THE INFLUENCE OF MOULDING MOISTURE CONTENT ON THE ENGINEERING PROPERTIES OF AGGREGATE-LIME-NATURAL POZZOLAN MIXES

by

ARIP-KITUYAN EMMANUEL OLEKAMBAINEI

A dissertation submitted in partial fulfilment of the requirements for the degree of

MASTER OF ENGINEERING

(TRANSPORTATION ENGINEERING)

in the

Faculty of Engineering, Built Environment and Information Technology
University of Pretoria, PRETORIA

October 2004

THE INFLUENCE OF MOULDING MOISTURE CONTENT ON THE ENGINEERING PROPETIES OF AGGREGATE-LIME-NATURAL POZZOLAN MIXES

AKE OLEKAMBAINEI

Supervisor: **Professor AT Visser**

Department: **Civil and Biosystems Engineering**
University of Pretoria

Degree: **Master of Engineering (Transportation Engineering)**

SUMMARY

The current trends in the road transport sector show a growth in axle loads as well as vehicle numbers on all types of roads in highly industrialised countries as well as in developing countries. This increase in axle loads and numbers has forced road agencies to amend their design standards adopting designs that provide roads with higher load bearing capacity. However, the rapid depletion of natural road construction gravel, as well as strict environmental conservation laws have resulted in many agencies in-charge of road construction and maintenance to resort to use of alternative materials that will be economically feasible and environmental friendly. The use of natural pozzolans for stabilising pavement layers fulfils this requirement. This research study was performed with the aim of evaluating the engineering properties of aggregate-lime-natural pozzolan (ALP) mixtures at varying compaction degrees of saturation and to compare them with conventional cement-stabilized aggregates. Two types of pozzolans found in Tanzania were used.

The laboratory investigation was carried out in two parts, namely a pilot investigation where the strength behaviour with time, shrinkage and CBR were determined at three degrees of saturation for a washed river sand specimen followed by the main investigation using two different types of sands at four varying degrees of saturation.

The study showed that the compaction degree of saturation for ALP mixes plays an important role in their tensile and compressive strengths development regardless of their optimum moisture contents. The ratio between tensile and compressive strengths for ALP mixes was also found to closely obey the relation given by Fulton (2001) for concrete.

The ALP mixes were also observed to develop their strength similar to cement mixes with the formation of tobermorite crystals with the additional of water and appropriate activator. Both pozzolan mixes developed significant tensile and compressive strength after 28 days of curing similar to cement mixes.

High CBR values for the two ALP mixes were obtained in mixes moulded at degrees of saturation lower than that corresponding to their optimum. Similarly, the shrinkage of the mixes was found to decrease with a decrease in the degrees of saturation. The CBR and shrinkage of the ALP mixes were found to show similar trend to that of the control cement mixes.

The ALP mixes showed no significant strength loss with an increase in the fines content in unwashed sand mix in comparison with that of washed sand mixes. No significant strength loss was observed in the ALP mixes as in the control cement mixes at all moulding degrees of saturation.

Finally the study concluded that the ALP mixes could be used in stabilization of pavement layers. However, care must be taken in deciding the compaction degrees of saturation as the specifications used in conventional cement stabilization does not necessarily yield desirable strength development in ALP mixes.

KEY WORDS:

Natural pozzolan, Tanzania, aggregate-lime-pozzolans, compressive strength, tensile strength, shrinkage, CBR

To my Cousin, MAGILANI *“Levava”* **MOLLEL, in loving memory**

“When I was at home, I was in a better place, but travellers must be content”
-WILLIAM SHAKESPEARE,
As You Like It, Act II, Scene iv

ACKNOWLEDGEMENT

I am most grateful to the following:

First and foremost to GOD ALMIGHTY in whom all things are possible,

Prof. Alex Visser, my supervisor for going that extra mile with me throughout the research study,

My parents and family for the sponsorship of my masters degree studies making it possible for the research work to be done and for all the moral support throughout my studies,

COWI Tanzania Limited for their financial support in sampling and transportation of pozzolan samples used in the study from Arusha and Mbeya regions (in Tanzania) to the University of Pretoria, South Africa,

The Department of Civil and Biosystems Engineering, the University of Pretoria for the financial support in procuring other test materials used in the study as well as allowing me to make use of its laboratories,

Mr. Chris Brits and the staff of the Africon Laboratories in Pretoria for their technical assistance as well as allowing me to make use of laboratory facilities not available at the University of Pretoria,

Mr. Andre Botha of Microscopy and Microanalysis Unit, the University of Pretoria for assisting with chemical analysis and scanning electron microscopy (SEM) of the test mixes,

Specialised Road Technologies, for the use of their laboratory jaw crusher for crushing of the pozzolan sample from Mbeya,

Last but not least to my fiancée, Joan “Nasiwa” for encouragements during those times when the going got tough and for believing in me.

TABLE OF CONTENTS

SUMMARY .... (i)

ACKNOWLEDGEMENTS .... (v)

LIST OF TABLES .... (x)

LIST OF FIGURES .... (xi)

LIST OF PLATES .... (xii)

ABBREVIATIONS .... (xiii)

CHAPTER 1: INTRODUCTION 1-1

2.1 Background 1-1
2.2 Problem Statement 1-3
2.3 Objectives of the Research 1-4
2.4 Scope of the Study 1-5
2.5 Organization of the Dissertation 1-6

CHAPTER 2: AGGREGATE-LIME-POZZOLANS: LITERATURE REVIEW 2-1

2.1 Introduction 2-1
2.1.1 Definition of Pozzolans 2-1
2.1.2 Background 2-2
2.1.3 Types of Pozzolans 2-3

2.2 Pozzolan Activity 2-3

2.3 Uses of Pozzolans 2-5

2.4 Natural Pozzolans in Tanzania 2-6
2.4.1 Pozzolans from Oldonyo Sambu (Arusha region) 2-9
2.4.2 Pozzolans from Mbeya region 2-11
2.4.3 Pozzolans from Kilimanjaro region 2-12

2.4.4 Chemical Composition 2-12
2.4.5 Lime-Pozzolan Reactivity 2-12

2.5 Engineering Properties of ALP 2-13
2.5.1 California Bearing Ratio (CBR) 2-15
2.5.2 Strength Development 2-15
2.5.3 Durability 2.17
2.5.4 Strength Characteristics of Tanzanian Pozzolans 2-20

2.6 Concluding Remarks on Tanzanian Pozzolans 2-21

2.7 Test Methods for Evaluation of Stabilized Soils 2-22
2.7.1 Moulding Moisture Contents 2-22
2.7.2 Compressive Strength Determination 2-22
2.7.3 Tensile Strength Determination 2-23
2.7.4 Shrinkage Determination 2-25

2.8 Guidance Gained from the Literature Review for this Study 2-26

CHAPTER 3: EXPERIMENTAL AND TEST PROGRAMME 3-1

3.1 Introduction 3-1
3.2 Test Materials 3-1
3.3 Experimental Design 3-4
3.4 Curing Conditions 3-7
3.5 Research Methodology 3-7

CHAPTER 4: DISCUSSION OF THE PILOT STUDY RESULTS 4-1

4.1 Introduction 4-1

4.2 Indicator Tests 4-2
4.2.1 Sieve Analysis for virgin pozzolans 4-2
4.2.2 Scanning Electron Microscopy (SEM) 4-4
4.2.3 Chemical Composition of Pozzolans 4-7

4.3 Pilot Laboratory Investigations 4-8
4.3.1 Statistical Analysis 4-9
4.3.2 Initial Lime Consumption (ICL) of Sand 4-10
4.3.3 Sieve Analysis 4-10
4.3.4 MDD and OMC 4-12
4.3.5 Tensile Test Methods Comparison 4-15
4.3.6 California Bearing Ratio 4-19
4.3.7 Compressive Strength 4-23
4.3.8 Tensile Strength 4-34
4.3.9 Comparison of the UCS and Tensile Strengths 4-44
4.3.10 Shrinkage Determination 4-49
4.3.11 Scanning Electron Microscopy (SEM) 4-56

4.4 Findings from the Pilot Study 4-58

CHAPTER 5: DISCUSSION OF THE MAIN STUDY RESULTS 5-1

5.1 Introduction 5-1

5.2 Indicator Tests 5-1
5.2.1 Sieve Analysis of Mixes 5-2
5.2.2 MDD and OMC 5-6

5.3 Compressive Strength 5-8

5.4 Scanning Electron Microscopy (SEM) 5-17

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS 6-1

6.1 Conclusions 6-1

6.2 Recommendations for Further Research 6-4

6.3 Recommendations for Construction of ALP mixes 6-6

LIST OF REFERENCES **7-1**

BIBLIOGRAPHY **7-8**

Appendix A: Chemical Analysis Results for Virgin Pozzolans A-1
Appendix B: SEM Images for Pilot Study Mixes B-1
Appendix C: SEM Images for Main Study Mixes C-1
Appendix D: Typical Arusha mix SAS input file (Pilot study) D-1
Appendix E: Summary of Pilot study SAS analysis E-1
Appendix F: Summary of Main study SAS analysis F-1

LIST OF TABLES

Table 3.1: The Pilot Experimental Mix Design Matrix 3-4
Table 3.2: The Main Experimental Mix Design Matrix 3-6
Table 4.1: Summary of Chemical Composition, Selected Components 4-7
Table 4.2: The Actual Degrees of Saturation Achieved 4-13
Table 4.3: Comparison between ITS and DPTT for Arusha pozzolan mix 4-18
Table 4.4: CBR results for the design mixes after 24 hours of curing 4-21
Table 4.5: CBR measured at 95% Mod AASHTO compaction 4-22
Table 4.6: The actual degrees of saturation for pilot study 4-23
Table 4.7: Summary of compressive strength results 4-25
Table 4.8: Summary of double punch tensile strengths results 4-35
Table 5.1: MDD and OMC for the main design mixes 5-7
Table 5.2: Compressive strength results 5-8
Table 5.3: UCS groupings for Arusha pozzolan mixes 5-11
Table 5.4: Density groupings for Arusha pozzolan mixes 5-12
Table 5.5: UCS groupings for Mbeya pozzolan mixes 5-14
Table 5.6: Density groupings for Mbeya pozzolan mixes 5-15

LIST OF FIGURES

Figure 4.1: Grain size distribution curve for virgin materials 4-3
Figure 4.2: Grain size distribution curve for mixes 4-11
Figure 4.3: Tensile strength test methods comparison for Arusha mix 4-18
Figure 4.4: UCS development for Arusha pozzolan mixes 4-26
Figure 4.5: UCS development for Mbeya pozzolan mixes 4-29
Figure 4.6: UCS development for Cement mixes 4-32
Figure 4.7: Tensile strength development for Arusha pozzolan mixes 4-36
Figure 4.8: Tensile strength development for Mbeya pozzolan mixes 4-39
Figure 4.9: Tensile strength development for Cement mixes 4-42
Figure 4.10: UCS and ITS comparison for Arusha mixes 4-45
Figure 4.11: UCS and ITS comparison for Mbeya mixes 4-46
Figure 4.12: UCS and ITS comparison for Cement mixes 4-47
Figure 4.13: UCS and ITS comparison for all mixes 4-48
Figure 4.14: 28day shrinkage/swell behaviour of Arusha pozzolan mixes 4-51
Figure 4.15: 28day shrinkage/swell behaviour of Mbeya pozzolan mixes 4-52
Figure 4.16: 28day shrinkage/swell behaviour of Cement mixes 4-53
Figure 4.17: 28day shrinkage/swell behaviour of all design mixes 4-54
Figure 5.1: Grain size distribution for virgin materials 5-3
Figure 5.2: Grain size distribution for washed sand mixes 5-4
Figure 5.3: Grain size distribution for unwashed sand mixes 5-5
Figure 5.4: UCS development for both Arusha mixes 5-10
Figure 5.5: UCS development for both Mbeya mixes 5-13
Figure 5.6: UCS development for both Cement mixes 5-16

LIST OF PLATES

Plate 2.1: Geological map of Tanzania 2-7
Plate 2.2: Index to the Geological map of Tanzania 2-8
Plate 2.3: Stratification of pozzolanic layers (Arusha region) 2-10
Plate 2.4: Stratification of pozzolanic layers (Mbeya region) 2-11
Plate 4.1: SEM: Arusha pozzolan, virgin state 4-4
Plate 4.2: SEM: Mbeya pozzolan, virgin state 4-5
Plate 4.3: SEM: Arusha pozzolan, mixed with lime (1:3 ratio) 4-6
Plate 4.4: SEM: Mbeya pozzolan, mixed with lime (1:3 ratio) 4-6
Plate 4.5: Positioning of the test specimen in ITS 4-16
Plate 4.6: Positioning of the specimen in DPTT 4-16
Plate 4.7: Two-plane failure of DPTT specimen 4-17
Plate 4.8: Three-plane failure of DPTT specimen 4-17
Plate 4.9: The Demec gauge with shrinkage specimen 4-49

LIST OF ABBREVIATIONS

AASHTO:	American Association of State Highway and Transportation Officials
ACI:	American Concrete Institute
ALP:	Aggregate-Lime-Pozzolan mixes
ASTM:	American Society of Testing Materials
BET:	Blaine Specific Surface Area
CANMET:	The Canada Centre for Mineral and Energy Technology
CBR:	California Bearing Ratio
COWI:	COWI Consulting Engineers and Planners A/S, Denmark
CSH:	Calcium Silica Hydrate gel (Tobermorite crystals)
DANIDA:	Danish International Development Assistance
DoT-SA:	South African Department of Transport
DPTT:	Double Punch Tensile Test method by Fang & Chang (1971)
GBFS:	Granulated Blast Furnace Slag
GM:	Grading Modulus
ICL:	Initial Consumption of Lime
ITDG:	Intermediate Technology Development Group, United Kingdom
ITS:	Indirect Tensile Test
MC:	Moisture Content
MDD:	Maximum Dry Density
Mod AASHTO:	Modified Proctor compaction effort using a 4.54kg hammer, falling though 457mm with 5layers each compacted by 55blows yielding a total energy of 0.68kWh/m^3
MOW:	Ministry of Works, Tanzania
N% Sat:	N percent saturation
NRB:	National Road Board compaction effort using a 4.54kg hammer, falling though 457mm with 5 layers each compacted by 25 blows yielding a total energy of 0.34kWh/m^3
OMC:	Optimum Moisture Content
P*n*:	Percentage smaller than the *n* mm sieve size
Proctor:	Standard Proctor compaction effort using a 2.49kg hammer, falling

	though 305mm with 3 layers each compacted by 55 blows yielding a total energy of 0.15kWh/m^3
PSI:	Present Serviceability Index
SABS:	South African Bureau of Standards
SAS:	Statistical Analysis Systems
SEM:	Scanning Electron Microscopy
SIDO:	Small Industries Development Organization, Tanzania
UCS:	Unconfined Compressive Strength
VOLCON:	A three-year project started in 1998 that includes investigation of volcanic rocks as aggregate and in building blocks of specialized uses, in the countries of Kenya, Tanzania, Ecuador, Costa Rica and Montserrat.

CHAPTER 1

INTRODUCTION

INDEX

1.1 Background 1-1
1.2 Problem Statement 1-3
1.3 Objectives of the Research 1-4
1.4 Scope of the Study 1-5
1.5 Organization of the Dissertation 1-6

CHAPTER 1

INTRODUCTION

1.1. Background

The current trends in the road transport sector show a growth in axle loads as well as vehicle numbers on all types of roads in highly industrialized countries as well as in developing countries. These trends are natural and important for economic and social development of the countries but they also call for an increase in bearing capacity of the existing roads. It should be noted that roads in most countries around the world were constructed during the post-World War II period following a boom in industrialization and trade. These roads were constructed based on the old standard specifications that catered for lower axle load limits as well as the number of axles that the roads were to carry throughout their design life. However, the rapid increase in trade caused by globalisation, resulted in a rapid increase in the number of axles and axle load mass at a time when the existing road networks are in the need for rehabilitation.

The increase in the axle loads and number has forced road agencies to amend their design standards adopting designs that provide roads with higher load bearing capacity. This solution would have been feasible if the availability of funds was certain and satisfactory. However, this is not the case in most developing countries that are running out of funds for road construction and maintenance. More cost effective solutions must be found.

Besides the savings in cost, another factor that made road agencies around the world opt for the use of marginal materials as well as utilization of various materials that were initially considered as unsuitable for road industry is the rapid depletion of the high-quality natural resources (Natt & Joshi, 1984). Strict environmental conservation laws have also made it impossible to continue with the old practices of road construction.

Currently, research is conducted worldwide to find road construction materials that will be both economically feasible as well as environmentally friendly. The use of in-situ and non-industrial materials in road construction was observed to be the best solution especially by including various stabilization techniques and chemicals. These stabilization methods have proved to be useful and economical in improving the quality of materials initially regarded as being marginal, despite the high costs associated with cement and other chemical stabilizers.

The environmental influences such as pollution in the manufacture of cement, and an excess of industrial by-products such as fly ash from coal fired power stations and granulated blast furnace slag from the steel industry led to many agencies charged with the responsibility of designing and constructing highways to start using by-product pozzolanic materials (Sharpe *et al*, 1985). These commonly regarded as low-strength binders (pozzolanic materials) have been used fairly extensively as stabilizing agents around the world especially in the USA, France, Turkey, South Africa and England where utilization of industrial by-products such as granulated blast furnace slag and fly ash has been observed.

Tanzania, like other developing countries, is faced with the urgency to rehabilitate its paved road network. However, uncertainty in availability of funds for conventional pavements led to the country to start investigating alternative solutions for road construction materials. Utilization of pozzolanic binders in Tanzania was achieved in the recently finished Dar es Salaam - Mlandizi road upgrading project. Here, gypsum activated granulated blast furnace slag (GBFS) imported from France was used to stabilize natural sand base course (Eriksen, 2000). The experience from the project was a 20% saving in the total construction cost (Eriksen *et al*, 1999) and much slower setting and hardening times with a reduction in shrinkage implying longer working times in the fresh state and reduced risks of later reflective cracking (Eriksen & Larsen, 2000).

The cost saving was achieved due to:

- Reduction of the total pavement thickness as compared to the conventional crushed stone base,

- Reduction in construction traffic associated with haulage of crushed stone from Lugoba quarry situated approximately 100 km from the site, and
- Reduction in environmental pollution if the conventional cement stabilization option was adopted.

After the successes obtained from the use of pozzolanic binders in the Dar es Salaam - Mlandizi road project, the Royal Danish Embassy in Tanzania through the Danish International Development Assistance (DANIDA) in co-operation with the Tanzanian Ministry of Works (MOW) contracted a private consulting firm, COWI Tanzania to perform a pilot study on the possible use of naturally occurring pozzolanic materials. The study was supposed to base its research on the volcanic ash that is abundantly available in many parts of the country. The study took place between June 1999 and May 2000 with the aim of (COWI, 2000):

- Identifying sources of natural pozzolanic material in Tanzania and selection of local aggregate for the pilot study,
- Laboratory tests on materials and mixtures, and
- Evaluating the expected use and possible limitations for the application of natural pozzolans binder in road construction in Tanzania.

This current research project is conducted following the pilot study for pozzolan use in road construction done by COWI (2000), and the experience from the Dar es Salaam - Mlandizi road project (Eriksen, 2000).

1.2. Problem Statement

The use of pozzolanic binders in stabilization as it is and other cementitious stabilizing materials in road pavements normally results in cracks forming in the layer that, depending on the thickness of the wearing course, eventually reflects to the surface. These cracks can lead to the ingress of moisture and atmospheric air that contain carbon dioxide (CO_2) and other gases possibly leading to rapid deterioration of pavement sub layers. In the Dar es Salaam - Mlandizi road project, cracks have been observed in places to reflect through the 100 mm asphalt concrete wearing course within a period of 2 years after construction. The study will evaluate the moisture related shrinkage properties of ALP mixes to determine its influence in the cracking of natural pozzolan-stabilized pavements.

Although these pozzolanic binders are economically feasible in road construction, it was found that using such materials can result in failures such as premature cracking and breakdown of the stabilized layers (Hoffman *et al*, 1976). Most of these failures have been related to factors such as poor construction techniques, slow strength development of pozzolanic materials and moulding moisture content that in turn influence the maximum dry density (Chikwira, 1991).

With the existence of strict construction standards and quality control measures and availability of various techniques to activate the strength development of pozzolanic binders, moulding moisture content is still believed to have the greatest influence on the performance of soils stabilized by pozzolanic binders.

1.3. Objectives of Research

The objective of this research is to investigate the influence of degree of saturation on the strength and shrinkage characteristics of different aggregate-lime-pozzolan mixes prepared from washed and unwashed river sands, and comparison will be made with the control specimens made from conventional cement stabilization. The degree of saturation was used in the study since the optimum moisture contents for sandy materials are not clearly defined.

Further objective of this study is to compare an alternative tensile strength testing method, called Double Punch Tensile Test (DPTT) with the commonly used indirect tensile test (ITS) to determine its possibility for use in tensile strength determination in practice.

Based on the findings of this study, recommendations for construction of ALP mixes in practice will be proposed.

1.4. Scope of Study

This research is part of the on-going study to evaluate the possibility of utilizing pozzolan of volcanic origin obtained in many parts of Tanzania as a stabilizing agent in road construction. It is thus essential to perform a thorough literature review to study all available relevant information concerning the characteristics of materials stabilized by pozzolans as well as factors that affect these characteristics. This will also include reviewing the performance of these pozzolanic materials in other uses such as in the cement industry to find their properties as binders in the cement and concrete industry.

This was followed by a series of laboratory tests during which compressive and tensile strength behaviour of mixes made up of different types of sand, lime and pozzolan was studied, and compared with cement-stabilized control samples.

Finally, conclusions are drawn from the study and recommendations for further research on aspects regarding the utilization of the pozzolanic material of volcanic origin obtained in Tanzania in the road industry.

In this study, an emphasis has been placed on evaluating the effects of degrees of saturation on the performance of laboratory prepared aggregate-lime-pozzolan mixes. Furthermore, the effects of sand types with varying fines content will be evaluated at varying degrees of saturation for mixes made from pozzolans.

The influence of other properties such as pozzolan-lime ratio and lime distribution have not been taken into account due to the availability of results from other studies performed on the subject and also due to the time and financial constraints of this study.

1.5. Structure of Dissertation

This dissertation is structured as follows:

- **Chapter 1: Introduction**

  This chapter provides a brief history on uses of marginal materials in the road construction industry by stabilizing them with various affordable materials such as industrial wastes, natural cementitious materials and other economic stabilizing agents. The objective of the research, problem statement and scope of the study are given in this chapter.

- **Chapter 2: Aggregate-Lime-Pozzolans: Literature Review**

  This chapter gives the information on pozzolans including the definition, types, properties, reactivity and uses of pozzolans. Details regarding the natural pozzolans in Tanzania are also given in this chapter.

  Further, a brief definition, composition, background and uses of the Aggregate-Lime-Pozzolan (ALP) mixes are presented. Brief information on the engineering properties as well as behaviour of ALP under various physical and chemical conditions is given in this chapter.

- **Chapter 3: Experimental Program and Test Procedures**

  This chapter outlines the methodology followed in the study, types of laboratory test standards and procedures adopted, and moulding and curing conditions to be adhered to during the laboratory experiments.

- **Chapter 4: Discussion of the Pilot Study Results**

  This chapter gives a description of the pilot experimental strength investigation. Here, pilot study for determination of the strength - degree of saturation relationship over time was investigated to determine the curing duration required in the study.

This chapter also presents the discussion of the shrinkage, CBR and SEM observation results from the pilot laboratory investigations.

- **Chapter 5: Discussion of the Main Study Results**

  This chapter gives a description of the work that was done in the main experimental section to evaluate the findings of the laboratory study. The behaviour of the ALP mixes made from different sand types was investigated and compared with that of cement mixes to determine their potential in the pavement stabilization process.

- **Chapter 6: Conclusions and Recommendations**

  This chapter presents the conclusions derived from the experimental investigations. Finally the recommendations for further studies on the subject will be provided in this section.

CHAPTER 2

AGGREGATE-LIME-POZZOLANS: LITERATURE REVIEW

INDEX

2.1 Introduction 2-1
2.1.1 Definition of Pozzolans 2-1
2.1.2 Background 2-2
2.1.3 Types of Pozzolans 2-3
2.2 Pozzolan Activity 2-3
2.3 Uses of Pozzolans 2-5
2.4 Natural Pozzolans in Tanzania 2-6
2.4.1 Pozzolans from Oldonyo Sambu (Arusha region) 2-9
2.4.2 Pozzolans from Mbeya region 2-11
2.4.3 Pozzolans from Kilimanjaro region 2-12
2.4.4 Chemical Composition 2-12
2.4.5 Lime-Pozzolan Reactivity 2-12
2.5 Engineering Properties of ALP 2-13
2.5.1 California Bearing Ratio (CBR) 2-15
2.5.2 Strength Development 2-15
2.5.3 Durability 2.17
2.5.4 Strength Characteristics of Tanzanian Pozzolans 2-20
2.6 Concluding Remarks on Tanzanian Pozzolans 2-21
2.7 Test Methods for Evaluation of Stabilised Soils 2-22
2.7.1 Moulding Moisture Contents 2-22
2.7.2 Compressive Strength Determination 2-22
2.7.3 Tensile Strength Determination 2-23
2.7.4 Shrinkage Determination 2-25
2.8 Guidance Gained from the Literature Review for this Study 2-26

CHAPTER 2

AGGREGATE-LIME-POZZOLANS: LITERATURE REVIEW

2.1. Introduction

This chapter gives a brief overview regarding ALP mixes. A literature review on the definition and types of pozzolanic materials and background information on their use in the construction industry will be briefly presented. Brief overview of ALP mixes will also be given in terms of their engineering properties. A review of various test methods for evaluation of engineering properties of stabilized materials in practice will be performed to determine the method to be used in this study.

In this study emphasis is on the natural pozzolans occurring in Tanzania since it is based on the Tanzanian experience with pozzolans and the ongoing efforts for its development.

2.1.1. Definition of Pozzolans

Pozzolan is defined as "pyroclastic rocks, essentially glassy and sometimes zeolitised, which occurs either in the neighbourhood of Pozzouli (the ancient Puteoli of the Romans times) or around Rome" (Hewlett, 1998).

Pozzolan also is defined as "all those inorganic materials, either natural or artificial, which harden in water when mixed with calcium hydroxide (lime) or with materials that can release calcium hydroxide such as Portland cement clinker" (Hewlett, 1998).

According to the American Concrete Institute, pozzolan is defined as "either raw or calcined natural material that has pozzolanic properties (e.g. volcanic ash or pumicite, opaline, chert and shales and some diatomaceous earths)" (ACI Committee 232, 1994).

ACI also defines pozzolan as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties" (ACI Committee 232, 1994).

Originally, the term pozzolana applied to the incoherent pyroclastic-sialitic rocks occurring in the neighbourhood of Pozzouli, and has subsequently been extended to include a wide range of both natural and inorganic materials differing in nature, composition and structure (Costa & Baroni, 1994).

However, within the cement manufacturing process the term has eventually acquired a specific technical meaning and is generally attributed to materials which, at ordinary temperatures, are capable of:

- Reacting with calcium hydroxide in the presence of water;
- Forming hydrated products with binding properties (Costa & Baroni, 1994).

2.1.2. Background

The cementitious properties of volcanic ash, lime and water were recognized over two thousand years ago by Roman and Chinese builders (Fairweather, 1975). Initially, ancient Romans and Chinese used volcanic ash in stabilizing road bases; this has been utilized in countries where volcanic ash is plentiful. However, countries like USA, France and England have been using the same technology to stabilize their road bases by using fly ash, a waste product of coal-burning power plants, and the industrial counterpart of volcanic ash for the past 15 to 25 years (Fairweather, 1975).

The Greeks and Romans used calcined limestone and later developed pozzolanic cements by grinding together lime and volcanic ash (Sharpe *et al*, 1985).

2.1.3. Types of Pozzolans

According to Lea's Chemistry of Cement and Concrete, pozzolans are classified based in their origin (Hewlett, 1998) namely natural pozzolans and artificial pozzolans. Natural pozzolans represent all those naturally occurring materials such as volcanic ash, diatomaceous earth, shales, opaline chert, pumicite and tuffs. Artificial pozzolan represent all those pozzolans formed from processing of materials originally without pozzolanic properties such as fly ash, blast furnace slag, burned clay or shale, micro silica and silica fumes. Furthermore, Lea classified natural pozzolans into three groups based on their origin, namely volcanic origin, sedimentary origin and those of mixed origin.

Although there are many methods of classification of pozzolans currently in practice, pozzolans may differ in the following respects (Philleo, 1989):

- Composition (chemical, mineralogical, physical),
- Geographical distribution,
- Amount of processing required,
- Properties (cementitious, pozzolanic),
- Economics,
- Methods of use, and
- Specification requirements.

2.2. Pozzolan Activity

A number of parameters are significant in evaluating the activation of pozzolan with lime, cements or any other material that constitute or generate calcium hydroxide to develop strength.

The term "pozzolanic activity" covers all reactions occurring among the active constituents of pozzolan, lime and water (Hewlett, 1998). The process of pozzolanic reaction is commonly evaluated in terms of diminution of free lime in the system or increase in the silica + alumina soluble in acids by using the Florentin attack method (Price, 1996). The term 'pozzolanic activity' includes two parameters, namely the maximum amount of lime that a pozzolan can combine with and the rate at which such combination occurs (Hewlett, 1998).

There is general agreement that the overall amount of combined lime essentially depends on the following (Hewlett, 1998):

- The nature of the active phases;
- Their content in pozzolana;
- Their silica content
- Their lime/pozzolana ratio in the mix;
- Length of curing;

Whereas the combination rate also depends on

- The specific surface area (BET) of pozzolana;
- Water/solid mix ratio;
- Temperature.

The above properties are essential in the performance of pozzolan-lime mixes. Each property has an effect on the durability and strength of the resulting mix.

2.3. Uses of Pozzolans

There is evidence that use of pozzolan in the construction industry is reported to date back to the 1500-2000 B.C with the Minoan structures of Crete Island, which contained potsherds (i.e., calcined clay) in a lime mortar (Lea, 1971).

Pozzolans are in worldwide use as cement replacement material due to the fact that they improve the durability of cement, reduce the cement production costs and also reduces the environmental degradation associated with cement production as well as disposal of various industrial wastes effectively. Common pozzolanic materials that have been utilized are fly ash, sludge and in some countries, natural pozzolans. Fly ash is in wide use whereas the addition of natural pozzolans of volcanic origin is used to a lesser extent.

The use of blast furnace slag (GBFS) in road bases and sub base stabilization has been observed in recent years. However, GBFS is widely confused for pozzolans. It should be noted that GBFS requires calcium ions activator (Ca^{++}) to hydrate compared with pozzolans that requires high pH environment to hydrate.

Depending on the type, fineness and amount of pozzolan addition, concrete properties can be tailored to required strength (Rols *et al*, 1999) and durability (Mehta, 1997).

Pozzolans are also greatly used in the road construction industry as a stabiliser in the construction of ALP mix pavements. This mix has successfully been used for road bases and shoulders in various parts of America especially in Pennsylvania (Hoffman *et al*, 1976). ALP mixes, in comparison with crushed stone aggregate, provides a stiff base that considerably reduce rutting. Furthermore, the PSI-value decreased much slower in the ALP pavement, and cracks developed much earlier and propagated much faster in the crushed-stone pavement (Wang & Kilareski, 1979).

Currently, the type of pozzolan used in ALP mixes has been restricted to availability, with fly ash being common in countries such as the US, France and South Africa while countries like Italy and Turkey are using natural pozzolans of volcanic ash origin.

Although pozzolan stabilized pavements give good performance, they are associated with failures requiring further research of the material. These failures have been observed in many states in the US such as in Pennsylvania where premature failures due to frost were observed to be associated with the construction cut-off date prior to the winter season (Hoffman *et al*, 1976). In Tanzania where granulated blast-furnace slag was used, excessive longitudinal cracking was observed. The cracks were considered to be associated with shrinkage of the material due to moisture variation in the pavement and partly due to inadequate compaction during construction. Thus it can be concluded that although ALP have shown a promising future in road industry, more research should be done to evaluate their performance behaviour under different environmental and traffic conditions.

2.4. Natural Pozzolans in Tanzania

The literature on pozzolans from various research institutions in Tanzania and other local contributors concentrates on the occurrence of natural pozzolans in two major areas surrounding the volcanic mountains (COWI, 2000), i.e. Mounts Kilimanjaro, Oldonyo Lengai and Meru in the north-eastern part of the country (Arusha and Kilimanjaro regions) as well as Mounts Rungwe and Ngozi in the southern highlands (Mbeya region). Plates 2.1 and 2.2 give the geological map of Tanzania and index to the geological map of Tanzania respectively.

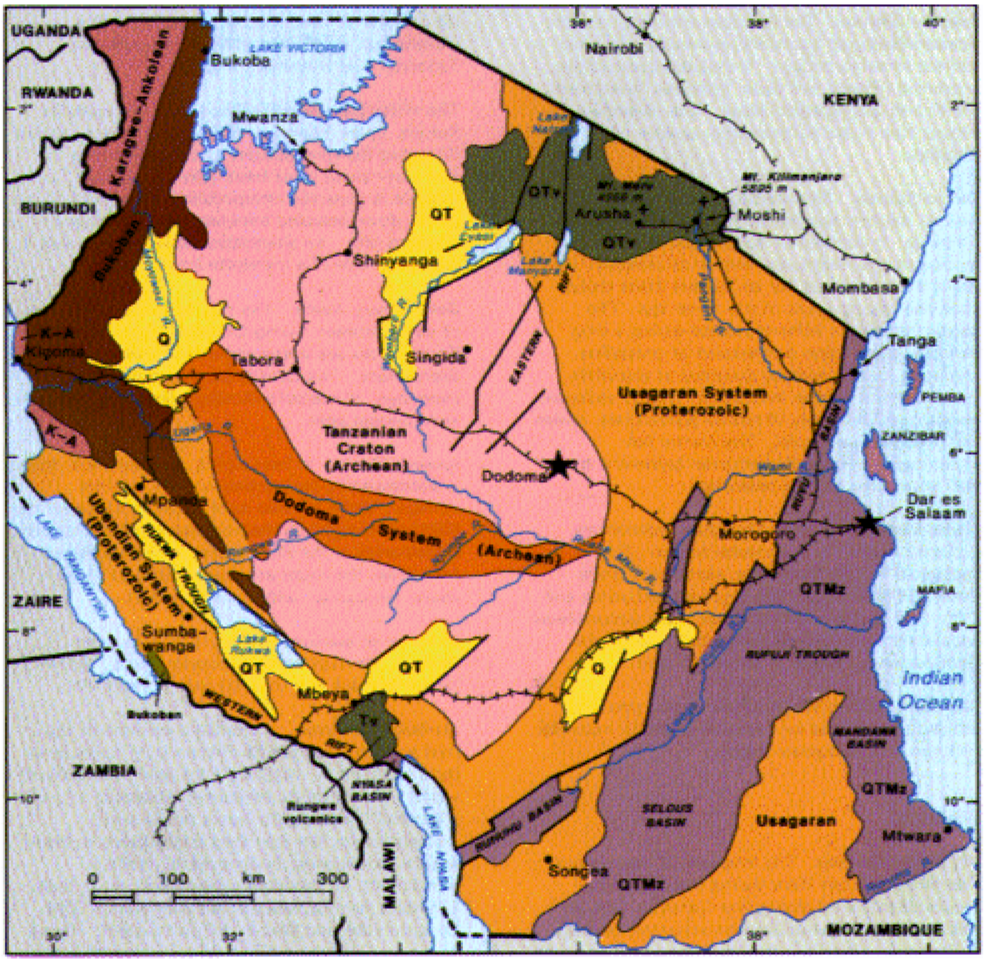


Plate 2.1: Geological map of Tanzania (http://tanzania.sgu.se)

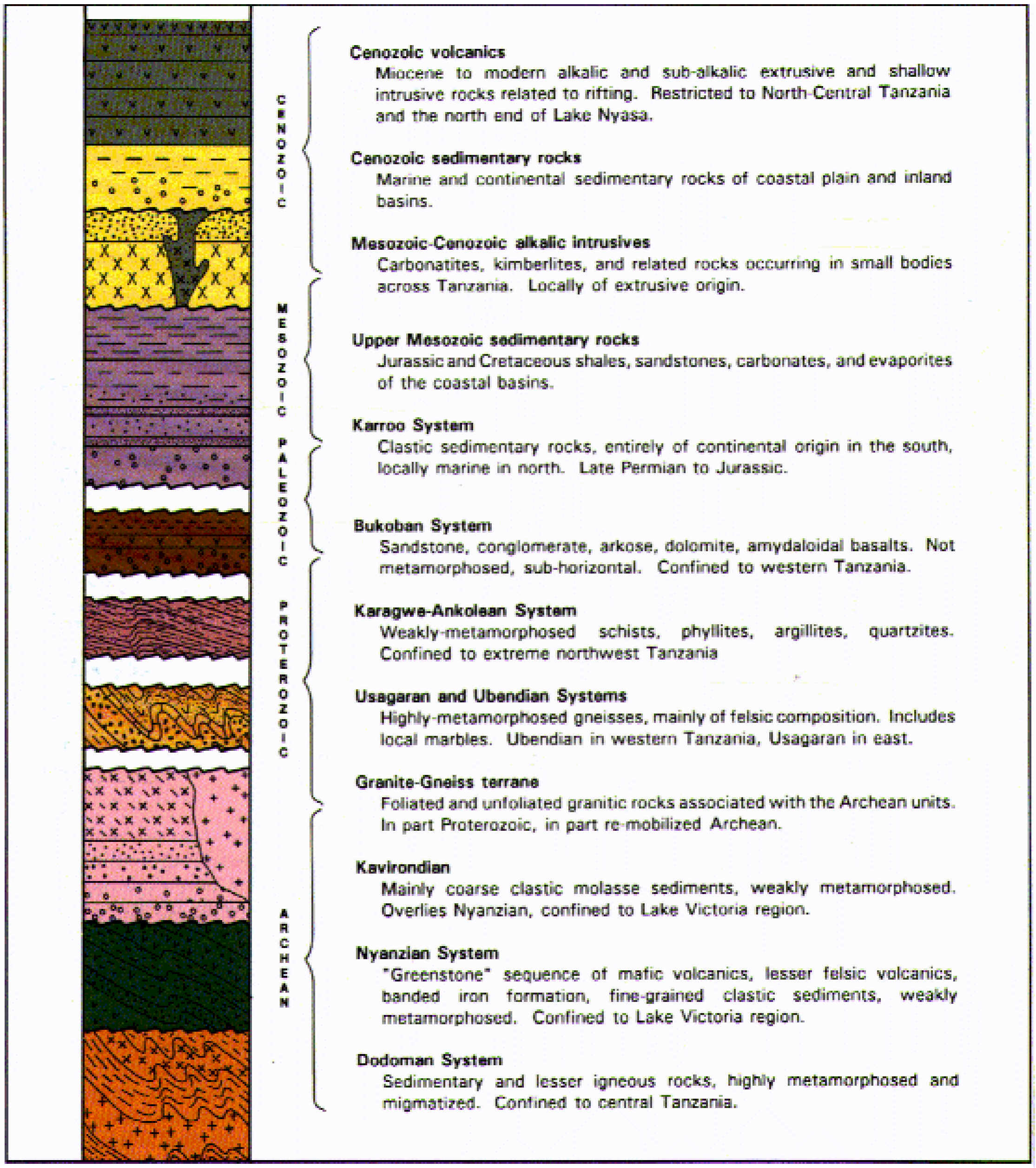


Plate 2.2: Index to the Geological map of Tanzania (http://tanzania.sgu.se)

The use of these pozzolans dates back to 1970's when the Small Industries Development Organisation (SIDO) established a lime-pozzolan project with expert assistance from the Intermediate Technology Development Group (ITDG). The aim of this project was to start up small industries producing "pozzolime" a (dry) mixture of hydrated lime and pozzolan to be used as alternative to Portland cement for low strength building purposes. Local production of lime was part of the project.

The project was set up in the Oldonyo Sambu area in the Arusha region along the Nairobi Road and it started production in 1979. However, production ceased a few years after handing over of the project to the Government of Tanzania in the early 1980's. A demonstration building that was constructed using blocks and pozzolime binder still exists in Oldonyo Sambu with some blocks showing some signs of deterioration while most are still intact.

As part of the SIDO-project pre-investigations for initiating a similar pozzolime production project in Mbeya region took place in the late 1970's (Sakula & Sauni, undated). The project to produce pozzolime was not started. Initial studies had indicated that a number of pozzolan sources in the region were identified and their reactivity indicated (observed to be about half the level for pozzolans found in Oldonyo Sambu area in Arusha region).

Since the early 1980's no utilization of pozzolans in combination with lime on an industrial level seems to have taken place in Tanzania. Currently, industrial utilization of natural pozzolan in the country is in cement manufacturing (i.e., Portland pozzolana cement manufacturing by Mbeya Cement Company in the Mbeya region).

The Government of the United Republic of Tanzania and DANIDA contracted COWI Tanzania to conduct a study to determine the feasibility of using volcanic ash in road construction. The study started in 1999, where a pilot study was conducted to determine the engineering properties of pozzolans obtained from various sources in Tanzania. Samples were collected from Arusha, Kilimanjaro and Mbeya regions where the literature indicated that the sources contained high activity pozzolans (COWI, 2000).

The results of the study are discussed in the next subsections.

2.4.1. Pozzolans from Oldonyo Sambu (Arusha region)

Oldonyo Sambu is a village located approximately 35km from Arusha town along the Arusha - Nairobi (Kenya) highway. The area is covered by a layer of mantling ash or tuff, with several volcanic cones placed in the vicinity of the area. The area consists of rolling barren land, which is known to be dusty during the dry season and flooded in rainy season. Vegetation consists mainly of scattered shrubs, trees and grass.

During the pilot study for possible use of pozzolans (COWI, 2000), a total of six samples of pozzolans were taken for laboratory investigations. The samples were found to originally exist in two forms, either fine grained volcanic ash or whitish pebble pumice. The two pozzolans from Arusha region were generally found to occur in two layers with the upper three meters being finer volcanic ash overlaying a layer of whitish pebble pumice (plate 2.3).

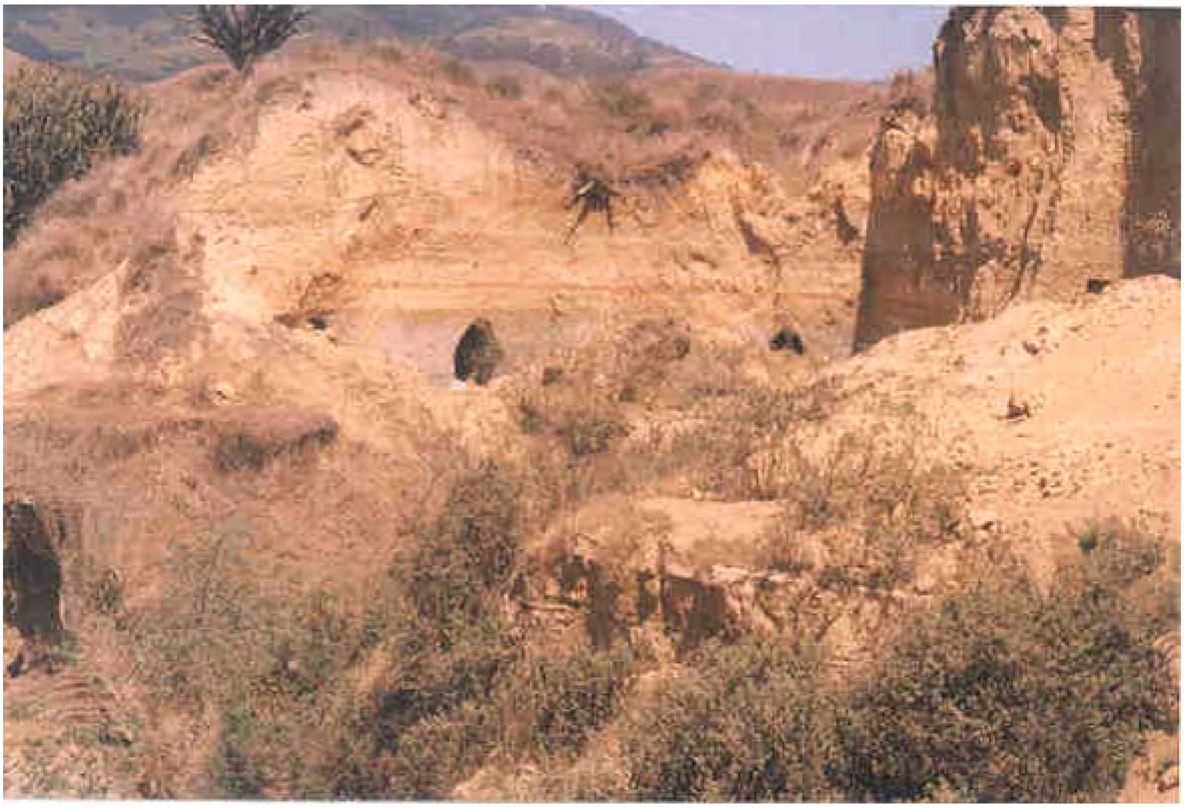


Plate 2.3: Stratification of pozzolanic layers (Arusha region)

Although only pozzolans from Oldonyo Sambu were tested during the pilot study (COWI, 2000), pozzolan deposits are recorded to exist in other parts of the region especially in areas around Lengijave and Kisongo. The other pozzolan deposits will need further investigation to determine their reactivity as compared to the Oldonyo Sambu deposits.

2.4.2. Pozzolans from Mbeya region

During the pilot study for possible use of pozzolans (COWI, 2000), samples were collected from various known sources around Mbeya region. The samples were collected between Songwe about 30km southwest of Mbeya town and Tukuyu, about 80km southeast of Mbeya town. Collected samples were mostly pumice that consisted of hard-cemented lumps and fine-grained volcanic ash (plate 2.4).

Plate 2.4: Stratification of pozzolanic layers (Mbeya region)

Currently, commercial utilization of pozzolan materials in Mbeya region is known to be restricted to cement manufacturing industry (i.e., Mbeya Cement Company that manufactures Portland pozzolan cement).

2.4.3. Pozzolans from Kilimanjaro region

During the pilot study for possible use of pozzolans (COWI, 2000), five samples were collected in the Kilimanjaro region. Three samples were obtained from Holili and two from Uchira (near Himo/Moshi) areas. Kilimanjaro pozzolans were observed to be very hard, rock-like material that will need blasting for extraction and heavy crushing to make them suitable for stabilization.

Currently no information is available regarding the commercial utilization of pozzolanic materials in the Kilimanjaro region.

2.4.4. Chemical Composition

Chemical analysis showed distinct differences among the three regions where samples were taken, leading to a different classification of the pozzolanic materials based on mineralogy. Referring to the classification after Best, used by British Geological Survey in the VOLCON project; the following applies to the investigated pozzolans (COWI, 2000):

- Mbeya pozzolans are classified as Intermediate (52 – 66% SiO_2),
- Oldonyo Sambu pozzolans are classified as Basic (42 - 52% SiO_2), and
- Kilimanjaro pozzolans are classified as Ultra basic (<42% SiO_2).

2.4.5. Lime-Pozzolan Reactivity

A microscopic investigation was done to determine the reactivity's of the pozzolan samples. The following conclusions were made (COWI, 2000):

- Aluminosilicate glass phase in the pozzolan is the main reactive phase with lime.
- The more weathered (no or low alkali content) and fine-grained/porous the aluminosilicate phase, the more reactive it is with lime forming mainly calcium

aluminates (C_4AH_{13}), possibly gehlenite (C_2ASH_8) and calcium silicates (CSH-gel). Calcium ratios between 1:1 and 3:1 in CSH were observed (ratio near 2:1 was the most frequent).

- Aluminosilicate glass phase (not weathered and containing alkalies, e.g. pumice of various types) seems to have a slower reaction rate but results in similar products as above.

- Calcium, iron, magnesium, titanium and other metals found in crystalline particles (with silicium and/or aluminium in plagioclase, augite, biotite and olivine). These crystalline particles were observed not to participate in the reactions with lime.

Most pozzolans from Oldonyo Sambu and Mbeya were found to form similar reaction products of expected composition, however, in varying amounts. These pozzolans were recommended for further investigation as useful in road construction.

Some pozzolans from Mbeya regions did not show any significant amounts of reaction and were concluded as not being pozzolanic. However, these samples were observed to contain glass phase material of similar composition as the other pozzolan samples from the area.

Pozzolans from Kilimanjaro region were observed to contain only small amounts of material expected to be pozzolanic (glass phase) and has a high content of crystalline particles (not pozzolanic). No significant reaction products with lime were detected. These pozzolans were also observed to differ from other pozzolans from Oldonyo Sambu and Mbeya in composition as well as its content of ferrite and calcium rich glass phase. Reaction products (CSH-phase) have, however, been detected in these pozzolans.

2.5. Engineering Properties of ALP

Road building materials are considered suitable if they can be used to make roads that can withstand all loads occurring routinely during the scheduled period of utilization without any environmental repercussions. The following concepts have been proposed for a good road building material (Schmidt & Vogel, 1990):

- The strength of the material must be sufficient to withstand all external and internal loadings occurring during use without any premature fatigue,
- The material must give adequate resistance to influences from the weather or the atmosphere, so that no damage is caused by moisture, frost or thawing salts,
- The material must have constancy of volume, i.e., it must not swell or expand to an unacceptable extent when penetrated by moisture, and it must not shrink excessively when dried out. Moreover, it must not contain any constituents which prevent or impair hardening in the event of being used in cement-bound building materials, or which react with bituminous or hydraulic building agents in a fresh or hardened condition that could lead to alterations in volume, and
- Finally, the material - whether in a stabilized or un-stabilized form must have a sufficient workability.

Generally, the performance of stabilized layers is evaluated by considering factors such as durability (Andres *et al*, 1976), strength (Cumberledge *et al*, 1976), volume stability and workability (Higgins *et al*, 1998). Studies have shown that the performance of ALP is influenced by:

- Aggregate gradation,
- Lime plus fly ash content,
- Ratio of lime plus fly ash to total fines,
- Curing conditions and time,
- Fly ash content, and
- Moulding moisture content.

The above factors were observed to greatly influence the compressive strength, tensile strength, California bearing ratio, freeze-thaw and shrinkage properties of the ALP mixes.

2.5.1. California Bearing Ratio (CBR)

The main criterion for any stabilized layer during its fresh state is that a CBR-value (unsoaked, without surcharge) of at least 50% is obtained (French Highways Directorate, 1997). This value should be fulfilled based on the type of material used as a stabilizer. In cases where Portland cement is used, CBR can be measured a few hours after mixing.

According to Yoder & Witczak (1975), the principal soil factors affecting the CBR are soil texture, moisture and density. To obtain an indicative CBR value for lime-pozzolan stabilized layers, tests should be done after allowing time for initial reaction to take place due to the slow reaction rates observed with pozzolanic binders.

In practice, the CBR test has been used as a basic criterion in the determination of fresh state bearing capacity of any soil to be used in road construction (Nicholson *et al*, 1994).

2.5.2. Strength Development

The compressive strength of stabilized materials is commonly considered their most valuable property (Natt & Joshi, 1984). Two characteristics are used to evaluate stabilized soils, compressive and tensile strengths. However, compressive strength testing has been preferred in practice over tensile strength testing due to difficulties associated with tensile strength testing (Natt & Joshi, 1984). However, mechanistic analyses have shown that a stabilized layer usually fails in tension (fatigue) at the bottom of the layer (DoT-SA, 1997).

The use of ALP layers in roads as a substitute for Portland cement or lime has resulted in a reduction of early strengths in samples up to 28 days old. However, higher strengths have been recorded for ALP mixes compared with Portland cement and lime stabilized layers for periods beyond 28 days.

Strength development of ALP has been observed to depend on the following factors:

- Dry Density and Moulding Moisture Content

The compressive strength of stabilized materials is highly influenced by moulding moisture content and dry density. Research has shown that the densest material requires the least water to prepare and was found to have the highest compressive strength. Similarly, the least dense material tends to have the highest water intake resulting in the lowest compressive strength. The strength of fly ash stabilization can be related to the amount of water needed to perform a standard Proctor test and is dependent on the type of fly ash used (Fraay *et al*, 1989). Literature has also indicated that a higher water content of the mix results in a higher rate of lime combination (Hewlett, 1998). The above statements have brought about the confusion regarding the quantity of water needed for pozzolanic reaction.

- Curing

It has been observed that curing plays an important role in strength development of any stabilized material (Vandewalle & Mortelmans, 1992). Longer curing times and curing type results in higher strengths when compared with shorter curing times for the same mix. ALP mixes are mostly affected by curing due to their slow strength gain as compared to conventional stabilizing techniques such as cement or lime. Curing also influences the permeability of pozzolanic mixes that reflects in the durability of the structure (Saricimen *et al*, 1992).

- Fines Content

The reactivity of pozzolan is greatly influenced by the fines content of the reacting particles, i.e. pozzolan and lime. It is expected that the fineness of lime-pozzolan cement, and particularly the fineness of the natural pozzolan used, will have a significant effect on early strength development (Day & Shi, 1994).

- pH Value of the pore water

Research has shown that an increase in the compressive strength of aggregate-lime-pozzolan mix starts at a pH value of 12.8 (Fraay *et al*, 1989). An increase in pH above this value has been observed to influence the rate of reaction resulting in strength increase. Various methods of increasing the pH of the mix have been used in research to determine their influences. It was however, observed that a limiting concentration is essential for the strength development of the mixes. It was found that the addition of an alkaline solution (NaOH was used in this study) should be within the range not exceeding 5% (Fraay *et al*, 1989).

- Testing conditions

Research has shown that for pozzolanic stabilized aggregate samples, water immersion before testing resulted in a reduction of the compressive and tensile strengths. The effect was observed when test specimens were immersed in water 24 hours before testing. Immersion doubled the water content of the specimens to 9% and decreased strength to 40% of the non-immersed specimens (Pylkkanen, 1995).

- Initial Consumption of Lime (ICL)

The amount of lime required to satisfy the ICL will be sufficient to permit modification to take place completely but little if any stabiliser will be available for cementation or pozzolanic action (Ballantine & Rossouw, 1989). The lime required to produce any strength in the long term must be in addition to that required to satisfy the ICL.

2.5.3. Durability

Durability of any stabilization material is an important factor in determining their suitability for use. The road materials are supposed to have a great resistance against the aggressive environmental and traffic conditions. Materials that are not durable have been associated with a shorter service life that leads to over-spending of funds for premature maintenance of roads. Studies have shown that in order to gain sufficient durability of cement bound road bases it is recommended to increase the required compressive strength, which will require in a higher cement content (Fleischer *et al*, 1994).

In the case of aggregate-lime-pozzolan mixes, it has been found that the most significant property in terms of pavement performance is durability (Andres *et al*, 1976). Durability of aggregate-lime-pozzolan mixes has been observed to be greatly influenced by shrinkage in their fresh state, permeability in their finished state (Saricimen *et al*, 1992) and frost action (Andres *et al*, 1976).

Durability can be determined by the mass loss exhibited during 12 standard freeze-thaw cycles, during which specimens are subjected to uniaxial temperature gradients similar to those measured in in-service pavements (Andres, 1976). The durability of stabilized materials can be evaluated by the compressive strength loss after five freeze-thaw cycles in the test unit (Dempsey, 1972). Shrinkage, frost resistance and permeability tests have been used successfully to evaluate the durability of stabilized layers. For African conditions, frost resistance test is not of importance due to the climatic condition also due to the fact that overlying layers that inhibit ingress of water and air always protect underlying stabilised layers.

- <u>Shrinkage</u>

Shrinkage of base and sub-base materials is one of the important factors contributing to the pavement cracking especially when stabilized. Infiltration of water through these cracks further damages the base leading to premature failure of the pavement.

Research has shown that the type of material stabilized significantly affects the shrinkage characteristics (Nakayama & Handy, 1967). However, it has been observed that moisture content plays a major role in the shrinkage behaviour of stabilized materials. Moreover, research has shown that shrinkage in stabilized materials occurs during the first 28 days and thereafter little or no shrinkage has been observed (Natt & Joshi, 1984).

- Permeability

Permeability is an important physical property that directly relates to the durability of aggregate-lime-pozzolan mixes due to the fact that it will lead to the entry of gases, liquids and solutions into the layer, which can result in expansion and cracking. Permeability of pozzolanic pastes is initially higher than that of Portland cement ones, but it tends to become lower as the curing time increases (Hewlett, 1998). Thus durability of aggregate-lime-pozzolan mixes can be improved by reducing the permeability by increasing the curing time for up to 28 to 90days (Saricimen *et al*, 1992).

- Frost Action

The freeze-thaw property of stabilized materials is important in determining their durability for materials that are constructed in frost susceptible areas.

Research has shown that the addition of fly ash to cement stabilizers results in the ability of these materials to retain and regain compressive strengths under repeated cycles of freezing and thawing and subsequent curing (Natt & Joshi, 1984). This property explains the fact that pozzolanic stabilized materials are less susceptible to freeze-thaw provided that they are constructed timely and gain tensile strength before the beginning of winter as compared with other stabilization techniques (Hoffman *et al*, 1976). Failure during freezing occurs when inter particulate, cementitious bonds break under tensile stresses induced by freezing pore water or the formation of ice lenses or both.

Therefore, freeze-thaw effect of ALP's can be reduced by taking into account the construction termination dates based on the available air temperatures and other local conditions (Hoffman *et al*, 1976). Lea has proposed that frost resistance of concrete can be decreased by increasing the water/cement ratio and shortening the wet curing before exposure to freezing (Hewlett, 1998).

On the other hand, the fineness of pozzolan has been related to the reduction in freezing and thawing property of concrete due to reduction in entrained air. This implies that finer ground pozzolans impair freezing and thawing durability of concrete unless adequate air entrainment agents are incorporated in the mix (Mehta, 1987).

However, research has shown that the failure of aggregate-lime-pozzolan material in the field due to freeze-thaw action can be related to a tensile strength being insufficient to sustain the induced tensile strain produced by freeze-thaw action. Thus, an improvement in tensile strength properties of the material can result in a better resistance to freeze-thaw action (Cumberledge *et al*, 1976).

2.5.4. Strength Characteristics of Tanzanian Pozzolans

Detailed laboratory testing was conducted in pozzolan-lime-sand mixes to determine their strength characteristics. The tests included determination of their fresh state strength as well as in hardened mixtures. The following conclusions were made (COWI, 2000):

- Properties in the fresh state

The main criterion for the lime-pozzolan-sand mixtures in the fresh state is that a CBR-value of at least 50% is obtained (French Highways Directorate, 1997). All tested pozzolan mixtures fulfilled this criterion. CBR values during the study (COWI, 2000) were observed to range from 66 - 150% in specimens made from all design mix ratios.

The OMC is typically between 7 and 10%, and the MDD was found to vary between 1850 and 2040 kg/m^3. This variation is expected due to the variation in fines content of different pozzolans tested.

The maximum dry density variations are expected to have a significant influence on strength results besides those variations that may result from the various pozzolan's individual reactivity and fineness.

- <u>Strength testing</u>

During the pilot study (COWI, 2000), comparable mixtures were prepared with dry mass mix ratio of 4:12:84 lime-pozzolan-sand respectively, cured at 20^0C and stored in closed containers.

The strength results showed that all pozzolan samples from Oldonyo Sambu, Mbeya and Kilimanjaro (except for one sample each from Mbeya and Kilimanjaro) obtained compressive strengths at 20 to 60 days similar to or higher than the reference sand-slag mixture with 1% gypsum activator. The same was observed for split tensile strengths. Moreover, a further significant development of strengths from 60 to 90 days was noted as compared to sand-slag mix that showed a very limited increase in the same period.

In some mixtures, 1% dry mass of finely ground gypsum from a natural source was added for the purpose of determining its effects to ALP mixes. This has, however, not been found to have any effect on the strength characteristics of the lime-pozzolan-sand mixes.

2.6. Concluding Remarks on Tanzanian Pozzolans

The results reported in the pilot study (COWI, 2000) showed that several pozzolan occurrences in Tanzania more than fulfil the requirements for use as a binder for road construction purposes as well as other possible uses in the building industry.

A difference in fines content of the natural fine pozzolans versus the ground pumice types is a parameter influencing strength development. The nature of pozzolan was also observed to influence strength development.

Furthermore, it was noted that chemical composition or determination of glass content would not be sufficient to determine the reactivity of a pozzolan. This means that strength tests should be used to analyse pozzolan reactivity.

2.7. Test Methods for Evaluation of Stabilized Soils

Various methods are used in practice to determine strength and shrinkage characteristics of stabilized layers. Strength testing has been a commonly used means of determination of the quality and performance evaluation of stabilized layers. In practice, compressive and tensile strengths are used in evaluating the strength characteristics of stabilized and unstabilized soils. However, compressive strength is preferred to tensile strength due to the complications associated with tensile strength testing (Natt & Joshi, 1984).

2.7.1. Moulding Moisture Contents

From Savage & Visser (2001), the maximum dry density is influenced by the following factors:

- Size and shape of the mould,
- Mould support,
- Sample preparation,
- Type, magnitude and distribution of compaction effort
- Temperature,
- Layer thickness, and
- Degradation of particles.

The above factors indicate that the MDD from the reference test may vary significantly, even with constraints of the standard test procedures. Furthermore, the reference test may not be representative of field compaction conditions.
Any uncertainty in the MDD may influence the optimum moulding moisture content in practice.

2.7.2. Compressive Strength Determination

Generally, unconfined compressive strength (UCS) test is used in determination of compressive strength of stabilized materials in the laboratory. The compressive strength of the material is determined by subjecting cylindrical or cubical test specimens to an increasing load until failure.

2.7.3. Tensile Strength Determination

Tensile strength of stabilized and unstabilized materials is determined by double punch tensile (DPTT) test (Cumberledge *et al*, 1976), flexural strength tests (Natt & Joshi, 1984) and indirect tensile strength (ITS) tests. Currently, confusion exists among researchers on which method gives the most appropriate prediction of the tensile strength of a stabilized material. Available methods were evaluated as to the benefits of using them in this research program.

- <u>ITS (Splitting tests)</u>

This method covers the determination of the splitting tensile strength of cylindrical soil, gravel or crushed stone specimens such as moulded cylinders or drilled cores. This test method measures the resistance of a cylindrical prepared or cored specimen when a load is applied to the curved sides of the specimen (DoT-SA, 1986). The test is done by applying a diametrical compressive force on a cylindrical specimen placed with its axis horizontal between the platens of a testing machine.

This method has been used in tensile strength determination by many road agencies and researchers due to its simplicity and reliability (DoT-SA, 1986). The method however, has been pointed out to measure the value of tensile strength across a predetermined failure plane, resulting in measurements of a tensile strength that does not represent the true value (Cumberledge *et al*, 1976). Also in the method, tensile stress at failure is calculated on the false assumption that the specimen remains elastic up to failure (Thanikachalam, 1973). Further, the split-tensile test does not provide a loading condition to resemble that in the field, nor does it permit determination of tensile strain during loading (Wang & Huston, 1971).

Therefore, the measured strength is unlikely to be a direct function of tensile strength but may be a complex function of the strength and size of the test specimen (Hannant, 1972).

- Flexural tests (beam tests)

This test is done so as to give the flexural strength of the material in tension. This test is done on laboratory cast prisms of 100 x 100 x 500mm dimensions. The testing is done by supporting the prism on two rollers spaced at 400mm and applying two concentrated loads midway spaced at 200mm on a 5t transverse testing machine. The load is applied continuously and uniformly, with a minimum loading rate increment. The maximum load at crack development is noted, and the flexural strength is determined (Gambhir, 1992).

This is an effective method of determination of composite tensile and compressive capacity of a soil sample through the determination of flexural strength (Natt & Joshi, 1984). In the test, elastic behaviour of the beam is assumed. This results in an overestimation of the tensile strength (Thanikachalam, 1973). Also, it has been shown experimentally that the tensile strength depends on the dimension of the beam; the larger the beam the lower the tensile strength (Wright, 1955). In practice, flexural strength is usually estimated as a fraction of compressive strength of the test specimen (Meyers *et al*, 1976).

- Double Punch Tensile Tests (DPTT)

This method was developed at Lehigh University in the early 1970s. In the method, two steel discs centered on both top and bottom surfaces of a cylindrical soil specimen, are used. A vertical load is then applied slowly on the discs until the specimen reaches failure. The tensile strength of the specimen is then calculated from the maximum load by the theory of perfect plasticity (Fang & Chen, 1971).

The method depends on specimen size and material types making it possible to be adjusted based on the type of material used and specimen size. The method also put into consideration the effect of loading rate (Fang & Chen, 1972), punch size, and the ratios of the diameter of the specimen to the diameter of the disk (punch) (Winterkorn & Fang, undated). However, both theoretical (Chen, 1970) and laboratory studies show that the shape of the specimen does not affect the double punch tensile results (Cumberledge *et al*, 1976).

Study has shown that, because the test is of a penetration type on unconfined soil mass, cracks developed always travel in the shortest distance from the centre of the punch. The advantage of this is that the method causes failure on the weakest plane (random failure plane), resulting in a measurement of true tensile strength of the soil. Because of a random failure plane, the double-punch test is a useful and sensitive method for studying the consistency characteristics and classification of soil, stabilized soils, and other construction materials (Cumberledge *et al*, 1976).

2.7.4. Shrinkage Determination

Currently, shrinkage of various stabilized layers is determined by using mechanical measuring devices such as verniers and strain gauges. Also, researchers have developed methods that can be used in the laboratory to determine the shrinkage of stabilized samples. Most of these methods were observed to be associated with errors that lead to incorrect shrinkage values.

Recently, a method was developed that can be used to determine the shrinkage of stabilized layers in the laboratory. The method can be used to determine very small shrinkage measurements (0,002mm), as well as the determination of shrinkage values for coarse graded lightly stabilized soils (Grobler, 1994).

Test specimens are compacted statically with a 90kN static load. Clout nails, 25mm in length are inserted vertically into the material at predetermined positions and the load was re-applied and increased until the required volume of material is obtained resulting in a specimen with the required density. The mould is then removed and dismantled by removing the retaining bolts, sides and end plates. The specimen is then slid from the base plate onto a glass support smeared with grease to reduce friction between the specimen and its support. Reference targets are then glued to the heads of the clout nails with quick setting glue. A 'Demec' measuring device is used to measure the movement between the reference targets to an accuracy of 0.002mm.

Although the method has been found to give accurate shrinkage results, there are minor problems that can be expected while using the method. The problems are associated with measurement anomalies that can arise from variability in the temperature and relative humidity during the storage of the specimen and as specimen-related errors that are influenced by the individual physical characteristics of specimens. However, the method is advantageous for the fact that in case of crack occurrence, the position is easily identified and measurements can continue between the remaining reference targets.

2.8. Guidance Gained from the Literature Review for this Study

From the literature survey, the following was learnt and used as guidelines in the laboratory testing:

Aggregate-lime-pozzolan properties

- The maximum dry density is influenced by many factors that in turn reflect on the optimum moisture content during compaction test. The use degree of saturation represents a better idea of water required for practical purposes when samples are moulded in large quantities. In this study, the degree of saturation was used.

- The properties of individual materials, aggregates, lime and pozzolans were observed to influence the performance of the mixes. Literature showed that the individual physical and chemical properties play an important role in the strength and durability of the ALP mixes. In this study, the individual properties of pozzolans were assumed to have no influence on the performance of the ALP mixes and hence were neglected.

- The reaction rate of ALP mixes is greatly influenced by the temperature at mixing, fineness of the pozzolan-lime fraction, pH of the moulding moisture, pozzolan-lime ratio and moulding moisture content. Here, all the physical and chemical conditions at mixing, curing and testing were standardized for all test specimens.

- The strength of the ALP mixes is greatly influenced by curing condition and duration. This is due to the fact that pozzolanic reaction takes place at a slower rate requiring extended curing times as compared to the conventional cement and lime stabilization. Here, the curing conditions and durations were standardised for all design mixes.

- No evidence was recorded that linked mixing methods with strength and durability characteristics of the ALP mixes. Literature has however indicated the importance of attaining a uniform mix for better results. Extra care was taken while mixing the materials in the laboratory to ensure uniformity by mixing lime and pozzolan before being added into sand, followed by the required water content. Thus during this study, mixing was done in a mechanical pan mixer for preparation of all test specimens. Test mixes were dry mixed for 1 minute after which, the required moulding moisture content was added and mixing continued for a further 2 minutes before being placed in airtight plastic bags to avoid any moisture loss during the casting process.

- The compaction effort and duration was observed to influence strength development of ALP mixes. Thus, in this study the Mod AASHTO compaction was used for compacting all test specimens. The Mod AASHTO compaction was achieved using a dynamic compactor for all the strength test cylindrical specimens. However, the prism specimens for shrinkage determination were compacted by placing an exact mass required into the mould provided with a wooden frame with a mass placed on top, the excess material was vibrated into the mould using the vibrating table until all excess material was compacted into the mould.

- ICL of the materials to be stabilized influences the quantity of lime available to activate the pozzolan material after mixing. Thus it was rendered important to determine ICL of the material to be stabilized before making a final decision on the lime requirement.

- The literature showed that the reactivity of pozzolan depends on its glassy structure. Thus it was important for this study, to perform scanning electro microscopy (SEM) analysis of the two pozzolanic samples to determine differences between the two samples for comparison purposes, both before and during the hydration process.

- No literature on the strength development of ALP mixes for short curing duration was obtained. Hence it was found that it is important to perform a pilot study to evaluate the most economical curing duration for ALP mixes if a large number of soil types was to be included in the study.

Laboratory program

- The degree of saturation was used instead of moulding moisture contents in the pilot study
- The ICL of sand to be stabilized was done following the method described by Ballantine & Rossouw (1989)
- Various test methods were reviewed from the literature for the determination of tensile strength of stabilized layers. It was found that DPTT is simpler than other methods discussed and provides results that closely represent the actual tensile strength of the material. In this research program, DPTT was used to determine the tensile strength of the mixes
- Compressive strength of soils is determined by using UCS test that was also utilised in this study
- Many shrinkage determination methods are available in practice. However, the method by Grobler (1994) was considered to effectively measure shrinkage accurately and was used in this research program
- Immersion of test specimens before testing reduces the compressive and tensile strengths. Thus samples in this research program were dry tested and there was no prior immersion of specimens in water

- A pilot study was performed to evaluate the strength development of ALP mixes after 28, 90 and 180 days of curing. The results were used in the selection of a curing duration representing the medium term curing duration to be adopted in the study.

CHAPTER 3

EXPERIMENTAL AND TEST PROGRAMME

INDEX

3.1 Introduction 3-1
3.2 Test Materials 3-1
3.3 Experimental Design 3-4
3.4 Curing Conditions 3-7
3.5 Research Methodology 3-7

CHAPTER 3

EXPERIMENTAL AND TEST PROGRAMME

3.1. Introduction

This research study was conducted as part of the on-going research to evaluate the feasibility of using volcanic ash obtained from Tanzania as pozzolan for road construction. Volcanic ashes from two regions in Tanzania, Arusha in the north-eastern and Mbeya in the south-western parts of the country, were imported for the evaluation based on the pilot study for possible pozzolan use for road construction (COWI, 2000). The volcanic ashes are activated using lime, and then utilized in stabilization of gravel or sand for road bases and sub bases.

During the study, the California bearing ratio (CBR), compressive strength and double punch tensile strength determination was done on the laboratory prepared pilot specimens made from washed river sand moulded at varying degrees of saturation to determine their behaviour. The shrinkage behaviour of the same pozzolan and cement mixes at varying degrees of saturation was also evaluated. The findings of the pilot laboratory investigations were further used in the compressive strength – degrees of saturation relationships for two sandy materials similar to those typically used in road construction in Tanzania.

A comparison between ALP mixes moulded at degrees of saturation and control cement-sand mixes moulded at similar varying degrees of saturation was performed for all design mixes.

3.2. Test Materials

In this study specimens were made from volcanic ash, hydrated lime, cement, water and sand.

- **Pozzolan**

Pozzolanic materials used in this study were volcanic ash obtained from two different natural sources in Tanzania, Oldonyo Sambu and Songwe in Arusha and Mbeya regions respectively. The two pozzolans were collected separately in July 2002, packed and transported by air to Pretoria. The packing was in airtight bags to avoid loss of fine particles and stored in a cool dry place free from moisture and contamination by foreign matter. According to COWI (2000), the pozzolan samples for strength testing should satisfy a target minimum required fines content (< 0.075mm) of 30% and maximum grain size of 1mm (requirement 95% < 1mm).

- Pozzolans from Arusha region

The sample was taken from Oldonyo Sambu village, close to the Police post; excavated from a depth of approximately 0.5m below topsoil. The material consisted of fine-grained volcanic ash, silty-texture with a brownish colour. The material was sampled from an approximately 3m thick layer overlying a layer comprising pumice pebbles, whitish in colour. This sample did not require any grinding prior to testing.

- Pozzolans from Mbeya region

The sample was taken in Songwe area from a borrow pit owned by Mbeya Cement Company. The material consisted of ash/tuff, hard-cemented lumps and fine-grained material with a greyish brown colour. The Mbeya pozzolan sample required crushing to obtain the target minimum fines content as given by COWI (2000). During this study, the pozzolan sample from Mbeya was crushed using an electric powered laboratory jaw crusher at the Specialised Road Technologies (SRT) laboratory located on the University of Pretoria South campus. The crusher was capable of crushing up to approximately 80% passing through a 2mm sieve material. The crushed material was then sieved through a 2mm sieve (due to the small sample available the conservative limit of 1mm was compromised to 2mm) to obtain the target material with 95% passing through a 2mm sieve (COWI, 2000).

- **Lime**

Lime used in this study was Calcium Hydroxide Supercalco 97 manufactured by Crest Chemicals in Johannesburg, South Africa. The lime is specified to contain 97.73% pure lime that is within the specified minimum lime purity of 90% (COWI, 2000).

- **Cement**

Cement used in this program was CEM II/B-V 32.5N Portland fly ash cement conforming to SABS EN197, manufactured by LAFARGE Cements obtained from a commercial cement supplier in Pretoria.

- **Sand**

In this study three sandy materials were selected based on their grading:

- Washed river sand specified to have 99% material finer than 4.75mm – used in the pilot study,
- Unwashed river sand specified to have 99% material finer than 4.75mm, and
- Washed river sand specified to have 99% material finer than 6.50mm.

The use of washed river sand as the pilot soil type was because it was required to study the influence of degree of saturation on the lime-pozzolan reaction. The presence of a fraction finer than 0.075mm might lead to the presence of clay particles that might react with lime contributing to the strength of the mixes.

All sands were obtained from a commercial supplier in Pretoria, South Africa, and were stored in 200 litre steel drums until use.

- **Water**

Tap water, potable without any dissolved salts or chemicals, was used in the study.

3.3. Experimental Design

This study consisted of two parts, namely a pilot and main studies to determine the influence of the degrees of saturation on the strength behaviour of ALP mixes made from three sand types and comparison was made with similar sand mixes made from cement.

- **The pilot study**

Here, the strength (tensile and compressive) behaviour of design mixes (shown in table 3.1) made from washed river sand was determined for specimens cured after 28, 90 and 180 days. The specimens were moulded at degrees of saturation varying between 60 and 100%.

The use of high binder contents (16%) in this study was based on the findings by COWI (2000), that comparable strength results between tensile and compressive could be attained at higher binder contents.

Table 3.1 gives the summary of the experimental mix designs for the pilot laboratory testing.

Table 3.1: The pilot experimental mix design matrix

Target degree of saturation (%)	Lime:pozzolan:sand ratios by mass		Control mix
	Arusha	Mbeya	Percentage cement
60	1:3:21	1:3:21	4%
80	1:3:21	1:3:21	4%
100	1:3:21	1:3:21	4%

For each of the combinations of the experimental design matrix (table 3.1) the following were measured:

- <u>CBR</u>

The unsoaked CBR, without surcharge for cement stabilized sand, plain sand as well as Arusha and Mbeya pozzolan mixes was determined, 24hours after mixing. The test was conducted on specimens made from all the design mixes as specified in table 3.1 with a control sand specimen compacted at a degree of saturation corresponding to the maximum dry density. The delay in CBR determination was to allow for binder to start reacting giving a rough indication as to the time for allowing the traffic on a newly constructed layer on-site.

The CBR was determined without soaking and surcharge based on the findings by Yoder & Witczak (1975) that for soils the CBR is dependent on soil texture, compacting moisture and density rather than swelling especially for granular materials.

All CBR tests were done based on method A9 given in the TMH 1 (1986).

- <u>Compressive Strength</u>

The mixes were prepared, cured and their compressive strength determined after 28, 90 and 180 days. UCS for control specimens made of cement-stabilized sand was also determined.

For each cell of the matrix (table 3.1), three identical specimens were crushed at the specified age and the average value (irrespective of results outliers etc.) taken as the compressive strength of the batch.

The UCS test was determined following method A14 of the TMH 1 (1986) without soaking prior to testing avoiding strength reduction associated with soaking (Plykannen, 1995).

- <u>Tensile strength</u>

The mixes were prepared, cured and the tensile strength was determined after 28, 90 and 180 days. Tensile strength for control specimens made of cement-stabilized sand was also determined.

For each cell of the matrix (table 3.1), three samples were crushed at the specified age and the average value (irrespective of results outliers etc.) taken as the compressive strength of the batch.

The tensile strength of the mixes was determined following the DPTT method by Fang & Chen (1971).

Shrinkage determination

One specimen was prepared from each experimental design matrix mix. Preparation of test specimens and procedure followed a method by Grobler & Visser (1996).

It should be noted that during this study, specimens were not allowed to dry freely and were placed in an ambient chamber maintained at 35^{0}C and 60% humidity. Thus shrinkage measurements taken are of autogenous shrinkage that is generally considerably less than drying shrinkage and is most probably not representative of the actual shrinkage occurring in stabilized layers in the field.

- **The main study**

The compressive strength for all ALP and cement mixes moulded at degrees of saturations corresponding to the moisture contents shown in table 3.2 was determined for specimens cured under ambient conditions. The percentage moulding moisture contents in this study were selected to cover a wide range of degrees of saturation around the maximum dry density.

Table 3.2: The main experimental mix design matrix

Target moulding mositure conditions		Lime:pozzolan:sand ratios by mass		Control mix
Target moisture content, %	**Target degrees of saturation**	**Arusha**	**Mbeya**	**Cement**
3	25	1:3:21	1:3:21	4%
6	50	1:3:21	1:3:21	4%
9	75	1:3:21	1:3:21	4%
12	100	1:3:21	1:3:21	4%

For every design mix given in table 3.2, specimens were made from both washed and unwashed sands. The compressive strength for all design specimens cured for 28 days in controlled temperature and humidity conditions was determined.

3.4. Curing Conditions

All strength specimens were wrapped in plastic shrink-wrap and placed in an ambient room maintained at 55% humidity and 30^0C until prior to testing. The selection of the humidity and temperature conditions for curing was made based on the approximated operating conditions in the tropics (Tanzania in particular). Moisture contents as well as the estimated dry density at testing for all strength specimens were determined.

All specimens prepared for shrinkage evaluation were wrapped in shrink-wrap and placed in another ambient chamber maintained at an average of 60% humidity and 35^0C for 7 days. On the eighth day, the specimens were removed from the wrap and left uncovered in the same room where the shrinkage measurements were taken every 24 hours. The curing conditions for the shrinkage specimens followed those specified for concrete testing.

3.5. Research Methodology

To characterize each of the combinations, the following tests were done:

- **Index properties determination.** The grain size distribution for the pozzolan samples was determined. Grain size distribution was determined by performing a wet sieve analysis to check if the pozzolan samples satisfy the target minimum required fines content (< 0.075mm) of 30% and maximum grain size of 1mm (requirement 95% < 1mm) (COWI, 2000). Those pozzolans that did not meet the target fines content were crushed before proceeding with further testing. Dry sieve analysis was also done on the virgin pozzolans and aggregates as well as for all design mixes. The tests were done following TMH 1 method A1(b) (1986).

- **SEM and chemical composition of pozzolan samples.** Electron microscopy analysis was performed on virgin dry pozzolanic samples from both Arusha and Mbeya regions for evaluation of surface texture and structure of the materials by determination of their chemical composition. The analysis was later done on small specimens taken from mixes after strength testing and curing for 28, 90 and 180 days. The chemical composition of the two pozzolans was also done to determine if there is any distinct difference between the two-pozzolan samples as well as with the aim of classifying the materials based on mineralogy as per the VOLCON (Evans *et al*, 1999).

- **ICL.** This test was done in accordance with method given by Ballantine & Rossouw (1989). The test was conducted to check if the target mix ratio proposed by COWI (2000) is sufficient to both activate the pozzolan as well as satisfy the ICL value of sand.

- **Determination of MDD and OMC.** For all mixes in the experimental design matrix (table 3.1), the moisture-density relationship (OMC/MDD) was determined. The OMC for each mix as well as its MDD was determined following TMH 1 method A7 (1986) with Mod AASHTO compaction effort. All test mixes were allowed to stand covered in a damp cloth for 30 minutes before compaction to allow for even moisture distribution in the sample.

- **Shrinkage determination.** Shrinkage was determined by following the method by Grobler & Visser (1996).

- **Strength development with time.** The strength development of ALP mixes with time should be studied for one type of sand to save time and costs of extending the research into many sand types that requires varying curing lengths. In this study, pilot specimens were prepared from ALP and cement mixes at varying degrees of saturation where their tensile and compressive strength, shrinkage and CBR were determined. The specimens were prepared from washed river sand so as to avoid the presence of any clay particles that could possibly react with lime resulting in higher strengths.

CHAPTER 4

DISCUSSION OF PILOT STUDY RESULTS

INDEX

4.1 Introduction 4-1

4.2 Indicator Tests 4-2

4.2.1 Sieve Analysis for virgin pozzolans 4-2

4.2.2 Scanning Electron Microscopy (SEM) 4-4

4.2.3 Chemical Composition of Pozzolans 4-7

4.3 Pilot Laboratory Investigations 4-8

4.3.1 Statistical Analysis 4-9

4.3.2 Initial Lime Consumption (ICL) of Sand 4-10

4.3.3 Sieve Analysis 4-10

4.3.4 MDD and OMC 4-12

4.3.5 Tensile Test Methods Comparison 4-15

4.3.6 California Bearing Ratio 4-19

4.3.7 Compressive Strength 4-23

4.3.8 Tensile Strength 4-34

4.3.9 Comparison of the UCS and Tensile Strengths 4-44

4.3.10 Shrinkage Determination 4-49

4.3.11 Scanning Electron Microscopy (SEM) 4-56

4.4 Findings from the Pilot Study 4-58

CHAPTER 4

DISCUSSION OF THE PILOT STUDY RESULTS

4.1. Introduction

The aim of this investigation was to determine the effect of variation in degree of saturation on the behaviour of laboratory prepared ALP mixes. The study was done by evaluating the compressive and tensile strength development, shrinkage and CBR at different degrees of saturation around the optimum for different sand types typically used in road construction in Tanzania. The scanning electron microscopy analysis to study the hydration pattern of the mixes was also done for all design mixes.

The decision to use the degree of saturation instead of moulding moisture content in this study was due to the fact that the exact optimum moisture contents of sands are not clearly defined as it varies with different factors such as size and shape of mould, degradation of particles, temperature and layer thickness (Savage & Visser, 2001). In this study, aggregates used were mostly sandy materials that are easily drainable whose compaction result in successive peaks (maximum dry densities) making it impossible to determine their exact optimum moisture contents. A further objective of the research was to compare the behaviour of cement-stabilized soils with ALP mixes in terms of compressive strength development and shrinkage characteristics.

The design mixes for the pilot investigations are as shown in table 3.1 in chapter 3. Sand used in the pilot study was obtained in damp condition and hence its hygroscopic moisture content was determined and the required moisture content was added. This however did not result in the exact moulding moisture contents for the target degrees of saturation for individual design mixes.

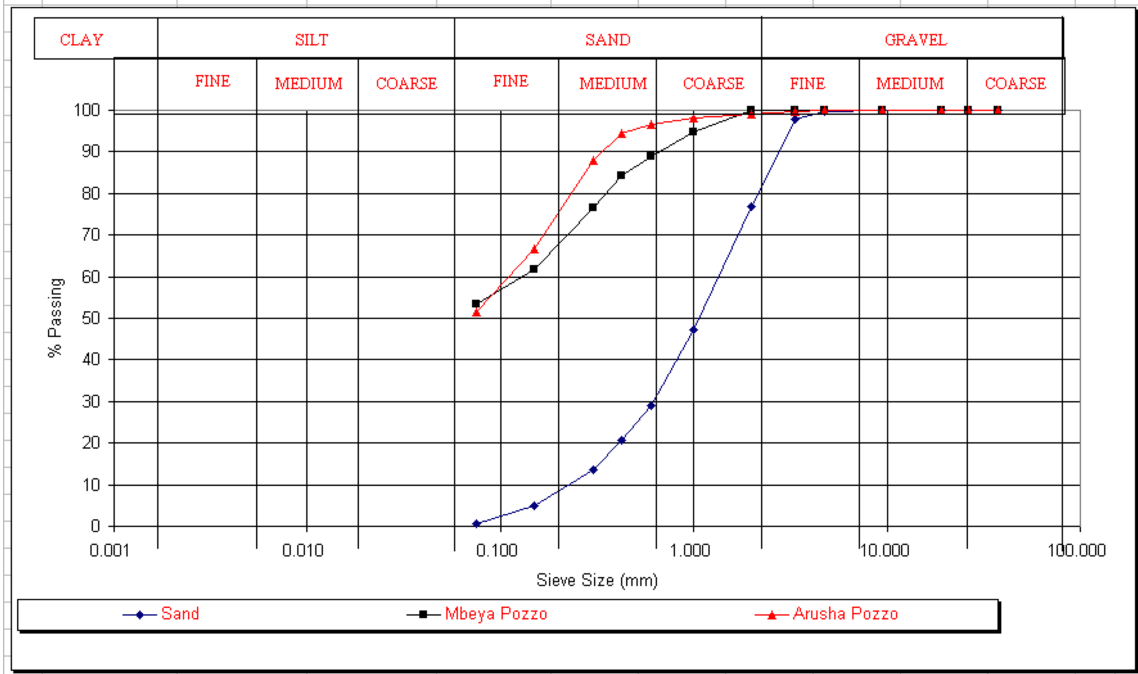
4.2. Indicator Tests

In this section result of indicator tests such as sieve analysis, chemical composition and SEM analysis of virgin pozzolans are presented and discussed.

4.2.1. Sieve analysis for virgin pozzolans

A sieve analysis was done on virgin sand similar to that typically used in Tanzania, Mbeya and Arusha pozzolan samples separately.

Results of the sieve analysis for the virgin materials are presented in figure 4.1.



	MATERIAL TYPE			SAND	ARUSHA POZZOLAN	MBEYA POZZOLAN
	Sieve	Size	ASTM	% Passing	% Passing	% Passing
Grain Size Distribution	37.50	mm	(1½")	100	100	100
	26.50	mm	(1")	100	100	100
	19.00	mm		100	100	100
	9.50	mm		100	100	100
	4.75	mm		100	100	100
	3.35	mm		98	100	100
	2.00	mm		77	99	100
	1.00	mm	(No. 30)	47	98	95
	0.600	mm (est.)	(No. 40)	29	97	89
	425	micron	(No. 50)	21	94	84
	300	micron (est.)	(No. 70)	14	88	77
	150	micron	(No. 100)	5	67	62
	75	micron	(No. 200)	1	52	53
% Fine Sand				4	15	9
% Medium Sand				9	21	15
% Coarse Sand				7	7	7
M.D.Density	*g drymax. (kg/m^3)*		Mod	1956	-	-
O.M. Cont.	*w opt. (%)*		AASHTO	7	-	-
SOIL CLASSIFICATION			GM	2.02	0.55	0.63

Figure 4.1: Grain size distribution curve for virgin materials

In figure 4.1 it is shown that the Arusha pozzolan has a slightly finer graded envelope than the Mbeya pozzolan. However, the percentage fines ($P_{0.075}$) found in Arusha pozzolan was similar to that found in Mbeya pozzolan.

4.2.2. Scanning Electron Microscopy (SEM)

SEM analysis was done on the Arusha and Mbeya pozzolans to evaluate the crystal shape and texture of the virgin pozzolans as well as in the mixed state. SEM photographs were taken of both pozzolans in their fresh (virgin) form.

Plates 4.1 and 4.2 give the SEM images of the virgin pozzolans at a magnification of 20 000.

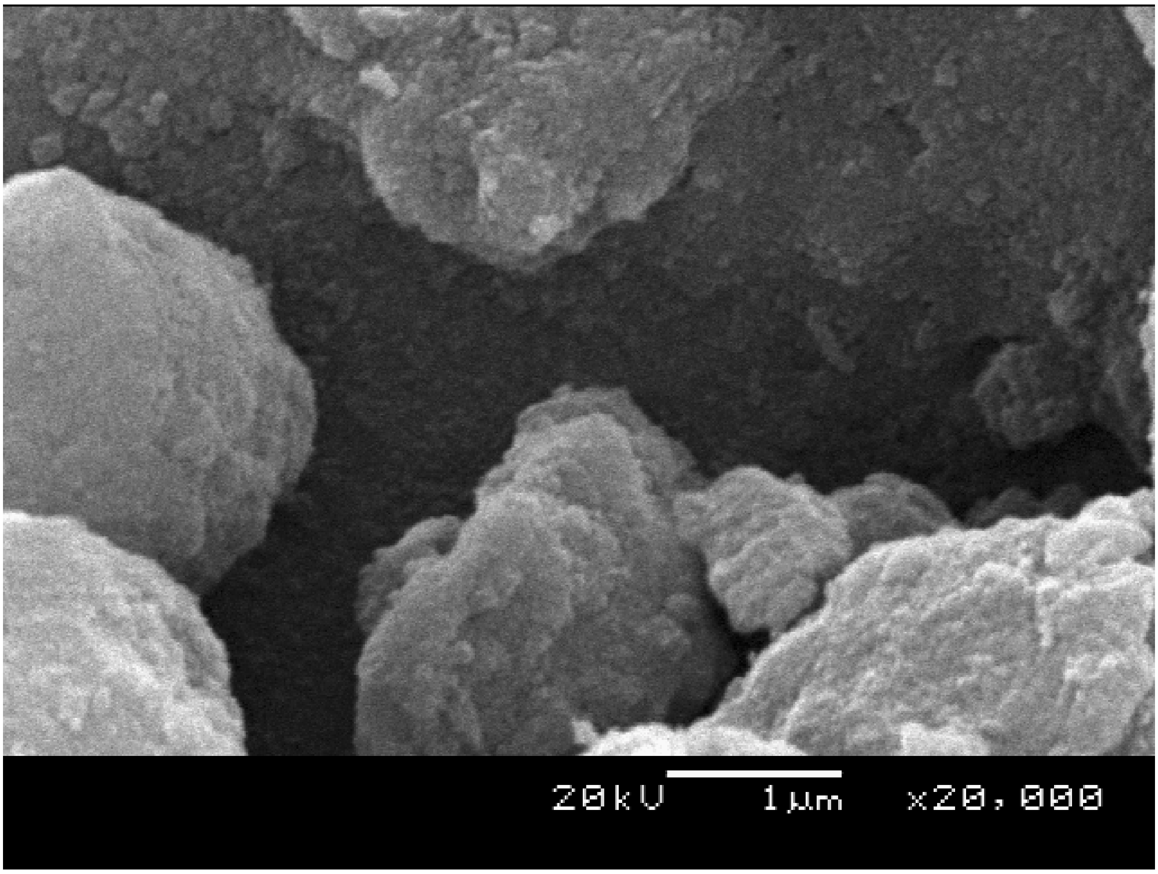


Plate 4.1: Arusha Pozzolan, virgin state

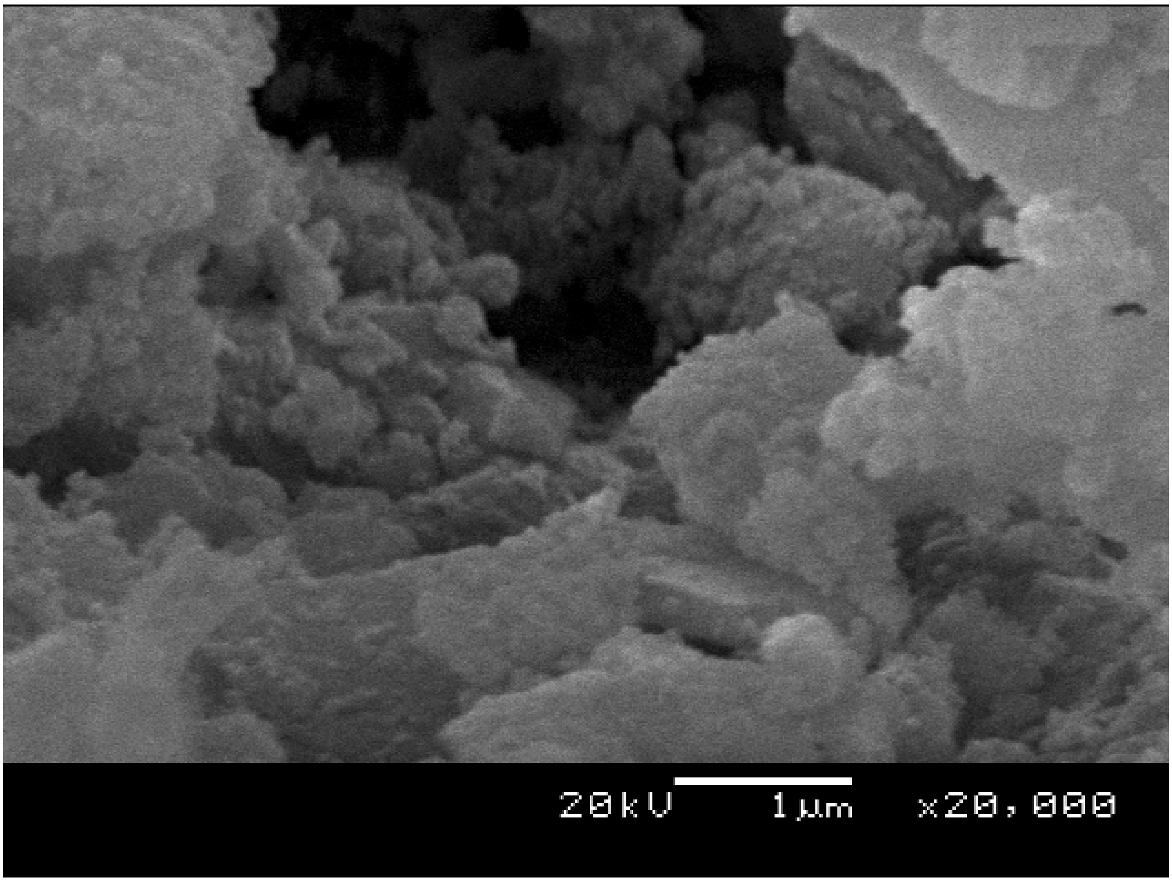


Plate 4.2: Mbeya Pozzolan, virgin state

From plates 4.1 and 4.2 it is evident that the Arusha pozzolan showed a dense and more rounded shape particle than the Mbeya pozzolan that showed an irregular texture comprising rounded and plate like shapes. Both pozzolans were found to have rough surfaces indicating a coarse texture making them easily reactive when lime and moisture are added.

The irregular shape found in Mbeya pozzolans can be due to the crushing done in the laboratory to obtain the required percentage finer than 0.075mm.

It was found to be of importance to compare the reactive phases for the pozzolans from Arusha and Mbeya. Small representative samples of Mbeya and Arusha pozzolan was weighed and mixed with lime in the ratio of 1:3 left to react and dry freely in the laboratory for 7 days. Plates 4.3 and 4.4 gives the SEM images for the two mixes.

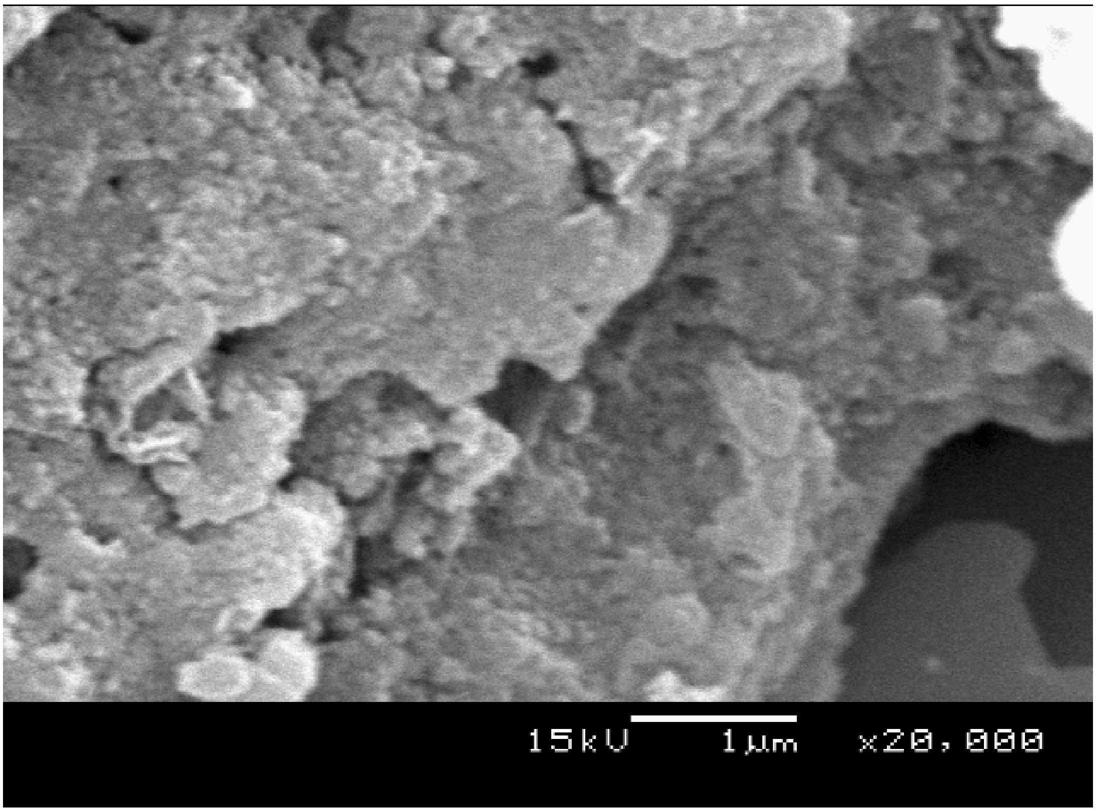


Plate 4.3: Arusha Pozzolan, mixed with lime (1:3 ratio)

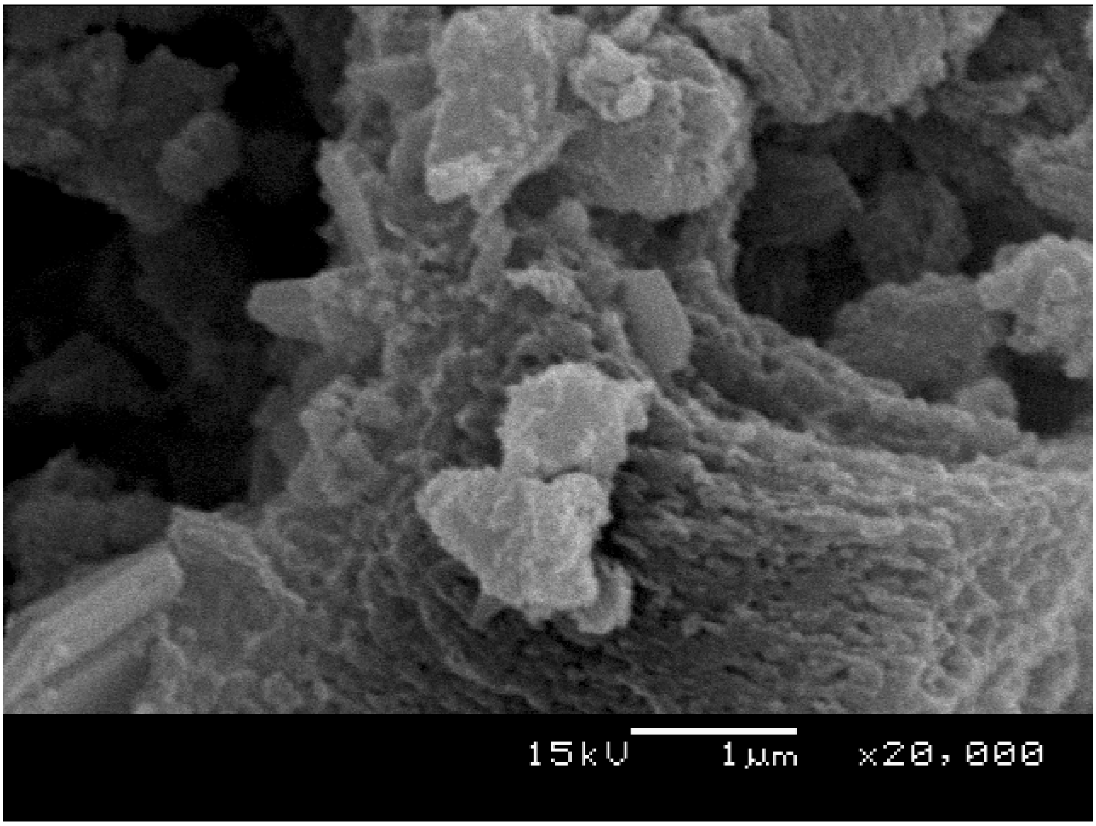


Plate 4.4: Mbeya Pozzolan, mixed with lime (1:3 ratio)

Plates 4.3 and 4.4 show that both pozzolans have rough irregular surface texture after the addition of lime. Arusha pozzolan, however, shows a denser and more uniform texture than that of Mbeya pozzolan.

The above differences between the Arusha and Mbeya pozzolans suggest a possible higher hydration rate for the Arusha pozzolan compared to that of the Mbeya pozzolan.

4.2.3. Chemical Composition of Pozzolans

A quantitative analysis was done to determine the chemical composition of the virgin pozzolans on three different parts of the slide. For both pozzolans, indistinctive difference between individual analysis points existed suggesting that the materials are thoroughly mixed in nature as shown in appendix A.

It was observed that the chemical composition of the two pozzolans differs mostly in their silica, aluminium and iron contents. Arusha pozzolans had a silica content of 48% as compared to Mbeya pozzolans with 61% as shown in table 4.1. According to COWI (2000) the pozzolans are classified from their composition as Intermediate and Basic for Mbeya and Arusha pozzolans respectively.

Table 4.1: Summary of Chemical composition, selected components

	Element mass, %	
Element	**Arusha**	**Mbeya**
Si	48.19	61.43
Al	22.06	17.25
Fe	12.61	7.98
K	6.58	8.26
Na	4.08	2.3
Ca	3.58	0.94
Ti	2.19	1.47
Mg	0.57	0.23
Cl	0.08	0.13
P	0.07	0.01

Table 4.1 give the chemical composition of Arusha and Mbeya pozzolans respectively. A distinct difference is observed between the two pozzolanic materials. Arusha pozzolans shows higher percentage of Aluminium (Al) and Iron (Fe) compared to Mbeya pozzolans. Higher degree of Al and Fe can explain the highest initial strength found in Arusha pozzolan mixes after 28 days of curing compared to that in Mbeya pozzolan mixes. This is due to the fact that Al and Fe crystallizes rapidly leading to initial strength development.

On the other hand, Mbeya pozzolans are found to have higher silica (Si) content compared to Arusha pozzolan. The high Si content in Mbeya pozzolans could have resulted in the higher long-term strength development (after 90 days) as compared to the Arusha pozzolan mixes.

4.3. Pilot Laboratory Investigations

The pilot laboratory investigation was done on washed river sand to evaluate the influence of varying degrees of saturation on the compressive and tensile strengths, CBR and shrinkage behaviour of ALP mixes. The tests were done mainly to:

- Compare the variations between the compressive and tensile strength developments in ALP mixes
- Determine the most suitable curing duration for strength development of ALP mixes. This was done in order to find a standard curing duration to be used during the main laboratory investigation where more ALP mixes would be studied mainly for minimizing the costs
- Determine the strength development of lime-pozzolan mix by using washed river sand. This will inhibit any lime reacting with the aggregate since no clay content was present in the sand.

4.3.1. Statistical Analysis

During the study, statistical analysis was done for all design mixes in the pilot and main studies. The analysis was done using SAS for windows Release 8.02 TS Level 02MO. The general linear model (GLM) procedure was used to determine the influence of degrees of saturation and curing duration on the tensile and compressive strengths development, shrinkage, dry density (at mixing and testing) and CBR. A grouping model was run using the Waller-Duncan K-ratio test and Duncan's multiple range tests for the UCS, DPTT, DBR and ratio between UCS and DPTT. The model groups the means of the test results that are not significantly different and thereby establishes a trend with varying degrees of saturations and curing times.

The model was run for each individual mix separately (i.e. Arusha, Mbeya and cement mixes). The conclusions were drawn for each mix without comparing the mixes due to significant differences in their respective chemical compositions and mix ratios. Thus only the degrees of saturation and curing duration were used to evaluate each individual mix.

Appendix D gives the typical SAS input files for Arusha pozzolan mix. Three tables each for DPTT, UCS and density have been constructed from regression analysis for each individual design mix for both pilot and main study and are given in Appendix E. The tables illustrate what occurs when different degrees of saturation are used in each mix with respect to curing duration.

From the SAS results presented in Appendix E, the regression values were estimated using a general formula for ANOVA given as:

$$\text{Regression value} = \text{Interceptor} + \text{Estimator } X_1 * Y_1 + \text{Estimator } Y_1 * X_1 + \ldots + \text{Estimator } X_n * Y_n + \text{Estimator } Y_n * X_n$$

Where,

X_1 to X_n – Actual curing duration, days

Y_1 to Y_n - Actual degrees of saturation, %

If the regression analysis is done for say, Y_1 then Y_1 is replaced with 1 and the rest of the equation with Zero. Similarly when evaluating the X values the same is adopted. This means the others will fall away and addition is only done on the remaining.

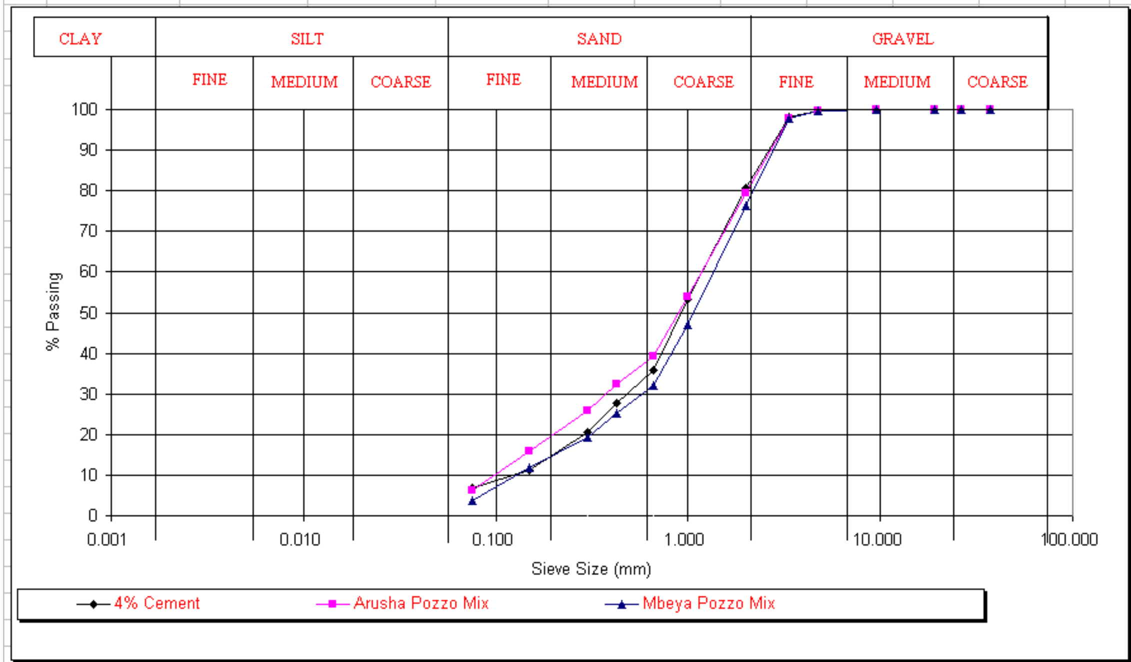
The individual SAS findings for every design mix are discussed in subsequent chapters.

4.3.2. Initial Consumption of Lime (ICL) of Sand

The ICL for the sand used in the pilot study was found to be 1.5% pure lime ($Ca(OH)_2$). During this study, 4% lime by mass was used. The low ICL value suggests that no amorphous silica was present in the sand material showing a possibility that the sane sample was from a quartzitic nature. This quantity satisfied the stabilized material's initial lime requirements and the remaining 2.5% was available as an activator for the pozzolans, and little or no strength contribution from the lime was expected.

4.3.3. Sieve Analysis

Sieve analysis for design mixes was done to establish the influence of grading on the strength and density. Figure 4.2 gives the grading of the design mixes.



	MIX TYPE			4% CEMENT	ARUSHA POZZO MIX	MBEYA POZZO MIX
	Sieve	Size	ASTM	% Passing	% Passing	% Passing
Grain Size Distribution	37.50	mm	(1½")	100	100	100
	26.50	mm	(1")	100	100	100
	19.00	mm		100	100	100
	9.50	mm		100	100	100
	4.75	mm		100	100	100
	3.35	mm		98	98	98
	2.00	mm		81	79	76
	1.00	mm	(No. 30)	53	54	47
	0.600	mm (est.)	(No. 40)	36	39	32
	425	micron	(No. 50)	28	33	25
	300	micron (est.)	(No. 70)	20	26	19
	150	micron	(No. 100)	11	16	12
	75	micron	(No. 200)	7	6	4
% Fine Sand				4	10	8
% Medium Sand				9	10	8
% Coarse Sand				7	7	6
M.D.Density	*g drymax. (kg/m^3)*		Mod	1996	1993	2067
O.M. Cont.	*w opt. (%)*		AASHTO	9.7	10.1	8.0
SOIL CLASSIFICATION			GM	1.85	1.82	1.95

Figure 4.2: Grain size distribution curve for mixes

In figure 4.2 it is shown that all mixes were found to have a similar grading, however, Mbeya pozzolan shows a slightly coarser mix compared to that of the cement and Arusha mixes. The grading moduli of the Arusha pozzolans and cement mixes were also shown in figure 4.2 to be significant and slightly lower than that of Mbeya pozzolan mix.

Therefore, sieve analysis of the design mixes show that Arusha pozzolan mix has similar grading as that of cement mix. Mbeya pozzolan mix was found to have a coarser grading as compared to the two mixes.

4.3.4. MDD and OMC

Compaction was conducted on all pozzolan and cement mixes using Mod AASHTO energy. It was decided to determine the MDD and OMC of untreated sand compacted at the same energy in the laboratory to determine the influence of addition of binders to the material. The results of compaction tests were used to determine the quantity of moisture to be added to attain the target degrees of saturation for all design mixes.

The degrees of saturation were calculated using an equation given by Punmia (1994):

$$\gamma_d = \frac{G\gamma_w}{1 + {}^{w}\!/_{S_r}}$$

Where,

S_r – degree of saturation

G – specific gravity of soil

w – moisture content

γ_d – dry unit mass of soil

γ_w – dry unit mass of water

Table 4.2 gives the maximum dry densities and optimum moisture contents for all pilot design mixes. The degrees of saturation corresponding to OMC are also given in the table. These results are discussed next.

The specific gravity for all design mixes was taken as 2.65. The value of 2.65 was taken based on the fact that the sand samples are quartzitic with the quartz content assumed to be above 90% (Barton *et al*, 2001 & Punmia, 1994).

Table 4.2: The actual degrees of saturation achieved

Type of Mix	OMC, %	MDD, kg/m^3	Specific Gravity, G	Actual degree of saturation, %
Plain sand	6.5	1956	2.65	49
Arusha mix	8.6	1993	2.65	69
Mbeya mix	8.0	2067	2.65	73
Cement mix	7.4	1996	2.65	60

- <u>Untreated sand</u>

From table 4.2, the MDD of 1956kg/m^3 at OMC of 6.5% was obtained for untreated sand. The maximum dry density is achieved at 49% degree of saturation. No significant increase in the maximum dry density was observed with an increase in degree of saturation confirming that the material (sand) is freely draining cohesionless soil.

- <u>Arusha pozzolan mix</u>

The Arusha pozzolan mix was observed to have an MDD of 1993kg/m^3 obtained at OMC of 8.6%. The MDD was achieved at 69% saturation (table 4.2). A slight increase in the maximum dry density for Arusha pozzolan mix compared to that of plain river sand with the degree of saturation being nearly 1.5 times that of sand (49% sand and 69% Arusha mix), indicates that the addition of fines from Arusha pozzolan greatly modifies drainability of sand with an insignificant improvement in the compactability at the same effort.

- <u>Mbeya pozzolan mix</u>

A MDD of 2067 kg/m^3 was attained at 8% OMC as shown in table 4.2. The addition of Mbeya pozzolan to sand has resulted in a higher MDD compared to that of Arusha mix. It is assumed that this is due to the coarser grading found in the Mbeya mix. The degree of saturation resulting in the maximum dry density was found to be approximately 73%. The significant increase on the maximum dry density of the Mbeya pozzolan mix as well as the increase in the degree of saturation for maximum dry density compared to that of plain sand indicates that the addition of Mbeya pozzolan modifies both the drainability and compactability of the sand at the same effort.

The maximum dry density for Mbeya mix was found to be higher than that of Arusha mix implying that mixes made from Mbeya pozzolans provide a denser mix due to its grading compared to those from Arusha. Higher degree of saturation for attainment of maximum dry density found in Mbeya mix compared with that of Arusha indicates that Mbeya pozzolans have higher water absorption potential compared with that of Arusha. This can have an influence in the quantity of water required in the hydration of Arusha pozzolan as compared to that of Mbeya pozzolans.

- <u>Cement mix</u>

From table 4.2, the MDD of 1996kg/m^3 was obtained at an OMC of 7.4% for the cement-sand mix. The MDD for the cement mix was obtained at 60% saturation. The addition of cement to the sand is shown to result in a slight increase in the MDD as well as the OMC. A higher degree of saturation for attainment of maximum dry density indicates that the addition of cement reduces the drainability of sand.

The following conclusions are derived from the compaction test:

- The binder addition to plain sand has an influence on both drainability and density.
- The addition of fine graded cement and Arusha pozzolans to the sand resulted in lower MDD values compared with that resulting from addition of the Mbeya pozzolans. This was influenced by the coarser grading found in the Mbeya pozzolans providing a denser mix of all the design mixes.
- High degrees of saturation for achieving MDD found in Arusha and Mbeya

pozzolan mixes compared with the cement mix can be related to the low drainability of pozzolan mixes.

- A slightly higher degree of saturation required for achieving MDD for Mbeya mix compared with that of Arusha mix can be related to the denser grading found in the Mbeya mix. This has resulted in a requirement of higher moisture content for lubricating the materials for achieving MDD using the same compaction compared with Arusha mix.

4.3.5. Tensile Test Methods Comparison

In this study, DPTT was used in the evaluation of the tensile strength of the specimens. The decision to use the method was reached after evaluation of various test methods based on the reliability as well as simplicity. However, it was found important to compare the test method with the conventional ITS in order to relate the results to the conventional methods. Pozzolan stabilized specimens mixed at the same moisture contents were tested by DPTT and ITS respectively after curing for 100 days under ambient conditions of 30^0C and 55% humidity.

In the ITS test method, the tensile strength is determined by measuring the resistance to failure of a cylindrical specimen when a load is applied to the curved sides of the specimen TMH 1 (DoT, 1986). The test set-up is as shown in plate 4.5 consisting of two loading strips of hardened steel, 19 x 20 x 200mm, with the 19mm face ground concave to a radius of 76±1mm, together with a frame suitable designed to align the loading strips on the test specimen.

The specimen and the frame is placed on a compression testing machine capable of applying a load of at least 30kN at a rate of 40kN/min and capable of measuring the load accurately to 0.1kN.

Plate 4.5: Positioning of the test specimen in ITS (Courtesy Africon Labs)

For the DPTT test, two circular discs are placed on the top and bottom of the specimen by the use of centring blocks and placed in a compression machine as shown in Plate 4.6. The specimen fails at the maximum failure load in either of the two patterns as shown in Plates 4.7 and 4.8 respectively.

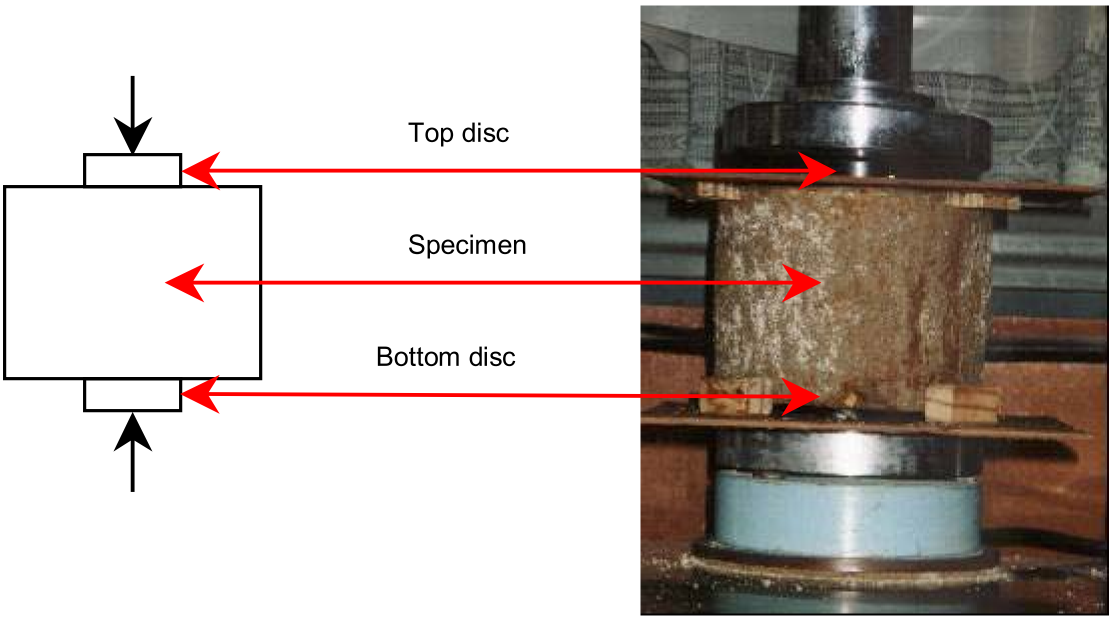


Plate 4.6: Positioning of test specimen in DPTT (Courtesy University of Pretoria)

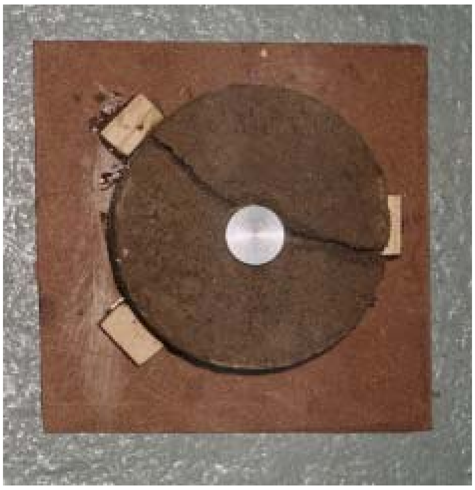


Plate 4.7: Two-plane failure of DPTT specimen (Note centring block and disc)

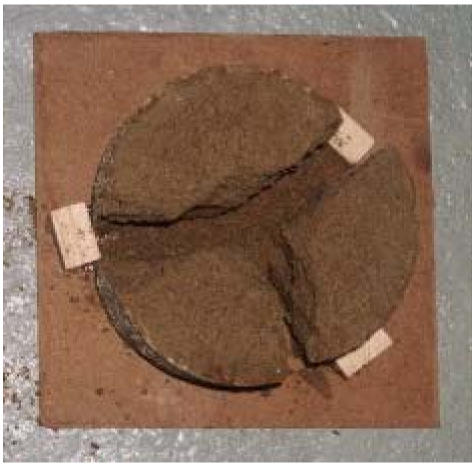


Plate 4.8: Three-plane failure of DPTT specimen

Table 4.3: Comparison between ITS and DPTT for Arusha pozzolan mix after100days

Sl. No.	Mould. MC, %	Mass at testing, (g)		Moisture content, %		Dry Density at test, kg/m3		Failure Force, kN		Avg. Strength, kPa	
		DPTT	ITS	DPTT	ITS	DPTT	ITS	DPTT	ITS	DPTT	ITS
1	10.7	4949	4958	7.0	7.5	2000	1996	32	30	1076	995
2	10.7	4958	4944	7.2	7.1	2001	1997	34	33	1161	1071
3	10.7	5009	4933	7.1	7.1	2024	1994	37	33	1243	1078
Average values		**4972**	**4945**	**7.1**	**7.2**	**2008**	**1996**	**34**	**32**	**1160**	**1048**

Insignificant tensile strength difference was observed in the specimens failing in either two or three-plane failures modes.

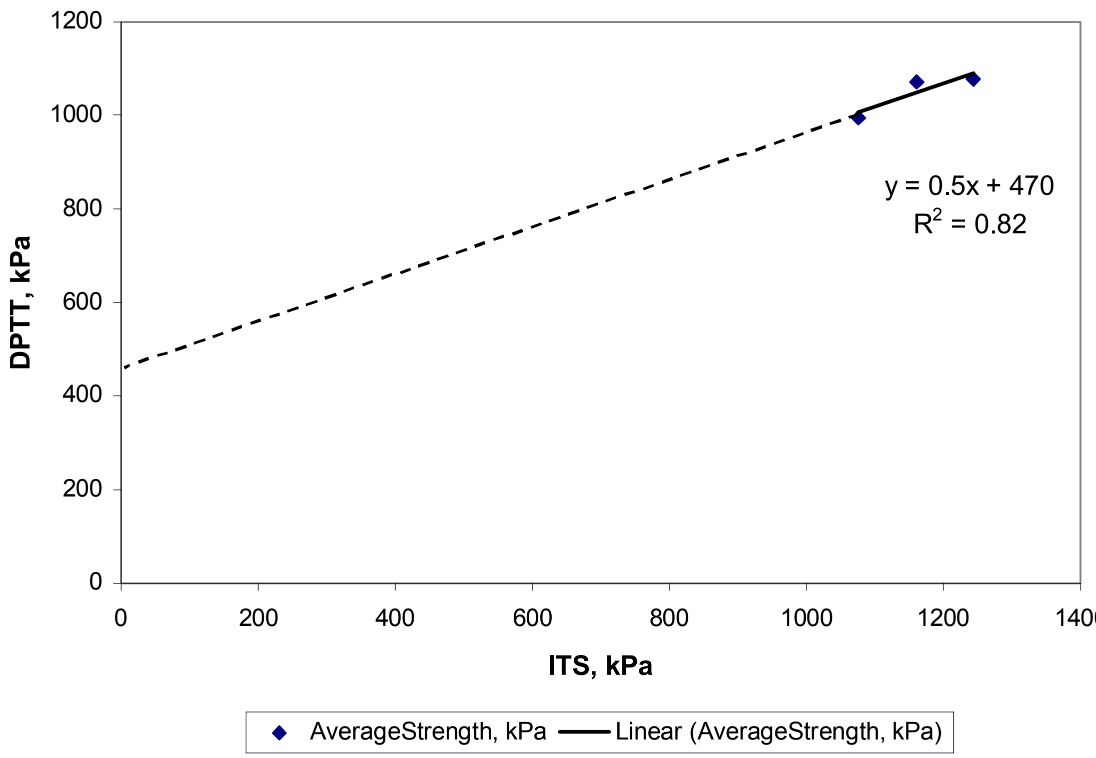


Figure 4.3: Tensile strength test methods comparison for Arusha mix

Figure 4.3 shows a relationship between ITS and DPTT for Arusha pozzolan ALP mix after 100 days of curing in an ambient chamber. Arusha pozzolan mix was selected for tensile strength test methods comparison based on its high short-term tensile strength development as compared with Mbeya mixes found in the pilot study by COWI (2000). Thus Arusha pozzolan mixes were chosen for the tensile strength methods comparison due to the time constraint.

The figure 4.3 shows a high degree of consistency between the two methods. Statistical regression analysis of the data returns an average R^2 value of 0.82 indicating a reasonable correlation between the two test methods. Due to the small sample size, the relation between ITS and DPTT did not cross the axes at the origin and hence an extrapolation line (doted) was used to indicate the expected relationship between the two methods.

Thus it was concluded that the DPTT method is useful in evaluation of tensile strength of stabilized layers as it gives results that are correlated with those obtained when using conventional methods. The method was also proved to be simple using standard laboratory equipments with simple additions that can be easily made.

4.3.6. California Bearing Ratio (CBR)

The literature review revealed that CBR of the stabilized materials is important in giving an indication of their strength in a fresh state. According to Yoder & Witzack (1975), soaking of specimens during CBR testing for granular soils is of no importance as these materials have little swelling potential. In this study, un-soaked CBR without surcharge was determined for all design mixes 24 hours after compacting.

- **Preparation of Specimens**

During this study, specimens were compacted using dynamic compactor at Africon Laboratories - Pretoria. The preparation of specimens is summarized below:

- Specimens tested were two ALP mixes made from Arusha and Mbeya pozzolans respectively, and a 4% cement stabilized control specimen. Three specimens were prepared for each sample, moulded at varying moisture contents as specified in the mix design matrix

- Specimens were compacted at three compaction efforts, namely Mod AASHTO, NRB and Proctor as specified by method A8 in TMH1-1986
- Samples were weighed, and dry mixed in the laboratory pan mixer for 2 minutes. Then, the moulding moisture content resulting in the design degree of saturation was administered while continuing mixing for another 1 minute
- The specimens were then compacted and placed in an empty trough, covered by moistened sawdust and left to cure for 24 hours
- After 24 hours, unit loads at 2.54, 5.08 and 7.62mm penetrations by CBR press were recorded for every specimen. Later, CBR values were determined following the method A9 in the TMH1 (1986).

The CBR values at 2.54, 5.08 and 7.62 penetrations for all mixes compacted at different compaction efforts are presented in table 4.4. Plain sand was compacted at the degree of saturation corresponding to the maximum dry density.

Table 4.4: CBR Results for the design mixes after 24 hours curing

Material type	Actual degree of saturation	Calculated CBR at Penetrations								
		2.54 mm			5.08 mm			7.62 mm		
		MOD	NRB	Proct	MOD	NRB	Proct	MOD	NRB	Proct
Sand	50	47	35	28	60	42	35	65	45	38
Cement	60	187	127	112	265	167	130	296	193	140
Cement	80	90	112	76	140	155	110	174	176	127
Cement	100	35	58	50	56	80	82	80	98	108
Arusha	60	202	187	102	270	207	107	300	211	110
Arusha	80	41	89	97	80	139	129	118	171	140
Arusha	100	16	25	16	27	37	27	38	50	43
Mbeya	60	210	131	90	260	145	106	284	148	107
Mbeya	80	102	127	99	154	162	109	190	177	118
Mbeya	100	19	36	55	42	72	84	68	103	105

- **CBR Measurements**

As per the TMH1 (1986), the CBR at 2.54mm penetration is used as the standard penetration to determine the CBR at a specified density. In this study, CBR values will be given at 97% Mod AASHTO specified for base layers (upper and lower) for cemented materials in TRH13 (DoT, 1986).

The CBR values measured for the design mixes at 2.54mm penetration at 97% and 100% Mod. AASHTO compactions are given in table 4.5.

Table 4.5: CBR measurement

Mix type	**Compaction degree of saturation, %**	**Calculated CBR**	
		At 2.54 mm Penetration	
		97% Mod. AASHTO	**100% Mod. AASHTO**
Cement	60	140	187
Cement	80	100	90*
Cement	100	58	35*
Arusha pozzolan	60	190	202
Arusha pozzolan	80	90	41*
Arusha pozzolan	100	25	16*
Mbeya pozzolan	60	160	210
Mbeya pozzolan	80	127	102*
Mbeya pozzolan	100	55	19*
* At this moisture content, it appeared that excess pore water pressure resulted in a drop in CBR with an increase in compaction effort			

The SAS analysis showed that all design mixes showed a significantly high CBR in specimens moulded at 60% saturation. A significant decrease in CBR with an increase in degree of saturation was also confirmed by the analysis.

From table 4.5 and SAS analysis, the following conclusions are drawn:

- CBR of all design mixes significantly depend on the degree of saturation. An increase in the degree of saturation beyond 60% results in a significant decrease in the CBR in all design mixes.

- All mixes attain highest CBR values when compacted at 60 percent degree of saturation.

- Possible dissipation of excess pore water pressure took place in all specimens moulded at degrees of saturation above 60 percent resulting in a decrease in the CBR with an increase in compaction effort from 97 to 100% Mod. AASHTO.

- The CBR's for the Arusha mixes are more affected by an increase in the degrees of saturation compared with the Mbeya pozzolan and cement mixes.

4.3.7. Compressive Strength

UCS was used to determine the compressive strength of the design mixes in the study. UCS tests were carried out on all design mixes after 28, 90 and 180days of curing in an ambient chamber respectively.

Table 4.6: The actual degrees of saturation achieved for pilot strength test specimens

Description of the binder in the mix	Degree of Saturation (%)				
	Target	DPTT		UCS	
		Actual	% dev.	Actual	% dev.
Arusha pozzo					
Arusha mix 1	60	69	15	69	15
Arusha mix 2	80	80	0	83	4
Arusha mix 3	100	100	0	100	0
Cement					
Cement mix 1	60	56	-7	60	0
Cement mix 2	80	67	-16	67	-16
Cement mix 3	100	100	0	100	0
Mbeya pozzo					
Mbeya mix 1	60	66	10	62	3
Mbeya mix 2	80	76	-5	73	-9
Mbeya mix 3	100	94	-6	94	-6

Table 4.6 gives the actual degrees of saturation achieved in specimens cast for tensile and compressive strengths determination and shrinkage evaluation for pilot study. A slight deviation of up to 15% of actual degree of saturation from the target degree of saturation was observed. However, the difference being small compared with the target range and was neglected and hence no correction was made.

- **Preparation of Specimens**

During this study, dynamic compaction was used for compacting all UCS test specimens. The preparation of the specimens is summarised below:

- UCS was determined for all design mixes, two ALP mixes and a 4% cement stabilized control sample. For each sample, three sets of specimens were moulded at varying degrees of saturation as specified in the design matrix (table 4.4).

- A Mod AASHTO compaction effort was used. All specimens were compacted using standard methods for UCS determination given in method A14 in TMH1-1986.

- For each sample, the required contents were weighed and dry mixed for 2 minutes in a laboratory pan mixer, and then the required water was added and mixed for 1 minute. The mixtures were then stored in airtight plastic bags to avoid moisture loss during compaction.

- After compacting, specimens were removed from the moulds, wrapped in shrink-wrap and placed in an ambient chamber maintained at 30^0C and 55% humidity until prior to testing.

- On testing day, specimens were taken out of the curing chamber, unwrapped and weighed before crushing in a standard compression-testing machine. A representative sample was taken after crushing for determination of moisture content at testing for determination of the moisture used during hydration.

- **UCS Measurements**

The UCS of all specimens was determined. The results of UCS testing are presented in table 4.7.

From table 4.7 it is noted that there is a significant testing moisture content reduction between 28 and 90 day cured specimens for both pozzolan mixes as compared with that of cement mixes. The difference reduces slightly between 90 and 180 days. This rapid reduction of testing moisture content in ALP mixes, compared with cement mixes suggests either rapid absorption of water or conversion of the H_2O into OH^- ions that just ordinary drying as that occurring in cement mixes.

Table 4.7: Summary of Compressive Strength Results

Design Mix Properties			28 days of Curing		90 days of Curing		180 days of Curing		Percentage Strength Increase	
Mix Type	**Moisture added, %**	**Target Degree of Saturation, %**	**Testing Moisture content, %**	**Average Strength, kPa**	**Testing Moisture content, %**	**Average Strength, kPa**	**Testing Moisture content, %**	**Average Strength, kPa**	**28-90 days**	**90-180 days**
Cement	14.0	100	13.0	837	11.8	1301	10.7	1356	56	4
Cement	8.3	80	7.4	2510	6.7	3026	4.7	3712	21	23
Cement	7.0	60	5.0	3357	4.8	4368	3.0	6061	30	39
Arusha	13.1	100	10.3	5081	8.0	5848	6.0	7128	15	22
Arusha	10.0	80	8.0	6950	7.0	7551	5.0	9859	9	31
Arusha	8.6	60	8.1	8116	6.0	9775	3.0	12070	20	23
Mbeya	10.1	100	9.0	5294	5.0	8164	5.0	11040	54	35
Mbeya	8.0	80	7.0	7395	5.0	10686	5.3	15310	45	43
Mbeya	7.0	60	4.0	3544	4.0	4417	3.0	5360	25	21

- **Arusha Pozzolan Mixes**

Figure 4.4 gives the UCS development for Arusha pozzolan mixes moulded at different degrees of saturation after 28, 90 and 180 days of curing.

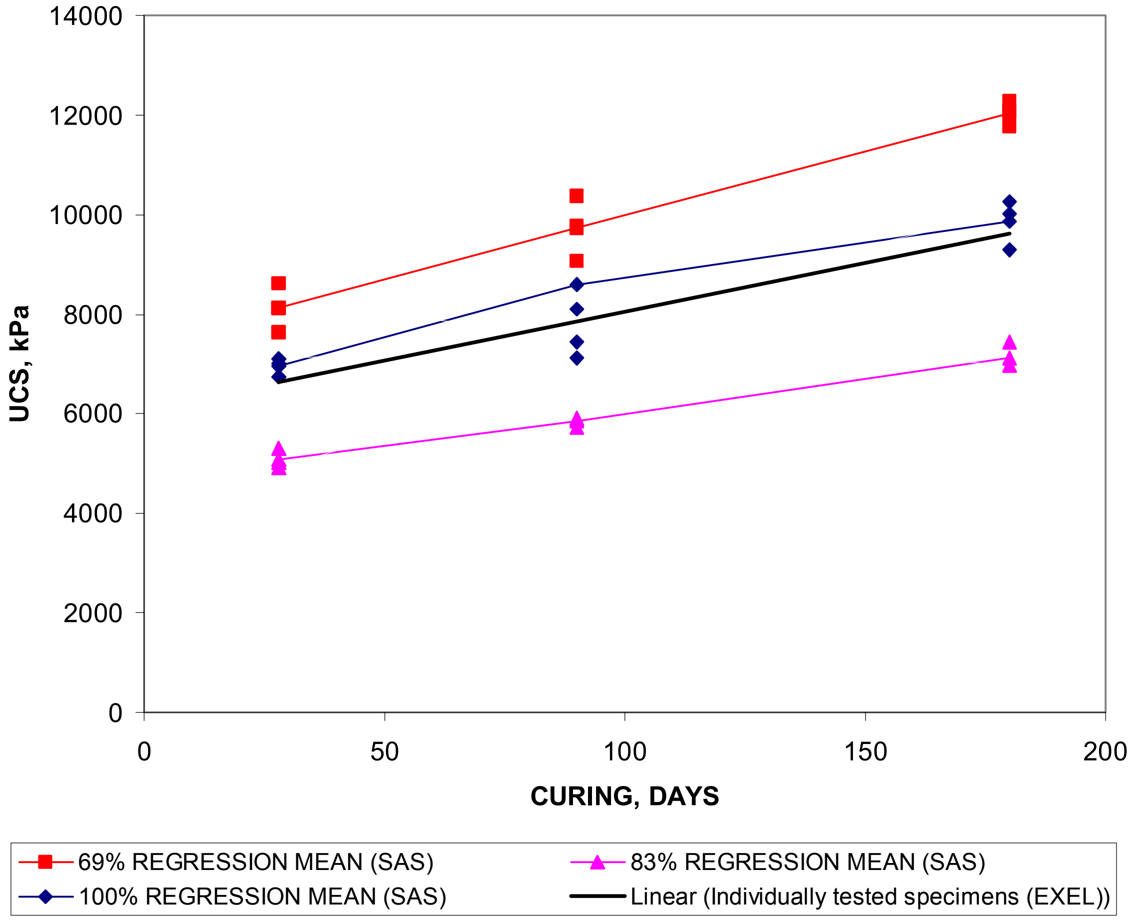


Figure 4.4: UCS Development for Arusha pozzolan mixes

From figure 4.4, the Exel regression analysis of all Arusha pozzolan mixes are plotted by the trend line "Linear (Individually tested specimens)". The values are for all individually tested specimens made from Arusha pozzolan mixes moulded at varying degrees of saturation at 69, 83 and 100% cured after 28, 90 and 180 days.

- Influence of curing duration and degree of saturation

Figure 4.4 shows that UCS for ALP mixes made from Arusha pozzolans depends on the degrees of saturation. A significant increase in UCS with a decrease in degree of saturation is observed.

The optimum moisture content for Arusha mix is around 69% saturation. This suggests that higher UCS for Arusha pozzolan mixes are achieved in specimens moulded at OMC. However, specimens moulded at 83% saturations show a lower UCS as compared to that obtained in specimens moulded at 100% saturation.

A significant increase in UCS at all compaction moisture contents is noted in all specimens between 28 and 180 days. This shows that the UCS development in Arusha pozzolan mixes follows a linear UCS development between 28 and 180 days of curing regardless of compaction degrees of saturation.

The SAS analysis results for all Arusha pozzolan mixes at varying curing durations are also plotted in figure 4.4. The SAS results confirm the findings of the actual UCS plots that there exists a significant dependency of UCS on the curing duration and variation in degrees of saturation for Arusha mixes.

- The UCS development

A significant linear increase in UCS at all degrees of saturation was noted with an increase in curing duration between 28 and 180days. No significant strength flattening out is noted throughout the curing duration of 180days. This is unlikely for conventional stabilization where flattening out of the UCS is expected between 50 and 60 days of curing.

Based on the conventional stabilization, draft TRH 13 (DoT, 1986) gives the ratio between the 28 and 7-days UCS for cemented materials between 1.4 and 1.7. The average UCS values of between 5 and 8Mpa were obtained for Arusha mixes at the design degrees of saturation. Taking an average of 6.5MPa, a corresponding 7-day UCS of between 3.8 and 4.6MPa are obtained for all Arusha pozzolan mixes.

TRH 4 (DoT, 1996) suggests cemented bases (C3) for relatively heavy trafficked roads and subbases for heavy trafficked roads. C3 is designed for UCS values between 1.5 and 3MPa at 100% Mod. AASHTO (7-days). Arusha pozzolan mixes attain UCS values between 3.8 and 4.6MPa, which is nearly 1.5 times the minimum specified C3 strength.

Hence, the UCS development for Arusha pozzolan mixes is found to be higher than that specified for C3.

The following conclusions are drawn on the UCS development for Arusha pozzolan mixes:

- Specimens moulded at degrees of saturation close to optimum results in highest UCS development.
- The degree of saturation plays a significant role in the UCS development of ALP mixes made from Arusha pozzolans in specimens cured between 28 and 180 days.
- A significant linear UCS increment with an increase in curing duration is observed regardless of the moulding degree of saturation for all Arusha pozzolan mixes.
- The UCS development in Arusha pozzolan mixes continues linearly beyond 180days of curing unlike in conventional stabilization where flattening-off is observed after 50 to 60days of curing.
- Arusha pozzolan mixes can be designed to attain UCS values required for C3 category base and sub base layers.

- **<u>Mbeya Pozzolan Mixes</u>**

Figure 4.5 gives the UCS values at different degrees of saturation for Mbeya pozzolan mixes after 28, 90 and 180 days of curing.

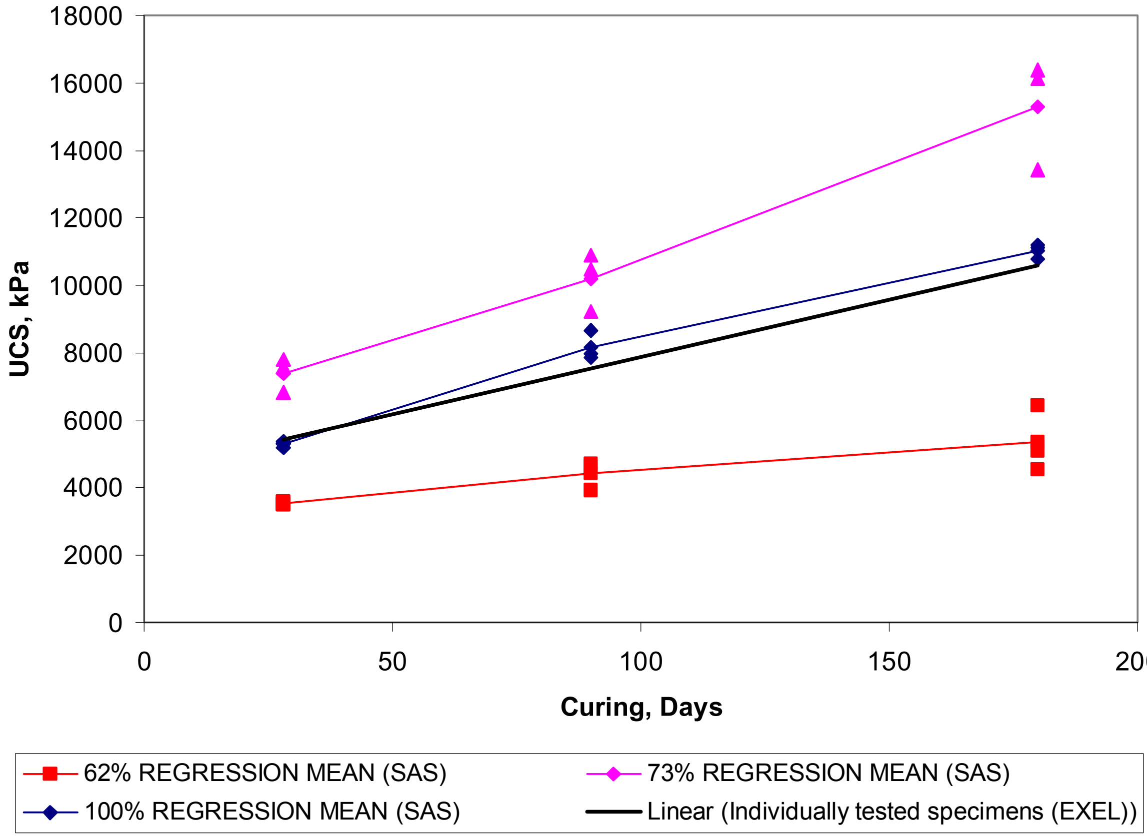


Figure 4.5: UCS development for Mbeya Pozzolan mixes

From figure 4.5, the Exel regression analyses of all Mbeya pozzolan mixes are plotted by the trend line “Linear (Individually tested specimens)”. The values are for all individually tested specimens made from Mbeya pozzolan mixes moulded at varying degrees of saturation at 62, 73 and 100% cured after 28, 90 and 180 days.

- Influence of curing duration and degree of saturation

From figure 4.5 it is shown that UCS for Mbeya pozzolan mixes attains significantly high UCS when moulded around 73% saturation. Mbeya pozzolan mixes attain their maximum dry density at 73% saturation. Hence, the maximum UCS is achieved in specimens moulded at moisture contents close to the optimum. An increase in the degrees of saturation results in a higher UCS compared with a decrease in the saturation. This implies that Mbeya pozzolans should be moulded at degrees of saturations corresponding to their optimum moisture contents or higher.

There is a significantly gradual UCS increase in Mbeya pozzolan mixes between 28 and 180 days. However, significant high UCS values are obtained after 28 days of curing for all mixes.

- The UCS development

A significant linear increase in UCS at all degrees of saturation was noted with an increase in curing duration between 28 and 180days. No significant strength flattening out is noted throughout the curing duration of 180days. This is unlikely for conventional stabilization where flattening out of the UCS is expected between 50 and 60 days of curing.

Based on conventional stabilization, draft TRH 13 (DoT, 1986) gives the ratio between the 28 and 7-days UCS for cemented materials between 1.4 and 1.7. For all design mixes made from Mbeya pozzolan, UCS values of between 3.5 and 7.4MPa were attained. Taking an average of 5.5MPa, a 7-day UCS between 3.0 and 4.0MPa was achieved by all Mbeya pozzolan mixes.

The estimated 7-day strength for Mbeya pozzolan mixes are observed to be significantly higher than that specified for C3 (TRH 4) material at 100% Mod. AASHTO (7-days) between 1.5 and 3MPa.

The following can be concluded on the UCS development for ALP mixes made from Mbeya pozzolans:

- Mbeya pozzolan mixes attain highest UCS when moulded at degrees of saturation close to their optimum.

- The degree of saturation plays a significant role in the UCS development of ALP mixes made from Mbeya pozzolans in specimens cured between 28 and 180 days.

- A significant linear UCS increment with an increase in curing duration is observed regardless of the moulding degree of saturation for all Mbeya pozzolan mixes.

- The UCS development in Mbeya pozzolan mixes continues linearly beyond 180days of curing unlike in conventional stabilization where flattening-off is observed after 50 to 60days of curing.

- High UCS values attained by Mbeya pozzolan mixes suggest that the mixes can be designed to C3 category base and sub bases specifications.

- **Cement Mixes**

Figure 4.6 gives the UCS development at different degrees of saturation for cement mixes after 28, 90 and 180 days of curing.

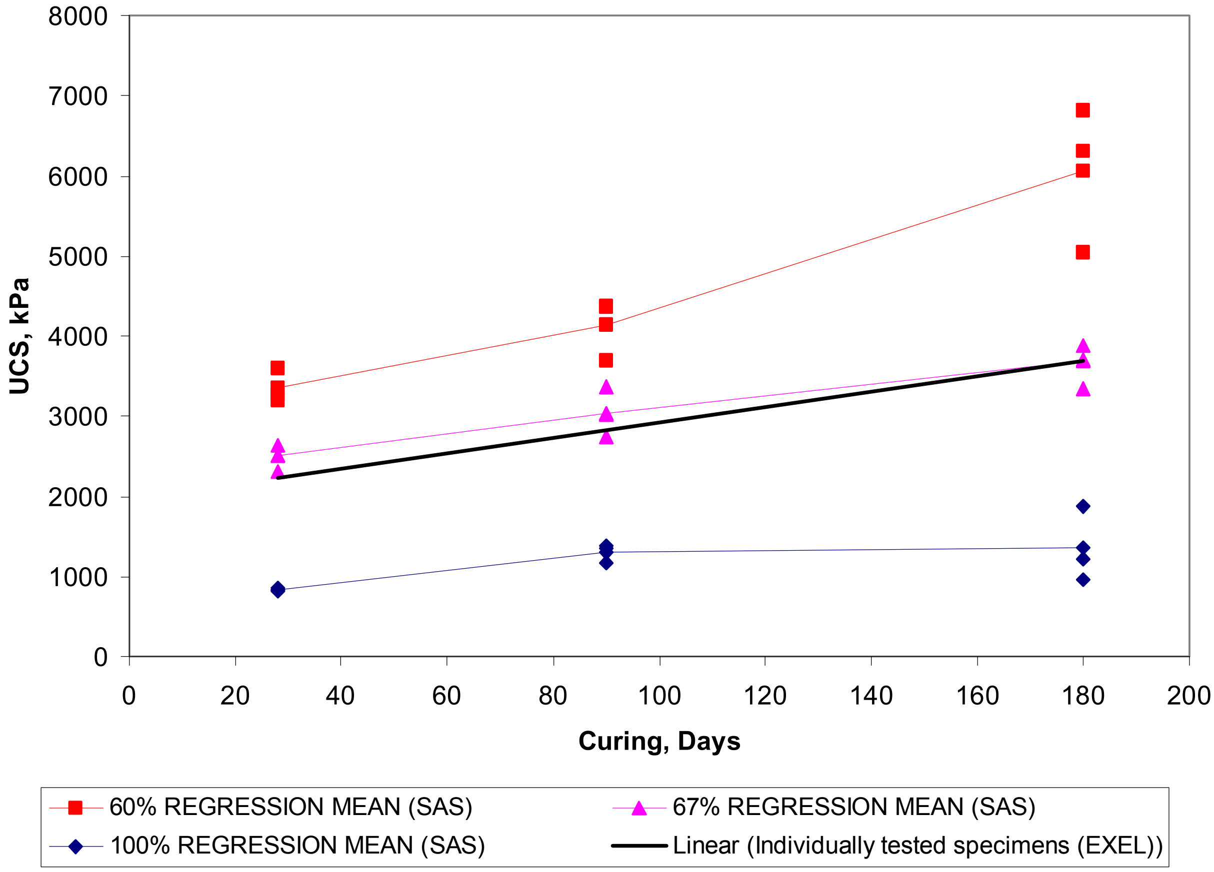


Figure 4.6: UCS development for Cement mixes

From figure 4.6, the Exel regression analysis of all control cement mixes are plotted by the trend line “Linear (Individually tested specimens)”. The values are for all individually tested specimens made from cement mixes moulded at varying degrees of saturation at 60, 67 and 100% cured after 28, 90 and 180 days.

- Influence of curing duration and degree of saturation

Figure 4.6 shows a decrease in UCS with an increase in degree of saturation for cement mixes after 28, 90 and 180 days of curing. An increase in UCS corresponding to a decrease in degree of saturation is observed in all cement mixes. The optimum moisture content for cement mix was observed to exist around 60% saturation. This suggests that there is a significant decrease in UCS with an increase in degree of saturation.

- The UCS development

From draft TRH 13 (DoT, 1986), the ratio between the 28 and 7-days UCS for cemented materials is between 1.4 and 1.7. The average UCS values of between 0.8 and 3.4MPa at all moulding degrees of saturations were obtained for all design cement mixes. Taking an average of 2.1MPa, a 7-day UCS between 1.2 and 1,5MPa was achieved by all design cement mixes.

The estimated 7-day strength for cement mixes suggest that the cement mixes used in the study slightly relates to the specifications for C4 (TRH 4) materials UCS of between 0.75 and 1.5MPa at 100% Mod. AASHTO (7-days).

The following can be concluded on the UCS development for cement mixes:

- The maximum UCS for cement mixes is obtained mixes moulded around optimum.
- The degree of saturation plays a significant role in the UCS development of cement mixes cured between 28 and 180 days.
- All cement mixes show a significant increase in strength with an increase in curing duration for all mixes moulded at varying degrees of saturation.

- An increase in the moulding degree of saturation has a harmful effect on the UCS development in cement mixes cured between 28 and 180 days.
- The cement mixes used in the study are observed to relate to the UCS specifications for C4 materials. However, the quantity of cement used in the study was only for indication purposes and should not necessarily correspond to the UCS obtained by the Arusha and Mbeya pozzolan mixes.

- **Findings related to UCS Development**

During this study, the behaviour of UCS for ALP mixes was studied, a comparison was made with a control cement stabilized mix. The following conclusions can be made from the UCS development in individual mixes:

- The degree of saturation plays an important role in the UCS development of Arusha and Mbeya pozzolan and cement mixes.
- The Arusha and Mbeya pozzolan mixes develops significant UCS after 28 days of curing. The UCS development continues linearly beyond 180 days. UCS development in the two pozzolanic mixes continues linearly beyond 180days unlike to the conventional stabilization where the UCS development flattens-off after 50 to 60days of curing.
- The maximum UCS for all mixes exists in specimens moulded at degrees of saturation corresponding to the OMC. All mixes showed a significant decrease in UCS in specimens moulded at saturations corresponding to moisture contents above OMC.

4.3.8. Tensile Strength

The DPTT (Fang & Cheng, 1971) method was used to determine the tensile strength of the design mixes in the study. DPTT tests were carried out on all design mixes after 28, 90 and 180days of curing respectively.

The specimens for tensile strength testing were prepared and cured similarly to those for UCS testing. The actual degrees of saturation achieved during compaction of design specimens are given in table 4.4. Specimens were not soaked prior to testing. The specimens were crushed as per DPTT method using a standard compression machine.

- **Tensile Strength Measurements**

The tensile strengths of all specimens are presented in table 4.8.

Table 4.8: Summary of Double Punch Tensile Strengths Results of the Mixes

Design Mix Properties			28 days of Curing		90 days of Curing		180 days of Curing		Percentage Strength Increase	
Mix Type	Moisture Added, %	Target Degree of Saturation, %	Testing Moisture Content, %	Average Strength, kPa	Moisture Content, %	Average Strength, kPa	Moisture Content, %	Average Strength, kPa	28-90 days	90-180 days
Cement	14.0	100.0	13.0	58	11.9	153	10.7	191	165	25
Cement	8.3	80.0	7.3	231	6.7	346	4.8	377	50	9
Cement	7.4	60.0	5.4	301	5.5	532	3.1	749	77	41
Arusha	13.0	100.0	10.2	762	8.1	1003	5.5	910	32	-9
Arusha	10.3	80.0	7.8	1212	6.9	1233	4.9	1268	2	3
Arusha	8.6	60.0	8.1	941	6.4	1321	2.7	1103	40	-16
Mbeya	9.9	100.0	8.6	510	5.5	1071	4.6	1262	110	18
Mbeya	7.7	80.0	6.8	959	5.1	1444	4.8	1689	51	17
Mbeya	6.6	60.0	4.8	210	4.3	453	3.5	489	116	8

- **Arusha Pozzolan Mixes**

Figure 4.7 gives the tensile strength development for Arusha pozzolan mix specimens moulded at different moisture contents for mixes cured after 28, 90 and 180 days.

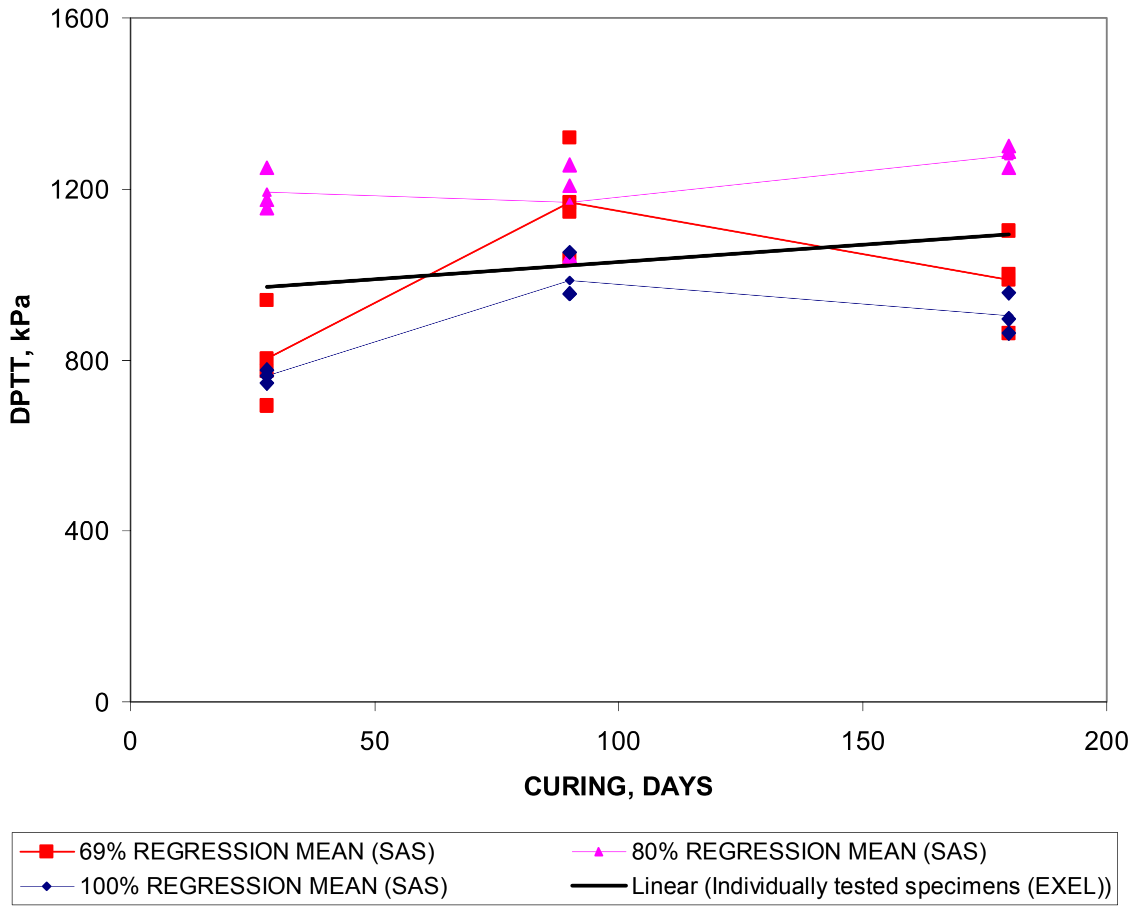


Figure 4.7: Tensile Strength Development for Arusha Pozzolan mixes

The exel regression analysis for DPTT results of all Arusha design mixes is plotted in figure 4.7 as "Linear (Individual tested specimens)". The regression analysis was performed on all mixes moulded at 69, 80 and 100% saturation cured after 28, 90 and 180-days respectively.

- Influence of curing duration and degree of saturation

All specimens made from Arusha pozzolans moulded at varying moisture contents showed an insignificant DPTT increment between 90 and 180 days of curing. This implies the Arusha pozzolan mixes attains most of their DPTT strength after 28 days of curing, after which little strength development is expected.

However, figure 4.7 shows that Arusha pozzolan mix specimens moulded at 69 and 100% saturation show a significant increase in DPTT between 28 and 90 days as compared to those moulded at 80%. The specimens moulded at 80% showed a constant DPTT between 28 and 180 days of curing suggesting that the specimens attained most of their DPTT strength within the first 28 days of curing.

The SAS analysis results for all Arusha pozzolan mixes at varying curing durations are also plotted in figure 4.4. The SAS results confirm the findings of the actual DPTT plots that there exists a significant dependency of DPTT on the variation in degrees of saturation for Arusha mixes.

- The DPTT development

The highest DPTT is achieved in specimens moulded at 80% saturation. The optimum moisture content for Arusha mix exists around 60% saturation. This suggests that highest DPTT for Arusha pozzolan mixes can be achieved in specimens moulded at degrees of saturation slightly higher than that corresponding to the optimum moisture content.

From draft TRH 13 (DoT, 1986), the 28 days ITS for cemented materials is between 1.4 and 1.7 of the 7-day strength. Arusha mixes developed DPTT values between 762 and 1212kPa at all moulding degrees of saturation. From the tensile strength methods comparison, corresponding ITS values between 625 and 994kPa respectively after 28 days of curing are obtained. Considering the least strength development in the Arusha mixes (625kPa), the corresponding 7-day ITS for the mixes of between 368 and 446 kPa should be expected.

TRH 4 (DoT, 1996) suggests cemented bases C2 and C3 for heavy trafficked roads. The stabilized C2 and C3 are designed for ITS values of 400 and 250kPa respectively. Arusha pozzolan mixes attain DPTT values between 368 and 446kPa, indicating that Arusha pozzolan mixes can be designed and used as C2 and C3 road bases regardless of their moulding degrees of saturation.

The following conclusions are made on the ITS development for Arusha pozzolan mixes:

- The degree of saturation plays a significant role in the DPTT development of ALP mixes made from Arusha pozzolans in specimens cured between 28 and 180 days.
- An insignificant DPTT increment with an increase in curing duration should be expected after 28 days of curing in specimens moulded at varying degrees of saturation.
- Highest tensile strength in Arusha pozzolan mixes can be achieved in mixes moulded at 80% saturation regardless of the optimum moisture content.
- An increase in the moulding degree of saturation has a no harmful effect on the DPTT development in Arusha pozzolan mixes in specimens cured between 28 and 180 days. However, above 80% the strength is found to decrease with an increase in saturation.
- Arusha pozzolan mixes can be used as category C2 and C3 road base materials.

- **<u>Mbeya Pozzolan Mixes</u>**

Figure 4.8 gives the tensile strength development for Mbeya pozzolan mix specimens moulded at different degrees of saturation for mixes cured after 28, 90 and 180 days.

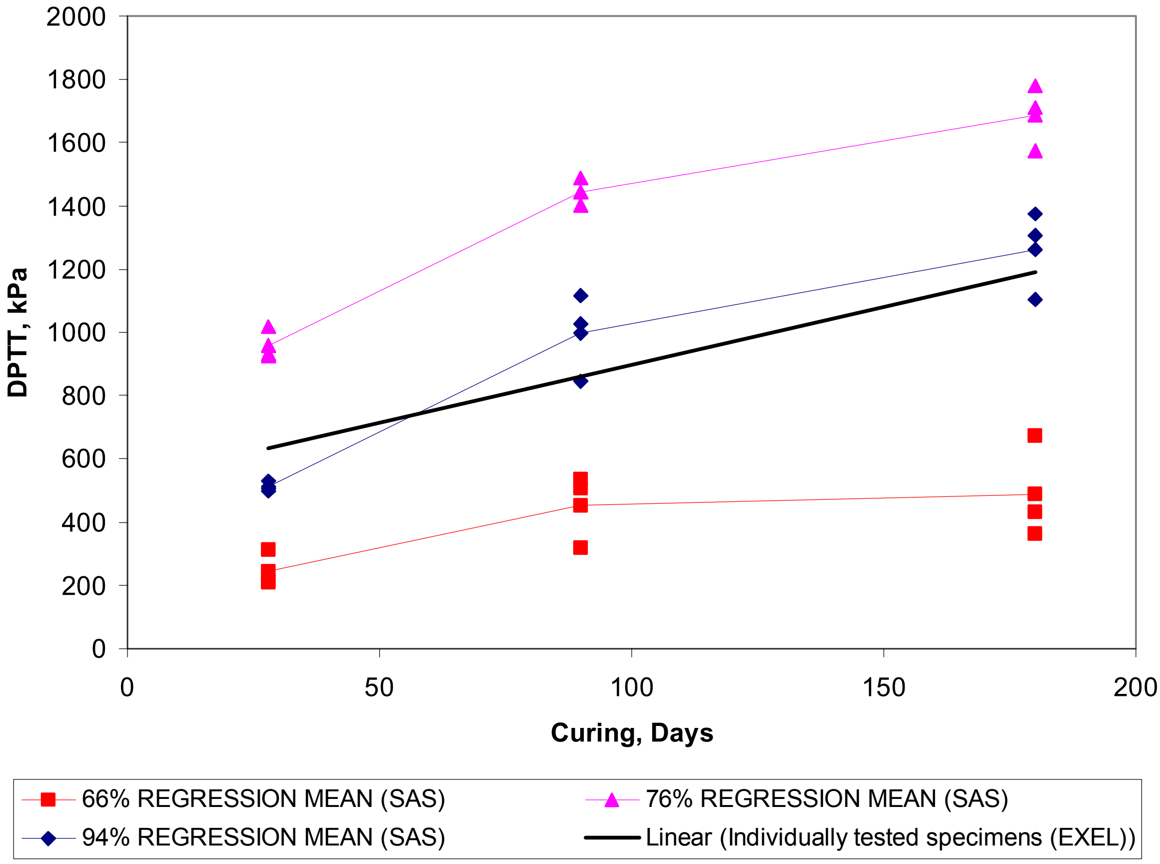


Figure 4.8: Tensile Strength Development for Mbeya Pozzolan mixes

The SAS analysis for DPTT of all Mbeya pozzolan is plotted in the same figure 4.8. The figure shows that the regression of the data follows a similar trend of strength increase with an increase in curing time. The regression analysis also indicates that the moulding degree of saturation influences the tensile strength development.

- Influence of curing duration and degree of saturation

From figure 4.8 it is shown that the DPTT for all Mbeya pozzolan mixes highly depend on the curing duration. Significant DPTT increment is noted in all mixes with an increase in curing duration between 28 and 180 days. However, the DPTT increment is observed to be independent of the moulding degree of saturation between 28 and 90 days. Mixes moulded at 66 percent saturation are found to attain a constant DPTT after 90 days of curing compared with the other mixes moulded at 76 and 94 percent saturation that show a continuous strength increase.

- The DPTT development

Figure 4.8 shows that the DPTT development in Mbeya pozzolan is highly influenced by the variation in moulding degree of saturation. Specimens moulded at 66 percent saturation are observed to achieve significantly low DPTT compared with those moulded at 76 and 94 percent. This suggests that the DPTT development in Mbeya pozzolan mixes is highly influenced by the moulding degree of saturation.

Mbeya pozzolan mixes attained DPTT values of between 210 and 959kPa at all moulding degrees of saturations. This corresponds to ITS values between 172 and 786kPa respectively. Considering the least strength development in Mbeya pozzolan mixes at all moulding degrees of saturation (i.e. 172kPa), the expected 7-day ITS is between 101 and 123kPa. Similarly, the corresponding 7-day ITS for the best moulding condition (taking 28-day ITS of 786kPa), is between 462 and 561kPa.

TRH 4(1996) suggests 7-day ITS values of 200, 250 and 400kPa for natural gravel stabilized layers corresponding to C4, C3 and C2 respectively. Based on the requirements of the TRH 4 (1996) for stabilized natural gravels, it is observed that Mbeya pozzolan mixes can be used in stabilization of bases and sub base layers. However, great care should be taken in deciding the moulding degree of saturation to avoid using low degrees of saturations that can lead to low tensile strength development.

The following can be concluded on the ITS development for ALP mixes made from Mbeya pozzolans:

- The degree of saturation plays a significant role in the DPTT development of ALP mixes made from Mbeya pozzolans in specimens cured between 28 and 180 days.

- High DPTT values are obtained in specimens moulded at 76% saturation compared that obtained at 66 and 94 percent for all specimens cured after 28, 90 and 180 days. Low degrees of saturation (66% and below) result in significantly low tensile strength development.

- Significant DPTT strength is developed in ALP Mbeya mixes between 28 and 180 days regardless of the moulding degrees of saturation.

- Mbeya pozzolan mixes can be used as stabilized materials of categories C2 – C4. However, care must be taken in deciding the moulding degree of saturation. The tensile strength for these mixes shows a high variability with variation in moulding degree of saturation.

- **<u>Cement Mixes</u>**

Figure 4.9 gives the tensile strength development for specimens moulded at different moulding degrees of saturations for mixes cured after 28, 90 and 180 days.

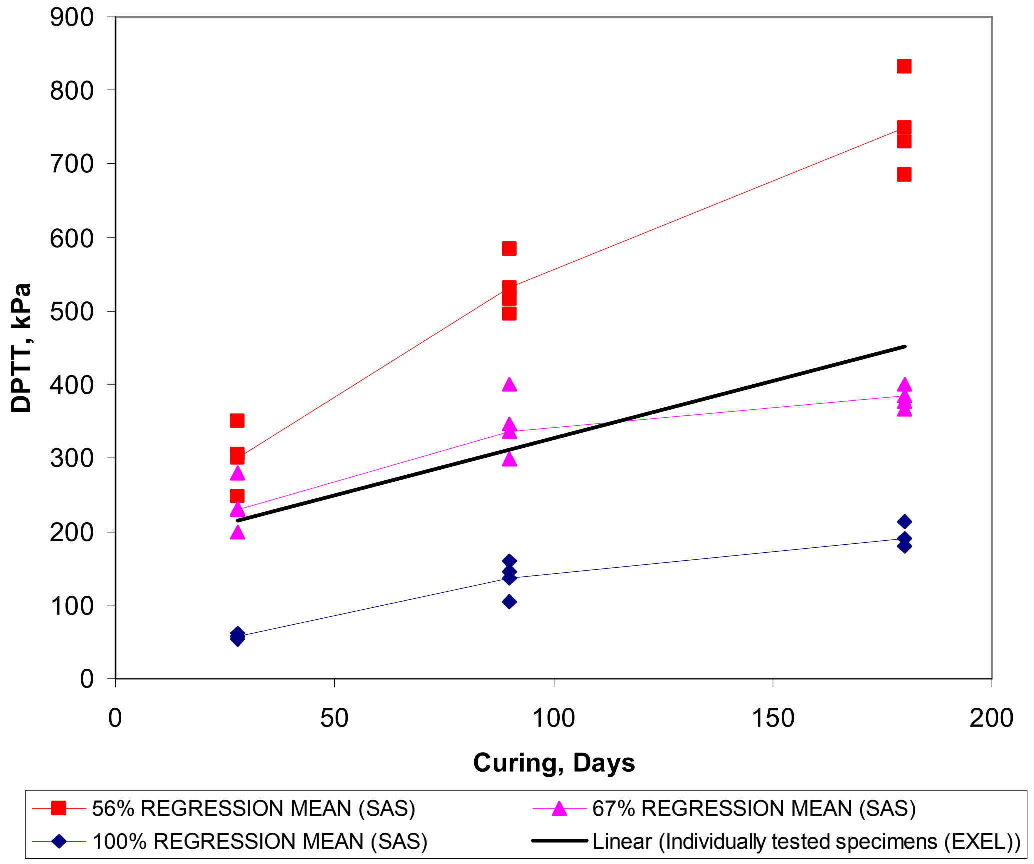


Figure 4.9: Tensile Strength Development for Cement mixes

- Influence of curing duration and degree of saturation

Figure 4.6 shows a decrease in DPTT with an increase in degree of saturation for cement mixes after 28, 90 and 180 days of curing. An increase in DPTT corresponding to a decrease in degree of saturation is observed in all cement mixes. The optimum moisture content for cement mix was observed to exist around 60% saturation. This suggests that there is a significant decrease in DPTT with an increase in degree of saturation beyond the optimum moisture.

- The DPTT development

The average DPTT values between 58 and 301kPa at all moulding degrees of saturations correspond to ITS values between 48 and 247kPa. The ITS values for cement mixes were not considered as the actual ITS development and were not used in comparison as the mix ratio were used only as indicative.

However, the all cement mixes showed a significant dependency of tensile strength development on the curing duration and moulding degrees of saturation. The tensile strength is observed to increase with an increase in curing time but decreases with an increase in the moulding degree of saturation.

The following can be concluded on the DPTT development for cement mixes:

- The degree of saturation plays a significant role in the DPTT development of cement mixes cured between 28 and 180 days.
- All cement mixes show a significant increase in strength with an increase in curing duration for all mixes regardless of the moulding degrees of saturation.
- An increase in the moulding degree of saturation has a harmful effect on the DPTT development in cement mixes cured between 28 and 180 days.

- **Findings related to Tensile Strength Development**

During this study, tensile strength behaviour the ALP mixes was studied, a comparison was made with a control cement stabilized mix. The following conclusions were drawn:

- Mixes made from both Arusha and Mbeya pozzolans show a significant dependency on the moulding degree of saturation. However, different behaviours were observed in the 2-pozzolan mixes.
- Mixes made from Arusha and Mbeya pozzolan attain significant 7-day tensile strength when moulded at varying saturations. However, Arusha pozzolan indicates lower strength after 28 days of curing compared with Mbeya pozzolan

mixes that showed a continuous strength increment up to 180 days of curing. Generally both Arusha and Mbeya pozzolan mixes showed slightly lower strength development between 90 and 180days of curing slightly resembling that of cement mixes.

- Both Arusha and Mbeya pozzolan mixes attain highest tensile strengths when moulded around 80 percent saturation regardless of their optimum.
- With appropriate selection of moulding degrees of saturation, both Arusha and Mbeya pozzolan mixes can be used as stabilized base categories C2, C3 and C4.
- The tensile strength development of pozzolanic mixes from Arusha and Mbeya show different moisture dependency compared with that of cement.

4.3.9. Comparison of the UCS and Tensile Strength Developments

During the study, UCS and tensile strengths for Arusha, Mbeya pozzolan and cement mixes moulded at different moulding degrees of saturation and cured for 28, 90 and 180 days were determined. Results are discussed in the previous sections.

In this study, the comparison between compressive and tensile strengths was done using the UCS results from the study and the equivalent ITS results estimated from the actual DPTT results using the relationship obtained from tensile tests method comparison in sub-chapter 4.3.5.

Figures 4.10 to 4.12 give tensile and compressive strengths relationship for Arusha, Mbeya and cement mixes. A corresponding plot of the ITS vs. UCS evaluated from the equation given by Fulton (2001) for the ratio between the compressive strength and the ITS, both being in MPa, for concrete.

$$f_t = 0.24\, f_c^{2/3}$$

Where,

f_t – ITS, MPa

fc – UCS, MPa

The comparison is done on the relationship given by Fulton (2001) and the actual relationship for each mix is given. Fulton relationship for concrete was used for an indication purpose, as no similar relation for stabilized materials is available. In the figures 4.10 to 4.13 the actual ITS vs. UCS are plotted with an assumption that they follow a linear pattern.

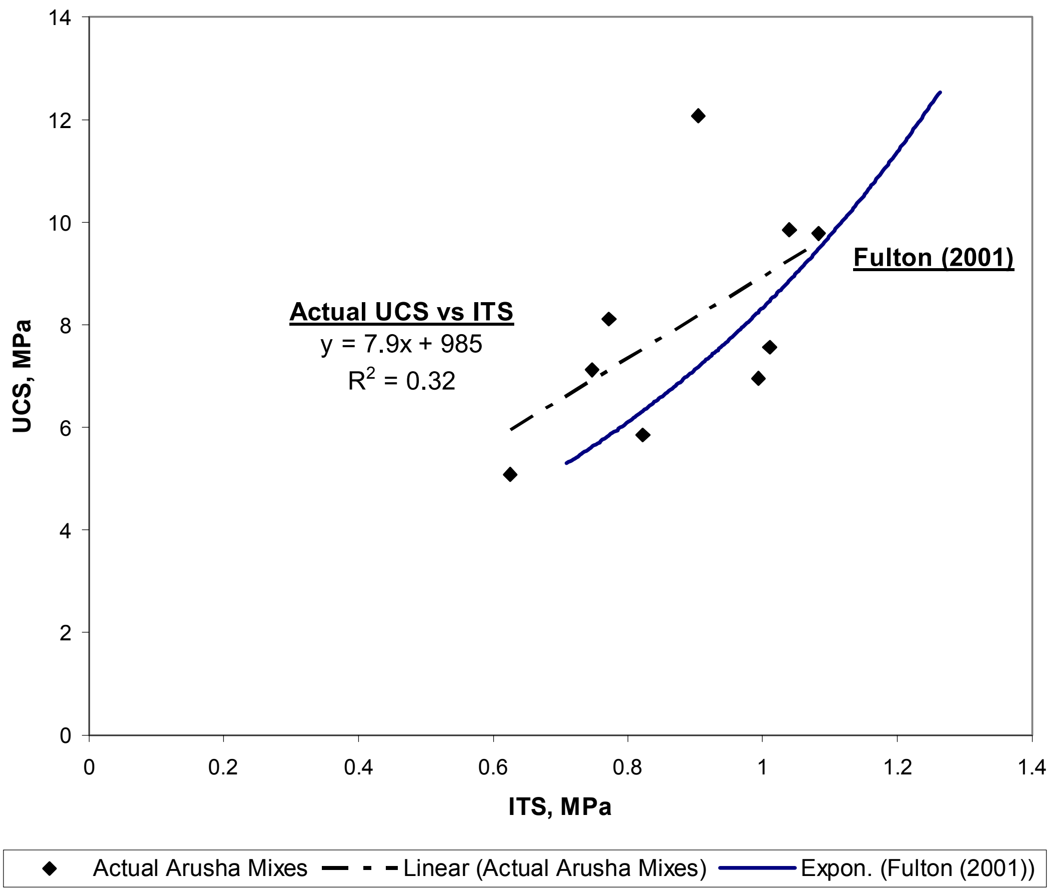


Figure 4.10: UCS and ITS comparison for Arusha mixes

From figure 4.10 it is shown that for Arusha pozzolan mixes, the actual ratio between UCS and corresponding ITS significantly resembles the plot of the estimated ITS from the actual UCS data using the relationship given by Fulton (2001).

The regression analysis between the actual UCS and ITS for Arusha mixes returned an R^2 value of 0.32 suggesting a low correlation of the results. The low correlation between UCS and ITS for Arusha mixes can be related to the fact that for Arusha mixes, little or no strength development was noted in mixes after 28 days of curing.

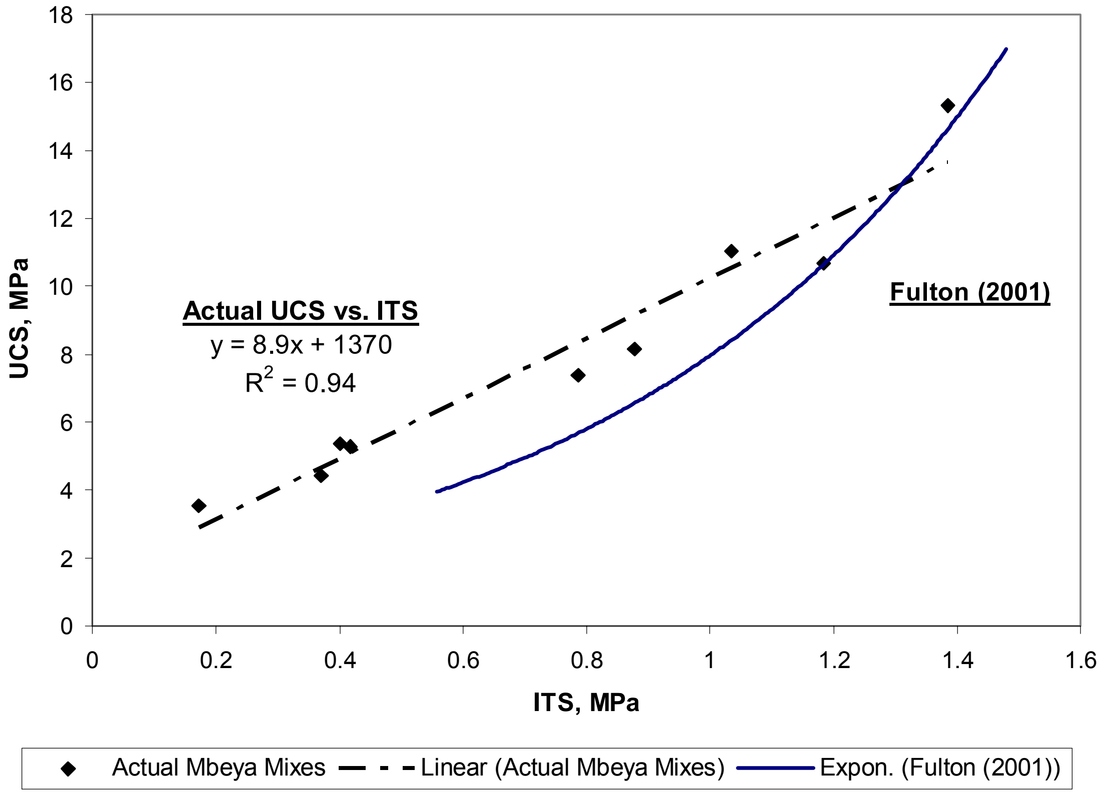


Figure 4.11: UCS and ITS comparison for Mbeya mixes

From figure 4.11, the plot of the actual ratio between UCS and ITS Mbeya pozzolan mixes significantly resemble that of the actual UCS to the estimated ITS as calculated using the relationship by Fulton (2001).

The actual UCS vs. ITS for Mbeya pozzolan mixes is shown in the figure. The regression analysis of the UCS vs. ITS returns an R^2 value of 0.94 implying a higher correlation between UCS and ITS compared to that returned by the Arusha pozzolan mixes.

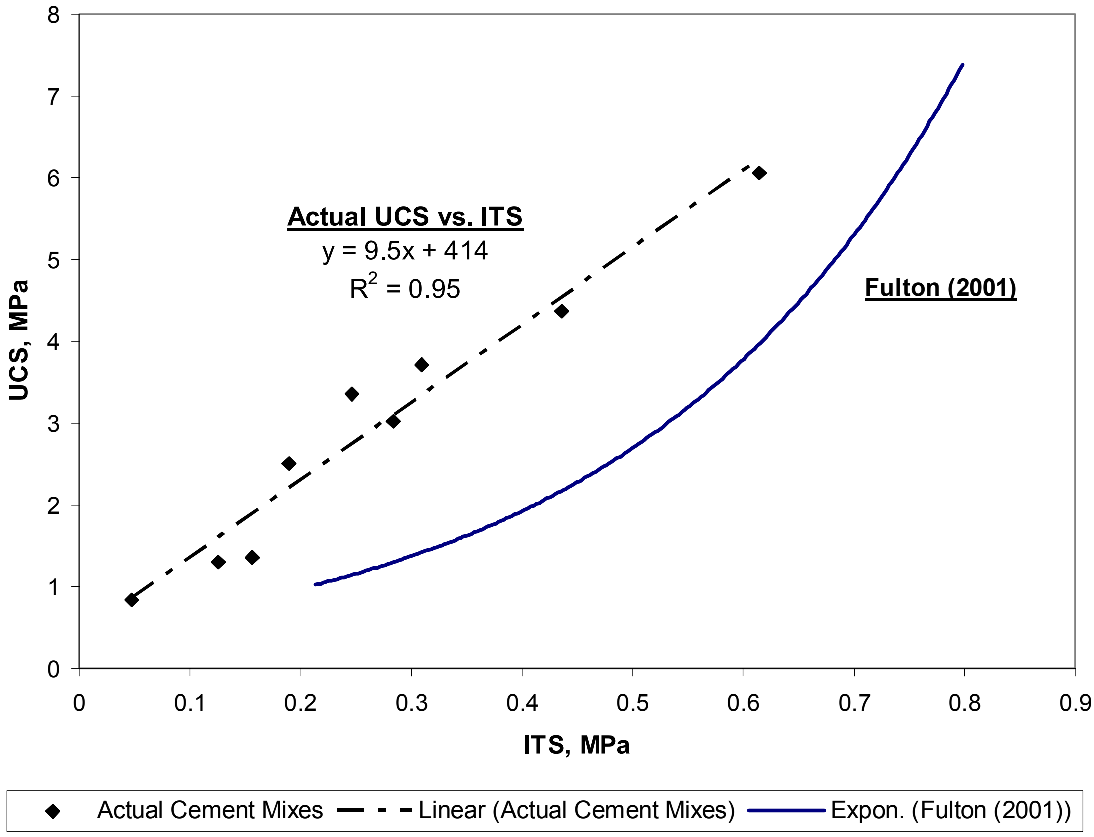


Figure 4.12: UCS and ITS comparison for cement mixes

The figure 4.12, gives the plot of the ratio between the actual UCS vs. ITS and that of the actual UCS vs. calculated ITS using the formula given by Fulton (2001). From the figure it is found that the two plots closely resembles each other for the exception that that of the actual UCS vs. ITS ratio is slightly higher than that from Fulton (2001).

The regression analysis between the actual UCS and ITS for the cement mixes returned R^2 value of 0.95 suggesting a high correlation between the two.

The general UCS and DPTT comparison for all mixes is shown in figure 4.13.

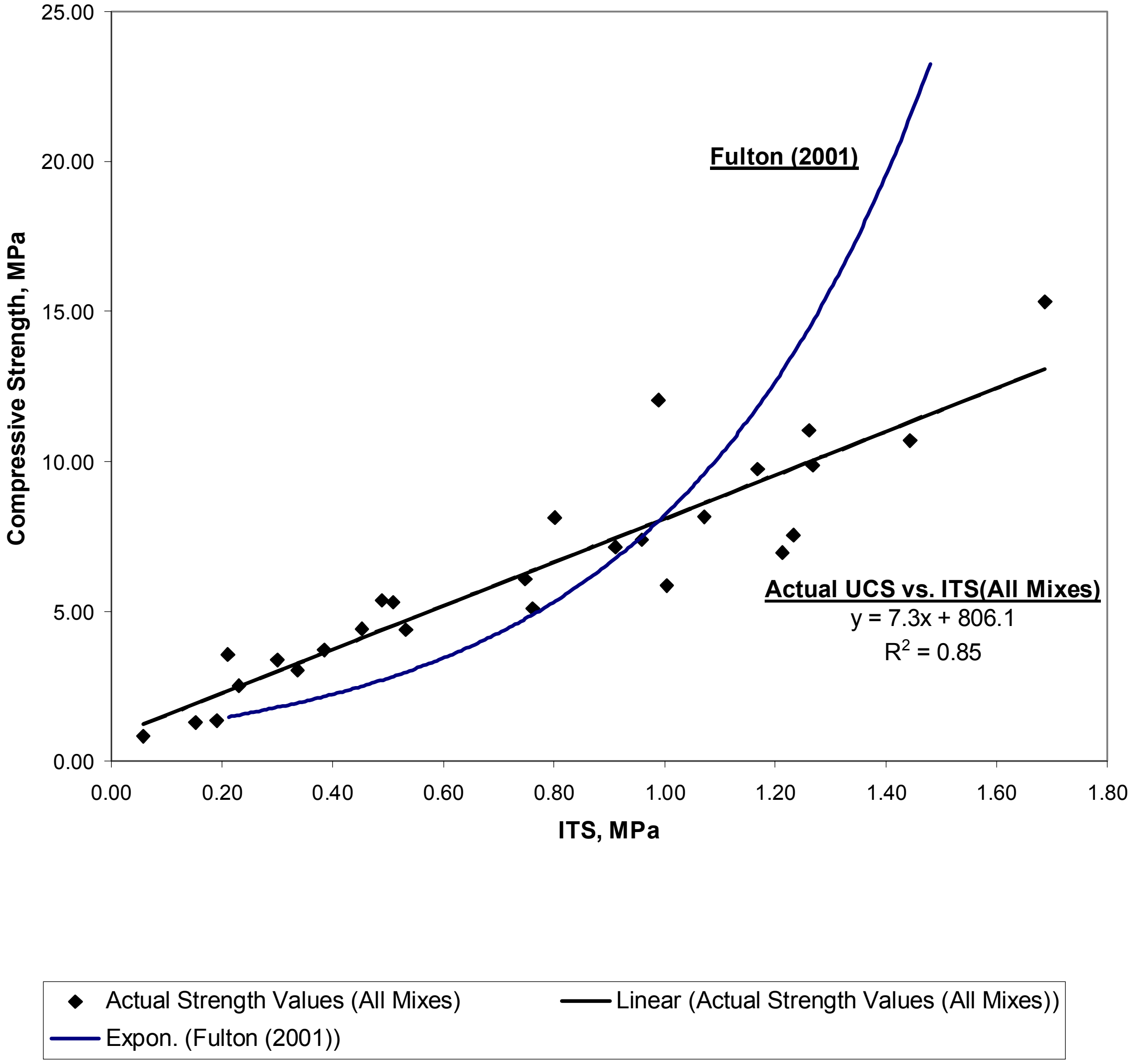


Figure 4.13: UCS and ITS comparison for all design mixes

From figure 4.13, it is observed that the plot of actual UCS vs. ITS for all design mixes returns an R^2 value of 0.85. High R^2 values indicates a high correlation between UCS and ITS for the design mixes.

The plot of the ratio between the actual UCS and the estimated ITS according to Fulton (2001), is plotted along the same graph in figure 4.13. The two plots, the actual UCS vs. ITS and that from Fulton (2001), are observed to significantly resemble each other. The resemblance between the two plots suggests that for all design mixes, the ratio between UCS and ITS follows the relationship proposed by Fulton (2001) for concrete mixes.

4.3.10. Shrinkage Determination

In this study, shrinkage was determined by using the method developed by Grobler and Visser (1994). The method was selected due to its accuracy in measuring shrinkages to 0.002mm. The method consisted of a Demec measuring gauge with targets glued on two sides of the specimen (Plate 4.9).

Plate 4.9 gives the layout of the reference targets and the Demec gauge.

Plate 4.9: The Demec gauge with shrinkage specimen
(Note the positioning of the Targets)

Shrinkage Measurements

A “Demec” measuring device (W.H Mayes & Son) was used to measure the movement between the different targets shown on Plate 4.9. The device is capable of measuring movements with an accuracy of 0.002 mm.

All design specimens were cast and left in the moulds for 7 days. On the 8th day the specimens were demoulded and the first (zero) reading recorded. The measurements were taken every 24 hours for a total of 28 days using the Demec gauge both on the right and left side of the specimens.

Shrinkage measurements were taken on both sides of the specimen as shown in figures 4.14 to 4.15 for different mixes. There was no significant difference between the measurements taken on either side of all specimens indicating that shrinkage occurs uniformly on both sides of the specimens.

All ALP mixes as well as cement stabilized control specimens showed a similar shrinkage pattern with reduced shrinkage in specimens moulded at 60 percent saturation followed by those moulded at 80 percent with those moulded at 100 percent saturation having the highest shrinkage values.

Specimens were found to shrink uniformly from day 8 to 28; however, a slight swelling was observed between 24 and 26 days in all the mixes at all moisture contents. This can be assumed to be possibly a result of the laboratory being busy by students leading to the ambient room being left open for extended times resulting in temperature and humidity variations.

- <u>Arusha Pozzolan Mixes</u>

Arusha pozzolan mixes were found to have similar shrinkage patterns for all specimens moulded at varying moisture contents. The shrinkage measurements for Arusha pozzolan mixes over 28 days of curing are given in figure 4.14.

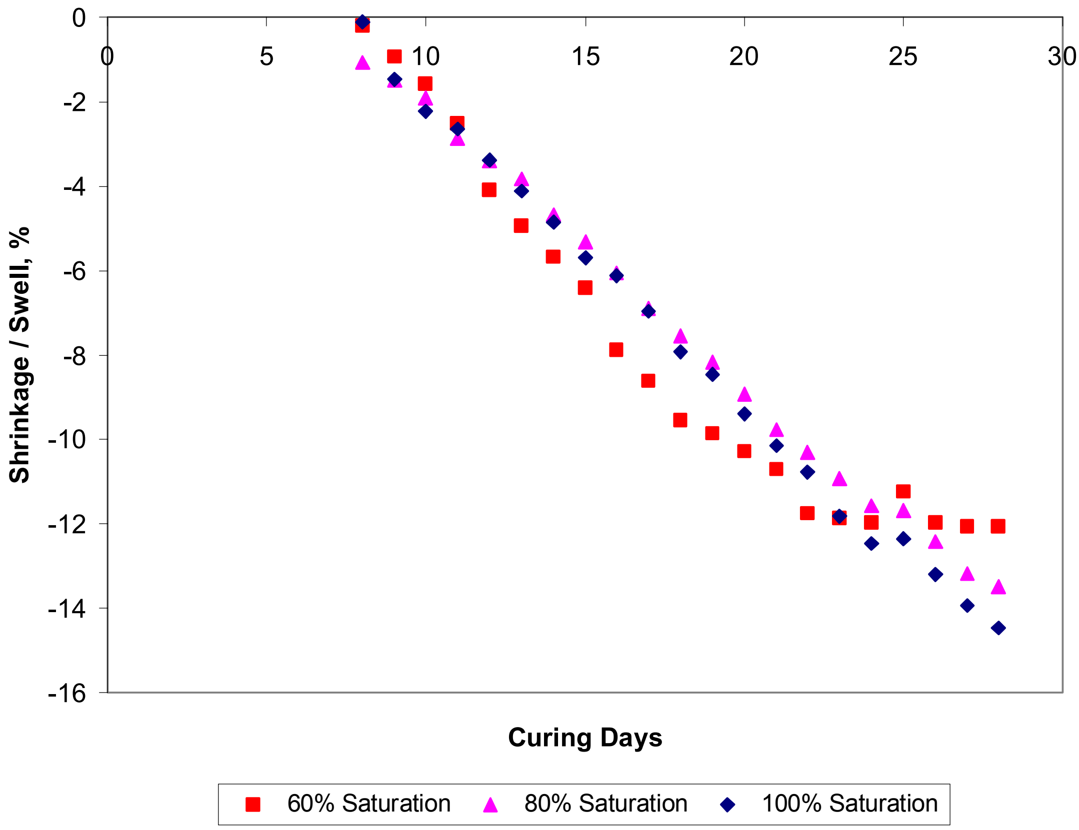


Figure 4.14: 28 day Shrinkage/Swell behaviour of Arusha pozzolan mixes

From Figure 4.14 an insignificant increase in shrinkage with increase in moulding degree of saturation is observed in the Arusha pozzolan mixes. However, shrinkage measurements taken on both sides of the each specimen were found to differ slightly but both were found to follow similar behaviour.

Also Figure 4.14 shows that shrinkage in Arusha pozzolan mixes is lowest when moulded at OMC. Moulding of specimens 20% above OMC results in a significant increase in shrinkage for ALP mixes made from Arusha pozzolans.

For Arusha pozzolan mixes compacted at 60% saturation the shrinkage values are observed to flatten-off after the 24th day. However, the specimens compacted at 80 and 100% saturations continue to shrink. This suggests that after Arusha pozzolan mixes shrinks gradually from the casting until the 24th day after which no significant shrinkage is expected. However, when high compaction moistures are used, Arusha pozzolan mixes should be expected to continue shrinking past 28 days of curing.

- Mbeya Pozzolan Mixes

Shrinkage measurements on Mbeya pozzolan mix specimens are presented in figure 4.15.

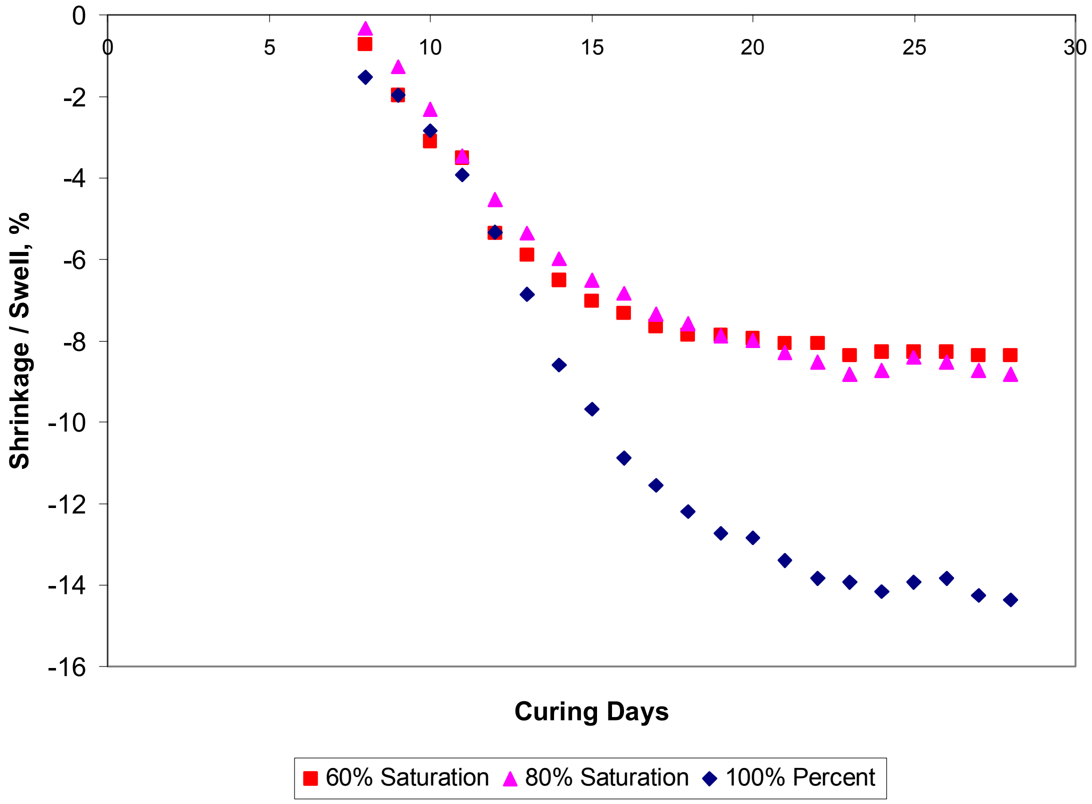


Figure 4.15: 28 day Shrinkage/Swell behaviour of Mbeya pozzolan mixes

From Figure 4.15 a significant increase in shrinkage with an increase in moulding degree of saturation was observed in the mixes. The specimens moulded at 60 and 80% shows the lowest shrinkage compared to the specimens moulded at 100%.

Shrinkage measurements for all specimens made from Mbeya pozzolan flatten-off after 18th day of curing. Although the shrinkage measurements for specimens moulded at 100% saturation are higher than for those of specimens moulded at 60 and 80%, the shrinkage values flatten-off at the 18th day.

- Cement Stabilized Sand

Shrinkage values for cement stabilized sand specimens moulded at varying moulding moisture contents are presented in figure 4.16.

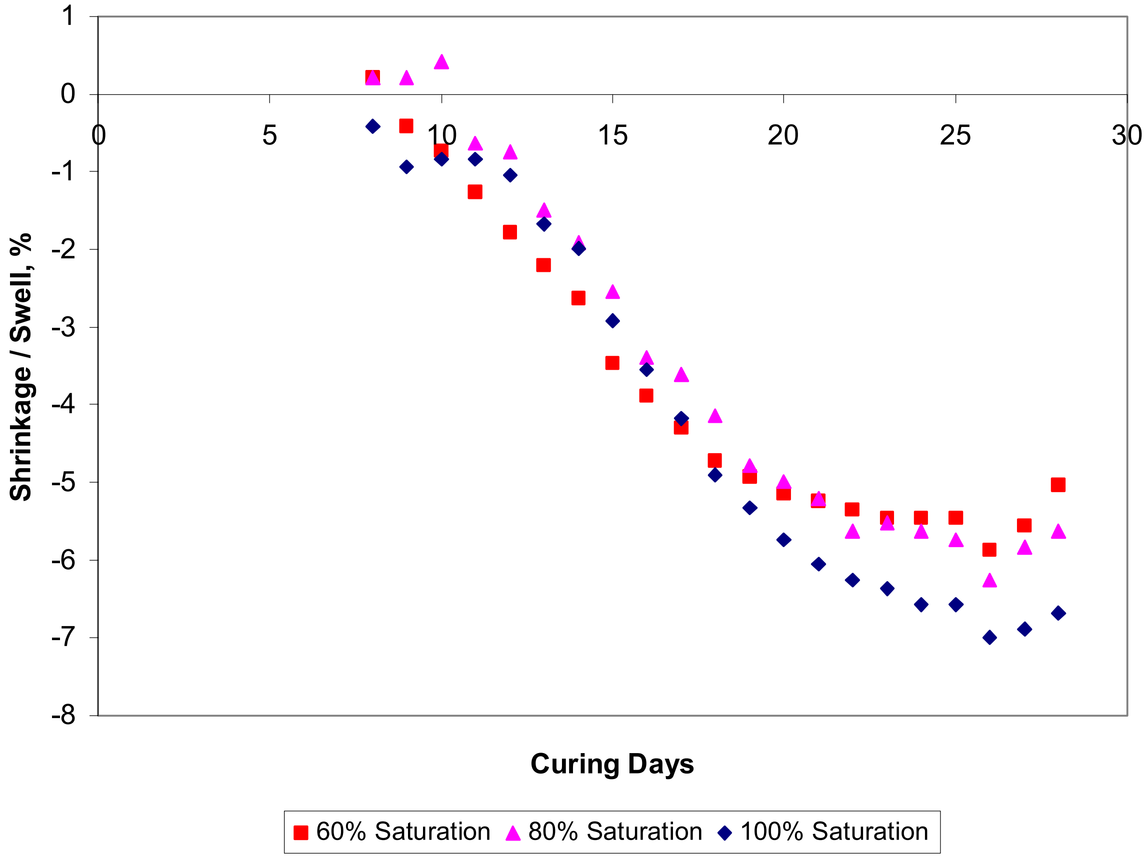


Figure 4.16: 28 day Shrinkage/Swell behaviour of Cement mixes

Figure 4.16 shows the shrinkage measurements for cement mixes moulded at varying degrees of saturation are significantly the same. An increase in the degree of saturation from 60 to 100% results in slightly higher shrinkage values.

Slight swell is observed in cement mixes between 8 and 10 days of curing. However, all specimens show gradual shrinking after 10 days.

For all cement mixes, the shrinkage values flatten-off after 21 days of curing. This suggests that shrinkage of cement mixes flatten-off at the 21st days regardless of the moulding degree of saturation.

The shrinkage behaviour for all design mixes was found to be significantly low for all specimens moulded at 60% saturation. Hence, the shrinkage values for specimens moulded at 60% saturation were plotted in Figure 4.17 for comparison of the two ALP mixes and cement mixes.

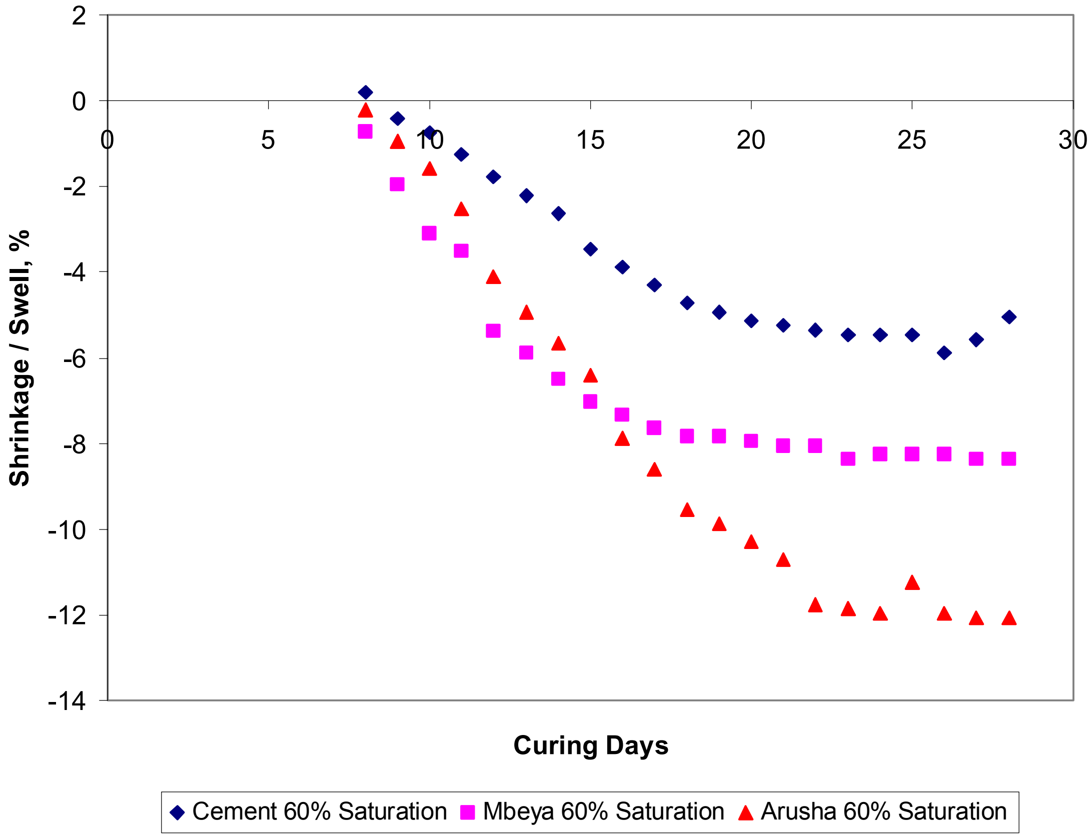


Figure 4.17: 28 day Shrinkage/Swell behaviour of all design mixes

From figure 4.17 it is observed that cement mixes show the lowest shrinkage behaviour compared with Arusha and Mbeya pozzolan mixes for mixes moulded at 60 percent saturation. Mbeya pozzolan mixes showed a lower shrinkage potential compared to Arusha pozzolan mixes.

- **Findings related to Shrinkage Behaviour**

The above observations showed that:

- All design mixes revealed that minimum shrinkage could be achieved when mixes are moulded at lower degrees of saturation. High increase in shrinkage for mixes moulded at 80 and 100 percent saturations suggests that there is a rapid moisture loss in the mixes compared with those moulded at 60 percent saturation where more of the moisture is used in the hydration process.

- Shrinkage values flatten-off between 18 and 24 days of curing for all cement mixes. Both ALP mixes from Arusha and Mbeya pozzolans shows slight shrinkage potential after 24 days for mixes moulded at 100% saturation. This indicates that possible hydration continues after 24 days in ALP mixes compared with cement mixes that shows reduction in hydration. This suggests that for durability purpose (Saricimen *et al*, 1992), stabilized mixes made from Arusha and Mbeya pozzolans should be cured for at least 24 days to avoid any rapid dryness.

- Arusha pozzolan mixes are found to shrink slightly more than the Mbeya pozzolan mixes. This can be associated with high UCS and DPTT achieved by the Arusha pozzolan mixes compared to the Mbeya mixes.

- Both pozzolan mixes show higher shrinkages than that found in cement mixes. The high UCS and DPTT strengths developed in the ALP mixes compared with that of cement mixes can explain this phenomenon which can possibly also be attributed to the high binder ratio used in ALP mixes compared with that in cement mixes.

4.3.11. Scanning Electron Microscopy (SEM)

Specimens used for SEM analysis were collected after crushing of tensile strength specimens for all design mixes after 28, 90 and 180 days of curing respectively. The specimens were stored in airtight sample cans for 24 hours after being collected. The specimens were then placed on SEM analysis plates and coated with gold in three layers each for 20 seconds before being scanned under the electron microscope.

Specimens were studied under the microscope at varying magnification factors between 1 000 and 30 000. After carefully studying the images, it was decided that clear textures for all specimens could be observed best under a 20 000-magnification factor.

The SEM images at a 20 000-magnification factor are presented at the end of this report as Appendix B, and the following subsections gives the details of observations made for all design mixes at varying moulding degrees of saturation after 28, 90 and 180 days of curing.

- **Observation after 28 days of curing**

All cement mixes showed a presence of tobermorite crystals when observed after 28 days of curing. Mbeya mixes showed no clear tobermorite crystals at 28 days compared to Arusha mixes moulded at 100 percent saturation.

- **Observations after 90 days of curing**

A significant increase in tobermorite crystals was noted between 28 and 90 days of curing in all cement mixes. The Mbeya pozzolan mixes moulded at 80 and 100 percent saturation both showed a significant presence of tobermorite crystals. Arusha pozzolan mixes however showed a rather irregular shaped structure for specimens moulded at 80 and 100 percent saturation. Arusha mixes moulded at 60 percent saturation showed a presence of tobermorte crystals similar to those observed in cement mixes.

- **Observations after 180 days of curing**

All design mixes show a significant presence of tobermorite crystals when cured after 180 days. Cement mixes were found to have the highest degree of the tobermorite crystals at all moulding degrees of saturation explaining the fact that the mixes have achieved the highest hydration state at that period.

Arusha pozzolan mixes also showed a high degree of tobermorite crystals at this stage as compared with those observed after 90 days. This implies that at this stage, hydration of the pozzolan is nearly completed and most of its strength is expected to have developed. There was insignificant difference between mixes moulded at 60, 80 and 100 percent saturation. This implies at this stage the moulding moisture has no influence to the hydration of the pozzolans.

Mbeya pozzolan mixes were observed to have the highest presence of tobermorite crystals in those moulded at 80 and 100 percent saturation compared to those found in the mixes moulded at 60 percent. This suggests that hydration of Mbeya pozzolans requires high moulding moisture.

- **Observations after 1 year of curing**

Three specimens made from Arusha and Mbeya pozzolan and cement mixes were left in an ambient chamber for SEM imaging after 1 year of curing. All mixes were moulded at 80 percent saturation.
The cement mix showed little or no significant signs of tobermorite crystals possibly due to complete hydration. Arusha and Mbeya pozzolan mixes both showed a significant presence of tobermorite crystals.
The presence of tobermorite crystals in the two ALP mixes suggests the slow hydration rate in the mixes compared with that of cement mixes.

- **Conclusions on SEM imaging**

The following conclusions can be drawn from the above observations:

- The hydration of pozzolan-lime mixes results in the formation of tobermorite crystals as observed in cement mixes but with a slower rate.

- Both Arusha and Mbeya pozzolans have high rates of hydration when moulded around 80 percent saturation.
- Hydration of Arusha and Mbeya pozzolan mixes showed a significant increase between 90 and 180 days. This means that the hydration process in these mixes is expected to continue beyond 180 days.
- Hydration of Mbeya pozzolan mixes is highly influenced by moulding degree of saturation.
- ALP mixes showed presence of tobermorite crystals after 1 year indicating ongoing hydration of the mixes compared with cement mixes.

4.4. Findings from the Pilot Study

From the pilot laboratory investigations, the following was observed and were used in the main laboratory investigation:

- There is a good correlation between the tensile and compressive strength development for all pozzolan and cement mixes. Thus in the main laboratory study, the UCS test was used to evaluate the strengths for ALP and cement mixes.

- ALP mixes from Mbeya and Arusha pozzolans were observed to attain high tensile and compressive strengths when moulded at drier moulding saturations than their corresponding OMC. This agrees with the findings of other researchers that for sandy soils-lime-fly ash mixes the best compacting moistures are on the drier side (Mateos & Davidson, 1963).

- All ALP and cement mixes achieve appreciable tensile and compressive strength after 28 days of curing. Therefore, in the main study the strength for all mixes was determined after 28 days curing.

- The strengths of all design mixes show a greater dependency on the moulding degree of saturation than on the curing durations. Therefore, in the main laboratory investigation the moulding saturation range was increased for further evaluation of this influence.

- There is a high correlation between the UCS and ITS for ALP mixes made from Arusha and Mbeya pozzolans. The correlation closely resembles that given by Fulton (2001) for concrete.

- Shrinkage of mixes made from Arusha and Mbeya pozzolans becomes constant after 18 and 21 days of curing respectively.

- High pozzolan ratios in ALP resulted in the high shrinkage values found in ALP mixes compared to cement mixes. Significant reduction in shrinkage should be expected when lower pozzolan ratios are used but care must be taken not to jeopardise the resulting strengths.

- A distinct difference is observed between the two pozzolanic materials. Arusha pozzolans shows higher percentage of Aluminium (Al) and Iron (Fe) compared to Mbeya pozzolans. Higher degree of Al and Fe can explain the highest initial strength found in Arusha pozzolan mixes after 28 days of curing compared to that in Mbeya pozzolan mixes. This is due to the fact that Al and Fe crystallizes rapidly leading to initial strength development. On the other hand, Mbeya pozzolans are found to have higher silica (Si) content compared to Arusha pozzolan. The high Si content in Mbeya pozzolans could have resulted in the higher long-term strength development (after 90 days) as compared to the Arusha pozzolan mixes.

CHAPTER 5

DISCUSSION OF THE MAIN STUDY RESULTS

INDEX

5.1 Introduction 5-1

5.2 Indicator Tests 5-1

5.2.1 Sieve Analysis of Mixes 5-2

5.2.2 MDD and OMC 5-6

5.3 Compressive Strength 5-8

5.4 Scanning Electron Microscopy (SEM) 5-17

CHAPTER 5

DISCUSSION OF THE MAIN STUDY RESULTS

5.1. Introduction

The aim of this study was to determine the effect of variation in degree of saturation on the strength behaviour of laboratory prepared ALP mixes. The study was divided in two sections. The pilot study to determine the shrinkage behaviour of pozzolan mixes, relationship between tensile and compressive strengths and CBR behaviour for a single washed sand type. The decision to use one type of sand in the pilot was based on the sand type commonly used in road stabilization in Tanzania.

The results of the pilot study were used in the main study with an addition of two types of sand with different fines fractions and grading. The aim of the main study was to determine the effect of variation in degrees of saturation on the strength properties of laboratory prepared ALP mixes made from the two sand types. The study was done by evaluating the compressive strength development at different degrees of saturation centred around the degree of saturation corresponding to the optimum moisture content for the two sand types commonly used in road construction in Tanzania, washed and unwashed.

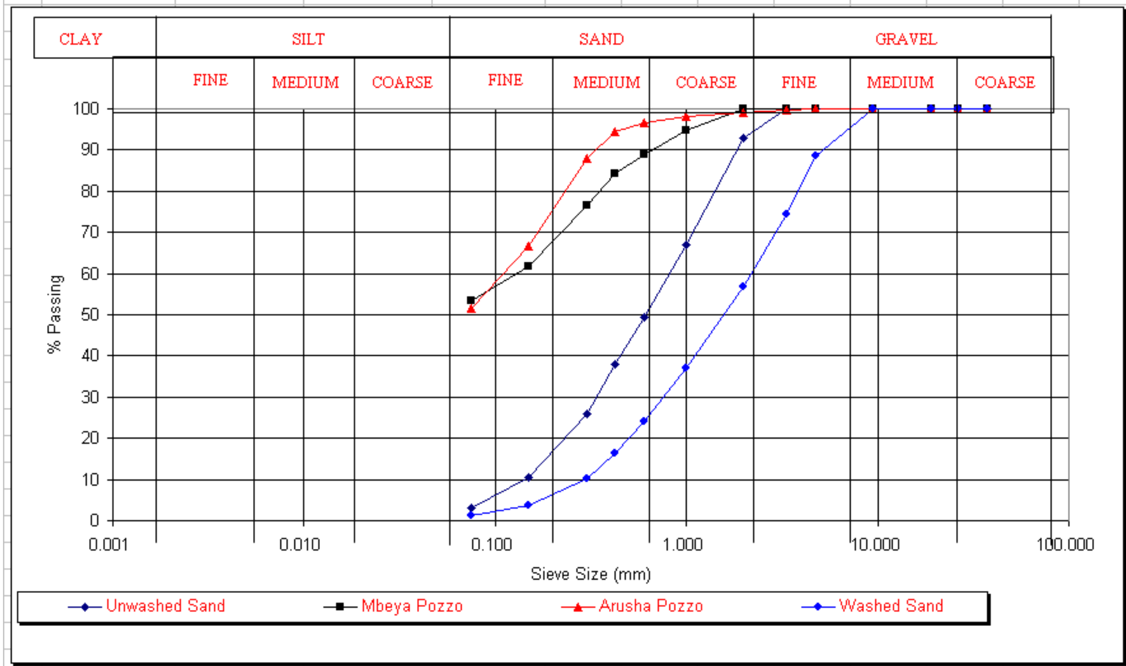
The design mixes for the main laboratory investigations are as shown in table 3.2 in chapter 3.

5.2. Indicator Tests

In this section the results of compaction tests, sieve analysis and SEM observations of both pozzolan mixes made from the two sand samples are presented and discussed. The tests included sieve analysis of the two sandy materials and compaction of the mixes. Although indicator tests were conducted during the pilot study, the same was done for the main study mixes due to the different materials used in the main study.

5.2.1. Sieve Analysis of Mixes

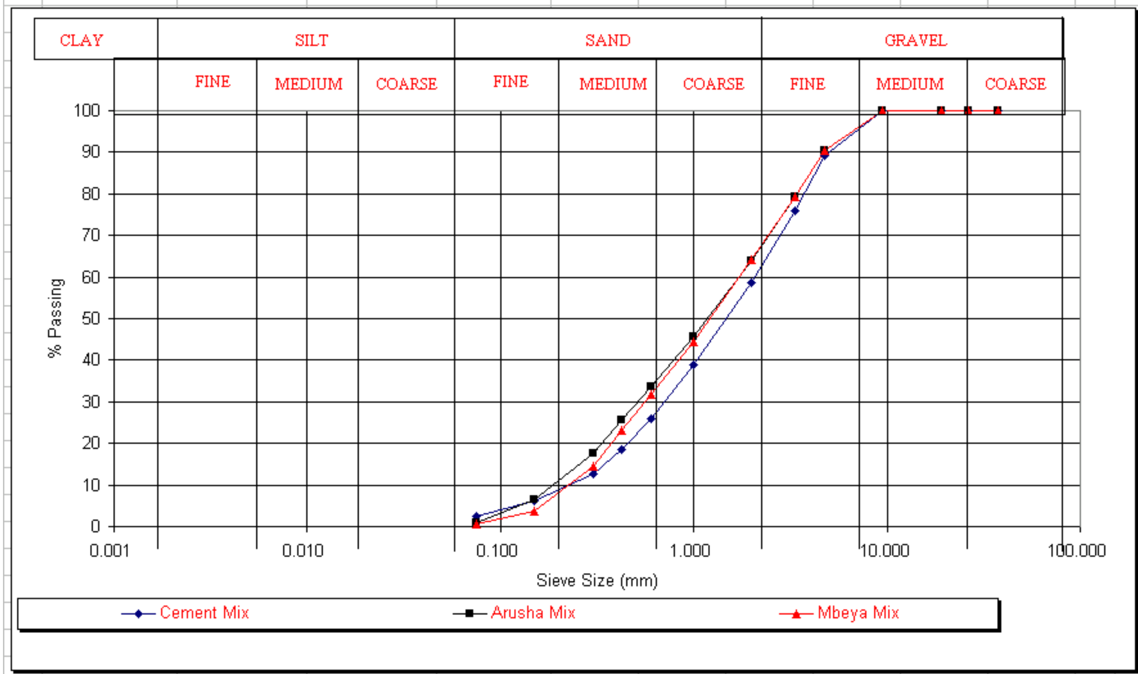
The sieve analysis for the two soils and all main design mixes was determined. Figures 5.1 to 5.3 give the sieve analysis results.



	MATERIAL TYPE			UNWASHED SAND	ARUSHA POZZOLAN	MBEYA POZZOLAN	WASHED SAND
	Sieve	Size	ASTM	% Passing	% Passing	% Passing	% Passing
Grain Size Distribution	37.50	mm	(1½")	100	100	100	100
	26.50	mm	(1")	100	100	100	100
	19.00	mm		100	100	100	100
	9.50	mm		100	100	100	100
	4.75	mm		100	100	100	89
	3.35	mm		100	100	100	75
	2.00	mm		93	99	100	57
	1.00	mm	(No. 30)	67	98	95	37
	0.600	mm (est.)	(No. 40)	49	97	89	24
	425	micron	(No. 50)	38	94	84	16
	300	micron (est.)	(No. 70)	26	88	77	10
	150	micron	(No. 100)	10	67	62	4
	75	micron	(No. 200)	3	52	53	1
% Fine Sand				7	15	9	2
% Medium Sand				16	21	15	6
% Coarse Sand				12	7	7	6
M.D.Density	*g drymax. (kg/m^3)*		Mod	2045	-	-	2041
O.M. Cont.	*w opt. (%)*		AASHTO	5.0	-	-	1.9
SOIL CLASSIFICATION			GM	1.66	0.55	0.63	2.26

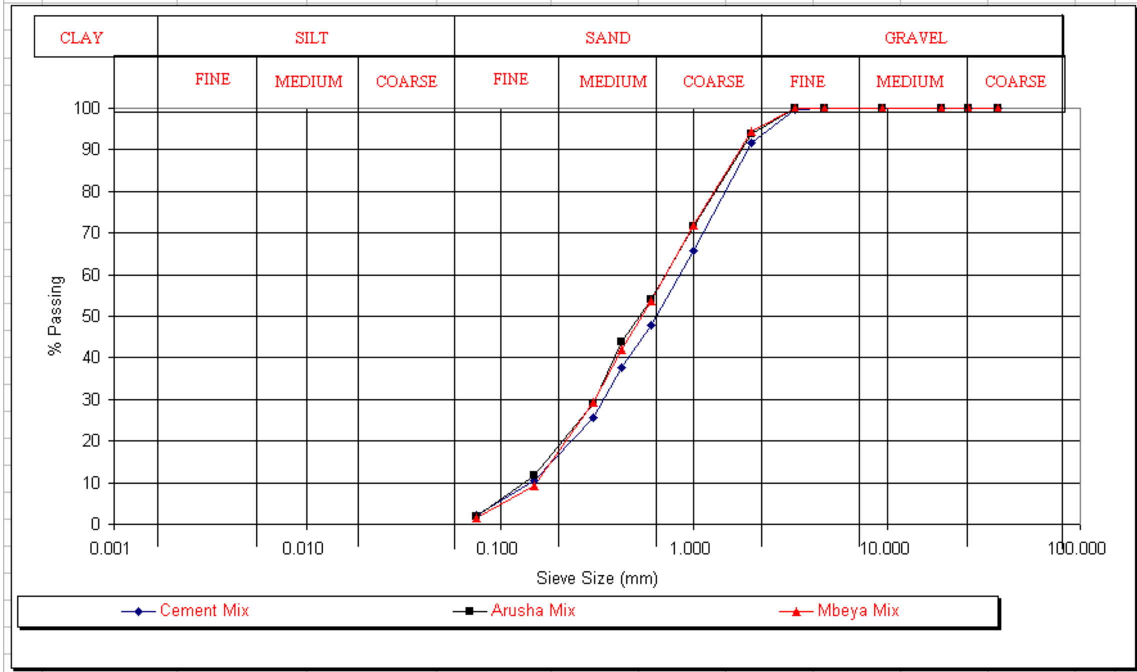
Figure 5.1: Grain size distribution for virgin materials

Figure 5.1 shows that the two sands have a similar fraction of material passing through 0.075mm sieve. However, the washed sand shows a higher percentage of materials between 4.75 and 2.0mm sieves compared with the higher percentage of the materials between 2mm and 0.075mm found in the unwashed sand.



	MATERIAL TYPE			CEMENT MIX	ARUSHA MIX	MBEYA MIX
	Sieve	Size	ASTM	% Passing	% Passing	% Passing
Grain Size Distribution	37.50	mm	(1½")	100	100	100
	26.50	mm	(1")	100	100	100
	19.00	mm		100	100	100
	9.50	mm		100	100	100
	4.75	mm		89	90	90
	3.35	mm		76	79	79
	2.00	mm		59	64	64
	1.00	mm	(No. 30)	39	46	45
	0.600	mm (est.)	(No. 40)	26	34	32
	425	micron	(No. 50)	19	26	23
	300	micron (est.)	(No. 70)	13	18	15
	150	micron	(No. 100)	6	6	4
	75	micron	(No. 200)	2	1	1
% Fine Sand				4	6	3
% Medium Sand				7	11	11
% Coarse Sand				6	8	9
M.D.Density	*g drymax. (kg/m³)*		Mod	2082	2040	2088
O.M. Cont.	*w opt. (%)*		AASHTO	4.6	9.2	8.0
SOIL CLASSIFICATION			GM	2.20	2.10	2.12

Figure 5.2: Grain size distribution for washed sand mixes



	MATERIAL TYPE			CEMENT MIX	ARUSHA MIX	MBEYA MIX
	Sieve	Size	ASTM	% Passing	% Passing	% Passing
Grain Size Distribution	37.50	mm	(1½")	100	100	100
	26.50	mm	(1")	100	100	100
	19.00	mm		100	100	100
	9.50	mm		100	100	100
	4.75	mm		100	100	100
	3.35	mm		100	100	100
	2.00	mm		92	94	94
	1.00	mm	(No. 30)	66	72	72
	0.600	mm (est.)	(No. 40)	48	54	54
	425	micron	(No. 50)	38	44	42
	300	micron (est.)	(No. 70)	26	29	29
	150	micron	(No. 100)	10	12	9
	75	micron	(No. 200)	2	2	1
% Fine Sand				8	10	8
% Medium Sand				15	17	20
% Coarse Sand				12	15	13
M.D.Density	g drymax. (kg/m^3)		Mod	2098	1991	2012
O.M. Cont.	w opt. (%)		AASHTO	6.8	9.2	7.6
SOIL CLASSIFICATION			GM	1.68	1.61	1.62

Figure 5.3: Grain size distribution for unwashed sand mixes

From figures 5.2 and 5.3 it is shown that all cement, Mbeya and Arusha pozzolan mixes all shows similar grading proportions in both washed and unwashed sand mixes. Cement mix was found to be slightly coarser as compared to Arusha and Mbeya pozzolan mixes in both the washed and unwashed sand mix. However, all six mixes were found to have the same percentage of material passing through the 0.075mm sieve.

The fraction of materials passing through the 0.075mm sieve for all design pozzolan mixes is observed to be lower than that expected based on the quantity of fines added from lime and pozzolans. The low fines content (< 0.075mm sieve) could possibly be influenced by:

- Rapid initial setting due to high Al content in both pozzolanic materials;
- Possible initial hydration of the pozzolans due to dampness of the pozzolanic samples since no oven-drying was done prior to sieve analysis; and
- Possible static of pozzolanic materials to lime/sand during sieving.

5.2.2. MDD and OMC

The maximum dry densities (MDD's) and optimum moulding moisture contents (OMC's) were determined for all the main design mixes shown in table 3.2. Table 5.1 gives the compaction results with the corresponding degrees of saturation for all design mixes.

In determination of the degrees of saturations, the specific gravity for all design mixes was taken as 2.65 (Barton *et al*, 2001 & Punmia, 1994).

Table 5.1: MDD and OMC for the main design mixes

Type of Mix	OMC, %	MDD, kg/m3	Degrees of saturation, %
Unwashed sand	5.0	2045	42
Washed sand	1.9	2041	16
Arusha + unwashed sand	9.2	1991	75
Arusha + washed sand	9.2	2040	77
Cement + unwashed sand	6.8	2098	59
Cement + washed sand	4.6	2082	40
Mbeya + unwashed sand	7.6	2012	63
Mbeya + washed sand	8.0	2088	69

Table 5.1 gives the OMC and MDD with their corresponding degree of saturation for all design mixes. Significant difference in optimum moisture content is found between the washed and unwashed sand. The difference in optimum moisture contents between the washed and unwashed sand mixes indicate that for unwashed mixes, the finer fraction results in a higher particle surface area and hence higher moisture requirement for lubrication during compaction.

However, Arusha mixes show similar optimum moisture contents for both unwashed and washed sand mixes. This can be related to high fines contents (< 0.075mm) found in Arusha pozzolan. This fraction is therefore dominant in both sand mixes and hence higher moisture contents are required for lubricating the Arusha pozzolan mixes regardless of the sand type used.

In Mbeya pozzolan mixes, the unwashed sand mix showed a slightly higher optimum compared with the washed mixes. This can be related to coarsely graded Mbeya pozzolans that require slightly lower compaction moisture compared with the finely graded Arusha pozzolan mixes. Here, the fines content of individual sands tend to dictate the optimum moisture of the mixes.

The cement mixes show a higher optimum moisture content in unwashed sand mix compared to the washed sand mix which is reflected in the degree of saturation for the two cement mixes.

5.3. Compressive Strength

Based on the findings of the pilot laboratory study, UCS was used to determine the strength development of all design mixes. The UCS was determined on specimens moulded at varying moulding moisture contents and tested after 28 days of curing. The specimens were prepared in a similar manner to the pilot study specimens and tested after 28 days. The compressive strength results for all main study specimens are given in Table 5.2.

Table 5.2: Compressive Strength results

Mix Properties		Actual compaction degree of saturation, %		Average UCS, kPa	
Mix Type	Compaction moisture added, %	Washed sand mix	Unwashed sand mix	Washed Sand	Unwashed Sand
Cement	3	26	26	86	2600
Cement	6	51	52	3370	5424
Cement	9	77	77	3617	3324
Cement	12	100	100	2118	1757
Arusha	3	25	24	0	2059
Arusha	6	50	49	4309	5949
Arusha	9	75	73	8669	8924
Arusha	12	100	98	5001	5303
Mbeya	3	26	25	1532	2630
Mbeya	6	51	49	6952	6178
Mbeya	9	77	74	4408	4256
Mbeya	12	100	99	2960	2134

From table 5.2, the SAS regression analysis results of UCS for Arusha pozzolan mix made from washed sand moulded at 25% saturation is zero. The value zero was given since all specimens moulded disintegrated before testing. This implies that no significant UCS was developed in the specimens made from the mix.

The UCS results for all design mixes were analysed using the SAS program. The statistical analysis was done using the GLM method (General Linear Methods) where the main and individual effects for the variations in the degrees of saturation for each mix influencing the UCS development was studied.

Figures 5.4 to 5.6 gives the plots of the SAS regression analysis against the compaction degrees of saturations for individual design mixes. A best-fit line is plotted to connect the SAS regression values for all compaction degrees of saturation in each individual mix.

The following subsections discuss the UCS behaviour for each design mix.

- **Arusha Pozzolan Mixes**

Figure 5.4 shows the UCS results for Arusha pozzolan mixed with both washed and unwashed sands moulded at varying degrees of saturation.

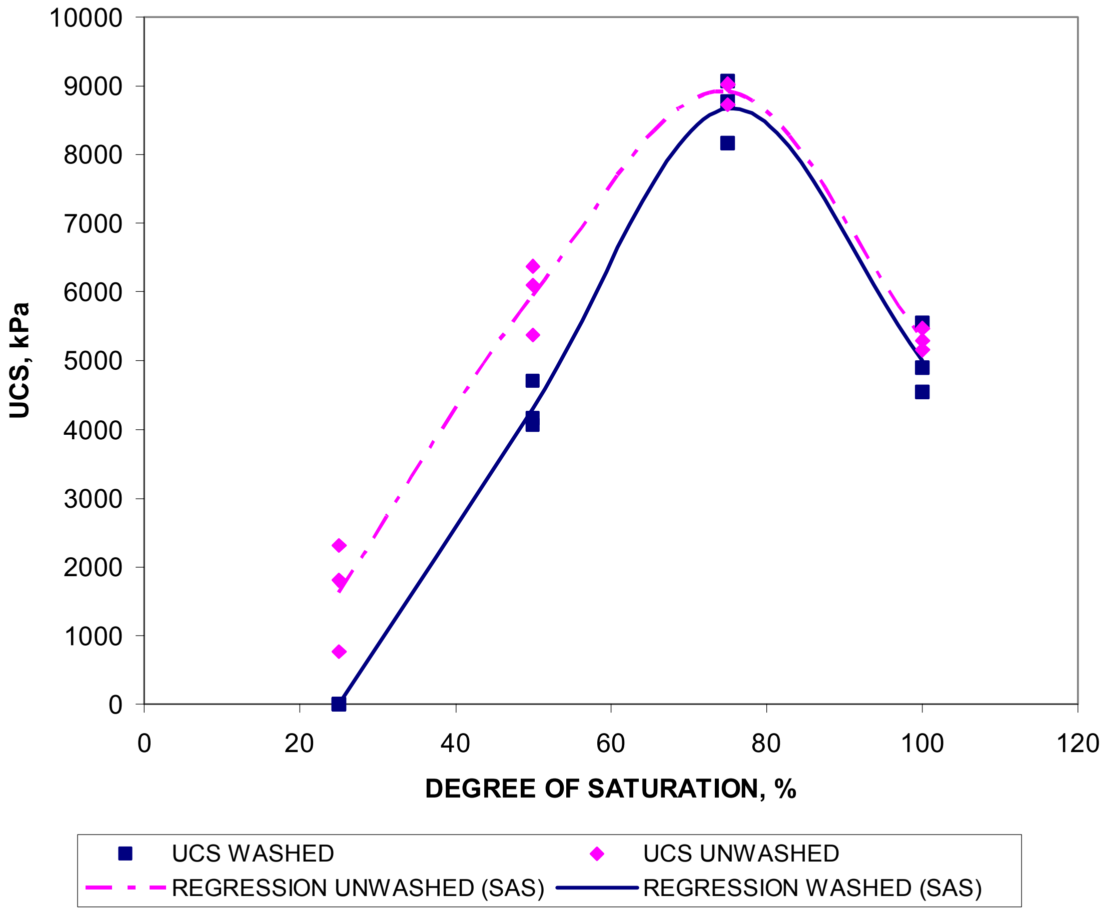


Figure 5.4: UCS development for both Arusha mixes

From the figure 5.4 it is observed that mixes made from Arusha pozzolan with both washed and unwashed river sand attain the highest UCS in specimens moulded at about 76% saturation, corresponding to their optimum moisture contents of 9.2%. This implies that the maximum UCS for both Arusha pozzolan mixes is obtained in specimens moulded at degrees of saturation close to their optimum moisture contents. The figure also show that the UCS decreases uniformly with an increase or decrease in the degrees of saturation corresponding to the OMC. However, care must be taken that further reduction in the degrees of saturation below 60% can result in a significant strength loss.

The SAS analysis results for both mixes are plotted on the same graph as the actual UCS results. The statistical analysis results confirm that Arusha pozzolan mixes attain the highest UCS values regardless of the type of sand used.

Figure 5.4 show that the UCS results for specimens moulded around 25% saturation are lower than those for specimens moulded at 100% saturation. This implies that for Arusha pozzolan mixes, moulding moistures around 76%, result in the highest UCS values. A slight increase in moistures resulting in degree of saturation above 76% should rather be opted for than lower moistures that result in strength decrease. Also, high compaction moistures are advised due to expected moisture loss during compaction and mixing related to field conditions.

The figure also shows that the unwashed-sand Arusha pozzolan mix attains higher UCS values compared to that of the washed-sand Arusha pozzolan mix. However, both mixes show a similar pattern with insignificant difference in peak UCS.

Unwashed sand mixes are observed to attain higher UCS values compared with the washed sand mixes. According to Barton *et al* (2001), there is a relatively small difference in the maximum dry density for washed and unwashed sands. The UCS values in this study for Arusha pozzolan shows that there is no significant difference between washed and unwashed sand mixes. Hence, for Arusha pozzolan the type of sand (washed or unwashed) has little influence to the strength development of the ALP mixes.

Tables 5.3 and 5.4 give the UCS and testing density groupings for Arusha pozzolan mixes. The statistical groupings of the mixes made from Arusha pozzolan with both fine and coarse sand were found to be similar.

Table 5.3: UCS groupings for Arusha pozzolan mixes.

Degree of Saturation, %	Mean UCS, kPa	Waller-Duncan Grouping	Duncan Multiple range grouping
75	8797	A	A
100	5152	B	B
50	5129	B	B
25	814	C	C

Note: Means with same letter are not significantly different.

From the table, it is observed that the UCS for specimens moulded at 75% is observed to have significantly higher strength than that for the other three mixes. However, a statistically comparable UCS development is observed for the mixes moulded at 50 and 100% saturation respectively. The statistical comparison was done by SAS using the Waller-Duncan K-ratio test and Duncan's multiple range tests grouped the two mixes to be statistically the same. The mixes moulded at 25% saturation have the lowest UCS and grouped the lowest.

Table 5.4: Density groupings for Arusha pozzolan mixes.

Degree of Saturation, %	Density, kg/m^3	Waller-Duncan Grouping	Duncan Multiple range grouping
75	2047	A	A
50	1962	B	B
25	1960	B	B
100	1960	B	B

Note: Means with same letter are not significantly different.

It is observed from Table 5.4 that the densities for the mixes moulded at 25, 50 and 100% saturation are statistically the same. However, the density for the mixes moulded at 75% saturation is found to be slightly higher than that for the other mixes.

From the observations from Tables 5.3 and 5.4, it is observed that for Arusha pozzolan mixes, the highest UCS is obtained in mixes moulded at degrees of saturation corresponding to their OMC. However, the influence is significantly low. The Table 5.4 also shows that no significant difference exists in mixes moulded at degrees of saturation around the optimum. The insignificant density variation in the Arusha pozzolan mixes moulded at varying degrees of saturation implies that there is no significant UCS-density relationship.

- **<u>Mbeya Pozzolan Mixes</u>**

Figure 5.5 gives the UCS curves for Mbeya pozzolan mixed with both washed and unwashed river sands moulded at varying moisture contents and cured for 28 days.

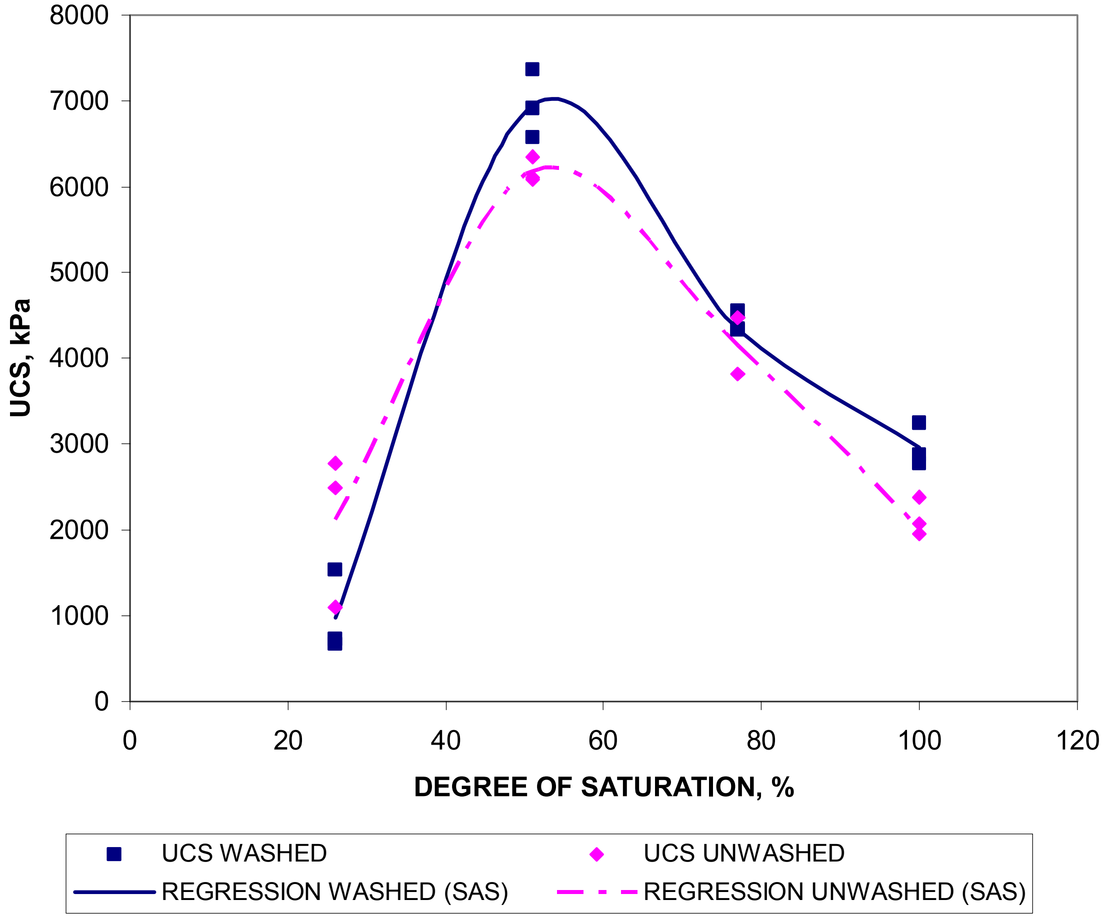


Figure 5.5: UCS development for Mbeya mixes

From figure 5.5 it is observed that both Mbeya pozzolan mixes attain highest UCS values in specimens moulded at degrees of saturation around 50%. The OMC for both Mbeya pozzolan mixes was attained around 70% saturation. This implies that for both washed and unwashed sands, Mbeya pozzolans attain their highest UCS when moulded at degrees of saturation slightly lower than their OMC. SAS analysis confirms the behaviour of both ALP mixes from Mbeya pozzolans as shown in figure 5.5.

For Mbeya pozzolan mixes, the highest UCS is found to be achieved in washed sand mixes. However at lower moulding degrees of saturation the unwashed sand mixes attained slightly higher UCS compared with the washed sand. Table 5.1 shows that the MDD's for Mbeya pozzolan mixes with washed sand mixes are slightly higher than those of the unwashed sand. The difference in MDD is small confirming the findings by Barton *et al* (2001) that there is a slight variation between washed and unwashed sands. However, the difference is accounted for a difference in UCS between the two sand mixes of nearly 1000kPa.

The statistical comparison was done on Mbeya pozzolan mixes with both washed and unwashed sand. The comparison was done on the UCS and density for all design mixes. Tables 5.5 and 5.6 gives the statistical grouping for UCS and densities for the Mbeya mixes.

Table 5.5: UCS grouping for Mbeya pozzolan mixes

Degree of Saturation, %	Mean UCS, kPa	Waller-Duncan Grouping	Duncan Multiple range grouping
51	6565	A	A
77	3812	B	B
100	2697	C	C
26	1547	D	D

Note: Means with same letter are not significantly different.

From Table 5.5 it is shown that a significant difference between all Mbeya pozzolan mixes moulded at varying degrees of saturations exists. Mixes moulded at 51% saturation achieve the highest strength. The optimum moisture content for both pozzolan mixes is achieved at degrees of saturation around 65%. The SAS analysis shows that for Mbeya pozzolan mixes made from washed and unwashed sand, the maximum strength will be achieved when moulded at degrees of saturation around 50%; i.e. drier than their optimum.

Mbeya pozzolan mixes moulded at 26% saturation were found to have the lowest strength compared with all mixes. This suggests that ALP mixes from Mbeya pozzolan require certain minimum moisture content for the hydration process.

Table 5.6: Density grouping for Mbeya pozzolan mixes

Degree of Saturation, %	Density, kg/m^3	Waller-Duncan Grouping	Duncan Multiple range grouping
51	2056	A	A
26	1969	A	A
77	1652	B	B
100	1498	B	B

Note: Means with same letter are not significantly different.

The SAS analysis given in table 5.6 show that the density of the mixes compacted at 51 and 26% saturation are not significantly different. Similarly, the mixes compacted at 77 and 100% saturation are similar. However, an increase in degree of saturation between 77 and 100% result in a significant density reduction compared with that of mixes moulded at 26 and 51% saturation.

Table 5.5 show that there is significant UCS differences for all the design mixes made from Mbeya pozzolans. The UCS of mixes moulded at 26% saturation attained the lowest strength but their density is found to be higher than that of the mixes moulded at 77 and 100%. This suggests that the strength for Mbeya pozzolan mixes is independent of the maximum dry density for both washed and unwashed sand mixes.

- <u>Cement Mixes</u>

Figure 5.6 gives the UCS curves for cement mixed with both washed and unwashed river sands moulded at varying moisture contents and cured for 28 days.

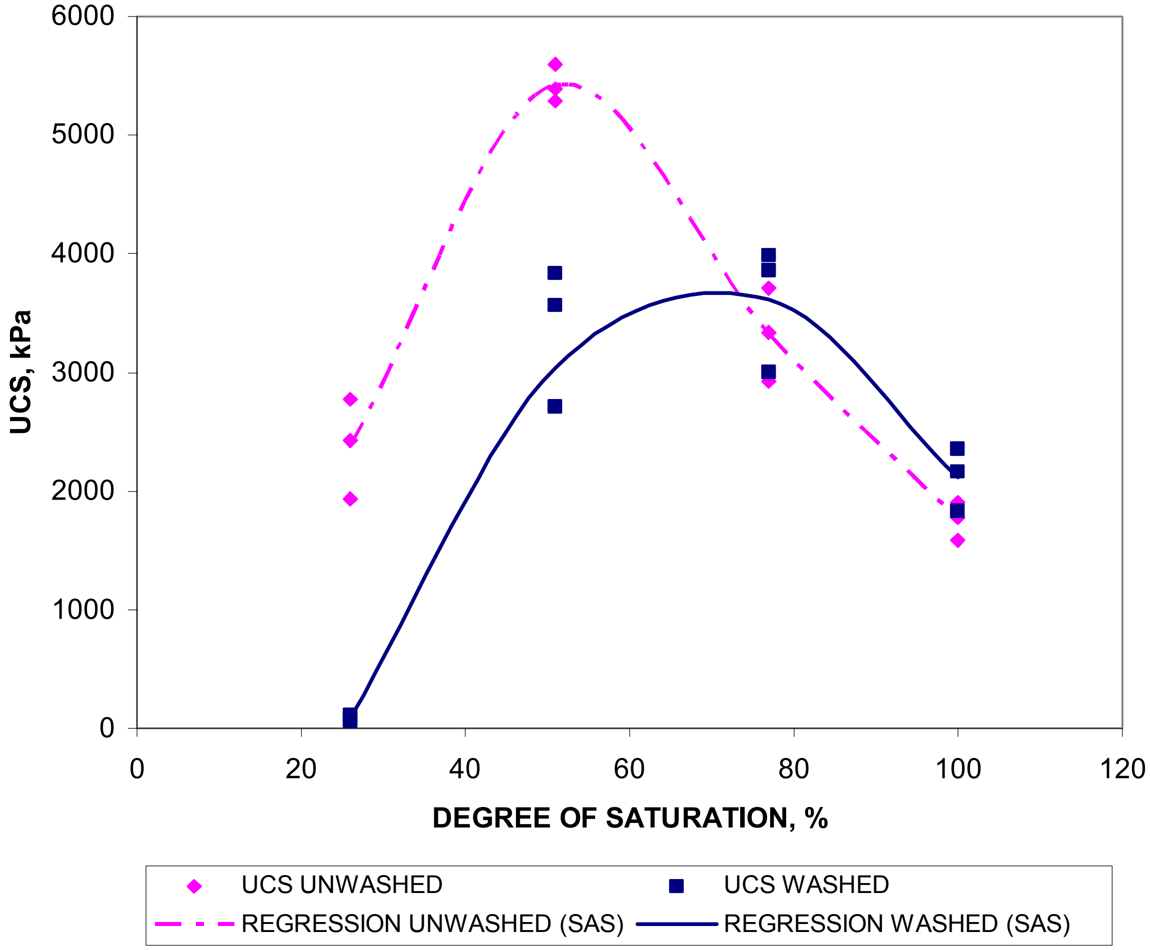


Figure 5.6: UCS development for Cement mixes

From figure 5.6 it is observed that the cement mixes attain their maximum UCS at different degrees of saturation. The cement mixes made from washed sand attain their highest UCS when moulded around 70% saturation compared with the unwashed sand mixes around 50%. The difference in degrees of saturation corresponding to the maximum UCS for the two mixes suggests that the performance of cement stabilized sand mixes depends significantly on the grading of sand.

- **Findings related to UCS determination**

During this study, the behaviour of UCS for ALP mixes was studied; a comparison was made with a control cement stabilized mix. The following conclusions can be made from the UCS development in the individual mixes:

- The fines content of the sand insignificantly influences the performance of ALP sand mixes. However, in cement stabilization, the fines content of the sand significantly influences their performance. This suggests that in natural pozzolan stabilization of sandy materials, the quantity of fines has insignificant role in the strength development.
- Mbeya pozzolan mixes should be moulded at degrees of saturation around 50 percent regardless of their OMC for the best performance.
- Arusha pozzolan should be moulded at degrees of saturation closer to their OMC for the best performance.

5.4. SEM Analysis

Scanning electron microscopy was done in all design mixes after testing. A small representative sample was taken from all specimens tested and was studied under the microscopy. The SEM images are presented in Appendix C at the end of this report.

- **Arusha pozzolan mixes**

All Arusha pozzolan mix specimens showed significant tobermorite crystals present for both washed and unwashed sands. However, more tobermorite crystals presence was found to depend rather on the moulding degrees of saturation than the type of sand used.

The highest tobermorite crystals were found in specimens moulded around 75% saturation for both washed and unwashed sand mixes. This corresponds to the highest UCS that was achieved by the specimens moulded at this saturation.

- **Mbeya pozzolan mixes**

All Mbeya pozzolan mixes showed the presence of tobermorite crystals for both washed and unwashed sandy mixes similar to Arusha mixes. Similarly, the degree of the tobermorite crystals development was found to depend on the degree of saturation than on the sand type.

Higher tobermorite crystals were found in mixes made from unwashed sand than in washed sand materials. In unwashed sand mixes, the crystals are observed in all design mixes compared with only in mixes moulded at 77 and 100 percent for washed sand mixes.

- **Cement mixes**

All cement mixes made from both washed and unwashed sand mixes showed a high presence of tobermorite crystals compared with the two-pozzolan mixes.

- **Findings related to SEM analysis**

- The tobermorite crystal formation in the pozzolanic materials depends on the degrees of saturation and not on the type of sand used. This agrees with the fundamental definition of pozzolan hydration is dependent on the quantity of lime and not their particle size.
- Higher tobermorite crystals formation determines the UCS development in pozzolanic mixes.
- The well-defined tobermorite crystals found in all cement mixes suggest that although the hydration of pozzolanic materials results in the formation of tobermorite crystals, the degree of the crystals developed by cement mixes is higher than that for pozzolanic materials regardless of the strength achieved by the mixes.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

INDEX

6.1 Conclusions 6-1

6.2 Recommendations for Further Research 6-4

6.3 Recommendations for Construction of ALP mixes 6-6

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1. Conclusions

The literature survey indicated that the use of ALP as road bases and sub bases gives an economical yet environmental friendly solution to the current road construction problems. The following conclusions are drawn from the laboratory studies:

- **The moulding degree of saturation plays an important role in performance of stabilized layers.**

The study found that the tensile and compressive strength development of ALP mixes depends significantly on the moulding degree of saturation regardless of the type of sand used. All pozzolans have high compressive and tensile strengths when moulded at degrees of saturation around 50 and 70% regardless of their OMC.

- **The correlation of tensile and compressive strengths.**

The study showed that a good correlation between tensile and compressive strengths in ALP mixes exists, which is similar to that of cement mixes. Hence, the UCS can be used in evaluating the strength development of the ALP mixes following the specifications set for cement stabilization.

- **ALP mixes form tobermorite crystals during hydration.**

ALP mixes from Arusha and Mbeya pozzolan were found to form tobermorite crystals similar to those in cement mixes during their hydration. The degree of tobermorite crystal formation influences the strength development in the ALP mixes. Both Arusha and Mbeya pozzolan mixes were found to show similar patterns to cement mixes as observed under SEM.

- **ALP mixes attain significant strength after 28 days of curing.**

Both Mbeya and Arusha pozzolan mixes attain significant strength after 28 days of curing similar to conventional cement mixes. Thus in pozzolanic stabilization, no extended curing duration is necessary after 28 days.

- **Double punch tensile test method is suitable for use in lightly stabilized soils.**

The DPTT test method is suitable for determination of the tensile strength of stabilized materials. The method is simple and can be performed in the laboratory using normal laboratory equipment.

- **Fines content of sand has no significant influence on the strength development of ALP mixes.**

The fines content of sandy materials does not affect the strength development of ALP mixes. Both Arusha and Mbeya pozzolans were found to develop effectively the same strength results when used to stabilize both washed and unwashed sands. However, a significant UCS difference exists between cement mixes made from washed and unwashed sands suggesting a possible strength gain in cement mixes made with finer materials.

- **Chemical composition of the pozzolan has an influence on the rate of strength development of ALP mix**

Chemical composition of an individual pozzolan has a significant influence in the rate of strength development of the resulting ALP mix. Higher degrees of Al and Fe can result in highest initial strength due to the fact that Al and Fe crystallizes rapidly leading to initial strength development. On the other hand, higher Si content results in the higher long-term strength development beyond 90 days of curing.

- **CBR in ALP mixes should not be used to predict the fresh strength.**

The study showed that high CBR values for ALP mixes were achieved in mixes moulded at the lowest degrees of saturation. This suggests that the CBR is mainly due to high inter-particle friction existing in the mixes with lowest moistures compared with those with high moisture degrees that provide lubrication to the particles. Hence, as in cement mixes, CBR tests should not be used as the final strength predictor in the ALP mixes. Also the slow strength development in ALP mixes does not contribute to the CBR of these mixes in the fresh state.

- **Shrinkage of ALP mixes.**

During the study, high binder ratios were used in all ALP mixes resulting in high strength and shrinkages. It is known that the addition of pozzolanic materials results in reduction in shrinkage but it was not the case in this study due to the high binder ratios used. Thus the shrinkage measurements obtained in this study should not be regarded as the representative behaviour of ALP mixes due to the high binder quantity used.

The two pozzolans from Arusha and Mbeya showed slightly different shrinkage behaviours in terms of flatten-off duration. Shorter flatten-off time should be expected in Mbeya pozzolan as compared to Arusha possibly due to significant chemical composition existing between the two sources.

- **Strength development of ALP mixes resembles that of cement mixes.**

The study showed that the strength development of ALP mixes from Arusha and Mbeya pozzolans significantly resembles that of cement mixes. However, significant differences exist in terms of moulding moisture contents for maximum strength and hence the specifications used for cement mixes should not be directly utilised in ALP mixes. Changes must be done in the specifications to suit the ALP behaviour.

The final compaction moistures for the two pozzolanic materials should be determined separately due to the significant chemical composition difference between the two. High initial strength development should be expected in Arusha pozzolan than in Mbeya mixes due to high Aluminium and Iron content in Arusha pozzolans. However, high long-term strengths should be expected in Mbeya pozzolan mixes due to possible high silica content in Mbeya pozzolans compared to Arusha pozzolans.

- **The ratio between UCS and ITS closely resembles that given by Fulton**

The relationship between the compressive and tensile strengths for ALP mixes closely follows the relationship given by Fulton (2001) for lightly stabilized layers.

The comparison between the ITS and UCS was assumed to be linear due to the fact that the actual comparison could not be obtained and hence linearity was assumed.

- **SEM Analysis.**

The formation of tobermorite in the pozzolanic materials depends on the degrees of saturation and not on the type of sand used. Higher tobermorite formation determines the UCS development in pozzolanic mixes.

The well-defined tobermorite crystals found in all cement mixes suggests that although the hydration of pozzolanic materials results in the formation of tobermorite crystals, the degree of the crystals developed by cement mixes is higher than that for pozzolanic materials regardless of the strength achieved by the mixes.

6.2. Recommendations for Further Research

The conclusions drawn from this experimental work indicated that lower moulding moisture contents have a significant effect on the strength of fresh and hardened ALP mixes. However, the following recommendations are proposed for further research for the use of ALP mixes in road construction:

- Further research is required to identify an economic activator for the pozzolanic materials. This is due to the fact that the current practice of using industrial manufactured calcium hydroxide (pure lime) as an activator is found to be costly compared with cement stabilization in terms of the unit mass. Similarly, the environmental degradation associated with lime production is that from cement production. Thus, the need for obtaining a cheaper but environmentally friendlier activator is important.

- Further research should be conducted in determining curing effects on the strength behaviour of ALP mixes. The influence of specimen size, curing temperature and humidity as well as the compaction effects should be studied with respect to fresh state strength (CBR) and long-term strength (compressive and tensile strengths).

- Further research on the drying shrinkage of ALP mixes in order to determine the true shrinkage behaviour related to the actual field conditions of free drying (uncontrolled conditions). During this study, all specimens were cast and cured in controlled conditions that are not a true representation of aggressive field conditions.

- Use of natural pozzolans in stabilization of other materials such as crushed sand, silts and clays should be studied as this study utilised only washed and unwashed river sands. The selection was based on the experience from Tanzania where natural sand has been used in pozzolanic stabilization.

- The durability of ALP mixes should be evaluated by considering the influence of varying compaction moisture in relation to the wet-dry test and accelerated carbonation of the mixes.

- The individual properties of natural pozzolans should be studied and compared to those of fly ash in terms of autogenous shrinkage.

- A proper UCS vs. ITS model should be determined since the model given by Fulton (2001) was found to give a vague indication of the relationship.

6.3. Recommendations for Construction of ALP mixes

This study showed that ALP mixes made from natural pozzolans from Arusha and Mbeya regions in Tanzania are feasible for use in stabilization of sandy materials.

The following are recommended for their construction:

- **Differences exists between the Arusha and Mbeya pozzolans**

The study has shown that there is a significant difference between the behavior of the stabilized mixes made from Arusha and Mbeya pozzolans. The two mixes showed different strength development with variation in the compaction degree of saturation. Hence during utilization of the pozzolanic mixes of Arusha and Mbeya origin, different compaction moisture specifications should be used.

- **Shrinkage of both Arusha and Mbeya pozzolans is greatly influenced by the quantity of binder used.**

The study showed that high shrinkage measurements were observed in both Arusha and Mbeya pozzolanic mixes compared to that of cement mixes. However, this could be related to the significantly high binder content used in the ALP mixes as compared to that used in cement mixes. Thus during construction, the quantity of binder selected should be carefully selected to avoid excessive cracking, while still maintaining the minimum strength and durability requirements.

- **Strength development for ALP mixes resembles that of cement but at a slower rate**

The tensile and compressive strength development for ALP mixes resembles that showed by cement mixes, the rate of strength development is significantly slower and continues for a longer period (beyond 180 days). Thus in ALP mixes construction, care should be taken not to use high binder content for achievement of target 28 day strength similar to that of cement but rather allowance should be made for that extra long-term strength development. However, care needs to be taken that construction traffic or early opening of the road does not cause premature damage to the stabilized layers.

REFERENCES

INDEX

List of references 7-1

Bibliography 7-8

References

- ACI Committee 232, 1994. Use of Natural Pozzolans in Concrete, American Concrete Institute, report no. ACI 232.1R-94.

- Andres R. J., Gibala R., Barenberg E. J. 1976. Some Factors Affecting the Durability of Lime-Fly Ash-Aggregate Mixtures, Transportation Research Record 560, Transportation Research Board, Washington DC.

- Ballantine R. W., Rossouw A. J. 1989. Stabilisation of Soils - A practical manual detailing all up-to-date techniques in all aspects of the subject, Pretoria Portland Cement Limited, Johannesburg, South Africa.

- Barton M.E., Cresswell A., Brown R. 2001. Measuring the Effect of Mixed Grading on the Maximum Dry Density of Sands, Geotechnical Testing Journal, London, UK.

- Chen W. F. 1970. Extensibility of Concrete and Theorems of Limit Analysis, Journal of Engineering Mechanics Division, Proc. ACSE, vol. 96, no. EM3.

- Chikwira S. 1991. Stabilisation of Kalahari and river sand with lime and fly ash, Mining and Engineering, September.

- Costa U., Baroni F. 1994. Pozzolanic Activity of Rhyolites from Monte Arci (Sardinia), il Cemento, submitted to Rivista Scientifica E Di Informazione Industriale Dell'Associazione Italiana Tecnico Economica Del Cemento (A.I.T.E.C).

- COWI Tanzania Ltd. 2000. Pilot Study for Possible use of Locally Available Pozzolan in Tanzania, submitted to Ministry of Works, the United Republic of Tanzania.

- Cumberledge G., Hoffman G. L., Bhajandas A. C. 1976. Curing and Tensile Strength Characteristics of Aggregate-Lime-Pozzolan, Transportation Research Record 560, Transportation Research Board, Washington DC.

▪ Day R. L., Shi C. 1994. Influence of the Fineness of Pozzolan on the Strength of Lime-Natural Pozzolan Cement Pastes, The University of Calgary, submitted to Cement and Concrete Research.

▪ Dempsey E. J. 1972. A Programmed Freeze-Thaw Durability Testing Unit for Evaluating Paving Materials, Journal of Materials, vol. 7, no. 2.

▪ Department of Transport RSA. 1986. Standard Methods of Testing Road Construction Materials, TMH 1, 2nd ed., National Institute for Transport and Road Research (NITRR), Pretoria, South Africa, ISBN: 0 7988 3653 9.

▪ Department of Transport RSA. 1985. Guidelines for Road Construction Materials, TRH14, Pretoria, South Africa, ISBN: 0 7988 3311 4.

▪ Department of Transport. 1997. Draft TRH 12: Flexible Pavement Rehabilitation Investigation and Design, Pretoria, South Africa.

▪ Department of Transport. 1986. Draft TRH 13: Cementitious stabilizers in road construction, Pretoria, South Africa.

▪ Department of Transport. 1996. Draft TRH 4: Structural design of flexible pavements for interurban and rural roads, Pretoria, South Africa.

▪ Eriksen K. I. 2000. Rehabilitation of the Dar es Salaam - Mlandizi Road and Improvement of Wami bridge Approaches, COWI Consulting Engineers and Planners, submitted to Ministry of Works, The United Republic of Tanzania, report no. P-030346S-E3-01.

▪ Eriksen K., Larsen E. E. 2000. Natural Pozzolans and Blast-Furnace Slag in Road Construction - Experience from Tanzania, Proc. Pavement Seminar for the Middle East and North Africa Region, Amman, Jordan.

▪ Eriksen K., Zhang W., Thogersen F., Macdonald R. A. 1999. Feasibility of Pozzolan-Stabilised Pavements in Developing Countries, Proc. First Road Technology Transfer Conference in Africa, Arusha, Tanzania.

- Evans E.J., Inglethorpe S.J., Wetton P.D. 1999. Evaluation of Pumice and Scoria Samples from East Africa as Lightweight Aggregates, submitted to British Geological Survey, report no. WG/99/15R.

- Fairweather V. 1975. Fly ash Pavements, Runways to take off? American Society of Civil Engineers, vol. 45, no. 8.

- Fang H. Y., Chen W. F. 1972. Further Study of Double-Punch Test for Tensile Strength of Soils, Proc. 3rd Southeast Asia Conference on Soil Engineering.

- Fang H. Y., Chen W. F. 1971. New Method for Determination of Tensile Strength of Soils, Transportation Research Record 345, Transportation Research Board, Washington DC.

- Fleischer W., Sodeikat C., Springenschmid R. 1994. Conclusions from the Long-Time Behaviour of Cement-Bound Road Bases, Proc. Seventh International Symposium on Concrete Roads, Vienna, Austria.

- Fraay A., Vogelaar P., Zeilmaker J. 1989. The Use of Fly ash produced from the Combustion of Powdered Coal as a Stabilization Material, Proc. Third CANMET / ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Trondheim, Norway.

- French Highways Directorate. 1997. French Design Manual for Pavement Structures, Translation of the December 1994 French version of the technical guide: Conception et dimensionnement des structures de chaussee - Guide techniques ed., Laboratoire Central des Ponts et Chausse'es & Service d'Etudes Techniques des Routes et Autoroutes, Centre de la Securite et des Techniques Routieres, Paris, France.

- Fulton's Concrete Technology, 2001. 8th edition, edited by Addis B., Owens G., 2001. Cement and Concrete Institute, Midrand, South Africa.

- Gambhir M. L. 1992. Concrete Manual, 4th edition, Dhanpat Rai & Sons, New Delhi, India.

▪ Grobler J. A. 1994. The Influence of Compaction Moisture on Shrinkage in Stabilized Materials, University of Pretoria, Dissertation submitted in partial fulfilment of the requirements for degree of Master of Engineering.

▪ Grobler J. A., Visser A. T. 1996. The influence of compaction moisture on shrinkage in stabilized materials, Journal of South African Institution of Civil Engineering, vol. 38, no. 3.

▪ Hannant D. J. 1972. The Tensile Strength of Concrete, A Review Paper, The Structural Engineer, vol. 50, no. 7.

▪ Hewlett P. C. 1998. LEA's Chemistry of Cement and Concrete, 4th edition, Arnold, London, ISBN: 0 340 56589 6.

▪ Higgins D. D., Kinuthia J. M., Wild S. 1998. Soil Stabilization using Lime-Activated Ground Granulated Blast Furnace Slag, Proc. Sixth CANMET / ACI International Conference, vol. 2, Bangkok, Thailand.

▪ Hoffman G. L., Cumberledge G., Bhajandas A. C. 1976. Establishing a Construction Cut-off Date for Placement of Aggregate-Lime-Pozzolan, Transportation Research Record 593, Transportation Research Board, Washington DC.

▪ Khanna S. K., Justo C. E. G. 1991. Highway Engineering, edited by Mrs. Indira Khana & Mrs. Lalitha Justo, 7th ed., Nem Chand & Bros, Roorkee (Uttar Pradesh), India, ISBN: 81-85240-43-4.

▪ Lea F. M. 1971. The Chemistry of Cement and Concrete, Chemical Publishing.

▪ Mateos M., Davidson D.T. 1963. Compaction Characteristics of Soil-Lime-Fly Ash Mixtures, Highway Research Record 29, Highway Research Board, Washington DC.

▪ Mehta K. 1997, Durability - Critical Issues for the Future, Concrete International, vol. July.

▪ Mehta P. K. 1987. Natural Pozzolans, edited by V.M. Malhorta ed., The Canada Centre for Mineral and Energy Technology (CANMET), Ottawa, Canada.

▪ Meyers J. F., Pichumani R., Kapples B. S. 1976. Fly-Ash - A Highway Construction Material, submitted to U.S. Department of Transportation, Implementation Package no. 76-16.

▪ Nakayama H., Handy R. L. 1967. Factors Influencing Shrinkage of Soil-Cement, Transportation Research Record 86, Transportation Research Board, Washington DC.

▪ Natt G. S., Joshi R. C. 1984. Properties of Cement and Lime-Fly Ash Stabilized Aggregates, Transportation Research Record 998, Transportation Research Board, Washington DC.

▪ Nicholson P. G., Kashyap V., Fujii C. F. 1994. Lime and Fly Ash Admixture Improvement of Tropical Hawaiian Soils, Transportation Research Record 1440, Transportation Research Board, Washington DC.

▪ Philleo R. E. 1989. Slag or Other Supplementary Materials? Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, Proc. 3rd International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Trondheim, Norway.

▪ Price B. 1996. Stronger, bigger, better, Concrete.

▪ Punmia B.C. 1994. Soil Mechanics and Foundations, edited by Jain A.K. & Jain A.K., 13th ed, Laxmi Publications Pvt. Ltd, New Delhi, India.

▪ Pylkkanen K. 1995. Granulated Blast Furnace Slag in Base Course of Low-Volume Roads, Proc. Sixth International Conference on Low-Volume Roads, vol. 2, Minneapolis, Minnesota USA.

▪ Rols S., Mbessa M., Ambroise J., Pera J. 1999. Influence of Ultra-Fine Particle Type on Properties of Very-High-Strength Concrete, Proc. Second CANMET / ACI International Conference, Granado, RS, Brazil.

▪ Sakula J. H., Sauni J. T. M. Undated. Feasibility Study for Lime and Pozzolana Production in Mbeya, University of Dar es Salaam, Tanzania.

▪ Saricimen H., Maslehuddin M., Shamim M., Khan M. S., Al-Tayib A. J. 1992. Effect of Curing on the Permeability of Plain and Pozzolanic Concrete, Proc. Fourth CANMET / ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul, Turkey.

▪ Savage P.F., Visser A.T., 2001. Stabilised Materials and Compaction, Course notes at the University of Pretoria (SGC 788), South Africa.

▪ Schmidt M., Vogel P. 1990. Pavements with Industrial By-Products: Technological, Environmental and Constructive Aspects, Proc. Sixth International Symposium on Concrete Roads, Madrid, Spain.

▪ Sharpe G. W., Deen R. C., Southgate H. F., Anderson M. 1985. Pavement Thickness Designs Using Low-Strength (Pozzolanic) Base and Sub base Materials, Transportation Research Record 1043, Transportation Research Board, Washington DC.

▪ Thanikachalam V. 1973. Discussion of paper Measurement of Tensile Strength of Compacted Soils, Southeast Asian Society of Soil Engineering, vol. 4, no. 1.

▪ Vandewalle L., Mortelmans F. 1992. The Effect of Curing on the Strength Development of Mortar Containing High Volumes of Fly Ash, Proc. Fourth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, vol. 1, Istanbul, Turkey.

▪ Wang M. C., Huston M. T. 1971. Direct Tensile Stress and Strain of a Cement Stabilized Soil, Transportation Research Record 379, Transportation Research Board, Washington DC.

▪ Wang M. C., Kilareski W. P. 1979. Field Performance of Aggregate-Lime-Pozzolan Base Material, Transportation Research Record 725, Transportation Research Board, Washington DC.

▪ Winterkorn H. F., Fang H. Y. Undated. Soil Technology and Engineering Properties of Soils: Foundation Engineering Handbook ed., Van Nostrand Reinhold Co.

- Wright P. J. F. 1955. The Effect of the Method of Test on the Flexural Strength of Concrete, Concrete Research, no.11.

- Yoder E. J., Witczak M. W. 1975. Principles of Pavement Design, 2 ed., John Willey & Sons Inc, New York, USA, ISBN: O-471-97780-2.

Bibliography

The following papers and publications provide further background to the topic:

- Aiqin W., Chengzhi Z., Ningsheng Z. 1999. The theoretic analysis of the influence of the particle size distribution of cement system on the property of cement, CEMENT and CONCRETE RESEARCH, no. 29, USA.

- Akman M. S., Mazlum F., Esenli F. 1992. A Comparison Study of Natural Pozzolans Used in Blended Cement Production, Proc. Fourth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul, Turkey.

- Alegre S., Lechunga J. A., Perxas J. 1992. Correlation between Water Permeability and Compressive Strength of Concrete with Spanish Cercs Fly Ash, Proc. Fourth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul, Turkey.

- Anday M. C. 1963. Curing Lime-Stabilized Soils, Stabilization of Soil with Lime and Fly Ash, Transportation Research Record 29, Transportation Research Board, Washington DC.

- Bergeson K. L., Pitt J. M., Demirel T. 1984. Increasing Cementitious Products of a Class C Fly Ash, Transportation Research Record 998, Transportation Research Board, Washington DC.

- Biernacki J. J., Williams P. J., Stutzman P. E. 2001. Kinetics of Reaction of Calcium Hydroxide and Fly Ash, ACI Materials Journal, Technical Paper, no. 98-M37.

- Clark I. H., Heckroodt R. O. 1987. Evaluation of Western Cape clay materials as artificial pozzolans, The CIVIL ENGINEER in South Africa, September.

- Corte J. F., Havard H., Kergoet M. 2001. The French Technical Guide on Soil Stabilisation with Lime and Hydraulic Binders, Proc. First International Symposium on Subgrade Stabilisation and In-Situ Pavement Recycling Using Cement, Salamanca, Spain, 1 - 4 October.

- COWI Tanzania Ltd. 2001. Draft report on Feasibility Study for usage of Local Materials for Roads and Airstrip on Mafia Island, Tanzania, February.

- COWI Tanzania Ltd. 2000. Seminar reports to present the Findings of the Pilot Study for Possible Use of Locally Available Pozzolan in Tanzania, Tanzania, June.

- Dawson A. R., Elliot R. C., Rowe G. M., Williams J. 1995. Assessment of Suitability of Some Industrial By-Products for Use in Pavement Bases in the United Kingdom, Transportation Research Record 1486, Transportation Research Board, Washington DC.

- de Sensale G. R., Dal Molin D. C. C. 1999. Study of Influence of Rice-Husk Ash on Compressive Strength of Concrete at Different Ages, Proc. Second CANMET/ACI International Conference, Gramado, RS, Brazil.

- Downey P., Dratva T. 1989. 30 Years Experience with Natural Pozzolans in Chile, Proc. Third CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag & Natural Pozzolans in Concrete, Trondheim, Norway.

- Fang H. Y., Hirst T. J. 1973. A method for determining the strength parameters of soils, Transportation Research Record 463, Transportation Research Board, Washington DC.

- Gaspar L., Gaspar L. Jr. 1995. Use of Industrial By-Products in Economical Standard Low-Volume Road Pavements, Proc. Sixth International Conference on Low-Volume Roads, Minneapolis, Minnesota, USA, 25 - 29 June.

- Ghosh S. N., Sharma R. N., Mathur V. K., Sharma R. C., Mohan L. 1992. Activation of Indian Fly Ash, il Cemento, vol. 2.

- Gichaga F. J. 1991. Deflections of Lateritic Gravel-Based and Stone-Based Pavements of a Low-Volume Tea Road in Kenya, Transportation Research Record 1291 vol. 2, Transportation Research Board, Washington DC.

▪ Giergiczny Z., Werynska A. 1989. Influence of Fineness of Fly Ashes on Their Hydraulic Activity, Proc. Third International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Trondheim, Norway.

▪ Gregory C. A., Saylak D., Ledbetter W. B. 1984. The Use of By-product Phosphogypsum for Road Bases and Sub bases, Transportation Research Record 998, Transportation Research Board, Washington DC.

▪ Halstead W. J. 1990. Overview of the Use of Fly Ash Concrete in Highway Construction, Transportation Research Record 1284, Transportation Research Board, Washington DC.

▪ Hanehara S., Tomosawa F., Kobayakawa M., Hwang K. 2001. Effects of water/powder ratio, mixing ratio of fly ash, and curing temperature on pozzolanic reaction of fly ash in cement paste, CEMENT and CONCRETE RESEARCH, no. 31, USA.

▪ Hooton R. D. 1987. The Reactivity of Hydration Products of Blast-Furnace Slag, edited by V. M. Malhorta, CANMET, Ottawa, Canada.

▪ Jambor J. 1990. Pore Structure and Strength Development of Cement Structures, CEMENT and CONCRETE RESEARCH, vol. 20, no. 6, USA.

▪ Joshi R. C., Lohtia R. P. Undated. Types and Properties of Fly Ash, Progress in Cement and Concrete, Mineral Admixtures in Cement and Concrete, vol. 4, edited by Dr. S. L. Sarkar, New Delhi, India.

▪ Joubert P. B., Van Steenderen W., Maree J. H. 1987. Experience with Ash in Road Construction at the Matla Power Station, Ash - a valuable resource papers, vol. 4, Ash handling/transportation-Roads-Engineering Fill-Marketing, CSIR Conference Centre, Pretoria, South Africa, 2-6 February.

▪ Justnes H., Ardoullie B., Hendrix E., Sellevold E. J., Van Gemert D. 1998. The Chemical Shrinkage of Pozzolanic Reaction Products, Proc. Sixth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slags and Natural Pozzolans in Concrete, Bangkok, Thailand.

- Khanna S.K., Justo C. E. G. 1997. Highway Material Testing (Laboratory Manual), 4th edition, Nem Chand & Bros, Roorkee (Uttar Pradesh), India, ISBN: 81-85240-21-3.

- Kirk S., Edwards A., Sese J. Undated. Making Good Use of Volcanic Ash in the Philippines, Transport Research Laboratory.

- Kolias S., Karahalios A. 2001. Mechanical Properties of Soils Stabilised with High Calcium Fly Ash and Cement, Proc. First International Symposium on Subgrade and in-Situ Pavement Recycling Using cement, Salamanca, Spain, 1 - 4 October.

- Lefort M. 1996. Technique for limiting the consequences of shrinkage in hydraulic-binder-treated bases, Reflective Cracking in Pavements, Design and Performance of performance of overlay systems, Proc. Third International RILEM Conference, Maastricht, The Netherlands, 2 - 4 October.

- Lim S. N., Wee T. H. 2000. Autogenous Shrinkage of Ground-Granulated Blast-Furnace Slag Concrete, ACI Materials Journal, no. 97-M67, October.

- Lochhart J., Marso M., Ozimok J. 2000. Pennsylvania Turnpike Reconstruction - Part III, Lime-Pozzolan meets the Reconstruction Challenge, USA.

- Mateos M., Davidson D. T. 1963. Compaction Characteristics of Soil-Lime-Fly Ash Mixtures, Highway Research Record 29, Highway Research Board, Washington DC.

- Marusin S. L. 1992. Influence of Fly Ash and Moist Curing Time on Concrete Permeability, Proc. Fourth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul, Turkey, May.

- Mehta P. K. 1998. role of Pozzolanic and Cementitious Material in Sustainable Development of the Concrete Industry, Proc. Sixth CANMET/ACI International Conference in Fly Ash, Silica Fume, Slag and Natural Pozzolans in Tanzania, Bangkok, Thailand.

- Miller R. H., Couturier R. R. 1963. Measuring Thermal Expansion of Lime-fly Ash-Aggregate Compositions Using SR-4 Strain Gages, Highway Research Record 29, Highway Research Board, Washington D. C., USA.

▪ Mitchell G. A. 1990. Fly Ash in Road Construction, Proc. Annual Seminar of the Institution of Materials Technicians, Durban, South Africa, August.

▪ Moore D. 1999. The Roman Pantheon: The Triumph of Concrete, First edition.

▪ Mora E. P., Paya J., Monzo J. 1992. Influence of Different Sized Fractions of a Fly Ash on Workability of Mortars, Proc. Fourth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Istanbul, Turkey.

▪ Mtui A. L., Kawiche G. M. 1983. Portland Pozzolana Cement, working report no. 31, National Housing and Building Research Unit Dar es Salaam, Tanzania.

▪ Murphy H. W. 1998. Cement Treated Pavements-From the unpredictable to the dependable, Proc. Fourteenth ARRB Conference on Materials and Testing, Canberra, Australian, 28 August – 2 September.

▪ Nwaboukei S. O., Lovell C. W. 1984. Comparisons of Shear Characteristics of Laboratory and Field-Compacted Soil, Transportation Research Record 998, Transportation Research Board, Washington DC.

▪ Osbaeck B. 1989. Ground Granulated Blast Furnace Slags: Grinding Methods, Particle Size Distribution, and Properties, Proc. Third International Conference in Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Trondheim, Norway.

▪ Papadakis V. G., Fardis M. N., Vayenas C. G. 1992. Hydration and Carbonation of Pozzolanic Cements, ACI Materials Journal, no. 89-M13, March-April.

▪ Philleo R. E. 1970. Summary of the American Concrete Institute Symposium on Creep, Shrinkage and Temperature, New York, USA, April 1970, Proc. Symposium on Design of Concrete Structures for Creep, Shrinkage and Temperature Changes, Madrid, Spain.

▪ Raad L. 1985. Behavior of Stabilized Layers Under Repeated Loads, Transportation Research Record 1022, Transportation Research Board, Washington DC.

- Ranganath R. V., Sharma R. C., Krishnamoorthy S. 1995. Influence of Fineness and Soluble Silica Content of Fly Ashes on Their Strength Development with Respect to Age, Proc. Fifth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Milwaukee, Wisconsin, USA.

- Ranganath R. V., Bhattacharjee B., Krishnamoorthy S. 1998. Influence of Size Fraction of Ponded Ash on its Pozzolanic Activity, CEMENT and CONCRETE RESEARCH, vol. 28, no. 5, USA.

- Robnett Q. L. 1976. Effect of Lime Treatment on the Resilient Behavior of Fine-Grained Soils, Transportation Research Record 560, Transportation Research Board, Washington DC.

- Savage P. F. 1995. Use of In-Situ Materials in Low Cost Road Construction - time for a Rethink, Proc. International Road Federation (IRF) Regional Conference, Fourways, Johannesburg, South Africa, 23-25 October.

- Scholen D. E. 1995. Stabilizer Mechanisms in Non-standard Stabilizers, Proc. Sixth International Conference on Low-Volume Roads, Minneapolis, Minnesota, USA, June 25-29.

- Scholen D. E., Coghlan S. 1991. Non-standard Stabilization of Aggregate Road Surfaces, Transportation Research Record 1291, Transportation Research Board, Washington DC.

- Shahid M. A., Thom N. H. 1996. Performance of cement bound bases with controlled cracking, Reflective Cracking in Pavements, Design and performance of overlay system, Proc. Third International RILEM Conference, Maastricht, The Netherlands, 2 - 4 October.

- Shen C. K., Smith S. S. 1976. Elastic and Viscoelastic Behavior of Chemically Stabilized Sand, Transportation Research Record 593, Transportation Research Board, Washington DC.

- Shi C., Day R. L. 1993. Acceleration of Strength Gain of Lime-Pozzolan Cements by Thermal Activation, CEMENT and CONCRETE RESEARCH, vol. 23, no. 4, USA.

- Steiger R. W. 1995. Roads of the Roman Empire, Concrete Construction, November.

- Sybertz F. 1989. Comparison of Different Methods for Testing the Pozzolanic Activity of Fly Ashes, Proc. Third International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, vol. 1, edited by V. M. Malhorta, Trondheim, Norway.

- Tokyay M., Hubbard F. H. 1992. Mineralogical Investigations of High-Lime Fly Ashes, Proc. Fourth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolan, Istanbul, Turkey, May.

- Turner J. P. 1997. Evaluation of Western Coal Fly Ash for Stabilization of Low-Volume Roads, Testing Soil Mixed with Waste or Recycled Materials, ASTM Publication, edited by M. A. Wasemiller & K. B. Hoddinott, USA.

- Uribe-Afif R., Rodriguez-Camacho R. E. 1995. Mexican Natural Pozzolans and the Evaluation of the Specifications Related to Their Use, Proc. Fifth International Conference on Fly Ahs, Silica Fume, Slag and Natural Pozzolans in Concrete, vol. 1, Milwaukee, Wisconsin, USA.

- Usmen M. A., Moulton L. K. 1984. Construction and Performance of Experimental Base Course Test Sections Built with Waste Calcium Sulphate, Lime, and Fly Ash, Transportation Research Record 998, Transportation Research Board, Washington DC.

- Vieira M., de Almeida I. R., Goncalves A. F. 2000. Influence of Moisture Curing on Durability of Fly Ash Concrete for Road Pavements, Proc. Fifth International Conference on Durability of Concrete, Barcelona, Spain.

- Visser A.T. 2002. Pavement design course notes (SGC 781), University of Pretoria, South Africa.

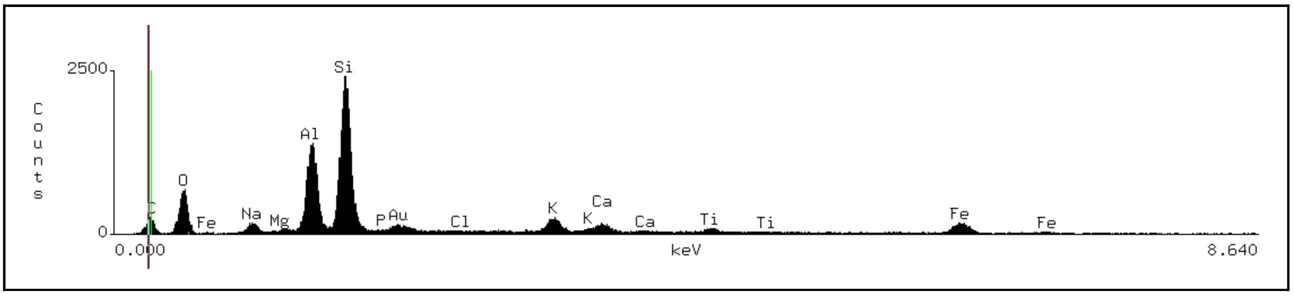
- Wang M. C. 1982. Performance Analysis for Flexible Pavements with Stabilized Base, Transportation Research Record 888, Transportation Research Board, Washington DC.

- Wang M. C., Gramling W. L. 1980. Distress Behavior of Flexible Pavements That Contain Stabilized Base Courses, Transportation Research Record 755, Transportation Research Board, Washington DC.

APPENDIX A

CHEMICAL ANALYSIS RESULTS FOR VIRGIN POZZOLANS

Arusha pozzolan position 1:



NRA-1

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35°
Dead Time: 14.062

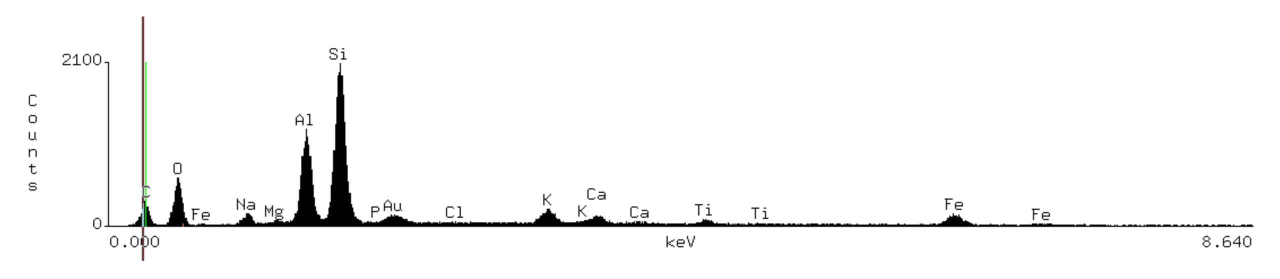
Quantitative Analysis

NRA-1

Chi-sqd = 1.24 Livetime = 100.0 Sec.

Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0200	2.034	5.40	4.06	+/- 0.17
Mg-K	0.0025	1.622	0.51	0.40	+/- 0.12
Al-K	0.1622	1.395	25.67	22.63	+/- 0.29
Si-K	0.3038	1.578	52.24	47.95	+/- 0.46
P -K	0.0000	2.166	0.01	0.01	+/- 0.24
Cl-K	0.0006	1.564	0.08	0.09	+/- 0.11
K -K	0.0474	1.269	4.70	6.01	+/- 0.16
Ca-K	0.0339	1.197	3.10	4.06	+/- 0.29
Ti-K	0.0174	1.212	1.35	2.11	+/- 0.30
Fe-K	0.1125	1.126	6.94	12.67	+/- 0.68
Total			100.00	100.00	

Arusha pozzolan position 2:



NRA-2

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35°
Dead Time: 12.743

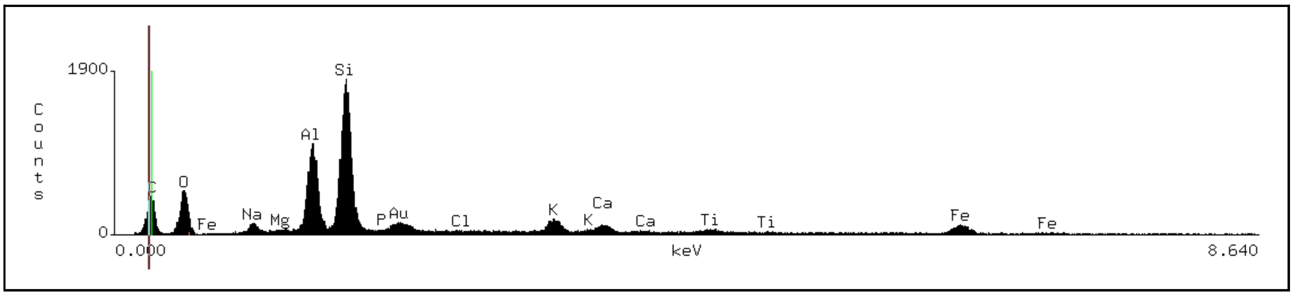
Quantitative Analysis

NRA-2

Chi-sqd = 1.30 Livetime = 100.0 Sec.

Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0201	2.033	5.44	4.08	+/- 0.19
Mg-K	0.0035	1.622	0.71	0.57	+/- 0.13
Al-K	0.1577	1.398	25.02	22.06	+/- 0.32
Si-K	0.3069	1.570	52.51	48.19	+/- 0.50
P -K	0.0003	2.170	0.07	0.07	+/- 0.27
Cl-K	0.0005	1.559	0.07	0.08	+/- 0.12
K -K	0.0518	1.269	5.15	6.58	+/- 0.32
Ca-K	0.0298	1.200	2.73	3.58	+/- 0.17
Ti-K	0.0181	1.212	1.40	2.19	+/- 0.33
Fe-K	0.1120	1.126	6.91	12.61	+/- 0.76
Total			100.00	100.00	

Arusha pozzolan position 3:



NRA-3

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35°
Dead Time: 12.979

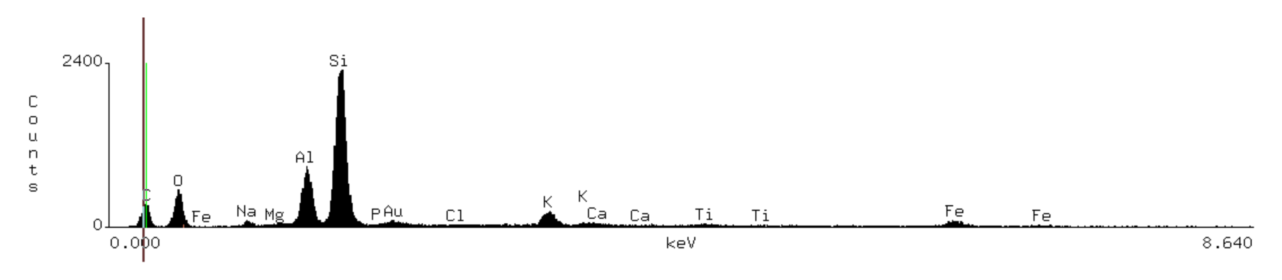
Quantitative Analysis

NRA-3

Chi-sqd = 1.34 Livetime = 100.0 Sec.

Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0183	2.019	4.92	3.70	+/- 0.21
Mg-K	0.0028	1.604	0.56	0.45	+/- 0.14
Al-K	0.1632	1.384	25.57	22.58	+/- 0.34
Si-K	0.3098	1.570	52.92	48.64	+/- 0.54
P -K	0.0009	2.178	0.20	0.20	+/- 0.31
Cl-K	0.0000	1.538	0.00	0.00	+/- 0.00
K -K	0.0498	1.271	4.95	6.33	+/- 0.35
Ca-K	0.0305	1.201	2.79	3.67	+/- 0.19
Ti-K	0.0170	1.213	1.32	2.06	+/- 0.21
Fe-K	0.1098	1.127	6.77	12.37	+/- 0.49
Total			100.00	100.00	

Mbeya pozzolan position1:



NRM-4

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35.928°
Dead Time: 12.937

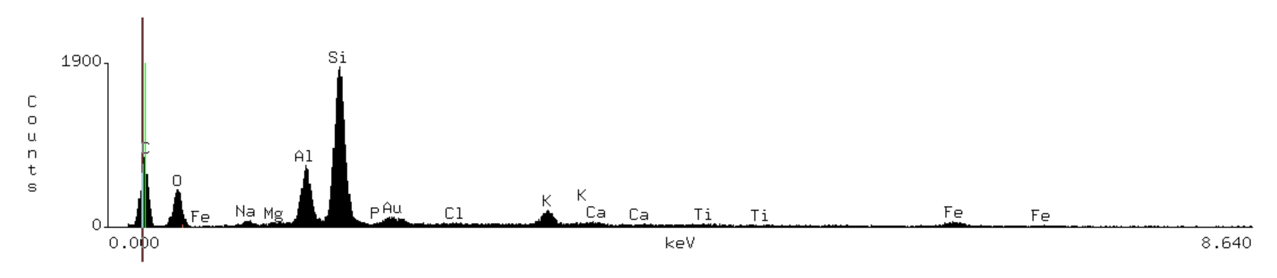
Quantitative Analysis

NRM-4

Chi-sqd = 1.24 Livetime = 100.0 Sec.

Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0116	1.845	2.81	2.15	+/- 0.17
Mg-K	0.0013	1.457	0.23	0.18	+/- 0.12
Al-K	0.1338	1.276	19.01	17.08	+/- 0.30
Si-K	0.4305	1.404	64.65	60.46	+/- 0.54
P -K	0.0000	2.207	0.00	0.00	+/- 0.00
Cl-K	0.0017	1.597	0.22	0.26	+/- 0.13
K -K	0.0703	1.296	7.00	9.11	+/- 0.37
Ca-K	0.0074	1.235	0.69	0.92	+/- 0.17
Ti-K	0.0100	1.228	0.77	1.22	+/- 0.19
Fe-K	0.0760	1.134	4.63	8.62	+/- 0.79
Total			100.00	100.00	

Mbeya pozzolan position 2:



NRM-5

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35.928°
Dead Time: 13.276

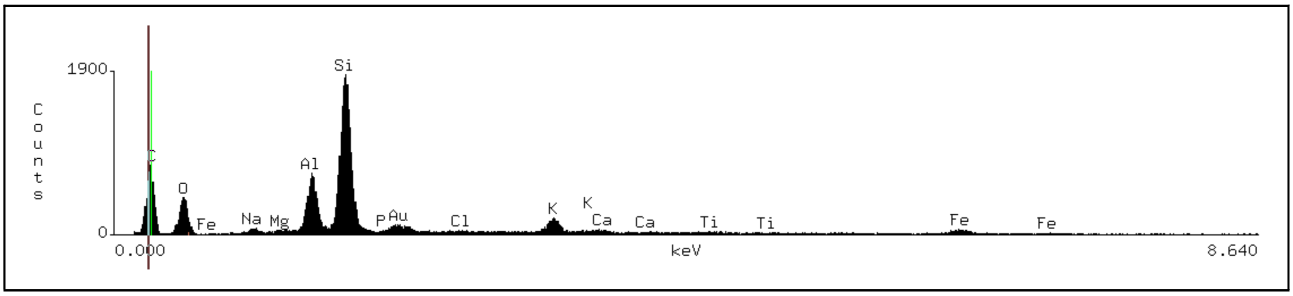
Quantitative Analysis

NRM-5

Chi-sqd = 1.31 Livetime = 100.0 Sec.

Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0127	1.736	2.83	2.20	+/- 0.21
Mg-K	0.0035	1.393	0.60	0.49	+/- 0.15
Al-K	0.1407	1.239	19.09	17.43	+/- 0.36
Si-K	0.4497	1.387	65.66	62.39	+/- 0.65
P -K	0.0005	2.251	0.10	0.11	+/- 0.40
Cl-K	0.0043	1.612	0.58	0.69	+/- 0.19
K -K	0.0675	1.308	6.68	8.83	+/- 0.26
Ca-K	0.0092	1.244	0.84	1.14	+/- 0.21
Ti-K	0.0053	1.237	0.40	0.65	+/- 0.25
Fe-K	0.0532	1.137	3.21	6.06	+/- 0.96
Total			100.00	100.00	

Mbeya pozzolan position 3:



NRM-5

Accelerating Voltage: 20 KeV
Live Time: 100 seconds

Take Off Angle: 35.928°
Dead Time: 13.276

Quantitative Analysis

NRM-5

Chi-sqd = 1.31 Livetime = 100.0 Sec.

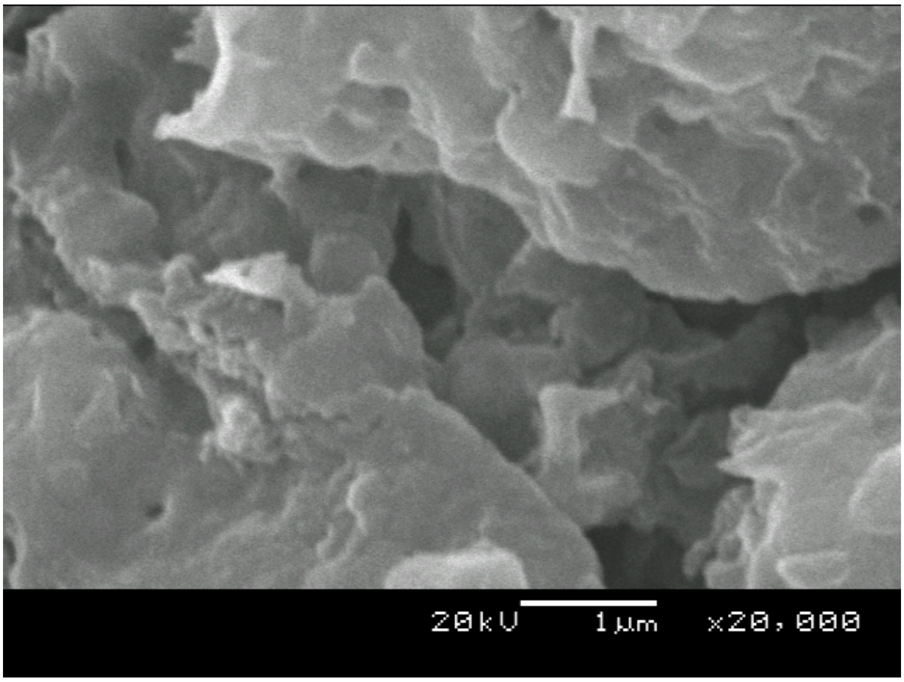
Element	k-ratio (calc.)	ZAF	Atom %	Element Wt %	Wt % Err. (1-Sigma)
Na-K	0.0127	1.736	2.83	2.20	+/- 0.21
Mg-K	0.0035	1.393	0.60	0.49	+/- 0.15
Al-K	0.1407	1.239	19.09	17.43	+/- 0.36
Si-K	0.4497	1.387	65.66	62.39	+/- 0.65
P -K	0.0005	2.251	0.10	0.11	+/- 0.40
Cl-K	0.0043	1.612	0.58	0.69	+/- 0.19
K -K	0.0675	1.308	6.68	8.83	+/- 0.26
Ca-K	0.0092	1.244	0.84	1.14	+/- 0.21
Ti-K	0.0053	1.237	0.40	0.65	+/- 0.25
Fe-K	0.0532	1.137	3.21	6.06	+/- 0.96
Total			100.00	100.00	

APPENDIX B

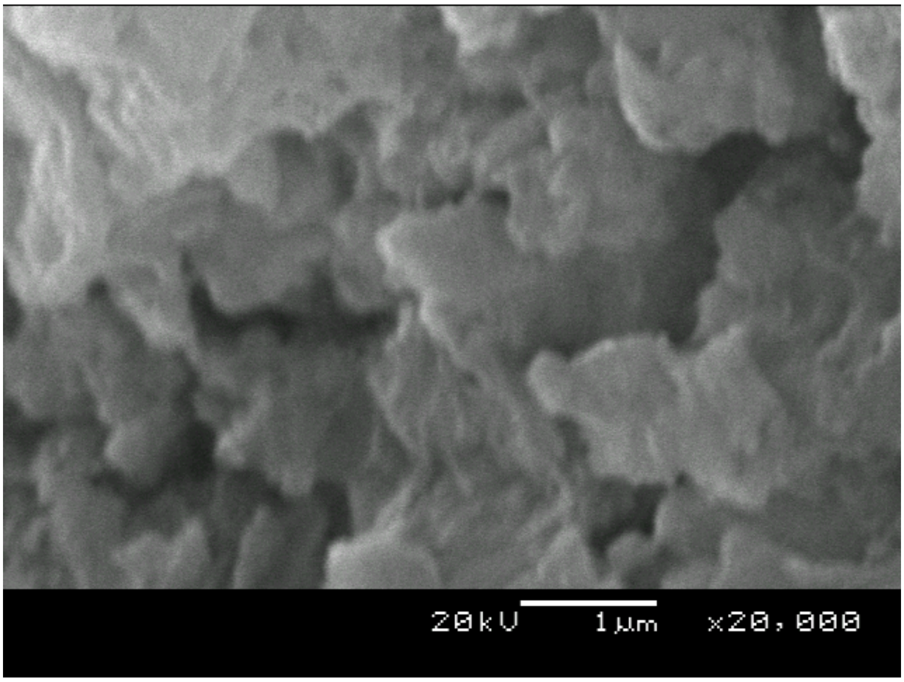
SEM IMAGES FOR PILOT MIXES

Appendix B-1: 28 days of curing

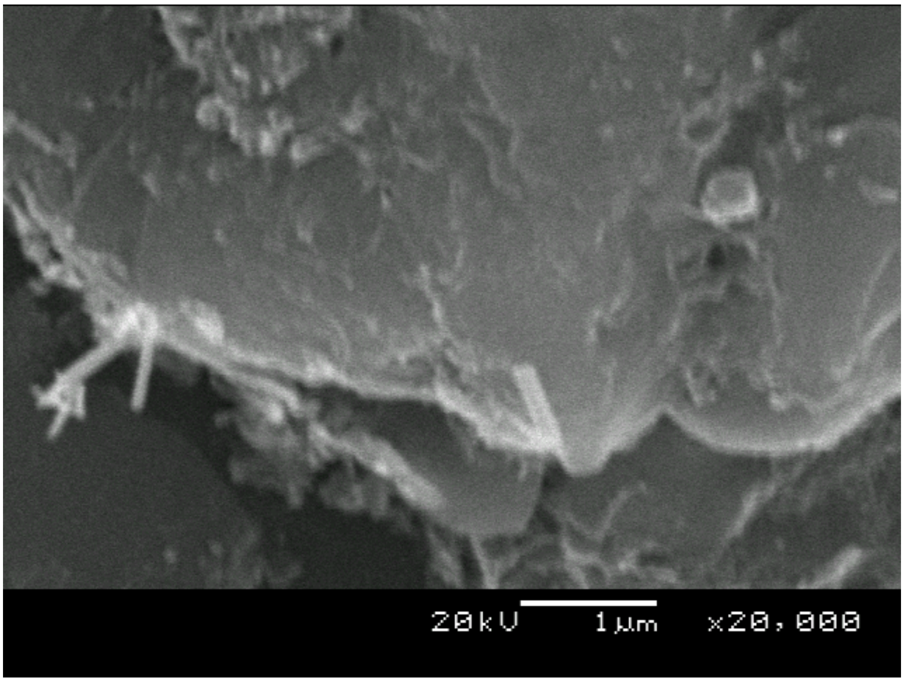
Arusha pozzolan mixes:



Arusha pozzolan: 60% Saturation (28 days)

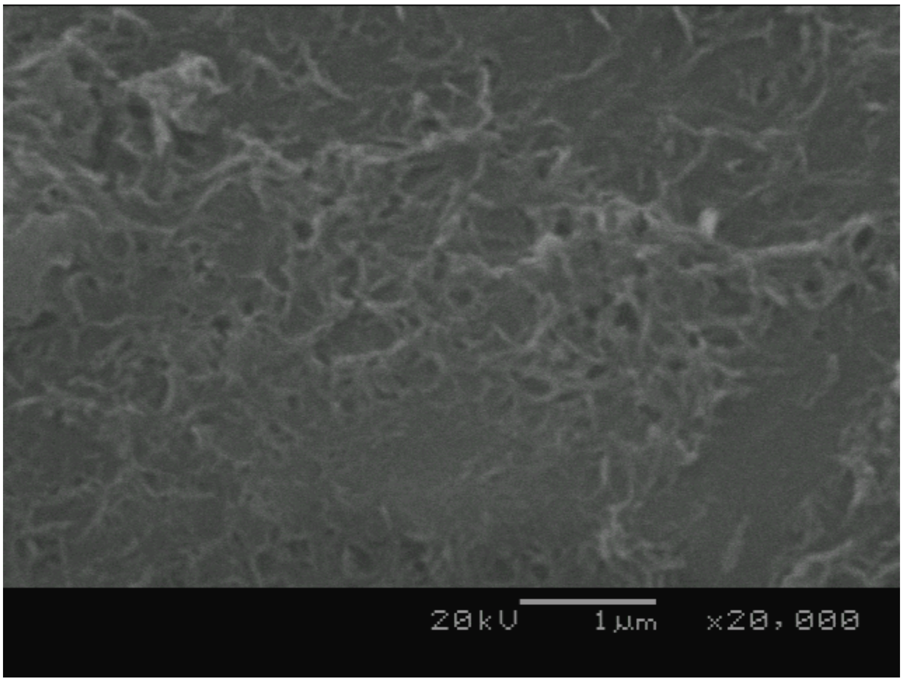


Arusha pozzolan: 80% Saturation (28 days)

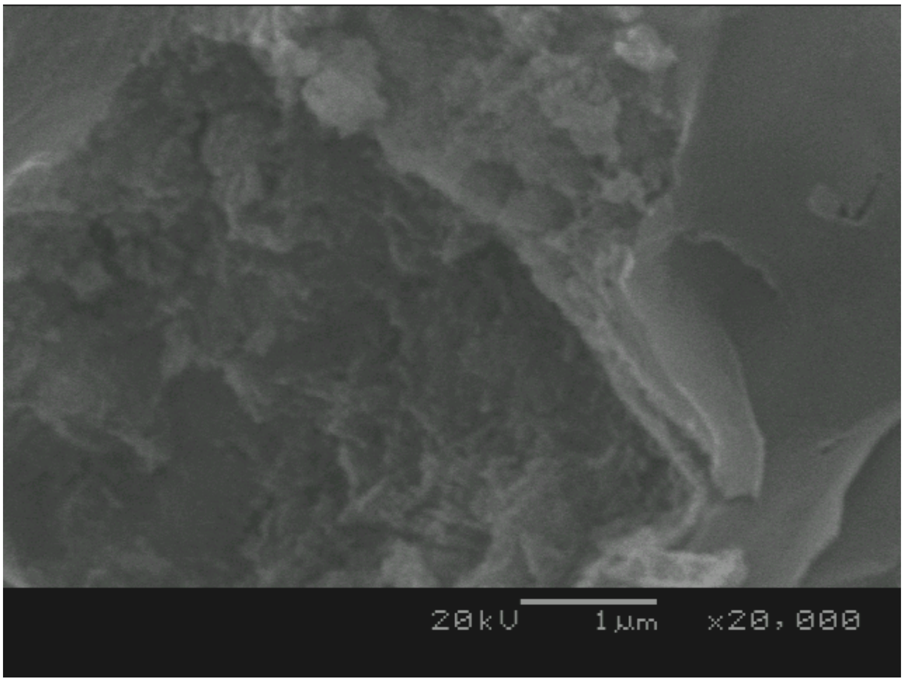


Arusha pozzolan: 100% Saturation (28 days)

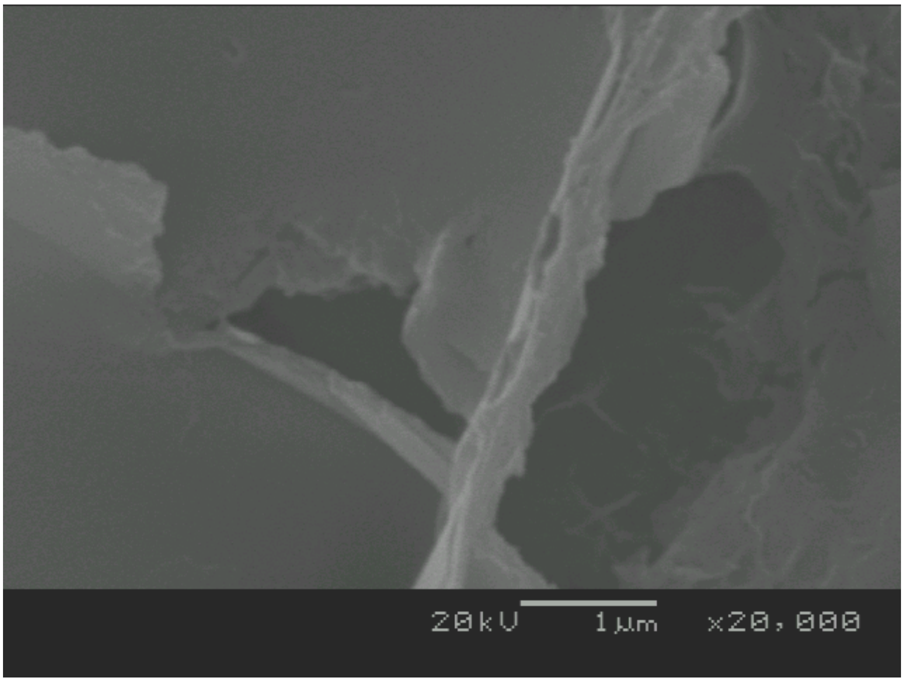
Mbeya pozzolan mixes:



Mbeya pozzolan: 60% Saturation (28 days)

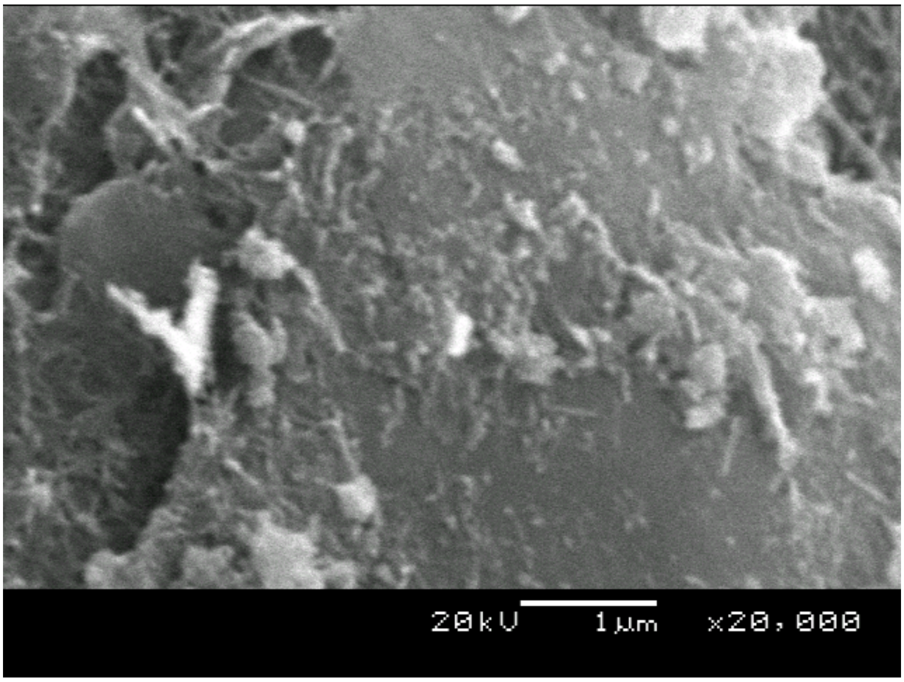


Mbeya pozzolan: 80% Saturation (28 days)

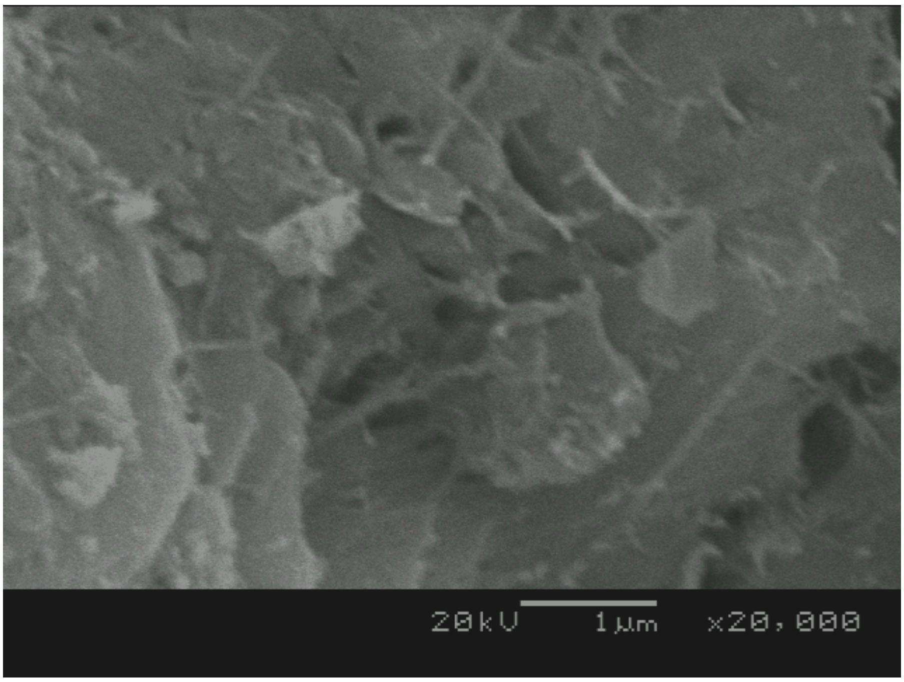


Mbeya pozzolan: 100% Saturation (28 days)

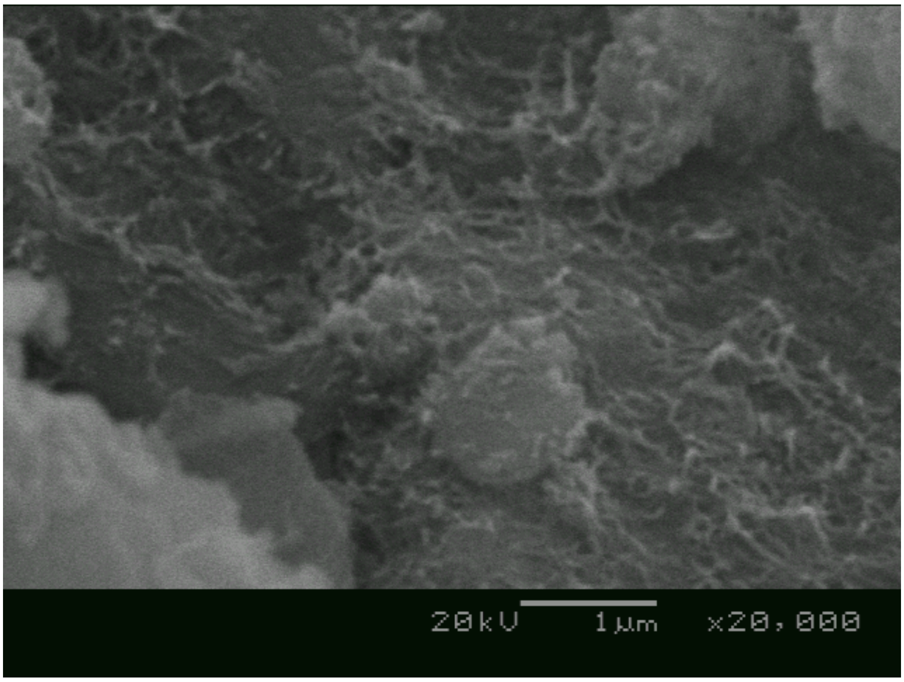
Cement mixes:



Cement: 60% Saturation (28 days)



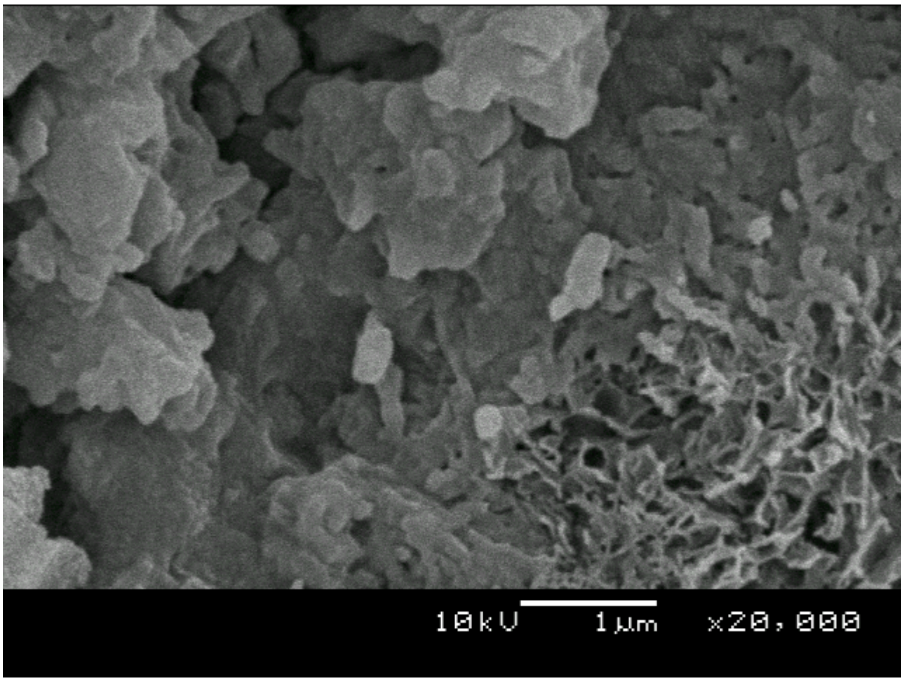
Cement: 80% Saturation (28 days)



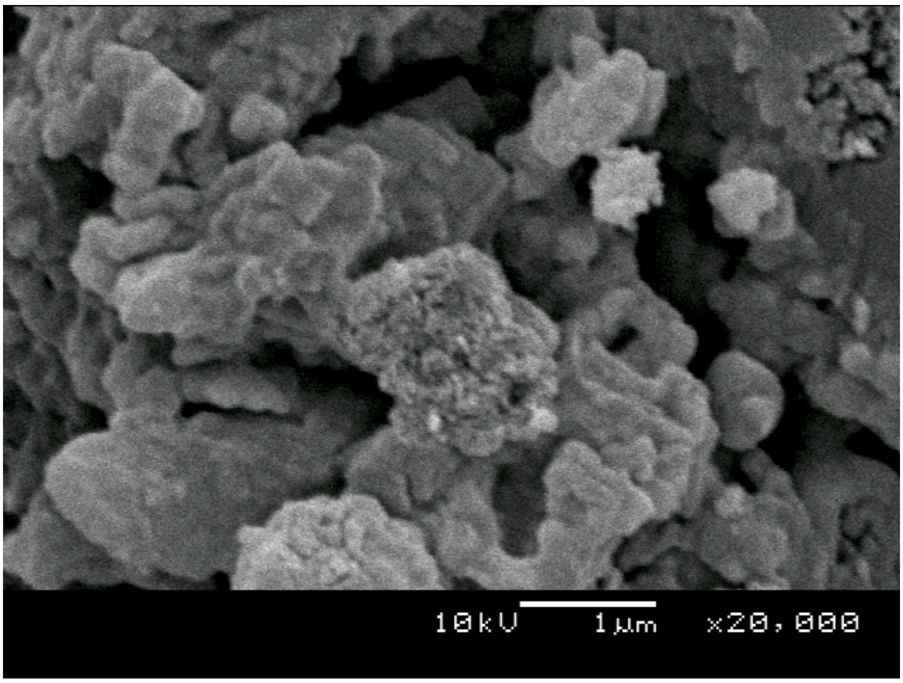
Cement: 100% Saturation (28 days)

Appendix B-2: 90 days of curing

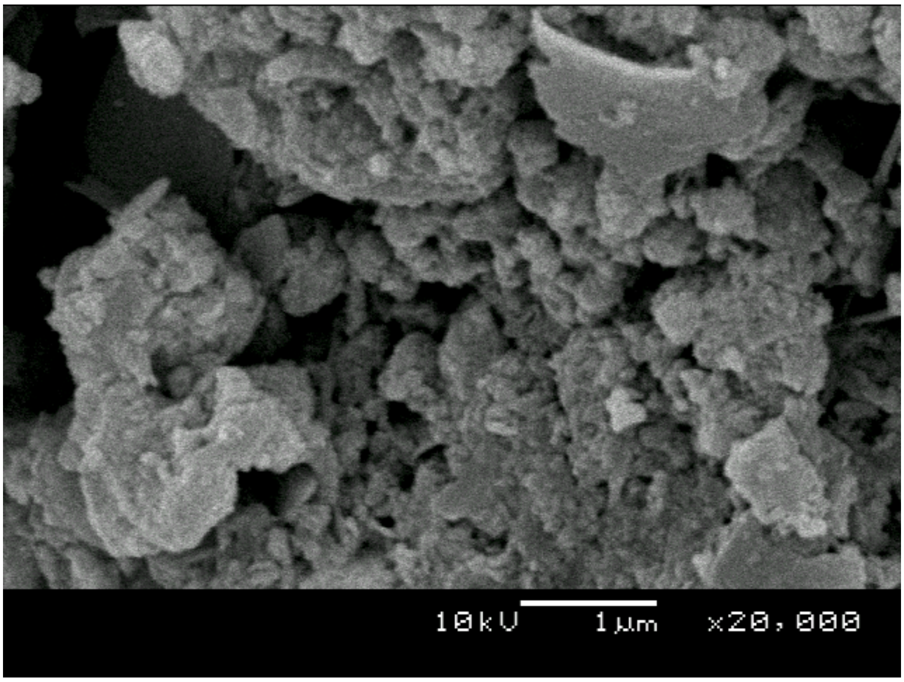
Arusha pozzolan mixes:



Arusha pozzolan: 60% Saturation (90 days)

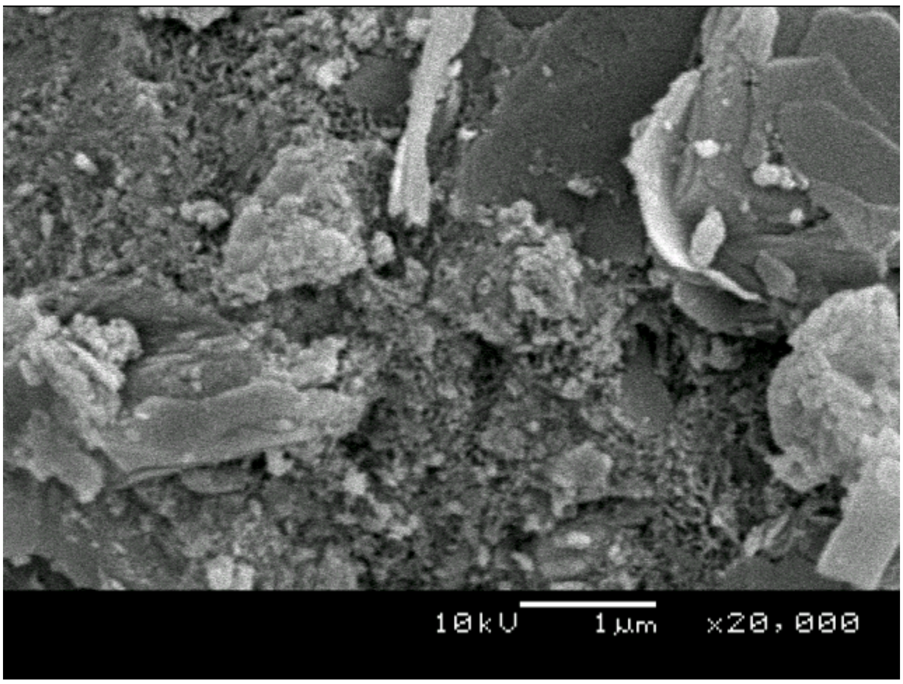


Arusha pozzolan: 80% Saturation (90 days)

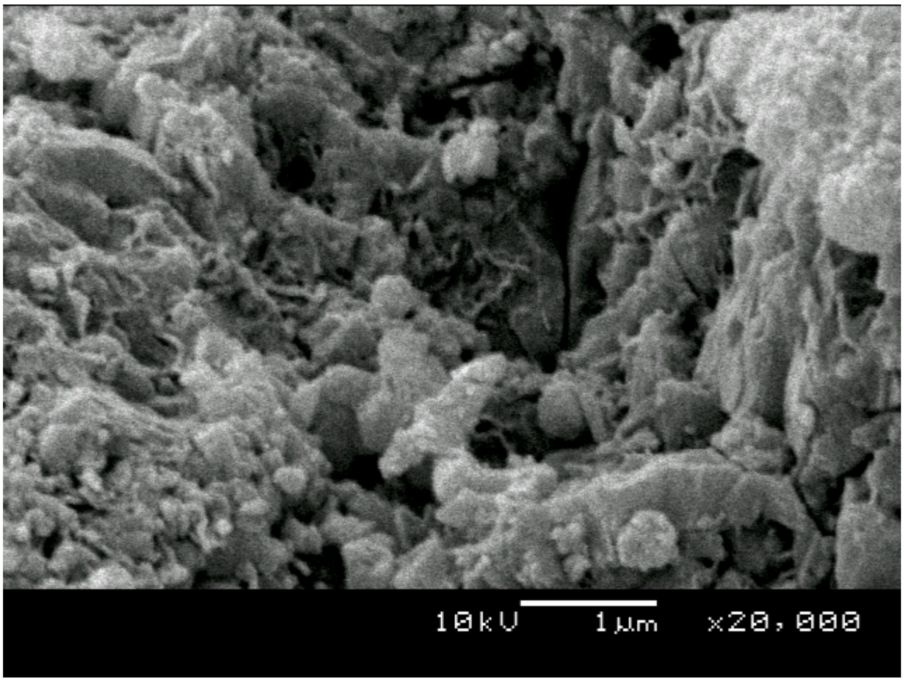


Arusha pozzolan: 100% Saturation (90 days)

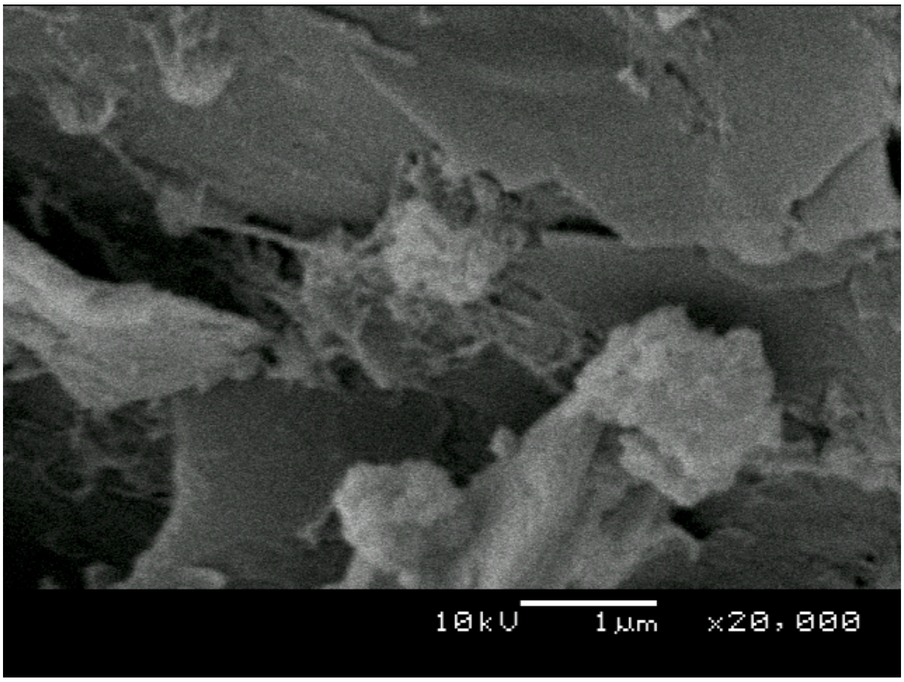
Mbeya pozzolan mixes:



Mbeya pozzolan: 60% Saturation (90 days)



Mbeya pozzolan: 80% Saturation (90 days)

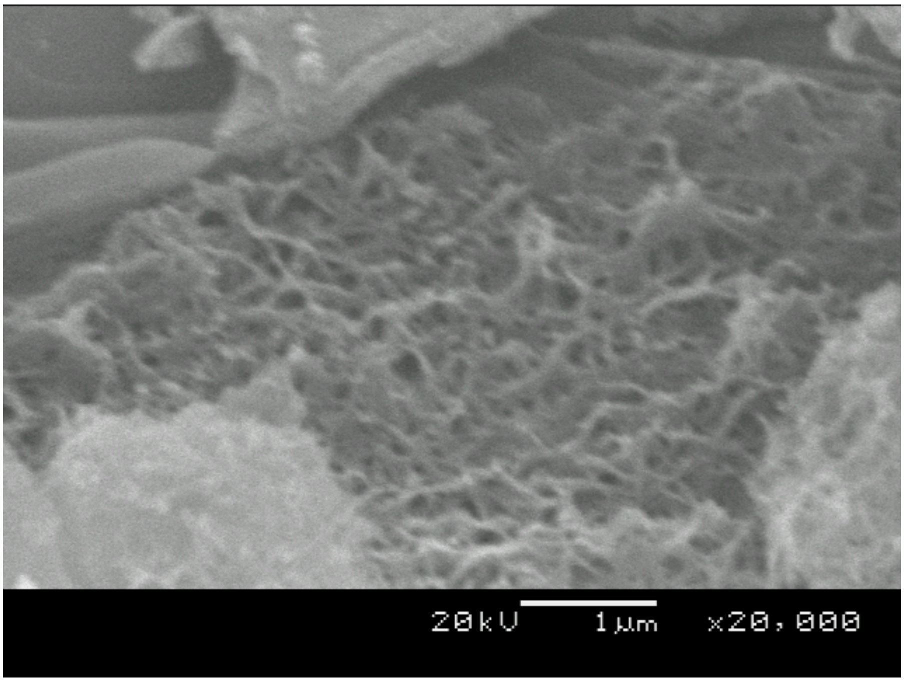


Mbeya pozzolan: 100% Saturation (90 days)

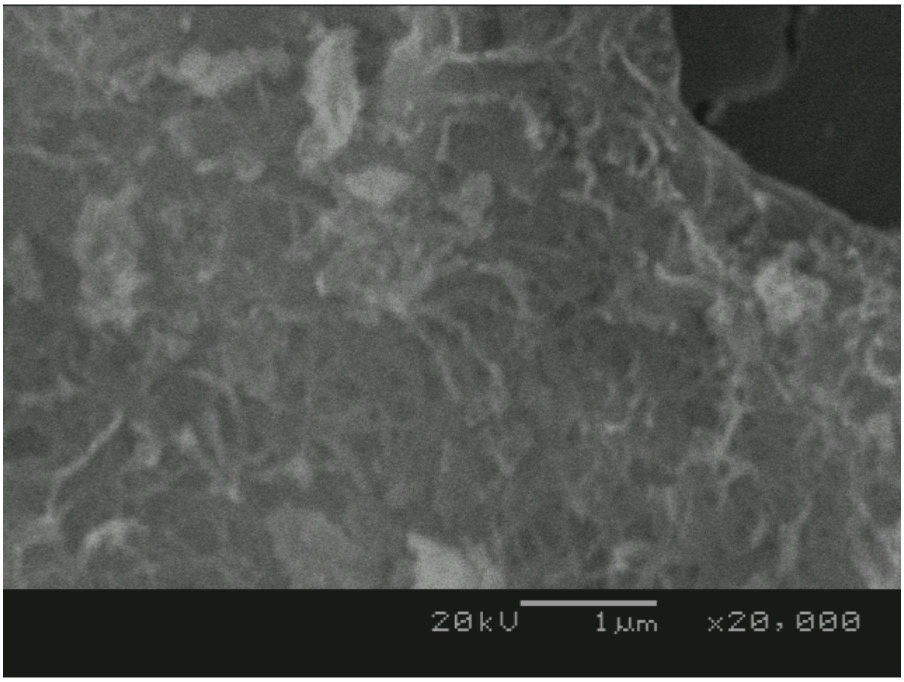
APPENDIX B2

SEM IMAGES FOR PILOT MIXES

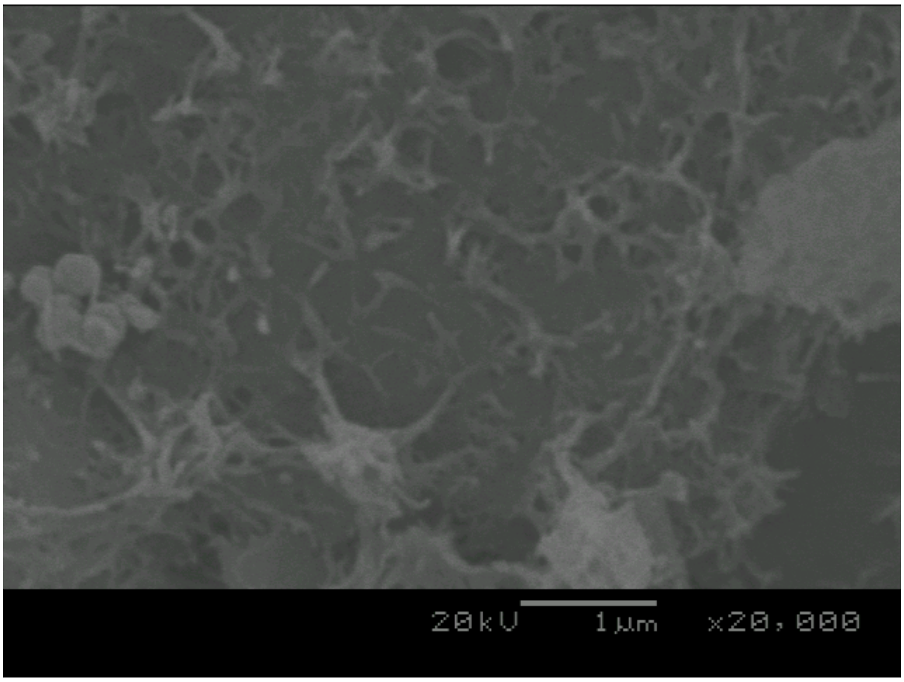
Cement mixes:



Cement: 60% Saturation (90 days)



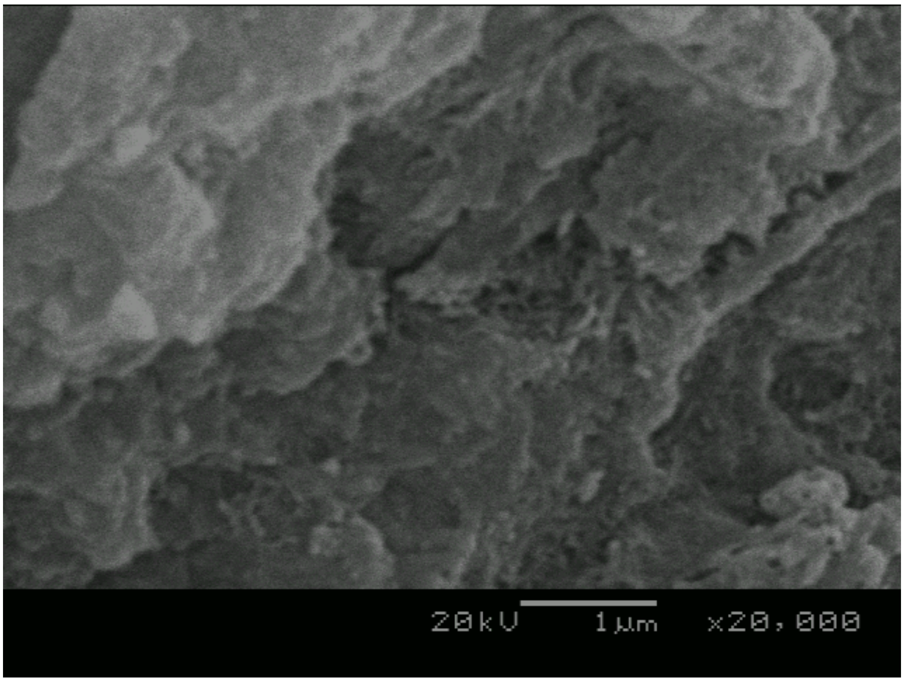
Cement: 80% Saturation (90 days)



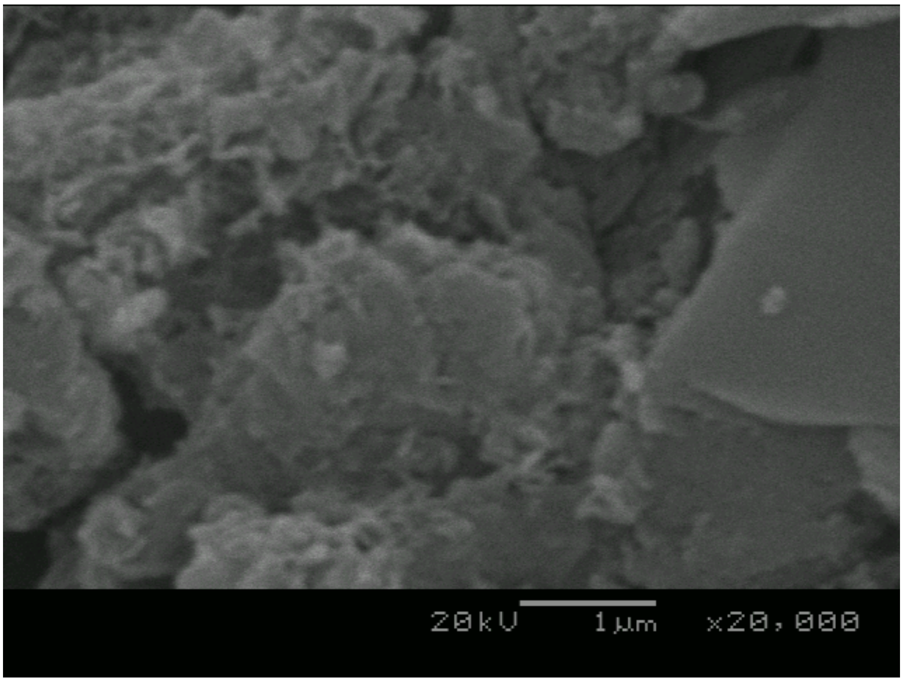
Cement: 100% Saturation (90 days)

Appendix B-3: 180 days of curing

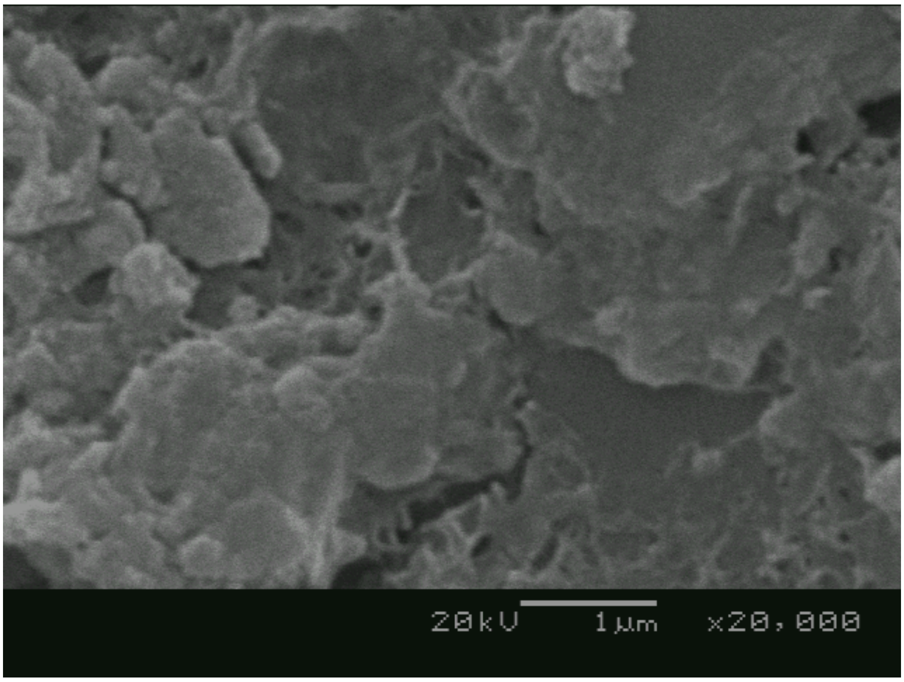
Arusha pozzolan mixes:



Arusha pozzolan: 60% Saturation (180 days)

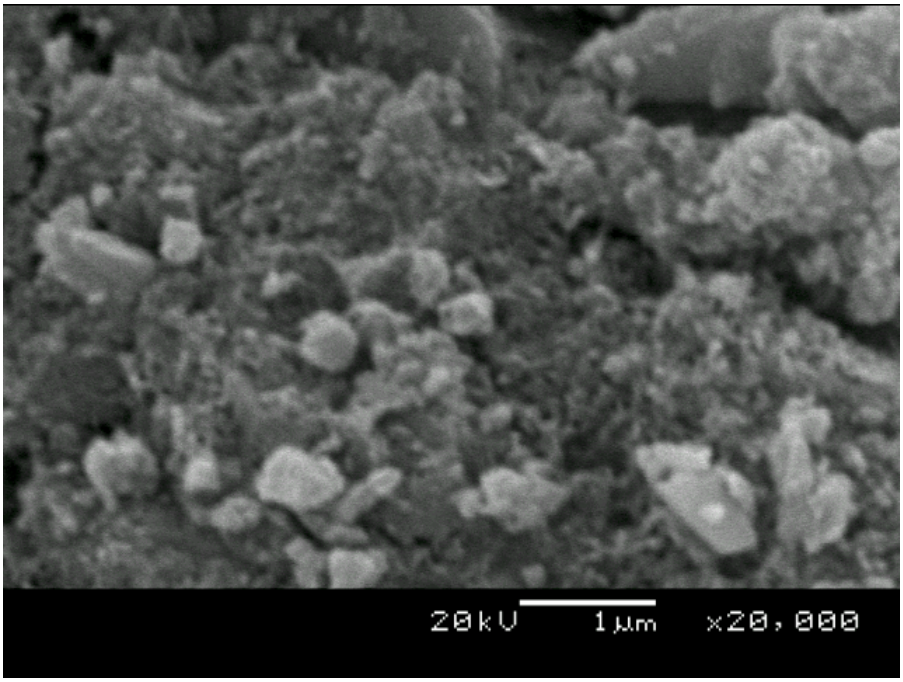


Arusha pozzolan: 80% Saturation (180 days)

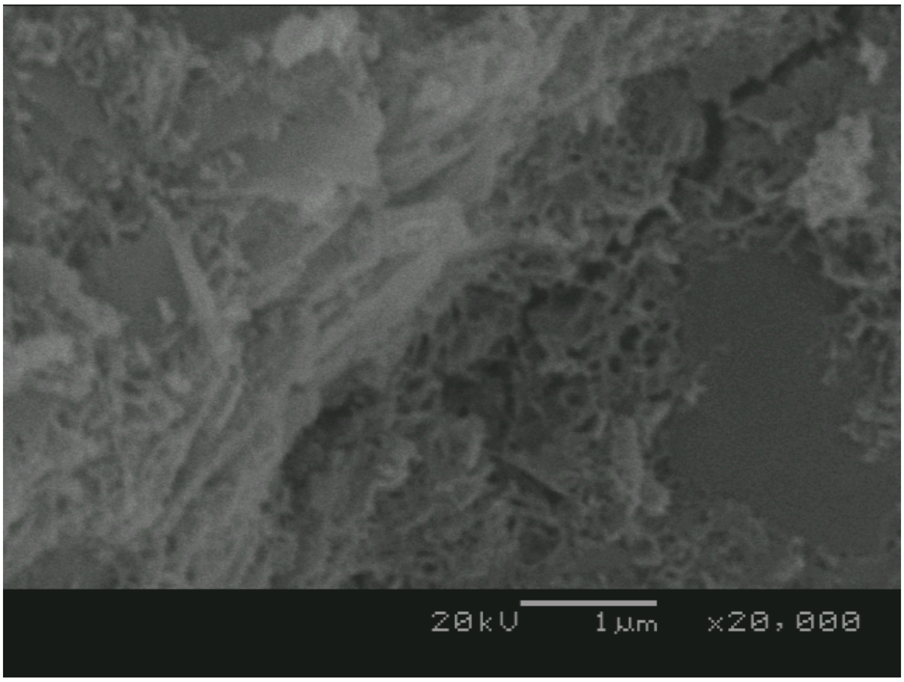


Arusha pozzolan: 100% Saturation (180 days)

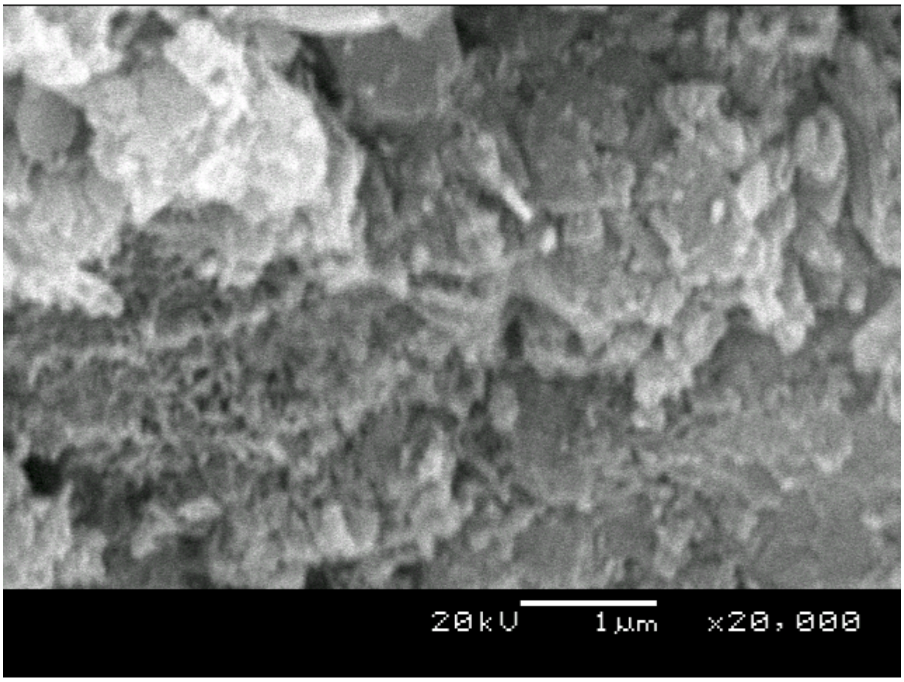
Mbeya pozzolan mixes:



Mbeya pozzolan: 60% Saturation (180 days)

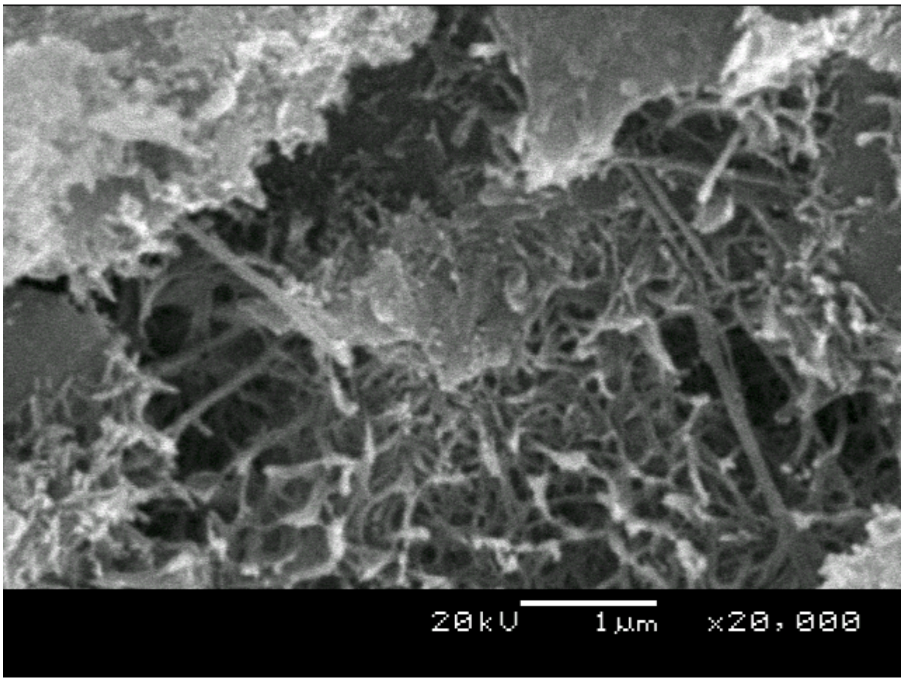


Mbeya pozzolan: 80% Saturation (180 days)

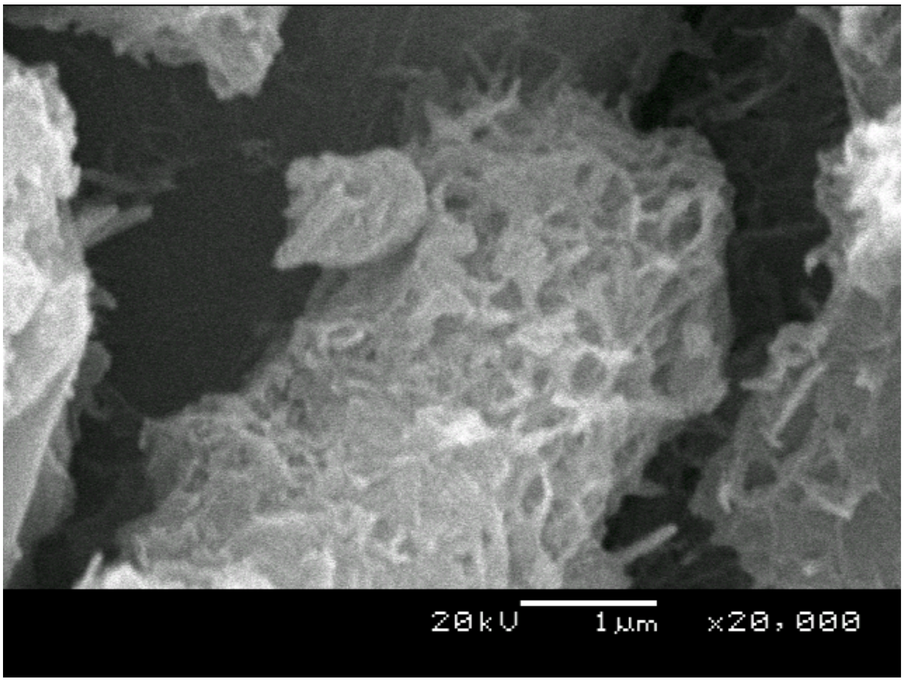


Mbeya pozzolan: 100% Saturation (180 days)

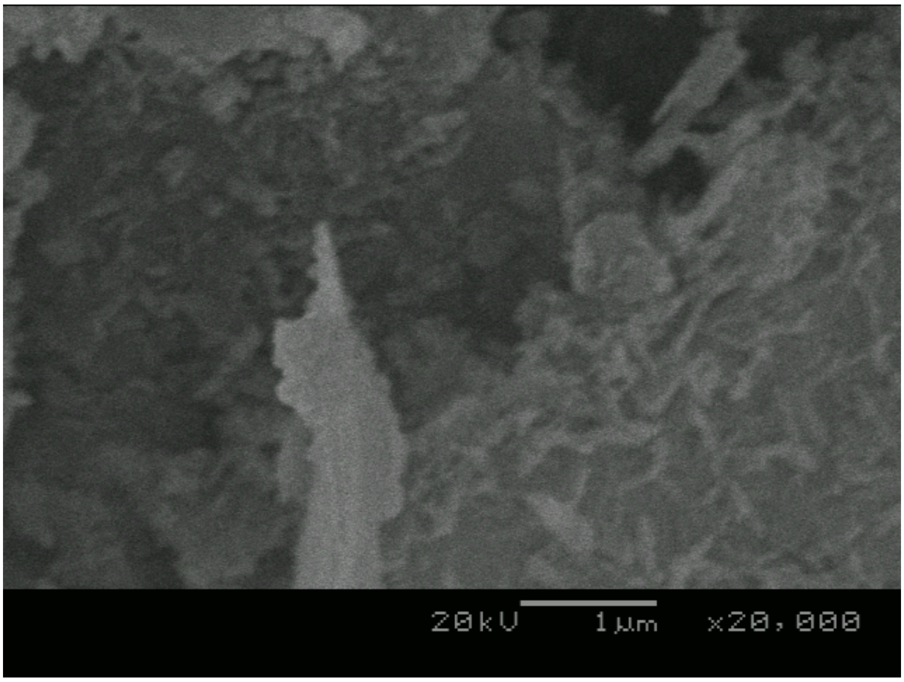
Cement mixes:



Cement: 60% Saturation (180 days)



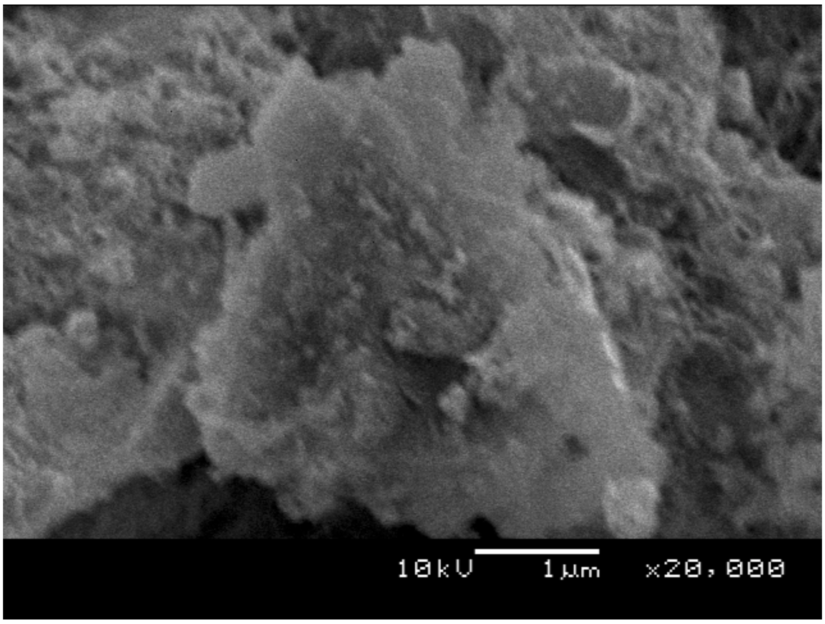
Cement: 80% Saturation (180 days)



Cement: 100% Saturation (180 days)

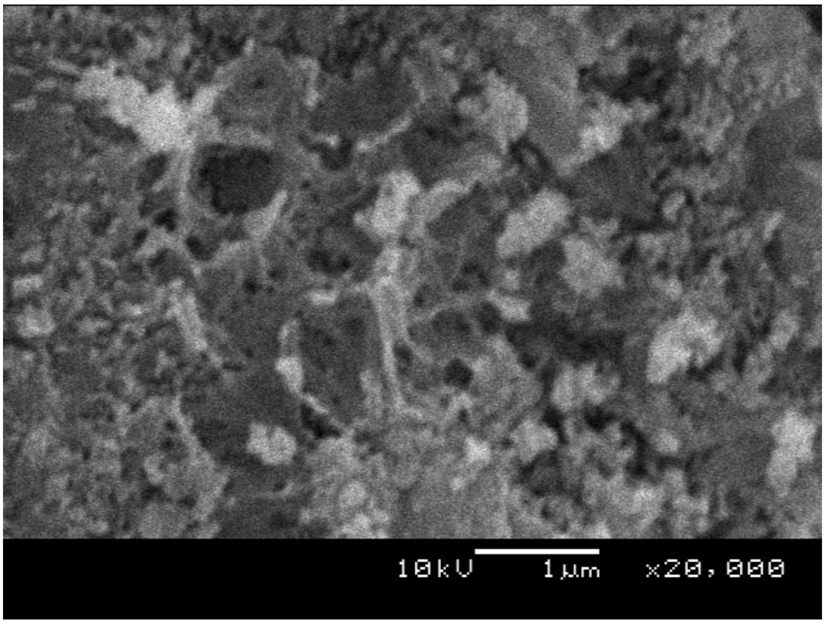
Appendix B-4: 1 year of curing

Arusha pozzolan mixes:



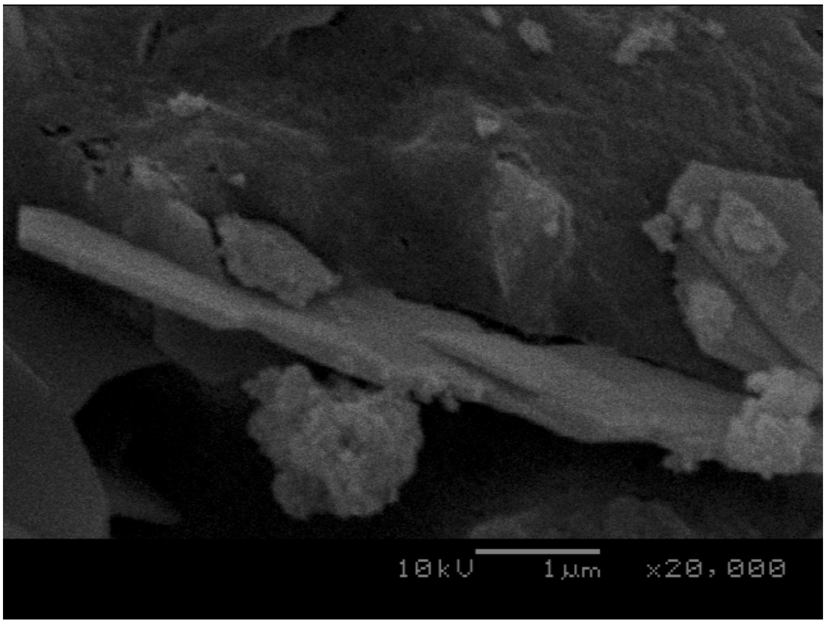
Arusha pozzolan: 80% Saturation (1 year)

Mbeya pozzolan mixes:



Mbeya pozzolan: 80% Saturation (1 year)

Cement mixes:



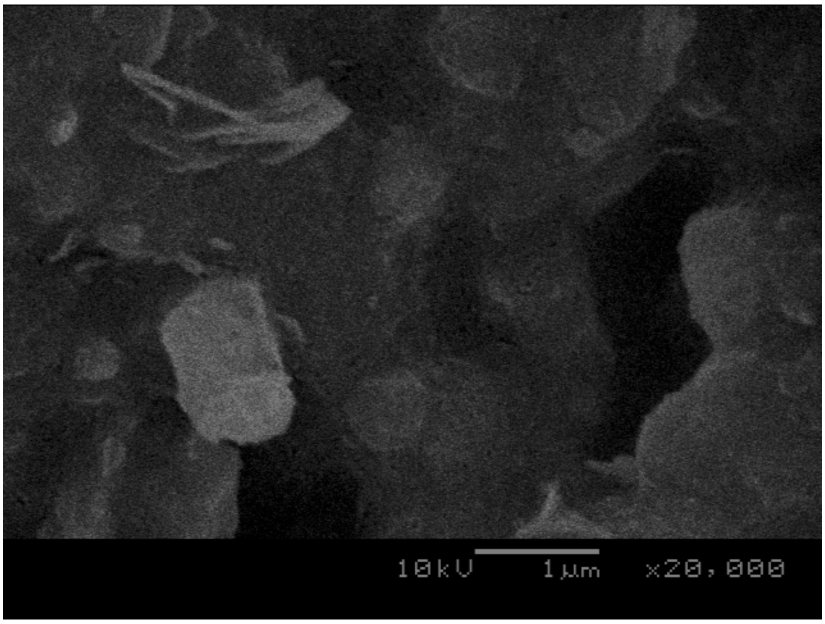
Cement: 80% Saturation (1 year)

APPENDIX C

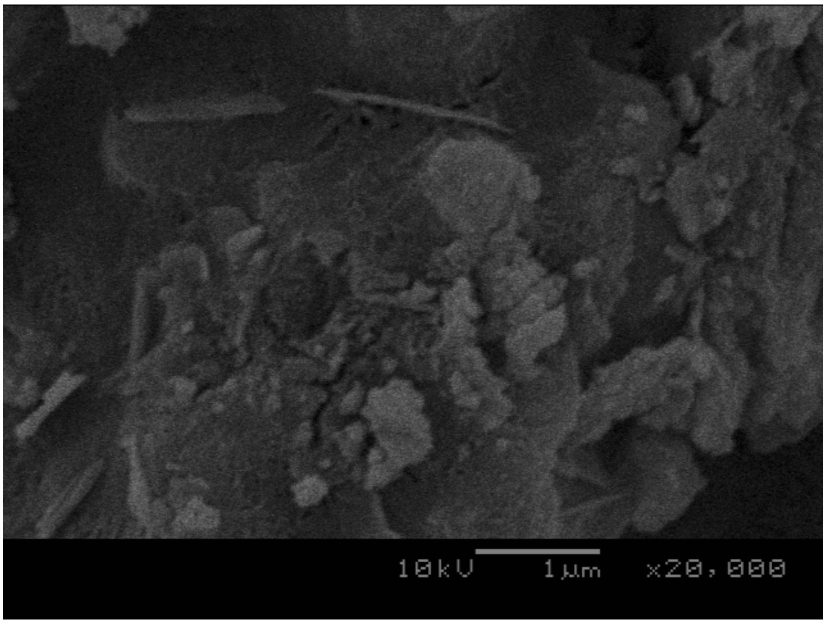
SEM IMAGES FOR MAIN STUDY MIXES

Appendix C1: Arusha pozzolan mixes

1. Arusha pozzolan washed sand mixes:

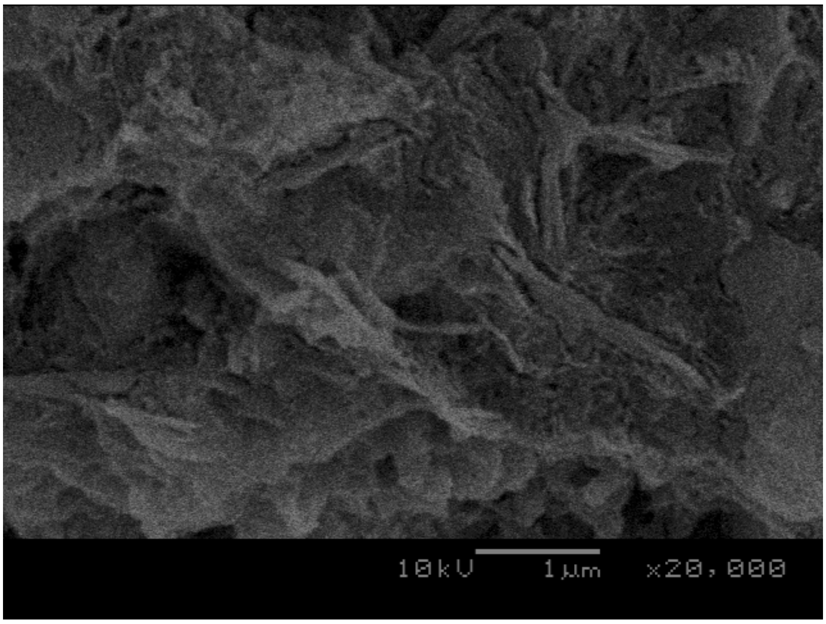


Arusha pozzolan: 25% Saturation (28 days)

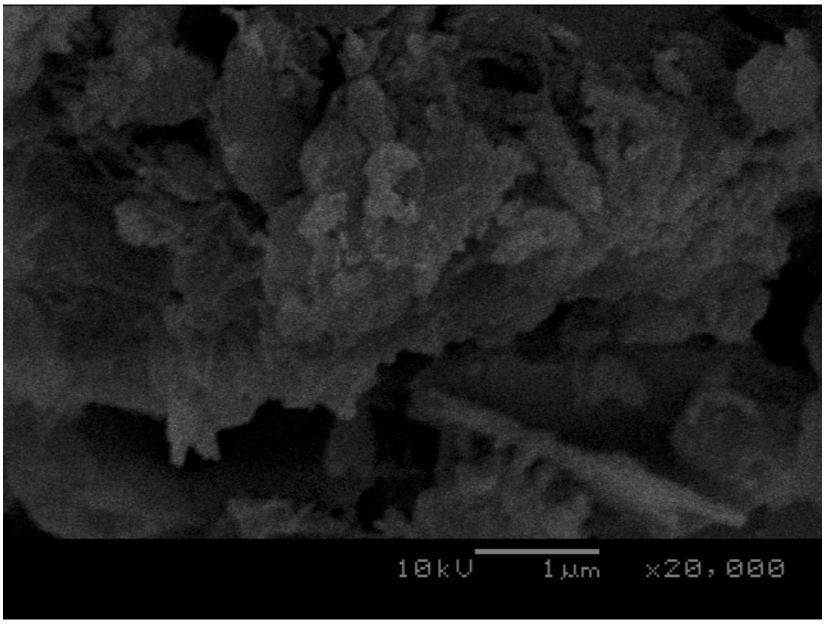


Arusha pozzolan: 50% Saturation (28 days)

Arusha pozzolan washed sand mixes

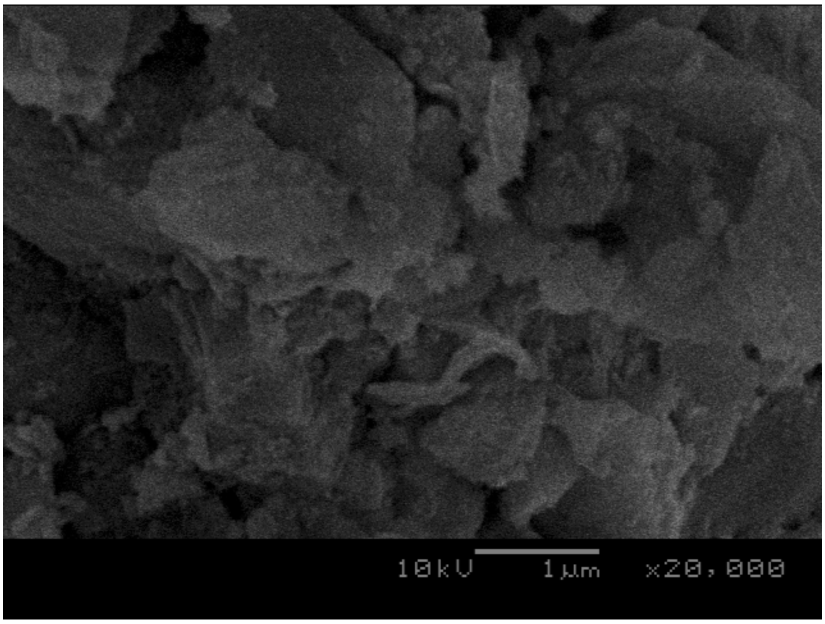


Arusha pozzolan: 75% Saturation (28 days)

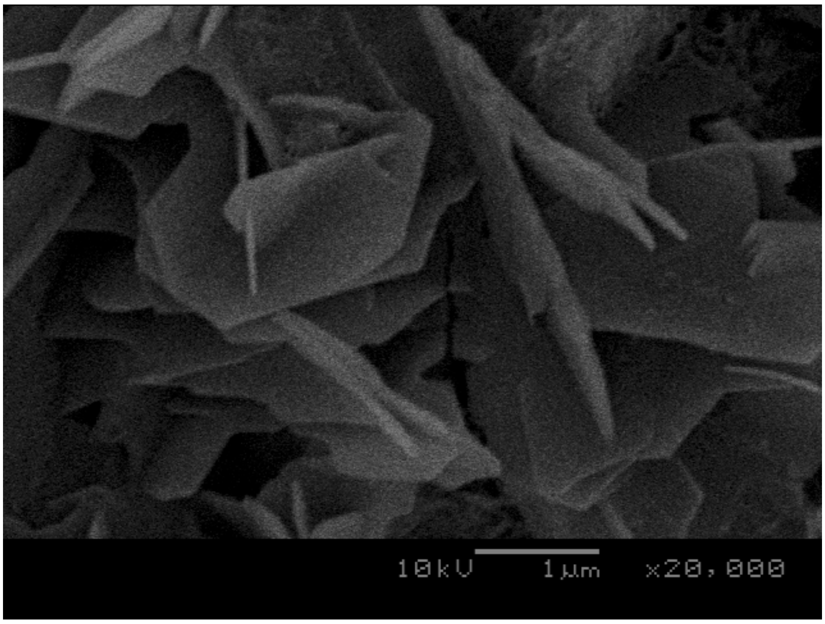


Arusha pozzolan: 100% Saturation (28 days)

2. Arusha pozzolan unwashed sand mixes:

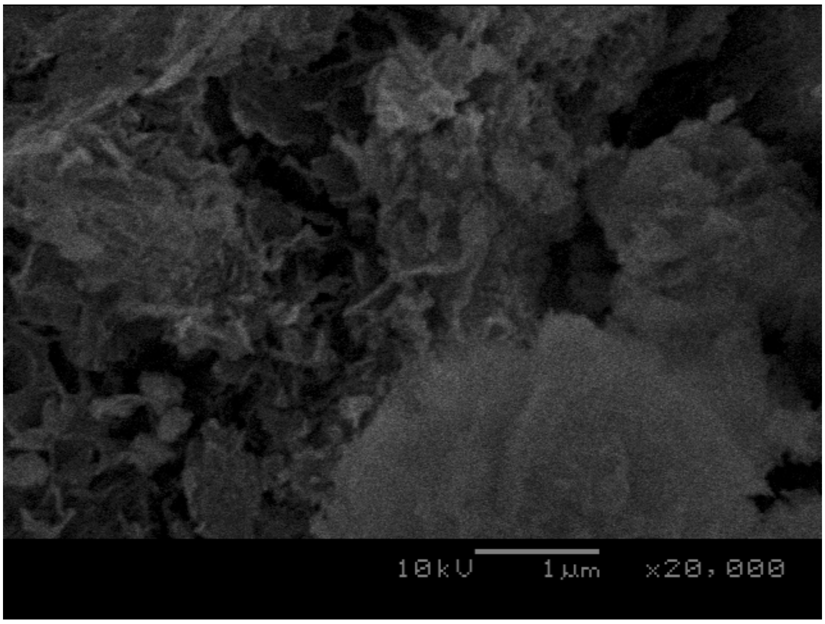


Arusha pozzolan: 24% Saturation (28 days)

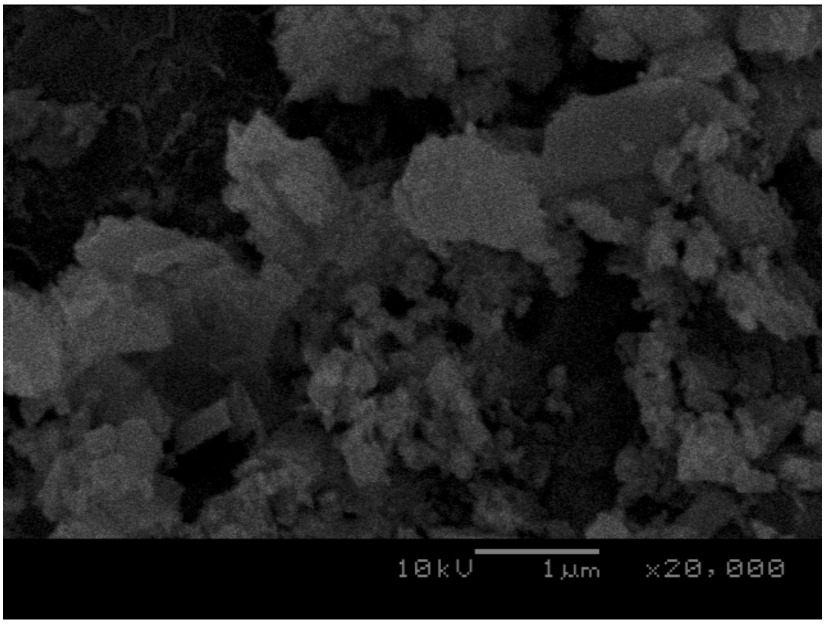


Arusha pozzolan: 49% Saturation (28 days)

Arusha pozzolan unwashed sand mixes



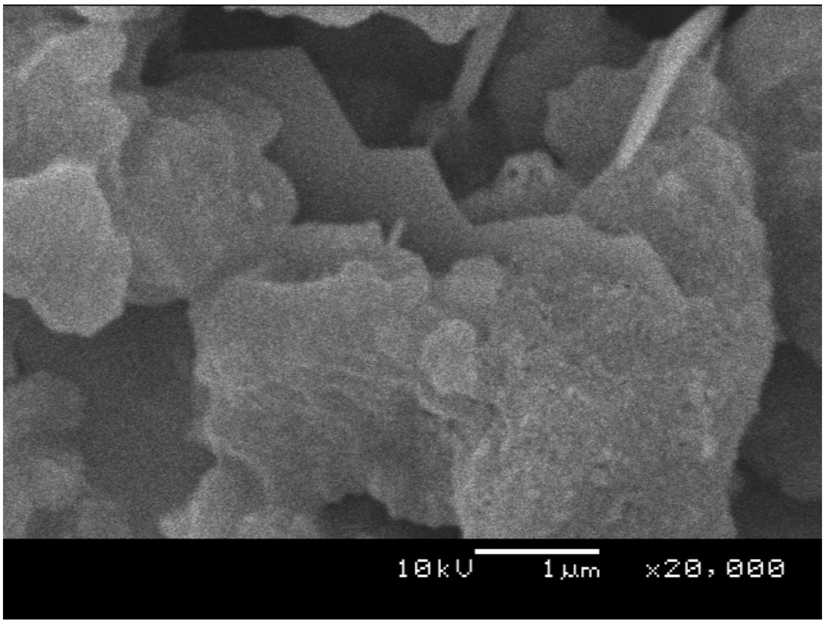
Arusha pozzolan: 73% Saturation (28 days)



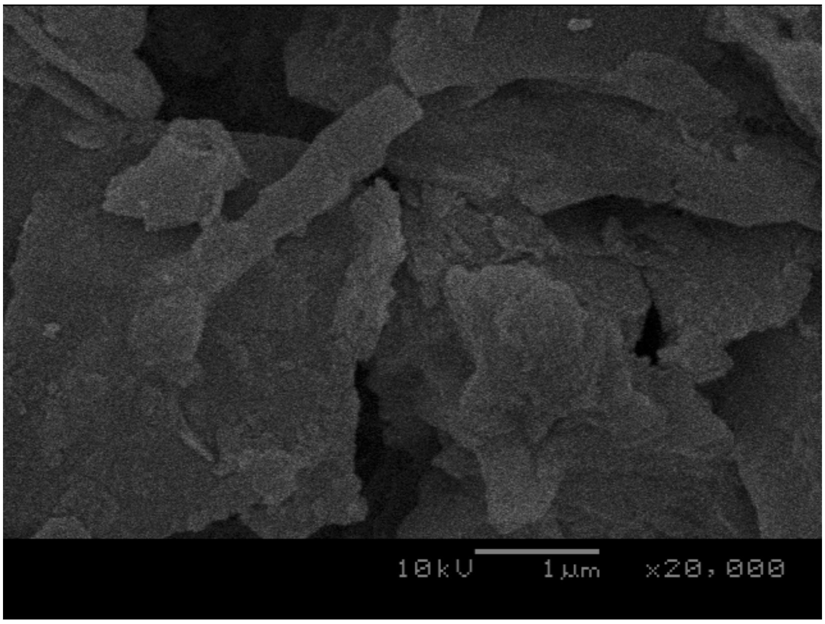
Arusha pozzolan: 100% Saturation (28 days)

Appendix C2: Mbeya pozzolan mixes

1. Mbeya pozzolan washed sand mixes:

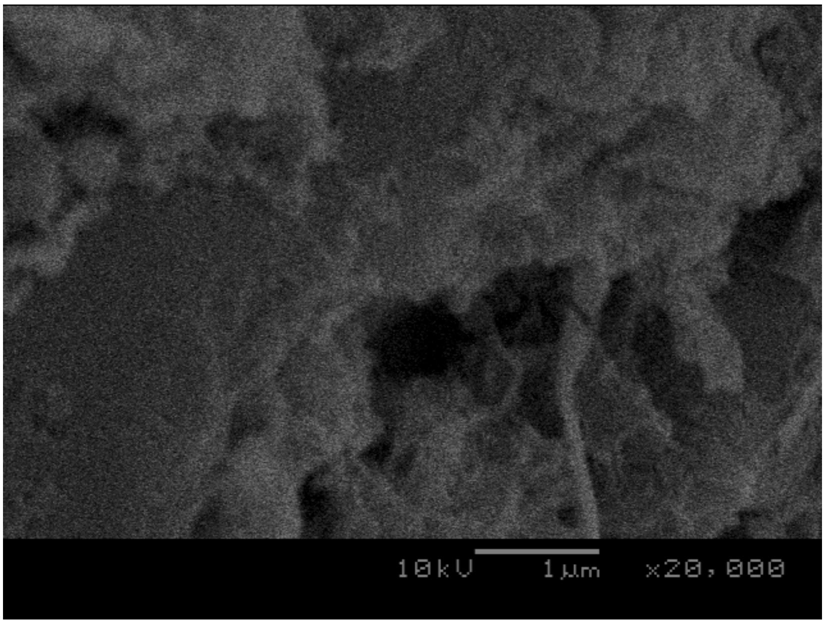


Mbeya pozzolan: 26% Saturation (28 days)

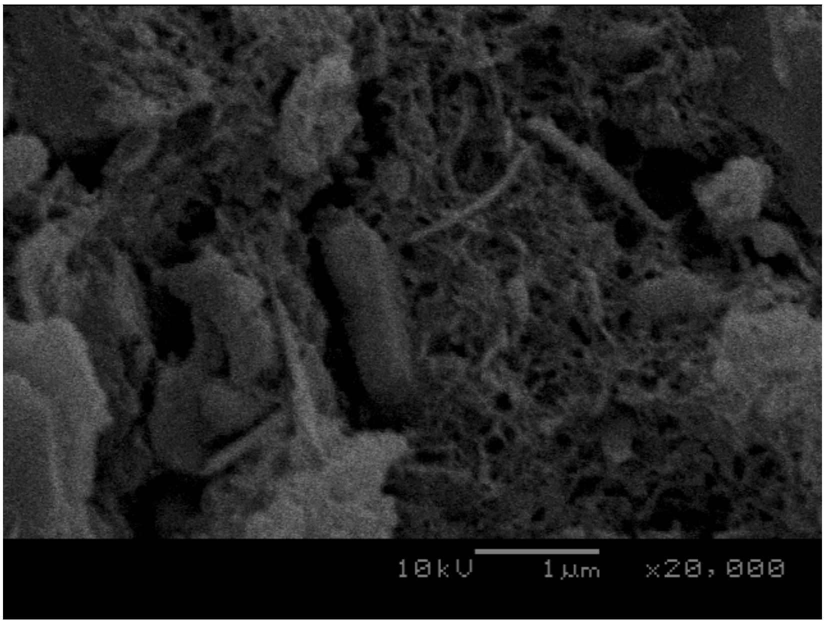


Mbeya pozzolan: 51% Saturation (28 days)

Mbeya pozzolan washed sand mixes

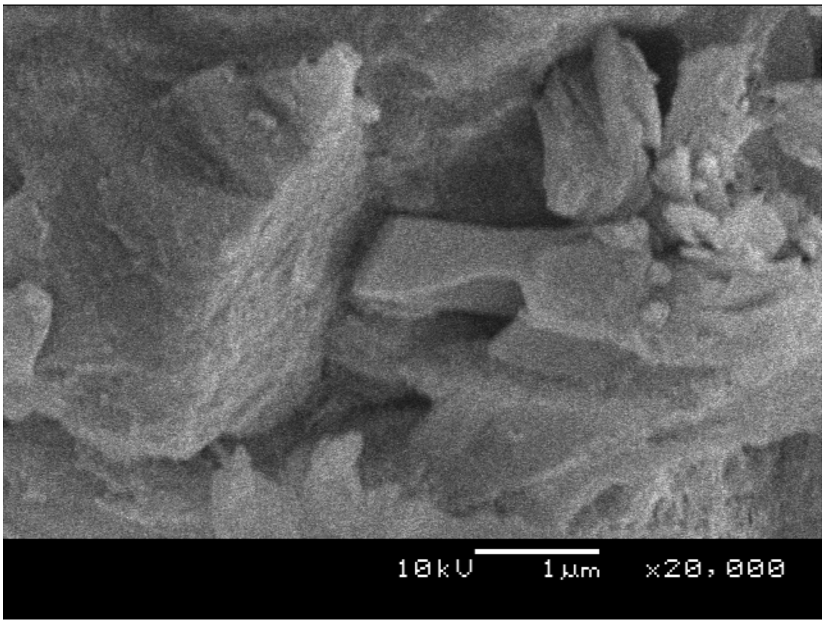


Mbeya pozzolan: 77% Saturation (28 days)

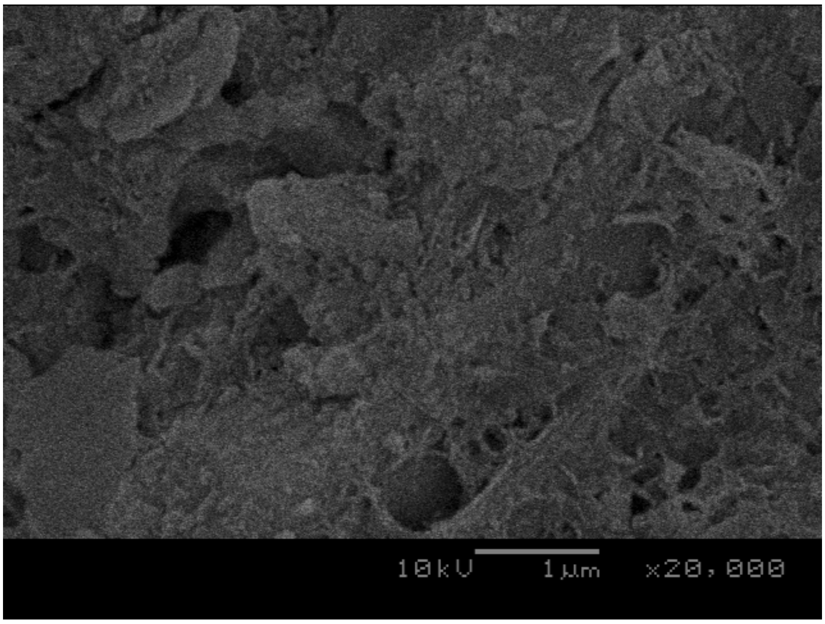


Mbeya pozzolan: 100% Saturation (28 days)

2. Mbeya pozzolan unwashed sand mixes:

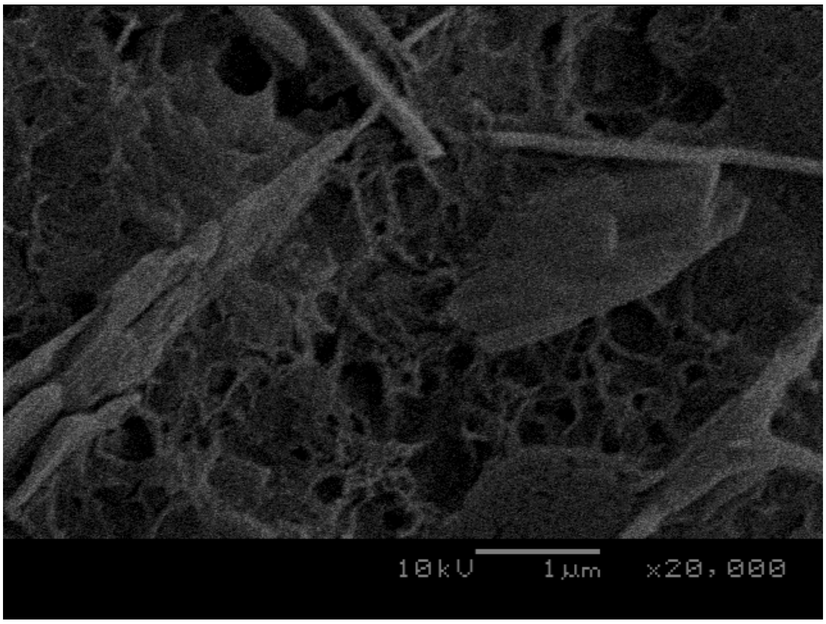


Mbeya pozzolan: 25% Saturation (28 days)

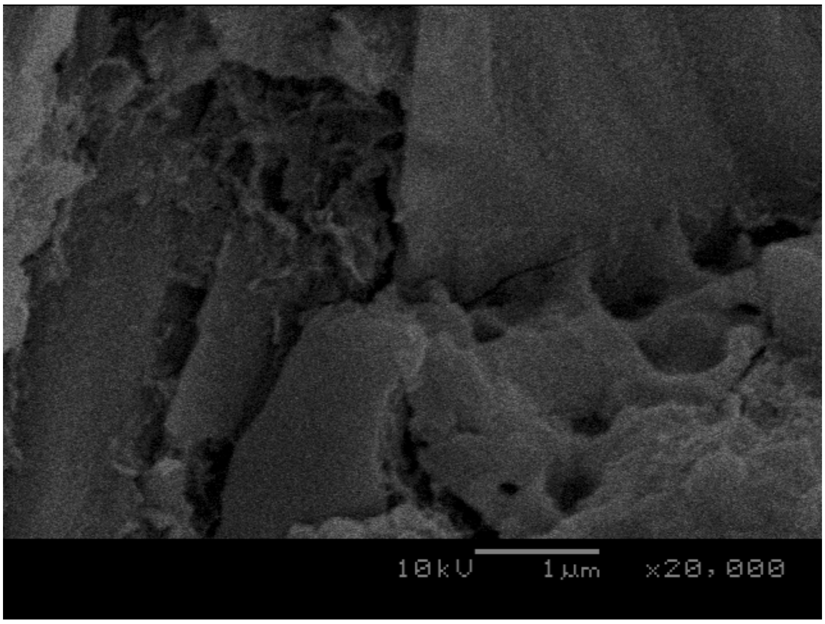


Mbeya pozzolan: 49% Saturation (28 days)

Mbeya pozzolan unwashed sand mixes



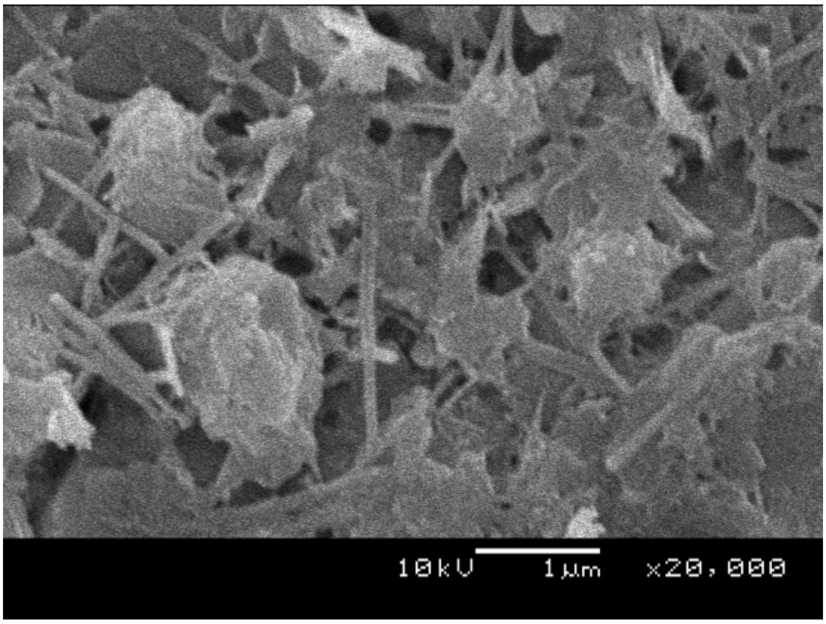
Mbeya pozzolan: 74% Saturation (28 days)



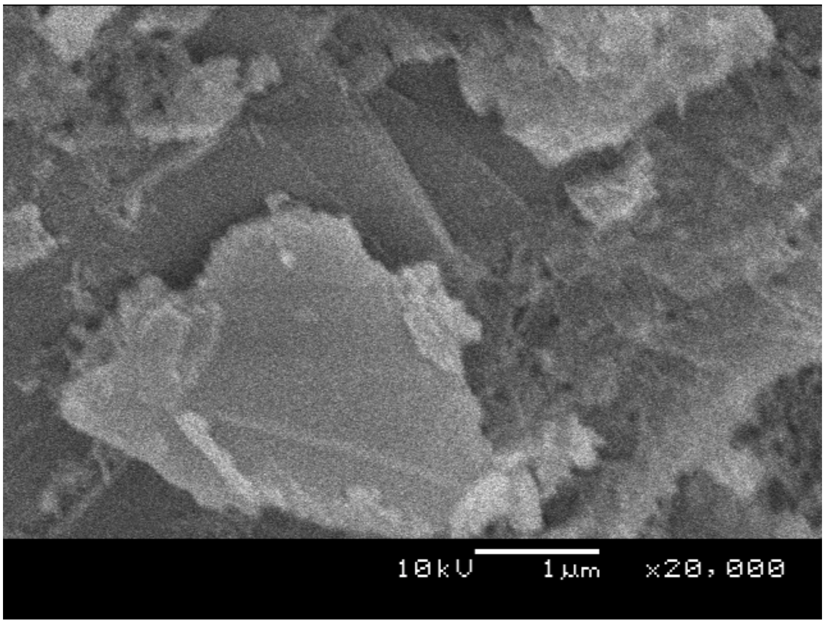
Mbeya pozzolan: 100% Saturation (28 days)

Appendix C3: Cement mixes

1. Cement washed sand mixes

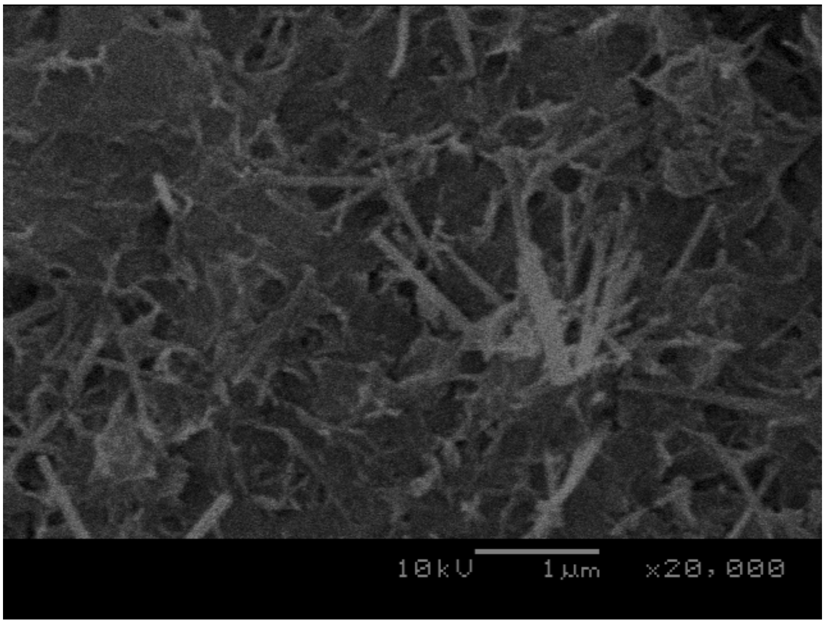


Cement: 26% Saturation (28 days)

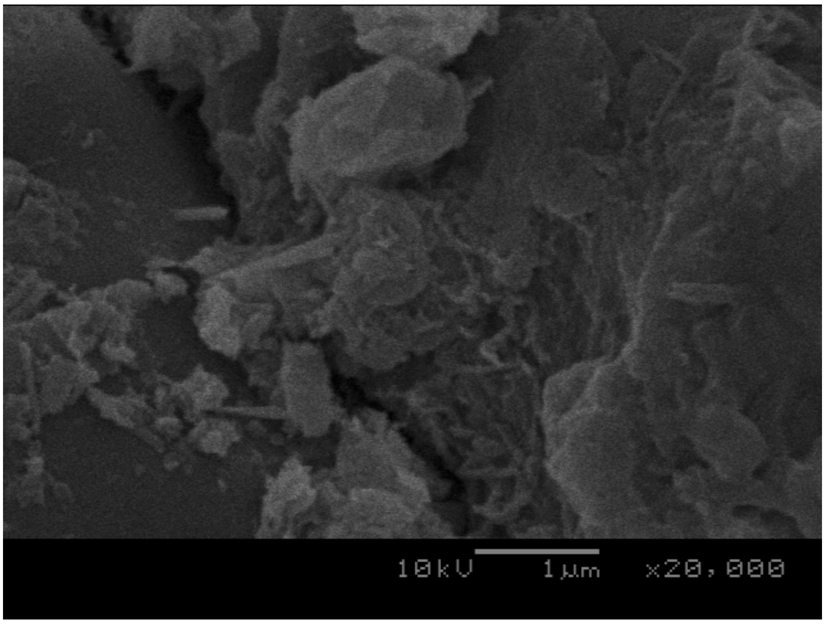


Cement: 51% Saturation (28 days)

Cement washed sand mixes

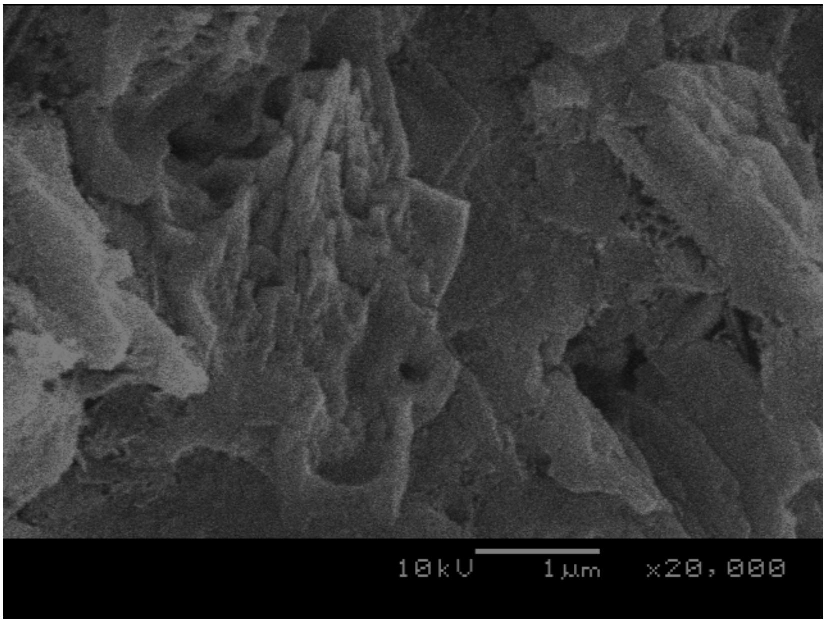


Cement: 77% Saturation (28 days)

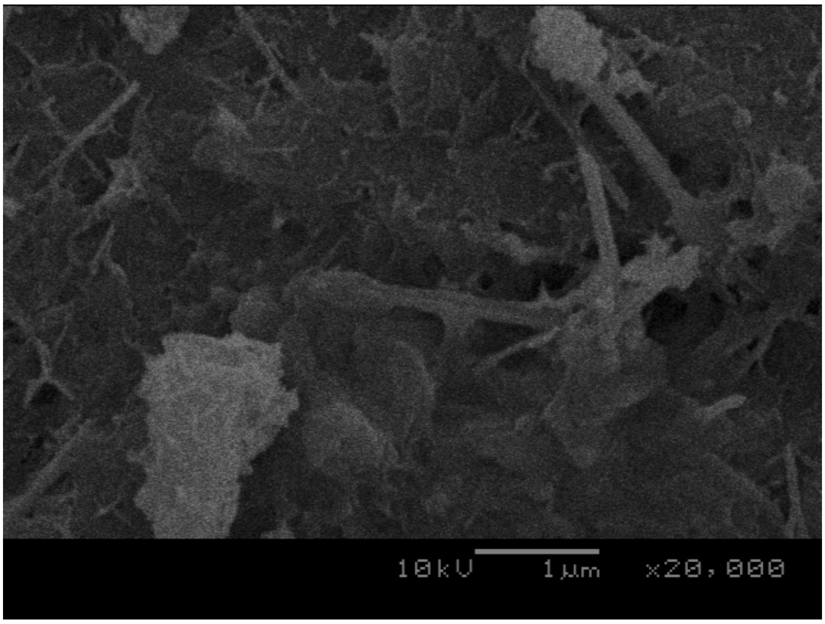


Cement: 100% Saturation (28 days)

2. Cement unwashed sand mixes

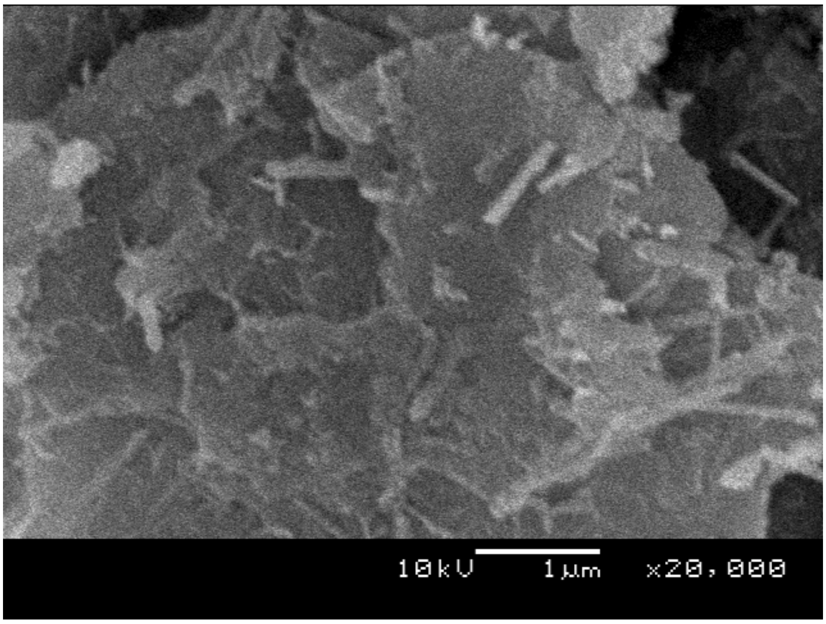


Cement: 26% Saturation (28 days)

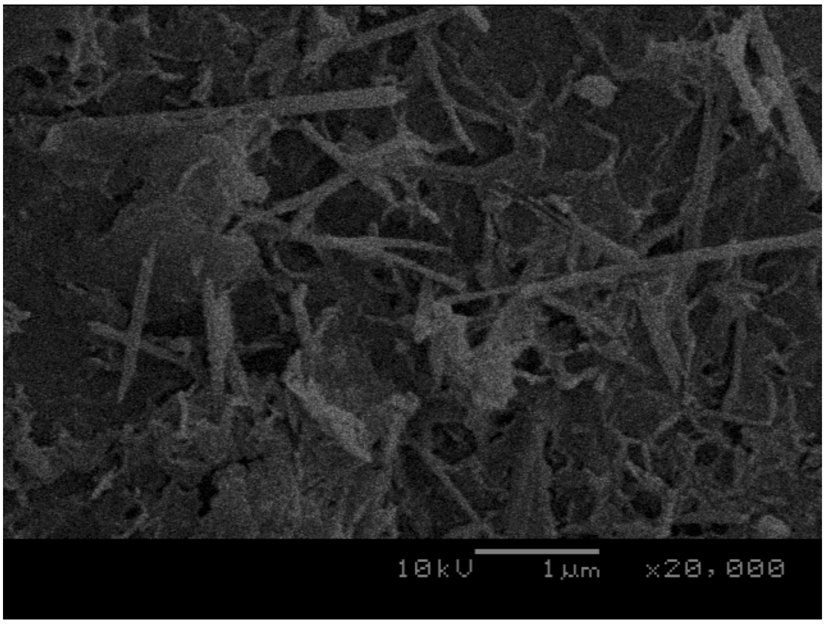


Cement: 52% Saturation (28 days)

Cement unwashed sand mixes



Cement: 77% Saturation (28 days)



Cement: 100% Saturation (28 days)

APPENDIX D

TYPICAL SAS INPUT FILE

Appendix D1: Notations used in the SAS model

Table A: Pilot Study (Degrees of Saturation):

Target Degree of Saturation, %	Y-Value
60	Y1
80	Y2
100	Y3

Table B: Pilot Study (Curing Durations):

Curing Duration, days	X-Value
28	X1
90	X2
180	X3

The tables A and B above presents the notations used in the statistical analysis for the pilot study. For each individual mix (i.e. Arusha, Mbeya and cement mixes), separate statistical analysis was performed.

Table C: Main Study (Degrees of Saturation):

Target Degree of Saturation, %	Y-Value
25	Y1
50	Y2
75	Y3
100	Y4

Table D: Main Study (Sand Type):

Sand Type	X-Value
Washed Sand	X1
Unwaashed Sand	X2

The tables C and D presents the notations used in the main study statistical analysis for each individual mix.

Appendices E and F provide the summaries of analysis for both Pilot and Main studies.

Appendix D2: Typical SAS input file (Pilot study - Arusha)

```
TITLE ARUSHA POZZOLAN A RANDOMIZED COMPLETE BLOCK (PILOT STUDY);
DATA RCB;
INPUT CUR MOIS $ DPTT UCS CBR DEN SHR;
CARDS;
1 69 693 8116 190 1974 -12.067
1 80 778 4915 90 2015 -13.482
1 100 1175 6744 25 2043 -14.467
1 120 1175 6744 25 2043 -14.467
1 69 941 7628 190 1968 -12.067
1 80 747 5019 90 1979 -13.482
1 105 1249 7007 25 2044 -14.467
1 120 1249 7007 25 2044 -14.467
1 69 774 8622 190 1970 -12.067
1 80 764 5310 90 2003 -13.482
1 105 1154 7100 25 2040 -14.467
1 120 1154 7100 25 2040 -14.467
3 69 1036 9775 190 1989 -12.067
3 80 1053 5898 90 1996 -13.482
3 105 1256 7435 25 2041 -14.467
3 120 1256 7435 25 2041 -14.467
3 69 1321 9061 190 1999 -12.067
3 80 954 5733 90 2016 -13.482
3 105 1042 10253 25 2041 -14.467
3 120 1042 10253 25 2041 -14.467
3 69 1148 10379 190 1993 -12.067
3 80 954 5914 90 2017 -13.482
3 105 1209 8100 25 2045 -14.467
3 120 1209 8100 25 2045 -14.467
6 69 862 12070 190 1977 -12.067
6 80 957 7446 90 2016 -13.482
6 105 1287 10022 25 2020 -14.467
6 120 1287 10022 25 2020 -14.467
6 69 1103 12273 190 1984 -12.067
6 80 862 6969 90 1988 -13.482
6 105 1249 10253 25 2041 -14.467
6 120 1249 10253 25 2041 -14.467
6 69 1002 11763 190 1969 -12.067
6 80 896 6969 90 2029 -13.482
6 105 1300 9303 25 2049 -14.467
6 120 1300 9303 25 2049 -14.467
9 69 862 12070 190 1977 -12.067
9 80 957 7446 90 2016 -13.482
9 105 1287 10022 25 2020 -14.467
9 120 1287 10022 25 2020 -14.467
9 69 1103 12273 190 1984 -12.067
9 80 862 6969 90 1988 -13.482
9 105 1249 10253 25 2041 -14.467
9 120 1249 10253 25 2041 -14.467
9 69 1002 11763 190 1969 -12.067
9 80 896 6969 90 2029 -13.482
9 105 1300 9303 25 2049 -14.467
9 120 1300 9303 25 2049 -14.467;
DATA RCB1;
SET RCB;
X1=0; X2=0; X3=0;
Y1=0; Y2=0; Y3=0;
```

```
IF MOIS=69 THEN Y1=1;
IF MOIS=80 THEN Y2=1;
IF MOIS=105 THEN Y3=1;
IF MOIS=120 THEN Y4=1;
IF CUR=1 THEN X1=1;
IF CUR=3 THEN X2=1;
IF CUR=6 THEN X3=1;
IF CUR=9 THEN X4=1;
RATIO = DPTT/UCS;
PROC GLM;
MODEL RATIO = X1 Y1 X2 Y2 X3 Y3
X1*Y1 X1*Y2 X1*Y3
X2*Y1 X2*Y2 X2*Y3
X3*Y1 X3*Y2 X3*Y3;
PROC GLM;
MODEL DPTT = X1 Y1 X2 Y2 X3 Y3
X1*Y1 X1*Y2 X1*Y3
X2*Y1 X2*Y2 X2*Y3
X3*Y1 X3*Y2 X3*Y3
;
PROC GLM;
MODEL UCS = X1 Y1 X2 Y2 X3 Y3
X1*Y1 X1*Y2 X1*Y3
X2*Y1 X2*Y2 X2*Y3
X3*Y1 X3*Y2 X3*Y3
;
PROC GLM;
MODEL DEN = X1 Y1 X2 Y2 X3 Y3
X1*Y1 X1*Y2 X1*Y3
X2*Y1 X2*Y2 X2*Y3
X3*Y1 X3*Y2 X3*Y3
;
PROC GLM;
MODEL CBR = Y1 Y2 Y3
;
PROC GLM;
MODEL SHR = Y1 Y2 Y3
;
PROC SORT DATA = RCB1; BY MOIS;
PROC MEANS DATA = RCB1;
BY MOIS;
VAR DPTT UCS DEN RATIO;
PROC SORT DATA = RCB1; BY CUR;
PROC MEANS;
BY CUR;
VAR DPTT UCS DEN RATIO;
PROC GLM DATA = RCB1;
CLASS MOIS CUR;
MODEL DPTT UCS DEN RATIO = MOIS CUR MOIS*CUR;
PROC GLM DATA = RCB1;
CLASS MOIS CUR;
MODEL DPTT UCS DEN SHR CBR RATIO = MOIS CUR MOIS*CUR;
MEANS MOIS/DUNCAN WALLER;
MEANS CUR/DUNCAN WALLER;
PROC PRINT DATA = RCB1;
RUN;
```

APPENDIX E: Summary of Pilot study SAS analysis

APPENDIX E1: ARUSHA POZZOLAN MIXES (PILOT STUDY)

ARUSHA UCS

PARAMETER	ESTIMATOR	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	9859	9859	9859	9859	9859	9859	9859	9859	9859	9859	9859	9859	9859	9859
X1	-2909	-2909	0	0	0	-2909	-2909	-2909	0	0	0	0	0	0
X2	-1263	0	-1263	0	0	0	0	0	-1263	-1263	-1263	0	0	0
Y1	2176	0	0	2176		2176		0	2176	0	0	2176	0	0
Y2	-2731	0	0	0	-2731		-2731	0	0	-2731	0	0	-2731	0
X1*Y1	-1004	0	0	0	0	-1004	0	0	0	0	0	0	0	0
X1*Y2	862	0	0	0	0	0	862	0	0	0	0	0	0	0
X2*Y1	-1034	0	0	0	0	0	0	0	-1034	0	0	0	0	0
X2*Y2	-16	0	0	0	0	0	0	0	0	-16	0	0	0	0
Total		6950	8596	12035	7128	8122	5081	6950	9738	5848	8596	12035	7128	9859

ARUSHA DPTT

Parameter	Estimator	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
Interceptor	1279	1279	1279	1279	1279	1279	1279	1279	1279	1279	1279	1279	1279	1279
X1	-86	-86	0	0	0	-86	-86	-86	0	0	0	0	0	0
X2	-110	0	-110	0	0	0	0	0	-110	-110	-110	0	0	0
Y1	-290	0	0	-290	0	-290	0	0	-290	0	0	-290	0	0
Y2	-374	0	0	0	-374	0	-374	0	0	-374	0	0	-374	0
X1*Y1	-100	0	0	0	0	-100	0	0	0	0	0	0	0	0
X1*Y2	-56	0	0	0	0	0	-56	0	0	0	0	0	0	0
X2*Y1	289	0	0	0	0	0	0	0	289	0	0	0	0	0
X2*Y2	192	0	0	0	0	0	0	0	0	192	0	0	0	0
TOTAL		1193	1169	989	905	803	763	1193	1168	987	1169	989	905	1279

ARUSHA DENSITY

PARAMETER	ESTIMATE	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	2037	2037	2037	2037	2037	2037	2037	2037	2037	2037	2037	2037	2037	2037
X1	6	6	0	0	0	6	6	6	0	0	0	0	0	0
X2	6	0	6	0	0	0	0	0	6	6	6	0	0	0
Y1	-60	0	0	-60	0	-60	0	0	-60	0	0	-60	0	0
Y2	-26	0	0	0	-26	0	-26	0	0	-26	0	0	-26	0
X1*Y1	-12	0	0	0	0	-12	0	0	0	0	0	0	0	0
X1*Y2	-18	0	0	0	0	0	-18	0	0	0	0	0	0	0
X2*Y1	11	0	0	0	0	0	0	0	11	0	0	0	0	0
X2*Y2	-7	0	0	0	0	0	0	0	0	-7	0	0	0	0
TOTAL		2042	2042	1977	2011	1971	1999	2042	1994	2010	2042	1977	2011	2037

APPENDIX E2: MBEYA POZZOLAN MIXES (PILOT STUDY)

MBEYA DPTT

Parameter	Estimator	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
Interceptor	1262	1262	1262	1262	1262	1262	1262	1262	1262	1262	1262	1262	1262	1262
X1	-751	-751	0	0	0	-751	-751	-751	0	0	0	0	0	0
X2	-266	0	-266	0	0	0	0	0	-266	-266	-266	0	0	0
Y1	-773	0	0	-773	0	-773	0	0	-773	0	0	-773	0	0
Y2	426	0	0	0	426	0	426	0	0	426	0	0	426	0
X1*Y1	506	0	0	0	0	506	0	0	0	0	0	0	0	0
X1*Y2	21	0	0	0	0	0	21	0	0	0	0	0	0	0
X2*Y1	230	0	0	0	0	0	0	0	230	0	0	0	0	0
X2*Y2	21	0	0	0	0	0	0	0	0	21	0	0	0	0
TOTAL		511	996	489	1688	244	958	511	453	1443	996	489	1688	1262

MBEYA UCS

PARAMETER	ESTIMATOR	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	11040	11040	11040	11040	11040	11040	11040	11040	11040	11040	11040	11040	11040	11040
X1	-5746	-5746	0	0	0	-5746	-5746	-5746	0	0	0	0	0	0
X2	-2875	0	-2875	0	0	0	0	0	-2875	-2875	-2875	0	0	0
Y1	-5680	0	0	-5680	0	-5680	0	0	-5680		0	-5680	0	0
Y2	4270	0	0	0	4270		4270	0		4270	0	0	4270	0
X1*Y1	3930	0	0	0	0	3930	0	0	0	0	0	0	0	0
X1*Y2	-2169	0	0	0	0	0	-2169	0	0	0	0	0	0	0
X2*Y1	1933	0	0	0	0	0	0	0	1933	0	0	0	0	0
X2*Y2	-2239	0	0	0	0	0	0	0		-2239	0	0	0	0
Total		5294	8164	5360	15310	3544	7395	5294	4417	10196	8164	5360	15310	11040

MBEYA DENSITY

PARAMETER	ESTIMATE	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	2097	2097	2097	2097	2097	2097	2097	2097	2097	2097	2097	2097	2097	2097
X1	-6	-6	0	0	0	-6	-6	-6	0	0	0	0	0	0
X2	-3	0	-3	0	0	0	0	0	-3	-3	-3	0	0	0
Y1	-155	0	0	-155	0	-155	0	0	-155	0	0	-155	0	0
Y2	-42	0	0	0	-42	0	-42	0	0	-42	0	0	-42	0
X1*Y1	23	0	0	0	0	23	0	0	0	0	0	0	0	0
X1*Y2	11	0	0	0	0	0	11	0	0	0	0	0	0	0
X2*Y1	-11	0	0	0	0	0	0	0	-11	0	0	0	0	0
X2*Y2	-1	0	0	0	0	0	0	0	0	-1	0	0	0	0
TOTAL		2092	2095	1942	2055	1959	2060	2092	1928	2052	2095	1942	2055	2097

APPENDIX E3: CEMENT MIXES (PILOT STUDY)

CEMENT UCS

PARAMETER	ESTIMATOR	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	1357	1357	1357	1357	1357	1357	1357	1357	1357	1357	1357	1357	1357	1357
X1	-520	-520	0	0	0	-520	-520	-520	0	0	0	0	0	0
X2	-55	0	-55	0	0	0	0	0	-55	-55	-55	0	0	0
Y1	4704	0	0	4704	0	4704	0	0	4704	0	0	4704	0	0
Y2	2335	0	0	0	2335	0	2335	0	0	2335	0	0	2335	0
X1*Y1	-2184	0	0	0	0	-2184	0	0	0	0	0	0	0	0
X1*Y2	-662	0	0	0	0	0	-662	0	0	0	0	0	0	0
X2*Y1	-1861	0	0	0	0	0	0	0	-1861	0	0	0	0	0
X2*Y2	-604	0	0	0	0	0	0	0	0	-604	0	0	0	0
Total		837	1301	6061	3692	3357	2510	837	4144	3033	1301	6061	3692	1357

CEMENT DPTT

Parameter	Estimator	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
Interceptor	191	191	191	191	191	191	191	191	191	191	191	191	191	191
X1	-134	-134	0	0	0	-134	-134	-134	0	0	0	0	0	0
X2	-54	0	-54	0	0	0	0	0	-54	-54	-54	0	0	0
Y1	558	0	0	558	0	558	0	0	558	0	0	558	0	0
Y2	194	0	0	0	194	0	194	0	0	194	0	0	194	0
X1*Y1	-314	0	0	0	0	-314	0	0	0	0	0	0	0	0
X1*Y2	-22	0	0	0	0	0	-22	0	0	0	0	0	0	0
X2*Y1	-163	0	0	0	0	0	0	0	-163	0	0	0	0	0
X2*Y2	6	0	0	0	0	0	0	0	0	6	0	0	0	0
TOTAL		58	137	749	385	301	230	58	532	336	137	749	385	191

CEMENT DENSITY

PARAMETER	ESTIMATE	X1	X2	Y1	Y2	X1*Y1	X1*Y2	X1*Y3	X2*Y1	X2*Y2	X2*Y3	X3*Y1	X3*Y2	X3*Y3
INTERCEPT	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959	-1959
X1	-9	-9	0	0	0	-9	-9	-9	0	0	0	0	0	0
X2	-4	0	-4	0	0	0	0	0	-4	-4	-4	0	0	0
Y1	70	0	0	70	0	70	0	0	70	0	0	70	0	0
Y2	2	0	0	0	2	0	2	0	0	2	0	0	2	0
X1*Y1	22	0	0	0	0	22	0	0	0	0	0	0	0	0
X1*Y2	55	0	0	0	0	0	55	0	0	0	0	0	0	0
X2*Y1	-1	0	0	0	0	0	0	0	-1	0	0	0	0	0
X2*Y2	47	0	0	0	0	0	0	0	0	47	0	0	0	0
TOTAL		-1968	-1964	-1890	-1957	-1876	-1912	-1968	-1895	-1915	-1964	-1890	-1957	-1959

APPENDIX F: Summary of Main study SAS analysis

APPENDIX F1: ARUSHA POZZOLAN MIXES (MAIN STUDY)

UCS - Arusha Pozzolan Mixes

PARAMETER	ESTIMATOR	X1*Y1	X1*Y2	X1*Y3	X1*Y4	X1*Y5	X2*Y1	X2*Y2	X2*Y3	X2*Y4	X2*Y5
INTERCEPT	5303	5303	5303	5303	5303	5303	5303	5303	5303	5303	5303
X1*Y1	-5303	-5303	0	0	0	0	0	0	0	0	0
X1*Y2	-994	0	-994	0	0	0	0	0	0	0	0
X1*Y3	3366	0	0	3366	0	0	0	0	0	0	0
X1*Y4	-302	0	0	0	-302	0	0	0	0	0	0
X1*Y5	0	0	0	0	0	0	0	0	0	0	0
X2*Y1	-3676	0	0	0	0	0	-3676	0	0	0	0
X2*Y2	646	0	0	0	0	0	0	646	0	0	0
X2*Y3	3621	0	0	0	0	0	0	0	3621	0	0
X2*Y4	0	0	0	0	0	0	0	0	0	0	0
X2*Y5	0	0	0	0	0	0	0	0	0	0	0
		0	4309	8669	5001	5303	1627	5949	8924	5303	5303

DENSITY - Arusha Pozzolan Mixes

PARAMETER	ESTIMATOR	Y1	Y2	Y3	Y4	Y5
INTERCEPT	1954	1954	1954	1954	1954	1954
Y1	6	6	0	0	0	0
Y2	7	0	7	0	0	0
Y3	93	0	0	93	0	0
Y4	9	0	0	0	9	0
Y5	0	0	0	0	0	0
		1960	1962	2047	1963	1954

APPENDIX F2- MBEYA POZZOLAN MIXES (MAIN STUDY)

UCS - Mbeya Pozzolan Mixes

PARAMETER	ESTIMATOR	X1*Y1	X1*Y2	X1*Y3	X1*Y4	X2*Y1	X2*Y2	X2*Y3	X2*Y4
INTERCEPT	2003	2003	2003	2003	2003	2003	2003	2003	2003
X1*Y1	-1029	-1029	0	0	0	0	0	0	0
X1*Y2	4949	0	4949	0	0	0	0	0	0
X1*Y3	2333	0	0	2333	0	0	0	0	0
X1*Y4	957	0	0	0	957	0	0	0	0
X2*Y1	117	0	0	0	0	117	0	0	0
X2*Y2	4175	0	0	0	0	0	4175	0	0
X2*Y3	2144	0	0	0	0	0	0	2144	0
X2*Y4	0	0	0	0	0	0	0	0	0
		974	6952	4336	2960	2120	6178	4147	2003

DENSITY - Mbeya Pozzolan Mixes

PARAMETER	ESTIMATOR	Y1	Y2	Y3	Y4	Y5
INTERCEPT	1954	1954	1954	1954	1954	1954
Y1	6	6	0	0	0	0
Y2	7	0	7	0	0	0
Y3	93	0	0	93	0	0
Y4	9	0	0	0	9	0
Y5	0	0	0	0	0	0
		1960	1962	2047	1963	1954

APPENDIX F3: CEMENT MIXES (MAIN STUDY)

UCS - Cement Mixes

PARAMETER	ESTIMATOR	X1*Y1	X1*Y2	X1*Y3	X1*Y4	X2*Y1	X2*Y2	X2*Y3	X2*Y4
INTERCEPT	1758	1758	1758	1758	1758	1758	1758	1758	1758
X1*Y1	-1672	-1672	0	0	0	0	0	0	0
X1*Y2	1278	0	1278	0	0	0	0	0	0
X1*Y3	1859	0	0	1859	0	0	0	0	0
X1*Y4	360	0	0	0	360	0	0	0	0
X2*Y1	620	0	0	0	0	620	0	0	0
X2*Y2	3666	0	0	0	0	0	3666	0	0
X2*Y3	1566	0	0	0	0	0	0	1566	0
X2*Y4	0	0	0	0	0	0	0	0	0
		86	3036	3617	2118	2378	5424	3324	1758

DENSITY - Cement Mixes

PARAMETER	ESTIMATOR	Y1	Y2	Y3	Y4	Y5
INTERCEPT	1954	1954	1954	1954	1954	1954
Y1	6	6	0	0	0	0
Y2	7	0	7	0	0	0
Y3	93	0	0	93	0	0
Y4	9	0	0	0	9	0
Y5	0	0	0	0	0	0
		1960	1962	2047	1963	1954