ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

IMPROVING ENGINEERING PROPERTIES OF EXPANSIVE SOILS OF ADDIS ABABA BY BLENDING WITH POZZOLANIC SAND FOUND AROUND MEKI TOWN

A Thesis in Geotechnical Engineering

By Wolde Ketema Muleta

Addis Ababa

Advisor: Dr.Ing:- Samuel Tadesse

A Thesis Submitted to

School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

(Geotechnical Engineering)

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

This to certify that the thesis prepared by Wolde Ketema entitled "**Improving the Engineering properties of Expansive Soil of Addis Ababa by blending with Pozzolanic sand found around meki town**", submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering complies with the regulations of the University and meets standards with respect to originality and quality.

Approved by the Board of Examiners

Dr.Ing:- Samuel Tadesse

____________________	____________________	____________________
Advisor	**Signature**	**Date**
____________________	____________________	____________________
Internal Examiner	**Signature**	**Date**
____________________	____________________	____________________
External Examiner	**Signature**	**Date**
____________________	____________________	____________________
Chairperson	**Signature**	**Date**

UNDERTAKING

I the undersigned, certify that research work titled "**Improving Engineering properties of Expansive Soil of Addis Ababa by blending with Pozzolanic sand found around meki town**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Wolde Ketema

____

December 2020 G.C

Addis Ababa University

Addis Ababa, Ethiopia

ABSTRACT

Black Cotton Soils exhibit high swelling and shrinking when exposed to changes in moisture content and are most troublesome soils leading to failure of civil engineering structures constructed on such soils. The purpose of this study is to investigate the effect of pozzolanic sand on improving the engineering properties of expansive soil collected from Addis Ababa around Koyefeche condominium site.

In this research, pozzolanic sand found around Meki Town was used to treat problematic expansive soil. The pozzolanic sand used for treating the expansive soil was pulverized to pass certain sieve sizes. In this particular research, to consider the size effect of pozzolanic sand on treating the expansive soil, pozzolanic sand passing 150μm and 75μm sieve size were used to treat the expansive soil. The pozzolanic sand crushed to a size less than 150μm was blended to the soil up to 25% at 5% percentage increments whereas the pozzolanic sand crushed to a size less than 75μm was blended with soil to a maximum of 15% at 2.5% percentage increments by dry weight of the soil.

Soil treated with the pozzolanic sand showed improvements to plasticity, strength and swelling. As soil is treated with pozzolanic sand the plasticity of the soil decreased, the dry density of the soil increased, the optimum moisture content decreased and the swelling pressure also decreased. Besides, the compressive strength of the soil increased as the percentage of pozzolanic sand is increasing. For the sizes of pozzolanic sand considered the pozzolanic sand crushed to a size less than 75μm improved the engineering properties of expansive soil than the pozzolanic sand crushed to a size less than 150μm when blended with the expansive soil.

Keywords: Expansive Soil, Pozzolanic Sand, Soil Improvement, Atterberg Limit, Compaction, Swelling Pressure, UCS, Free Swell Value.

ACKNOWLEDGEMENT

First of all, I would like to thank the Almighty God for giving me the courage to work on such long-lasting thesis work. Next, my deepest gratitude goes to my thesis advisor **Dr.Ing:- Samuel Tadesse** for giving me a chance to work on such interesting title, made me interested to work on with his valuable guidance and advise through the work of this thesis above all he suggested me to think in a way that I haven't seen on my side.

I would also like to thank all staffs of the geotechnical laboratory who were candid to me during my laboratory work and giving me extra time to work in the laboratory and supported me on difficulties I faced during my work.

Last but not least to my friends those who discussed my problems during the work of the thesis by spending more time with me for discussion.

TABLE OF CONTENTS

ABSTRACT I

ACKNOWLEDGEMENT II

TABLE OF CONTENTS III

LIST OF TABLES VIII

LIST OF FIGURES IX

LIST OF SYMBOLS AND ABBREVIATIONS XI

CHAPTER 1 INTRODUCTION 1

1.1 Background 1
1.2 Statement of the problem 2
1.3 The objective of the study 2
1.3.1 General Objective 2
1.3.2 Specific Objective 2
1.4 Methodology 2
1.4.1 Size of pozzolanic sand used for treating the expansive soil 3
1.4.2 Sample preparation and percentage of pozzolanic sand 4
1.5 Organization of the Thesis 5

CHAPTER 2 LITERATURE REVIEW 6

2.1 Expansive soils 6
2.1.1 Origin and formation of expansive clay soils 6
2.1.2 Characteristics of Expansive clay Soil 7
2.1.3 Distribution of expansive soil 7
2.1.4 Distribution of expansive soils in Addis Ababa 8
2.1.5 Clay Mineralogy 9
2.1.5.1 Kaolinite Group 10
2.1.5.2 Mica-like group 11
2.1.5.3 The Smectite group 11
2.1.6 Identification and classification of expansive clay soil 13

2.1.6.1 General ... 13
2.1.6.2 Identification of expansive clay soil ... 13
2.1.6.2.1 Field Identification ... 13
2.1.6.2.2 Mineralogical Methods ... 15
2.1.6.3 Engineering Classification methods ... 15
2.1.6.3.1 Indirect methods ... 15
2.1.6.3.2 Direct methods ... 16
2.1.6.3.3 Factors affecting the swelling potential and swelling pressure ... 17
2.1.7 Classification methods to expansive soil ... 18
2.1.7.1 Skempton Method ... 18
2.1.7.2 U.S.B.R. Classification Method ... 20
2.1.7.3 Seed's classification Method (Activity Method) ... 20
2.1.7.4 The unified soil classification method ... 20
2.1.8 Overview of damage due to expansive soils ... 21
2.1.8.1 General ... 21
2.1.8.2 Recognition of damages due to expansive soil ... 22
2.1.8.3 Damage due to Expansive soil in Addis Ababa ... 23
2.2 Properties of natural pozzolan and its origin ... 24
2.2.1 Sources of natural pozzolan ... 24
2.2.1.1 Volcanic ash ... 24
2.2.1.2 Clay minerals ... 24
2.2.1.3 Diatomaceous earth ... 25
2.2.2 Chemical composition of natural pozzolan ... 25
2.2.3 Mineralogical composition of natural pozzolan ... 26
2.2.4 Particle characteristics of natural pozzolans ... 26
2.2.5 Classification of natural pozzolans ... 27
2.2.6 Natural pozzolans in Ethiopia ... 27

2.3 Review on Improvements of expansive soil 28
2.3.1 Soil stabilization 28
2.3.2 Mechanical Stabilization 29
2.3.3 Chemical Stabilization....29
2.3.3.1 Lime stabilization 30
2.3.3.2 Cement stabilization 31
2.3.4 Previous studies on the use of pozzolans as a stabilizing agent 32
2.3.4.1 Pumice 32
2.3.4.2 Stone dust 33
2.3.4.3 Use of other natural pozzolanas for stabilization 33
CHAPTER 3 EXPERIMENTAL STUDY ON PROPERTIES OF NATURAL SOIL AND POZZOLANIC SAND 35
3.1 General....35
3.2 Material....35
3.2.1 Expansive (Natural) soil 35
3.2.2 Pozzolanic Sand (Stabilizing Agent) 36
3.2.3 Water....36
3.3 Laboratory tests on the characterization of natural/expansive soil....36
3.3.1 Grain size analysis test....36
3.3.2 Atterberg limits tests....37
3.3.3 Free Swell Test for Natural soil 38
3.3.4 Specific Gravity Test 38
3.3.5 Moisture Density Relations (Compaction) Tests of Natural Soil 39
3.3.6 Unconfined Compression Strength (UCS) Test for Natural Soil 40
3.3.7 Swell and Consolidation Test 41
3.4 Classification of the Natural soil 41
3.4.1 Unified Soil Classification System (USCS) 41
3.4.2 Classification of Soils based on its Free Swell value and plasticity Index....42

3.4.3 Classification of soil Based on Activity Number, Ac....42
3.5 Laboratory Tests on Characterization of Pozzolanic Sand....44
3.5.1 Grain Size Analysis for PS....44
3.5.2 The Atterberg Test Results of the Pozzolanic sand....46
3.5.3 Moisture Density Relation for Pozzolanic Sand....47
3.5.4 Unconfined Compression Strength Test (UCS) for Pozzolanic Sand....48
3.5.5 Chemical Composition of Pozzolanic Sand....49
CHAPTER 4 RESULTS AND DISCUSSION ON LABORATORY TESTS OF BLENDED SOIL....50
4.1 Introduction....50
4.2 Effect of Pozzolanic Sand on Free-Swell of Expansive Soil....50
4.3 Effect of Pozzolanic Sand on the specific gravity of Expansive Soil....52
4.4 Effect of Pozzolanic sand on Atterberg limit....53
4.4.1 Effect of Pozzolanic sand on plasticity index....53
4.4.2 Effect of Pozzolanic sand on soil classification....54
4.5 Effect of Pozzolanic Sand on Compaction Characteristics of soil....56
4.5.1 Effect of PS on Dry Density of Expansive Soil....57
4.5.2 Effect of PS on Optimum Moisture Content of Expansive Soil....58
4.6 Effect of Pozzolanic Sand on Unconfined Compressive Strength of Expansive Soil 59
4.7 Effect of Pozzolanic sand on Swelling Pressure of Expansive Soil....61
CHAPTER 5 CONCLUSION AND RECOMMENDATION....63
5.1 Conclusion....63
5.2 Recommendation....65
REFERENCES....66
APPENDIX A: ATTERBERG LIMITS FOR BLENDED SOIL....69
APPENDIX B: COMPACTION CURVE FOR BLENDED SOIL....80
APPENDIX C: UNCONFINED COMPRESSION STRENGTH TESTS FOR BLENDED SOIL....91

APPENDIX D: SWELL-CONSOLIDATION TESTS FOR BLENDED SOIL......102

APPENDIX E: LABORATORY TEST PHOTOS 110

APPENDIX F: CHEMICAL COMPOSITION OF POZZOLANIC SAND...........115

LIST OF TABLES

Table 1-1:- Proportions of expansive soil and pozzolanic sand used in the laboratory test 4
Table 2-1: Expansion potential of soil and plasticity index (Nelson, Chao, Overton, & Nelson, 2015).. 16
Table 2-2: Classification of expansive soil based on activity (Nelson & Miller, 1992). . 19
Table 2-3: Activity of clay minerals (Nelson & Miller, 1992).. 19
Table 2-4: Relationship between the degree of swell and clay properties (Tefera & Leikun, 1999).. 20
Table 2-5: Relationship between the degree of swell and swelling potential (Tefera & Leikun, 1999).. 20
Table 2-6: Typical chemical and mineralogical analysis of natural pozzolan (Desta, 2003) 26
Table 2-7: Standard Specification of natural pozzolans according to ASTM C 618 27
Table 3-1: Grain size analysis test results (Sieve and Hydrometer)................................ 37
Table 3-2: Results of Atterberg limit tests for natural soil 38
Table 3-3: Summary of properties of natural soil.. 43
Table 3-4: Test conducted on pozzolanic sand and test results (Unpulverized)............... 44
Table 3-5: Complete silicate analysis test result.. 49

LIST OF FIGURES

Figure 2-1: Distribution of Expansive soil in Ethiopia (Mada, 2016)................................8
Figure 2-2: Engineering geological map of Addis Ababa (Simon, 2016)..........................9
Figure 2-3: Diagrammatic representation of Kaolinite group (Baser, 2009)....................10
Figure 2-4: Diagrammatic Sketch of the illite (Baser, 2009) 11
Figure 2-5: Diagrammatic sketch of the smectite group (Murray, 2007).........................12
Figure 2-6: Schematic representation of clay minerals (Baser, 2009).............................13
Figure 2-7: Cracked surface during the dry season (Rogers, Olshansky, & Rogers, 2016) 14
Figure 2-8: Activity chart (Tefera & Leikun, 1999)..19
Figure 2-9: Effect of swelling on Structure (Mir, 2015) 22
Figure 2-10: Damage to house supported on shallow piers (Rogers, Olshansky, & Rogers, 2016)..23
Figure 3-2: Pozzolanic sand deposit to the west side of Meki Town 36
Figure 3-3: Particle Size distribution for the sample tested...37
Figure 3-4: Flow curve for Natural soil 38
Figure 3-5: Compaction curve for Natural/Native soil..40
Figure 3-6: Axial Stress Vs Axial Strain plot of UCS test result for the Natural soil......40
Figure 3-7: Void ratio Vs Log (pressure) curve for natural soil.......................................41
Figure 3-8: Location of Natural soil on Plasticity chart as per USCS ASTM D-2487-0042
Figure 3-9: Particle Size distribution of PS as brought from the quarry. 45
Figure 3-10: Particle size distribution curve for pulverized pozzolanic sand 45
Figure 3-11: Liquid limit determination for pozzolanic sand..47
Figure 3-13: Compaction curve for the pozzolanic sand...48
Figure 3-14: Pozzolanic sand under compression 48
Figure 3-15: Axial stress Vs Axial strain plot of UCS test result for the pozzolanic sand. 49
Figure 4-1: Effect of pozzolanic sand on the free swell value of expansive soil 51
Figure 4-2: Variation of specific gravity with the percentage of PS 52
Figure 4-3: Variation of plasticity index with the percentage of pozzolanic sand 54
Figure 4-4: Location of soil on the plasticity chart with varying percentage of pozzolanic sand for classification. 55

Figure 4-5: Location of soil on the plasticity chart with varying percentage of pozzolanic sand for classification. 56
Figure 4-6: Summary of Compaction Curves for PS<75μm 57
Figure 4-7: Summary of Compaction curves for PS<150μm 57
Figure 4-8: Variation of maximum dry density with a pozzolanic sand content.............58
Figure 4-9: Effect of pozzolanic sand on OMC of expansive soil 59
Figure 4-10: Effect of pozzolanic sand on UCS of expansive soil..................................60
Figure 4-11: Effect of pozzolanic sand on the swelling pressure 62

LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO	American Association of Highway and Transportation Officials
A_c	Activity Number
Al	Aluminum
ASTM	American Standards for Testing and Materials
c	Cohesion
CH	Fat Clay or Clay of High plasticity
D	Diameter
e	Void Ratio
ES	Expansive Soil
FSV	Free Swell Value
G_t	Specific Gravity at test temperature
Gs	Specific Gravity
$G_{20^{o}c}$	Specific Gravity at room temperature
H	Height
K	Temperature Coefficient
LL	Liquid Limit
MDD	Maximum Dry Density
MH	Silt with High Plasticity
M_s	Mass of Soil
M_a	Mass of Pycnometer filled with Water at test temperature
M_b	Mass of Pycnometer filled with water and Soil at test temperature
N	Number of blows
OMC	Optimum Moisture Content
p	Pressure
PI	Plasticity Index
PL	Plastic Limit
PS	Pozzolanic Sand
q_u	Ultimate Compressive Strength
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
SL	Shrinkage Limit
SP	Swelling Pressure
Vs	Initial Volume
V_f	Final Volume
cc	Centimeter Cube
g	Gram
Kg	Kilogram
ml	milliliter
m	meter
mm	Millimeter
µm	micrometer
KN	Kilo newton
KPa	Kilo Pascal
ºC	Degree Celsius

CHAPTER 1 INTRODUCTION

1.1 Background

Expansive soils are fine-grained soil or decomposed rocks that show large volume change when exposed to fluctuations of moisture content. Swelling-shrinkage behaviour is likely to take place near the ground surface where it is directly subjected to seasonal and environmental variations. The expansive soils are most likely to be unsaturated and have highly reactive clay minerals comprising of montmorillonite. Most of the severe damage concerning expansive soils is dependent on the amount of monovalent cation absorbed to the clay minerals. (Nelson & Miller, 1992), (Bel, 1996), (Sisay, 2004). Construction of residential buildings and other civil engineering structures such as highways, bridges, airports on expansive soil is highly risky in that such soil is susceptible to cycles of drying and wetting, inducing shrinkage and swelling behaviour under pavements and building foundations, which results in cracking to structural and non-structural elements.

An increasing number of ground improvement techniques have been suggested for dealing with problematic soil such as the application of sand cushion technique, belled piers and granular pile-anchors. Besides, chemical stabilization is the most popular method utilized to enhance the physical and mechanical properties of problematic soils consisting of soft soil and expansive soil. The chemical ground improvement approach is a proven technique in improving engineering properties of problematic soils and is highly applicable for lightly loaded structures such as road pavements and low-rise residential buildings. In recent years, a considerable number of laboratory and field experiments have been carried out and extensive studies have been conducted on reactive soil using various additives such as cement and lime.

Several by-products including fly ash, rice husk ash, bagasse ash, just to name a few, were investigated by using each alone or in combination with other additives. Although a growing number of investigations have been undertaken to reinforce reactive soil using waste by-products to diminish the effects of the swelling-shrinkage characteristics and enhance the mechanical properties, there are still not adequate studies on the influence of sand, particularly in pozzolanic sand stabilized expansive soil.

1.2 Statement of the problem

Expansive soils have a pronounced effect on the damages caused to building due to soil movement or deformation that lead to the improper functioning of structural and non-structural elements. Most of the time in structures which have been affected by shrinking and swelling of the soil visible cracks or wide openings occur on the walls and ground floors, failure of structural connection between structural elements such as beam-column connections, failure of truss elements at the roof levels, difficulty in closing or opening of doors and windows are among the signs of defects that expansive soils impose on structures. But all these defects can be mitigated by stabilizing the expansive soil underlying the structural system by applying some stabilizing methods before construction of every infrastructure.

1.3 The objective of the study

1.3.1 General Objective

The main objective of this thesis is to evaluate the influence of pozzolanic sand found around Meki Town to enhance the engineering properties of expansive soils of Addis Ababa.

1.3.2 Specific Objective

The specific objective of the study is

- ✓ To determine the physical or engineering properties of the soil under study, identifying whether the soil is expansive or not at its natural.
- ✓ Studying some of the engineering properties of pozzolanic sand
- ✓ Evaluating the engineering properties of the soil after treatment and identifying the optimum amount of pozzolanic sand for improving the engineering properties of expansive soil.

1.4 Methodology

To achieve the above-mentioned objectives, field observation of certain active construction sites where expansive soil exists was done according to geological maps of Addis Ababa to get samples easily for laboratory testing. Representative samples of soil for testing were collected from Akaki Kality sub-city, Koye condominium site where building with five-story is under construction. The soil found was completely black in colour and the building was to be constructed on mat foundation which was placed at 4.7m below normal ground level. Certain laboratory tests were conducted to identify whether the soil collected was expansive or not.

Pozzolanic sand was collected from around Meki town which is located in East African Great Rift Valley where natural pozzolanic materials are abundantly available. Experience on the treatment of expansive soil with similar pozzolanas was reviewed and method of usage of treated soil for the proposed purpose. Damage due to expansive soil was also reviewed here in Addis Ababa from previously done research.

From collected representative samples, the following laboratory tests were done to characterize the engineering properties of natural or untreated soil and soil blended with pozzolanic sand.

- Grain Size analysis
- Specific gravity test
- Free swell test
- Atterberg Limit test
- Moisture-Density relation (Compaction) test
- Unconfined compression strength test
- Swell-consolidation tests

All the above tests were conducted for the natural soil and the soil treated with pozzolanic sand. After completing the laboratory tests analyzing the test results and discussion on the engineering properties of treated soil to evaluate the effect of pozzolanic sand on engineering properties of expansive soil were carried out.

1.4.1 Size of pozzolanic sand used for treating the expansive soil

To investigate the effect of pozzolanic sand size, two-particle sizes of pozzolanic sand were considered. The first particle size of the pozzolanic sand considered is 150µm and the second is 75µm, in this regard, it is intended to determine which particle size of pozzolanic sand is appropriate for improving the engineering properties of expansive soil.

1.4.2 Sample preparation and percentage of pozzolanic sand

Samples collected from the field were brought to the laboratory and prepared for the study according to the predetermined amount of the pozzolanic sand, and different percentage rates of pozzolanic sand were used to study the effect it has on the engineering properties of the expansive soils collected from the site. The pozzolanic sand was crushed to make it more reactive with the natural soil and to take into account the effect of particle size of pozzolanic sand on improving the engineering properties of blended soil. For this purpose, the pozzolanic sands were crushed to pass 150µm and 75µm sieve size. The size with 150µm contains all particles of pozzolanic sand less than 150µm including finer particles less than 75µm but those that passed sieve size 75µm contains all particles less than 75µm but excluding those greater in size than 75µm. The percentages rates of the pozzolanic sand range from 0 to 25% by dry weight of the natural soil at 5% intervals for sand crushed and passed sieve size 150µm and 0 to 15% at 2.5% interval for sand crushed to a particle size less than 75µm. As curing enhances the mixture to be uniform and to give adequate time for a reaction to take place between the natural soil and pozzolanic sand the mix will be cured for seven days.

These two cases of pozzolanic sand sizes were selected to take into account the effects of sizes of the stabilizing agent on engineering properties of the expansive soil. The test was conducted for uncured blended samples and blended samples cured for seven days, and all possible outcomes of the test were discussed in the report for the research.

The prepared mix proportions of the test samples are as follows:

Table 1-1:- Proportions of expansive soil and pozzolanic sand used in the laboratory test

PS<150µm	PS<75µm
untreated soil	
5%PS +95%ES	2.5% PS + 97.5% ES
10%PS +90%ES	5% PS + 95% ES
15%PS +85%ES	7.5% PS + 92.5% ES
20%PS +80%ES	10% PS + 90% ES
25%PS +75%ES	12.5% PS + 87.5% ES
	15% PS + 85% ES

1.5 Organization of the Thesis

The thesis is organized into six chapters, where the first chapter deals with the introduction, objectives, and methodologies of the study. Chapter two emphasizes on review of literature which focuses mainly on origin, behaviour, and distribution of expansive soil in Addis Ababa city. Moreover, literature that focuses on damages caused by expansive soils and possible remedial measures was reviewed in this chapter. Improvement of engineering properties of expansive soil with similar material like lime, cement, Pumice, Stone Dust and other pozzolanic materials were reviewed in this chapter. Chapter three deals with the experimental investigation on the engineering properties of the native soil and pozzolanic sand. The classification of the native soil and the pozzolanic sand was following ASTM standards for soil. Laboratory test results of blended soil that is the effect of pozzolanic sand; on the free swell of expansive soil, on the specific gravity of expansive soil and Atterberg limits are discussed in chapter four. The effect of pozzolanic sand on compaction characteristics of expansive soil, on unconfined compressive strength of expansive soil and swelling pressure of expansive soil, are also discussed in chapter four. Moreover, all physical and mechanical properties of the blended soil are discussed and corresponding engineering classifications are given in chapter four. Chapter five presents the conclusion of the study and some recommendations.

CHAPTER 2 LITERATURE REVIEW

2.1 Expansive soils

2.1.1 Origin and formation of expansive clay soils

Expansive soil is a term generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture condition, such soils which have a potential to swell due to moisture absorption are the reason for settlement to occur as they shrink upon on moisture loss. Generally, the term expansive soil and swell potential are used in a universal sense to refer to soils that both shrink and swell. The primary problem that arises concerning expansive soils is that deformations are significantly greater than elastic deformations and they cannot be predicted by classical elastic or plastic theory i.e. the deformation is usually in an uneven pattern (differential movement) with such magnitude to cause severe damage to the structures and pavements resting on them (Nelson & Miller, 1992), (Tefera & Leikun, 1999).

The parent material that can be associated with expansive soils is classified into two groups. The first group comprises the basic igneous rocks, which are comparatively low in silica, generally about 45% to 52%. Rocks which are rich in a metallic base such as the pyroxenes, amphiboles, biotitic and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glass. The second group comprises of the sedimentary rocks that contain montmorillonite as constituent including shale and clay stones which breaks down physically to form expansive soil (Fikadu, 2015), (Chen, 1975). Limestone and marls, rich in magnesium can also weather to clay. These constituents of the shale and clay stones contain varying amount of volcanic ash and glass, which were subsequently weathered to montmorillonite. Some of the fine-grained sediments which accumulated to form these rocks also contain montmorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas as clouds of ash from volcanic eruptions can fall on continents and sea. The constituents of the parent materials during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods (Fikadu, 2015).

2.1.2 Characteristics of Expansive clay Soil

Expansive soils owe their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil. Swelling clays can control the behaviour of virtually any type of soil if the percentage of clay is more than about 5 % by weight. Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties (Rogers, Olshansky, & Rogers, 2016). Soil characteristics may be considered either as microscale (Mineralogical & chemical properties of the soil) or macroscale (engineering properties of the soil, which in turn are dictated by micro-scale factors) (Chen, 1975). At the microscale level, the swelling capacity of an entire soil mass depends on the amount and type of clay minerals in the soil, the arrangement and specific area of the clay particles, the chemistry of the soil water surrounding those particles. The macroscale soil properties reflect the microscale nature of the soil because they are more conveniently measured in engineering work than microscale factors, macroscale characteristics are primary indicators of swelling potential. Soil plasticity and density are commonly determined macroscale factors to provide insight regarding the expansive potential of soil. Above all soil consistency is the most widely used indicator of expansive potential as most expansive soils can exist in a plastic condition over a wide range of moisture content, which results from the capacity of expansive clay minerals to contain large amounts of water between particles (Nelson & Miller, 1992), (Chen, 1975), (Fikadu, 2015).

2.1.3 Distribution of expansive soil

Potentially expansive soils can be formed almost anywhere in the world; it is widely spread throughout the five continents (United States, South America, Africa, Asia and Australia). In underdeveloped countries, the effect of expansive soils on structures is not well recognized rather when the construction is increasing. Ethiopia is one of the countries to have expansive soils distributed in some major cities of the country and major trunk roads. In Ethiopia, expansive soils are observed in areas such as central Ethiopia, following the major trunk roads of Addis-Ambo, Addis-Welliso, Addis-Tarmaber, Addis-Gohastion and Addis-Modjo. Similarly regional states such as Mekelle, Gambella (Mada, 2016).

Figure 2-1: Distribution of Expansive soil in Ethiopia (Mada, 2016)

2.1.4 Distribution of expansive soils in Addis Ababa

Addis Ababa is one of the cities in the country it has been found that there are expansive soils located in different parts of the capital as outlined in the geological map of Addis Ababa, and the most southern, southwestern and southeastern parts of Addis Ababa are found to be covered with expansive soils as shown in Figure 2-2. As depicted in Figure 2-2 areas like Bole, Mekenisa, Lideta and NefasSilk are places covered with expansive soil (Simon, 2016).

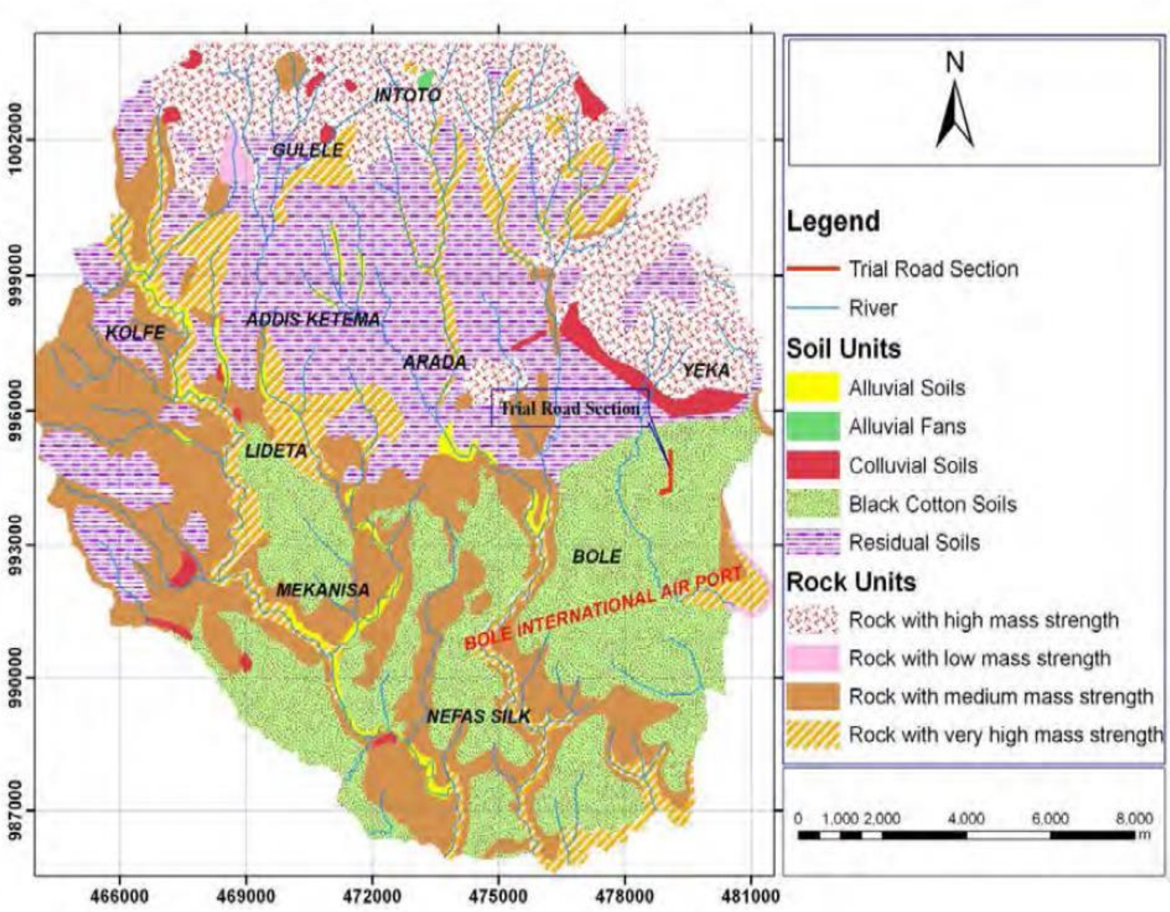


Figure 2-2: Engineering geological map of Addis Ababa (Simon, 2016)

2.1.5 Clay Mineralogy

Clay minerals refer to a group of hydrous alumino-silicates that predominate the clay-sized fraction of the soils. These minerals are similar in chemical and structural composition to the primary minerals that originate from the earth's crust. However, transformations in the geometric arrangement of atoms and ions within their structures occur due to weathering.

The most important structural groups of clay minerals which are considered as microscale factors for engineering purpose generally includes

- Kaolinite group- generally non-expansive.
- Mica-like group- includes illites and vermiculies, which can be expansive, but generally do not pose significant problems.
- Smectite group – includes montmorillonites, which are highly expansive and are the most troublesome clay minerals.

All of these groups have a layered crystal structure. The mineralogical distinction is based on the physical arrangement of the different layers and how the individual structural units are bonded together (Nelson & Miller, 1992).

2.1.5.1 *Kaolinite Group*

The kaolinite group has a structural unit made up of alumina sheets joined to silica sheets which consists of many such layers stacked one on top of the other. The bond that exists between the layers is tight which makes separation of layers difficult (Tefera & Leikun, 1999). The 1:1 layer mineral contains one tetrahedral and one octahedral sheet in their basic structural unit, exhibiting Al^{3+} octahedral and Si^{+4} tetrahedral coordination. The sheets are held together by van der Waals bonds between the basal oxygens of the tetrahedral sheet and the hydroxyls of the octahedral sheet. Layers are held together tightly by hydrogen bonding, which restricts expansion and limits the reactive area to the external (Baser, 2009). Due to this kaolinite is relatively stable and water is unable to penetrate between the layers as a result kaolinite shows little swelling on wetting.

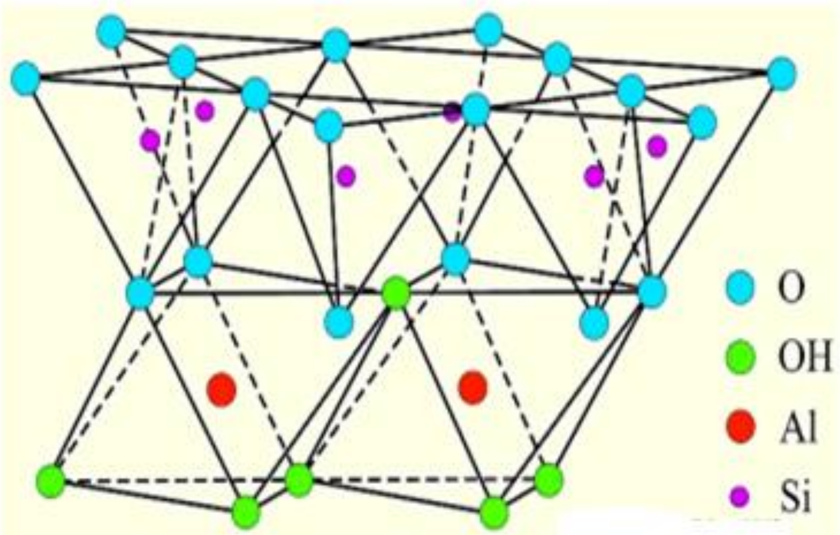


Figure 2-3: Diagrammatic representation of Kaolinite group (Baser, 2009)

2.1.5.2 *Mica-like group*

Illites

The illites have the basic structure in which the octahedral Alumina sheet is sandwiched between two tetrahedral silicate sheets. These illite units are bonded together by potassium ions which are non-exchangeable, that is why the illite units are reasonably stable and the mineral swells less when it is in contact with water but more severe than the kaolinite group (Tefera & Leikun, 1999). The structure is 2:1 layer in which the interlayer cation is Potassium, the size, charge, and coordination number of potassium is such that it fits closely in the hexagonal ring of oxygens of the adjacent silica tetrahedral sheets. This gives the structure a strong interlocking ionic bond which holds the individual layers together and prevents water molecules from entering and occupying the interlayer position as it does in the smectites (Murray, 2007) (Barton & Karathanasis, 2002). In the octahedral sheet, there is a partial substitution of aluminium by magnesium and iron, and in the tetrahedral sheet, there is a partial substitution of silicon by aluminium. In general, the illites can be expressed as potassium smectite (Baser, 2009).

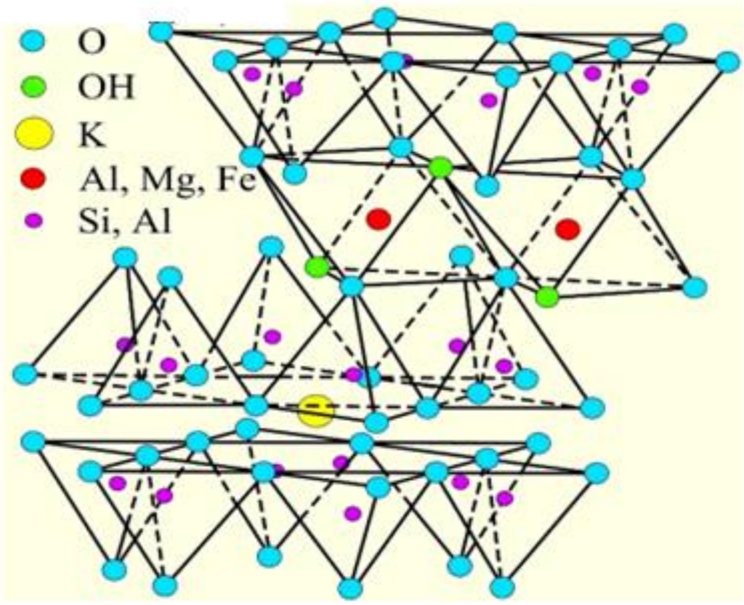


Figure 2-4: Diagrammatic Sketch of the illite (Baser, 2009)

2.1.5.3 *The Smectite group*

The major smectite minerals are sodium montmorillonite, calcium montmorillonite, magnesium montmorillonite, lithium montmorillonite and aluminium montmorillonite.

Like the illite group, these minerals are composed of two silica tetrahedral sheets with a central octahedral sheet and forming structure having 2:1 layer mineral. The water molecules and cations occupy the space between the 2:1 layers. In the smectites, there is a considerable substitution in the octahedral sheet and some in the tetrahedral sheet. In the tetrahedral sheet, there is a substitution of aluminium for silicon and in the octahedral sheet, magnesium and iron for aluminium (Chen, 1975), (Murray, 2007), (Barton & Karathanasis, 2002). The most common smectite mineral is calcium montmorillonite in which the layer charge deficiency is balanced by the interlayer cation calcium and water. But when the layer charge deficiency is balanced by sodium ions and water it is sodium montmorillonite. The major difference between calcium and sodium montmorillonite is that calcium montmorillonite has two water layers in the interlayer position and sodium montmorillonite have one water layer (Murray, 2007). The 2:1 layers in smectites are held together by van der Waals bonds and weak cation-to-oxygen linkages. The existence of exchangeable cations located between water molecules in the interlayer allows for expansion of the crystal lattice as the mineral hydrates. Upon saturation the basal spacing between the layers increases and reduces on drying which leads to the shrink-swell phenomena which are the main reason for crack formation and general instability of the soil surface. In general, smectite refers to a group of expandable clay minerals with montmorillonite the most common member of the group (Barton & Karathanasis, 2002), (Mukherjee, 2013), (Tefera & Leikun, 1999).

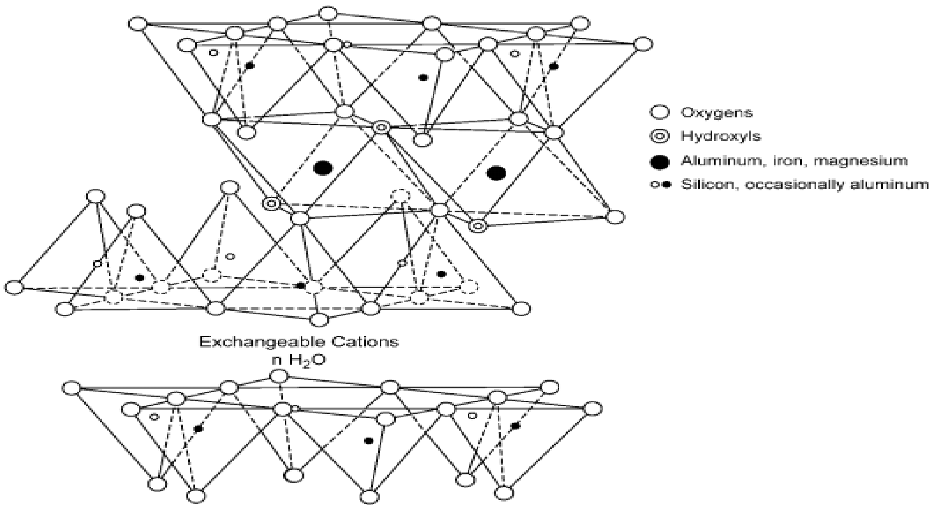


Figure 2-5: Diagrammatic sketch of the smectite group (Murray, 2007).

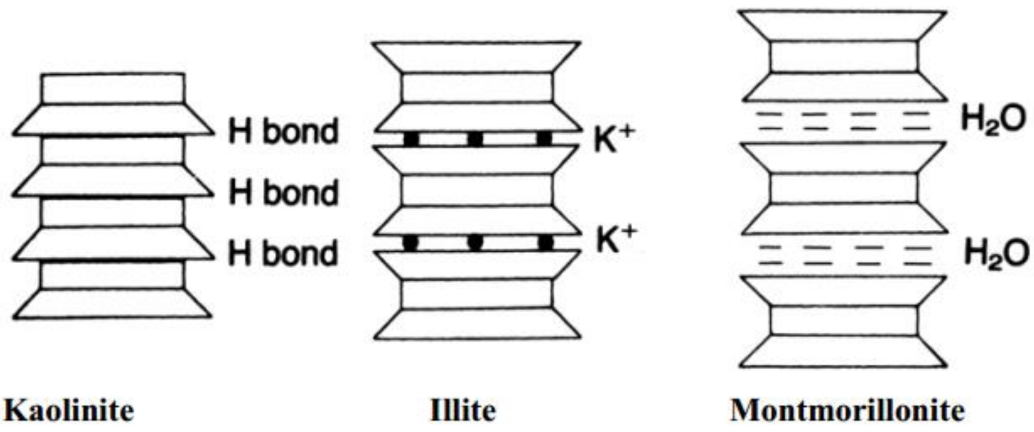


Figure 2-6: Schematic representation of clay minerals (Baser, 2009).

2.1.6 **Identification and classification of expansive clay soil**

2.1.6.1 ***General***

Early identification of expansive soils, during the reconnaissance and preliminary stages of a project, is essential to allow for appropriate sampling, testing and design in later stages. Thus, the investigation must comprise two important phases. The first is the recognition and identification of the soil as expansive soil and the second is sampling and measurement of material properties to be used as the basis for design predictions. In engineering practice, the common identification schemes are based on standard classification results, such as grain size analysis, Atterberg limits, free swell, and potential volume change which deals with physical properties of expansive soil. But there are also mineralogical methods which are used for identification and recognition of expansive clay soil which deals with mineralogical and chemical properties such as clay content, cation exchange capacity and specific surface area (Chen, 1975), (Nelson, Chao, Overton, & Nelson, 2015).

2.1.6.2 ***Identification of expansive clay soil***

2.1.6.2.1 Field Identification

Expansive soils can be identified in the field by visual inspection as it has unique properties that can be visible to the necked eye. The most physical properties that are visible to the necked eye are related to its shrink-swell characteristics and stickiness of the soil when it is in contact with water and loses water, so a good knowledge of local geology and site location relative to other problem areas is very useful in the identification of potential problems. Awareness of those particular formations that exhibit high swelling potential is necessary for the local practising geotechnical engineers.

Field observations made during the reconnaissance and preliminary investigation phases can provide valuable data and can be obtained easily, even by relatively inexperienced professionals with some on-the-job training. The field characteristics to look for will be somewhat localized, some of the important observations that can be obtained by the field engineer are as follows (Nelson & Miller, 1992), (Tesema, 2016):

- They usually have black and grey in colour.
- Wide and deep cracks during the dry season.
- They have high dry strength and low wet strength which indicates high plasticity.
- Stickiness and low trafficability when wet.
- Scraped or cut surfaces have a glazed or shiny appearance, like soap.
- Evidence of low permeability indicated by surface drainage and infiltration features.
- Cracking appears in nearby structures (mainly on walls, foundations and grade beams) of buildings and longitudinal cracks especially appear near road shoulders and around the centerline of highways. Transversal crack emerges in minor drainage structures like culverts.

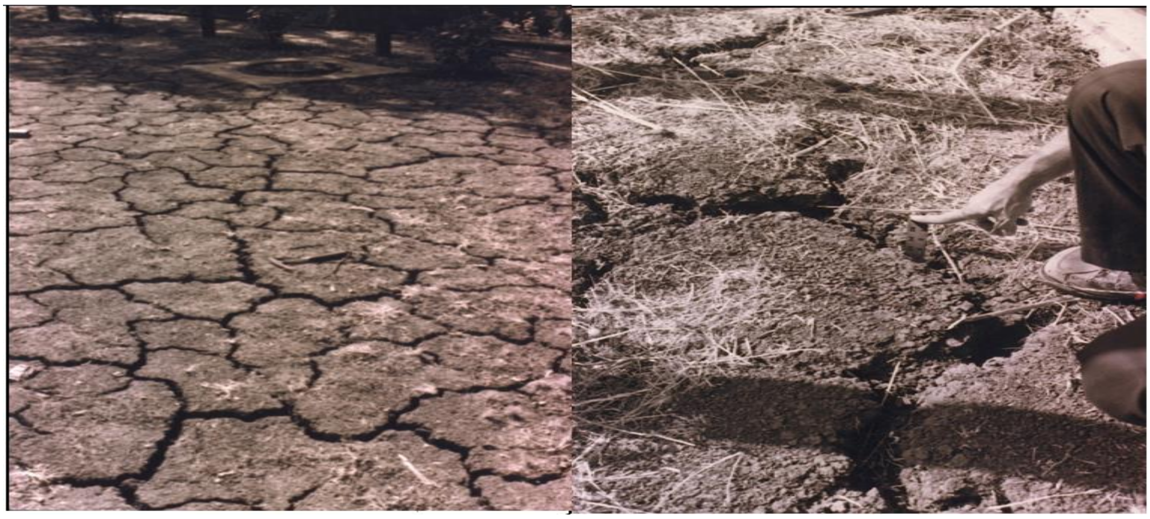


Figure 2-7: Cracked surface during the dry season (Rogers, Olshansky, & Rogers, 2016)

2.1.6.2.2 Mineralogical Methods

The mineralogical methods are useful in the identification of expansive clay soil but by itself is not sufficient in characterizing the clay soil for engineering classification (Chen, 1975). Many of the methods are used to identify the presence of clay minerals which causes expansion of the soil but do not consider the insitu physical properties of the soil such as moisture content and density. Thus these methods cannot identify whether the soil is expansive or not and unable to quantify the expansion potential of the soil. But the mineralogical methods gives the insight to investigate the potential for the expansiveness of the soil (Nelson, Chao, Overton, & Nelson, 2015).

Clay minerals can be identified using a variety of techniques, the more common are:

- X-Ray diffraction
- Differential Thermal Analysis (DTA)
- Dye Adsorption
- Chemical Analysis
- Electron Microscope Resolution

These mineralogical identification methods are applicable in a research laboratory but are impractical for practising engineers as it needs more experienced personnel for interpretation of test results. Besides, the mineralogical identification methods are time taking and uneconomical. The apparatus required for conducting tests are costly and not available easily in most soil testing laboratories as they impose high investment cost in equipment (Tefera & Leikun, 1999) (Chen, 1975).

2.1.6.3 *Engineering Classification methods*

2.1.6.3.1 Indirect methods

1. **Atterberg limits**

Classification tests for soil index properties such as grain size distribution, clay content and plasticity are the most widely used in practice for identifying and classifying expansive soils. Two useful indices may be computed from the Atterberg limits and the natural moisture content. These are the plasticity index (PI) and the liquidity index (LI). The PI is used extensively for classifying expansive soils and should always be determined during the preliminary investigation. The more expansive soils tend to exhibit higher plasticity (Chen, 1975), (Nelson & Miller, 1992), (Nelson, Chao, Overton, & Nelson, 2015). Based on its plasticity index the soil can be classified as shown in Table 2-1 below.

Table 2-1: Expansion potential of soil and plasticity index (Nelson, Chao, Overton, & Nelson, 2015)

Plasticity index (%)	Expansion Potential
0-15	Low
0-35	Medium
20-55	High
>35	Very high

2. **Free swell Test**

The free swell can be considered as one of the methods for measuring the volume of clay soil upon saturation. The free swell test consists of placing a known volume of oven-dry soil passing the No. 40(425µm) sieve into a graduated cylinder filled with water and measuring the swelled volume after the soil-water mixture has completely settled. The free swell of the soil is determined as the ratio of the volume change noted after 24hrs to the initial volume expressed as a percentage.

$$FSV = \frac{V_f - V_i}{V_i} x100 \tag{2-1}$$

Where

FSV- Free Swell Value, V_i – Initial Volume, V_f – Final Volume

Experiments indicate that a good grade of high swelling commercial bentonite has a free swell value of 1200 to 2000%. Soil with free swell values as low as 100% can cause considerable damage to lightly loaded buildings. Soils having free swell less than 50 % exhibits volume changes even under light loadings (Chen, 1975), (Nelson, Chao, Overton, & Nelson, 2015).

2.1.6.3.2 Direct methods

The most convenient method of characterizing expansive soil is a direct measurement of the swelling potential and swelling pressure of the soil. These can be done by using the conventional one–dimensional consolidometer (Chen, 1975). The swelling pressure or swelling potential can be determined for the sample that is undisturbed or remoulded to maximum dry density with its optimum moisture content obtained from standard compaction test. The soil sample is confined in a metallic ring which gives the lateral confining to the soil. The sample confined in the ring is enclosed between two porous plates which give the soil access to water during saturation. The sample is given access to water both from the top and from the bottom to allow vertical expansion under a surcharge load. Readings of vertical expansion are taken at specified time intervals until the sample ceases to swell (Tefera & Leikun, 1999), (Chen, 1975), (Nelson, Chao, Overton, & Nelson, 2015), (Nelson & Miller, 1992).

The dimension of the ring used in most standard one-dimensional consolidometer has a diameter to height ratio of 2.5. In some standards, the height of the ring cannot be less than 18mm and not more than 0.4 times the internal diameter of the ring.

The main soil characteristics obtained from the direct measurements are the swelling potential and swelling pressure.

The swelling pressure can be defined as the pressure which is required to prevent the soil from swelling or pressure required to compress swelled soil back to its initial volume. The swelling potential is the vertical expansion of the soil sample in consolidation ring expressed as a percentage of the initial height of the soil sample which has been soaked under a surcharge load of 7kPa (Nelson & Miller, 1992), (Tefera & Leikun, 1999), (Chen, 1975), (Nelson, Chao, Overton, & Nelson, 2015), (Al-Rawas & Goosen, 2006). After the sample has reached its maximum volume increase the sample is loaded in increment as in the case of conventional consolidation test to determine the swelling pressure.

2.1.6.3.3 Factors affecting the swelling potential and swelling pressure

The volume change that takes place in expansive soil can be either expansion or shrinkage. The expansion occurs when the soil gets in contact with water which initially was dry, but shrinkage takes place when the soil loses moisture which initially was wet (Tefera & Leikun, 1999). The most common factors which affect the swelling characteristics of expansive soil are the following:

1. **Amount and type of clay**

The swelling potential and swelling pressure of expansive soil are greatly affected by the amount and type of clay present in the soil. The swelling potential increases with the increasing percentage of clay in the soil. Based on the type of clay the montmorillonite clay has higher swelling potential than kaolinite and illite clay types. The same is true for swelling pressure (Tefera & Leikun, 1999).

2. **Placement Conditions**

The placement condition involves initial water content, initial density and confining pressure. The swelling potential of the expansive soil increases with a decrease in initial moisture content for a given dry density and the reverse occurs when the initial moisture content increases for given dry density. When the initial dry density increases the swelling potential increases for given initial moisture content. Also, percent expansion decreases with increase in confining pressure (Tefera & Leikun, 1999), (Chen, 1975). The same thing is true for swelling pressure.

3. **Time allowed to swell**

For a given initial moisture content and initial dry density, the percent expansion increases with time for which the sample is allowed to swell (Tefera & Leikun, 1999).

2.1.7 Classification methods to expansive soil

2.1.7.1 *Skempton Method*

The plasticity characteristics of soils are directly related to the amount of colloidal sized particles in the soil. Colloidal sized particles are those in which electrostatic and adsorptive forces control their properties rather than gravitational forces. Colloidal particles are generally smaller than 0.001mm in size. Most clay particles are considered as colloids because of their irregular shapes and large surface area. For a given clay type the amount of swell will increase with the increasing amount of colloid sized particle (Chen, 1975), (Nelson, Chao, Overton, & Nelson, 2015), (Tefera & Leikun, 1999). Atterberg limits and clay content can be combined into a single parameter called Activity, to determine the degree of activity or expansiveness of expansive soil (Nelson & Miller, 1992), (Tefera & Leikun, 1999), (Nelson, Chao, Overton, & Nelson, 2015).

$$\text{Activity } (A_c) = \frac{\text{Plasticity index}}{\text{\% by weight finer than 2 micrometer}} \qquad \textbf{(2-2)}$$

Based on activity Skempton suggested three classes of expansive soil as follows:

Table 2-2: Classification of expansive soil based on activity (Nelson & Miller, 1992).

Activity (Ac)	Degree of Activity
<0.75	Inactive
0.75<Ac<1.25	Normal
Ac>1.25	Active

This classification method can also be presented in the form of a chart called activity chart as shown in Figure 2-8 (Tefera & Leikun, 1999).

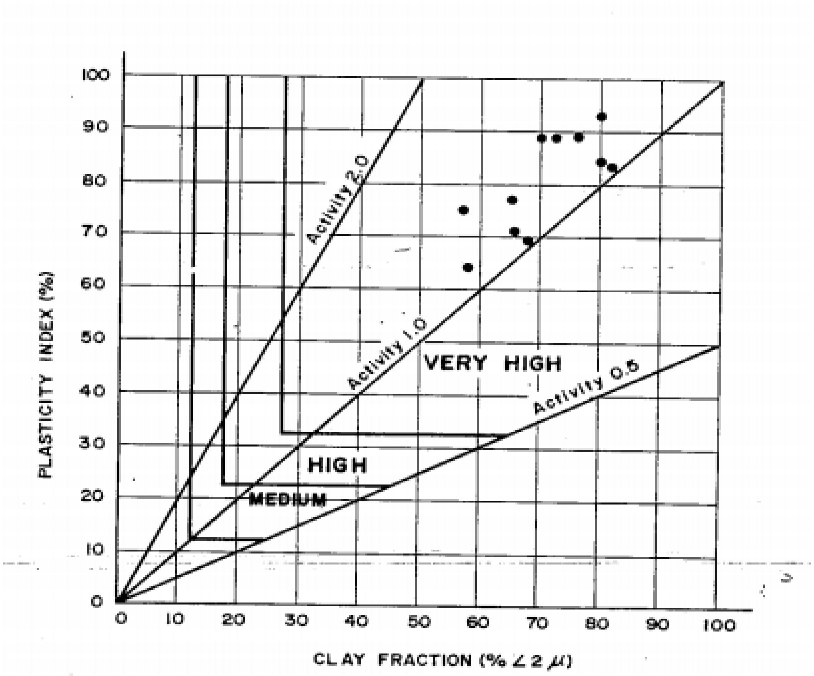


Figure 2-8: Activity chart (Tefera & Leikun, 1999)

Typical values of activities for various clay minerals are as follows: (Nelson & Miller, 1992).

Table 2-3: Activity of clay minerals (Nelson & Miller, 1992)

Mineral	Activity
Kaolinite	0.33 to 0.46
Illite	0.9
Montmorillonite (Ca)	1.5
Montmorillonite (Na)	7.2

2.1.7.2 *U.S.B.R. Classification Method*

This method was developed by Holtz and Gibbs to establish the degree of expansion of soil based on simultaneous consideration of shrinkage limit (SL), plasticity index (PI), percent smaller than 0.001mm, free swell value(FSV) and percent swell under the pressure of 7kPa. Based on these parameters the clay soil was assigned to degree of expansion ranging from very high to low degree of expansion as illustrated in the following table.

Table 2-4: Relationship between the degree of swell and clay properties (Tefera & Leikun, 1999).

Degree of Expansion	Swell in Oedometer under the pressure of 7kPa	Shrinkage limit (SL),%	Plasticity Index (PI)	Percent Smaller than 1μm	Free swell (%)
Very High	>30	<10	>32	>27	>100
High	20-30	6-12	23-45	18-37	>100
Medium	10-20	8-18	12-34	12-27	50-100
Low	<10	>13	<20	<17	<50

2.1.7.3 *Seed's classification Method (Activity Method)*

Seed et al .classify expansive soil based on swelling potential defined as percent vertical swell under a pressure of 7kPa of laterally confined sample compacted to maximum dry density and optimum moisture content following AASHTO compaction method. This classification scheme utilizes the activity of clay and colloid content of the soil in determining the swelling potential. Based on this method of classification method the soil is categorized into four degrees of expansion as indicated in the following table.

Table 2-5: Relationship between the degree of swell and swelling potential (Tefera & Leikun, 1999).

Degree of swell	Swelling Potential,%
Low	0-1.5
Medium	1.5-5
High	5-25
Very High	>25

2.1.7.4 *The unified soil classification method*

This classification method is based on laboratory determination of particle-size distribution, liquid limit, and plastic limit and based on the plasticity chart (Tefera & Leikun, 1999). According to this method, fine-grained soils are classified as organic or inorganic clays of high plasticity if the plot of the plasticity index vs liquid limit falls above A-line and the liquid limit is greater than 50%. Organic or inorganic clays of low plasticity if the plot of the plasticity index vs liquid limit falls above A-line and the liquid limit is less than 50% but greater than 30%. If the plot of the plasticity index vs liquid limit falls below the A-line the soil is classified as organic or inorganic silt of high plasticity and the liquid limit is greater 50%. For the plot of plasticity index vs Liquid limit located below the A-line with liquid limit less than 50% but greater than 30% the soil is classified as organic or inorganic silt of low plasticity.

2.1.8 Overview of damage due to expansive soils

2.1.8.1 *General*

The problem of expansive soil is worldwide, which induces damage to infrastructure constructed on such type of soils. As reported on research concerning problems of expansive soils the cost incurred due to damages caused by expansive soils are more than that caused by the combination of Floods, Hurricanes, Earthquakes and Tornados on an annual average basis in the USA (Chen, 1975). Extensive cracks and heave in floor slabs constructed on expansive soil areas are due to lack of proper design of concrete by not considering the effect of soil movement, besides, the lack of expansion joints results in a crack in floor slab (Chen, 1975), (Sisay, 2004). Lightly loaded structures constructed in expansive soil areas are not vulnerable to cracks due to settlement as the soil is generally stiff. At the same time, there are horizontal and vertical cracks in basement walls not due to foundation heave rather due to lateral earth pressure from the soil retained and seepage pressure (Chen, 1975).

In general, expansive soils are always blamed for the damage caused to buildings. But faulty design, poor construction, failure to maintain damaged parts and uncontrolled construction activities near to existing structure on expansive soil or not completing the construction in one season due to financial case aggravates the problem associated with such type of soil (Sisay, 2004).

In the case of buildings with basement wall, cracks in the basement wall are due to careless construction crews, Backhoe or other earth moving equipment bumping soil against the wall before restraining the wall top and bottom level. Sometimes expansive soils are blamed for arching of basement walls with no proper reinforcement and restraint, failure to control such problems result in wall arching. Without taking care of such phenomenon it is mistakenly interpreted as horizontal swelling pressure against the wall due to the backfill soil but as the backfill is loosely compacted it is uncommon for the wall to experience horizontal swelling pressure (Chen, 1975).

2.1.8.2 ***Recognition of damages due to expansive soil***

The damages due to alternate heave and shrinkage or differential heave of the foundation soil are manifested through the crack of floors and walls, stacked windows and doors, bulged floors and tilted walls and structures. The magnitude of the damages can be extended even to the extent of the failure of one part and/or the whole structure by decreasing the structural safety of the building. Maintenance and repair cost can also exceed the original cost of the foundation and creates a financial burden to the homeowner. Generally, the damage will create an economic loss for building owners and the country at large (Sisay, 2004), (Mir, 2015).

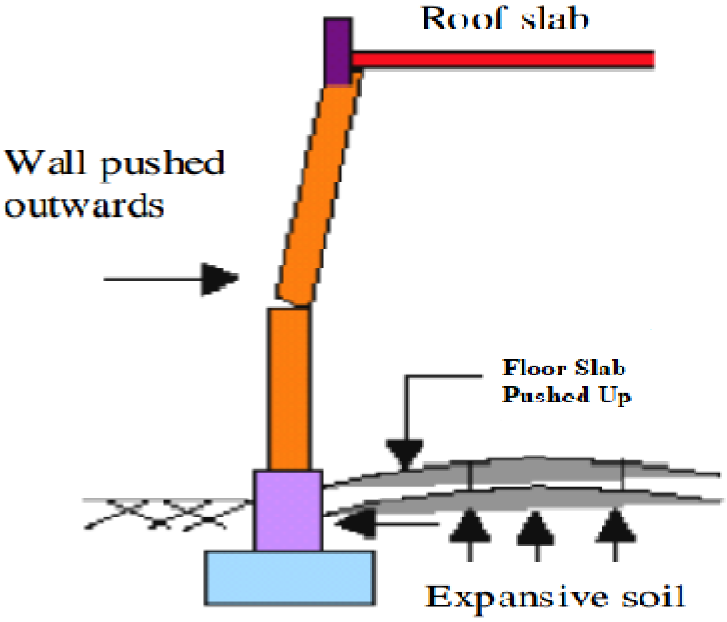


Figure 2-9: Effect of swelling on Structure (Mir, 2015)

For buildings supported on shallow piers, the building foundation was effective due to skin friction between the pier and the surrounding soil when the soil was dry. In the wet season or during the rainy season water starts to enter the soil around the pier, as a result, the wetted soil starts to swell lifting the pier and the building. In the dry season, the groundwater table falls and the soil dries and contracts. As tension cracks grow around the pier, the skin friction is reduced and the effective stress of the soil increases (due to drying). When the building load exceeds the remaining skin friction or the effective stress of the soil increases to an all-time high, adhesion is broken by this straining and the pier sinks which results in an overall foundation settlement.

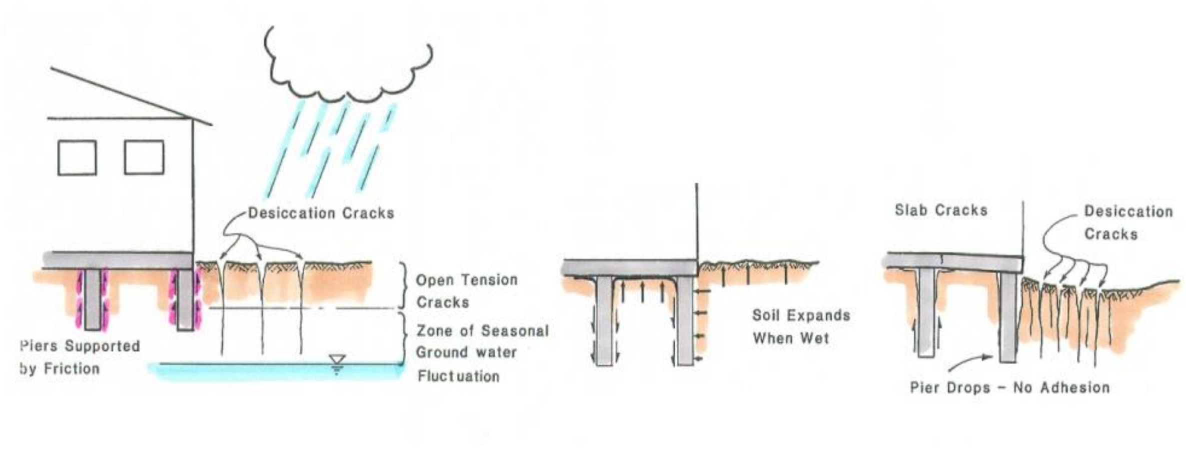


Figure 2-10: Damage to house supported on shallow piers (Rogers, Olshansky, & Rogers, 2016)

2.1.8.3 *Damage due to Expansive soil in Addis Ababa*

Based on the research conducted on the damage analysis of ninety-six buildings sixty-nine of which were damaged (Sisay, 2004). The observations are carried out concerning many parameters such as Foundation type, Foundation depth, subsurface drainage condition, Structural system and Building type (Sisay, 2004). Buildings for which the walls constructed from the hollow concrete block were more susceptible to crack compared to the one constructed of brick, 67 % of the wall of the building constructed from hollow concrete block showed crack and 40% of the building's wall constructed from the brick wall also showed crack. Most of the damages caused were due to moisture variation under the building foundation as a result of poor surface drainage system hence disturbs moisture equilibria. From the sixty- nine damaged buildings 84% were damaged due to poor subsurface drainage system (Sisay, 2004). The other factors that cause damage are vegetation around the building, leaking of pipes, splashing of downpipes to the foundation soil and new adjacent construction. All these factors were found to aggravate the damage that expansive soil poses on buildings.

2.2 Properties of natural pozzolan and its origin

Pozzolanas are a broad class of siliceous or siliceous and aluminous materials which, in themselves, possess little or no cementitious value but which will, in finely divided form and with the presence of water, react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties. Consequently, a natural pozzolan is also defined as either a raw or calcined natural material that has pozzolanic properties (like volcanic ash, pumicites, opaline chert and shales, tuff and some diatomaceous earth).

2.2.1 Sources of natural pozzolan

2.2.1.1 *Volcanic ash*

Except for diatomaceous earth, natural pozzolanic materials are derived from volcanic rocks. Volcanic eruptions throw into the atmosphere large quantities of molten lava, which is composed mainly of aluminosilicates. Quick cooling of lava results in the formation of amorphous phases (glass) with disordered microstructure or poorly crystalline minerals. Also, the escaping gases and water vapour may impart to the volcanic material a porous texture with a high surface area. A combination of glassy or poorly crystalline structure and a high surface area is the cause for the reactivity of the alumino-silicate phases present in the volcanic ash with calcium hydroxide at normal temperature (Desta, 2003), (Malhotra & Mehta, 1996).

2.2.1.2 *Clay minerals*

Clay minerals that are not pozzolanic are formed by progressive alteration of volcanic glass. The crystalline structure of alumino-silicate minerals present in clay or shale can be destroyed by heating it with a temperature of 700-800ºC, resulting in calcined clays and shales that are pozzolanic.

Another pozzolan obtained from clay is the metakaolin, which is made from high purity kaolin clay by low-temperature calcination. The product is pulverized to very fine particle size (average 1-2µm) to make it more pozzolanic. The non-reactive impurities of kaolin clay could be removed by washing and the resulting pure kaolin is calcined at a specific temperature to produce highly reactive metakaolin. However, nowadays the production of reactive pozzolans from calcination of clays is not favoured because of the needs of high energy (Desta, 2003).

2.2.1.3 *Diatomaceous earth*

This type of source of natural pozzolana is characterized by materials of organic origin. It is hydrated amorphous silica and composed of skeletal shells from the cell walls of many varieties of microscopic aquatic algae. Pure diatomite is pozzolanic, but the material is usually contaminated with clays and therefore must be calcined to increase the pozzolanic activity.

Diatomite is highly reactive to lime, but their skeletal microstructure accounts for a high water requirement, which could be harmful to the strength and durability of concrete (Desta, 2003), (Malhotra & Mehta, 1996).

2.2.2 Chemical composition of natural pozzolan

Natural pozzolans are quarried from natural deposits. Therefore, their properties differ from one deposit to another depending on the variable proportions of the chemical and mineralogical composition and physical characteristics. Most natural pozzolans, however, contain a considerable amount of constituents such as alumina, iron oxide and alkalies, other than silica, which also reacts with calcium, sodium and potassium hydroxides to form more complex compounds. But the quantity and molecular structure of silica is very important to determine the pozzolanic activity (Malhotra & Mehta, 1996).

Even though there is no distinct boundary between amorphous and non-amorphous material, amorphous silica reacts with calcium hydroxide and other alkalis more rapidly as compared to other silica in a crystalline form. The reactivity of the silica depends on the particle size. As in the case of solid-state chemical reaction, larger particles show a slower rate of reaction as compared to particles with smaller particle size (particles with the higher specific surface area). From this, the chemical composition of the pozzolan alone does not determine its reactivity with calcium hydroxide and other alkalis (Desta, 2003).

Natural pozzolan with sources of volcanic glass and zeolitic tuffs produce good pozzolans like calcium silicate hydrate, hydrated calcium aluminates and calcium aluminosilicates upon mixing with lime (Desta, 2003). Chemical and mineralogical composition of some typical natural pozzolans is given in Table 2.6.

Table 2-6: Typical chemical and mineralogical analysis of natural pozzolan (Desta, 2003)

Pozzolan	Percentage Mass						Ignition loss, Percentage
	SiO2	Al2O3	Fe2O3	CaO	MgO	Alkalies	
Santorin earth	65.1	14.5	5.5	3.0	1.1	6.5	3.5
Rheinisch trass	53.0	16.0	6.0	7.0	3.0	6.0	-
Photolite	55.7	20.2	2.0	4.2	1.1	10.8	3.6
Roman tuff	44.5	18.9	10.1	10.3	4.4	6.7	4.4
Neapolitan glass	54.5	18.3	4.0	7.4	1.0	11.0	3.1
Opaline shale	65.4	10.1	4.2	4.6	2.7	1.4	6.3
Diatomite	86.0	2.3	1.8	-	0.6	0.4	5.2
Phyolite pumicite	65.7	15.9	2.5	3.4	1.3	6.9	3.4
Jalisco pumice	68.7	14.8	2.3	-	0.5	9.3	5.6

2.2.3 Mineralogical composition of natural pozzolan

Natural pozzolan with unaltered volcanic ashes obtains their activity from the aluminosilicate glass. Mineralogical analysis of most natural pozzolan reveals more than 80% aluminosilicate glass from which natural pozzolans gain pozzolanic activity. Santorin earth, for instance, is composed of fragments of quartz, feldspar and mica. Volcanic tuff and trass also have significant amounts of quartz, feldspar and clay in a glassy matrix as obtained from the mineralogical analysis, but these minerals have undergone alterations to zeolite minerals (Desta, 2003). Also, Rheinisch trass contains a glassy matrix of about 50% which altered to zeolite minerals while diatomaceous earth predominantly contains opaline silica (hydrous non-crystalline silica).

2.2.4 Particle characteristics of natural pozzolans

The pozzolanic activity and reactivity of natural pozzolans depend mainly on the size, shape and texture of the particle than on the chemical composition. For instance, the water demand and workability are controlled by particle size distribution, packing effect, and smoothness of the surface texture of the pozzolan when used as cement replacement in concrete. The pozzolanic and cementitious activity can be enhanced by controlling the particle size distribution of the pozzolans (Desta, 2003), (Malhotra & Mehta, 1996). ASTM C618 specifies the size of natural pozzolan to be used us pozzolanic admixture. As specified in the standard, the largest particle size to be used as the pozzolanic additive is 45μm, besides, the percentage retained on 45μm is limited not exceed 34%.

In general, the pozzolanic activity of natural pozzolan is directly proportional to the particle size distribution. Where there is a particle of lesser sizes are abundantly available in a mixture there is more pozzolanic reactivity (Malhotra & Mehta, 1996).

2.2.5 Classification of natural pozzolans

Naturally occurring pozzolan deposits are mainly of volcanic origin. They occur as tuffs, volcanic ash and pumice and are found abundantly in areas with a geological history of volcanic activity. A good natural pozzolan contains low quantities of unreactive minerals, alkali feldspar and quartz and high quantities of reactive components such as zeolite minerals and volcanic glass.

The materials that have been investigated or even commercially applied as natural pozzolans the recent years are classified as follows:

i. Volcanic Ash and Pumice
ii. Scoria
iii. Tuffs
iv. Diatomite
v. Hydrothermal siliceous sinters
vi. Bentonite
vii. Perlite

Table 2-7: Standard Specification of natural pozzolans according to ASTM C 618

Properties	Limits
$SiO_2 + Al_2O_3 + Fe_2O_3$, %	≥70
SO_3, %	≤4.0
Moisture content, %	≤3.0
Loss on Ignition,%	≤10
Amount retained when wet-sieved on 45 µm(No.325) sieve,%	≤34
Available alkalis,%	≤1.5

2.2.6 Natural pozzolans in Ethiopia

As Ethiopia was one of the sources of volcanic action, raw natural volcanic ashes are found all over the country, especially in the rift valley area. Apart from the volcanic ashes, there are also deposits of diatomaceous earth in the vicinity of most of the lakes that are found in the rift valley area. Even though there are several types of deposits available in the area, large quantities of pumice, scoria and diatomite are also found predominantly. Pumice which is a light, porous volcanic rock that is found as a result of explosive volcanic eruptions and it resembles like a sponge. According to information obtained from the Institute of Ethiopian Geological survey, several deposits of pumice are located in the Rift Valley area, in areas like Metehara, Koka, and Kimbibit with the percentage of major oxide exceeding 68.6%. It is mostly used as an aggregate in the production of non-load bearing concrete blocks (Desta, 2003).

Another natural pozzolan is scoria, which is vesicular glassy lava of basaltic to andesitic composition ejected from a vent during an explosive volcanic eruption. The scoria, as in the case of pumice, is also located in many parts of the Main Ethiopian Rift valley areas. It covers areas like Tulu Dimtu, Wonji, and Metehara. Scoria in these areas has major oxide composition exceeding 69.2% by mass. It is commonly used as a foundation for small rural roads, as a lightweight concrete aggregate and in the production of the non-load bearing concrete hollow block. It is also, used as backfill material in the construction of the foundation for dwelling houses, especially on expansive soils (Desta, 2003).

2.3 Review on Improvements of expansive soil

Soils found in the construction site is often problematic, such soils may not meet construction requirements. These types of soil need improvements in their engineering properties to meet the desired construction requirement. In its broadest sense, soil improvement is defined as the process of changing one or more of the engineering properties of the soil to get the desired performance of the soil to be used as a construction material. One method of improving the engineering properties of soil is soil stabilization. Soil stabilization can be done by mechanically and chemically altering the properties of the soil.

2.3.1 Soil stabilization

There are two types of soil stabilization.

1. Mechanical Stabilization
2. Chemical Stabilization

2.3.2 Mechanical Stabilization

Mechanical stabilization can be done through compaction, Excavation and replacement with non-problematic soil and Mixing of different soils.

Compaction is a process in which solid matters of soil are packed into the denser state by reducing air voids. The purpose of compaction is to produce soil having physical properties appropriate for construction. Compacted soils are less vulnerable to settlements, percolation of water. Also, compaction increases the shear strength of the soil.

Excavation and replacement of problematic soil with non-problematic soil is another alternative way of improving the mechanical properties of soil. This type of mechanical stabilization method is effective and economical if problematic soil thickness is small and located near to the ground surface. In such cases, it is better to remove the soil and replace it with well-graded soil that can be compacted to a dense state.

Mixing of different soils is usually undertaken when the natural soil at the construction site is of uniform size i.e. either too coarse or too fine which is difficult to be compacted to a dense structure to serve as sub-base in highway construction or floor foundation for building construction.

2.3.3 Chemical Stabilization

Chemical stabilization is suitable for fine-grained soils as the aim of this stabilization method is to alter the chemical properties of the soil to achieve desirable engineering properties. Fine-grained granular materials are the easiest to stabilize due to their large surface area to their particle diameter. A clay soil compared to others has a large surface area due to flat and elongated particle shapes that is why it is easy to apply chemical stabilization to clay soil. On the other hand, silty materials are sensitive to a small change in moisture and may pose difficulty in effective stabilization (Makusa, 2012).

Different researchers have studied several stabilization chemicals and some naturally available inert material and discussed the results of their study, the engineering properties of expansive soil have been improved by blending with crushed and natural sand (Simon, 2016), So many other naturally available materials and industry by-products were studied in the past such as Bagasse Ash (Worku, 2015), Sugarcane Molasses (Tesema, 2016), Marble Waste Powder (Mada, 2016), Bagasse and hydrated lime mix (Dang, Hasan, Fatahi, Jones, & Khabbaz, 2016), Potassium Chloride (Lemi, 2015) has improved the Engineering properties of expansive soils.

2.3.3.1 *Lime stabilization*

Applying lime to expansive soil improve the strength, stiffness and durability of the soil, also adding lime produces lower maximum dry density and higher optimum moisture content, lowers the plasticity index and greatly reduces shrink-swell characteristics of expansive soil (Tefera & Leikun, 1999). All the properties of soil lime mixtures vary and depend upon the character of the clay soil, the type and length of curing, and the methods and quality of construction (Bel, 1996). Upon addition of lime to clay soil, it has an immediate effect on the properties of the soil due to cation exchange occurring between the metallic ions on the surfaces of clay particles and calcium ions of the lime (Bel, 1996).

The cation exchange alters the density of electrical charges around clay particles which is a means for clay particles attracted to each other to form flocs by the process of flocculation, the process primarily responsible for the improvement of engineering properties of clay soil stabilized with lime (Bel, 1996), (Worku, 2015). In addition to cation exchange reaction occurs between the silica and alumina of the clay minerals. This reaction contributes to flocculation by bonding adjacent soil particles together which leads to an increase in strength of soil on curing (Bel, 1996). The spacing between particles are larger in a flocculated structure which results in the reduction of the dry density of expansive soil and increases in optimum moisture content, in other words, reduces the swelling potential of expansive soil (Nelson, Chao, Overton, & Nelson, 2015).

It was observed in (Bel, 1996), the effectiveness of lime stabilization depends on the type of clay mineral present in the expansive soil. Based on this study the montmorillonite clay minerals respond quickly in terms of consistency limits. The liquid limit decrease for such type of clay minerals, but the plastic limit increases up to the lime fixation point i.e. 4% by dry weight of lime, after passing this point the plastic limit decreases. Generally, the plasticity of montmorillonites is reduced and are more workable up on stabilization with lime. But when kaolinites are treated with lime the liquid limit increases but the plastic limit decreases, which results in increases in plasticity (Bel, 1996). The same is true for quartz-like material like that of kaolinite when treated with lime.

From a strength point of view the lime stabilized expansive soil showed an increase in unconfined compressive strength, the increase is more noticeable on kaolinite and quartz clay minerals. For the montmorillonite and kaolinite clay minerals, the optimum lime is at lime fixation point after this optimum content the strength starts to decrease. But in the case of quartz clay minerals, the strength increase is marginal after this optimum lime content (Bel, 1996).

The quantity of lime required for stabilization varies from 2 to 10% by dry weight of the soil. However, if the lime is used only to modify some of the physic-chemical characteristics of the soil, the quantity of lime is about 1 to 3%.

The following amount may be used as a rough guide (Lemi, 2015).

i. 2 to 5% for clay gravel having less than 50% silt – clay fraction.

ii. 5 to 10% for soil with more than 50% silt – clay fraction.

iii. For soils having particle size intermediate between one and two above, the quantity of lime required is between 3 to 7%.

iv. About 10% for heavy clays used as bases and sub-bases lime stabilization is not effective for sandy soils. However, these soils can be stabilized in combination with clays, fly ash or other pozzolanic materials which serve as hydraulically reactive ingredients

2.3.3.2 *Cement stabilization*

Under special situations such as soil bases of concrete pavements for highways, airfields and to protect dams against erosion by waves and as cores earth dams cement can be used as a stabilizing agent, but the soil-cement mixture is affected by the soil type, cement content, ageing, curing condition, compaction condition etc. All inorganic soil that can be pulverized effectively can be stabilized with cement, well-graded soil is most suitable for cement stabilization and the strength of soil-cement mix increases with curing (Tefera & Leikun, 1999).

2.3.4 Previous studies on the use of pozzolans as a stabilizing agent

Several types of natural pozzolans were in use to improve the engineering properties of expansive soil. These pozzolans, most of the time were considered as an economical substitute to other known stabilizing admixtures such as cement and lime. They are used in combination with these two main stabilizing admixtures. Above all, every engineering activity depends on the availability of suitable local materials. Especially, in the case of soil stabilization, the most common issue is the presence of pozzolan in the nearby areas to the expansive soil to be stabilized and/or that can be transported to the site with minimum cost.

2.3.4.1 *Pumice*

Pumice treated expansive soil showed improvements on their plasticity. The addition of pumice to the expansive soil reduced the liquid limit and plastic limit of the soil which in turn reduces the plasticity index of the soil and makes the soil more workable (Birhane, 2018). The compaction characteristics of the expansive soil are also influenced by the addition of pumice to the expansive soil at different percentages by weight. The maximum dry density and the optimum moisture content of the soil decreased as the percentage of pumice increases which is the main reason for swelling potential and swelling pressure to decrease for the blended soil (Birhane, 2018). The decrease in dry density is because pumice is a lightweight material and the strength of the blended soil decreased as the percentage of pumice increased. Due to this, the pumice has no significant effect on strength characteristics of expansive soil. Swelling characteristics of the expansive soil is also improved when the expansive soil was blended with pumice i.e. the swelling pressure and swelling potential of expansive soil decreased upon blending with pumice. Here the particle size of pumice used was limited to sieve size less than 4.75mm.

(Ömür Çimen, 2015), investigated the effect of adding pumice crushed to a size less than 425µm to the expansive soil. On this investigation, it was found that the maximum dry density increased 1.19 g/cc to 1.46 g/cc and the optimum moisture content decreased from 38% to 27% as the percentage of pumice increases from 0 to 50% by dry weight of the expansive soil. The plasticity characteristics of the expansive soil were also decreased as the percentage of the pumice increased. According to the investigation by (Ömür Çimen, 2015), the unconfined compressive strength of the expansive soil increased from 202.1kPa to 435.6kPa as the percentage of the pumice rises to 30% by dry weight of the expansive soil. After this percentage of pumice, the unconfined compressive strength of the soil-pumice mixture decreased. The effect of pumice on swelling pressure was also investigated, which showed a decrease from 304.11kPa to 196.2kPa as the percentage of pumice increased from 0 to 50%. Surprisingly, the pumice that brought this much improvement to the expansive soil has a specific gravity of 1.22, maximum dry density and optimum moisture content of 1.07g/cc and 16% respectively.

2.3.4.2 *Stone dust*

Stone dust which is obtained from the quarry site where Stones quarried for various building purposes was used as an additive to treat expansive clay (Reddy, Tahasildar, & Rao, 2015). The compaction characteristics of the expansive soil and the swelling characteristics were also influenced on blending the expansive soil with Stone dust. The dry density of the expansive soil increased and the optimum moisture content gets reduced when the percentage of the stone dust is increasing from 10% to 50% at 5% interval. The swelling pressure and the swelling potential of the expansive soil also reduced upon adding the Stone dust to the expansive soil at various percentages (Reddy, Tahasildar, & Rao, 2015). The reduction in swelling was because the stone dust acts as a fill material within the pore space of the soil mass as it reduces the affinity of the soil-dust mixture to water and also creates contrast in the density of particles between stone dust and parent soil.

2.3.4.3 *Use of other natural pozzolanas for stabilization*

(Al-Swaidani, Hammoud, & Meziab, 2016)**,** investigated the effect of adding natural pozzolan has on engineering properties of lime stabilized clay soils. In their study, they found that the addition of natural pozzolana to the clay-lime mixture increased the maximum dry density and reduced the optimum moisture content of the soil. Besides, the Atterberg limit characteristics were also get reduced with the increasing percentage of natural pozzolan on blending. Based on this study, the maximum dry density increased from 1.41g/cc to 1.5g/cc and the optimum moisture content decreased from 31% to 26.09% as the percentage of the natural pozzolan increased from 0 to 20% for soil stabilized with 4% lime. When the natural pozzolan was added to clay soil stabilized with 8% lime, the maximum dry density increased 1.35g/cc to 1.5g/cc and the optimum moisture content decreased from 33% to 26.21% with the increasing percentage of natural pozzolan from 0 to 20%. The decrease in optimum moisture content was attributed to the lower affinity of natural pozzolan to water.

(Harichane, Ghrici, Kenai, & Khaled, 2011), studied the effect of natural pozzolana, lime or a combination of both on the physical and mechanical characteristics of cohesive soils. Natural pozzolana, lime and natural pozzolana-lime were added to the cohesive soils at ranges of 0-20% and 0-8% respectively. According to this study, plasticity characteristics of the cohesive clay soil decreased when stabilized with natural pozzolana and lime alone. The addition of natural pozzolana with percentage ranges of 0-20% to the cohesive clay soils enhanced their workability by reducing the plasticity index from 52 to 46%. The combination of natural pozzolana and lime has a marginal effect on reducing the plasticity index compared to the use of lime alone. The compaction characteristics of the cohesive clay soil are also influenced by stabilizing with natural pozzolana, lime and their combination. The addition of the natural pozzolana to the soil increased the maximum dry density but decreased the optimum moisture content and the same happens when the natural pozzolana was combined with lime to stabilize the clay soil. Also, the unconfined compressive strength of the soil increased with the addition of natural pozzolana alone and/or combined with lime.

(K.M.A. Hossain*, 2007)**,** evaluated the influence of volcanic ash on the engineering properties of expansive soil and expansive soil stabilized with lime and cement. Based on this investigation, the volcanic ash alone increased the optimum moisture content and decreased the maximum dry density of the expansive soil. Also, the unconfined compressive strength of the expansive soil increased as the percentage of volcanic ash is increasing from 0 to 20%. The addition of volcanic ash and lime to the soil rises the optimum moisture content and lowers the maximum dry density of the soil. The increase in optimum moisture content is due to the pozzolanic reaction of volcanic ash and lime with the constituents of the soil. From a strength point of view, the unconfined compressive strength of lime stabilized expansive soil (percentage of lime 0 to 4%) increased with the percentage of volcanic ash increasing from 0 to 20%.

Many other pozzolanas were used by different researchers to investigate the effect they have on improving the engineering properties of expansive soil. Fly Ash, Rice Husk Ash, Sugarcane Bagasse Ash were used and show better performance on improving the engineering properties of expansive soil.

CHAPTER 3 EXPERIMENTAL STUDY ON PROPERTIES OF NATURAL SOIL AND POZZOLANIC SAND

3.1 General

The durability and good performance of pavement subgrades and foundation of existing buildings are dependent on the quality of soil underneath in terms of its engineering properties. In most cases, soils underlying the foundation of buildings located in flat low lying areas are expansive soils. The soil mass may vary from non-expansive to highly expansive which imposes considerable problems to buildings foundation if some mechanism of improvement was not applied to the soil.

So for improving the engineering properties of the soil it is required to properly identify all the physical and mechanical properties of natural or untreated soil. Therefore the investigation conducted is the major factor for selecting appropriate improvement methods on an economic and technical basis.

For the present study, several laboratory tests were conducted to characterize natural or untreated soil. To select the representative samples for laboratory testing preliminary field investigation was carried out visually to identify the soil based on its colour as most of the expansive soils are black to grey in colour and texture of the soil. Laboratory tests conducted include: Grain size analysis, Specific gravity, Atterberg limits, free swell, standard compaction, unconfined compression strength and swell-consolidation tests were conducted. These tests were deemed enough to characterize the soil for the present research work.

The material used for this laboratory test was as identified in the material and methods section of this document.

3.2 Material

3.2.1 Expansive (Natural) soil

The expansive soil was collected from Akaki Kality sub-city, Koye condominium site. The area is generally flat and it is the area where potential expansive soil exists according to the geological map of Addis Ababa. Representative samples were collected from a depth of 2m from the natural ground level. The samples have black to dark-grey colour.

3.2.2 Pozzolanic Sand (Stabilizing Agent)

Pozzolanic sand was brought from around Meki Town which is located in Oromia regional state, Eastern Shoa, Dugda Wereda 130 Km southeast of Addis Ababa. The sand is in use for hollow concrete block (HCB) production by the enterprises found in Meki Town. The sand is not river sand but it is available as a natural hill to the west of Meki Town. Representative samples are collected and transported to Addis Ababa and used for the proposed laboratory test in the research.

a. Hill of pozzolanic sand b. Pozzolanic sand collected

Figure 3-1: Pozzolanic sand deposit to the west side of Meki Town

3.2.3 Water

According to American Standards for Testing Materials (ASTM) the water to be used to conduct laboratory tests like Atterberg limit, specific gravity is demineralized or distilled water. But due to lack of such water in the geotechnical engineering laboratory of Addis Ababa institute of technology water used for conducting the tests was tap water.

3.3 Laboratory tests on the characterization of natural/expansive soil

3.3.1 Grain size analysis test

The grain size distribution can be determined by two methods i.e. the sieve and hydrometer analysis of which the sieve analysis is used for soils in which sands are predominant in percentage whereas hydrometer analysis is used for soils which have silt/clay as a predominant particle in percentage. For this particular research, both methods were applied to get overall or combined grain size distribution of the soil under investigation as per ASTM D 422-63.

Table 3-1: Grain size analysis test results (Sieve and Hydrometer)

Particle Size (mm)	4.75	2.36	1.18	0.6	0.3	0.15	0.075	0.040	0.028
Percentage Finer	99.84	99.63	99.53	99.32	99.14	98.95	98.70	94.57	92.94
Particle Size (mm)	0.018	0.011	0.008	0.005	0.004	0.003	0.002	0.001	
Percentage Finer	88.04	83.15	78.91	75.65	73.37	71.09	69.78	66.20	

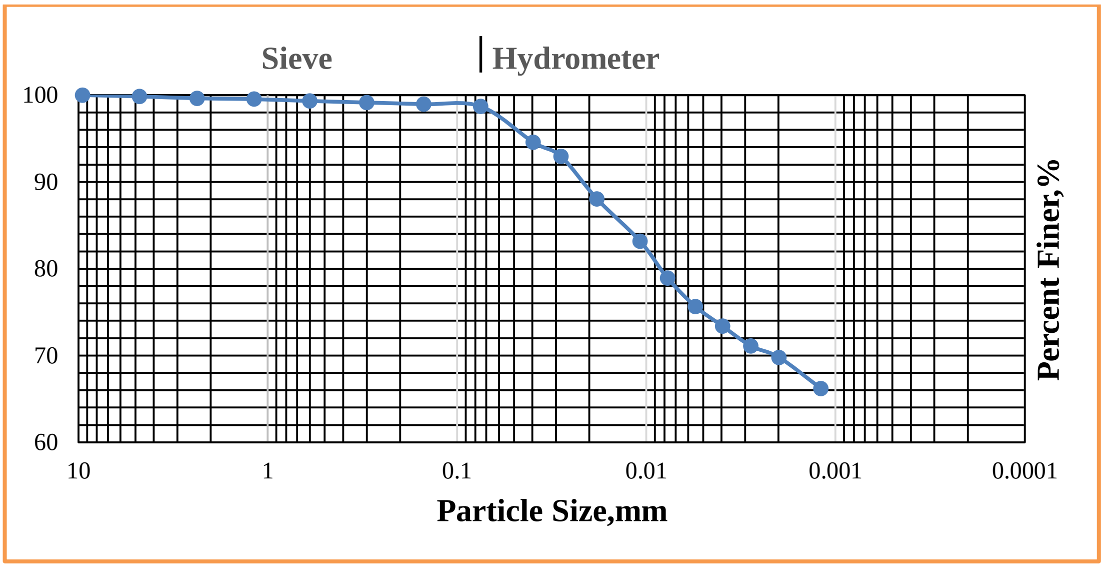


Figure 3-2: Particle Size distribution for the sample tested

3.3.2 Atterberg limits tests

The liquid limit and plastic limit together with the natural water content can be used to determine the relative consistency or liquidity index of clay soils and can also be used with percentage finer than 2micron size to determine the activity number of clay soil. The results of the tests are indicated in Figure 4-4 and Table 4-2 as per ASTM D 4318-98

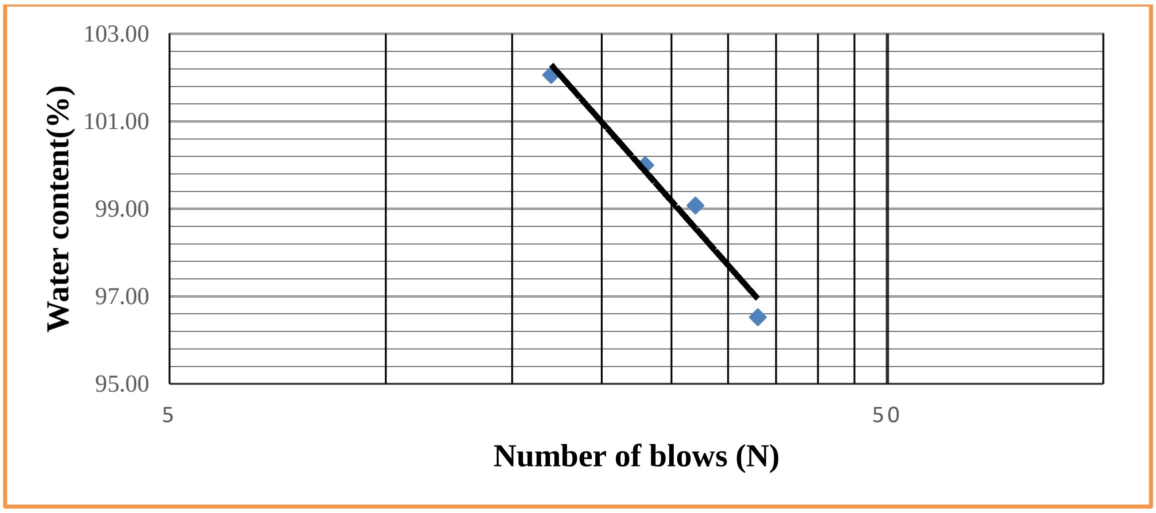


Figure 3-3: Flow curve for Natural soil

Table 3-2: Results of Atterberg limit tests for natural soil

LL, %	99.1
PL, %	36.5
PI, %	62.6

The activity number for the soil is determined with plasticity index and percent finer than 2 µm size particles, which can be expressed as a ratio of plasticity index to percentage finer than 2-µm size particles according to (Skempton, 1953). Based on the data obtained in the laboratory the Activity number (Ac) for the natural soil is equal to 0.9.

3.3.3 Free Swell Test for Natural soil

This test was conducted to determine volume change that the soil show when it comes in contact with moisture or the moisture within the soil varies. The test is one that is simple and cannot take into account any stress effects as on-site soils are mainly under stress due to building loads and/or pavement loads, overburden pressures. Here in this test 10cc(V_s) of dry soil passing No. 40(425µm) sieve is poured slowly into 100ml graduated cylinder then filled with water and allowed to settle for 24hr which gives final volume (V_f) of the soil. After obtaining the final volume, the free swell can be calculated as a ratio of final volume (V_f) minus initial volume (V_s) to initial volume (V_s) expressed as a percentage. From which free swell value for natural/untreated soil was obtained to be 200%.

3.3.4 Specific Gravity Test

The specific gravity (Gs) of soil is the measure of the heaviness of the soil particles. Mathematically speaking, it is the ratio of the mass of a unit volume of a soil solids to the mass of the same volume of gas-free distilled water at room temperature i.e. 20°C.

The specific gravity of the soils was determined according to ASTM D854-98 using calibrated water pycnometer in which the mass of pycnometer filled with gas-free distilled water and mass of pycnometer filled with soil-water slurry to graduate-level i.e. 100cc was measured to determine the specific gravity of the soil. In all cases, the entrapped air in the soil-water slurry was removed by using air-compressor. The temperature at which the mass was measured must be noted to take effect of temperature into account as the density of water varies with temperature.

The specific gravity soil solids at test temperature were determined using the following equation:

$$G_t = \frac{M_s}{(M_s + (M_a - M_b))} \quad 3\text{-}1$$

The specific gravity of soil solids at standard room temperature i.e. 20ºC is determined by the following:

$$G_{20^o C} = K.G_t \quad 3\text{-}2$$

Where K is the temperature coefficient given in Table 2 of ASTM D854-10, according to this specific gravity of the soil was found to be 2.68.

3.3.5 Moisture Density Relations (Compaction) Tests of Natural Soil

The compaction tests of untreated soil were conducted according to ASTM D698-91 to determine the maximum dry density and optimum moisture content at which the maximum dry density is attained. Soils are compacted mostly to obtain satisfactory engineering properties such as shear strength, compressibility or permeability that is why foundation soils are compacted to improve their engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved. For current soil under investigation compaction test was conducted to determine its optimum moisture content and maximum dry density as indicated in Figure 4-5.

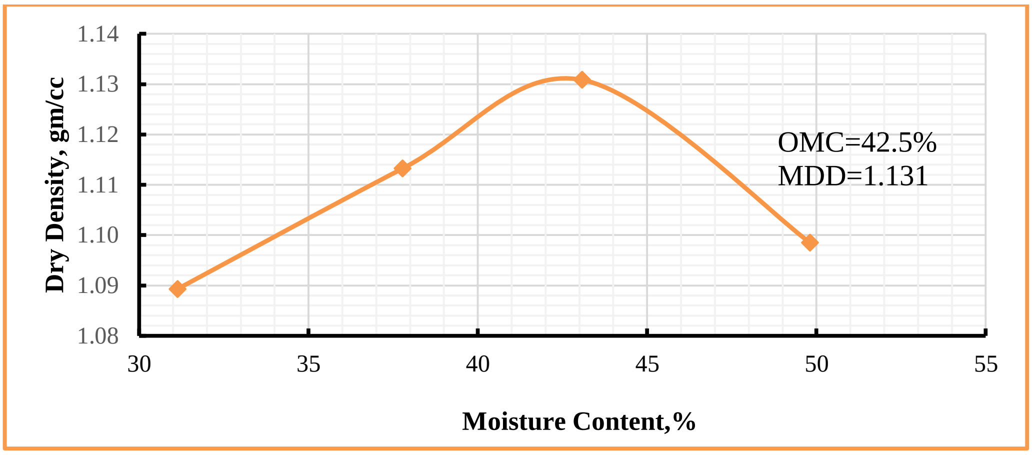


Figure 3-4: Compaction curve for Natural/Native soil

3.3.6 Unconfined Compression Strength (UCS) Test for Natural Soil

The unconfined compression strength test is a compression test in which soil sample (remoulded or undisturbed) is subjected to axial load without any lateral confinement or pressure as in the triaxial test. It is also known as a quick test or undrained unconsolidated test as it doesn't allow water to drain from the sample and doesn't wait for the sample to consolidate. For the expansive soil under consideration once the maximum dry density and optimum moisture content were obtained in the standard compaction it paves the way to determine its compressive strength by remoulding the sample at a maximum dry density and optimum moisture content in a mould with diameter D = 38mm and H = 76mm as per ASTM D 2166-98. The unconfined compressive strength of the soil was found to be 191.2 kPa.

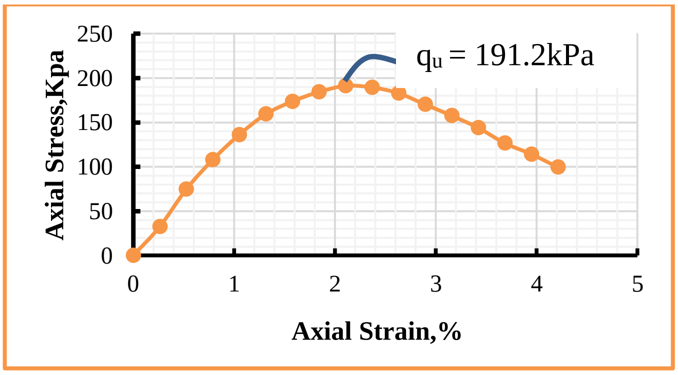


Figure 3-5: Axial Stress Vs Axial Strain plot of UCS test result for the Natural soil.

From this test results, the unconfined compression strength of the soil is q_u=191.2kPa and the shear strength of the soil is half of the unconfined compression strength which is equal to 95.6 kPa.

3.3.7 Swell and Consolidation Test

These tests are conducted to determine the swelling pressure of natural soil in the laboratory by remoulding the sample to maximum dry density and with its optimum moisture content in consolidation ring. The swelling pressure is the pressure which required to prevent the soil in a consolidation ring from swelling or it is a pressure which is required to recompress the sample to its original height or void ratio. Here in this test, the sample is allowed to swell for 72hrs under a seating load of 7kPa which is assumed to represent insitu overburden pressure which the soil is expected to experience on-site. After the swelling was completed the sample in a ring was subjected to loads in increments until it attains its initial height or initial void ratio. In this manner, the pressure which the soil exerts on building foundations was determined in the laboratory found to be 325kPa. In all situations, the soil was given access to water.

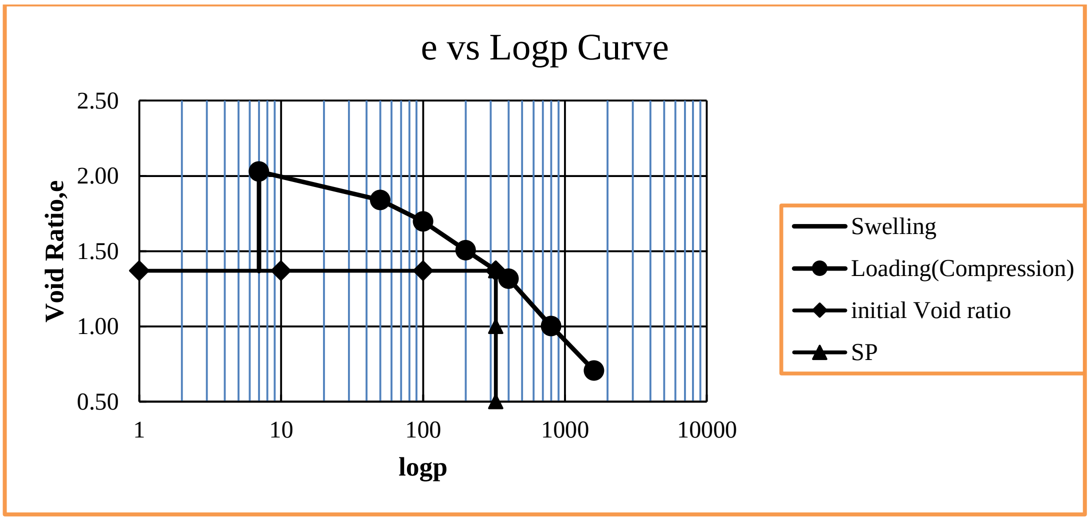


Figure 3-6: Void ratio Vs Log (pressure) curve for natural soil

3.4 Classification of the Natural soil

3.4.1 Unified Soil Classification System (USCS)

For this particular soil under investigation, the plot of plasticity index versus liquid limit falls above the A-line and with more than 30% fines passing the #200 sieve, so that the soil is classified as clay of high plasticity with group Symbol of CH. Further, the soil contains less than 15%, particles coarser than #200 sieve, in particular, the soil is classified as fat clay. The location of the plasticity index versus Liquid limit is indicated in the plasticity chart as shown in Figure 3-8.

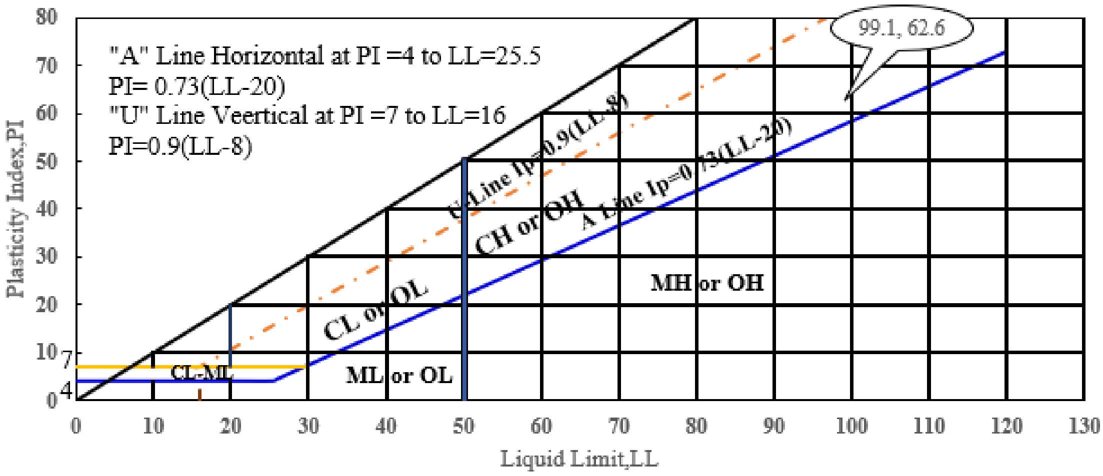


Figure 3-7: Location of Natural soil on Plasticity chart as per USCS ASTM D-2487-00

3.4.2 Classification of Soils based on its Free Swell value and plasticity Index

Free swell tests consist of placing a known volume i.e. 10cc of dry soil in water and noting the swelled volume after the material settles at the end of 24 hours, without any surcharge, to the bottom of a graduated cylinder of volume 100cc filled with water soil mixture to the graduate point. The difference between the final and initial volume, expressed as a percentage of the initial volume, is the free swell value.

Furthermore, the free swell is an indicator of the expansiveness of soil. According to Holtz and Gibbs (1956) soils having free swell value as low as 100% can cause considerable damage to lightly loaded structures. Since the value of the free swell for the soil under investigation is above 100% (200%), the soil is under the category of expansive soil with a high degree of expansion (Tefera & Leikun, 1999).

According to Chen (1988) the expansive soil is classified based on its plasticity index for which he outlined different ranges of plasticity index to classify the soil as Low, Medium, High and Very high Swelling Potential. Based on this the soil is categorized expansive soil with very high swelling potential.

3.4.3 Classification of soil Based on Activity Number, Ac

The activity number is the relation between plasticity index colloidal content of the soil which is percent finer than 2µm size particles. According to Skempton (1953), the activity number was used to categorize clay soils as active, Normal and In-Active, the activity number for the soil under investigation is 0.9 as determined earlier in this document based on this classification the clay soil is Normal, but in the same method the soil can also be classified based on the activity chart which indicates the soil under investigation has a very high degree of expansiveness.

Table 3-3: Summary of properties of natural soil

Colour	Black to Gray
Specific Gravity	2.68
Percent Passing 75μm (%)	98.7
Percent finer than 2μm (%)	69.78
Liquid Limit (%)	99.1
Plastic Limit (%)	36.5
Plasticity Index (%)	62.6
Free Swell Value (%)	200
Activity Number(Ac)	0.9
Classification (USCS)	CH
Optimum Moisture content (%)	42.5
Maximum Dry Density(g/cc)	1.131
Unconfined Compressive Strength(kPa)	191.2
Swelling Pressure(kPa)	325

3.5 Laboratory Tests on Characterization of Pozzolanic Sand

3.5.1 Grain Size Analysis for PS

The pozzolanic sand was collected from around Meki Town where it is found abundantly. Basic properties of sand were investigated by conducting tests in the laboratory. Sieve analysis, water content, and the specific gravity of pozzolanic sand were determined. These tests were conducted on the sand with its natural grain size without crushing or pulverizing the sand into finer sizes to get its properties. To conduct similar tests the sand was crushed to pass through 150µm and 75µm to study the effect of the size of pozzolanic sand on the engineering properties of the natural/expansive soil upon blending.

Table 3-4: Test conducted on pozzolanic sand and test results (Unpulverized)

Tests Conducted		Test Results
Gradation, Percent Passing, %	19 mm	100
	9.5 mm	100
	4.75 mm	99.80
	2.36 mm	96.79
	1.18 mm	40.9
	0.6 mm	10.12
	0.3 mm	2.24
	0.15 mm	0.92
	0.075	0.34
Specific Gravity, Gs		1.77
Natural Moisture Content, %		23.97
Liquid Limit		Could not be determined
Plastic limit		Could not be determined

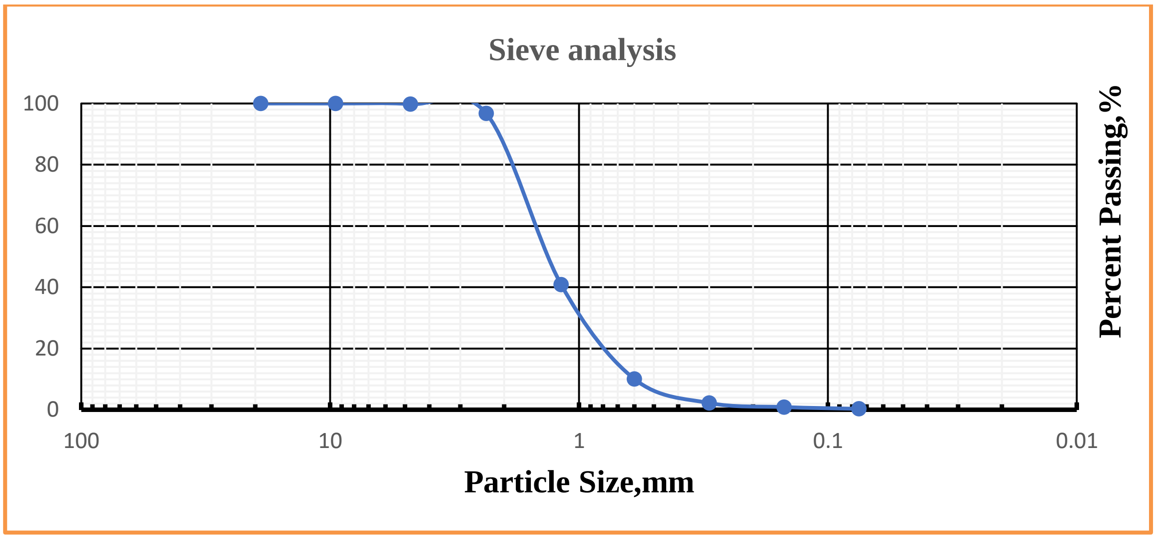


Figure 3-8: Particle Size distribution of PS as brought from the quarry.

As depicted in figure 4-9 above, the pozzolanic sand as brought from the quarry has 98.80 % passing 4.75mm and 0.34% passing 75µm sieve size, with 59.1% retained on 1.18mm, 89.88% retained on 0.6mm, and 97.76% retained on 0.3mm sieve sizes, which indicates that the pozzolanic sand as brought from the quarry is uniformly graded based on sieve analysis test only.

The sand was crushed/pulverized to pass certain sieve size to make it finer and of uniform size. For this particular research, the sand was crushed and sieved to pass 150µm and 75µm sieve size. The particle size distribution of pulverized pozzolanic sand was done using the hydrometer method of particle size analysis according to ASTM D422-98. The results of the analysis were as indicated in Figure 3-9.

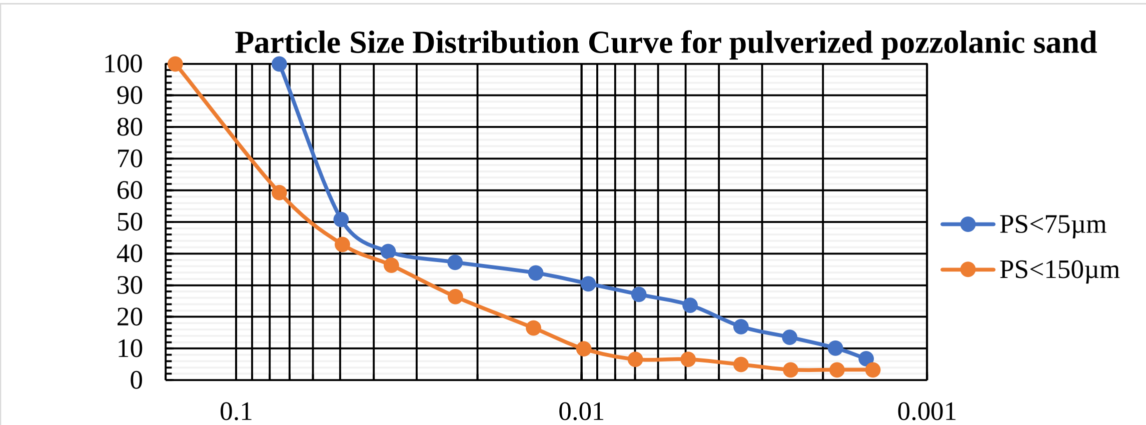


Figure 3-9: Particle size distribution curve for pulverized pozzolanic sand

As depicted in figure 3-9, 50.83% of the total sample used has a size of 50µm in the case of pozzolanic sand crushed to a particle size less than 75µm, but the remaining 49.17% ranges in particle size from 50µm to 75µm which are these particles sizes settled rapidly out of suspension to the bottom of the hydrometer testing cylinder. Similarly, in the case of pozzolanic sand crushed to a particle size less than 150µm, 42.88% of the sample used in the test has a size less than 50µm in which the sample settled out of suspension ranges in size from 50µm to 150µm in the hydrometer test. Also, pozzolanic sand crushed to a size less than 150µm contains particle sizes retained on 75µm and which is 40.72% as obtained from sieve analysis of representative samples of PS with size less than 150µm, using set of sieve containing 150µm,75µm and pan. In both cases, the pozzolanic sand with particle sizes greater than 50µm settles to the bottom of the hydrometer test cylinder.

The specific gravity of the pozzolanic sand crushed and passed through 150µm and 75µm sieve size was obtained to 2.54 and 2.44 respectively.

3.5.2 The Atterberg Test Results of the Pozzolanic sand

The liquid limit for pozzolanic sand was tried to be determined in the laboratory for pozzolanic sand passing sieve size 425µm using the Casagrande apparatus but at low water content the sand slides over the surface of the Casagrande cup and even it is difficult to make a groove across the surface of the placed soil pat. As water content is increasing the sand with a groove on the cup will close the groove after just it has been grooved. Sometimes it tends to close the groove with one or two number of blows of the Casagrande cup. Several trials of such tests have been done all the number of drops to close the groove was below 25 so the liquid limit of the pozzolanic sand cannot be determined and it is non-plastic material according to ASTM D4318-00 whether it was cured or uncured.

Figure 3-10: Liquid limit determination for pozzolanic sand

3.5.3 Moisture Density Relation for Pozzolanic Sand

Compaction of the pozzolanic sand was carried out in the laboratory without curing according to ASTM D698-98 to determine the maximum dry density and optimum moisture content of pozzolanic sand using the standard compaction effort, as all sand passes 4.75mm sieve size upon gradation. But in this case, it was difficult to obtain the maximum dry density and optimum moisture content of the sand because it cannot have any tendency to get compacted, upon doing several trials the water starts to flow/drain out of the compacted sand while the compaction rammer was hitting the sand from height specified for standard compaction effort. Which makes it difficult to obtain the water content in each trial of compaction.

But when the sand was crushed and passed through smaller sieve sizes 150µm there was little bit tendency to get compacted, as the water for each trial increases the mass of the compacted sand trimmed to the height of the compaction mould increases and started to decrease after the sixth trial. Based on this test the maximum dry density and Optimum Moisture content of the pozzolanic sand passed 150 µm sieve size was found to be 1.07g/cc and 40%, respectively.

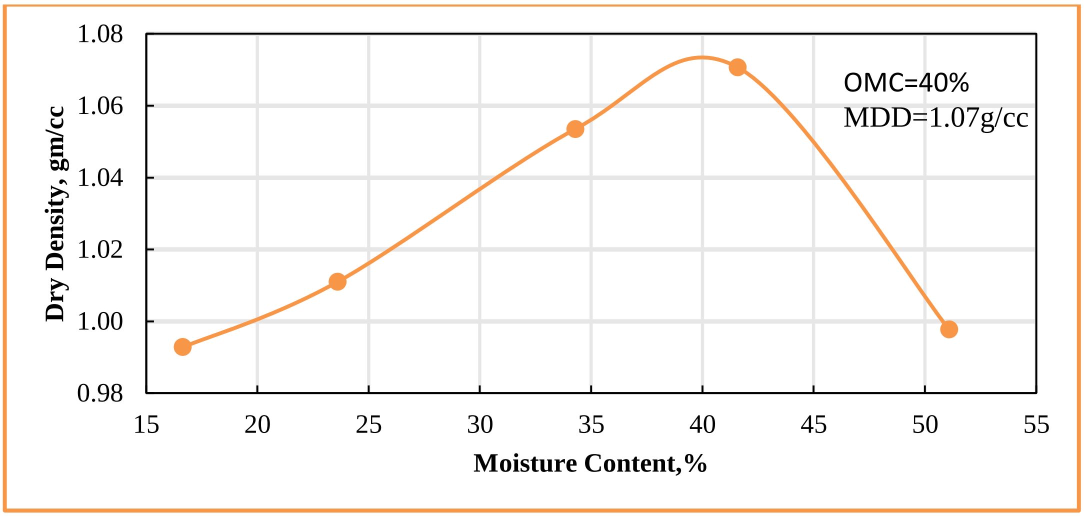


Figure 3-11: Compaction curve for the pozzolanic sand.

3.5.4 Unconfined Compression Strength Test (UCS) for Pozzolanic Sand

The unconfined compression strength test is a compression test in which soil sample (remoulded or undisturbed) is subjected to axial load without any lateral confinement or pressure as in the case of the triaxial test. For the pozzolanic sand under investigation with the maximum dry density and optimum moisture content known the compressive strength was determined by remoulding the sample with D=38mm and H=76mm at a maximum dry density and optimum moisture content. The unconfined compressive strength of the sand was found to be 157 kPa.

Figure 3-12: Pozzolanic sand under compression

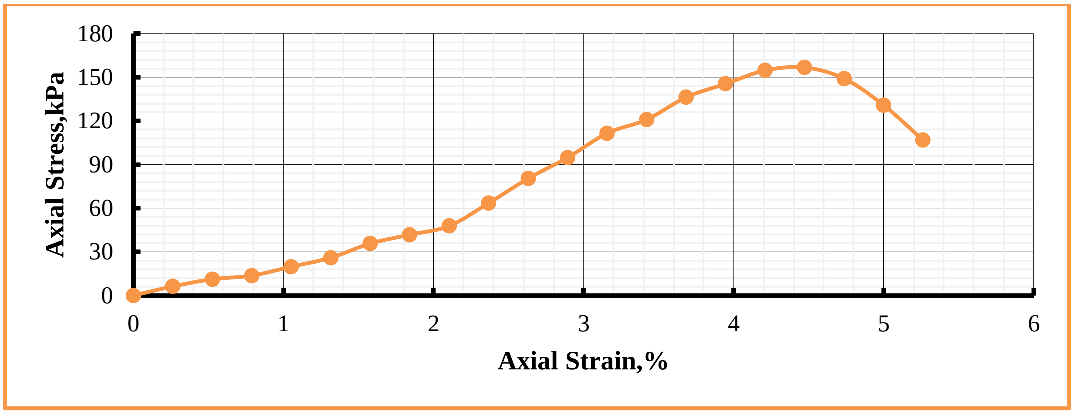


Figure 3-13: Axial stress Vs Axial strain plot of UCS test result for the pozzolanic sand.

3.5.5 Chemical Composition of Pozzolanic Sand

Since the chemical composition of pozzolanic sand shall be known, silicate analysis was conducted to estimate the oxide composition. SiO_2, Al_2O_3 and Fe_2O_3 are the prominent oxides present in pozzolanic materials. Therefore, their presence and amount are used to identify pozzolanic materials. The pozzolanic sand used in this research has 89.03% of prominent oxides which makes the sand pozzolanic as per ASTM C-618. Table 4-6 shows the chemical composition of pozzolanic sand used, test result obtained from the Geological Survey of Ethiopia, Geochemical Laboratory, see Appendix F.

Table 3-5: Complete silicate analysis test result

Chemical Oxide	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
Percentage (%)	74.84	8.09	6.10	<0.01	<0.01	3.80	2.76	0.20	0.12	0.05	0.62	3.70

CHAPTER 4 RESULTS AND DISCUSSION ON LABORATORY TESTS OF BLENDED SOIL

4.1 Introduction

In this chapter, all laboratory test results are discussed as the main objective of the thesis is to investigate the effect of pozzolanic sand on engineering properties of expansive soil. As outlined in Section 3.3 of this document, the engineering properties of expansive soil were determined and laboratory test results were discussed there in detail but here in this chapter, the engineering properties of blended soil under both sizes of pozzolanic sands are discussed. The laboratory tests done for the blended soil includes free swell, Atterberg limit, Specific Gravity, Standard compaction, unconfined compressive strength, and Swell-Consolidation tests for each percentage of pozzolanic sands added to the expansive soil. The mix proportions of blending were as outlined in Table 1.1, where the amount of pozzolanic sand was percentage by dry mass of natural/expansive soil.

4.2 Effect of Pozzolanic Sand on Free-Swell of Expansive Soil

The free swell is one of the preliminary tests, to check whether the given soil is expansive or not. This test is not standardized in ASTM but it was suggested by Holts and Gibbs. For this particular soil under investigation, the free swell value was found to be 200% which indicates the soil is highly expansive. Upon adding the pozzolanic sand, which is non-expansive material, the free swell value of the soil decreased with increasing the amount of pozzolanic sand. For the same class of soil, (Fusheng Zha, 2008) observed that the free swell decreased from 55% to 32.5% for the addition of 15% fly ash. Similar results were also observed by (Birhane, 2018) on adding pumice to the expansive soil. The pozzolanic sand used in this experimental study is primarily composed of silicate, aluminium and iron oxides. This PS has the potential to provide multivalent cations (Al^{3+}, Fe^{3+}, etc.), which promote flocculation of clay particles (Fusheng Zha, 2008). As a result, the specific surface area and water affinity of the samples decrease, which indicates a reduction in the free swell values of the soil.

Even though the free swell was decreased upon addition of pozzolanic sand its effect is more noticeable when the sand was crushed to a size less than 75µm as compared to that crushed to a size less than 150µm. As shown on Figure 5-1, for the same percentage of pozzolanic sand sizes added to the expansive soil, the free swell value of the expansive soil decreased when the soil was blended with pozzolanic sand passing sieve size 75µm than blending the expansive soil with pozzolanic sand passing sieve size 150µm. The free swell value decreased from 200% to 80% when 15% of PS <75µm was added to the expansive soil but for PS<150µm the free swell value decreased to 115% upon addition of 25% pozzolanic sand to the expansive soil.

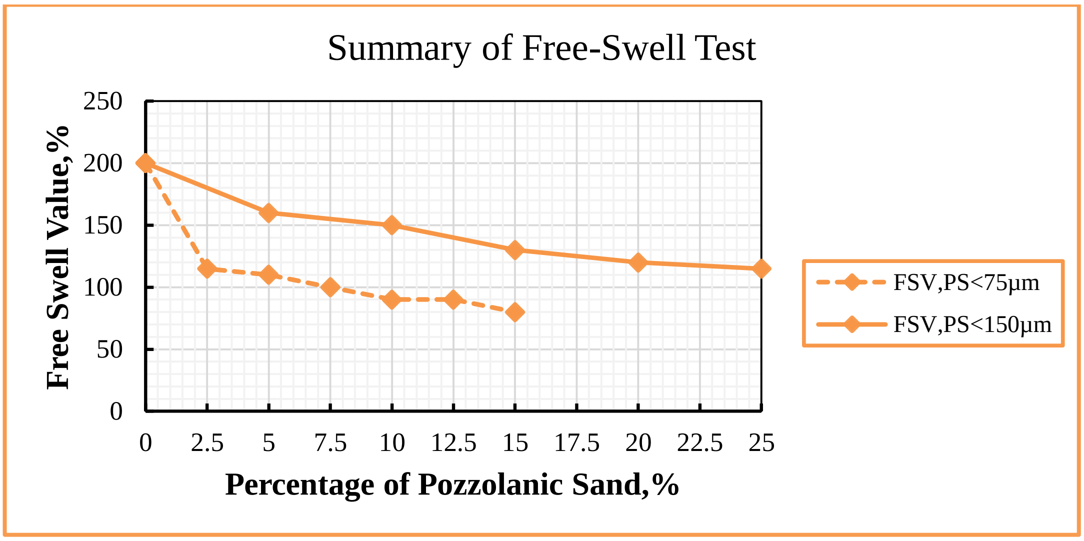


Figure 4-1: Effect of pozzolanic sand on the free swell value of expansive soil

As seen from the above test results, the swelling of the expansive soil decreased, when the soil was blended with pozzolanic sand crushed to a size less than 75µm than that crushed to a size less than 150µm. The decrease in free swell value for PS<75µm was because pozzolanic sand which is non-swelling material partially replaces some of the volumes which were previously occupied by the expansive clay minerals and the chemical reaction between clay minerals of expansive soil and the pozzolanic sand. Besides, the pozzolanic sand cements the soil particles together. In this regard, treatment of expansive soil with pozzolanic sand crushed to a particle size less than 75µm was more effective in reducing the free swell than pozzolanic sand crushed to a size less than 150µm.

4.3 Effect of Pozzolanic Sand on the specific gravity of Expansive Soil

The effect of pozzolanic sand on the specific gravity of expansive soil is shown in Figure 5-2. The specific gravity of the expansive soil increased from 2.68 to 2.78 when 10% of PS<150µm was added to the expansive soil after that the specific gravity decreased to 2.57, but for PS< 75µm the specific gravity increased from 2.68 to 2.88 upon addition of 5% pozzolanic sand to the expansive soil, after this point as the percentage of pozzolanic sand is increasing the specific gravity starts to decline and reached to 2. 81 when 15% PS was added to the expansive soil but not less than the specific gravity of expansive soil.

After 10% for PS<150µm and 5% for PS<75µm, the specific gravity is in an indirect proportion with the percentage of pozzolanic sand added to the expansive soil. When the percentage of pozzolanic sand passed these two values, the pozzolanic sand which is lightweight material starts to dominate and replace the heavier particles of expansive soil. Due to that the specific gravity for the size of PS<150µm decreased to 2.57 which is less than the specific gravity of expansive soil i.e. equal to 2.68 when 25% by dry weight of expansive soil was replaced by the pozzolanic sand. But for the size of PS<75µm, even though it was decreasing it doesn't go below the specific gravity of the expansive soil when 15% pozzolanic sand was added to the expansive soil.

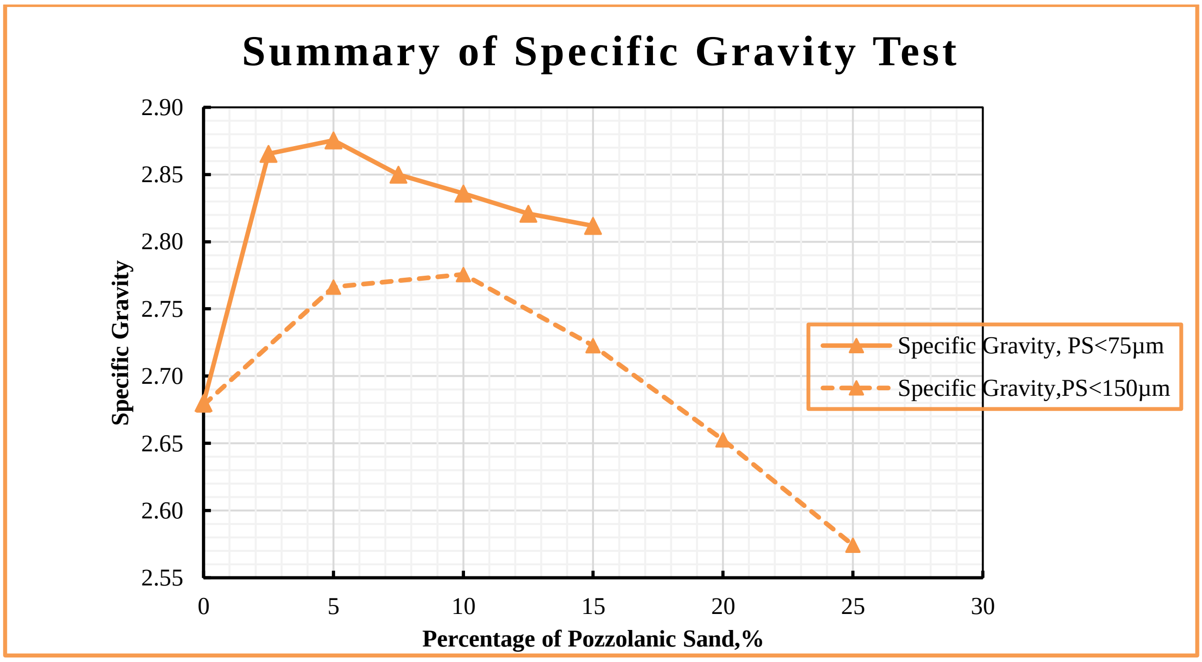


Figure 4-2: Variation of specific gravity with the percentage of PS

Similar scenarios occurred concerning the effect of pozzolanic sand on the specific gravity of the expansive soil for both particle sizes considered. The pozzolanic sand has the effect of filling the void space between clay particles at lower percentages, until the percentage of blending reaches 5% and 10% for PS<75μm and 150μm respectively. This may be the reason for the increase of the specific gravity of the soil. But, at higher percentages of the pozzolanic sand blended to the expansive soil, the specific gravity starts to decline. This may be due to the fact, the pozzolanic sand starts to coat individual particles of clay which resulted in aggregated nature of the expansive clay soil-PS mixture. This means that the single solid soil particles of natural clay were transformed into the aggregated particles of soil–PS consisting of several single solid particles with voids in between. This may be the reason for the decrease of specific gravity for both sizes of pozzolanic sand blended to the expansive soil.

4.4 Effect of Pozzolanic sand on Atterberg limit

The Atterberg limit defines the moisture content between the states of consistency of fine-grained soil, and clay soils, which mainly exist in four consistency states i.e. liquid, plastic, semi-solid and solid states. The liquid limit and plasticity index can be used in determining the swelling characteristics of expansive soil. The Atterberg limits are in direct relationship with the swelling potential of expansive soil, i.e. when the plasticity of soil increases, the swelling potential of the soil increases as well (Chen, 1975). But for soil with low plasticity index, the swelling potential is low (Chen, 1975).

4.4.1 Effect of Pozzolanic sand on plasticity index

The effect of pozzolanic sand on the plasticity index is shown in Figure 5-3 for both sizes of pozzolanic sand. The effect of curing on the plasticity index is also shown in the same figure. Similar results were observed by (Harichane, Ghrici, Kenai, & Khaled, 2011), (Fusheng Zha, 2008), (K.M.A. Hossain*, 2007). As shown in Figure 4-3 the plasticity index of the blended soil decreases, as the percentage of pozzolanic sand increase, which is more pronounced for the case of pozzolanic sand crushed to a particle size less than 75μm which implies that there was more pozzolanic reaction between pozzolanic sand and the expansive soil.

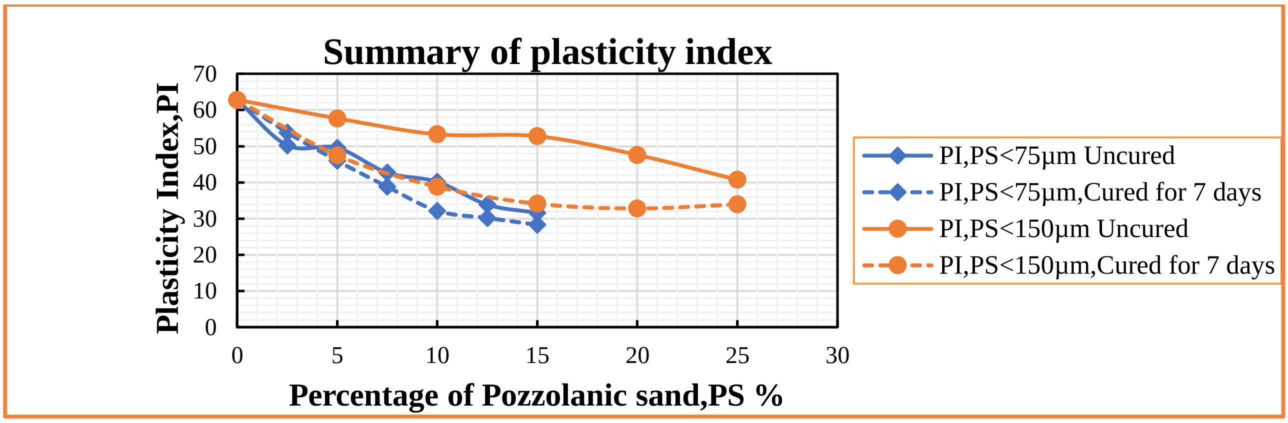


Figure 4-3: Variation of plasticity index with the percentage of pozzolanic sand

The plasticity index decreases because more plastic clay was replaced with pozzolanic sand which is non-plastic material as indicated in section 3.5.2., as depicted in Figure 4-3 the effect of curing is more or less the same for both sizes of pozzolanic sand i.e. the plasticity index decreases in the same trend. For the pozzolanic sand crushed to a particle size less than 150µm the plasticity index of blended soil starts to increase after 15% by dry weight of expansive soil was replaced with pozzolanic sand but not reaching the value of the uncured soil as well as that of untreated expansive soil. Details of Atterberg limit test results are given in Appendix A-3. The main reason for a reduction in plasticity is due to flocculation of clay particles, a decrease in specific surface area and a decrease in water affinity of PS-soil mixture.

Concerning the particle size of pozzolanic sand, the plasticity index reduction is more pronounced for expansive soil blended with pozzolanic sand crushed to a size less than 75µm.

4.4.2 Effect of Pozzolanic sand on soil classification

As indicated in Figure 4-4, the plot of plasticity index versus liquid limit falls below the A-line for all percentages of pozzolanic sand crushed to a particle size less than 75µm blended to the expansive soil. But in the case of pozzolanic sand crushed to a particle size less than 150µm the plot of plasticity index versus liquid limit slightly falls above A-Line until the percentages of pozzolanic sand reach 20% after that it falls below the A-Line.

As depicted in Figure 4-4, the plasticity of the expansive soil was more affected when the clay minerals were replaced with finer material, in this case, the pozzolanic sand crushed to a particle size less than 75µm. From this, the swelling potential of the soil decreases as the plasticity index of the soil is decreasing on replacing the part of expansive soil with non-expansive material.

Upon curing the plasticity of the soil decreases in both sizes of pozzolanic sand as the percentage is increasing as indicated in Figure 4-5, with pozzolanic sand crushed to a particle size less than 75μm more effective in reducing the plasticity of the expansive soil upon blending.

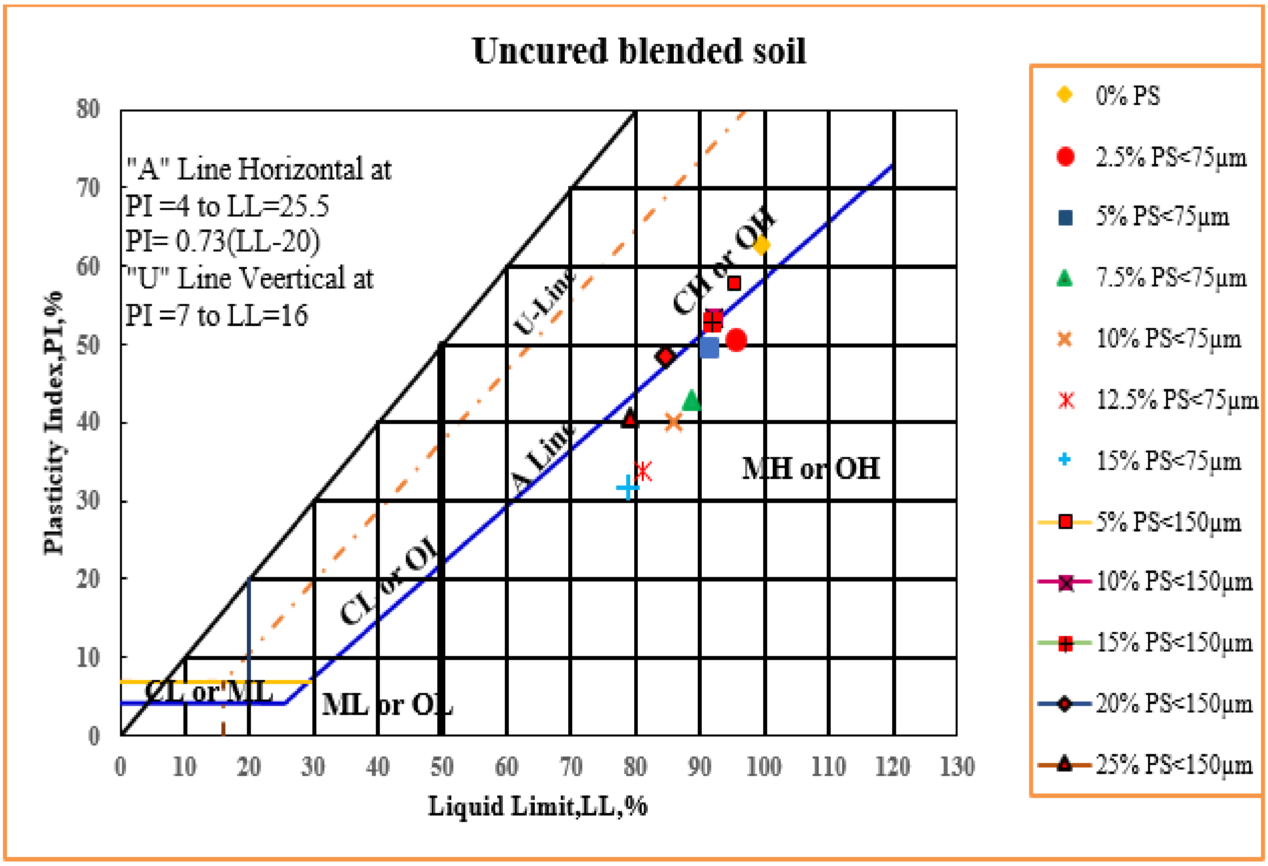


Figure 4-4: Location of soil on the plasticity chart with varying percentage of pozzolanic sand for classification.

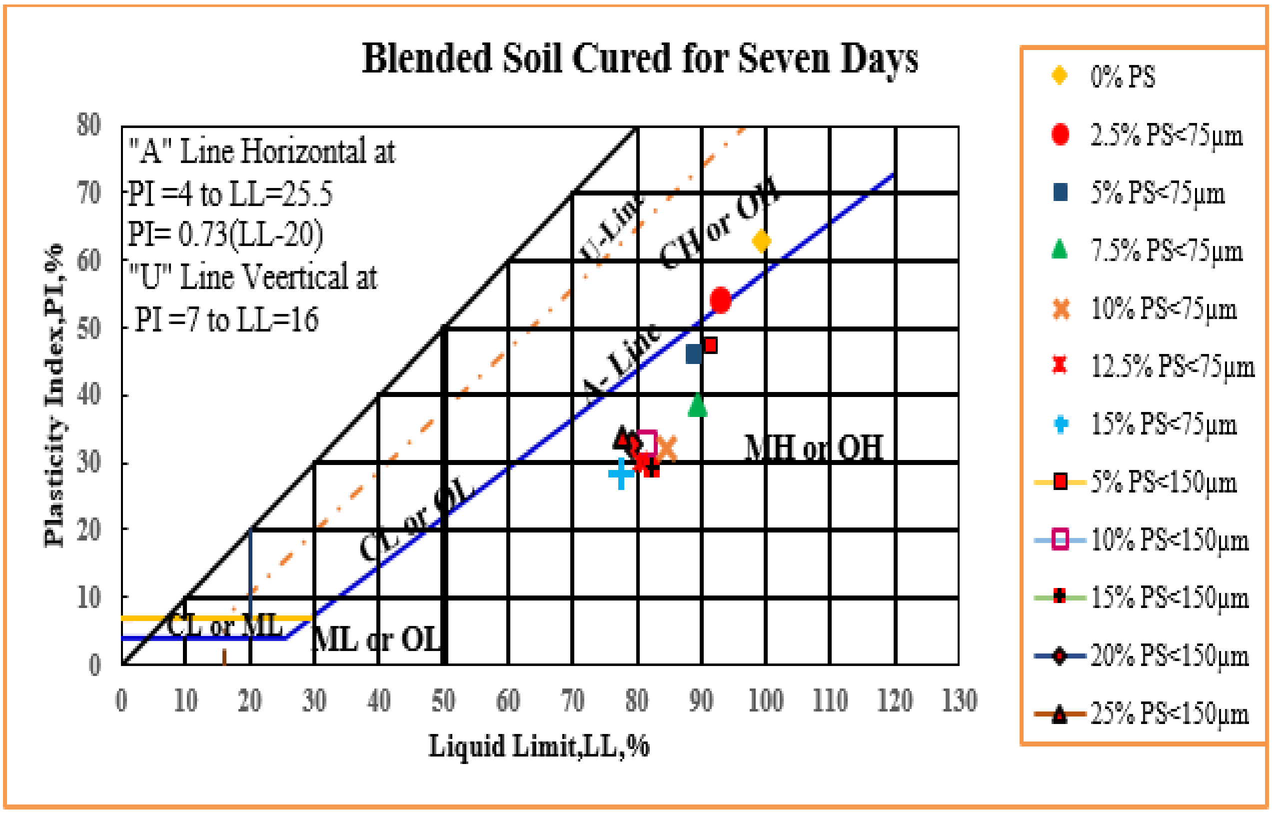


Figure 4-5: Location of soil on the plasticity chart with varying percentage of pozzolanic sand for classification.

4.5 Effect of Pozzolanic Sand on Compaction Characteristics of soil

All pozzolanic sand treated soil compaction curves fall to the dry side of the compaction curve of the natural soil as the percentage of the pozzolanic sand is increasing. But for 5% of expansive soil replaced with pozzolanic sand the OMC obtained was higher than the OMC of the natural soil for both sizes of pozzolanic sand. The effect is more noticeable when the native soil is blended with the pozzolanic sand crushed to a particle size less than 150μm when the pozzolanic sand with such particle size is added to the soil it replaces the clay soil which is uniform and finer in size. It was known that when a courser material is added to finer soil it improves the gradation of the soil which in turn improves the dry density of the soil.

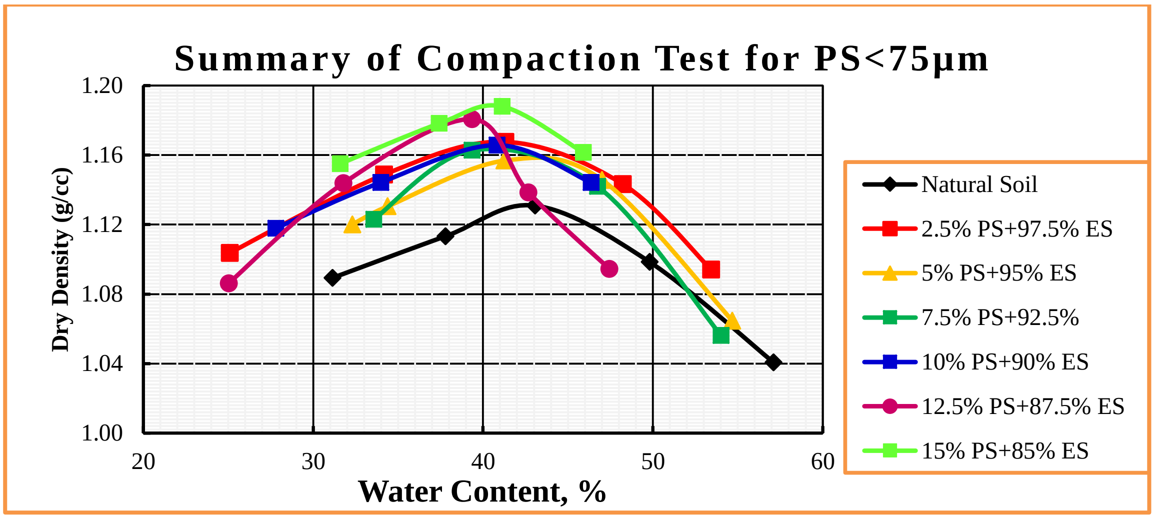


Figure 4-6: Summary of Compaction Curves for PS<75μm

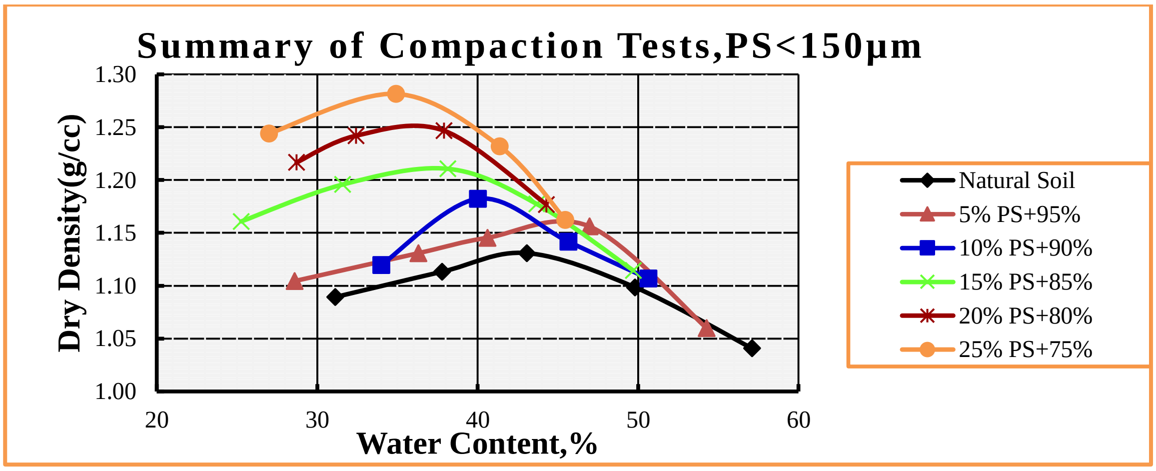


Figure 4-7: Summary of Compaction curves for PS<150μm

4.5.1 Effect of PS on Dry Density of Expansive Soil

The effect of pozzolanic sand on the dry density of the expansive soil is shown in Figure 4-8. As shown in figure 4-8, the maximum dry density increases as the percentage of pozzolanic sand increases, which in turn shows the improvement of the soil. The increase in the dry density is more pronounced in the case adding PS<150μm to the expansive soil than replacing a portion of the expansive soil with PS<75μm. The reason for the increase in dry density is because the finer clays are replaced with relatively courser pozzolanic sand which in turn modifies the gradation of the expansive soil as the soil is uniform in size as indicated in Figure 3-3 in section 3.3.1.

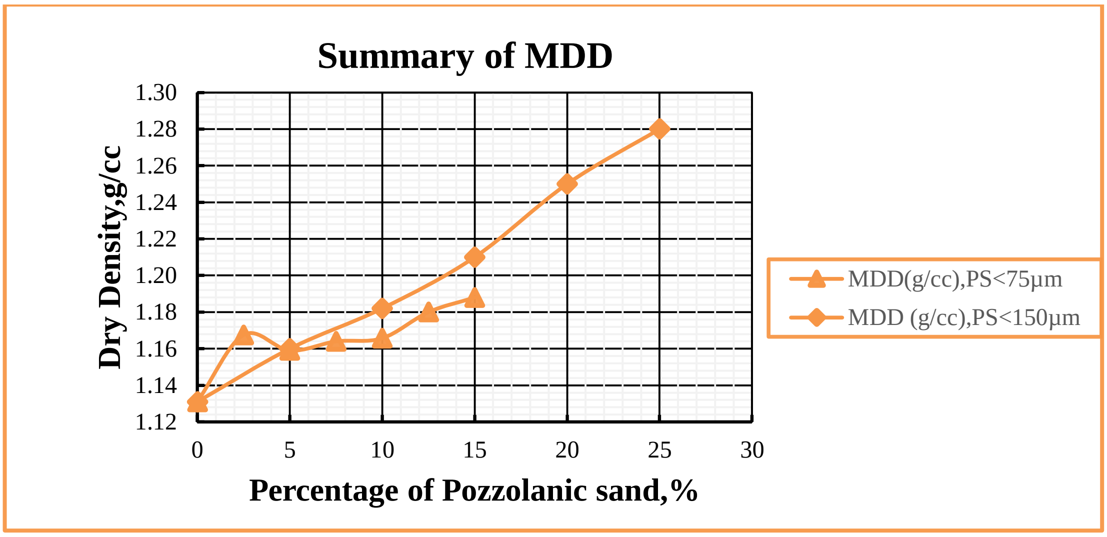


Figure 4-8: Variation of maximum dry density with a pozzolanic sand content

The pozzolanic sand crushed to a particle size less than 75µm has an immediate effect on increasing the maximum dry density when it replaces 2.5% of the expansive soil. This increase may result from a filler effect of small particles of pozzolanic sand which filled void space between clay particles and allowing denser packing.

4.5.2 Effect of PS on Optimum Moisture Content of Expansive Soil

The optimum moisture content of the soil decreases with increasing the percentage of pozzolanic sand. As depicted in Figure 4-9, the effect on moisture content is more noticeable when the pozzolanic sand was crushed to a particle size less than 150µm than that crushed to a particle size less than 75µm. The reason for the decrease in optimum moisture content is that due to lower affinity of pozzolanic sand for water as the pozzolanic sand fills the pore spaces between soil particles, which prevents water from entering these pore spaces.

Even though, the optimum moisture content is decreasing there are uncertainties which may lead to experimental errors. The errors may arise from the mode of mixing of pozzolanic sand and the natural soil as it was done manually. But efforts were made to mix pozzolanic sand and soil as much as possible to get a uniform mixture.

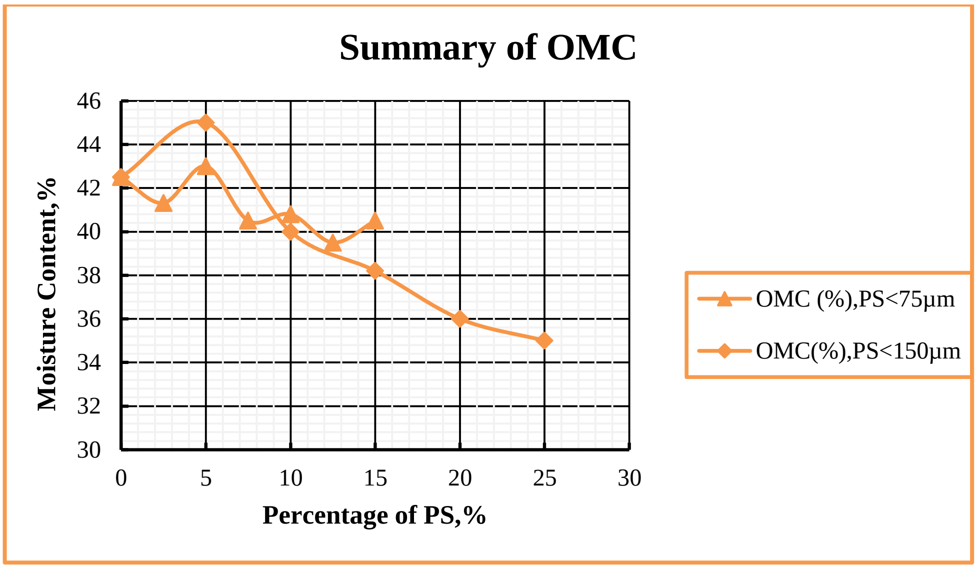


Figure 4-9: Effect of pozzolanic sand on OMC of expansive soil

4.6 Effect of Pozzolanic Sand on Unconfined Compressive Strength of Expansive Soil

The trend of change in unconfined compressive strength (UCS) with the addition of a variable percentage of pozzolanic sand is shown in Figure 4-10. As depicted in Figure 4-10, the addition of pozzolanic sand crushed to a particle size of less than 75μm increases the compressive strength of blended soil specimens. The increase in compressive strength is not linear with the percentage of pozzolanic sand blended to the soil. The compressive strength increases when 2.5% of pozzolanic sand is blended with the natural soil up to the percentage of the pozzolanic sand reaches 7.5% at which the compressive strength increases to 1.9 times that of natural soil after that the compressive strength decreases. Even though, the compressive strength of blended soil is decreasing, it is greater than the compressive strength of the natural soil.

In the case of pozzolanic sand crushed to a particle size less than 150μm, the unconfined compressive strength increases with the increase of pozzolanic sand content. In this case, the unconfined compressive strength increases to 1.86 times that of the strength of natural soil when 25% pozzolanic sand by dry weight of the natural soil was added but with a slight difference from the previous percentage of pozzolanic sand added which increases the unconfined compressive strength 1.8 times.

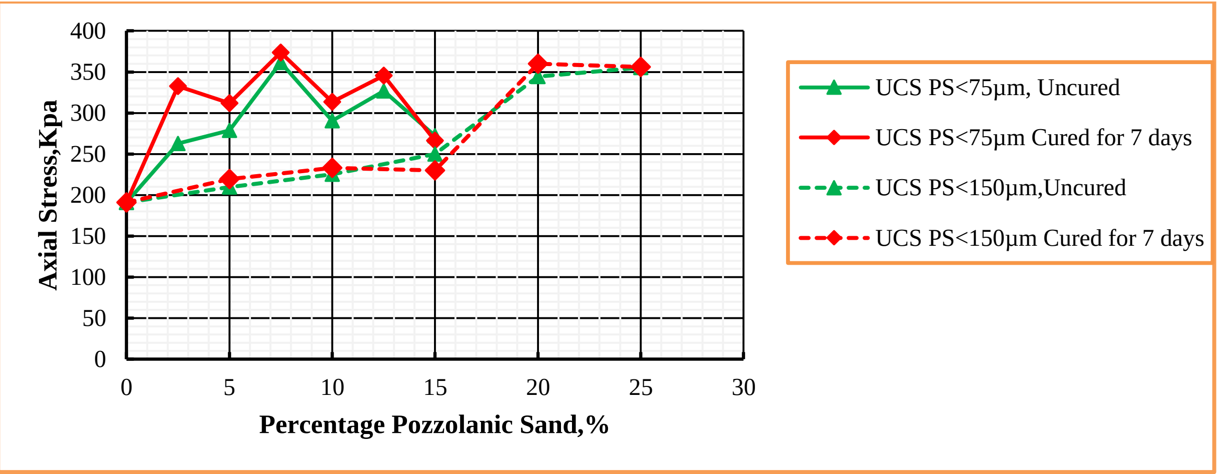


Figure 4-10: Effect of pozzolanic sand on UCS of expansive soil

Upon curing, the compressive strength of the blended soil increases to some extent which is not that much significant as compared to strength obtained before curing. But the result shows the compressive strength increases with age. Similar results were observed by (K.M.A. Hossain*, 2007), (Harichane, Ghrici, Kenai, & Khaled, 2011) using similar material. Moreover, the increase on strength is more pronounced on soil blended with pozzolanic sand crushed to a particle size less than 75µm than crushed to a particle size less than 150µm. For instance, curing increases the strength by 1.12 times that of the uncured blended soil when 5% pozzolanic sand crushed to a particle size less than 75µm was used as compared to soil blended with pozzolanic sand crushed to a size less than 150µm.

According to the laboratory test result, the compressive strength of the soil increased when pozzolanic sand of 7.5% was added to the natural soil. However, the addition of more than 7.5% pozzolanic sand resulted in the decrease of the compressive strength of the blended soil when the pozzolanic sand was crushed to a particle size less than 75µm. In this case, the decrease in strength is because, as the pozzolanic sand gets finer it has neither friction nor cohesion for resistance. Similar results were observed by (Bel, 1996) in which excessive addition of stabilizer reduced compressive strength. As noted from Figure 4-10, the compressive strength of soil blended with PS<75µm increased until 7.5% PS content is reached beyond which the strength continues to increase at a slower rate or begins to decline. This may be due to non-homogeneity of the soil-PS mixture which resulted from non-uniform distribution of PS.

But for pozzolanic sand crushed to a particle size less than 150µm, increases the unconfined compressive strength of the blended soil with the increasing percentage of pozzolanic sand. The reason for increasing the compressive strength is that the coarser material that replaced the finer clay particles gives frictional resistance to the soil grain structure.

In general, pozzolanic sand crushed to a particle size less than 75µm improved the compressive strength of the blended soil at a lower percentage of pozzolanic sand as compared to the pozzolanic sand pulverized to a particle size less than 150µm.

4.7 Effect of Pozzolanic sand on Swelling Pressure of Expansive Soil

Swelling is one of the physical properties which determines the severity of expansive soil. Soils which tends to swell exerts pressure on building foundations, pavements and airfields when the soil is in contact with moisture which in turn results in uplifting of structures. These same soils also tend to shrink when there is a loss of moisture through drying. Upon losing moisture the soil starts to shrink which in turn results in settlement of the structure as moisture loss is a major reason for soil to shrink.

In this particular research, investigating the swelling pressure of the natural soil blended with pozzolanic sand is the major objective. Samples for the determination of swelling pressure were prepared by compacting the blended soil to maximum dry density with optimum moisture content in a consolidation ring. Those samples were allowed to swell in an oedometer under a light surcharge load of 7kPa by giving access to water or submerging in water for 72hrs or more until swelling ceases. In this particular work, 72hrs for swelling was used and followed by incremental loading as in conventional consolidation test until the samples get compressed to its initial volume/void ratio or less. The reduction in swelling pressure of the soil with various percentage of pozzolanic sand is indicated in Figure 4-11. As depicted in the figure, the swelling pressure decreases as the percentage of the pozzolanic sand increases for sizes of the pozzolanic sand considered in both cases.

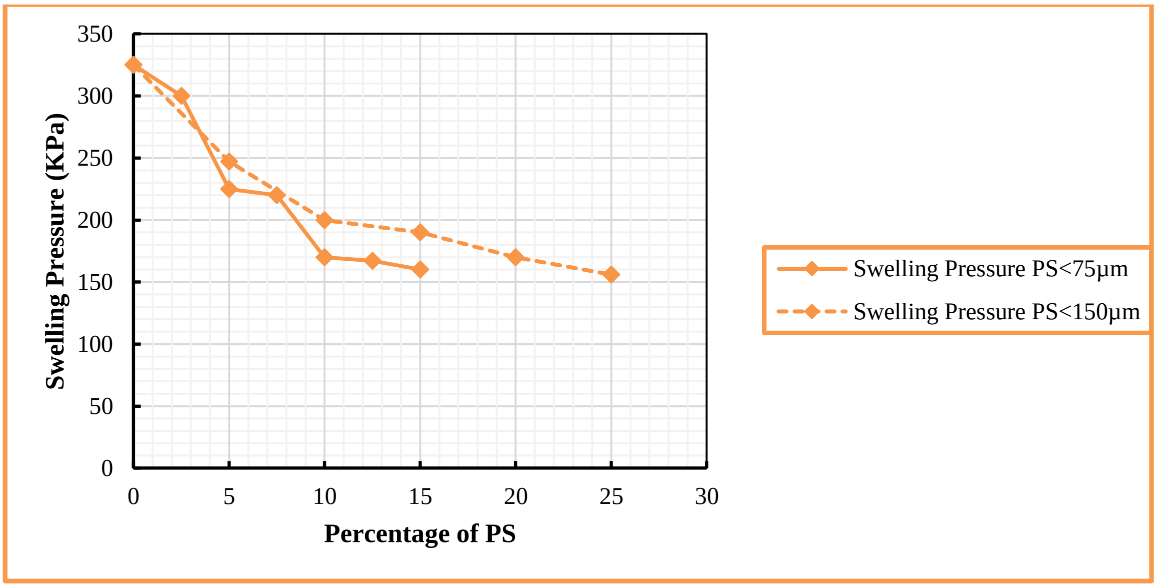


Figure 4-11: Effect of pozzolanic sand on the swelling pressure

For the size of pozzolanic sand crushed to a size of less than 75μm, the swelling pressure of blended soil decreases. The decrease is because the finer pozzolanic sand replaces clays that exists and which gives the soil the tendency to swell. The swelling pressure and swelling potential also depend on the amount of clay that presents in the expansive soil (Tefera & Leikun, 1999), (Chen, 1975). As the pozzolanic sand gets finer and blended with expansive soil it replaces those clay particles which exists in the soil. The chemical reaction between the expansive soil and pozzolanic sand also has an immediate effect in reducing the swelling pressure of the soil. (Ömür Çimen, 2015), observed similar results on stabilization of clay with pumice. As depicted in the figure, reduction in swelling pressure is more significant when the expansive soil is blended with pozzolanic sand crushed to a particle size less than 75μm as compared to expansive soil blended with PS<150μm.

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Based on the experimental study on treatment of expansive soil by blending with pozzolanic sand the following conclusions were drawn.

1. The plasticity of the expansive soil decreased when blended with an increasing percentage of pozzolanic sand. The plasticity index of the natural soil was observed to decrease from 62.8% to 31.6% when 15% by dry weight of expansive soil is replaced with pozzolanic sand crushed to a particle size less than 75μm. In the case of replacing a portion of expansive soil with 25% pozzolanic sand crushed to a particle size less than 150μm, the plasticity index was decreased from 62.8% to 40.74% which results in 35% reduction in plasticity index of the expansive soil. Upon curing the soil-sand mixture the plasticity index reduced significantly which reduced the plasticity index by 55% and 46% for addition of 15% and 25% pozzolanic sand crushed to a particle size less than 75μm and 150μm, respectively. Based on this the pozzolanic sand crushed to a particle size less than 75μm improves the workability of the expansive soil, which indicates the pozzolanic sand crushed to a particle size less than 75μm can be used to improve the plasticity of expansive soil.
2. Pozzolanic sand improves the swelling characteristics such as free swell of the expansive soil upon blending. The free swell reduced from 200% to 80% when 15% of the expansive soil was replaced with pozzolanic sand crushed to a particle size less than 75μm, which results in 60% in reduction of free swell value of the expansive. But for pozzolanic sand crushed to a particle size less than 150μm the percentage reduction in free swell amounts 42.5%, that is the free swell reduced from 200% to 115% when 25% of the expansive soil was replaced with the pozzolanic sand. As observed here, the pozzolanic sand crushed to a particle size less than 75μm was effective in reducing the free swell of the expansive soil as it partially replaces clay minerals and chemically react with those clay minerals.
3. The optimum moisture content decreased while the maximum dry density increased with an increasing percentage of pozzolanic sand. But the decrease in

optimum moisture content and increase in dry density is more significant for the pozzolanic sand crushed to a particle size less than 150μm than that crushed to a size less than 75μm.

4. The unconfined compressive strength of the expansive soil increased when the pozzolanic sand was blended with native soil. But the increase in unconfined compressive strength is more pronounced for soil blended with pozzolanic sand crushed to a particle size less than 75μm than that of crushed to a particle size less than 150μm. For instance, the unconfined compressive strength of the expansive soil increased by 52% when 10% pozzolanic sand with size less than 75μm was blended to the expansive soil than blending the same percentage of pozzolanic sand crushed to a particle size less than 150μm to the expansive soil, which increased the unconfined compressive strength of the expansive soil by 18%. The compressive strength starts to decrease when the percentage of pozzolanic sand by dry weight of expansive soil exceeded 7.5% which increased the unconfined compressive strength of the expansive soil by 89%, this may give clue for optimizing pozzolanic sand for increasing the compressive strength of the expansive soil. Curing also increases the unconfined compressive strength of the blended soil.
5. The swelling pressure of the expansive soil decreases when the percentage of the pozzolanic sand increases upon blending. The decrease in swelling pressure is more noticeable when pozzolanic sand crushed to a particle size of less than 75μm is used, which indicates that sizes of additives affect the swelling characteristics of the expansive soil.
6. From the above, the pozzolanic sand crushed to a particle size less than 75μm improves the engineering properties of expansive soil than pozzolanic sand crushed to a size less than 150μm. In general, the pozzolanic sand crushed to a particle size of less than 75μm can be used as a treatment for improving the engineering properties of expansive soil.

5.2 Recommendation

.The fineness of the pozzolanic sand affects the engineering properties of expansive soil as obtained by comparing engineering properties of soil blended with pozzolanic sand crushed to a particle size less than 75µm and 150µm. Based on this the following topics are recommended for future studies.

- Improving the engineering properties of expansive soil by blending with pozzolanic sand crushed to a particle size finer than considered in this research.

Besides, a combination of pozzolanic sand with other stabilization additives containing lime can also be used to improve the expansive soil, because pozzolanas react with lime to form cementitious compounds which are suitable for soil improvements. Based on this, the following can be considered for further study

- Using pozzolanic sand in combination with lime for improving the engineering properties of expansive soil.
- Using pozzolanic sand in combination with cement for improving the engineering properties of expansive soil.
- Using pozzolanic sand in combination with marble dust for improving the engineering properties of expansive soil.
- Using pozzolanic sand in combination with cement kiln dust for improving the engineering properties of expansive soil.
- Using pozzolanic sand in combination with ceramic powders for improving the engineering properties of expansive soil.

REFERENCES

Al-Rawas, A. A., & Goosen, M. F. (2006). *Expansive Soils; Recent Advances in Characterization and Treatment.* LONDON / LEIDEN / NEW YORK / PHILADELPHIA / SINGAPORE: Taylor & Francis Group.

Al-Swaidani, A., Hammoud, I., & Meziab, A. (2016). Effect of adding natural pozzolana on geotechnical properties of lime-stabilized clayey soil. *Journal of Rock Mechanics and Geotechnical Engineering*. doi:10.1016/j.jrmge.2016.04.002

Barton, C. D., & Karathanasis, A. D. (2002). Clay minerals. *Encyclopedia of soil science*, 187-192.

Baser, O. (2009). *Stabilization of Expansive soil Using waste Marble Dust.* MSc. Thesis Submitted to the Graduate School of Natural and Applied Science of Middle East Technical University.

Bel, F. (1996). Lime stabilization of clay minerals and soils. *Engineering Geology, 42*, 223-237.

Birhane, D. (2018). *Investigation on Improving the Geotechnical Properties of Black Cotton Soil by blending with pumice.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Chen, F. H. (1975). *Foundation on Expansive Soil.* Amsterdam: ELSEVIER SCIENTIFIC PUBLISHING COMPANY,INC.

Dang, L. C., Hasan, H., Fatahi, B., Jones, R., & Khabbaz, H. (2016). Enhancing the Engineering Properties of Expansive Soil Using Bagasse Ash and Hydrated Lime. *International Journal of Geomate, 11*(25), 2447-2454.

Desta, S. K. (2003). *Utilization of Ethiopian Natural Pozzolans.* Trondheim: Norwegian University of Science and Technology, Faculty of Engineering Science and Technology, Department of Structural Engineering.

Fikadu, A. (2015). *Relationship between Swelling and Consolidation characteristics of Expansive Soils of Gelan Town.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Fusheng Zha, S. L. (2008). Behavior of expansive soil stabilized with fly ash. *Natural Hazards, 47*, 509-523.

Harichane, K., Ghrici, M., Kenai, S., & Khaled, G. (2011). Use of Natural Pozzolana and Lime for Stabilization of Cohesive Soils. *Geotechnical and Geological Engineering*(29), 759–769.

K.M.A. Hossain*, M. L. (2007). Stabilized soils for construction applications incorporating natural resources. *Resources, Conservation and Recycling, 51*, 711-731.

Lemi, A. (2015). *Stabilization of Expansive Clay Soil Using Potassium Chloride.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Mada, T. (2016). *Application of Marble Dust to Improve the Engineering Properties of Expansive Soils to be Used as Road Bedding Material.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Makusa, G. P. (2012). *Soil Stabilization Methods and Materials, State of the art review*. Lulea University of Technology, Department of Civil, Environmental and Natural Resources Engineering, Division of Mining and Geotechnical Engineering, Lulea, Sweden.

Malhotra, V. M., & Mehta, P. K. (1996). *Pozzolanic and cementtious materials* (Vol. 1). (V. M. Malhotra, Ed.) Taylor & Francis Group.

Mir, B. A. (2015). Challenges Associated with Expansive Soils and Remedial Measures. *50th Indian Geotechnical Conference.* Pune, Maharashtra, India.

Mukherjee, S. (2013). *The Science of Clays, Application in industry, Engineering and Environment.* New Delhi, India: Capital Publishing Company.

Murray, H. H. (2007). *Applied Clay Mineralogy, Occurrences, Processing and Application of Kaolins, Bentonites, Palygorskite-Sepiolite, and Common Clays.* Amsterdam: Elsevier.

Nelson, J. D., & Miller, D. J. (1992). *Expansive soils Problems and Practice in Foundation and Pavement Engineering.* New York: JOHN WILEY & SONS, INC.

Nelson, J. D., Chao, K. C., Overton, D. D., & Nelson, E. J. (2015). *Foundation Engineering for Expansive Soils.* Hoboken, New Jersey: John Wiley & Sons, Inc.

Ömür Çimen, M. S. (2015). Stabilization of clayey subgrade with waste pumice for road infrastructure. *Science and Engineering of Composite Materials, 22*(5), 583-590.

Rahman, M. A. (1986). The potentials of some stabilizers for the use of lateritic soil in construction. *Building and Environment, 21*(1), 57-61.

Reddy, N. G., Tahasildar, J., & Rao, B. H. (2015). Evaluating the Influence of Additives on Swelling Characteristics of Expansive soil. *International Journal of Geosynthesis and Ground Engineering, 1*(7), 1-13. doi:10.1007/s40891-015-0010-x

Rogers, J. D., Olshansky, R., & Rogers, R. B. (2016). Damage to Foundations from Expansive soils.

Simon, H. (2016). *An Alternative for Stabilization of Addis Ababa Expansive Soil by Crushed and Natural Sand.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Sisay, A. (2004). *Assessment of Damage of buildings constructed in expansive soil areas of Addis Ababa.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

Tefera, A., & Leikun, M. (1999). *Soil Mechanics.* Faculty of Technology, Addis Ababa University.

Tesema, G. (2016). *Stabilization of Expansive soils by Sugarcane Molasses.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

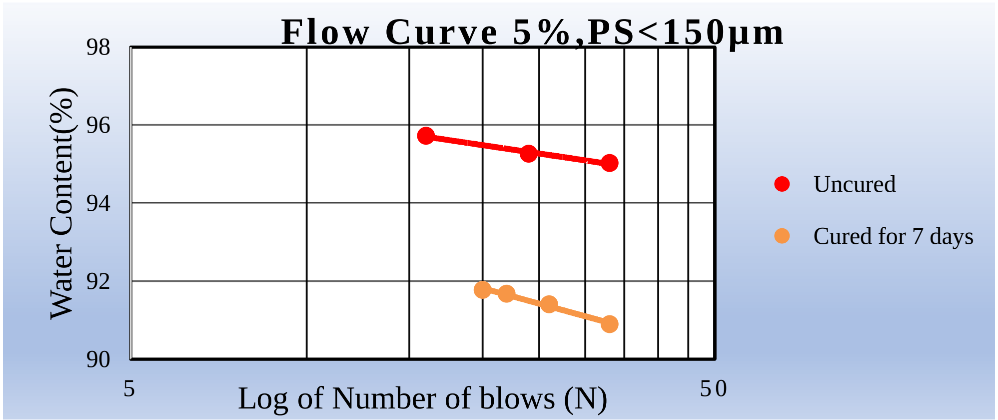
Worku, B. (2015). *Stabilization of Lateritic Soil Using Sugarcane Bagasse Ash.* MSc. Thesis, School of Graduate Studies, Addis Ababa University.

APPENDIX A: ATTERBERG LIMITS FOR BLENDED SOIL

A-1 Atterberg limits for soil blended with pozzolanic sand crushed to a particle size less than 150µm

A. 5% pozzolanic sand

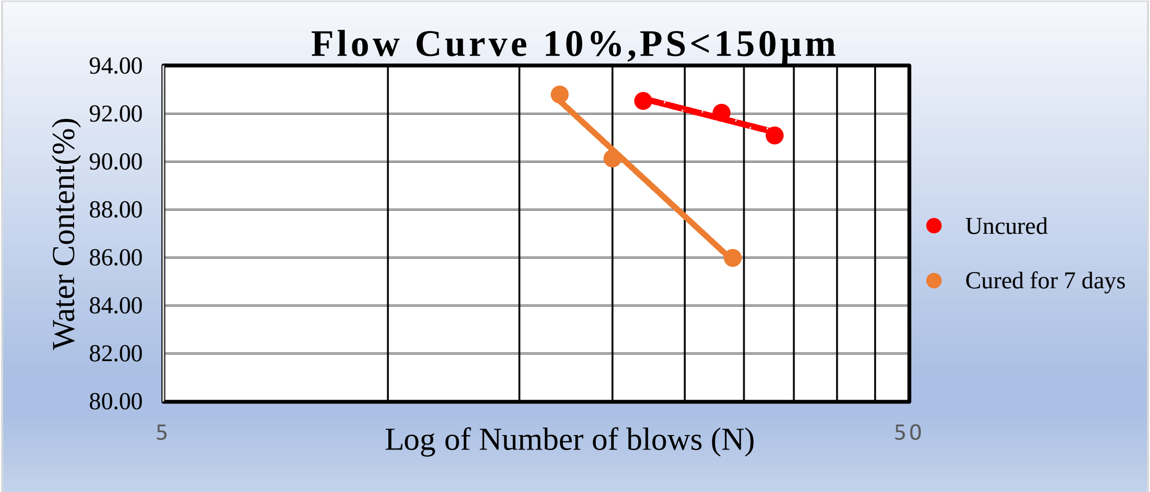
Liquid Limit/Measurement & calculations	Uncured			Cured for 7 Days			
Trial No	**1**	**2**	**3**	**1**	**2**	**3**	**4**
Can No	13	P-3	LL-3	W_5	PL-2	12	D_{32}
Mass of Can, g	15.62	15.73	15.8	15.536	15.813	20.685	20.675
Mass of Can + Moist Soil, g	37.58	41.7	40.9	33.9	34.228	37.886	39.576
Number of Blows	**16**	**24**	**33.0**	**33**	**26**	**22**	**20**
Mass of Can + Dry Soil, g	26.84	29.03	28.6	25.156	25.434	29.659	30.531
Mass of Water, g	10.74	12.67	12.2	8.744	8.794	8.227	9.045
Mass of Dry soil, g	11.22	13.3	12.9	9.62	9.621	8.974	9.856
Moisture Content, w (%)	**95.72**	**95.26**	**95.03**	**90.89**	**91.40**	**91.68**	**91.77**



Plastic Limit/ Measurement & Calculation	Uncured			Cured for 7 days		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	W_5	W_o	51	K_6	L_1	10
Mass of empty can, g	15.528	15.611	15.668	15.211	15.668	15.62
Mass of can + Wet soil, g	19.909	22.02	21.515	19.821	19.119	18.661
Mass of can + Oven dry soil,	18.733	20.223	19.936	18.449	18.042	17.731
Water content,%	36.69	38.96	37.00	42.37	45.37	44.05
Plastic Limit,%	**38**			**44**		
Liquid Limit,%	**95.25**			**91.40**		
Plasticity index,%	**57.7**			**47.4**		

B. 10% pozzolanic sand

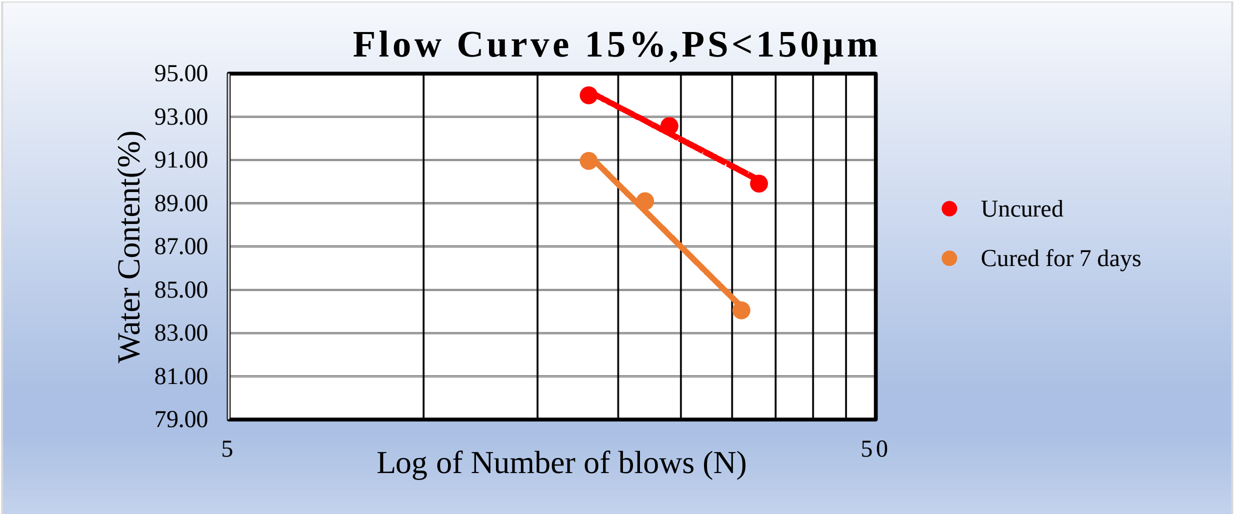
Liquid Limit/Measurement & calculations	**Uncured**			**Cured for 7 days**		
Trial No	1	2	3	1	2	3
Can No	12	W_9	4	10(8)	W_{10}	17
Mass of Can, g	16.08	15.73	15.5	15.683	15.826	15.784
Mass of Can + Moist Soil, g	40.55	35.01	41.43	32.101	31.323	36.401
Number of Blows	**22**	**28**	**33**	**17**	**20**	**29**
Mass of Can + Dry Soil, g	28.79	25.77	29.07	24.199	23.977	26.869
Mass of Water, g	11.76	9.24	12.36	7.902	7.346	9.532
Mass of Dry soil, g	12.71	10.04	13.57	8.516	8.151	11.085
Moisture Content, w (%)	**92.53**	**92.03**	**91.08**	**92.79**	**90.12**	**85.99**



Plastic Limit/Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	101	52	PL-2	W_5	54	W_{30}
Mass of empty can, g	15.96	15.71	20.67	15.54	15.67	15.44
Mass of can + Wet soil, g	21.60	20.85	25.43	18.89	20.94	20.06
Mass of can + Oven dry soil, g	20.01	19.42	24.10	17.80	19.22	18.52
Water content,%	39.41	38.38	38.79	48.56	48.40	50.05
Plastic Limit,%	**38.9**			**49.00**		
Liquid Limit,%	**92.15**			**87.8**		
Plasticity index,%	**53.25**			**38.80**		

C. 15% pozzolanic sand

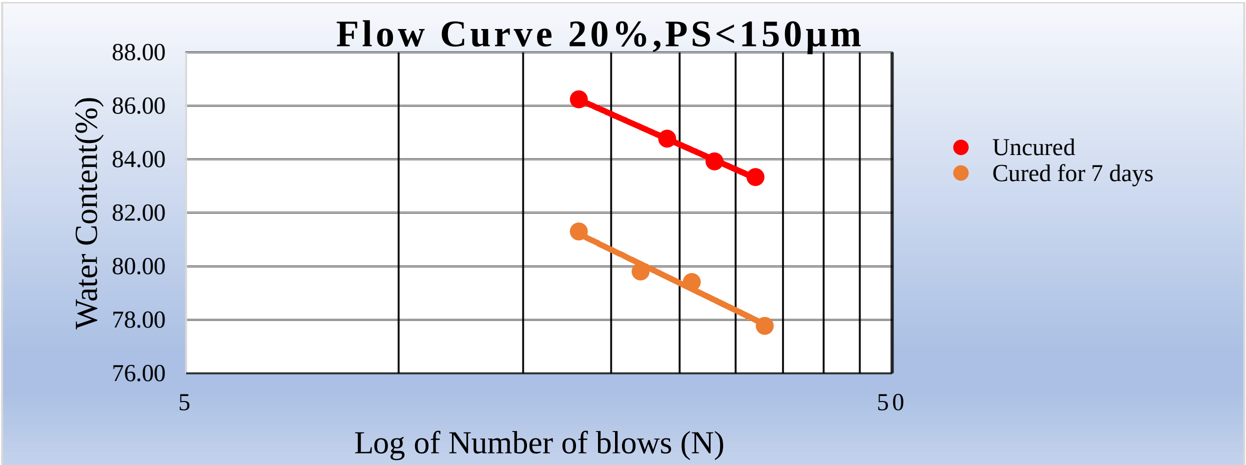
Liquid Limit/Measurement & calculations	**Uncured**			**Cured for 7 days**		
Trial No	1	2	3	1	2	3
Can No	51	101	33	101	W_{14}	W_{15}
Mass of Can, g	15.67	15.962	15.485	15.962	15.698	15.56
Mass of Can + Moist Soil, g	33.675	36.082	34.51	32.169	31.015	34.212
Number of Blows	**33**	**24**	**18**	**31**	**22**	**18**
Mass of Can + Dry Soil, g	25.151	26.41	25.292	24.768	23.798	25.328
Mass of Water, g	8.524	9.672	9.218	7.401	7.217	8.884
Mass of Dry soil, g	9.481	10.448	9.807	8.806	8.1	9.768
Moisture Content, w (%)	**89.91**	**92.57**	**93.99**	**84.04**	**89.10**	**90.95**



Plastic Limit/ Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	W_{21}	W_{32}	W_{37}	W_{31}	Co-14	W_2
Mass of empty can, g	15.94	15.59	15.65	16.01	15.97	15.42
Mass of can + Wet soil, g	21.79	23.39	19.33	19.68	19.30	20.80
Mass of can + Oven dry soil, g	20.13	21.20	18.30	18.37	18.17	18.95
Water content, %	39.49	38.83	38.74	55.20	51.16	52.32
Plastic Limit, %	**39.02**			**52.89**		
Liquid Limit, %	**91.80**			**87.00**		
Plasticity index, %	**52.78**			**34.11**		

D. 20% pozzolanic sand

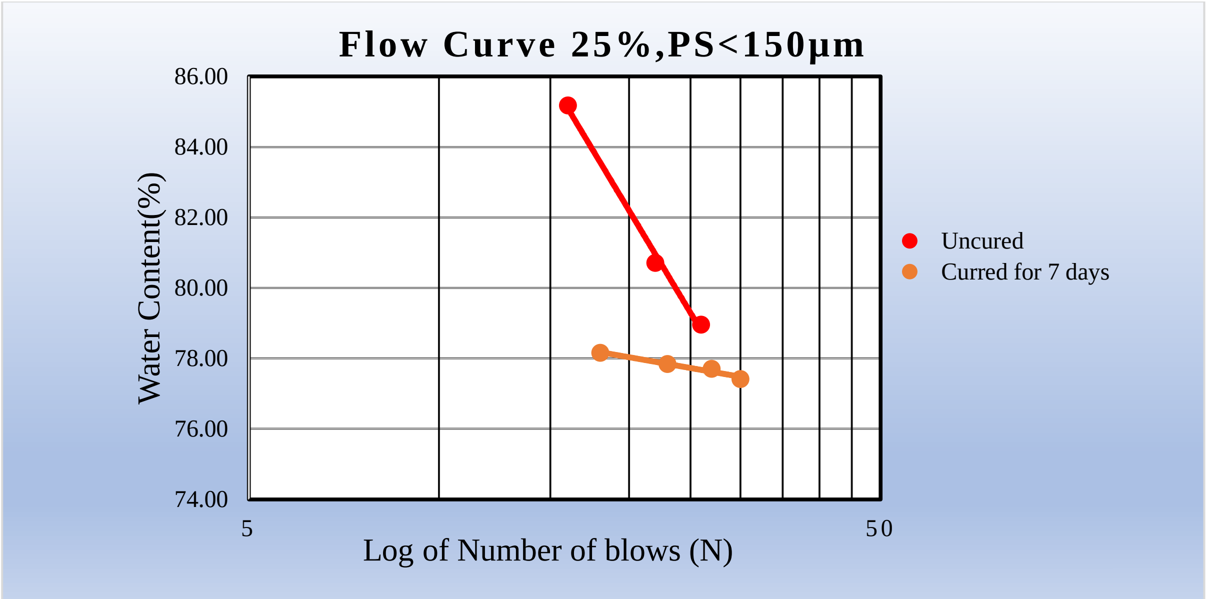
Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	1	2	3	4	1	2	3	4
Can No	60	Co-9	Co-11	D	61	Co-11	W_{13}	W_{11}
Mass of Can, g	15.848	15.86	15.831	15.719	15.818	15.839	15.282	15.564
Mass of Can + Moist Soil, g	36.764	33.98	35.772	36.023	31.9977	32.6	32.665	31.873
Number of Blows	**18**	**24**	**28**	**32**	**18**	**22**	**24**	**33**
Mass of Can + Dry Soil, g	27.079	25.667	26.673	26.794	24.742	25.161	24.971	24.738
Mass of Water, g	9.685	8.313	9.099	9.229	7.25572	7.439	7.694	7.135
Mass of Dry soil, g	11.231	9.807	10.842	11.075	8.924	9.322	9.689	9.174
Moisture Content, w (%)	**86.23**	**84.77**	**83.92**	**83.33**	**81.31**	**79.80**	**79.41**	**77.77**



Plastic Limit/ Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	W_{35}	W_{22}	W_{28}	Co-4	W_{25}	W_{26}
Mass of empty can, g	15.60	15.69	15.75	15.91	15.70	15.79
Mass of can + Wet soil, g	21.55	21.89	19.73	20.73	21.24	20.39
Mass of can + Oven dry soil, g	19.98	20.25	18.68	19.18	19.42	19.01
Water content, %	35.96	35.93	36.10	47.29	48.90	42.82
Plastic Limit, %	**36.00**			**46.34**		
Liquid Limit, %	**84.60**			**79.15**		
Plasticity Index, %	**48.60**			**32.81**		

E. 25% pozzolanic sand

Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**
Can No	D	Co-9	16	60	W_{16}	60	W_{17}	50
Mass of Can, g	15.721	15.863	15.81	15.879	14.593	15.891	15.904	15.586
Mass of Can + Moist Soil, g	33.628	36.768	36.848	36.198	32.086	34.166	32.509	32.871
Number of Blows	**31**	**26**	**22**	**16**	**30**	**27**	**23**	**18**
Mass of Can + Dry Soil, g	25.856	27.545	27.452	26.852	24.453	26.175	25.241	25.288
Mass of Water, g	7.772	9.223	9.396	9.346	7.633	7.991	7.268	7.583
Mass of Dry soil, g	10.135	11.682	11.642	10.973	9.86	10.284	9.337	9.702
Moisture Content, w (%)	**76.68**	**78.95**	**80.71**	**85.17**	**77.41**	**77.70**	**77.84**	**78.16**

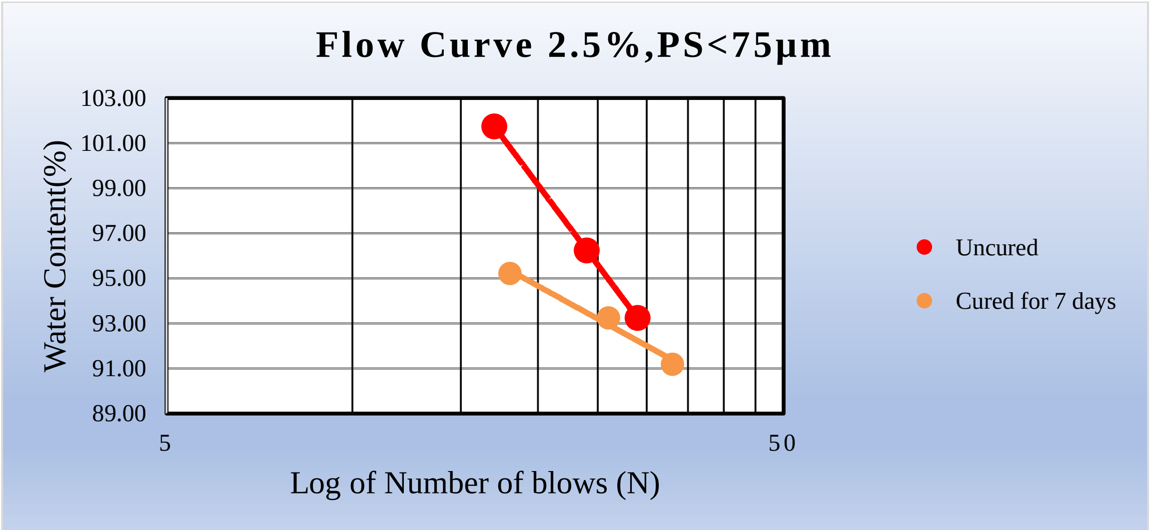


Plastic Limit/ Measurement &Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	W_{25}	W_{26}	W_{34}	W_{22}	W_{34}	W_{6}
Mass of empty can, g	15.71	15.80	15.46	15.67	15.46	14.45
Mass of can + Wet soil, g	21.00	22.58	20.30	18.33	21.26	18.09
Mass of can + Oven dry soil, g	19.61	20.68	18.89	17.51	19.51	16.99
Water content, %	35.62	38.91	41.00	44.39	43.32	43.55
Plastic Limit, %	**38.51**			**43.75**		
Liquid Limit, %	**79.25**			**77.70**		
Plasticity Index, %	**40.74**			**33.95**		

A-2 Atterberg limits for soil blended with pozzolanic sand crushed to a particle size less than 75µm

A. 2.5 % pozzolanic sand

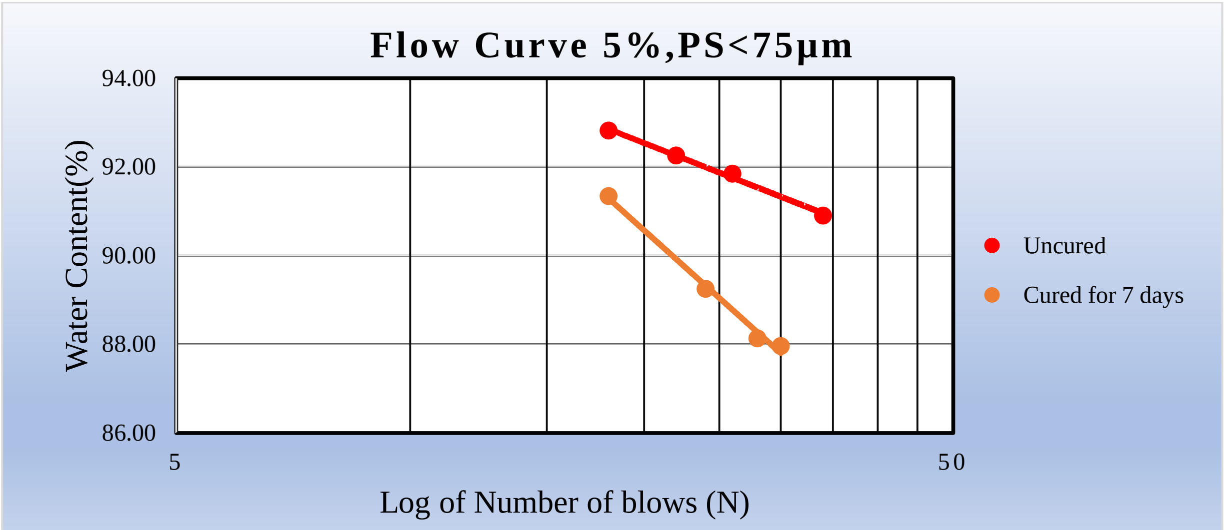
Liquid Limit/Measurement & calculations	**Uncured**			**Cured for 7 days**		
Trial No	1	2	3	1	2	3
Can No	752	753	100	7511	W_{29}	W_0
Mass of Can, g	15.639	15.8	15.837	15.665	15.243	15.656
Mass of Can + Moist Soil, g	27.902	33.0	33.616	33.163	31.916	33.992
Number of Blows	**29**	**24**	**17**	**33**	**26**	**18**
Mass of Can + Oven dry soil, g	21.985	24.6	24.65	24.817	23.8705	25.048
Mass of Water, g	5.917	8.5	9.0	8.346	8.04547	8.944
Mass of Dry soil, g	6.346	8.8	8.8	9.152	8.62753	9.392
Moisture Content, w (%)	**93.24**	**96.23**	**101.74**	**91.19**	**93.25**	**95.23**



Plastic Limit/ Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	751	752	753	758	759	760
Mass of empty can, g	14.632	15.639	15.787	15.26	15.68	15.78
Mass of can + Wet soil, g	18.872	20.286	19.896	18.578	18.585	18.454
Mass of can + Oven dry soil, g	17.576	18.793	18.616	17.645	17.757	17.697
Water content, %	44.02	47.34	45.25	39.12	39.87	39.49
Plastic Limit, %	**45.5**			**39.5**		
Liquid Limit, %	**95.8**			**93.3**		
Plasticity index	**50.3**			**53.8**		

B. 5% pozzolanic sand

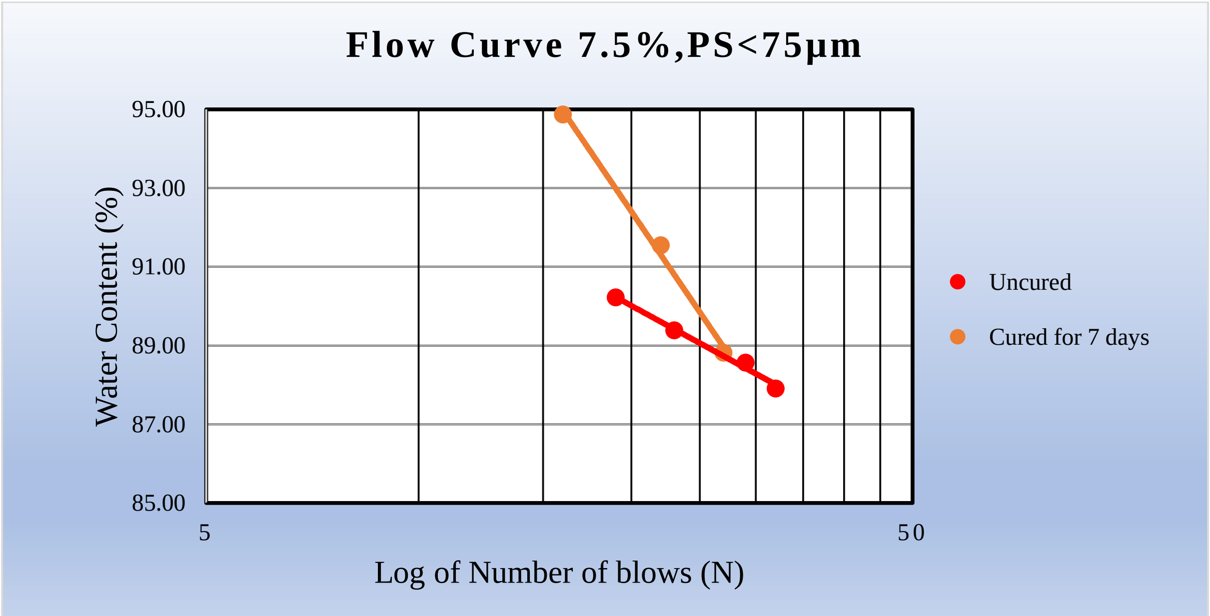
Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	1	2	3	4	1	2	3	4
Can No	W_{12}	W_2	W_{20}	754	75	68	CO-4	W_{21}
Mass of Can, g	15.839	15.37	13.883	15.618	15.82	15.973	15.652	15.916
Mass of Can + Moist Soil, g	29.08	31.105	28.2	30.322	31.084	34.615	36.481	36.579
Number of Blows	**34**	**26**	**22**	**18**	**30**	**28**	**24**	**18**
Mass of Can + Dry Soil, g	22.775	23.572	21.33	23.244	23.941	25.882	26.658	26.715
Mass of Water, g	6.305	7.533	6.87	7.078	7.143	8.733	9.823	9.864
Mass of Dry soil, g	6.936	8.202	7.447	7.626	8.121	9.909	11.006	10.799
Moisture Content, w (%)	**90.90**	**91.84**	**92.25**	**92.81**	**87.96**	**88.13**	**89.25**	**91.34**



Plastic Limit/ Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	100	W_{12}	W_{20}	W_{12}	756	W_{13}
Mass of empty can, g	15.837	15.839	13.883	15.866	15.976	15.67
Mass of can + Wet soil, g	20.216	19.466	18.491	19.802	19.677	19.49
Mass of can + Oven dry soil, g	18.915	18.418	17.089	18.592	18.585	18.339
Water content, %	42.27	40.64	43.73	44.39	41.86	43.12
Plastic Limit, %	**42.2**			**43.1**		
Liquid Limit, %	**91.8**			**89.10**		
Plasticity Index, %	**49.6**			**46**		

C. 7.5 % pozzolanic sand

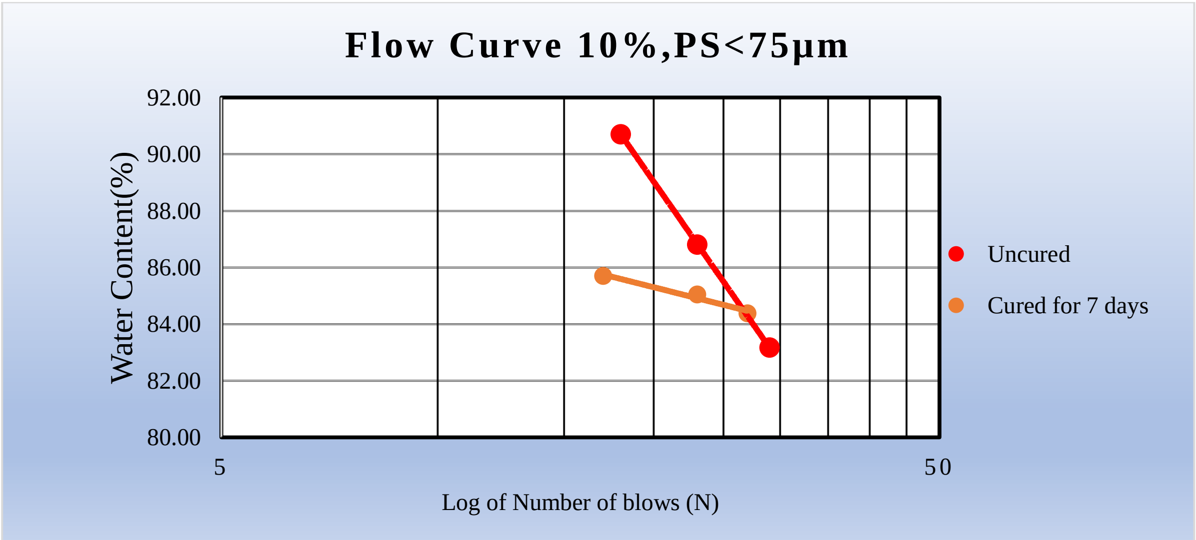
Liquid Limit/Measurement & calculations	Uncured				Cured for 7 days		
Trial No	1	2	3	4	1	2	3
Can No	755	W_{19}	756	W_1	22	W_{25}	C-5
Mass of Can, g	15.664	15.557	15.848	15.602	15.644	15.641	16.024
Mass of Can + Moist Soil, g	34.707	32.803	31.8806	29.96	31.38	29.737	33.747
Number of Blows	**32**	**29**	**23**	**19**	**27**	**22**	**16**
Mass of Can + Dry Soil, g	25.798	24.703	24.3136	23.15	23.978	23	25.119
Mass of Water, g	8.909	8.1	7.567	6.81	7.402	6.737	8.628
Mass of Dry soil, g	10.134	9.146	8.4656	7.548	8.334	7.359	9.095
Moisture Content, w (%)	**87.91**	**88.56**	**89.39**	**90.22**	**88.82**	**91.55**	**94.87**



Plastic Limit/Measurement & Calculation	Uncured			Cured for 7 days		
Trial No	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	754	W_2	755	757	755	756
Mass of empty can, g	15.618	15.37	15.815	15.744	15.66	15.85
Mass of can + Wet soil, g	19.418	19.835	20.856	21.251	19.224	20.947
Mass of can + Oven dry soil, g	18.23	18.417	19.246	19.402	18.023	19.234
Water content, %	45.48	46.54	46.93	50.55	50.83	50.62
Plastic Limit, %	**46.32**			**51**		
Liquid Limit, %	**89.00**			**89.5**		
Plasticity index, %	**42.7**			**38.5**		

D. 10% pozzolanic sand

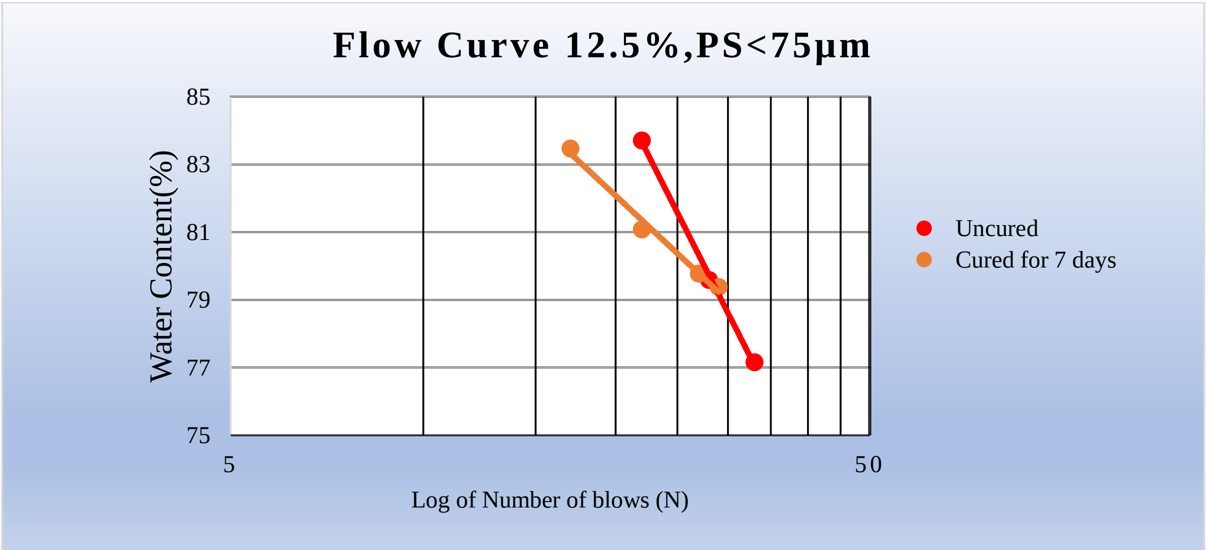
Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	1	2	3	4	1	2	3	4
Can No	759	758	757	22	66	L_1	67	W_5
Mass of Can, g	15.648	15.213	15.701	15.622	15.869	15.647	15.631	15.527
Mass of Can + Moist Soil, g	32.95	31.858	30.99	33.306	32.67	36.945	29.819	39.773
Number of Blows	**34**	**29**	**23**	**18**	**33**	**27**	**23**	**17**
Mass of Can + Dry Soil, g	25.155	24.3	23.885	24.895	25.022	27.198	23.298	28.583
Mass of Water, g	7.795	7.558	7.105	8.411	7.648	9.747	6.521	11.19
Mass of Dry soil, g	9.507	9.087	8.184	9.273	9.153	11.551	7.667	13.056
Moisture Content, w (%)	**81.99**	**83.17**	**86.82**	**90.70**	**83.56**	**84.38**	**85.05**	**85.71**



Plastic Limit/Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial No	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	756	w19	w1	5	P5	P6
Mass of empty can, g	15.933	15.557	15.602	15.1	15.782	15.687
Mass of can + Wet soil, g	20.522	20.486	19.887	17.958	22.398	21.875
Mass of can + Oven dry soil, g	19.096	18.926	18.547	16.989	20.055	19.785
Water content, g	45.08	46.30	45.50	51.30	54.83	51.00
Plastic limit, %	**45.6**			**52.4**		
Liquid limit, %	**85.8**			**84.5**		
Plasticity index, %	**40.2**			**32.1**		

E. 12.5% pozzolanic sand

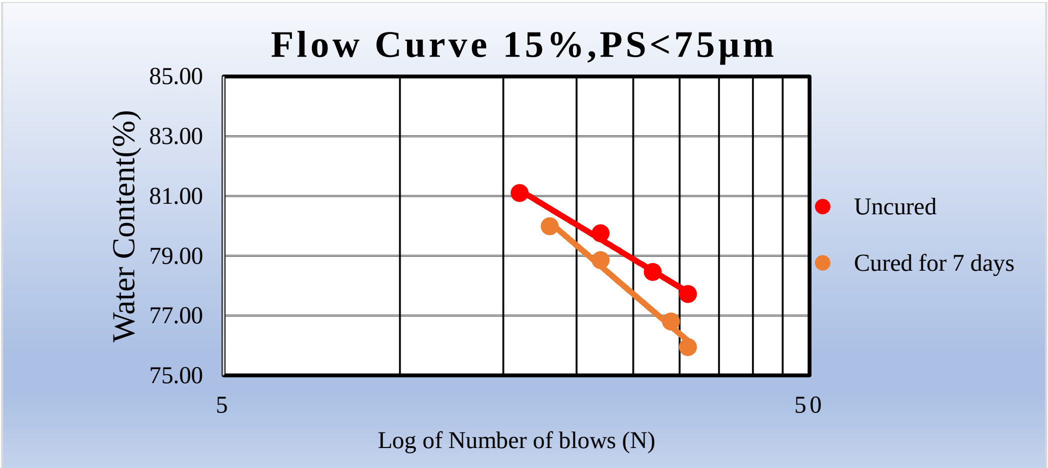
Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**
Can No	7510	7511	Co-4	W_6	7510	W_2	65	754
Mass of Can, g	15.596	15.647	15.775	14.437	15.589	15.347	15.705	15.604
Mass of Can + Moist Soil, g	32.144	32.183	31.765	32.77	34.393	29.058	33.95	33.619
Number of Blows	**33**	**28**	**22**	**20**	**29**	**27**	**22**	**17**
Mass of Can + Dry Soil, g	24.937	24.855	24.479	24.378	26.072	22.974	25.781	25.423
Mass of Water, g	7.207	7.328	7.286	8.392	8.321	6.084	8.169	8.196
Mass of Dry soil, g	9.341	9.208	8.704	9.941	10.483	7.627	10.076	9.819
Moisture Content, w (%)	**77.15**	**79.58**	**83.71**	**84.42**	**79.38**	**79.77**	**81.07**	**83.47**



Plastic limit/Measurement & Calculation	**Uncured**			**Cured for 7 days**		
Trial No	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	757	22	758	S_3	W_{24}	W_{12}
Mass of empty can, g	15.701	15.622	15.213	13.883	15.494	15.645
Mass of can + Wet soil, g	19.299	20.731	19.915	17.213	19.63	18.782
Mass of can + Oven dry soil, g	18.201	19.203	18.25	16.107	18.238	17.734
Water content, %	43.92	42.67	54.82	49.73	50.73	50.17
Plastic limit, %	**47.1**			**50.2**		
Liquid limit, %	**81.0**			**80.4**		
Plasticity index, %	**33.9**			**30.2**		

F. 15% pozzolanic sand

Liquid Limit/Measurement & calculations	**Uncured**				**Cured for 7 days**			
Trial No	1	2	3	4	1	2	3	4
Can No	W_{24}	W_{22}	18	W_{25}	W_6	13	W_7	18
Mass of Can, g	15.495	15.651	15.926	15.657	14.454	15.572	15.822	15.951
Mass of Can + Moist Soil, g	32.389	31.882	31.823	32.762	29.598	31.945	33.332	32.916
Number of Blows	**31**	**27**	**22**	**16**	**31**	**29**	**22**	**18**
Mass of Can + Dry Soil, g	25.001	24.746	24.77	25.102	23.061	24.833	25.612	25.377
Mass of Water, g	7.388	7.136	7.053	7.66	6.537	7.112	7.72	7.539
Mass of Dry soil, g	9.506	9.095	8.844	9.445	8.607	9.261	9.79	9.426
Moisture Content, w (%)	**77.72**	**78.46**	**79.75**	**81.10**	**75.95**	**76.80**	**78.86**	**79.98**



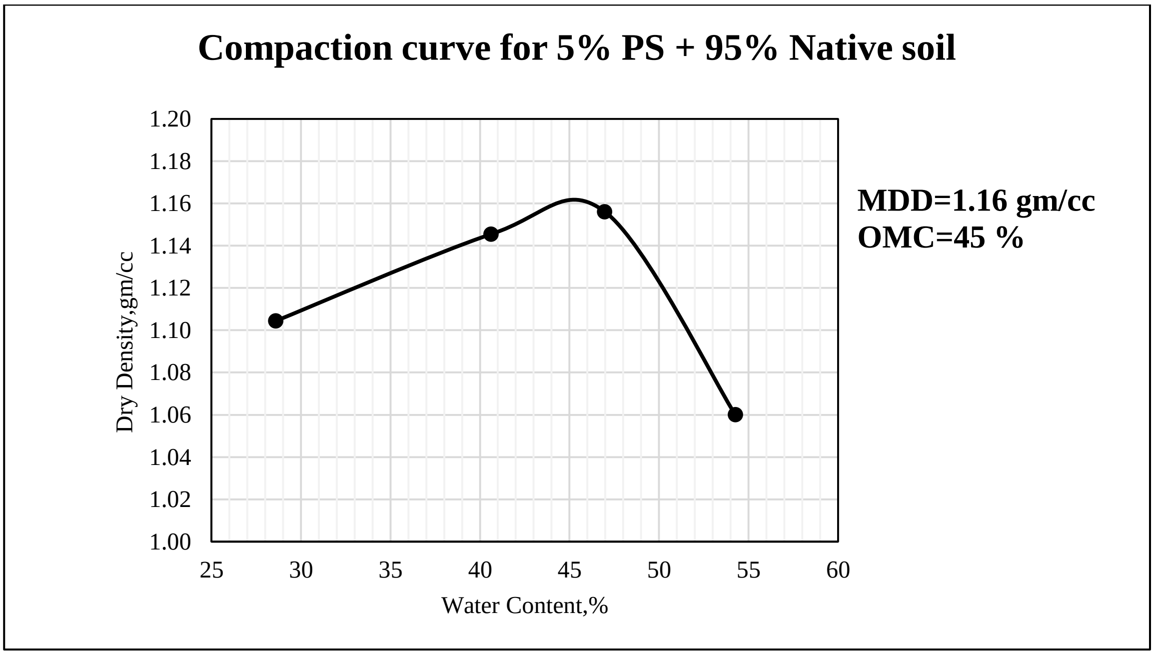
Plastic limit/Measurement & Calculation	**Uncured**			Cured for 7 days		
Trial No	**1**	**2**	**3**	**1**	**2**	**3**
Can Number	759	7510	7511	12	W_{20}	W_{19}
Mass of empty can, g	15.648	15.596	15.647	15.669	13.9	15.879
Mass of can + Wet soil, g	18.876	23.26	22.938	18.956	17.453	19.705
Mass of can + Oven dry soil, g	17.851	20.764	20.615	17.866	16.285	18.442
Water content, %	46.53	48.30	46.76	49.61	48.97	49.28
Plastic limit, %	**47.2**			**49.29**		
Liquid limit, %	**78.80**			**77.60**		
Plasticity index, %	**31.60**			**28.31**		

APPENDIX B: COMPACTION CURVE FOR BLENDED SOIL

B-1: Compaction curve for soil blended with pozzolanic sand crushed to a particle size less than 150µm

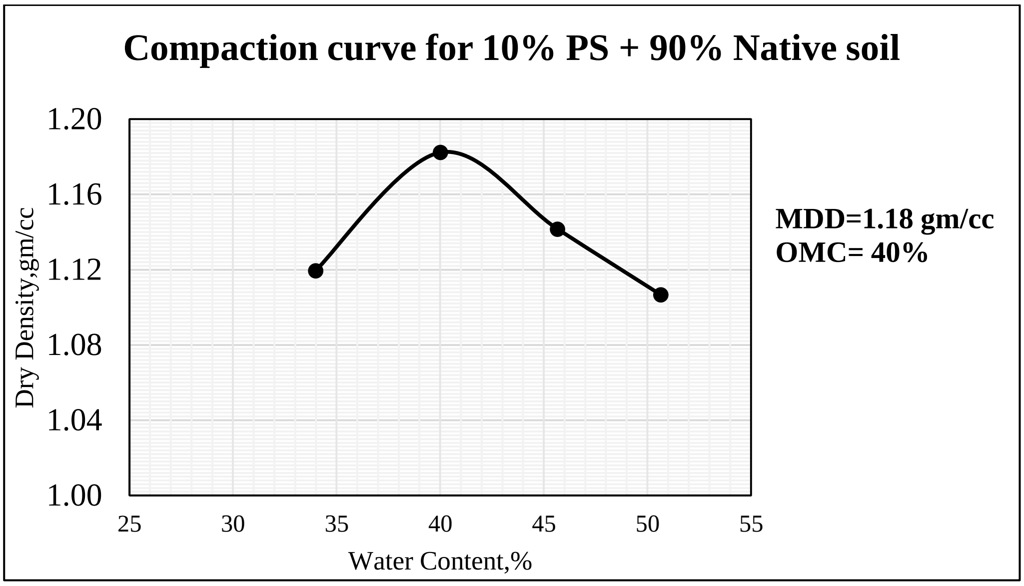
A. 5% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of Soil & Mold, g	4457		4571		4636		4719		4659	
Mass of Mold, g	3123.8		3123.8		3123.8		3123.8		3123.8	
Mass of Soil, g	1333.2		1447.2		1512.2		1595.2		1535.2	
Wet density, g/cm^3	1.420		1.541		1.611		1.699		1.635	
Can Number	P-3	13	D	12	4	52	Co-9	LL-3	P-8	18
Mass of empty can, g	15.75	15.82	15.72	16.01	15.73	15.74	15.87	15.75	15.90	16.05
Mass can & wet soil, g	45.68	53.61	54.08	55.65	52.81	52.81	50.91	48.33	61.77	62.77
Mass can & dry soil, g	39.17	45.03	44.29	44.66	42.08	42.12	39.71	37.92	45.62	46.35
Mass of soil, g	23.42	29.21	28.57	28.65	26.35	26.38	23.84	22.17	29.72	30.30
Mass of water, g	6.51	8.58	9.79	10.99	10.73	10.69	11.20	10.41	16.15	16.42
Water Content, %	27.80	29.37	34.27	38.36	40.71	40.53	46.98	46.96	54.34	54.19
	28.59		**36.31**		**40.62**		**46.97**		**54.27**	
Dry Density, g/cm^3	**1.10**		**1.13**		**1.15**		**1.16**		**1.06**	



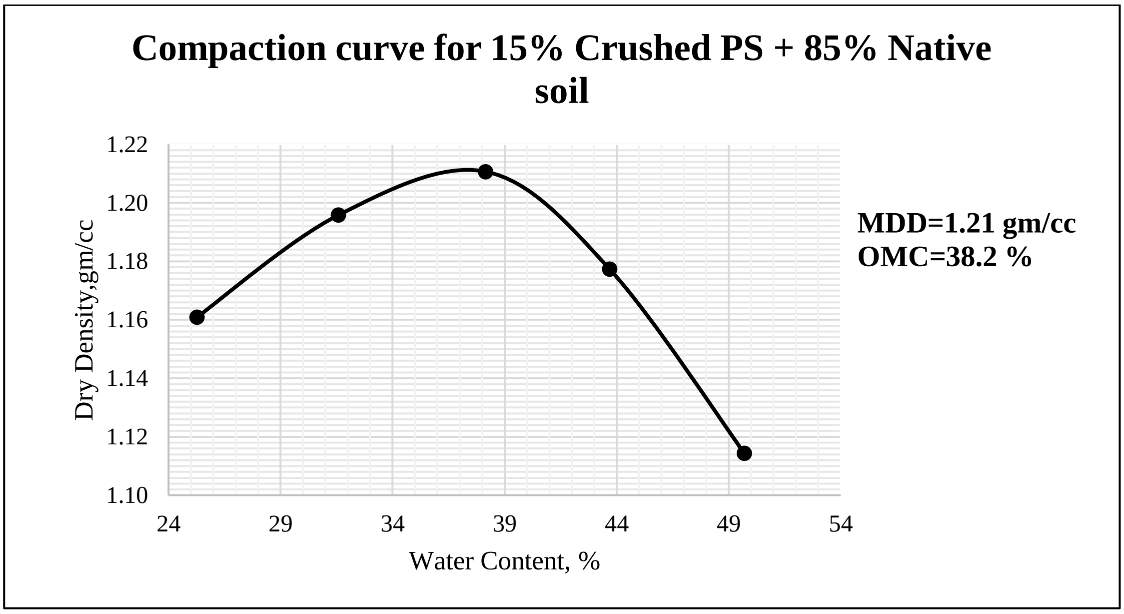
B. 10% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of Soil & Mold(g):	4532		4607.8		4685		4689		4646	
Mass of Mold (g):	3123.8		3123.8		3123.8		3123.8		3123.8	
Mass of Soil (g):	1408.2		1554.2		1561.2		1565.2		1522.2	
Wet Density (g/cm^3):	1.500		1.655		1.663		1.667		1.621	
Can Number	P_5	51	15	W_4	E_1	C_4	PL-2	Co-10	Co-9	LL-3
Mass of empty Can (g):	15.8	15.68	15.71	15.47	14.32	15.35	20.69	15.85	15.9	15.76
Mass can & wet soil (g):	41.93	46.83	61.52	48.24	53.5	60.22	52.9	54.76	51.87	58.82
Mass can & dry soil (g):	35.42	38.79	48.24	39.01	41.37	45.98	42.03	41.73	39.03	43.49
Mass of soil (g):	19.62	23.11	32.53	23.54	27.05	30.63	21.34	25.88	23.13	27.73
Mass of water (g):	6.51	8.04	13.28	9.23	12.13	14.24	10.87	13.03	12.84	15.33
Water content *w* (%):	33.18	34.79	40.82	39.21	44.84	46.49	50.94	50.35	55.51	55.28
	33.985		**40.017**		**45.667**		**50.642**		**55.398**	
Dry density (g/cm^3):	**1.119**		**1.182**		**1.142**		**1.107**		**1.043**	



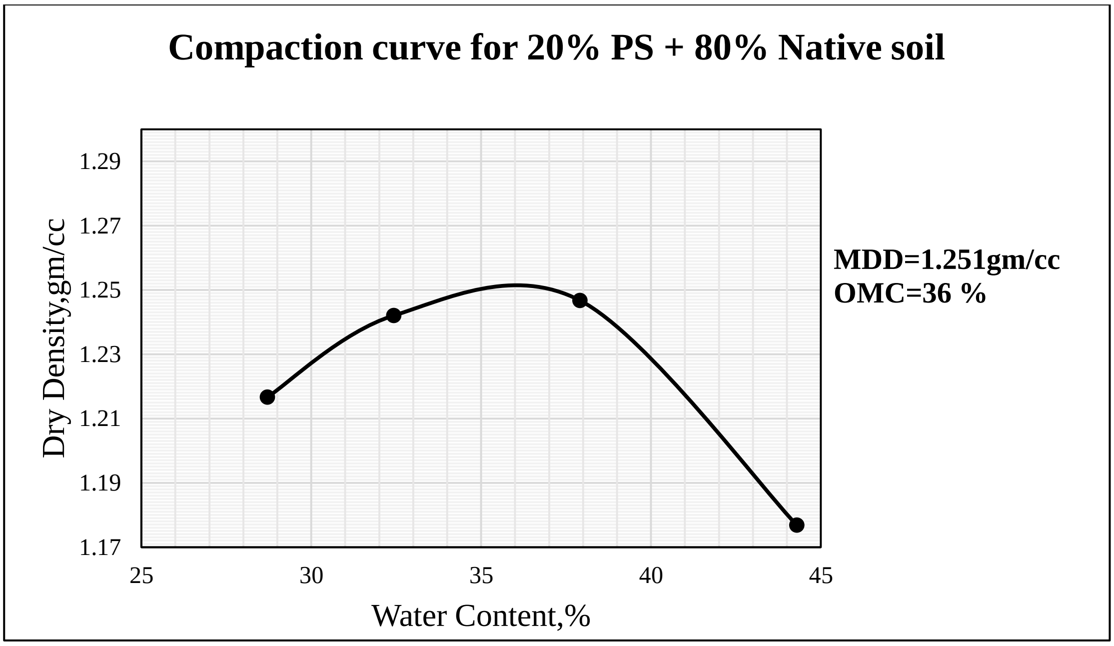
C. 15% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4490		4602		4695		4713		4691	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1365.2		1477.2		1570.2		1588.2		1566.2	
Wet density (g/cm³):	1.454		1.573		1.673		1.692		1.668	
Can number	100	102	LL-3	101	P-5	Co-9	W_4	15	Co-10	PL-3
Mass of empty can (g):	15.83	15.73	15.77	15.95	15.79	15.89	15.44	15.71	15.83	20.67
Mass can & wet soil (g):	48.46	55.32	46.47	54.30	55.37	45.80	55.30	56.25	58.15	59.37
Mass can & dry soil (g):	42.32	46.82	39.33	44.82	44.28	37.66	42.94	44.18	43.95	46.66
Mass of soil (g):	26.49	31.09	23.56	28.87	28.49	21.77	27.50	28.47	28.12	25.99
Mass of water (g):	6.14	8.51	7.14	9.49	11.09	8.14	12.36	12.08	14.19	12.72
Water content *w* (%):	23.18	27.35	30.31	32.85	38.92	37.39	44.96	42.42	50.47	48.93
	25.265		**31.582**		**38.150**		**43.689**		**49.701**	
Dry density (g/cm³):	**1.161**		**1.196**		**1.211**		**1.177**		**1.114**	



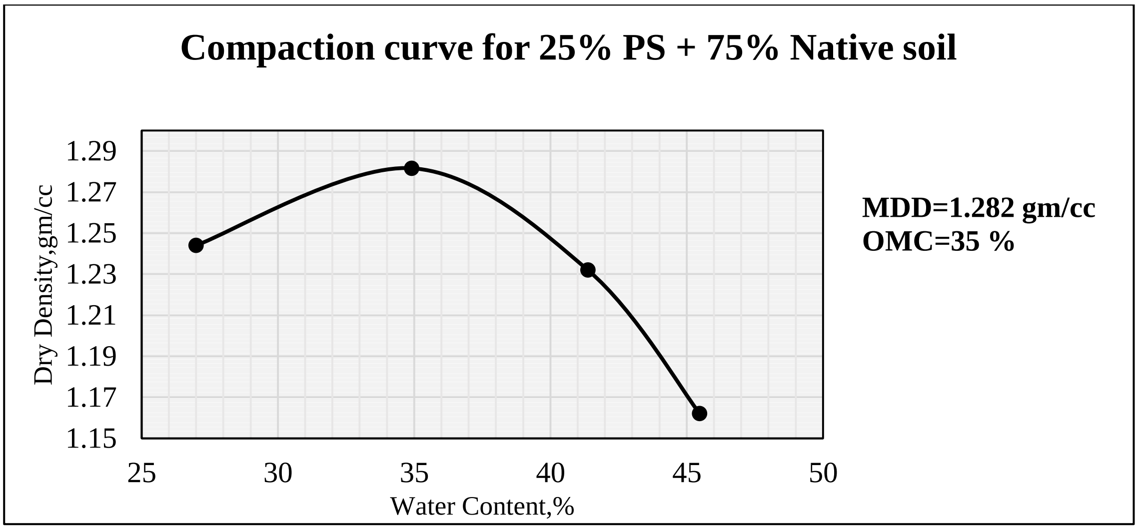
D. 20% pozzolanic sand

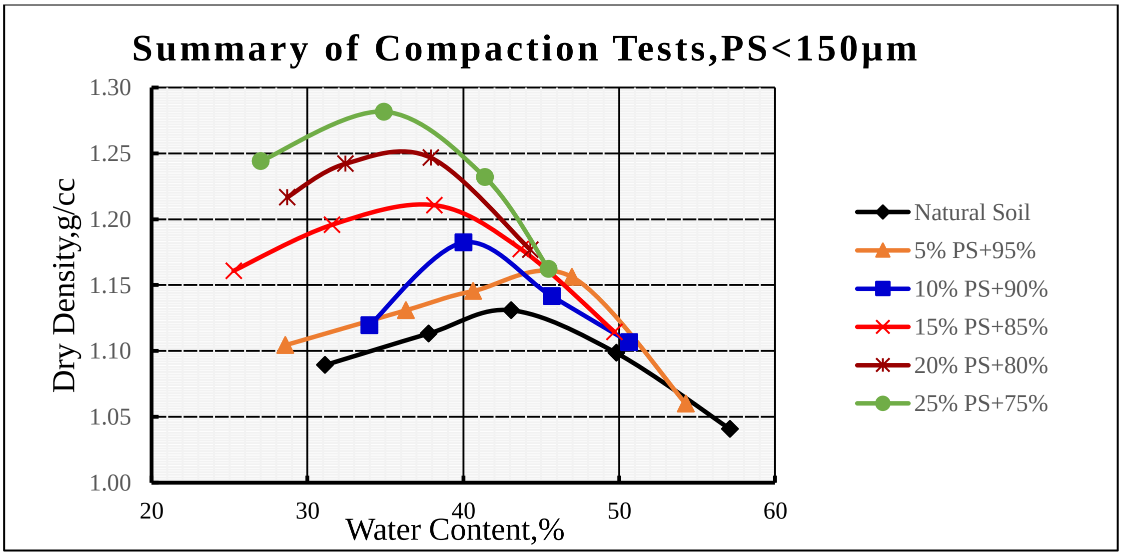
Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4595		4669		4739		4719		4719	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1470.2		1544.2		1614.2		1594.2		1594.2	
Wet density (g/cm^3):	1.566		1.645		1.719		1.698		1.698	
Can number	12	100	16	4	LL-3	P-3	33	13	15	W_A
Mass of empty can (g):	16.03	15.86	15.80	15.72	15.75	15.74	15.49	15.63	18.66	19.07
Mass can & dry soil (g):	46.58	50.95	62.80	52.99	62.10	54.92	52.17	62.75	77.42	106.05
Mass can & dry soil (g):	39.78	43.10	51.55	43.67	49.20	44.28	40.93	48.26	56.09	74.10
Mass of soil (g):	23.76	27.25	35.74	27.94	33.45	28.54	25.44	32.64	37.43	55.03
Mass of water (g):	6.80	7.85	11.25	9.32	12.90	10.64	11.24	14.49	21.33	31.95
Water content *w* (%):	28.62	28.80	31.49	33.37	38.56	37.27	44.18	44.40	56.99	58.06
	28.712		**32.428**		**37.912**		**44.293**		**57.523**	
Dry density (g/cm^3):	**1.217**		**1.242**		**1.247**		**1.177**		**1.078**	



E. 25% pozzolanic sand

Trial number	1		2		3		4	
Mass of soil & mold (g):	4608		4748		4760		4712	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1483.2		1623.2		1635.2		1587.2	
Wet density (g/cm^3):	1.580		1.729		1.742		1.691	
Can number	Co-9	Co-11	D	Wo	P3	W-6	15	PL-2
Mass of empty can (g):	15.87	15.85	15.73	15.63	15.87	14.44	15.72	20.68
Mass can & wet soil (g):	48.00	46.37	60.13	56.68	58.18	63.82	72.36	63.33
Mass can & dry soil (g):	41.10	39.95	48.55	46.14	50.00	45.54	54.55	50.08
Mass of soil (g):	25.22	24.10	32.82	30.52	34.13	31.10	38.83	29.40
Mass of water (g):	6.90	6.42	11.58	10.54	8.18	18.28	17.81	13.25
Water content *w* (%):	27.37	26.62	35.28	34.53	23.97	58.78	45.87	45.07
	26.995		**34.904**		**41.373**		**45.470**	
Dry density (g/cm^3):	**1.244**		**1.282**		**1.232**		**1.162**	

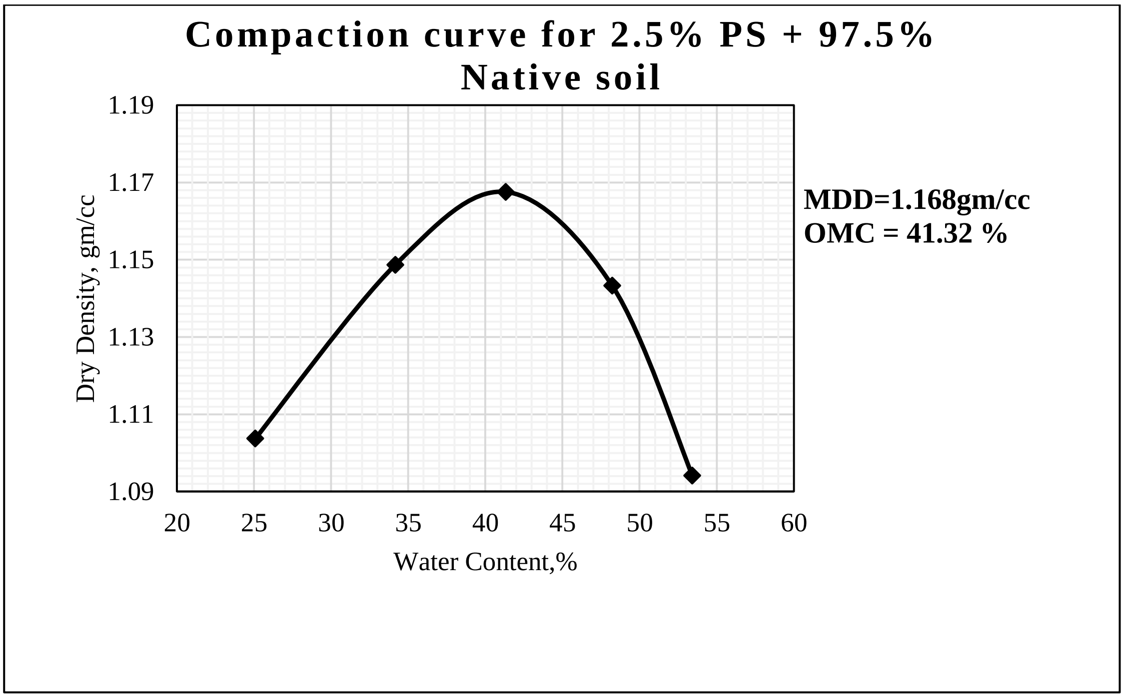




B-2: Compaction curves for soil blended with pozzolanic sand crushed to a particle size less than 75μm

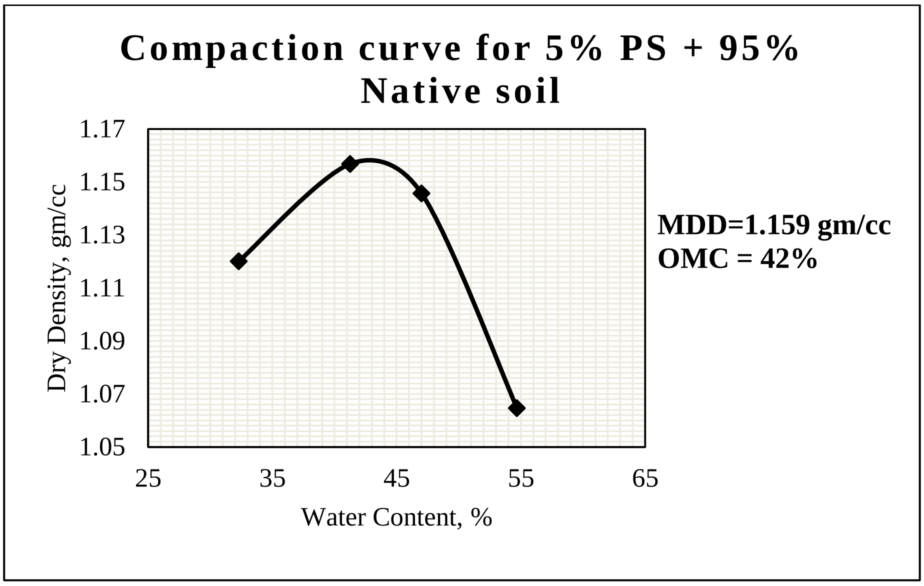
A. 2.5% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4420		5812		6154		6196		6181	
Mass of mold (g):	3123.8		4365		4605		4605		4605	
Mass of soil (g):	1296.2		1447		1549		1591		1576	
Wet density (g/cm^3):	1.381		1.541		1.650		1.695		1.679	
Can number	756	755	F-6	753	67	Co-12	F-5	12	F-1	F-3
Mass of empty can (g):	15.96	15.64	15.54	15.80	15.58	14.46	15.65	15.94	15.59	15.80
Mass can & wet soil (g):	42.82	44.81	38.44	39.20	49.37	37.68	41.14	42.31	44.22	45.00
Mass Can & dry soil (g):	37.68	38.70	32.44	33.41	39.42	30.94	32.87	33.71	34.17	34.91
Mass of soil (g):	21.71	23.06	16.90	17.61	23.83	16.49	17.21	17.78	18.58	19.11
Mass of water (g):	5.14	6.11	5.99	5.79	9.96	6.74	8.27	8.60	10.05	10.08
Water content *w* (%):	23.69	26.49	35.46	32.88	41.77	40.86	48.05	48.39	54.09	52.76
	25.089		**34.173**		**41.317**		**48.221**		**53.422**	
Dry density (g/cm^3):	**1.104**		**1.149**		**1.168**		**1.143**		**1.094**	



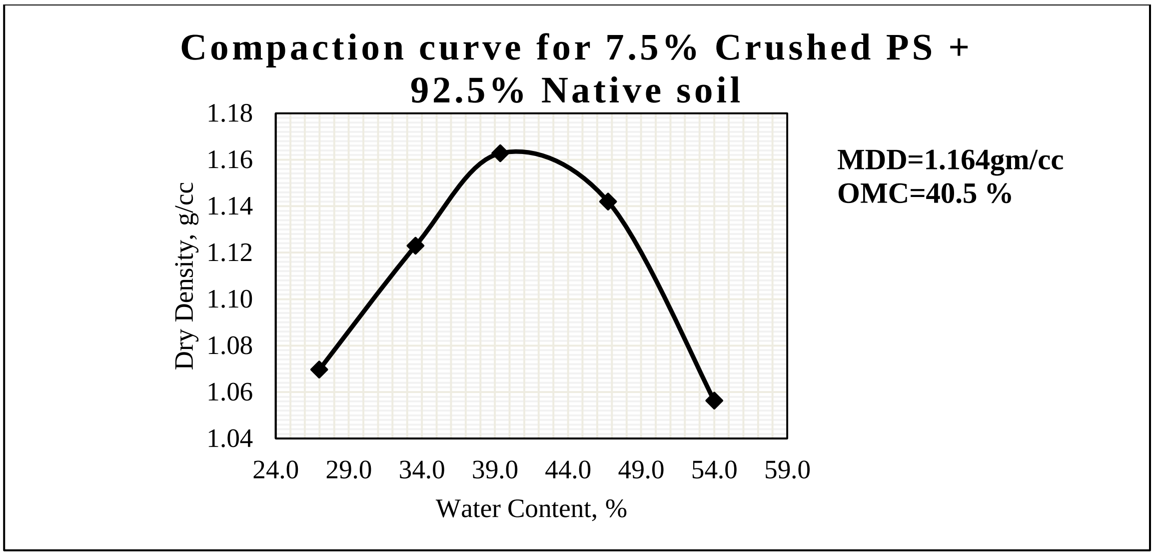
B. 5% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4515		4550		4658		4705		4670	
Mass of mold (g):	3123.8		3123.8		3123.8		3123.8		3123.8	
Mass of soil (g):	1391.2		1426.2		1534.2		1581.2		1546.2	
Wet density (g/cm^3):	1.482		1.519		1.634		1.684		1.647	
Can number	7511	751	18	22	754	7510	W_{19}	100	W_2	Co-4
Mass of empty can (g):	15.64	14.63	15.93	15.63	15.60	15.58	15.56	15.84	15.38	15.72
Mass can & wet soil (g):	44.78	37.90	46.20	41.30	43.56	41.08	47.72	40.25	51.42	42.14
Mass can & dry soil (g):	38.13	31.86	38.30	34.86	35.17	33.84	37.52	32.38	38.63	32.84
Mass of soil (g):	22.49	17.23	22.37	19.24	19.57	18.26	21.96	16.55	23.24	17.12
Mass of water (g):	6.65	6.04	7.90	6.44	8.39	7.24	10.20	7.87	12.79	9.30
Water content *w* (%):	29.57	35.03	35.30	33.46	42.88	39.65	46.45	47.55	55.03	54.33
	32.301		**34.381**		**41.265**		**47.000**		**54.682**	
Dry density (g/cm^3):	**1.120**		**1.130**		**1.157**		**1.146**		**1.065**	



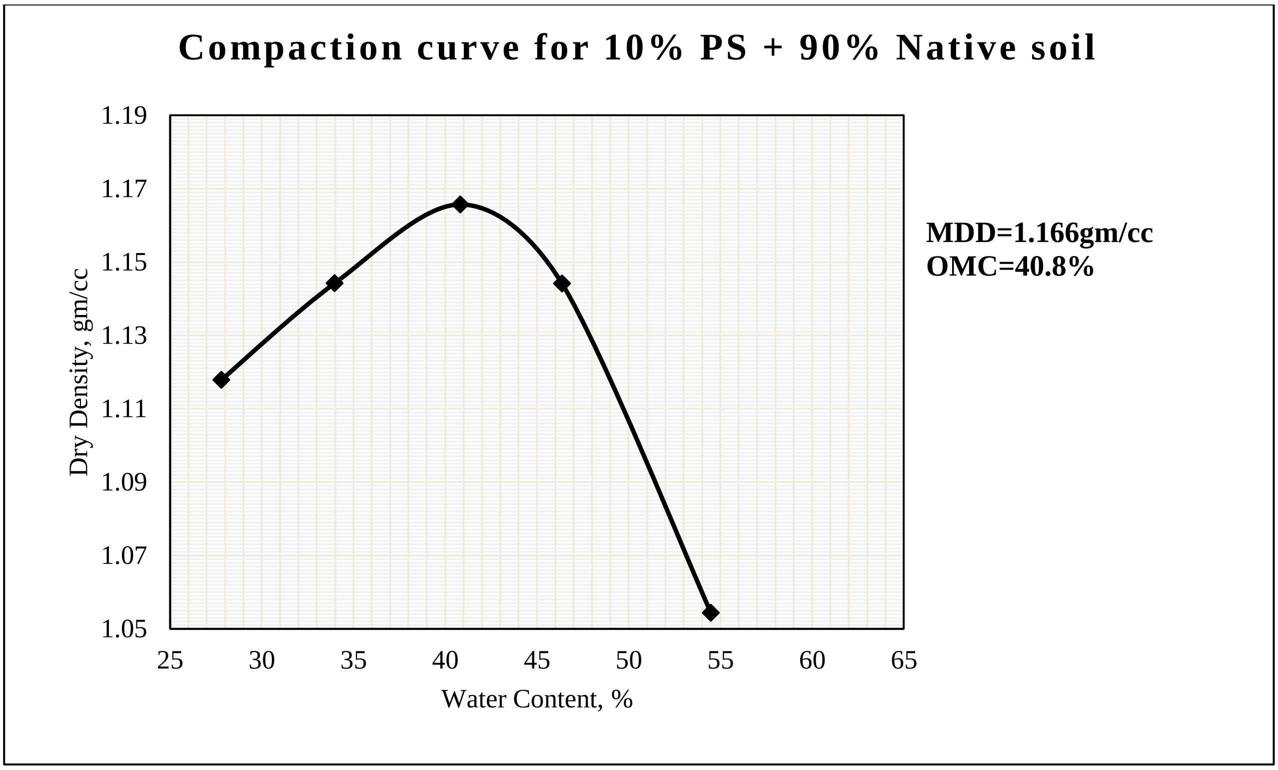
C. 7.5% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4400		4533		4646		4698		4652	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1275.2		1408.2		1521.2		1573.2		1527.2	
Wet density (g/cm^3):	1.358		1.500		1.620		1.676		1.627	
Can number	50	13	W_{29}	W_o	68	67	W_5	75	66	65
Mass of empty can (g):	15.63	15.56	15.24	15.56	15.95	15.65	15.52	15.82	15.92	15.90
Mass can & wet soil (g):	38.52	37.44	42.52	37.95	42.89	43.44	45.71	47.78	39.78	44.90
Mass can & dry soil (g):	33.71	32.74	35.64	32.35	35.14	35.75	36.19	37.50	31.50	34.63
Mass of soil (g):	18.08	17.18	20.40	16.78	19.18	20.10	20.67	21.68	15.58	18.73
Mass of water (g):	4.81	4.70	6.88	5.61	7.76	7.69	9.53	10.28	8.28	10.28
Water content *w* (%):	26.62	27.35	33.74	33.40	40.43	38.28	46.08	47.41	53.15	54.87
	26.988		**33.570**		**39.356**		**46.747**		**54.012**	
Dry density (g/cm^3):	**1.070**		**1.123**		**1.163**		**1.142**		**1.056**	



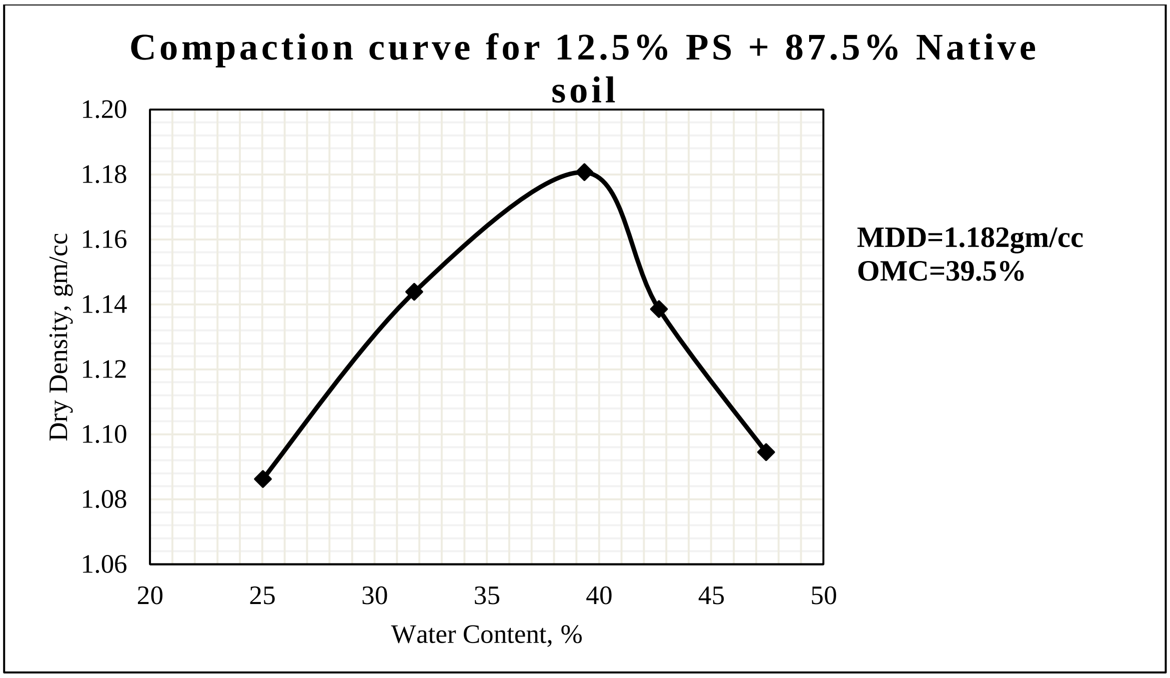
D. 10% pozzolanic sand

Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4466		4564		4666		4697		4654	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1341.2		1439.2		1541.2		1572.2		1529.2	
Wet density (g/cm^3):	1.429		1.533		1.642		1.675		1.629	
Can number	P_6	C_2	L-1	W_6	W_7	C-5	W_{25}	752	51	W_{21}
Mass of empty can (g):	11.04	15.58	15.66	14.43	15.81	16.04	15.63	15.64	15.66	15.91
Mass can & soil (g):	27.59	36.84	41.44	37.81	39.97	47.62	41.29	43.47	43.75	46.53
Mass can & dry soil (g):	23.97	32.23	34.96	31.83	33.07	38.33	33.02	34.81	33.79	35.79
Mass of soil (g):	12.94	16.66	19.30	17.40	17.26	22.29	17.39	19.17	18.13	19.88
Mass of water (g):	3.61	4.61	6.48	5.98	6.90	9.29	8.27	8.66	9.96	10.74
Water content *w* (%):	27.91	27.69	33.59	34.36	39.97	41.68	47.54	45.18	54.97	54.00
	27.797		**33.974**		**40.826**		**46.361**		**54.486**	
Dry density (g/cm^3):	**1.118**		**1.144**		**1.166**		**1.144**		**1.054**	



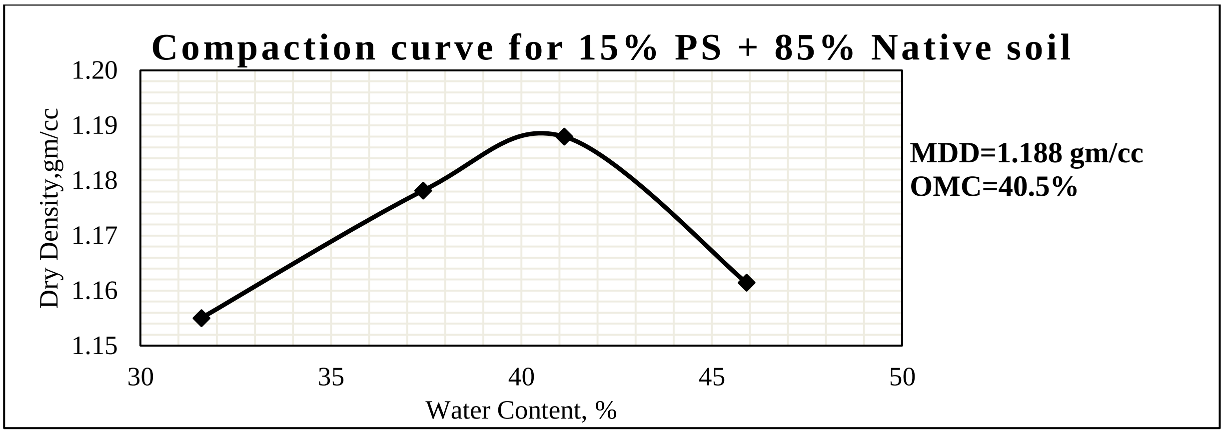
E. 12.5% pozzolanic sand

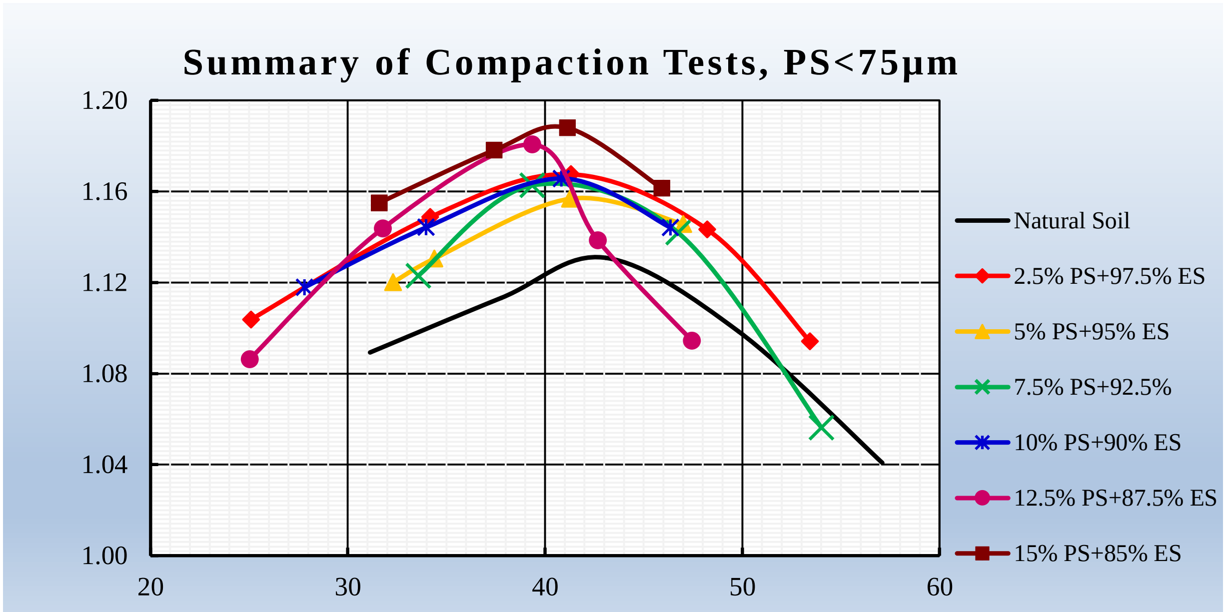
Trial number	1		2		3		4		5	
Mass of soil & mold (g):	4399.8		4539.8		4669.4		4649.8		4639.8	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8		3124.8	
Mass of soil (g):	1275		1415		1544.6		1525		1515	
Wet density (g/cm^3):	1.358		1.507		1.645		1.624		1.614	
Can number	W_6	66	W_5	7510	W_{37}	7511	67	50	13	W_{29}
Mass of empty can (g):	14.46	15.87	15.54	15.60	15.65	15.67	15.63	15.62	15.58	15.22
Mass can & wet soil (g):	32.56	42.82	40.95	34.81	43.05	45.61	44.59	42.30	45.97	46.67
Mass can & dry soil (g):	29.01	37.32	34.71	30.26	35.97	36.49	36.04	34.22	36.16	36.57
Mass of soil (g):	14.55	21.45	19.18	14.67	20.31	20.82	20.41	18.60	20.58	21.36
Mass of water (g):	3.55	5.50	6.24	4.55	7.08	9.13	8.55	8.08	9.80	10.09
Water Content *w* (%):	24.40	25.65	32.53	31.02	34.87	43.84	41.91	43.43	47.63	47.26
	25.025		**31.774**		**39.353**		**42.674**		**47.446**	
Dry Density (g/cm^3):	**1.086**		**1.144**		**1.181**		**1.139**		**1.094**	



F. 15% pozzolanic sand

Trial number	1		2		3		4	
Mass of soil & mold (g):	4551.8		4644.8		4698.8		4715.8	
Mass of mold (g):	3124.8		3124.8		3124.8		3124.8	
Mass of soil (M_s) (g):	1427		1520		1574		1591	
Wet density (g/cm^3):	1.520		1.619		1.677		1.695	
Can number	Co-9	13	W_{25}	50	Co-4	759	W_{29}	W_{34}
Mass of empty can (g):	15.88	15.57	15.63	15.61	15.60	15.68	15.21	15.44
Mass can & wet soil (g):	40.81	37.53	40.72	47.35	36.80	45.55	47.78	44.21
Mass can & dry soil (g):	34.77	32.31	33.94	38.64	30.81	36.59	37.59	35.10
Mass of soil (g):	18.89	16.73	18.31	23.04	15.21	20.91	22.38	19.66
Mass of water (g):	6.04	5.22	6.78	8.71	5.99	8.96	10.19	9.10
Water content *w* (%):	31.98	31.21	37.03	37.81	39.39	42.87	45.52	46.31
	31.597		**37.421**		**41.130**		**45.912**	
Dry density (g/cm^3):	**1.155**		**1.178**		**1.188**		**1.161**	



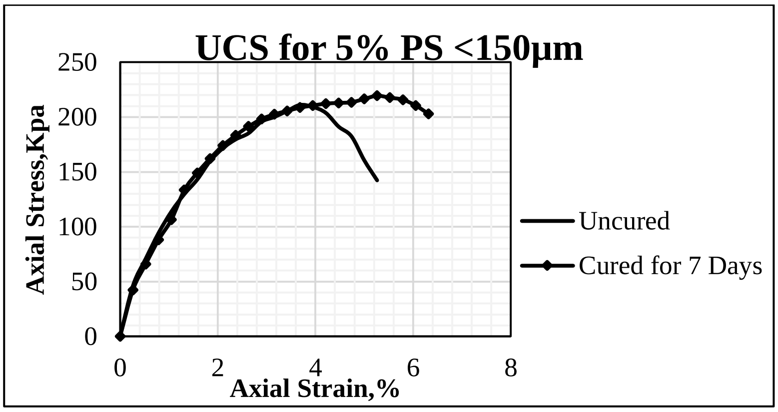


APPENDIX C: UNCONFINED COMPRESSION STRENGTH TESTS FOR BLENDED SOIL

C-1: Unconfined strength tests for soil blended with Pozzolanic sand crushed to a particle size less than 150μm

A. UCS for 5% pozzolanic sand

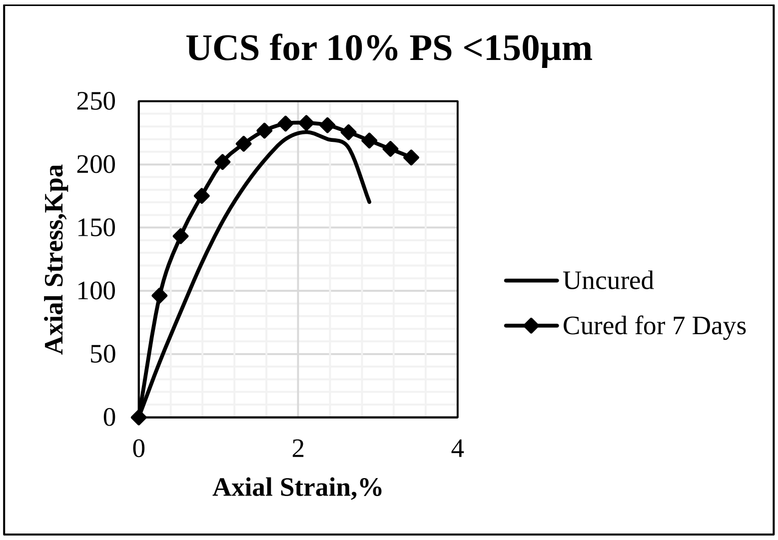
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	37	0.053	46.20	34	0.048	42.46
0.4	0.53	0.001140	57	0.081	70.99	53	0.075	66.01
0.6	0.79	0.001143	76	0.108	94.41	71	0.101	88.20
0.8	1.05	0.001146	92	0.131	113.98	86	0.122	106.55
1	1.32	0.001149	105	0.149	129.74	108	0.153	133.45
1.2	1.58	0.001152	116	0.165	142.95	121	0.172	149.11
1.4	1.84	0.001155	130	0.185	159.77	132	0.187	162.23
1.6	2.11	0.001159	140	0.199	171.60	142	0.202	174.05
1.8	2.37	0.001162	147	0.209	179.70	150	0.213	183.36
2	2.63	0.001165	152	0.216	185.31	157	0.223	191.40
2.2	2.89	0.001168	161	0.229	195.75	163	0.231	198.18
2.4	3.16	0.001171	165	0.234	200.07	167	0.237	202.49
2.6	3.42	0.001174	170	0.241	205.57	170	0.241	205.57
2.8	3.68	0.001177	175	0.249	211.04	173	0.246	208.63
3	3.95	0.001181	174	0.247	209.26	175	0.249	210.46
3.2	4.21	0.001184	170	0.241	203.89	177	0.251	212.29
3.4	4.47	0.001187	160	0.227	191.37	178	0.253	212.90
3.6	4.74	0.001191	153	0.217	182.49	179	0.254	213.51
3.8	5.00	0.001194	135	0.192	160.58	182	0.258	216.48
4	5.26	0.001197	120	0.170	142.34	185	0.263	219.44
4.2	5.53	0.001200				184	0.261	217.65
4.4	5.79	0.001204				183	0.260	215.86
4.6	6.05	0.001207				179	0.254	210.56
4.8	6.32	0.001211				173	0.246	202.93



5%	Uncured	Cured for 7 Days
q_u(kPa)	211	219
c(kPa)	105.5	109.5

B. UCS for 10% pozzolanic sand

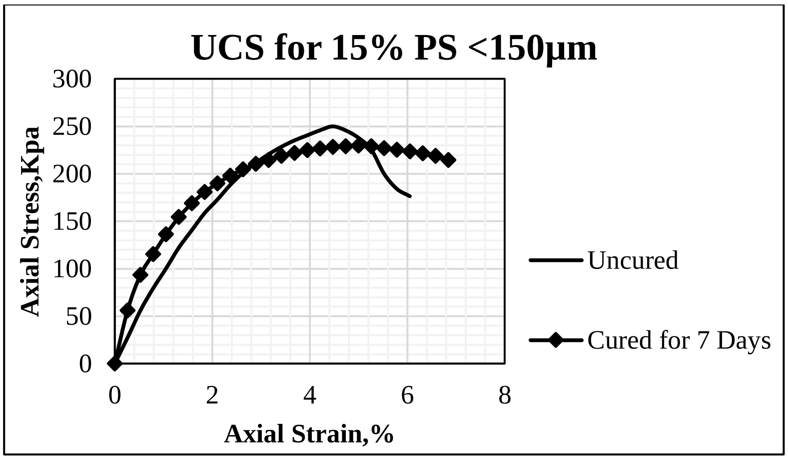
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0	0.00
0.2	0.26	0.001137	35	0.050	43.71	77	0.10934	96.16
0.4	0.53	0.001140	67	0.095	83.45	115	0.1633	143.23
0.6	0.79	0.001143	98	0.139	121.73	141	0.20022	175.15
0.8	1.05	0.001146	125	0.178	154.86	163	0.23146	201.94
1	1.32	0.001149	147	0.209	181.63	175	0.2485	216.23
1.2	1.58	0.001152	165	0.234	203.33	184	0.26128	226.74
1.4	1.84	0.001155	179	0.254	219.99	189	0.26838	232.28
1.6	2.11	0.001159	184	0.261	225.53	190	0.2698	232.89
1.8	2.37	0.001162	180	0.256	220.04	189	0.26838	231.04
2	2.63	0.001165	175	0.249	213.35	185	0.2627	225.54
2.2	2.89	0.001168	140	0.199	170.22	180	0.2556	218.85
2.4	3.16	0.001171	100	0.142	121.25	175	0.2485	212.19
2.6	3.42	0.001174				170	0.2414	205.57



10%	Uncured	Cured for 7 Days
q_u(kPa)	226	233
c(kPa)	113	117

C. UCS for 15% pozzolanic sand

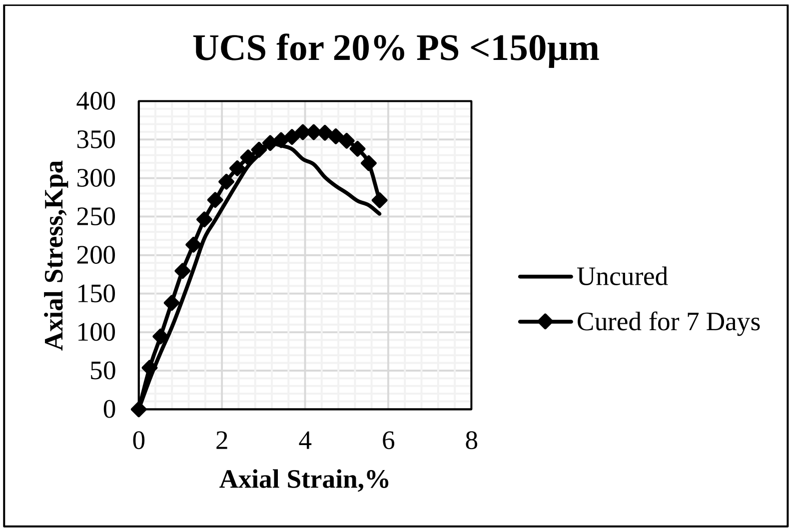
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	22	0.031	27.47	45	0.064	56.20
0.4	0.53	0.001140	45	0.064	56.05	75	0.107	93.41
0.6	0.79	0.001143	64	0.091	79.50	93	0.132	115.52
0.8	1.05	0.001146	81	0.115	100.35	110	0.156	136.28
1	1.32	0.001149	99	0.141	122.32	125	0.178	154.45
1.2	1.58	0.001152	114	0.162	140.48	137	0.195	168.83
1.4	1.84	0.001155	129	0.183	158.54	147	0.209	180.66
1.6	2.11	0.001159	141	0.200	172.83	155	0.220	189.99
1.8	2.37	0.001162	154	0.219	188.25	162	0.230	198.03
2	2.63	0.001165	165	0.234	201.16	168	0.239	204.81
2.2	2.89	0.001168	174	0.247	211.55	173	0.246	210.34
2.4	3.16	0.001171	182	0.258	220.68	177	0.251	214.62
2.6	3.42	0.001174	189	0.268	228.55	181	0.257	218.87
2.8	3.68	0.001177	195	0.277	235.16	184	0.261	221.89
3	3.95	0.001181	200	0.284	240.53	187	0.266	224.90
3.2	4.21	0.001184	205	0.291	245.87	189	0.268	226.68
3.4	4.47	0.001187	209	0.297	249.98	191	0.271	228.45
3.6	4.74	0.001191	206	0.293	245.71	192	0.273	229.01
3.8	5.00	0.001194	200	0.284	237.89	193	0.274	229.57
4	5.26	0.001197	190	0.270	225.37	193	0.274	228.93
4.2	5.53	0.001200	169	0.240	199.91	192	0.273	227.11
4.4	5.79	0.001204	156	0.222	184.02	191	0.271	225.30
4.6	6.05	0.001207	150	0.213	176.44	190	0.270	223.50
4.8	6.32	0.001211				189	0.268	221.70
5	6.58	0.001214				187	0.266	218.73
5.2	6.84	0.001217				184	0.261	214.62



15%	**Uncured**	**Cured for 7 Days**
q_u(kPa)	250	230
c(kPa)	125	115

D. UCS for 20% pozzolanic sand

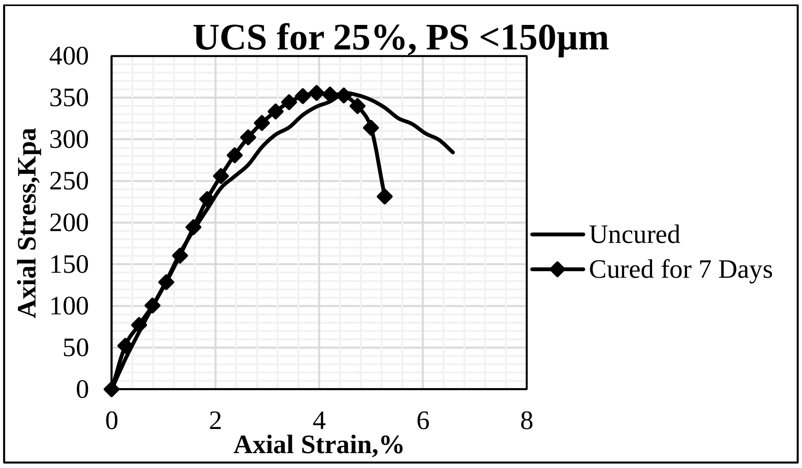
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	31	0.044	38.71	43	0.061	53.70
0.4	0.53	0.001140	59	0.084	73.48	76	0.108	94.66
0.6	0.79	0.001143	85	0.121	105.59	111	0.158	137.88
0.8	1.05	0.001146	115	0.163	142.47	145	0.206	179.64
1	1.32	0.001149	147	0.209	181.63	173	0.246	213.76
1.2	1.58	0.001152	180	0.256	221.82	200	0.284	246.46
1.4	1.84	0.001155	200	0.284	245.80	221	0.314	271.61
1.6	2.11	0.001159	220	0.312	269.66	241	0.342	295.40
1.8	2.37	0.001162	240	0.341	293.38	256	0.364	312.94
2	2.63	0.001165	259	0.368	315.75	268	0.381	326.73
2.2	2.89	0.001168	272	0.386	330.71	277	0.393	336.79
2.4	3.16	0.001171	284	0.403	344.36	285	0.405	345.57
2.6	3.42	0.001174	283	0.402	342.22	289	0.410	349.47
2.8	3.68	0.001177	280	0.398	337.67	293	0.416	353.34
3	3.95	0.001181	270	0.383	324.72	299	0.425	359.59
3.2	4.21	0.001184	265	0.376	317.83	300	0.426	359.81
3.4	4.47	0.001187	252	0.358	301.41	300	0.426	358.82
3.6	4.74	0.001191	243	0.345	289.84	297	0.422	354.25
3.8	5.00	0.001194	236	0.335	280.72	293	0.416	348.52
4	5.26	0.001197	228	0.324	270.45	285	0.405	338.06
4.2	5.53	0.001200	224	0.318	264.97	270	0.383	319.38
4.4	5.79	0.001204	215	0.305	253.61	230	0.327	271.31



20%	Uncured	Cured for 7 Days
q_u(kPa)	344	360
c(kPa)	172	180

E. UCS for 25% pozzolanic sand

			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	29	0.041	36.21	42	0.060	52.45
0.4	0.53	0.001140	55	0.078	68.50	62	0.088	77.22
0.6	0.79	0.001143	80	0.114	99.38	81	0.115	100.62
0.8	1.05	0.001146	105	0.149	130.08	104	0.148	128.85
1	1.32	0.001149	132	0.187	163.10	130	0.185	160.63
1.2	1.58	0.001152	155	0.220	191.01	158	0.224	194.70
1.4	1.84	0.001155	176	0.250	216.31	186	0.264	228.60
1.6	2.11	0.001159	197	0.280	241.47	209	0.297	256.18
1.8	2.37	0.001162	209	0.297	255.49	230	0.327	281.16
2	2.63	0.001165	221	0.314	269.43	248	0.352	302.34
2.2	2.89	0.001168	239	0.339	290.58	263	0.373	319.76
2.4	3.16	0.001171	252	0.358	305.56	275	0.391	333.45
2.6	3.42	0.001174	260	0.369	314.40	285	0.405	344.63
2.8	3.68	0.001177	273	0.388	329.22	292	0.415	352.14
3	3.95	0.001181	282	0.400	339.15	296	0.420	355.99
3.2	4.21	0.001184	288	0.409	345.42	295	0.419	353.81
3.4	4.47	0.001187	297	0.422	355.23	295	0.419	352.84
3.6	4.74	0.001191	296	0.420	353.06	285	0.405	339.94
3.8	5.00	0.001194	292	0.415	347.33	264	0.375	314.02
4	5.26	0.001197	285	0.405	338.06	195	0.277	231.30
4.2	5.53	0.001200	275	0.391	325.29	190	0.270	224.75
4.4	5.79	0.001204	270	0.383	318.49	186	0.264	219.40
4.6	6.05	0.001207	261	0.371	307.01	176	0.250	207.03
4.8	6.32	0.001211	255	0.362	299.11	168	0.239	197.06
5	6.58	0.001214	243	0.345	284.24	160	0.227	187.15

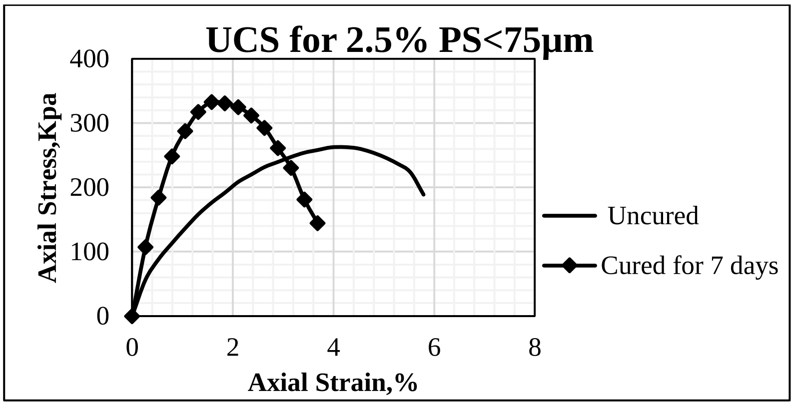


25%	Uncured	Cured for 7 Days
q_u(kPa)	355.23	355.99
c(kPa)	177.62	178.00

C-2: Unconfined strength tests for soil blended with Pozzolanic sand crushed to a particle size less than 75µm

A. UCS for 2.5% pozzolanic sand

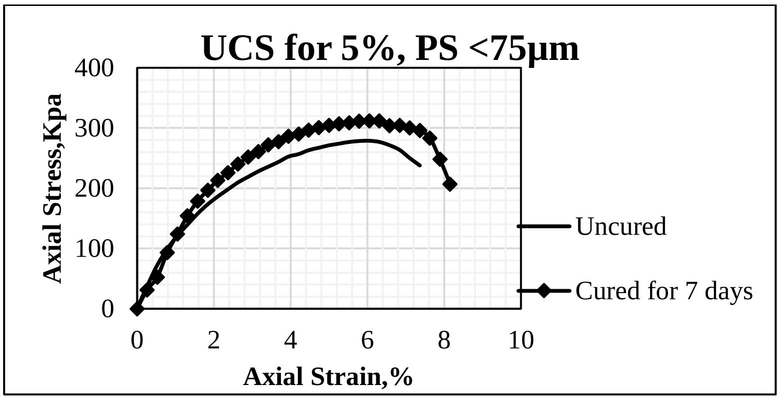
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	45	0.064	56.20	86	0.122	107.40
0.4	0.53	0.001140	71	0.101	88.43	148	0.210	184.33
0.6	0.79	0.001143	91	0.129	113.04	200	0.284	248.44
0.8	1.05	0.001146	110	0.156	136.28	232	0.329	287.42
1	1.32	0.001149	128	0.182	158.16	257	0.365	317.55
1.2	1.58	0.001152	143	0.203	176.22	270	0.383	332.72
1.4	1.84	0.001155	156	0.222	191.73	269	0.382	330.60
1.6	2.11	0.001159	170	0.241	208.37	265	0.376	324.82
1.8	2.37	0.001162	180	0.256	220.04	255	0.362	311.72
2	2.63	0.001165	190	0.270	231.63	240	0.341	292.59
2.2	2.89	0.001168	197	0.280	239.52	215	0.305	261.40
2.4	3.16	0.001171	204	0.290	247.36	190	0.270	230.38
2.6	3.42	0.001174	210	0.298	253.94	150	0.213	181.39
2.8	3.68	0.001177	214	0.304	258.07	120	0.170	144.71
3	3.95	0.001181	218	0.310	262.18			
3.2	4.21	0.001184	219	0.311	262.66			
3.4	4.47	0.001187	218	0.310	260.74			
3.6	4.74	0.001191	214	0.304	255.25			
3.8	5.00	0.001194	208	0.295	247.41			
4	5.26	0.001197	200	0.284	237.24			
4.2	5.53	0.001200	189	0.268	223.57			
4.4	5.79	0.001203809	160	0.227	188.73			



2.5%	Uncured	Cured for 7 days
q_u(kPa)	263	333
C(kPa)	131.5	166.5

B. UCS for 5% pozzolanic sand

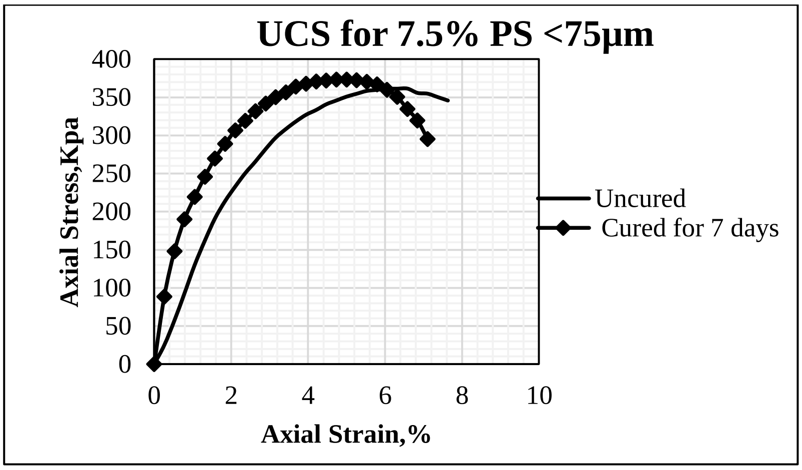
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	30	0.043	37.46	25	0.036	31.22
0.4	0.53	0.001140	58	0.082	72.24	42	0.060	52.31
0.6	0.79	0.001143	80	0.114	99.38	75	0.107	93.16
0.8	1.05	0.001146	98	0.139	121.41	100	0.142	123.89
1	1.32	0.001149	113	0.160	139.62	125	0.178	154.45
1.2	1.58	0.001152	128	0.182	157.74	145	0.206	178.68
1.4	1.84	0.001155	141	0.200	173.29	160	0.227	196.64
1.6	2.11	0.001159	152	0.216	186.31	174	0.247	213.27
1.8	2.37	0.001162	162	0.230	198.03	185	0.263	226.15
2	2.63	0.001165	172	0.244	209.69	197	0.280	240.17
2.2	2.89	0.001168	180	0.256	218.85	207	0.294	251.68
2.4	3.16	0.001171	188	0.267	227.96	215	0.305	260.70
2.6	3.42	0.001174	195	0.277	235.80	225	0.320	272.08
2.8	3.68	0.001177	202	0.287	243.60	230	0.327	277.37
3	3.95	0.001181	210	0.298	252.56	238	0.338	286.23
3.2	4.21	0.001184	214	0.304	256.66	242	0.344	290.24
3.4	4.47	0.001187	220	0.312	263.13	248	0.352	296.62
3.6	4.74	0.001191	224	0.318	267.18	252	0.358	300.58
3.8	5.00	0.001194	228	0.324	271.20	256	0.364	304.51
4	5.26	0.001197	231	0.328	274.01	259	0.368	307.22
4.2	5.53	0.001200	234	0.332	276.79	261	0.371	308.73
4.4	5.79	0.001204	236	0.335	278.38	264	0.375	311.41
4.6	6.05	0.001207	237	0.337	278.78	265	0.376	311.72
4.8	6.32	0.001211	236	0.335	276.83	266	0.378	312.02
5	6.58	0.001214	232	0.329	271.37	260	0.369	304.12
5.2	6.84	0.001217	226	0.321	263.61	261	0.371	304.43
5.4	7.11	0.001221	215	0.305	250.07	258	0.366	300.08
5.6	7.37	0.001224	205	0.291	237.76	255	0.362	295.75
5.8	7.63	0.001228	195	0.277	225.52	245	0.348	283.35
6	7.89	0.001231	188	0.267	216.81	215	0.305	247.94
6.2	8.16	0.001235	180	0.256	206.99	180	0.256	206.99



5%	Uncured	Cured for 7 days
q_u(kPa)	278.8	312.0
c(kPa)	139.4	156.0

C. UCS for 7.5% pozzolanic sand

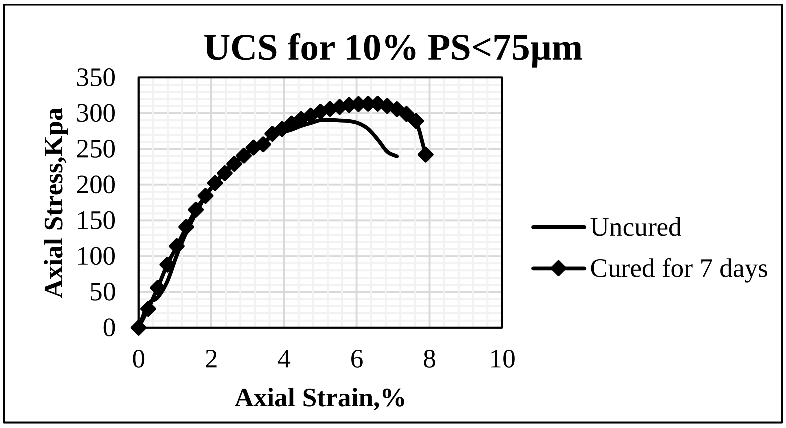
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	20	0.028	24.98	71	0.101	88.66
0.4	0.53	0.001140	46	0.065	57.29	119	0.169	148.21
0.6	0.79	0.001143	75	0.107	93.16	153	0.217	190.06
0.8	1.05	0.001146	105	0.149	130.08	177	0.251	219.28
1	1.32	0.001149	131	0.186	161.86	199	0.283	245.88
1.2	1.58	0.001152	155	0.220	191.01	219	0.311	269.88
1.4	1.84	0.001155	174	0.247	213.85	235	0.334	288.82
1.6	2.11	0.001159	190	0.270	232.89	250	0.355	306.43
1.8	2.37	0.001162	205	0.291	250.60	261	0.371	319.05
2	2.63	0.001165	218	0.310	265.77	272	0.386	331.60
2.2	2.89	0.001168	232	0.329	282.07	281	0.399	341.65
2.4	3.16	0.001171	245	0.348	297.07	289	0.410	350.42
2.6	3.42	0.001174	255	0.362	308.36	295	0.419	356.73
2.8	3.68	0.001177	264	0.375	318.37	302	0.429	364.20
3	3.95	0.001181	272	0.386	327.12	306	0.435	368.01
3.2	4.21	0.001184	278	0.395	333.42	309	0.439	370.60
3.4	4.47	0.001187	285	0.405	340.88	311	0.442	371.98
3.6	4.74	0.001191	290	0.412	345.90	313	0.444	373.34
3.8	5.00	0.001194	295	0.419	350.89	314	0.446	373.49
4	5.26	0.001197	299	0.425	354.67	314	0.446	372.46
4.2	5.53	0.001200	303	0.430	358.41	313	0.444	370.24
4.4	5.79	0.001204	305	0.433	359.77	311	0.442	366.85
4.6	6.05	0.001207	307	0.436	361.12	306	0.435	359.95
4.8	6.32	0.001211	308	0.437	361.28	299	0.425	350.73
5	6.58	0.001214	309	0.439	361.44	286	0.406	334.54
5.2	6.84	0.001217	305	0.433	355.75	274	0.389	319.60
5.4	7.11	0.001221	305	0.433	354.75	254	0.361	295.43
5.6	7.37	0.001224	302	0.429	350.27	238	0.338	276.04
5.8	7.63	0.001228	299	0.425	345.80	226	0.321	261.37



7.50%	Uncured	Cured for 7 days
q_u(kPa)	361.44	373.49
c(kPa)	180.72	186.75

D. UCS for 10% pozzolanic sand

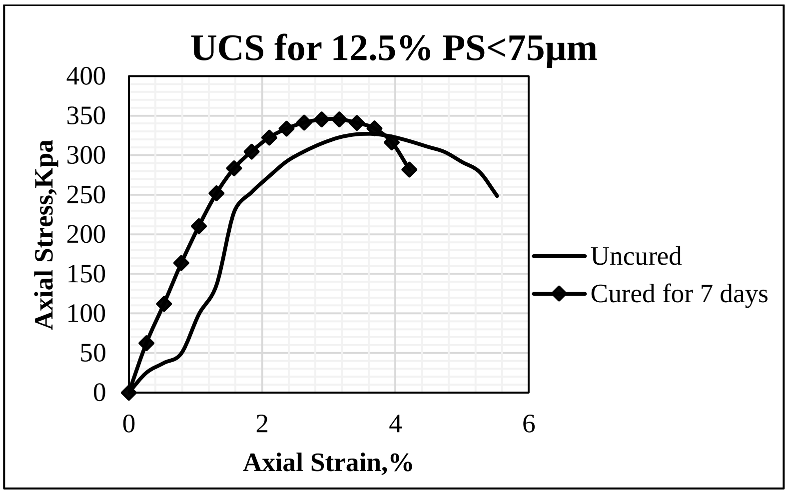
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	26	0.037	32.47	21	0.030	26.22
0.4	0.53	0.001140	34	0.048	42.35	45	0.064	56.05
0.6	0.79	0.001143	52	0.074	64.59	71	0.101	88.20
0.8	1.05	0.001146	82	0.116	101.59	92	0.131	113.98
1	1.32	0.001149	108	0.153	133.45	114	0.162	140.86
1.2	1.58	0.001152	130	0.185	160.20	134	0.190	165.13
1.4	1.84	0.001155	150	0.213	184.35	150	0.213	184.35
1.6	2.11	0.001159	165	0.234	202.24	165	0.234	202.24
1.8	2.37	0.001162	179	0.254	218.81	177	0.251	216.37
2	2.63	0.001165	190	0.270	231.63	188	0.267	229.20
2.2	2.89	0.001168	198	0.281	240.73	198	0.281	240.73
2.4	3.16	0.001171	206	0.293	249.78	208	0.295	252.21
2.6	3.42	0.001174	214	0.304	258.78	212	0.301	256.36
2.8	3.68	0.001177	222	0.315	267.72	225	0.320	271.34
3	3.95	0.001181	227	0.322	273.00	231	0.328	277.81
3.2	4.21	0.001184	231	0.328	277.05	238	0.338	285.45
3.4	4.47	0.001187	236	0.335	282.27	244	0.346	291.84
3.6	4.74	0.001191	240	0.341	286.26	249	0.354	297.00
3.8	5.00	0.001194	244	0.346	290.23	254	0.361	302.13
4	5.26	0.001197	245	0.348	290.61	258	0.366	306.03
4.2	5.53	0.001200	245	0.348	289.81	261	0.371	308.73
4.4	5.79	0.001204	245	0.348	289.00	264	0.375	311.41
4.6	6.05	0.001207	243	0.345	285.84	266	0.378	312.89
4.8	6.32	0.001211	237	0.337	278.00	267	0.379	313.19
5	6.58	0.001214	225	0.320	263.18	268	0.381	313.48
5.2	6.84	0.001217	211	0.300	246.11	266	0.378	310.26
5.4	7.11	0.001221	206	0.293	239.60	263	0.373	305.90
5.6	7.37	0.001224	195	0.277	226.16	258	0.366	299.23
5.8	7.63	0.001228	182	0.258	210.49	250	0.355	289.13
6	7.89	0.001231	176	0.250	202.97	210	0.298	242.18



10%	Uncured	Cured for 7 days
q_u(kPa)	290.61	313.48
c(kPa)	145.31	156.74

E. UCS for 12.5% pozzolanic sand

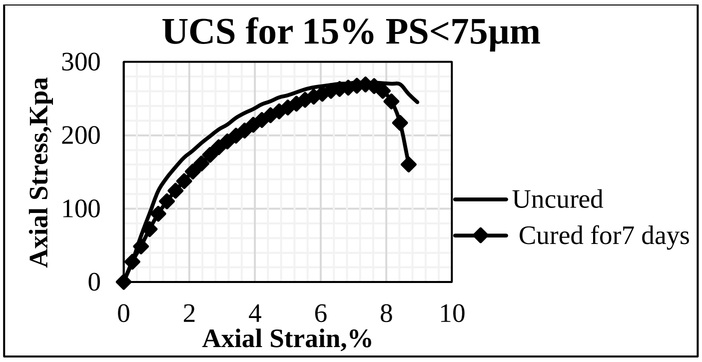
			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0	0.00
0.2	0.26	0.001137	20	0.028	24.98	50	0.071	62.44
0.4	0.53	0.001140	30	0.043	37.36	90	0.1278	112.09
0.6	0.79	0.001143	40	0.057	49.69	132	0.18744	163.97
0.8	1.05	0.001146	80	0.114	99.11	170	0.2414	210.61
1	1.32	0.001149	110	0.156	135.92	204	0.28968	252.06
1.2	1.58	0.001152	185	0.263	227.98	230	0.3266	283.43
1.4	1.84	0.001155	206	0.293	253.18	248	0.35216	304.80
1.6	2.11	0.001159	223	0.317	273.34	263	0.37346	322.36
1.8	2.37	0.001162	239	0.339	292.16	273	0.38766	333.72
2	2.63	0.001165	250	0.355	304.78	280	0.3976	341.36
2.2	2.89	0.001168	259	0.368	314.90	284	0.40328	345.30
2.4	3.16	0.001171	266	0.378	322.54	285	0.4047	345.57
2.6	3.42	0.001174	270	0.383	326.50	282	0.40044	341.01
2.8	3.68	0.001177	271	0.385	326.81	277	0.39334	334.05
3	3.95	0.001181	269	0.382	323.51	263	0.37346	316.30
3.2	4.21	0.001184	265	0.376	317.83	235	0.3337	281.85
3.4	4.47	0.001187	260	0.369	310.98			
3.6	4.74	0.001191	255	0.362	304.16			
3.8	5.00	0.001194	245	0.348	291.42			
4	5.26	0.001197	235	0.334	278.75			
4.2	5.53	0.001200	210	0.298	248.41			



12.5%	Uncured	Cured for 7 days
q_u(kPa)	326.81	345.57
c(kPa)	163.41	172.79

F. UCS for 15% pozzolanic sand

			Uncured			Cured for 7 days		
Axial Deformation	Axial Strain	Corrected Area (m2)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)	Proving Ring Reading	Axial Load (KN)	Vertical Stress (kPa)
0	0.00	0.001134	0	0.000	0.00	0	0.000	0.00
0.2	0.26	0.001137	22	0.031	27.47	22	0.031	27.47
0.4	0.53	0.001140	50	0.071	62.27	39	0.055	48.57
0.6	0.79	0.001143	75	0.107	93.16	58	0.082	72.05
0.8	1.05	0.001146	100	0.142	123.89	75	0.107	92.92
1	1.32	0.001149	115	0.163	142.09	89	0.126	109.97
1.2	1.58	0.001152	127	0.180	156.50	101	0.143	124.46
1.4	1.84	0.001155	138	0.196	169.60	112	0.159	137.65
1.6	2.11	0.001159	146	0.207	178.95	123	0.175	150.76
1.8	2.37	0.001162	155	0.220	189.48	132	0.187	161.36
2	2.63	0.001165	163	0.231	198.72	142	0.202	173.12
2.2	2.89	0.001168	171	0.243	207.91	151	0.214	183.59
2.4	3.16	0.001171	177	0.251	214.62	158	0.224	191.58
2.6	3.42	0.001174	185	0.263	223.71	165	0.234	199.53
2.8	3.68	0.001177	191	0.271	230.34	171	0.243	206.22
3	3.95	0.001181	196	0.278	235.72	178	0.253	214.07
3.2	4.21	0.001184	202	0.287	242.27	184	0.261	220.68
3.4	4.47	0.001187	206	0.293	246.39	190	0.270	227.25
3.6	4.74	0.001191	211	0.300	251.67	195	0.277	232.59
3.8	5.00	0.001194	214	0.304	254.55	200	0.284	237.89
4	5.26	0.001197	218	0.310	258.59	205	0.291	243.17
4.2	5.53	0.001200	222	0.315	262.60	210	0.298	248.41
4.4	5.79	0.001204	225	0.320	265.41	214	0.304	252.43
4.6	6.05	0.001207	227	0.322	267.02	218	0.310	256.43
4.8	6.32	0.001211	229	0.325	268.62	222	0.315	260.41
5	6.58	0.001214	231	0.328	270.20	225	0.320	263.18
5.2	6.84	0.001217	232	0.329	270.61	227	0.322	264.77
5.4	7.11	0.001221	234	0.332	272.17	230	0.327	267.52
5.6	7.37	0.001224	234	0.332	271.40	232	0.329	269.08
5.8	7.63	0.001228	235	0.334	271.78	231	0.328	267.16
6	7.89	0.001231	235	0.334	271.01	226	0.321	260.63
6.2	8.16	0.001235	235	0.334	270.23	214	0.304	246.09
6.4	8.42	0.001238	235	0.334	269.46	189	0.268	216.71



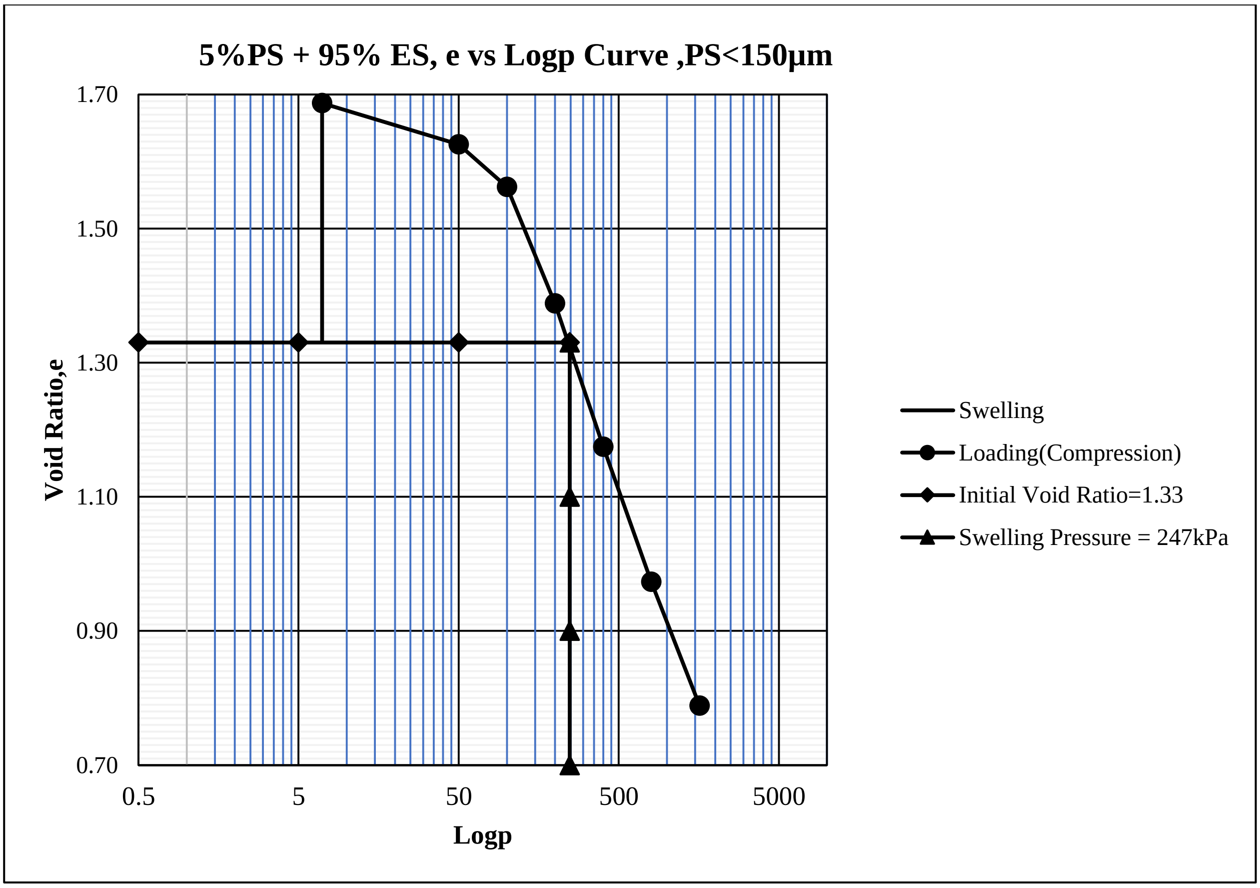
15%	Uncured	Cured for 7 days
q_u(kPa)	272.17	269.08
c(kPa)	136.08	134.54

APPENDIX D: SWELL-CONSOLIDATION TESTS FOR BLENDED SOIL

D-1: Swell-consolidation tests for soil blended with pozzolanic sand crushed to a particle size less than 150µm

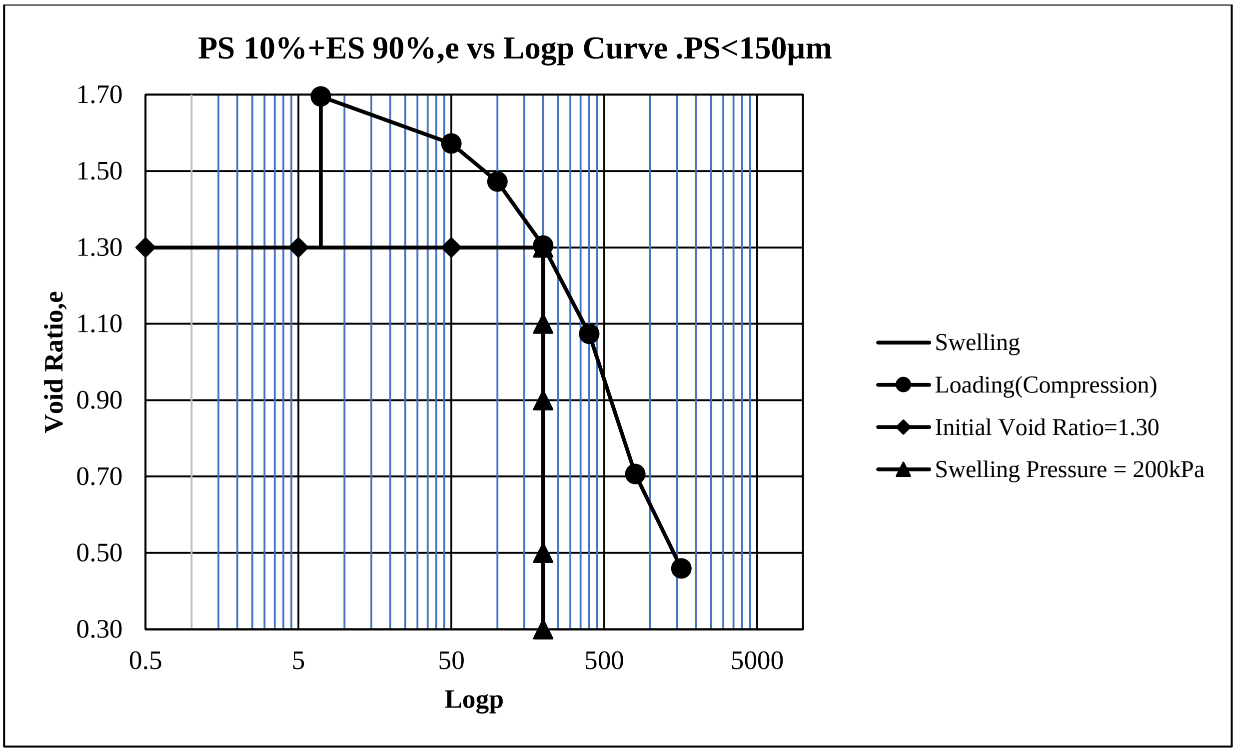
A. Swell-Consolidation test for soil blended with 5% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.33	Swelling
7	1.69	
50	1.63	Loading (Compression)
100	1.56	
200	1.39	
400	1.17	
800	0.97	
1600	0.79	



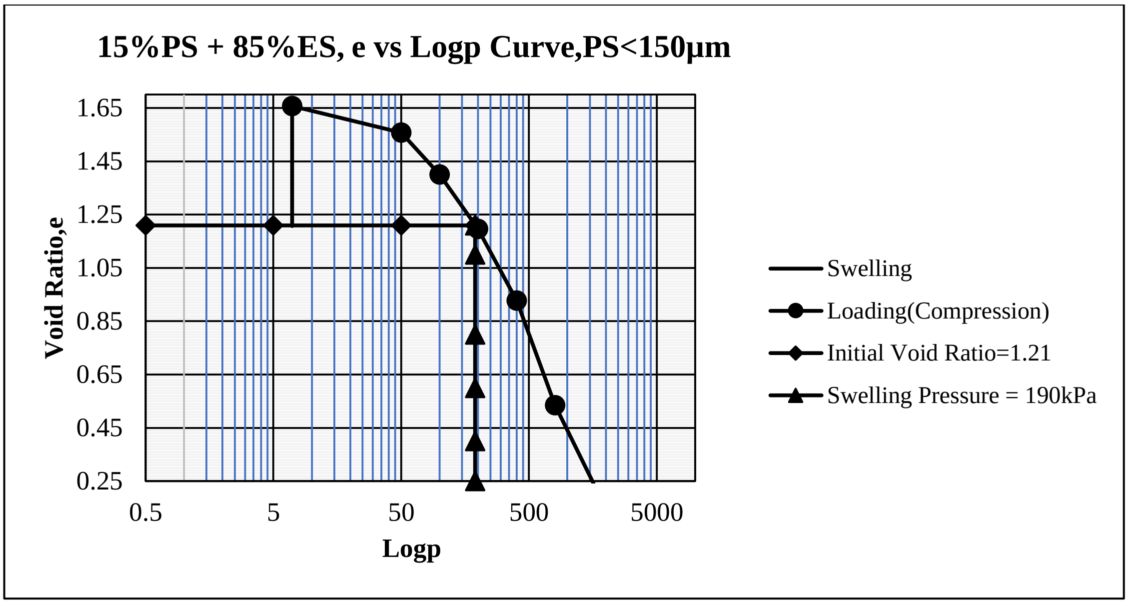
B. Swell-Consolidation test for soil blended with 10% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.30	Swelling
7	1.70	
50	1.57	Loading (Compression)
100	1.47	
200	1.30	
400	1.07	
800	0.71	
1600	0.46	



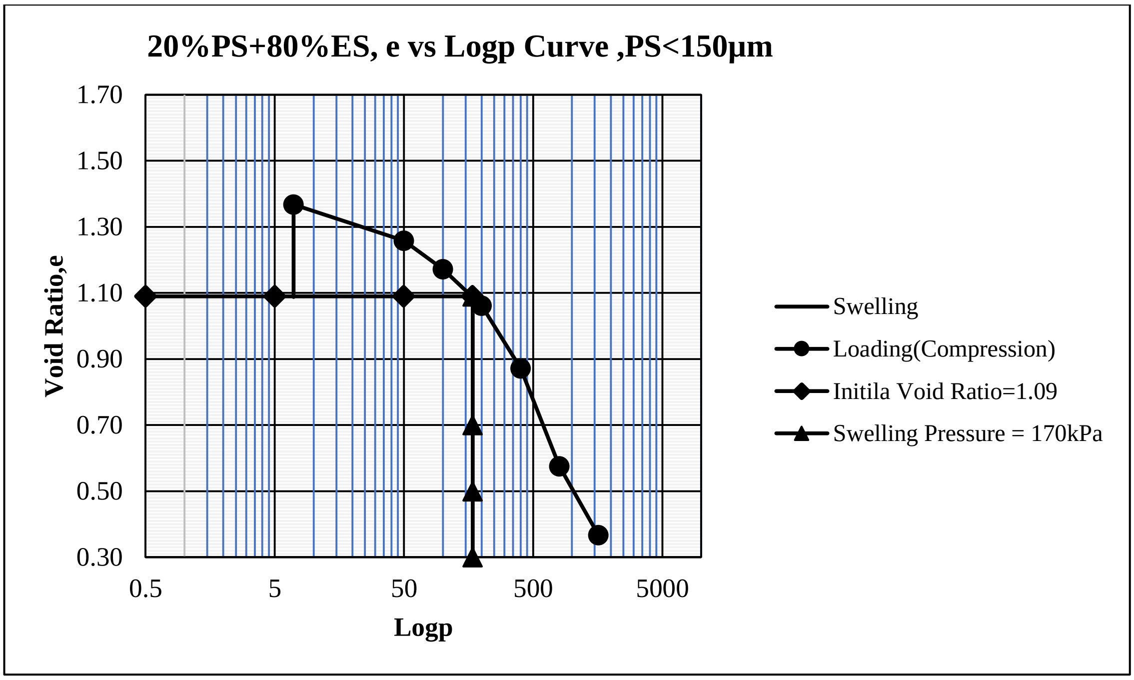
C. Swell-consolidation test for soil blended with 15% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.21	Swelling
7	1.66	
50	1.56	Loading (Compression)
100	1.40	
200	1.20	
400	0.93	
800	0.54	
1600	0.24	



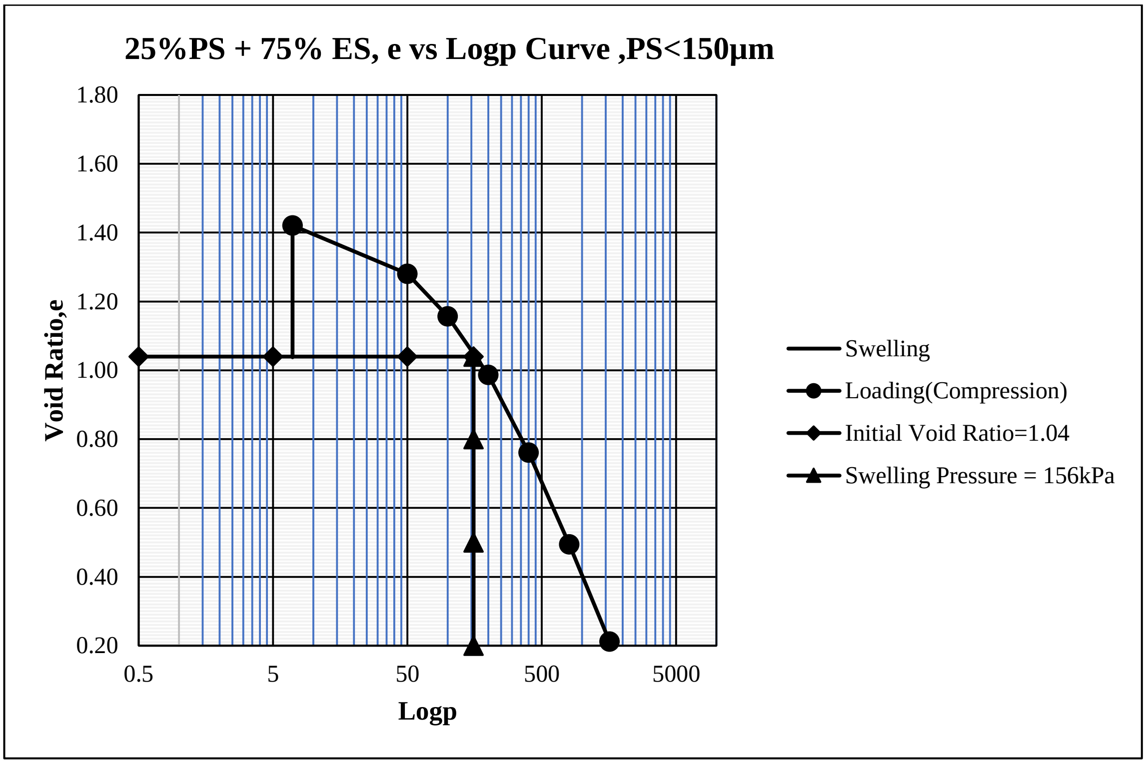
D. Swell-consolidation test for blended soil with 20% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.09	Swelling
7	1.37	
50	1.26	Loading (Compression)
100	1.17	
200	1.06	
400	0.87	
800	0.57	
1600	0.37	



E. Swell-consolidation test for soil blended with 25% pozzolanic sand

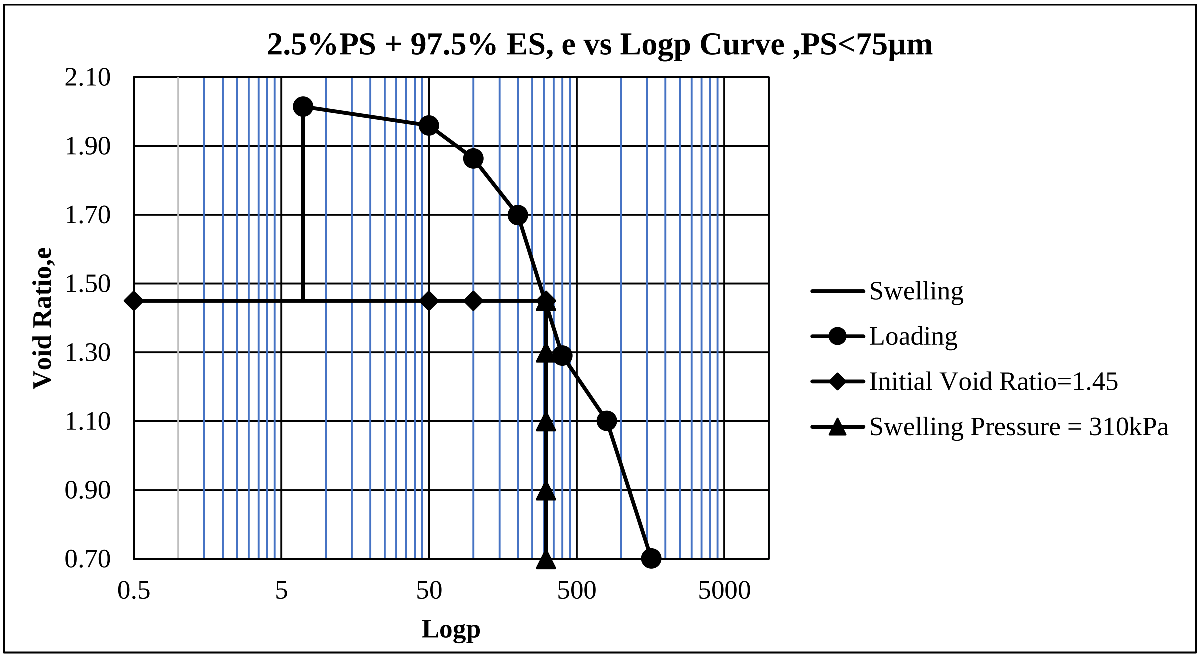
Pressure (kPa)	Void Ratio	
7	1.04	Swelling
7	1.42	
50	1.28	Loading (Compression)
100	1.16	
200	0.99	
400	0.76	
800	0.49	
1600	0.21	



D-2: Swell-consolidation tests for soil blended with pozzolanic sand crushed to a particle size less than 75µm

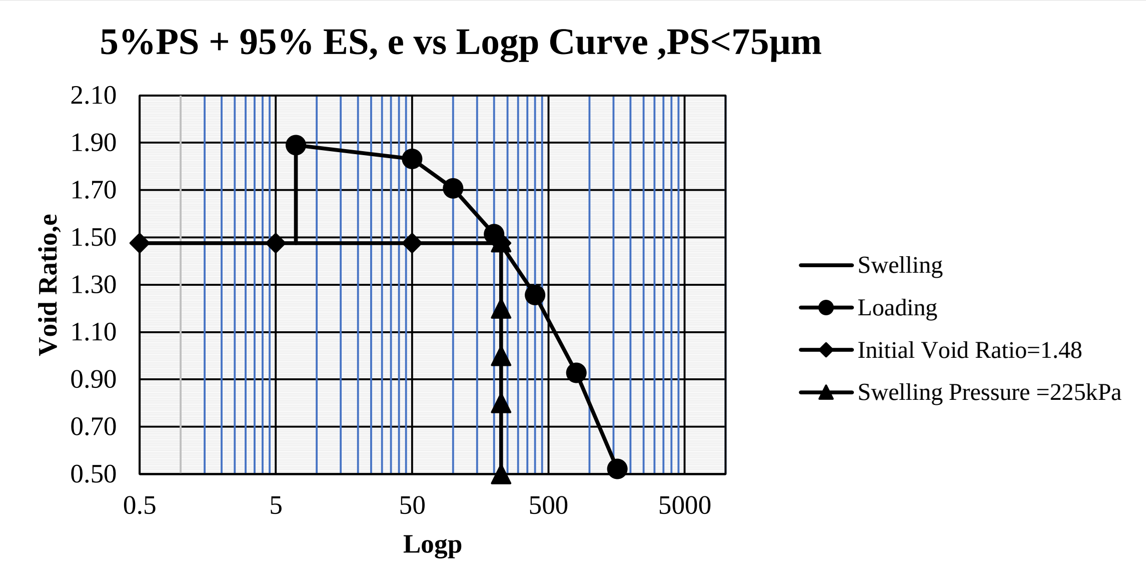
A. Swell-consolidation test for soil with 2.5% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.45	Swelling
7	2.01	
50	1.96	Loading (Compression)
100	1.86	
200	1.70	
400	1.29	
800	1.10	
1600	0.70	



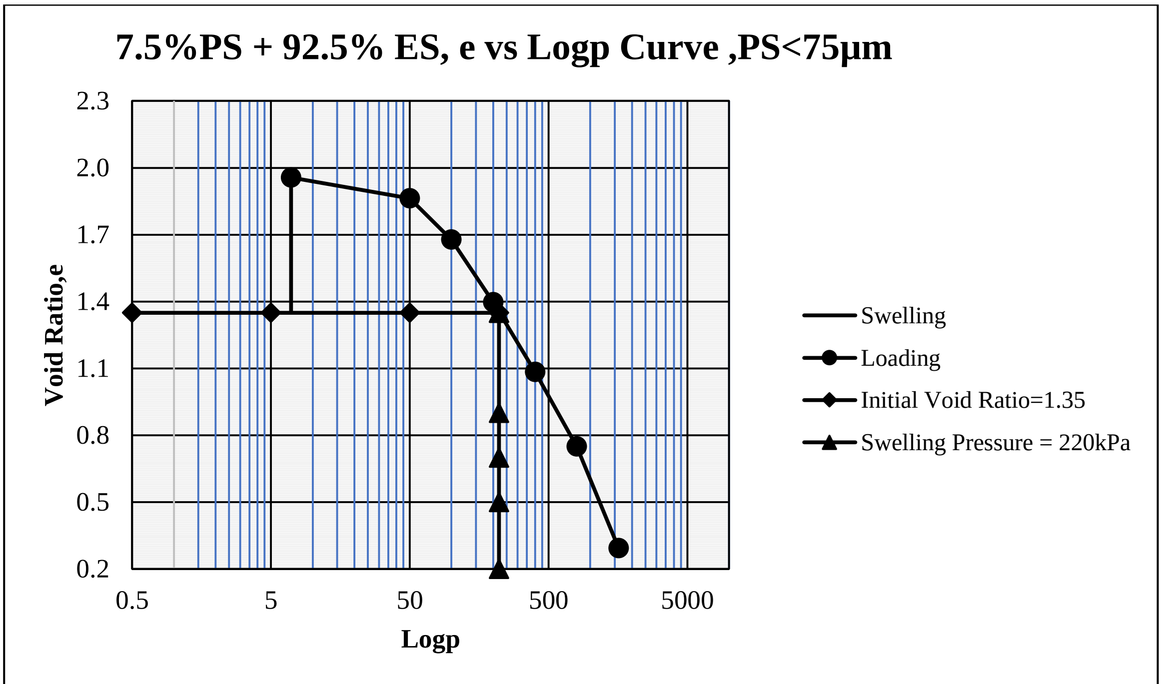
B. Swell-consolidation test for soil blended with 5% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.48	Swelling
7	1.89	
50	1.83	Loading (Compression)
100	1.71	
200	1.51	
400	1.26	
800	0.93	
1600	0.52	



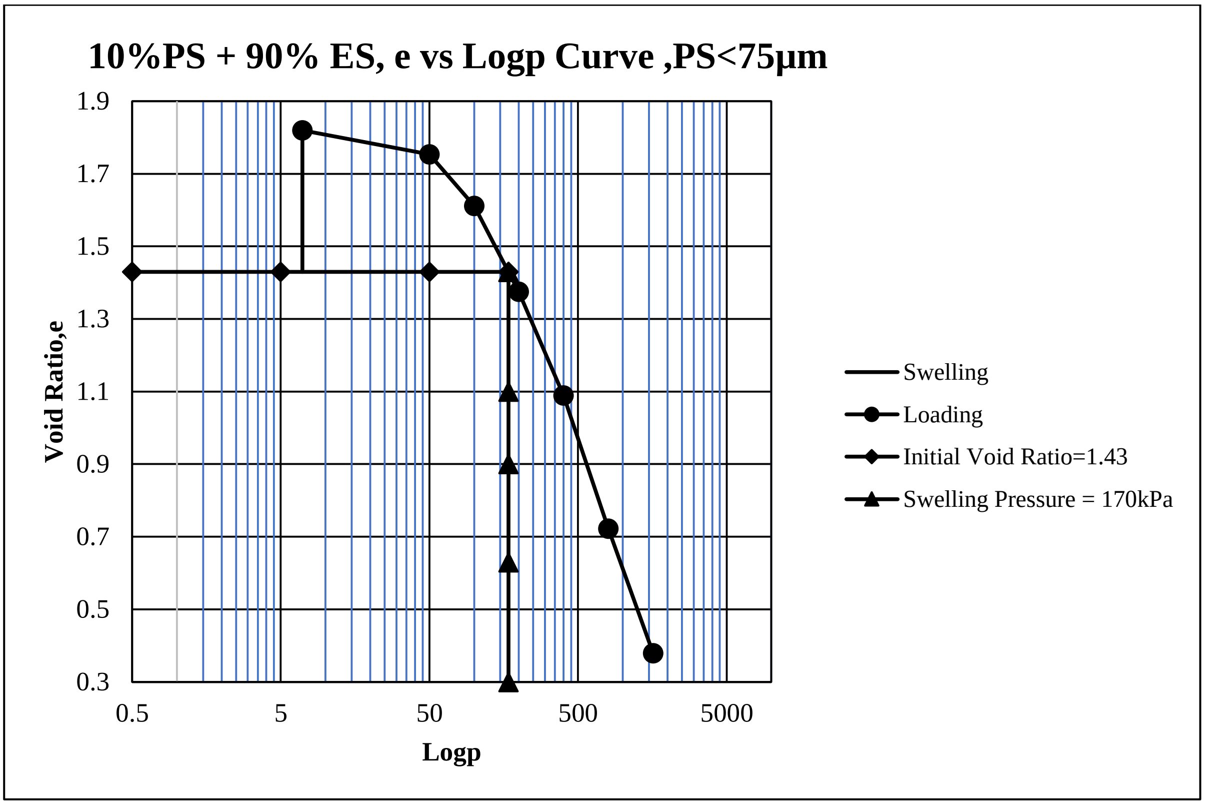
C. Swell-consolidation test for soil blended with 7.5% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.35	Swelling
7	1.96	
50	1.86	Loading (Compression)
100	1.68	
200	1.40	
400	1.08	
800	0.75	
1600	0.29	



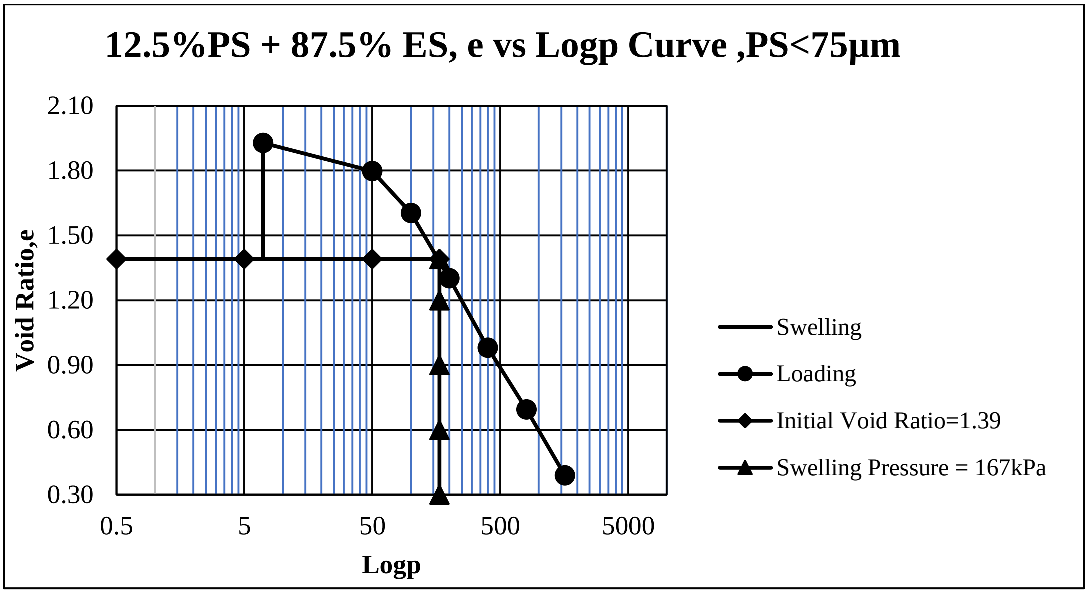
D. Swell-consolidation test for soil blended with 10% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.43	Swelling
7	1.82	
50	1.75	Loading (Compression)
100	1.61	
200	1.37	
400	1.09	
800	0.72	
1600	0.38	



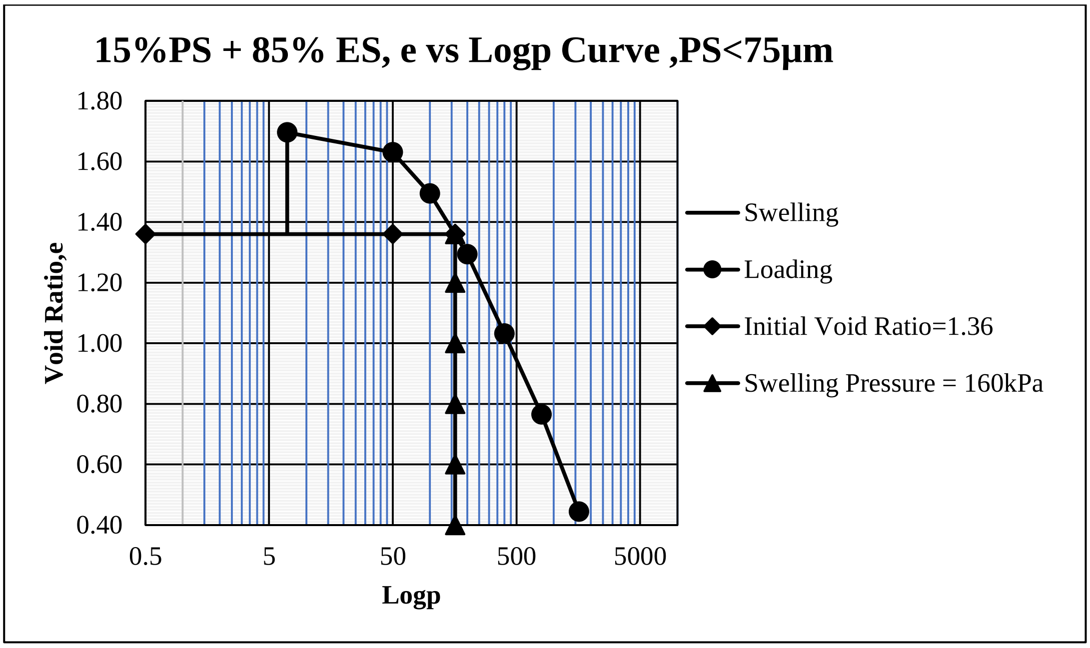
E. Swell-consolidation test for soil blended with 12.5% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.39	Swelling
7	1.93	
50	1.80	Loading (Compression)
100	1.60	
200	1.30	
400	0.98	
800	0.70	
1600	0.39	



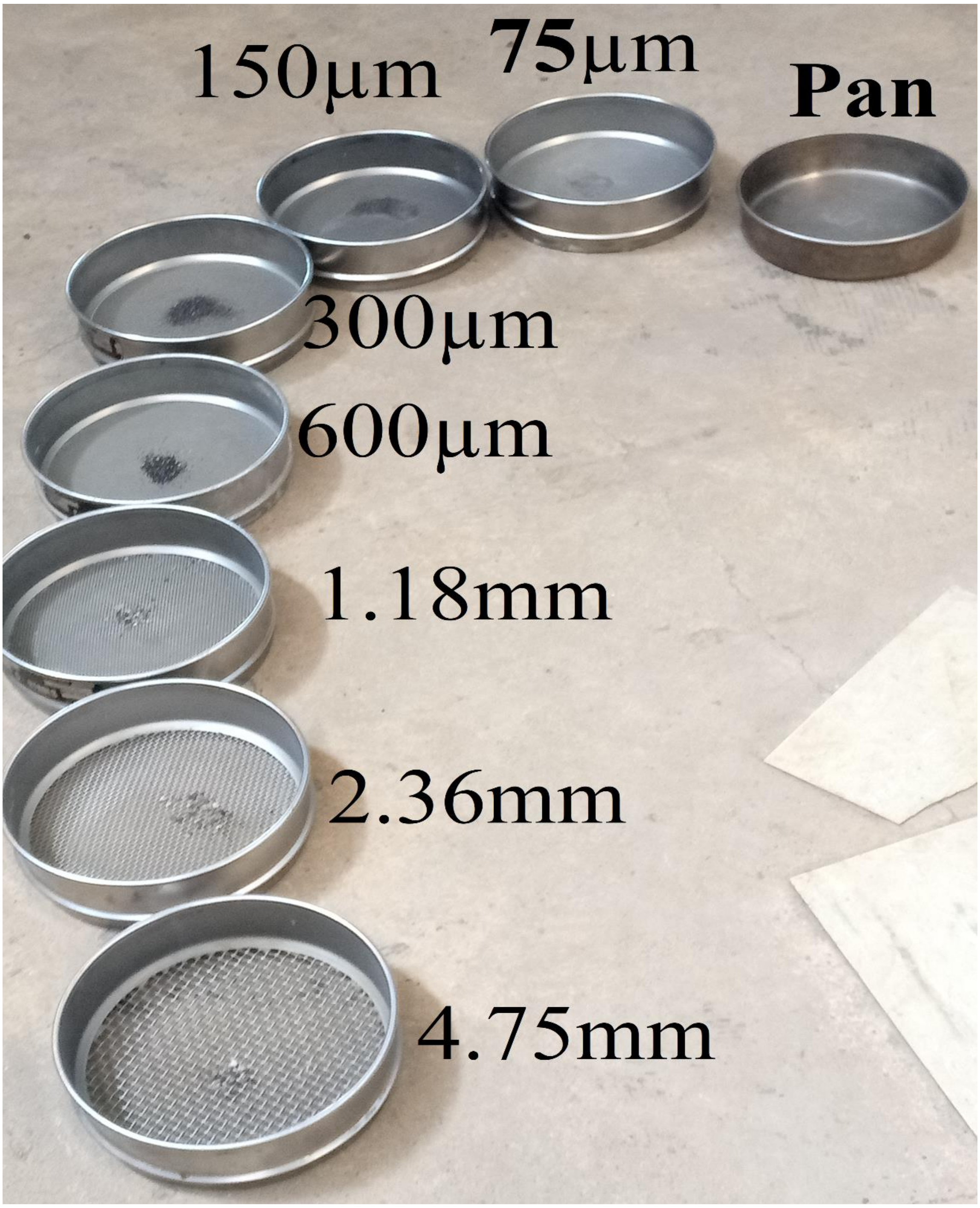
F. Swell-consolidation test for soil blended with 15% pozzolanic sand

Pressure (kPa)	Void Ratio	
7	1.36	Swelling
7	1.70	
50	1.63	Loading (Compression)
100	1.49	
200	1.29	
400	1.03	
800	0.77	
1600	0.44	



APPENDIX E: LABORATORY TEST PHOTOS

E-1: Sieve Analysis and samples retained on each sieve



E-2: Sample Preparation for compaction

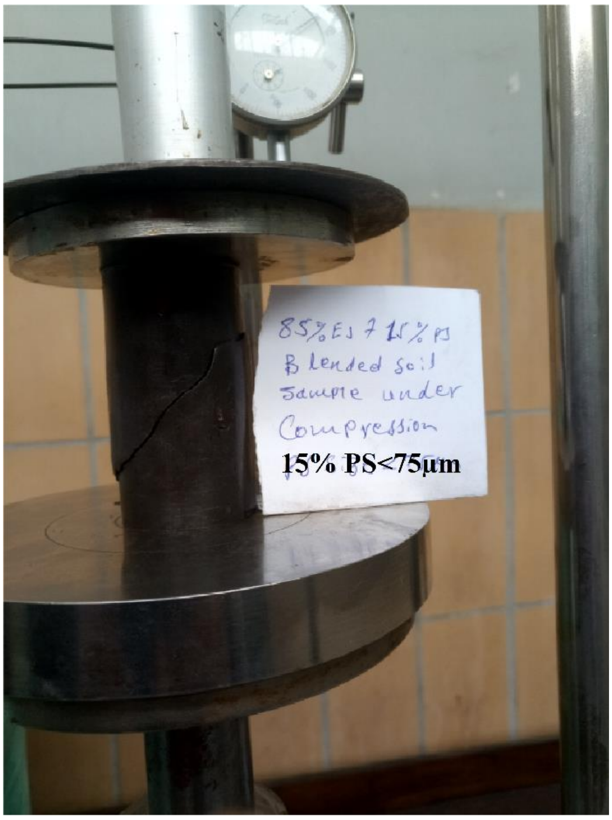
E-3: Samples under Compression for soil blended with PS<150µm


Treated Soil under Compression 15% Pozzolanic Sand
15% PS<150µm


Treated Soil under Compression 20% Pozzolanic Sand.
20% PS<150µm


Treated Soil under Compression 25% Pozzolanic
25% PS<150µm

E-4: Samples under Compression for soil blended with PS<75μm


Blended soil
sample under
Compression
15% PS<75μm

APPENDIX F: CHEMICAL COMPOSITION OF POZZOLANIC SAND

	GEOLOGICAL SURVEY OF ETHIOPIA	Doc.Number: GLD/F5.10.2	Version No: 1
	GEOCHEMICAL LABORATORY DIRECTORATE		Page 1 of 1
Document Title:	Complete Silicate Analysis Report	Effective date:	May, 2017

Issue Date: -09/07/2020

Customer Name: -Wolde Ketema.

Request No:- GLD/RQ/417/20

Report No:- GLD/RN/401/20

Sample type:- Rock

Sample Preparation: - 200 Mesh

Date Submitted: - 19/05/2020

Number of Sample:- One (1)

Analytical Result: In percent (%) Element to be determined Major Oxides & Minor Oxides

Analytical Method: $LiBO_2$ FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
Wolde 3854	74.84	8.09	6.10	<0.01	<0.01	3.80	2.76	0.20	0.12	0.05	0.62	3.70

Note: - This result represent only for the sample submitted to the laboratory.

Analysts	Checked By	Approved By	Quality Control
Yirgalem Abriham Tizita Zemene Nigist Fikadu	Yohannes Getachew	Gosa Haile	Negash Worku