and low electrolyte content of the clay mineral give rise to a very low conductivity for Na+-smectite. On the other hand, the conductivity of Ca2+- smectite is slightly higher because of its low densities. Hydraulic conductivity is depended on density at fluid saturation for different clay minerals. If the hydraulic gradient is high, the particles can also move and this affects the hydraulic conductivity. Thus, particle sand aggregates, that are set free, can be transported by flowing pore water to narrow parts of the pore spaces and cause clogging (Hansbo, 1960).

2.7 Stabilizing Agents

These are hydraulic (primary binders) or non-hydraulic (secondary binders) materials that when in contact with water or in the presence of pozzolanic minerals reacts with water to form cementitious composite materials. The stabilizers used in this research are Lime and Sugar Cane Straw Ash.

2.7.1 Soil Stabilization Using Lime

Lime provides an economical way of soil stabilization. Lime modification describes an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reaction (Sherwood, 1993). In soil modification, as clay particles flocculates, transforms natural plate like clays particles into needle like interlocking metalline structures. Clay soils turn drier and less susceptible to water content changes (Roger et al, 1993). Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds (Sherwood, 1993, EuroSoilStab, 2002). The effect can be brought by either quicklime, CaO or hydrated lime, Ca (OH)2. Slurry lime also can be used in dry soils conditions where water may be required to achieve effective compaction (Hicks, 2002). Quicklime is the most commonly used lime; the followings are the advantages of quicklime over hydrated lime (Rogers et al, 1996). - Higher available free lime content per unit mass - denser than hydrated lime (less storage space is required) and less dust - generates heat which accelerate strength gain and large reduction in moisture content according to the reaction equation below

I. $CaO + H_2O \rightarrow Ca(OH)_2$ + Heat (65kJ/ mol) (Equation 1)

Quicklime when mixed with wet soils, immediately takes up to 32% of its own weight of water from the surrounding soil to form hydrated lime; the generated heat accompanied by this reaction will further cause loss of water due to evaporation which in turn results into increased plastic limit of soil i.e. drying out and absorption (EuroSoilStab, 2002; Sherwood, 1993).

Sherwood (1993) investigated the decrease in plasticity as brought about in first instance by cation exchange in which cations of sodium and hydrogen are replaced by calcium ions for which the clay mineral has a greater water affinity. Even in soils (e.g. calcareous soils) where, clay may be saturated with calcium ions, addition of lime will increase pH and hence increase the exchange capacity. Like cement, lime when reacts with wet clay minerals result into increased pH which favors solubility of siliceous and aluminous compounds. These compounds react with calcium to form calcium silica and calcium alumina hydrates, a cementitious product similar to those of cement paste. Natural pozzolanas materials containing silica and alumina (e.g. clay minerals, pulverized fly ash, PFA, blast furnace slag) have great potential to react with lime. Lime stabilizations technology is mostly widely used in geotechnical and environmental applications. Some of applications include encapsulation of contaminants, rendering of backfill (e.g. wet cohesive soil), highway capping, slope stabilization and foundation improvement such as in use of lime pile or lime-stabilized soil columns (Ingles and Metcalf, 1972). However, presence of sulphur and organic materials may inhibit the lime stabilization process. Sulphate (e.g. gypsum) will react with lime and swell, which may have effect on soil strength.

Lime stabilization enhances engineering properties of soils, such as improved strength, higher resistance to fracture, fatigue, and permanent deformation, enhanced resilient properties, reduction in swelling; and resistance to the harmful effects of moisture. The most considerable improvements in these properties are observed in moderately to highly plastic clays.

Al-Kiki, Mohammed and Devashish, (2013) acknowledged that over the time, the properties of treated soil affect the strength gain. Soil pH, organic content, the quantity of exchangeable sodium, clay mineralogy, natural drainage, weathering conditions, extractable iron, carbonates and silica-alumina ratio are some of the properties which influence the gain in strength. The stabilization of acidic soil using lime, resulted in lower compressive strength than that of alkaline soil. Broderick and Daniel, (2013) reported that the lime and cement stabilized soils are less vulnerable to attack by organic chemicals in comparison to untreated soils Haraguchi, Manoj and Mamatha, (2010). Haraguchi, Manoj and Mamatha, (2010) investigated the variation of the engineering properties of freshly cement-stabilized decomposed granite soil cured in water and in 0.2N acid solutions, and indicated that the CBR obtained from the specimens cured in the 0.2N acid solution was lower than that cured in water (Little, 1999). The strong alkaline conditions were able to release silica and alumina from the clay mineral and eventually react with lime to form new cementation products. The success of the lime treatment process is highly dependent on the available lime content, curing time, soil type, soil pH and clay minerals Eadesand and Grim, (1966).

Limited research has been conducted to determine whether pH variations will affect properties of lime-stabilized soils. Additional studies are therefore necessary to explain the erosion mechanism of lime-stabilized soils due to pH variations (Phanikurmar, 2009: Bell, 1989: Kassim, 2009). However, experience shows that lime will react with many medium-, moderately fine- and fine-grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength.

2.7.2 Sugar Cane Straw Ash

Sugarcane straw is the one of the organic wastes obtained from sugar industry during the process of sugar manufacturing. It's use in agriculture as organic fertilizer for crop production. Sugarcane straw ash as a good source of micronutrients like, Fe, Mn, Zn and Cu. It can also be used as soil additive in agriculture due to its capacity to supply the plants with small amounts of nutrients. In many situations, soils in natural state do not present adequate geotechnical properties to be used as road service layers, foundation layers and as filter material. In order to make deficient soils useful and meet geotechnical engineering design requirements researchers have focused more on the use of potentially cost-effective materials that are locally available from industrial and agricultural waste in order to improve the properties of deficient soils and also to minimize the cost of construction.

Table 2.0: Chemical Composition of Sugar Cane Straw Ash (Osinubi et al, 2014).

Compound	Percentage
SiO_2	48.65
Al_2O_3	1.22
CaO	4.52
Fe_2O_3	2.85
MgO	1.46
Na_2O	0.52
K_2O	4.12
SO_3	1.53

2.8 Factors Affecting Strength of Stabilized Soils

Presence of organic matters, sulphates, sulphides and carbon dioxide in the stabilized soils may contribute to undesirable strength of stabilized materials (Netterberg and Paige-Green, 1984, Sherwood, 1993).

2.8.1 Organic Matter

In many cases, the top layers of most soil constitute large amount of organic matters. However, in well drained soils organic matter may extend to a depth of 1.5 m (Sherwood, 1993). Soil organic matters react with hydration product e.g. calcium hydroxide (Ca(OH)2) resulting into low pH value. The resulting low pH value may retard the hydration process and affect the hardening of stabilized soils making it difficult or impossible to compact.

2.8.2 Sulphates

The use of calcium-based stabilizer in sulphate-rich soils causes the stabilized sulphate rich soil in the presence of excess moisture to react and form calcium sulphoaluminate (ettringite) and or thamausite, the product which occupy a greater volume than the combined volume of reactants. However, excess water to one initially present during the time of mixing may be required to dissolve sulphate in order to allow the reaction to proceed (Little and Nair, 2009; Sherwood, 1993).

2.8.3 Sulphides

In many of waste materials and industrial by-product, sulphides in form of iron pyrites (FeS_2) may be present. Oxidation of FeS_2 will produce sulphuric acid, which in the presence of calcium carbonate, may react to form gypsum (hydrated calcium sulphate) according to the reactions (i) and (ii) below

i. $2FeS_2 + 2H_2O + 7O_2 = 2FeSO_4 + 2H_2SO_4$ (Equation 1)

ii. $CaCO_3 + H_2SO_4 + H_2O = CaSO_4.2\,H_2O + CO_2$ (Equation 2)

The hydrated sulphate so formed, and in the presence of excess water may attack the stabilized material in a similar way as sulphate (Sherwood, 1993). Even so, gypsum can also be found in natural soil (Little and Nair, 2009).

2.8.4 Compaction

In practice, the effect of addition of binder to the density of soil is of significant importance. Stabilized mixture has lower maximum dry density than that of unstabilized soil for a given degree of compaction. The optimum moisture content increases with increasing binders (Sherwood, 1993). In cement stabilized soils, hydration process takes place immediately after cement comes into contact with water. This process involves hardening of soil mix which means that it is necessary to compact the soil mix as soon as possible. Any delay in compaction may result in hardening of stabilized soil mass and therefore extra compaction effort may be required to bring the same effect. That may lead to serious bond breakage and hence loss of strength. Stabilized clay soils are more likely to be affected than other soils (Figure 2.0) due to alteration of plasticity properties of clays (Sherwood, 1993). In contrary to cement, delay in compaction for lime-stabilized soils may have some advantages. Lime stabilized soil require mellowing period to allow lime to diffuse through the soil thus producing maximum effects on plasticity. After this period, lime stabilized soil may be remixed and given its final compaction resulting into remarkable strength than otherwise (Sherwood, 1993).

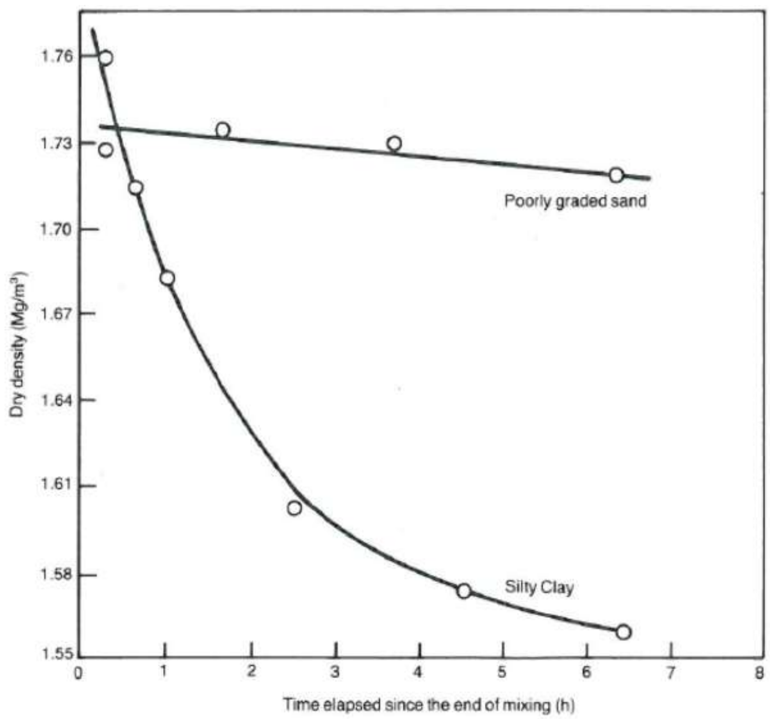


Figure 2.0: Dry density versus time lapsed since the end of mixing of two materials stabilized with 10% cement (Sherwood,1993)

2.8.5 Moisture Content

In stabilized soils, enough moisture content is essential not only for hydration process to proceed but also for efficient compaction. Fully hydrated cement takes up about 20% of its own weight of water from the surrounding (Sherwood, 1993); on other hand, Quicklime (CaO) takes up about 32% of its own weight of water from the surrounding (Roger et al, 1993; Sherwood, 1993). Insufficient moisture content will cause binders to compete with soils in order to gain these amounts of moisture. For soils with great soil-water affinity (such as clay, peat and organic soils), the hydration process may be retarded due to insufficient moisture content, which will ultimately affect the final strength.

2.8.6 Temperature

Pozzolanic reaction is sensitive to changes in temperature. In the field, temperature varies continuously throughout the day. Pozzolanic reactions between binders and soil particles will slow down at low temperature and result into lower strength of the stabilized mass. In cold regions, it may be advisable to stabilize the soil during the warm season (Sherwood, 1993; Maher et al, 1994).

2.8.7 Freeze-Thaw and Dry-Wet Effect

Stabilized soils cannot withstand freeze-thaw cycles. Therefore, in the field, it may be necessary to protect the stabilized soils against frost damage (Maher et al, 2003; Al-tabbaa and Evans, 1998).

Shrinkage forces in stabilized soil will depend on the chemical reactions of the binder. Cement stabilized soil are susceptible to frequent dry-wet cycles due to diurnal changes in temperature which may give rise to stresses within a stabilized soil and, therefore, should be protected from such effects (Sherwood, 1993; Maher et al, 2003).

2.9 Review of Previous Works

Soil Stabilization is frequently termed soil improvement, which in its broadcast sense is the alternation of any property of a soil to improve its engineering performance. It has been suggested by different researches to reflect their aim.

Winter Korn (1946) defined soil Stabilization as the collective term for any physical, chemical or biological method or any combination of method employed to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose.

Loughnan(1969) defined stabilization as process by which a soil material is improved and made more stable. The goals of stabilization are therefore to improve the soil strength, to improve the bearing capacity and durability under adverse moisture and stress condition, and to improve the volume stability of a soil mass.

Seed (1954) also defined soil Stabilization as the process of making a soil support an intended design load without excessive deformation.

O' Flahertry (1974) reviewed the definition by the American highway research board (A. H. R. B) in 1938 and concluded that a stabilization fill, subgrade road surface or road base is one that will put. This stabilization is the process by which it has been made that way.

Bowled (1938) defined soil stabilization as a process of improving the undesirable physical properties of a soil to achieve the desired strength and make the soil more stable.

(William and Lambe) defined soil stabilization as the process of changing any property of a soil to enhance its engineering performance.

(T.N.W. Akroyd) defined stabilized soil as one which has been treated so as to improve or beneficially alter it's properties.

CHAPTER THREE

3.0 MATERIALS AND METHODS

This section will review the materials and methods employed towards achieving the research aim and these are stated below:

3.1 Collection and Preparation of Materials

3.1.1 Clay

The representative soil samples used for the experimental study were natural clay soil designated as NC. This sample was collected at Nnamdi Azikiwe University Campus with the aid of a digger and shovel at a depth representative of the soil stratum and not less than 1m below the natural ground level. The clay sample passed all physical tests necessary to classify them as clay in that, it was sticky, constitute of substantial amount of lumps and with the individual particles not visible to the naked eyes. This soil sample was collected in two empty cement bag, marked indicating the sampling depth, soil description, sampling date and conveyed to geotechnical laboratory of Department of Civil Engineering Nnamdi Azikiwe University. After conveyance, the natural moisture content of the clay sample was determined and was thereafter air-dried in a corrugated roofing sheet for one week to allow partial elimination of natural water which may affect analysis. After drying, the lumps in the samples were slightly pulverized with minimal pressure in order not to damage the individual particles, the samples were passed through sieve No 4 (4.75mm) and the materials passing through the sieve were stored in cement bags in a safe location preparatory for laboratory testing.

3.1.2 Lime

The lime employed in the experimental was designated as LM; this lime was procured at Onitsha Market and conveyed to geotechnical laboratory of Department of Civil Engineering Nnamdi Azikiwe University. Upon arrival, confirmatory test were conducted on the lime sample, the lime was thereafter stored in a safe location preparatory for various laboratory testing. The lime will be used to partially replace the natural clay sample in increasing percentages of 2% by dry weight of clay sample.

3.1.3 Sugar Cane Straw Ash (SCSA)

The sugar cane straw ash designated as SCSA was obtained from a sugar cane plantation in Enugu-Agidi Anambra State Nigeria. The sugar cane straw was washed using water to remove unwanted material such as sand grain, spread out by drying in air for 2 to 3 days. After drying, the sugar cane straw was converted into ash by burning openly into ash at a temperature of 550^0C (Plate 3.1) and collected in polythene bags, stored under room temperature until use. The sugar cane straw ash was sieved through Sieve No 200 (0.075mm) so as to obtain the fine ash (Plate 3.2). Enough care was exercised to ensure that the sugar cane straw ash remained covered before and after use to prevent entry of moisture and other contaminate materials. The clay will be partially admixed with sugar cane straw ash in a stepped increase of 2% to 10% by dry weight of clay starting from 0% so as to establish six different specimens.

Plate 3.0: Photographs of Sugar Cane Straw (Source: Image Obtained from research material)

Plate 3.1: Photograph of Burning Process of Sugar Cane Straw (Source: Image Obtained during experimentation phase)

Plate 3.2: Photograph of Sugar Cane Straw Ash (Source: Image Obtained during experimentation phase)

3.2 Method of Study

The laboratory tests to be conducted are as follows:

3.2.1 Particle Size Distribution (Sieve Analysis)

Sieve analysis is a procedure used to assess the particle size distribution of a granular material (sand, gravel). The size distribution is often of critical importance to the behaviour of the material during use. Sieve analysis can performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)
6. Thermostatically controlled Oven (With temperature of about $80^{O}C$-$110^{O}C$).
7. Masking tape for identification of sample.
8. Exercise book and pen for recording of result.
9. The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

Percentage retained (%) = × 100

Cumulative percentage retained =

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

Coefficient of Curvature =

Coefficient of Uniformity =

Where

D10= particle size such that 10% of the soil is finer than the size

D30= particle size such that 30% of the soil is finer than the size.

D60= particle size such that 60% of the soil is finer than the size**.**

Plate 3.3 Apparatus for Particle Size Distribution Test (Sieve Analysis).

Plate 3.4 Apparatus for Particle Size Distribution Test (Sieve Analysis).

Test Procedure

1. Clean properly the stack of sieves to be used for the experiment using hand brush.
2. Weigh about 500g of air-dried soil sample on a weighing balance.
3. Pour the weighed soil sample into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.
4. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.
5. Remove the sample from the oven and determine it weight (net weight) by deducting the weight of plate from the weight of plate and soil.
6. Arrange the stacks of sieve in the ascending order, place in a mechanical sieve shaker, and thereafter pour the sample and connect the shaker for about 10-15 minute.
7. Disconnect the sieve shaker and determine the mass retained on each of the sieve sizes.
8. Determine the percentage retained, Cumulative percentage retained and Cumulative percentage finer.
9. Plot the graph of sieve Cumulative percentage finer against sieve sizes.
10. Determine D10, D30 and D60 from the plotted graph.

11. Determine the Coefficient of Curvature and Coefficient of Uniformity and classify the soil using the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

3.2.2 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined by (Braja, 2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

1. Density bottle of 50ml capacity and a stopper.
2. Desiccator containing anhydrous silica gel.
3. Thermostatically controlled oven with temperature of about 80-110^{O}C.
4. Weighing balance of 0.01g sensitivity.
5. Mantle heater.
6. Plastic wash bottle.
7. Distilled water.
8. Funnel
9. Thin glass rod for stirring.
10. 425um Sieve.
11. Dry piece of cloth for cleaning.
12. Masking tape for identification of sample.
13. Exercise book and pen for recording of result.

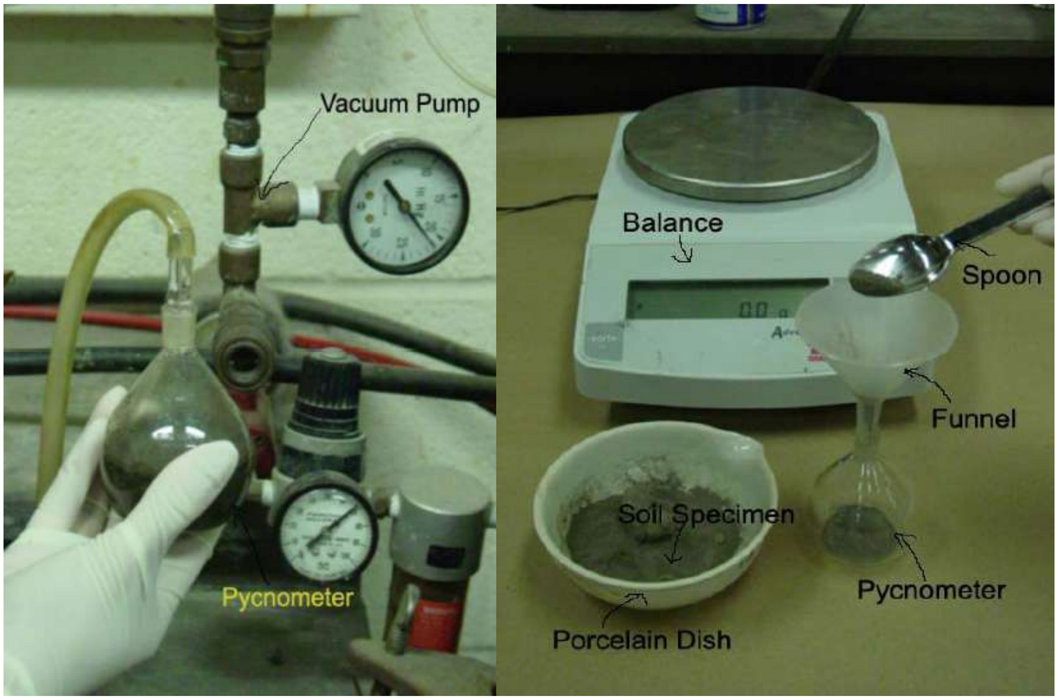


Plate 3.5 Apparatus used for Specific Gravity Test.

Test Procedure

1. Firstly clean the density bottle properly and rinse it with distilled water, oven- dry the clean density bottle with stopper, then cool it in a desiccator so as to remove any moisture present.
2. Weigh and record the weight of the empty clean and dry density bottle say (M_1)
3. Place 10-15g of soil passing through 425um sieve inside the density bottle, weigh and record the weight of density bottle +dry soil + stopper say (M_2).
4. Add distilled water to fill about half to three-fourth of the density bottle, soak the sample for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).
5. Gently stir the density bottle using thin glass rod and thereafter connect to a mantle heater to de-air the sample, do not allow the sample to boil over.
6. After agitation, allow to cool at room temperature and fill it with distilled water up to the specified mark (at lower meniscus level), clean the exterior surface of the density bottle

with a clean dry cloth and determine the weight of the density bottle + stopper +soil filled with water say (M_3).

7. Empty the density bottle clean and rinse with distilled water, then fill it with distilled water up to the same mark. Clean the exterior surface of the density bottle with a clean dry cloth and determine the weight of the density bottle filled with distilled water + stopper say (M_4).
8. Repeat the procedure for two more trials and take the average specific gravity value obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result obtained are as follows:

Specific gravity (G_S) =

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

M_3= Weight of density bottle filled with water + wet soil + stopper.

M_4= Weight of density bottle filled with water + stopper

3.2.3 Moisture Content Determination

The moisture content of a soil sample is defined as the mass of water in the sample expressed as a percentage of the dry mass. Water is present in most naturally occurring soil and this have a profound effect in the soil behavior. Knowledge of this moisture content is used as a guide to the classification of soil. It is also used as a subsidiary to almost all other field and laboratory test of soil. The methods used for moisture content determination are the oven dry method and sand bath method. The oven dry method is a more definitive method used for determining the moisture content of soil, the sand bath method on the other hand is mostly used when the oven dry method is not possible, mainly on site. However for the course of this research the oven dry method is to be adopted.

The apparatus adopted for this research is outlined below:

1. Thermostatically controlled drying oven with a temperature of 80-110^{O}C.
2. Moisture content tins.
3. Weighing balance of 0.01g sensitivity.
4. Distilled water
5. Graduated measuring cylinder.
6. Corrosion resistant tray.
7. Masking tape for identification of tin.
8. Exercise book and pen for recording

Test Procedure

1. Identify and clean the moisture content tins and weigh say (W_1).
2. Weigh a known mass of the wet soil say 50g and place in a tray.
3. Place part of the wet crumbled soil in a moisture content tin and weigh say (W_2).
4. Dry the sample in an oven at a temperature of 80-110^{O}C for a period of 16-24hrs.
5. Remove the oven dried sample and weigh say (W_3).
6. Repeat the test for two more trials for the same sample and take the average moisture content value for the samples.

The Procedure employed for the Computation of the Test Result is as Follows:

Moisture content=×100 =×100

Where W_1 = weight of empty tin

W_2 = weight of tin + wet soil.

W_3 = weight of tin + oven dried soil.

3.3.4 Atterberg limit test.

The behavior of soils especially fine grained soils differs considerably in the presence of water. Clay in the presence of water may almost take a liquid or can be quite hard. Consistency is the property of soil that offers resistance to deformation, it denote the degree of firmness of a soil and can be explained in terms of plasticity and stickiness of soil. Stickiness is the ability of soil especially fine grained soil to adhere to other materials while plasticity on the other hand is the ability of soils to undergo a change in shape under the action of an impressed force without a change in volume.

Stickiness of soils especially fine grained soils can be identified practically by mixing of an air-dried soil with a given quantity of water and then interposing the soil between the thumb and the fore finger (index finger), thereafter the following inference are made as it regards to the observation and this includes:

1. **Non-Sticky:** If the wet soil falls freely between the thumb and the forefinger without leaving any remain or without stretching.
2. **Slightly Sticky:** If the wet soil falls slowly with an infinitesimal traces of remains but without stretching.
3. **Sticky:** If the wet soil falls quite slowly with visible remains and apparent stretching.
4. **Very Sticky:** If the wet soil stretches between the thumb and the fore finger without falling.

The plasticity of soils can be identified practically by rolling a known weight of wet soil into a 3mm uniform diameter thread and the following inferences based on the observation are made and they are as follows:

1. **Non-plastic:** If the wet soil cannot be rolled into thread.
2. **Slightly plastic:** If the wet soil can be rolled into thread but crumbles easily under application of little pressure.
3. **Plastic:** If the wet soil can be rolled into 3mm thread but crumbles under intense application of pressure and cannot be reformed.
4. **Very plastic:** If the wet soil can be rolled into 3mm diameter thread but crumbles under intense application of pressure and can be reformed.

The atterberg limit is a limit characterized by visible transition of soil (especially fine grained soils) from liquid-plastic-semi-solid-solid state consequent upon the variation of moisture content. This test was developed by Albert Atterberg a Swedish agricultural scientist in 1911. This test is divided into three limits namely:

1. Liquid Limit (LL)
2. Plastic Limit (PL)
3. Shrinkage Limit (SL)

Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Cassagrande type)
2. Grooving tool
3. Moisture content tins
4. Porcelain evaporating dish
5. Spatula or pellet knife

6. Thermostatically controlled oven
7. Weighing balance sensitive to 0.01g
8. Plastic wash bottle containing distilled water
9. Paper towels
10. Masking tape for identification of tin.
11. Exercise book and pen for recording of data
12. 425um Sieve
13. Airtight container

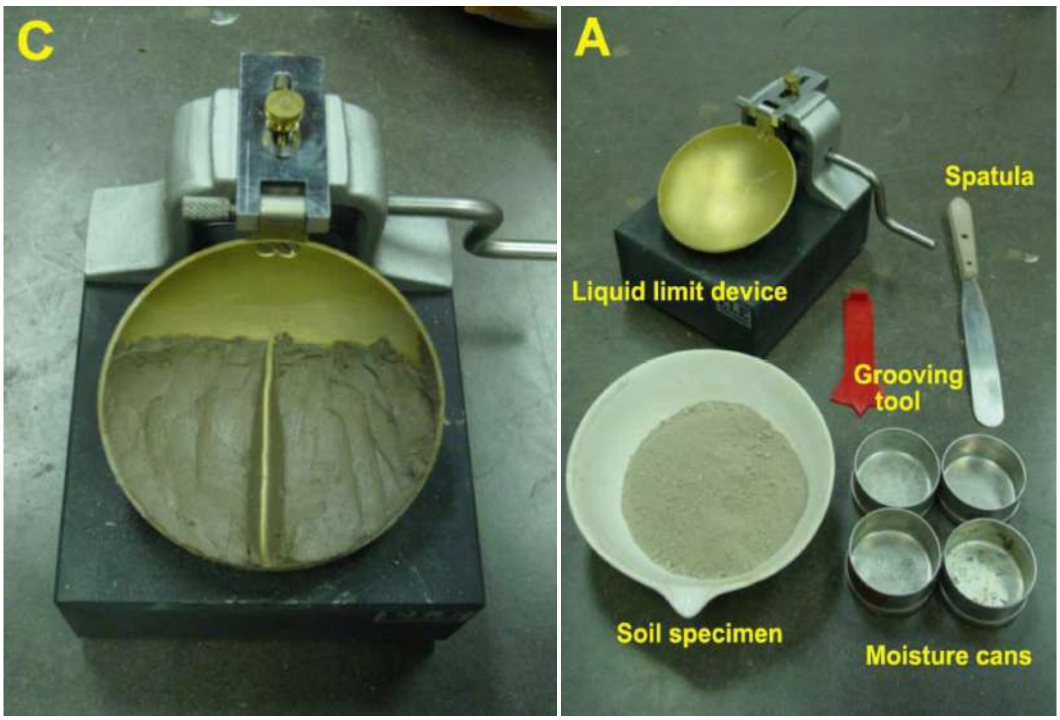


Plate 3.6 Apparatus for Atterberg Limit Test.

Test Procedure

1. Prepare the sample by weighing about 150g of soil passing through 425um sieve, mix the sample with distilled water in a glass plate mixing with pellet knife, remove any coarse particle by hand and mix to form a thick homogenous paste, place the mixed soil in an airtight container and leave to mature for 24hrs.

2. Determine the mass of four moisture content tins say (W_1)
3. Place the matured sample on an evaporating dish and add little water using the plastic squeeze bottle, mix the soil properly to ensure uniform distribution of moisture.
4. Place a portion of the paste (mixed soil) on the liquid limit device and level the mixture so as to obtain a maximum depth of 1cm.
5. Using the grooving tool, cut a groove along the symmetrical axis of the cup holding the tool perpendicular to the cup.
6. Turn the crank or rotate the handle of the liquid limit device at the rate of 2 revolution per second and count the no of blows required to close the groove at a distance of 13mm. Closing of the groove should be as a result of plastic flow of the soil and not by sliding, if sliding occurs repeat the test.
7. Take about 10g of soil in the closed groove and put in the moisture content tins for moisture content determination, weigh the sample say (W_2)
8. Remove the rest of the soil in the cup and use paper towel to clean the cassagrande cup.
9. Alter the water content of the soil and the repeat the process to get the no of blows in the range of 15-40 blows.
10. Plot the graph of moisture content against the log of no of blows, the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content =×100 =

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried soil.

Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.0 is used to classify soil based on the ranges of it plasticity index.

Table 3.1: Plasticity Ratings for Fine grained Soil (Braja, M.Das, 2002).

Plasticity Index	Plasticity
0	Non-Plasticity
<7	Low Plasticity
7-17	Medium Plasticity
17-35	High Plasticity
>35	Very High Plasticity

1. The apparatus used for this experiment includes:
2. A smooth glass plate about 300mm square and 10mm thick.
3. A palette knife or spatula
4. A short length of 3mm metal rod
5. Moisture content tins
6. Plastic squeeze bottle
7. Weighing balance with 0.01g sensitivity
8. Veneer caliper
9. Masking tape for tin identification
10. Exercise book and pen for recording of result.

Test Procedure

1. Prepare the sample by the method described in the liquid limit using the sample passing 425um sieve.
2. Identify and weigh the empty moisture content tins say (W1).
3. Take about 20g of the prepared soil paste on a porcelain evaporating dish, add water from the plastic squeeze bottle and mix thoroughly until the paste is plastic enough to be rolled into a ball.

4. Take a portion of the ball and roll it on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward.
5. Continue rolling and remolding until the thread just start to crack at a distance of 3mm.
6. Collect the small crumbed pieces, place in a moisture content tin and weigh say (W2).
7. Place the tin in the oven at a constant temperature of 80-110^{O}C for a period of 16-24hrs.
8. After 24hrs, remove the tin from the oven and determine the weight of the dry soil plus the tin say (W_3).
9. Repeat the test for at least two trials and take the average plastic limit value for all the trials.

The Computation for Plastic Limit is as follows:

Plastic limit = =

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

3.2.5 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil. According to (Shruthi, 2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing it dry density. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the dry density of the soil. It is achieved through the reduction of the volume of air void in the soil

with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

In civil engineering practice soil compaction is essential for the following reasons:

1. Increasing the bearing strength of foundation
2. Provide stability to slope and foundation.
3. Prevention of undesirable settlement of structures
4. Reduction of water seepage from structure

The compaction methods to be adopted for this research are:

1. British Standard Light for the natural clay samples and also the specimen (clay partially admixed with lime and sugar cane straw ash).

Details of British Standard Compaction Process

Table 3.2: Details of Compaction Mould.

Type	**Diameter (mm)**	**Height (mm)**	**Volume(cm^3)**
British Standard	**105**	**115.5**	**1000**

Table 3.3: Details of Compaction Procedure.

Type of test	**Mould (cm^3)**	**Rammer(kg)**	**Drop (mm)**	**No of layers**	**Blow per layer**
BS light	**1000**	**2.5**	**300**	**3**	**27**
BS heavy	**1000**	**4.5**	**450**	**5**	**27**

The mechanical energy applied in each type of British Standard in term of work done is given as follows:

British Standard Light

Mechanical energy = = = 60.75kgm =60.75.81Nm=596j

Work done per unit volume of soil = =596kj/m^3

British Standard Heavy

Mechanical energy = =2652j

1. The apparatus used for the test are as follows:
2. Compaction mould with a detachable base plate and removable extension collar.
3. Metal rammer (either 2.5kg or 4.5kg)
4. Measuring Cylinder 200ml or 500ml
5. Large Metal tray (600mm600mm 600mm)
6. Balance up to 10kg readable to 1g
7. Small tools such as palette knife, steel straight edge about 300mm long.
8. Drying oven temperature of 105-110^{O}C
9. Apparatus for moisture content determination

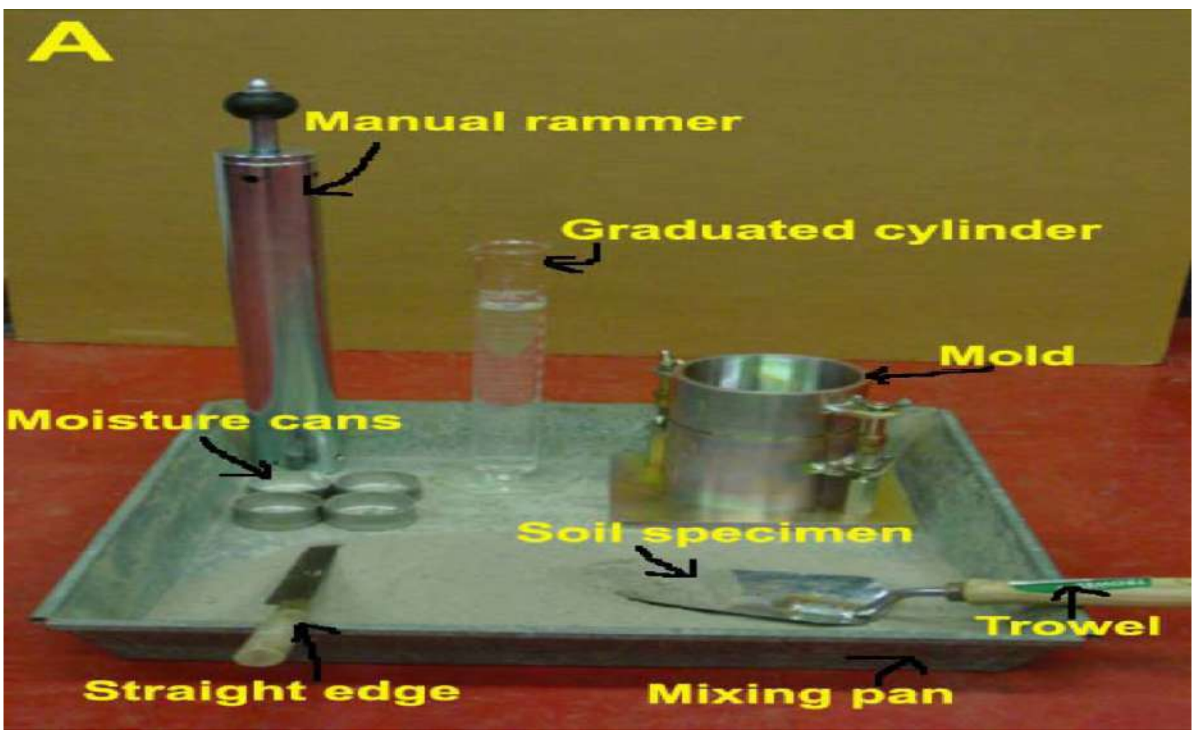


Plate 3.7 Apparatus employed for Compaction Test.

Test Procedure.

1. Check to see if the mould, extension collar and base plate are clean and dry. Measure the dimension and weigh to the nearest 1kg check if the rammer falls freely.
2. Grease the internal surface of the mould
3. Attach the extension collar to the mould.
4. Weigh about 3kg of the soil sample on a weighing balance
5. Add about 4% water to the soil sample, mixing it thoroughly and separating the soil into three layers for British Standard Light and five layers for British Standard Heavy.
6. Pour the wet soil into the mould and compact by applying the required no of blow using either a 2.5kg or 4.5kg rammer falling freely from a height of 300mm. The blow must be distributed uniformly over the surface of the mould.
7. After completion of the compaction operation remove the extension collar and level carefully the top of the mould by means of a straight edge.
8. Weigh the mould with the compacted soil to the nearest 1kg, record the weight as W_2.
9. Determine the moisture content of the representative sample of the specimen; record the moisture content as M.
10. Repeat the procedure for 8%, 12%, 16% and 20% of water to be added and record the value obtained.
11. Plot the graph of dry density against moisture content and determine the maximum dry density (MDD) of the soil at the corresponding optimum moisture content (OMC).

The Computation of the result obtained is as follows:

Determination of Dry Density (P_d).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (M^3) = W_4

Bulk Density (kg/m^3) = =

Moisture Content (%) =

Dry Density (kg/m^3) = =

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W_1

Wt of tin + wet soil = W_2

Wet of wet soil (kg) = $W_3 = W_2 - W_1$

Wt of tin + dry soil (kg) = W_4

Wt of dry soil (kg) = $W_5 = W_4 - W_1$

Wt of water (kg) = $W_6 = W_3 - W_5$

Moisture Content (%) = =

3.2.6 California Bearing Ratio Test

The California bearing ratio test was originally developed by the California division of highway in 1938, for the design of highway thickness. The test is used for evaluating the suitability of materials used in sub-grade, sub-base and base course respectively. The test result has been correlated with the thickness of various materials required for flexible pavement construction. The test may be conducted on a prepared specimen in a mould or on the soil in-situ condition.

In the test the load required to push a plunger into a soil specimen at a controlled rate is measured, then the load on the plunger at a certain depth is recorded as a percentage of a standardized load. The load necessary to push a plunger to a certain depth into the soil is expressed as a percentage of the load required to force the same plunger to the same depth into a standard sample of compacted crush stone. The construction of highway pavement requires a California Bearing Ratio value for 2.5mm and 5mm penetration respectively, with that of 2.5mm penetration being comparatively higher than that of 5mm penetration. The Federal Ministry of work Standard Specification for roads and bridges (1997) state that road construction material should have a CBR value of 10%, 20% and 80% for use as sub-grade, sub-base and base course respectively. The material to be used for the test will be subjected to 48 hours soaking in other to ascertain it behavior under worst condition (flooding as a result of intense rainfall).

Table 3.4: Standard load adopted for different penetration on a standard material with CBR value of 100%.

Penetration of plunger (mm)	Standard Load (kg)
2	**1150**
2.5	**1320**
4	**1760**
5	**2000**
6	**2220**
7.5	**2630**
8	**2650**
10	**3180**
12.5	**3600**

1. The apparatus used for the test are outlined below:
2. A cylindrical corrosion resistant mould 152mm127mm having a diameter of 150-152mm with a detachable base plate and a removable extension collar.
3. A compressive device for static compaction of applying a force of at least 300KN
4. Metal plugs 150mm 0,5mm and 50mm thick.
5. Metal rammer 2.5kg or 4.5kg.
6. Dial gauge of 0.01g sensitivity.
7. Soaking tank.
8. A steel rod of about 16mm diameter and 600mm long and a straight edge of 300mm steel stripe and 3mm thick with one beveled edge.
9. Weighing balance of 25kg accuracy and a spatula.
10. Filter paper
11. Apparatus for moisture content determination.
12. Masking tape used for identification of moisture content tins.
13. Exercise book and pen for recording.

Plate: 3.8 California Bearing Ratio testing machine.

Test Procedure

The methods used for California Bearing Ratio Test are:

1. Compression with tamping.
2. Recompaction with known maximum dry density (MDD) and optimum moisture content (OMC).
3. For this course of study the method for recompacted sample with known maximum dry density (MDD) and optimum moisture content (OMC) is to be adopted and the procedure is outlined below:

4. Carry out Compaction test using 6kg of soil sample, varying the moisture content at a particular percentage say 4%, determine the maximum dry density and optimum moisture content.
5. Clean properly and grease the internal surface of the CBR mould.
6. Weigh 6kg of soil mixing with the optimum moisture content determined from compaction test.
7. Divide the soil into 5 equal layer (CBR Heavy) and seal in an airtight container until requested for use.
8. Stand the mould assembly in a solid base, place the first soil portion and compact using 4.5kg rammer for 62 even blows.
9. Repeat using the remaining four portion of soil in turn so that the level of the soil is not more than 6mm above the top of the mould body.
10. Remove the collar and trim the soil flush with mould with the scrapper or knife edge.
11. Weigh the mould, soil and base plate to the nearest kg.

Preparation for Soaking

Soil may soften when load is placed on it due to flooding or increase in moisture content. Soaking of the sample is done primarily to determine the strength (load bearing strength) of the soil under worst condition (rainy season).Below are the list of apparatus used for CBR Soaking:

1. Perforated base plate fitted to CBR mould in place of normal base plate.
2. Perforated swell plate with an adjustable stem to provide a sealing for the stem of the dial gauge.
3. Tripod mounting to support dial gauge
4. Soaking tank
5. Annular Surcharge discs with internal diameter of 52-54mm and external diameter of 145mm to 150mm.
6. Petroleum jelly.
7. The Soaking procedures are enumerated as follows:
8. Remove the base plate and replace with perforated base plate.

9. Fit the collar to the other end of the mould, pack the screw thread with petroleum jelly to make it water tight.
10. Place the mould assembly in soaking, place the filter paper in the sample, the perforated swell plate, then annular surcharge disc.
11. Mount dial gauge on top of the extension collar, secure the dial gauge in place and adjust the stem in the perforated base plate to give zero.
12. Fill the immersion tank with water just below the extension collar. Start the timer when water has just covered the base plate.
13. Record the time taken for water to appear at the top of the sample if it does occur within two days. Flood the top of the sample and leave to soak for a day.
14. Plot the swelling against elapsed time or square root of time. Flattening curve indicates that swelling is complete.
15. Take off the dial gauge and its support; remove the mould assembly and leave to drain for 15min.
16. Remove the Surcharge discs, perforated plate and collar, then fit the other base plate.
17. Weigh the sample + mould + base plate if density is required after soaking is completed.
18. If the sample has swollen, trim it to the level of the mould and reweigh
19. Test the sample by adjusting the dial gauge to start at zero and take the reading at interval of 0.5mm for every 30seconds till 7mm penetration.
20. Record the load at penetration of 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0mm and express these forces as percentage of the standard load.
21. Calculate the CBR for 2.5 and 5mm penetration; repeat the same procedure for both top and bottom, the higher CBR value will be used as the CBR for the material.
22. Plot the graph of force (KN) against penetration (mm).
23. The normal curve is convex upward, but if the initial part is concave upward applies the necessary correction to the curve.

Mathematically it is expressed as

Where

Test load = dial gauge reading proof ring constant

CHAPTER FOUR

4.0 RESULT AND DISCUSSION

During the experimentation phase of the research, certain findings were obtained which was useful in classifying and determining the strength characteristics of the samples and additives, these findings are presented in Table 4.0 below:

Table 4.1: Summary of Index Properties of NC, LM and SCSA.

Properties	NC	LM	SCSA
Specific Gravity	2.66	2.71	2.48
Liquid Limit (%)	54.46	55.48	42.14
Plastic Limit (%)	33.66	37.76	35.29
Plasticity Index (%)	20.9	17.72	6.85
Plasticity Ratings	High	High	Low
Moisture Content (%)	34.58	-	-
% Passing Sieve No 200 (0.075mm)	74.71	-	-
AASHTO Classification System	A-7-6	-	-
USCS Classification System	CH	-	-

Table 4.1.1: Summary of Index Properties of NC+LM

Properties	NC + 2%LM	NC + 4%LM	NC + 6%LM	NC + 8%LM	NC + 10%LM
Specific Gravity (Gs)	2.7	2.72	2.75	2.76	2.91
Liquid Limit (%)	50.42	59.34	62.81	67.38	72.14
Plastic Limit (%)	34.83	38.55	42.04	41.62	50.2
Plasticity Index (%)	15.59	20.79	20.77	25.76	21.94
Plasticity Ratings	Medium	High	High	High	High
Moisture Content (%)	36.22	38.65	41.13	37.89	41.38

Table 4.1.2: Summary of Index Properties of NC+SCSA

Properties	NC + 2%SCSA	NC + 4%SCSA	NC + 6%SCSA	NC + 8%SCSA	NC + 10%SCSA
Specific Gravity (Gs)	2.66	2.62	2.58	2.55	2.48
Liquid Limit (%)	57.46	61.84	56.48	52.62	43.5
Plastic Limit (%)	30.79	27.25	24.73	21.8	20.72
Plasticity	26.67	34.59	31.75	30.82	22.78

Index (%)					
Plasticity Ratings	**High**	**High**	**High**	**High**	**High**
Moisture Content (%)	**35.99**	**33.63**	**31.76**	**28.43**	**21.61**

Table 4.2: Compaction and CBR Characteristics of NC + LM + SCSA

Properties	**NC + 0%LM + 0%SCSA**	**NC + 2%LM + 2%SCSA**	**NC + 4%LM + 4%SCSA**	**NC + 6%LM + 6%SCSA**	**NC + 8%LM + 8%SCSA**	**NC + 10%LM + 10%SCSA**
Maximum Dry Unit Weight (kN/m^3)	**20.29**	**21.71**	**21.72**	**21.72**	**20.75**	**20.27**
Optimum Moisture Content (%)	**17.78**	**13.15**	**14.04**	**13.98**	**15.05**	**16.42**
CBR (Soaked)	**14**	**16**	**17**	**13.5**	**12.5**	**10.5**
CBR (Unsoaked)	**14.5**	**18.5**	**20.0**	**20.5**	**22.0**	**16.5**

Table 4.2.1: Compaction and Characteristics of NC + LM

Properties	NC + 2%LM	NC + 4%LM	NC + 6%LM	NC + 8%LM	NC + 10%LM
Maximum Dry Unit Weight (kN/m^3)	21.24	21.66	21.71	21.73	22.19
Optimum Moisture Content (%)	14.61	8.24	12.94	15.09	12.21

Table 4.2.2: Compaction and Characteristics of NC + SCSA

Properties	NC + 2%SCSA	NC + 4%SCSA	NC + 6%SCSA	NC + 8%SCSA	NC + 10%SCSA
Maximum Dry Unit Weight (kN/m^3)	20.27	18.81	18.32	17.34	16.9
Optimum Moisture Content (%)	16.23	16.71	16.98	17.26	22.57

4.2 Analysis of Results

4.2.1 Particle Size Distribution (Sieve analysis)

Figure 4.0 is a semi-logarithmic plot of particle size distribution of natural clay samples, the percentage passing sieve no 200 (0.075mm) is 74.71 and according to AASHTO classification system, it is classified as A-7-6 and CH (clay of high plasticity) according to Unified Soil Classification System (USCS). Since more than 35% passes through sieve no 200 (0.075mm), according to AASHTO sub-grade rating for soil samples, this sample will therefore constitute a fair to poor soil if used at the sub-grade level of pavement construction and therefore need to be sufficiently stabilized.

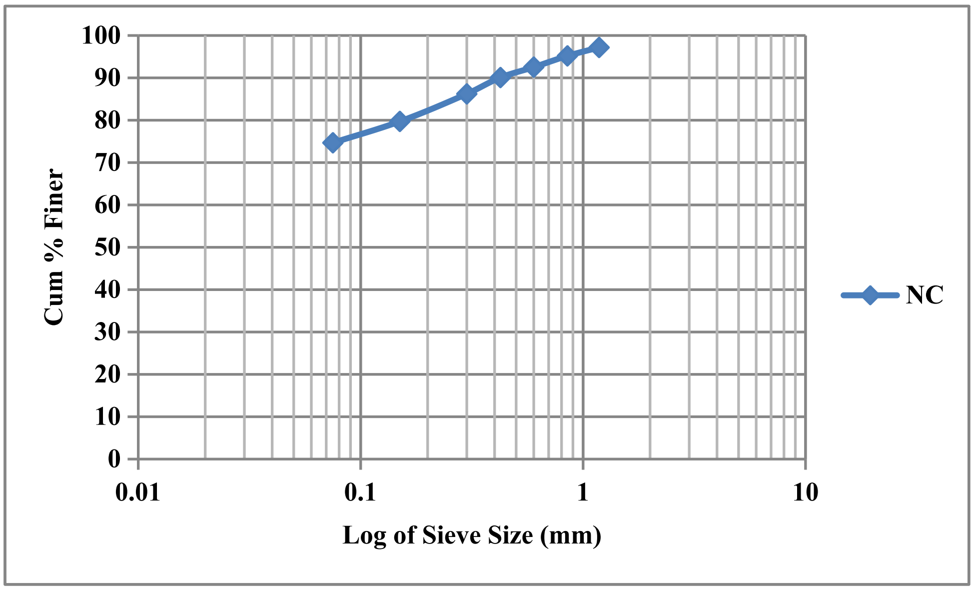


Figure 4.0: Particle Size Distribution Curve for NC.

4.2.2 Moisture Content

The moisture content of the soil is defined as the ratio of weight of water expressed as percentage of weight of dry soil. Knowledge of the in-situ moisture content of soil help in proper understanding of the amount of water to be used prior to laboratory testing. Result obtained from combination of natural clay to lime suggest that addition of lime to natural clay increases the moisture content of natural clay from it natural value of 34.58% to 41.38% after 10% addition of lime content. The consistent increase can be attributed to the fine texture of lime which provides larger surface area for it hydration with clay. Result obtained from natural clay to sugar cane straw ash suggest that addition of sugar cane straw ash to clay raises the moisture content of clay samples from 34.58% to 35.99% after 2% addition of sugar cane straw ash, beyond 2%SCSA content, the moisture content of clay sample decreased consistently while the addition of lime and sugar cane straw ash to clay decreased the moisture content of natural clay from it natural value of 34.58% to 31.17% at 6%LM an SCSA content beyond this point, the moisture content increased. The later increase in moisture content can be attributed to the increasing lime content which became evident at 8% of it addition. This finding is in correlation with the work of (Asma and Darius, 2013).

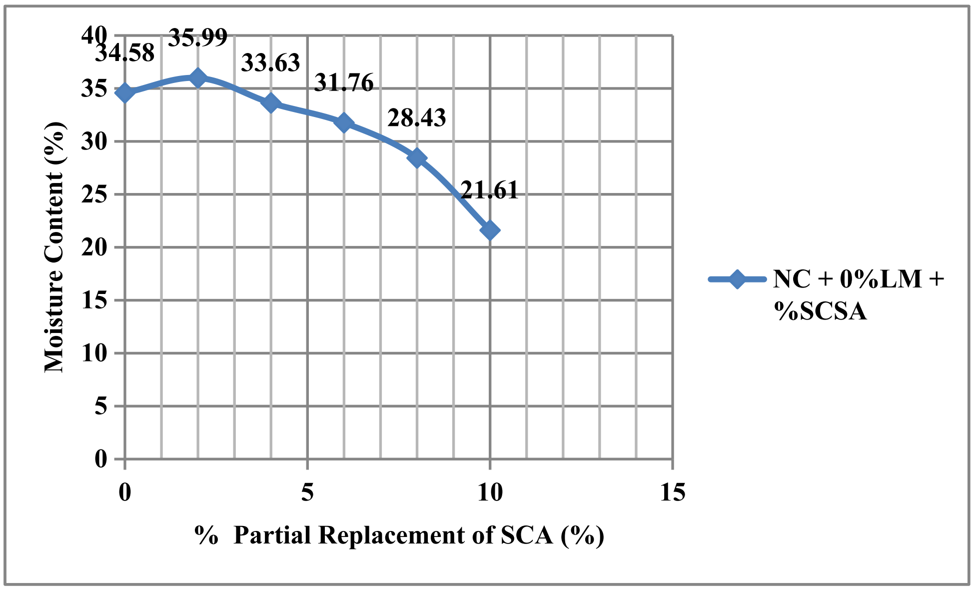


Figure 4.1a: Moisture Content Graph for NC + 0%LM + %SCSA

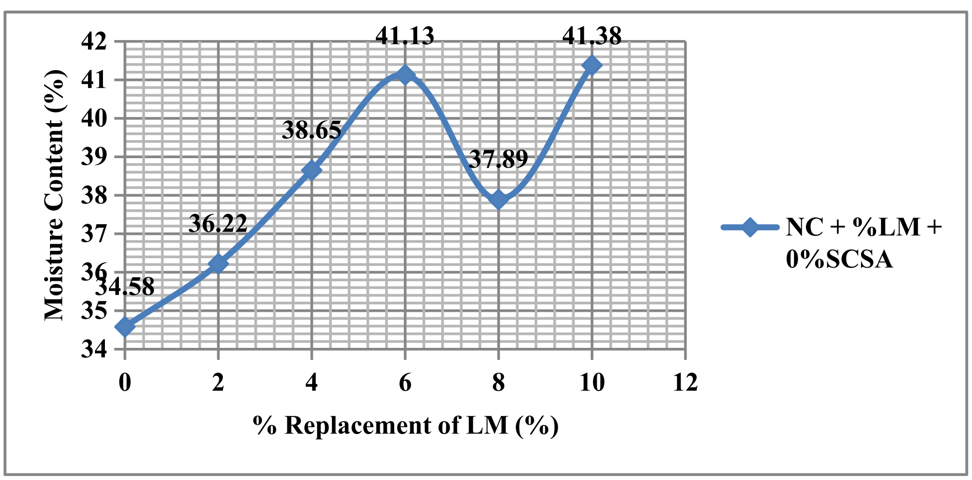


Figure 4.1b: Moisture Content Graph for NC + %LM + 0%SCSA

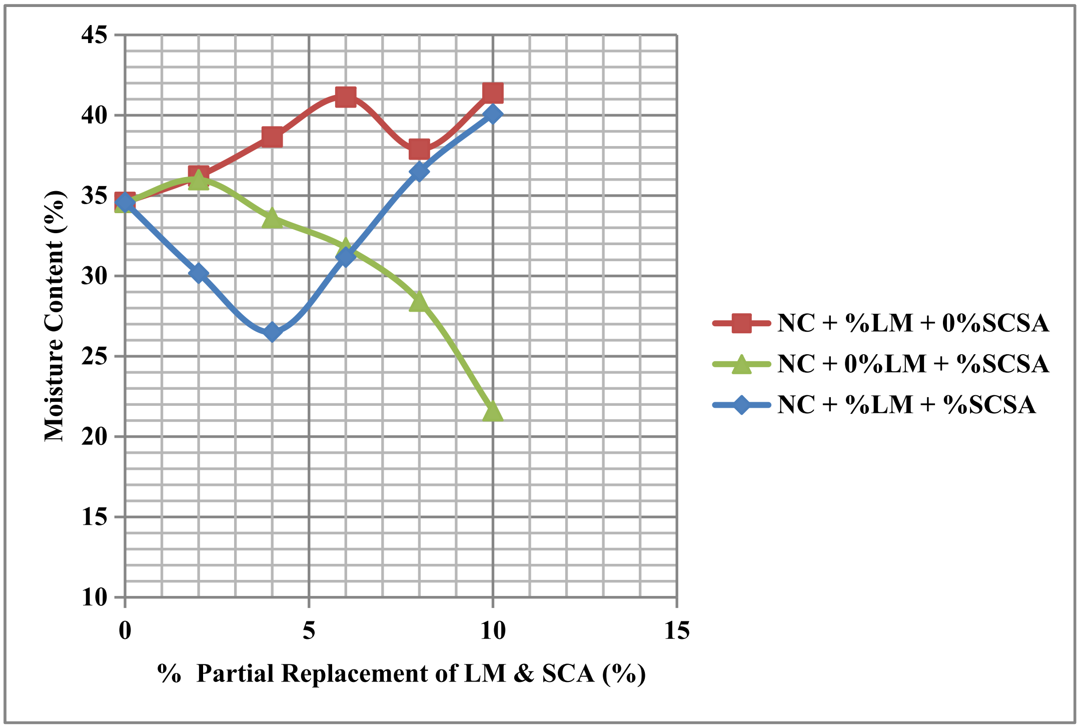


Figure 4.1c: Combined Moisture Content Graph for NC + %LM + %SCSA

4.2.3 Specific Gravity

The specific gravity of the soil is defined as the ratio of weight of the soil to the rate of equal volume of water; it is used to obtain the unit weight of soil in the presence of water. The specific gravity result obtained from combination of natural clay to lime suggest that addition of lime to natural clay increased the specific gravity of natural clay from it natural value of 2.66 to 2.91 after 10% addition of lime content. The consistent increase can be attributed to the high specific gravity value of lime (2.7) which infers that lime is not light weight material and has value which falls within the specific gravity range for clay materials as stated by De-Graft Johnson, (1969) to be in the range of 2.60-3.40 and 2.60-2.90 by Das, (1990). Result obtained from natural clay to sugar cane straw ash combination, suggest that addition of sugar cane straw ash to natural clay maintained the specific gravity value after 2% of it addition but beyond 2%SCSA content, the specific gravity value decreased, the later decrease can be attributed to the low specific gravity value of sugar cane straw ash (2.48) which suggest that it is a relatively lightweight material. The result obtained from the composite mixture (NC + %LM + %SCSA) suggest a decline in specific gravity value of natural clay from 2.66 to 2.52 at 6%LM & SCSA content but beyond this point, the specific gravity value of natural clay increased. The overall increase in specific gravity falls within the range (2.5 to 2.75) specified by federal ministry of work standard and specification for roads and bridges (1997).

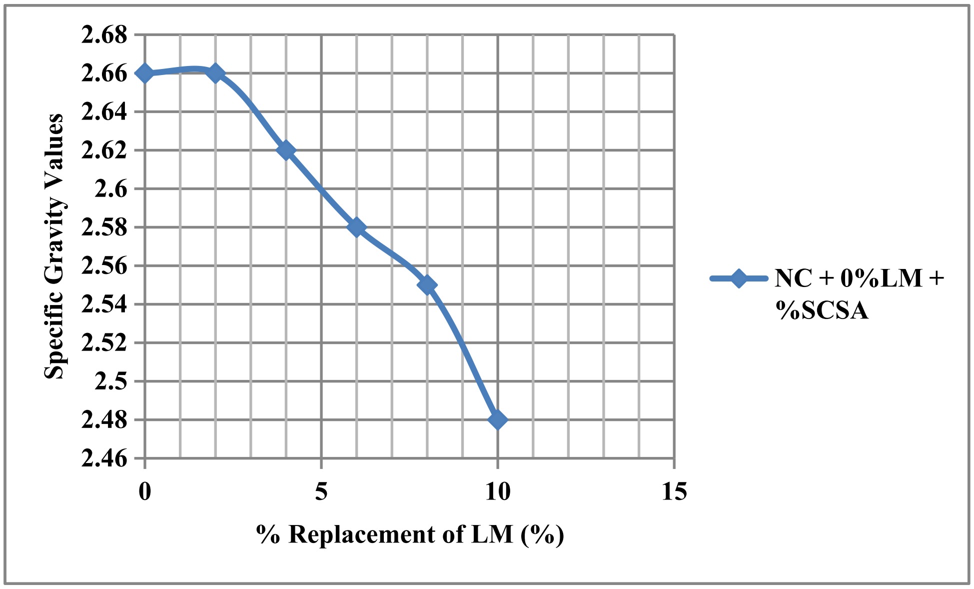


Figure 4.2a: Specific Gravity Graph for NC + 0%LM + %SCSA

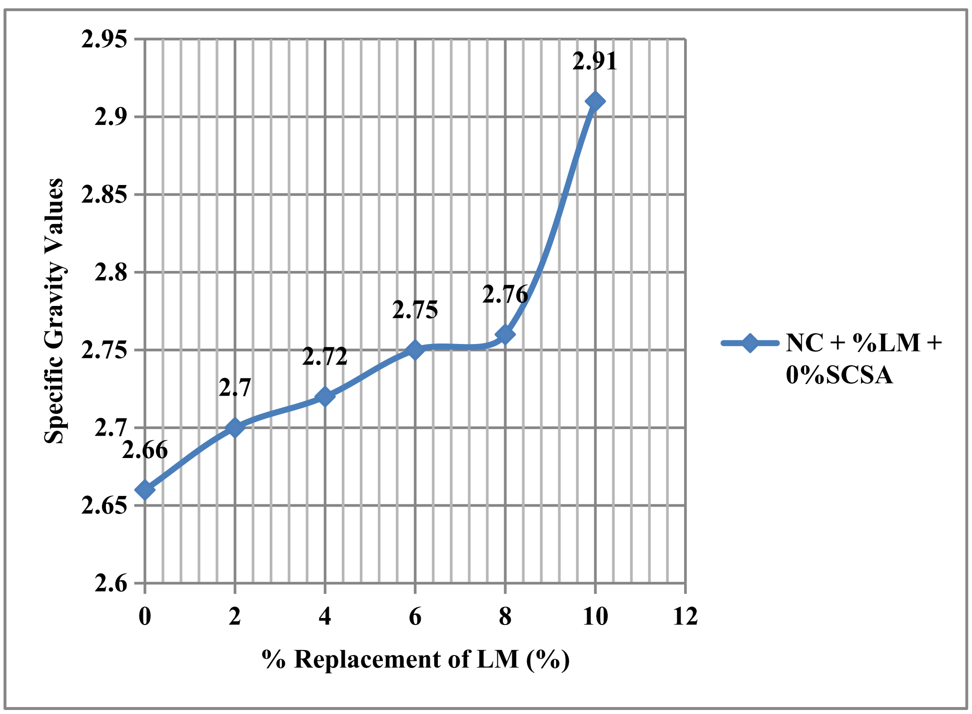


Figure 4.2b: Specific Gravity Graph for NC + 0%LM + %SCSA

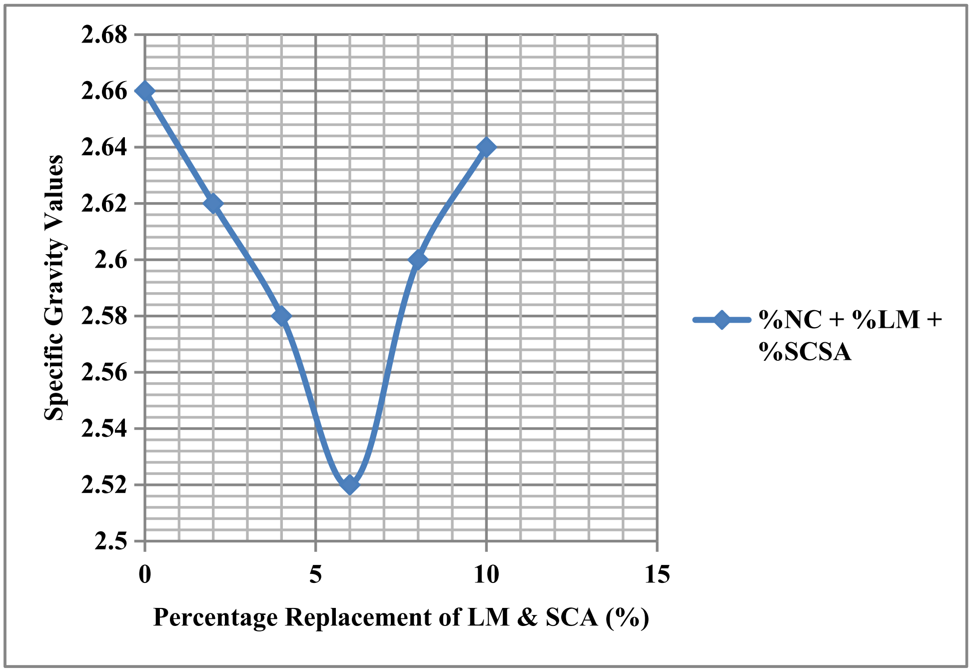


Figure 4.2c: Combined Specific Gravity Graph for Composite Mixture

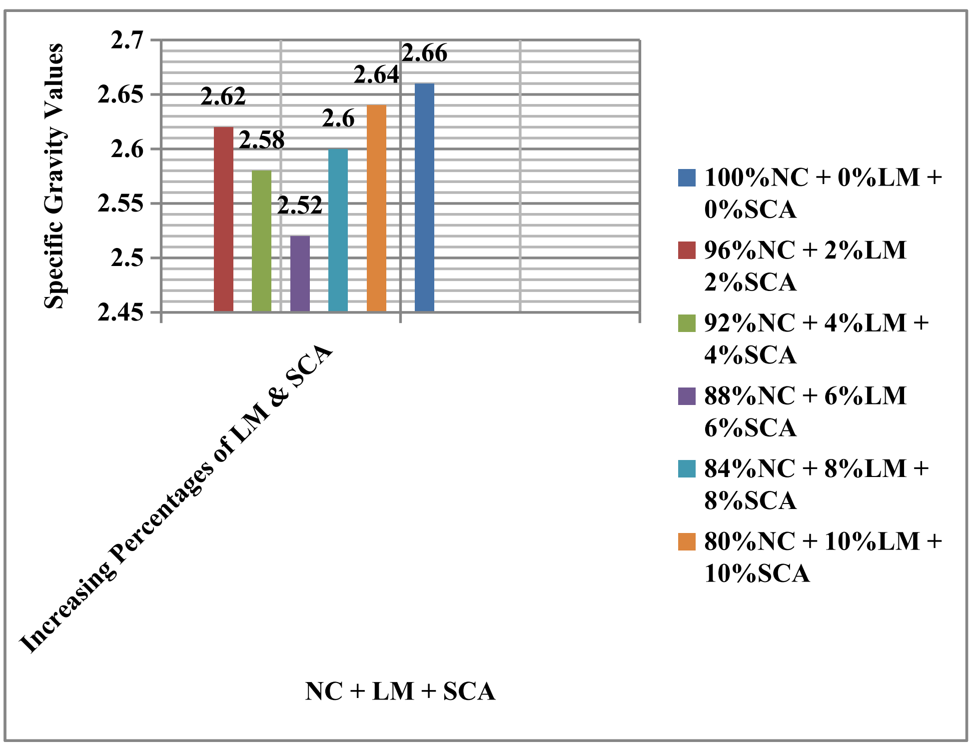


Figure 4.2d: Combined Specific Gravity Chart for Composite Mixture

4.2.4 Liquid Limit

Atterberg limit (liquid limit, plastic limit and shrinkage limit) are indices of the quantity of clay sized particles and their mineralogical composition. Typically higher liquid limit and plasticity index values are linked with soils having significant amount of clay particles or particles having higher surface activity. Also, lower hydraulic conductivities is associated with soil having higher liquid limit or plasticity index (Benson et al,.1994). Result obtained from the liquid limit test for the combination of natural clay and lime suggest that addition of lime to natural clay was characterized by initial decline in the natural value of clay sample at 2%LM content beyond this point, the liquid limit increased while that obtained from natural clay to sugar cane straw ash combination shows that the liquid limit of natural clay increased from 54.46% to 61.84% at 4%SCSA content beyond this point, the liquid limit decreased. The subsequent decrease can be attributed to the increasing SCSA content which provides lesser surface area for hydration and also due to it porous texture. The result obtained from the composite mixture (NC + %LM + %SCSA) suggest a decreasing trend in liquid limit value of natural clay samples. The later decrease in liquid limit value of the composite mixture is advantageous as the final value obtained after 10%LM & SCSA addition falls within the range specified by Federal Ministry of Works Standard and Specification for roads and bridges (1997) which state that the liquid limit of sub-grade material should not exceed 80%. This finding is in agreement with the works of Azeem, et al. (2020).

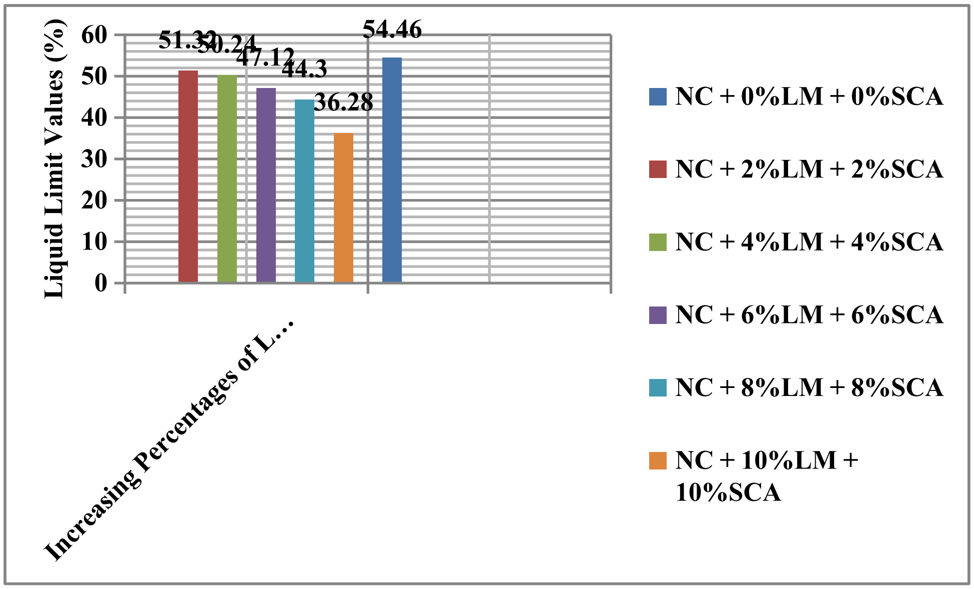


Figure 4.3a: Liquid Limit Chart for the Composite Mixture

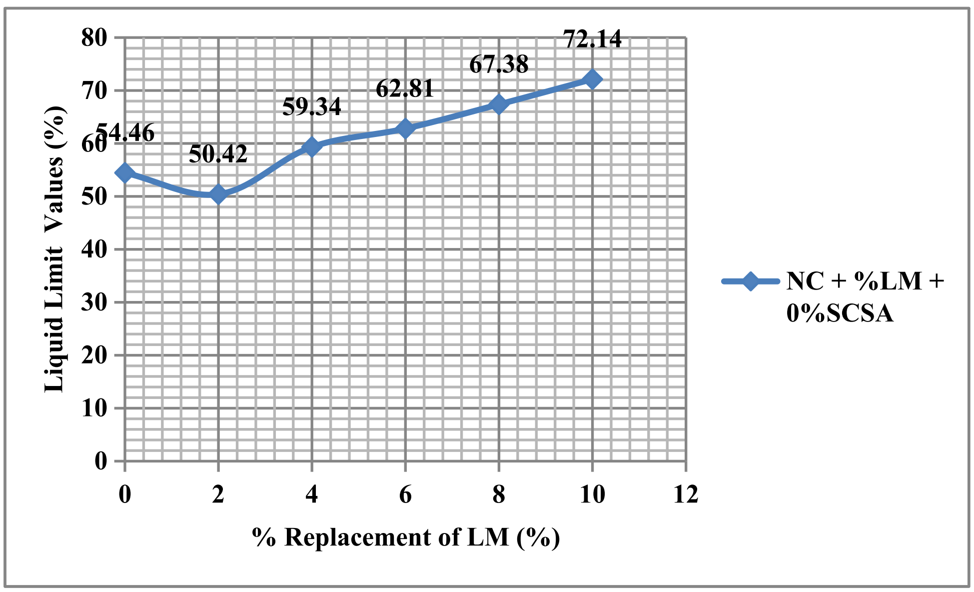


Figure 4.3b: Liquid Limit Graph for the NC + %LM + 0%SCS

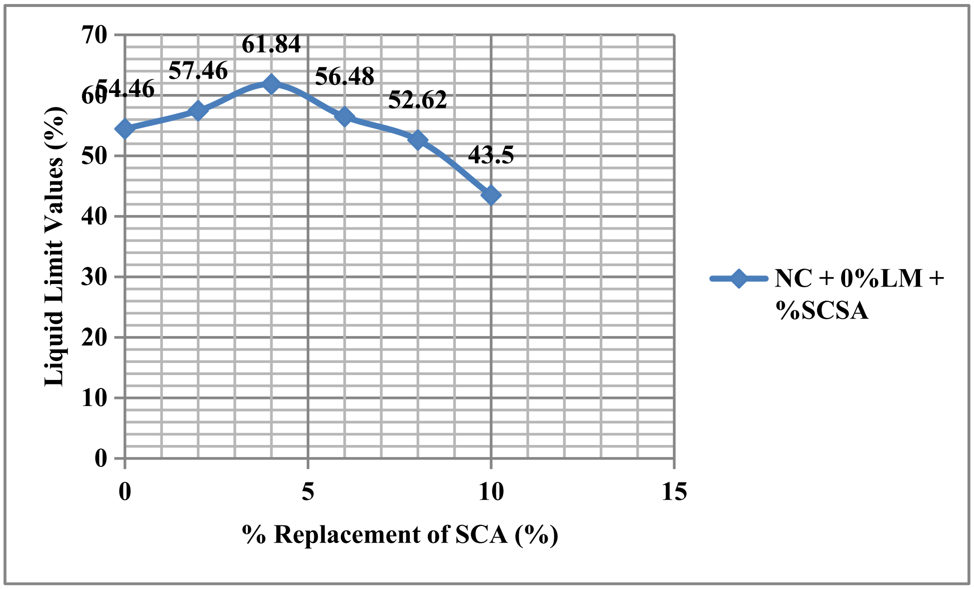


Figure 4.3c: Liquid Limit Graph for the NC + 0%LM + %SCSA

4.2.5 Plastic Limit

Figure 4.4a-c shows the plastic limit graphs for NC + LM, NC + SCSA and NC + LM + SCSA combination. The plastic limit result obtained from natural clay to lime combination indicate an overall increasing trend in plastic limit value of natural clay from it natural value of 33.66 to 50.2% at 10%LM content while for natural clay to sugar cane straw ash combination, an overall decreasing trend in plastic limit value of natural clay was observed. This decrease can be attributed to the relatively non-plastic nature of SCSA. The plastic limit value obtained for the composite mixture mirrors that of natural clay to sugar cane straw ash combination as an overall decreasing trend in plastic limit value of natural clay were equally observed. This finding contradicts the works of Asma and Darius, (2013).

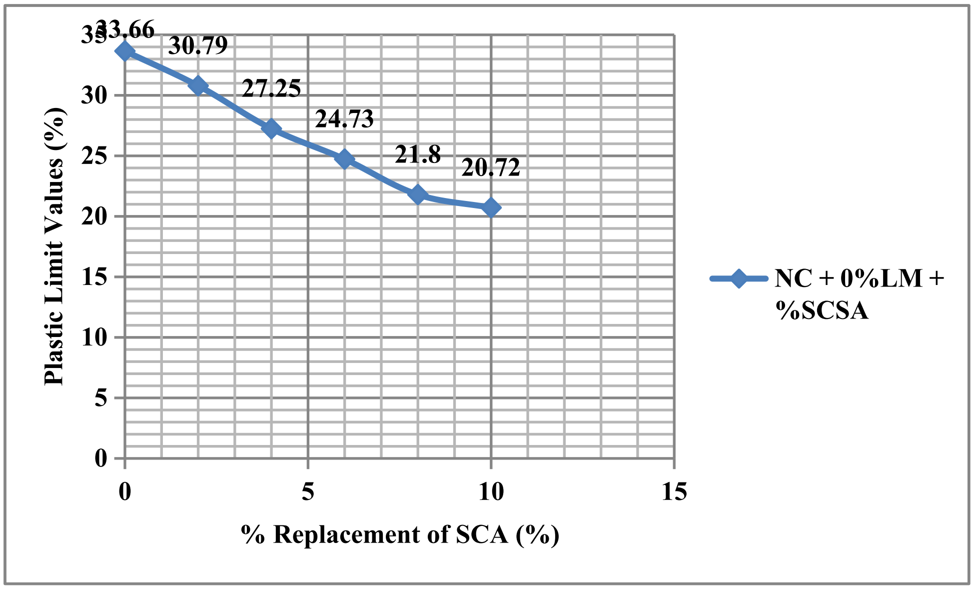


Figure 4.4a: Plastic Limit Graph of NC + 0%LM + %SCSA.

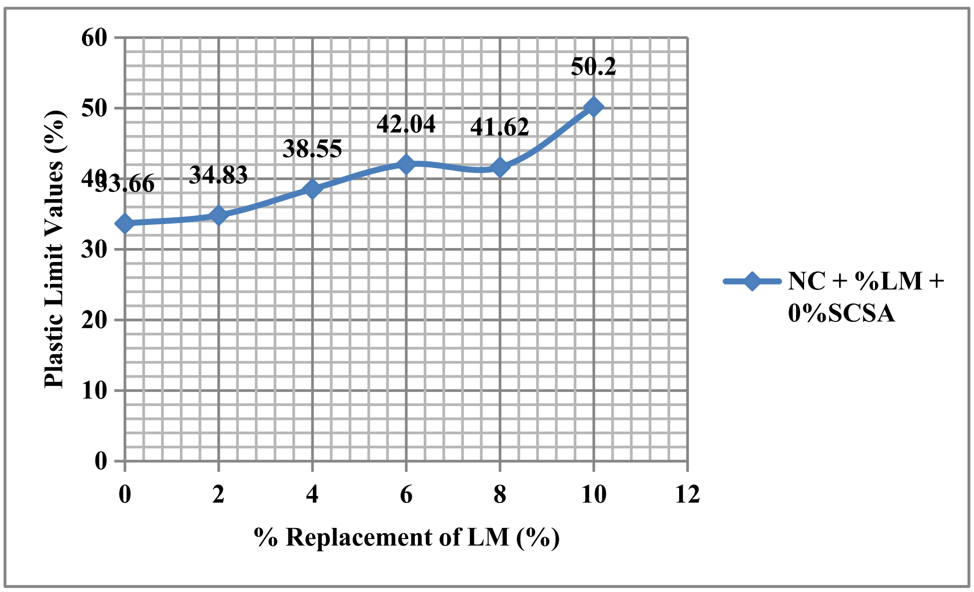


Figure 4.4b: Plastic Limit Graph of NC + %LM + 0%SCSA.

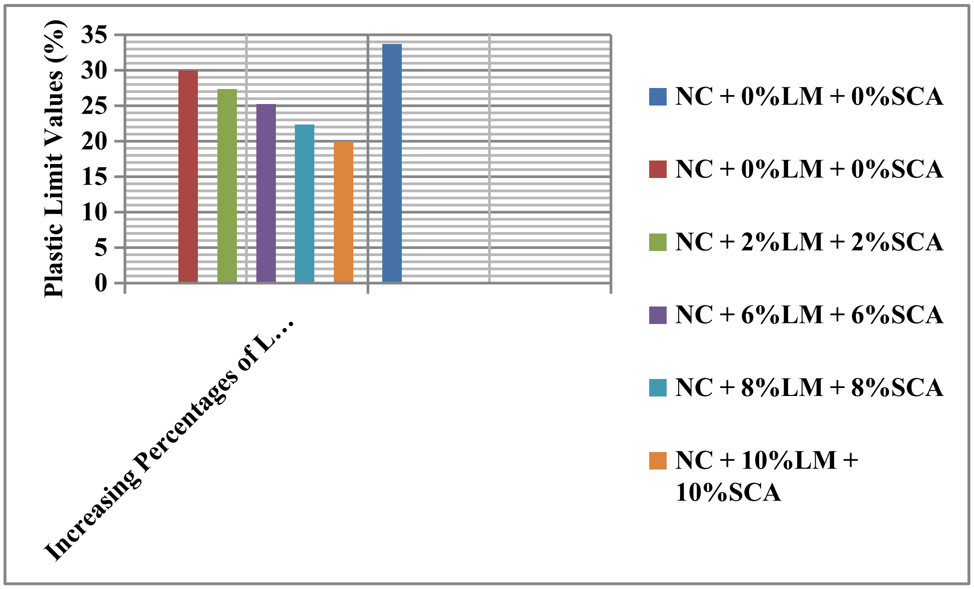


Figure 4.4c: Chart Showing the Plastic Limit of Composite Mixture

4.2.6 Plasticity Index

The plasticity index is the numerical difference between the liquid limit and plastic limit of soils, it is an index of the amount of clay and it mineralogical composition. The plasticity index result obtained for natural clay to lime combination suggest that the plasticity index value of natural clay initially decreased from 20.9 to 15.59% at 2%LM addition but beyond this point, the plasticity index of natural clay increased with the plasticity rating been relatively high between (17-35%) as suggested by Braja, (2002). The plasticity index result obtained for natural clay to sugar cane straw ash addition suggest a consistent increase in plasticity index value of natural clay from 20.9% to 30.82% from 0%SCSA to 8%SCSA content which was ended by a decrease at 10%SCSA content. The plasticity index of the composite mixture (NC + %LM + %SCSA) was characterized by an increase from 0%LM & SCSA content to 6%LM & SCSA content beyond this point, the plasticity index of the mixture decreased. The overall trend in increase and decrease in plasticity index ultimately conform to the specification by the Federal Ministry of Works Standard and Specification for road and bridges (1997) which states that the plasticity index of sub-grade material should not exceed 55%.

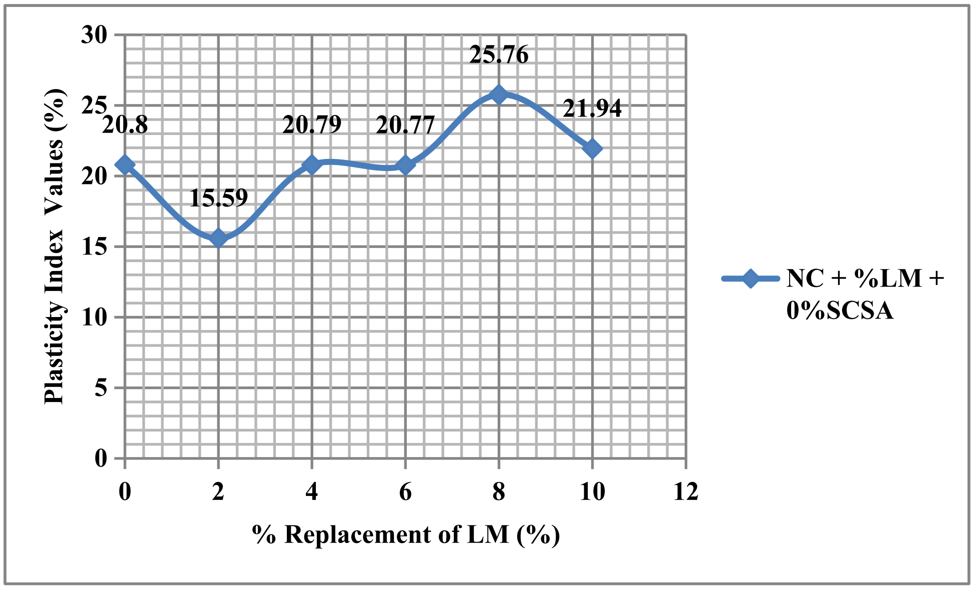


Figure 4.5a: Graph Showing the Plasticity Index Value of NC + %LM + 0%SCSA.

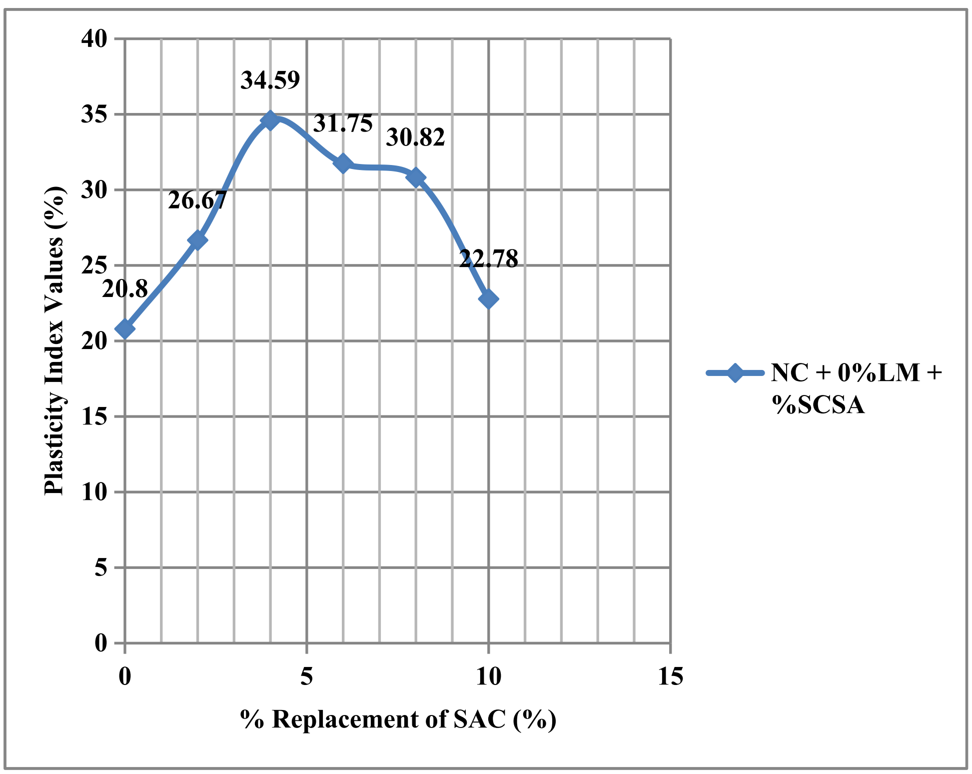


Figure 4.5b: Graph Showing the Plasticity Index Value of NC + 0%LM + %SCSA.

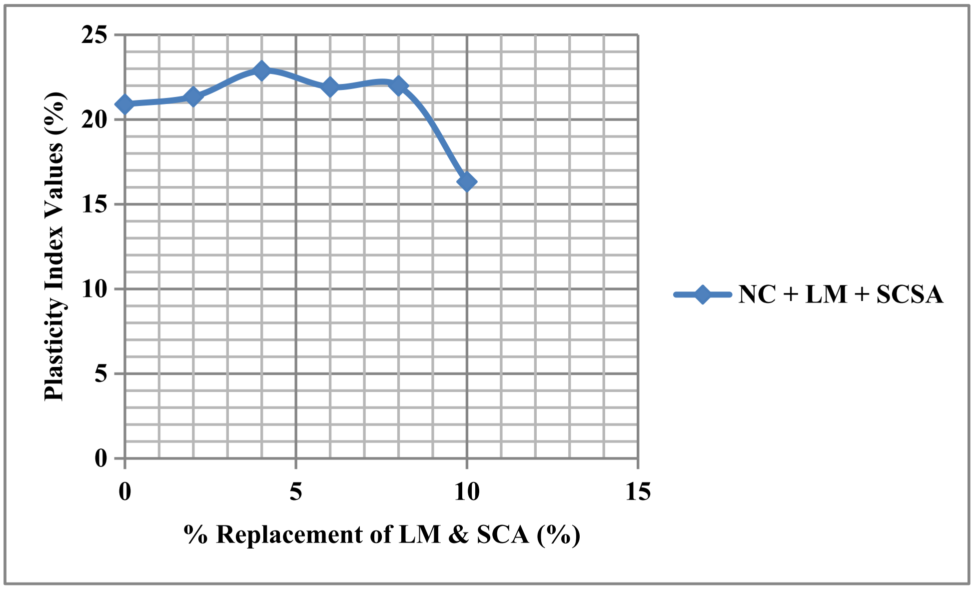


Figure 4.5c: Graph Showing the Plasticity Index Value of Composite Mixture.

4.2.7 Compaction

Figure 4.6a-f shows the compaction curve for NC + LM, NC + SCSA and NC + LM + SCSA combination. Result obtained from natural clay to lime combination suggest overall consistent increase in the maximum dry unit weight of natural clay from 20.29kN/m^3 to 22.19kN/m^3 after 10% addition of lime which was characterized with a fluctuation in optimum moisture content value. The increased can be attributed to the high specific value of lime. It has been reported by Osinubi et al., (2015) and Etim et al., (2017) that specific gravity of the admixtures significantly influences the density of the compacted soil. The higher specific gravity of admixtures gives rise to the higher density of the compacted soil which favours its suitability as a road construction material. The increase in specific gravity shows an improvement in the geotechnical properties of the treated soil. Therefore, such improvement makes the soil more suitable for use in the construction of roads and as a fill material, as increased density automatically increases the strength and shear resistance of the material (Osinubi et al., 2015; Etim et al., 2017).

Result obtained from compaction test of natural clay to sugar cane straw ash combination suggest an overall decreasing trend in maximum dry unit weight of natural clay characterized with an increasing optimum moisture content which is in agreement with the of works of This agrees with Proctor (1933), Venkatramaiah (2006), Rowe (2000) and other concluded research work where maximum dry unit weight was found to decrease with an increasing optimum moisture content. Result obtained from the overall combination of natural clay, lime and sugar cane straw ash suggest an improvement in the maximum dry unit weight of natural clay up to 6%LM & SCSA content which was characterized by later decrease.

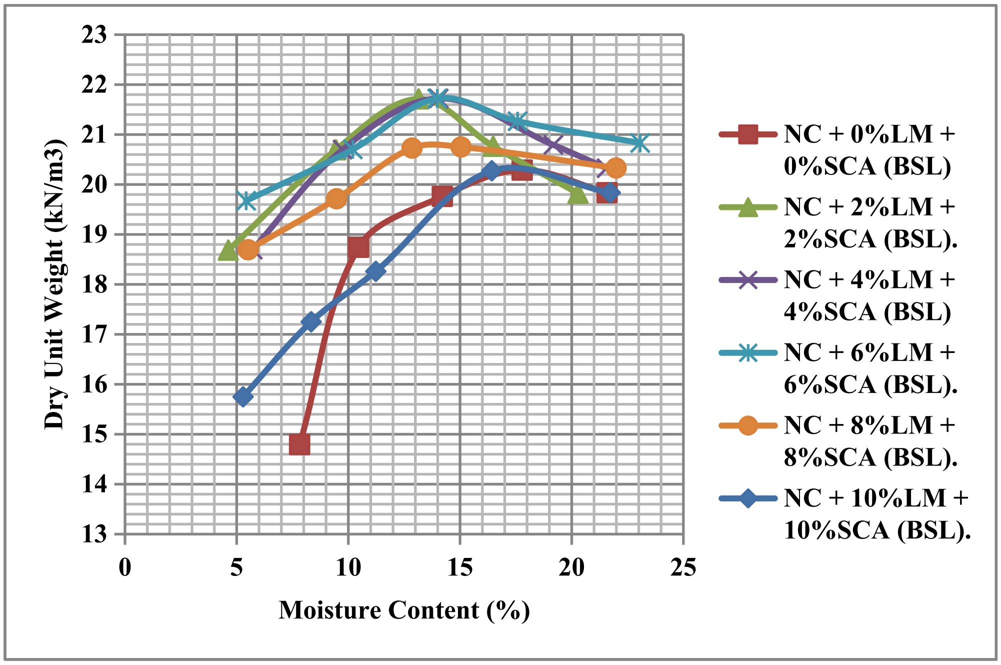


Figure 4.6a: Compaction Curve for the Composite Mixture

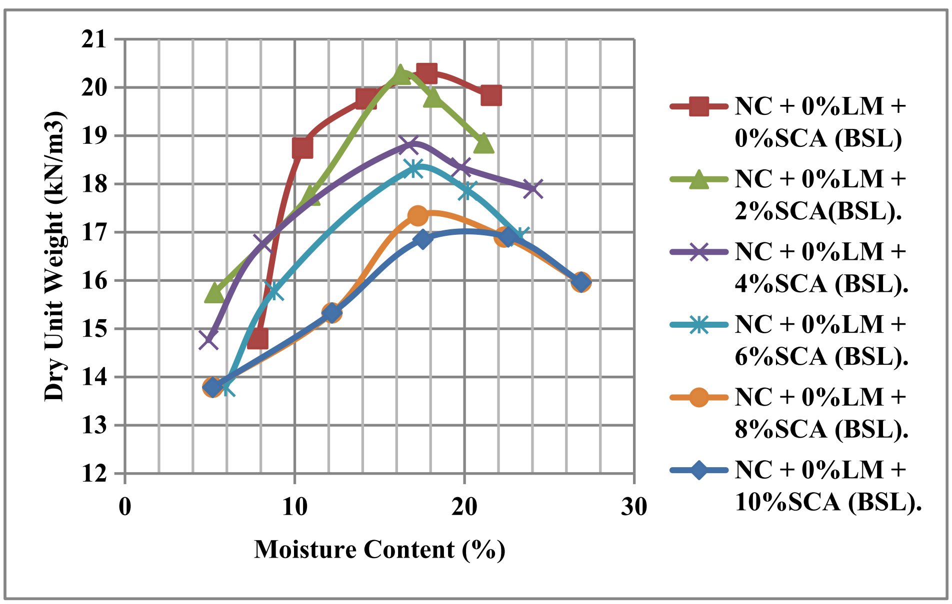


Figure 4.6b: Compaction Curve for NC + 0%LM + %SCSA.

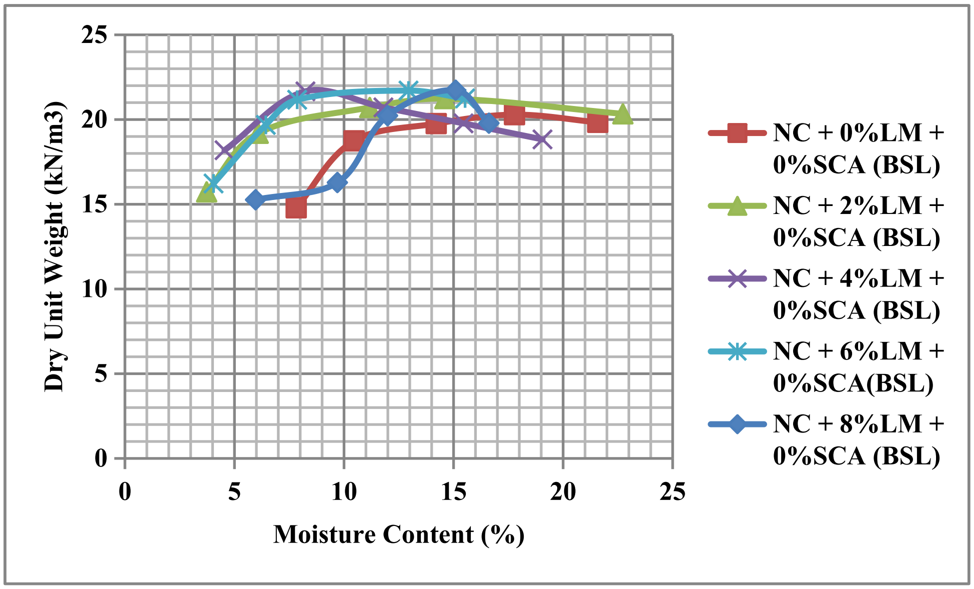


Figure 4.6c: Compaction Curve for NC + %LM + 0%SCSA.

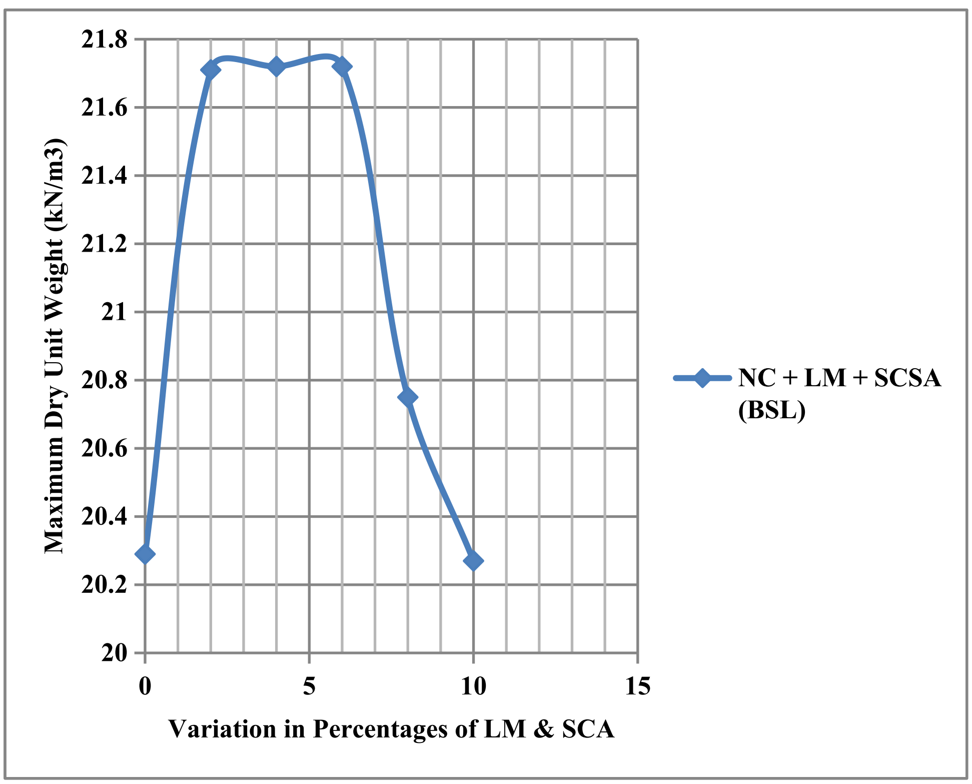


Figure 4.6d: Graph of MDUW against Variation in % of LM & SCSA

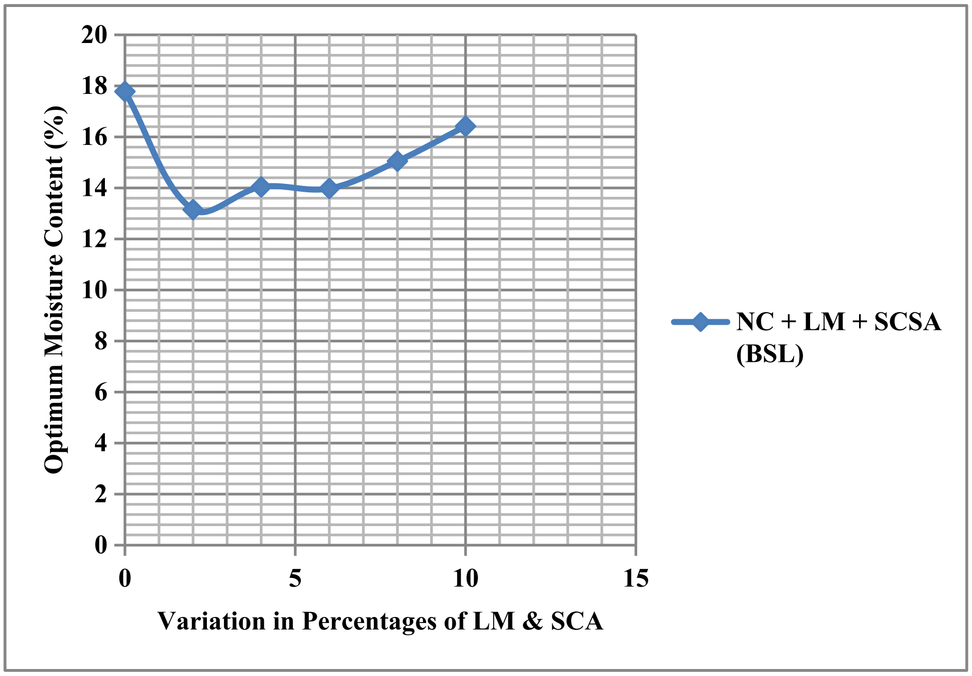


Figure 4.6e: Graph of OMC against Variation in % of LM & SCSA

4.2.8 California Bearing Ratio

The California bearing of the mixture obtained after 10% addition of lime and sugar cane straw ash the California bearing ratio of natural clay increased from 14% to 17% after 4% addition of lime and sugar cane straw ash to natural clay but beyond 4%LM & SCSA content, the California bearing ratio value decreased, this result was obtained after two days of soaking while for the unsoaked sample, the California bearing ratio increased from 14.5% to 22% up to 8% Lm & SCSA content which culminated with a sharp drop at 10%LM & SCSA content. The strength loss in the soaked sample can be attributed to the high water absorption capacity of both natural clay and lime. It can also be deduced the California bearing ratio result obtained for the soaked sample justifies the use of the material at the sub-grade level of pavement construction as it conforms to the specification given by the Federal Ministry of Works Standard and Specification for construction of roads and bridges (1997) which state that the California bearing ratio of a soaked sample must exceed 10% for use at the pavement level and 20% for unsoaked samples. Therefore the California bearing ratio obtained for both soaked and unsoaked sample justifies the samples for use at the sub-grade level of pavement construction. It was also observed that the California bearing ratio obtained for unsoaked sample was comparatively higher than that obtained for soaked sample, this finding mirror the works of Azeem et al. (2020).

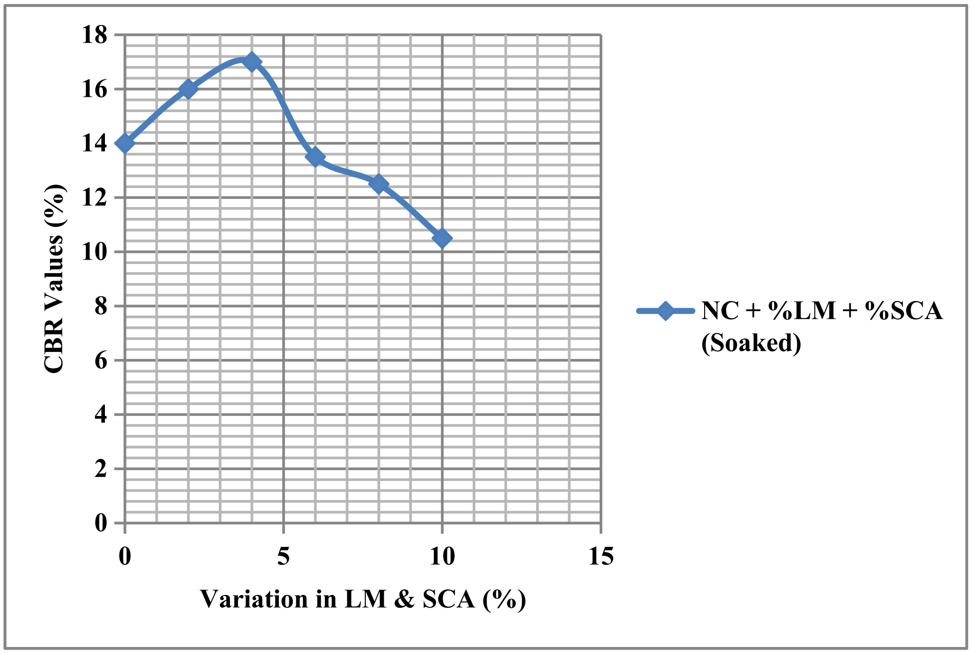


Figure 4.7a: Graph Showing the CBR Values of NC + %LM + %SCSA (Soaked).

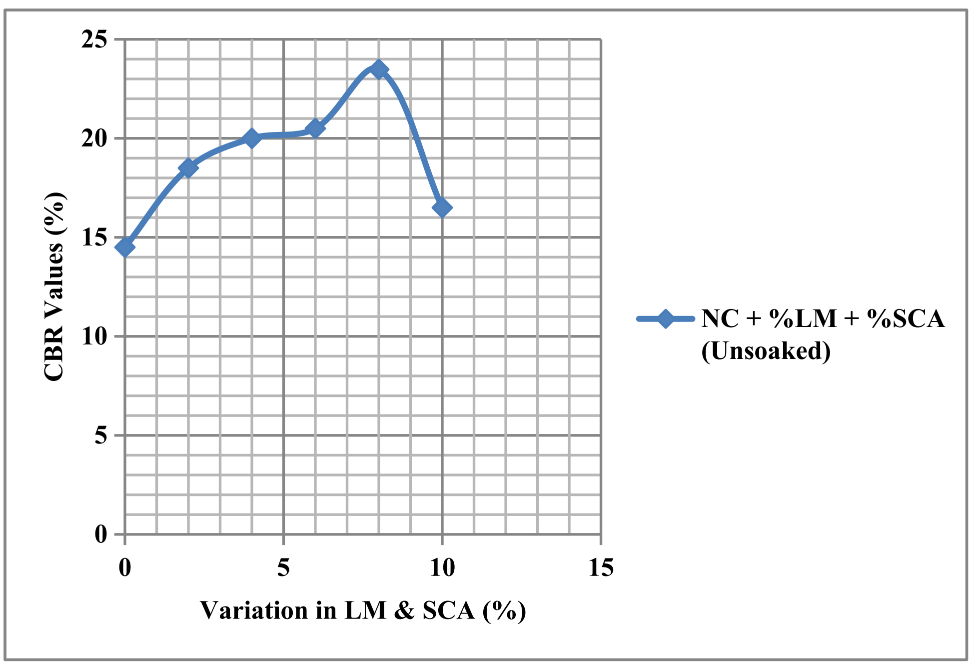


Figure 4.7b: Graph Showing the CBR Values of NC + %LM + %SCSA (Unsoaked).

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the findings obtained the following conclusion can be drawn:

1 Sieve analysis test shows that the percentage passing sieve no 200 (0.075mm) is 74.71 and according to AASHTO classification system, it is classified as A-7-6 and CH (clay of high plasticity) according to Unified Soil Classification System (USCS). Since more than 35% passes through sieve no 200 (0.075mm), according to AASHTO sub-grade rating for soil samples, this sample will therefore constitute a fair to poor soil if used at the sub-grade level of pavement construction and therefore need to be sufficiently stabilized.

2 Result of moisture content obtained from combination of natural clay to lime suggest that addition of lime to natural clay increases the moisture content of natural clay from it natural value of 34.58% to 41.38% after 10% addition of lime content. Result obtained from natural clay to sugar cane straw ash suggest that addition of sugar cane straw ash to clay raises the moisture content of clay samples from 34.58% to 35.99% after 2% addition of sugar cane straw ash, beyond 2%SCSA content, the moisture content of clay sample decreased consistently while the addition of lime and sugar cane straw ash to clay decreased the moisture content of natural clay from it natural value of 34.58% to 31.17% at 6%LM an SCSA content beyond this point, the moisture content increased.

3 The specific gravity result obtained from combination of natural clay to lime suggest that addition of lime to natural clay increased the specific gravity of natural clay from it natural value of 2.66 to 2.91 after 10% addition of lime content.

4 Result obtained from the liquid limit test for the combination of natural clay and lime suggest that addition of lime to natural clay was characterized by initial decline in the natural value of clay sample at 2%LM content beyond this point, the liquid limit increased while that obtained from natural clay to sugar cane straw ash combination shows that the liquid limit of natural clay increased from 54.46% to 61.84% at 4%SCSA content beyond this point, the liquid limit decreased.

5 The plastic limit result obtained from natural clay to lime combination indicate an overall increasing trend in plastic limit value of natural clay from it natural value of 33.66 to

50.2% at 10%LM content while for natural clay to sugar cane straw ash combination, an overall decreasing trend in plastic limit value of natural clay was observed.

6 The plasticity index result obtained for natural clay to lime combination suggest that the plasticity index value of natural clay initially decreased from 20.9 to 15.59% at 2%LM addition but beyond this point, the plasticity index of natural clay increased with the plasticity rating been relatively high between (17-35%) as suggested by Braja, (2002).

7 Compaction test result obtained from natural clay to lime combination suggest overall consistent increase in the maximum dry unit weight of natural clay from 20.29kN/m^3 to 22.19kN/m^3 after 10% addition of lime which was characterized with a fluctuation in optimum moisture content value. Result obtained from compaction test of natural clay to sugar cane straw ash combination suggest an overall decreasing trend in maximum dry unit weight of natural clay characterized with an increasing optimum moisture content.

8 The California bearing of the mixture obtained after 10% addition of lime and sugar cane straw ash the California bearing ratio of natural clay increased from 14% to 17% after 4% addition of lime and sugar cane straw ash to natural clay but beyond 4%LM & SCSA content, the California bearing ratio value decreased, this result was obtained after two days of soaking while for the unsoaked sample, the California bearing ratio increased from 14.5% to 22% up to 8% Lm & SCSA content which culminated with a sharp drop at 10%LM & SCSA content.

9 It can be deduced that the stabilizer (LM & SCSA) are feasible and effective since an improvement in both strength and compaction properties of natural clay was observed.

10 The stabilization of natural clay with lime and sugar cane straw ash produces specimen that satisfies the requirement for use at the sub-grade level of pavement construction.

5.2 Recommendation

From the findings gleaned from the research, the following recommendation can be made:

1. Natural clay stabilized with lime and sugar cane straw ash can only be used at the sub-grade level of pavement construction as advancement from this stage might result to the development of pavement distresses.

2. The recommendation 1 can be subject to further research to ascertain whether other materials can be economically added so as to justify the use of natural clay beyond the sub-grade level.

REFERENCE

AASHTO (1986). Standard Specifications for Transportation Materials and Methods of Sampling and Testing. 14th edition, American Association of State Highway and Transportation Officials, Washington, DC.

Al-kiki, S., Mohammed, F., & Devashish P. (2013). "Soil stabilization using lime", International Journal of Innovative Research in Science, Engineering and Technology, Vol. 2, Issue 2, ISSN: 2319-8753.

Aroja, M., & Nagaraj, K. (2017). "Soil Stabilization Using Geosynthetic Material, Visvesvaraya Technological University Belgium (Pp.9).

Arora, K. R. (2011). "Soil Mechanics and Foundation Engineering", Sixth Edition (Revised and Enlarged).

Asiagwu,G., Newill D, & Schreiner H. D. (2002). "Expansive soils: TRL's research strategy", in Proc. 1st Int. Symposium on Engineering Characteristics of Arid Soils, London, UK, pp. 247-60.

Asma, M., & Darius, W. (2013). Effect of Lime Stabilisation on the Strength and Microstructure of Clay. IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 6, Issue 3 PP 87-94.

Azeem,A., Saurabh, S., Divya, S., & Shahnawaz, A. (2020). Effect of Bagasse ash and Lime on the Compaction and Strength Properties of Black Cotton Soil. International Journal of Engineering Research & Technology (IJERT), Vol. 9 Issue 05, ISSN: 2278-0181.

Benson, P., Schaeffer, B., Temgoua, E., Gratier, M., & Steinman, G. (1994). "Assessment of soilcompaction using shrinkage curve measurement and modelling". Experimental results and perspectives. Soil and Tillage Research 88: 65–79.

Bowled, P. (1938). Stabilization of expansive clay using lime and sugarcane bagasse ash. International Journal of Science and Research (IJSR), Vol. 4, No. 4, pp. 2112-2117.

Braja, M. (2002). Soil Mechanics Laboratory Manual 6th Edition, New York, Oxford University Press.

Braja, M. (2006) "Specific Gravity" In Principle of Geotechnical Engineering, 7th Edition, Cengage Learning (Pp.34).

Broderick A. & Daniel, S. (2013). “Effect of Lime on the Index Properties of Black Cotton Soil and Mine tailings mixtures.”IOSR,, Vol.3,No.4. Pp.1-7.

Phanikumar, B.R. (2009). Effect of lime and fly ash on swell, consolidation and shear strength characteristics of expansive clays: a comparative study, Journal of Geomechanics and Geoengineering, 4(2), 175-181.

British Standard 1377. (1990). Methods of Test for Soil for Civil Engineering Properties (BS 1377). British Standard Institution: London, UK, 143p.

Butt, M. Vijay and D. Keerthi, D. (2016). Agricultural wastes as soil stabilizer. International Journal of Earth Sciences and Engineering, 04(06): 50-51.

Chen, F.H. (1988). Foundations on Expansive Soils, seconded. Elsevier Scientific Publishing Co., Amsterdam, The Netherlands.

Das, B. M. (1990). Principles of Geotechnical Engineering. Seventh Edition, PWS-KENT Publishing Company, Boston. pp 34-35.

De. Graft-Johnson, J.W.S. and Bhatia, H.S. (1969). Engineering properties of lateritic soils. General Report, Specific Session on Engineering Properties of lateritic soils, Seventh International Conference on Soil Mechanics and Foundation Engineering Mexico City, Vol. 1:117-128.

Edil, D and Craig, J., (2007). Improvement of subgrade CBR value by using Bagasse ash and Eggshell powder. International Journal of Advanced Structures and Geotechnical Engineering Vol. 4, No. 2, pp. 86-91.

Etim, R.K., Eberemu, A.O., & Osinubi, K.J (2017).Stabilization of Black Cotton Soil with Lime Iron Ore Tailings Admixture, Journal of Transportation Geotechnics. Elsevier.10:85–95.http://dx.doi.org/10.1016/j.trgeo.2017.01.00

Federal Ministry of Works and housing, (1997) “General Specifications For roads and bridges” Volume II, Federal highway department Lagos Nigeria. Pp: 145.

F. G. Bell(1989) Lime Stabilization of Clay Soils, Bulletin of Engineering Geology and the Environment, 39(1), 67-74.

Hansbo, H. (1969). “Expansive soils: TRL’s research strategy”, in Proc. 1st Int. Symposium on Engineering Characteristics of Arid Soils,, London, UK, pp. 247-60.

Haraguchi.H.N, Manoj.K.V and Mamatha.H.V (2010), “Compaction and Strength Behaviour of Lime treated Black Cotton Soil” Geomechanics and Engineering, Vol.2, No.1, pp 19-28.

Hicks, A. (2002). Expansive soils, ICE manual of geotechnical engineering. London: ICE Publishing, 2012, pp. 413-441.

Hossain, K. M. A., & Mol, L. (2011). Some engineering properties of stabilized clayey soils incorporating natural pozzolans and industrial wastes. Construction and Building Materials, 25(8), 3495-3501.

Ingles, O.G. , Metcalf, J.B. and Woods, R. (1993). Soil Stabilization Principles and Practice. Butterworth, Sydney

Ikeagwani, C.C., Nwonu, D.C., 2018. Emerging trends in expansive soil stabilization – a review.

J.L. Eadesand R.E. Grim(1966) A Quick Test to Determine Lime Requirements of Lime Stabilisation, Highway Research Record 139, pp. 61-72. J. Rock Mech. Geotech. Eng. 11 (2), 423–440.

K. A. Kassim(2009) The Nanostructure Study on the Mechanism of Lime Stabilised Soil, Research Vot No: 78011, Department of Geotechnics and Transportation, UniversitiTeknologi Malaysia.

K.J. Osinubi, V. Bafyau, A.O. Eberemu, and O. Adrian, (2014). Bagasse Ash Stabilization of Lateritic Soil. E.K. Yanful (ed.), Appropriate Technologies for Environmental Protection 281in the Developing World. [Accessed 12th December, 2022].

Khalid, R., Hasan H., Fatahi B., Jones R., and Khabbaz H., (2012). Enhancing the engineering properties of expansive soil using bagasse ash and hydrated lime. International Journal of GEOMATE, Vol. 11, No. 25, pp. 2447 - 2454.

Krishna, R. (2002) “Specific Gravity Determination” http// users, rowan, edu/Surkmaran/geotechnical/notes/Experiments %204-Specific %20 Gravity.pdf

Little, D. N. (1999) Evaluation of Structural Properties of Lime stabilised soils and aggregates. The national lime association, Vol. 1.

Little D., and Nair, F., (2009). Evaluation of structural properties of lime stabilized soils and aggregates, National Lime Association.

Loughman, B. (1969). Sugar cane wastes as pozzolanic materials: applications of mathematical model. American Concrete Institute.

Maher, D., Sobha C. & Babu.T. (1994) "Effect Of Inclusion Of Coir Fiber On The Shear Strength Of Marine Clay" Proceedings Of Indian Geotechnical Conference Kochi (Paper No.H-070)

Maher, M.H. and Woods, R.D. (1990). Dynamic Response of Sand Reinforced with Randomly Distributed Fibers. Journal of Geotechnical Engineering, Vol. 116, No.7, pp. 1116-1131.

Misari, S.M., Busari, L.D. & Agboire, S. (1996). Current Status of Sugar Cane Research and Development in Nigeria. Proceedings of National Co-ordinate Research Programme on Sugar Cane, NCRI, Badeggi, pp. 2–12.

Nevile, G. (2000). Effect of sugarcane straw ash on cement stabilized lateritic soil for use as flexible pavement material. Journal of Emerging Technologies and Innovative Research (JETIR), Vol. 5, No. 2, pp. 563-567.

O'Flaherty, C.A (21974). Highways: The location, design and maintenance of road pavements. 4th ed. Butterworth Heinemenn, Jordan Hill Oxford.

Osinubi, K. J., Yohanna, P & Eberemu, A. O. (2015). Cement Modification of Tropical Black Clay Using Iron Ore Tailing as Admixture. Journal of Transportation Geotechnics. 5:35-49.

Perry , H.N., Krishnaiah A.J. and Shilpa, S., (1997). Effect of Lime on the Index Properties of Black Cotton Soil and Mine tailings mixtures."IOSR,, Vol.3,No.4. Pp.1-7.

Phanikumar, B.R., Nagaraju, T.V. (2018). Influence of fly ash (FA) and rice husk ash (RHA) on properties of expansive clays-a comparative study. Geotech. Geol. Eng. https://doi.org/10.1007/s10706-018-0544-5.

Roger,AK, Kumar P & Ransinchung G. D. (1993) . Use of various agricultural and industrial waste materials in road construction", 2nd Conference of Transportation Research Group of India, Pp. 264-73.

Rowe, K. (2000). Geotechnical and Geo-environmental Engineering Handbook:Kluver Academic Publishers.

Seed, G. (1954). "Lime stabilization of clay minerals and soils", Engineering Geology, vol. 42, pp. 223-237.

Sherwood, P.T. (1993). Soil Stabilization with Cement and Lime. HMSO Publications Center, London, pp. 14–55.

Shruthi, H. (2017). Compaction Characteristics of Soil. Sanjivani College of Engineering Kopargaon (Pp .3) (http://www.slideshares. Net/ Shruthi Hire maths/ Compaction Characteristics of soil).

Ventatramaiah, C. (2006). Geotechnical Engineering, Revised 3rd Edition, New Age International Publishers New Delhi.

Young, F & Ouhadi, G. (2007). Expansive soils: problems and practice in foundation and pavement engineering. New York: Wiley-Interscience.

Winterkorn, J. (1946). Stabilisation of clayey soils with industrial by-products: part A. Proceedings of the Institution of Civil Engineers-Ground Improvement, 163(3), 149–163.

APPENDICES

APPENDIX A

SPECIFIC GRAVITY TEST

Table A1. Specific Gravity Result for 100%NC + 0%LM + 0%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.76	25.64	25.90
Wt of bottle + dry soil, W_2 (g).	34.74	35.63	35.90
Wt of bottle + soil + water, W_3 (g).	84.33	85.15	85.79
Wt of bottle + water, W_4 (g).	78.07	78.94	79.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100%NC + 0%LM + 0%SCSA

Trial 1 (G_{S1}) = = = 2.68

Trial 2 (G_{S2}) = = = 2.64

Trial 3 (G_{S3}) = = =2.65

Specific Gravity = = = 2.66

Table A2: Specific Gravity Result for 96%NC + 2%LM + 2%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	23.87	25.21	25.54
Wt of bottle + dry soil, W_2 (g).	33.86	35.21	35.54
Wt of bottle + soil + water, W_3 (g).	82.91	81.13	79.94
Wt of bottle + water, W_4 (g).	76.72	74.95	73.77

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 96%NC + 2%LM + 2%SCSA

Trial 1 (G_{S1}) = = = 2.63

Trial 2 (G_{S2}) = = = 2.62

Trial 3 (G_{S3}) = = =2.61

Specific Gravity = = = 2.62

Table A3: Specific Gravity Result for 92%NC + 4%LM + 4%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density	25.12	24.86	25.42

bottle, W_1 (g).			
Wt of bottle + dry soil, W_2 (g).	**35.11**	**35.85**	**35.42**
Wt of bottle + soil + water, W_3 (g).	**80.09**	**82.75**	**79.71**
Wt of bottle + water, W_4 (g).	**73.96**	**76.63**	**73.60**

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 92%NC + 4%LM + 4%SCSA

Trial 1 (G_{S1}) = = = 2.59

Trial 2 (G_{S2}) = = = 2.58

Trial 3 (G_{S3}) = = =2.57

Specific Gravity = = = 2.58

Table A4: Specific Gravity Result for 88%NC + 6%LM + 6%SCSA.

Determinants	**Trial 1**	**Trial 2**	**Trial 3**
Wt of density bottle, W_1 (g).	**25.41**	**24.70**	**24.11**

Wt of bottle + dry soil, W_2 (g).	35.41	34.68	34.11
Wt of bottle + soil + water, W_3 (g).	81.32	83.41	78.88
Wt of bottle + water, W_4 (g).	75.25	77.37	72.88

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 88%NC + 6%LM + 6%SCSA.

Trial 1 (G_{S1}) = = = 2.54

Trial 2 (G_{S2}) = = = 2.53

Trial 3 (G_{S3}) = = =2.50

Specific Gravity = = = 2.52

Table A5:Specific Gravity Result for 84%NC + 8%LM + 8%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.14	25.88	25.26
Wt of bottle + dry soil, W_2 (g).	36.14	35.87	35.25

Wt of bottle + soil + water, W_3 (g).	86.14	84.26	82.14
Wt of bottle + water, W_4 (g).	79.96	78.13	76.02

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 84%NC + 8%LM + 8%SCSA.

Trial 1 (G_{S1}) = = = 2.62

Trial 2 (G_{S2}) = = = 2.59

Trial 3 (G_{S3}) = = = 2.58

Specific Gravity = = = 2.60

Table A6: Specific Gravity Result for 80%NC + 10%LM + 10%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.52	25.94	25.16
Wt of bottle + dry soil, W_2 (g).	34.52	35.93	35.15
Wt of bottle + soil + water, W_3 (g).	84.92	82.19	86.24

Wt of bottle + water, W_4 (g).	78.71	75.94	80.05

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80%NC + 10%LM + 10%SCSA.

Trial 1 (G_{S1}) = = = 2.63

Trial 2 (G_{S2}) = = = 2.67

Trial 3 (G_{S3}) = = = 2.63

Specific Gravity = = = 2.64

Table A7: Specific Gravity Result for SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.44	24.69	25.12
Wt of bottle + dry soil, W_2 (g).	34.43	34.68	35.12
Wt of bottle + soil + water, W_3 (g).	82.51	84.65	81.07

Wt of bottle + water, W_4 (g).	76.54	78.75	75.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for SCSA.

Trial 1 (G_{S1}) = = = 2.49

Trial 2 (G_{S2}) = = = 2.50

Trial 3 (G_{S3}) = = =2.46

Specific Gravity = = = 2.48

Table A8: Specific Gravity Result for LM

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.24	26.21	24.68
Wt of bottle + dry soil, W_2 (g).	34.23	36.21	34.68
Wt of bottle + soil + water, W_3 (g).	83.95	84.69	82.45
Wt of bottle + water, W_4 (g).	77.67	78.35	76.15

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LM

Trial 1 (G_{S1}) = = = 2.69

Trial 2 (G_{S2}) = = = 2.73

Trial 3 (G_{S3}) = = =2.7

Specific Gravity = = = 2.71

Table A9: Specific Gravity Result for NC + 2%LM + 0%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.82	25.48	26.38
Wt of bottle + dry soil, W_2 (g).	34.82	35.46	36.38
Wt of bottle + soil + water, W_3 (g).	80.44	78.46	81.79
Wt of bottle + water, W_4 (g).	74.17	72.15	75.49

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90% NC + 10%BN

Trial 1 (G_{S1}) = = = 2.68

Trial 2 (G_{S2}) = = = 2.72

Trial 3 (G_{S3}) = = = 2.7

Specific Gravity = = = 2.7

Table A10: Specific Gravity Result for NC + 4%LM + 0%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.54	26.28	24.48
Wt of bottle + dry soil, W_2 (g).	35.54	36.26	34.47
Wt of bottle + soil + water, W_3 (g).	81.11	83.19	82.09
Wt of bottle + water, W_4 (g).	74.8	76.92	75.71

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 4%LM + %SCSA

Trial 1 (G_{S1}) = = = 2.71

Trial 2 (G_{S2}) = = = 2.69

Trial 3 (G_{S3}) = = = 2.76

Specific Gravity = = = 2.72

Table A11: Specific Gravity Result for NC + 6%LM + 0%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.82	26.12	25.42
Wt of bottle + dry soil, W_2 (g).	35.81	36.11	35.42
Wt of bottle + soil + water, W_3 (g).	83.48	84.14	86.17
Wt of bottle + water, W_4 (g).	77.15	77.77	79.78

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 6%LM + 0%SCSA

Trial 1 (G_{S1}) = = = 2.73

Trial 2 (G_{S2}) = = = 2.76

Trial 3 (G_{S3}) = = = 2.77

Specific Gravity = = = 2.75

Table A12 Specific Gravity Result for NC + 8%LM + 0%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.42	25.83	24.91
Wt of bottle + dry soil, W_2 (g).	36.42	35.82	34.91
Wt of bottle + soil + water, W_3 (g).	83.33	86.78	85.14
Wt of bottle + water, W_4 (g).	76.99	80.41	78.74

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 8%LM + 0%SCSA

Trial 1 (G_{S1}) = = = 2.73

Trial 2 (G_{S2}) = = = 2.76

Trial 3 (G_{S3}) = = = 2.78

Specific Gravity = = = 2.76

Table A13: Specific Gravity Result for NC + 10%LM + 0%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.48	26.36	25.72
Wt of bottle + dry soil, W_2 (g).	35.48	36.36	35.71
Wt of bottle + soil + water, W_3 (g).	82.44	78.88	79.24
Wt of bottle + water, W_4 (g).	75.88	72.18	72.81

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 10%LM + 0%SCSA

Trial 1 (G_{S1}) = = = 2.90

Trial 2 (G_{S2}) = = = 3.03

Trial 3 (G_{S3}) = = = 2.79

Specific Gravity = = = 2.91

Table A14: Specific Gravity Result for NC + 0%LM + 2%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.76	25.64	25.90
Wt of bottle + dry soil, W_2 (g).	34.74	35.63	35.90
Wt of bottle + soil + water, W_3 (g).	84.33	85.15	85.79
Wt of bottle + water, W_4 (g).	78.07	78.94	79.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 0%LM + 2%SCSA

Trial 1 (G_{S1}) = = = 2.68

Trial 2 (G_{S2}) = = = 2.64

Trial 3 (G_{S3}) = = =2.65

Specific Gravity = = = 2.66

Table A15: Specific Gravity Result for NC + 0%LM + 4%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	23.87	25.21	25.54
Wt of bottle + dry soil, W_2 (g).	33.86	35.21	35.54
Wt of bottle + soil + water, W_3 (g).	82.91	81.13	79.94
Wt of bottle + water, W_4 (g).	76.72	74.95	73.77

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 0%LM + 4%SCSA

Trial 1 (G_{S1}) = = = 2.63

Trial 2 (G_{S2}) = = = 2.62

Trial 3 (G_{S3}) = = =2.61

Specific Gravity = = = 2.62

Table A16: Specific Gravity Result for NC + 0%LM + 6%SCSA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.12	24.86	25.42
Wt of bottle + dry soil, W_2 (g).	35.11	35.85	35.42
Wt of bottle + soil + water, W_3 (g).	80.09	82.75	79.71
Wt of bottle + water, W_4 (g).	73.96	76.63	73.60

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 0%LM + 6%SCSA.

Trial 1 (G_{S1}) = = = 2.59

Trial 2 (G_{S2}) = = = 2.58

Trial 3 (G_{S3}) = = =2.57

Specific Gravity = = = 2.58

Table A17: Specific Gravity Result for NC + 0%LM + 8%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.50	25.32	25.12
Wt of bottle + dry soil, W_2 (g).	34.48	35.31	35.10
Wt of bottle + soil + water, W_3 (g).	84.43	86.39	85.03
Wt of bottle + water, W_4 (g).	78.35	80.32	78.93

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 0%LM + 8%SCSA.

Trial 1 (G_{S1}) = = = 2.56

Trial 2 (G_{S2}) = = = 2.55

Trial 3 (G_{S3}) = = =2.53

Specific Gravity = = = 2.55

Table A18: Specific Gravity Result for NC + 0%LM + 10%SCSA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.44	24.69	25.12
Wt of bottle + dry soil, W_2 (g).	34.43	34.68	35.12
Wt of bottle + soil + water, W_3 (g).	82.51	84.65	81.07
Wt of bottle + water, W_4 (g).	76.54	78.75	75.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for NC + 0%LM + 10%SCSA.

Trial 1 (G_{S1}) = = = 2.49

Trial 2 (G_{S2}) = = = 2.50

Trial 3 (G_{S3}) = = =2.46

Specific Gravity = = = 2.48

APPENDIX B

SIEVE ANALYSIS TEST

Table B1: Sieve Analysis Result for NC.

Sieve Sizes (mm)	Mass Ret (g)	% Mass Ret	Cum % Ret	Cum% Finer
1.18	14.13	2.826	2.826	97.17
0.85	9.97	1.994	4.82	95.18
0.6	13.24	2.648	7.468	92.53
0.425	12.3	2.46	9.928	90.07
0.3	19.31	3.862	13.79	86.21
0.15	32.48	6.496	20.286	79.71
0.075	25.04	5.008	25.294	74.71
Tray	0.16	0.032	25.326	74.67
Total	500			

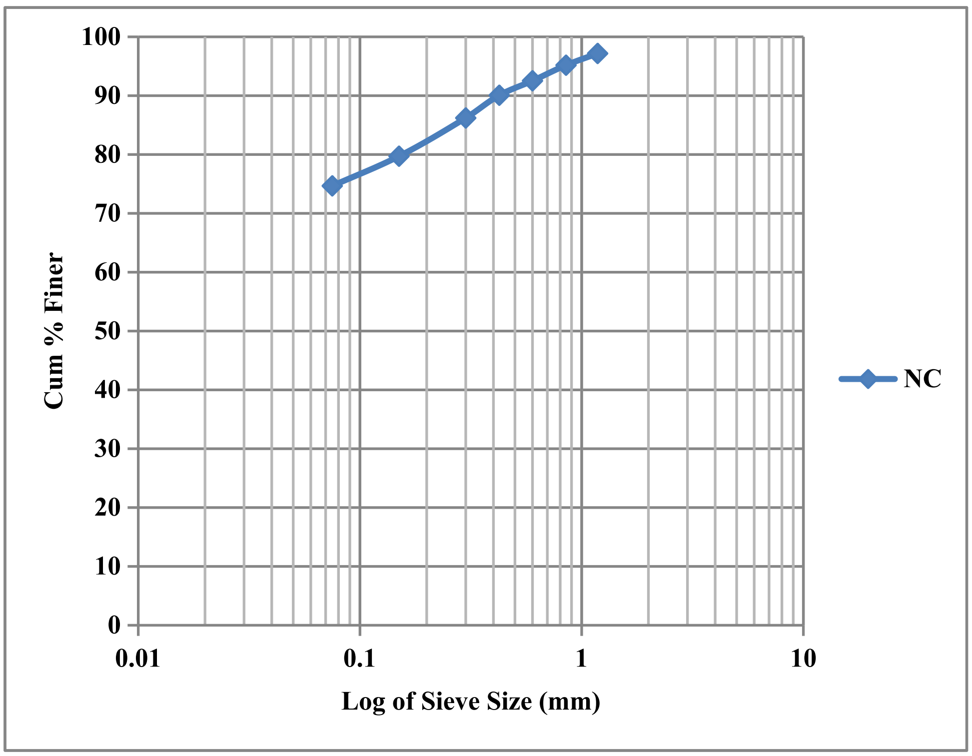


Figure B1: Particle Size Distribution Curve for NC

APPENDIX C

MOISTURE CONTENT TEST

Table C1: Moisture Content Result for NC + 0%LM + 0%SCSA

NC + 0%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.52	16.84	15.21
Wt of tin + wet soil (g)	31.88	25.64	36.81
Wt of wet soil (g)	17.36	8.8	21.6
Wt of tin + dry soil (g)	27.05	23.69	31.01
Wt of dry soil (g)	12.53	6.85	15.8
Wt of water (g)	4.83	1.95	5.8
Moisture Content (%)	38.55	28.47	36.71

Table C2: Moisture Content Result for NC + 2%LM + 2%SCSA

NC + 2%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.21	14.58	13.86
Wt of tin + wet soil (g)	24.46	30.28	35.79
Wt of wet soil (g)	9.25	15.7	21.93
Wt of tin + dry soil (g)	21.87	26.29	30.02
Wt of dry soil (g)	6.66	11.71	16.16
Wt of water (g)	2.59	3.99	5.77
Moisture Content (%)	38.89	34.07	35.71

Table C3: Moisture Content Result for NC + 4%LM + 4%SCSA

NC + 4%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.82	14.21	16.32
Wt of tin + wet soil (g)	30.9	28.37	33.91
Wt of wet soil (g)	16.08	14.16	17.59
Wt of tin + dry soil (g)	26.18	24.74	28.89
Wt of dry soil (g)	11.36	10.53	12.57
Wt of water (g)	4.72	3.63	5.02
Moisture Content (%)	41.55	34.47	39.94

Table C4: Moisture Content Result for NC + 6%LM + 6%SCSA

NC + 6%LM + 0%SCA	Test 1	Test 2	Test 3

Wt of empty tin (g)	**14.82**	**14.21**	**16.32**
Wt of tin + wet soil (g)	**25.15**	**20.38**	**22.47**
Wt of wet soil (g)	**10.33**	**6.17**	**6.15**
Wt of tin + dry soil (g)	**22.01**	**18.94**	**20.44**
Wt of dry soil (g)	**7.19**	**4.73**	**4.12**
Wt of water (g)	**3.14**	**1.44**	**2.03**
Moisture Content (%)	**43.67**	**30.44**	**49.27**

Table C5: Moisture Content Result for NC + 8%LM + 8%SCSA

NC + 8%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	**15.21**	**16.44**	**17.14**
Wt of tin + wet soil (g)	**28.45**	**30.51**	**32.88**
Wt of wet soil (g)	**13.24**	**14.07**	**15.74**
Wt of tin + dry soil (g)	**24.49**	**27.66**	**27.95**
Wt of dry soil (g)	**9.28**	**11.22**	**10.81**
Wt of water (g)	**3.96**	**2.85**	**4.93**
Moisture Content (%)	**42.67**	**25.40**	**45.61**

Table C6: Moisture Content Result for NC + 10%LM + 10%SCSA

NC + 10%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	**15.21**	**14.42**	**17.04**
Wt of tin + wet soil (g)	**31.23**	**35.84**	**36.28**
Wt of wet soil (g)	**16.02**	**21.42**	**19.24**
Wt of tin + dry soil (g)	**26.05**	**31.14**	**30.02**
Wt of dry soil (g)	**10.84**	**16.72**	**12.98**
Wt of water (g)	**5.18**	**4.7**	**6.26**
Moisture Content (%)	**47.79**	**28.11**	**48.23**

Table C7: Moisture Content Result for NC + 0%LM + 0%SCSA

NC + 0%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.52	16.84	15.21
Wt of tin + wet soil (g)	31.88	25.64	36.81
Wt of wet soil (g)	17.36	8.8	21.6
Wt of tin + dry soil (g)	27.05	23.69	31.01
Wt of dry soil (g)	12.53	6.85	15.8
Wt of water (g)	4.83	1.95	5.8
Moisture Content (%)	38.55	28.47	36.71

Table C8: Moisture Content Result for NC + 0%LM + 2%SCSA

NC + 0%LM + 2%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.14	16.24	15.21
Wt of tin + wet soil (g)	26.99	32.18	34.26
Wt of wet soil (g)	11.85	15.94	19.05
Wt of tin + dry soil (g)	23.74	28.25	29.07
Wt of dry soil (g)	8.6	12.01	13.86
Wt of water (g)	3.25	3.93	5.19
Moisture Content (%)	37.79	32.72	37.45

Table C9: Moisture Content Result for NC + 0%LM + 4%SCSA

NC + 0%LM + 4%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	13.28	17.05	16.28
Wt of tin + wet soil (g)	30.84	22.88	28.17
Wt of wet soil (g)	17.56	5.83	11.89
Wt of tin + dry soil (g)	26.58	21.46	24.98
Wt of dry soil (g)	13.3	4.41	8.7
Wt of water (g)	4.26	1.42	3.19
Moisture Content (%)	32.03	32.20	36.67

Table C10: Moisture Content Result for NC + 0%LM + 6%SCSA

NC + 0%LM + 6%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	13.28	15.21	16.28
Wt of tin + wet soil (g)	34.66	38.28	40.12
Wt of wet soil (g)	21.38	23.07	23.84
Wt of tin + dry soil (g)	29.49	33.02	34.09
Wt of dry soil (g)	16.21	17.81	17.81
Wt of water (g)	5.17	5.26	6.03
Moisture Content (%)	31.89	29.53	33.86

Table C11: Moisture Content Result for NC + 0%LM + 8%SCSA

NC + 0%LM + 8%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	13.28	14.47	16.08
Wt of tin + wet soil (g)	37.77	42.19	25.68
Wt of wet soil (g)	24.49	27.72	9.6
Wt of tin + dry soil (g)	32.62	36.49	23.31
Wt of dry soil (g)	19.34	22.02	7.23
Wt of water (g)	5.15	5.7	2.37
Moisture Content (%)	26.63	25.89	32.78

Table C12: Moisture Content Result for NC + 0%LM + 10%SCSA

NC + 0%LM + 10%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.16	15.28	17.14
Wt of tin + wet soil (g)	36.99	40.16	28.64
Wt of wet soil (g)	22.83	24.88	11.5
Wt of tin + dry soil (g)	32.62	36.29	26.51
Wt of dry soil (g)	18.46	21.01	9.37
Wt of water (g)	4.37	3.87	2.13
Moisture Content (%)	23.67	18.42	22.73

Table C13: Moisture Content Result for NC + 0%LM + 0%SCSA

NC + 0%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.52	16.84	15.21
Wt of tin + wet soil (g)	31.88	25.64	36.81
Wt of wet soil (g)	17.36	8.8	21.6
Wt of tin + dry soil (g)	27.05	23.69	31.01
Wt of dry soil (g)	12.53	6.85	15.8
Wt of water (g)	4.83	1.95	5.8
Moisture Content (%)	38.55	28.47	36.71

Table C14: Moisture Content Result for NC + 2%LM + 2%SCSA

NC + 2%LM + 2%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.18	16.84	17.12
Wt of tin + wet soil (g)	34.96	28.25	40.44
Wt of wet soil (g)	19.78	11.41	23.32
Wt of tin + dry soil (g)	30.06	26.08	34.51
Wt of dry soil (g)	14.88	9.24	17.39
Wt of water (g)	4.9	2.17	5.93
Moisture Content (%)	32.93	23.48	34.10

Table C15: Moisture Content Result for NC + 4%LM + 4%SCSA

NC + 4%LM + 4%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	16.24	13.79	17.02
Wt of tin + wet soil (g)	35.28	30.26	32.11
Wt of wet soil (g)	19.04	16.47	15.09
Wt of tin + dry soil (g)	31.08	27.66	28.41
Wt of dry soil (g)	14.84	13.87	11.39
Wt of water (g)	4.2	2.6	3.7
Moisture Content (%)	28.30	18.75	32.48

Table C16: Moisture Content Result for NC + 6%LM + 6%SCSA

NC + 6%LM + 6%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.21	14.28	17.02
Wt of tin + wet soil (g)	30.42	26.88	35.18
Wt of wet soil (g)	15.21	12.6	18.16
Wt of tin + dry soil (g)	26.79	24.55	30.04
Wt of dry soil (g)	11.58	10.27	13.02
Wt of water (g)	3.63	2.33	5.14
Moisture Content (%)	31.35	22.69	39.48

Table C17: Moisture Content Result for NC + 8%LM + 8%SCSA

NC + 8%LM + 8%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.21	14.28	16.42
Wt of tin + wet soil (g)	28.28	30.44	40.18
Wt of wet soil (g)	13.07	16.16	23.76
Wt of tin + dry soil (g)	24.76	26.72	33.07
Wt of dry soil (g)	9.55	12.44	16.65
Wt of water (g)	3.52	3.72	7.11
Moisture Content (%)	36.86	29.90	42.70

Table C18: Moisture Content Result for NC + 10%LM + 10%SCSA

NC + 10%LM + 10%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.28	16.16	17.05
Wt of tin + wet soil (g)	30.52	34.68	42.24
Wt of wet soil (g)	16.24	18.52	25.19
Wt of tin + dry soil (g)	26.08	30.01	33.97
Wt of dry soil (g)	11.8	13.85	16.92
Wt of water (g)	4.44	4.67	8.27
Moisture Content (%)	37.63	33.72	48.88

APPENDIX D

CBR TEST

Table D1 CBR Result for NC + 0%LM + 0%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.3	0.5	0.8	1.1	1.4	1.7	2	2.2	2.5	2.8	3.1	3.4	3.7	4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.1	0.4	0.7	0.9	1.1	1.4	1.7	2.0	2.2	2.5	2.8	3.1	3.4	3.7

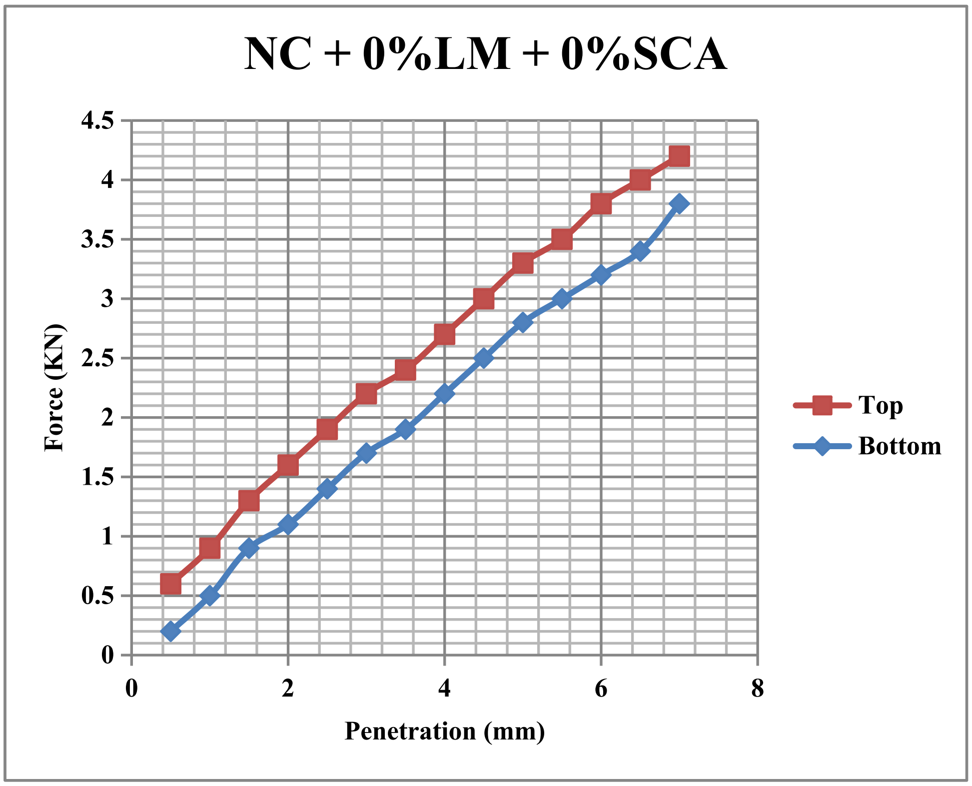


Figure D1: Graph of Force against Penetration for NC + 0%LM + 0%SCSA

Table D2 CBR Result for NC + 2%LM + 2%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.6	0.9	1.1	1.5	1.8	2.1	2.4	2.6	2.9	3.2	3.5	3.8	4.1	4.4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.4	0.6	0.9	1.1	1.3	1.7	1.9	2.2	2.5	2.8	3.1	3.4	3.6	3.9

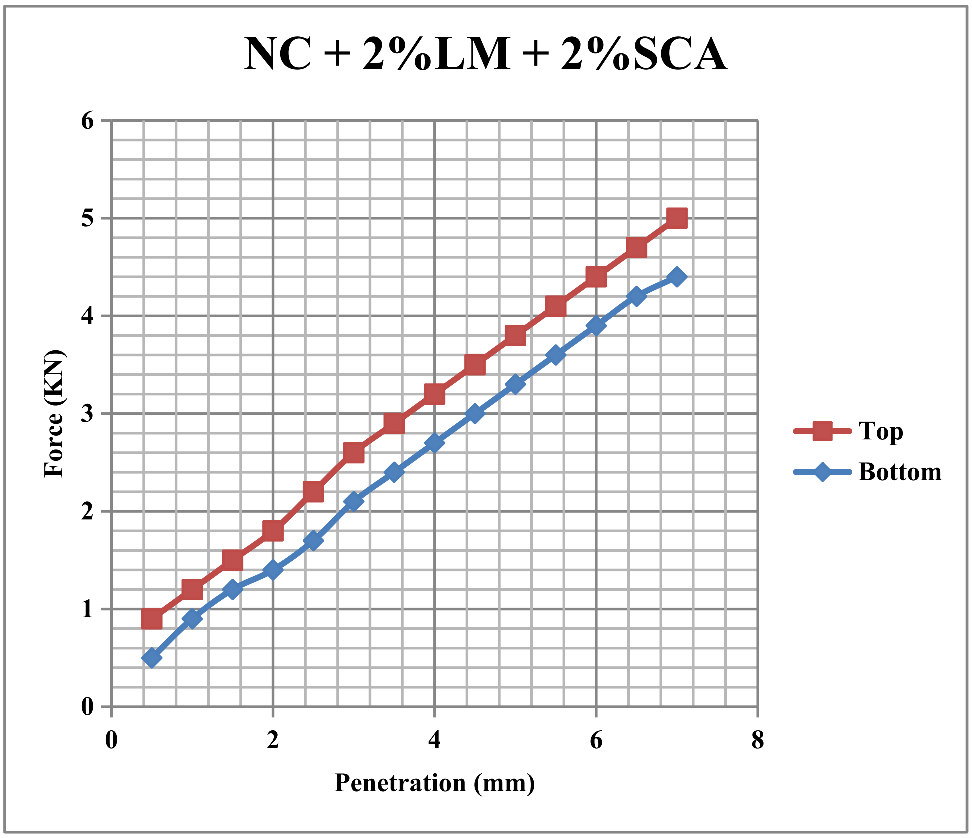


Figure D2: Graph of Force against Penetration for NC + 2%LM + 2%SCSA

Table D3 CBR Result for NC + 4%LM + 4%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.8	1.1	1.4	1.7	2	2.3	2.6	2.9	3.1	3.4	3.7	4	4.3	4.6
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.3	0.7	0.9	1.2	1.4	1.7	2	2.1	2.4	2.7	2.9	3.1	3.4	3.7

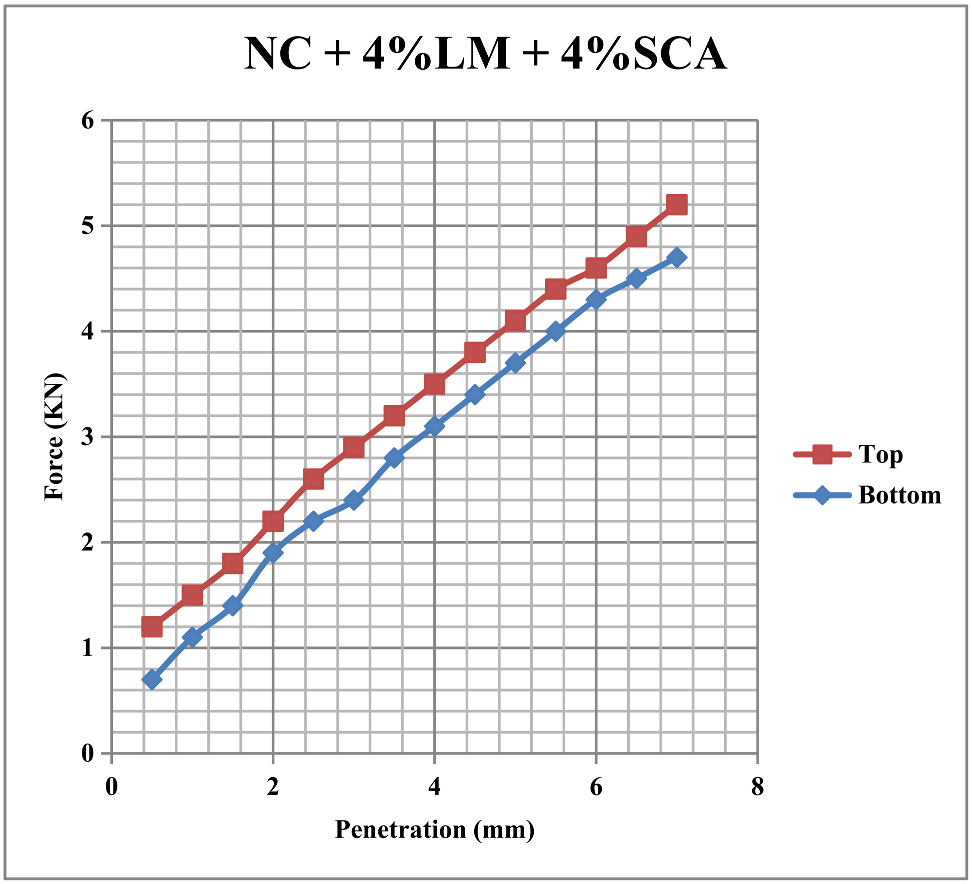


Figure D3: Graph of Force against Penetration for NC + 4%LM + 4%SCSA

Table D4 CBR Result for NC + 6%LM + 6%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.4	0.6	0.9	1.2	1.4	1.7	2	2.2	2.4	2.7	3	3.3	3.5	3.8
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.2	0.5	0.7	1	1.2	1.4	1.7	2	2.2	2.5	2.8	3.1	3.3	3.5

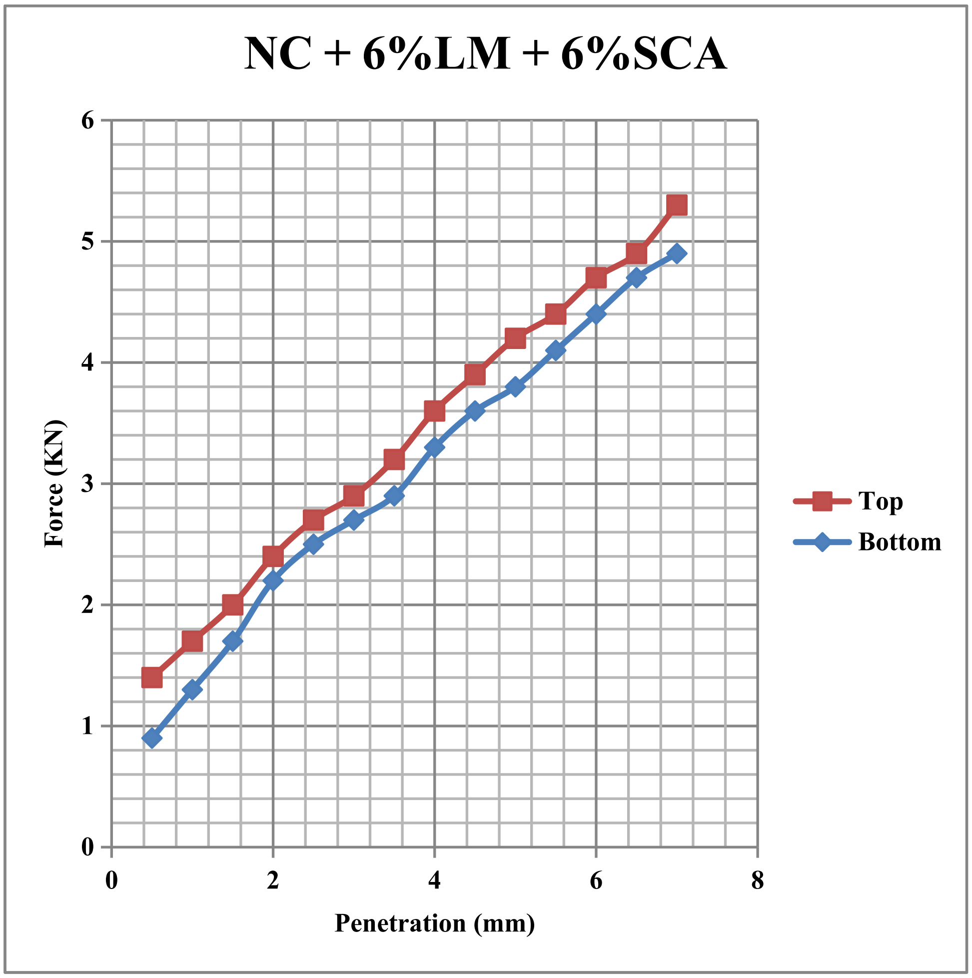


Figure D4: Graph of Force against Penetration for NC + 6%LM + 6%SCSA

Table D5 CBR Result for NC + 8%LM + 8%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.2	0.5	0.7	1.1	1.3	1.4	1.7	2	2.2	2.5	2.8	3.1	3.4	3.7
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.2	0.4	0.6	0.9	1.2	1.4	1.6	1.8	2.1	2.4	2.7	3	3.3	3.5

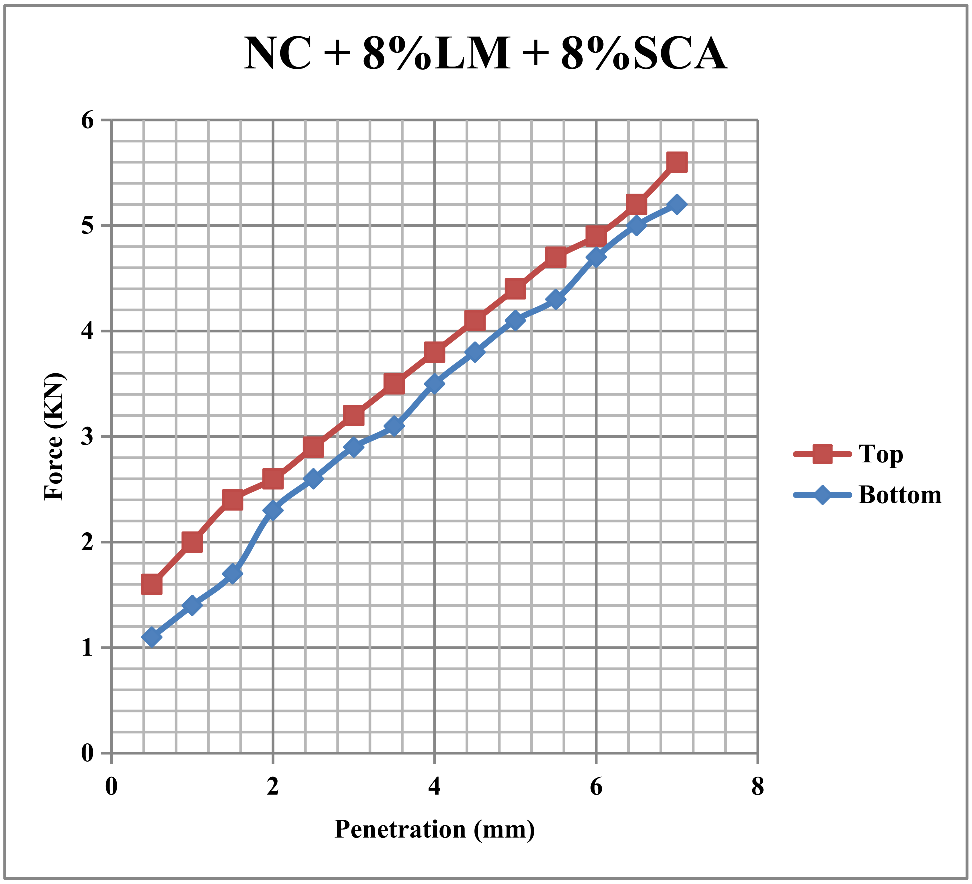


Figure D5: Graph of Force against Penetration for NC + 8%LM + 8%SCSA

Table D6 CBR Result for NC + 10%LM + 10%SCSA (Soaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.2	0.4	0.6	0.9	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.5	2.8	3
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.2	0.4	0.5	0.7	0.9	1.1	1.3	1.4	1.6	2	2.1	2.2	2.3	2.4

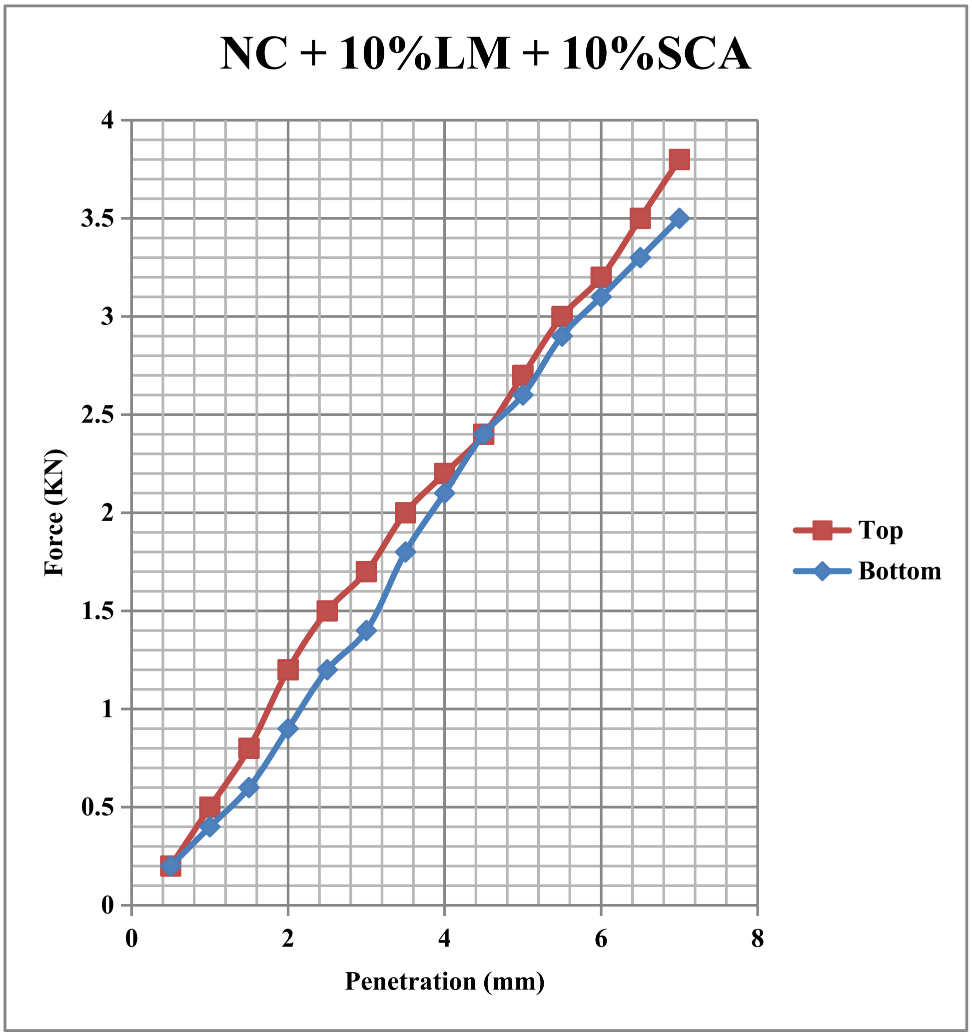


Figure D6: Graph of Force against Penetration for NC + 10%LM + 10%SCSA

CALIFORNIA BEARING RATIO RESULT FOR UNSOAKED SAMPLES

Table D7 CBR Result for NC + 0%LM + 0%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.5	2.7	2.9	3.1	3.4	3.6	4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.6	0.9	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.5	2.8	3.1	3.4	3.7

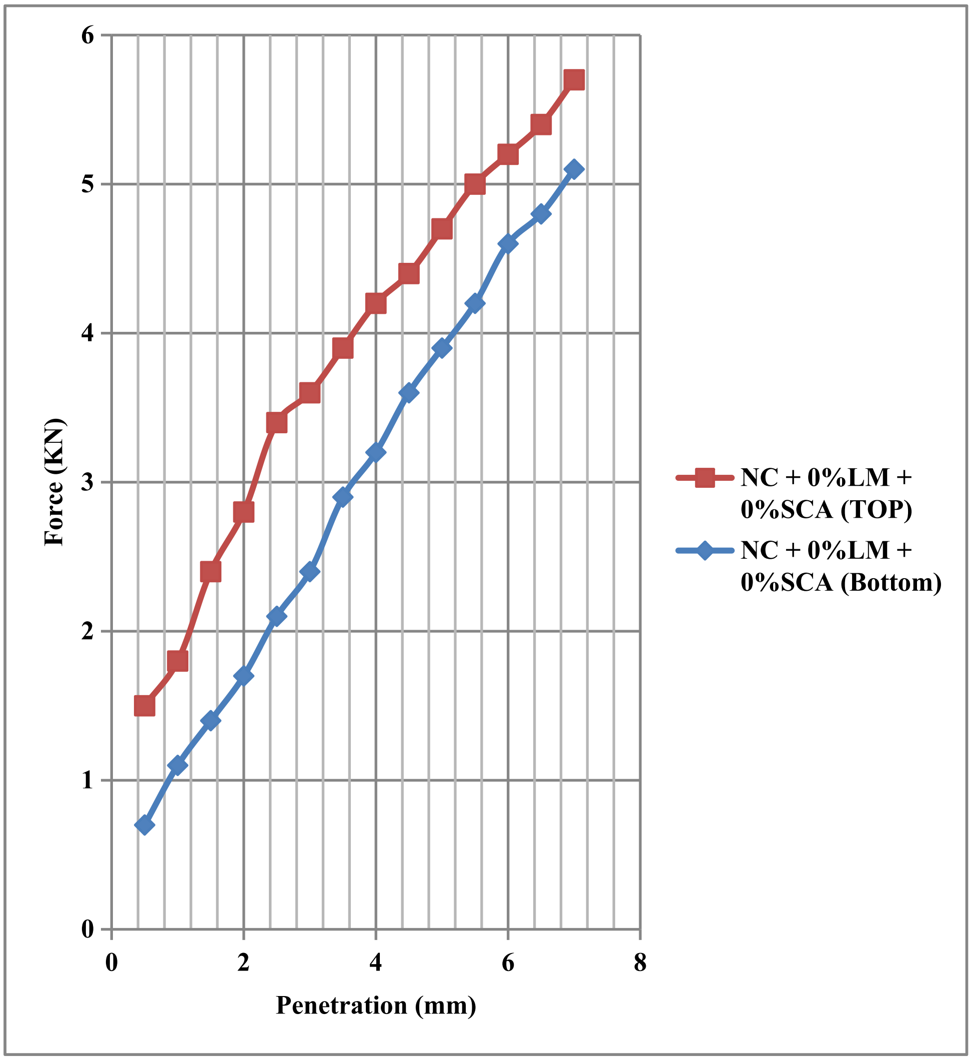


Figure D7: Graph of Force against Penetration for NC + 0%LM + 0%SCSA

Table D8 CBR Result for NC + 2%LM + 2%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.3	1.6	1.9	2.2	2.5	2.7	2.9	3.1	3.4	3.7	3.9	4.1	4.3	4.5
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.7	1	1.3	1.5	1.7	2	2.3	2.5	2.8	3	3.2	3.4	3.6	3.8

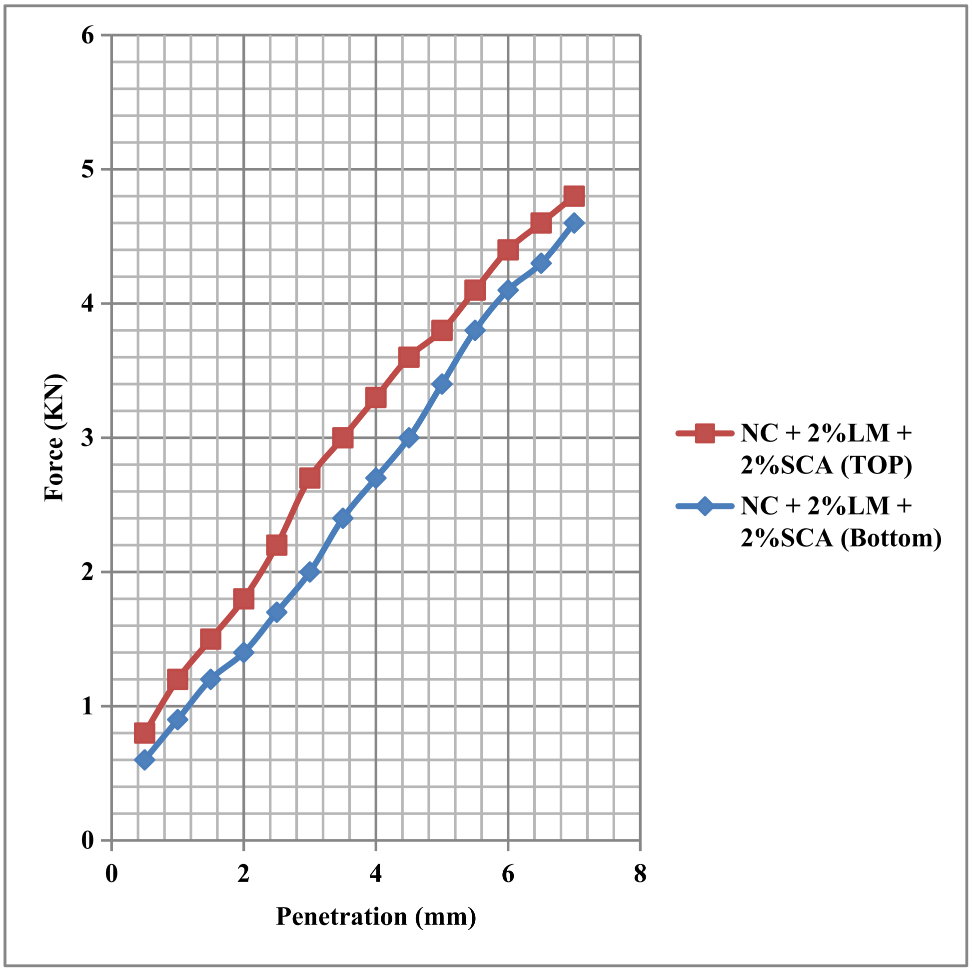


Figure D8: Graph of Force against Penetration for NC + 2%LM + 2%SCSA

Table D9 CBR Result for NC + 4%LM + 4%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.5	1.7	2	2.3	2.6	2.8	3.1	3.4	3.7	4	4.2	4.4	4.6	4.9
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.9	1.1	1.3	1.5	1.8	2.1	2.3	2.5	2.8	3.1	3.3	3.4	3.7	4

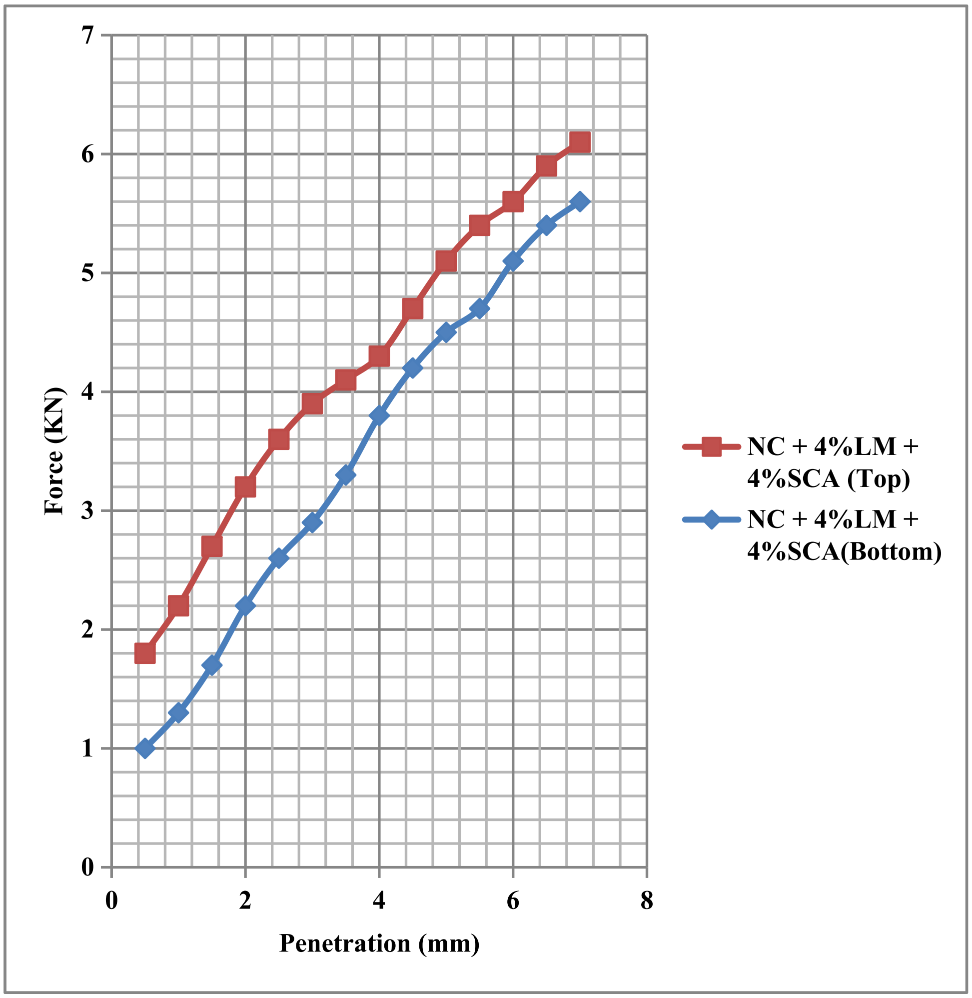


Figure D9: Graph of Force against Penetration for NC + 4%LM + 4%SCSA

Table D10 CBR Result for NC + 6%LM + 6%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.7	2	2.3	2.5	2.7	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.9
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.9	1.1	1.3	1.5	1.8	2.1	2.3	2.5	2.8	3.1	3.3	3.4	3.7	4

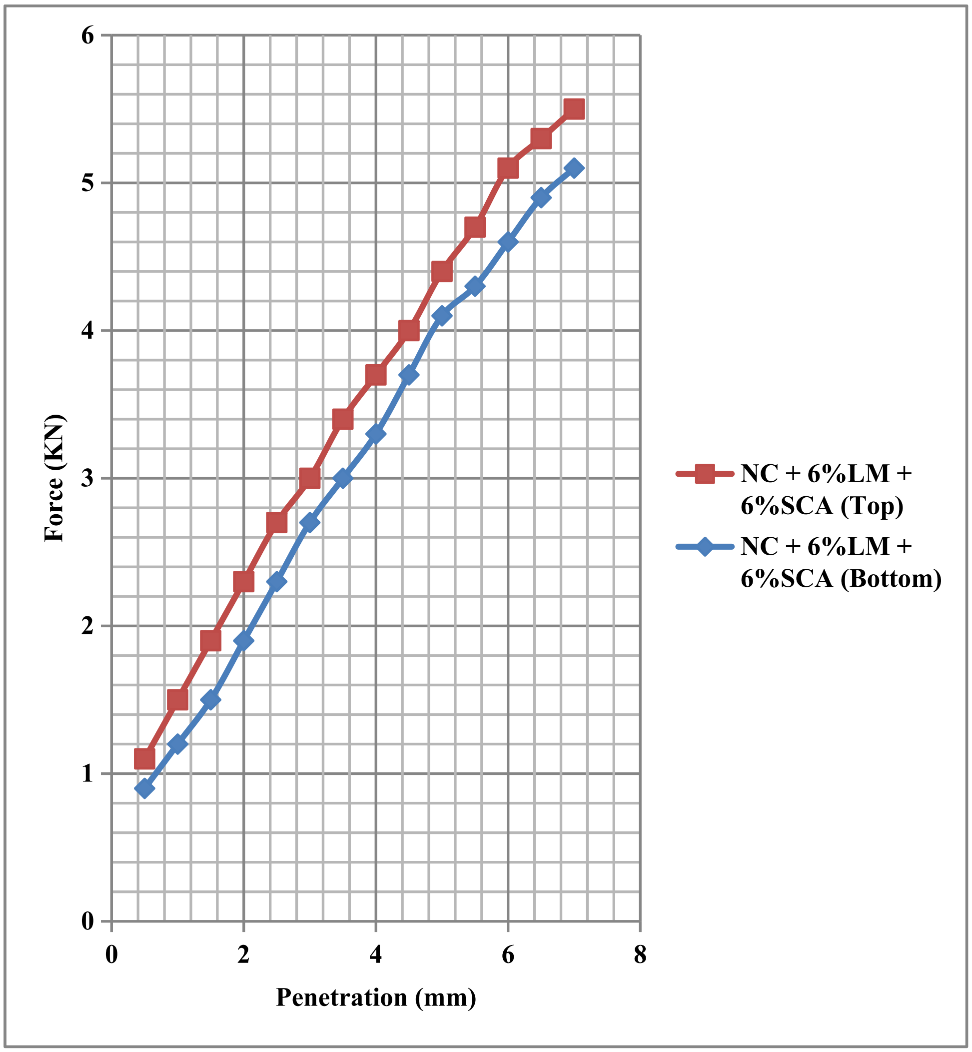


Figure D10: Graph of Force against Penetration for NC + 6%LM + 6%SCSA

Table D11 CBR Result for NC + 8%LM + 8%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.9	2.2	2.5	2.8	3.1	3.3	3.5	3.8	4.1	4.4	4.5	4.8	5.1	5.4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.2	1.4	1.7	2	2.3	2.5	2.8	3.1	3.4	3.7	4	4.3	4.6	4.8

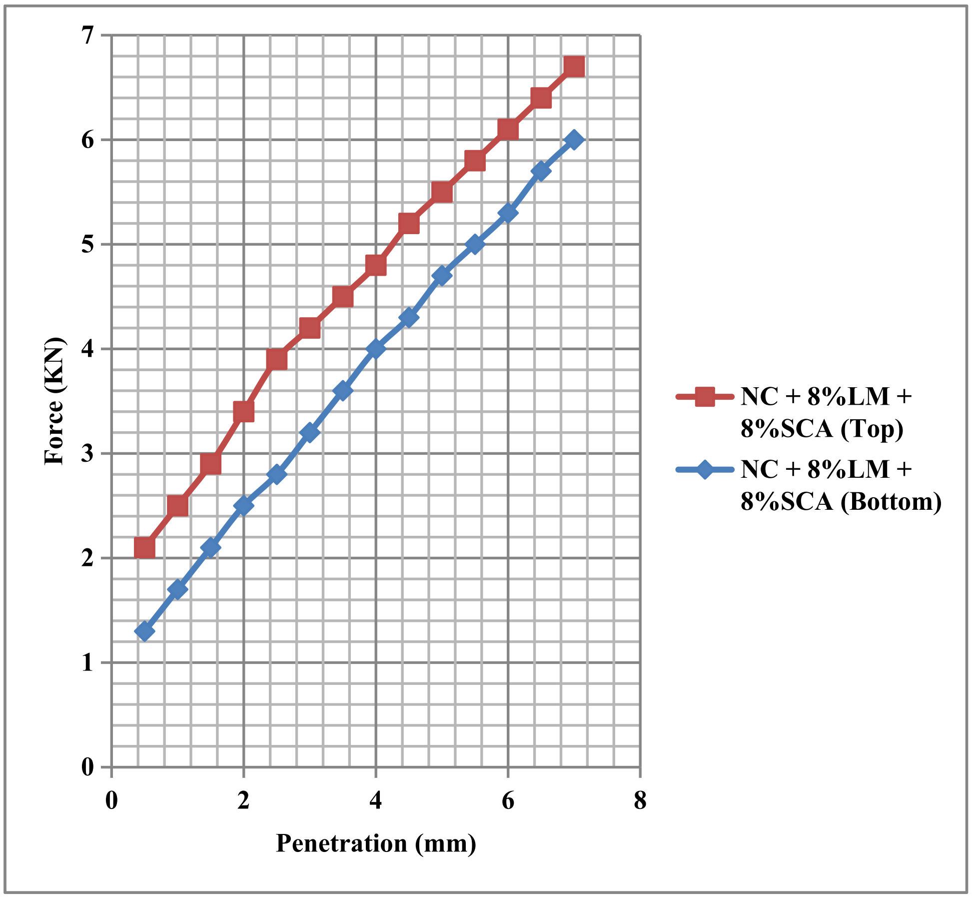


Figure D11: Graph of Force against Penetration for NC + 8%LM + 8%SCSA

Table D12 CBR Result for NC + 10%LM + 10%SCSA (Unsoaked)

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.3	1.5	1.7	1.9	2.1	2.3	2.6	2.8	3.0	3.3	3.6	3.8	4.1	4.4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.9	1.1	1.3	1.5	1.8	2.1	2.3	2.5	2.8	3.1	3.3	3.4	3.7	4

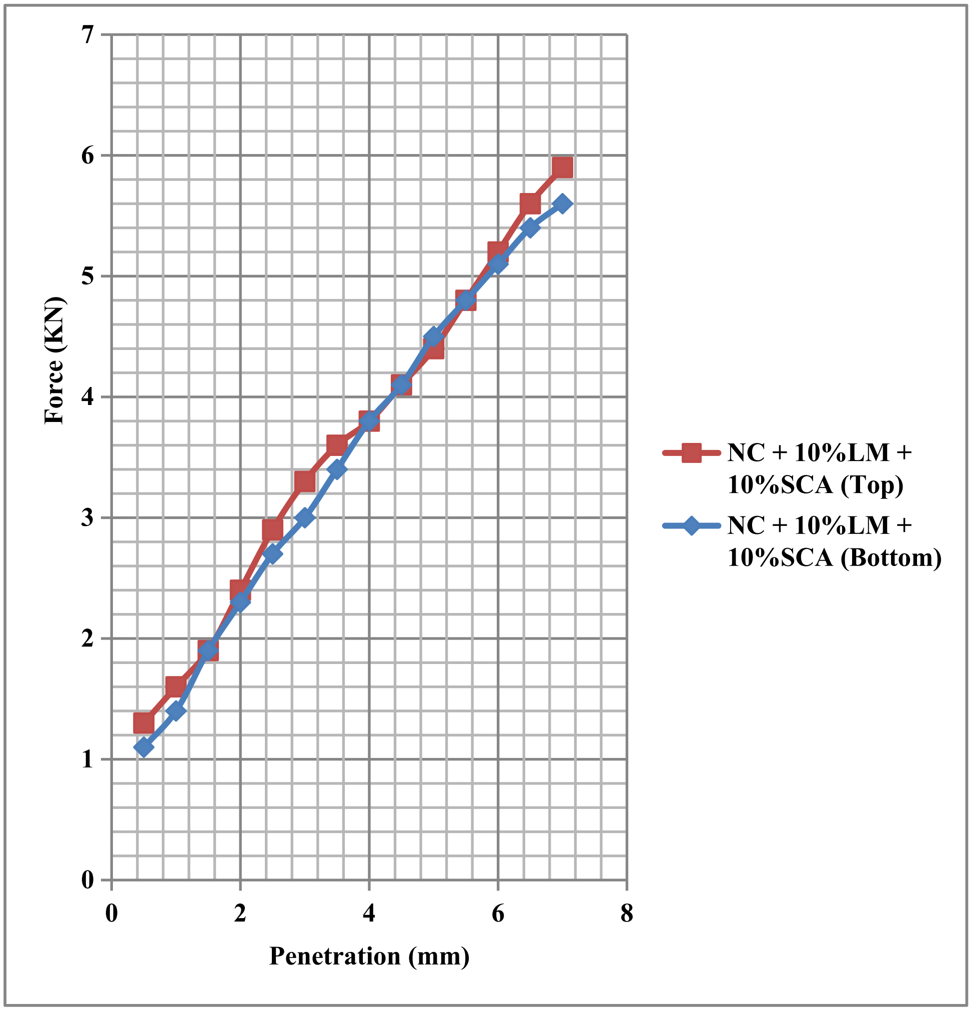


Figure D12: Graph of Force against Penetration for NC + 10%LM + 10%SCSA

APPENDIX E

LIQUID LIMIT TEST

Table E1: Liquid Limit Result for NC + 0%LM + 0%SCSA

NC + 0%LM + 0%SCA	32	26	21	18	13
Wt of empty tin (g)	14.21	14.69	16.34	14.52	17.02
Wt of tin + wet soil (g)	39.42	44.28	46.82	36.92	45.82
Wt of wet soil (g)	25.21	29.59	30.48	22.4	28.8
Wt of tin +dry soil (g)	31.79	34.9	36.5	27.66	33.21
Wt of dry soil (g)	17.58	20.21	20.16	13.14	16.19
Wt of water (g)	7.63	9.38	10.32	9.26	12.61
Moisture Content (%)	43.40	46.41	51.19	70.47	77.89

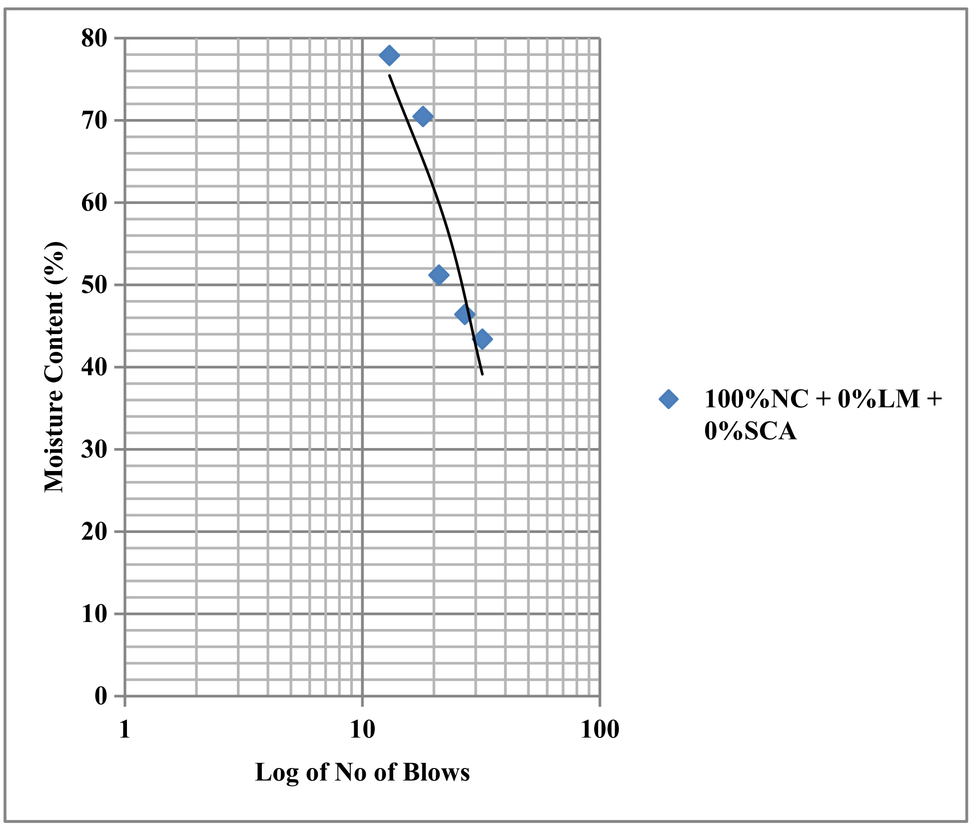


Figure E1: Liquid Limit Graph for 100%NC + 0%LM + 0%SCSA

Table E2: Liquid Limit Result for NC + 2%LM + 2%SCSA

BLOWS	33	27	23	18	14
Wt of empty tin (g)	18.24	15.32	14.12	15.38	17.12
Wt of tin + wet soil (g)	52.86	46.92	50.14	38.66	51.28
Wt of wet soil (g)	34.62	31.6	36.02	23.28	34.16
Wt of tin +dry soil (g)	42.91	36.98	38.17	29.65	36.98
Wt of dry soil (g)	24.67	21.66	24.05	14.27	19.86
Wt of water (g)	9.95	9.94	11.97	9.01	14.3
Moisture Content (%)	40.33	45.89	49.77	63.14	72.00

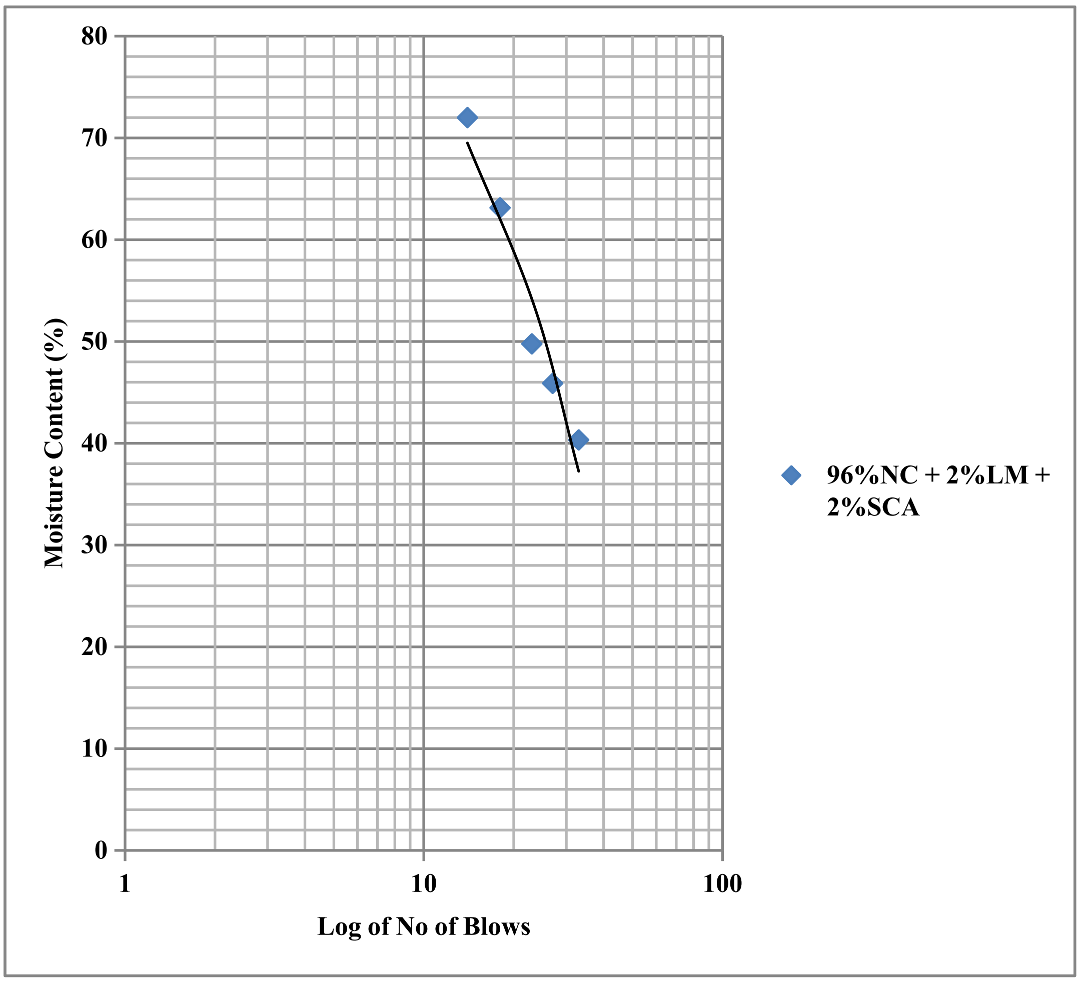


Figure E2: Liquid Limit Graph for 100%NC + 2%LM + 2%SCSA

Table E3: Liquid Limit Result for NC + 4%LM + 4%SCSA

BLOWS	32	27	23	17	14
Wt of empty tin (g)	17.24	14.26	15.13	16.48	14.38
Wt of tin + wet soil (g)	48.76	42.85	60.12	52.86	55.92
Wt of wet soil (g)	31.52	28.59	44.99	36.38	41.54
Wt of tin +dry soil (g)	40.28	34.04	45.63	39.55	40.09
Wt of dry soil (g)	23.04	19.78	30.5	23.07	25.71
Wt of water (g)	8.48	8.81	14.49	13.31	15.83
Moisture Content (%)	36.81	44.54	47.51	57.69	61.57

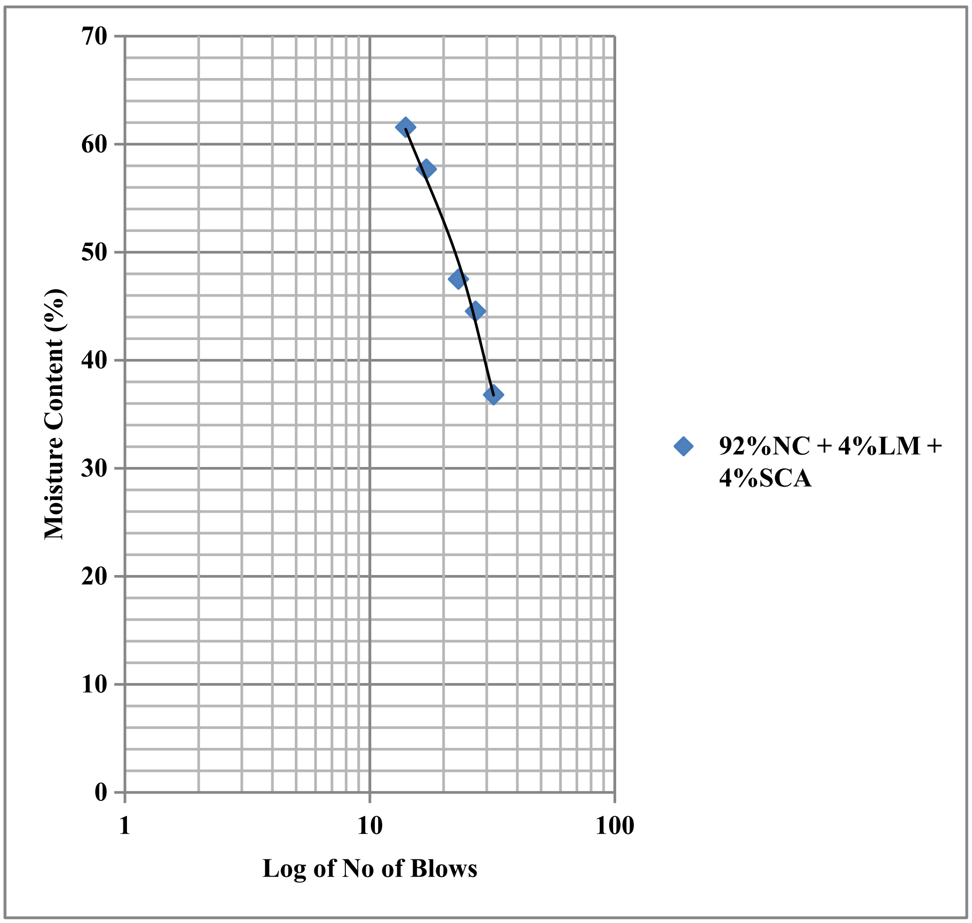


Figure E3: Liquid Limit Graph for 92%NC + 4%LM + 4%SCSA

Table E4: Liquid Limit Result for NC + 6%LM + 6%SCSA

BLOWS	33	28	23	18	14
Wt of empty tin (g)	16.24	15.15	14.68	15.34	15.28
Wt of tin + wet soil (g)	51.24	45.88	50.82	46.78	52.08
Wt of wet soil (g)	35	30.73	36.14	31.44	36.8
Wt of tin +dry soil (g)	42.64	36.72	39.71	36.15	38.52
Wt of dry soil (g)	26.4	21.57	25.03	20.81	23.24
Wt of water (g)	8.6	9.16	11.11	10.63	13.56
Moisture Content (%)	32.58	42.47	44.39	51.08	58.35

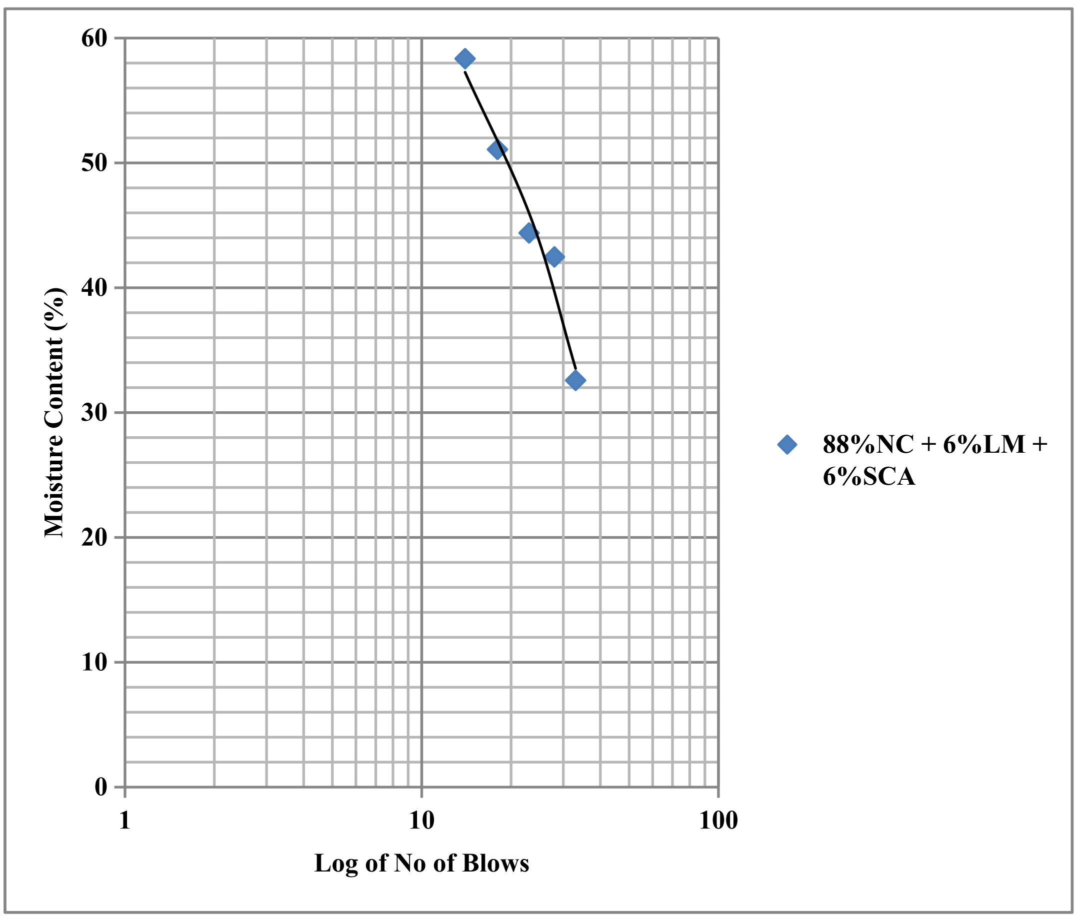


Figure E4: Liquid Limit Graph for 88%NC + 6%LM + 6%SCSA

Table E5: Liquid Limit Result for NC + 8%LM + 8%SCSA

BLOWS	33	28	23	18	14
Wt of empty tin (g)	16.24	16.2	15.24	15.88	15.32
Wt of tin + wet soil (g)	50.43	45.88	50.82	46.78	52.08
Wt of wet soil (g)	34.19	29.68	35.58	30.9	36.76
Wt of tin +dry soil (g)	42.64	38.33	40.37	36.99	39.53
Wt of dry soil (g)	26.4	22.13	25.13	21.11	24.21
Wt of water (g)	7.79	7.55	10.45	9.79	12.55
Moisture Content (%)	29.51	34.12	41.58	46.38	51.84

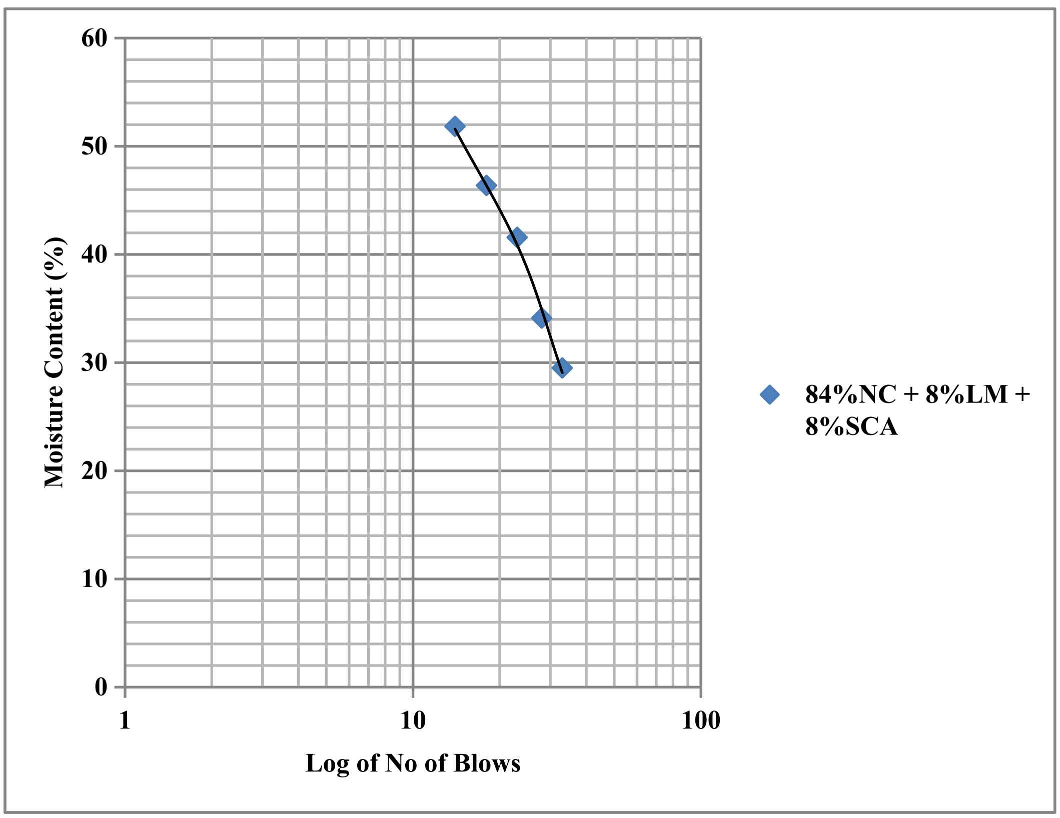


Figure E5: Liquid Limit Graph for 84%NC + 8%LM + 8%SCSA

Table E6: Liquid Limit Result for NC + 10%LM + 10%SCSA

BLOWS	34	28	23	19	14
Wt of empty tin (g)	15.23	14.47	16.12	15.69	17.72
Wt of tin + wet soil (g)	51.42	48.88	52.49	50.65	60.08
Wt of wet soil (g)	36.19	34.41	36.37	34.96	42.36
Wt of tin +dry soil (g)	44.56	41.08	43.28	40.03	46.27
Wt of dry soil (g)	29.33	26.61	27.16	24.34	28.55
Wt of water (g)	6.86	7.8	9.21	10.62	13.81
Moisture Content (%)	23.39	29.31	33.91	43.63	48.37

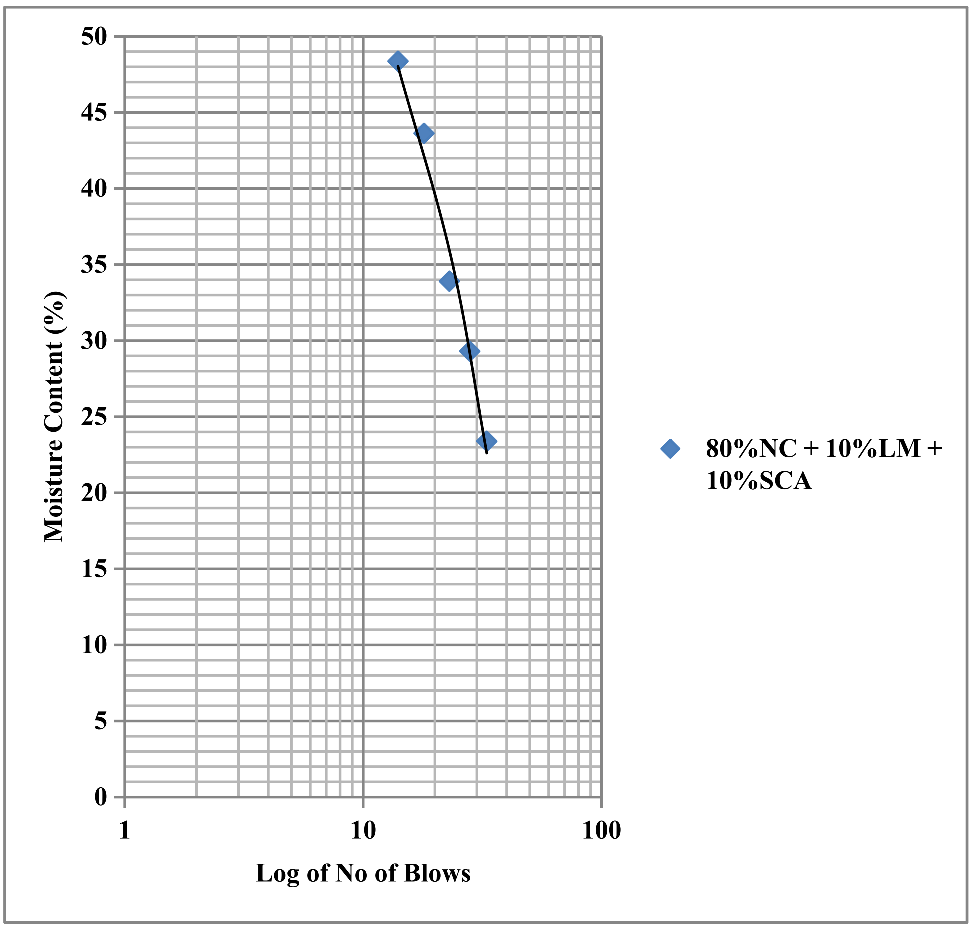


Figure E6: Liquid Limit Graph for 80%NC + 10%LM + 10%SCSA

APPENDIX F

PLASTIC LIMIT TEST

Table F1: Plastic Limit Result for 100%NC + 0%LM + 0%SCSA

100%NC + 0%LM + 0%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.24	16.49	17.14
Wt of tin + wet soil (g)	34.56	47.84	36.37
Wt of wet soil (g)	19.32	31.35	19.23
Wt of tin + dry soil (g)	29.64	40.44	31.29
Wt of dry soil (g)	14.4	23.95	14.15
Wt of water (g)	4.92	7.4	5.08
Plastic Limit (%)	34.17	30.90	35.90

Table F2: Plastic Limit Result for 96%NC + 2%LM + 2%SCSA

96%NC + 2%LM + 2%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	16.42	17.34	14.22
Wt of tin + wet soil (g)	28.68	34.62	32.18
Wt of wet soil (g)	12.26	17.28	17.96
Wt of tin + dry soil (g)	25.64	31.05	27.94
Wt of dry soil (g)	9.22	13.71	13.72
Wt of water (g)	3.04	3.57	4.24
Plastic Limit (%)	32.97	26.04	30.90

Table F3: Plastic Limit Result for 92%NC + 4%LM + 4%SCSA

92%NC + 4%LM + 4%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.52	16.84	15.21
Wt of tin + wet soil (g)	31.88	25.64	36.81
Wt of wet soil (g)	17.36	8.8	21.6
Wt of tin + dry soil (g)	27.93	23.89	32.11

Wt of dry soil (g)	**13.41**	**7.05**	**16.9**
Wt of water (g)	**3.95**	**1.75**	**4.7**
Plastic Limit (%)	**29.46**	**24.82**	**27.81**

Table F4: Plastic Limit Result for 88%NC + 6%LM + 6%SCSA

88%NC + 6%LM + 6%SCA	**Test 1**	**Test 2**	**Test 3**
Wt of empty tin (g)	**14.52**	**16.24**	**17.18**
Wt of tin + wet soil (g)	**34.68**	**30.06**	**32.48**
Wt of wet soil (g)	**20.16**	**13.82**	**15.3**
Wt of tin + dry soil (g)	**30.49**	**27.37**	**29.4**
Wt of dry soil (g)	**15.97**	**11.13**	**12.22**
Wt of water (g)	**4.19**	**2.69**	**3.08**
Plastic Limit (%)	**26.24**	**24.17**	**25.20**

Table F5: Plastic Limit Result for 84%NC + 8%LM + 8%SCSA

84%NC + 8%LM + 8%SCA	**Test 1**	**Test 2**	**Test 3**
Wt of empty tin (g)	**14.82**	**15.38**	**17.18**
Wt of tin + wet soil (g)	**38.92**	**26.88**	**34.26**
Wt of wet soil (g)	**24.1**	**11.5**	**17.08**
Wt of tin + dry soil (g)	**34.87**	**24.78**	**30.91**
Wt of dry soil (g)	**20.05**	**9.4**	**13.73**
Wt of water (g)	**4.05**	**2.1**	**3.35**
Plastic Limit (%)	**20.20**	**22.34**	**24.40**

Table F6: Plastic Limit Result for 80%NC + 10%LM + 10%SCSA

80%NC + 10%LM + 10%SCA	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.21	15.38	16.05
Wt of tin + wet soil (g)	22.94	30.15	33.88
Wt of wet soil (g)	7.73	14.77	17.83
Wt of tin + dry soil (g)	21.75	27.52	30.91
Wt of dry soil (g)	6.54	12.14	14.86
Wt of water (g)	1.19	2.63	2.97
Plastic Limit (%)	18.20	21.66	19.99

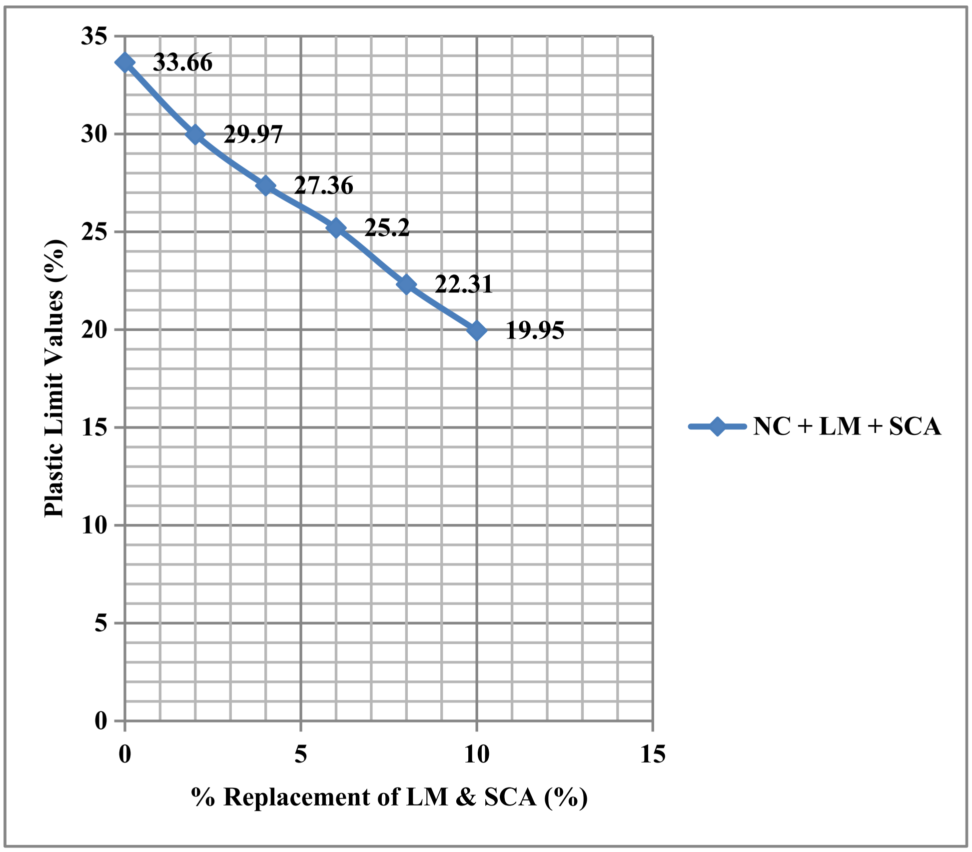


Figure F1: Plastic Limit Graph for NC + %LM + %SCSA

APPENDIX G

COMPACTION TEST

Table G1: Dry Density Determination for NC + 0%LM + 0%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.5	1.5	14.72	7.82	14.79
8	0.001	4	5.9	1.9	18.64	10.46	18.74
12	0.001	4	6	2	19.62	14.21	19.76
16	0.001	4	6.05	2.05	20.11	17.78	20.29
20	0.001	4	6	2	19.62	21.58	19.84

Table G1a: Moisture Content Determination for NC + 0%LM + 0%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.65	34.58	17.93	33.28	16.63	1.3	7.82
8	15.88	47.69	31.81	44.95	29.07	2.74	9.43
12	15.3	51	35.7	46.74	31.44	4.26	13.55
16	14.18	70.95	56.77	62.5	48.32	8.45	17.49
20	15.08	65.43	50.35	56.86	41.78	8.57	20.51

Table G1b: Moisture Content Determination for NC + 0%LM + 0%SCSA Bottom (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.85	29.22	14.37	28.57	13.72	0.65	4.74
8	17.82	58.27	40.45	54.1	36.28	4.17	11.49
12	17.82	46.41	28.59	42.71	24.89	3.7	14.87
16	15.52	64.84	49.32	57.29	41.77	7.55	18.08
20	16.64	67.76	51.12	58.32	41.68	9.44	22.65

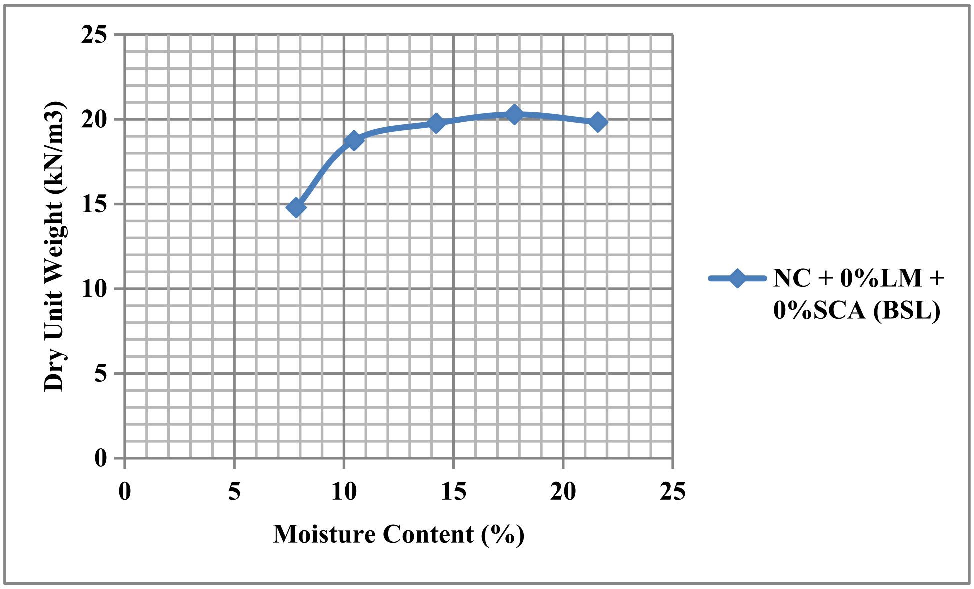


Figure G1: Compaction Curve for NC + 0%LM + 0%SCSA

Table G2: Dry Density Determination for NC + 2%LM + 2%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	4.62	18.69
8	0.001	4	6.1	2.1	20.60	9.44	20.70
12	0.001	4	6.2	2.2	21.58	13.15	21.71
16	0.001	4	6.1	2.1	20.60	16.49	20.77
20	0.001	4	6	2	19.62	20.28	19.82

Table G2a: Moisture Content Determination for NC + 2%LM + 2%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.51	45.67	30.16	44.37	28.86	1.3	4.50
8	14.64	38.42	23.78	36.29	21.65	2.13	9.84
12	14.9	41.76	26.86	38.43	23.53	3.33	14.15

16	**15.24**	**49.15**	**33.91**	**44.19**	**28.95**	**4.96**	**17.13**
20	**16.62**	**57.69**	**41.07**	**51.11**	**34.49**	**6.58**	**19.08**

Table G2b: Moisture Content Determination for NC + 2%LM + 2%SCSA Bottom (BSL)

Percentages of Water	**Wt of tin**	**Wt of tin + wet**	**Wt of wet Soil (g)**	**Wt of tin + dry Soil**	**Wt of dry**	**Wt of Water (g)**	**Moisture**
(%)	**(g)**	**Soil (g)**		**(g)**	**Soil (g)**		**Content (g)**
4	**15.31**	**38.93**	**23.62**	**37.86**	**22.55**	**1.07**	**4.75**
8	**16.08**	**45.73**	**29.65**	**43.27**	**27.19**	**2.46**	**9.05**
12	**15.76**	**49.29**	**33.53**	**45.66**	**29.9**	**3.63**	**12.14**
16	**14.57**	**54.14**	**39.57**	**48.73**	**34.16**	**5.41**	**15.84**
20	**14.92**	**60.15**	**45.23**	**52.15**	**37.23**	**8**	**21.49**

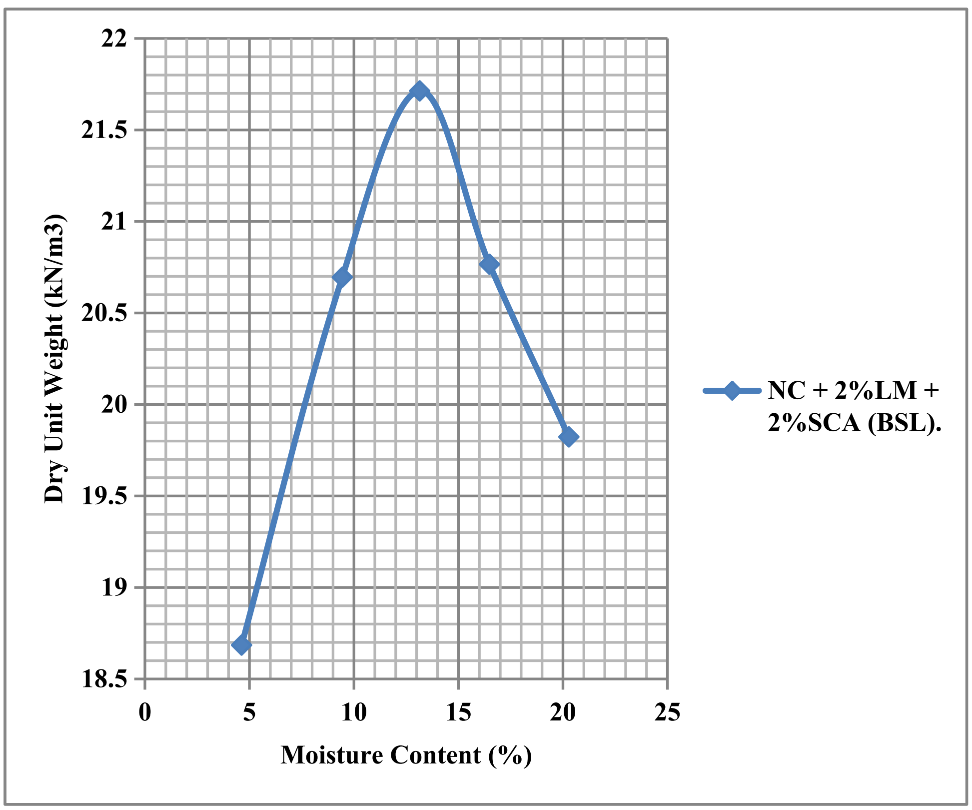


Figure G2: Compaction Curve for NC + 2%LM + 2%SCSA

Table G3: Dry Density Determination for NC + 4%LM + 4%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	5.72	18.70
8	0.001	4	6.1	2.1	20.60	9.74	20.70
12	0.001	4	6.2	2.2	21.58	14.04	21.72
16	0.001	4	6.1	2.1	20.60	19.18	20.79
20	0.001	4	6.05	2.05	20.11	21.56	20.33

Table G3a: Moisture Content Determination for NC + 4%LM + 4%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.79	46.87	31.08	44.91	29.12	1.96	6.73
8	14.38	53.27	38.89	49.96	35.58	3.31	9.30
12	15.84	63.09	47.25	57.67	41.83	5.42	12.96
16	14.8	48.99	34.19	43.18	28.38	5.81	20.47
20	16.77	60.42	43.65	52.11	35.34	8.31	23.51

Table G3b: Moisture Content Determination for NC + 4%LM + 4%SCSA Bottom (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.82	65.34	50.52	63.07	48.25	2.27	4.70
8	17.08	39.16	22.08	37.12	20.04	2.04	10.18
12	16.99	56.94	39.95	51.69	34.7	5.25	15.13
16	15.37	50.5	35.13	45.17	29.8	5.33	17.89
20	14.04	73.27	59.23	63.56	49.52	9.71	19.61

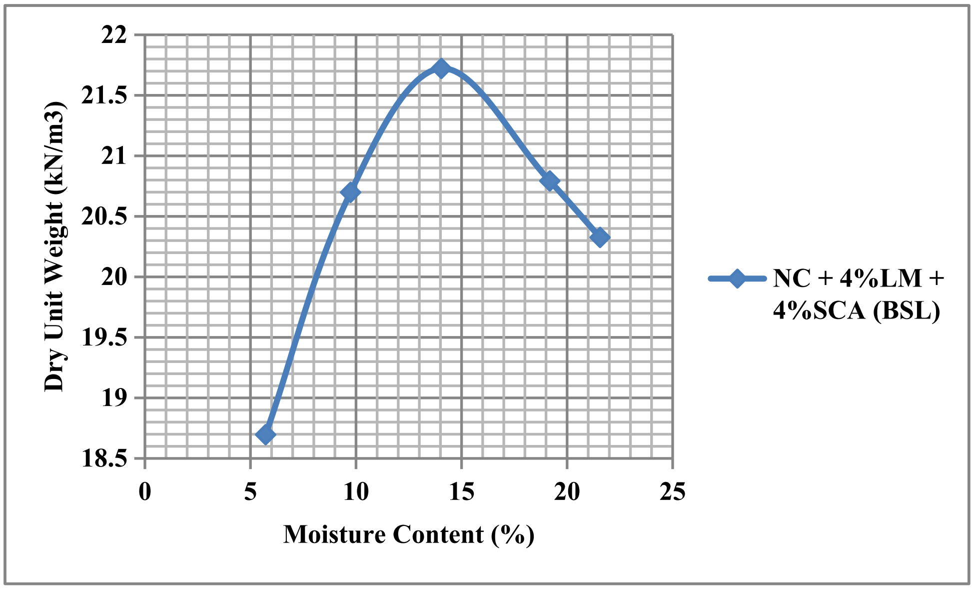


Figure G3: Compaction Curve for NC + 4%LM + 4%SCSA

Table G4: Dry Density Determination for NC + 6%LM + 6%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	6	2	19.62	5.42	19.67
8	0.001	4	6.1	2.1	20.60	10.20	20.70
12	0.001	4	6.2	2.2	21.58	13.98	21.72
16	0.001	4	6.15	2.15	21.09	17.58	21.27
20	0.001	4	6.1	2.1	20.60	23.06	20.83

Table G4a: Moisture Content Determination for NC + 6%LM + 6%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.04	42.19	26.15	40.68	24.64	1.51	6.13
8	15.36	51.69	36.33	48.58	33.22	3.11	9.36
12	15.73	49.05	33.32	44.76	29.03	4.29	14.78

16	**14.21**	**67.23**	**53.02**	**59.03**	**44.82**	**8.2**	**18.30**
20	**15.12**	**37.49**	**22.37**	**33.26**	**18.14**	**4.23**	**23.32**

Table G4b: Moisture Content Determination for NC + 6%LM + 6%SCSA Bottom (BSL)

Percentages of Water	**Wt of tin**	**Wt of tin + wet**	**Wt of wet Soil (g)**	**Wt of tin + dry Soil**	**Wt of dry**	**Wt of Water (g)**	**Moisture**
(%)	**(g)**	**Soil (g)**		**(g)**	**Soil (g)**		**Content (g)**
4	**15.41**	**61.34**	**45.93**	**59.27**	**43.86**	**2.07**	**4.72**
8	**16.72**	**47.91**	**31.19**	**44.81**	**28.09**	**3.1**	**11.04**
12	**16.28**	**58.61**	**42.33**	**53.68**	**37.4**	**4.93**	**13.18**
16	**15.75**	**52.76**	**37.01**	**47.42**	**31.67**	**5.34**	**16.86**
20	**14.92**	**68.33**	**53.41**	**58.41**	**43.49**	**9.92**	**22.81**

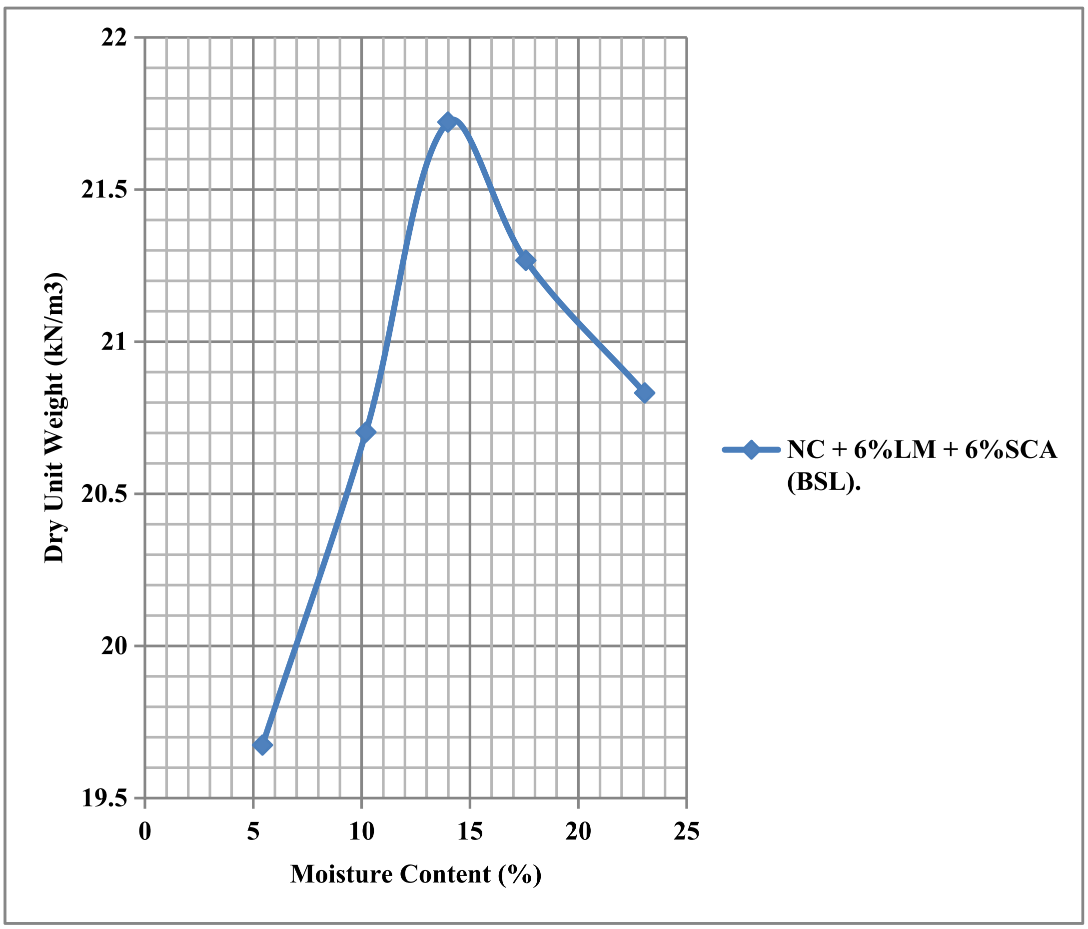


Figure G4: Compaction Curve for NC + 6%LM + 6%SCSA

Table G5: Dry Density Determination for NC + 8%LM + 8%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	5.52	18.69
8	0.001	4	6	2	19.62	9.47	19.71
12	0.001	4	6.1	2.1	20.60	12.85	20.73
16	0.001	4	6.1	2.1	20.60	15.05	20.75
20	0.001	4	6.05	2.05	20.11	21.99	20.33

Table G5a: Moisture Content Determination for NC + 8%LM + 8%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.47	56.27	41.8	54.03	39.56	2.24	5.66
8	14.76	53.16	38.4	49.81	35.05	3.35	9.56
12	15.16	48.99	33.83	44.59	29.43	4.4	14.95
16	15.84	47.34	31.5	43.08	27.24	4.26	15.64
20	15.74	52.91	37.17	46.32	30.58	6.59	21.55

Table G5b: Moisture Content Determination for NC + 8%LM + 8%SCSA Bottom (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.61	53.88	38.27	51.93	36.32	1.95	5.37
8	15.28	42.66	27.38	40.31	25.03	2.35	9.39
12	16.33	45.17	28.84	42.37	26.04	2.8	10.75
16	16.27	57.21	40.94	52.04	35.77	5.17	14.45
20	17.05	49.15	32.1	43.27	26.22	5.88	22.43

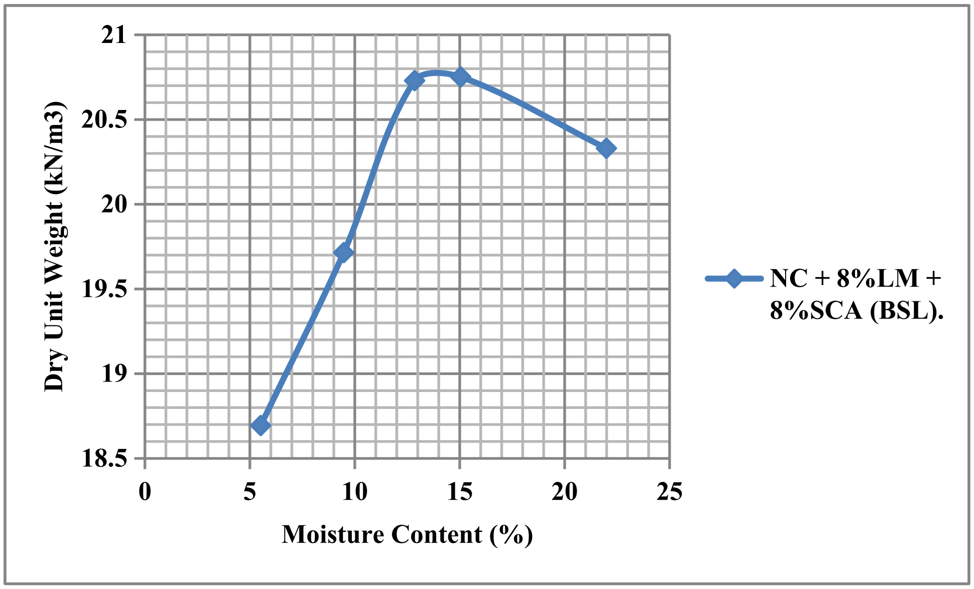


Figure G5: Compaction Curve for NC + 8%LM + 8%SCSA

Table G6: Dry Density Determination for NC + 10%LM + 10%SCSA (BSL)

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.6	1.6	15.70	5.29	15.75
8	0.001	4	5.75	1.75	17.17	8.34	17.25
12	0.001	4	5.85	1.85	18.15	11.23	18.26
16	0.001	4	6.05	2.05	20.11	16.42	20.27
20	0.001	4	6	2	19.62	21.72	19.84

Table G6a: Moisture Content Determination for NC + 10%LM + 10%SCSA Top (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.59	44.87	30.28	43.47	28.88	1.4	4.85
8	14.07	54.32	40.25	51.06	36.99	3.26	8.81
12	14.76	56.17	41.41	51.67	36.91	4.5	12.19

16	15.83	73.28	57.45	64.74	48.91	8.54	17.46
20	16.33	50.52	34.19	44.41	28.08	6.11	21.76

Table G6a: Moisture Content Determination for NC + 10%LM + 10%SCSA Bottom (BSL)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.84	68.17	51.33	65.39	48.55	2.78	5.73
8	15.88	45.53	29.65	43.37	27.49	2.16	7.86
12	16.04	76.84	60.8	71.18	55.14	5.66	10.26
16	17.33	43.28	25.95	39.82	22.49	3.46	15.38
20	16.93	60.41	43.48	52.66	35.73	7.75	21.69

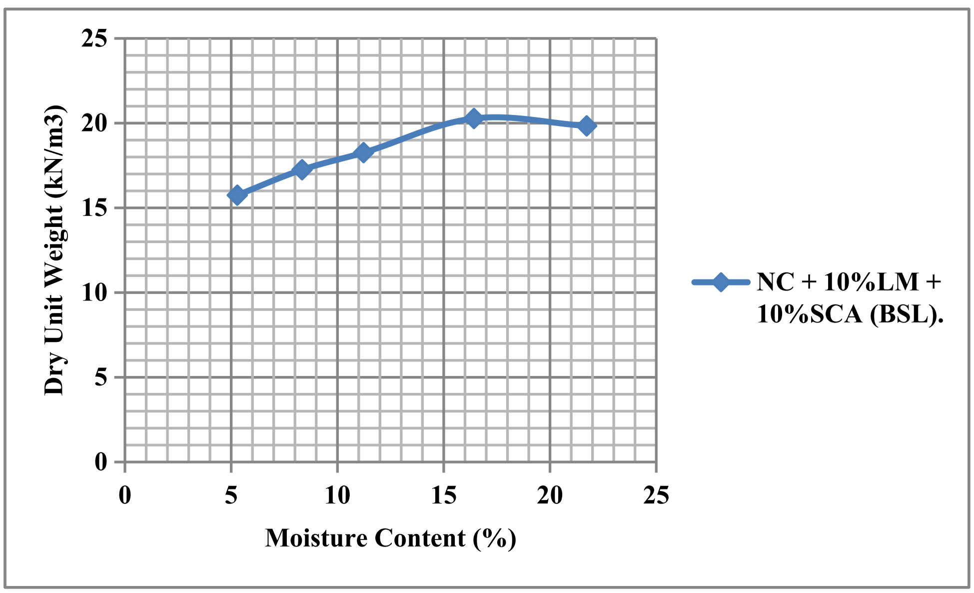


Figure G6: Compaction Curve for NC + 10%LM + 10%SCSA

DESICCATION INDUCED SHRINKAGE BEHAVIOUR OF LATERITIC SOILS

BY

UMEADI ANTHONY CHUKWUGOZIE

NAU/2015224072

A RESEARCH PROJECT REPORT PRESENTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF BACHELOR OF ENGINEERING (B.ENGR.) IN CIVIL ENGINEERING

SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING, FACULTY OF ENGINEERING, NNAMDI AZIKIWE UNIVERSITY, AWKA.

FEBRUARY, 2022

CERTIFICATION

This is to certify that this project work titled "DESICCATION INDUCED SHRINKAGE BEHAVIOUR OF LATERITIC SOILS", was carried out by UMEADI ANTHONY CHUKWUGOZIE, Registration Number: 2015224072. It is original and has never been submitted or published anywhere.

... **.......................................**

UMEADI ANTHONY CHUKWUGOZIE **Date**

Student

APPROVAL

This is to approve that this is an original work titled “**"DESICCATION INDUCED SHRINKAGE BEHAVIOUR OF LATERITIC SOILS,”** was carried out by **UMEADI ANTHONY CHUKWUGOZIE, Registration Number: 2015224072,** and has been prepared in accordance with the regulations governing the preparations and presentation of projects in Civil Engineering in Nnamdi Azikiwe University, Awka.

______________________________ ______________________________

ENGR. MRS. EZEMA N.M. Date

Supervisor

______________________________ ______________________________

ENGR. DR. EZEAGU A. Date

Head, Department of Civil Engineering

______________________________ ______________________________

External Invigilator Date

DEDICATION

This report is dedicated to God Almighty for his protection, guidance, wisdom and provision in my life and throughout the course of this project.

ACKNOWLEDGEMENTS

Firstly, I will start by expressing acknowledgement to God Almighty for his guidance and grace in my life and towards my work.

I wish to express my profound gratitude to ENGR MRS EZEMA N.M. for her earnest guidance and support towards my work.

My regards to my amazing mom, MRS UMEADI NMA VICTORIA who through her advice, prayers and financial support helped me throughout my project work and to my beloved siblings, I say, remain blessed by God Almighty. Also I will love to say thank you to myself for putting in the effort. Well done

Special thanks also go to my lecturers in the Department of Civil Engineering, for their seasoned lectures, to them all, I say be blessed, Amen.

ABSTRACT

Specimens prepared from three lateritic soil samples were subjected to drying under laboratory conditions. Volumetric shrinkage strains were measured at the end of the drying period. Results of this study indicate that, for the lateritic soils tested, volumetric shrinkage strains are influenced by soil composition and compaction conditions. Volumetric shrinkage strain increased with higher compaction water content. The influence of compaction water content on measured volumetric shrinkage strain was more pronounced in specimens with higher fines content. A regression equation was developed from the data to estimate volumetric shrinkage strain given the compaction water content relative to optimum, plasticity index, fines content and compactive effort.

TABLE OF CONTENTS

Title Page	i
Certification	ii
Approval	iii
Dedication	iv
Acknowledgments	v
Abstract	vi
Table of Content	vii
List of Tables	xi
List of Figures	xii
List of Plates	xiii
CHAPTER ONE:	
1.0 Introduction	1
1.1 Background of Study	1
1.2 State of Problem	4
1.3 Aim and objectives	4
1.4 Scope of Study	5
1.5 Significance of Study	5

CHAPTER TWO:

2.0 Literature Review 6

2.1 Lateritic Soil 6

2.1.1 Physical Characteristics 8

2.1.2 Chemical Characteristics 9

2.1.3 Formation and Development of Lateritic Soil 9

2.1.4 Composition of Laterite 12

2.2 Desiccation 14

2.2.1 Physical Processes involved by Desiccation of Soils 15

2.3 Shrinkage 17

2.3.1 The Morphological Characteristics of Shrinkage 18

2.3.2 Mechanism for Shrinkage Behaviour 19

2.3.3 Volumetric Shrinkage and Cracking Formation Process 19

2.4 Properties of Desiccated Soil 20

2.4.1 Volume Change Behaviour 20

2.4.2 Shear Strength 20

2.4.3 Cohesion Values 21

CHAPTER THREE: MATERIALS AND METHODS

3.1 Introduction 22

3.2 Sampling and Sample Location 22

3.2.1 Sampling Locality 22

3.2.2 Geology of Study Area 23

3.2.3 Climate 23

3.3 Materials 24

3.4 Methods of Testing 24

3.4.1 Specific Gravity Test 24

3.4.2 Index Properties 27

3.4.3 Atterberg Limit Test 29

3.4.4 Compaction Test 34

3.4.5 Volumetric Shrinkage Test 36

CHAPTER FOUR: RESULT AND DISCUSSION

4.1 Specific Gravity 40

4.2 Particle Size Distribution (Sieve Analysis) 41

4.3 Consistency Limit Test 42

4.4 Classification of Soil Samples 43

4.5 Compaction Results 43

4.6 Volumetric Shrinkage Result 45

4.6.1 Effect of Time on Volumetric Shrinkage Strain 45

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1 Conclusion 48

5.2 Recommendation 49

References 50

Appendix 58

LIST OF TABLES

Table 1: Composition of Lateritic Soil 13

Table 3.1: Coordinates of Sample Materials 22

Table 3.2: Details of Compaction Method 34

Table 4.1: Result of Average Specific Gravity 40

Table 4.2: Consistency Limit Result 42

Table 4.3: Index Properties of Lat 1, Lat 2 and Lat 3 43

Table 4.4: Compaction Test Result of Lat 1, Lat 2 and Lat 3 for BSL and BSH 43

Table 4.5: Volumetric Shrinkage Table of Lat 1 45

Table 4.6: Volumetric Shrinkage Result Table of Lat 2 46

Table 4.7: Volumetric Shrinkage Result Table of Lat 3 46

LIST OF FIGURES

Fig 4.1: Combined Sieve Analysis Graph 41

Fig 4.2: Compaction Curve for BSL and BSH of Lat 1 44

Fig 4.3: Compaction Curve for BSL and BSH of Lat 2 44

Fig 4.4: Compaction Curve for BSL and BSH of Lat 3 44

Fig 4.5: Volumetric Shrinkage Graph of Lat 1 47

Fig 4.6: Volumetric Shrinkage Graph of Lat 2 47

Fig 4.7: Volumetric Shrinkage Graph of Lat 3 47

LIST OF PLATES

Plate 3.1: Density Bottle and Funnel 25

Plate 3.2: Image of Sieve Shaker Equipment and Wire Cloth Sieves 28

Plate 3.3: Apparatus for Atterberg Limit Test 31

Plate 3.4: Apparatus Employed for Compaction Test 35

Plate 3.5: Practical Picture 1 38

Plate 3.6 Practical Picture 2 38

Plate 3.7: Practical Picture 3 39

Plate 4.1: Picture showing the Scattering of Compacted Specimen at dry side of OMC 45

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background of Study

The term laterite was originally suggested by Buchanan (1807) as a name for highly ferruginous deposits first observed in Malabar in India. Buchanan described a material that was soft enough to be cut with knife when in situ but hardened in exposure; it was being quarried to make bricks, and he derived the term from the latin Later, a brick. Laterite is usually reddish brown has a moderately high density 2.5-3.6; usually contains secondary aluminium, may contain quartz and kaolinite but low in other forms of silica, exchangeable bases and humus are absent. An iron-rich mottled clay which hardens on exposure to air, or repeated wetting and drying, is called soft laterite (Coka 2001).Laterite soils are considered as a firm soils and it commonly use as barrier in the Malaysia landfill liner system. However, Lateritic soils in the landfill area cannot perform satisfactory as barrier because of its high hydraulic conductivity. Hence, modification of soils to improve their engineering properties becomes necessary (Safiuddin, *et al*., 2011).

Lateritic soils are products of intense weathering which are devoid of gravel sized components and occur under tropical climatic condition resulting in the accumulation of hydrated oxides of iron and Aluminum (Akpokodje 2001). Lateritic soils are one of important soils and are widespread in tropical areas and subtropical climates. They are the most highly weathered soils in the classification system. The significant features of the lateritic soils are their unique color, poor fertility, and high clay content and lower cation exchange capacity. In addition, lateritic soils possess a great amount of iron and aluminum oxides (Shaw, 2001). Iron oxides, existing mainly in the amorphous and crystalline inorganic forms, are one of major components in many soil orders. It is thus very important to understand the properties of lateritic soils when they are going to be a commercial product for industrial application (Ko, 2008). Parent material is a key factor affecting the iron and mineral composition and distribution for lateritic soils. Different parent materials also bring the different physical and chemical properties (Ko *et al*., 2006).

A lateritic soil profile is characterized by the presence of three major horizons include the pallid or leached zone overlying the parent rock uioxide, the sesquioxide rich lateritic horizon and the mottled zone with evidence of enrichment of sesq. Brown and violet to black, ochre through red are the varieties colour of Lateritic soils (Safiuddin, *et al*., 2011). According to OSullivan and Quigley (2002), the colour of the soils depends largely on the concentration of iron oxides and the presence of hematite and goethite. If soil sample consist high amount of iron oxides, the sample of laterite gives reddish to brown in colour.

Lateritic soils abound in virtually all parts of the tropical world. They find wide applications not only as foundations for structures but more importantly as construction materials. Although, lateritic soils possess geotechnical, mineralogical and physical characteristics that make them fair to good engineering soils (Gidigasu, 2000), their properties may need to be improved upon. In some cases, soil stabilization, as the process of improvement, has over the years gained wide popularity among users of soils.

Lateritic soils have wider applications in the Nigerian construction industry, especially in road construction projects where they are utilized as fill materials and flexible pavement foundations. Their usage as sub-base and base construction materials is mainly because they are easy to manipulate on the road surface and have natural stable grading with a suitable proportion to act as binders. One of the major causes of a road accident is a bad road which is usually caused by wrong application of constructional materials, especially laterite as base and sub-base material by construction companies (Oke *et al*., 2009a; Nwankwoala *et al*., 2013). For a material to be used as either a base course or sub-base course depends on its strength in transmitting the axle-load to the sub-soil and or sub-grade. The degree of success in each case depends on the genetic characteristic of the soils and the specific purpose for which they have been used. The performance of lateritic soils as foundations for structure is varied and appears to depend on the nature of the soil, the degree of the weathering, topography, the drainage condition and more importantly on the type of foundation, and the amount of loads imposed (Ojo *et. al*, 2016).

In lateritic deposits, it may be possible to build ordinary structure on suitable design footings located a few feet below the ground surface. However, heavier structures may have to be based on firm layers, which are determined by sub-soil investigations. The actual design bearing values will depend on the degree of weathering of the lateritic soil and the geotechnical characteristics of the soil layers of a particular site. Lateritic soils develop from many rock types under different climatic and geochemical conditions. Hence in many instances, the properties to the parent materials are not lost on the resulting products of laterization. The process of laterization involves the breaking down of silicate materials such as illite and kaolinite leading towards the formation of hydrous oxides of iron and aluminum. The dominance of iron oxides gives laterites the characteristic reddish brown or dark brown color with a unique set of physical, chemical and engineering properties. (Omotoso, 2010). The mineralogical composition of the lateritic soil has an influence on the geotechnical parameters such as specific gravity, shear strength, swelling potential, Atterberg limits, bearing capacity and petrograpic properties (Amadi *et al*., 2012). A literature review has revealed that the geo-technical characteristics and engineering behavior of red soils depend mainly on the genesis and degree of weathering. Omotoso (2010) worked on laterite soil in connection with construction of road, highways and airfields.

Desiccation from Latin “de” which means “thoroughly” and “siccare” which means “to dry” is the state of extreme dryness or the process of extreme drying (Wikipedia). Desiccation in laterites soil is the process in which wet soils dry and soil moisture content decreases as the moisture evaporates into the surrounding environment, leading ultimately to cracking of the ground surface.

Desiccation cracks which occur due to volume changes resulting from moisture variation are common phenomena in soils, and can create pathways for percolation of fluids (Albrecht and Benson, 2001; Rayhani *et al*., 2007; Allaire *et al*., 2009; Taha and Taha, 2012). This phenomenon of self-healing can weaken the strength of the soil, causing shrinkage and reduction in crack dimensions during wetting, thus a panacea in waste containment facilities (Chertkov, 2000; Tang *et al*., 2011). The resulting loss in pore water leads to shrinkage of the soil mass and subsequently cracking and desiccation as the attractive forces within the clay cause individual clods to form. In some geotechnical applications such as landfills; this could be a serious problem. Therefore, volumetric shrinkage and desiccation cracking of compacted soils used as liners or hydraulic barriers have received much attention by researchers (Kodikara *et al.*, 2000; Osinubi and Nwaiwu, 2006; Eberemu *et al*., 2011; Moses and Afolayan, 2013).

1.2 Statement of Problem

The importance of desiccation-induced shrinkage study on lateritic soil is very significant. Undue volume changes in varying extraordinary climate conditions result to cracks being formed at their surface and subsequently spread deeper inside their matrix. This could result in the numerical increment in of hydraulic conductivity values. Shrinkage in soil causes considerable damages to civil engineering structures because of the pressures that are associated with heaving and differential settlement. Pressures are strong enough to crack foundations, floors, walls, roads, and pipelines. Many factors govern the shrinkage behavior of such soils like changes in water content, amount and type of clay size particles, type of soil, dry density, moisture content, magnitude of the surcharge pressure, and amount of non-expansive material such as gravel or cobble size particles (Hobart, 2015). Civil engineering projects that are constructed with problematic soils are one of the most common problems over the world. Therefore researchers look for alternative methods to improve soil properties before construction, so as to meet the engineering requirements of specific projects.

1.3 Aim and Objectives

The aim of this study is to evaluate the potential of desiccation inducing shrinkage behavior on three different lateritic soil

Objectives

1. To determine the index properties and compaction properties of the lateritic soil using two different compactive efforts.

2. To evaluate the volumetric shrinkage strain of three different lateritic soils

3. To determine the influence of compaction water content in measured volumetric shrinkage strain .

1.4 Scope of Study

The study covers examination of desiccation induced shrinkage behaviour of different lateritic soils. It will bring to focus the roles of desiccation in inducing shrinkage in lateritic soil. For this purpose, an experimental series of tests will be carried out to explore the effects of desiccation on shrinkage behaviour of lateritic soil. Material employed in this study include three(3) different lateritic soils and portable water. The index properties and compaction properties of the soils will be evaluated. These tests will be done in accordance with British standards (BS 1377:1990). The compacted soils samples will be air dried freely and their changes in height and diameter will be documented. These values will be employed in evaluating the volumetric shrinkage strains of each sample.

1.5 Significance of study

Engineering properties of soils play a significant role in civil engineering construction works particularly in road constructions, foundations, embankments and dams. This made imperative, the testing of soil, on which a foundation or superstructure is to be laid. This would determine its geotechnical suitability as a construction material. From literature lateritic soils exhibits alternative swelling and shrinkage behavior. The behavior poses serious threat to the foundation of buildings. Hence, to protect the structures from damage and to increase its design life, it is necessary to understand the properties of the soil before using for engineering work. Therefore this study will identify the effect of desiccation induced shrinkage on the behavior of lateritic soils.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 LATERITIC SOIL

The recognition of laterite as an earth material, with unique properties, dates back to 1807 when Buchanan first encountered a material in India which he called laterite and defined as soft enough to be readily cut into blocks by an iron instrument, but which upon exposure to air quickly becomes as hard as brick, and is reasonably resistant to the action of air and water. Since Buchanan's time, the word laterite has been used to describe a wide variety of tropical soils without reaching an agreement on the exact origin, composition and properties of laterites.

The term Laterite could mean brick earth in some local dialects but the name laterite got its meaning from a Latin word, meaning brick and so relating solely to the use of these soils in block making (Gonzalez de Vallejo and Ferrer, 2011). There have been so many arguments, criticism and learned discussion on the definitions of laterite and lateritic soils by different authors and workers giving different definitions in terms of its physical nature, chemistry, origin and morphology (Bourman and Ollier, 2002).By implication, a lateritic soil could then be described as that which has an underdeveloped laterite horizon capable of becoming true laterite given the appropriate conditions and sufficient time (Olanipekun, 2000). Giorgis *et al*., (2014) attempted to re-define laterite, by proposing a new definition and classification based on rock chemistry; basically on the Si/(Al + Fe) ratio in comparison to the chemical composition of the underlying parent rock. This was widely accepted, but later criticised strongly by several authors like Bourman and Ollier, (2002 and 2003) and Schellmann, (2003), where Bourman and Ollier proposed that the term laterite should have been abandoned. Therefore lateritic soil is used to describe the highly weathered tropical or sub-tropical residual soil, which is well-graded and usually coated with sesquioxide rich concretions. The colour may vary from liver brown to rusty red. Based on this, a laterite could therefore be regarded as fitting exactly the definition although non-residual soils are often included. High moisture content and temperature cause intense chemical weathering that produces well developed residual soils (Gonzalez de Vallejo and Ferrer, 2011). Their geotechnical behaviour is controlled by mineralogical composition, micro-fabric and geochemical environmental conditions. Where high iron (Fe) and aluminium (Al) content are present, laterites are formed and when drainage is poor, black cotton soils may develop, which have high smectite clay content. Alternate wet (rainy) and dry seasons favours the formation of lateritic soils, as leaching of the parent rock takes place in the rainy season while during the dry season, capillary action transports solutions of leached ions to the surface from where it evaporates with the salts left behind to be washed down the following wet season. Thus the whole zone is progressively depleted of the more mobile elements like Na, K, Ca. Olanipekun, (2000) observed that the high proportion of $Fe3+$ oxides in laterites signifies a left-over accumulation as a result of removal of silica and alkalis.

When compared with active clay soils, lateritic soils presents attractive option because of its greater shear strength properties, chemical resistance, better workability and availability within economic haul distance in tropical latitudes where they occur in abundance (Frempong, E. M., and Yanful, 2008; Osinubi, K. J. and Nwaiwu, 2008).

Chemical weathering progresses more rapidly in warm than in cool climates and provided there is good drainage, it is more prevalent in wet than dry climates. (Mitchell and Soga, 2005). This explains the more pronounced laterisation processes observed in the tropics compared to arid regions and also, the formation of laterites in the south-western region of Nigeria compared to the black cotton soils of the north-eastern part of the country. Laterites are mostly residual soils formed directly or almost directly on the parent rock making it possible for laterite/lateritic soils to retain some of the characteristics of the parent rocks. Kasthurba *et al*., (2007) reported a downward softening of material as a result of decline in sesquioxide cementation and increase in the amount of clay filled pore spaces. This brings about lateritic profiles which are characterized by accumulation of sesquioxides in the upper horizons and kaolinitization at lower levels

The process of laterisation around south-western Nigeria was observed by Emofurieta, & Salami, (1993) while studying the geochemical dispersion patterns associated with the laterisation process at Ile-Ife. It was reported that the soils derived from the melanocratic bands in gneiss bedrock are

SiO_2 rich, compared to soils derived from the leucocratic bands. Based on their average $SiO_2/Al_2O_3 + Fe_2O_3$ ratios, the soils derived from the melanocratic bands are lateritic whilst the leucocratic derivatives were described as non-laterite. In the author's opinion, those leucocratic derivatives could still be described as lateritic soils, as they could also reach a matured stage of laterisation given favourable conditions over time.

In the laterization processes, sesquioxides of iron and aluminum are absorbed onto the surfaces of the clayey constituents through the interaction of the positively charged sesquioxides and the negatively charged clay particles. The sesquioxides cause a physical cementation of fine particles into coarser aggregations resulting in a granular structure (Osinubi and Nwaiwu, 2008). The consequences of these processes on the engineering characteristics of lateritic soils include low plasticity, high permeability, and low swelling potential.

2.1.1 Physical Characteristics

Laterites occur principally in a vesicular or a pisolitic form but may be found in various intermediate types (Sharp, *et al*, 2001). Their physical characteristics may be categorized under the following four general aspects.

a. Induration: The hardness of laterites varies from a scarcely cohesive state to hard blocks. The degree of hardness can be attributed to the relative amount of sesquioxides and iron, and to the degree of hydration. The extent of aging and the arrangement of the various constituents have also an important effect on the induration of laterites.
b. Structure: The various structures of laterites may be reduced to the three following patterns, the indurated elements form a continuous, cohesive skeleton; the indurated elements are free concretions or nodules in an earthy matrix; and the indurated elements cement preexisting materials.
c. Color: Coloration of laterites is usually caused by the presence of iron oxides and sometimes also by manganese. The iron oxides impart a characteristic red color. When combined with other components of various colors, the red tint is changed in various degrees producing shades that can be listed as pink, ochre, rose, brown, violet, yellow, green, and black. Coloration is sometimes an indication of the level of evolution of the

laterite. As age progresses, ferruginous laterites change from red to brown to black, and aluminous laterites tend to become lighter.

d. Specific Gravity: Specific gravities of laterites vary in the range of 2.5 to 3.6, and may vary with the size fraction. It increases with iron content with age of formation and with cementation, and may either increase or decrease with depth of the various levels in the deposit.

2.1.2. Chemical Characteristics

Laterites are characterized by the high content of sesquioxides of iron and aluminum in relation to the other components. Bases are generally absent in most cases and combined silica in the form of kaolinite is very low, but neither can be considered an absolute criterion. Alumina is sometimes the principal constituent, but more commonly the iron oxide or the sesquioxides of iron and aluminum are the major constituents. In aluminous laterites, combined water, as determined by loss on ignition, is generally higher than in ferruginous late rites (Paigegreen, 2007).

Quartz can be a major constituent depending on the characteristic of the parent rock. Other minerals, such as titanium and manganese, may be present in sufficient quantities to justify exploitation as a mineral. Vanadium and chromium appear only in small or insignificant quantities. In general, however, a relationship between the components is difficult to establish owing to the fact that the composition tends to vary between the different size fractions (Pinard, 2011).

2.1.3 Formation and Development of Lateritic soil

Laterite is the product of a humid tropical weathering process, has the following effects:

- The parent material is chemically enriched with iron and aluminium oxides and hydroxides (sesquioxides)
- The clay mineral component is largely kaolinitic
- The silica content is reduced

The above processes usually produce yellow, brown, red or purple materials, with red being the predominant colour. While tropical weathering in oxidizing conditions generally leads to reddening, this does not necessarily produce a lateritic material hence the widespread confusion concerning laterite and its behaviour. Laterite formation requires particular conditions which concentrate the iron- and aluminium- rich weathering products sufficiently to allow concretionary development, often progressing to a cemented horizon within the weathering profile (Fookes,1997). Three phases of action are necessary to produce concretionary laterite:

(A) Humid Tropical weathering

In the humid, tropical regions of the world, chemical decomposition (the chemical alteration of the primary minerals into secondary and residual products) rather than physical disintegration, is the dominant mode of weathering which is especially effective in the presence of water and high temperature (Sharp, *et al*, 2001). The weathering process originates with rock exposure at or near the earth's surface in a physical environment, quite different from that in which it was originally formed. The minerals which constitute the rocks may react chemically with rainwater, ground water, and dissolved solids and gases of the new near surface environment to form new minerals which are more nearly in equilibrium with the surface conditions. The end result of these changes is to convert the upper portion of the rock into a residual debris more soil-like than rock-like in character and with chemical, mineralogical and physical properties entirely different from those of the original rock (WIDDOWSON, 2003).

(B) Concentration of minerals

Before the concretionary development of true laterite can take place, an additional process is required, the concentration of the weathering products within the residual soil/completely weathered zones (paigngreen, 2007). Processes are involved in the concentration of minerals into discrete horizons by leaching and residual accumulation or solution and precipitation

(C) Concretionary development

The degree of development of concretions in laterite materials significantly governs their engineering properties and is reflected in the appearance of the material. The harder these concretions are, the stronger is the material. The strength of the concretions depends mainly on the content of iron and aluminium oxide (sesquioxides) present and how much the concretionary particles have been dehydrated (Pinard, 2011). In order for the concretions to develop, they need sufficient concentration of hydrated oxides of iron and aluminium to act as catalysts for cementation or precipitation growth to start.

The concentrated, uncemented, partially self-hardened, horizon of material is referred to as plinthite by pedologists (a term that has gained wide acceptance among soil scientists as an unambiguous one for Buchanan's laterite). The hardening or concretionary development after the iron enrichment seems to proceed by a number of mechanisms including chemical precipitation, loss of water of crystallization (dehydration) and the development of a continuous fabric of cementing materials (Paigngreen,2007). The physical condition in which the processes develop is thought to be due to the fluctuation of the groundwater level, or simply distinct wet and dry seasons, which causes alternately reducing and oxidizing conditions. Only under the oxidizing conditions, provided by drying or the lowering of the groundwater level, can precipitation and dehydration take place. The current climatic conditions at which concretionary laterites are found are not necessarily those at the time of formation. Conditions of drying or alternate wetting and drying appear to be necessary for the precipitation of the sesquioxides (Pinard and Netterberg, 2012).

In light of the above, a laterite may be qualitatively described as a soil that has been impregnated with, cemented by or partly replaced by, another material. This process is mainly due to the precipitation of fine particles of sesquioxides which, with time, concentrate to the point whereby soft discrete nodules or concretions of soil cemented by the sesquioxides are likely to form. If the process continues the nodules coalesce and a spongy, hard mass full of cavities with a honeycomb structure is formed (Paigngreen, 1999). The cavities may be filled with remnants of the host soil although the cementing material may have already become quite hard. The filling of these cavities with the precipitate eventually results in rocklike hardpan which, when weathering, breaks down into boulders (Pinard, 2011). The filling and cementation of the material in the cavities eventually results in a massive, rock-like, hardpan. If this hardpan is subjected to a new cycle of weathering, a secondary deposit of lateritic material will be formed. Erosion and transportation play an important role in this process.

2.1.4 Composition of Laterite

Laterite is composed of both cohesion-less and cohesive soils. This forms the basis of laterites being referred to as C-φ (C-Phi) soils. The cohesion-less portion consist of gravel, sand and silts while the cohesive portion includes fine particles usually in silt and clay sizes. Lateritic soils behave in a unique way with some laterites changing volume when exposed to humidity variations while others are not affected. Hence, some components are referred to as stable i.e gravel and sand, while silt and clay are referred to as unstable. Stability in this sense is based on their ability to withstand variations in terms of moisture without a significant change in its properties, which is of course fundamental in materials for building construction. Composition of Laterites are essentially two-component mixtures of the original host or parent material and the authigenic cementing, replacing or relatively accumulated minerals (Netterberg, 1994). As the laterite develops, so the authigenic mineral content increases until it may constitute almost the whole material. Thus, hardpan laterite can be expected to have a higher content of sesquioxides (Al_2O_3 + Fe_2O_3) than a nodular laterite. The citrate-carbonate-dithionite (CBD)-extractable iron content (a measure of the total free iron oxide and hydroxide minerals present) of hardpans laterites ranges between 43 and 77 % (Widdoson, 2003). In the case of lateritic soils and gravels the content of Fe_2O_3 increases and that of Al_2O_3 decreases with particle size, while SiO2 is highest in intermediate fractions (Pinard, 2003). Rigassi, (1995) described the properties of each of these components of lateritic soil as follows:

a) Gravels: composed of fragments of rock of varying hardness, whose size fall between 2 - 20 mm with stable mechanical properties when it comes in contact with water.

b) Sands: composed of mineral particles, with size ranging between 0.06 - 2 mm. Stable, though lacks cohesion when dry, it has an appreciable degree of internal friction, which means, it offers a great resistance (i.e. mechanical) to intra-particle movement. It is normally characterised by apparent cohesion when wet due to the surface tension of the water present in the void spaces.

c) Silts: consist of grain particles ranging from 0.002 to 0.06 mm; cohesion is low when dry and it offers lower resistance to intra-particle movement than sands. Silts are characterized by cohesion in wet condition and susceptible to swell and shrinkage on exposure to varying levels of humidity, leading to appreciable change in volume. In the dry state, they have very poor cohesion and therefore cannot be used independently as main material for building.

d) Clays: the finest of the particle sizes in lateritic soil, generally smaller than 0.002mm. Their characteristics differ completely from the larger sized particles in that they consist mainly of microscopic clay minerals which include: kaolinite, illite and montmorillonite. Clay particles are usually coated in a thin-film of absorbed water molecules and since they are microscopic, they tend to be very light in comparison to surface tension forces acting on the film of absorbed water. Clays unlike gravel and sand, are not stable and quite sensitive to varying humidity. Due to strong attraction of clay to water, its volume increases due to increase in moisture content as a result of thick films of absorbed water (Mitchell and Soga, 2005). On the other hand; as clay dries out, shrinkage cracks may appear in the clay mass with a reduction in strength. The cracks also form pathways for water during subsequent wetting up events. This creates a major problem when clay is being used independently as a building construction material. Thus, a combination of the stable constituent's i.e gravel and sand with silt and clay forms good soil material for construction purposes. Lateritic soils appear to be best suited in this regard because it is made up of all these different particle sizes in varying proportions (well-graded).

Table One: Composition of Lateritic Soil

Component	% By Mass	Main form of occurrence
SiO_2	5 - 70	Quartz, feldspar, clay minerals
Al_2O_3	5 - 35	Feldspar, clay minerals, gibbsite
Fe_2O_3	5 - 70	Goethite, hematite
TiO_2	0 - 5	Anatase, rutile
MnO	0–5	
P_2O_5	0-1	
H_2O +	5 - 20	Clay minerals, goethite, gibbsite
Loss on Ignition	5 - 30	Clay minerals, goethite, gibbsite, organic matter
Organic matter	0,2 - 2	Organic matter

Source: Netterberg F. 1994

2.2 DESICCATION

Drying of soils and the ensuing cracking are a crucial issue in geo-environmental engineering. Drying fractures strongly affect permeability and may compromise the integrity of structures, such as clay buffers for nuclear waste isolation. In addition, cracking is the cause of substantial damage in foundation-supported structures. Compressibility increases substantially while the rate of consolidation decreases with the appearance of desiccation cracks (Morris *et al.*, 1992). Cracks are also a possible precursor for inception of a failure surface at the top of dams and embankments.

During desiccation, the bulk water pressure within the soil pores will become negative with respect to the atmospheric pressure. This depression of pressure (i.e., the difference between atmospheric pressure and bulk water pressure) is known as the soil matric suction and is associated with the formation of curved water menisci within soil pores. On the basis of the capillary tube principle, the soil suction that can be sustained within a soil pore meniscus may be represented as $2T\cos\theta/R$, where T is the water surface tension, θ is the wetting angle, and Ris the radius of wetted part of soil pore. Therefore, the smaller the pore size, the higher the suction that can be sustained before soil becomes dry.

Daniel and Wu (1993), investigated a clayey soil in order to define ranges of water content and dry unit weight at which compacted test specimen would have low hydraulic conductivity, adequate shear strength and minimal shrinkage. According to their findings, an acceptable limiting value of volumetric shrinkage strain to prevent desiccation of these soil was less than or equals to 4%. In a similar work, laboratory tests were carried out by Osinubi and Nwaiwu, (2008) using compacted lateritic and clayey sand soils subjected to drying under room condition. The changes in volume were determined at the 7, 14 and 21 days of the drying process. The results from the experiment showed that volumetric shrinkage strain was influenced most by the clay content, compaction condition, drying process, wetting and drying cycles, soil particle orientation, unit weight, pore fluid and exchangeable ions (Yesiller *et al.*, 2000; Osinubi; Moses and Afolayan, 2013).Albrecht and Benson, (2001) found that cracking could increase the hydraulic conductivity of clay liner material by sometimes as large as three fold due to a larger flow path.

2.2.1 Physical processes involved by desiccation of soils

I. Drying

Drying results from an initial thermodynamic imbalance between the soil moisture and its surroundings, which causes evaporation and a transfer of fluids within the soil. In the general case, the fluid movement is accomplished through both liquid and gaseous phases. According to equilibrium thermodynamics laws, the phase change between pore liquid and vapour occurs instantaneously at the interface between the phases, so that the specific vapour and liquid Gibbs potentials remain equal (Coussy *et al*., 1998; Mainguy *et al*., 2001). This generates liquid pressure decay in this zone according to Kelvin’s law, and at the same time, a gradient of suction within the body. The liquid movement is due to the spatial differences in liquid head that then arise. External pressures, high temperatures and shrinkage deformations are also responsible for additional pore fluid pressure generation and subsequent fluid movements.

II. Shrinkage

At the micro-scale it may be considered that the basic process behind the shrinkage of a soil is a decrease in liquid pressure and more generally an increase in suctions, caused by the evaporation at the location of interphase menisci, and which acts as an attractive force between the components of the matrix. In the initial, saturated stage of the process the menisci are located at the external boundary of the soil body. At the macro scale, this translates into application of suction at the boundary and a resulting increase of effective stress compression throughout the soil the matrix and the ensuing sample shrinking. For fine soils with no swelling minerals, and following Mitchell and Soga (2005), it is reasonable to assume that the mechanisms related to the adsorbed water are not prevalent (i.e. capillary processes predominate), at least for a large range of water content and associated shrinkage strains. Shrinkage strain then can be seen as the consequence of an effective stress increase. At the macro-scale the effective stress embeds the contributions of externally applied stresses and internal pore fluid pressures. The Bishop’s generalized effective stress is a standard and broadly recognized effective stress for partially saturated soils (Nuth and Laloui, 2008).

III. Cracking

Desiccation macro-cracks are likely to occur if the drying shrinkage is constrained. Typically, these constraints can arise from different causes (Hueckel, 1992):

(i) a frictional or any other traction or displacement boundary conditions;

(ii) Any eigen-stress concentrations within the soil sample; and

(iii) Intrinsic soil inhomogeneity factors, such as soil texture and soil structure.

In the field, cause (i) can arise from any restraining structure. cause (ii) from soil moisture gradients, which do not respect the strain compatibility conditions (Kowalski, 2003). Irrespective of soil internal structure, any soil element that is allowed to dry and is not constrained in its movement by adjacent elements or boundary conditions, would not crack. In early studies, stress generation and subsequent cracking was related to the change in the rheological properties of soil during drying. Because of the liquid loss, soil loses its ability to relieve the tensile forces generated by the mechanisms. An additional build-up of stress arises, which is finally relieved by tensile failure and cracking (Kim and Hwang, 2003). When the shrinkage deformations at a macroscale are expressed through a phenomenological approach, i.e. directly relating the development of shrinkage deformations to a non-mechanical convenient variable such as water content or suction.

Desiccation cracking is a complex phenomenon involving the coupled interaction of soil and atmosphere (Cui et al., 2013). Over recent decades, substantial research efforts have been devoted to developing experimental, theoretical and numerical approaches to investigate the fundamental cracking mechanism and characterizing the cracking behavior of soils. Several review papers have been published in the past on this topic: Morris et al. (1992) reviews the occurrence and morphology of cracks in the dry-climate regions, and developed theoretical solutions to capture the crack depths observed in the field; Péron et al. (2009c) described the physical processes associated with the desiccation cracking of soils and discussed the initiation and propagation of desiccation cracks; and Kodikara and Costa (2013) presents a summary of historical field observations, laboratory modeling and identified mechanisms. Bordoloi et al. (2020) provided an insight into the influence of vegetation on soil cracking in the context of the soil-water-plant interaction. Wei et al. (2020) summarized part of experimental work and mechanism study of desiccation cracking behavior. Comprehensive state-of-the-art reviews emphasizing lab- and field-scale investigation approaches, cracking dynamics, and influencing factors of desiccation cracking behaviors remain unavailable.

Desiccation cracking behavior of soil is a complex process influenced by a large variety of factors, which can be categorized into four major groups, including intrinsic properties, boundary constraints, environmental factors, and soil admixtures.

2.3 SHRINKAGE

These fissures separate the soil surface and gradually close down into the deeper soil and in turn give rise to different problems of stability. Clayey soil is observed to give different characteristics in wet and dry conditions, it possess desirable sorption characteristics when wet and crack with dust emitted when dry (Osinubi and Eberemu, 2010). These cracks break down continuity of soil mass, thereby reducing its strength and soil stability is affected. It grants surface water easy infiltration into the soil. When laterite is soaked with infiltrated water, it collapses and loose strength. Structures built on a soil with stability problem, especially pavements, will in turn collapse with the soil.

It is commonly recognized that shrinkage form in lateritic soils during desiccation when the tensile stress caused by suction is larger than the tensile strength of soil. Shrinkage can greatly weaken the engineering properties of clayey soils in various applications, such as geotechnical, water conservancy, and environmental engineering (Tay *et al*, 2001; Wang *et al*, 2016).In many conditions, there is direct inducement of damage for many soil-based structures. Shrinkage can affect soil's hydro mechanical properties such as strength, bearing capacity, permeability, and compressibility. The overall stability of Earth structures and foundations (slopes, buildings, dams, and embankments) can also be influenced by the consequences of shrinkage. In addition, shrinkage can create preferential path for water in slopes, which greatly influence the transport rate and velocity of moisture, solutes, and microorganisms in soil profile, and affect the crops growth and production.

2.3.1 The Morphological Characteristics of Shrinkage.

Desiccation have been observed in situ and in laboratory, respectively. Desiccation are characterized in different clayey soils, for example, different layers of lacustrine clays, desiccated hydraulic fills, silty sand, and moderately to highly plastic clayey soil. Parameters such as crack patterns, orientations, and depths of the cracks are analyzed (Wang *et al* 2018). In northwestern China, there is large-scale distribution of clayey loess. Hierarchical pattern formation in clayey soils can be observed in situ. It is concluded that the crack patterns are not formed simultaneously (Peng *et al*, 2006). Otherwise, they formed in a sequential and hierarchical way. The scale of cracks can also be influenced by in homogeneous water content. With larger water content, the cells tend to be bigger. In a homogeneous soil matrix, a crack will develop perpendicularly to the direction of maximum stress. Any subsequent crack tends to propagate to reach the preexisting crack perpendicularly because the stress in the vicinity of an existing crack is parallel to its developing direction (Wei *et al*, 2016; Philip *et al*, 2002). Most of physical observations are consisted in geological papers, and measurable parameters that control soil cracking are not investigated. Analyses of quantitative relationship between crack spacing and these physical factors are still insufficient (Auvray *et al*, 2014; Peng *et al*, 2006).

In the field of geology and geophysics, the geometric and morphologic characteristics of the crack pattern reflect the drying process and may be able to inform us details about historic climatic conditions (Wang et al., 2018b). Glennie (1970) observed that desiccation crack pattern could be preserved after formation if sediments filled the spaces. If layers of soil continue to build up, a historical record of desiccation cracks can then be preserved underground. Understanding the conditions necessary for the formation of desiccation cracking patterns shed light on the potential correlation between underground water activity and debatable climatic conditions. Style et al. (2011) pointed out the possibility of quantifying local windblown (aeolian) sediment levels based on soil desiccation cracking patterns. The formation of desiccation cracks dramatically increases the surface roughness of soil, reduces the wind speed threshold at which sediment particles are picked up from the soil surface, and thus facilitates sediment entrainment. Researchers have also compared the desiccation-induced soil polygons on Earth and the polygonal cracking patterns on Mars, to analyze the historic climate and mineralogical conditions on Mars and provide possible evidence of ancient playa settings (El Maarry et al., 2012; El-Maarry et al., 2013, El-Maarry et al., 2014). Moreover, desiccation cracks considerably modify the hydrologic flow path conditions, increase the weathering of soils, and impair their water retention capabilities, resulting in the aggravation of soil erosion and the destruction of local environmental ecology (Zeng et al., 2020).

2.3.2 Mechanism for Shrinkage Behaviour

Several studies have been performed with the purpose of giving a better understanding of the characteristics on the morphology of the shrinkage related to desiccation. Different techniques have been used in order to quantify the shrinkage behaviors in soils (Lima and Grismer, 1992). However, it is not easy to quantify the shrinkage with irregular form and complicated geometry. For the purpose of better analyzing the characteristics of shrinkage, digital imaging methods have been applied to quantify the parameters of shrinkage (Tang *et al*, 2010; Hallaire, 1994). Digital image correlation (DIC) has become an efficient method, which can better quantitatively analyze the variation of cracking parameters during desiccation.

In addition, for better understanding the relationship between physical properties of soil materials and macroscopic shrinkage behavior, meso structural and microscopic researches have been increasingly carried out (Chertkov, 2002).

2.3.3 Volumetric Shrinkage and Cracking Formation Process.

Soil shrinkage is in response with the stress state. The initiation and propagation of cracks correspond tightly with volumetric shrinkage of soils. Clayey soils shrink mainly due to the hydrophilic clay minerals (Yesiller *et al*, 2000). During drying, water evaporates from the surface in a fully saturated homogeneous clayey soil. In the beginning of desiccation, the superficial water evaporates first and the stress-strain states are not influenced. A water-air meniscus between clay particles formed when the water-air interface gradually reached the clay surface. Thus, the capillary suction appears in the upper layer of soil sample. With the evaporation of water, the capillary suctions among the clay particles increase (Costa *et al*, 2013). At the same time, the curvature of capillary meniscus increases. Thence the clay shrinks and consolidates. From the micro scale aspect, it can be imagined that each of soil grains on the layer surface is controlled by tensile force, which is due to the increase of capillary suction from the nearby particles. A tensile stress field is therefore developed in the upper layer. As soon as the tensile stress is larger than the tensile strength of soils, cracks appear on the surface (Tang *et al*, 2011). At the microscopic scale, the clayey particles are reorganized and are closer to each other, with decreasing void ratio. At the macroscopic scale, this phenomenon is presented by volumetric shrinkage, which can be demonstrated by void ratio water content curve. Soil shrinks with the increase of soil plasticity. For example, for the same content of clay minerals in a soil, compared with kaolinites, montmorillonite suffers a greater volume of change during drying and wetting (Cordero *et al*, 2017). It is expected that, for two samples from a given material which are with the same initial water content and different soil fabrics, the one with dispersed structure shrinks most. Other parameters that affect shrinkage of clays are also investigated by researchers. It is identified that the amount of sand reduces the drying shrinkage in soils. The strains caused by shrinkage were initially proportionate to water content and did not rely on dry density of soil samples (Nahlawi and Kodikara, 2006; Villar *et al*, 2010). Besides, the dielectric constant of the pore fluid is also one factor, which influences shrinkage of soils.

2.4 PROPERTIES OF DESICCATED SOIL

2.4.1 VOLUME CHANGE BEHAVIOR

Experimental work indicated that repeated desiccation results in interparticle bonding which is also chemical in nature. Such bonding results in increased resistance to compression and greater permeability. Depending on the degree of bonding, an expansive soil may display either reduced expansiveness or even behave as a non-expansive soil in the undisturbed condition.

2.4.2 SHEAR STRENGTH

The behavior of both saturated and unsaturated soils is affected by the pore-water pressures in the soils. Unsaturated soils have the negative pore-water pressures (i.e., metric suctions).
Suctions have a major effect on shear strength and volume change for unsaturated soils.

The very high suction is produced by evaporation near ground surface.

2.4.3 COHESION VALUES

It is now common knowledge that if soil becomes desiccated it demonstrates a significant change in cohesion. Preliminary test data from the soil tests undertaken on many projects indicate that the cohesive strengths of a normal saturated clay can increase by as much as $100kN/m^2$ as the soil becomes desiccated

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

This chapter describes the details of the laboratory experiments carried out. A wide range of laboratory tests was carried out to investigate desiccation induced shrinkage behavior of lateritic soils from three different sample locations designated as (LAT 1, LAT 2 and LAT 3). The test conducted were specific gravity test, sieve analysis test, consistency limit test, compaction test (BSL and BSH) and volumetric shrinkage test.

3.2 sampling and sample location

Three samples were selected for the various testing technique from different locations within the geological location of Anambra state and Oji Local government area in Enugu state. The three samples were sampled designated as (LAT 1, LAT 2, LAT 3) and were all collected at depths representative of the soil stratum. They were kept safe and dry in sack bags in the Geotechnical Laboratory of the Department of Civil Engineering Nnamdi Azikiwe University, Awka.

3.2.1 Sampling Locality

Table 3.1: coordinates of sample materials

S/N	Samples	Latitude	Longitude	Location
1	LAT 1	6.2552N	7.1535E	Ugwuoba
2	LAT 2	6.2228N	7.0537E	Okpuno
3	LAT 3	6.2060N	7.0258E	Nawfia

The lateritic soil samples used for this project were disturbed samples collected from a burrow pit located at different locations across Awka and Oji RIVER local government. LAT 1 was collected from Ugwuoba in Oji River local government area in Enugu State; LAT 2 was collected at Okpuno burrow pit in Awka South local government, while LAT 3 was collected from Nawfia in Njikoka Local Government Area of Anambra State.

3.2.2 Geology of Study Area

Ugwuoba in Oji River local government area is located few kilometers from Enugu, the capital city of Enugu state. The study area is underlain by the Ajali Sandstone and Nsukka Formation. The Nsukka Formation (Upper Maastricht an) covers 25% of the study area. It lies conformably on top of the Ajali Sandstone (Maastricht an), which covers a total of 65% of the study area. The Ajali formation has a thickness of about 45-50 meters and consists of thin band of white mudstone and shale occurring at interval and increasing in number toward the base. It is also characterized by gently sloping topography, the nature and slope of the area makes it vulnerable to erosion, while Okpuno and Nawfia lies below 300 meters above sea in a valley on the plains of the Mamu River. Two ridges or cuesta, both lying in a North-South direction, form the major topographical features of the area. The ridges reach the highest point at Agulu just outside the Capital Territory.

3.2.3 Climate

Ugwuoba, the inversion the tropical mass causes total annual rainfall range from 1600m more than 2000m. The area falls within the tropical rainforest belt Nigeria with temperature ranges from 300c 320c is characterized two seasons, the dry and rainy season. The climate favorable for farming. The dry season usually from (November-March), is marked an average rainfall of about 60mm.

The highest annual rainfall in south eastern Nigeria received around the eastern highlands due the convectional and orthographic nature the rains received. While that of Okpuno and Nawfia in the tropical zone of Nigeria and experiences two distinct seasons brought about by the two predominant winds that rule the area: the south western monsoon winds from the Atlantic Ocean and the North east dry winds from across the Sahara Desert. The Monsoon winds from the Atlantic creates seven months of heavy tropical rains which occur between April and October which are then followed by five months of dryness (November March). The harmattan also known as Ugulu in Igbo is a particularly dry and dusty wind which enters Nigeria in late December or in the early part of January and is characterized by a grey haze limiting visibility and blocking the sun's rays. The temperature in Awka is generally a comfortable 27-30 degrees Celsius between June and December but rises to 32-34 degrees between January and April with the last few months of the dry season marked by intense heat.

3.3 Materials

Lateritic sample were obtained from borrow pits at Nawfia, Ugwuoba and Okpuno. The choice of the site is justified by the fact that it is from where construction companies obtain their materials for road construction and earth filling work. The laterite samples obtained were collected with the aid of a digger and a shovel at a depth of 300mm (1 foot). The sample passed all the physical test that could classify them as lateritic soils which are: it is reddish-brown in colour, fine grained in texture and could become hard during the dry season. These samples were collected in bags and were conveyed via commercial transport to the school laboratory. The in-situ moisture content of the sample was determined using oven-dried method before air-drying for a period of two weeks in an open area using corrugated roofing sheets so as to ensure complete and even drying of moisture from the samples. Upon drying, the sample needed to be segregated by means of crushing the agglomerate with the help of wooden mortar and pestle. Enough care was exercised to ensure that the individual particles were not crushed into smaller sizes. This was achieved by pressing the pestle on the sample agglomerate and not pounding the soil with the pestle.

3.4 Methods of testing

3.4.1 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined by (Braja, 2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

1. Density bottle of 50ml capacity and a stopper.

2. Desiccator containing anhydrous silica gel.

3. Thermostatically controlled oven with temperature of about 80-110OC.

4. Weighing balance of 0.01g sensitivity.

5. Mantle heater.

6. Plastic wash bottle.

7. Distilled water.

8. Funnel

9. Thin glass rod for stirring.

10. 425um Sieve.

11. Dry piece of cloth for cleaning.

12. Masking tape for identification of sample.

13. Exercise book and pen for recording of result.

Plate 3.1 Density bottle and Funnel

The Procedure for Computation of result obtained are as follows:

Specific gravity (GS) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

M_3= Weight of density bottle filled with water + wet soil + stopper

M_4= Weight of density bottle filled with water + stopper

Test Procedure

The density bottle used was properly cleaned and was rinsed with distilled water before been placed in a desiccator to remove any moisture present. The empty bottle was weighed together with the stopper and the weight recorded as (M_1). About 10-15g of soil passing through sieve 425micrometer was poured into the density bottle and the weight of density bottle +dry soil + stopper recorded as (M_2). Distilled water was added to fill about half to three-fourth of the density bottle and it was allowed to saturate for 24hours. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample. The density bottle was allowed to cool at room temperature and then filled with distilled water up to the specific mark (at lower meniscus level). The exterior surface of the density bottle was dried with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and weight recorded as (M_3) and thereafter the density bottle was emptied, cleaned and rinsed with distilled water. The density bottle was filled with distilled water up to the same mark after which the exterior surface was dried. The weight of the density bottle filled with distilled water + stopper was determined and the weight recorded as (M_4). The procedure was repeated for two more trials after which the average specific gravity was determined.

3.4.2 Index Properties

The test will be done in accordance with BS 1377 part 2.

Sieve analysis is a technique used to determine the particle size distribution of a granular soil material (sand, gravel). This method is performed by sifting soil sample through a stack of wire mesh sieves, separating it into discrete size ranges. A sieve shaker is used to vibrate the sieve stack for a specific period of time. Vibration allows irregularly shaped particles to reorient as they fall through the sieves. Additionally, agitation of the sieves serves to break apart weak Agglomerates, allowing for a more reliable measurement of the particle size distribution. Care should be taken to choose an appropriate agitation time, so that particle fracture does not occur. The particle size distribution of a powder serves as an indication of flowability. Soils with a broad size distribution tend to be poorer flowing than those with a narrow size distribution.

The grain size analysis is widely used in classification of soils. The data obtained from the grain distribution curve is used in the design of fillers for earth dams and to determine the suitability of soil for road construction, air field etc. Information obtained from grain size analysis can be used to predict soil water movement although permeability test is more generally used. Soil gradation is very important to geotechnical engineering; it is an indication of other engineering properties such as shear strength, compressibility and hydraulic conductivity. In a design, the gradation of the in-situ- soil help in the selection of filler material for the construction of highway embankment and it also controls the design and ground water drainage of site. A poorly graded soil (one with predominantly one-sized particle) will have better drainage property than the well graded soil (soil with varieties of particle sizes) because of the relatively higher magnitude of void present. A well graded can be easily compacted more than a poorly graded soil. However most engineering project may have gradation requirement that must be satisfied before the soil is to be used is accepted for construction work. When options for ground remediation technique are to be considered the soil gradation is a controlling factor.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.

2. Mechanical sieve shaker.

3. Weighing balance of 0.01g sensitivity.

4. Hand brush

5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)

6. Thermostatically controlled Oven (With temperature of about 80OC-110OC).

7. Masking tape for identification of sample.

8. Exercise book and pen for recording of result.

9. Hand brush

Plate 3.2 Image of sieve shaker equipment and wire cloth sieves.

The Procedure for Computation of result obtained are as follows:

1. Percentage retained (%) = $mass\ of\ soil\ retained\ in\ the\ sie(g)total\ mass\ of\ soil\ sample(g)$ × 100

2. Cumulative percentage retained = $\Sigma Percentage\ retained$ (%)

3. Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

4. Coefficient of Curvature = $\frac{\mathrm{D60}}{\mathrm{D10}}$

5. Coefficient of Uniformity = $\frac{(\mathrm{D30})\hat{}2}{(\mathrm{D10})\times(\mathrm{D60})}$

Where

6. D10= particle size such that 10% of the soil is finer than the size
7. D30= particle size such that 30% of the soil is finer than the size.
8. D60= particle size such that 60% of the soil is finer than the size.

Test Procedure

The sieves to be used for the experiment were cleaned properly using hand brush and it was arranged in decreasing order of aperture on a mechanical sieve shaker. 500g of air-dried soil sample was weighed out using a weighing balance after which it was washed inside sieve No. 200 under steady supply of water until the water passing through the sieve became clear. After washing, the washed sample was placed inside a thermostatically controlled oven at a temperature of 80-110˚C for 24hrs. The sample was brought out and allowed to cool before weighing, the weight of the soil sample was determined by deducting the weight of plate from the weight of plate and soil. The dried sample was poured into the stack of sieves with sieve of highest aperture on top, the sieve shaker was connected to electricity and allowed to shake for about 10-15 minutes. The sample retained on each sieve were weighed and the weights were recorded. Percentage retained, cumulative percentage retained and cumulative percentage finer were determined. A graph of cumulative percentage finer against sieve sizes was plotted.

3.4.3 Atterberg limit test

This test is done as per standards in BS 1377 part 2: 1990

Atterberg Limits test is to determine the water content at distinct transitions between different states of soil consistency and is widely used in the design stage of construction to ensure that the soils being used exhibit the proper consistency to support structures even as their moisture levels change. Soils for engineering use are often classified based on properties relative to foundation support or how they might perform under pavements.

These limits can be determined with the three tests that make up the Atterberg limits tests. They are:

I. Liquid Limit (LL)

II. Plastic Limit (PL)

III. Shrinkage Limit (SL)

i. Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. Liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Casagrande type)

2. Grooving tool

3. Moisture content tins

4. Porcelain evaporating dish

5. Spatula or pellet knife

6. Thermostatically controlled oven

7. Weighing balance sensitive to 0.01g

8. Plastic wash bottle containing distilled water

9. Paper towels

10. Masking tape for identification of tin.

11. Exercise book and pen for recording of data

12. 425um Sieve

13. Airtight container

Plate 3.3 Apparatus for Atterberg Limit Test.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content = $Weight\ of\ water\ weight\ of\ dry\ soil$×100 = $\frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried soil.

Test procedure

150g of soil sample passing through 425micometer sieve was weighed out and was placed on a glass plate, thereafter, distilled water was added and pallet knife was used to mix it thoroughly to form a thick homogenous paste after which the sample was place in an airtight container and was left to mature for 24hours. Five moisture content tins were weighed and the mass recorded as (M_1). The matured sample was placed on an evaporating dish and little water were added using plastic squeeze bottle after which the soil was properly mixed to ensure uniform distribution of moisture. A portion of the paste was placed on the liquid limit device and it was levelled to obtain a maximum depth of 1cm. The grooving tool was used to cut a groove along the symmetrical axis of the cup. The handle of the liquid limit device was rotated at the rate of 2 revolution per second and the number of blows that closed the groove at a distance of 13cm were counted. About 10g of the sample was taken out from the closed groove for moisture content determination, weigh the sample + moisture content tin and record the mass as (M_2). The rest of the soil in the liquid limit device cup were removed and the cup was cleaned properly using towel. The water content of the sample was altered and the process were repeated to get the number of blows for a specific range of blows. A graph of moisture content against the log of number of blows was plotted and the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

ii. Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil.

The apparatus used for this experiment includes:

1. A smooth glass plate about 300mm square and 10mm thick.

2. A palette knife or spatula

3. A short length of 3mm metal rod

4. Moisture content tins

5. Plastic squeeze bottle

6. Weighing balance with 0.01g sensitivity

7. Veneer caliper

8. Masking tape for tin identification

9. Exercise book and pen for recording of result

The Computation for Plastic Limit is as follows:

Plastic limit = $\frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W_3 = Weight of tin plus oven-dried soil

Test procedure

The sample was prepared by the method described in the BS 1377: 2: 1990. The moisture content tin to be used was weighed and the mass recorded as (M_1). About 20g of prepared soil paste on a porcelain dish was taken out, thereafter, it was mixed thoroughly with water until the paste became plastic enough to be rolled into ball. A portion of the ball was rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward. The rolling and remolding were continued until the thread just start to crack at a distance of 3mm. the crumbled pieces were collected and placed in a moister content tin and weighed and the mass was recorded as (M_2). The moisture tin was placed in the oven at a constant temperature of 80-110˚C for a period of 16-24hours, after 24hours the tin was removed from the oven and the mass of the dry soil plus the tin was determined and the mass recorded as (M_3). The test was repeated for two trials and the average plastic limit value was determined.

3.4.4 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry unit weight of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

The compaction methods to be adopted for this research are:

British Standard Light (BSL) and British Standard Heavy (BSH) for the natural samples of laterite collected from Okpuno, Ugwuoba and Nawfia.

Table 3.2 Details of Compaction method.

Type of test	Mould (cm^3)	Rammer (kg)	Drop (mm)	No. of layers	Blow per layer
BS light	1000	2.5	310	3	27
BS heavy	1000	4.5	450	5	56

The apparatus used for the test are as follows:

1. Compaction mould with a detachable base plate and removable extension collar.

2. Metal rammer (2.5kg and 4.5kg)

3. Measuring Cylinder 200ml or 500ml

4. Large Metal tray (600mm×600mm ×600mm)

5. Balance up to 10kg readable to 1g

6. Small tools such as palette knife, steel straight edge about 300mm long.

7. Drying oven temperature of 105-110OC

8. Apparatus for moisture content determination

Plate 3.4 Apparatus employed for Compaction Test.

The Computation of the result obtained is as follows:

Determination of dry unit weight (Pd).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (m^3) = W_4

Bulk Density (kg/m^3) = mass $of\ wet\ soil$ - mass of mould ÷ volume of mould = $\frac{W2-W1}{W4}$

Moisture Content (%) = $mosture\ content\ (top$ + bottom) ÷ 2

Dry unit weight (kN/m^3) = $\frac{\text{Pb}}{(1+\frac{w}{100})}$

Test procedure.

Firstly, the mould, extension collar and base plate were cleaned and greased, the mould was weighed and the mass was recorded as (M_1), the rammer was checked for free fall. The extension collar was attached to the mould. 3kg of soil sample was weighed out using a weighing balance, after which the weighed sample was placed in a mixing tray. 2% of water was added, thereafter, it was mixed thoroughly before been shared into three layers for British Standard Light (BSL) compaction. The first layer was poured inside the mould and was compacted by applying 27blows using 2.5kg rammer falling freely from a height of 300mm. the blows were distributed uniformly over the surface of the mould. After completion of the compaction, the extension collar was removed and the mould was carefully levelled off by means of straight edge. The mould plus the compacted soil was weighed and the mass recorded as (M_2). Small portions of the sample were collected from the top and bottom of the mould for moisture content determination. The procedures were repeated for 4%, 6%, 8% and 10% of water and the values obtained were recorded. A graph of dry unit weight (DUW) against moisture content were plotted and optimum moisture content (OMC) corresponding to maximum dry unit weight (MDUW) was determined.

3.4.5 Volumetric shrinkage test.

Volumetric shrinkage is the decrease in volume (expressed as a percentage of the soil mass when dried) of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

The apparatus used for the test are as follows:

1. Compaction mould with a detachable base plate and removable extension collar.

2. Metal rammer (2.5kg)

3. Measuring Cylinder 200ml or 500ml

4. Large Metal tray (600mm×600mm ×600mm)

5. Balance up to 10kg readable to 1g

6. Small tools such as palette knife, steel straight edge about 300mm long.

7. Drying oven temperature of 105-110OC

8. Extruding machine.

Test procedure.

The compaction mould, extension collar and base plate were cleaned and greased, the mould was weighed and the mass was recorded as (M_1), the rammer was checked for free fall. The extension collar was attached to the mould. 3kg of soil sample was weighed out using a weighing balance, after which the weighed sample was placed in a mixing tray. +2% of optimum moisture content was added, thereafter, it was mixed thoroughly before been shared into three layers for British Standard Light (BSL) compaction. The first layer was poured inside the mould and was compacted by applying 27blows using 2.5kg rammer falling freely from a height of 300mm. the blows were distributed uniformly over the surface of the mould. After completion of the compaction, the extension collar was removed and the mould was carefully levelled off by means of straight edge. The mould plus the compacted soil was weighed and the mass recorded as (M_2). The mould plus the compacted soil was taken to the metallic extruder for the extrusion of the compacted soil. The extruder and the mould were firmly tighten and was ensured to be in-line before extruding. After the extrusion, the diameter and height of the extruded soil were taken. These procedures were repeated for +4%, -2% and -4% of optimum moisture content.

Formula used in computation of volumetric shrinkage

Volume of specimen = $\pi r^2 H$ (cm^3)

Radius = r (cm)

Height = H (cm)

Volumetric shrinkage (%) = $\frac{\text{Initial volume} - \text{Final volume}}{\text{Initial volume}}$

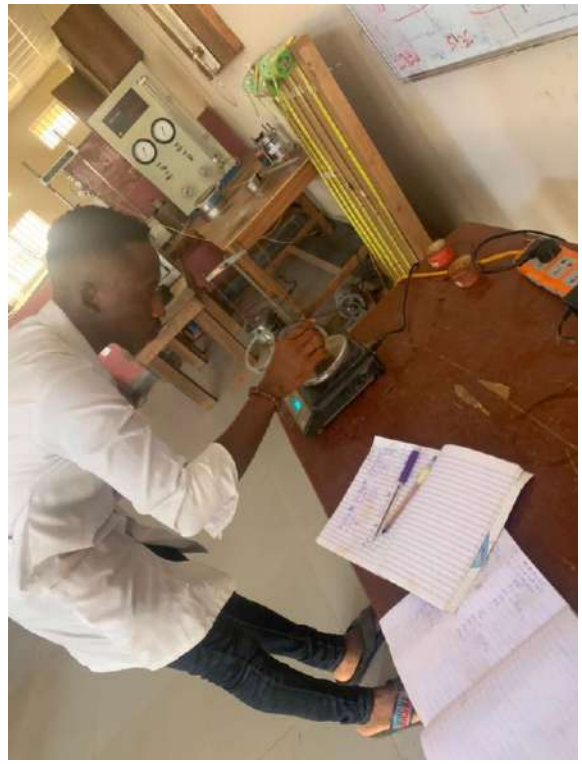


Plate 3.5 Practical picture 1

Plate 3.6 Practical picture 2

Plate 3.7 Practical picture 3

CHAPTER FOUR

RESULT AND DISCUSSION

The results analysis was carried out in this section to relate the properties of the soil tested and its relation to the aim of the project work. Test results to be discussed include: sieve analysis, specific gravity, compaction and volumetric shrinkage limit test.

4.1 Specific gravity

The specific gravity test was conducted on the samples in accordance with the code of practice BS-1377 Part-2, 1990. The specific gravity of the soil is defined as the ratio of weight of the soil to the weight of equal volume of water. it is used to obtain the unit weight of soil in the presence of water. For the soil samples designated as **Lat 1**, **Lat 2** and **Lat 3**, their average specific gravity values are computed in the table below.

Table 4.1 Result of Average specific gravity

Soil sample	**Av. Specific gravity**
Lat 1	2.65
Lat 2	2.72
Lat 3	2.64

The value obtained for **Lat 1** is 2.65, **Lat 2** is 2.72 and **Lat 3** is 2.64. These values fall within the specific gravity range of lateritic soil (i.e. from 2.65 to 2.75). This suggests the presence of clay or silt, which can be of advantage at the sub-grade and sub-base level of road construction. According to the Federal Ministry of work Standard Specification for roads and bridges (1997) a good sub-grade material should have specific gravity value ranging from 2.5 to 2.75. The values obtained for the three samples as shown in table 4.0 suggest that the soil samples satisfy this requirement.

4.2 Particle size distribution (sieve analysis)

The sieve analysis conducted on clay sample was in accordance with the code of practice BS-1377 Part-2, 1990. Figure 4.0 is the logarithmic plot of the particle size distribution of the soil samples designated as **Lat 1**, **Lat 2** and **Lat 3**. From the graph, the coefficient of uniformity (Cu) of **Lat 1** is 0 and the coefficient of curvature (Cc) is 0.1, the percent passing Sieve No 200 (0.075mm) is 13.97, percent passing sieve No. 40 (0.4225mm) is 65.34 and percent passing sieve No.10 (2mm) is 98.39. According to AASHTO M 145, the soil is classified as A-3 since more than 11.35% is retained on Sieve No 200 (0.075mm). For **Lat 2**, the value of (Cc) is 0.061 and (Cu) is 0, the percent passing sieve No. 10 is 99.97, percent passing sieve NO. 40 is 72.36 and percent passing sieve No.200 is 25.13. According to AASHTO M 145, the soil is classified under A-2-4 since more than 11.07% is retained on Sieve No 200 (0.075mm). According to ASTM D-2487, the soil is classified as poorly graded. For **Lat 3**, the value of (Cc) is 0.36 and (Cu) is 0, the percent passing sieve No. 10 is 99.35, percent passing sieve NO. 40 is 64.77 and percent passing sieve No.200 is 20.68. According to AASHTO M 145, the soil is classified under A-2-4 since more than 4.53% is retained on Sieve No 200 (0.075mm).

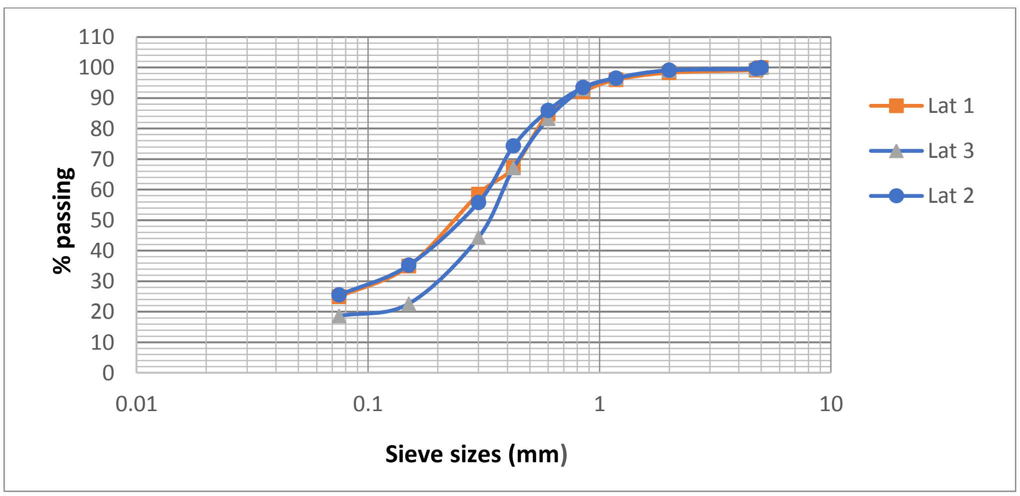


Fig 4.1 combined sieve analysis graph

4.3 Consistency Limit Test

Testing of Atterberg limits is performed only on the soil fraction passing through a No. 40 sieve, according to ASTM D4318-00 (ASTM,2003). Consistency limits is a basic measure of the critical water content of fine-grained soils. These **tests** include shrinkage limit, plastic limit, and liquid limit, which are outlined in ASTM D4318. The liquid limit, plastic limit and plasticity index values of **Lat 1**, **Lat 2** and **Lat 3** are presented in table 4.2 below.

Table 4.2 consistency limit result

Sample	Liquid limit (LL) (%)	Plastic limit (PL) (%)	Plasticity index (PI) (%)
Lat 1	28.8	16.94	11.86
Lat 2	32.4	21.98	10.42
Lat 3	30.4	21.29	9.11

From the above, the plasticity index values of **Lat 1**, **Lat 2** and **Lat 3** fall within 7-17% PI therefore, according to ASTM D 4318 classification, the three samples are said to be medium – plastic which is an indication that there is little amount of clay content in the lateritic soil samples.

4.4 Classification of soil samples

The table below summarizes the index properties of the **Lat 1**, **Lat 2** and **Lat 3**.

Table 4.3 index properties of Lat 1, Lat 2 and Lat 3

Properties	Lat 1	Lat 2	Lat 3
Specific gravity (Gs)	2.65	2.72	2.64
Liquid limit (%)	28.8	32.4	30.4
Plastic limit (%)	16.94	21.98	21.29
Plasticity index (%)	11.86	10.42	9.11
Plasticity	medium	medium	medium
% passing sieve No. 10	98.39	99.97	99.35
% passing sieve No. 40	65.34	72.36	64.77
% passing sieve No. 200	13.97	25.13	20.68
Uniformity coefficient (Cu)	0	0	0
Coefficient of curvature (Cc)	0.1	0.061	0.36
AASHTO classification	A-3	A-2-4	A-2-4
USCS classification	SC	SC	SC

4.5 Compaction Results

The method used for this test is in accordance with the code of practice BS-1377 part 4-1990.

Table 4.4 Compaction Test Result of Lat 1, Lat 2 and Lat 3 for BSL and BSH

	British Standard Light		British standard Heavy	
Sample	Max. dry unit weight (kN/m^3)	Optimum moisture content (%)	Max. dry unit weight (kN/m^3)	Optimum moisture content (%)
Lat 1	19.82	6.25	20.2	4.25
Lat 2	19.82	6	20	6.6
Lat 3	18.3	7.8	19.45	8.1

The BSL and BSH compaction test result of **Lat 1**, **Lat 2** and **Lat 3** are presented in the table above. For BSL, the MDUW and OMC obtained for **Lat 1**, **Lat 2** and **Lat 3** are 19.82kN/m^3, 20 kN/m^3 and 18.3kN/m^3 respectively with corresponding 6.25%, 6% and 7.8% OMC values. While for BSH, the MDUW obtained were 20.2kN/m^3, 19.82kN/m^3 and 19.45kN/m^3 at 4.25%, 6.6%

and 8.1% OMC respectively. From the graphs shown below, it compares the MDUW of BSL and BSH of each soil sample. It is observed that for most lateritic soil as the compactive effort increases, there is an increase in the MDUW. This is in agreement with most previous works done in this subject of compaction (Mauzu, 2007; Kumar and Sharma, 2004).

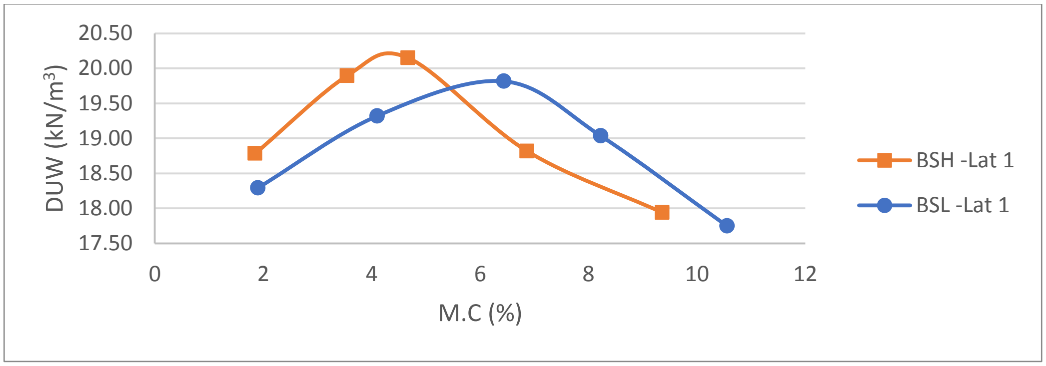


Fig 4.2 Compaction curve for BSL and BSH o Lat 1

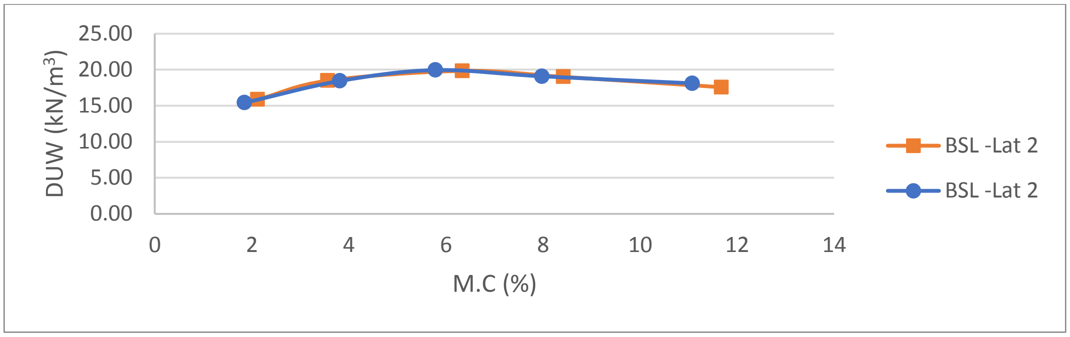


Fig 4.3 Compaction curve for BSL and BSH o Lat 2

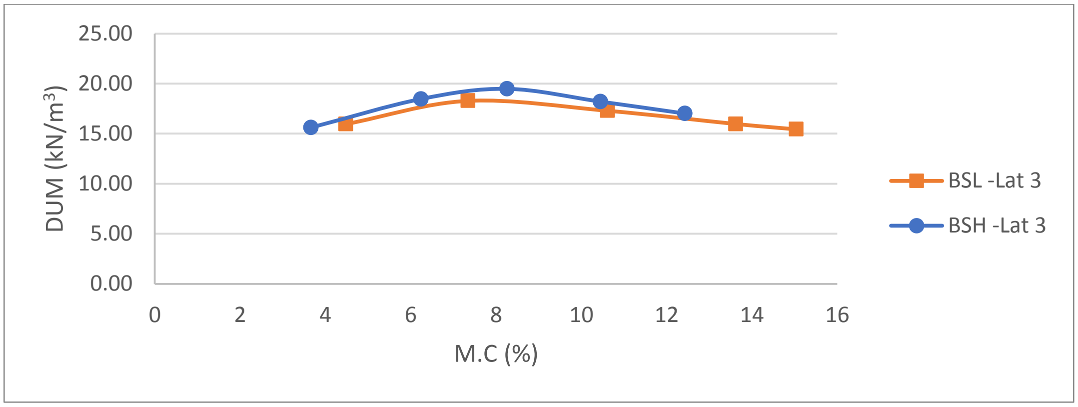


Fig 4.4 Compaction curve for BSL and BSH of Lat 3

4.6 Volumetric Shrinkage Result

The volumetric shrinkage of compacted soil liner was in accordance with the code of practice BS-1377 Part-2, 1990. This test is to investigate the effect of time on volumetric shrinkage strain of **Lat 1**, **Lat 2** and **Lat 3.** The samples were compacted at +2, +4 and -2, -4 % values of OMC obtained from BSL and BSH compactions. The samples compacted at moulding water content of -2, -4 % values of OMC could not be extruded as a result of the dryness of the specimen. Therefore, analysis was not made on the dry side of the optimum moisture content.

Plate 4.1 Picture showing the scattering of compacted specimen at dry side of OMC

4.6.1 Effect of time on Volumetric Shrinkage Strain

The volumetric shrinkage values of **Lat 1**, **Lat 2** and **Lat 3** at +2%, +4% of OMC are presented in the tables below.

Table 4.5 Volumetric shrinkage table of Lat 1

Lat 1	Volumetric shrinkage (%)	
Time (days)	+2% MWC	+4% MWC
0	0	0
7	3.8	6.6
14	4.9	7.8
21	5.4	8.2

Table 4.6 Volumetric shrinkage table of Lat 2

Lat 2	Volumetric shrinkage (%)	
Time (days)	**+2% MWC**	**+4% MWC**
0	0	0
7	3.2	6.1
14	5.3	8
21	6.1	8.2

Table 4.7 Volumetric shrinkage table of Lat 3

Lat 3	Volumetric shrinkage (%)	
Time (days)	**+2% MWC**	**+4% MWC**
0	0	0
7	3.34	5.7
14	5.14	6.8
21	5.94	7.2

The changes in volumetric shrinkage strain (VSS) with time at different Moulding water content (MWC) for **Lat 1**, **Lat 2** and **Lat 3** are shown in fig 4.5, 4.6 and 4.7 below. There is sharp increase in VSS within the first 7 days followed by the 14 days of drying in all gradations. Thereafter, slight increase is observed at 21 days drying. Volumetric shrinkage strain is proportional to moulding water content i.e the higher the moulding water content, the higher the volumetric shrinkage strain. The laterite soils compacted at moulding water content on the wet side of the OMC achieve a low hydraulic conductivity in the soil liners (Amadi, 2012). It also observed that the hydraulic integrity of fine-grained soils is often lost due to desiccation cracking during their designed expectancy span.

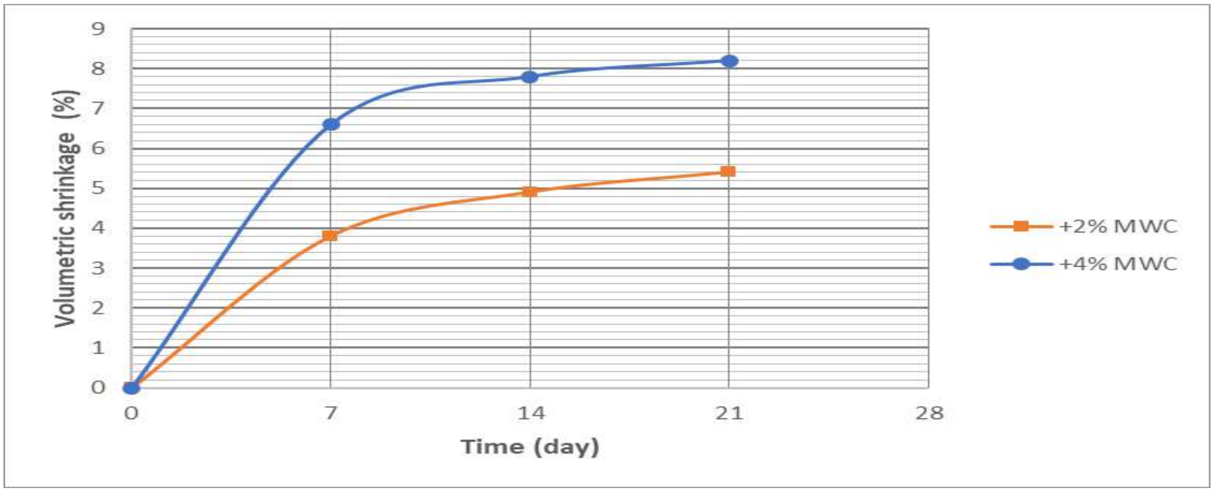


Fig 4.5, Volumetric Shrinkage Graph of Lat 1

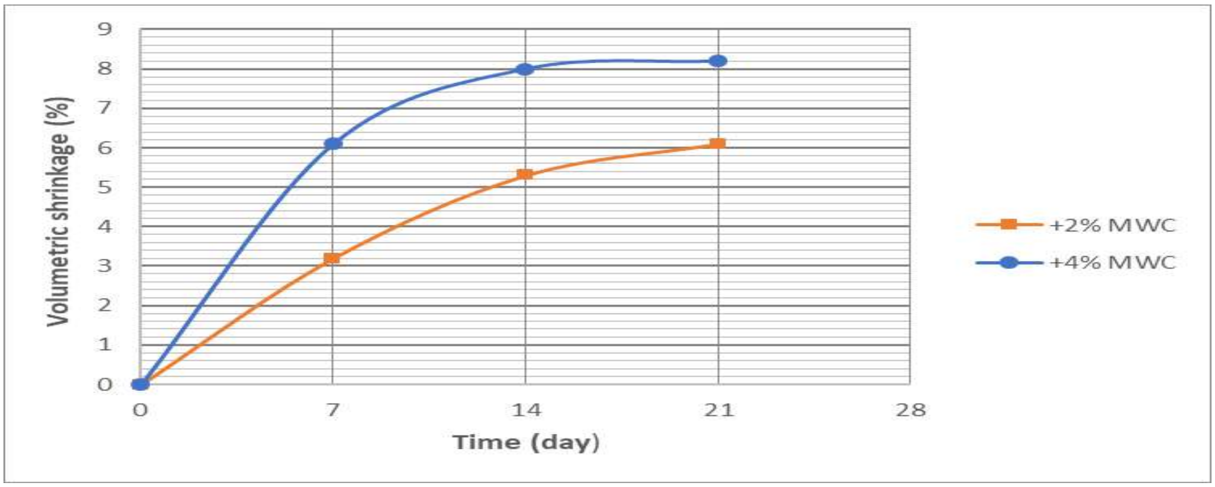


Fig 4.6 Volumetric Shrinkage Graph of Lat 2

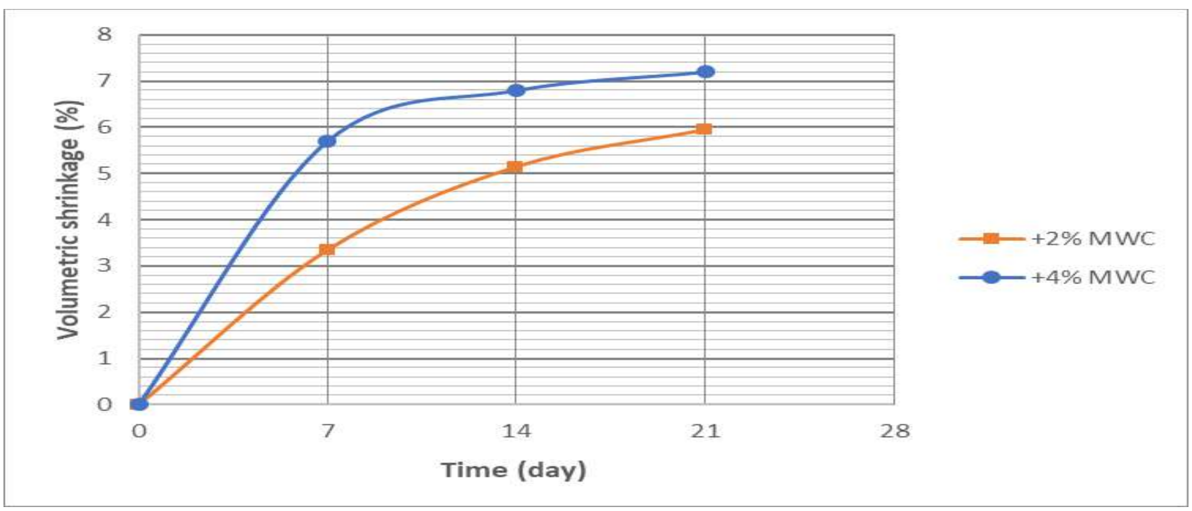


Fig 4.7 Volumetric Shrinkage Graph of Lat 3

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion.

The compaction and strength characteristics of the three lateritic soil samples collected around the geographical location of Okpuno, Nawfia and Ugwuoba borrow pits in Anambra and Enugu state respectively have been investigated in this study. Graphical analyses show that the geotechnical properties of these soils are sensitive to variations in compactive effort, compaction water content, as well as to dry unit weight. Atterberg limit, specific gravity, particle size and volumetric shrinkage are properly investigated. The particle distribution analysis shows that less than 35% particles passed BS No. 200. Therefore, it can be affirmed that the entire samples are suitable as base and sub base materials. Specific gravity value for the lateritic soil samples range between 2.65 up to 2.75. It implies that the soil can be used as sub-base in road construction even when the soils were not added any admixture. The results of the investigation carried out show that the soil samples are better classified as inorganic clays or inorganic silt of medium plasticity.

The compaction characteristics of the soil samples show that **Lat 1**, **Lat 2** and **Lat 3** attain maximum dry unit weight at 6.25%, 6% and 7.8% respectively for BSL compaction and 4.25%, 6.6% and 8.1% for BSH compaction. From the graphical illustration, it can be deduced that for most lateritic soil that as the compactive effort increases, the MDUW will also increase. The volumetric shrinkage properties of the samples shown that volumetric shrinkage strain varies directly to the moulding water content and that lateritic soil compacted at wet side of the OMC achieves low hydraulic conductivity in the soil liner. Hairy cracks were noticed around the compacted specimen and the cracks were observed to be increasing with shrinkage time.

5.2 Recommendation

This work is recommended for any geotechnical investigation that will be carried out on lateritic soils. Reference can be sited from this work for any future investigation in any of Ugwuoba, Okpuno and Nawfia borrow pits. Soil samples can be collected at various locations to compare the geotechnical characteristics of the samples at various depths because of the anisotropic nature of soil.

Further studies can consider increasing the initial water content used in compacting the soil to understand its effect on the dessication induced volumetric shrinkage properties of the soil.

Increasing moulding water content is suggested for further studies.

Finally, higher institutions should equip their soil laboratories in order to enable students carry out laboratory test with relative ease and accuracy.

REFERENCES

Abu H. A., (1993) Desiccation theory for soft cohesive soils, PhD. Thesis, University of Colorado, Boulder.

Akpokodje E.G, (2001) Introduction to Engineering Geology, Pam Publishing Coy Ltd, Port Harcourt, pp. 239- 246

Albrecht, B. A., Benson, C. H. (2001). Effect of desiccation on compacted natural clays. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(1), 67–75.

Alhassan, M. Alhaji, M. M. (2017) Utilization of Rice Husk Ash for Improvement of Deficient Soils in Nigeria: A Review, *Nigerian Journal of Technology*, Vol. 36, No. 2, pp. 386 – 394.

Allaire, S. E., Roulier, S. Cessan, A. (2009). Quantifying preferential flow in soils: A review of different techniques. *J. Hydrol.* 378: 179204

Amadi, A. N., Eze, C. J., Igwe, C. O., Okunlola, I. A., Okoye, N. O., (2012). Architect's and Geologist's view on the causes of building failures in Nigeria *Modern Applied Science*, 6 (6), 31-38.

American Association of State Highway And Transport Officials (AASHTO). (2011). ASH TOM14765. Materials for aggregate and Soil Aggregate subbase, base, and surface courses. In Standard Specifications for Transportation materials and methods of sampling and testing .

Auvray, R., Rosin-Paumier, S., Abdallah, A., Masrouri, F (2014) Quantification of soft soil cracking during suction cycles by image processing, *European Journal of Environmental and Civil Engineering*, vol. 18, no. 1, pp. 11–32.

Bachar, M., Azzouz, L., Rabehi, M., Mezghiche, B., (2014). Characterization of a stabilized earth concrete and the effect of incorporation of aggregates of cork on its thermo-mechanical properties: experimental study and modeling. Constr. Build. Mater. http://dx.doi.org/10.1016/j.conbuildmat.2014.09.106.

Bourman, R.P., Ollier, C.D., (2002). A critique of the Schellmann definition and classification of laterite. Catena. http://dx.doi.org/10.1016/S0341-8162(01)00178-3.

Bourman, R.P., Ollier, C.D., (2003). Reply to the Discussion of A critique of the Schellmann definition and classification of laterite. Catena 52, 81-83. http://dx.doi.org/10.1016/S03418162(02)00180-7.

Buchanan F., (1807). - A journey from Madras through the countries of Mysore, Canara and Malabar, Vol. 2, 436-60, Rast-India Co. London.

Chertkov, V. Y (2000) Modeling the pore structure and shrinkage curve of soil clay matrix, *Geoderma*, vol. 95, no. 3-4, pp. 215–246.

Chertkov, V. Y (2002) Modelling cracking stages of saturated soils as they dry and shrink, *European Journal of Soil Science*, vol. 53, no. 1, pp. 105–118.

Chertkov, V.Y. (2000). Using surface crack spacing to predict crack network geometry in swelling soils. *Soil Sci. Soc. Am. J.* 64, 1918-1921

Coka, E. (2001) Use of class C fly ashes for the stabilization of an expansive soil. *J. Geotech. Geoenviron Engr*., 127 (7):568 - 573.

Cordero, J. A., Useche, V., Prat, P. C., Ledesma, A., Santamarina, J. C.(2017) Soil desiccation cracks as a suctioncontraction process, *G´eotechnique Letters*, vol. 7, no. 4, pp. 272–278.

Costa, S., Kodikara, J., Shannon, B (2013) Salient factors controlling desiccation cracking of clay in laboratory experiments, *G´eotechnique*, vol. 63, no. 1, pp. 18–29.

Coussy O., Eymard R., Lassabatère T., (1998) Constitutive modelling of unsaturated drying deformable media, *J. Eng. Mech*. ASCE, vol. 124, 6, p. 658-667

Daniel, D. E., Wu, Y. K. (1993). Compacted clay liners and covers for arid sites. *Journal of Geotechnical Engineering*, 119(2), 223–237

Deng G., Shen, Z. J. (2006) Numerical simulation of crack formation process in clays during drying and wetting, *Geomechanics and Geoengineering*, vol. 1, no. 1, pp. 27–41.

Eberemu, A. O., Amadi, A. A., Sule, J. (2011). Desiccation Effect on Compacted Tropical Clay Treated with Rice Husk Ash. Retrieved from http://link.aip.org/link/ascecp/v397/i41165/p122/s1

Emofurieta, W.O., Salami, A.O., (1993). A comparative South-western, study on two Kaolin deposits in Nigerian. *J. Min. Geol*. 24, 15-27

Fookes, P.G. (1997). Tropical residual soils. Geol. Soc. Eng Group Working Party Revised re port, *Geol. Soc., London* 184 pp.

Frempong, E. M., Yanful, E. K. (2008) Interaction between three tropical soils and municipal solid waste landfill leachate. *J. Geotech and Goenv. Engrg*. Vol. 134. No.3, pp 379–396.

Gidigasu, M.D (2000) Review of Identification of Problem Lateritic Soils in Highway Engineering. Transport Research Board Washington 497, 96-111.

Giorgis, I., Bonetto, S., Giustetto, R., Lawane, A., Pantet, A., Rossetti, P., Thomassin, J.H., Vinai, R., (2014). The lateritic profile of Balkouin, Burkina Faso: geochemistry, mineralogy and genesis. *J. Afr. Earth Sci.* http://dx.doi.org/10.1016/j.jafrearsci.2013.11.006.

Gonzalez de Vallejo, L.I., Ferrer, M., (2011). Geological Engineering. CRC Press/Balkema, Leiden, The Netherlands.

Gui Y., Zhao, G.-F.(2015) Modelling of laboratory soil desiccation cracking using DLSM with a two-phase bond model, *Computers and Geotechnics*, vol. 69, pp. 578–587.

Hallaire, V. (1994) Description of microcrack orientation in aclayey soil using image analysis,"in Soil Micromorphology: Studies in Management and Genesis. Developments in Soil Science ,A.J. Ringrose-oase and G. S. Humphreys, Eds., vol. 22, pp. 549– 557, Elsevier, Amsterdam, Netherland.

Hobart, k. (2015) Expansive Soil and Expansive Clay: The hidden force behind basement and foundation problems. *Geology*.

Hueckel T., (1992) on effective stress concept and deformation in clays subjected to environmental loads: Discussion, *Canadian Geotechnical Journal*, vol. 29, p. 1120-1125.

Islam, M.S., Salokhe, V.M., Gupta, C.P., Hoki, M. (1994). Effects of PTO Powered Disk Tilling on Some Physical Properties of Bangkok Clay Soil. *Soil and Tillage Research* 32, 93-104.

Kasthurba, A.K., Santhanam, M., Mathews, M.S., (2007). Investigation of laterite stones for building purpose from Malabar region, Kerala state, SW India Part 1: field studies and profile characterisation. *Constr. Build. Mater* 21, 73-82.

Kim T., Hwang C., (2003) Modelling of tensile strength on moist granular earth material at low water content, *Engineering Geology*, vol. 69, p. 233-244

Ko, T. H., Chu, H., Lin, H. P., Peng, C Y (2006) Red soil as a regenerable sorbent for high temperature removal of hydrogen sulfide from coal gas, *Journal of Hazardous Materials*,vol.136, no.3,pp.776–783.

Ko, T. H. (2008) Removal of hydrogen sulfur from coal-derived gas by iron oxides in various oxisols, *Environmental Engineering Science*, vol.25,no.7, pp. 969–973.

Ko, T.H., (2014). Nature and properties of lateritic soils derived from different parent materials in Taiwan. *Sci. World J*. http://dx.doi.org/10.1155/2014/247194.

Kodikara, J. K., Barbour, S. L., Fredlund, D. G. (2000). Desiccation cracking of soil layers. *Unsaturated Soils for Asia*, 90(5809), 139

Kodikara, J., Rajeev, P.,Rhoden, N. J. (2011) Determination of thermal diffusivity of soil using infrared thermal imaging, *Canadian Geotechnical Journal*, vol.48, no. 8, pp.1295–1302.

Konrad J. M., Ayad, R. (1997) Desiccation of a sensitive clay: Field experimental observations, *Canadian Geotechnical Journal*, vol. 34, no. 6, pp. 929–942.

Konrad J.M., Ayad R., (1997) An idealized framework for the analysis of cohesive soils undergoing desiccation, *Can. Geotech. J.*, vol. 34, p. 477-488.

Kowalski S.J., (2003) Thermomechanics of Drying Processes, *Springer Verlag*.

Li, J. H., Li, L., Chen, R., Li, D. Q. (2016) Cracking and vertical preferential flow through landfill clay liners, *Engineering Geology*, vol. 206, pp. 33–41.

Li, J. H., Lu, Z., Guo, L. B., Zhang, L. M. (2017) Experimental study on soil-water characteristic curve for silty clay with desiccation cracks, *Engineering Geology*, vol. 218, pp. 70–76.

Lima L. A., Grismer, M. E (1992) Soil crack morphology and soil Salinity1, *Soil Science*, vol.153, no. 2, pp.149–153, 1992.

Mainguy M., Coussy O., Baroghel-Bouny V., (2001) Role of air pressure in drying of weakly permeable materials, *J. Eng. Mech.* ASCE, vol. 127, 6, p. 582-592

Millogo, Y., Traor_e, K., Ouedraogo, R., Kabor_e, K., Blanchart, P., Thomassin, J.H., (2008). Geotechnical, mechanical, chemical and mineralogical characterization of a lateritic gravels of Sapouy (Burkina Faso) used in road construction. *Constr. Build. Mater*. http://dx.doi.org/10.1016/j.conbuildmat.2006.07.014

Mitchell, J.K., Soga, K., (2005). Fundamentals of Soil Behavior, third ed. John Wiley & Sons, Hoboken, N.J.

Morris P.H., Graham J., Williams D.J., (1992) Cracking in drying soils, *Can. Geotech. J*., vol. 29, p. 262-277.

Moses, G., Afolayan, J. O. (2013). Desiccation-Induced Volumetric Shrinkage of Compacted Foundry Sand Treated with Cement Kiln Dust. *Geotechnical and Geological Engineering*, 31(1), 163–172.

Nahlawi H., Kodikara, J. K. Laboratory experiments on desiccation cracking of thin soil layers, *Geotechnical and Geological Engineering*, vol. 24, no. 6, pp. 1641–1664, 2006.

Netterberg, F. (1994). Low cost local road materials in southern Africa. *Geotech. Geol. Engng,* 1 2, 35- 42.

Nogami, J. S., Villibor, D. F. (1991) Use of lateritic fine grained soils in road pavement base courses. *Geotech. Geol. Eng.* 9(3-4):167-182

Nuth M., Laloui L., (2008) Effective stress concept in unsaturated soils: Clarification and validation of a unified framework, *Int. J. Numer. Anal. Met. Geomech*, vol. 32, p. 771-801

Nwankwoala, H.O., Amadi, A.N. (2013), Geotechnical Investigation of Sub-soil and Rock Characteristics in parts of ShiroroMuya-Chanchaga Area of Niger State, Nigeria. *International Journal of Earth Sciences and Engineering*, 6(1), 8 – 17.

Odumade, A. O., Ezeah, C., Ugwu, O. O. (2018) Performance analysis of cement-stabilised laterite for road construction in the tropics, *Environmental Geotechnics*. https://doi.org/10.1680/jenge.17.00026

Ojo, G.P., Igbokwe, U.G., Nwozor, K.K., Egbuachor, C. J. (2016). Geotechnical Properties of Lateritic Overburden Materials on the Charnockite and Gneiss Complexes in Ipele-Owo Area, Southwestern Nigeria *American Journal of Engineering Research* (AJER) 5(9) pp-53-59.

Oke, S,A., Okeke, O.E., Amadi, A.N., Onoduku, U.S. (2009), Geotechnical Properties of the Subsoil for Designing Shallow Foundation in some selected parts of Chanchaga area, Minna, Nigeria, *Journal of Environmental Science,* 1(1), 45 – 54.

Olanipekun, E.O., (2000). Kinetics of leaching laterite. *Int. J. Min. Process*. http://dx.doi.org/10.1016/S0301-7516(99)00067-8

Omotoso, I. (2010). An Investigation into the geotechnical and engineering properties of some laterites of Eastern Nigeria *Journal of Nigeria Mining Geology and Metallurgical Society*, 1:10-110.

Osinubi, K. J., Eberemu, A. O. (2010) Desiccation induced shrinkage of compacted lateritic soil treated with blast furnace slag. *Geotechnical and Geological Engineering*, 28:537-547. https://doi.org/10.1007/s10706-010-9308-6

Osinubi, K. J., Eberemu, A. O., Adzegah, B. (2012) Effect of Fines Content on the Engineering Properties of Reconstituted Lateritic Soils in Waste Containment Application *Nigerian Journal of Technology*, Vol. 31, No.3, , pp 277–287

Osinubi, K. J., Nwaiwu, C. M. O. (2008). Desiccation-induced shrinkage in compacted lateritic soils. *Geotechnical and Geological Engineering*, 26(5), 603–611.

Osinubi, K. J., Nwaiwu, C. M.O. (2006). Design of compacted lateritic soil liners and covers. *Journal of Geotechnical and Geoenvironmental Engineering*, 132(2), 203–213.

Osinubi, K.J., Eberemu, A. O., Amadi, A. A. (2009) Compacted lateritic soil treated with blast furnaces slag as hydraulic barriers in waste containment. *Int. J. Risk Assessment and Management*, 13 (2):177 - 188. Osinubi, K.J., Kundiri, A.M. (2008). Hydraulic Conductivity of Compacted Clayey Sand”. Proceedings of Bi-Monthly meetings/Workshops. Materials Society of Nigeria (MSN), Zaria Chapter.

OSullivan, D., Quigley, P. (2002). Geotechnical engineering and environmental aspects of clay liners for landfill projects, Fehily Timoney & Co. and Irish Geotechnical Services Ltd. Institution of Engineers of Ireland.

Oyediran, I.A., Okosun, J., (2013). An attempt to improve geotechnical properties of some highway lateritic soils with lime. 60, 287-296.

Paigegreen, P (1999). Materials for and construction of sealed low volume roads. Transpor tation Research Record, 1652, TRB, Washington, DC, 1999, Vol 2, pp10- 15.

Paigegreen, P (2007). Improved material specifications for unsealed roads. Quarterly Journal *o f Engineering Geology and Hydrogeology*, Vol 40: 2, 175- 179.

Pasricha, K., Wad, U., Pasricha, R., Ogale, S. (2009) Parametric dependence studies on cracking of clay, Physical: Statistical Mechanics and its Applications,vol.388(8),pp.1352–1358.

Peng, X., Horn, R., Peth, S., Smucker, A. (2006) Quantification of soil shrinkage in 2D by digital image processing of soil surface, *Soil and Tillage Research*, vol. 91, no. 1-2, pp. 173–180.

Peron, H., Hueckel, T., Laloui, L., Hu, L. B (2009) Fundamentals of desiccation cracking of finegrained soils: Experimental characterisation and mechanisms identification, *Canadian Geotechnical Journal,* vol. 46, no. 10, pp. 1177–1201.

Philip, L. K., Shimell, H., Hewitt, P. J., Ellard, H. T (2002) Afieldbased test cell examining clay desiccation in landfill liners, *Quarterly Journal of Engineering Geology and Hydrogeology*, vol. 35, no. 4, pp. 345–354.

Pinard, M. I., Netterberg, F (2012) Comparison of trst methods and implications on materials selection for road condtruction. 2nd AFCAP practioners' conference, Maputo.

Pinard, M. I (2011) Performance Review of Design Standards and Technical Specifications for Low Volume Sealed Roads in Malawi, Lilongwe, Malawi: Roads Authority (Final Report AFCAP Project MAL/016).

Qasim, M., Bashir, A., Tanvir, M., Anees, M. M. (2014) Effect of Rice Husk Ash on Soil Stabilization, *Bulletin of Energy Economics*, 3(1), 10-17

Rayhani, M.H., Yanful, T., Fakher A (2007). Desiccation-induced cracking and its effect on the hydraulic conductivity of clayey soils from Iran Can. *Geotech. J,* 44 (3), pp. 276– 283 http://dx.doi.org/10.1139/T06-125.

Rigassi, V., (1995). Compressed earth blocks: a publication of Deutsches Zentrum für Entwicklungs technologien - GATE, a division of the Deutsche Gesellschaft für Technische

Safiuddin L.O., Heris V., Wirman R.P., Bijaksana S., (2011). A Preliminary Study on the Magnetic Properties on Laterite Soils as Indicators of Pedogenic Processes. *Latinmag Letters*, 1(1), 1-15.

Schellmann, W., (2003). Discussion of "A critique of the Schellmann definition and classification of laterite". In: Bourman, R.P., Ollier, C.D. (Eds.), (Catena 47, 117e131). CATENA, 52, pp. 77-79. http://dx.doi.org/10.1016/S0341-8162(02) 00178-9.

Sharp, K., Vuong, B., Rollings, R., Baran, E., Foley, G., Johnsonclarke, J, Metcalf J. (2001). An evaluation of the field and laboratory properties of lateritic gravels. Research Report ARR 343. ARRB Transport Research. Ltd. 65pp.

Shaw, J. N. (2001) Iron and aluminum oxide characterization for highly-weathered Alabama ultisols, *Communications in Soil Science andPlantAnalysis*, vol.32,no.1-2,pp.49–64.

Sunil, B.M., Shrihari, S., Sitaram, N. (2009). Shear strength characteristics and chemical characteristics of leachate-contaminated lateritic soil. *Engineering Geology*, 106, (1-2): 2025.

Taha, M. R., Taha, O. M. (2012). Influence of Nano-material on the expansive and shrinkage soil behavior. *Journal of Nanoparticle Research*, 14(10), 1–13.

Taha, M.R, Desa, H.M., Kabir, H. (2002). The Use of a Granite Residual Soils as a Landfill Liner Material. Proceedings 2nd World Engineering Congress, 264-267. Sarawak, Malaysia.

Taha, M.R., Hossain, M.K., Mofiz, S.A. (2000). Behaviour and modeling of granite residual soil in direct shear test. *Journal of Institution of Engineers Malaysia*. 61(2); 27- 40.

Tang, C. S., Cui, Y. J., Shi, B., Tang, A.M., Liu, C. (2011). Desiccation and cracking behaviour of clay layer from slurry state under wetting drying cycles. *Geoderma*, 166(1), 111–118

Tang, C. S., Shi, B., Cui, Y. J., Liu, C., Gu, K (2012) Desiccation cracking behavior of polypropylene fiber-reinforced clayey soil, *Canadian Geotechnical Journal*, vol. 49, no. 9, pp. 1088–1101.

Tang, C. S., Shi, B., Liu, C., Suo, W. B., Gao, L. (2011) Experimental characterization of shrinkage and desiccation cracking in thin clay layer, *Applied Clay Science*, 52(1-2), pp. 69– 77.

Tang, H., Dong, V., Wang, T., Dong, Y. (2019) Simulation of strain localization with discrete element-Cosserat continuum finite element two scale method for granular materials, *Journal of the Mechanics and Physics of Solids,* vol. 122, pp. 450–471.

Tang, S. C., Cui, Y.J., Tang, A. M., Shi, B. (2010) Experiment evidence on the temperature dependence of desiccation cracking behavior of clayey soils, *Engineering Geology*, vol. 114, no. 3-4, pp. 261–266.

Tang, S. C., Wang, D. Y., Zhu, C., Zhou, Q.Y., Xu, S. K., Shi, B. (2018) Characterizing dryinginduced clayey soil desiccation cracking process using electrical resistivity method, *Applied Clay Science*, vol. 152, pp. 101–112.

Tay,Y. Y., Stewart, D. I., Cousens, T. W. (2001) Shrinkage and desiccation cracking in bentonitesand landfill liners, *Engineering Geology*, vol. 60, no. 1–4, pp. 263–274.

Thorne, R., Roberts, S., Herrington, R., (2012). The formation and evolution of the Bitincke nickel laterite deposit, Albania. Min. Depos. http://dx.doi.org/10.1007/ s00126-012-0411-x.

Villar, M. V., Gomez, R., Lloret, A, (2010) Experimental investigation into temperature effect on hydro-mechanical behaviours of bentonite, *Journal of Rock Mechanics and Geotechnical Engineering*, vol. 2, no. 1, pp. 71–78.

Wang, D, Y., Tang, C.S., Shi, B., Li, J (2016) Studying the effect of drying on soil hydromechanical properties using micro penetration method, *Environmental Earth Sciences*, vol. 75, no. 12, pp. 1–13.

Wang, L. L., Tang, C. S., Shi, B., Cui, Y.J., Zhang, G. Q., Hilary, I (2018) Nucleation and propagation mechanisms of soil desiccation cracks, *Engineering Geology*, vol. 238, pp. 27– 35.

Waziri, B., Shehu, L., Alhaji, Mala, M, (2013). Properties of compressed stabilized earth blocks (CSEB) for low- cost housing construction: a preliminary investigation. *Int. J. Sustain. Constr. Eng. Technol.* 4, 2180-3242

Wei, X., Hattab, M., Bompard, P., Fleureau, J. M. (2016) Highlighting some mechanisms of crack formation and propagation in clays on drying path, *G´eotechnique*, vol. 66, no. 4, pp. 287– 300.

Widdowson, M (2003). Ferricrete. In: Goudie, A. S. ed. Encyclopedia of Geomorphology, V olume 1. London: Routledge, pp. 365–367.

Xu, L., Coop, M. R (2016) Influence of structure on the behavior of a saturated clayey loess, *Canadian Geotechnical Journal*, vol. 53, no. 6, pp. 1026–1037.

Yesiller, N., Miller, C. J., Inci, G., Yaldo, K. (2000). Desiccation and cracking behavior of three compacted landfill liner soils. *Engineering Geology*, 57(1), 105–121.

Zolfaghari, Z., Mosaddemhi, M.R., Ayobi, S., Kelishadi, H. (2015). Soil Atterberg limits and consistency indices as influenced by land use and slope position in Western Iran. *J. Mt. Sci*. 12(6): 1471- 1483.

APPENDIX 1

Specific gravity

Lat 1

SPECIFIC GRAVITY TEST

Density Bottle	(W1) Wt. of Bottle (g)	(W2) Bottle + dry Soil(g)	(W3) Bottle + sat. Soil (g)	(W4) Bottle + Water (g)	Sp. Gravity	Av. Sp. Gravity
S1	24.73	39.44	88.93	79.84	2.62	2.65
S2	24.11	39.56	88.53	78.87	2.67	
S3	27.62	40.48	90.12	82.08	2.67	

Lat 2

SPECIFIC GRAVITY TEST

Density Bottle	(W1) Wt. of Bottle (g)	(W2) Bottle + dry Soil(g)	(W3) Bottle + sat. Soil (g)	(W4) Bottle + Water (g)	Sp. Gravity	Av. Sp. Gravity
S1	24.13	42.46	90	78.43	2.71	2.72
S2	24.74	39.34	88.4	79.2	2.70	
S3	27.57	39.65	89.4	81.74	2.73	

Lat 3

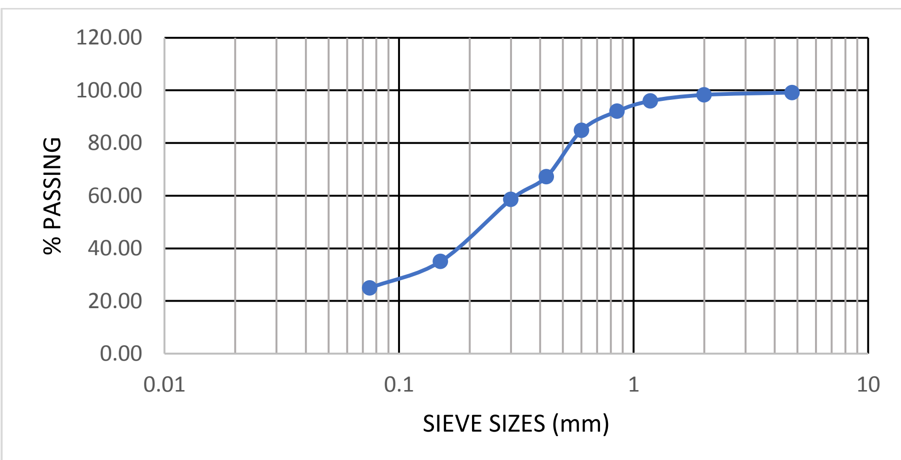
SPECIFIC GRAVITY TEST

Density Bottle	(W1) Wt. of Bottle (g)	(W2) Bottle + dry Soil(g)	(W3) Bottle + sat. Soil (g)	(W4) Bottle + Water (g)	Sp. Gravity	Av. Sp. Gravity
S1	25.1	41	89.68	79.84	2.62	2.64
S2	24.82	39.56	87.46	78.27	2.66	
S3	27.73	36.92	88.1	82.4	2.63	

Particle size distribution

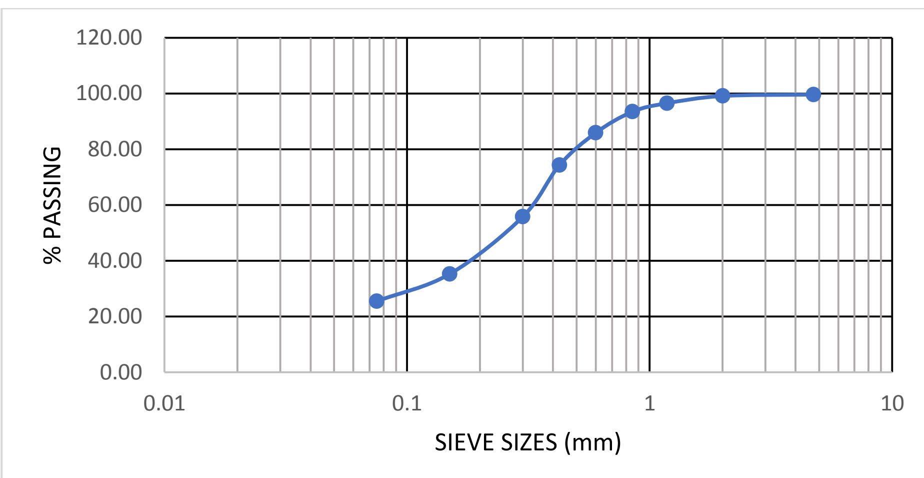
Lat 1

sieve sizes (mm)	Mass Retained (g)	Quantity passing (g)	% Retained	Cum % Retained	% passing
5	0	500	0	0	100
4.75	4.14	495.86	0.83	0.49	99.17
2	4.45	491.41	0.89	1.38	98.28
1.18	11.63	479.78	2.33	3.71	95.96
0.85	19.56	460.22	3.91	7.62	92.04
0.6	36.28	423.94	7.26	14.87	84.79
0.425	88.12	335.82	17.62	32.50	67.16
0.3	43.02	292.8	8.60	41.10	58.56
0.15	117.97	174.83	23.59	64.70	34.97
0.075	50.04	124.79	10.01	74.70	24.96
Tray	2.67	122.12	0.53	75.24	24.42
	373.74				



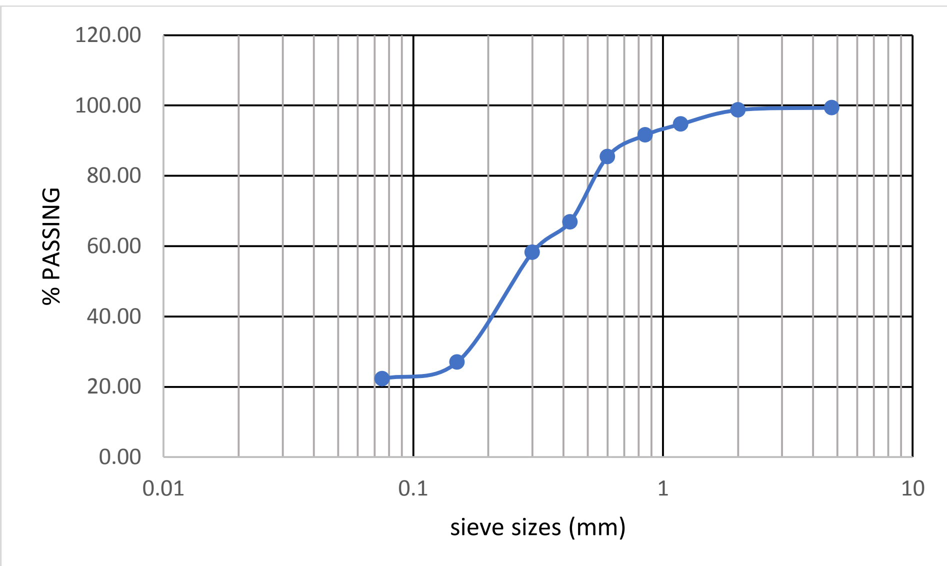
Lat 2

sieve sizes (mm)	Mass Retained (g)	Quantity passing (g)	% Retained	Cum % Retained	% passing
5	0	500	0	0	100
4.75	1.94	498.06	0.39	0.39	99.61
2	2.54	495.51	0.51	0.90	99.10
1.18	13.03	482.49	2.61	3.50	96.50
0.85	15.18	467.31	3.04	6.54	93.46
0.6	37.80	429.51	7.56	14.10	85.90
0.425	58.04	371.47	11.61	25.71	74.29
0.3	92.54	278.93	18.51	44.22	55.79
0.15	102.62	176.31	20.52	64.74	35.26
0.075	48.81	127.51	9.76	74.50	25.50
Tray	0.32	127.19	0.06	74.56	25.44
	370.87				



Lat 3

sieve sizes (mm)	Mass Retained (g)	Quantity passing (g)	% Retained	Cum % Retained	% passing
5	0	500	0	0	100
4.75	3.08	496.93	0.62	0.49	99.39
2	3.43	493.50	0.69	1.18	98.70
1.18	20.07	473.43	4.01	5.19	94.69
0.85	15.53	457.90	3.11	8.30	91.58
0.6	30.66	427.24	6.13	14.43	85.45
0.425	92.70	334.54	18.54	32.97	66.91
0.3	43.61	290.94	8.72	41.69	58.19
0.15	155.89	135.05	31.18	72.87	27.01
0.075	23.66	111.38	4.73	77.60	22.28
Tray	1.56	109.82	0.31	77.91	21.96
	390.18				



Atterberg limit test

Lat 1

ATTERBERG LIMIT TEST

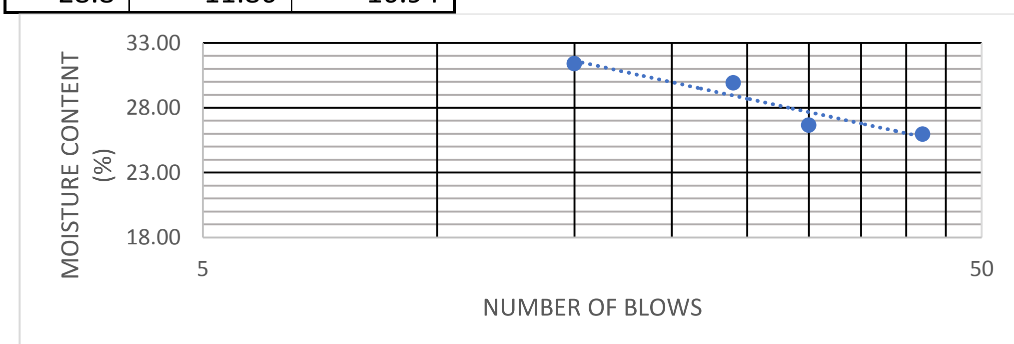
LIQUID LIMIT

No. of Blows	Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)
15	16.57	49.21	32.64	41.41	24.84	7.8	31.40
24	15.06	41.5	26.44	35.41	20.35	6.09	29.93
30	17.56	47.44	29.88	41.15	23.59	6.29	26.66
42	18.46	50.57	32.11	43.95	25.49	6.62	25.97

PLASTIC LIMIT

Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)	Av. Moisture
17.62	21.86	4.24	21.41	3.79	0.45	11.87	11.87
14.49	18.64	4.15	18.2	3.71	0.44	11.86	

LL	PL	PI
28.8	11.86	16.94



Lat 2

ATTERBERG LIMIT TEST

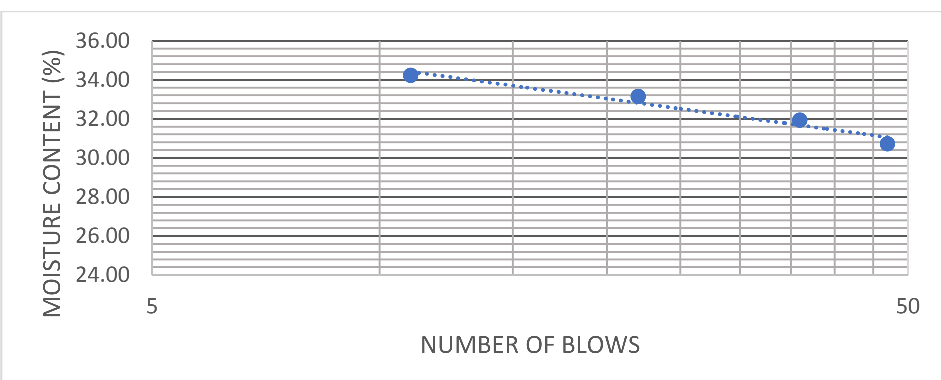
LIQUID LIMIT

No. of Blows	Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)
11	16.6	40.1	23.5	34.11	17.51	5.99	34.21
22	15.87	39.9	24.03	33.92	18.05	5.98	33.13
36	16.22	39.2	22.98	33.64	17.42	5.56	31.92
47	15.85	37.6	21.75	32.49	16.64	5.11	30.71

PLASTIC LIMIT

Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)	Av. Moisture
16.74	18.1	1.36	17.85	1.11	0.25	22.52	21.98
15.89	17.25	1.36	17.01	1.12	0.24	21.43	

LL	PL	PI
32.4	21.98	10.42



Lat 3

ATTERBERG LIMIT TEST

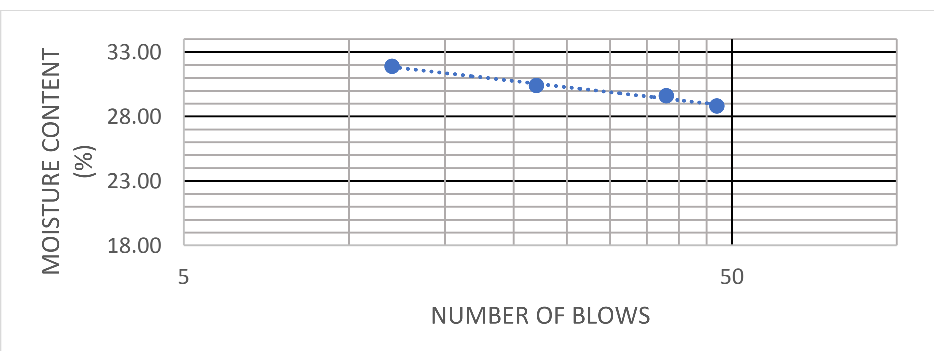
LIQUID LIMIT

No. of Blows	Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)
12	16.15	39.02	22.87	33.49	17.34	5.53	31.89
22	16.11	38.37	22.26	33.18	17.07	5.19	30.40
38	16.67	37.86	21.19	33.02	16.35	4.84	29.60
47	16	36.21	20.21	31.69	15.69	4.52	28.81

PLASTIC LIMIT

Wt.of Can (g)	Can + Wet soil (g)	Wt. of wet soil	Can + Dry soil	Wt. of Dry soil	Wt. of Water	Moisture content (%)	Av. Moisture
16.62	19.17	2.55	18.72	2.1	0.45	21.43	21.29
16.22	18.74	2.52	18.3	2.08	0.44	21.15	

LL	PL	PI
30.4	21.29	9.11

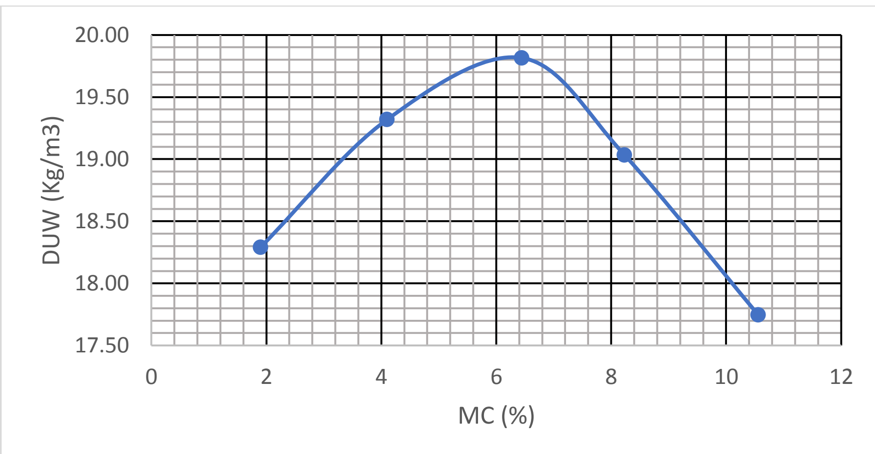


Compaction BSL and BSH

Lat 1

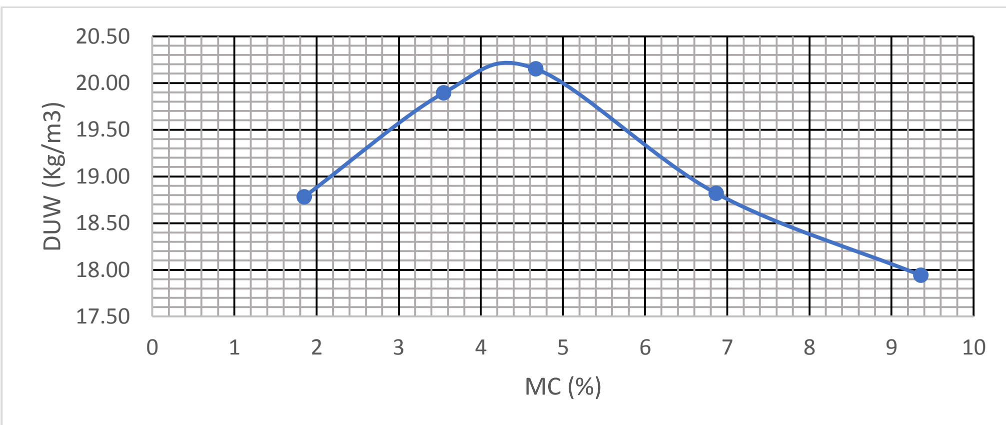
BSL

% of Water	Vol. of mould (m3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m3)	Moisture (%)	Dry density (Kg/m3)	Dry unit weight (Kg/m3)
2	0.001	2.4	4.3	1.9	1900	1.9	1864.57	18.29
4	0.001	2.4	4.45	2.05	2050	4.1	1969.26	19.32
6	0.001	2.4	4.55	2.15	2150	6.44	2019.92	19.82
8	0.001	2.4	4.5	2.1	2100	8.23	1940.31	19.03
10	0.001	2.4	4.4	2	2000	10.56	1808.97	17.75



BSH

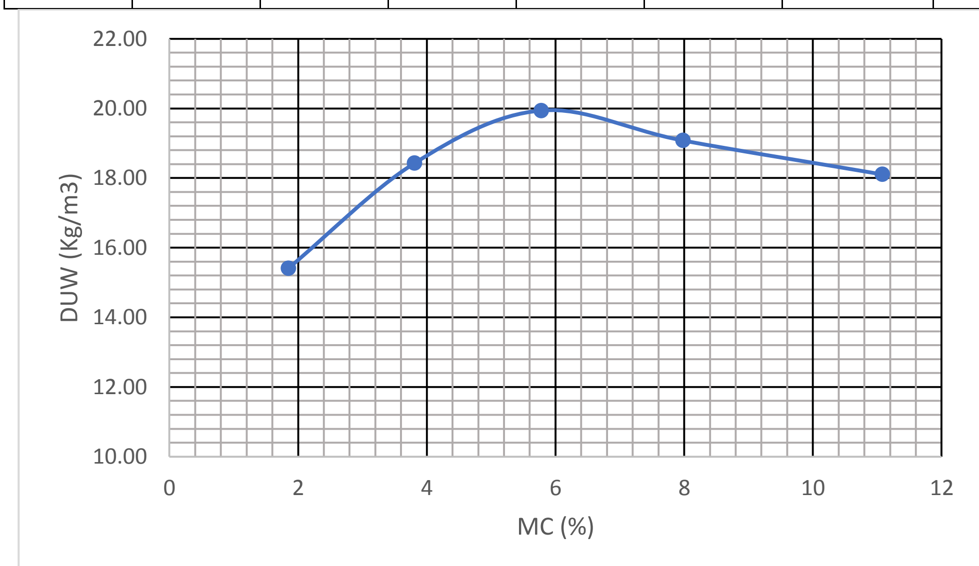
% of Water	Vol. of mould (m3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m3)	Moisture (%)	Dry density (Kg/m3)	Dry unit weight (Kg/m3)
2	0.001	4	5.95	1.95	1950	1.85	1914.6	18.78
4	0.001	4	6.1	2.1	2100	3.55	2028.0	19.89
6	0.001	4	6.15	2.15	2150	4.67	2054.1	20.15
8	0.001	4	6.05	2.05	2050	6.87	1918.3	18.82
10	0.001	4	6	2	2000	9.36	1828.8	17.94



Lat 2

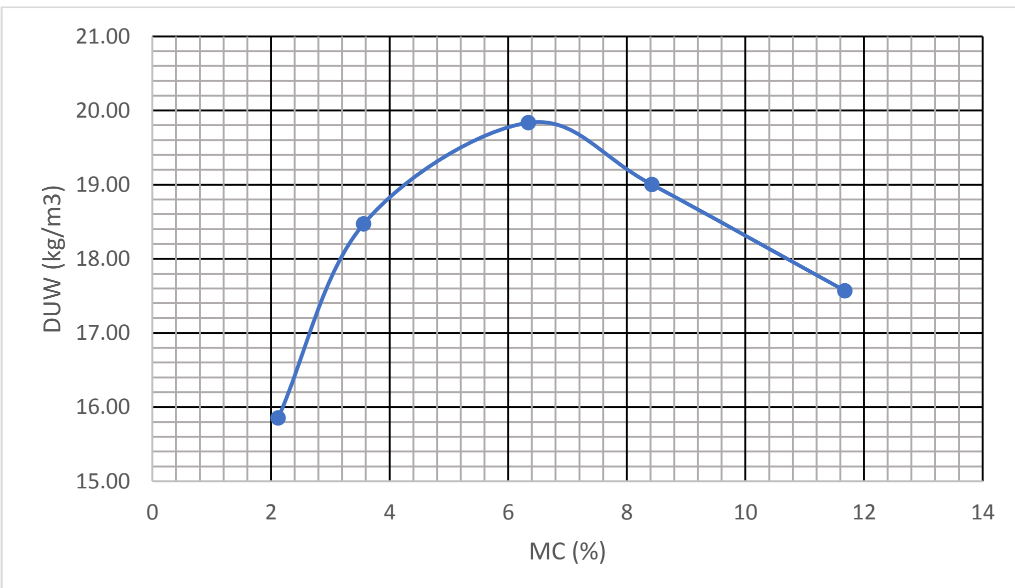
BSL

% of Water	Vol. of mould (m3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m3)	Moisture (%)	Dry density (Kg/m3)	Dry unit weight (Kg/m3)
2	0.001	2.4	4	1.6	1600	1.85	1570.94	15.41
4	0.001	2.4	4.35	1.95	1950	3.81	1878.43	18.43
6	0.001	2.4	4.55	2.15	2150	5.78	2032.52	19.94
8	0.001	2.4	4.5	2.1	2100	7.98	1944.80	19.08
10	0.001	2.4	4.45	2.05	2050	11.08	1845.52	18.10



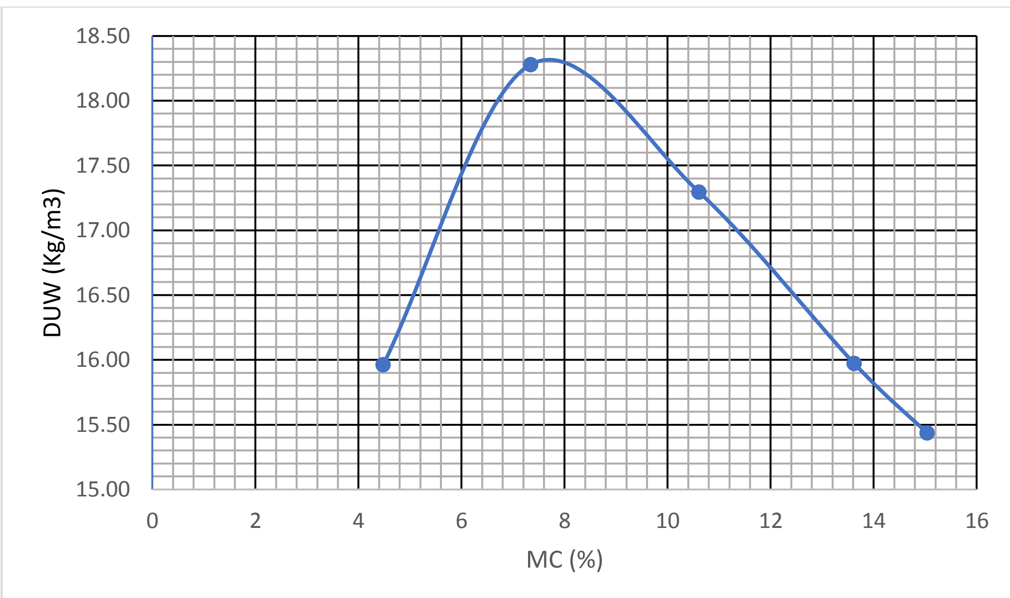
BSH

% of Water	Vol. of mould (m3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m3)	Moisture (%)	Dry density (Kg/m3)	Dry unit weight (Kg/m3)
2	0.001	4	5.65	1.65	1650	2.12	1615.75	15.85
4	0.001	4	5.95	1.95	1950	3.564	1882.89	18.47
6	0.001	4	6.2	2.15	2150	6.34	2021.82	19.83
8	0.001	4	6.1	2.1	2100	8.423	1936.86	19.00
10	0.001	4	6.0	2	2000	11.68	1790.83	17.57



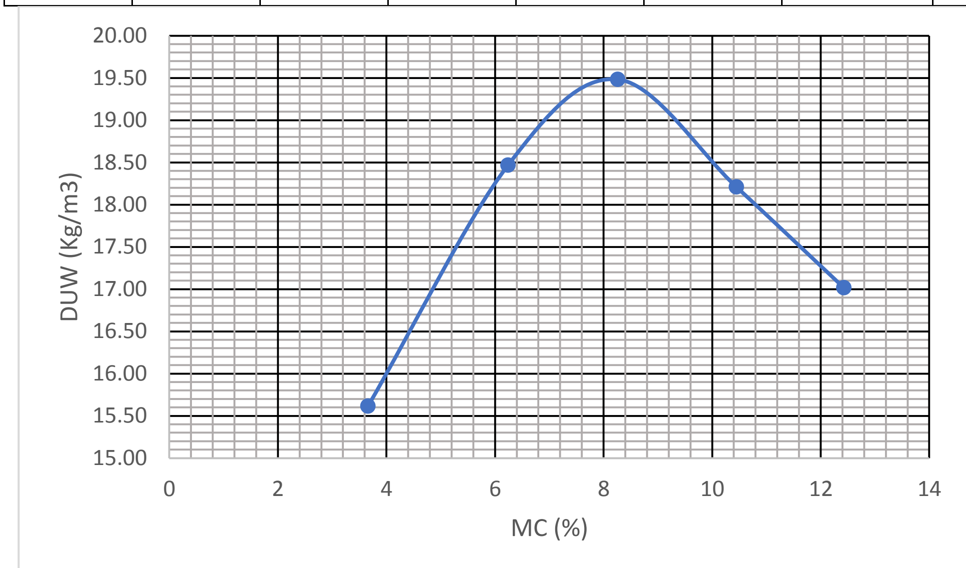
Lat 3

% of Water	Vol. of mould (m^3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m^3)	Moisture (%)	Dry density (Kg/m^3)	Dry unit weight (Kg/m^3)
2	0.001	2.4	4.1	1.7	1700	4.48	1627.11	15.96
4	0.001	2.4	4.4	2	2000	7.34	1863.24	18.28
6	0.001	2.4	4.35	1.95	1950	10.61	1762.95	17.29
8	0.001	2.4	4.25	1.85	1850	13.62	1628.23	15.97
10	0.001	2.4	4.21	1.81	1810	15.04	1573.37	15.43



BSH

% of Water	Vol. of mould (m3)	Wt. of mould (Kg)	Mould + Wet soil (Kg)	Wt. of wet soil (Kg)	Bulk density (Kg/m3)	Moisture (%)	Dry density (Kg/m3)	Dry unit weight (Kg/m3)
2	0.001	4	5.65	1.65	1650	3.66	1591.74	15.61
4	0.001	4	6.0	2	2000	6.24	1882.53	18.47
6	0.001	4	6.15	2.15	2150	8.26	1985.96	19.48
8	0.001	4	6.05	2.05	2050	10.45	1856.04	18.21
10	0.001	4	5.95	1.95	1950	12.43	1734.41	17.01



Volumetric shrinkage

Lat 1

		+2 %MWC		
lat 1	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.35	10.2	10.12	10.1
height (cm)	11.4	11.29	10.71	10.7
volume (cm^3)	959.13	922.5	912.13	908.4
VSS(%)	0	3.8	4.9	5.4

		+4 %MWC		
lat 1	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.4	10.28	10.21	10.1
height (cm)	11.55	10.65	10.62	10.6
volume (cm3)	981.16	916.4	904.63	900.7
VSS(%)	0	6.6	7.8	8.2

Lat 2

		+2 %MWC		
lat 2	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.35	10.29	10.21	10.19
height (cm)	11.5	10.76	10.67	10.65
volume (cm3)	967.5	936.54	916.22	908.48
VSS(%)	0	3.2	5.3	6.1

		+4 %MWC		
lat 2	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.4	10.13	10.08	10.08
height (cm)	11.5	11.39	11.26	11.24
volume (cm3)	976.9	917.3	899	896.7
VSS(%)	0	6.1	8	8.2

Lat 3

		+2 %MWC		
lat 3	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.35	10.2	10.13	10.1
height (cm)	11.3	11.24	11.17	11.14
volume (cm3)	950.72	918.96	901.85	894.24
VSS(%)	0	3.334	5.14	5.94

		+4 %MWC		
lat 3	**0day**	**7days**	**14days**	**21days**
diameter (cm)	10.5	10.2	10.2	10.1
height (cm)	11.13	11.08	11.05	11.05
volume (cm3)	963.74	908.8	898.2	894.35
VSS(%)	0	5.7	6.8	7.2

TITLE PAGE

INVESTIGATION ON THE ECONOMIC AND LIFE CYCLE COST BENEFITS OF CONCRETE OVER ASPHALT PAVEMENT (USING ISIEKE EBONYI STATE, AS A CASE STUDY)

B. ENG. PROJECT SUBMITTED

BY

NWANKWOEKE CHINEDU SHEDRACK

2017224054

IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR OF ENGINEERING (E.ENG) IN CIVIL ENGINEERING.

TO THE

DEPARTMENT OF CIVIL ENGINEERING

NNAMDI AZIKIWE UNIVERSITY, AWKA.

FEBRUARY, 2022

CERTIFICATION

This is to certify that I Nwankwoeke Chinedu Shedrack with Registration 2017224054 personally carried out this project till completion for the award of Bachelor's degree in Engineering (B.ENG), Department of Civil Engineering, Nnamdi Azikiwe University Awka,

This work to the best of my knowledge has not been to any form submitted for the same purpose in the field of Civil Engineering.

______________________________ ____________________

Nwankwoeke Chinedu Shedrack Date

APPROVAL PAGE

This research work has been assessed and approved by the Department of Civil Engineering, Nnamdi Azikiwe University, Awka.

……………………………   ………………
Engineer Mrs. B. A. Njotea
(Project Supervisor)   Date

……………………………   ………………
Engr. Dr. C.A. Ezeagu   Date
(Head of Department)

……………………………   ………………
Engr. Prof D.O Onwuka
(External Examiner)   Date

DEDICATION

This work is dedicated to the creator of the whole universe, the Almighty God, He has been wonderful to me.

ACKNOWLEDGMENT

My profound gratitude to my loving parents Madam Rose and Emmanuel Nwankwoeke and Mr. Nwankwoeke Okechukwu who have always been a source of encouragement to me to see that I achieve my dreams.

Special thanks and total appreciation to my project supervisor, Engineer Mrs. B. A. Njotea for her relentless help and direction throughout the period of this work.

Also, my utmost gratitude to the Head of department of Civil Engineering, Engr. Dr. C.A. Ezeagu and all my lecturers who have never failed in their teachings and who through their academic impact got me to the level and saw me through to the completion of this work.

I also appreciate my friends, Engr Orie Chinedu, Onwe, Theophelus, Nwachukwu Fabian, Nte Okechukwu Ajah, Ojon Matthew and my colleague's, Agbago Solomon, Ogbonna Iswell they have been wonderful to me.

Thank you all and God bless you.

ABSTRACT

Life cycle cost analysis is a method for assessing the total cost of the facility ownership. It takes into account all cost acquiring, owning and disposing of a building, or building system. The cost of road construction consists of design expense, material extraction, construction equipment, maintenance cost, rehabilitation cost, salvage cost initial construction cost, fuel cost, time saving cost and operations cost over the entire service life of concrete over asphalt pavement. An economic analysis process known as Life Cycle Cost Analysis (LCCA) is used to evaluate the cost efficiency of alternative based on the Net Present Value (NPV) concept. It is important to evaluate the above mention cost – aspect in order to obtain optimum pavement life-cycle costs. However, pavement managers are often unable to consider each important element that may be required for performing failure maintenance tasks. The life cycle cost of concrete over asphalt pavement was investigated using Isieke Ebonyi State as a case study. The result obtained from the life cycle cost of both alternatives indicate that the concrete pavement has a longer service life than asphalt pavement and for 1km (kilometer) road length, the life cycle cost of the concrete pavement is lower by a value of 17.3million in naira than that of asphalt pavement in forty (40) years analysis period. The 1km (kilometer) concrete pavement is therefore adjudged as an economic viable alternative over asphalt pavement due to it relatively lower life cycle cost and higher service life and as a result must be given due consideration by prospective investors. Current Life Cycle Cost Analysis (LCCA) method are analysed in research. LCCA research will become more robust of improvement are made, facilitating private industries and government agencies to accomplish their economic aims.

TABLE OF CONTENTS

Title page	i
Approval page	ii
Dedication	iii
Acknowledgement	iv
Abstract	v
Table of contents	vii
List of tables	x
List of figures	xi

CHAPTER ONE

1.1 Introduction	1
1.2 Statement of the problem	2
1.3 Objectives of the research	3
1.3.1 General objective	3
1.3.2 Specific objective	3
1.4 Scope	4
1.5 Relevance of the study	6

CHAPTER TWO

2.0 Literature review	7
2.1 Overview of the study	7
2.2 Definition of some major terms	9
2.3 Experience and practices of life cycle cost analysis (LCCA)	11

2.3.1 Practices from asphalt associations 11

2.3.2 Practices from concrete associations 12

2.3.3 Experience from the U.S. department of transportation, federal highway administration 13

2.4 Types of concrete pavements 14

2.4.1 Jointed unreinforced concrete pavement 14

2.4.2 Jointed reinforced concrete pavement 14

2.4.3 Continuously reinforced concrete pavement 15

2.5 Materials for concrete pavement 16

2.5.1 Cementitious materials 16

2.5.1.1 Portland cement 16

2.5.1.2 Supplementary cementitious materials 18

2.6 Historical background of the study 20

CHPTER THREE

3.0 Research Design And Methodology 22

3.1 Main research 22

3.1.1 Data collection 24

3.1.2 Analysis period 24

3.1.3 Discount rate 24

3.1.4 Traffic analysis and pavement design 24

3.1.5 Project data used 25

3.1.6 Data analysis 25

CHAPTER FOUR

4.0 Result and discussion 26

4.1 Results 26

4.2 Analysis of results 30

4.2.1 Construction cost 30

4.2.2 Maintenance cost 31

4.2.3 Rehabilitation cost 33

4.2.4 Salvage cost 33

4.2.5 User cost. 34

4.2.6 Life cycle cost. 36

CHAPTER FIVE

5.0 Conclusion and recommendation 38

5.1 Conclusion 38

5.2 Recommendation 39

Reference 40

Appendix A: Construction cost 42

Appendix B: Maintenance and rehabilitation cost 46

Appendix C: Agency and user cost 48

Appendix D: Life cycle cost 51

LIST OF TABLE

Table 4.1a Summary of Initial Construction Cost of 1km Concrete Pavement at Isieke Ebonyi State (Sourced and adjusted from Ministry of Works Ebonyi State). 26

Table 4.1b: Summary of Bill of Engineering Measurement and Evaluation (BEME) for initial construction cost of flexible pavement at Isieke Ebonyi State (Adjusted from proposed Ebonyi State Ministry of Works, BEME 2010). 27

Table 4.2a Agency (Authority) Cost of Concrete Pavement (Computed from Ebonyi State Ministry of Works Data). 29

Table 4.2b Agency (Authority) Cost of Flexible Pavement (Computed from Ebonyi State Ministry of Works Data). 29

Table 4.3 Life Cycle Cost of Concrete and Flexible Pavement 30

LIST OF CHARTS

Fig 4.0: Chart showing the initial construction cost of both pavement against Analysis Period. 31

4.1a: Chart showing the routine maintenance cost of both pavements against analysis period. 32

4.1b: Chart showing the periodic maintenance cost of both pavements against analysis period. 32

Fig 4.2: Chart showing the rehabilitation cost of both pavements against analysis period. 33

Fig 4.3: Chart showing the salvage cost of both pavements against analysis period. 34

Fig 4.4a: Chart showing the time saving cost of both pavements against analysis period. 35

Fig 4.4b: Chart showing the time delay cost of both pavements against analysis period. 35

Fig 4.4b: Chart showing the fuel cost of both pavements against analysis period. 36

Fig 4.5a: Chart showing the life cycle cost of both pavements against analysis period in Naira. 36

Fig 4.5b: Chart showing the life cycle cost of both pavements against analysis period in USD. 37

CHAPTER ONE

1.1 INTRODUCTION

A highway pavement is composed of a system of overlaid states of chosen processed materials that is positioned on the in-situ soil termed the subgrade. Its basic requirement is the provision of a uniform skid-resistant running surface with adequate life and requiring minimum maintenance, the main structural purpose of the pavement is the support of vehicle wheel loads applied to the carriageway and the distribution of them to the subgrade immediately underneath. If the road is in cut, the subgrade will consist of the in-situ soil. If it is constructed on fill, the top layer of the embankment structure are collectively termed the subgrade. (Martin Rogers et al 2013).

Highway pavement construction, maintenance and rehabilitation costs are rising dramatically. It is important for highway agencies to utilize tools and approaches that facilitate proper decision-making by applying economics and operations research such as Life-Cycle Cost Analysis (LCCA) to achieve economically reasonable long-term investments. LCCA is a method based on principles of economic analysis. It improves the estimation of the total long-term economic viability of different investment options. This method finds significant application in pavement design and management. A number of agencies employ the LCCA approach to estimate the economic feasibility of pavement designs over the long haul. Thus, it is very important for agencies to realistically evaluate pavement economics in order to provide suitable input to the LCCA. (Taylor et al. 2016).

As a concept, it was in the 1950s that benefit-cost analysis (BCA) was initially applied as a selection factor for various pavement design options. Then in the 1970s, LCCA principles started being implemented in some key projects at the local and national state levels for pavement design and pavement type selection.

Considering the mostly inadequate funding under normal circumstances, road authorities are consistently challenged with funding projects due to insufficient resource. Moreover, with the increasing demand for new road infrastructure, the demand for efficient management of old and new roads is on the rise as well, along with safety demands, accessibility and the implementation of advanced traffic management systems for decreasing socio-economic costs by mitigating maintenance-related environmental effects, traffic issues, and losses. Maintenance backlogs nonetheless increase too. Road authorities thus emphasize more on better efficiency and lower expenses due to limited funds. Since maintenance expenditures normally comprise half the annual road infrastructure funds, it is very important to prioritize efficiency in road maintenance. Thus, with respect to road objects, life-cycle costs (LCCs) are regarded as having higher priority than simply investments. Hence, road authorities are expected to realize the importance of Life cycle cost agency (LCCA) and maintain a calculation system. LCCs are also deemed to be a restraint in road design selection or the assessment of tenders. When calculating LCCs, both road authority costs and costs of socio-economic nature should be taken into account. Road agency (authority) costs comprise expenses for planning, construction, design, maintenance, and rehabilitation. All these costs are usually the government's responsibility to cover using tax earnings. Socio-economic costs comprise of agency costs, user costs (e.g. delay costs, accident costs and vehicle operation costs), and environmental costs.

1.2 STATEMENT OF THE PROBLEM

The dramatic increase in traffic volume in built-up areas, such as the Capital road, Federal Trunk Roads of Nigeria results in more and more construction of new roads and modernization of old ones. Therefore, this requires further studies on how road pavement types are selected.

Road Authorities could make more informed and better investment decisions, because pavement type has a significant impact on future cost and service quality. Traffic growth, especially in heavy axle load can cause damage to pavements much quicker than expected, in turn causing more maintenance and thereby increasing agencies and users costs.

Pavement type choice is usually based on traffic level, soil conditions, atmospheric factors and costs. In many cases, the initial construction cost is the main consideration; the future maintenance and rehabilitation costs may sometimes be forgotten.

Life cycle cost analysis (LCCA) is a process of evaluating economic performance of a pavement over its entire life, sometimes known as whole cost accounting or total cost of ownership of firms.

It balance initial monetary investment with the long term expense and operating the pavement. According to the American Association of State Highway and Transportation Officials (AASHTO) Guide for the Design of Pavement Structures, life cycle costs “refer to all costs which are involved in the provision of a pavement during its complete life cycle”. That means that all pavement options are evaluated by taking into account different agencies and users costs.

Agencies costs include initial construction costs as well as future costs of rehabilitation, maintenance and facility operation. User costs are a result of many different issues, for instance increased delay costs, increased vehicle operating costs or charges in accident costs due to future maintenance actions.

All types of asphalt binding materials used in our country are imported with hard currency and its cost is becoming increasingly high from time to time. On the other hand cement materials have high potential of production in Nigeria and it is hoped that the price will decrease. Asphalt pavement requires heavy maintenance activities starting from early stages of its service life, but Cement Concrete as an Alternative Pavement Material Over Asphalt pavement in Arterial Roads of Nigeria (Isieke Ebonyi State); Hence, this study will try to address the problem and show economic advantages of using locally available resource for sustainable pavement construction. (AASHTO 2013).

1.3 AIM AND OBJECTIVES OF THE STUDY

1.3.1 AIM

The general aim of the study is to identify the sustainable and economical pavement material in arterial roads of Nigeria by making life cycle cost comparisons and economic analysis of Cement Concrete with Asphalt Concrete pavement materials in selected representative arterial roads of the Isieke Ebonyi State.

1.3.2 Specific objective;

• To identify the initial cost for both concrete and asphalt pavements.

• To identify the required types of life time maintenances for both concrete and asphalt pavements.

• To identify the varying costs related to different maintenances involved in concrete and asphalt pavements.

• To carry out economic evaluation of concrete and asphalt pavements on selected segments of roads in Nigeria for forty years and to determine which pavement type is more economical and sustainable.

• To draw conclusions and forward recommendations based on the findings of the study.

1.4 SCOPE

The primary objectives of road construction project planning are to optimize quality, cost and time. In Nigeria, this construction industry and its management is at developing stage, fulfilling these requirements is difficult and challenging. This research work therefore, focuses on one of the basic requirements i.e. cost. Hence, the scope of the study is restricted to the identification of Cement Concrete as an Alternative Pavement Material Over Asphalt Concrete in Arterial Roads of Nigeria;

The scope of the study is, therefore, limited to evaluation of two alternative pavement types based on life cycle cost and economic advantages for sustainable road construction projects. Environmental impact and societal benefits from this sustainable road construction projects will not be fully quantifiable in this study.

This research work started with problem identification, followed by literature review, formal and informal discussion with professionals in the federal road construction sectors, such as concrete and asphalt pavement

Concrete Pavement material against Asphalt Concrete material in trunk road of Isieke Ebonyi State roads of Nigeria. Literatures include magazines, books, journals, internet etc. In parallel with literature review, an in-depth desk study has been conducted to identify construction costs, maintenance costs, environmental impacts and road user benefits of each pavement material type. During the desk study, various documents such as design manuals of different countries, technical specifications, feasibility studies reports, Engineering design reports, correspondences, progress reports, completion reports, payment certificates, statements on final account, road asset management documents etc. has been critically evaluated. Whenever there is unclear primary data or ambiguity during the desk study, further explanation or information has been obtained through informal interviews with professionals involving in road construction projects especially pavement or material Engineers, in order to maximize the clarity and to gain adequate understanding of the data for its use in analysis of cost of concrete over asphalt pavement, using Ebonyi State manual guide 2013.

The document search was intended to collect pavement design trends, values of initial/construction costs consumed and annual maintenance or rehabilitation costs allocated from some randomly selected upgrading and rehabilitation trunk type road construction projects which are completed/substantially completed.

Then analysis and discussion has been conducted based on the primary & secondary data obtained. Finally, conclusions have been drawn and recommendations forwarded based on the finding of the study and literature reviews. The study has been conducted on projects with high traffic volumes of Trunk type Federal Roads of Isieke Ebonyi State of Nigeria.

Life Cycle Cost Analysis provides a methodology for computing the cost of a product or service during its lifetime. It is used to compare competing design alternatives over the lives of each alternative, considering all significant costs and benefits, expressed in equivalent monetary units. For infrastructure assets such as roads, a large proportion of the total cost over the lifetime of these assets is incurred after construction, i.e. during their service lives. It is possible to avoid most of the "unknown" costs by introducing long-term costs into the pavement valuation processes instead of comparing only initial material and construction costs. (Akbar et al. 2012).

Analysis period of 40 years has been determined based on the recommendation of AASHTO 1993 and in order to utilize the full design life of concrete pavement for proper comparison of the two pavement type alternatives.

1.5 RELEVANCE OF THE STUDY

In the area of road construction, proper planning of projects is important to highway organizations (authorities) as their construction program outlines how highway funds are to be spent over time and to be sustainable in its serviceability for the comfort of road users, any deviation from the established program often brings a quick response from the public, the press, and politicians. When this occurs, the highway organization loses creditability. On the other hand, if a highway organization can produce realistic program estimates that it is able to attain, then the image of the agency is enhanced.

Therefore, it is the responsibility of the Authorities to make an accurate project planning in the selection of economical pavement materials for the construction of sustainable roads for allocation of justifiable budgets. Thus beneficiaries of this research are;

- The implementing authorities (Ebonyi State minister of works)

- consulting and construction firms plus practicing Engineers,

- Educational institutions, which use the information for academic purposes,

- Investors to focus on manufacturing of ready mix concrete for pavement works.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 OVERVIEW OF THE STUDY

Transport and mobility are essential for economic and social development. For this reason, developed countries have devoted considerable resources to the development of high-quality transport networks which need to be adequately maintained. Current road construction methods lead to significant maintenance requirements, which can only be met at a very high cost. The continued growth in road traffic and axle loads and the pressure to restrain government spending put growing pressures on road authorities to come up with new solutions. At the same time, the cost to economics due to congestion and disruption during road works on high volume roads has become unacceptably high.

There are growing pressures for long-life road infrastructures that require minimal maintenance. Pavements are the critical elements of an efficient highway transportation system for moving people and goods. Without well-performing pavements, the transportation infrastructure cannot effectively function, road users suffer (in terms of increased costs, travel time, and unsafe roads) and the overall economy suffers (in terms of higher costs for goods and commodities).

Modern societies cannot function without mobility, and mobility requires well-performing pavements. In recent years, innovation in the road sector has focused on economic and organizational structures, while changes in road paving techniques have been much less dramatic. Rather, they have at best been incremental. Yet, in order to optimize national highway budgets, whole-life costing methods are increasingly used to determine how, where and when to best spend budget funding on road construction and maintenance. Within this framework, the shift to full maintenance contracting has helped reduce costs, and the adoption of long-term contracts has helped establish an environment in which the development of more durable pavement types could be stimulated.

Therefore, long-lasting pavements that are safer, smoother, and environmentally sensitive and can be cost-effectively constructed and maintained are an important part to modernize the

Nigeria transportation system. In Ethiopia, billions of ETB are spent every year to construct, maintain, preserve, and rehabilitate the Nation's highway pavement infrastructure.

The accumulated investment in our roadway pavements is above billion for the last 16 years of Road Sector development program phase (RSDP). This investment needs to be protected and managed efficiently so that future generations of our citizens can have high standard infrastructure which is to be one of the best highway systems in Africa and to enjoy the benefits of it. (Francis et al. 2016)

Initial cost of construction, maintenance cost of pavements and road user costs play major role for deciding which pavement will be more economical. There is no intensified life cycle cost comparison research conducted in this regard in Nigeria, but various studies have been conducted worldwide in different countries in this subject matter through different researchers.

It is becoming increasingly apparent that a host of human activities and development practices are negatively affecting the economic, environmental and social well-being of the planet, putting future generations of human being, as well as other species, at risk. Confronted with this reality, stakeholders in the pavement industry are being challenged to adopt practices that maintain economic vitality while balancing environmental and social needs. (Francis et al. 2016).

At the same time, stakeholders are facing other challenges: pavements are aging and deteriorating; 26% of the paved road system are not in a good condition. Traffic volumes and axle loads continue to increase, putting more demands on the already stressed pavement system in the country. Road way authority budgets continue to fall short of funds needed to substantially improve pavement conditions. The trend and budget allocation attention for pavement condition improvement, comparing with new projects, is very poor in a developing country like Nigeria.

The people responsible for the management, design, construction, maintenance and rehabilitation of the deteriorating network of pavements are overwhelmed, recognizing that the current approach to solving problems inherent in the countries pavement infrastructure is not sustainable. What is needed is a new approach, the implementation of truly sustainable pavement solutions that result in reduced cost over the life cycle, lessened environmental impact and enhanced societal benefit. The main purpose of this research is to evaluate the life cycle cost benefit of concrete over asphalt pavements.

2.2 DEFINITION OF SOME MAJOR TERMS

Concrete is basically a mixture of two components: aggregates and paste. The paste, comprised of Portland cement and water, binds the aggregates (usually sand and gravel or crushed stone) into a rocklike mass as the paste hardens because of the chemical reaction of the cement and water. Design Manual 2013 required a 28-day characteristic compressive strength of 30, 35 or 40 MPa for pavement construction. However, the 28-day concrete flexural strength is a principal design factor. The suitable value for road pavement construction is 40 Mpa.

2.2.1 Concrete pavements: (also called rigid pavements), as the name implies, are Concrete and very strong in compression. The strength of the pavement is contributed mainly by a concrete slab, unlike asphalt pavements where successive layers of the pavement contribute cumulatively. The rough surface required for an adequate resistance to skidding in wet conditions can be provided by dragging stiff brooms transversely across the newly-laid concrete or by cutting shallow randomly spaced grooves in the surface of the hardened concrete slab.

As the name ‘Concrete’ implies, the deflections under a loaded wheel are very small compared with the deflections observed in flexible pavements and the stresses within the underlying sub-base and subgrade are also comparatively small. Concrete pavements therefore deteriorate through quite different mechanisms from those that affect asphalt pavements. This constitution implies the following advantages:

It is feasible to design Concrete pavements for longer design lives, up to 60 years.

Little maintenance is generally required

Concrete pavement deforms less under traffic as compared to asphalt pavements

A relatively thin pavement slab distributes the load over a wide area due to its high Concretively. Localized low strength sub-grade materials can be overcome due to this wider distribution area. Concrete is very resistant to abrasion making the anti-skidding surface texture last longer.

In the absence of deleterious materials (either in the aggregate or entering the concrete in solution from an external source), unlike with asphalt pavements, concrete does not suffer deterioration from weathering. Neither its strength nor its stiffness material affected by temperature changes.

2.2.2 Asphalt concrete can be described as a combination of bitumen and aggregates which are mixed together, spread and compacted while hot, to form a pavement surface. The strength of asphalt is derived from friction between the aggregate particles, the viscosity of the bitumen under operating conditions and the cohesion within the mass resulting from the bitumen itself and the adhesion between the bitumen and aggregate.

In reality asphalt concrete is visco-elastic in nature due to the bitumen in the mixture. However, at the normal operate rates and magnitude of loading and temperature, asphalt may be considered as an elastic isotropic material.

Initial cost is generally the major factor in deciding the type of the pavement in design. The Planners often think that flexible pavement is cheaper than Concrete pavements. In fact this is not always the case. In the last decade the price of bitumen which is the main ingredient of flexible pavement has increased because of the increase in crude oil prices. However the Concrete pavement's main ingredient cement price is relatively stable

2.2.3 Life Cycle Cost Analysis; means a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future costs, such as maintenance, user costs, reconstruction, rehabilitation, restoring and resurfacing costs over the life of the project segment.

According to the American Association of State Highway and Transportation Officials (AASHTO) guide for the design of pavement structures, life cycle costs "refer to all costs which are involved in the provision of a pavement during its complete life cycle". That means that all pavement options are evaluated by taking into account different agencies' and users' costs. Agencies' costs include initial construction costs as well as future costs of rehabilitation, maintenance and facility operation. User costs are a result of many different issues, for instance increased delay costs, increased vehicle operating costs or changes in accident costs due to future maintenance actions.

Life Cycle Cost Analysis provides a methodology for computing the cost of a product or service during its lifetime. It is used to compare competing design alternatives over the lives of each alternative, considering all significant costs and benefits, expressed in equivalent monetary units. For infrastructure assets such as roads, a large proportion of the total cost over the lifetime of these assets is incurred after construction, i.e. during their service lives. It is possible to avoid most of the “unknown” costs by introducing long-term costs into the pavement valuation processes instead of comparing only initial material and construction costs.

Pavement type decisions are usually based on traffic level, soil conditions, atmospheric factors and construction costs. Usually, construction cost is the main consideration; the future maintenance and rehabilitation costs may be forgotten. (Velado, M. et al. 2012).

2.3 EXPERIENCE AND PRACTICES OF LIFE CYCLE COST ANALYSIS (LCCA)

2.3.1 Practices from Asphalt Associations

In 2004, the Asphalt Pavement Alliance (APA) released a position paper that concluded that pavement type selection should be a road user-oriented process, not an industry-oriented process It states that the system used to select pavement type should be objective, defensible, understandable, and based on historical records, primarily driven by economics and periodically reviewed.

APA supports the use of life-cycle cost analysis in the decision making process and recommends the methodology developed by the Federal Highway Administration (FHWA) in Demonstration Project. The Net Present Value (NPV) is used for the purpose of comparing alternatives. Initial costs, maintenance costs, and salvage value are recommended for consideration in the life-cycle cost analysis. APA recommends a 40-year analysis period when comparing asphalt with concrete pavements. APA states that asphalt pavements possess many advantages when compared with concrete pavements including low initial cost, low maintenance costs, flexibility and speed of construction, the ability to handle heavy loads, a long life, and complete recyclability.

APA conducted another study in 2005 to determine the average service life of flexible pavements to reach an unacceptable surface condition. Researchers considered six types of distresses in the analysis including fatigue cracking, longitudinal cracking in the wheel path area and longitudinal cracking outside the wheel path area, transverse cracking, rut depth, and smoothness measured by the International Roughness Index (IRI). Data for the analysis were extracted from the Long-Term Pavement Performance (LTPP) database. The median age of the 643 sections considered in the study was 17 years with 109 sections that were older than 20 years. An analysis to determine the probability of failure was performed for each distress type. According to the study, the expected service life to a moderate distress level exceeds 20 years for all distresses.

2.3.2 Practices from Concrete Associations

The American Concrete Pavement Association (ACPA) states that concrete pavements are a better choice than asphalt pavements because they have advantages in several areas including safety, durability, smoothness, versatility, and value. On safety, it provides better visibility, reduced wet spray since concrete never ruts, and provides the best traction grip. On durability, concrete hardens over time, and outlasts flexible materials since their average lifespan is 30 years. On smoothness, concrete stays smoother longer, creating safer, comfortable transportation surfaces and saving fuel. On versatility, concrete pavements can be; 1) designed to last from 10 to 50 years, 2) used to rehabilitate old asphalt pavement using white topping, or 3) used to rehabilitate a worn concrete pavement. On value, concrete pavements provide the best long-term value due to their longer life, they are easy to repair, and they can be built and opened to traffic in less than 12 hours.

In 2002, ACPA published a guide for comparing alternate pavement designs using LCCA. The guide describes the LCCA process factors that influence the results including agency costs (initial cost, maintenance and rehabilitation costs, and salvage value), user costs (delay of user-costs, roadway deterioration costs, and accident crash cost), discount rate, selection of rehabilitation activities, use of comparable sections, and length of the analysis period. Present worth (PW) and the equivalent uniform annual cost (EUAC) are mentioned as economic indicators used to express LCCA results. APA recommends EUAC because all costs are expressed in terms of an annual cost over the analysis period. The guide also presents a brief summary of life-cycle cost and performance studies conducted with historical data in Michigan, Minnesota, Iowa, Florida, Tennessee, South Dakota, Utah, and Georgia. According to these studies, concrete sections lasted between 1.6 and 2.6 times longer than the asphalt sections and were from 14 percent to 250 percent more effective than the asphalt pavements.

2.3.3 Experience from the U.S. Department of Transportation, Federal Highway Administration

In 1998, the Federal Highway Administration (FHWA) published an Interim Technical Bulletin with recommended procedures for conducting life-cycle cost analysis (LCCA) of pavements.

The bulletin discusses how to address alternative pavement design strategies, length of performance periods and activity timing, agency costs (initial cost, maintenance and rehabilitation cost, residual value), and user costs (delay costs, vehicle operating, and crash cost) in LCCA. The Net Present Value is proposed as the economic indicator for comparing alternatives. The Federal highway Administration (FHWA) encourages risk analysis. A sensitivity analysis is recommended as a minimum to study the impact of the individual outputs on LCCA results. The discount rate is one of the major factors considered in the sensitivity analysis. The use of simulation techniques incorporated into LCCA such as Monte Carlo, is recommended to account for the variability of the input values and their influence in the results of the analysis.

In 2002, the office of Asset Management at the Federal Highway Administration (FHWA) published a Life-Cycle Cost Analysis Primer. The primer was intended to provide background information to evaluate infrastructure investment alternatives. The LCCA approach considers total user and agency costs when comparing alternatives. The application of Benefit Cost Analysis (BCA) to account for benefits in the comparison of alternatives is discussed in the primer. If expected benefits provided by the alternatives under comparison are different, then BCA is considered more appropriate than LCCA. A description of the LCCA process steps is included with a discussion on how to establish design alternatives, determine activity timing, estimate costs (agency and user), compute life- cycle costs, and analyze the results. The use of the equivalent uniform annual cost or the present value is recommended as economic indicators to compare alternatives.

2.4 TYPES OF CONCRETE PAVEMENTS

According to the American Concrete Pavement Association, concrete pavement can be classified into three types: jointed unreinforced concrete pavement (JUCP), jointed reinforce concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP). This classification was done based on the joint spacing and use of reinforcement.

2.4.1 Jointed Unreinforced Concrete Pavement

In Jointed Unreinforced Concrete Pavements (JUCP), the pavement consists in an unreinforced concrete slab cast in place continuously and divided into bays of predetermined dimensions by the construction of joints. The bays dimensions are made sufficiently short (usually 15 times the slab thickness) so as to ensure that they do not crack. The bays are linked together by tie bars, the main function of which is to prevent horizontal movement (i.e. the opening of warping joints) and thus ensure load transfer through aggregate interlock.

2.4.2 Jointed Reinforced Concrete Pavement

In Jointed Reinforced Concrete Pavements (JRCP) the pavement consists generally in a cast in place concrete slab divided in reinforced concrete bays separated by joints at every 25 m. The reinforcement is made to prevent developing cracks from opening. This allows designing much larger bays than with JUCP. The bays are linked together by tie bars to prevent horizontal movement and thus ensure load transfer through aggregate interlock. The longitudinal reinforcement is the main reinforcement. A transverse reinforcement though not absolutely necessary in most cases is usually added to facilitate the placing of longitudinal bars.

Jointed reinforced concrete pavement, or JRCP, is distinguished from JUCP by longer slabs and light reinforcement in the slabs. The slab steel content is typically in the range of 0.10–0.25 percent of the cross-sectional area, in the longitudinal direction, with less steel in the transverse direction. This light reinforcement is often termed temperature steel

2.4.3 Continuously Reinforced Concrete Pavement

Continuously reinforced concrete pavement, or CRCP, is characterized by heavy steel reinforcement and an absence of joints. Much more steel is used for CRCP than for JRCP, typically on the order of 0.4–0.8 percent by volume in the longitudinal direction. Steel in the transverse direction is provided in a lower percentage as temperature steel

Continuously Reinforced Concrete Pavement (CRCP) is made of a cast in place reinforced concrete slab without joint. The expansion and contraction movements are prevented by a high level of sub-base restraint. The frequent transverse cracks are held tightly closed by a large amount of continuous high tensile steel longitudinal reinforcement.

Basically, the use of the different types of Concrete pavement is as follows:

JUCP is suitable for all levels of traffic, whenever the risk of sub-grade movement is low.

JRCP is suitable for all levels of traffic and is used when the risk of settlements of the sub-grade cannot be neglected.

CRCP shall basically be considered only for rather high design traffic (>30 million ESAL). They can also be included for less heavily trafficked schemes where the advantage of lower maintenance throughout the design life may be worthwhile. They are particularly suitable where settlement of the sub-soil is expected (Martin Rogers et al. 2013).

2.5 MATERIALS FOR CONCRETE PAVEMENT

Careful selection of concrete materials and careful mixture design and proportioning, combined with good design and construction practices, is a key contributor to success.

2.5.1 Cementitious Materials

Portland cements are hydraulic cements composed primarily of hydraulic calcium silicates. Hydraulic cements set and harden by reacting chemically with water. During this reaction, called hydration, cement combines with water to form a stone like mass, called paste. When the paste (cement and water) is added to aggregates (sand and gravel, crushed stone, or other granular material) it acts as an adhesive and binds the aggregates together to form concrete, the world's most versatile and most widely used construction material.

Hydration begins as soon as cement comes in contact with water. Each cement particle forms a fibrous growth on its surface that gradually spreads until it links up with the growth from other cement particles or adheres to adjacent substances. These fibrous build up results in progressive stiffening, hardening and strength development. The stiffening of concrete can be recognized by a loss of workability that usually occurs within three hours of mixing, but is dependent upon the composition and fineness of the cement, any admixtures used, mixture proportions, and temperature conditions. Subsequently, the concrete sets and becomes hard.

2.5.1.1 Portland Cement

Different types of Portland cement are manufactured to meet various normal physical and chemical requirements for specific purposes. Portland cements are manufactured to meet the specifications of ASTM C 150, AASHTO M 85, or ASTM C 1157. ASTM C 150, Standard Specification for Portland cement, provides for eight types of Portland cement using Roman numeral designations as follows: (William James Wilde, et al. 2013).

Type I Normal

Type IA Normal, air-entraining

Type II Moderate sulfate resistance

Type IIA Moderate sulfate resistance, air-entraining

Type III High early strength

Type IIIA High early strength, air-entraining

Type IV Low heat of hydration

Type V High sulfate resistance

AASHTO M 85, Specification for Portland cement, also uses type designations I through V for Portland cement. The requirements of M 85 are almost identical to ASTM C 150. AASHTO specifications are used by some state departments of transportation in lieu of ASTM standards. ASTM C 1157, Performance Specification for Hydraulic Cements, provides for six types of Portland cement as discussed under "Hydraulic Cements" below. A detailed review of ASTM C 150 and AASHTO M 85cements follows.

Type I Portland cement is general-purpose cement suitable for all uses where the special properties of other types are not required. Its uses in concrete include pavements, floors, reinforced concrete buildings, bridges, tanks, reservoirs, pipe, masonry units, and precast concrete products.

Type II Portland cement is used where precaution against moderate sulfate attack is important. It is used in normal structures or elements exposed to soil or ground waters where sulfate concentrations are higher than normal but not unusually severe. Type II cement has moderate sulfate resistant properties because it contains no more than 8% tricalcium-aluminate (C_3A). Sulfates in moist soil or water may enter the concrete and react with the hydrated C_3A, resulting in expansion, scaling, and cracking of concrete. Some sulfate compounds, such as magnesium sulfate, directly attack calcium silicate-hydrate. Use of Type II cement in concrete must be accompanied by the use of a low water to cementitious materials ratio and low permeability to control sulfate attack (William James Wilde, et al. 2013).

Type III Portland cement provides strength at an early period, usually a week or less. It is chemically and physically similar to Type I cement, except that its particles have been ground finer. It is used when forms need to be removed as soon as possible or when the structure must be put into service quickly. In cold weather its use permits a reduction in the length of the curing period.

Type IV Portland cement is used where the rate and amount of heat generated from hydration must be minimized. It develops strength at a slower rate than other cement types. Type IV cement is intended for use in massive concrete structures, such as large gravity dams, where the temperature rise resulting from heat generated during hardening must be minimized. Type IV cement is rarely available.

Type V Portland cement is used in concrete exposed to severe sulfate action; principally where soils or ground waters have high sulfate content. It gains strength more slowly than Type I cement. Sulfate concentrations requiring the use of Type V cement. The high sulfate resistance of Type V cement is attributed to low tricalcium-aluminate content, not more than 5%.

2.5.1.2 Supplementary Cementitious Materials

Supplementary cementitious materials, or mineral admixtures, include fly ash, slag, and silica fume, as well as other materials. Fly ash and slag are commonly used in pavements.

Blended cements are produced by intimately and uniformly inter-grinding or blending ordinary Portland cement (OPC) with one or more supplementary cementitious materials (SCMs). Most SCMs, such as ground granulated blast-furnace slag (GGBFS) or fly ash (FA), are industrial by-products. These materials are generally not used as cements by themselves, but when blended with OPC, they make a significant cementing contribution to the properties of hardened concrete through hydraulic or pozzolanic activity.

Typically, these materials retard early strength gain of concrete, but improve the ultimate strength and durability. Overall heat of hydration and the rate of heat buildup are both reduced. Workability is improved, and the concrete surface is often easier to finish. SCMs are increasingly used in concrete because of following benefits:

Reduction of economic and environmental concerns by utilizing industrial wastes, reducing carbon dioxide emissions, and lowering energy requirements for OPC clinker production; and

Improvements in concrete properties, such as workability, impermeability, ultimate strength, and durability, including enhanced resistance to alkali-silica reactions, corrosion of steel, salt scaling, delayed ettringite formation, and sulfate attack .

Ebonyi State of Nigeria, most blended cements used for pavement concrete contain 20% to 35% Ground Granulated Blast Furnace Slag (GGBFS). In addition, 15% Class C fly ash is often employed together with the blended cements to improve concrete workability and further reduce cost. The cementitious material (CM) containing fly ash, slag, and Portland Cement (OPC) were defined as ternary cement. If a CM consisted of only fly ash and OPC, it was defined as binary cement.

As discussed above, most SCMs are byproducts from other industries that beneficially react with Portland cement to enhance the performance of concrete. The effective use of SCMs reduces not only the amount of Portland cement required but also the need to dispose of what otherwise would be industrial waste.

The two most commonly used SCMs in paving concrete are fly ash and slag cement. Fly ash is a byproduct of burning pulverized coal for the generation of electrical power. The rock embedded in the coal melts in the furnace and is carried up the stack in the flue gases. As it rapidly cools, small glassy spheres are formed that are collected before the flue gases are emitted to the air. Because of the small size, glassy form, and chemical composition of the ash, it dissolves and reacts with the cement paste to contribute to the performance of the mixture. About 63 million tons of fly ash were produced in the United States in 2009, of which about 12 million tons were used either to make cement or in concrete.

Fly ash is currently specified in AASHTO M 295/ASTM C 618 in two classes based on the chemical composition. The differences are generally influenced by the source of the coal. In general, Class C fly ash is higher in lime content (CaO) and tends to be more reactive at early ages than Class F. The higher CaO content is beneficial for early strength gain but can have negative effects on alkali-silica reactivity and sulfate resistance. It should be noted that the specification for fly ash is broad and that two ashes from different sources, albeit of the same class, are likely to perform very differently; therefore, performance testing should be conducted to determine if the chosen fly ash is behaving as desired. Dosage of fly ash is typically between

15 and 40 percent by mass of cement. The amount of fly ash that can be used is often limited by concerns of delayed setting times and lower early strength gain. In some cases, there may be a potential for undesirable incompatibilities and a perceived increase in the risk of salt scaling. Judicious increases in dosage can be accommodated with attention to detail in mix proportioning and construction workmanship. Some factories use coal in our country, but byproduct collection and testing for fly ash is not yet practiced.

Slag cement, formerly known as ground granulated blast furnace slag (GGBFS), is the material left after extraction of iron from iron ore. When quenched from the molten state and ground to the fineness of cement, it is an extremely effective SCM. Slag cement is specified by ASTM C 989, and about 3 million tons are used in concrete in the United States every year. It is generally used in pavements in dosages up to 50 percent but is limited by concerns of early strength gain, especially when placed during cooler ambient temperatures, and scaling resistance. As with fly ash, usage tends to be regional because of limitations on the cost effectiveness of transporting it long distances. But slag cement is not available in Nigeria currently to consider for our cost comparison (William James Wilde, et al. 2013).

2.6 HISTORICAL BACKGROUND OF THE STUDY

The American Association of State Highway Officials (AASHO) introduced the concept of life-cycle cost-benefit analysis in its "Red Book" in 1960. The LCCA was introduced for highway investment decisions, and as such, formed the notion of economic evaluation of highway upgrades during the planning stage. The next progress step was made by Winfrey who combined data available on the cost of vehicle operations in a system to be utilized when highway planners are developing life-cycle costing processes. Moreover, two projects in the 1960s introduced the utilization of LCC principles for pavement type selection and pavement design. In the first project, the Centre for Highway Transportation Research and the Texas Transportation Institute (TTI) developed the Flexible Pavement System (FPS), a computer-based approach for analyzing and rating alternative flexible pavement designs through the overall life-cycle cost. The second project was by the National Cooperative Highway Research Program (NCHRP), which examined the promotion of the LCCA concept. Subsequently, the Concrete Pavement System (RPS) was developed by Texas DOT, which is identical to FPS with regard to how Life-Cycle Cost Analysis of Concrete pavements is carried out. RPS also ranks alternative designs according to their total life-cycle costs.

The use of LCC concept is supported in the different AASHTO Pavement Design Guide editions, which also include detailed discussions regarding costs that should be considered in LCCA. The current study presents an overview of the basic life-cycle costing theories, with explanations of the various user and agency costs associated with highway pavement projects, as well as the discount rates and economic feasibility of systems.

CHPTER THREE

3.0 RESEARCH DESIGN AND METHODOLOGY

The aim of the research was to identify the sustainable and economical pavement material in arterial roads of Nigeria by making life cycle cost comparisons and economic analysis of Concrete with Asphalt pavement materials in selected representative roads of Isieke Ebonyi State of Nigeria; considering asphalt pavement as a base case and project case of concrete pavement for comparison. The research design and methodology followed towards this end are discussed as follows.

A comprehensive literature review was made to understand the previous efforts, practical situations which include the review of textbooks, design manuals, commercial websites, previous studies by different professionals in different countries, periodicals and academic journals, seminar, conference and research papers.

3.1 MAIN RESEARCH

In this research, two pavement section of 1kilometer span road way in Isieke Ebonyi State was considered using life cycle cost analysis methodology and the test was carried out by comparing concrete and asphalt pavement.

A two pavement behavior scenarios was built: one for asphalt pavement and the other for concrete pavement.

Plate . 3.1 Concrete pavement

Plate 3.2 Asphalt pavement

LCCA applied here includes all costs that are involved in the manufacture and use of the product during its lifetime; it was decided to compare alternatives by using the Present Worth method. A more detailed description of LCCA can be found in chapter 2.

The components of LCCA were divided into two categories, agencies' costs and user costs. Agencies' costs include initial construction, rehabilitation and maintenance costs. Others costs, such as engineering design and land acquisition, were not considered. User costs such as vehicles operating costs, accident costs, discomfort costs etc. were considered equal for both pavement types. The only user costs to be considered will be fuel consumption costs and straight light requirement costs, because other data for user costs are difficult to collect and quantify due to lack of availability.

.

3.1.1 Data Collection

Traffic data were collected from Ebonyi State ministry of works historic data from manual. I took initial traffic data annual average daily traffic (AADT) from those manual for each specific road segment.

Construction material, labor and machinery unit costs were taken from current market values during the time of data collection, November 2021.

3.1.2 Analysis Period

Experience in the US shows the life of concrete pavement is often more than 20 years, while the life of asphalt pavement in Ethiopia is around 15 to 20 years, depending, of course, on traffic intensity and type of surfacing applied. The Federal Highway Administration (FHWA) recommends an analysis period of at least 35 years. An analysis period of 40 years was chosen for this research so that it could include full design period for concrete pavement, depending on the traffic level.

3.1.3 Discount Rate

Discount rate is used to convert the future benefits and costs of projects to present value. The higher the discount rate, the lower the net present worth of future costs will be. Thus, higher rates render initially expensive projects less profitable, while lower rates render them more so. A discount rate of 10% was used in this study as recommended by Ministry of Finance for evaluation of project feasibilities in Ebonyi State.

3.1.4 Traffic Analysis and Pavement Design

Traffic analysis and forecast were conducted using the applicable trend of traffic projection practices of the country. Two different traffic classes were selected for equivalent pavement design works for the two alternative pavement types.

Assessment was made on the existing pavement design type selection practices and conditions in our country in relation to international practices. Equivalent pavement design works were carried out for the two pavement alternatives using different traffic volumes based on the traffic analysis data.

3.1.5 Project Data Used

A 1 km length and 7.3 m width representative arterial road Ebonyi State were chosen from the five out let roads at the out skirts of Abakaliki the two alternative pavement options.

3.1.6 Data Analysis

It is the part and parcel of the main research to integrate and make analysis of the collected and computed data to come in to picture of this research. Hence, the following core tasks were carried out in the data analysis scheme.

The Present Worth method was used for cost comparison and economic analysis purpose.

Initial cost estimates were carried out for each alternative pavement structure based on current market rates of each specific cost components.

Life cycle maintenance and rehabilitation costs were adopted from international practices with some modifications to be suited for our specific country. User benefits and sustainability issues were assessed in this study. Cost comparison and economic analysis of concrete and asphalt pavement roads were conducted. The results were presented in Tables and chart form and interpretation and discussion were made on the research findings. Based on the findings conclusions are drawn.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 RESULTS

During the course of the research, certain data valuable to the research were sourced, processed and analyzed with the outcome used as basis for evaluating the economic and life cycle cost of Concrete over asphalt pavement at Isieke Ebonyi State, this data are presented in the Table below:

Table 4.1a Summary of Initial Construction Cost of 1km Concrete Pavement at Isieke Ebonyi State (Sourced and adjusted from Ministry of Works Ebonyi State).

Item No	Description	Total Cost (Naira)
1	Sub-grade level	3,240,000.00
2	Sub-base level	4,724,000.00
3	Concrete Pavement	48,728,000.00
4	Texturing and Curing	640,740.00
5	Joints	80,480.00
6	Reinforcement bars	4,458,380.00
7	Separation membrane	480,855.40
8	Subtotal	62,353,455.40
9	Contingency allowance	6,235,245.54
10	Total Cost per km	68,587,700.94

Table 4.1b: Summary of Bill of Engineering Measurement and Evaluation (BEME) for initial construction cost of asphalt pavement at Isieke Ebonyi State (Adjusted from proposed Ebonyi State Ministry of Works, BEME 2010).

Item No	Description	Unit	Qty	Rate	Amount
Bill No 1: Pavement and Sub-grade work.					
1.01	Compact earthen sub-grade material to the required level using vibratory rollers	m^2	9.800.00	300.00	2,940,000.00
	Total bill no 1: carried to summary				2,940,000.00
Bill No 2: Pavement and Sub-base work.					
2.01	Import and compact fill material at sub-base level to required depth	m^2	10,400.00	300.00	3,120,000.00
	Total bill no 2: carried to summary				3,120,000.00
Bill No 3: Pavement and Road base work.					
3.01	Import and compact granular material at base course level to required depth	m^2	13,200	350.00	4,620,000.00
	Total bill no 3: carried to summary				4,620,000.00

	Bill No 4: Pavement and Surfacing				
4.01	Provide and apply bituminous emulsion tack coat on the entire carriageway.	m^2	12,500.00	300.00	3,750,000.00
4.02	Provide and lay 100mm compacted thickness of hot rolled asphalt (HRA) wearing course constituting of 12% bitumen content.	m^2	12,800.00	2,600.00	33,280,000.00
	Total Bill No 4: carried to summary.				37,030,000.00
	General Summary				
	Bill No 1: Pavement and Sub-grade work				2,940,000.00
	Bill No 2: Pavement and Sub-base work.				3,120,000.00
	Bill No 3: Pavement and Road-base work				4,620,000.00
	Bill No 4: Pavement and Surfacing				37,030,000.00
	Subtotal				47,710,000.00
	Contingency allowance				4,771,000.00

	Total cost per km				52,481,000.00

Table 4.2a Agency (Authority) Cost of Concrete Pavement (Computed from Ebonyi State Ministry of Works Data).

S/NO	Activity	Cost (Naira)	Remark
1	Construction Cost	68,587,700.94	---------
2	Routine Maintenance Cost	2,346,740.00	Every ten years interval
3	Periodic Maintenance Cost	845,750.75	Every fourteen years interval
4	Rehabilitation Cost	950,780.50	After forty years
5	Salvage Cost	475,390.25	-----------

Table 4.2b Agency (Authority) Cost of Flexible Pavement (Computed from Ebonyi State Ministry of Works Data).

S/NO	Activity	Cost (Naira)	Remark
1	Construction Cost	52,481,000.00	--------------
2	Routine Maintenance Cost	5,608,350.00	Twice every five years
3	Periodic Maintenance Cost	2,378,000.00	Once every eight years
4	Rehabilitation Cost	3,330,304.97	At every ten years interval.
5	Salvage Cost	1,665,152.485	--------------

Table 4.3 Life Cycle Cost of Concrete and Asphalt Pavement

Description	Concrete (Naira)	Asphalt (Naira)	Difference
Initial Construction Cost	68,587,700.94	52,481,000.00	16,106,700.94
Routine Maintenance Cost	3,060,148.96	29,253,153.6	26,193,004.64
Periodic Maintenance Cost	827,144.23	3,876,140.00	3,048,995.77
Rehabilitation Cost	309,954.443	4,342,717.68	4,032,763.237
Salvage Value	-154,977.22	-542,839.71	387,862.49
Fuel Cost	132,214,779.00	131,849,105.7	365,673.3
Time Saving Cost	-1,494,675.00	-597,870.00	896,805.00
Total Cost in 40 years	203,350,075.353	220,661,407.27	17,311,331.917

4.2 ANALYSIS OF RESULTS

4.2.1 Construction Cost

The data sourced on the initial construction cost of Concrete and flexible pavement, indicate that after thorough adjustment using the current inflationary rate, the Concrete pavement was relatively higher than the flexible pavement with a cost difference of about 16.1 million. This can be attributed to the cost of materials that constitute a Concrete pavement and also from this result, it can be deduced that there is a relative economy in the construction cost of flexible pavement than that of Concrete pavement. This result mirrors the work of Yonas et al, (2016).

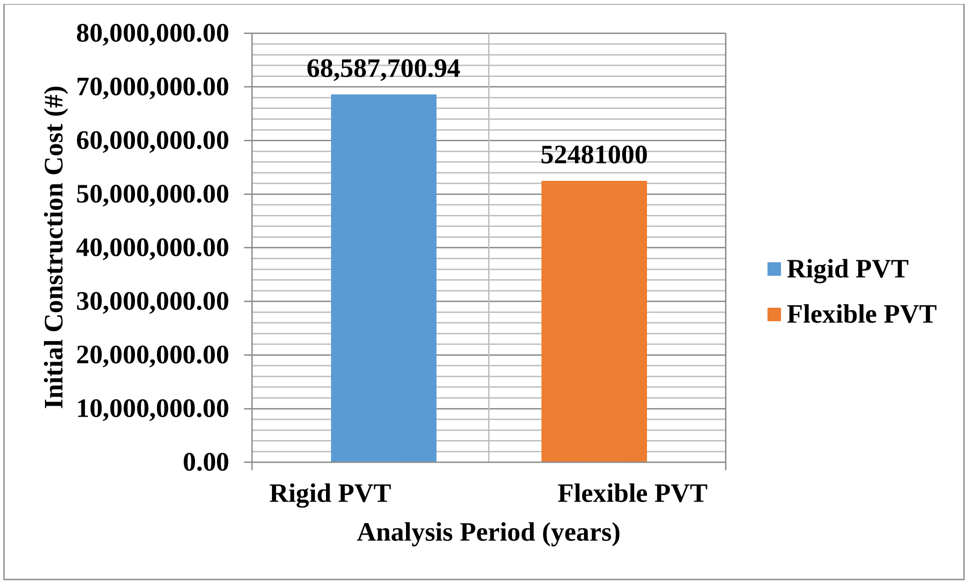
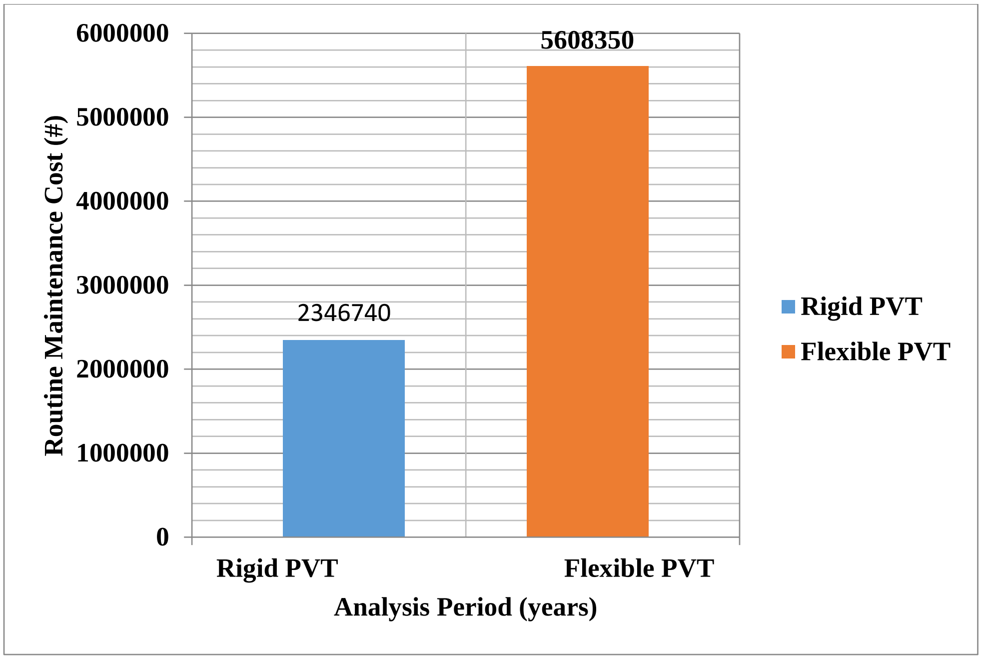


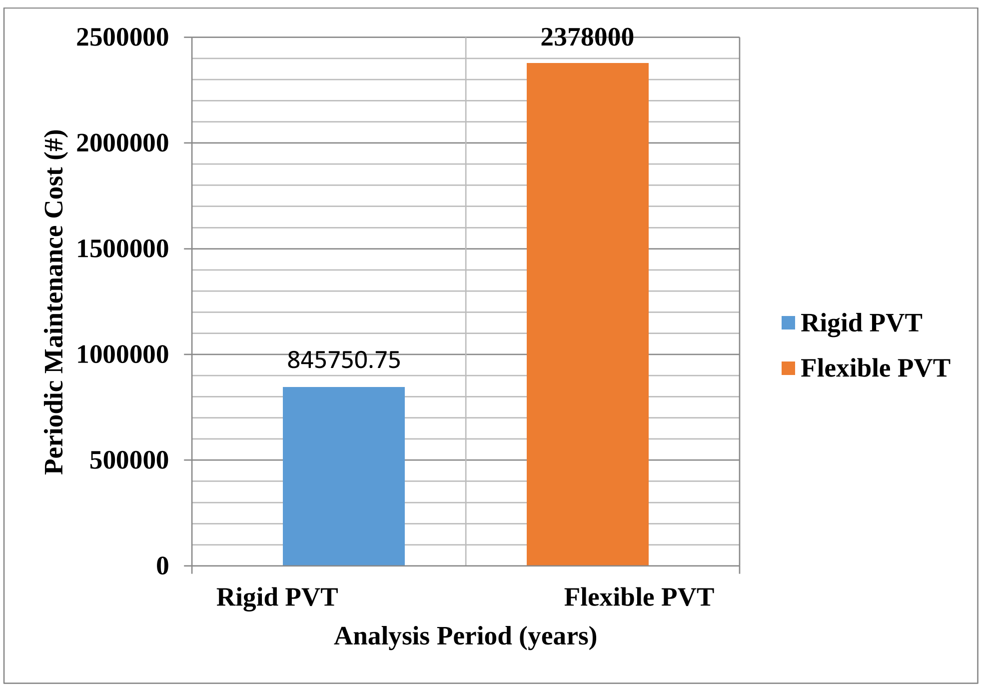
Fig 4.0: Chart showing the initial construction cost of both pavement against Analysis Period.

4.2.2 Maintenance Cost

The life time maintenance of both Concrete and flexible pavement was categorized into periodic and routine maintenance schedule. The periodic and routine maintenance of Concrete pavement was conducted at every 14 and 10 years interval while that of the flexible pavement was carried out every eight and five years respectively. This maintenance was conducted before and after the period of high rainfall intensity. Result obtained from the maintenance cost suggests that the periodic and routine maintenance cost of flexible pavement was considerably higher than that of Concrete pavement. This is mainly due to the frequency at which this maintenance was conducted as flexible pavement are susceptible to distress even at the early stage of it service life. This result is in agreement with the works of Yonas et al, (2016) and Audu et al, (2015).



4.1a: Chart showing the routine maintenance cost of both pavements against analysis period.



4.1b: Chart showing the periodic maintenance cost of both pavements against analysis period.

4.2.3 Rehabilitation Cost

The rehabilitation cost is the cost of full depth repair in Concrete pavement or removal and replacement of asphalt (resurfacing) in asphalt pavement. According to the rehabilitation activities schedule specified in the work, the Concrete pavement will be rehabilitated at the end of the analysis period (40 years) while the flexible pavement will be rehabilitated at every ten years intervals. Comparative deduction extracted from the rehabilitation cost of the competing alternatives suggests that the rehabilitation cost of flexible pavement was appreciably higher than that of Concrete pavement with a substantial discrepancy in cost. This development can be attributed to the higher frequency in rehabilitation exercise for flexible pavement which cumulatively amount to cost. Studies indicative of the results are the works of Yonas et al, (2016) and Audu et al, (2015).

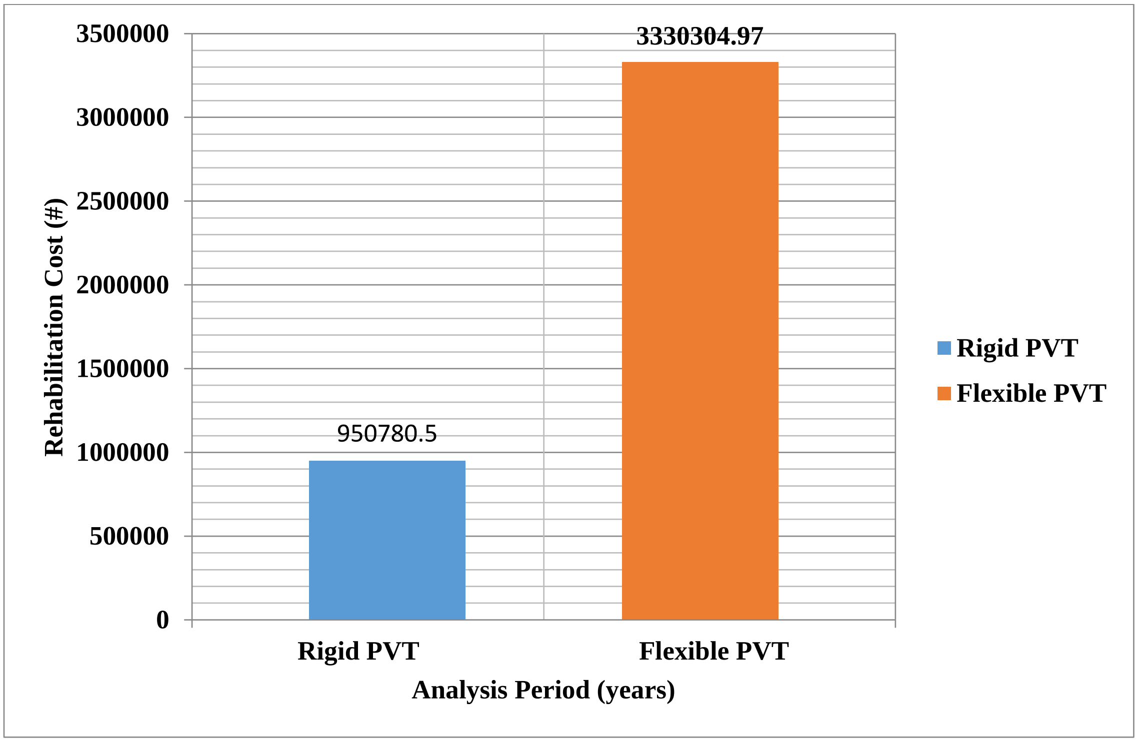


Fig 4.2: Chart showing the rehabilitation cost of both pavements against analysis period.

4.2.4 Salvage Cost

Salvage cost is the cost recovered from expenses incurred due to the rehabilitation of the competing alternatives. It is expressed mathematically as rehabilitation cost multiplied by expected remaining life of the rehabilitation cost divided by total expected life of the rehabilitation cost. This cost is deducted from the total cost incurred due to the agency and user cost of both pavements. Result obtained indicates that the salvage cost of asphalt pavement was appreciably higher than that of Concrete pavement implying higher recovery from rehabilitation cost.

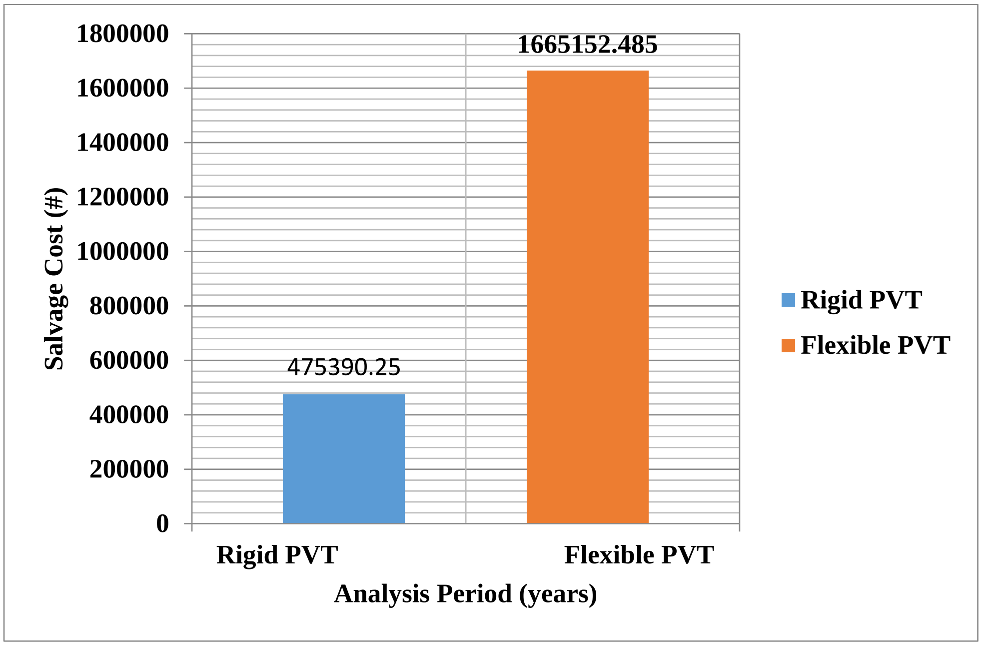


Fig 4.3: Chart showing the salvage cost of both pavements against analysis period.

4.2.5 User Cost.

The user cost comprises of both vehicle time saving and delay cost and also fuel cost for the two pavement options. The user cost was computed from the annual average daily traffic data collected from ministry of works Ebonyi state. Results obtained indicate that the time saving and delay cost of Concrete pavement were comparatively higher than that of flexible pavement while the fuel cost of flexible pavement is higher than that of Concrete pavement. This result implies that there is higher energy consumption (fuel usage) and less delay cost in flexible pavement than that of Concrete pavement.

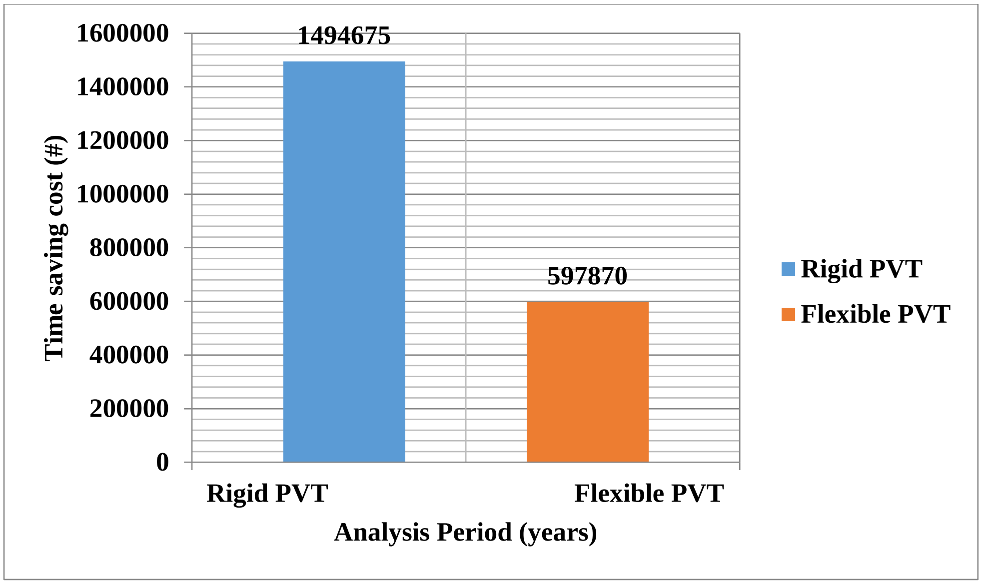


Fig 4.4a: Chart showing the time saving cost of both pavements against analysis period.

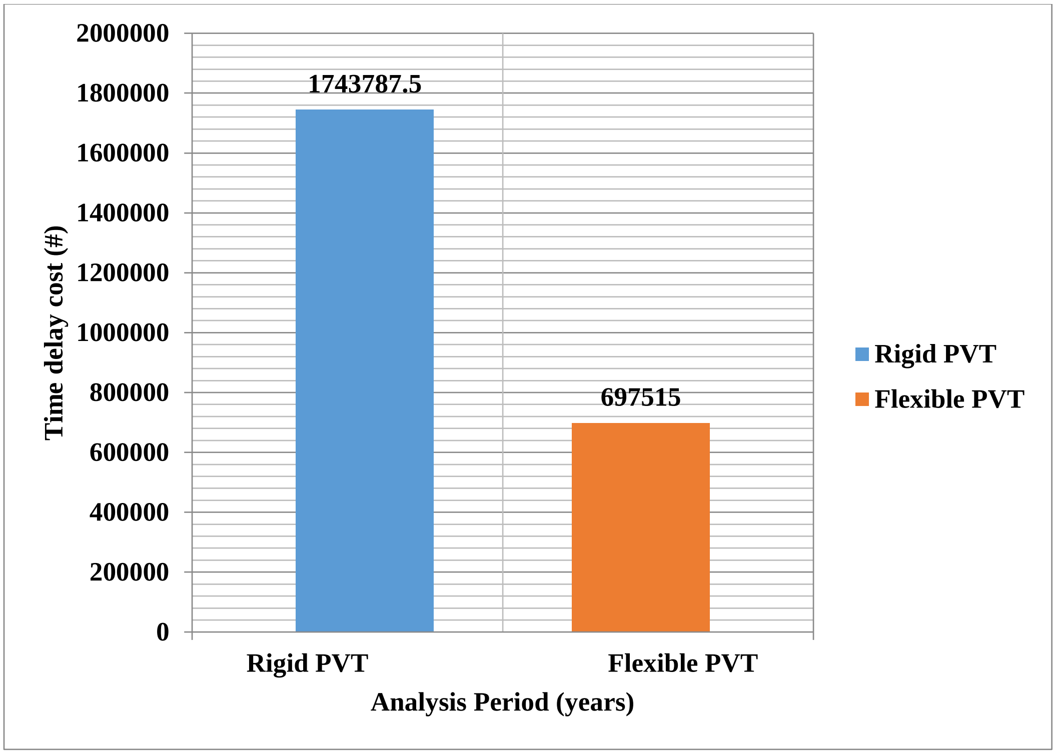


Fig 4.4b: Chart showing the time delay cost of both pavements against analysis period.

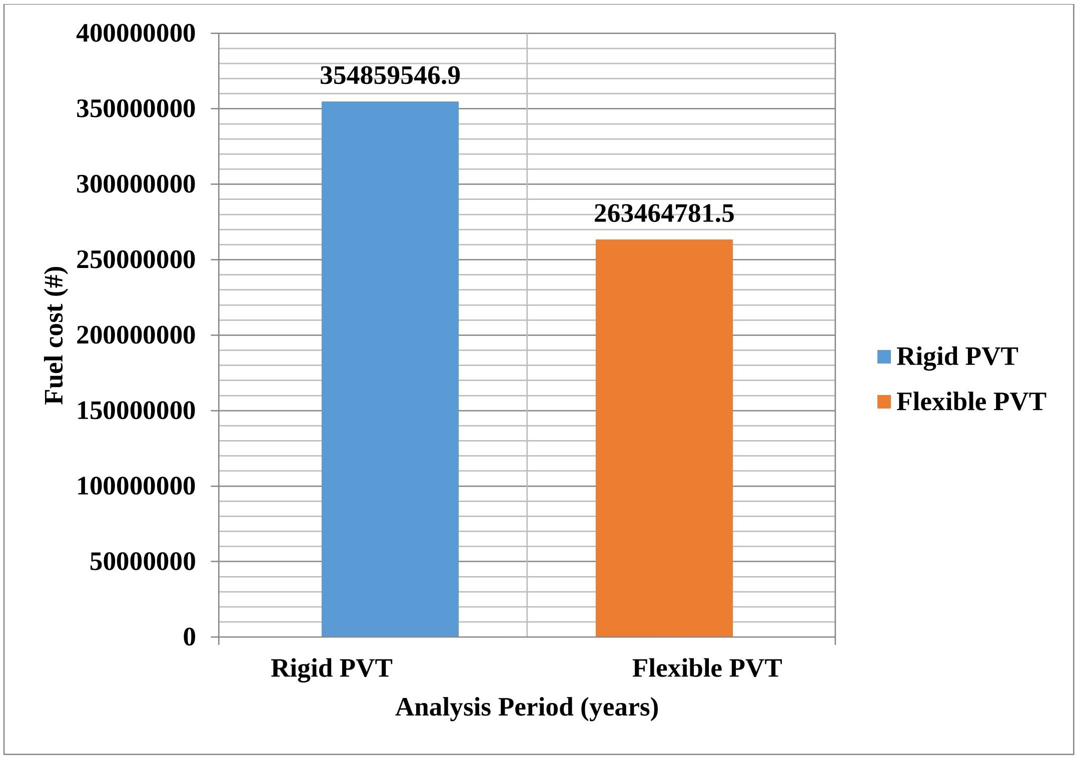


Fig 4.4b: Chart showing the fuel cost of both pavements against analysis period.

4.2.6 Life Cycle Cost.

The comparison of the economic worth of the competing alternatives was done from the findings obtained from the life cycle cost of both pavement types with an analysis period of 40 years. This result is employed as a supporting tool for investment decision on economic viable alternatives. Result obtained from the life cycle cost of both alternatives indicate that the Concrete pavement has longer service life than flexible pavement and for 1km (kilometer) road length, the life cycle cost of the Concrete pavement is lower by a value of 17.3 million in Naira than that of the asphalt pavement in forty (40) years analysis period. This result suggests that the Concrete pavement is an economically viable alternative than the asphalt pavement due to it lower life cycle cost and longer service life. Life cycle cost comparison in currency suggest that 1km (kilometer) road length in USD (United State Dollar) for both asphalt and Concrete pavement requires 537267 and 495129 USD respectively, this result is in correlation with the works of Yonas et al, (2016) and Audu et al, (2015).

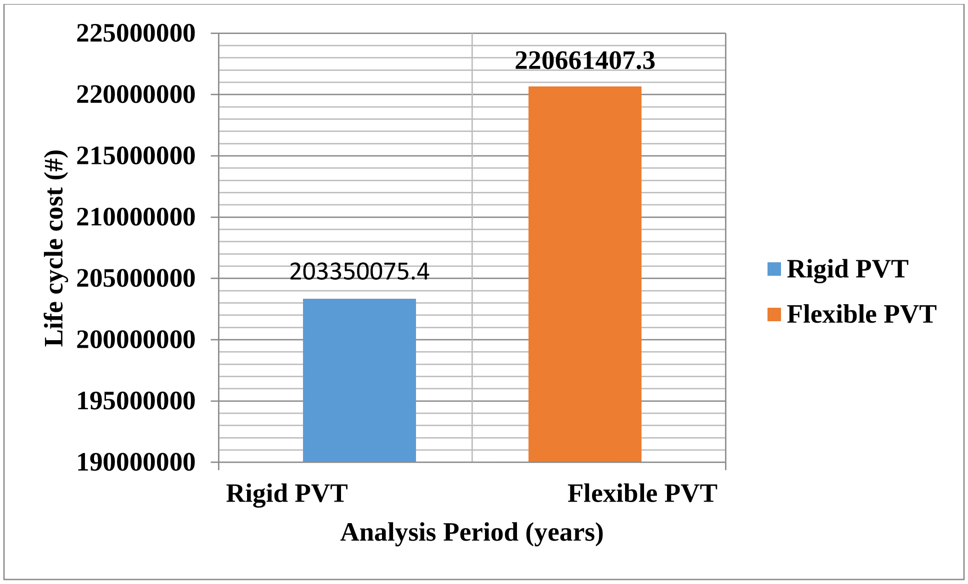


Fig 4.5a: Chart showing the life cycle cost of both pavements against analysis period in Naira.

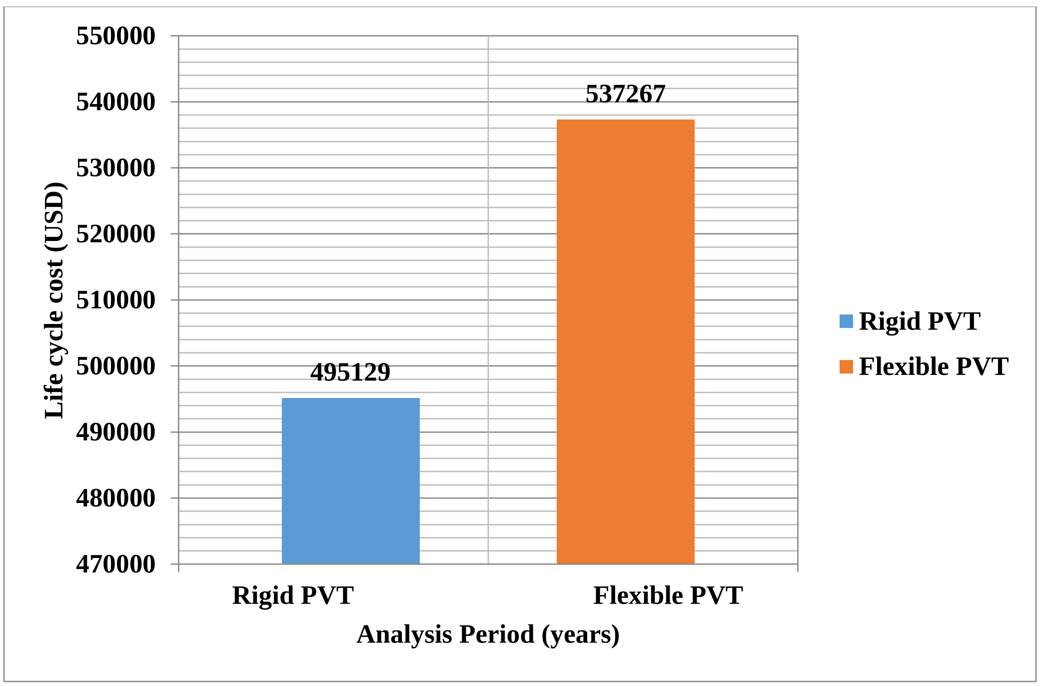


Fig 4.5b: Chart showing the life cycle cost of both pavements against analysis period in USD.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

From the findings obtained the following conclusion was made:

1. The data sourced on the initial construction cost of Concrete and asphalt pavement, indicate that after thorough adjustment using the current inflationary rate, the Concrete pavement was relatively higher than the flexible pavement with a cost difference of about 16.1 million.
2. This maintenance conducted before and after the period of high rainfall intensity suggests that the periodic and routine maintenance cost of asphalt pavement was considerably higher than that of Concrete pavement. This is mainly due to the frequency at which this maintenance was conducted as asphalt pavement are susceptible to distress even at the early stage of it service life
3. Comparative deduction extracted from the rehabilitation cost of the competing alternatives suggests that the rehabilitation cost of flexible pavement was appreciably higher than that of Concrete pavement with a substantial discrepancy in cost.
4. The salvage cost derived from the rehabilitation cost indicates that the salvage cost of asphalt pavement was appreciably higher than that of Concrete pavement implying higher recovery from rehabilitation cost.
5. Results obtained from the user cost indicate that the time saving and delay cost of Concrete pavement were comparatively higher than that of asphalt pavement while the fuel cost of asphalt pavement is higher than that of Concrete pavement.
6. Result obtained from the life cycle cost of both alternatives indicate that the Concrete pavement has longer service life than asphalt pavement and for 1km (kilometer) road length, the life cycle cost of the Concrete pavement is lower by a value of 17.3 million in Naira than that of the asphalt pavement in forty (40) years analysis period.
7. The 1km (kilometer) Concrete pavement is therefore adjudged as an economically viable alternative over asphalt pavement due to it relatively lower life cycle cost and higher service life and as a result must be given due consideration by prospective investors.

5.2 RECOMMENDATION

The following recommendation can be made from the research findings obtained:

1. Concrete pavement is adjudged as an economically feasible alternative over asphalt pavement due to it relatively lower life cycle cost and higher service life.
2. For widespread construction of Concrete pavement over asphalt pavement, this study recommends that consulting and contracting firms must convince their client over the economic and life cycle benefit of Concrete pavement over flexible pavement.
3. Academic institution must promote the economic and life cycle benefit of Concrete pavement over asphalt pavement through robust tutelage and professional bodies (NSE) must raise awareness on the necessity of Concrete pavement over asphalt pavement.

REFERENCE

Akbar, H. Energy Saving Potential and Her quality benefits of Urban Heat Island Mitigation. In first International Conference on passive and how Energy Cooking for the built Environment, Hlens, Concrete, 2010.

American Association of State Highway and Transportation Official (AASHTO) Guide for Design of pavement Structure.

American Concrete Pavement Association, white Topping State of the Practice Engineering Bulletin 210p.

Audu, H., Aniekwu, A., and Oghorodge, E. (2015). Life Cycle Cost Analysis (LCCA) Delivery Model for an Urban Flexible Pavement. Journal of Civil & Environmental Engineering. DOI: 10.4172/2165-784X.1000175.

Ministry of Works Manual (2010). Cost Specification for Construction and Maintenance of 1km (kilometer) Concrete and flexible pavement in Isieke Ebonyi State.

Tayhr and Francis Group Concrete Pavement Design Construction and Performance, Hand Book, London, 2010. Taylor and Francis, Concrete Pavement Design, Construction and Performance Book 270. Madison Ave, New Tack, Nig 10016, USA 2072

Taylor Consulting Additional Analysis of the effect of pavement structure on track fuel consumption action plan 2010 on Climate Change, Concrete Roads Advisory Committee, Government of Canada 2020.

Taylor, P.C. "New Technologies for sustainable Concrete" in Proceeding Concrete for a sustainable Environment, Concrete Society of Southern Africa Johanna Sturg S.A. 2010.

Tom Van Dan, wr L, Lye Cycle Cost Analysis of Portland Cement Concrete Pavement. Research Report, University of Texas, Andfor 2010.

Van Dan T. J. Geopolymer Concrete, FHWA Concrete Pavement Technology Program Washington, D.C. 2010.

Walls J and Smoth, M.R. Life-Cycle Cost Analysis in Pavement Design, in Search of better investment Decision, U.S. Department of Transportation, Washington D.C. 2012.

Yomas, K., Emer, T. and Getachew, K., (2016). Cost and Benefit Analysis of Concrete and Flexible Pavement: A Case Study at Chancho –Derba-Becho Road Project. International Journal of Scientific & Engineering Research, Volume 7, Issue 10, October-2016 181 ISSN 2229-5518.

APPENDICES

APPENDIX A

CONSTRUCTION COST

Table A1 Summary of Initial Construction Cost of 1km Concrete Pavement at Isieke Ebonyi State (Sourced and adjusted from Ministry of Works Ebonyi State).

Item No	Description	Total Cost (Naira)
1	Sub-grade level	3,240,000.00
2	Sub-base level	4,724,000.00
3	Concrete Pavement	48,728,000.00
4	Texturing and Curing	640,740.00
5	Joints	80,480.00
6	Reinforcement bars	4,458,380.00
7	Separation membrane	480,855.40
8	Subtotal	62,353,455.40
9	Contingency allowance	6,235,245.54
10	Total Cost per km	68,587,700.94

Table A2: Summary of Bill of Engineering Measurement and Evaluation (BEME) for initial construction cost of flexible pavement at Isieke Ebonyi State (Adjusted from proposed Ebonyi State Ministry of Works, BEME 2010).

Item No	Description	Unit	Qty	Rate	Amount
Bill No 1: Pavement and Sub-grade work.					
1.01	Compact earthen sub-grade material to the required level using vibratory rollers	m^2	9.800.00	300.00	2,940,000.00
	Total bill no 1: carried to summary				2,940,000.00
Bill No 2: Pavement and Sub-base work.					
2.01	Import and compact fill material at sub-base level to required depth	m^2	10,400.00	300.00	3,120,000.00
	Total bill no 2: carried to summary				3,120,000.00
	Bill No 3: Pavement and Road base work.				
3.01	Import and compact granular material at base course level to required depth	m^2	13,200	350.00	4,620,000.00
	Total bill no 3: carried to summary				4,620,000.00

	Bill No 4: Pavement and Surfacing				
4.01	Provide and apply bituminous emulsion tack coat on the entire carriageway.	m^2	12,500.00	300.00	3,750,000.00
4.02	Provide and lay 100mm compacted thickness of hot rolled asphalt (HRA) wearing course constituting of 12% bitumen content.	m^2	12,800.00	2,600.00	33,280,000.00
	Total Bill No 4: carried to summary.				37,030,000.00
	General Summary				
	Bill No 1: Pavement and Sub-grade work				2,940,000.00
	Bill No 2: Pavement and Sub-base work.				3,120,000.00
	Bill No 3: Pavement and Road-base work				4,620,000.00
	Bill No 4: Pavement and Surfacing				37,030,000.00

	Subtotal				47,710,000.00
	Contingency allowance				4,771,000.00
	Total cost per km				52,481,000.00

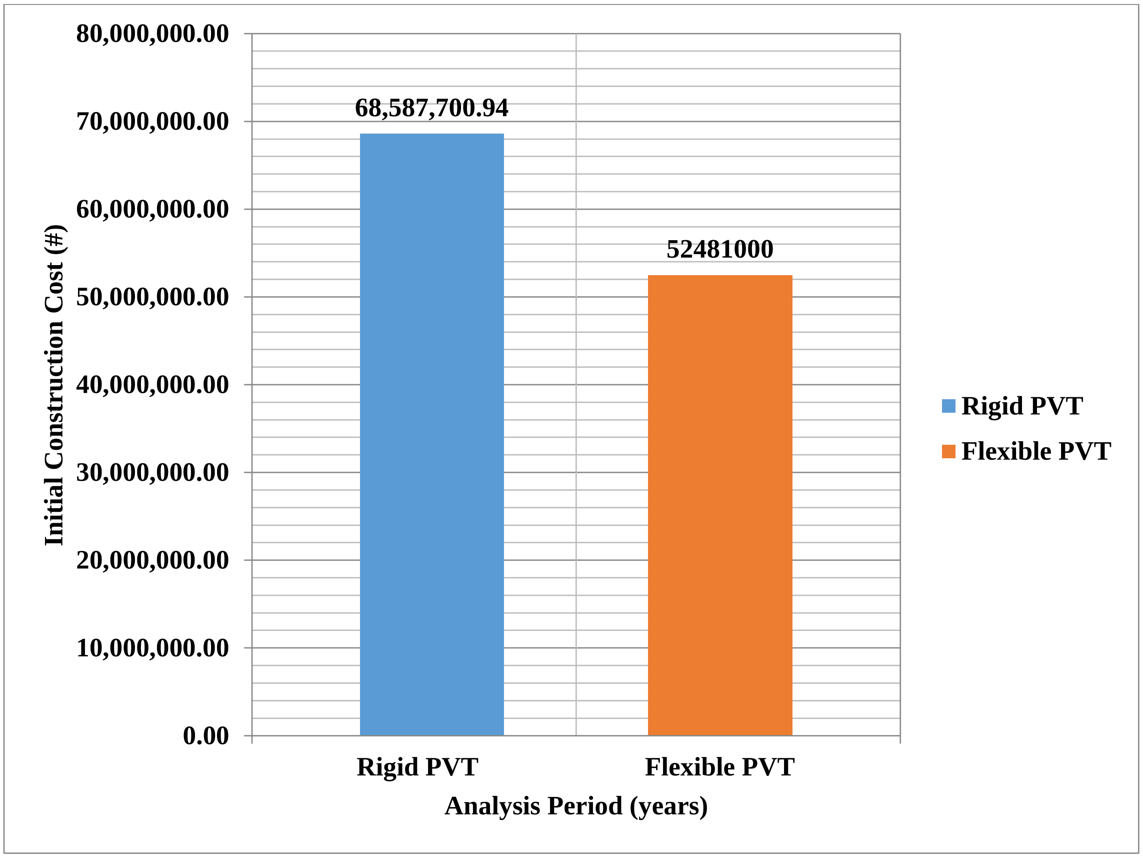


Fig A1: Chart showing the construction cost of both pavements against analysis period.

APPENDIX B

MAINTENANCE AND REHABILITATION COST

Table B1 Flexible Pavement Routine Maintenance Schedule (Sourced and adjusted from Ministry of Works Ebonyi State).

Frequency in a year	Month of a year	Routine Maintenance Activities	Cost (Naira)	Remark
1st	March	Potholes and chip seals	2,484,350.00	Twice (2) a year before and after the rainy season.
	April			
	May			
2nd	October	Potholes Patching	3,124,000.00	
	November			
	December			
			5,608,350	

Table B2 Flexible Pavement Periodic Maintenance Schedule (Sourced and adjusted from Ministry of Works Ebonyi State).

Frequency in a year	**Month of a year**	**Periodic Maintenance Activities**	**Cost (Naira)**	**Remark**
1st	**March**	**Pothole repair and repainting of white line.**	**2,378,000.00**	**Once every year before and after the rainy season.**
	April			
	May			
			2,378,000.00	

Table B3 Schedule of Maintenance and Rehabilitation Cost of 1km Concrete Pavement (Sourced and Modified from Ministry of Works Ebonyi State.)

Maintenance Type	**Activity**	**Cost (Naira)**	**Frequency in years**
Routine Maintenance	Joint sealing and crack sealing	2,346,740.00	Every 10 years interval
Periodic Maintenance	Partial depth repair	845,750.75	Every 14 years interval
Rehabilitation	Full depth repair	950,780.50	At 40 years

APPENDIX C

AGENCY AND USER COST

Table C1 Agency (Authority) Cost of Concrete Pavement (Computed from Ebonyi State Ministry of Works data).

S/NO	Activity	Cost (Naira)	Remark
1	Construction Cost	68,587,700.94	---------
2	Routine Maintenance Cost	2,346,740.00	Every ten years interval
3	Periodic Maintenance Cost	845,750.75	Every fourteen years interval
4	Rehabilitation Cost	950,780.50	After forty years
5	Salvage Cost	475,390.25	-----------

Table C2 Data of Vehicle Time Saving Cost Computation for both Pavement (Data Sourced from Ministry of Works Ebonyi State).

Vehicle Type	AADT (2021-2035)	Composition (%)	No of Vehicles	Time Saving/day/km	Time Saving/day
Small bus	454	42	191	0.3	57.3
Medium bus	454	4	18	0.3	5.4
Large bus	454	3	14	0.3	4.2
Medium Truck	454	26	118	0.3	35.4
Heavy Truck	454	13	59	0.3	17.7
Truck Trailer	454	12	55	0.3	16.5
Total	454	100	455	18	136.5

Table C3 Agency (Authority) Cost of Flexible Pavement (Computed from Ebonyi State Ministry of Works data).

S/NO	Activity	Cost (Naira)	Remark
1	Construction Cost	52,481,000.00	
2	Routine Maintenance Cost	5,608,350.00	Twice every five years
3	Periodic Maintenance Cost	2,378,000.00	Once every eight years
4	Rehabilitation Cost	3,330,304.97	At every ten years interval.
5	Salvage Cost	1,665,152.485	

APPENDIX D

LIFE CYCLE COST

Table D1 Life Cycle Cost of 1km Concrete and Flexible Pavement.

Description	**Concrete (Naira)**	**Flexible (Naira)**	**Difference**
Initial Construction Cost	68,587,700.94	52,481,000.00	16,106,700.94
Routine Maintenance Cost	3,060,148.96	29,253,153.6	26,193,004.64
Periodic Maintenance Cost	827,144.23	3,876,140.00	3,048,995.77
Rehabilitation Cost	309,954.443	4,342,717.68	4,032,763.237
Salvage Value	-154,977.22	-542,839.71	387,862.49
Fuel Cost	132,214,779.00	131,849,105.7	365,673.3
Time Saving Cost	-1,494,675.00	-597,870.00	896,805.00
Total Cost in 40 years	203,350,075.353	220,661,407.27	17,311,331.917

INVESTIGATION INTO THE SAFETY CONSIDERATIONS FOR PEDESTRIANS ALONG THE AWKA SEGMENT OF ENUGU-ONITSHA EXPRESSWAY

BY

OKAFOR FAVOUR KASIEMOBI

(NAU/ 2016224034)

SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING, FACULTY OF ENGINEERING

NNAMDI AZIKIWE UNIVERSITY AWKA, ANAMBRA STATE

IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR IN ENGINEERING (B.ENG) DEGREE

CIVIL ENGINEERING

FEBRUARY, 2022

CERTIFICATION

This is to certify that the research project on "Investigation into the safety considearations for pedestrians along Enugu-Onitsha expressway" was carried out by Okafor Favour Kasiemobi with registration number (NAU/2016224034) and submitted to the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State, Nigeria.

______________________________	______________________________
Okafor Favour Kasiemobi	Date
(Student)	

APPROVAL PAGE

This research work titled “Investigation into the safety considerations for pedestrians along Enugu-Onitsha expressway” was carried out by Okafor Favour Kasiemobi with registration number (NAU/2016224034) and submitted to the Department of Civil Engineering, Nnamdi Azikiwe University, Awka for approval in partial fulfillment for the award of Bachelor of Engineering (B. Eng) Degree.

Rev. (Dr) C. M. Nwakaire
(Project Supervisor)

Date

Engr.Dr. C. A. Ezeagu
(Head of Department)

Date

Engr. Prof. D.O. Onwuka
(External Examiner)

Date

DEDICATION

This research project is solely dedicated to my Abba Father, the Almighty God, for his undeserved grace and love he showed me throughout my stay in this university.

ACKNOWLEDGEMENTS

I express my profound gratitude to God the father, the son and the sweet Holy Spirit for their role in making my dream come through.

I wish to say a special thank you to my mum Mrs patience Okafor Ukaamaka ,my uncle Mr Aneke Chinwuba and my sibling Joseph and Joshua Okafor for their love and commitment towards my growth and journey. God bless you all.

My immense gratitude also goes to my lecturer and project supervisor Rev.(Dr) C. M. Nwakaire under whose guidance and supervision this research work was carried out. I also thank him for his relentless effort, tolerance, encouragement and professional criticism throughout this research process.

I am also grateful to all the staff and lecturer in the department of civil engineering for the role they played in lecturing and guiding my academic life in the university.

I also appreciate the help of my senior colleagues for their acceptance and teachings during this course. I also wish to thank the entire physical infrastructure department of Tertiary education trust fund (TETFUND) for their help during my industrial training.

Lastly, I must acknowledge my friends and course mates; Stella, Flower, Cynthia, Onyinye, Ebube, Ayomide, Chisom, Winnifred Divine, Evans and all others. God continue to uplift you all for me.

ABSTRACT

This research is an attempt to investigate into the safety consideration for pedestrians along the Awka segment of the Enugu-Onitsha express way in Anambra Nigeria. The Awka segment of the highway was of interest, Awka is the capital of Anambra state with obvious commercial activities going on at both sides of the carriageway. Problems such as congestion, collisions and hostile vehicle to pedestrian relationship have been observed on this research location. Therefore it is important to investigate and find possible remedies to this. There are three major intersections along this location, they are; Regina, Aroma and Unizik junction. The volume of pedestrians with respect to behaviors that affect their safety were gotten at this junctions by manual counting , direct and indirect interviews were also carried out on pedestrians. The Regina junction recorded the highest volume pedestrians crossing the carriageway, which is over 50% of the total pedestrian volume at that time. The Aroma junction had 745 people as the highest number of pedestrians walking along the junction during a one hour period. This junctions lack crossing or walking facilities, hence obvious difficulties were observed for pedestrians using these intersections. The Unizik junction recorded the most number of pedestrians using their mobile phones on the road, which was over 35% of the pedestrians crossing at that time. From the interviews carried out, it was analyzed that motorists have a hostile attitude towards pedestrians which makes using this junctions without traffic and safety facilities difficult for pedestrians . It was recommended that there should be provision of facilities that would resolve pedestrian related issues, awareness campaign for road users on the right attitude to increase safety and maintenance of already existing facilities.

TABLE OF CONTENT

Title Page……………………………………….……………………..……….……i

Certification……………………………………….…………………...…………….ii

Dedication …………………………………….....……………….……………iii

Approval page………………………...………….…………………………....iv

Acknowledgements……………..……………..……………………………..…v

Abstract……………………..……………………………….…………………..vi

Table of content…………..………………………………………………....vii

List of tables……………….………………………………………..…………..ix

List of figures………………..…………………………………………………x

List of plates………………………………………………………… …………..xi

CHAPTER ONE: INTRODUCTION

1.1 Background of study…………..……………..……………………………..1

1.2 Statement of problem………………..…….………………………….…3

1.3 Aims and objectives of study……………..……...……………………………5

1.4 Scope of work………………...……………………………..……………..5

CHAPTER TWO: LITERATURE REVIEW

2.1 Pedestrians and their challenges……………………………………......……..7

2.2 Age as a factor that affects pedestrians safety…………....……………………..8

2.3 Distraction from mobile phones………………...…………………….11

2.4 Vulnerable road users (VRU)……………….…….……….….……....14

2.5 Safety enforcement measures for pedestrians…….…….…………….15

2.6 Significance of creating and maintaining a planned road network…...16

2.7 Maintenance of road network…………………...………………….18

2.8 Design consideration for pedestrians…….……………….……..……19

2.9 Difficulties for pedestrians…………………….. ..………………....23

2.10 Considerations for pedestrians……………..……………………..…..27

2.11 Summary of literature review……………….. ..………………………29

CHAPTER THREE: MATERIALS AND METHODS

3.1 Method of collecting data……..........………………………………..34

3.2 Selected survey location…………………........………………………37

CHAPTER FOUR: RESULTS AND ANALYSIS

4.1. General data analysis 40

4.2. Descriptive analysis.......................... 40

4.3. Result analysis from the questionnaire...43

4.4 Analysis on direct interview……………...........…………......……….53

4.5 Other analysis on this research location…..…............…...……………..55

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1 Conclusion ……………………………………………………………….60

5.2 Recommendation……………………………..…………………………61

REFERENCES

APPENDICES

LIST OF TABLES

Table 4.1: descriptive statistics of pedestrian count…..…………………………………….....41

Table 4.2: Questionnaire data result…....……………………….………………………… 46

LIST OF FIGURES

Figure1.1 map view of awka segment, enugu Onitsha express way………………………4

Figure 3.1. Flow chart of methodology for data collection…………..…………………….34

Figure 4.1 Chart analysis of pedestrians on Sunday…………………………..….....…43

Figure 4.2 Chart analysis of pedestrians on Thursday……………………...………..…45

Figure 4.3 chart analysis comparing pedestrian walking along and crossing Regina and Unizik junction……………………......................................…………………………..…..........49

Figure4.4 chart analysis on Tuesday……….….................…………………………..50

Figure 4.5 Chart analysis of total pedestrian volume on different days. ….........................51

LIST OF PLATES

Plate 3.1 picture of Aroma junction intersection…………………….……………......38

Plate 3.2 Picture of the Regina junction intersection………………….……………..39

Plate 3.4. Picture of the Unizik junction ....,...…………..39

Plate 4.1 Pedestrian carrying a heavy load, walking along the junction…............40

Plate 4.2 Motorists parking in front of pedestrians waiting to cross the road. ..…40

Plate 4.3 pictures of pedestrians crossing the Regina junction carriage way . ..…41

Plat 4.4 Potholes on the refuge island …………….……………………………..48

CHAPTER ONE

INTRODUCTION

1.1 Background of study

A pedestrian is considered as a person rather walking than travelling with a vehicle. Walking and crossing roadblocks is a minor part of pedestrian activity on the road, but this is known to be one of the activities with the most risk uncertainty due to collision with vehicles. Most of these pedestrian collisions occur at crosswalks, which has been recorded numerous times as the location with the highest risk (Hamed,2010).

A pedestrian crosswalk is a segment of the road that is allocated to pedestrians. The crosswalks can be located at the end of an intersection or middle of the street block.

Hamed (2021) had observed that whilst there are several types of crosswalks, marked crosswalks which is one of the most convenient is not seen. The location of roads, crosswalks, and behavior of pedestrians are factors that have a significant effect on regularities and level of crashes. They also determine the impact of the crash outcome on the pedestrian.

The behavior of a pedestrian on the road is influenced by many factors, and demographic variables is one of the major factors. This covers the gender and age of pedestrians. Age is considered as one of the elements that could affect pedestrian behavior on the highway. Li et al. (2013) analyzed characteristics of three groups of pedestrians pair while crossing the street in Albany, Oregon United States of America (USA) ;adults, adult-child pairs, and children alone. Coffin and Morrall (1995) found that the walking speed of elderly pedestrians above 60 years of age at the same street at midblock crossings and marked intersections overseen to be around 3.3 ft./s (1.01 m/s) to 3.9 fits./s (1.19 m/s). Gates et al. (2006) evaluated the performance of 1947 pedestrian using an 11 crossing Intersections in Eugene, Oregon USA, and found that elderly pedestrians greater than 65years of age were the slowest among all age groups. Groups of pedestrians crossed the freeway at 0.40 ft./s (0.12 m/s) to 0.60 ft./s (0.18 m/s) slower than pedestrians crossing alone. Thus, a recommended walking speed of 4.0 ft./s (1.22 m/s) was not sufficient time to cross the road for most vulnerable road users (aged, children, persons with disability) or large groups of pedestrians. Hence, there is a need for special considerations for vulnerable road users(elderly, children, and disabled), and this also has an effect on the frequency of road accidents that involves pedestrians.

The frequency of road accidents in Nigeria has been a critical concern. The death toll shows that every twelve hours, not fewer than eight lives are lost on Nigerian roads. The daily toll is 15 persons and a cumulative of 426 persons every month. Every year, about 20,000 of the 11.854 million vehicles in the country are involved in accidents, 20% of lives lost are due to pedestrian casualties, an analysis of data released by the National Bureau of Statistics (NBS)and the Federal Road Safety Commission(FRSC). In research shown by the FRSC (2016), there were 12,077 road accidents of which 5,400 lives were lost in the year 2015.There were 11,363 crashes with 5,053 deaths in 2016 as reported by NBS; and in 2017, a total of 10,026 crashes and 5,049 deaths.

According to Hamed (2012), distraction of pedestrians at roadway crossing increased the Hazard of pedestrian-vehicle crashes. Advanced Placement warnings, pavement markings, and traffic signals are being utilized as different traffic control equipments to increase pedestrians 'safety at crosswalks. Some elements could be introduced at crosswalks to make the pedestrian more visible to the motorist and increase the awareness of the motorist to stop on the approach to the crosswalks. These countermeasures comprise slower vehicle speeds, shorter crossing distances, enhanced visibility for both pedestrians and vehicles, and the adequate configuration of traffic control devices.

1.2 Statement of problem

Awka is the capital of Anambra state in Nigeria, a high rising and fast-developing city. This city is known as a core for hospitality, this matches with the number of high and beautiful structures, ranging from hotels, business plazas, learning institutions, hospitals, malls and event spaces. Along the area of study (Onitsha-Enugu expressway), particularly from the Unizik express gate to the Unizik junction, many important infrastructures that links to the highway exist, this is an area with high traffic volume of pedestrians during morning hours, especially in front of the university gate, Aroma junction and Unizik junction. Furthermore, the availability of shops, cafes, eateries, hotels, and infrastructure where people visit during different hours of the day, brings constant traffic of people during the night and day time, hence the road has a high recording of accidents in Anambra state. Map view of this research location is shown in Figure 1.1. The lack of adequate traffic system on this road is a major contributor to the accident and collision of pedestrians.

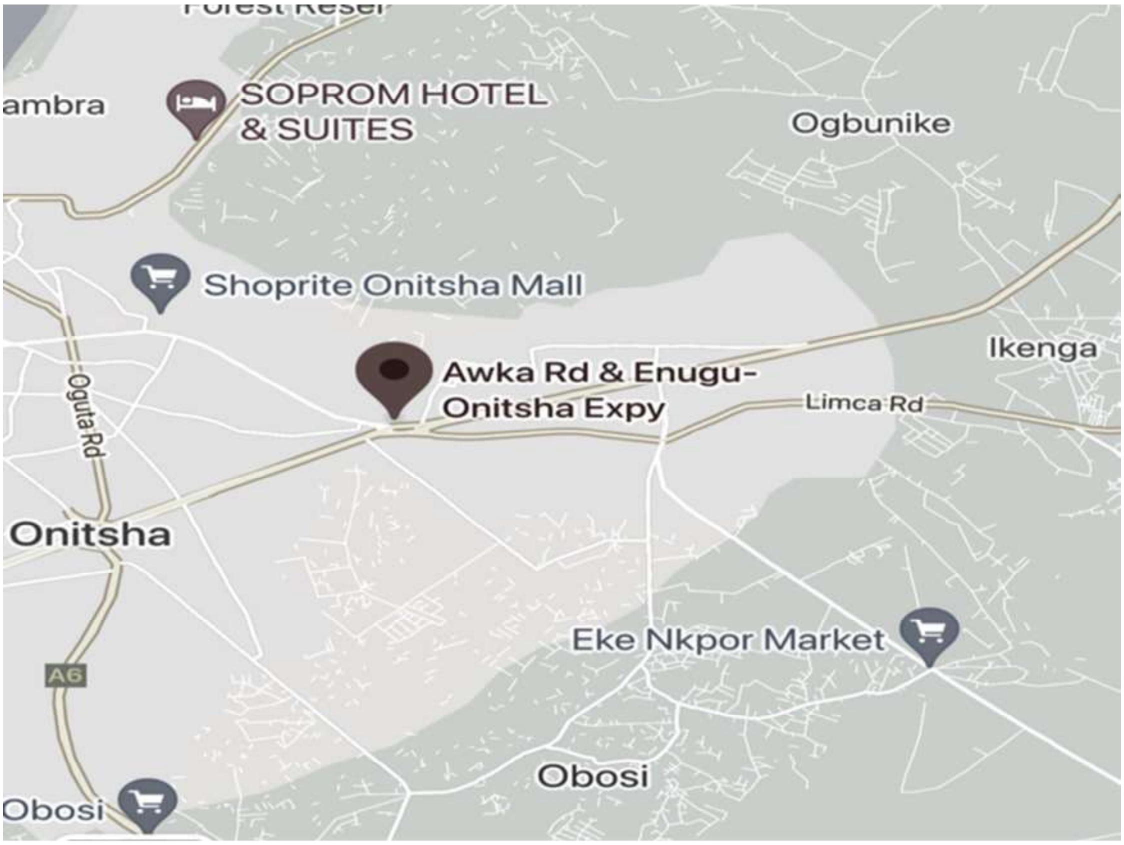


Figure 1.1 View of Awka segment , Enugu Onitsha express way.

This research area lacks enough crossing opportunities like signaled controlled crossings and pedestrian bridges that are supposed to be provided at regular convenient intervals to improve safety of crossing. The provision of these controlled crossings and implementation of a good and effective traffic system with the consideration of pedestrians will lessen the accident experienced on this road. Some sections of this research area already have signaled controlled traffics but they are not in use due to lack of maintenance, some of the traffic lights have not been working for a long period, and this has affected the traffic systems on this intersection.

In some sections of this research area, the road designs do not put into consideration pedestrians and vulnerable road users. A major provision lacking in this area is accessible paths for road users like sidewalks, transit stops, pedestrian bridges and refuge islands. Pedestrians are seen walking on the sides of the roadway due to lack of a special path for them and this has had a high contribution to collisions.

1.3 Aim and Objectives

The aim of this study is to investigate into the safety considerations for pedestrians along the Awka segment of Enugu Onitsha expressway. The objectives are;

i. To identify pedestrian behaviors that cause and increases more crashing outcome on this research location, through pedestrian count and observation along the major intersection.

ii. To offer countermeasures that reduce risk of pedestrian-vehicle crashes, by distributing questionnaires along this research location and making necessary evaluation of pedestrian problems.

iii. To identify and analyze design facilities needed on this research location, by observation of already existing and non existing facilities, major pedestrian activity and needs on different intersection in the research location.

1.4. Scope of work.

This project research is concerned with general considerations that can be adopted for the pedestrians in the area, the need to adapt facilities like pedestrian bridges, pedestrian crossings, and road keens. Crossings controlled by authorized persons and other measures needed for a planned road network and how to effectively implement them is strongly canvassed throughout the text. It contains five chapters covering some aspect of pedestrian considerations from planning feasibility study and environmental impact assessment to traffic facilities and recommendations for management of these facilities.

This research brings pedestrians into limelight as it pertains to road design. The text also represents more background information that gives a clear overview of the pedestrian to motorists relationship.

This study includes an analysis of distracted pedestrian crossing behavior, identifying the problems pedestrians face on the highway that affect their crossing speed and pedestrians experience with motorists. In brief, road network that gives equal consideration and attention to both pedestrians’ safety and other traffic demands.

CHAPTER TWO
LITERATURE REVIEW

2.1 Pedestrians and their challenges

Contrary to popular beliefs that pedestrians are larger victims of road accidents, pedestrians are also a big factor of casualties on the road, they inflate the risk of road accidents and they are often flexible and unstable on the highway. In many analyses, some pedestrian activities like talking phone calls while crossing the street, disregarding road signals like traffic lights, crossing a red light, and crossing at unsuitable places (crossing dangerous lines) are referred to as pedestrians' unsafe behaviors. Furthermore, low perceived risk of unsafe behaviors, consumption of alcoholic beverages, poor vision on the roads, and minor distraction are also reported as other behavioral factors that contribute to pedestrian accidents. Lack of adequate traffic indications and environmental factors are also a cause of unsafe conditions for pedestrians. As an instance, there's a sense of insecurity experienced by older pedestrians as a result of inadequate crossing facilities on the crosswalk. prior to this literature, some factors like ignoring pedestrians needs on the road by drivers, avoiding the use of facilities that assist pedestrians to flow on the road, lacking a sense of safety in using the facilities during dark hours of the day, lack of pedestrian facilities like escalators and bridges, and the pile-up of traders along pedestrian bridges are also reported as hurdles to pedestrian safety (Hakeem, 2010).

There are vulnerable road users in pedestrians, but pedestrians as a whole are the most vulnerable on the highway.25% of road accident deaths, injuries and casualties recorded in India are pedestrians. Pedestrian deaths and injuries are often preventable and yet pedestrian safety does not command the awareness it warrants (Salem, 2018).

According to the Haghighi et al.(2012) the challenges associated to pedestrian safety were evaluated under six sectors including "Challenges related to pedestrians", "Challenges related to drivers", "Management system challenges", "Environmental infrastructure challenges", "Educational and media challenges" and "Challenges of legislation and enforcement". According to these outcomes, it was concluded that in developing countries, pedestrian safety was a formidable subject in urban transport systems with evident complexities, particularly in the supervision network system.

2.2 Age as a factor that affects pedestrians safety

The walking speed of pedestrians on the highway can be affected by gender, age, the level of urgency to the pedestrian, if he/she is carrying a heavy load, accompanied by a child, or has a disability. Statistics have then shown that younger people face more risk than older people, however, older people are prone to suffer more injuries. Older people are more at risk as a pedestrian because albeit the collision rate among older pedestrians been lower,(census data of Britain 2001 shows that 20% of the population are over 60 years old), injuries to older people over 65 years old tend to be more severe and fatal than that of other age groups (department for transport 2004, AA foundation 1994 and TRL unpublished research undertaken as part of the study by the wall, 2000).

In 2003, National statistics shows that 0.5% of collisions to child pedestrians under 16 years old resulted in severe casualties, while 21% of adults over 60years old involved in road collisions were recorded deceased.

In 2003, there were 36405 pedestrians collisions reported on the Great Britain freeway.60% of those involved were male and the remaining 40% we're female pedestrians according to the government statistics . Research has also indicated that men, most especially young men are mostly involved in collision than women.

In research by Zhuo and Sixian (2019), In four peak hours, 4196 pedestrians are observed, among them, 328 pedestrians (7.82%) are using their mobile phones encompassing of162male pedestrians and 166 female pedestrians. The average phone use rate in different age groups is presented as follows: 4.49% are teenagers , 10.69% are youth, 6.87% are middle-aged , and 1.15% are old -aged . Age is a significant factor in terms of a pedestrian character on the crosswalk.

Statistically significant relationships were observed between age and the perceived crossing behaviors. Waiting time is at the lowest for individuals in their twenties, and then steadily increased as the age increases. Waiting time ranged from 19.0 s been the minimum for pedestrians in their twenties to a maximum of 59.1 s for pedestrians in their seventies (Ferenchak,2015).

In research by Niebuhr (2016), using some assumptions, a model-based approach to injury risk, allowing for the specification of individual injury risk parameters for individuals is presented. The data set is divided into three age groups; children from 1-14 years, adults from 15-60 years of age, and older adults older than 60. Individual risk curves are computed for every group .The derived model addresses the influence of age on the outcome of pedestrians to car collisions. The results show that compared to younger people, older people have an increased injury risk at all collision speeds. The injury risk for children behaves is low compared to other age groups, their injury risk is lower at lower collision speeds but substantially increases once a threshold has been exceeded. At this stage in life, younger people are allowed and perceived to make decisions by themselves, which can affect their health and well-being in the long term and short term.

The research suggests that in spite of their supposition that they know what to do around roads and traffic, some factors combine to put young people at greater risk of injury than the middle and older age groups. Risks associated with being young include:

1. Oversee other pedestrian crossing the road.
2. New levels of independence.
3. Lack of experience with alcohol and experimentation with alcohol and drugs
4. Still cultivating maturity, hazard perception and decision-making skills
5. Peer pressure.
6. Risk-taking tendencies.
7. Catching public transport.

2.3 Effect of gender on the behavior of pedestrians

The gender of a pedestrian is a very important characteristic considered in deducing the pedestrian behaviors such as waiting time and inclination towards risk. In particular

r, male pedestrians have been perceived to be keener to violate rules and make unsafe crossing decisions. They are also less likely to perceive risk when crossing a roadway in the presence of a motor. This harmony between males and higher rates of risky behavior has even been exhibited in young kids aged 5-8 years old. Female pedestrians also tend to wait for longer amounts of time than male pedestrians when crossing a roadway. Likewise, male pedestrians have reasonably faster-walking speeds than their female equivalents, probably linking to their lesser waiting times. As is predicted, males make up to 80% of pedestrian casualties. In extension to pedestrian behavior being strongly conditional on biological gender, it has also been seen to depend on the psychological masculinity of a person.

According to a research by Ferencliak (2015), the normal waiting time of female pedestrians is 34.4 s, which is 88.0% higher than the average waiting time of male pedestrians of 18.3 s. In terms of the available crossing treatments used in the research, only 39.8% of males and 53.2% of females used functional crosswalks. This demonstrates that females are over one-third more likely to use crossing infrastructure than males. The most remarkable discovery related to gender is the difference between males and females in terms of conflict with motorists while crossing. While only 14.3% of females caused a conflict with a motor vehicle, 33.1% of males caused a conflict. This shows a 131.5% rise in the probability of causing a conflict, which is statistically critical at the 99% confidence level and a strong trend that demonstrates significant safety behavior differences between the two genders.

2.4 Distraction from mobile phones.

A combination of the attitude toward the behavior, biased norm, and awareness of behavioral control is known as Intention. These three variables will take out the most regular factor that influences the intention of the pedestrian to cross the road Distractions while walking constitute to factors that influence the intention and behaviors of pedestrians. The average pedestrian is seen to use their mobile phone while walking rather than pay attention, even when crossing a road, Using a phone is a risk to the drivers' and pedestrian safety, and also a major distraction to the pedestrians, especially for the new generation. Records has reflected that Pedestrians were less conscious of the traffic, took off in a less safe time between their crossing and the next arriving vehicle, experienced more collisions with vehicles, close calls with incoming traffic, and waited longer before starting to cross the street because they've been distracted by the phone usage (Hameed, 2021).

According to Zhou and Sixian (2019), talking on the phone is the main usage of mobile phones at the intersection located at commercial district , while text messaging or playing games is the most common use type at crossings located at the school and residential area.

In this research, Some data research is performed to evaluate the relationship between the use of mobile phones and pedestrians' observing behavior. Among all the pedestrians who do not use the phone, the percentage ratio of pedestrians who observe the traffic to those who do not observe be the traffic is 68% to 31%.Also for the pedestrians who use the phone, the ratio of those who observe the traffic is 39.02%, while 60.98% of them do not observe the traffic .The significance of the topic of using mobile phones and headphones by pedestrians is validated by the number of available papers and documents that denote the existence of this dilemma. The use of mobile phones and headphones is characterized as a major cause of pedestrian accidents. For example in the document Pedestrian safety, a survey was conducted in New York in 2018 by the University of Northern Arizona, according to the results, 9% of pedestrians were on their mobile phones when crossing the road. Regardless, according to statistics for the entire United state of America (USA) for the duration 2010-2015, just 0.2% of pedestrians sustained severe accidents caused by the use of electronic gadgets, and a minimum of related cases was documented in the city. In addition, the New York City Transportation Department (NYCTD) is committed in different educational activities such as the Cross this Way campaign, which focuses on the threats of using mobile gadgets by road users , so they chose to centralize on speed, based on these facts, the law has ultimately not been adopted.

The same problems have been recognized on the other side of the globe, in Australia. In 2018, the Pedestrian Council put forward a proposal to the government of Australia to fine pedestrians using electronic gadgets while crossing the road with charges of up to USD 200. This penalty also applies if a pedestrian crosses a road at a signalized crossing when the light is green. Certain charges for pedestrians have already been initiated, For instance, a sanction of up to USD 80 has been imposed for crossing the road outside the pedestrian crossing. Harold Scruby, chairman of the Pedestrian Council, insisted that the fine for pedestrians for using mobile devices and headphones at the crossings must be increased, it would be one of the major ways to ensure pedestrians' safety. Speed management is much more than setting and enforcing appropriate speed limits. It employs a range of regulations in engineering, enforcement, and education, to balance safety and efficient vehicle speed on the road network. Younger people have a bigger percent of speed limit problems, but information on a likely law for this group of pedestrians ensued as early as 2020. (Mikusova, Wachnicka & Zukowska, 2020). Granted that mobile phones are increasingly common among all age groups in daily life, and considering their dominant use in traffic scenarios (e.g., drivers, pedestrians), the problem may even worsen in the future. To reduce pedestrian fatality due to distraction-related to mobile gadget use, it is essential to observe such unsafe behavior among road users, along with their intention to cross under the condition of being diverted.

2.5 Vulnerable road users (VRU).

According to the road statistics by World health organization (WHO), more than 3000 deaths are caused by traffic accidents worldwide per day, among which half are VRUs (pedestrians, children, cyclist, aged, etc.), showing a strong need for clues to protect VRUs.

Vulnerable road users typical walking speed is 1.4m/s (5 km/h) . Pedestrians may walk unaccompanied or in groups of different sizes. Walking speed of VRU varies by age and physical proficiency or disability. Based on VRU's physical features , they may further be categorized into the following groups:

1. Aged—This group holds to the typical qualities of VRU, such as low speed and consistency.
2. Children/teenagers—This group may show characteristics such as unpredictable consistency, delay in walking or running.
3. physically disadvantaged persons—This group may demonstrate characteristics such as slow walking and may use some aid (e.g., cane, wheelchair, or a guide dog).

Pedestrians embody a significant faction of road users and are one of the most vulnerable groups. According to the most recent findings, vulnerable road-users account for 70% of road deaths in urban societies. Pedestrians represent 21% of total casualties. They account for a vast amount of road traffic fatalities and their vulnerability index is also on the highest level. (Mikusova, Wachnicka & Zukowska 2020)

According to WHO, Each year about a million people die, and ten times that figure

are severely injured on the world's roads. The World Health
Organization has indicated that, for people above 3 years to 35 years of age, road traffic crashes are now the leading cause of their death and disability.

2.6 Safety enforcement measures for pedestrians.

According to Salem (2018), In an article on measures to guarantee pedestrians, safety, some of the important measures are :

1. Minimizing pedestrians exposure to traffic; There are a number of engineering regulations to ensure that pedestrians have minimal to no exposure to vehicular traffic. A great number of this measures involve operating vehicles from pedestrians, hence reducing traffic volume, building of usable sidewalks for pedestrians, building pedestrian overpasses and under passes, design of mass transport routes , putting up signaled and marked crossings. Marked crossings help to indicate pedestrian right-of-way and drivers need to yield to pedestrians at these points of intersection, The main aim of the marked crossings is to specify the most convenient location for pedestrians to cross, and signaled crossings help shoe the best time for pedestrians to cross, hence giving them a feeling of safety. Pedestrian safety is a key issue to consider in the design of any highway transport system, including paths and stops.

2. Reducing vehicle speeds: measures put in place to reduce vehicular speed is one of major ways to ensure pedestrians safety. Speed supervision is more than just setting and implementing a given speed limit, it entails a range of restriction on the highway, enforcement, and empowerment in engineering to harmonize safety, security and efficient vehicle speed on the highway system.

3. Improving pedestrian visibility: Low lightening condition on the highway has

caused a high percentage of deaths and casualties on the highway for pedestrians. There are so many measures and details put in place to ensure an improved lightening on the highway, this helps make pedestrians visible to drivers especially and night, dusk or dawn.

4) Empowering pedestrian and motorist about safety awareness and behavior: altering the attitude and behavior of drivers and pedestrians is a huge long term measure undertaken to bring a series of improvement on the highway. Interventions that can be put in place for this changes are; Education, outreach and empowerment programs, training and campaigns. This will result to better pedestrian behaviour and a reduction in pedestrian to vehicle collision , but this also depends on the community support, exposure and risk perception, social models and norms, engineering enforcement laws and regulations. Strategic and planned media awareness, social media campaigns enlightening the public about the pedestrian risk factors and the necessary laws to abide by to enhance the driver and pedestrian relationship, and improve understanding of traffic problems such as ignoring traffic signs and marked crosswalks for all road users.

2.7 Significance of creating and maintaining a planned road network.

Transport has become a significant subject issue for geographers for two major motives. Firstly, transport is a very crucial human activity with a vital spatial element. Lastly, it is a significant component influencing the spatial distinction of several public and economic activities in the world. Transport is a means by which people and products are carried from one location to another by a number of ranging physical modes like water, roads, airlines, railways, and pipelines. So transport in one form or the other is a fundamental and crucial aspect of the daily pulse of life throughout the world. Transport is considered an important part of life and a very essential feature of all modern economies. In social terms, as the economy of a country grows and develops, it develops more intent relationship with its transport sector, Development of the road transport network and highest sector plays a significant role in the economic development of a country and, therefore the kilometers-age of paved roads existing in a country is constantly used as an index to classify the degree of its development (Aldagheiri, 2007).

Roads make a sufficient amount of contribution to economic development and progress and bring significant social benefits. They are importance factors required in order to make nation maintain development and growth .Improving and giving access to employment, health, social , civic and education services makes a road network vital enough in revolting against poverty. Roads introduce more areas and community, it also promotes economic and social development. This makes highway infrastructure and development the most important of all national asset. Maintaining and improving road maintenance will attract so many benefits by preserving current assets whilst also lowering future costs of maintenance for citizens, pedestrians, motorists, tax payers and road owners. This can only be valuable and effective if the maintenance of this assets are done periodically and on time (Gülay, 2015).

Road transportation investment is frequently based on multiple purposes, such as providing high tiers of mobility, protection, dependability, improving social equity, and guaranteeing elevated standards of environmental protection. Besides, highway infrastructure maintenance and building are recognized by the public opinion and by many citizens in high decision ranking as an important instrument to promote economic development (and regional growth). In fact, these objectives are, in general, considered by most decision makers as the ultimate goals for developing massive road infrastructure investments plans. However, regardless of the public consensus about the positive impact that road investment has on economic development, available empirical studies suggest that the impact is difficult to ascertain (Bruno, 2009).

2.8 Maintenance of road network.

Road property management and supervision involve the application of engineering, physical, financial, and management practices to optimize the level of service result in exchange for the most cost-effective economic input. Certainly, the main purpose is to apply the right service at the right time to obtain the desired degree of contribution, reflecting that the highway infrastructure is a financial asset for the community and the economy. Concentrating on road maintenance will bring substantial benefits by conserving current assets, and also reducing future costs for citizens, pedestrians, motorists, taxpayers, and road owners. This would prevent these assets from depreciating in value, provided that timely services are carried out in time (Gülay,2015)

Road network planning comprises of the optimization of a road network by either improving the existing roads , adding new roads, or connecting existing road networks. In most of the works reported in the literature, the objective is to reduce travel costs for a given travel demand, while respecting a particular budget limitation. There are several other objectives and intents contemplated in the road network planning literature e.g., the maximization of user benefits, the maximization of accessibility, and the maximization of equity in benefit distribution (Santos, 2009),

2.9 Design consideration for pedestrians.

Drivers safety are neglected in designing for pedestrians, but they are more essential in designing for pedestrians' safety. Acceleration and deceleration time, characteristics of drivers reaction time and vehicular dimension, this are all vital in drivers safety, but function better as an element of pedestrians safety.

Significant considerations in design for pedestrians are; their basic movement patterns, road facilities like roundabouts, junction configuration, crossing details, design details, signaled crossing and interaction with other pedestrians. New approaches been considered in deigning for pedestrians are ;combined bicycle and pedestrians route, new and rebranded street network system to reduce vehicle speed and encourage pedestrians moving with a feeling of security, measurement methods to determine the levels of benefit and auditing of new and existing systems.

After the pandemic in the year 2020, there has been a consultation draft aimed at reducing the dangers of pedestrians and cyclists; the launch of a gear change strategy by the government policy of England promising funds and measures for the walking and cycling infrastructure to aid the establishment of active travel England, an offset for road users to enforce elevated standards on the road. The department of transport has published strong traffic measures that includes: traffic level managements, low traffic neighborhood, where motor vehicles are filtered out of urban areas .Institution of civil engineers,(ICE)(2021).

According to a report by the guardian,(2018). On the 27 March 2016, the UK government published a draft on the cycling and walking investment strategy to be implemented in the other to reverse a recent decline in pedestrians interaction on the street, to make cycling or walking a shorter paths and a better choice for a journey that have longer paths, this strategy has an objective of ; doubling the rate of cycling activity by 2025, reducing the rate of pedestrians killed or seriously injured on the road, increase the percentage of vulnerable road users especially school children that use the road on a normal day and reverse the decline of walking activity, the strategies put in place are; creating a different route for cyclists, traffic systems and cards, putting campaigns and programs in place to get information and recommendation from the pedestrians and cyclists themselves .

In designing to improve pedestrian safety, it is important to observe and appreciate some behavioral characteristics of pedestrians on the highway, the body area, walking rates, pedestrians ability and disability and the capacity for pedestrian-related facilities. Sight distance is a critical design element if considering pedestrians and it takes forms of stopping sight distance , decision sight distance and passing sight distance. Cross section elements like lane width , shoulders, curbs, traffic barriers, walk away, and medians should be taken into consideration. The intersection on a road network system is a vital instrument in easing the insecurity of pedestrians on the road, moreover, adding other systems to this intersection such as refuge and channelization islands, curb ramps, flared curbs, turning radii, traffic control devices, drainage, bus stops, street furniture and parking way.

Transportation engineers should consider vulnerable road users in order to design well efficient facilities for pedestrians. The good and efficient pedestrian design promotes the smooth cooperation of pedestrians into the flow of traffic among larger vehicles. Most facilities needed to aid the use of the road by pedestrians with disability are in operation with design consideration for well-to-do pedestrians but at this state efficiently designed with their disabilities in mind .

The basis of a pedestrian friendly environment is the community structure , the community transportation system provides for all the modes of transport action in a balanced manner whilst creating a pedestrian safe environment and this is possible through appropriate land use and development. The recommended regulations needed to arrive at a pedestrian-oriented growth in a community, such as; sub regions, regions, whole cities, or sub region of cities should be surveyed in order to produce a pedestrian-oriented community. Community structure considers use of land, physical details, and the circulation structure. It is also made up of construction blocks generated at a pedestrian scale: regions, hallways, centers, neighborhoods, horizons, seams, and layers(Owens, 1998).

Designing, enforcing, maintaining and operating on efficient road networks and pedestrians facility is a critical element in the highway system. The solutions required to aid a better design for the pedestrian facility are organized into the following ; roadway and building design , ITS technology ,Trail design, Traffic measures and management, on-street parking improvement, designing for pedestrians with special needs, and street crossing. Among the many factors that are put as the basis for pedestrians design, if engineers do not consider disabled pedestrians, the elements of the design is not complete. A high percent of pedestrians with full life expectancy will at some point experience some sort of disability. Design elements specially considered for pedestrians with disabilities include; clear side walk width, wheel chair ramp, side walk cross slope, temporary work zones, tightening curb , adequate median, pedestrian signal timing ,construction zones, placement and designs .

In a research by Nwakaire et al (2017), On the design considerations necessary for pedestrian environment in Nigerian roads, a manual and demographic traffic count was carried out along the research location to know the policies needed for relating the efficiency of pedestrian activity and to formulate recommendable elements on the road. The result showed that pedestrians engaged in more walking than crossing activity notwithstanding the lack of pedestrians facilities in the study area. The difficulties on this road includes ; lack of sidewalks , lack of walk ways, inadequate traffic signals and road markings, poor lightening ,hostile relationships between pedestrians and motorist, poor road aesthetics and lack of sit-outs and shades.

According to this research, the remedies for this difficulties in the study location requires help from both government and non government organizations, and this could be carried out if a great reputation is given to the pedestrian safety . Some of this remedies include; improved design and road materials , increased illumination and lighting during dark hours, public information and highway campaigns or programs, and enforcement of traffic laws.

A report on the design of a 2 way road to accommodate pedestrians used manual counting to determine the traffic volume of pedestrians and vehicles on the road.

This measurement was done to obtain the sidewalk and crosswalk volume demand used in the design, te facilities provided after the design were recommended to be less than the yearly count in other to save cost and enhance the economical factor of the design. The other pedestrian facilities recommended for this road network are; zebra and wombat crossings, children crossings, finalized intersections, design of islands, and design of bus shelters (Braimoh & Gana 2018)

In a research by Muhammad (2020), the crossing facilities, usage and contributions to pedestrians safety in Damaturu, a local town in Nasarawa state, Nigeria, were evaluated. In this research, the method of the research was divided into two segments, the manual pedestrian count and crossing count survey done in 4 sectors of the study location. The result of this survey shows that there were over 3000 pedestrian in this location, and 80% were walking with traffic while 20% we're walking against the traffic signals, 20 of pedestrians used the designated crossing facility while 800 use the undesignated facility. Also Amongst 1000 respondents only 10% could identify at least 4 different types of pedestrian crossing signs, while 30, 40 and 20% could identify only1, 2, and 3 different types respectively. Zebra crossing happened to be the most recognized crossing sign, as 60% of the respondents understand what a zebra crossing is for. Fifty percent (50%) of the respondents do not understand what children crossing meant for.

2.10 Difficulties for pedestrians.

Pedestrian to car collision has a high result in fatal injury, this is a major problem in the Nigerian traffic system. Pedestrians death have a higher percentage in urban areas than in rural areas. Regardless, the ratio of death to injury is higher in rural area due to the impact of high speed on rural roads compared to urban roads. A demographic of pedestrian accident shows that elderly pedestrians are more likely to die after a minor accident compared to younger pedestrians, even though younger pedestrians are more involved in pedestrian to vehicle collisions.

Accidents occur largely at intersections then any other location, but accidents that affect pedestrians occur in almost every part of the road. And a major attribute of this can be related to some of the following reasons;.

1. Over speeding: speeding not only means driving too fast, or above the posted limit, it also means driving too fast for a particular road condition or weather like ice or rain, it can also mean driving too fast for a road without traffic facility or under construction. over speeding is one of the major causes of accidents on the highway. Over speeding is relative to the law and speed limit of that particular path. The most dangers caused by this reckless act of speeding includes ;increased accident, high severity of accident injuries or sometimes death, increasing the stopping distance of a vehicle , and a reduced reaction time. There is an 85% of death chance when a pedestrian is involved in a motor collision at a speed of 45mph, a 45% chance at a speed of 32mph, and a 5% chance at a speed of 20mph. This goes to show how much over-speeding affects pedestrians' experience on the highway. There are so many reasons why motorists speed, sometimes not related to aggressive behavior, most times it's due to the road condition or traffic situation at that particular period. Some reasons why motorists speed includes running late due to traffic congestion, trying to avoid a specifically timed congestion ahead of them, lack of awareness of the consequence of speeding, or simple disregard for the highway rules and regulations. It has been observed that drivers often mismanaged the provision of smooth roads and in turn, use this as an avenue for speeding. Very good and smooth roads have been seen to be a major factor that wills drivers to Over speed.
2. Careless driving: Careless driving falls below the standard expected of drivers

and driving without consideration for other road users. careless driving is as a result of negligence by the driver, attentiveness to the road ahead, intoxicated drivers and aggressive driving. This occurs when drivers do not consider pedestrians on the highway, they often forget that there are vulnerable road users that their actions may affect. Careless driving includes overtaking at the wrong time and path, running a red light, forgetting to use traffic signals, using intersections for the wrong purpose, tailgating, sudden braking, diving on the pedestrian path, failing to check mirrors before overtaking, staying in an unsafe distance with other vehicle and generally disregarding traffic laws. According to the Highway traffic acts (HTA) Ontario, careless driving includes the driver not driving carefully enough, the driver not paying enough attention to their driving, driving without due care and consideration for other persons using the roadway, or not considering the road conditions. (Santos, 2001)

3. Crossing and waiting time of pedestrians; pedestrians crossing speed and waiting time plays a major role in their safety and the design of traffic facility. It has been observed that the waiting time of pedestrians varies with the intersection, gender, age, place, waiting for location, and vehicle flow. Elderly pedestrians have a slower crossing speed and waiting time which makes them more susceptible on the road when crossing. Findings show that younger pedestrians do not wait more than 20-30seconds to cross the road, unlike elderly pedestrians. Traffic signals are designed to regulate the movement of pedestrians and improve their safety, enough time should be provided for their waiting time and crossing time to make them comfortable to the necessary extent when crossing the road. Waiting time for pedestrians should be considered differently for their specific nature (disabled, aged and children),

because this different pedestrians are vulnerable in different ways. The crossing vulnerability of pedestrians during waiting time should be considered. Furthermore the vulnerability of pedestrians increases with their increase in waiting time, and over increased waiting time make impatient pedestrians violate traffic rule which puts them at risk. For crossing speed, this is a major factor of unsafe pedestrian relationships on the highway, but it is mostly by the pedestrians themselves (Owen,2007)

4. Poor highway and road facilities: lack of highway facilities or poor effectiveness of the facilities has proven to be a major cause of casualties on the highway. Lack of street lights, pedestrian bridges, crosswalks, low lighting and lack of signs and road aesthetics. This facilities do not only give a safe and secure movement of the pedestrians, it also helps motorists understand and use the road in favour of them and pedestrians. Road and highway facilities include garage parking lots, telephone boots and services stations. Transportation is a major part of our lives as humans, we spend a high percentage of the day on the road, this demands a safe and standardized road facilities .
5. Poor pedestrian behavior : According to Hameed (2001), pedestrians have a role to play in their safety also, but most of them do not place importance to this. Bad road behaviors like using mobile phones, head phones, talking, and careless crossing affect the rate of accident and cavities on the highway,
6. Pedestrian volume at intersections; crash prediction models developed (Eenink et al., 2008). The study shows that the number of crashes increases with an increasing traffic volume. This increase is not a proportional one, however. This means that the crash rate, which is here defined as ‘the number of crashes per motor vehicle kilometer, decreases with increasing traffic volume.

However, this traffic volume to crash rate depends a lot on the road design and intersections.

The highway as a national facility was created and developed for all citizens, both pedestrians and motorists, but the crash data in a way contradicts this. Pedestrians have a high percentage on the crash data released annually, more pedestrians loose their live than the number of vehicle damages. This data brought the burden of this research topic along this major road in Awka city.

However, there are no available data that shows the time and volume relationship pedestrians have on the highway to fully understand the causes and triggers to this road casualty affecting pedestrians. The average time pedestrians spend on the highway, The volume of pedestrians, Their crossing speed, their waiting time, the behavior that put them more at risk, the facilities needed to ensure pedestrian safety, and the necessary measures that can be imposed to reduce this casualty.

2.11 Considerations for pedestrians.

The term considerations according to the Cambridge University dictionary refer to the act of carefully thinking about a particular act or subject before making a decision. It might be thinking about the feelings of people and the consequences an action might have on them. In the term of this research, considerations refer to the thoughts and actions that should be put into place during design to help pedestrians have a more secure and safe movement on the pathway of Enugu-Onitsha express road.

Design considerations are areas that may affect the outcome, requirement, policies, effectiveness and operational concept of a design system and should be part of the engineering process all through the preconstruction stage.

Some considerations are concepts that assist the system and accommodate every action taken, while others are limitations, constraints, and boundaries that affect the system and need to be negotiated on to have an effective design.

Design considerations are required to bring to the attention of the designer the accessibility and design systems to building and facilities.They are guidelines and tools needed by the designer, sometimes they are gotten from relevant design manuals while some other times they're inspired by the needs of the facility users. In addition to engineering principles and manuals, these considerations have to also incorporate practical findings, analysis from surveys in the case studies, interviews from the users and professionals. Architectural service departments. (2018)

This research area is a major road passing through the city of awka, connecting the Anambra state to a neighboring state. The critical locations on this major road are ; the Unizik express school gate, Unizik junction, Aroma junction, and Regina. There are various groups of pedestrians using this road due to the infrastructure inside this critical location, they include; University students, primary and secondary school kids, workers, the aged, and disabled. These pedestrians have and need special considerations to ensure they use

2.12 Summary of literature review.

This literature review covers the effect of pedestrian behavior on their relationship with other road users, their safety on the road, challenges and considerations necessary for a well planned road network. The significance of vulnerable road users safety, their challenges and difficulties is discussed in details. From analysis of research recorded in this chapter, it can be concluded that pedestrians are often neglected in road planning and design.

Considerations necessary for a safe and well planned road network and pedestrians safety needs were also recommended.

CHAPTER THREE

METHODOLOGY

3.1 Method of collecting data.

As a way to explain the traffic problems in a research location and provide solutions that'll aid better consideration and systems, engineers are required to collect data and information. The focus is aimed at collecting data related to the traffic volume, pedestrian behavior and traffic facilities available. Pedestrian traffic volume of pedestrians is important and can be applicable in different ways. For instance it could be used to forecast future trends in society, decide safety analysis study, determine position of traffic infrastructure and help in modification of this infrastructure. Traffic surveys are important in planning, designing, and making major organized decisions on the highway. Location for pedestrian survey are important. They include intersections, sidewalks, and midblock crossing (Alutman ,2009).

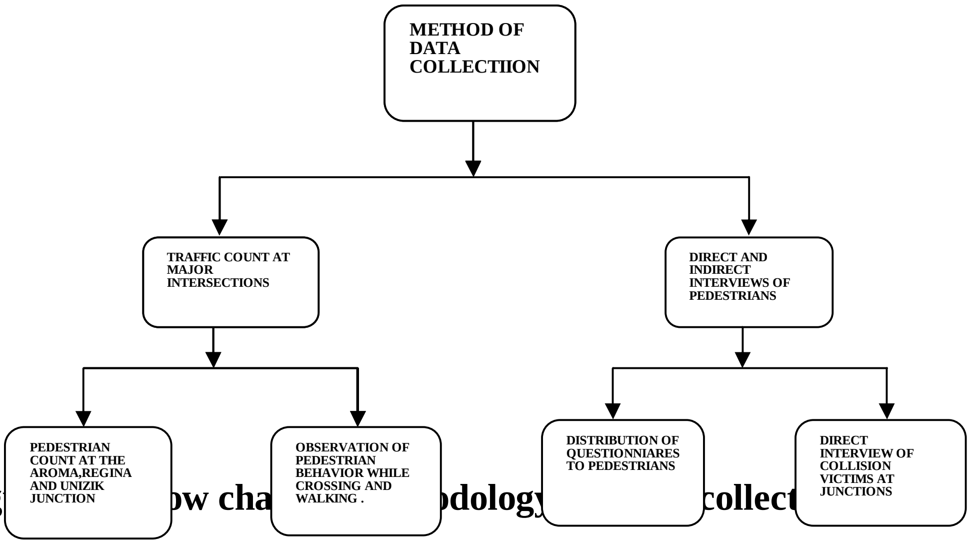


Fig ... ow cha ... dolog ... collect

Data collection was done through manual observation first. A pedestrian count survey was conducted along the Awka segment of the Enugu onitsha express way. The survey were on four days; Tuesday to Thursday representing weekdays and Sunday representing weekends. The traffic count of people was carried out over a one hour period for every of this location on the four days; 4-5pm, 5-6pm and 6;7pm. Three of this locations have different pedestrian facility features. The Aroma junction is with faulty traffic lights, the Regina junction has no pedestrian facility, while the Unizik junction has a pedestrian overhead bridge. The pedestrian count data gotten from this count is used to access the volume of pedestrians along the intersection that the questionnaires and pedestrian observation was conducted, pedestrian to vehicle relationship, congestion, and also compare the pedestrian count on the intersection with pedestrian facility to the intersection without any. Furthermore, direct interviews were conducted on pedestrians that have experienced collision on the intersection without pedestrian facilities.

The pedestrian count conducted on this road was done with respect to pedestrian behaviors; their use of phones, if the pedestrian was with a child, if the pedestrian is just walking along the junction or crossing the intersection. This count was done this way to help observe the major behavior of pedestrians on this research location that makes pedestrians more vulnerable to accidents of any form. The count of pedestrians using their mobile phones and carrying loads were only done for pedestrians walking along the junction of Regina and Aroma only, since it was noticed that pedestrians are often focused while crossing the carriage way without any crossing facility. For the Unizik traffic count and observation, the observation for pedestrian using their mobile phones and carrying loads were recorded for those crossing the pedestrian bridge and those walking along. Children were excluded from this count, because the observation during this count were done for pedestrians conciopus of their actions.

On the traffic count done on Sunday, that of Regina and Unizik were done at the same time, this was done this way to help compare data at this two intersections since they're 1km apart and have a difference of the availability of pedestrian facility.

Direct and indirect interview were also carried out in the data collection of this research study. A hardcopy interview guide in form of a questionnaire was shared, received and analyzed.

The questionnaire (appendix 1.0) was shared among 100 pedestrians walking along the Regina and Aroma junction intersection; this questionnaire had covered a few questions regarding pedestrian problems, thoughts, their preferences, and exposure on pedestrian safety and facilities. 92 of these questionnaires were collected back from the pedestrians. This survey question covered pedestrians preference on traffic facilities , their view on motorist attitude and safety issues along the Regina and Aroma junction. This interview guide was shared with only participants along the Regina and Aroma junction, and this is because they have no pedestrian facility and are closer to the Unizik junction with a pedestrian bridge. This was considered to help give the questionnaire to pedestrians with better knowledge and experience of this questions asked. To increase data validity, two criteria including dependability and credibility were considered. This participants were pedestrians seen present on the two different days of the traffic count survey, hence they are considered to be regular road users of this location.

After the collection of the questionnaire from the pedestrians given, a direct interview on 4 pedestrians that were once victim of accidents on this intersections was carried out. 10 minutes personal interview was conducted with them, were they shared their collision experience and the assumed causes .

3.2 Selected survey location

Awka is a fast developing city in Anambra state, with different roads and road intersections. The research area connects 3 busy junctions Aroma, Regina and Unizik to many other intersection leading to towns in Anambra state. This expressway is a two way , three lane highway . Hence it is characterized by a heavy traffic volume daily. The research survey was done on the tempsite junction with a pedestrian overhead bridge, Regina and aroma junction; this are the two major junctions with intersections and no or poor pedestrian facility. The Aroma and Regina junction are shown in plate 3.1 and 3.2 respectively.

This selected survey locations are the major locations with observed high traffic volume. The aroma junction has a traffic light at the intersections as shown in Plate 3.2, but this facilities are not been used as supposed, because they are faulty.

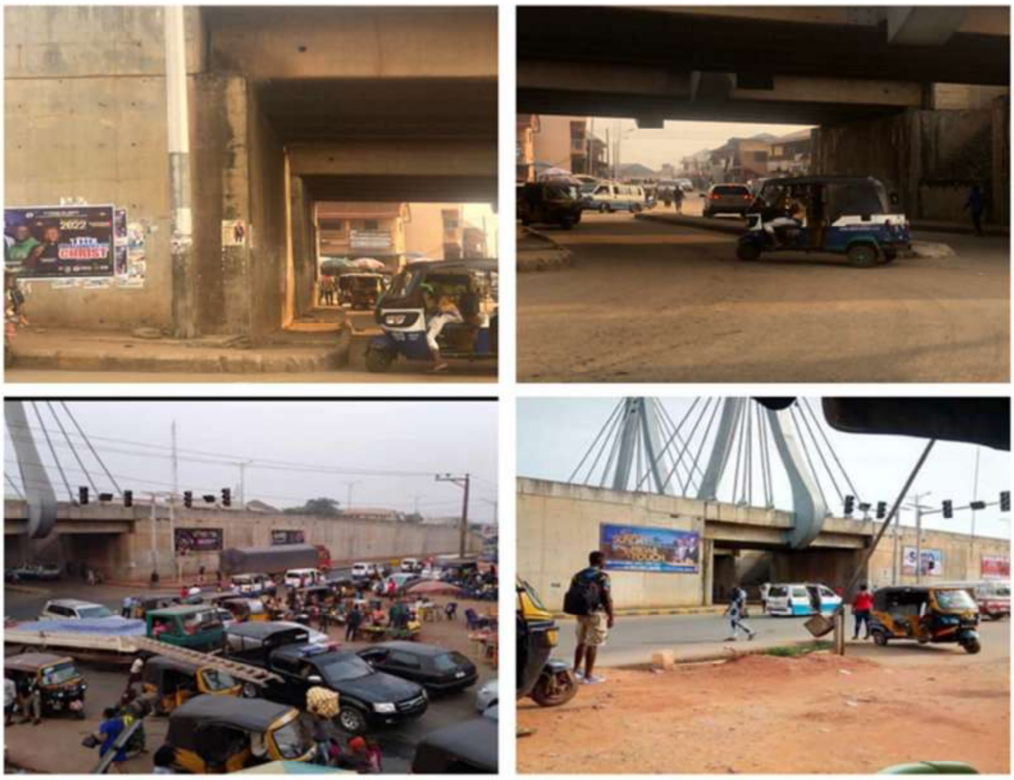


Plate 3.1 picture of Aroma junction intersection.

Plate 3.2 Picture of the Regina junction intersection

Plate3.3. Picture of the Unizik junction pedestrian bridge .

CHAPTER FOUR

RESULT AND ANALYSIS

4.1. General data analysis

As explained in the methodology, three approaches were taken in an attempt to give sustainable considerations for pedestrians in this research location.

The first approach is the traffic count, the second is the direct and indirect interview and the third is the direct observations of pedestrian behaviour during pedestrian count , and all this approaches were taken along the major intersection of the research location.

4.2 Descriptive analysis

The highest pedestrian volume recorded in this survey during the one hour viewing time is 1295. This was observed at the Regina junction on the Sunday evening (Table 4.1). The vehicular volume observed was high, and there was congestion at the major intersection . 275 of the pedestrians were holding kids or load, 108 were on their mobile phones, 233 pedestrians were walking along the intersection , and the rest of the pedestrians were crossing the junction without distractions from their phones or extra weights.

The lowest traffic volume was recorded at the Unizik junction with an overhead bridge , during the one hour viewing period on a Thursday evening. A total pedestrian count of 872 was recorded ,104 pedestrians were using their mobile phones, 376 were walking, 106 were carrying loads or kids that delayed their movement and the rest of this pedestrians were observed using this pedestrian bridge without distraction. The result of the pedestrian count at the 3 intersections on the different days are shown in Table 4.1

Table 4.1: **Descriptive statistics of pedestrian count**

The analysis of data from the traffic count shows that the intersections without

Pedestrian statistics (Volume).	**Sunday**	**Tuesday**	**Wednesda**	**Thursday**
Total pedestrians at Aroma	**898**	**1002**	**907**	**872**
Pedestrians using their mobile device	121	103	123	104
Pedestrians with loads or kid(s)	93	141	92	106
Pedestrians walking along the junction	401	501	358	376
Pedestrians crossing	283	257	334	296
Total pedestrians at Regina	**1295**	**1261**	**985**	**1080**
Pedestrians using their mobile device	108	131	103	147
Pedestrians with loads or kid(s)	275	274	271	316
Pedestrians walking along the junction	233	451	244	391
Pedestrians crossing	679	465	570	574
Total pedestrians at Unizik junction	**996**	**1064**	**902**	**789**
Pedestrians using their mobile device	218	304	321	262
Pedestrians with loads or kid(s)	139	278	122	95
Pedestrians walking along the junction	320	198	171	143
Pedestrians crossing the intersection	310	284	288	289

pedestrian facilities have an even higher pedestrian volume than the intersection without pedestrian facility. From the table analysis , the volume of pedestrians at the regina and aroma junction without pedestrian facility and the Unizik junction with a pedestrian bridge has the highest difference of 397 people. The first descriptive analysis done comparing locations , is with respect to the feeling of safety pedestrians experience on the intersections with facilities to that without facilities, and this is analyzed based on the number of pedestrians using their mobile phones on this different intersection. The Unizik junction with a pedestrian bridge records the highest number of pedestrians using their mobile phones while crossing and walking along this intersection on every day of this count. This result can be analyzed to say that pedestrians felt safe enough to use their phone on the facility created for them, using this pedestrian bridge, the fear of collision is less since there are no vehicles. The highest number of pedestrians using their mobile device was recorded on Sunday as shown in Figure 4.1.

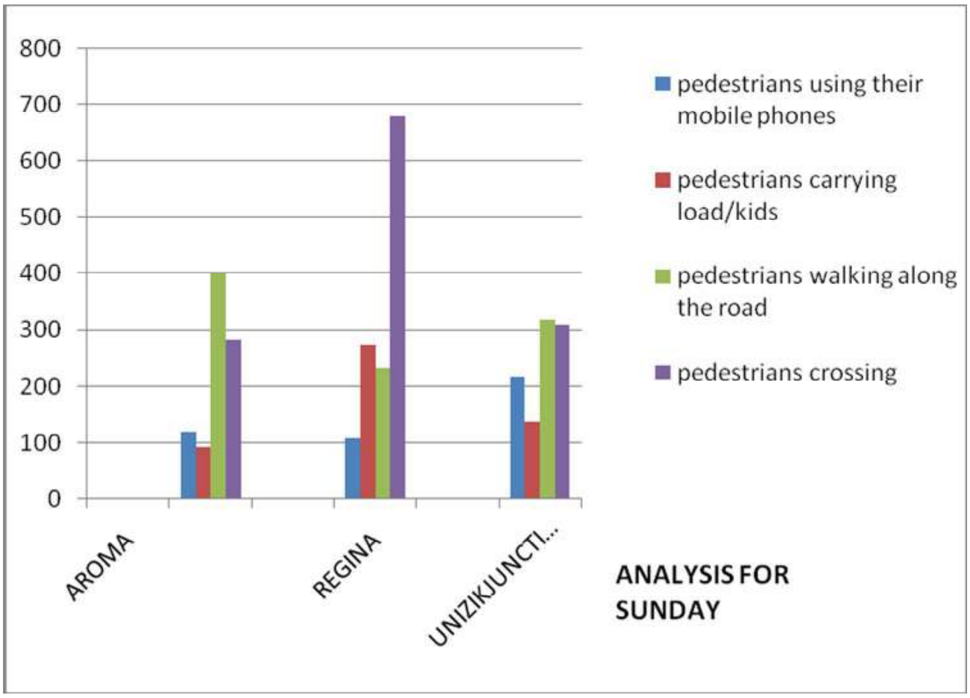


Figure 4.1 Chart analysis of pedestrians on Sunday

This intersections are leading inside and along places and communities, so pedestrians are seen either walking along these junctions or crossing to the other side of the carriage way. From the chart analysis, the volume of pedestrians crossing this survey intersections take more than 50% of the total volume of pedestrian count at the survey period in every location. This shows that the main pedestrian activity on this intersection is crossing,the highest crossing activity was recorded on Thursday at Regina junction as seen in Figure 4.2. This can be linked with the Eke awka market day(the community market of the awka city) been Thursday.the highest number of pedestrians carrying loads was also recorded on the same day as seen in this chart (Figure 4.2).

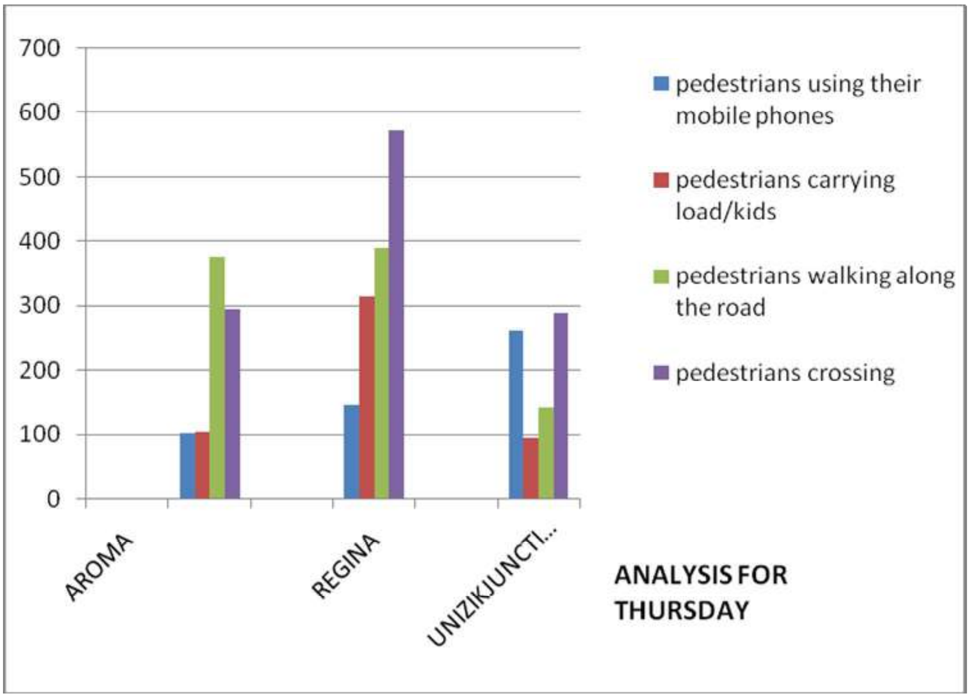


Figure 4.2 Chart analysis of pedestrians on Thursday

From Figure 4.3, the statistics shows that on the day and time with Regina junction having the highest pedestrian volume, the Unizik junction that is 1km after the Regina junction and with a pedestrian safety facility(pedestrian bridge) had one third of the volume of pedestrians crossing and walking recorded in Regina junction. This predicts that distance can be a factor pedestrians consider when faced with a decision of using the pedestrian facility or crossing the carriage way without any pedestrian, and also creates an assumption that pedestrians will choose to save their time if need be, instead of walking down to use safety facilities that are far from them. Pedestrians will choose time management over the level of safety experienced while crossing the road.

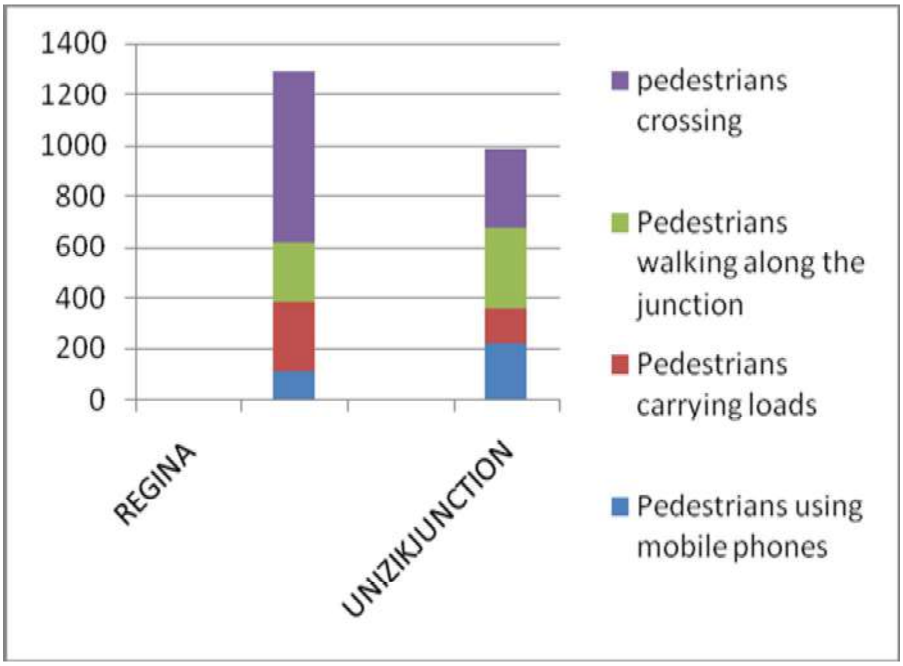


Figure 4.3 chart analysis comparing pedestrian walking along and crossing Regina and Unizik junction.

From Figure 4.2, the Regina junction has the highest volume of pedestrians, which can be linked to a major observation during this survey. During this survey, it was observed that the Regina at both sides of the junction is the major road entrance to eateries, hotels, eats out spots, churches and hotels. Analysis from all the locations shows that Regina junction also has the highest number of pedestrians crossing. Polished without any traffic facility at the Regina junction, the volume of pedestrians crossing was high, and this has obvious difficulties. A good number of pedestrians in each of this survey locations were seen with kids and loads, most of this weighed them down, affected their crossing speed, waiting time, and their relationship with other pedestrians. One of the reason for this pedestrian count in this research is to ascertain the volume of pedestrians using this intersection even with no availability of the pedestrian facility, and from the result, it is obvious that there is a high number of vehicular movement on these intersections, it is safe to say that pedestrians experience difficulties using this location. A picture of pedestrian with heavy loads observed during this survey is shown in plate 4.1.

Plate 4.1 Pedestrian carrying a heavy load, walking along the junction.

During this survey, many abusive relationships between motorists and pedestrians were experienced. Motorists were seen parking in front of pedestrians and disrupting their waiting and crossing time, as shown in Plate 4.2.

Plate 4.2 Motorists parking in front of pedestrians waiting to cross the road.

It is unfair that this location lacks pedestrian facility, this location only has facilities aimed at making the road movement efficient for just vehicles, no sidewalks, pathways, Island, signaling or separate walkways. As of last year, an attempt was made to give pedestrians an easy movement by the presence of road safety officers that helped students and vulnerable pedestrians cross the road, but this didn't last enough seeing that none were present throughout this survey,

Plate 4.3 pictures of pedestrians crossing the Regina junction carriage way .

Pedestrians are left with no other but to cross the road with an average carriageway of 33m and a central drainage ditch of 2.7m opening. During the waiting time at this location, pedestrians are seen hiding close to try cycles to avoid vehicles over speeding into the intersection at this very location. Plate 4.3 shows a view of pedestrians crossing the road during an observation. On this road, pedestrians with disabilities, the elderly, and kids were seen struggling to cross the road too. Pedestrian safety is often neglected on the highway, and this survey proved that this research location is amongst them.

Furthermore, from this survey, it was analyzed from figure 4.4 that the highest number of pedestrians walking along the road was observed at the Aroma junction on Tuesday. This is predictable because along the Aroma junctions are many firms and organizations.

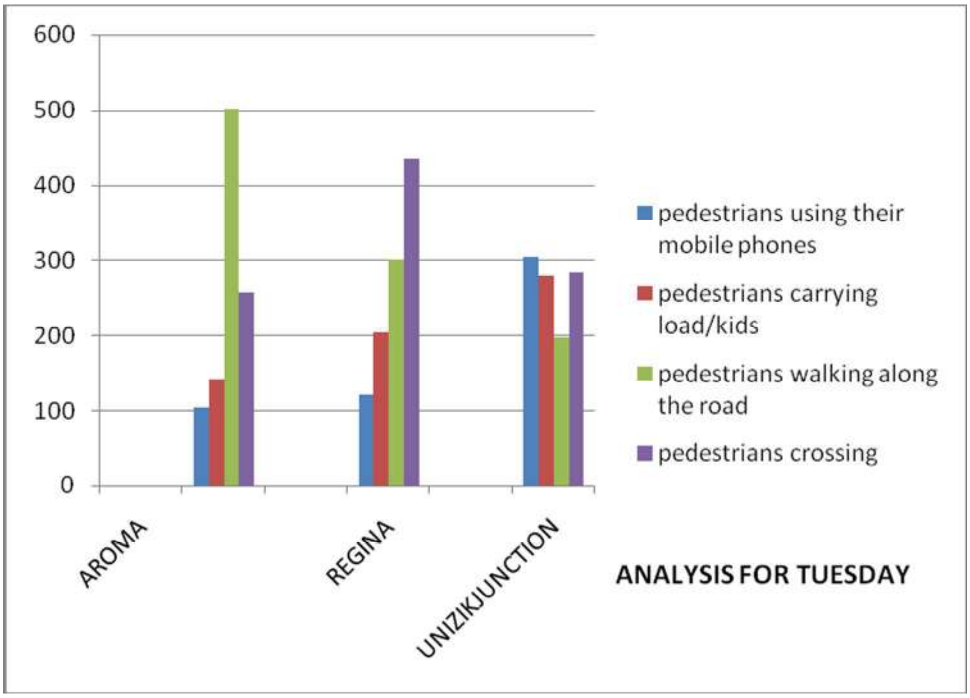


Figure 4.4 chart analysis of pedestrian volume with respect to character observation on tuesday.

Since the "sit at home" order in the eastern part of Nigeria, Tuesdays are now observed as the first work day of the week with highest traffic volume from workers coming to or going home from work. This can also explain the chart analysis from Figure 4.5 showing Tuesday as the day with the highest total pedestrian volume during.

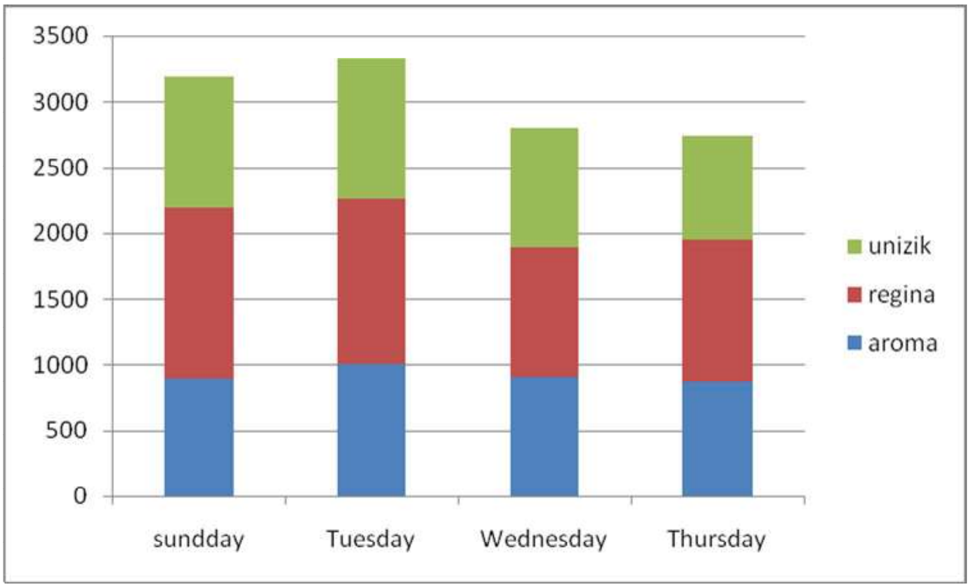


Figure 4.5 Chart analysis of total pedestrian volume on different days.

4.3. Result analysis from the questionnaire

From the answers given by this pedestrians in the questionnaire survey, it was observed that ;

1. From the answers and analysis, with most of the answers to the first question on table 1 as "poor", this shows that there is really a lack in pedestrian facility and the pedestrians are very aware of this .

2. The second question on table 1. was included to help ascertain if pedestrian will choose their safety over time management and the stress of walking 1km in order to use the overhead bridge. From this results, it's quite obvious that this participants representing the pedestrians on this road will choose their a shorter time.

3. The third question on this survey was included to help determine how safe pedestrians feel using marked signals, and the ratio of those that prefer to use this marked crossings to those that do not wasn't quite high. This shows that the least marked crossings will be well appreciated and used on this road by pedestrians. A number of this participant couldn't give a definite answer to this question because

they had no knowledge of what a marked signal stands for.

4. The fourth question helps us understand how pedestrians feel about motorists attitude towards them, from this interview, from this answers and analysis, it is observed that most pedestrians experience hostility from motorists.

5. The last question was to help us know the frequency of accident in this intersection. 62 pedestrians marked that they've never witnessed an accident on this intersection. While 38 indicated that they have, with 4 persons specifically stating that they've has been a victim of this accident . this 6 persons are females, they were reached out to for a direct interview which 4 agreed to.

Table 4.2 Questionnaire data analysis.

Question	**Answers**
What do you think about the availability of pedestrian facility in awka? a. Satisfactory b. Good c. Poor d. Very poor	**20 participants choose "satisfactory", 32 choose "poor" and 40 choose "very poor"**
So you prefer walking down to the overhead bridge using the pedestrian bridge or crossing the carriageway at regina? a. Yes, I'll rather use the bridge b. I prefer crossing the carriageway at regina.	**65 pedestrians choose "b" while the rest went for "b"**

Can you confidently use a zebra crossing mark on the highway when you see one ? a. Yes, I can b. No, I'll rather use a bridge	**49 pedestrians said they'll use the zebra crossing mark confidently, while 34 said they'll not. The remaining 9 pedestrians did not know what a zebra marking indicated, hence they couldn't make any decision.**
How will you rate the motorists attitude towards you during crossing? a. Hostile b. Satisfactory c. Normal	**42 pedestrians choose the option a,** **30 choose the option b** **20 choose the option c**
Have you ever witnessed any collision at the Regina or Aroma junction? a. Yes I have b. no I haven't	**38 choose 'a',** **48 choose "b"** **And 6 out of this pedestrians indicated to been a victim at this intersection.**

4.4 Analysis on direct interview

To understand the perception of pedestrians on the facilities and their relationship with other road users, it was necessary to delve into their experience on these major intersections. The 4 pedestrians that indicated to been a victim of accident at the Regina junction was interviewed. In this analysis, they'll be referred to as respondent A,B,C, and D respectively. Before the interview started, they were asked if the accidents were major or just caused minor injuries. They all said it wasn't severe, but respondent C and B said they spent some nights at the hospital to heal properly. Respondents were first asked what time of the day these accidents happened. 3 out of 4 of them said these accidents occurred at night. When asked if they were crossing or walking along the side of the road, 4 of them said they were in the process of crossing the road when this happened. Then lastly, they were asked to express their opinion on who contributed majorly to this accident between them and the motorist involved. In response, respondent A stated that it wasn't the fault of either her or the motorist. In her words *"I checked my left and right hand side before crossing, but this Aroma junction has plenty turns, so I didn't see the car coming, and I'm sure he didn't see me coming, it felt like we both rushed into the lane of this accident at the same time,Hence the collision"*

Respondent B stated ;

"*I was crossing the carriageway and even though I saw the car coming, It was still far to me and I felt I could finish crossing before it gets too close. But it didn't go as I planned, the car came with speed and stopped abruptly because of the break. There was no physical collision nut I was I shock and I fainted. I'll say it's the fault of the driver, he was impatient. And most drivers on these roads are"*

This in a way shows how much pedestrians expect from the motorists as their fellow road users. Regina junction is a location with an intersection on both sides; this means that vehicles use the intersection where pedestrians often wait before crossing the road. The only hint they have as to if a vehicle is turning or moving straight is the vehicle indicator light, which not all road users can detect.

Respondent C shared her experience after;

" it was wholly my fault, I was waiting with my load for drivers to reduce their speed so I can cross, I waited and It didn't look like they wanted to stop, so I walked into the road thinking the vehicles will stop, I've seen people do it, and vehicles stopped for them"

She went further to explain how frustrating it can get when the road feels like it is meant for motorists alone.

The last respondent shared her experience "the road was very congested and looked easy to cross between vehicles, when I started crossing ,the congesting started reducing and the driver ignorantly started driving without looking carefully. Its all the drivers fault"

After her narration of the story, she went ahead to tell us that the driver involved in this accident put on his light for her to see that his engine has started, but in her words *"I've never drove a car, so I don't know of signs and marks for pedestrians"*.

4.5 Other analysis on this research location.

The road was solely built with only vehicles in mind.

Summary of some other observed difficulties faced by pedestrians on this road are;

1. No road markings: road markings are form of traffic signs that are on the road surface to help regulate traffic flows and movement time for both motorists. This aesthetically pleasing item could be in form of white lines on the road surface used to indicate curbside parking, intersections, pedestrian

or bicycle lane and other features. An example of a marking important when considering pedestrians as the zebra mark. Zebra markings are used to determine the pedestrian right of way, this mark helps pedestrians move with a sense of security. On this Research location, this mark was not found.

2. Poor road pavement; this is an issue that affects all road users, but during the observation of pedestrian behavior, the state of the pavement was observed to have a slowing down effect on the pedestrians, there are potholes on the refuge island pedestrians stand at some of this intersection.

Plate 4.4 Potholes on the refuge island

3. Uneven side walks and always : sidewalks are paths on the side available for pedestrians to walk on the road, while walkways are made specifically for pedestrians traffic. This aids better movement and gives a sense of security to pedestrians when walking on the road. In these research locations, the availability of sidewalks was seen to be very low, and the paths with sidewalks were uneven, the sidewalks were available on just one path of the

road. This will cause difficulties among pedestrians, and also reduce the feeling of security the pedestrian should experience. The pedestrian friendly path of the road in these research locations has been used to create ditches and wide trapezoidal drainage channels. Pedestrians on this road are forced to walk on the side of the carriageway. Most walkways found in this road are unpaved, unkempt and filled with grass.

4. Absence of pedestrians bridges ; during this survey, it was found that in the whole city of awka, there is only one pedestrian bridge, which is at one of the survey location, Unizik junction. This pedestrian bridge is highly utilized by the pedestrians and according to the survey, pedestrians prefer using the bridge to crossing the carriageway.
5. Hostile attitude of drivers towards pedestrians: according to the survey in this location, the motorists have a very poor attitude towards the pedestrians, they treat the pedestrians as though the road is not meant for their use also, and this can be understandable from the design consideration on this highway that excludes the pedestrians.
6. Poor night lightning: the road lightning at night on this highway is either very poor or absent. This is a major issue because pedestrians and motorists use the road every hour of the day, and some pedestrians have little or no knowledge of wearing bright clothing at night to help motorists notice them.
7. No traffic signals and signs: Through out this survey, it was either that there was no traffic lights or signals on this research location. Traffic signs help minimize traffic congestion and improves the vehicle to pedestrian relationship. The lack of this traffic facility made it very difficult for pedestrians to use some of the road intersection. The traffic wardens who

operated on some locations were found absent on all their usual locations.

8. Lack of sit out and pedestrian shades : on this locations, there were not enough sit out or pedestrian shades, this shades and sit out are one of the most important facility that improves vehicle to pedestrian relationships, pedestrians use this when waiting for vehicles. The few sit-outs on the road were occupied by hawkers and traders.
9. Careless and reckless driving: motorists exhibited careless driving on this research location. During the survey here, I experienced so much careless and unsafe driving. Driving carelessly shows great negligence of other road users.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The research work done to investigate into pedestrian problems and safety needs along the Enugu Onitsha express way. To identify pedestrian behavior that causes collision on this research location, a pedestrian traffic count on three of the major junctions in this research location was carried out. This count entailed recording of pedestrian volume with respect to the behaviors likely to cause collision. It was observed that pedestrians engaged in operating their mobile device while either crossing or walking along this junction at the expense of their safety. Pedestrians were also seen carrying heavy loads that might affect their focus and further distract them from paying attention at these junctions with traffic volume.

In order to analyze and offer countermeasures that should reduce risk of pedestrians-vehicle crashes, questionnaires were distributed to pedestrians and direct interviews were carried out on some pedestrians that have been victims of collision. This was done to get a better and direct view of pedestrians experience and problems from them the major challenges analyzed from these interviews stated that drivers show very hostile attitudes towards pedestrians on these junctions without pedestrian facility. It was also observed that some pedestrians have little or no knowledge of attitudes and characters to exhibit on the road that increases their safety.

Furthermore, to better recommend measures and facilities needed in this research location, observation of the use of pedestrian facilities in these major junctions along the research location was done. It was analyzed that pedestrians felt safer using the pedestrian bridge which is the only pedestrian facility along this location.

The traffic light along the aroma intersection that should be serving the road users by regulating traffic flow from all sides is not properly maintained and currently faulty.

5.2 Recommendation

From the analysis and result gotten from this investigation, it can be said that pedestrian safety is neglected in this study location. This has an effect on the awareness of motorists that the road is for both pedestrians and them, and the feeling of safety pedestrians experience while using this road. The following recommendations can be used to give a better relationship between the road, pedestrians and motorists;

1. Pedestrian awareness program and campaign: due to the lack of awareness and knowledge on pedestrian safety on this intersection, campaigns and programs focused on educating pedestrians on necessary information they should know as road users. This campaigns should also be focused on enlightening pedestrians on the fact that they are major road users too, and they should always put their safety as priority. In these programs, Pedestrians should be more exposed to Road signs and facilities that increases safety.
2. Designs should be made with pedestrians in mind, every road construction plan should have the facilities for safety of pedestrians as a major factor for a complete design. Facilities like road signs, traffic lights , crossings marks, pedestrian islands / refuge, hallway and crosswalks. Junctions with more than two intersections should have traffic lights to help control pedestrian and vehicle movement.
3. There should be a maintenance of already existing facilities, facilities at the aroma intersection should be repaired and maintained. The faulty traffic light and pedestrian refuge island should be maintained , and signs added to it.

4. Motorists should be more enlightened about their relationship with pedestrians and how it affects vehicular flow. They should be better exposed to the fact that pedestrians are also road users like them.

REFERENCES

Abdul, H., Suhaila, S., Kamel, N., and Aimil, S., (2015). Factors that Influences

Pedestrian Intention to Cross a Road While using Mobile Phone. I*nternational Journal of Economics and Financial Issues*.10-11.doi: 10.1016/j.aap.2015.10.026.

Aldagheiri, M., (2007).The role of the transport road network in the economic development of Saudi Arabia. *WIT Transactions on The Built*. Prev5(1); 32-35

Alutman, L., Lane, D., and Lambert RR., (2009). Assessing the Impact of Weather and Season on Pedestrian Traffic Volumes. *Report of the Transportation Research Board (TRB), University of Vermont*; . Vol.9, No.1.

Briamoh, T., and Gana, T., (2018). Highway traffic safety for children. *Accident anal* .vol3:131-137.

Bruno, F., and Santos, (2009). Road Network Planning with Efficiency, Equity, and Robustness Objectives. *Canadian journal of civil engineering*, Vol 45(7), 594-604.

Charles V. and Zeagar, (2012). Design and safety of pedestrian facilities. *Traffic engineering committee* TENC-5A-5, vol2 , no 17.

Coffin, R., and Morall, (1995). Using GIS for evaluation of pedestrian neigbhourhood accessibility. J. Urban stud 40(8), 1471-1485.

David, M.(2013). Communication for Vulnerable Road Users: Survey, *Design Considerations, and Challenges. Sensors.* 19; 2.

David K., and Flach A., (2010). CAR-2-X and Pedestrian Safety, *in IEEE Vehicular Technology Magazine*, vol. 5, no. 1, pp. 70-76.

Dedham Herald. (2019). Distractions shouldn’t be deadly. New York City *Department of Transportation journal.* Vol. 2 ;23-24.

Eenink, R. (2008). Accident Prediction Models and Road Safety Impact Assessment: recommendations for using these tools. *Final report. Deliverable D2 of the RiPCORD-iSEREST project*. 3(4),. 2:2-3 doi: 10.2495/UT090251.

Ferencliak, N., (2016). Pedestrian age and gender in relation to crossing behavior at midblock crossings in India. *Journal of Traffic and Transportation Engineering (English Edition).* volume 3:10.

Gates, NJ., Hoel L., and Arnold L., (2006). Assessing walkability in Eugene,Oregon. *public health,* Prev. 45, 383-391.

Gülay, Malkoç., (2015). Importance of road maintainance. *Road networking journal of engineering mechanics,* vol137,no 2, pp 138-146.

Hamed, T., (2021). Death on the highway. *Safe streets,liveable streets.*urban stud.36(4), 553-542.

Hubin and Vincent J., (2009). "Pedestrian Traffic Counts," T*he Appraisal Journal*. *American Institute of Real Estate Appraisers.* July, Vol. 137, no. 2, pp.138-146.

Institution of civil engineering, (2021). Machine learning and pedestrians safety upgrade. *Municipal engineering.* Vol175, no1

Li M., Valencia, V., and Ma, M., (2013). crashing model of vehicles. *Accident anal* prev78,146-147.

Muhammed T., (2020) . Considerations for pedestrians in nasarrawa state Nigeria. *Medvile*. Vol4:23-56.

Niebuhr, T., Junge, M., and Rosen E., (2016). Pedestrian injury risk and the effect of age. *Accid Anal* Prev.8 :121-123.

Nwakaire, CM., Chuku, DE., Udemba J.,(2017). The challenges and possible remedies to poor pedestrian safety considerations along and across Nigerian busy roads. *MOJ Civil Eng*;2(1):36-41. DOI: 10.15406/mojce.2017.02.00025.

Owens, P., (1998). Planning and designing for pedestrians.Pedestrian-oriented development pattern*; Rockridge neighborhood, Oakland, CA.* 6 (5), 42-49.

Saleem, M., (2018). Seven measures to ensure pedestrian safety ., tech *rep. Decham herald.*vol. 5, no. 1, pp. 70-76.

Sewalkar, P., and Seitz, J., (2019). Vehicle-to-Pedestrian relationship. MOJ civ eng:3(1):21-23.

The guardian - UK cycling and walking, www.thegurduanuktransport.com. (23rd may 2018).

Wachnicka, J., and Zukowska, J., (2021). Research on the Use of Mobile Devices and Headphones on Pedestrian Crossings—*Pilot Case Study from Slovakia*. Vol.10.33-91,doi.org/10.3390.

WHO, “Global Status Report On Road Safety 2013: supporting a decade of action,” *World Health Organization, Tech. Rep*., 2013.

Zhuo, R., and Sixian D., (2019), effect of phone usage in pedestrian death. *Tech rep.* vol 2:23-24.

APPENDIX A.

1.What do you think about the availability of pedestrian facility in awka?

a. Satisfactory b. Good c. Poor d. Very poor

2. So you prefer walking down to the overhead bridge using the pedestrian bridge or crossing the carriageway at regina?

a. Yes, I'll rather use the bridge b. I prefer crossing the carriageway at regina.

3. Can you confidently use a zebra crossing mark on the highway when you see one ?

a. Yes, I can b. No, I'll rather use a bridge

4. How will you rate the motorists attitude towards you during crossing?

a. Hostile b. Satisfactory c. Normal

5. Have you ever witnessed an accident at the Regina or Aroma junction?

a. Yes I have, b. No I haven't c. I've been a victim.

TITLE PAGE

EVALUATION OF BANANA PEEL POWDER AS NATURAL COAGULANT FOR TREATMENT OF WASTEWATER.

BY

ANYAKORA SOMTO VALENTINE

REG. NO: NAU/2016224049

A RESEARCH WORK SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR DEGREE IN ENGINEERING (B.ENG.)

TO THE

DEPARTMENT OF CIVIL ENGINEERING
FACULTY OF ENGINEERING AND TECHNOLOGY
NNAMDI AZIKIWE UNIVERSITY,
AWKA.

FEBRUARY, 2022.

CERTIFICATION PAGE

This is to certify that this project titled "EVALUATION OF BANANA PEEL POWDER AS NATURAL COAGULANT FOR TREATMENT OF WASTEWATER" was carried out by ANYAKORA SOMTO VALENTINE, with Registration Number 2016224049 in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka:

____________________________ _____________________

Anyakora Somto Valentine **Date**

APPROVAL PAGE

This is to certify that this project work titled "Evaluation of Banana Peel Powder as Natural Coagulant for Treatment of Wastewater" is an authentic academic work undertaken by ANYAKORA SOMTO VALENTINE with Registration Number **2016224049** in the Department of Civil Engineering, Faculty of Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

________________________________ ______________

Engr. Prof. O.E. Ekenta **Date**

(Project Supervisor)

________________________________ ______________

Engr. Dr. A.C. Ezeagu **Date**

(Head of Department)

________________________________ ______________

Engr. Prof. D.O. Onwuka **Date**

(External Examiner)

DEDICATION

This work is dedicated to the Gentle Spirit of God who guided me all through my stay in school and in whose inspiration I have written this work. And also to my Late Dad Engr. Sebastine Anyakora.

ACKNOWLEDGEMENTS

My profound gratitude goes to the Almighty God for His provision and protection all through my stay in school.

I am most grateful to Mr & Mrs Darlington Ngonadi for their financial support throughout the years. May the Almighty God continue to keep you and shower His Blessings on you. And I am also grateful to Mrs Felicia Ezeanyi for being there for me.

I will always appreciate the support, advice and instructions of my supervisor, Engr. Prof. O.E. Ekenta and Engr Samson/ Engr Johnbosco the laboratory technologists who assisted me in the laboratory work. I am also very grateful to the Head of the Department of Civil Engineering Engr. Ezeagu and all he staff members of Civil Engineering Department, Nnamdi Azikiwe University, Awka for their contribution throughout my studies.

My unrivaled gratitude goes to my Mother Mrs Uju Anyakora and also to my sweet sisters Precious and Nkiru Anyakora for their spiritual, emotional and moral support throughout my academic programme.

ABSTRACT

This study evaluated Banana peel powder as a natural coagulant for treatment of wastewater. The demand for fresh water has continued to increase at a rapid pace due to the growing population, increasing urbanization and the constant economic growth. Fresh water scarcity can be overcome to an extent by using fresh water sources for drinking purposes and treated wastewater for various domestic purposes. Conventional treatment techniques are extremely expensive for developing countries like Nigeria. Therefore, an urgent need for cost effective methods of treatment and recycling of wastewater are highly desirable. The wastewater was gotten from Juhel Pharmaceuticals and was used for the laboratory analysis. The Jar test was used to determine the optimum coagulant dosage of banana peel powder. Banana peels are capable of absorbing various metals and other pollutants present in industrial waste waters. This project also reports an investigation regarding activation of banana peels for this purpose. Banana peels were dried and burned at a specific heat. This experimental method found Banana peel powder coagulant optimum dosage to be **0.3mg/L.**

TABLE OF CONTENTS

Title page	i
Certification	ii
Approval	iii
Dedication	iv
Acknowledgements	v
Abstract	vi
Table of Contents	vii
List of symbols	viii

CHAPTER ONE

1.0 INTRODUCTION	1
1.1 Background of Study	1
1.2 Aim	3
1.3 Study Objective	4
1.4 Scope of Study	4
1.5 Limitation of Study	5
1.6 Statement of Problem	5
1.7 Justification of Problem	6

CHAPTER TWO

2.0 Literature Review	7
2.1 Water Quality Constituents	7
2.2 Impurities in Water	7
2.3 Water Quality Characteristics	7
2.4 Turbidity	10
2.5 Nutritional Value of Banana Peel Powder	10

2.6 Antibacterial and phytochemical analysis of Banana peel 12
2.7 Nutrient and Heavy Metal Composition of Banana (Musa paradisiaca) Peels 12
2.8 Chemical Properties of Banana Peel 14
2.9 Waste Water 15
2.10 What Does Coagulant Mean? 18
2.11 What is coagulation? 19
2.12 Essence of Coagulation 19
2.13 Factors affecting Coagulation 20
2.14 Mechanism of coagulant function 21
2.15 Flocculants 21
2.16 Particle Settling Theory 22
2.17 Chemical coagulation 23
2.18 Natural coagulants for wastewater treatment 25
2.18.1 Advantages of natural coagulants 27
2.19Applications, Advantages and Disadvantages of Banana Peel As a Coagulant 28
2.19.1 Advantages of banana peel as a coagulant 28
2.19.2 Disadvantages of banana peel as a coagulant 29
2.20 Mechanism of coagulation by natural coagulants 29
2.21 Potential coagulants presents in banana peel powder and possible mechanism of coagulation 31

CHAPTER THREE
3.0 Material and Methods 33
3.1 Sample Collection and Preparation 33
3.2 Apparatus 33

3.3 Materials 33

3.4 Precautions 34

3.5 Banana Peel Structure 34

3.6 Procedure 35

CHAPTER FOUR

4.0 Result Analysis and Discussion 38

4.1 Jar Test Result 38

4.2 Determination of Optimum Dosage 38

4.3 Data Analysis 39

4.4 Turbidity Removal Efficiency Calculation 41

4.5 Efficiency 44

4.6 Colour of End Product 44

CHAPTER FIVE

5.0 Conclusion and Recommendation 45

5.1 Conclusion 45

5.2 Recommendation 45

REFERENCES 47

APPENDIX 49

LIST OF SYMBOLS, ABBREVIATIONS AND UNITS

NTU - Nephrometric turbidity Unit

Mg - Milligramme 10^{-3} gramme

L - Litre

Conc - Concentration

LIST OF TABLES

Table 3.1 Values of Coagulant Dosage and the initial Turbidity 37

Table 4.1 Values of Coagulant dosage, Initial Turbidity, Final Turbidity, Turbidity Removal and Turbidity Efficiency. 39

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background of Study

Water is one of the essential requirements for life. All living things need water for their survival. Water is used for a variety of purposes, including drinking, food preparation, irrigation and manufacturing. Although water covers more than 70% of the Earth’s surface, less than 1% of that resource is available as fresh water – and this is not evenly distributed throughout the world. More than one billion people worldwide, mostly in developing countries, lack safe drinking water. Apart from the scarcity of water, there are many other challenges in providing a safe, adequate and reliable water supply in many parts of the world. (Subashree et al., 2017).

The numerous methods being used in treating wastewater is based on its toxicity in order to protect the aquatic systems (Kukić et al., 2018) such as coagulation, adsorption, dissolved air flotation, membrane technology, biological systems, filtration and so on (Kukić et al., 2018; Tetteh and Rathila 2019; Tetteh and Rathila 2020).

Coagulation is an ancient practice which became necessary as sedimentation could only take care of those particles with a mass density higher than that of the surrounding water.

The first use of coagulant in water treatment was recorded in 77 AD (internet). Coagulation originated from a Latin word ‘’coagular” which means to gather together in order to gain mass, that is to become denser. This process is aimed towards removing colloids, as well as suspended impurities in water, including organic and inorganic substances which are principal contributors to turbidity and colour.

Coagulation is a physical–chemical process to remove turbidity of drinking water and wastewater. Conventionally, chemical-based coagulants such as alum (AlCl3), ferric chloride

(FeCl3), polyaluminium chloride and synthetic polymers (polyacrylamide) are used to remove the turbidity of water (Alwi et al. 2013; Choy et al. 2014). Coagulation has been a vital mechanism in physio-chemical wastewater treatment processes, which involves the addition of chemicals. Most of these chemicals are commonly known as coagulants (polymeric, natural, organic and inorganic) responsible for destabilizing and agglomerating the contaminants (Beyene et al., 2016; Teh et al., 2014). Generally, aluminum and iron-based coagulants are employed in pre-treatment processes in water and wastewater treatment facilities to efficiently reduce the organic load prior to subsequent treatment processes (Kukić et al., 2018; Teh et al., 2014). However, these types of coagulants are associated with the formation of large volumes of sludge associated with complex metals. Generation of excessive non-biodegradable sludge is the major issue of using chemical coagulants for wastewater treatment (Carvalho et al. 2016). As a result, this poses great threat to agriculture, human health (memory loss, intestinal constipation, abdomen colic, spasms) and aquatic life if not treated before being disposed into the environment (Beyene et al., 2016; Teh et al., 2014). An alternative greener and sustainable approach is use of natural coagulants for turbidity removal. Natural coagulants from plant-based materials or renewable sources are attracting a lot of attention due to their various advantages over chemical counterpart. They are biodegradable (Asrafuzzaman et al. 2011), non-toxic, non-corrosive (Swati and Govindan 2005) and cheaper than chemical coagulants. Since they produce lesser sludge with high nutritional value (Choy et al. 2014), the sludge handling and treatment cost is minimal. Despite these advantages, natural coagulants are not commercialized so far (except few such as Moringa oleifera seeds) (Choy et al.

2014). Challenges in harvesting and processing of natural coagulants from plants might be the major factors limiting their commercialization (Carvalho et al. 2016).

Waste or non-useful materials such as orange peel, banana pith and neem leaf powder have also been utilized as natural coagulants. Tones of lemon and banana peels were discarded and send to garbage as useless materials and it is very significant and even essential to find applications and uses for these peels, as the management of wastes nowadays is becoming a very serious environmental issue. These waste peels are low cost, non-hazardous and environment friendly bio-materials which can be used as coagulant in water treatment (Anju and Mophin-Kani 2016; Kakoi et al. 2016).

Therefore, the improvement of the coagulation process by the use of cost-effective or biodegradable or natural coagulants is worth investigating, since there is limited information and studies on natural coagulants used for wastewater treatment. This study design to evaluates the usefulness of banana peel as a coagulant in waste water treatment.

1.2 Aim

The aim of this research is to investigate the effectiveness of banana peel as a potential coagulant to reduce turbidity of synthetic wastewater that will be gotten from Juhel pharmaceutical drug producing company by adding different dosage of coagulant and to determine the most effective dosage of this coagulant.

1.3 Study Objective

1. The objectives is to investigate absorption capacity and application of Banana peel and its combusted product in removal of colour and turbidity from wastewater and also to investigate the application of banana peel as a potential coagulant to reduce impurities in wastewater by adding different dosage of banana peel powder to the wastewater sample.

2. To conduct the standard jar test for determining the performance of extracted banana peel with respect to its coagulation activity.

3. To determine the most effective dosage of this coagulant.

1.4 Scope Of Study

Amongst the wide range of existing methods accessible for wastewater treatment, coagulation and flocculation process is the most preferable. This treatment is commonly being practiced as it is cost effective, reliable, simple and best regarded as low-energy consuming process. This established physical-chemical process removes colloidal, suspended and soluble particles efficiently by prompting aggregation of macro and micro particles into larger size proceeded by sedimentation. In conventional treatment processes, various types of coagulants are often used depending on chemical traits of the contaminants present in the wastewater. The inorganic and synthetic organic polymer coagulants include alum, ferric chloride, calcium carbonate as well as polyaluminium chloride were generally used in the wastewater treatment. Despite that, such treatment leads to disposal problems as the sludge obtained after the treatment using aluminium salts risks accumulation in the environment. At the same time, synthetic organic polymers like acrylamide possess carcinogenic and neurotoxic effects. The alternative solution to these problems is replacement of metal and synthetic coagulants with natural coagulants which are more environmental friendly.

1.5 Limitations Of Study

Plant extracts derived from seeds, leaves, root, barks and fruits have been used in water

purification process ever since ancient times. Natural coagulants exhibit various benefits which

include reduction in production of sludge, lower cost, restrain variations in the pH of treated water, iniquity and provide greater biodegradability. Researchers on biological originated coagulants that have been studied include okra, nirmali plant, Moringa Oleifera and chitosan.

In addition, Jahn (2001) has also conducted experimental study on the performance of apricot, peach kernel and beans in clarifying water. In the current study, banana peel is developed into a source of natural coagulant. Banana plants belong to the family of Musaceae and species of Musa which includes M. Acuminata, M. Sapientum, M. Paradisiaca, and M. Cavendishii are grown worldwide.Banana tree can bear fruit almost 3 to 20 hands in a cluster once in a lifetime which leads to huge amount of biomass waste generated from the tree. The Southeast Asian countries includingMalaysia, India and Japan has initially cultivated banana plant. Nevertheless, some species are regarded to be genetically connected with species of banana from Africa

1.6 Statement of problem

Coagulation is the primary treatment process for treating these fluids. It is a common process in treating both industrial wastewater and surface water. Dissolved and colloidal substances are removed in coagulation by overcoming inter-particle repulsive energy barrier (Prakash, Sockan and Jayakaran 2014, Shankar, et al. 2014). However, high procurement cost, detrimenta effect on human beings and environment are the major disadvantage of chemical coagulation. Also, large area requirement, high maintenance and long retention time are the major drawbacks associated with biological treatment (Alwi, et al. 2013). Therefore, natural coagulants are an alternative and if adopted, will reduce the health risks of using chemical coagulants.

1.7 Justification of problem

The natural coagulants are cost effective and thus reduce usage of chemicals and environmental pollution (Subramani, et al. 2018). Neem leaves powder, orange peel powder and alum were used as a coagulant in a study by (Anju and Kani 2016). The turbidity removal for neem leaves powder was about 98% and pH in the range of 8 and above but not less than 9 which is alkaline in nature. The turbidity removal for alum was about 98.8% and pH in the range of 5 which is acidic in nature. The effectiveness of neem is near to that done with alum as a chemical coagulant. Also, Neem leaf powder has high efficiency in removing oil and grease present in the textile wastewater. (Sasirekha, et al. 2018, Hassan, et al. 2018) Neem leaves were found to be efficient absorbent for the removal chromium ions (Islamuddin, et al. 2016).

CHAPTER TWO

2.0 LITERATURE REVIEW

Study on the Effectiveness of Banana Peel Coagulant in Turbidity Reduction of Wastewater.

2.1 Water Quality Constituents

Pure water is colourless, tasteless and odourless amongst other qualities. (Ababio 1990). As earlier said it is strong solvent and as a result dissolves many minerals it mixes with. This is why there is no such thing as 'pure water' in nature.

2.2 Impurities in Water

The impurities in natural water can be grouped under the following groups (Quasim, Motley & Zhu 2008).

1. Dissolved Organic and Inorganic Particle – these include:

(a) Cation and Anions, such as $Ca^{2+,}$ Mg^{2+} , Na^{+} , K^{+}, CI^{-}, $SO_4{}^{2-}$, F^{-} etc

i. Natural Organic matters that contain substances like hydroxyl and carboxyl groups which give hydrophilic properties.

ii. Colloids – They are very small to be seen with unaided eyes. They have diameter range that mis greater than 1mm and less than 10^{-3} mm. Examples includes microorganism like bacteria fungi, protozoa, parasitic worms etc.

(b) Suspended Particles – they include floating substances like leaves, debris, silt sand etc.

2.3 Water Quality Characteristics

Several water quality parameters are used to describe properties of water that is indicative of treated and quality nwater. G.N. Gray in his work "water Technology" grouped the parameters as follows:

- **Physical Parameters:** These include – turbidity, colour, odour, taste, temperature, total dissolved solids (TDS), dry solid content, conductivity etc.
- **Chemical Parameters:** To include cations, anions, molecules alkalinity, acidity, hardness, radioactivity, agressivity dissolved oxygen, penols etc.
- **Microbiological Parameters:** Water has long served as a medium of transmission of diseases. The most important of the water borne diseases are those of the intestinal tracts including typhoid fever, dysentery, cholera and some parasitic worms all caused by microorganisms content in water.

Water is one of the key factors for economic development worldwide as it is widely used in different productive sectors including industry, agricultural production, livestock and urban supply. The fast-paced industrialization, economic growth and population growth in developing countries has implicated in unpredicted water requirement in cities. In the last few decades, the water used capacity and wastewater production in domestic sectors has increased remarkably [1]. Wastewater treatment is a crucial process in sanitation system. Effective municipal wastewater resource treatment and
contamination control are necessary to ensure compliance to environment compliance and reutilization of resources.

Water is essential not only for survival but also contributes immensely to the quality of our lives. Since the dawn of time, human beings have harnessed water to improve their lives. Various reports have mentioned the direct and indirect toxic effects of metals in the form of tumors, cancers, and allergies. Hence, health problems caused by alum have been recently reported. In order to replace alum as coagulants, the natural coagulants like in the combination of banana peel and lemon peel are used in water treatment via coagulation-flocculation processes. pH and coagulants dosage were identified for banana peel. This investigation conducted found that banana peel can be a good coagulant which can absorb Biological Oxygen Demand. The Dehydration method is found to be more efficient for banana peels and the optimum contact time for banana peel is 90min. The optimum adsorbent dosage for banana peel is 0.3g and the optimum particle size for banana peels are 300µm.

Key Words: Natural Coagulants, Lemon, Banana, Turbidity.

The Worldwide water demand is increasing day by day due to rapid population, and on the other hand there is continuous decline in ground and surface water levels due to over exploitation. Efforts are being made to find the alternatives for water supply and one prominent solution is treatment and re-use of wastewater. The wastewater is a combination of kitchen waste water, washing and rinsing, bathing water which generates lot of wastewater which contains very high concentration of organic substances such as proteins, carbohydrates and lipids. Many technologies are in practice to treat the wastewater and in the present study, an attempt was made to investigate the application of low-cost adsorbents from banana peels for the treatment by considering the wastewater. Banana peel is an agricultural waste that is being discarded all over the world as a useless material. They cause waste management problems although they have some compost and cosmetics potentiality. The substance could be used for medicine as well as personal care and is known for anti-fungal and antibiotic properties. Besides that, banana peels also have adsorbent potentiality. It is very useful for purification and refining processes. It has adsorption capacities to remove chromium from wastewater, copper and also some dyes. Lemon peels works as a low cost natural coagulant agent for coagulation process in water treatment plant and act as a significant environmentally friendly product. The potential of lemon peels as a coagulant agent in removing turbidity in synthetic turbid water.

2.4 Turbidity

Turbidity is a measure of the light-transmitting properties of water. It is also a measure of the cloudiness of water and the cloudier the water, the greater the turbidity. It is a unity of measurement quantifying the degree to which light traveling through a water column is scattered by the suspended particles in wastewater discharges and natural waters.

The measurement of turbidity is based on comparison of the intensity of light scattered by a sample to the light scattered by a reference suspension under the same conditions (Standard Methods, 1998). Formation suspensions are used as the primary reference standard. The results of turbidity measurements are reprted as nephelometric turbidity units (NTU).

2.5 Nutritional Value Of Banana Peel Powder.

Banana is one of the most common crops grown in almost all tropical countries, including India. It is an abundant and cheap agriculture product. Banana chip and banana fig are the main products from banana flesh produced by a number of small and medium factories located nationwide. As industrial by-products, peels represent about 35-50 g/ 100 g of fruit weight. This resulted in 250 tons of waste from banana peels in India generated each day and these amounts tend to increase annually. The banana peels waste is normally disposed in municipal landfills, which contribute to the existing environmental problems. However, the problem can be recovered by utilizing its high-added value compounds, including the dietary fiber fraction that has a great potential in the preparation of functional foods. Kokum has multiple health and medicinal benefits. The fruits of kokum are an excellent source of antioxidants that prevent to free radicals thereby helps from different diseases. It is also known as cool king of India fruits. Dietary fiber has shown beneficial effects in the prevention of several diseases, such as cardiovascular diseases, diverticulosis, constipation, irritable colon colon cancer, and diabetes. The fruit fiber has a better quality than other fiber sources due to its high total and soluble fiber content, water and oil holding capacities, and colonic ferment ability as well as a lower phytic acid and caloric value content. A high dietary fiber content of banana peel (about 50g/ 100 g) is indicative of a good source of dietary fiber. Cellulose, lignin, and hemicelluloses contents of bananapeels, the components of the insoluble dietary fiber fractions, varied from 8 to 10 g/ 100 g ,7.4 to 9.6 g/ 100 g and 6.4 to 8.4 g/100 g respectively falling within the safety limits. These results indicated that banana peels were safe and valuable functional ingredients for human consumption. Several technological treatment applied to the fruit residue may affect dietary fiber compositions and functional properties. At present there are no published studies on the suitable processes to produce dietary fiber from banana peel. Therefore, the overall objective of this study was to investigate the influence of different preparation methods of banana peels powder and nutrient composition of banana peel juice and its drying characteristic which used as a medicinal purpose on the chemical compositions and properties of dietary fiber concentrate used as a raw material for functional food.

Advances in food industry have opened up numerous possibilities for the large scale production. Nowadays we gave an artificial beverage which is not safe for human health. The banana peel powder is prepared naturally consisting the nutritive value which are method were investigated on their effects on the chemical composition and properties of the banana peel dietary fiber concentrate (BDFC). Banana is common crop grown in almost all tropical countries including India. There are various nutrient content can be used in various ways in India. In the present study used banana peel which is good source of dietary fiber where dietary fiber can be extracted from banana peel powder can be obtained by using sun drying method and peel powder can be used as nutritional juice which is used for medicinal purpose also.

2.6 Antibacterial and phytochemical analysis of Banana peel.

The invitro antibacterial activity of ethanolic and aqueous extract of banana (Musasapientum) peels was investigated on both gram-positive and gram-negative bacteria using agar well diffusion technique. Phytochemical result showed ethanol to be a better solvent for the extraction of the bioactive agents in banana peels which include: glycosides, alkaloids, saponins, tannins, flavonoids and volatile oil. The presence of glycosides and alkaloids in Musa sapientum peels may be attributed to their use by traditional medicine practitioners in healthcare systems in the treatment of some bacterial infections such as cough, fever, cold and venereal diseases. Thus extracts from the peel can be used to control infections caused by Salmonella typhi, Escherichia coli, Klebsiella pneumoniae and Staphylococcus aureus. Infections such as bronchopneumonia, bacterial endocarditis and meningitis caused by Micrococcus Spp. and Pseudomonas aeruginosa will also find treatment with the extracts of this medicinal peel. Thus the use of banana peel by traditional medical practitioners is justified.

2.7 Nutrient and Heavy Metal Composition of Banana (Musa paradisiaca) Peels.

The higher amounts of K than Na in the peel samples investigated are considered of comparative advantage. This is because intake of diets with higher Na to K ratio has been related to the incidence of hypertension. Phosphorus is involved in several biological processes such as: bone mineralization, energy production, ell signaling and regulation of acid-base homeostasis. Findings from this study indicate that unripe plantain peel contains higher quantities of Zn than ripe plantain peel, unripe and ripe banana peels respectively. The considerable amount of Fe in unripe plantain peel is an important finding in this study. Iron is an essential component of hemoglobin and it is critical to the proper function of the immune system and the production of energy.

Mineral content in a banana peel is primarily consistent of potassium (78.10mg/g) and manganese (76.20mg/g). Other minerals present are sodium, calcium and iron at 24.30, 19.20 and 0.61 mg/g respectively. The peel's high potassium content, if taken orally, aids in maintaining normal blood pressure. About 91.50 percent of a banana peel is organic nutrient matter consisting of lipids, proteins, crude fiber and carbohydrates. About 31.70 percent of total mass is fiber with carbohydrates accounting for 59 percent and protein and lipids accounting for 0.9 and 1.7 percents respectively. The high fiber content is useful as a natural laxative.

2.7.1 Phytate

Phytate (myo-inositol hexaphosphate) content of a banana peel is 0.28mg/g, lower than in most whole grains. The only risk associated with phytate and dietary consumption comes from a lack of it. Low phytate consumption increases risk for osteoporosis and adding it to the diet increases bone density.

2.7.2 Saponins

Saponins are known for their foaming property and are another potentially dangerous constituent of a banana peel. The levels are high in banana at 24 percent, greatly exceeding the 3.00 percent level marked safe for consumption by animals. Saponins consumption at high levels can paralyze the sensory system and are known to increase cholesterol production in the body.

2.7.3 Oxalates

Oxalates are organic acids associated with kidney disease and are known to decrease the absorption of minerals, such as calcium, in the body by binding with them decreasing their availability. Eighty percent of all kidney stones occurring in adults in the United States are calcium oxalate stones. The oxalate level in a banana peel is 0.51mg/g, which is low and relatively non-threatening.

2.7.4 Hydrogen Cyanides

Of the anti-nutritive constituents the most poisonous is hydrogen cyanide. It is present in the peel at 1.33mg/g. The chemical can cause immediate death if taken in high dosages and in small dosages may cause stiffening of the throat and chest, heart palpitations and weak muscles. Amounts in a peel fall into the 0.5 to 3.5mg/g safe range.

2.8 Chemical Properties Of Banana Peel.

Musa sapientum (banana) peels have been used in conjunction with other substances to remedy the achy and painful symptoms of arthritis. They are composed of nutritive chemicals, minerals, and nonnutritive chemicals. Banana peels have both highly beneficial and highly dangerous constituents and can be manipulated to serve both as a remedy and a poison. According to [5] banana peel has been discovered to have nutrients and compounds including **protein, dietary fiber, potassium, polyunsaturated fatty acids and essential amino acids** as well as antioxidant compounds such as carotenoids, catecholamines and polyphenols. Mineral content in a banana peel is primarily consistent of **potassium (78.10mg/g) and manganese (76.20mg/g)**. Other minerals present are sodium, calcium and iron at 24.30, 19.20 and 0.61 mg/g respectively

2.9 Waste water

Rapid industrialization has posed many threats to the environment due to the wastewater generation through various industrial processes. Indiscriminate disposal of this wastewater (with or without an appropriate level of treatment) can cause water pollution and land pollution. This has a profound effect on the health of living beings apart from the impacts on the abiotic components such as soil (Chhonkar et al., 2000) and water (Pal et al., 2010). Some key sources of industrial wastewater are palm oil mills (Bha□a et al., 2007; Jagaba et al., 2020), paper and pulp industry (Ashra□ et al., 2015; Wang et al., 2011; Chaudhari et al., 2010), brewery and winery (Brito et al., 2004), cosmetic industry (Naumczyk et al., 2014), tannery industry (Rameshraja and Suresh, 2011), slaughterhouses (Bus□llo-Lecompte et al., 2015), paint industry (Aboulhassan et al., 2014), dairy industry (Triques et al., 2020), etc. A huge quantity of industrial wastewater is generated worldwide daily. Saha et al. (2005) claimed that a single unit of Indian distillery uses 1133.5 KLD of water, and, after processing, around 668 KLD of wastewater is generated. In another attempt to quantufy the wastewater generation in India, Majumder et al. (2014) estimated that every day around 13,468 MLD of wastewater is generated, out of which, more than 50% is discharged into the environment untreated. The quantity and quality of industrial effuent are functions of the ongoing industrial process, raw material utilized, and products manufactured in each industrial unit, and therefore the composition and constutution of wastewater generated are different for different industries. For instance, the textile dyeing processes in the textile industry generate colored wastewater rich in chemical dyes (Ari□n et al., 2009). On the other hand, the effuent from paper and pulp industry contains large quantites of sodium salts of organic acids, lignin, etc. (Wang et al., 2011). Wastewater from the cosmetic industry has significant concentrations of COD, oils, detergents, fats, and suspended solids (Tobajas et al., 2014). Muralimohan et al. (2014) studied the various processes contributing to the wastewater generation in the textile industry.The study revealed that processes like cleaning, de-sizing, etc. are responsible for toxic wastewater generation. The main high-risk constituents of this wastewater are chemicals, dyes, bleaching agents, oils, acids, bases, etc. Sometimes, wastewater contain heavy metals which are a potential cause of concern for the environment and human health (Ahemd et al., 2020). Prasad and Rao 2016 studied the impact of the cadmium pollution, caused due to the discharges from the mining activities. Elevated levels of cadmium can cause diseases like Itai-Itai; whereas, at low 17levels, it may cause problems like kidney damage, sterility among males, u disorders, and high blood pressure. The characteristics of wastewater have been studied in terms of key parameters like BOD (Sehar et al., 2013), TDS, COD (Rathi and Puranik, 2002), and color. These parameters help in gauging the level of potential hazard the effuent can pose to the environment and human health. Due to the potential toxicity, industrial effuents may require prior treatment before their release into the environment. Improper disposal of the wastewater can cost liability in addition to their environmental impacts (Elsheikh and Al-Hemaidi, 2013). Sahu and Chaudhari 2013 suggested that industrial wastewater management is necessary to eliminate the health and socio-economic concerns. Wastewater treatment technologies can be physical, chemical, or biological (Elsheikh and Al-Hemaidi, 2013). Among the physicochemical processes, coagulation-flocculation has been frequently used. In this process, the charge of the colloidal particles is destabilized with the help of coagulants (typically aluminium or iron salts) which results in floc formation due to collision of destabilized particles and their aggregation, which ultimately gets separated from the liquid phase. An alternative to the conventional coagulation-flocculation is the electrocoagulation process. Here, coagulants are formed due to electro-dissolution of the anode which causes hydrolysis products that result in the destabilization of the particles (Verma and Kumar, 2018).

Wastewater streams obtained from industries like tannery (Ari□n et al., 2009), hospital (Wang et al., 2011), paper and pulp mill (Wang et al., 2011), etc. have been treated by coagulation using synthetic chemicals. The use of chemical coagulants for coagulation of wastewater has various implications such as their potential to cause diseases (WHO Guidelines, 2010), the possibility of groundwater contamination or surface runo□ of treated water containing high residual aluminium concentration and therefore their use for the wastewater treatment is not an eco-friendly option. On the other hand, the use of natural substances for coagulation in place of chemicals is a promising alternative for the treatment of industrial wastewater. Natural coagulants are safe for consumption (owing to their plant-origins) and are biodegradable in the environment (Feria-Díaz et al., 2016; Nath, et al., 2020).

Natural coagulation has been used for treating wastewater from various industries like texile (Muralimohan et al., 2014; Shankar et al., 2014; Do□o et al., 2019; Prabhakaran et al., 2020), dairy (Sivakumar et al., 2014; Sivakumar, 2015; Sivakumar et al., 2016; Triques et al., 2020), tannery (Ahmed et al., 2020), etc. A compilation of the various chemical and natural coagulants used for the treatment of industrial wastewater, from literature studies, has been provided in Table 1. In comparison, some of the synthetic coagulants used for the treatment of domestic sewage include alum (Sarparastzadeh et al., 2007), ferric chloride (Sarparastzadeh et al., 2007), poly-aluminium chloride (El Samrani et al., 2008) and aluminium sulphate (Fabres et al., 2017). Several research studies have shown the potential of natural coagulants in treating the wastewaters originating from different industries and have shown them to be equivalent to chemical coagulants in terms of treatment efficiencies for various parameters of interest. Coagulation using natural coagulants has also been effectively used for the water treatment.

2.10 What Does Coagulant Mean?

According to corrosionpedia, This is an inorganic or organic substance that initiates or aids a congealing process during water treatment. A coagulant, together with other chemicals, are added in water to aggregate dissolved contaminants and tiny particles into larger particles so that filtration, clarification, or any other solid removal process may be used to remove them. Inorganic coagulants, except the sodium aluminate (which is basic), decrease alkalinity levels in water. This helps to reduce risks of corrosion attack in pipes transporting wastewater.

To further explain Coagulants;

A coagulant is used in colored, low pH or alkaline and low turbidity water. The optimum pH it generates helps in water purification. The coagulate dose used in purification produces a hydrolysis process that generates a pH suitable for coagulation. The metal hydroxide formed is what adsorbs the impurities (humus) to form compounds that become flocs. These flocs can later be filtered out since they are suspended solid bodies and when heavy they settle at the base.

The importance of a coagulant is to destabilize the acidity of the fluid and cause flocs formation and also to Purify the fluid by removing unwanted active metallic or non-metallic elements.

An inappropriate coagulant dose might lead to corrosion due to acidity, hence one must follow guidelines by the governing authority. (Corrosionpedia, 2018).

2.11 What is coagulation?

coagulation flocculation involves the addition of compounds that promote the clumping of fines into larger floc so that they can be more easily separated from the water. Coagulation is a chemical process that involves neutralization of charge. (Jiang and Jia-Qian 2015; Chekli et al., 2017). Coagulation is an effective, simple and widely practiced water treatment method. However, the usage of chemical coagulant pose detrimental effect on living organism and human health as well as producing large amount of toxic sludge. This study describes the utilization of banana peel as a natural coagulant for the treatment of household wastewater. The natural coagulant extracted from banana peel was prepared by using simple extraction method. The effectiveness of the natural coagulant was evaluated based on the reduction of turbidity during the treatment process.

2.12 Essence Of Coagulation

Untreated surface water contains several constituents nsuch as clay, inert solids, bacteria and others. Most of these constituents may cause several problems such as inhabiting disinfections, leaving the untreated water cloudy and causing problem in the distribution system. It is also known that these suspended solids in aqueous solution exhibit Brownian movement, which keeps them in constant motion, inhabiting settling. By coagulating or flocculating, the suspended aqueous solution is neutralized so that those like charges in the suspension are flocked together into large particles, since the larger the size, the faster the settlimg (WTAS Worldwide Water).

Morso, it can be noticed that the ability of particles to remain msuspended in water is a function of both the particle sizes and specific gravity. Large sized particles may generallys not require any coagulant and may settle in a matter of seconds, but colloidal particles ranges in sizes from 0.001 to 1umm in diameter may take days to months for complete settling and since detention time in water treatment processes are generally less that twelve hours, the rate of settling must be increased in the treatment process. This is accomplished in the coagulation process when tiny particles are brought closer into denser particles which will settle more quickly, thus the high surface area to mass ratio in the colloidal suspension is reduced or destabilized (Shaw, 1989). For effective coagulation, an ion that has opposite charge to that of the colloidal particle is usually added. Since colloidal particles are usually negatively charged the ions added as coagulants must be cations.

The coagulating power of an ion is dependent on its valence and magnitude of charge. When a coagulant is added to water, this results into a series of complicated reactions, firstly, the ions of the coagulant are hydrated, also anions present in the water such as hydroxide and sulphate ions attach themselves to the aluminum ions, leading to a large positively charged molecule with aluminium ions centered. As this continues, elation reaction takes place and this involves bridging of two or more of these larger molecukes to produce large, positively charged ions.

As this process proceeds with hydroxylation and elation, the masses become larger so that they are able to trap turbidity particles and settle under normal condition. It is also noted that the zeta potential at the surface of the colloid reduces to approximately 5 mill volts depending on the specific set of characteristics of the water but this potential cannot be zero for coagulation to be effective. This is so because, for coagulation to occur, forces of attraction must exercise some predominance (www,zeta mater.com).

2.13 Factors affecting Coagulation

Coagulation is affected by the type of coagulant used, its dose and mass; pH and initial turbidity of the water that is being treated; and properties of the pollutants present. (Ramavandi and Bahman 2014). The effectiveness of the coagulation process is also affected by pretreatments like oxidation. (Ayekoe et al., 2017).

2.14 Mechanism of coagulant function

In a colloidal suspension, particles will settle very slowly or not at all because the colloidal particles carry surface electrical charges that mutually repel each other. This surface charge is most commonly evaluated in terms of zeta potential, the + potential at the slipping plane. To induce coagulation, a coagulant (typically a metallic salt) with the opposite charge is added to the water to overcome the repulsive charge and "destabilize" the suspension. For example, the colloidal particles are negatively charged and alum is added as a coagulant to create positively charged ions. Once the repulsive charges have been neutralized (since opposite charges attract), van der Waals force will cause the particles to cling together (agglomerate) and form micro floc. (Wikipedia.net).

2.15 Flocculants

Flocculants are chemical that are used to promote flocculating by causing colloids and other suspended particles in liquids to aggregate, forming flocs. Flocculants are used in water treatment process to improve the sedimentation of small particles. For this reason, flocculants are sometimes called coagulants aid at water treatment operation (Tillman, 1996; Faust and Aly, 1999). Many flocculants are multivalent cations such as aluminum, iron, calcium or magnesium. These positively charge molecules interact with negatively charged particles and molecules to reduce the barriers to aggregation. In addition, many of these chemicals under appropriate Ph react with water to form insoluble hydroxides. These on precipitation link together to form long chains or mesh which traps small particles into larger proportion.

2.16 Particle Settling Theory

The settling of discrete, non-flocculating particles can be analysed by means of the classic law of sedimentation formed by Newton and Stoke. Newton's law yields the terminal particle velocity by equating the gravitational force of the particle to the frictional resistance or drag.

The gravitational force is given by

F_G = (Pp – Pw)g Vp

Where

F_G = gravitational force Kg M / S^2

Pw = density of water Kg/M^3

Pp = Density of particle Kg/m^3

G = acceleration dur to gravity (9.8 m/s)

Vp = Volume of particle (m^3)

The frictional drag force depends on the particle velocity, fluid density, fluid viscosity and drag coefficient C_d (dimensionless) and is given by Fd = $C_d A_p P_w V^2 p$

Where

Fd = frictional drag force (Kgm/s^2)

Ap = cross sectional or project area of particle in the direction of flow (m^2)

Vp = particle settling velocity (M/S)

The coefficient of drag C_d takes on different values depending on whether the flow regime surrounding the particles is laminar or turbulent. Although particle shape affects the value of the drag coefficient, for particles that are approximately spherical, the C_d is approximated by the equation;

$$Cd = 24/NR + 3/\{NR + 0.34$$

The Reynolds number for settling particle is defined as ;

$$NR = VpdpPw/N = Vpdp/v$$

Where N = dynamic viscocity (n-s/m^2)

V = kinetic viscocity (m^2/s)

Vp = particle velocity (m/s)

Dp = particle diameter (m)

2.17 Chemical coagulation

Chemical coagulation is the process of destabilizing the colloidal impurities in water or wastewater by using chemically produced substances. The process of formation of flocs on charge neutralization is known as flocculation (Hamawand, 2015). Aluminum sulfate (Quintero-Jaramillo et al., 2016), potassium aluminum sulfate, Iron (III) chloride hexahydrate, ferric sulfate (Karamany, 2010), etc. can be used as coagulants for water and wastewater treatment. Alum is the most commonly used chemical coagulant. Alum contains potassium and aluminum, and is chemically represented as KAl(SO4)2.12H2O. Alum or potassium aluminium sulfate is very frequently used for water and wastewater treatment. Alum is easily available and is an inexpensive alternative for the wastewater treatment. Madhavi et al. 2014 used alum for treating wastewater from the metal fabrication industry. Alum, at a concentration of 450mg/L, removed 99% of color from the wastewater at a pH of 8.0 (Madhavi et al., 2014). The effectiveness of alum can be increased by using it with other coagulants. In a study, alum, when used with polyacrylamide (PAA) and poly ferric sulfate (PFS), resulted in an increase in the efficiency of COD removal to 82% from 68% (Loloei et al., 2014). Jiang and Llyod 2002 reviewed the potential of ferrate salts for the treatment of wastewater and reported their potential application for wastewater treatment. Iron (III) chloride, also called as ferric chloride (represented as FeCl3), can also be used as a coagulant for wastewater treatment. Bogacki et al. 2011 used ferric chloride for the treatment of cosmetic industry wastewater with the aim of COD reduction. Using ferric chloride, up to 63.9% reduction of COD was achieved at a pH of 6.0. Poly aluminum ferric chloride can also be used as a coagulant (Ebrahimi et al., 2014). Liang et al. 2009 used ferric chloride for treating the molasses wastewater. At optimum conditions, 96% color and 86% COD could be removed from the wastewater (Liang et al., 2009).

2.17.1 Disadvantages of chemical coagulants: Health and environmental impacts

Chemical coagulation is carried out using synthetic chemical coagulants. This practice has the potential of leaving a bad impact on the environment and the public health. Chemical coagulants are non- biodegradable and remain in the water even after the coagulation process is completed. There is a possibility that the treated supernatant contains the traces of metals present in the chemical coagulants due to the presence of residual aluminum in the supernatant (Zouboulis and Tzoupanos, 2009). Use of chemical coagulants can cause neurological diseases like Alzheimer's disease (WHO Guidelines, 2010; Nique□e et al., 2004), Encephalopathy (Srinivasan et al., 1999) leading to dementia, Down's syndrome and staining of Hippocampal neurons (Walton, 2006). Parmar et al. 2012 in his study suggested that supernatant obtained from dairy industry using alum and ferrous sulfate was not suitable for discharge into the municipal drains due to the high values of various parameters like BOD and COD in the supernatant. Major issues with the use of aluminium-based coagulants are that they lead to increased concentration of residual aluminium in the supernatant (WHO Guidelines, 2010). This aluminum may either seep into the groundwater or may have a surface runo□ (WHO Guidelines, 2010). Conventional, water and wastewater treatment plants do not remove aluminium and water with elevated aluminium content (Walton, 2006) is supplied to the end consumers (Srinivasan et al., 1999). If aluminium entered the public distribution system, it could lead to precipitation of hydrous aluminium in the water, which is to be supplied to the consumers (Srinivasan et al., 1999). Residual aluminium in the treated water is found to negatively impact the health of consumers. Exposure to aluminium is linked to Alzheimer's disease (WHO Guidelines, 2010; Nique□e et al., 2004) as it stains the Hippocampal neurons (Watson, 2006). Aluminium is neurotoxic and is responsible for disorders like Parkinson's disease and Down's syndrome (Watson, 2006). Its accumulation in the bloodstream for the long term can result in severe Encephalopathy (Srinivasan et al., 1999), and consequently contributing to dementia (Oaks et al., 2004).

2.18 Natural coagulants for wastewater treatment

The potential environmental and human health hazards associated with the use of chemical coagulants has necessitated the need for the use of natural coagulants for industrial wastewater treatment. Natural coagulants are gaining a lot of attention these days as they are an effective alternative to the chemical coagulants (Yin, 2010). Natural coagulants are a sustainable approach Folkard et al., 1995). Plant-based materials have been investigated for treating industrial effuents from different industries. Plant-based substances like Moringa Oleifera (Chonde and Raut, 2017; Sivakumar, 2013), chitosan and chitin, Abelmoschus esculentus, Opuntia ficus-indica, Synchorous Potatorum, Prosopis laevigata Seed Gum, Hibiscus rosa-sinensis (Awang and Aziz, 2012), Acacia mearnsii (Beltrán-Heredia et al., 2011), etc. can be used as coagulants. Generally, the natural coagulants are directly used as a powder or a stock solution. In some cases, the deoiled powder is also used, after extraction of oil from the coagulant. The plant-based products (such as seeds, etc.) are first extracted from the plant, cleaned to remove any impurities that may interfere with coagulation, and then dried. The powder is then formed (with or without the oil extraction, as per need) by grinding (Miller et al., 2008). This powder may be directly used, or a stock solution can be prepared from it. In some cases, proteins may be extracted from the specific plant parts and used as a coagulant. This may require extensive extraction and purification steps (Kansal and Kumari, 2014). Kumar et al. 2017 and Tariq et al., 2015 reported that the microbial polysaccharides, starches, gelatin galactomannans, cellulose derivatives, chitosan, glues, and alginate can be used for wastewater treatment. Mohamed et al. 2014 used Moringa oleifera and Strychnos potatorum seeds as a natural coagulant for car wash wastewater. The turbidity and COD reduction efficiency of coagulants was studied. Using Moringa oleifera, 94% turbidity, 60% COD, 81% phosphorus removal were obtained, whereas using Strychnos potatorum, 97% turbidity, 54% COD, and 82% phosphorus removal were obtained. These results were compared with synthetic coagulants, and natural coagulants were suggested for coagulation process as they provide better treatment, are cost-effective and are safe for environment. Kani et al. 2016 used banana pith juice for textile wastewater treatment. At pH 4, 97.5% turbidity and 50.1% total solids were removed from the wastewater. There was a significant improvement in the electrical conductivity. The results confirmed that banana stem juice has an enormous potential for turbidity removal from the textile wastewater. Gandhi et al. 2013 used plant-based polyelectrolytes as coagulants (derived from the fruits of Opun□a □cus indica, fruits of Jatropha gossypifolia and Borassus □abelli□er) to remove chromium. It was concluded that there is a significant improvement in the physicochemical characteristics of wastewater and heavy metal chromium was successfully controlled by natural coagulants. These polyelectrolytes can be effectively used for removal of chromium as they destabilize and reduced the repulsive forces between the molecules. Prodanovic et al. 2015 used the common bean for dis□llery wastewater treatment and it was concluded that pH value of sillage influenced activity of the natural coagulant. The optimum pH for the treatment was reported to be 8.5.

2.18.1 Advantages of natural coagulants

Wastewater treatment using natural coagulants is an eco-friendly option. Natural coagulants are non-toxic, biodegradable, and environment friendly (Verma et al., 2012; Muralimohan et al., 2014). Unlike synthetic coagulants, treated water contains no residual aluminium. Prodanović et al. 2013 used common bean extract for the treatment of dis□llery wastewater treatment. The study claimed that anaerobic sludge contained no aluminium salt.

Need for natural coagulants Chemical coagulant used has raised controversial issues due to its toxic nature for living organisms and can be categorized into three types: hydrolyzing metallic salts, pre-hydrolyzing metallic salts, and synthetic cationic polymers(Freitas et al., 2018; Verma et al., 2012). Due to the low cost, easy handling, storage, and high availability, chemical coagulants are more prevalent in wastewater treatment processes.

Al2(SO4)3, Fe2(SO4)3, AlCl3, and FeCl3 are the most commonly used coagulant salts (Freitas et al., 2018; Matilainen et al., 2010; Sher et al., 2013). Despite the availability, low cost etc.; chemical coagulants are far behind in green chemistry due to high residual concentrations of aluminum found in treated wastewater(Freitas et al., 2018; Matilainen et al., 2010). According to Freitas et al., 2018; McLachlan 1995; Polizzi et al. 2002, Alzheimer's disease is linked with the neurotoxicity of aluminum. Synthetic polymer coagulants form hazardous secondary products such as acrylamide which is carcinogenic and neurotoxic, and also synthetic polymers have low biodegradability (Freitas et al., 2018;Kurniawan et al., 2020). Excessive concentrations of chemical coagulants such as aluminum reduce the pH of water tends and also, they can be accumulated to food chains (Kurniawan et al., 2020). Improper disposal of toxic sludge pollutes the groundwater and soil. Accumulation of toxic sludge, such as aluminum, iron etc., in natural water bodies causes adverse effects on aquatic organisms and plant species (Kurniawan et al., 2020). Hence there is a need for the efficient utilization of natural coagulants for water and wastewater treatment.

2.19APPLICATIONS, ADVANTAGES AND DISADVANTAGES OF BANANA PEEL AS A COAGULANT.

Applications of banana peel as a coagulant

- To identify a sustainable, simple, locally available.
- Ecofriendly water treatment technology which is more suitable for the earth to protect it from pollution caused by
- chemical coagulant.
- Evaluate the optimum dosages of banana powder for a different level to remove turbidity.
- Removal efficiency is very high in banana powder.

2.19.1 Advantages of banana peel as a coagulant.

- It is non-toxic and safe for consumption
- It is biodegradable
- Safe for consumption.

- Banana peel is rich in organic compounds
- Banana peel has high nutrients.
- Banana peel has lignin
- Banana peels are a good source of galacturonic acid
- Banana peel is consumer approachable and eco-friendly, a substitute for minor size water treatment.
- Banana peel is a renewable source that can be grown on a huge scale.
- Using Banana peel as an alternate coagulant that meets up water quality factors.
- Banana peel is simple to use, effortlessly available, easy to maintain and can be used as a domestic coagulant.
- The banana peel can remove Hardness, Chlorides, and Residual chlorine
- It is eco-friendly proficiency which has additional benefits more than other treatment alternatives.

2.19.2 Disadvantages of banana peel as a coagulant.

- Availability of dried peel is a bit difficult.
- It requires a large quantity of growing.
- The smell it may cause after using in water treatment.

2.20 Mechanism of coagulation by natural coagulants

Coagulation occurs between the coagulant added, the impurities, and the alkalinity of the water, resulting in the formation of insoluble flocs. Flocs are the agglomerations of particulate suspended matter in the raw water, reaction products of the added chemicals, colloidal and dissolved matter from the water adsorbed by these reaction products. Unprocessed water from the reservoir contains organic and inorganic impurities, such as silt, rotten substance, alga, bacterium, etc. Hence coagulation is the essential step in water purification. In addition, coagulants make suspensions in water to gather and reduce the turbidity of water (Z. Song et al., 2009). The successful coagulation of natural coagulants (Ang et al., 2020) stands on these three pillars: characteristics of coagulant used, characteristics of water to be treated, characteristics of mixing process (Ang et al., 2020; Kumar et al., 2017). As Fig. 1 shows, these coagulation factors play a significant role in determining the most efficient coagulant required for the treatment. Coagulants' molecular weight (Ang et al., 2020; Gautam and Saini, 2020), types of equipment and reagents used, chemical and physical properties of the pollutants such as zeta potential (Ang et al., 2020), color, the concentration of the colloidal particles, the presence or absence of impurities (trace elements and dissolved salts (ions and chemicals) also affect the coagulation process (Ang et al., 2020; Kumar et al., 2017; Muruganandam et al., 2017). If the natural coagulant contain positive surface charge, its coagulation activity against negatively charged suspended particles will be higher and vice versa for negatively charged natural coagulants with positively charged suspended particles. Functional groups also contribute to surface charge (Ang et al., 2020). Molecular weight of natural coagulant is very important in particle bridging. If the molecular weight of natural coagulant is higher, it can form strong bridges with the particles and it leads to the formation of strong flocs and improve settling (Ang et al., 2020). Mixing is another critical step in the coagulation process. Fast mixing increases the interactions between coagulants and suspended particles and forms micro flocs. Slow mixing leads to the aggregation of micro flocs into large flocs (Kurniawan et al., 2020). Coagulation also affects the other steps of the treatment process. An efficient and effective coagulation process favors the microbiological quality (Kumar et al., 2017) of the end product and increases the lifetime of filters (Kumar et al., 2017), reducing the total cost of treated water. Natural coagulants are composed of carbohydrates, protein, and lipids. The primary building blocks are the polymer of polysaccharides and amino acids. According to the previous research, the main mechanisms governing coagulation activity are charge neutralization and polymer bridging. Polymer bridging is preceded by polymer adsorption. Because of the affinity between long-chain polymers and colloidal particles, long-chained polymers can attach to the colloidal particle’s surface. A part of the polymer is attached to the particle while the other parts form loops and tails. These loops and tails are the main structure of polymer bridging loops, and tails allow attaching to other colloidal particles and form larger flocs. The basis of charge neutralization is known as the electrostatic patch mechanism. The patches of positive and negative regions on the particle’s surface cause the additional attraction between particles. Ionizable polymer (polyelectrolytes) is used as a coagulant in the charge neutralization mechanism. It stabilizes the negatively charged colloidal particles. Polycation is used to stabilize the particles, gaining near to zero zeta potential. The optimum dosage of polyelectrolyte needed will be determined by the charge density of the polyelectrolyte (Amran et al.,2018; Yin, 2010). Natural coagulants have varied mechanisms of action. Let us consider some of the coagulation mechanisms of natural coagulants.

2.21 Potential coagulants presents in banana peel powder and possible mechanism of coagulation

Banana peel powder was found to be a better coagulant than others in terms of coagulant activity (turbidity removal). Banana peel is composed of polymeric substances such as fiber (11.04%) and protein (10.14%) (Memon et al. 2008). The FTIR analysis of banana peels revealed various peaks of different functional groups such as carboxylic acid (C=O), hydroxyl (-OH) and aliphatic amines (N–H) indicating the presence of both positively and negatively charged species in the polymeric substances (Fig. S1). These functional groups of banana peel powder might be responsible for promoting the coagulation–flocculation by neutralizing both positively and negatively charged impurities in water. Memon et al. (2008) and Thirumavalavan et al. (2011) also reported the role of these functional groups in the removal of impurities from water.

CHAPTER THREE

3.0. MATERIALS AND METHODS

This chapter highlights the methods as well as the procedures used in the Evaluation of banana peel powder as natural coagulant for treatment of wastewater.

3.1 Sample Collection and Preparation

About 10 litres of the Juhel wastewater were collected and used throughout the experiment as the testing sample. The sample was taken to the laboratory immediately and initial analysis done so as to get the initial Turbidity without allowing the sample to settle. A total sum of five experiments were conducted.

3.2. Apparatus

1. Five Beakers
2. Volumetric pipette
3. 1 Litre measuring cylinder
4. Electronic weighing balance
5. Turbidity Meter
6. Electro-magnetic stirrer
7. Stop watch
8. Voltage regulator
9. Standby power generator

3.3 Materials

Banana (Musa Acuminata species) peels were bought from Eke Awka market, Awka Anambra state. The banana peels were cut into small pieces and washed thoroughly with tap water to remove any external dirt. The washed pieces of banana peels were **air-dried under** sunlight for 2 weeks and oven-dried for 24 h at 105 °C, and were finely powdered using electric grinder and then sieved through 2.36 mm IS sieves.

3.4 Precautions

The following precaution were taken so as to obtain accurate results

1. The sample was collected on the very day of the experiments so as not to allow it settle before the commencement of the tests.
2. A minimum of five readings were taken and the average obtained for a particular reading.
3. Other rules and principles in the use of various metres used were strictly followed

3.5 Banana Peel Structure

The spectrum of raw banana peels shown in Fig. 1 generated by FT-IR. Starting right hand side from 500 cm-1, the weak peeks are formed initially with very small gaps indicating inside structure. As wave number increases and reaches around 1000cm-1, the declining in the spectrum become high but peaks strength remains the same. If analysed further the spectrum become narrow and near wave number 1500 cm-1 the peaks become sharper and the number of peaks within this range become high. This increase in number of peaks in spectrum continues up to 2200 cm-1. After this the peaks strength again start getting weaker and number of peaks start decreasing and the spectrum start declining up to 3300 cm-1. As wavenumber further approaches to 3400 cm-1 and onwards spectrum pattern shows ascending behaviour. Number of peaks becomes sharper and increased in numbers near the end of spectrum. In Fig. 1 ascending and descending pattern of spectrum and number of peaks and their sharpness shows the adsorption

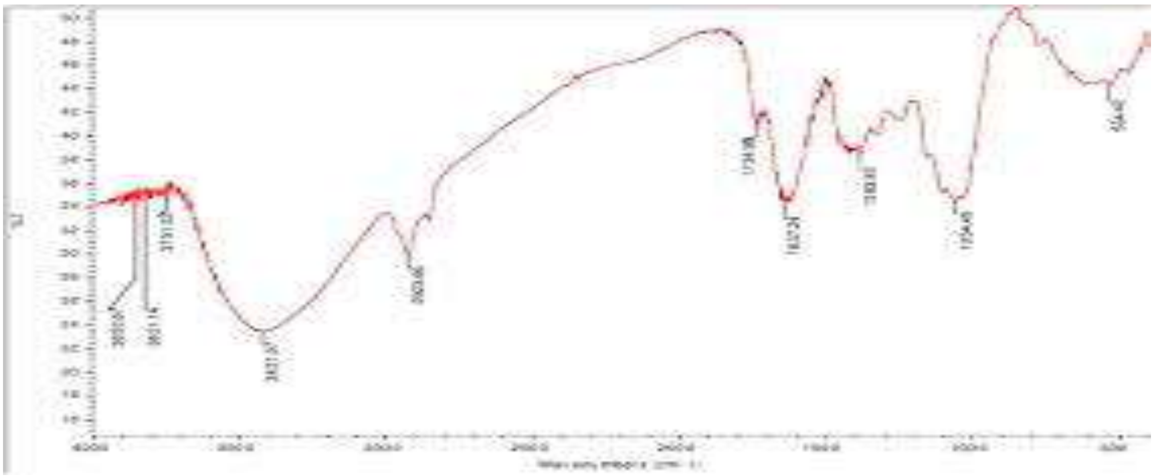


FIG 1.

properties of the sample under consideration. The portion of the spectrum where number of peaks are less sharp and decrease in number shows that the rays couldn't get enough space to penetrate inside the surface and reflect and finally the fewer peaks are formed. While sharper portion of the peaks are greater in number shows the penetration behaviour of ray inside the surface. Penetration of rays inside the structure shows porous surface and hence greater the penetration of peaks more will be the porosity factor. In untreated banana peels majority of sharp peaks were found in the range of 1600 cm-1 to 2100 cm-1. According to FT-IR wavenumber table in this range alkenyl C=C stretch, aryl substituted C=C and terminal alkyne C≡C functional groups are present [27]. Similarly, sharp peeks in raw banana peel are also formed from 3600 cm-1 and onwards. This range of wavenumber contains hydroxyl groups -OH with bond stretch. Stretch in bonds shows weak attractive forces between the bonds. So overall raw banana peel shows some tendency of adsorption process because of unsaturated double and triple bonded functional groups.

3.6 Procedure

The objective of the test is to determine the effectiveness of banana peel powder as a natural coagulant for treatment of wastewater. Coagulation is not yet an extract science, although recent advances have been made in understanding the mechanics of the process (Okoro 2007). Therefore, selection and optimum dosage of coagulants are determined experimentally by the jar test. The jar test must be performed on any water that is to be coagulation and must be repeated with each significant change in the quality of the water. The jar test was performed using a series of glass beakers of uniformed sizes and shapes. Five jar beakers were used with a stirring device that mixes the contents of the jar simultaneously with uniform power point. Each of the five beakers was properly washed and cleaned to ensure zero impurities in them, and afterwards filled to the 1000ml mark with the wastewater. Different dosages of banana peel powder were added ranging from 0.1mg – 0.7mg. One of the beakes were left untreated with no reagent to serve as control and the remaining four were treated with the banana peel powder. The aim of this is to determine the efficiency of these coagulants by comparing the turbidity of the treated ones at different time intervals with the turbidity of the untreated one (the one that settled naturally) after the same time interval. After chemical addition, each of the beaker containing the wastewater, coagulant and a magnetic stirrer was placed on the electro-magnetic stirrer with hot plate and was connected to a power supply. The mixture was rapidly mixed using the electro-magnetic stirred for three (3) minutes and followed by 25minutes of slow mixing. This is to keep flocs particles to suspend uniformly. The mixture was left for one hour to allow settling and some portions

TABLE 3.1: VALUES OF THE TURBIDITY AT DIFFERENT TIMES AND ALSO THE VALUE OF THE INITIAL TURBIDITY.

TIME	**COAGULANT DOSAGE**				**INITIAL TURBIDITY**
	0.1	O.3	0.5	0.7	
10	57	50	55	60	
20	55	47	54	57	
30	50	40	51	54	65 (NTU)
40	50	37	47	50	
50	48	40	45	47	
60	47	39	46	45	

of the settling water were pipette out at 2cm depth at different time intervals to

determine the Turbidity by using the Turbidity meter measured in Nephelometric

Turbidity Unit (NTU).

The turbidity reduction was calculated for each sample by using the Formula as in Eq. 1..

Reduction of turbidity (NTU) = initial turbidity (NTU) – final turbidity (NTU)....1

CHAPTER FOUR

4.0 RESULT ANALYSIS AND DISCUSSION

4.1 Jar Test Result

The graphical representation of the result was used in this chapter to make influence. The tables for the results are shown below.

4.2 Determination of Optimum Dosage

The optimum dosage should correspond to the column with the least turbidity value (Matti et al, 2004). And also the optimum dosage has the highest settling rate, that is, more particles were removed within the stated time.

Therefore, looking and comparing the values gotten by the use of different centrations of banana peel ash as shown in table 4.1 and represented in Fig. 4.1. **0.1mg/L** of coagulant gave a turbidity value of 57NTU, 55NTU, 50NTU, 50NTU, 48NTU and 47NTU after every ten minutes' intervals for 60minutes respectively. And by increasing the concentration to **0.3mg/L**, lower values were obtained. But further increase to **0.5mg/L** and more gave higher turbidity values showing that the optimum dosage of banana peel coagulant required for the coagulation of Juhel wastewater is **0.3mg/L.**

TABLE 4.1: RESULTS OF THE TURBIDITY REMOVAL AND TURBIDITY EFFICIENCY (%) FOR DIFFERENT COAGULANT DOSAGE.

Time	Coagulant Dosage(NTU)	Initial Turbidity (NTU)	Final Turbidity (NTU)	Turbidity Removal (NTU)	Turbidity Efficiency(%)
10min	0.1	65	57	8	12.31%
	0.3	65	50	15	23.08%
	0.5	65	55	10	15.38%
	0.7	65	60	5	7.69%
20min	0.1	65	55	10	15.38%
	0.3	65	47	18	27.69%
	0.5	65	54	11	16.92%
	0.7	65	57	8	12.31%
30min	0.1	65	50	15	23.08%
	0.3	65	40	25	38.46%
	0.5	65	51	14	21.54%
	0.7	65	54	11	16.92%
40min	0.1	65	50	15	23.08%
	0.3	65	37	28	43.08%
	0.5	65	47	18	27.69%
	0.7	65	50	15	23.08%
50min	0.1	65	48	17	26.15%
	0.3	65	40	25	38.46%
	0.5	65	45	20	30.77%
	0.7	65	47	18	27.69%
60min	0.1	65	47	18	27.69%
	0.3	65	39	26	40%
	0.5	65	46	19	29.23%
	0.7	65	45	20	30.77%

4.3 DATA ANALYSIS.

Initial turbidity = f1

Final turbidity = f2

Calculate for turbidity removal

Turbidity Removal = Initial turbidity – Final turbidity

$$\mathbf{T_R = f1 - f2}$$

Calculating for 10 minutes;

For 0.1ml dosage, T_R = 65 – 57 = 8

0.3ml dosage, T_R = 65 – 50 = 15

0.5ml dosage, T_R = 65 – 55 = 10

0.7ml dosage, T_R = 65 – 60 = 5

Calculating for 20 minutes;

For 0.1ml dosage, T_R = 65 - 55 = 10

0.3ml dosage, T_R = 65 – 47 = 18

0.5ml dosage, T_R = 65 – 54 = 11

0.7ml dosage, T_R = 65 – 57 = 8

Calculating for 30 minutes;

For 0.1ml dosage, T_R = 65 – 50 = 15

0.3ml dosage, T_R = 65 – 40 = 25

0.5ml dosage, T_R = 65 – 51 = 14

0.7ml dosage, T_R = 65 – 54 = 11

Calculating for 40 minutes;

For 0.1ml dosage, T_R = 65 – 50 = 15

0.3ml dosage, T_R = 65 – 37 = 28

0.5ml dosage, T_R = 65 – 47 = 18

0.7ml dosage, T_R = 65 – 50 = 15

Calculating for 50 minutes;

For 0.1ml dosage, T_R = 65 – 48 = 17

0.3ml dosage, T_R = 65 – 40 = 25

0.5ml dosage, T_R = 65 – 45 = 20

0.7ml dosage, T_R = 65 – 47 = 18

Calculating for 60 minutes;

For 0.1ml dosage, T_R = 65 – 47 = 18

0.3ml dosage, T_R = 65 – 39 = 26

0.5ml dosage, T_R = 65 – 46 = 19

0.7ml dosage, T_R = 65 – 45 = 20

4.4 Turbidity Removal Efficiency Calculation.

T.R.E = F1 – F2/F1 x 100 = Turbidity Removal/F1 x 100

Calculating for 10minutes;

For 0.1ml dosage = 8/65 X 100 = 12.31%

0.3ml dosage = 15/65 X 100 = 23.08%

0.5ml dosage = 10/65 X 100 = 15.38%

0.7ml dosage = 5/65 X 100 = 7.69%

Calculating for 20 minutes;

For 0.1ml dosage = 10/65 X 100 = 15.38%

0.3ml dosage = 18/65 X 100 = 27.69%

0.5ml dosage = 11/65 X 100 = 16.92%

0.7ml dosage = 8/65 X 100 = 12.31%

Calculating for 30 minutes;

For 0.1ml dosage = 15/65 X 100 = 23.08%

0.3ml dosage = 25/65 X 100 = 38.46%

0.5ml dosage = 14/65 X 100 = 21.54%

0.7ml dosage = 11/65 X 100 = 16.92%

Calculating for 40 minutes;

For 0.1ml dosage = 15/65 X 100 = 23.08%

0.3ml dosage = 28/65 X 100 = 43.08%

0.5ml dosage = 18/65 X 100 = 27.69%

0.7m dosage = 15/65 X 100 = 23.08%

Calculating for 50 minutes

For 0.1ml dosage = 17/65 X 100 = 26.15%

0.3ml dosage = 25/65 X 100 = 38.46%

0.5ml dosage = 20/65 X 100 = 30.77%

0.7ml dosage = 18/65 X 100 = 27.69%

Calculating for 60 minutes;

For 0.1ml dosage = 18/65 X 100 = 27.69%

0.3ml dosage = 26/65 X 100 = 40%

0.5ml dosage = 19/65 X 100 = 29.23%

0.7ml dosage = 20/65 X 100 = 30.77%

Taking average of 0.1ml dosage for all time intervals

$$\frac{12.31 + 15.38 + 23.08 + 23.08 + 26.15 + 27.69}{6}$$

= 21.28%

Taking average of 0.3ml dosage for all time intervals

$$\frac{23.08 + 27.69 + 38.46 + 43.08 + 38.46 + 40}{6}$$

= 35.13%

Taking average of 0.5mk dosage for all time intervals

$$\frac{15.38 + 16.92 + 21.54 + 27.69 + 30.77 + 29.23}{6}$$

= 23.59&

Taking average of 0.7ml dosage for all time intervals

$$\frac{7.69 + 12.31 + 16.92 + 23.08 + 27.69 + 30.77}{6}$$

= 19.75%

4.5 Efficiency

Having a control aimed at investigating the benefit of using these coagulants. And if sample is allowed to settle naturally, and at the end the turbidity value becomes low as the treated then the treated is of no use. Thus, the turbidity of the treated and untreated sample was checked at regular time intervals. Therefore, from the table above, we calculate the efficiency of each reagent by the formular given below.

Efficiency, S = Change in turbidity/Initial turbidity X 100

4.6 Colour of End Product

After the end of the test, the end product of the treated sample gave perfect colorless water at the end of the day.

Summarily, Banana peel powder has the following advantages.

1. It is effective and efficient
2. It produces a colorless end product thus no need for further treatment to remove colour.
3. Generally, it is very affordable and available

CHAPTER FIVE

5.0 CONLUSION, AND RECOMMENDATION

5.1 CONCLUSION

Jar test results were used to calculate the types and quantity of coagulant to be used in the treatment of the sample. It also served to illustrate the rate order of coagulation reaction of the sample.

Turbidity curve plotted for the experiment shows that as the concentration of the coagulant increases as the turbidity of the wastewater reduces. (Tiuman 1996). This obeyed until the optimum dosage is attained and the reverse becomes the case.

5.2 Recommendation

This experimental study has produced valuable results for characterizing the rate order of settling and idea of the dosage of the coagulant (banana peel powder) needed.

Due to the quality variation of natural water from time to time and from season to season this dose cannot be fixed. Again due to unknown factors influencing coagulation of water, it is impossible to predict the type and amount of reagents necessary to achiee desired result economically for every water except through tests. Thus, trial and error experimentation via the jar test procedure must be carried out.

Owing to the complexity of this research subject, further investigation is recommended.

1. There is need to carry out this study using particle counter method.
2. There is also the need to carry out study on other treatment processes since coagulation is just a part of water treatment.
3. Other factors that influence coagulation of this wastewater sample should be investigated.
4. Experimental techniques still need to be improved for more accurate measurement of coagulant dose and turbidity.

APPENDIX A

PLATE 1: DIGITAL TURBIDITY METER

APPENDIX B

PLATE 2:MAGNETIC STIRRER WITH HOT PLATE AND THE FIVE BEAKERS CONTAINING THE WASTEWATER SAMPLE.

PLATE 3: ELECTRO-MAGNETIC STIRRER.

APPENDIX C

PLATE 4: MAGNETIC STIRRER WITH A BEAKER CONTAINING THE SAMPLE AND A MAGNETIC STIRRER.

PLATE 5: MYSELF WHILE I WAS PERFORMING THE PRACTICAL.

APPENDIX D

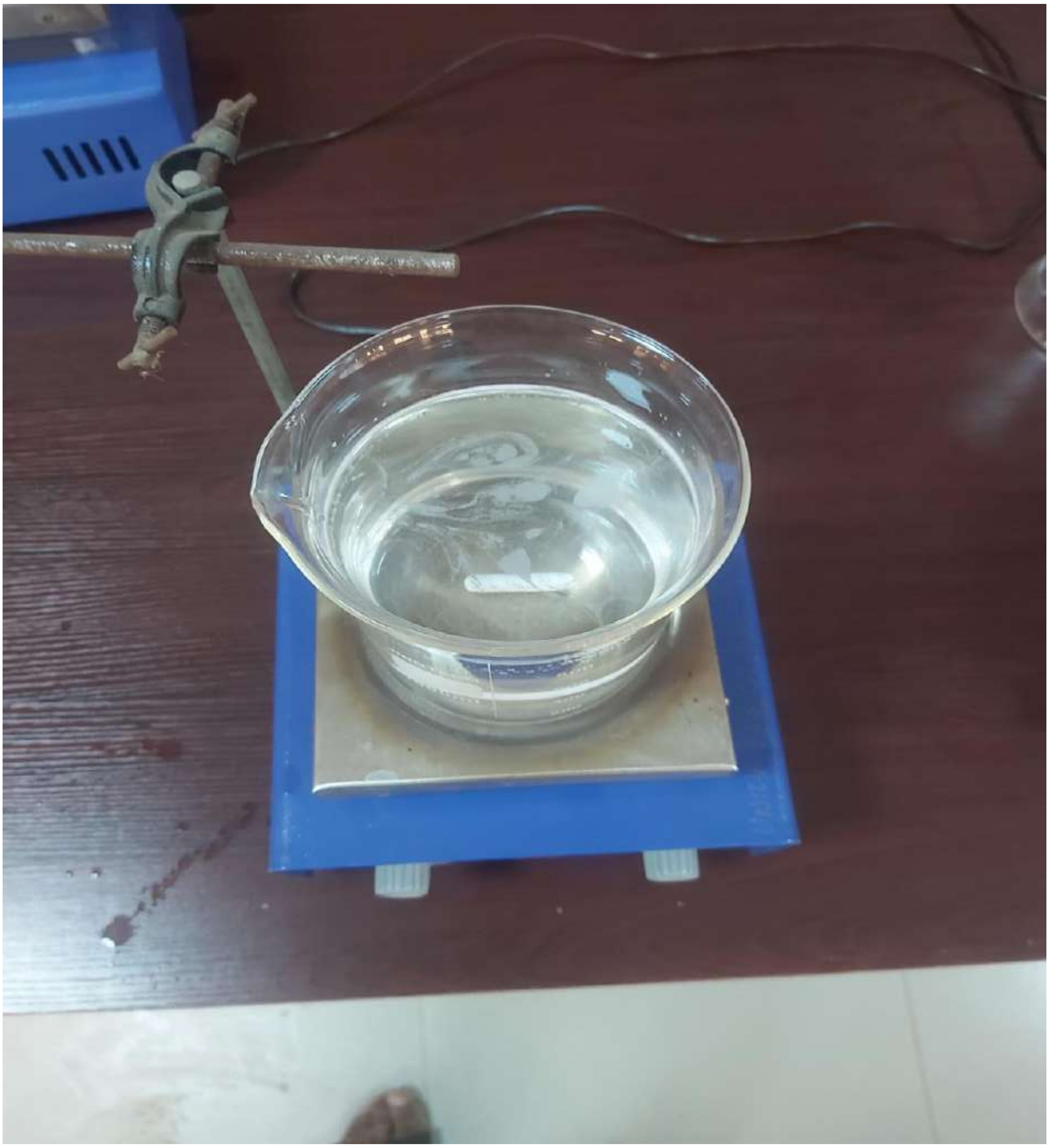


PLATE 6: A PICTURE SHOWING THE APPEARANCE OF THE WASTEWATER SAMPLE AFTER COAGULATION.

REFERENCES

1. Alwi H, Idris J, Musa M, Hamid KHK (2013) A preliminary study of banana stem juice as a plant-based coagulant for treatment of spent coolant wastewater. J. Chem 165057:7
2. Anju S, Mophin-Kani K (2016) Exploring the use of orange peel and neem leaf powder as alternative coagulant in treatment of dairy wastewater. IJSER 7(4):238–244
3. Asrafuzzaman M, Fakhruddin ANM, Alamgir Hossain M (2011) Reduction of turbidity of water using locally available natural coagulants. ISRN Microbiol 632189:1–6
4. Ayekoe, Chia Yvette Prisca; Robert, Didier; Lanciné, Droh Gone (2017-03-01). "Combination of coagulation-flocculation and heterogeneous photocatalysis for improving the removal of humic substances in real treated water from Agbô River (Ivory-Coast)". Catalysis Today. 281: 2–13. doi:10.1016/j.cattod.2016.09.024
5. Beyene, H. D., Hailegebrial, T. D. and Dirersa, W. B. 2016. Investigation of coagulation activity of cactus powder in water treatment. Journal of Applied Chemistry, 2016 (1): 1-9. https://doi.org/10.1155/2016/7815903
6. Carvalho MS, Alves BRR, Silva MF, Bergamasco R, Coral LA, Bassetti FJ (2016) CaCl2 applied to the extraction of Moringa oleiferaseeds and the use for Microcystis aeruginosa removal. Chem Eng J 304:469–475
7. Choy SY, Prasad KMN, Wu TY, Raghunandan ME, Ramanan RN (2014) Utilization of plant-based natural coagulants as future alternatives towards sustainable water clarifcation. J Environ Sci 26:2178–2189
8. Chekli, L.; Eripret, C.; Park, S. H.; Tabatabai, S. A. A.; Vronska, O.; Tamburic, B.; Kim, J. H.; Shon, H. K. (2017-03-24). "Coagulation performance and floc characteristics of polytitanium tetrachloride (PTC) compared with titanium tetrachloride (TiCl4) and ferric chloride (FeCl3)

in algal turbid water". Separation and Purification Technology. 175: 99–106. doi:10.1016/j.seppur.2016.11.019. hdl:10453/67246

9. Corrosionpedia.com The online hub for corrosion professionals. 2018

48

10. Kakoi B, Kaluli JW, Ndiba P, Thiong'o G (2016) Banana pith as a natural coagulant for polluted river water. Ecol Eng 95:699–705

11. Kukić, D., Šćiban, M., Prodanović, J., Vasić, V., Antov, M. and Nastić, N. 2018. Application of natural coagulants extracted from common beans for wastewater treatment. e-GFOS, 9 (16): 77-84. https://doi.org/10.13167/2018.16.7

12. R.Subashree, N.Surya Praba, Dr.G.Anusha.Investigation of Coagulation Activity of Lemon and Banana Peel Powder in Water Treatment Bannari Amman Institute of Technology, Sathyamangalam, Erode,Tamil Nadu (India)

13. Teh, C.Y., Wu, T.Y. and Juan, J.C., 2014. Potential use of rice starch in coagulation–flocculation process of agro-industrial wastewater: treatment performance and flocs characterization. Ecological engineering, 71, pp.509-519.

14. Tetteh, E.K. and Rathilal, S., 2019. Application of organic coagulants in water and wastewater treatment. IntechOpen

15. Tetteh, E.K. and Rathilal, S., 2020. Evaluation of different polymeric coagulants for the treatment of oil refinery wastewater. Cogent Engineering, 7(1), p.1785756. https://doi.org/10.1080/23311916.2020.1785756

16. vandi, Bahman (2014-08-01). "Treatment of water turbidity and bacteria by using a coagulant extracted from Plantago ovata". Water Resources and Industry. 6: 36–50. doi:10.1016/j.wri.2014.07.001.

ANALYSIS AND DESIGN OF TREATMENT FACILITY FOR WASTEWATER FROM BOY'S HOSTEL, NNAMDI AZIKIWE UNIVERSITY, AWKA USING WASTE STABILIZATION PONDS SYSTEM

OKALIWE DAVID FELIX

2016224007

DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

NNAMDI AZIKIWE UNIVERSITY, AWKA

JANUARY, 2022

ANALYSIS AND DESIGN OF TREATMENT FACILITY FOR WASTEWATER FROM BOY'S HOSTEL, NNAMDI AZIKIWE UNIVERSITY, AWKA USING WASTE STABILIZATION PONDS SYSTEM

OKALIWE DAVID FELIX

(2016224007)

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG) IN THE

DEPARTMENT OF CIVIL ENGINEERING

NNAMDI AZIKIWE UNIVERSITY, AWKA

JANUARY, 2022

CERTIFICATION PAGE

This project work "Analysis and Design of Treatment Facility for Wastewater from Boy's Hostel, Nnamdi Azikiwe University, Awka Using Waste Stabilization Ponds System" was carried out by me under the supervision of Engr. Prof. O.E. Ekenta and has not been submitted in part or full to this university or other institutions for the award of a degree.

Okaliwe David Felix   Date

APPROVAL PAGE

This is to certify that this project work written by Okaliwe David Felix with registration number 2016224007 has been supervised and approved by the Department of Civil Engineering, Nnamdi Azikiwe University, Awka:

Engr. Prof. O.E. Ekenta (Supervisor)	Date
Engr. Dr. A.C. Ezeagu (Head of Department)	Date
Engr. Prof. D.O. Onwuka (External Examiner)	Date

DEDICATION

This project work is dedicated to the Almighty God, without Whom I would not have come this far. Special dedication also goes to my irreplaceable parents, Dr. and Mrs. E.F. Okaliwe who have been fully supportive in every ramification throughout this academic journey.

ACKNOWLEDGEMENT

I wish to express my gratitude to my project supervisor, Engr. Prof. O.E. Ekenta for his support, tutelage and guidance which have led to the actualization of this project work.

Worthy of special recognition and appreciation are my lecturers who have imparted in me academically, morally and otherwise beginning at the Head of Department of this great Department of Civil Engineering, Engr. Dr. A.C. Ezeagu, whose guidance and effective administration has been of great help throughout this journey.

Furthermore, I would love to appreciate Engr. Prof. C.M.O. Nwaiwu, Engr. Prof. C.H. Aginam, Engr. Prof. N.E. Nwaiwu, Engr. Prof. A.C. Chidolue, Engr. Dr. B.O. Adinna, Engr. Dr. P.D. Onodagu, Engr. Dr. V.O. Odinaka, Engr. Dr. A.A. Ezewamma, Engr. Dr. A.I. Nwajuaku, Engr. Dr. M.C. Nwakaire, Engr. Mrs. I.I. Nwajuaku, Engr. Mrs. N.M. Ezema, Engr. B. Njotae, Engr. I.E. Omaliko, Engr. O.U. Ubani, J.I. Obiora, E.O. Mezie, F.C. Uzodimma for their various contributions towards my academic growth and wellbeing.

My appreciation also goes to all non-teaching staff, Laboratory attendants and technicians especially Mr S.S. Akerejola and Mr J.C. Onwuliri for their support and guidance throughout my stay especially during the course of this project.

I would not pass the opportunity to thank my special and amazing parents for their support, love and encouragement, your rewards are unquantifiable.

I appreciate my siblings, Engr. Daniel Okaliwe and Mrs Grace Ujah for their availability, concern and support. My friends and colleagues, especially Chuks and Peter, God bless you.

ABSTRACT

This project work presents the analysis and design of treatment facility for wastewater from Boy's Hostel, Nnamdi Azikiwe University, Awka using waste stabilization ponds system. The use of waste stabilization ponds to treat wastewater is an effective, low maintenance and low-cost method. Laboratory experiments were conducted in the Engineering Laboratory (water lab.) and Springboard Research Laboratory on parameters such as pH, dissolved solids, suspended solids, Bio-chemical Oxygen Demand (BOD) and Chemical Oxygen Demand (COD) from which values were obtained for the design of the ponds: anaerobic, facultative and maturation. The ponds were designed on the basis of a flow of 875m^3/d. A design temperature of 25^{0}C was adopted for the operation of the ponds. The anaerobic pond was designed on the basis of volumetric loading, has a retention time of 1.03days which reduces the BOD loading from 360mg/l in the influent to 108mg/l. The facultative pond was designed on the basis of surface volumetric loading, it has a retention time of 28.5 days and the influent BOD of 108mg/l reduces to 23mg/l and has an area of 25,000m^2. The maturation pond is designed according to Marias' method (1974) for faecal removal. It has a retention time of 7 days for complete decomposition and faecal coliform reduces to 224.08fc/100ml.

TABLE OF CONTENTS

Table of Contents		Page
Title Page		ii
Certification Page		iii
Approval Page		iv
Dedication		v
Acknowledgement		vi
Abstract		vii
Table of Contents		viii
List of Figures		xiv
List of Tables		xv
List of Symbols and Abbreviations		xvi
CHAPTER ONE: INTRODUCTION		
1.1	Background of the Study	1
1.2	Problem Statement	2
1.3	Aim and Objectives	3
1.4	Significance of the Project	3
1.5	Scope of the Study	4
1.6	Limitations of the Study	4
CHAPTER TWO: LITERATURE REVIEW		
2.1	Waste Stabilization Ponds	5
2.1.1	Arrangement of Waste Stabilization Ponds	7

2.1.2 Anaerobic Ponds 7

2.1.2.1 Sludge Accumulation in Anaerobic Ponds 11

2.1.3 Facultative Ponds 12

2.1.3.1 Influence of Algae 14

2.1.4 Maturation Ponds 16

2.1.4.1 Process Description 17

2.1.4.2 Quality Requirements for the Effluent 18

2.2 Waste Stabilization Pond Terminologies 19

2.2.1 Biochemical Oxygen Demand Removal Kinetics 19

2.2.2 Hydraulic Flow Regime 20

2.2.3 Complete Mixing 20

2.2.4 Plug Flow 21

2.2.5 Dispersed Flow 21

2.3 Review of Related Literature 22

2.3.1 Model Based Approaches of Waste Stabilization Pond Designs 23

2.3.1.1 Rules of Thumb 23

2.3.1.2 Regression Equations 23

2.3.1.3 First-Order Models 24

2.3.1.4 Mechanistic Models 24

2.4 Summary of Reviewed Literature 25

2.5 Literature Gaps 26

CHAPTER THREE: MATERIALS AND METHODS

3.1 Research Methodology 28

3.2 Environmental Description 28

3.3 Pond Diagrams 29

3.4 Supporting Equations for Waste Stabilization Ponds Design 30

3.4.1 Anaerobic Pond Design 30

3.4.1.1 Volumetric Organic Loading Rate 31

3.4.1.2 Anaerobic Pond Area 31

3.4.1.3 Retention Time 32

3.4.1.4 Depth 33

3.4.1.5 Geometry 33

3.4.1.6 Design Values for Anaerobic Ponds 33

3.4.1.7 Estimation of the Effluent BOD Concentration from the Anaerobic Pond 34

3.4.2 Facultative Pond Design 35

3.4.2.1 Depth 37

3.4.2.2 BOD Removal 37

3.4.2.3 Microbiological Quality 38

3.4.2.4 Human Intestinal Nematode Egg Removal 38

3.4.3 Maturation Pond Design 39

3.5 Test Parameters and Procedure 41

3.5.1 pH 41

3.5.1.1 Apparatus 41

3.5.1.2 Procedure 42

3.5.1.3 Result and Conclusion 42

3.5.2 Suspended Solids 42

3.5.2.1 Apparatus 43

3.5.2.2 Procedure 44

3.5.2.3 Result and Conclusion 44

3.5.3 Dissolved Solids 44

3.5.3.1 Apparatus 45

3.5.3.2 Procedure 46

3.5.3.3 Result and Conclusion 46

3.5.4 Biochemical Oxygen Demand 46

3.5.4.1 Apparatus 47

3.5.4.2 Procedure 47

3.5.4.3 Formular 48

3.5.5 Chemical Oxygen Demand 48

3.5.5.1 Apparatus 49

3.5.5.2 Procedure 49

3.5.5.3 Formular 50

3.5.6 BOD and COD Results 50

CHAPTER FOUR: PONDS DESIGN AND RESULT ANALYSIS

4.1 Ponds Design 51

4.1.1 System Design Parameters 51

4.1.2 Anaerobic Pond Design 52

4.1.2.1 Pond Area 52

4.1.2.2 Retention Time 53

4.1.2.3 Pond Volume 53

4.1.2.4 Pond Dimensions 53

4.1.2.5 Effluent BOD Concentration from the Anaerobic Pond 54

4.1.3 Facultative Pond Design 54

4.1.3.1 Pond Area 55

4.1.3.2 Retention Time 55

4.1.3.3 Pond Volume 55

4.1.3.4 Pond Dimensions 56

4.1.3.5 BOD Removal in the Facultative Pond 56

4.1.4 Maturation Pond Design 57

4.1.4.1 Pond Area 57

4.1.4.2 Retention Time 57

4.1.4.3 Pond Volume 57

4.1.4.4 Pond Dimensions 57

4.1.4.5 Faecal Coliform Removal 58

4.2 Pond Facility Design 58

4.3 Analysis of Results 60

4.4 Pond Maintenance 61

4.4.1 Desludging 62

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1 Conclusion 63

5.2 Recommendation 64

REFERENCES 65

APPENDIX A 70

APPENDIX B 74

LIST OF FIGURES

Figure 2.1	Mutual Relationship between Pond Algae and Pond Bacteria	7
Figure 2.2	Typical Scheme of a Waste Stabilization System: An Anaerobic, Facultative and Maturation Pond in Series	19
Figure 3.1	Diagram of an Anaerobic Pond	29
Figure 3.2	Diagram of a Facultative Pond	29
Figure 3.3	Diagram of a Maturation Pond	30
Figure 3.4	A pH metre	42
Figure 3.5	Heating Mantle	43
Figure 3.6	Desiccator	43
Figure 3.7	Electronic Weighing Scale	45
Figure 4.1	Cross Section of Waste Stabilization Ponds	59

LIST OF TABLES

Table 2.1	Anaerobic and Facultative Bacteria Responsible for the Various Stages	10
Table 3.1	Design Values for Anaerobic Ponds	33
Table 3.2	BOD Removal Efficiencies in Anaerobic Ponds as a Function of the Temperature	34
Table 3.3	Values of the First Order Rate Constant for Faecal Coliform Removal at Various Design Temperatures	40
Table 3.4	Dissolved Oxygen, BOD and COD Values Obtained from Laboratory Tests	50

LIST OF SYMBOLS AND ABBREVIATIONS

WSP - Waste Stabilization Pond

BOD - Biochemical Oxygen Demand

COD - Chemical Oxygen Demand

NEMA- National Emergency Management Agency

NIMET- Nigeria Meteorological Agency

FAS - Ferrous Ammonium Sulphate solution

DO - Dissolved Oxygen

TSS - Total Suspended Solids

WHO - World Health Organization

CFD - Computational Fluid Dynamics

SLR - Surface Loading Rate

HRT - Hydraulic Retention Time

CO_2 - Carbon Dioxide

N_2 - Nitrogen gas

NH_4^+ - Ammonium ion

H_2S - Hydrogen Sulphide

NH_3 - Ammonia

HCO_3- Bicarbonate ion

HS^- - Bisulphide ion

OH^- - Hydroxide ion

CH_4 - Methane

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

The activities of man give rise to a wide range of waste products, many of which become water borne and must be carefully treated before release to the environment. Such wastewater may contain excreta, household wastes, industrial discharges, agricultural run-off and urban storm drainage. These wastes, collectively or individually, can pollute the environment. There is an increasing need for low-cost methods of treating wastewater. The operations of such methods and their maintenance must be within the capacity of the developing urban centres and industries. For both the maintenance of public health and the conservation of water resources, it is essential that water pollution be controlled.

Waste management can never be under emphasized for any growing community like Nnamdi Azikiwe University, Awka (Unizik).

Waste stabilization pond (WSP) is a large shallow basin enclosed by earth embankments in which raw wastewater is treated by entirely natural processes involving both algae and bacteria for the removal of pathogenic organisms, Bio-chemical Oxygen Demand (BOD) and other pollutants.

WSPs can be applied when a central sewage system is adopted. In construction of WSPs, certain factors are to be considered such as climate, soil nature, availability of land and ground water level (Agunwamba, 2000). It is advisable that such WSPs should be situated at 300m to 500m away from residential areas to avoid hazardous effects from such ponds and away from areas with high prospect of future expansion development.

When a central sewage system is adopted, the effluents from the waste stabilization pond can serve as irrigation waters for farmers around the community to increase their crop yield, overall output and the centre can serve as a research centre for allied departments. Also, effluents from WSPs have been reported to be rich in potassium and phosphorus.

High efficiencies of WSPs have been reported with respect to removal of intestinal nematode (Saqqar and Pescod, 1992), organic compounds and faecal bacteria. In addition, it is also economical, simple to construct, operate and maintain and it does not require any input of external energy.

In addition to being useful in the treatment of sewage, waste stabilization pond is being applied in the treatment of industrial and agricultural wastes. Its long detention time; its relatively slow rates of sludge accumulation and its physiochemical conditions such as neutrality to alkaline pH, make it attractive in treating industrial wastewaters.

This project is mainly restricted to the study and design of the anaerobic, facultative and maturation ponds which will be extensively discussed.

1.2 Problem Statement

Nnamdi Azikiwe University, Awka, which is the study area, is a federal university with a population of over 30,000 people which include staff and students. This project deals with the design of sewage ponds in Unizik, Awka for waste water treatment. The treatment facilities will be for waste water that comes from toilets, kitchens, bathrooms, some of which contain faecal matter.

Over the years, Unizik has experienced increase in population and it is worthy of note that among challenges it has faced, waste management has not been left out. This study seeks to address and recommend a central sewage collection system and waste stabilization to address challenges faced in managing soluble waste around the community. If adopted, it will reduce the cost due to frequent desludging of septic tanks, reduce pollution of underground waters through contaminant transport, save the environment from dirt due to leakages and spills from septic tanks. These spills cause foul odours most often and cholera outbreak can occur if not properly managed.

1.3 Aim and Objectives

The aim of this project is to design anaerobic, facultative and maturation pond for the treatment of waste water in Nnamdi Azikiwe University, Awka. The specific objectives of this project are:

i. To test wastewater obtained within the school environment (Boy's hostel) and obtain values for pH, dissolved solids, suspended solids, Bio-chemical Oxygen Demand (BOD) and Chemical Oxygen Demand (COD).
ii. To effectively design anaerobic, facultative and maturation ponds using values obtained in (i) above.
iii. To achieve an efficient system for the removal of pathogens, BOD and other pollutants from waste water generated within the community.

1.4 Significance of the Project

There are many advantages of this system of waste water treatment in Unizik. WSPs do not require any input of external energy, are not expensive to construct and maintain and do not require any special skills to operate as it is entirely based on natural processes. The major significances of this project are as follows:

i. The adoption of a central sewage collection system within the school community.
ii. Effluents from the ponds to be used for irrigation which will improve crop yield of farmers within the community.

1.5 Scope of the Study

The scope of this project is the study and design of the 3 principal types of Waste Stabilization Ponds (WSP) for the treatment of waste water in Nnamdi Azikiwe University, Awka which are;

i. Anaerobic pond
ii. Facultative pond
iii. Maturation pond

1.6 Limitations of the Study

The limitations of WSPs as discovered in the course of research are;

i. There was not enough time for collecting research data and research data is limited.
ii. As a result of the absence of a central wastewater management system, wastewater flow was impossible to accurately measure.
iii. Research data (wastewater sample) was only collected from Boy's Hostel.

CHAPTER TWO

LITERATURE REVIEW

2.1 Waste Stabilization Ponds

Wastewater is composed of over 99% water. In a developing urban society, the wastewater generation is usually approximately 30-70m^3 per people per year. In a city of one million people, the wastewater generated would be sufficient to irrigate approximately 1500-3500 hectares (SIDA, 2000). Innovative and appropriate technologies can contribute to urban wastewater treatment and reuse.

Water contaminated by human, chemical or industrial wastes can cause a number of diseases through ingestion or physical contact. Water related diseases include dengue, filariasis, malaria, onchocerciasis, trypanosomiasis and yellow fever. Consequently, no other type of intervention has greater impact upon a country's development and public health than the condition of clean drinking water and the appropriate disposal of human waste (SIDA, 2000).

One approach to sustainability is through decentralization of the waste water management system and this approach leads to treatment and reuse of water, nutrients and by-products of the technology (i.e. energy, sludge and mineralized nutrients) in the direct location of the settlement.

Waste or Wastewater Stabilization Ponds (WSPs) are large, man-made water bodies in which blackwater, greywater or faecal sludge are treated by natural occurring processes and the influence of solar light, wind, microorganisms and algae. The ponds can be used individually or linked in a series for improved treatment.

Waste Stabilization Ponds (WSP) have proven to be an effective alternative for treating wastewater and the construction of low energy-consuming ecosystems that use natural processes, in contrast to complex high-maintenance treatment systems, will hopefully lead to more ecologically-sustainable wastewater treatment in the future. WSPs also have the capability of meeting the demand for a high percentage removal of pathogenic organisms, compared to conventional technologies. WSPs, joined with other technologies, may be important for even more improved performance of water cleaning systems. WSPs are now well-established methods for wastewater treatment in tropical climates such as in Nigeria.

There are three principal types of WSPs: Anaerobic, facultative and maturation ponds. Anaerobic and facultative ponds are designed for BOD (Biochemical Oxygen Demand) removal while maturation ponds are designed for faecal bacteria removal. Some removal of faecal bacteria (especially Vibrio Cholerae) occurs in anaerobic and facultative ponds which are also responsible for most of the removal of Helminth eggs. Some removal of BOD occurs in maturation ponds which also remove some of the nutrients from the wastewater.

Facultative and maturation ponds are photosynthetic ponds i.e. the oxygen needed by the pond bacteria to oxidize the waste water BOD is mainly supplied by micro-algae that grow naturally and profusely in these ponds (and gives them their characteristic green colour) and the carbon dioxide needed by the algae is mainly provided by the pond bacteria as an end product of their metabolism. Thus, there is a mutual or symbiotic relationship between the pond algae and the pond bacteria. The algae are also extremely important in creating conditions within these ponds for faecal bacterial die-off.

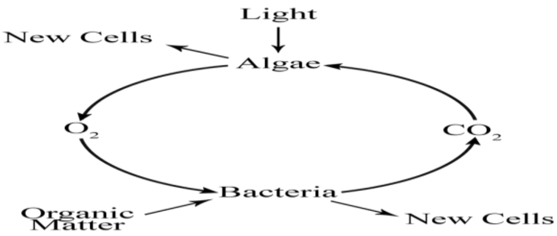


Figure 2.1: Mutual Relationship between Pond Algae and Pond Bacteria

2.1.1 Arrangement of Waste Stabilization Ponds

The different WSP types are arranged in a series- first, an aerobic pond, then a facultative pond and finally, the maturation ponds (where needed) to achieve required effluent quality. At any one site, there may be more than one series of WSP, each usually receiving an equal proportion of waste water flow.

Effluent from a series of ponds is of better quality than that from a single pond of the same size. This is because, even if the hydraulic flow regime in individual ponds is closer to complete mixing than it is to plug flow. The overall performance of a series of ponds approximates that of a plug flow reactor.

2.1.2 Anaerobic ponds

Anaerobic ponds are the first ponds used in a series of ponds. They are usually 2-5m deep and receive such a high organic loading usually >100g BOD/m^3 day. As a result of this high organic loading, they contain no dissolved oxygen and no algae although occasionally a thin film of Chlamydomonas may be present at the surface.

They function much like open septic tanks and their primary function is BOD removal. Anaerobic ponds work extremely well, a properly designed and not significantly under loaded anaerobic pond will achieve >60% BOD removal at 20^0C or >40% BOD removal at 10^0C. A shorter retention time of 1.0-1.5 days is usually used.

The conversion of organic matter under anaerobic conditions is slow, owing to the slow growth rate of anaerobic bacteria. This results from the fact that the anaerobic reactions generate less energy than the aerobic reactions for the stabilisation of organic matter. The temperature of the medium has a great influence in the biomass reproduction and substrate conversion rates, which makes warm-climate regions to be favourable for the utilisation of this type of pond.

In anaerobic ponds, BOD removal is achieved by sedimentation of solids and subsequent anaerobic digestion in the resulting sludge. The process of anaerobic digestion is more intense at temperatures above 15^0C. The anaerobic bacteria are usually sensitive to pH <6.2 thus, acidic waste water must be neutralized prior to its treatment in anaerobic ponds.

Anaerobic digestion proceeds in four stages;

a) **Hydrolysis:** the hydrolysis of complex waste water organics such as proteins, polysaccharides and fats.
b) **Acidogenesis:** the anaerobic oxidation of fatty acids and alcohols and the fermentation of amino acids and carbohydrates to volatile fatty acids (butyrates and propionates) and hydrogen gas.
c) **Acetogenesis:** the conversion of butyrates and propionates to acetases.
d) **Methanogenesis:** the conversion of acetates and hydrogen and CO_2 to methane.

In a simplified way, the anaerobic conversion takes place in two stages:

- liquefaction and formation of acids (through the acid-forming bacteria, or acidogenic bacteria)
- formation of methane (through the methane-forming organisms, or methanogenic archaea

In the first phase, there is no BOD removal, just the conversion of the organic matter to other forms (simpler molecules and then acids). It is in the second stage that BOD is removed, with the organic matter (acids produced in the first stage) being converted mainly to methane and carbon dioxide. The carbon is removed from the liquid medium by the fact that the methane (CH_4) escapes to the atmosphere.

The methane-forming organisms are very sensitive to the environmental conditions. If their reproduction rate is reduced, there will be the accumulation of the acids formed in the first stage, with the following consequences:

a. interruption of the BOD removal process and
b. generation of bad odours, because the acids are very fetid.

Therefore, it is essential that the appropriate balance between the two communities is guaranteed, ensuring the completion of both stages. For the adequate development of the methane-forming archaea, the following conditions should be met:

- absence of dissolved oxygen (methane-forming archaea are strict anaerobes and do not survive in the presence of dissolved oxygen)
- adequate temperature of the liquid (above 15°C)
- adequate pH (close to or above 7)

The anaerobic activity affects the nature of the solids, in such a way that, in the facultative pond, the solids are less prone to fermentation and flotation, besides decomposing more easily.

The anaerobic and facultative bacteria responsible for the various stages are as follows;

Table 2.1: The anaerobic and facultative bacteria responsible for the various stages

STAGE	MICROBES
Stage 1	Bacillus, Clostrudium, Proteus, Micrococus, Staphylococcus, Vibrio.
Stage 2	Butyrivibrio, Clostridium, Eubacterium.
Stage 3	Synthobacter, Synthrophomonas.
Stage 4	Methanothrix, Methanosarcina, Methanococcus (these are very slow growing, more sensitive to environment stress).

Anaerobic bacteria degrade organic materials in the absence of oxygen and produce methane and carbon dioxide. The methane can be reused as an alternative energy source (biogas). Other benefits include a reduction of total bio-solids volume of up to 50-80% and a final waste sludge that is biologically stable, can serve as rich humus for agriculture (Rose, 1999).

Much advantage is noticed in this treatment such as (Van Leir, 1998);

i. No or very low energy demand.
ii. Production of valuable energy in the form of methane.
iii. Low investment costs and low space requirement.
iv. Applicable at small as well as large scale.
v. Low production of excess sludge which is well stabilized.
vi. Low nitrogen and phosphorus requirements.
vii. High loading capacity (5-10 times that of aerobic treatment).
viii. High treatment efficiencies.
ix. Suitable for areas with long term periods without discharge of wastewater.
x. Effluents can contain valuable fertilizers (ammonium salts).

The existence of an anaerobic stage in an open reactor is always a matter of concern, owing to the possibility of the generation of bad odours. If the system is well balanced, the generation of bad smell should not be important, but occasional operational problems can lead to the release of hydrogen sulphide (H_2S), responsible for obnoxious odours. If the sulphate concentration in the influent is lower than 300 mg/L, the production of sulphide should not be problematic (in anaerobic conditions, sulphate is reduced to sulphide).

Additionally, if the pH in the pond is close to neutrality, most of the sulphide will be present in the form of the bisulphide ion (HS^-), which is odourless (Mara, 1997). Wastewaters with low pH values (industrial effluents or wastewater originated from a water that is soft, with low alkalinity, high acidity or without pH correction) may induce odour problems. As a result of the points above, the anaerobic-facultative ponds system should be located far away from houses (during all the operational life of the ponds).

2.1.2.1 Sludge Accumulation in Anaerobic Ponds

The aspects of sludge management in anaerobic ponds are different from facultative ponds. In the latter, the system can operate for several years, eventually during all of the design period, without needing to remove sludge (provided there is a good grit removal in the preliminary treatment). However, because of the smaller volume of the anaerobic ponds, the sludge accumulation manifests itself more rapidly, bringing about the need of an appropriate planning related to the sludge management.

The anaerobic ponds should be cleaned according to one of the following strategies:

- when the sludge layer reaches approximately 1/3 of the liquid depth.
- annual removal of a certain volume, in a pre-determined month, to include the cleaning stage in a systematic way in the operational strategy of the pond.

If the removal is not by emptying and drying inside the pond, the whole sludge mass should not be removed, since this would lead to a total loss of the biomass, requiring the anaerobic pond to start up again.

2.1.3 Facultative Ponds

Facultative ponds are the simplest of all WSPs. They consist of large shallow ponds (depth of 1 to 2m) with an aerobic zone to the surface and a deeper anaerobic zone.

The advantages are associated with the high operational simplicity and reliability. Natural processes are likely to be reliable: there is no equipment that can be out of order or the need for special operational schemes. However, nature is slow and needs long detention times so that the reactions are completed, which implies large land requirements. The biological activity is largely affected by temperature, mainly under the natural conditions of the ponds. As a result, the stabilization ponds are more appropriate where the land is cheap, the climate is favourable, and a treatment method that does not require equipment or a special training for the operators is desired (Arceivala, 1981).

Facultative ponds are of two types: the primary facultative ponds which receive raw wastewater (after preliminary treatment) and the secondary facultative ponds which receive settled wastewater, usually the effluent from anaerobic ponds.

They are designed for BOD removal on the basis of a relatively low surface BOD loading in the range of 100-400kg/ha/day. Facultative ponds are coloured dark green as a result of large number of micro-algae in them. They may also appear red or pink when they are overloaded due to the presence of anaerobic purple sulphide-oxidizing photosynthetic bacteria. The algae that tend to predominate in the turbid water of facultative ponds belong to motile Geneva (such as Chlamydomonas, Pyrobotrys and Euglena).

The motile algae can optimize their vertical position in the pond waster column in relation to incident light intensity more easily than non-motile forms (such as Chlorella which is also common in facultative ponds). The concentration of algae in a facultative pond depends on loading and temperature but is usually in the range of 500-2000µg Chlorophyll per litre.

As a result of the photosynthetic activities of the pond algae, there is a diurnal variation in the concentration of dissolved oxygen. After sunrise, the dissolved oxygen level gradually rises in response to photosynthetic activity to a maximum in mid-afternoon. After this, it falls to a minimum during the night when photosynthesis ceases and algal as well as bacterial respiratory activity consumes oxygen. The position of oxypause (i.e. the depth at which the dissolved oxygen concentration reaches zero) and the pH are subject to change. At peak algal activity, carbonate and bicarbonate ions react to provide more carbon dioxide for the algae, so leaving an excess of hydroxyl ions resulting in an increase in pH greater than 9.4. This condition rapidly kills most faecal bacteria.

Above the oxypause, aerobic conditions prevail, while below it, anoxic or anaerobic conditions predominate. The level of the oxypause varies during the 24 hours of the day, as a function of the variability of the photosynthesis during this period. At night, the oxypause level rises in the pond, while during the day it lowers down. The thickness of the aerobic zone, besides varying along the day, also varies with the loading conditions of the pond. Ponds with a greater BOD load tend to have a larger anaerobic layer, which can practically take up the whole pond depth during the night. The pH in the pond also varies with the depth and along the day. The pH depends on the photosynthesis and respiration, according to:

- Photosynthesis:
  - Consumption of CO_2.
  - Bicarbonate ion (HCO_3^-) of the wastewater is converted to OH^-.
  - pH rises.

- Respiration:
  - Production of CO_2.
  - Bicarbonate ion (HCO_3^-) of the wastewater is converted to H^+.
  - pH decreases.

During the day, in the hours of maximum photosynthetic activity, the pH can reach values around 9 or even more. In these conditions of high pH, the following phenomena can occur:

- Conversion of the ammonium ion (NH_4^+) to free ammonia (NH_3), which is toxic, but tends to be released to the atmosphere (nutrient removal).
- Precipitation of the phosphates (nutrient removal).
- Conversion of sulphide (H_2S), which may cause bad odours, to the odourless bisulphide ion (HS^-). At pH levels greater than 9 there is practically no H_2S.

2.1.3.1 Influence of Algae

Algae play a fundamental role in facultative ponds. Their concentration is much higher than that of bacteria, giving the greenish appearance of the liquid at the pond surface. In terms of dry suspended solids, their concentration is usually lower than 200 mg/L, although in terms of numbers they can reach counts in the range of 104 to 106 organisms per ml (Arceivala, 1981). The presence of algae is usually measured in the form of chlorophyll a, a pigment presented by all plants, and the main parameter for the quantification of the algal biomass (Konig, 2000). The chlorophyll a concentrations in facultative ponds depend on the applied load and temperature, but are usually located in the range from 500 to 2000 µg/L (Mara et al, 1992).

The main types of algae found in stabilisation ponds are (Mara et al, 1992; Silva Jr. and Sasson, 1993; Jordao and Pessoa, 1995):

- **Green algae** (Chlorophyta) and **pigmented flagellated** (Euglenophyta): These algae give the pond the predominant greenish colour. The main genera are Chlamydomonas, Chlorella and Euglena. Chlamydomonas and Euglena are usually the first to appear in the pond, tending to be dominant in cold periods, and possessing flagella, which gives them motility (optimisation of their position with relation to the incidence of light and to temperature).
- **Cyanobacteria** (previously called Cyanophyta or blue-green algae): In reality these organisms present characteristics of bacteria and algae, and are classified as bacteria. The cyanobacteria do not have locomotion organelles, such as cilia, flagella or pseudopodes, but are capable of moving by sliding. The nutrient requirements are very small: the cyanobacteria can proliferate in any environment that has at least CO2, N2, water, some minerals and light. These organisms are typical of conditions with low pH values and low nutrient availability in the wastewater. This environment (not typical in stabilization ponds) is unfavourable for the green algae, which may also serve as food for other organisms, such as protozoa, leading to the proliferation of the cyanobacteria. Oscillatoria, Phormidium, Anacystis and Anabaena are among the main genera that can be mentioned.

Other types that can be found are algae of the phyla Bacyllariophyta and Chrysophyta (Konig, 2000; Mara et al, 1992). The predominant species vary from place to place, and even with the position in the series of ponds (facultative ponds and maturation ponds).

The processes in anaerobic and secondary facultative ponds occur simultaneously in primary facultative ponds. It is estimated that about 30% of the influent BOD leaves the primary facultative pond in the form of methane (UNEP). A high proportion of the BOD that does not leave the pond as methane ends up in algae. This process requires more time, more land area, and possibly 2 -3 weeks water retention time, rather than 2 -3 days in the anaerobic pond. In the secondary facultative pond (and the upper layers of primary facultative ponds), sewage BOD is converted into ‘Algal BOD’ and has implications for effluent quality requirements. About 70 ± 90% of the BOD of the final effluent from a series of well-designed WSPs is related to the algae they contain.

The effluent from a facultative pond has the following main characteristics (CETESB, 1989):

- green colour due to the algae
- high dissolved oxygen concentration
- high suspended solids concentration, although these practically do not settle (the algae practically do not settle in the Imhoff-cone test).

2.1.4 Maturation Ponds

Maturation ponds are primarily designed for tertiary treatment i.e. the removal of pathogens, nutrients and possibly algae whereas anaerobic and facultative ponds are designed for BOD removal. Maturation ponds reduce the number of excreted pathogens typically faecal bacteria and viruses present in the effluent of facultative ponds to a level suitable for agricultural and/or aqua cultural re-use. Maturation ponds are typically aerobic throughout their depth. They retain suspended stabilized solids and the size and number of maturation ponds depend on the required bacteriological quality of the final effluent.

The principal mechanism for faecal bacterial removal in facultative and maturation ponds are retention time, temperature, high pH or radiation of the sun leading to solar disinfection. Maturation ponds are shallower (1 to 1.5m) with 1m being optimal the recommended hydraulic retention time can be 15 to 20 days if used in combination with algae and for fish harvesting, this type of pond is also effective at removing the majority of nitrogen and phosphorous from the effluent.

Maturation ponds lead to a polishing of the effluent from any of the stabilization pond systems previously described or, in broader terms, from any wastewater treatment system. The main objective of maturation ponds is the removal of pathogens, and not an additional BOD removal. Maturation ponds constitute an economic alternative to the disinfection of the effluent by more conventional methods, such as chlorination.

2.1.4.1 Process Description

The ideal environment for pathogenic organisms is the human intestinal tract. Outside it, in the sewerage system, sewage treatment plant or in the receiving water body, the pathogenic organisms tend to die.

Several factors contribute to the removal of the pathogenic organisms:

- bacteria and viruses: temperature, solar radiation, pH, food shortage, predator organisms, toxic compounds.
- protozoan cysts and helminth eggs: sedimentation.

Maturation ponds are designed in order to provide an optimal utilisation of these mechanisms, especially for the removal of bacteria and viruses, which can be represented by the coliforms as indicators. Some of these mechanisms are more effective with smaller pond depths, which justifies the fact that the maturation ponds are shallower, compared with other types of ponds.

Among the mechanisms associated to the low depth of the pond, the following can be mentioned (Van Haandel et Lettinga, 1994; Van Buuren et al, 1995; Cavalcanti et al, 2001):

- High penetration of the solar radiation (ultraviolet radiation).
- High pH (due to high photosynthetic activity).

- High DO concentration (favouring the aerobic community, which is more efficient in the removal of coliforms, besides increasing the removal rate due to other mechanisms, such as photooxidation).

The maturation ponds should reach high coliform removal efficiencies (E > **99.9** or **99.99%**), so that the effluent can comply with most uses of the water in the receiving water body, or for direct uses, such as irrigation. In order to maximize the coliform removal efficiency, the maturation ponds are designed with one of the following two configurations:

a. three or four ponds in series or
b. a single pond with baffles.

Regarding the other organisms of public health importance, which are not well represented by coliforms as indicators, the ponds usually reach complete (100%) removal of *protozoan cysts and helminth eggs* (Arceivala, 1981). The major removal mechanism is sedimentation.

2.1.4.2 Quality Requirements for the Effluent

Normally there are no discharge standards for coliforms. The water quality standards are usually with respect to the receiving body, as a function of its intended uses.

If the effluent is to be used for unrestricted irrigation (for cultures that can present contamination risks), the recommended values according to the World Health Organisation (WHO, 1989) are:

- faecal coliforms: ≤1,000 faecal coliforms/100 mL (geometric mean)
- helminth eggs: ≤1 egg/L (arithmetic average)

For restricted irrigation, there is a limit for only helminth eggs (≤ 1 egg/L), and no limits for coliforms.

In any case, in terms of the receiving body or for agricultural reuse, the coliform counts in the effluent should be very low. Considering that the faecal (thermotolerant) coliform concentrations are in the order from 10^6 to 10^9 org/100mL in the raw sewage, the removal efficiencies in the treatment should be extremely high. To comply with the above criteria, coliform removal efficiencies of the order of 3 to 6 log units (99.9 to 99.9999%) are necessary in the wastewater treatment plant.

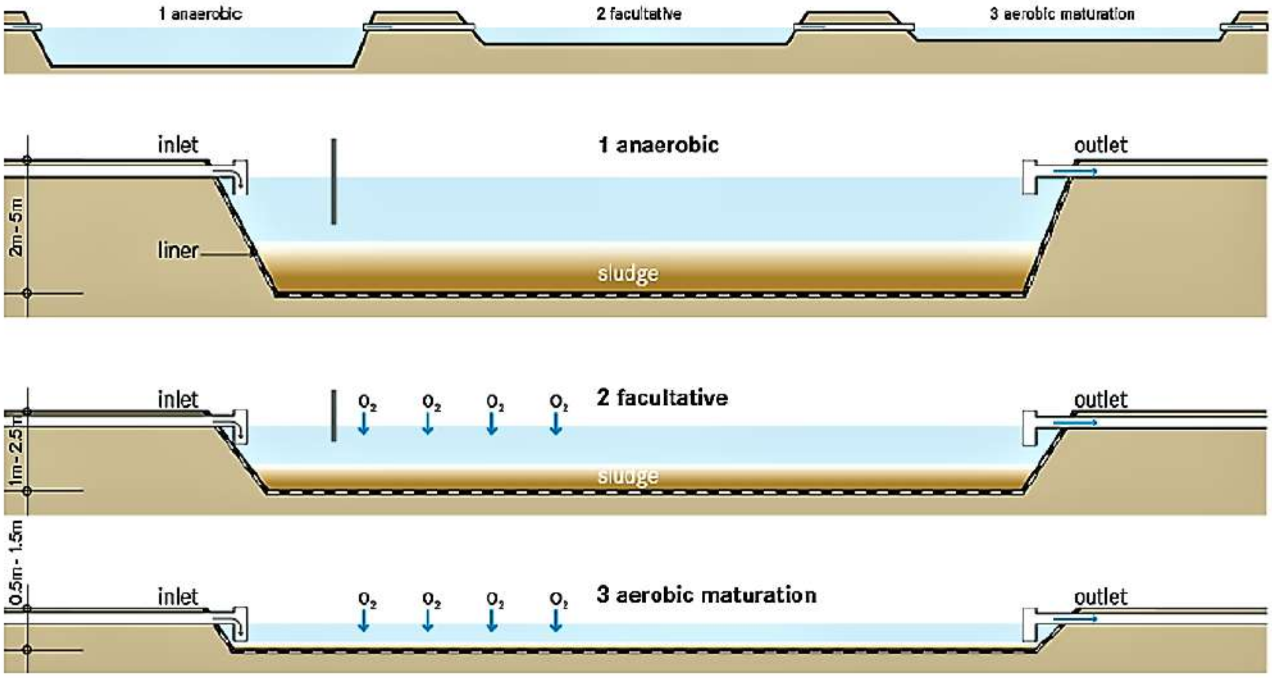


Figure 2.2: Typical scheme of a waste stabilization system: An anaerobic, facultative and maturation pond in series. Source: TILLEY et al. (2014)

2.2 Waste Stabilization Pond Terminologies

2.2.1 Biochemical Oxygen Demand Removal Kinetics

The rate at which organic matter is oxidized by bacteria is a fundamental parameter in the rational design of waste water treatment processes. It has been found that BOD removal often approximates first-order kinetics i.e. the rate of BOD removal (equals the rate of oxidation of organic matter) at any time is proportional to the amount of BOD present in the system at that time.

Mathematically,

$$\frac{dL}{dt} = -K, L \tag{2.1}$$

Where L = amount of BOD remaining (i.e., organic matter still to be oxidized) at time t

K_1 = the first order rate constant for BOD removal which has units reciprocal to time i.e. t^{-1}

The differential coefficient $\frac{dL}{dt}$ = the rate at which the organic matter is oxidized and the minus sign indicates a decrease in the value of L with time

2.2.2 Hydraulic Flow Regime

The flow of waste water through a microbiological reactor can approximate either complete mixing or plug flow. These two flow patterns represent two extreme or ideal conditions. In practice, the hydraulic regime lies between these two extremes and is described as dispersed flow.

2.2.3 Complete Mixing

The influent to this ideal reactor is completely and instantly nearly mixed with the reactor contents which are as a result of intense mixing, uniform in composition throughout. The effluent is identical therefore, in every respect to the reactor contents. The removal of BOD is described by;

$$\frac{L_e}{L_l} = \frac{1}{1+K_1\theta} \tag{2.2}$$

2.2.4 Plug Flow

The contents of this ideal reactor flow through the reactor in an orderly fashion characterized by the complete absence of longitudinal mixing. The concept of plug flow may be understood by imagining the waste water on arrival at the reactor to be placed in water tight packets which then travel along the length of the reactor- as if on a conveyor belt- with no transfer of material from one packet to another but with complete mixing within each packet. Since each packet receives no additional BOD and loses none to a neighbouring packet, the removal of BOD with each packet is essentially a batch process, so that BOD removal in a plug flow reactor is;

$$\boldsymbol{L_e = L_i e^{-k_1 \theta}} \tag{2.3}$$

2.2.5 Dispersed Flow

In practice, it is impossible to build a plug flow reactor in which there is no mixing between packets. Some degree of longitudinal mixing always occurs. The degree of inter-packet mixing that takes place is usually expressed in terms of a dimensionless 'dispersion number' δ defined as

$$\boldsymbol{\delta = \frac{D}{VL}} \tag{2.4}$$

Where D= coefficient of longitudinal dispersion (m^2/h)

V= mean velocity of travel of a typical particle in the reactor (m/h)

L= mean path length (m)

2.3 Review of Related Literatures

There are several guidelines for WSPs designs but most are outdated (Arthur and Mundial, 1983, Mara and Pearson, 1998, Mara, 2003, Kayombo, 2005, EPA, 2011). These guidelines focused on data-based approaches, i.e., rules of thumb or regression equations, to dimension pond treatment systems. Since WSPs have been applied on a global scale, the results of these approaches were relatively heterogeneous. Indeed, Shilton and Harrison (2003b) emphasized that the foundation of improving the treatment efficiency of WSPs is based on the insight of its fundamental mechanisms, not gross rules of thumb.

These authors presented a guideline for the hydraulic design of WSPs with a demonstration via the use of computational fluid dynamics (CFD) to test the engineering adjustments of pond configuration. In fact, these hydraulic models together with biokinetic models have been a main research focus for the last two decades, but frequently missing in previous guidelines.

To form an updated critical review of model-based designs of typical WSP systems including a series of anaerobic (AP), facultative (FP), and maturation ponds (MP), a lot of research papers were reviewed. More specifically, four different types from simple to more dedicated models were analyzed with their merits and disadvantages. Furthermore, the applicability of all design models on the future designs was tested via a case study in which we investigated the variability and uncertainty of their outputs in designing required area surface of a facultative pond. Besides the lack of a recent comparison between different modelling approaches, the number of studies on the incorporation of uncertainty analysis into WSP designs is also small.

Previously, to account for uncertainties in deterministic design procedures, a gross number is applied in the form of safety factors. However, due to stricter environmental regulations and an increasing demand for the effective treatment efficiency of WSPs, a new approach with more in-depth understanding of uncertainty sources is required.

2.3.1 Model Based Approaches of Waste Stabilization Pond Designs

2.3.1.1 Rules of Thumb

Rules of thumb is a so-called black box approach, in which pond dimensions are directly determined by criteria such as population, loading rate, retention time, etc. Although what happens inside the pond systems is mostly unknown, this simplest approach has been applied in thousands of applications throughout the world.

Loading rate (LR) and hydraulic retention time (HRT) are the most commonly used criteria in all three pond types of typical WSPs. In case of APs, the former criterion is evaluated as an irrelevant design approach because of its high degree of uncertainty whereas designs based on HRT is relatively common.

On the other hand, the surface LR (SLR) approach has been preferred by the majority of engineers for designing FPs since the performance of FPs is considerably influenced by algal photosynthetic activities occurring near the water surface (WHO, 1987). Additionally, temperature appears as a key factor influencing the behavior of the ponds as we observed a significant drop in BOD removal efficiency during the winter period despite the increase in pond HRT (Gloyna, 1971, Arceivala, 1973, EPA, 1983, WHO, 1987).

2.3.1.2 Regression Equations

Regression equations are based on an inductive methodology, in which observed data are applied to build empirical relationships between a dependent variable and covariates. For a complex system as WSPs, empirical models appear as a handy tool for design and operation purposes because of their advantages, such as few basic input data required and simple calculations for quickly obtaining reliable results at local scales. However, like rules-of-thumb, no standardized equations were developed, resulting in a wide variety of applicable equations in each pond type.

2.3.1.3 First-Order Models

First-order models are a process-based approach that attempts to simulate and quantify the combined effect of removal processes in pond systems based on their kinetics in standard reactors. Particularly the reaction rate of chemical and biological processes is assumed to follow first-order kinetics which is incorporated with boundary conditions representing the hydraulic regime of pond treatment systems in mass balance equation (Tchobanoglous and Schroeder, 1985).

2.3.1.4 Mechanistic Models

In the first-order models, the equation of mass balance consists of two parts: (1) an overall reaction rate equation coupled with (2) boundary conditions of standard reactor flows, resulting in a model with a narrow spatial resolution and simple concept of removal processes.

In mechanistic models, the complexity of these two parts is advanced to a higher level, which depends on the degree of conceptualizing and measuring biochemical/physical mechanisms occurring in the pond system. More specifically, Computational Fluid Dynamics (CFD) offering higher spatial resolutions and different reaction rates for different removal processes are applied.

CFD models have been increasingly applied on WSP systems over the last two decades to precisely simulate their fluid flow. Their degree of complexity has been growing, which started from the first 2D commercial model of Wood et al. (1995) and Wood et al. (1998), then advanced to the first 3D models of Shilton (2000) and Salter et al. (2000).

Subsequently, the impacts of external factors, i.e., wind, thermal stratification, or baffling, on the flow regime were included in later 3D models of Sweeney et al. (2003), Sweeney et al. (2005), Karteris et al. (2005), and Aldana et al. (2005). In other studies, i.e., Persson (2000), Shilton and Harrison (2003b), Li et al. (2013), and Passos et al.

(2014), the influences of different configurations of inlet/outlet (I/O) and dead zones of pond systems were also evaluated.

2.4 Summary of Reviewed Literatures

Having discussed recent developments and progress in technologies surrounding design and implementation of waste stabilization ponds, the following should be noted.

Rules of thumb is considered as an inappropriate tool for pond designs due to its low design specification and very high output variation. Some well-known regression equations are recommended for general pond designs, i.e. Mara equation (1975) or Gloyna method (1976).

Note that, in case of low pressure over land and moderate water quality required or at the beginning phase of design process, these equations are very useful to have an idea for pond dimensions. However, care should be given when extrapolating them on a global scale as hydraulic flow, microbiological and meteorological inputs should be taken into account.

Compared to mechanistic models, first-order models seem to be a good consensus between required efforts and confidence level in pond designs. Nonetheless, since a model can be applied in many stages of the life of a WSP, such as detailed design, process optimization, performance analysis, and pond upgrade, mechanistic models can prove their superiority.

The application of CFD models has become more and more essential on pond designs as many of their recommendations proved to be very effective for retrofitting/upgrading the pond systems, such as baffling, adjusting I/O configurations, and desludging procedures. These modifications certainly affect the removal efficiency of pond systems, which can be simulated in a comprehensive and integrated model expected to be developed quantitatively and qualitatively in the near future.

Minimizing the possibility of generating a wrong decision without exaggerating the safety factor in designs is another difficult question to solve for pond designers. By implementing an iterative procedure of deterministic models, probability designs quantify uncertainty in a more precise manner, which can significantly reduce investment and operational costs. However, over years of pond development, only few designs included this approach.

This lack of experience intimidates pond designers to employ probabilistic approaches in their models. For better cost-effective designs, a better understanding is required, hence, more studies are expected in this area in the future.

2.5 Literature Gaps

There are still many gaps in the understanding of the mechanisms of biochemical processes and the effects of hydraulic efficiency on them in WSPs. It is concluded in Verbyla and Mihelcic (2015) that there is no model that adequately and accurately describes virus removal processes in WSPs while low nutrient removal remains an issue in many systems due to the insufficient data for developing a comprehensive model (Frank and Joanne, 2014).

Very recently, Coggins et al. (2017) highlighted the need of further research on the interaction between sludge accumulation and other factors, such as inlet and outlet configurations, flow rates, wind, and pond geometry.

Other emerging issues which have only recently been concerned in WSP systems are greenhouse gas emissions and the removal efficiency of antibiotics.

Recently, Hernandez-Paniagua et al. (2014) and Glaz et al. (2016) concluded that WSPs release more greenhouse gases than conventional activated sludge system and other aquatic ecosystems and Moller et al. (2016) is the only one mechanistic model so far concerning the removal mechanisms of antibiotics in pond treatment systems.

In order to tackle these issues, the currently small number of complex integrated models is expected to increase in the very near future with the present availability of strong computational power.

There are also knowledge gaps in areas such as more advanced understandings on nitrogen and virus removals or the effects of different hydraulic regimes and sludge accumulations on biochemical processes.

Furthermore, there is a great need of a fully calibrated and validated model of WSP systems which requires a close attention on measuring campaigns to collect necessary and highly informative data.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Research Methodology

This section covers the detailed explanation of the methodology adopted in this project. The methodology used follows a step by step and systematic approach. It encompasses design methods with supporting equations adopted for Anaerobic, Facultative and Maturation ponds in order to achieve the intended system. Values obtained from laboratory tests on parameters such as pH, dissolved solids, suspended solids, BOD and COD are also incorporated in the design of the ponds.

3.2 Environmental Description

Nnamdi Azikiwe University, Awka, with coordinates of 6.242889^0N and 7.118289^0E has a vast land mass of about 250 acres, located in the suburban setting of the small city of Awka along Onitsha - Enugu expressway. This locale is largely populated with an enrollment range of 30,000 – 34,999 students and evenly distributed infrastructures across the vast land mass. The community is adorned with a lot of infrastructure which all have individual waste management systems and this typically is not ideal, hence the need for a central wastewater management system which are the waste stabilization ponds in this case.

This will aid in low-cost, low maintenance wastewater management, recycling and treatment for such uses as irrigation and research.

3.3 Pond Diagrams

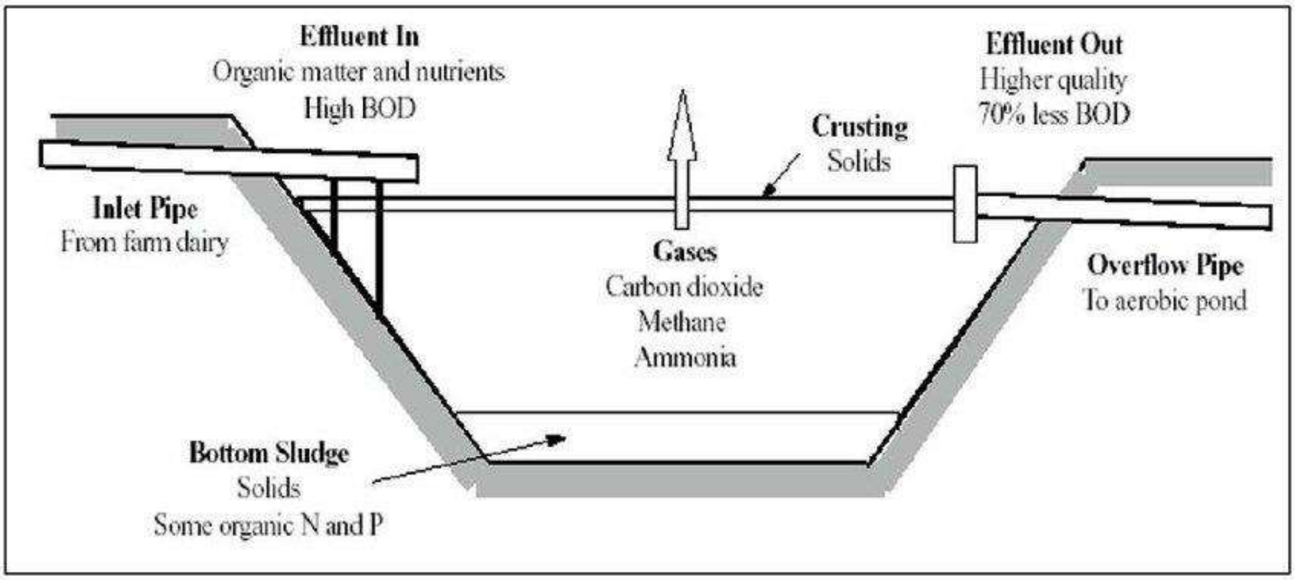


Figure 3.1 Diagram of an Anaerobic Pond

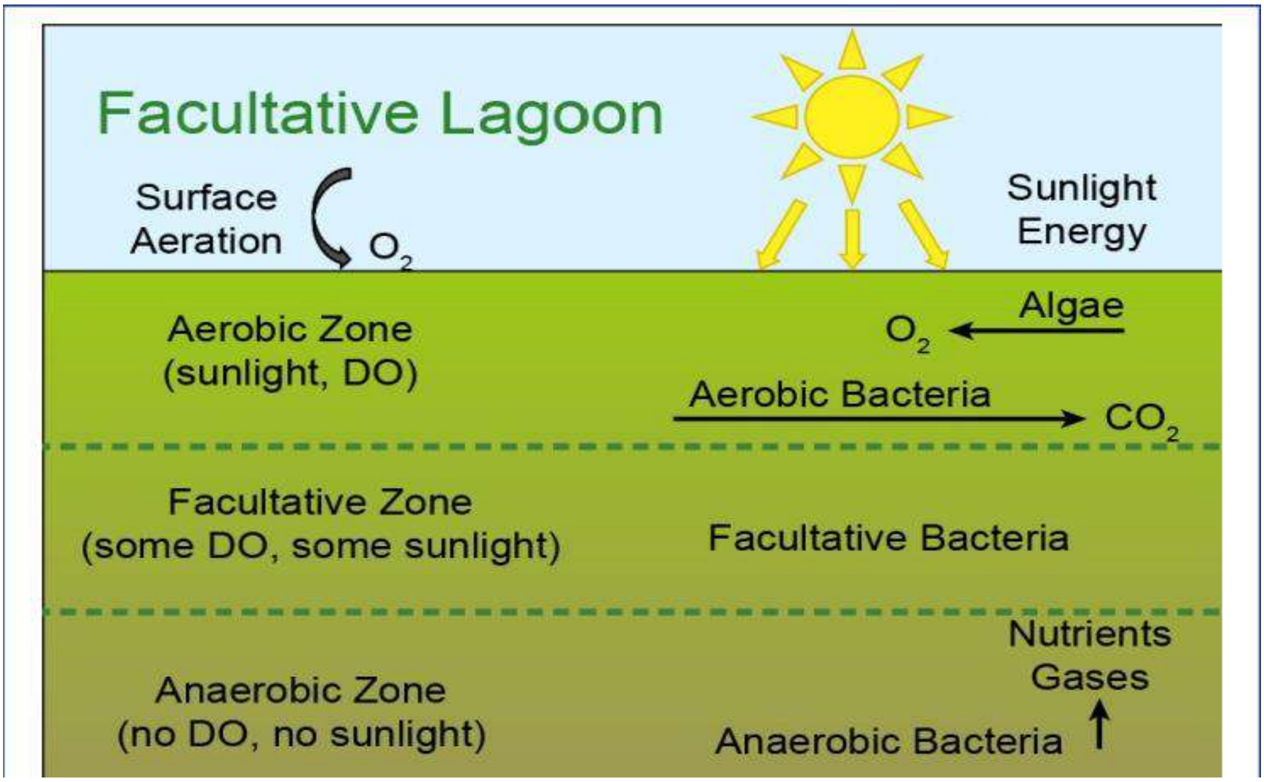


Figure 3.2 Diagram of a Facultative Pond

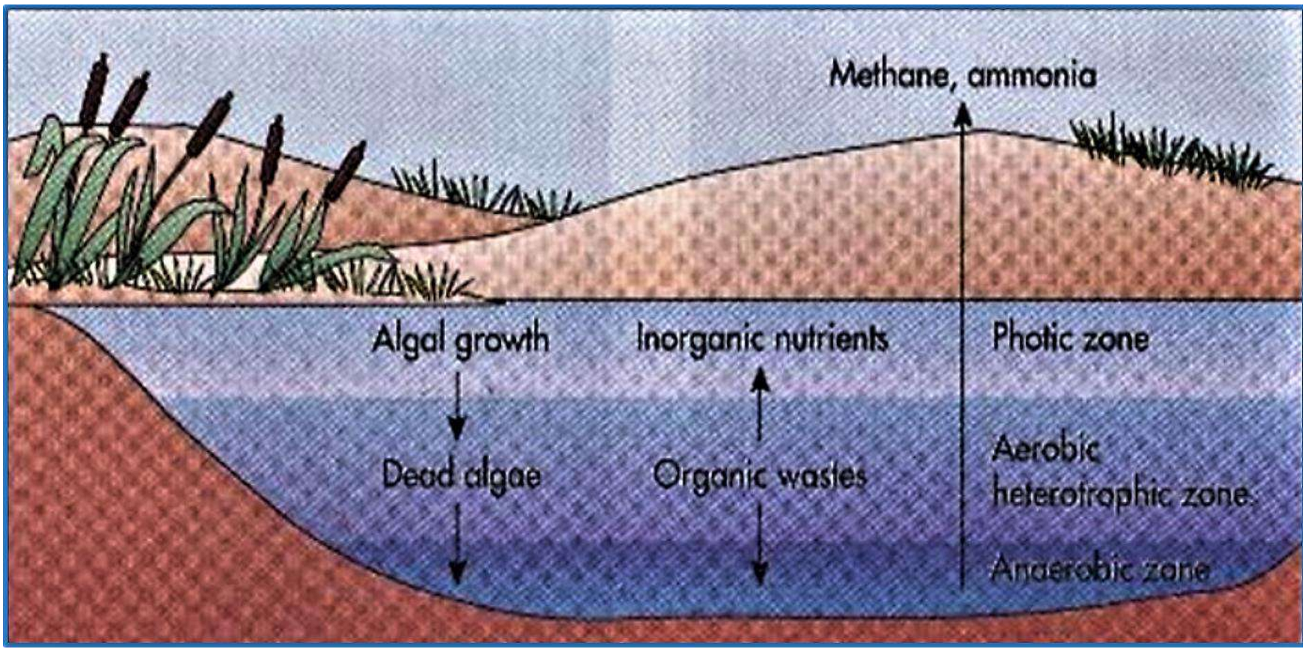


Figure 3.3 Diagram of a Maturation Pond

3.4 Supporting Equations for Waste Stabilization Ponds Design

This section looks at the different mathematical equations under the three principal types of waste stabilization ponds; anaerobic ponds, facultative ponds and maturation ponds. These equations ensure the ponds are designed to meet standard requirements and to operate with the required efficiency. This ensures that the objectives of this project are achieved.

3.4.1 Anaerobic Pond Design

Anaerobic ponds can be satisfactorily designed without the risk of odour release on the basis of volumetric BOD loading.

The main design parameters for anaerobic ponds are:

- Volumetric organic loading rate
- Retention time
- Depth

- Geometry (length / breadth ratio)

The criterion of the volumetric organic loading rate is the most important, and is established as a function of the need of a certain pond volume for the conversion of the applied BOD load. The criterion of the detention time is based on the time necessary for the reproduction of the anaerobic bacteria.

3.4.1.1 Volumetric Organic Loading Rate

The volumetric loading rate B_v, the main design parameter for anaerobic ponds, is a function of the temperature. Warmer locations allow a larger loading rate (smaller pond volume). The consideration of the volumetric load is important, because industrial wastewaters can vary widely in the relationship between flow and BOD concentration (load = flow × concentration). Therefore, only the detention time criterion is insufficient.

The volume required is given by:

$$\mathbf{B_v = L_i Q / V_1} \tag{3.1}$$

Where L_i = influent BOD (mg/L)

Q = flow rate (m^3 /day)

V_1 = anaerobic pond volume (m^3)

3.4.1.2 Anaerobic Pond Area

Anaerobic pond area is given by

$$\mathbf{A_a = Qt/d_1 = L_i Q / B_v d_1} \tag{3.2}$$

Where:

t = retention time, d_1 = assumed pond depth

3.4.1.3 Retention Time

For domestic sewage, the hydraulic retention time is usually within the following range:

$$t = 1.0 \text{ d to } 6.0 \text{ d}$$

In conventional anaerobic ponds (in which the inlet pipe is above the sludge layer), if the retention time is lower than 3.0 days, the methane-forming organisms may be washed out of the reactor. In these conditions, the maintenance of a stable bacterial population would not be possible. Apart from the efficiency of the anaerobic pond being reduced, the more serious aspect of imbalance between the acid-forming and methane-forming stages would occur. The consequences would be the accumulation of acids in the liquid, with the generation of bad odours, as a result of the small population of methane-forming organisms to continue the conversion of acids.

With retention times greater than 6 days, the anaerobic pond can behave occasionally as a facultative pond. This is undesirable, because the presence of oxygen is fatal for the methane-forming organisms. Anaerobic ponds must work as strict anaerobic ponds and cannot alternate between anaerobic, facultative and aerobic conditions.

After calculating the volume based on the volumetric loading rate (B_v), the resulting retention time is obtained by:

$$\mathbf{t = V/Q} \tag{3.3}$$

where:

t = retention time (d)

V = volume of the pond (m^3)

Q = average influent flow (m^3/d)

3.4.1.4 Depth

The depth of anaerobic ponds is high, in order to guarantee the predominance of anaerobic conditions, avoiding the pond to work as a facultative pond. In fact, the deeper the pond, the better. However, deep excavations tend to be more expensive.

Values usually adopted are in the range of:

$$H = 2.0 \text{ m to } 5.0 \text{ m}$$

The mid- depth area

$$\mathbf{A_1 = V_1 d_1} \qquad (3.4)$$

Where d_1 = assumed pond depth

3.4.1.5 Geometry (length / breadth ratio)

Anaerobic ponds are square or slightly rectangular, with typical length/breadth (L/B) ratios of:

Length / breadth ratio (L/B) = 2 to 1

3.4.1.6 Design Values for Anaerobic Ponds (Lugali 2012)

Table 3.1 Design Values for Anaerobic Ponds

Temperature (^{0}C)	Volumetric (g/m^3.d)	BOD removal (%)
<10	100	40
10-20	20T-100	2T+20
20-25	10T+100	2T+20
>25	350	70

3.4.1.7 Estimation of the Effluent BOD Concentration from the Anaerobic Pond

There are still no conceptual mathematical models in widespread use that allow an estimation of the effluent BOD concentration from anaerobic ponds. For this reason, these ponds have been designed mainly according to empirical criteria. Mara (1997) proposed the BOD removal efficiencies as a function of the temperature presented in the table below.

Table 3.2 BOD Removal Efficiencies in Anaerobic Ponds as a Function of the Temperature

Mean Air Temperature of the Coldest Month -T (^0C)	BOD Removal Efficiency E (%)
10 – 25	2T + 20
>25	70

Once the removal efficiency E has been estimated, the effluent concentration (BOD_{effl}) of the anaerobic pond is calculated using the formulas:

$$\mathbf{E = (S_o - BOD_{effl}).100/S_o} \qquad (3.5)$$

or

$$\mathbf{BOD_{effl} = (1 - E/100).S_o} \qquad (3.6)$$

where:

S_o = influent total BOD concentration (mg/l)

BOD_{effl} = effluent total BOD concentration (mg/l)

In this empirical approach, the effluent BOD considered is the total BOD, different from the calculations of facultative ponds, in which the effluent BOD is split in terms of soluble BOD and particulate BOD.

3.4.2 Facultative Pond Design

Facultative ponds are best designed on the basis of surface BOD loading which is given by;

$$\mathbf{B_s = 10L_iQ/A_2} \tag{3.7}$$

Where:

A_2 = facultative pond area (m^2)

The factor 10 arises from the units used

L_iQ = mass of BOD entering the pond (g/day)

So, $10^{-3}L_iQ$ is in Kg/day and the area in hectares is $10^{-4}A_2$ which is in m^2.

Surface loading is used for facultative ponds rather than volumetric loading (anaerobic ponds) because the light needed for algal photosynthesis arrives from the sun at the pond's surface. Thus, algal oxygen production is a function of area so the BOD loading (which is an oxygen demand) must also be a function of area.

The permissible design value of B_s increases with temperature which has a direct relationship with the climate. The earliest relationship between B_s and T is that given by Mchany and Pescod (1970). Their value of B_s is the maximum that can be applied to a facultative pond before it fails (i.e., before it becomes anaerobic). The relationship which is a failure envelope is given as;

$$\mathbf{B_s = 60(1.099)^T} \tag{3.8}$$

The Mchany and Pescod equation does not incorporate a factor of safety and as such should not be used for design.

A more universal equation given by Mara (1976) is;

$$\mathbf{B_s = 20T - 120} \tag{3.9}$$

where:

B_s = design BOD_5 loading (kg/ha/day)

T = design temperature (°C)

The Mara equation is based on a loading of 80kg/ha/d at ≤ 8^0C in European winters, a loading of 350kg/ha/d at 25^0C in northeast Brazil and an arbitrary loading of 500kg/ha/d at 35^0C.

On a suitable surface loading, B_s has been selected, the pond area is calculated from equation (3.7) above and the retention time;

$$\mathbf{T_2 = A_2D/Q_m} \qquad (3.10)$$

Where:

D = pond depth (usually 1.5m)

Q_m = mean flow (m^3/d)

The mean flow is the mean of the influent and effluent flows, Q_i and Q_e. The latter being the former, less net evaporation and seepage.

Therefore, the equation becomes;

$$\mathbf{T_2 = A_2D/[0.5(Q_i + Q_e)]} \qquad (3.11)$$

If seepage is negligible, Q_e is given by;

$$\mathbf{Q_e = Q_i - 0.001eA_2} \qquad (3.12)$$

Where:

e = net evaporation rate (mm/day) – from meteorological stations.

Therefore, retention time can be calculated as;

$$\mathbf{T_2 = 2A_2D/(2Q_i - 0.001eA_2)} \quad (3.13)$$

A minimum value of T_2 of 5 days should be adopted for temperatures $<20^0$C and 4 days for temperatures $\geq 20^0$C.

3.4.2.1 Depth

Facultative pond working (i.e., liquid) depths are usually in the range of 1-1.8m with 1.5m being the most commonly used. Depths of <1m will become an ideal breeding ground for mosquitoes while >1.8m will make the pond predominantly anaerobic.

3.4.2.2 BOD Removal

The BOD of the facultative pond effluent can be estimated using the equation below:

$$\mathbf{L_e = L_i/1 + K_1T_2} \quad (3.14)$$

Where:

L_i = BOD of either the raw wastewater (for primary facultative ponds) or anaerobic pond effluent (mg/l)

K_1 = first order rate constant for BOD removal

Design values for $K_{1(20)} = 0.3d^{-1}$ for primary facultative ponds and $0.1d^{-1}$ for secondary facultative ponds. The latter is lower because a lot of the BOD removal was done in the anaerobic ponds.

The term L_e is the unfiltered BOD which includes the BOD of the algae present in the facultative pond effluent. This algal BOD accounts for 70-90% of the total BOD of the effluent. Thus, the relationship between the filtered and unfiltered BOD (i.e., algal and algal total BOD) is;

$$\mathbf{L_e\ (filtered) = F_{na}\ (L_e\ unfiltered)} \qquad (3.15)$$

Where:

F_{na} = non algal fraction of the total BOD (varying from 0.1-0.3), 0.3 is the usual design value.

3.4.2.3 Microbiological Quality

After treatment with facultative ponds, it is necessary to assess the effluent quality in terms of suitability for re-use in restricted irrigation i.e., for numbers of human intestinal nematode eggs and E-Coli.

3.4.2.4 Human Intestinal Nematode Egg Removal

The WHO recommends that treated wastewaters used for crop irrigation should contain ≤human intestinal nematode egg/l or when children under 15 years are exposed, ≤0.1 egg/l. A derived equation for percentage egg removal is;

$$\mathbf{R = 100[1-0.14\ exp(-0.38T)]} \qquad (3.16)$$

Where:

R = % egg removal in anaerobic, facultative or maturation ponds

T = retention time in ponds

But the equation recommended for design is;

$$\mathbf{R = 100[1-0.41\ exp(-0.49T + 0.0085T^2)]} \qquad (3.17)$$

It is applied to each pond in series.

3.4.3 Maturation Pond Design

The main function of maturation ponds is to reduce the number of excreted pathogens, principally faecal bacteria and viruses present in the effluent of facultative ponds to a level suitable for agricultural and/ or aquacultural re-use. Although less than ideal for the purpose, faecal coliform bacteria are commonly used as indicators of excreted pathogens, so maturation ponds are usually designed to achieve a given removal of faecal coliforms.

Maturation ponds are typically aerobic throughout their depth.

The equation for a single maturation pond is;

$$\mathbf{N_e = N_i/(1 + K_T T_3)} \tag{3.18}$$

Where:

N_e = number of faecal coliforms per 100 ml effluent

N_i = number of faecal coliforms per 100 ml influent

K_T = first order rate constant for faecal coliform removal at T°C, d^{-1}

T_3 = maturation pond retention time

The value of K_T at various temperatures is given by the equation (see Table 3.3):

$$\mathbf{K_T - 2.6(1.19)^{T-20}} \tag{3.19}$$

Table 3.3: Values of the first order rate constant for faecal coliform removal at various design temperatures (from equation 3.19)

$T(^0C)$	K_T (day^{-1})	$T(^0C)$	K_T (day^{-1})
1	0.10	13	0.77
2	0.11	14	0.92
3	0.14	15	1.09
4	0.16	16	1.30
5	0.19	17	1.54
6	0.23	18	1.84
7	0.27	19	2.18
8	0.32	20	2.60
9	0.38	21	3.09
10	0.46	22	3.68
11	0.54	23	4.38
12	0.65	24	5.21

For a series of anaerobic, facultative and maturation ponds, equation 3.18 becomes:

$$\mathbf{N_e = N_i/[(1+K_TT_1)(1+K_TT_2)(1+K_TT_3)^n]} \qquad (3.20)$$

Where:

N_e and N_i now refer to the final effluent and raw wastewater respectively and n is the number of maturation ponds, which are assumed to be all of the same size (this is desirable as it optimizes removal efficiency (Marais 1974) but may not always be possible due to topographical constraints).

For unequally sized maturation ponds, the term $(l+K_TT_3)^n$ in equation 3.20 is replaced by $[(l+K_TT_{3a})(l+K_TT_{3b})(l+K_TT_{3n})]$.

3.5 Test Parameters and Procedure

The following parameters were tested for in the civil engineering laboratory and also at Springboard Research Laboratory upon obtaining wastewater samples from Boy's hostel, Block E, Unizik, Awka. The sample was gotten from a pipe connecting drains from both the bathrooms and kitchens.

i. pH
ii. Suspended solids
iii. Dissolved solids
iv. Biochemical Oxygen Demand (BOD_5)
v. Chemical Oxygen Demand (COD)

3.5.1 pH

pH is a measure of how acidic/basic water is. The range goes from 0 – 14, with 7 being neutral. pHs of less than 7 indicate acidity, whereas a pH of greater than 7 indicates a base. pH is really a measure of the relative amount of free hydrogen and hydroxyl ions in the water. Water that has more free hydrogen ions is acidic, whereas water that has more free hydroxyl ions is basic.

3.5.1.1 Apparatus

i. A pH metre
ii. Distilled water
iii. A glass stirring rod
iv. Standard buffer solution
v. Wide-mouth glass beaker
vi. A teaspoon or small scoop

Figure 3.4 A pH metre

3.5.1.2 Procedure

i. The sample was stirred vigorously using a clean glass stirring rod.
ii. 100ml of the sample was poured into the beaker.
iii. 100ml of distilled water was poured in a beaker and the buffer solution (Buffer 7) added and stirred with a stirring rod. This was used to standardize the pH metre.
iv. The electrode of the pH metre was immersed into the sample for about 30 seconds and the reading taken.
v. The pH value was read off the pH metre and recorded.

3.5.1.3 Result and Conclusion

The pH value was found to be 7.04

A pH value of 7.04 indicates that the sample is neutral.

3.5.2 Suspended Solids

Total suspended solids (TSS) is the dry-weight of suspended particles, that are not dissolved, in a sample of water that can be trapped by a filter that is analyzed using a filtration apparatus. It is a water quality parameter used to assess the quality of a specimen of any type of water or water body, ocean water for example, or wastewater after treatment in a wastewater treatment plant.

3.5.2.1 Apparatus

i. Conical flask
ii. Filter paper
iii. Wide-mouth glass beaker
iv. Heating mantle
v. Electronic weighing scale
vi. Desiccator
vii. Funnel

Figure 3.5 Heating mantle

Figure 3.6 Desiccator

3.5.2.2 Procedure

i. The filter paper was weighed and recorded.
ii. The conical flask was also weighed and recorded.
iii. 100ml of the sample was poured in the beaker.
iv. The measured sample was poured into the filter paper in the funnel and allowed to filter through.
v. The filter paper was removed and dried in the heating mantle then put in a desiccator.
vi. The filtrate inside the conical flask was put in the mantle too and allowed to evaporate to dryness then transferred to the desiccator.

3.5.2.3 Result and Conclusion

K_1 = Weight of filter paper = 0.991g

K_2 = Weight of filter paper + dried residue = 1.010g

$$\frac{\boldsymbol{K_2 - K_1}}{\mathbf{100}} \times \mathbf{10^6} \tag{3.21}$$

$$\frac{1.010 - 0.991}{100} \times 10^6$$

$$\frac{0.02}{100} \times 10^6$$

$$= 0.0002 \times 10^6$$

Total suspended solids = 200g/ml

3.5.3 Dissolved Solids

The term total dissolved solids refers to materials that are completely dissolved in water. These solids are filterable in nature. It is defined as residue upon evaporation of filterable sample.

3.5.3.1 Apparatus

i. Conical flask
ii. Wide-mouth glass beaker
iii. Heating mantle
iv. Desiccator
v. Funnel
vi. Electronic weighing scale

Figure 3.7 Electronic weighing scale

3.5.3.2 Procedure

i. The empty conical flask was weighed and recorded.
ii. The filtrate in the conical flask was evaporated to dryness in a heating mantle and put in a desiccator.
iii. The dried conical flask was weighed and recorded.

3.5.3.3 Result and Conclusion

W_1 = Weight of the empty conical flask = 111.78g

W_2 = Weight of the conical flask + dried filtrate = 111.87g

$$\frac{\mathbf{W_2 - W_1}}{\mathbf{100}} \times \mathbf{10^6} \tag{3.22}$$

$$\frac{111.87 - 111.78}{100} \times 10^6$$

$$\frac{0.09}{100} \times 10^6$$

$$= 0.0009 \times 10^6$$

Total dissolved solids = 900g/ml

$$\textbf{Total Solids TS = TDS + TSS} \tag{3.23}$$

900g/ml + 200g/ml = 1100g/ml

3.5.4 Biochemical Oxygen Demand (BOD)

The biochemical oxygen demand (BOD) test is a measurement of the quantity of oxygen required by bacteria to biologically oxidize organic material under aerobic conditions. BOD is usually expressed in mg/l but can also be expressed in lbs./day. The organic matter serves as food for the bacteria and the cell receives energy from the organic matter during its oxidation. By measuring the amount of oxygen consumed by the bacteria, the amount of BOD, or food for the bacteria, can be calculated.

3.5.4.1 Apparatus

i. BOD metre with probe for measurement of dissolved oxygen in 300 ml BOD bottles
ii. 300 ml stoppered BOD bottles
iii. Incubator, capable of maintaining 20 +/- 1°C
iv. 250 ml graduated cylinders
v. 100 ml graduated cylinders
vi. 25 ml measuring pipettes (wide-mouth)
vii. 10 ml measuring pipettes (wide-mouth)
viii. 100 ml beaker
ix. 1000 ml beaker
x. 250 ml Erlenmeyer flask
xi. Burette graduated to 0.1 ml
xii. Dilution water bottle of suitable volume for the number of tests to be performed
xiii. Pipette bulb

3.5.4.2 Procedure

For Dissolved Oxygen (Winkler Method);

i. The 300ml stoppered BOD bottle was filled to the brim with sample water.
ii. $1cm^3$ of 0.1M manganese sulphate solution was added followed by 0.1M alkali-iodide-azide solution. Care was taken by wiping the tips of the pipettes below the surface of the liquid and the stopper replaced immediately to avoid inclusion of air bubbles.
iii. The contents of the bottle were thoroughly mixed by inversion and rotation until a clear supernatant water was obtained.

iv. $1cm^3$ of concentrated sulphuric acid was added with the tip of the pipette below the level of solution and the stopper again replaced.

v. The contents of the bottle were mixed by rotation until the precipitation was completely dissolved.

vi. $100cm^3$ of the solution was pipetted into a $250cm^3$ conical flask and immediately titrated against standard sodium thiosulphate (0.025 mol/dm^3) using freshly prepared starch solution as the indicator. This was added when the solution became pale yellow.

vii. Titration was repeated in the duplicate stoppered BOD bottle.

3.5.4.3 Formular

$$\mathbf{D.O} = \frac{\textbf{mole of titrant x normality of titrant x 8000}}{\textbf{ml of sample used}} \qquad (3.24)$$

The general equation for the determination of BOD value (mg/l) is;

$$\frac{\mathbf{D_1 - D_2}}{\mathbf{P}} \qquad (3.25)$$

Where:

D_1 = initial DO of the sample

D_2 = final DO of the sample after 5 days

P = decimal volumetric fraction of sample used.

3.5.5 Chemical Oxygen Demand (COD)

COD is a measure of the oxygen equivalent of the organic matter in a water sample that is susceptible to oxidation by a strong chemical oxidant. COD is widely used as a measure of the susceptibility to oxidation of the organic and inorganic materials present in water bodies and in the municipal and industrial wastes. The COD test of natural water yields the total quantity of oxygen that is required for oxidation of a waste to carbon dioxide and water.

3.5.5.1 Apparatus

i. Culture tubes
ii. 25 ml measuring pipettes (wide-mouth)
iii. 10 ml measuring pipettes (wide-mouth)
iv. 100 ml beaker
v. 1000 ml beaker
vi. 250 ml Erlenmeyer flask
vii. Burette graduated to 0.1 ml

3.5.5.2 Procedure

i. 15ml of the sample was added into a 250ml beaker.
ii. 2.5ml of standard 5% $K_2C_rO_4$ was added slowly and mixed.
iii. 3.5ml of conc. Sulphuric acid reagent was added through the sides of the tubes and let go to the bottom.
iv. The contents were capped and mixed and transferred into a water bath at 50^0 C heat.
v. Distilled water was added to make the volume 50ml.
vi. 2 drops of phenanthroline indicator were added.
vii. It was titrated with 0.05M of ferrous ammonium sulphate solution, FAS (Mohr's salt).

3.5.5.3 Formular

$$\textbf{COD (mg/l)} = \frac{\mathbf{A-B\times M\times 8000}}{\mathbf{V_{sample}}} \tag{3.26}$$

Where:

A = volume of FAS used for blank (ml)

B = volume of FAS used for sample (ml)

M = molarity of FAS

8000 = milli equivalent weight of oxygen (8) ×1000 ml/l

3.5.6 BOD and COD Results

Table 3.4: Dissolved Oxygen, BOD and COD values obtained from laboratory tests

Parameters	Concentration	Reference Value
DO_1 mg/l	29.6	100
DO_2 mg/l	11.6	100
BOD_5 mg/l	360	100
COD mg/l	408	200

CHAPTER FOUR

PONDS DESIGN AND RESULT ANALYSIS

4.1 Ponds Design

In this chapter, the procedures involved in the actual design of the anaerobic, facultative and maturation ponds are explained and followed through in detail. The equations and formulas obtained and stated in the previous chapter are employed to achieve desired results and parameters.

4.1.1 System Design Parameters

Important parameters for WSP design used in these designs are;

- Temperature
- BOD
- Flow

Temperature: The design temperature is 25^0C which is the mean air temperature of the coolest month in Nigeria.

BOD: This value was obtained from laboratory test conducted on waste water from within the school community. A value of 360mg/l was obtained.

Flow: This was determined based on an estimated population of Nnamdi Azikiwe University, Awka which is 35,000 people.

Most wastewater comes from use of bathrooms, washing and some other activities. Therefore, if an estimated 8 litres is used and we use an average of 3 instances per person per day,

$$\text{Total amount used} = 8 \times 3 = 24 \text{ litres}$$

$$+1 \text{ extra litre for each person} = 25 \text{ litres}$$

Loading design approach;

Total volume of waste generated = population × average human waste contribution

$$= 35{,}000 \times 0.025\text{m}^3$$

Volume of waste generated = 875m^3/day

4.1.2 Anaerobic Pond Design

The sewage is domestic wholly with strength of 360mg/l BOD_5 and the flow rate is less than 10,000m^3/d, therefore preliminary treatment (Anaerobic) is not required (Mara, D.D. 1980) but this will be done for the purpose of this research.

Given;

L_i = 360mg/l

Q = 875m^3/day

T = 25^{0}C (mean air temperature of the coolest month in Nigeria)

B_v = 350g/m^3.d (from table 3.1)

D = 3m (assumed depth)

4.1.2.1 Pond Area

Pond area (from eqn. 3.2) = $\mathbf{L_iQ/B_vd_1}$

$$= \frac{360 \times 875}{350 \times 3}$$

$$= \frac{315000}{1050}$$

$$= 300\text{m}^2$$

4.1.2.2 Retention Time

$$= \frac{\textbf{pond area } \times \textbf{depth}}{\textbf{flow}}$$

$$= \frac{300 \times 3}{875}$$

$$= \frac{900}{875}$$

= 1.03d

4.1.2.3 Pond Volume

Pond Volume (from eqn. 3.1)

$$\mathbf{V_1} = \frac{\boldsymbol{L_i Q}}{\boldsymbol{B_v}}$$

$$= \frac{360 \times 875}{350}$$

$$= \frac{315000}{350}$$

= 900m^3

4.1.2.4 Pond Dimensions

Area = L × B for a rectangular section with ratio of 2:1

$$\frac{L}{B} = 2, \text{A} = 2\text{B}^2$$

300m^2 = 2B^2

Therefore, B = $\sqrt{\frac{300}{2}}$

The breadth (width) of the pond, B = 12.25m

Since L = 2B, L = 2 × 12.25 = 24.5m

Length of the pond, L = 24.5m

4.1.2.5 Effluent BOD Concentration from the Anaerobic Pond

For a mean air temperature of 25^0C, the removal efficiency, E is 70%. From equation (3.6),

$$\mathbf{BOD_{effl} = (1 - E/100).S_o}$$

where:

S_o = influent total BOD concentration (mg/l)

BOD_{effl} = effluent total BOD concentration (mg/l)

Therefore,

$$BOD_{effl} = \left(1 - \frac{70}{100}\right) \times 360$$

$$= 1 - 0.7 \times 360$$

$$= 108 \text{mg/l}$$

4.1.3 Facultative Pond Design

Facultative ponds are best designed on the basis of surface BOD loading (B_s kg/ha/day) which is given by equation (3.7);

$$\mathbf{B_s = 10L_iQ/A_2}$$

Where:

A_2 = facultative pond area (m^2)

The factor 10 arises from the units used

L_iQ = mass of BOD entering the pond (g/day)

So, $10^{-3}L_iQ$ is in Kg/day and the area in hectares is $10^{-4}A_2$ which is in m^2

From equation (3.9), B_s (surface BOD loading);

$$= 20(25) – 120$$

$$= 500 – 120$$

$$= 380 \text{ kg/ha/d}$$

4.1.3.1 Pond Area

Pond area from eqn. (3.7) = $\mathbf{10L_iQ/B_s}$

$$= \frac{10 \times 94.5}{380}$$

$$= \frac{945}{380}$$

$$= 2.5\text{ha} = 25{,}000\text{m}^2$$

4.1.3.2 Retention Time

From eqn. (3.13), $\mathbf{T_2 = 2A_2D/(2Q_i – 0.001eA_2)}$

Assume D = 1.0m

e = 5.3mm/d (Nigeria Meteorological Agency, NIMET) = 0.0053m/d

$$T_2 = \frac{2 \times 25000 \times 1.0}{[(2 \times 875) - (0.001 \times 0.0053 \times 25000}$$

$$= \frac{50000}{1750 - 0.1325}$$

$$= \frac{50000}{1749.9}$$

$$= 28.5\text{days}$$

4.1.3.3 Pond Volume

$$= \text{pond area} \times \text{depth}$$

$$= 25000 \times 1 = 25000\text{m}^3$$

4.1.3.4 Pond Dimensions

$$\text{Area} = L \times B \text{ for a rectangular section with ratio of } 2:1$$

$$\frac{L}{B} = 2, A = 2B^2$$

$$25000m^2 = 2B^2$$

$$\text{Therefore, } B = \sqrt{\frac{25000}{2}}$$

The breadth (width) of the pond, B = 111.8m

Since L = 2B, L = 2 × 111.8 = 223.6m

Length of the pond, L = 223.6m

4.1.3.5 BOD Removal in the Facultative Pond

The BOD of the facultative pond effluent can be estimated using equation (3.14) but K_1 (first order rate constant for BOD removal) has to be first determined;

$$\mathbf{K_1 = 0.1(1.05)^{T-20}}$$

$$= 0.1(1.05)^{25-20}$$

$$= 0.13d^{-1}$$

From eqn. (3.14), $\mathbf{L_e = L_i/1 + K_1T_2}$

$$= \frac{108}{[1+(0.13\times 28.5)]}$$

$$= \frac{108}{4.7}$$

= 23mg/l unfiltered BOD

Filtered BOD = $L_{e(filtered)} = 0.3(L_{e(unfiltered)})$

$$= 0.3 \times 23$$

$$= 6.9mg/l$$

4.1.4 Maturation Pond Design

4.1.4.1 Pond Area

This is given by; $\mathbf{Q \times t/D}$

Where t = 7days (minimum adopted)

D = 1.5m

$$= \frac{875 \times 7}{1.5}$$

$$= \frac{6125}{1.5}$$

$$= 4083.33\text{m}^2$$

4.1.4.2 Retention Time

$$\mathbf{T_{mat} = AD/Q}$$

$$= \frac{4083.33 \times 1.5}{875}$$

$$= \frac{6125}{875}$$

= 7days

4.1.4.3 Pond Volume

= pond area × depth

4083.33 × 1.5

$$= 6125\text{m}^3$$

4.1.4.4 Pond Dimensions

Area = L × B for a rectangular section with ratio of 2:1

$$\frac{L}{B} = 2, A = 2B^2$$

$$4083.33m^2 = 2B^2$$

$$\text{Therefore, } B = \sqrt{\frac{4083.33}{2}}$$

The breadth (width) of the pond, B = 45.2m

Since L = 2B, L = 2 × 45.2 = 90.4m

Length of the pond, L = 90.4m

4.1.4.5 Faecal Coliform Removal

This can be estimated from equation (3.18) $\mathbf{N_e = N_i/(1 + K_T T_3)}$.

Where:

N_i = 4 × 107fc/ml (adopted minimum)

K_T = 0.13

T_3 = 7days

$$N_e = \frac{4 \times 107}{1+0.13 \times 7}$$

$$= \frac{428}{1.91}$$

= 224.08fc/100ml

224.08 < 5000 fc/100ml effluent standard which implies that the design is satisfactory.

4.2 Pond Facility Design

- **Pond Geometry:** The hydraulic characteristics of rectangular and trapezoidal section ponds have been found to be superior to those of square, circular ponds and those with irregular geometry. Length to breath ration of 2:1 is adopted for stability. The ponds are to be trapezoidal in section and rectangular in plan.

- **Pond Base and Embankment:** The bottom of the pond should be impermeable, although the sludge layer is expected to seal up small pores in the soil. Sealing of the base is necessary to prevent ground water pollution. Therefore, plain in-situ concrete is adopted for the ponds base, sides and on top of embankment to protect it from erosion.

  An embankment slope of 1:3 is usually satisfactory in most soil conditions. If steeper slopes are used, their stability should be established by standard soil mechanics procedures. The plain-in-situ concrete stop vegetation growth down the banks and so prevent the breeding of mosquitoes and makes maintenance easier.

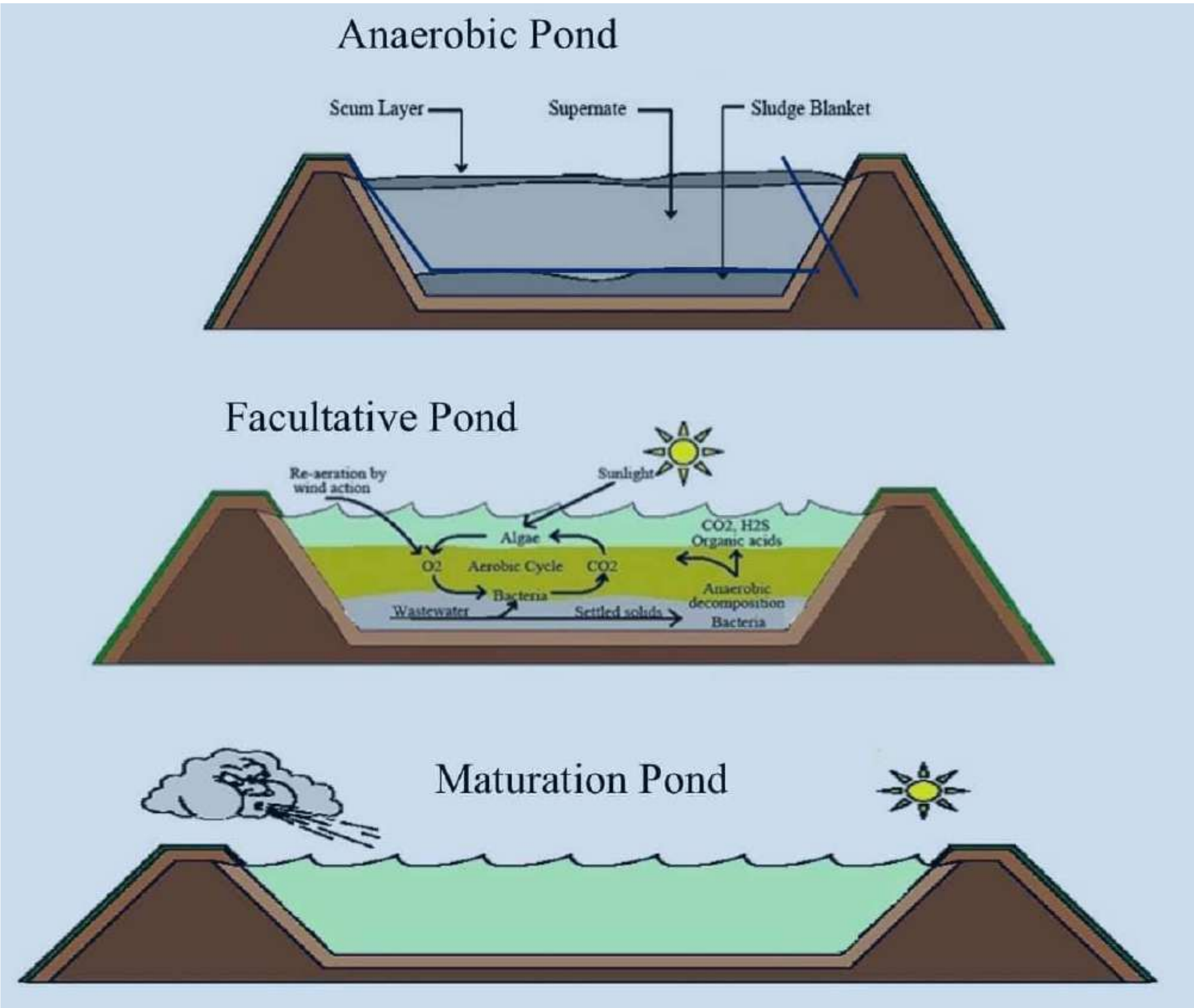


Figure 4.1 Cross section of waste stabilization ponds

4.3 Analysis of Results

- The anaerobic pond is designed for BOD removal according to volumetric loading to reduce the BOD loading of the waste water generated. The sludge is deposited at the bottom of the pond and broken down by the anaerobic bacteria.
- The pond is trapezoidal in shape so as to avoid erosion of the pond banks with a side slope of 1:1.
- A length-breadth ratio of 2:1 is used and this is to ensure that sludge banks do not from around the inlet.
- The pond has a retention time of 1.03days which reduces the BOD loading from 360mg/l in the influent to 108mg/l.
- The facultative pond is also used for BOD removal but is designed according to surface loading. The surface loading of the pond is based on temperature and called the empirical method, (mean coldest temperature in the year 25^0C.
- The sludge is also deposited at the bottom of the pond and the pond is trapezoidal in shape for the same reason as the anaerobic pond.
- A length-breadth ratio of 2:1 is used to ensure that sludge bank do not from around the inlet of the pond.
- The pond has a retention time of 28.5 days during which the influent BOD which is 108mg/l reduces to 23mg/l in the effluent thus meeting the standard value set by NEMA and an area of 25,000m^2 was obtained and therefore results from this design can be depended on.
- The maturation pond is designed according to Marias' method (1974) for faecal removal. It is also trapezoidal in shape to avoid erosion.
- The length-breadth ratio is 2:1 to better approximate plug flow conditions.
- The pond has a retention time of 7 days for complete decomposition and this time is enough to reduce the faecal coliform to 224.08fc/100ml which

falls within the acceptable standard of National Water and Sewage Corporation of 5000fc/100ml.

4.4 Pond Maintenance

The maintenance requirements of ponds are very simple but they must be carried out regularly otherwise, there will be serious odour, flies and mosquito nuisance. Maintenance requirements and responsibilities must therefore be clearly defined so as to avoid problems later.

Routine maintenance tasks are as follows:

- Removal of screenings and grit from the preliminary works.
- Cutting the grass on the embankments and removing it so that it does not fall into the pond (this is necessary to prevent the formation of mosquito-breeding habitats; the use of slow-growing grasses minimizes this task).
- Removal of floating scum and floating macrophytes from the surface of facultative and maturation ponds (this is required to maximize photosynthesis and surface re-aeration and obviate fly breeding).
- Spraying the scum on anaerobic ponds (which should not be removed as it aids the treatment process), as necessary, with clean water or pond effluent to prevent fly breeding.
- Removal of any accumulated solids in the inlets and outlets.
- Repair of any damage to the embankments caused by rodents, rabbits or other animals.
- Repair of any damage to external fences and gates.

4.4.1 Desludging

Anaerobic ponds require desludging when they are half full of sludge. This occurs every n years:

$$\mathbf{n = V_a/2Ps} \tag{4.1}$$

where:

V_a = volume of anaerobic pond (m^3)

P = population served

s = sludge accumulation rate (m^3/person/year)

Sludge removal can be readily achieved by using a raft-mounted sludge pump, which discharges into either an adjacent sludge lagoon or tankers that transport it to a landfill site, central sludge treatment facility or other suitable disposal location.

Although the microbiological quality of pond sludge is better than that from conventional sewage treatment works and its toxic chemical composition no worse, its disposal must still be carried out in accordance with the relevant local or regional regulations governing sludge disposal.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Throughout the years, the nature of water has been crumbling, predominantly because of the anthropogenic exercises, increase in population, impromptu urbanization, quick industrialization and unskilled utilization of natural water resources. Besides, the expanded consciousness of the significance of given effects because of the current environmental policies has driven the research community towards the advancement of robust, economically feasible and environmentally friendly techniques capable of eliminating contaminants from water and at same time, protecting the health of influenced communities.

Sewage can be a friend as well as an enemy. It is a friend if properly handled and an enemy if not handled properly. Looking at the use of effluents from the ponds, it is worth handling properly. As a matter of fact, sewage is expensive but it is worth having, as it helps in reducing water borne and water related diseases.

More so, the choice of waste stabilization ponds for the treatment of the sewage from the boy's hostel is not unconnected with its advantage over other biological treatment processes. Also, the comparatively low constructional and operating cost, simplicity and high treatment efficiency of the stabilization ponds, have rendered it undoubtedly the most widely applicable and advantageous method of sewage treatment especially in hot climates where the ambient temperature is an advantage.

The disadvantage is that they require larger area of land than other treatment methods.

The anaerobic pond has a design area of $300m^2$ and a retention time of 1.03days with the influent BOD at 360mg/l.

The facultative pond has an area of $25000m^2$ and a retention time of 28.5days. The effluent BOD is 23mg/l (unfiltered) which is less than the standard maximum of 50mg/l so the pond is effective.

The maturation pond has an area of $4083.33m^2$ and a retention period of 7days. The number of faecal E.coli in the effluent is 224.08fc/100mc, this meets the standard value which is <5000fc/100ml.

The research was able to reveal that the ponds have the capacity to reduce most of the organic pollutants to an appreciable percent.

5.2 Recommendation

A central sewage collection and treatment system should be adopted in the school community as this research has shown its numerous advantages and efficiency. Also, measures should be put in place to ensure and facilitate easy collection of data as related to monitoring and evaluation of wastewater in the community.

Upon adoption of a central sewage system and subsequent construction of the waste stabilization ponds, to make the environment a conducive place to live in, high sanitary conditions should be maintained. This helps immensely in keeping diseases away from the inhabitants of the community.

The ponds should also be enclosed so as to prevent accidents and other possible mishaps.

REFERENCES

Agunwamba J.C. (2000). Water engineering systems immaculate publication Ltd, Enugu state Nigeria.

Agunwamba J.C. (1992). Prediction of the dispersion number in waste stabilization ponds. Water Research.

Aldana G. J., Lloyd, B. J., Guganesharajah K. & Bracho N. 2005. The development and calibration of a physical model to assist in optimising the hydraulic performance and design of maturation ponds. Water Science and Technology, 51**,** 173-181.

Arceivala S.J. 1973. Simple Waste Treatment Methods: Aerated Lagoons, Oxidation Ditches, Stabilisation Ponds in Warm and Temperate Climates, Middle East Technical University.

Arceivala S.J. (1981). Wastewater treatment and disposal. Marcel Dekker, New York. 892 p.

Arthur J. P. & Mundial B. 1983. Notes on the design and operation of waste stabilization ponds in warm climates of developing countries. BIRF Technical Paper. World Bank.

Cavalcanti P.F.F., Van Haandel A.C., Kato M.T., Von Sperling M., Luduvice M.L., Monteggia L.O. (2001). Chapter 3: Post-treatment of anaerobic reactor effluents by polishing ponds. In: Chernicharo, C.A.L. (coordinator). Post-treatment of anaerobic reactor effluents. PROSAB/ABES, Rio de Janeiro. p. 105–170.

CETESB (1989) Operation and maintenance of anaerobic and facultative ponds. Environmental Sanitation Technology Company., Sao Paulo. 91 p.

Coggins L. X., Ghisalberti M. & Ghadouani A. 2017. Sludge accumulation and distribution impact the hydraulic performance in waste stabilisation ponds. Water Research, 110**,** 354-365.

Epa U. 1983. Design Manual: Municipal Wastewater Stabilization Ponds United States Environmental Protection Agency, Office of Research and Development.

Epa U. 2011. Principles of Design and Operations of Wastewater Treatment Pond Systems for Plant Operators, Engineers, and Managers, United States Environmental Protection Agency, Office of Research and Development.

Frank R. S. & Joanne, E. D. 2014. Wastewater Stabilization Ponds, CRC Press.

Glaz P., Bartosiewicz M., Laurion I., Reichwaldt E. S., Maranger R. & Ghadouani A. 2016. Greenhouse gas emissions from waste stabilisation ponds in Western Australia and Quebec (Canada). Water Research, 101**,** 64-74.

Gloyna E. F., Malina J. F. & Davis E. M. 1976. Ponds as a wastewater treatment alternative, Center for Research in Water Resources, College of Engineering, University of Texas at Austin.

Gloyna E.F. Waste stabilization ponds. Geneva, World Health Organization, 1971.

Hernandez-Paniagua I. Y., Ramirez-Vargas R., Ramos-Gomez M. S., Dendooven L., Avelar-Gonzalez F. J. & Thalasso F. 2014. Greenhouse gas emissions from stabilization ponds in subtropical climate. Environmental Technology, 35**,** 727-734.

Jordao E.P. & Pesso A. C.A. (1995) Domestic sewage treatment. ABES, 3a ed. 683 p.

Karteris A., Papadopoulos A. & Balafoutas G. 2005. Modeling the temperature pattern of a covered anaerobic pond with computational fluid dynamics. Water Air and Soil Pollution, 162**,** 107-125.

Kayombo S., Mbwette T.S.A., Katima J.H.Y., Ladegaar N. And Jørgensen S. E 2005. Waste Stabilization Ponds and Constructed Wetlands Design Manual. WSP and CW research project, College of Engineering and Technology, University of Dar es Salaam.

Konig A.M. (2000). Biology of stabilization ponds: algae. In: Mendonca, S.R Stabilization Pond systems. McGraw-Hill. Colombia.

Li M., Zhang, H., Lemckert C., Lu Z., Lei L. & Stratton H. 2013. Three-dimensional Investigation of Retention Time Distribution of Waste Stabilisation Ponds. 20th International Congress on Modelling and Simulation (Modsim2013)**,** 2723-2729.

Lugali Yvonne (2012): Design of sewage treatment system for Gulu University.

M.M. Saqqar, M.B. Pescod, Modelling coliform reduction in waste water stabilization pond. Water Sci. Technology, 26 (1992) 1667–1677.

Mara D. D. & Pearson H. W. 1998. Waste stabilization ponds: design manual for Mediterranean Europe. Waste stabilization ponds: design manual for Mediterranean Europe. World Health Organization. Regional Office for Europe.

Mara D. D. 1975a. Design Manual for Sewage Lagoons in the Tropics, East African Literature Bureau.

Mara D. D. 2003. Design manual for waste stabilization ponds in India, IWA.

Mara D.D. (1997). Design Manual for Waste Stabilization Ponds in India. Leeds: Lagoon Technology International.

Mara D.D., Alabaster G.P., Pearson H.W., Mills S.W. (1992). Waste stabilisation ponds. A design manual for Eastern Africa. Lagoon Technology International. Leeds. 121 pp.

Marais G.V.R. (1974). Faecal bacteria kinetics in stabilisation ponds. J. Env. Eng. Div., ASCE, 100 (EE1), p. 119.

Moller C. C., Weisser J. J., Msigala S., Mdegela R., Jorgensen S. E. & Styrishave B. 2016. Modelling antibiotics transport in a waste stabilization pond system in Tanzania. Ecological Modelling, 319**,** 137-146.

Passos R. G., Von Sperling M. & Ribeiro T. B. 2014. Hydrodynamic evaluation of a full-scale facultative pond by computational fluid dynamics (CFD) and field measurements. Water Science and Technology, 70**,** 569-575.

Persson J. 2000. The hydraulic performance of ponds of various layouts. Urban Water, 2**,** 243-250.

Salter H. E., Ta C. T. & Williams S. C. 2000. Three-dimensional computational fluid dynamic modelling of a facultative lagoon. Water Science and Technology, 42**,** 335-342.

Shilton A. & Harrison J. 2003a. Integration of coliform decay within a CFD (computational fluid dynamic) model of a waste stabilisation pond. Water Science and Technology, 48**,** 205-210.

Shilton A. 2000. Potential application of computational fluid dynamics to pond design. Water Science and Technology, 42**,** 327-334.

Shilton A. N. & Harrison J. 2003b. Guidelines for the hydraulic design of waste stabilisation ponds, Institute of Technology and Engineering, Massey University.

SIDA, 2000. Water and Wastewater Management in Large to Medium-sized Urban Centers.

Silva Jr. C., Sasson S. (1993) Biology 2: Living things, structure and function. (Caesar and Sezar). Current Publisher, Sao Paulo, 2a ed. 382 p.

Sweeney D. G., Cromar N. J., Nixon J. B., Ta C. T. & Fallowfield H. J. 2003. The spatial significance of water quality indicators in waste stabilization ponds - limitations of residence time distribution analysis in predicting treatment efficiency. Water Science and Technology, 48**,** 211-218.

Sweeney D. G., Nixon J. B., Cromar N. J. & Fallowfield H. J. 2005. Profiling and modelling of thermal changes in a large waste stabilisation pond. Water Science and Technology, 51**,** 163-172.

Tchobanoglous G. & Schroeder E. D. 1985. Water Quality: Characteristics, Modeling, Modification, Addison-Wesley.

Tilley E., Lüthi C., Morel A., Zurbrügg C. & Schertenleib R. 2014 Compendium of sanitation systems and technologies. Dübendorf: Swiss Federal Institute of Aquatic Science and Technology (EAWAG).

United Nations Environment Programme International Environment Technology Contest (UNEP-IETC) and the Danish International Development Agency, DANIDA).

Van Buuren J.J.L., Frijns J.A.G., Lettinga G. (1995). Wastewater treatment and reuse in developing countries. Wageningen Agricultural University.

Van Haandel A.C., Lettinga G. (1994). Anaerobic sewage treatment. A handbook for hot climates.

Van-Lier J., P. Seeman and Lettinga 1998. Decentralized urban sanitation concepts: perspectives for reduced water consumption and wastewater reclamation for reuse. EP & RC Foundation, Wageningen, Netherlands, Sub-Department of Environmental Technology, Agricultural University.

Verbyla M. E. & Mihelcic J. R. 2015. A review of virus removal in wastewater treatment pond systems. Water Research, 71**,** 107-124.

WHO (1989). Health guidelines for the use of wastewater in agriculture and aquaculture. Technical Report Series 778. Geneva: World Health Organization.

Wood M. G., Greenfield P. F., Howes T., Johns M. R. & Keller J. 1995. Computational fluid dynamic modelling of wastewater ponds to improve design. Water Science and Technology, 31**,** 111-118.

Wood M. G., Howes T., Keller J. & Johns M. R. 1998. 2-dimensional computational fluid dynamic models for waste stabilisation ponds. Water Research, 32**,** 958-963.

APPENDIX A

LABORATORY EXPERIMENTS CONDUCTED AT ENGINEERING LABORATORY (WATER LAB.)

1. pH

pH is a measure of how acidic/basic water is. The range goes from 0 – 14, with 7 being neutral. pHs of less than 7 indicate acidity, whereas a pH of greater than 7 indicates a base. pH is really a measure of the relative amount of free hydrogen and hydroxyl ions in the water. Water that has more free hydrogen ions is acidic, whereas water that has more free hydroxyl ions is basic.

Apparatus

1. A pH metre
2. Distilled water
3. A glass stirring rod
4. Standard buffer solution
5. Wide-mouth glass beaker
6. A teaspoon or small scoop

Procedure

1. The sample was stirred vigorously using a clean glass stirring rod.
2. 100ml of the sample was poured into the beaker.
3. 100ml of distilled water was poured in a beaker and the buffer solution (Buffer 7) added and stirred with a stirring rod. This was used to standardize the pH metre.
4. The electrode of the pH metre was immersed into the sample for about 30 seconds and the reading taken.
5. The pH value was read off the pH metre and recorded.

Result and Conclusion

The pH value was found to be 7.04

1

A pH value of 7.04 indicates that the sample is neutral.

2. SUSPENDED SOLIDS

Total suspended solids (TSS) is the dry-weight of suspended particles, that are not dissolved, in a sample of water that can be trapped by a filter that is analyzed using a filtration apparatus. It is a water quality parameter used to assess the quality of a specimen of any type of water or water body, ocean water for example, or wastewater after treatment in a wastewater treatment plant.

Apparatus

1. Conical flask
2. Filter paper
3. Wide-mouth glass beaker
4. Heating mantle
5. Electronic weighing scale
6. Desiccator
7. Funnel

Procedure

1. The filter paper was weighed and recorded.
2. The conical flask was also weighed and recorded.
3. 100ml of the sample was poured in the beaker.
4. The measured sample was poured into the filter paper in the funnel and allowed to filter through.
5. The filter paper was removed and dried in the heating mantle then put in a desiccator.
6. The filtrate inside the conical flask was put in the mantle too and allowed to evaporate to dryness then transferred to the desiccator.

Result and Conclusion

K_1 = Weight of filter paper = 0.991g

K_2 = Weight of filter paper + dried residue = 1.010g

$$\frac{\boldsymbol{K_2 - K_1}}{\mathbf{100}} \times \mathbf{10^6}$$

$$\frac{1.010 - 0.991}{100} \times 10^6$$

$$\frac{0.02}{100} \times 10^6$$

$$= 0.0002 \times 10^6$$

Total suspended solids = 200g/mol

3. DISSOLVED SOLIDS

The term total dissolved solids refers to materials that are completely dissolved in water. These solids are filterable in nature. It is defined as residue upon evaporation of filterable sample.

Apparatus

1. Conical flask
2. Wide-mouth glass beaker
3. Heating mantle
4. Desiccator
5. Funnel
6. Electronic weighing scale

Procedure

1. The empty conical flask was weighed and recorded.

3

2. The filtrate in the conical flask was evaporated to dryness in a heating mantle and put in a desiccator.
3. The dried conical flask was weighed and recorded.

Result and Conclusion

W_1 = Weight of the empty conical flask = 111.78g

W_2 = Weight of the conical flask + dried filtrate = 111.87g

$$\frac{\mathbf{W_2 - W_1}}{\mathbf{100}} \times \mathbf{10^6}$$

$$\frac{111.87 - 111.78}{100} \times 10^6$$

$$\frac{0.09}{100} \times 10^6$$

$$= 0.0009 \times 10^6$$

Total dissolved solids = 900g/mol

Total Solids TS = TDS + TSS

900g/mol + 200g/mol = 1100g/mol

Mr. Onwuliri J.C.

Lab. Technician

2/2/22

Mr. Akerejola S.S.

Lab. Technician

APPENDIX B

LABORATORY REPORT SHEET IMAGES FROM SPRINGBOARD RESEARCH LABORATORIES

Name of Customer: Okaliwe David
Sample: Bathroom/ Kitchen water
Test Required: BOD, COD
Date Received: 21st Jan ,2022.

Springboard Research Laboratories
Rd 3, House 1, Udoka Housing Estate,
Awka Anambra State.
08100382601, 046844587, 08022087994
springboardlab@yahoo.com

Parameters	Concentration	Reference value
DO_1 mg/l	29.6	100
DO_5 mg/l	11.6	100
BOD mg/l	360	100
COD mg/l	408	200

Okeke David Okechukwu
(PhD, MIPAN)
Public Analyst

Signature of analyst

Determination of Chemical Oxygen Demand

- 15ml of the sample into a 250 beaker
- 2.5ml standard 5% K_2Cr0_4 was added slowly and mix.
- 3.5ml of Conc. sulphuric acid reagent was added through side of the tubes and let it go to the bottom.
- Cap and mix the the contents.
- Transfer into a water bath and heat at 50degree
- Put a blank test also.
- Add distilled water to make the volume to 50ml.
- 1 -2 drops of phnanthronlein indicator.
- Titrate with 0.05m ferrous ammonium sulphate solution (Morh Salt)
- Calculation ; COD as mg/l $= \frac{A - B \times 8000}{\text{ml Sample}}$

A = Titre of blank
B = Titre of sample
M = Molarity of FAS (0.05m)

Dissolved Oxygen in Water determination

Carefully remove the stopper from the sample bottle and add in turn 1cm^3 of 0.1M manganous sulphate solution followed by 1cm^3 0.1 M iodide –azide solution. When introducing various reagents into the full bottle of sample, the tips of the pipettes should be wipe below the surface of the liquid. Replace the stopper carefully after each addition so as to avoid inclusion of air bubbles. Thoroughly mix the the contents by inversion and rotation until a clear supernatant water is obtained.

Add 1cm^3 concentrated sulphuric acid with the tip of the pipette below the level of solution and again replace the stopper. Mix well by rotation until the precipitation has completely dissolved.. Pipette into a 250cm^3 conical flask 100cm^3 of the solution and immediately titrate it against standard sodium thiosulphate (0.025 mol dm^3) using freshly prepared starch solution as the indicator (add when solution becomes pale yellow). Carry out the titration in duplicate.

Biochemical Oxygen Demand Determination by Winkler method

Dissolved oxygen

Carefully remove the stopper from the sample bottle and add in turn 1cm^3 manganous sulphate solution followed by 1cm^3 alkaline – iodide –azide solution. When introducing various reagents into the full bottle of sample, the tips of the pipettes should be wipe below the surface of the liquid. Replace the stopper carefully after each addition so as to avoid inclusion of air bubbles. Thoroughly mix the contents by inversion and rotation until a clear supernatant water is obtained.

Add 1cm^3 concentrated sulphuric acid with the trip of the pipette below the level of solution and again replace the stopper. Mix well by rotation until the precipitation has completely dissolved. Pipette into a 250cm^3 conical flask 100cm^3 of the solution and immediately titrate it against standard sodium thiosulphate (0.025 mol dm^3) using freshly prepared starch solution as the indicator (add when solution becomes pale yellow). Carry out the titration in duplicate.

$$D.O = \frac{\text{mole of titrant x normality of titrant x 8000}}{\text{Ml of sample used}}$$

The general equation for the determination of a BOD value is:

BOD (mg/l) = $D_1 - D_5$

Where D_1 = initial DO of the sample, D_2 = final DO of the sample after 5 days, and P = decimal volumetric fraction of sample used.
If 100 ml of sample are diluted to 300ml, then P= 0.33. Notice that if no dilution was necessary, P = 1.0 and the BOD is determined by $D_1 - D_2$
standard sodium thiosulphate (0.025 mol dm^3) using freshly prepared starch solution as the indicator (add when solution becomes pale yellow). Carry out the titration in duplicate.

- Siphen and read the pH using digital pH meter.

Reference

American Public Health Association (1995) 3112B,Cold – Vapour Atomic Absorption Spectrometric Method, Standard Methods for the Examination of Water and Wastewater,20th Edition, APHA, AWWA,WEF