ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

IMPROVING THE CHARACTERISTIC OF COLLAPSIBLE SOIL USING PUMICE

By: Yehamleshet Bekele

Advisor: Dr.-Ing. Samuel Tadesse

A Thesis Submitted to

School of Graduate Studies in Partial Fulfillment of the

Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

Improving the characteristic of collapsible Soil using pumice

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

"IMPROVING THE CHARACTERISTIC OF COLLAPSIBLE SOIL USING PUMICE"

By:

Yehamleshet Bekele Student	Signature	27/12/2022 Date
Dr. - Ing. Samuel Tadesse Advisor	Signature	27/12/2022 Date
Dr. Ing. Henok Fikre Internal Examiner	Signature	7/4/2023 Date
Dr. -Ing. Tensay Gebremedhin External Examiner	Signature	07/04/23 Date
School Dean	Abrham Gebre (Dr.) Dean, School of Civil & Environmental Engineering Signature	Date
Post Graduate Director	Signature	Date

Director of Postgraduate Program
Addis Ababa University
Addis Ababa Institute of Technology

AAiT School of Graduate Studies Page i

Improving the characteristic of collapsible Soil using pumice

DECLARATION

I hereby declare that the thesis entitled ''Improving the Geotechnical Property of Collapsible Soil Using Pumice'' is my original work performed under the supervision of Dr.- Ing. Samuel Tadesse submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirement for the Degree of Master of Science in Geotechnical Engineering. I further declare that this work has not been submitted to any other institutions for the award of any degree and all sources of materials used for the thesis have dully acknowledged.

Yehamleshet Bekele

Signature: ____

Addis Ababa, Ethiopia

2022

AAiT School of Graduate Studies Page ii

Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
CI	Collapsible index of soil
ERA	Ethiopian Roads Authority
LL	Liquid Limit
PL	Plastic Limit
MDD	Maximum Dry Density
NGL	Natural Ground Level
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
CaCO3	Calcium Carbonate
SiO2	Silicon Oxide
Al2O3	Aluminum Oxide
Fe2O3	Iron Oxide

UNITS

cm^3	Centimeter cube
gm	Gram
g/cm^3	Gram per centimeter cube
kN	Kilo Newton
kN/m^2	Kilo Newton per meter square
kpa	Kilo Pascal
mm	Millimeter

ACKNOWLEDGEMENT

First and foremost, praises and thanks to the God, the Almighty, for his help in all aspects of my life and for giving me courage to complete the research successfully.

I would like to express my deep and sincere gratitude to my research advisor Dr. Ing. Samuel Tadesse for giving me his valuable advice, support and supervision as well as providing the necessary information throughout this research work.

Special thanks go to Best Consulting Engineers Plc. for letting me to use and carry out relevant laboratory tests in their central Laboratory. In addition, my sincere thanks goes to the staff of the Best Consulting Engineers Laboratory, especially to Mr. Getachew Baro for his gentle helps.

I would like to thank Mr. Ashenafi Gezahighn, Senior material engineer on Ziway Arsi Negele road project, for his help in material, manpower and transport arrangement during sample extraction. I would like also to thank Dangote Cement Factory staffs at Meki site for providing me pumice.

To my beloved Mother and Father who was always so patient and understanding, it would not have been possible without their prayers and motivational words. To my wonderful brothers, sisters, and their families for all their love, support, and encouragement.

Last but not least, I would like to thank my friends who shared their knowledge and discussed my problems during the work of the project.

ABSTRACT

Collapsible soils, which are sometimes referred to as metastable soils, are unsaturated soils that undergo a large change in volume upon saturation with or without loading which pose problem to civil engineering structure. Many researches have been done and variety of methods and techniques have been proposed for improving the engineering property of this soil to make suitable for civil engineering structure built on it. Some techniques used for improving collapsible soils are partial excavation and replacement with compacted fill, dynamic compaction and chemical stabilization. Alternatively, use of other local abundant available pumice material as a means improving is one method which reduce damages on structure.

This study was undertaken on collapsible soil sample taken from Ziway-Adamitulu Woreda. The change in engineering properties of this soil investigated with addition of increasing percentage of pumice. Pumice is the most locally abundant available material around the aforementioned area. The preliminary investigation of the soil to be improved initially characterized and classified based on indicative tests and found to be A-4 according to AASHTO and SM class of soil according to USCS systems. Compaction, direct shear, CBR and collapsible potential tests were used to evaluate engineering properties the improved soil. Field density tests also conducted to determine the in-situ densities of the soil.

Analysis of the results shows that the collapse index of natural and compacted untreated soil 6.720% with moderate severe disorder soil at bulk and 2.235% with moderate disorder soil at OMC respectively. Treating this soil with pumice reduce collapsible index from moderate disorder soil (2.235%) to slight disorder soil (0.329) % at optimum content of 25 % pumice according to ASTM D5333. Similarly, MDD decrease and CBR, Shear strength and OMC values increased as pumice content is increased. Curing has significant effect on the geotechnical properties of pumice-stabilized soil. Therefore, this experimental result contributes in solving the problems of soil collapse by treating to enhance its resistance.

CHAPTER ONE: INTRODUCTION-

1.0 General Background

An increase in population levels has resulted in the extensive of land use for construction and may face problematic soils, like collapsible one. Construction over problematic soils is a common problem in many parts of the world. Therefore, it is critical for geotechnical engineers to have adequate knowledge to deal with these problematic soils.

According to Evans (2004) there are numerous types of naturally occurring soils having the potential to cause problems for a variety of reasons, including collapse, expansion, settlement, low bearing capacity and other factors that would cause difficulties for the civil or geotechnical engineer. Expansive clays, peat, quicksand, glacial deposit, and collapsible soils are all examples of naturally occurring problem soils, as well as man-made problematic soils and fills. The specific problems posed by naturally occurring soils with potential for collapse under loading (for example by a building or during construction works) or wetting (such as during the rise of groundwater) are an existing and on-going geological hazard that should be considered when engineering work is planned, conducted or completed. Different types of problematic soils have been recognized, however, one of the most significant ground related hazards to the built environment is collapsible soils.

Collapsible soils are metastable, unsaturated soils with a porous structure. They are often in the form of deposits. Because of the natural cementation between the grains, this type of soil can withstand high loads in the dry state; but, even without the addition of loads, saturation causes the inter-granular bonds to fail, resulting in a denser structure and the collapse of the soil. These soils are generally composed of a predominant percentage of silts and a small percentage of fine elements (Bellil et al., 2018).

Although collapsible soils can sustain high applied vertical stress without showing a significant change in volume in an unsaturated condition, the settlements associated with the percolation of water into the system often led to expensive repairs (Gaaver, 2012). This can yield disastrous consequences for structures built on such deposits.

Collapsible soils are found in many parts of the world, typically in arid and semi-arid climate areas. (e.g. United States of America, Brazil, Egypt, Kuwait, South Africa, and China). (Al-Rawas ,2000). Further to the natural collapsible soils, any type of soil compacted at ‘‘dry-of-optimum’’ conditions and at a low dry density may develop a collapsible fabric or metastable structure (Pereira et al., 2000).

A sudden reduction in volume, due to an increase in the water content, can cause damage to the lightly loaded structure including houses, buildings, roads, and pipelines; a failure in earth dams, and slope failure. There are several examples of construction failure associated with collapsible soils that have led to tremendous maintenance costs. Damage occurring as a direct or indirect result of collapse can be enormous, in terms of both economic and human loss (Lawton et al., 1992). Therefore, the identification, characterization and, prediction of the collapsible potential of the soil as well as proposing new solutions to mitigate the problems related to collapsible soils are critical issues for geotechnical engineers.

Methods for remedying collapsible soil are generally replacing the collapsible soils by suitable building materials, compaction of the soil, an appropriate drainage system to prevent the soil from getting wet, chemical stabilization or injection, pre-wetting or wetting the soil in a controlled manner, dynamic compaction, using pile foundations, using water explosion and compaction of collapsible soils by explosion energy (Moayed & Kamalzare, 2015).

Among the above-mentioned methods, in this study, more emphasis is given to reduce and minimize the effect of volume change and increase the bearing capacity and shearing resistance of these soils by stabilization using natural pumice material. The purpose of soil stabilization is to improve the engineering characteristics of the soil. A widely practiced method is to add cement, lime, bitumen, or special additives to bind the particles of the soil or strengthening the existing bonds. Nontraditional stabilizers such as Nano-silica and fly ash are also applied for soil stabilization (Hosseini et al., 2019). The use of granulated slag and natural pozzolan to treat collapsible soils resulted in a significant reduction in collapse potential (I_c) and a decrease in suction (Ziani et al., 2019).

Stabilization using naturally available materials has several advantages such as they are cost-effective and preserve the environment. The method used is exclusively experimental and conducted on samples of soils reconstituted in the laboratory. Such method has been selected for this study to improve the characteristics of the collapsible soil and to indicate that it is possible to reduce the collapse potential (C_p) to an acceptable level.

Very few researches were carried out here in our country to study the engineering properties of collapsible soils. However, no attempt was made so far about improving the characteristics of collapsible soil using pumice. This study investigates the engineering properties of collapsible soil and identifies problems associated with such soils and the possible ways/mechanism of improving soil collapse problem using natural pumice.

Tewodros (2019) discussed that no satisfactory research and investigation has been made and published on collapsible soils found in Ethiopia. Many of the focus areas of the researches undertaken so far have emphasized on expansive soils, among any other problematic soils, and less attention is given on the existence of collapsible soils. Due to the above-mentioned reasons, there is no extensive knowledge, even within the experienced engineering professionals, on the methods of identification, classification, and treatment of collapsible soils.

To summarize; in our country this type of soil is not studied sufficiently, but now days different infrastructures are under construction in areas where this soil is commonly found. So, in order to use this problematic soil as a good foundation support, different stabilizing techniques should be provided by researchers. Therefore, in this research an attempt was made to stabilize collapsible soil using pumice.

1.1 Statement of the Problem

Collapsible soils are unsaturated soils that undergo a large change in volume upon saturation with or without loading. Because of wetting, problems of collapse occur to civil engineering structure built on it with large and sudden reduction in volume. So, prevention or minimizing of damage due to collapse of such soil by improving its engineering characteristics make the structure stable built on it. Some of the mitigation measure for improving collapsible soils are partial excavation and replacement with compacted fill, dynamic compaction and chemical stabilization. Alternatively, Use of other local abundant available pumice material as a means improving is one method which reduce severity of collapse on structure.

This study was undertaken on collapsible soil sample taken from Ziway-Adamitulu Woreda. The change in engineering properties of this soil investigated in detail by mixing the collapsible soil with an increasing percentage of pumice in the laboratory.

1.2 Research Objective

1.2.1 General Objective

The main objective of this research is improving the engineering properties of collapsible soil using pumice.

1.2.2 Specific Objective

The specific objectives of the study are listed as follows:

- ✓ To assess the engineering properties of the natural collapsible soil by conducting index and classification tests as well as strength tests.
- ✓ To investigate the effects of pumice on engineering properties of collapsible soil. Such properties include compaction characteristics, collapse potential, CBR, and shear strength.
- ✓ To assess the collapsible potential of natural soil and to reduce the collapsible potential of the soil after stabilization.
- ✓ To reduce or minimize the effect of volume change (settlement) of collapsible soils.

1.3 Scope of the Research

This study has been supported by secondary data and a series of experiments. The experimental work carried out as part of this research consisted of field and laboratory investigation. The field portion involved the determination of in-situ density and collecting disturbed and undisturbed samples. The laboratory portion involved conducting different tests on untreated and pumice-treated samples. The tests included atterberg limits, specific gravity, grain-size analysis, moisture-density relationship, collapsible potential, direct shear, and one-point CBR tests.

The findings of the study are limited to soil samples considered in the research which are collapsible and specific to the type of additives used i.e. pumice. The research is limited to Ziway area soil around Adamitulu Woreda from a single test pit.

1.4 Application of the Result

Due to the severe damage inherent in collapsible soils, a better understanding of the problem is essential. Accordingly, a study of the natural behavior of collapsible soils and their possible remedy would be important. Therefore, the results obtained from this research will be important to overcome the problems associated with such type of soil, and reducing damages for the structures constructed on this collapsible soil.

In addition; the results from this study can be used to understand better about the nature of the collapse phenomenon through a detail investigation of the natural characteristics of collapsible soil and to introduce the effectiveness of pumice treatment in solving the problem.

The study can also be used as a guide for experts in the field, as well as an indicating how to improve collapsible soils in the construction industry while minimizing the initial costs of new projects.

1.5 Methodology

The research was conducted using a literature review on collapsible soils, field observations, collection of samples for laboratory tests, analysis of the laboratory test results.

The study started by reviewing secondary sources from the literature, journals, previous studies and the internet. Both primary data (collected personally) from the source itself (such as experiments, observations, and photograph records) and secondary data from different research works are collected and used for the analysis. The analysis of the data collected is qualitative and quantitative. In general, the following methodology is developed to achieve the objective of the research.

Literature review: various academic sources; such as textbooks, academic journals, seminars and research papers were reviewed in order to have different soil improvement techniques. Those literature reviews were conducted; to understand the engineering properties of collapsible soils, to identify problems associated with such soils and to investigate possible ways of improving the problematic nature of collapsible soils.

Sampling and laboratory testing

To assess the effects of collapsible soil improvement by pumice material, various tests have been conducted on the soil samples collected from the Ziway-Adamitulu areas. Soil sampling and testing methods are used to characterize the physical properties of the soil. These methods are undertaken as per ASTM and AASHTO standards.

The laboratory tests conducted before improvement on natural soil samples include: specific gravity, particle size determination, atterberg limits, moisture-density relationship, collapsible potential test, direct shear test, and one-point CBR test. The laboratory test that was conducted for blended samples are: moisture-density relationship, collapsible potential test, direct shear test, and one-point CBR test.

Steps for laboratory testing

- ✓ Preparation of soil samples and stabilizers (pumice).
- ✓ Carry out different laboratory tests on the soil samples.
- ✓ Carry out tests on different types of treated samples.

Analysis of the results

Based on theoretical backgrounds and laboratory test results, the optimum amount of pumice required to stabilize the soil under investigation was determined and, the engineering characteristics of the soil have been assessed for the collapsible soil and its combination with pumice. Using descriptive statistics, the data was analyzed and interpreted.

Conclusions and recommendations: are deduced based on critical interpretation of results.

1.6 Organization of the Thesis

The presentation of this thesis work is organized into six Chapters.

- **Chapter one** contains the introduction part, which includes the background of study, problem statement, objectives of the study, scope, methodology, application of the result, and organization of the thesis.
- **Chapter two** presents a literature review: It includes genesis of collapsible soils, collapse mechanism, characteristics of collapsible soil, identification of collapsible soils, classification of collapsible soil, distribution of collapsible soils, problems associated with collapsible soils, and mitigation measures against collapsible soils.
- **Chapter three** describes the study area according to climate, geology, and geographical location.
- **The fourth** Chapter deals with the characterization of materials used for the study, field and laboratory testing procedures followed.
- **Chapter five** comprises the test results obtained; analysis of results and discussions of results with respect to the theoretical background and findings of previous studies.
- **Chapter six** presents conclusions and recommendations drawn from this research.

CHAPTR TWO: LITERATURE REVIEW

2.1 Overview

Unfortunately, many soils can prove problematic in geotechnical engineering, because they expand, collapse, disperse, undergo excessive settlement, have a distinct lack of strength, or are corrosive. Soil compositions, the nature of their pore fluids, their mineralogy, or their fabric are the factors responsible for such characteristics (Bell and Culshaw, 2001). The present study is concerned on the collapsibility problems of soil.

This chapter is intended to review the literature about the collapse phenomenon, such as definition, origin, and collapsible mechanisms, distribution of collapsible soil, problems associated with collapsible soil, and mitigation measures against collapsible soil. It also covers methods of identification of collapsible soils used to measures the degree of collapse.

2.2 Definition

Many definitions of collapsible soils can be found in the literature; hence it is essential to demonstrate some of them for understanding its behavior. Collapsible soils can be defined as follows:

‘’Collapsible soils are those unsaturated soils that can withstand relatively high pressure without showing significant change in volume, however upon wetting; they are susceptible to a large and sudden reduction in volume’’ (Gaaver, 2012).

“Collapsible soils are natural materials where the combination of particle type and sedimentation mechanism give collapsibility. Collapsible soils are ones that appear to be strong and stable in their natural state, but collapse rapidly when wet, resulting in large and often unexpected settlements. This can yield disastrous consequences for structures built on such deposits. Such soils are often termed “collapsible” or “metastable” and the process of their collapse is often called any of “hydro-consolidation”, or “hydro-collapse.” (Abdelmohsen and Ali,2014).

Cementing agents that exist in the structure of collapsible soils stabilize the open and partially unstable fabric, and thereby give the soil high bearing capacity in the unsaturated state. However, the addition of water to the system, along with high pressure on top of the soils, softens the inter-particle bindings and leads to a critical reduction in volume (Bigdeli, 2018).

.

Pereira et al. (2000) outlined the factors that produce collapse as follows:

1. An open, partially unstable, unsaturated fabric
2. A high enough net total stress that will cause the structure to be metastable
3. A bonding or cementing agent that stabilizes the soil in the unsaturated condition,
4. As water is added to the soil, the bonding or cementing agent is reduced, and the inter-aggregate or inter-granular contacts fail in shear, resulting in a reduction in the total volume of the soil mass.

2.3 Characteristics of Collapsible Soils (Features of Collapsible Soils)

Zamani & Badv (2019) presented some of the common typical features that are found with most collapsible soils. The following characteristics are useful in determining the causes of collapsing soils:

- open physical structure
- high porosity (more than 40%)
- low dry density
- low saturation degree (less than 60%)
- young geological formations that had less time to obtain their natural density
- strongly affected by changes in moisture content or effective stress level
- high silt content (more than 30%, sometimes up to 90%) and low clay amount
- low resistance in inter-granular forces

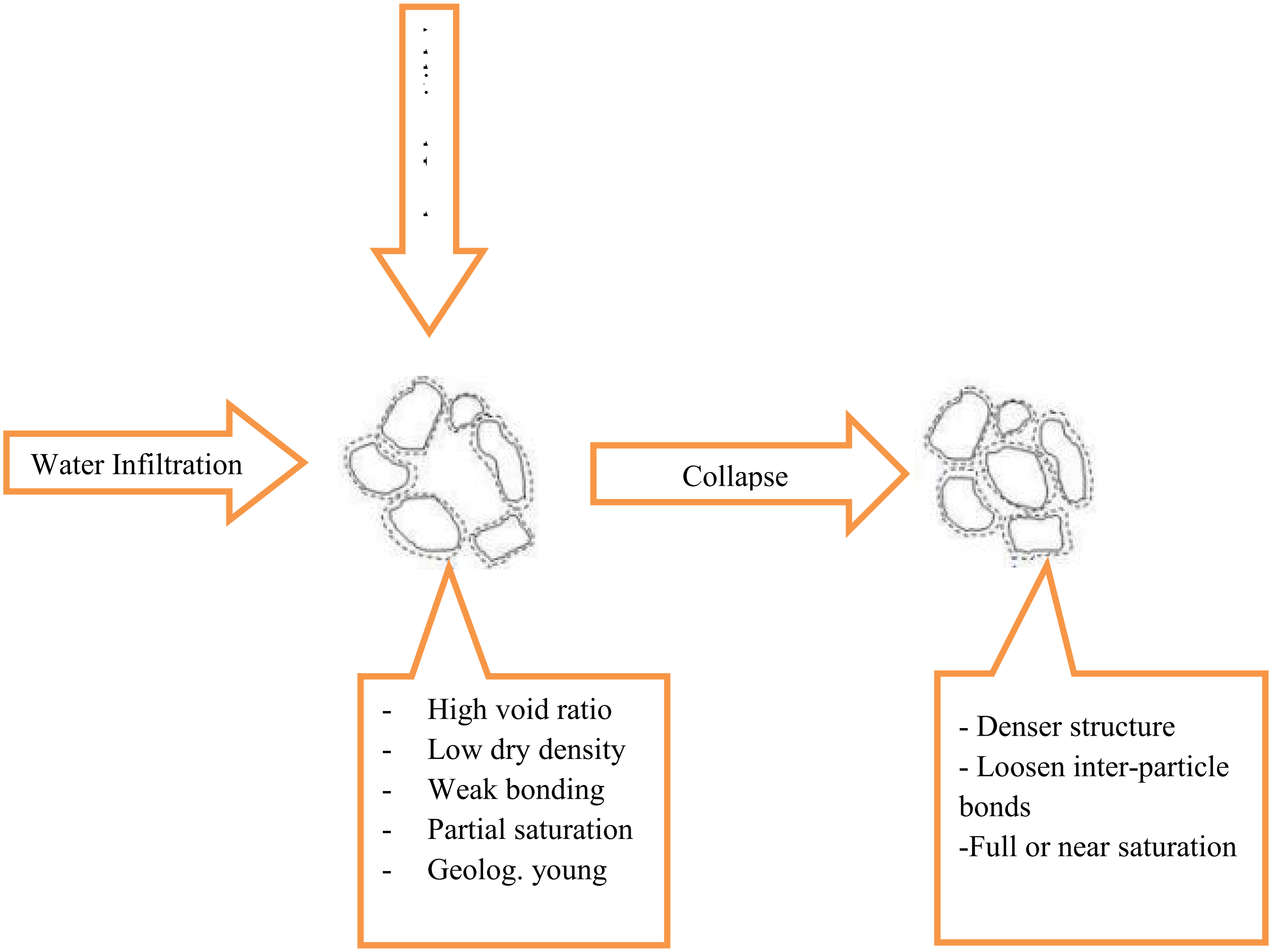


Figure 2. 1: Schematic view of the characteristics of collapsible soils before and after the collapse

According to Houston et al. (2001) in addition to natural soils that exhibit some of the above characteristics and can be classified as naturally deposited collapsible soils, engineered compacted fills may exhibit volume moisture sensitivity if compaction specifications and quality control are not appropriate. Compaction to low dry density of optimum produces the greatest susceptibility to densification upon wetting, but almost any compacted soil can exhibit collapse if the confining pressure is sufficiently high.

2.4 Conditions Leading to Collapse

Charles and Bruce (2007) have put some discussion on the conditions leading to collapse. In their books, they explained that under normal circumstances naturally occurring collapsible soils are not wetted to significant depth by precipitation. Rainfall either runs off or infiltrates only a short distance and then evaporates to the surface, particularly in arid regions. Problems with collapsible soils are almost always associated with man-induced changes in the surface water and groundwater regime. This is often coupled with a failure to identify the collapse condition before construction and/or a lack of knowledge of potential wetting sources. Collapse, triggered by increased water content, can typically be linked to engineering modifications and alteration of natural flow patterns. In some cases, the overburden stresses alone are sufficient to drive the collapse process when wetting occurs. When cementation is very strong or the collapsible soils are very shallow, additional stress due to a structure or foundation may be necessary for collapse to happen. Often collapsible soils go undetected until a structure is already in place. Even if collapsible soils are identified prior to construction, a lack of knowledge of potential sources of wetting can lead to incomplete mitigation of the problem. For example, due consideration of rising groundwater table may not be given, or infiltration may extend to a greater depth than assumed.

2.5 Mechanism of Collapse

Since the first, recognition of the collapse phenomenon, a great effort has been made to thoroughly understand the mechanism of collapse and the behavior of collapsible soils.

Al-Rawas (2000) has presented Casagrande's (1932) demonstration which states that a portion of the fine-grained fraction of the soil exists as bonding material for the larger-grained particles. These bonds undergo local compression in the small gaps between adjacent grains resulting in the development of strength.

Jennings and Knight (1957) substantiated and idealized Casagrande's concept to explain the collapse mechanism. When the soil is loaded at its natural moisture content, the soil structure remains reasonably unchanged and the bonding material compresses only slightly without relative movement of the soil grains. When the loaded area is wetted and certain critical moisture content is exceeded, the fine silt or clay bridges come to the stage where they can no longer resist the deformation forces. Collapse of the grain structure takes place with reduction of the volume of the soil (Alfi, 1984).

Barden et al. (1973) conducted a comprehensive electron microscopic study of the metastable structure of different types of collapsing soils from various origins. The various possible structures of collapsing soils are shown in Figure 2.2 Their study indicated that a common basic collapse mechanism applies to the different types of soil ranging from sand to clay. Various bonding between particles is possible. The most recognizable are capillary suction, silt and/or clay bridges or buttresses, and cementing agents. Any combination of these bonding sources and the various types of collapsing soil structures will form a collapsing mechanism. There are three main bonding mechanisms present in collapsible soils.

i. In cases when the soil consists of sand with a fine silt binder, it is assumed that simple capillary forces provide the silt-silt and silt-sand bond. Figure (2.2a & 2.2 b).
ii. Clay plates in the bonds between bulky sand and silt grains, on the other hand, are responsible for the majority of collapsing soils. Depending on the geologic origins and history of the soil, a number of structural arrangements of the clay plates are possible. As shown in Figure (2.2c), when the clay is formed in place by authigenesis, it may form a parallel plate onion-skin effect around the quartz particles. Alternatively, if the clay was originally in suspension in the pore water, then gradual evaporation would cause the clay plates to retreat with the water into the menisci at inter-particle contacts. Knight (1960) indicates that under such conditions the clay would form a random flocculated structure as illustrated in Figure (2.2d), giving a buttress type of support to the bulky grains.

iii. Chemical cementing agents such as iron oxide, calcium carbonate, and others, which are often the main agent in loessial soils, may have a significant bonding effect in certain collapsing soils. Figure (2.2e & 2.2f).

In all cases, the clay bond possesses high dry strength, which is reduced upon saturation. The reduction of strength may be due to loss of capillary tension, increase in water content, and physicochemical action; however, it is not known how much is due to each (Alfi, 1984).

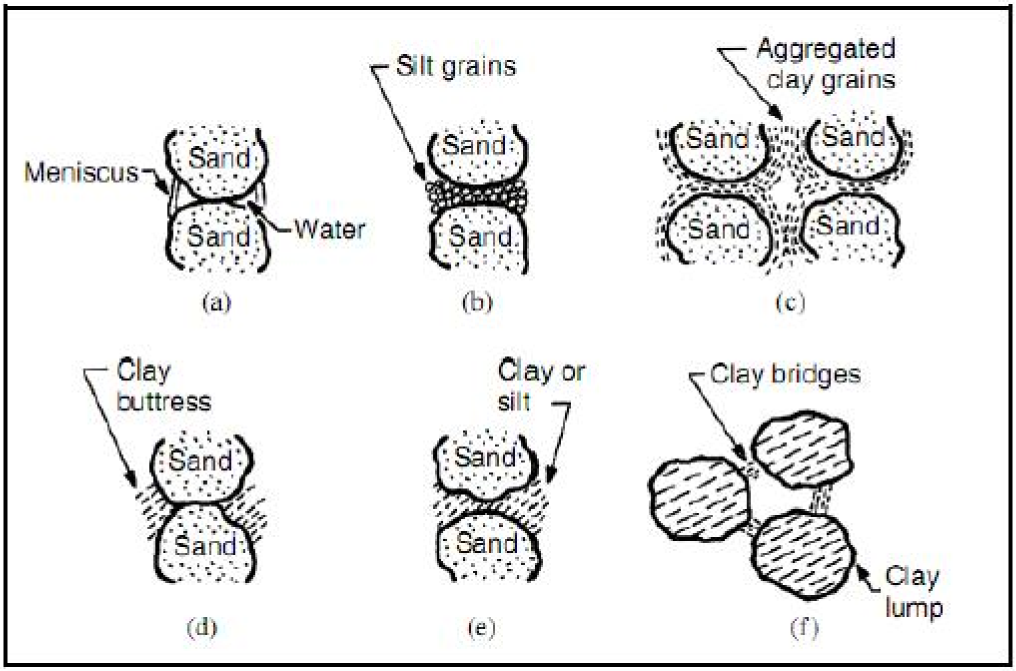


Figure 2. 2 :Typical collapsible Soil mechanism after Barden (1973): (a) capillary tension; (b) silt bond; (c) aggregated clay bond; (d) flocculated clay bond; (e) mudflow type of separation; (f) clay bridge structure.

2.6 Causes of Collapse

Kalantari (2013) describes that, Collapsible soils such as loess sands consist primarily of silt sized particles loosely arranged in a cemented honeycombed structure (Figure 2.3). Small amounts of water-softening or water-soluble cementing additives, such as clay minerals and calcium carbonate, hold the loose structure together. The introduction of water dissolves or softens the bonds between the silt particles and allows them to take a denser packing under any type of compressive loading.

The addition of water is a widely used for triggering soil collapse. Collapse can also occur, however, as the result of load application, or wetting, or both. Thus, collapse can occur either by increasing the stress above the soil strength or by lowering soil strength below the stress. Whatever; the physical basis of the bond strength, all collapsible soils are weakened by adding water. Decreased strength is immediate in cases where the grains are held together by capillary suction, slow in the case of chemical cementing, and much slower in the case of clay buttresses.

Thus, it may take long periods of time, even years, for the total collapse to take place (Knodel, 1992).

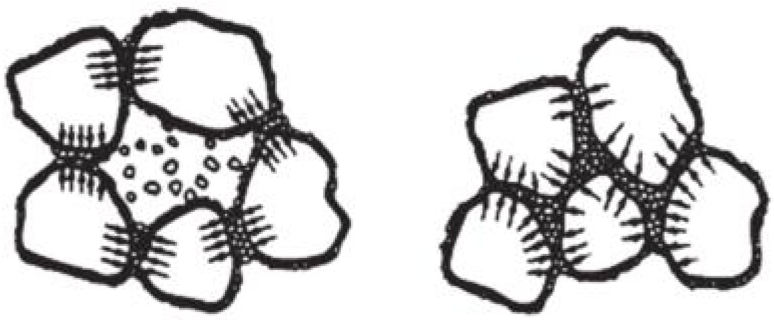


Figure 2. 3: a) Dry soil with honeycombed structure before inundation (b) Soil structure after inundation

2.7 Genesis of Collapsible Soils

According to Houston et.al. (2002), climates of arid regions and the typical soil types found in these areas give rise to soil collapse problems. In arid climates, evaporation greatly exceeds rainfall. Only the relatively near surface soils become wetted from normal rainfall. It is the combination of depositional processes and climatic conditions that lead to the formation of collapsible soils.

In addition to naturally deposited collapsible soil, this soil can be formed artificially through poor compaction control or where compaction is dry of optimum (Pereira, 2000).

2.8 Identification of Collapsible Soils

Identification of collapsible soils requires a special consideration because such soils are often overlooked during soil investigations. However, they have certain distinctive characteristics.

According to Al-Rawas (2000) the first task that a Geotechnical Engineer should perform in the evaluation of collapsible soils is to visually examine the soil encountered at a particular site. This step is followed by the evaluation of the soil behavior using index tests to determine the likelihood of the soil for collapse. Once the soil has shown indications of collapse by visual examination and index tests, further laboratory tests are required to determine the magnitude of collapse. Several collapse criteria based on some parameters such as dry density, atterberg limits, moisture content, void ratio, clay content and porosity.

2.8.1 Geological Reconnaissance

Local site geology and depositional processes should be considered first in assessing the collapse potential of the soil profile. Together with climatological data, this information can be a very useful tool in planning the site investigation. In the case of an existing roadway, the geomorphological clues can be coupled with knowledge of performance to help identify possible problem soil conditions. Geographical and geological information is strongly correlated with collapsibility and collapse potential. Engineering experience and familiarity with local infrastructure performance and geological conditions is an essential element of the site characterization (Houston and et al., 2002).

2.8.2 Indirect Correlations

Qualitative and semi-quantitative correlations have been developed between collapse potential and various index properties. Of course, a low initial density is a good predictor of collapse potential, but some soils with moderately high density have also exhibited significant collapse. The water content at saturation is another index property that is closely related to density. When the water content corresponding to full saturation significantly exceeds the liquid limit, major collapse potential is indicated. Correlations with seismic velocity, SPT N-values, and CPT tip resistance have also been attempted with low to moderate success. Many of these correlations with index properties have the drawback that they are typically weak, with considerable scatter, and in most cases the quality of the collapse potential prediction is not high enough to be reasonably used for subsequent settlement analyses (Houston et al., 2001).

Several investigators have proposed various methods for evaluating the physical parameters of collapsing soils for identification. Das (2011) provided a summary of some soil collapse criteria based on different parameters. Some of these methods are discussed briefly in the next section.

- Abelev (1948) was the first to suggest the following criterion for evaluation of soil collapse potential due to variation of soil void ratio before and after saturation as follows: (Esmaeli ,2008).

$\delta_s = (\Delta e / (e_L + 1))$ % (2.1)

Where: Δe = void ratio reduction during soil saturation

e_L= void ratio before soil saturation

Regarding the above criterion If δ_s is greater than 2 percent then the soil will be susceptible to collapse.

- Denisov (1951) used the void ratio at liquid limit to natural void ratio as follows:(Das,2011).
  K = void ratio at liquid limit / natural void ratio

  K=Coefficient of subsidence
  If, K= 0.5 – 0.75: highly collapsible soil;
  If, K= 1.0: non collapsible loam;
  If, K= 1.5 – 2.0: non collapsible soil

- Priklonski (1952) identified the collapsible soil by the follows: (Das, 2011).
  $K_D = (W_n - PL) / PI$ (2.2)
  where K_D= Liquidity Index, Wn= natural moisture content, PL= plastic limit, and PI= plasticity index
  If, $K_D < 0$: highly collapsible soils
  If, $K_D > 0.5$: non collapsible soils
  If, $K_D > 1.0$: swelling soils

- Clevenger (1958) proposed a criterion based on dry density, that is, if the density is less than 12.6kN/m^3, then the soil is liable to significant settlement. On the other hand, if the dry densiy is greater than 14.1kN/m^3, then the amount of collapse should be small, while at intermediate densities the settlements are transitional (Al-Rawas, 2000).
- Holtz and Hilf (1961) suggest that the density of collapsible soils together with their liquid limits (LL) may be used to estimate collapse potential. They provide a chart to evaluate collapsibility of various types of soils (Figure 2.4). The graph shows that soils that plot above the line are prone to collapsing when wet, and that as their dry densities decrease, the severity of collapsibility increases (Kalantari, 2013).
- Gibbs and Bara (1962) used the dry unit weight and liquid limit as criteria to distinguish between collapsible and non-collapsible soil types. The method is based on the premise that a

soil, which has enough void space to hold its liquid limit moisture content at saturation, is susceptible to collapse up on wetting (Charles and Bruce, 2007).

Collapse ratio = (water content at saturation)/liquid limit

A soil with a collapse ratio equal to or greater than unity will be near a liquid state when saturated and therefore subject to collapse.

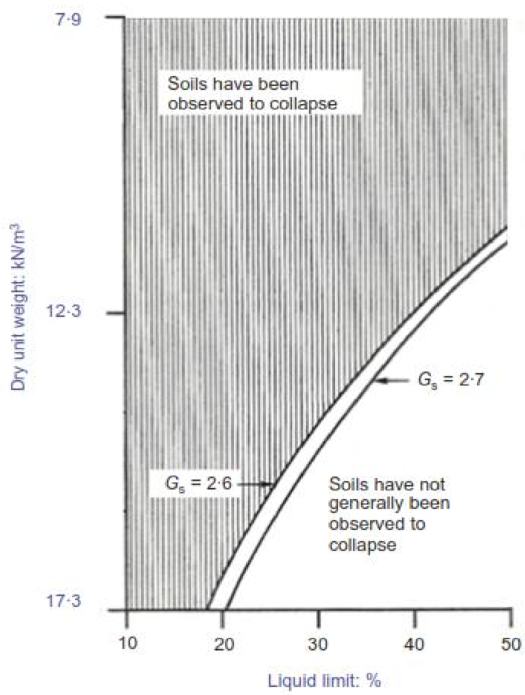


Figure 2. 4: Identification of collapsible and non-collapsible soils (After Holts and Hilf, 1961), Gs, specific gravity of soil material

- USSR 1962 Building Code. The soil is considered susceptible to collapse upon wetting if the in-situ degree of saturation (S) is less than 60 percent and $\lambda \geq -0.1$(Das, 2011).

  $\lambda = (e_o - e_L / 1+e_o)$ (2.3)

  Where (e_L) is the voids ratio corresponding to liquid limit and (eo) is the natural voids ratio.

- Feda (1964) identified collapsible soils as follows :(Das, 2011).

  $K_L = (w_o/S_r)–(PL)/(PI)$ (2.4)

Where: w_o= natural water content, S_r= natural degree of saturation, PL =plastic limit, and PI = plasticity index

For Sr< 100 %, if K_L> 0.85, the soil is collapsible.

- The percentage clay content or the ratio of liquid limit to saturation moisture content, according to Handy (1973), may be used to determine collapsibility (Das, 2011).
    - Clay content of less than 16 percent had a high probability for collapse;
    - Clay content of between 16 and 24 percent were probably collapsible;
    - Clay content between 25 and 32 percent had a probability of collapse of less than 50 percent;
    - Clay content which exceeded 32 percent was non-collapsible.
- Zur and Wiseman (1973) used the density versus liquid limit as a collapse criterion given by as follows:(Hamid, 2012).

  $D_o / D_{LL} < 1.1$; soil prone to collapse

  $D_o / D_{LL} > 1.3$; soil prone to swell

Where Do is the in-situ dry density and D_{LL} is the dry density of soil at full saturation and at moisture content equal to the liquid limit.

- Sabbagh (1982), According to (Zamani & Badv, 2019).

  $$Cp = \Delta H/H_0 \qquad (2.5)$$

  ΔH is the difference between the initial height of the sample (before saturation) and the final height of the sample (after saturation) and also H_0 is the initial height of the sample in the oedometer test

  If $0 < Cp \leq 2\%$, the soil is low collapsible (LC)

  If $2\% < Cp \leq 5\%$, the soil is moderately collapsible (MC)

  If $Cp > 5\%$, the soil is highly collapsible (HC)
- According to Lin and Wang (1988) criterion the collapsibility index of soil in self weight condition is defined as follow:

  $$i_{CZ} = h_Z - h_{ZS}/h_1 \qquad (2.6)$$

Where h_Z and h_{ZS} are the soil sample thicknesses in odometer test regarding overburden pressure in natural and saturation conditions respectively and h1 is initial soil sample thickness. The soil condition and tendency to be collapsed are summarized below: (Esmaeli, 2008).

- (0-1)% - No Collapsibility
- (1-5) % - Medium Collapsibility
- (5-10) %- High Collapsibility
- (10-20) % -Very High Collapsibility

- (>20) % - Extremely Collapsible

2.8.3 Direct Evaluation of Collapse Potential

Prediction of collapse potential and measurement of collapse strain could be outlined, either in laboratory by performing oedometer test or in field by performing plate load test.

2.8.3.1 Laboratory Testing

The most common laboratory collapse test is the one-dimensional response-to-wetting test, which is performed with conventional consolidation equipment.

However, performance of laboratory response to wetting tests on undisturbed specimens likely represents the most widely used and economical method for identification of collapsible soils. Of course, the quality of the nature specimen used in the laboratory oedometer test is of concern in making quantitative estimates of collapse potential.

Double Oedometer Test

Double oedometer method is conducted based on an assumption that the “deformations induced by wetting are independent of the loading-wetting sequence” (Lawton et al. 1992).

This method uses two parallel oedometer tests on identical samples, one being tested at its natural moisture content, whilst the other is tested under saturated conditions, the same loading sequence being used in both cases (Charles and Bruce,2007). The results of such tests are shown in Figure 2.5. Two stress versus strain curves will be generated, one for the “dry” soil and one for the saturated soil. If the soil is strongly hydro collapsible, the stress-strain response for the saturated curve will be significantly different than that of the dry soil. The vertical distance between the tests results represents the potential hydro-collapse strain. The advantage of the double-oedometer test is that through a single test one can obtain a large amount of data without repeating single oedometer tests at different stress levels.

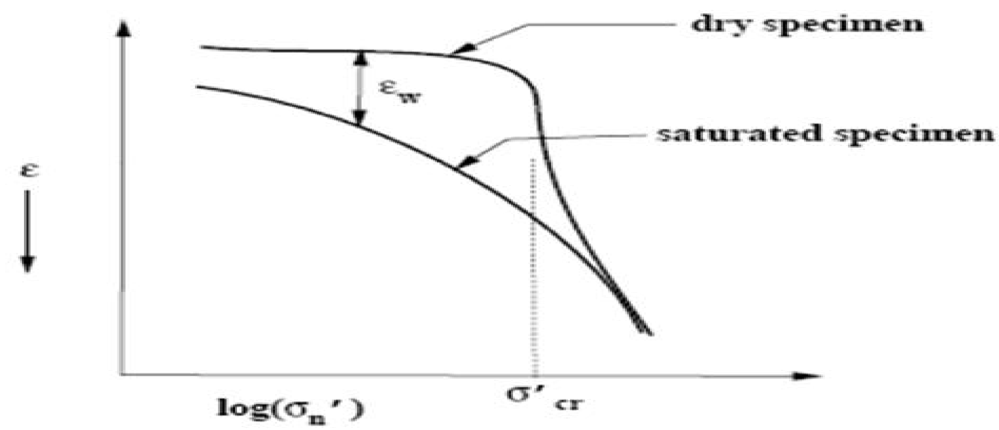


Figure 2. 5: Results of a double-oedometer test on a hydro-collapsible soil.

Single Oedometer Test

The single-oedometer test is conducted based on the ‘‘soaked-after-loading method’’ (Lawton et al. 1992). The undisturbed soil specimen at natural moisture content loaded in the conventional oedometer up to a given load. At this point the specimen is flooded and the resulting collapse strain, if any, is recorded (Figure 2.6). Then the specimen is subjected to further loading. The result of a single oedometer test will indicate whether a soil is collapsible and at the same time give a direct measure of collapse strain potential (Charles and Bruce, 2007).

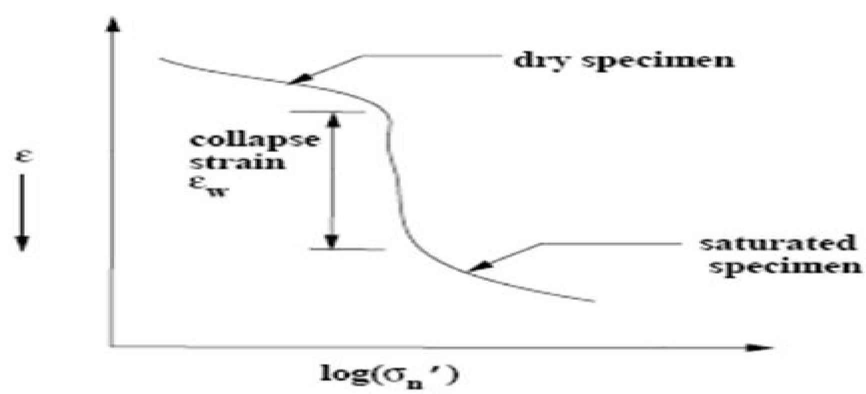


Figure 2. 6: Typical results from a single oedometer test on a hydro-collapsible soil specimen.

ASTM D5333 describes the procedure for the single odeometer test to measure the collapse potential of soils. The standard also introduces the collapse index (I_e), which is the wetting induced strain measured at a reference stress level of 200 kPa and the collapse potential (I_c), percent-relative magnitude of soil collapse determined at any stress level. ASTM D5333 specifies the severity of collapse based on the value of the collapse index according to Table 2.1.

$$I_e=\Delta h/ho*100 \tag{2.7}$$

Where: Δh = change in specimen height resulting from wetting, mm (in.) and ho = initial specimen height, mm (in.).

Table 2. 1: Classification of the degree of collapse using collapse index in accordance with ASTM D5333-03 standard

Degree of Specimen Collapse	Collapse Index I_e, %
None	0
Slight	0.1 to 2.0
Moderate	2.1 to 6.0
Moderately severe	6.1 to 10.0
Severe	>10

2.8.3.2 Field Testing

Field tests are often used to help identify and characterize collapsible soil deposits.

A. Sausage test

A very simple field test is the "sausage" test. A block of soil of about 500 cm^3 is taken from the test trial pit and broken into two pieces, and each is trimmed until they are approximately equal in volume. One specimen is then wetted and molded in the hands to form a damp ball. This ball's volume is then compared to the volume of the undisturbed specimen. If the wetted ball is obviously smaller, then collapse may be suspected. This test is only a guide as to whether or not a soil can collapse (Al-Rawas, 2000).

B. Plate load test

The most common field tests for evaluating allowable foundation pressures are plate load tests. Normally, these tests are carried out near the ground. Water is added to the loaded soil in this test, and the resultant displacement due to wetting is recorded.

Charles and Bruce (2007) reported that, in situ methods of testing for collapsible soils include shallow plate load test and down-hole plate load test. In situ measurements appear promising for identification of collapsible soils, particularly for difficult-to-sample materials. In general, sample disturbance may be greatly reduced, and a quantitative measure of collapse potential can be obtained. A quantitative measure of collapse potential in terms of strain is somewhat more difficult to obtain from in situ tests because the degree and extent of wetting, and therefore the zone of influence, is not routinely determined. Ideally, an adequate number of points within the profile can be tested so that a reasonable engineering estimate of collapse settlement potential can be made.

2.9 Classification of Collapsible Soils

Numerous soil types can fall in the general category of collapsible soils, including natural soils and compacted soils. Coduto (2001) reported that, collapsible soils are natural deposits formed by several geologic processes such as aeolian deposits, alluvial deposits, colluvial deposits and residual deposits. Lawton (1992) added that, collapse can occur also in compacted fills.

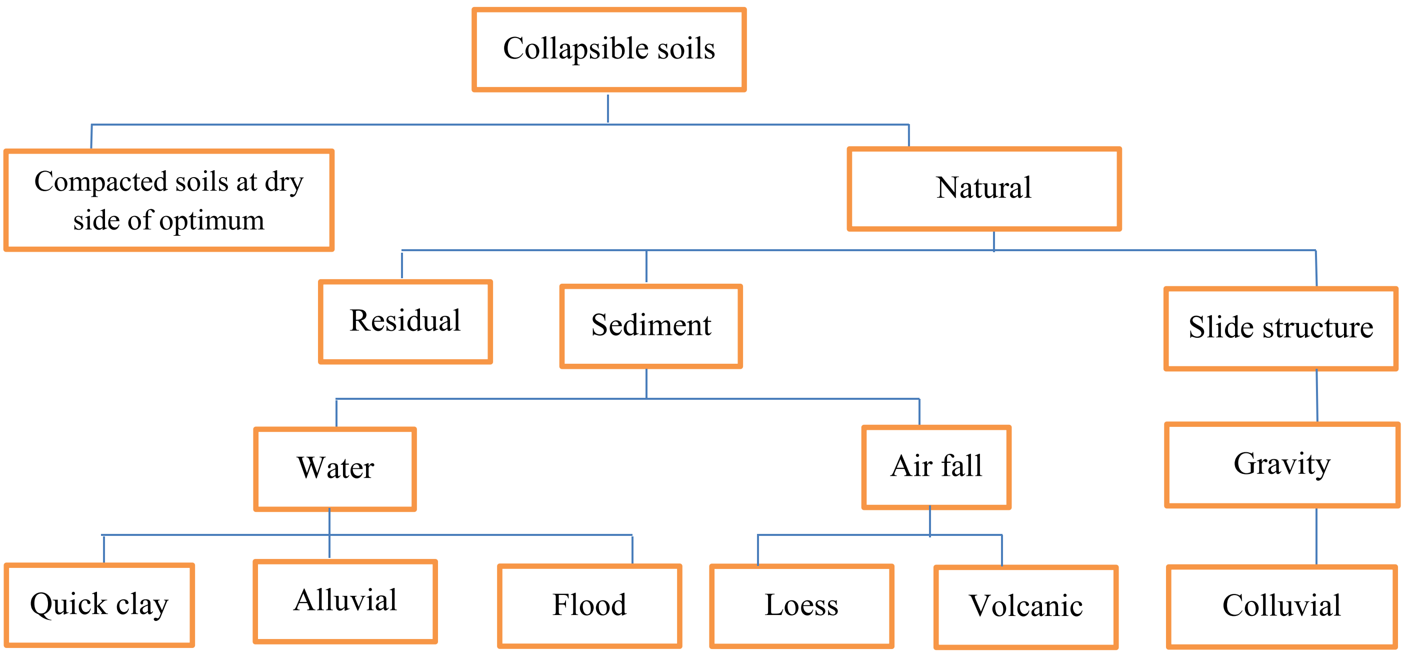


Figure 2. 7: General classification of collapsible soils (Rogers, 1995)

2.9.1 Natural Collapsible Soils Deposits

2.9.1.1 Collapsible Alluvial and Colluvial Soils

Some alluvial soils (i.e., soils transported by water) and some colluvial soils (i.e., soils transported by gravity) can be highly collapsible (Coduto,2001).

Alfi (1984) explained that, Alluvial deposits consist mainly of loose water-laid deposits, which include alluvial fans, flood-plain deposits, and mudflows. A major part of the flood-plain deposits is loess like deposits, which are sometimes mistaken for loess. Colluvial deposits are formed from deposits of loose rock debris that has accumulated at the base of a cliff or slope due to gravity effects only.

2.9.1.2 Collapsible Aeolian Soils

Soils deposited by wind are known as aeolian soils. These include windblown sand dunes, loess, volcanic dust deposits, as well as other forms. Loess (an aeolian silt or sandy silt) is the most common aeolian soil and it covers much of the Earth's surface. Collapsible loess has a very high porosity and a correspondingly low unit weight. Loess deposits are generally much less erratic than other types of collapsible soils, but they are often much thicker (Coduto, 2001).

2.9.1.3 Collapsible Residual Soils

Residual deposits are the product of weathering and mechanical alteration of the components of parent rocks in place. This process can sometimes involve the disintegration of rock minerals into clay minerals, which can then be leached away, leaving a honeycomb structure with a high void ratio. The soil is prone to collapse as this structure develops. The largest amount of spatial variation is likely to be found in residual soils, making it more difficult to predict the collapse potential (Coduto, 2001).

2.9.2 Collapsible Compacted Fills

Some deep fills can collapse even when they have been compacted to traditional standards. This suggests that the hydro-consolidation potential is dependent on both the void ratio and the normal stress. Very loose soils will collapse upon wetting even at low normal stresses, but denser soils will be collapsible only at higher stresses. It appears that this phenomenon is most likely to occur in soils that are naturally dry and compacted at moisture contents equal to or less than the optimum moisture content. We can reduce the collapse potential by compacting the fill to a higher dry unit weight at a moisture content greater than the optimum moisture content (Coduto, 2001).

2.10 Problems Associated with Collapsible Soils

The key geotechnical problem with collapsible soils to is the substantial loss of shear strength and volume reduction that occurs when they are exposed to water from any source (Ali, 2016), (Kalantari, 2013) discuss that foundations for various structures or pavements constructed on collapsible soils might undergo rapid and considerable amounts of settlement. When the foundation subsoil becomes saturated due to various forms of water intrusions, such as leakage from broken pipelines, sewer lines, pools and basins, as well as water from runoff or irrigations, these deformations occur.

According to Houston et al. (2002), non-homogenous subgrades, which include materials with varying degrees of collapse potential, and non-uniform distribution of wetting in subgrades materials cause differential collapse settlement across roadway sections. When subgrade soils differentially collapse, the consequence is a rough, wavy surface and potentially many miles of extensive damage highway. There have been several reported cases for which collapsible soils have been cited as the cause of excessive pavement waviness or highway bridge problems. Some examples include Interstate 10 near Benson, Arizona, sections of Interstate 25 in the vicinity of Algadonas, N.M., and the Steins Pass, Arizona bridge foundation problems.

The mechanism of wetting from downward infiltration is more likely to result in a shallow depth of wetting than wetting to great depth. The probability of relatively shallow partial wetting from sources such as landscape irrigation and broken pipelines is very high. In addition, increased infiltration due to urbanization can effectively change an arid or semi-arid climate into a humid climate. For example, in connection with a forensic investigation in San Diego, California, the annual precipitation was about 30 cm before a residential subdivision was built, but about 170 cm (counting landscape irrigation) after it was built. Significant settlements of the underlying compacted fill were caused by the change in the effective precipitation level. Despite the fact that this subdivision was situated in a semi-arid area, the lawns were spongy to walk on, and the street side curbs had moss growing on them due to the excessive landscape watering. In another example, in semi-arid New Mexico, a commercial building won an award from the city for the year's most beautiful lawn and landscaping. However, they also suffered about US$0.5 million dollars in foundation damage owing to differential settlement, having wetted the collapsible foundation soils to a depth of 15 m in some locations (Houston et al., 2001).

In addition, Ziani et.al. (2019) reported that recently, in Algeria, troubles due to the collapse phenomenon have been observed in several regions. In Algiers region, a collapse affected two buildings in of El Achour city and part of the Ben Aknoun highway in 2009 and 2013 respectively. In Sétif region (300 km east of Algiers), a hole of 5 m in diameter and 12 m in depth appeared an agricultural land because of the presence of a collapsible soil. The collapse which occurred in 2009 in Cheria, near Tebessa in the far east of Algeria, causes settlements of a magnitude of two and a half meters of dozens of buildings. Furthermore, in the south-east of Biskra region, the infiltration of water induced the degradation of several residential buildings.

2.11 Mitigation Measures Against Collapsible Soils

The methods for treating collapsibility of soils fall into two groups: (i) Mechanical remediation methods, which are mainly focused on increasing the compaction of loess soil and rearranging the soil particles to reduce collapse; and (ii) chemical remediation methods, which use chemicals to alter the behavior and improve the strength and stability of collapsible soils. Recently, a combination of the two methods using compaction of soil followed by chemical stabilization has been used to treat problematic soils (Bigdeli, 2018).
(Al-Rawas, 2000) summarized that some of the most common techniques with an explanation of the advantages and disadvantages of each method as shown below.

A. Partial excavation and replacement with compacted fill

Excavation, removal of collapsible soil, and replacement with compacted soil with improved mechanical properties is one of the easiest methods for dealing with collapsible soils. Only if the layer of problematic soil is thin and close to the surface can this approach be used (Bigdeli, 2018).
Rollins and Rogers (1994) reported that this method has three benefits: it reduces the amount of collapsible material in the zone of significant stress, it increases the depth to which water must percolate before reaching collapsible material, and it reduces the induced stress to which collapsible soil is subjected. This reduction in the induced stress many keep the stress below the critical value necessary to produce significant collapse settlement. Furthermore, this technique minimizes the differential settlement under the footing.
Ali (2016) proved that by replacing sand/crushed stone in collapsible soils, it is possible to control/mitigate the risk of sudden settlement when exposed to water. She also discovered that replacing soil with compacted cohesion less soil decreases foundation settlement by about half and increases bearing capacity by about (80%-100%). To achieve good results of higher bearing

capacity and low and uniform settlement, the subgrade should be improved with compaction and pre-wetting before placing the top compacted sand replacement. Within the tested range, the most effective thickness for the compacted sand layer was found to be equal to the plate width.

B. Pre-wetting

This method the collapsible soil is wetted with water via boreholes, trenches, or ponding. It has been routinely used to stabilize collapsible soils prior the structure is built, so that soil collapse will be minimized after the structure is built. Although pre-wetting is useful for canals and roadways where the induced loads are small, pre-wetting without preloading is not sufficient to prevent future foundation settlement. Pre-wetting causes the soil to collapse under its existing overburden pressure. Therefore, additional loads imposed by the foundation are not compensated for and will result in additional settlement (Rollins and Rogers, 1994).

C. Controlled wetting

Controlled wetting is similar to prewetting, except it takes place after the structure has been placed. The quantities of water should be approximately measured and added in increments When a structure suffers damage or tilts as a result of differential settlement, this technique is often used. To correct the tilt, the added water should be introduced in a carefully-monitored manner to correct the tilt (Al-Rawas, 2000).

D. Moisture control: This method purports to prevent water ingress into the ground. Controlling wetting can be accomplished by: (a) controlling water irrigation, (b) placing landscaping in watertight planter boxes, (c) restricting landscape vegetation adjacent to structure, (d) placing pavement or buried geo-membranes around the perimeter of the structures, (e) placing effective surface and buried drainage systems, (f) informing occupants of buildings of the problems associated with collapsible soils (Al-Rawas, 2000).

E. Compaction control

Compaction is one of the most practical and efficient methods of minimizing soil collapse. Compaction has been used for both shallow and deep collapsible soils. In southern California (USA), collapsible soil deposits of 6 and up to 10 m deep are frequently removed and re-compacted. However, the compaction process is expected to be effective only up to about 5 m depth, with the greatest improvement in the upper 3 m. If sufficiently large weights are dropped from sufficiently great heights, then the effectiveness can extend somewhat deeper. Compaction can be achieved by use of rollers, displacement piles, heavy tamping (dynamic compaction), and vibration (vibroflotation or deep blasting) (Al-Rawas, 2000).

F. Heat treatment and small explosions Gas and fuel are burned in pressurized boreholes to heat treat loess in this method. The boreholes are closely spaced and temperatures are generated up to 1000 °C, producing a stabilized soil column with a diameter of 1.5 - 2 m (Al-Rawas, 2000).

G. Chemical stabilization

Chemical stabilization is the modification of the soil properties to improve its engineering performance. Chemical stabilization with additives including sodium silicate and calcium chloride has been attempted for years with varying degrees of success. The method creates cementation within the soil structure, which prevents it from collapsing when wet.

Penetration of chemical solutions into the desired depth is essential for the success of the operation. The method is most applicable to fine sand deposits. Silicate's stabilization is generally costly. The injection of sodium silicate solution has been used extensively in the former Soviet Union and Bulgaria. Pre-wetting with a 2% sodium silicate solution will significantly reduce the compressibility and increase the strength of collapsible loessial soil deposits, according to field and laboratory tests performed in the former Soviet Union. This approach is used for both dry and wet collapsible soils that are supposed to subside as a result of the added weight of the structure to be constructed (Al-Rawas ,2000).

Lime stabilization and cement stabilization are the most common chemical stabilization methods. Cement, lime, fly-ash, polymers, silicates, and other stabilizing additives have all been tested for their ability to remediate collapse. Traditional stabilizers: Cement-based stabilizers were the first chemicals used to stabilize collapsible soils. Fly ash, cement kiln dust, and a combination of fly ash and rice husk ash have all shown promise in preventing soil collapse. However, a careful check must be done to ensure the cementitious materials have penetrated to the desired depth to strengthen the particles' bonding upon wetting (Bigdeli, 2018).

The stabilization of a collapsible soil by granulated slag and natural pozzolana carried out by (Ziani et.al. 2019) has been able to show that the influence of the addition of granulated slag and natural pozzolana on the collapsible soil led to a significant reduction in the collapse potential (Cp) followed by a decrease of suction. In this work, pumice is used as a stabilizing material to increase the strength of collapsible soils.

Table 2. 2: Summary of mitigation measures' benefits and drawbacks (Rollins and Rogers, 1994)

Treatment method	**Advantage**	**Disadvantage**
Pre-wetting	Low cost Ease of application	Excessive settlement without preloading Over excavation Failure to densify surface layers Differential settlement likely
Partial excavation and replacement with compacted fill	Relatively low cost Ease of application Reduction of induced stress on collapsible soil Minimal settlement for small volumes of water Minimization of differential settlement	Excessive settlement following wetting of deep zones
Dynamic Compaction at Natural Moisture	Dramatic reduction in collapse settlement Decrease in hydaulic conductivity Improvement to significant depths (>5 m)	Higher cost Potential for damage due to vibrations Non-uniformity of treatment Less contractor experience with method
Dynamic Compaction after Pre-wetting	Significant decrease in collapse settlement Increased compaction efficiency prior to liquefaction Reduction in level of vibrations Greater uniformity of densification Decrease in hydraulic conductivity Improvement to significant depths(>5 m)	Higher cost Increase in creep (long-term) settlement Potential for liquefaction when water content is high Difficult to withdraw weight after drop Drying time following treatment may be excessive Less contractor experience with method Difficult to measure improvement

2.12 Factors Affecting Collapse

2.12.1 Index Properties of Soil

A. Soil type: The structure of collapsible soils consists of silt-size particles bound together with clayey-bridge. In the unsaturated state, the clayey-bridge works as a cementitious agent between the soil particles which increases the shear strength of the soil. The higher the amount of clayey material, the higher the void ratio of the soil. Upon the addition of water to the system, the clayey-bridges dissolve and the soil experiences collapse. Therefore, a higher amount of clayey content results in a greater amount of collapse, the addition of clay to the soil leads to expansibility behavior. Later studies on collapsible soils with a high amount of clay (more than 30%) proved that clay content is not the key factor and there is no monotonic relation to the collapse potential of soil (Bigdeli, 2018).

B. Dry unit weight: The denser the soils are, the lower the initial void ratios are, thus, resulting in less collapse upon wetting. Furthermore, a dense state of soil would reduce the relative contribution of the metastable forces in supporting the soil structure. Although these forces, when reduced upon water addition, would decrease the soil resistance to load, this decrease is less profound when the density is higher (Basma and Tuncer, 1992).

C. Water content: The degree of saturation that causes collapse varies depending on the type of soil. The soil's initial moisture content as well as the applied pressure on the soil structure influence this aspect. The meta-stable structure of collapsible soils is weakened as the moisture content of the soil increases, allowing cementitious agents to dissolve. Consequently, the structure of soil fails and triggers the collapse. It is noteworthy that an increase in moisture content and applied load increases the collapse potential to a certain point beyond which no substantial settlement happens. The fact that every form of soil, including collapsible soils, has a maximum densification level explains this collapse behavior (Bigdeli 2018).

2.12.2 Applied Pressure at Wetting

The applied stress at wetting has a significant effect on the behavior of soil. Collapse potential increases with increasing wetting pressure up to a certain point (yield stress), after which collapse potential remains the same or decreases with further increase in wetting pressure. This can be attributed to the fact that there is a maximum degree of densification that collapse can trigger that may be attained at a certain pressure level. Any further increase in pressure beyond this level will, subsequently, cause little or no change in collapse potential. As the applied pressure increases, larger particles crush to form smaller silt-size grains that can move into the voids upon wetting, thus, reducing the void ratio and resulting in higher collapse (Lawton et al. 1989).

2.12.3 Suction of Soil

Stress state principal of soils can be used to describe the mechanical behavior of soils in both saturated and unsaturated conditions. The two stress state variables of unsaturated soils has been applied to investigations of volume change behavior caused by loading and wetting of collapsible soils. Collapse is attributed to the loss of strength associated with suction decrease as a result of wetting. To put it another way, collapse happens when the soil's stress state changes from unsaturated to saturated (Li et al., 2016).

2.13 Distribution of Collapsible Soils

Collapsible soils could occur almost anywhere (Rogers, 1995). On a world map, one could mark the most notable, and well known, sand deposits as:

- Granitic sands of South Africa.
- The collapsible Aeolian sands of the Saharan fringes.
- Kalahari sands of South Africa.

One can do the same for silt deposits, thus;

- Loess and volcanic dust of South America.
- Chinese loess.
- Eastern European and Central Asian loess.
- North American loess.

However, to these should be added the loessial soils of India, Pakistan, Thailand and New Zealand, to name but a few. Moving to finer soil;

- Volcanic ash of Japan and New Zealand.
- Quick clays of Scandinavia and Canada.

This list can easily be extended by considering the less well-known deposits that are found in the literature such as:

- Australian parna, a carbonate cemented sandy clay that collapses on leaching.
- The gypsum soils of the CIS.
- The cemented soils of Northern Nigeria.
- The saline soils of China.

This summary has concentrated so far on natural deposits, but can be further widened by including:

- Slide structures caused by slope instability, which can produce open structures.
- Soils compacted dry of optimum.

2.13.1 Distribution of Collapsible Soil in Ethiopia

The distribution of collapsible soil in Ethiopia was explained by ERA (2013). These can be summed up as follows:

Collapsible soils are found in areas where there are loess deposits (windblown silts). Loess deposits abound in Ethiopia's Omo River valley's southern portions.

Collapsible soils can also be found in the Ethiopian Rift Valley's central and southern sections, in the form of silty loess-type deposits. These soils contain a large amount of void space, and particles are held together by the clay component.

The existence collapsible soil in Ethiopia around Zeway, Shashemene and Hawassa is manifested by the occurrence of ground cracks and potholes during heavy rains or floods due to hydro-compaction. In Afar region, collapsible soils are present in the form of sand dunes (Tewodros, 2019).

2.14 Review of Improving of Collapsible Soil

The subject of collapsible soil and its improvement is relatively well researched in different parts of the world.

2.14.1 Index and Mechanical Properties of Collapsible Soils (A Case Study on Ziway-Arsi Negele Highway Project)

Tewodros (2019) studied index and mechanical properties of collapsible soils: A case study on ziway-arsi negele highway project. The experimental study was made by field density test atterberg limit test, compaction and collapsible potential tests. The findings and conclusions of the study can be summarized as follows:

- Collapse potential test done for the samples remolded at their bulk densities and subjected to stress level of 200 kPa have shown that the samples are moderately collapsible as per ASTM classification.
- Collapse potential test done for the samples remolded at their maximum dry densities and subjected to stress level of 200 kPa has shown that the samples are slightly collapsible as per ASTM classification.
- The soils have moderate collapse potential at their natural state and does not exhibit any kind of collapsibility at their compacted state.
- One-dimensional test results obtained at the same density but at different stress levels revealed that, the magnitude of the loading stress level had effect on the collapse potential of the samples. A higher stress level resulted a higher collapse index/potential.
- The collapse potential test results for samples remolded at their bulk and maximum dry densities, it was learnt that the collapse nature of the samples can be highly enhanced by improving their densities

2.14.2 Treatment of Collapsible Soils by Additions of Granulated Slag and Natural Pozzolana

Ziani (2019) has made a study on the effect of adding granulated slag and natural pozzolana, at different contents, on the suction and the collapse potential (Cp) of a soil. Suction and collapse potential (Cp) tests of a soil were used. The conclusions and findings of the study are;

- The obtained results showed that treated collapsible soils with granulated slag and natural pozzolana, lead to a significant reduction in the collapse potential (Cp) followed by a decrease of suction.
- Addition of granulated slag or natural pozzolan at dosages of [5−7%] by the weight of the treated soil, cause the stabilization of the collapsible soil.

2.14.3 Improving Physical Characteristics of Collapsible Soil (Case Study: Tehran-Semnan Railroad)

Moayed (2015) investigated the effect of using lime, cement and microsilca injection for improving the physical characteristic of Collapsible soil. The conclusions and findings drawn from the study are;

- Results of consolidation test on intact soil samples before and after injection show that the collapsing potential of the soil would decrease up to 70, 63 and 40 percent for lime, cement and microsilica injection respectively. It can be said that injection of proper materials significantly decreases the collapsing potential of the soil.
- Injection would increase the soil bearing capacity and increase NSPT up to 85% while it would decrease soil plasticity properties.
- Injection of lime, cement and micro silica considerably increase internal friction angle of soil and consequently shear strength properties of soil.

2.14.4 Feasibility of Using Electrokinetics and Nanomaterials to Stabilize and Improve Collapsible Soils.

Hosseini (2019) studied the utilization of an innovative method for stabilization and improvement of Gorgan loessial soil. This method uses electrokinetics and nanomaterials to instigate additives to move through soil pores, as an in situ remedial measure. The conclusion and findings drown from the study are;

- The oedometer tests on samples improved by 3% lime and 5% nanomaterials show considerable improvement of the collapse potential.
- The test results indicate that the resistance of the soil was highly dependent on the water content and matric suction of the soil.

2.14.5 Treatment of Collapsible Soils by Mixing with Iron Powder

Alshaba (2018) has conducted the major laboratory tests on natural collapsible soil and the treated samples with different percentage of iron powder and come up with the following conclusions;

- Mixing the collapsible soil with a specific ratio of iron powder results in an improvement of the collapsibility characteristics, and reduction of the collapse settlement upon wetting.
- Series of single Oedometer tests showed that the collapse potential decreases with either increasing the soil unit weight or adding iron powder with a specific ratio.
- Experimental tests using both Oedometer and laboratory experimental set-up show that the optimal ratio of iron powder that may be mixed with collapsible soil ranges from 5% to 6% of the soil weight.

2.14.6 Treatment of Collapsible Soil Using Bentonite–Cement Mixture

Bellil (2018) conducted the major laboratory tests on the natural collapsible soil and bentonite–cement blended sample and come up with the following conclusions;

- According to the compressibility tests, the collapse potential Cp decreases when the water content and the compaction energy increase. It is also found that the best optimum reduction is obtained for contents of bentonite and cement equaling 4% and 8%, respectively.
- The results show that treatment with a bentonite–cement mixture improves the geotechnical and mechanical characteristics, modifies the chemical composition of the soil, reduces the collapse potential and the consistency limits.

2.16 Availability of Pumice in Ethiopia

Pumice is amorphous foam produced during volcanic eruptions. It is constituted mostly of silica and alumina in relative amounts varying according to the geological area of origin, and also includes other chemical species, such as different oxides and water. Pumice is a volcanic rock of which porous structure is formed by dissolved gases precipitated during the cooling as the lava hurtles through the air. The connectivity of the pore structure may range from completely closed to completely open (Zerai, 2015). Following Italy, Chile and Ecuador; Ethiopia is the 4th leading producer of pumice and scoria aggregates in the world (Melese, 2015).

CHAPTER THREE: DESCRIPTION OF THE STUDY AREA

3.1 Study Area Description

Ziway is a town and separate woreda in central Ethiopia. It is located on the road connecting Addis Ababa to Nairobi in the East Shewa Zone of the Oromia Region of Ethiopia. Ziway has a latitude and longitude of $7^o56'$N / 38^0 43'E with an elevation of 1643 meters above sea level.

Lake Ziway is the closest major Rift Valley Lake to Addis Ababa, 162 km south of Addis Ababa. It is located on the rift floor with an altitude of 1636 m.a.s.l, the catchment area of 7380 km^2, a maximum depth of 9 m, a minimum depth of 2.5 and a volume of 1466 m^3. It is one of the freshwater Rift Valley lakes of Ethiopia. The woredas holding the lake's shoreline are Adami-Tullu and Jido-Kombolcha, Dugda-Bora, and Ziway-Dugda. (Danait, 2015). However, the study area is located in East Shewa Zone of Oromia Region, Adami-Tulu Woreda. Adjacent to Lake Ziway, the economy of the town is based on finishing and horticulture. Ziway is also home to a prison and a caustic soda factory.

Figure 3. 1: Location of sampling point area of collapsible soil on the map

Figure 3. 2: Location of sampling point area of pumice on the map

3.2 Climate

The climate of the area around Lake Ziway has arid characteristics for most of the year and monthly average rainfall never exceeds evaporation. The mean annual rainfall is 760 mm (MoWR, 2006). The mean daily temperature at Ziway is 19.3°C. The highest temperatures occur between March and June prior to the start of the main rains, though seasonal variation in daily temperature is relatively slight. The mean annual temperature of the area is within the range of 16-25^{0}C.Five years of Meteorological data was found from the Meteorological and Climatology Directorate from the year 2010- 2014. The highest mean rainfall was in July-August. (Danait, 2015).

3.3 Wind

The Lake Ziway area is known for its strong and persistent daytime winds. During the afternoon in the dry season, the prevailing northeasterly winds are reinforced to the west and south of the lake by local on-shore airflow. (Meron, 2007).

3.4 Soil Type

The areas around Lake Ziway are composed of volcanic rocks, with alkaline lavas, ashes and, ignimbrites, mainly of Tertiary and younger age. The sediments covering the volcanic rocks in the area are generally mixed deposits of sandstone, limestone, and silts, with frequent occurrences of evaporite minerals. In Ziway, area gentle levees are formed of sandy clay loams (Meron, 2007).

CHAPTER FOUR: MATERIAL DISCRIPTION AND LABORATORY METHODS

4.1 Introduction

In this chapter, the materials used and methods adopted for the research are briefly described with regard to their sources and their physical properties. All laboratory investigations on materials were done in Best Consulting Engineering laboratory. The properties of the selected material including specific gravity, atterberg limits, particle size distribution, collapsible potential, California bearing ratio (CBR), direct shear, and compaction characteristics are presented. It also presents field density tests for the samples that were taken from the one test pit. The engineering properties of the soil is assessed both for natural and improved soil samples separately.

The experimental program is planned to achieve the objectives of the study and testing procedures adopted for various laboratory tests are described in this chapter.

4.2 Materials

Collapsible soil and pumice were the main materials used in this research.

A. Collapsible soil

The collapsible soil sample used for this research work is collected from Ziway at (464603.1E, 873392.2N) and (464631.7E, 873508.8N) from two-test pit. Both disturbed and undisturbed collapsible soil samples are collected at (464603.1E, 873392.2N) with a depth of 1.7m for test pit one and (464631.7E, 873508.8N) with a depth of 1.5m for test pit two from the natural ground level. Samples were collected during dry season using pickaxe, shovel, and the samples have been placed in bags and labeled, and transported to the laboratory.

Picture 4. 1: Collapsible soil samples collected from the test pit

B. Pumice

Pumice as shown in Picture 4.2 was obtained from Meki town inside Dangote cement factory quarry site, which is located in Oromia Regional State around Ziway (484212E &905275N). It was properly packed in sacks and transported to the laboratory.

Picture 4. 2: Material site for pumice

4.3 Method

The experimental testing procedure was carried out to investigate and determine the properties of collapsible soils and analyze the effect of pumice stabilizer material on their geotechnical properties. The experimental work consisted of a field portion and a laboratory portion.

4.3.1 Field Test Program

The field test program of the soil included field (visual) description of the soil, disturbed and undisturbed soil sampling, determination of in situ density (field density test) and natural moisture content determination for laboratory tests.

4.3.3.1 Undisturbed Samples

A laboratory testing program is performed on undisturbed samples to evaluate the collapsibility index of the natural soil. According to (Alfi,1984), collapsible soils contain a large amount of air in their voids. It was made every possible effort to obtain undisturbed soil samples, which makes very difficult due to metastable structure. The low stability of the soil presents the danger of breaking apart the soil sample while pushing down the sampler mold in the field while transporting the sample to the laboratory, or while trimming the sample in the laboratory.

Picture 4. 3: Undisturbed sampling

4.3.3.2 Field Test

The natural dry density has been used as an indicator for the collapse potential in most of the previously mentioned collapse criteria. The field density test was conducted at 1.7m depth by using sand replacement method (sand cone method) according to ASTM – D 1668, to find out the natural dry density and moisture content.

Picture 4. 4: Field density test using sand replacement method

4.3.2 Laboratory Test Program

The laboratory testing program was initiated by preparing the samples for testing to find out the geotechnical properties of the soil. The conducted laboratory testing program on the disturbed soil samples includes; compaction test to find out the laboratory maximum dry density and optimum moisture content, particle size distribution tests to obtain the amount and distribution of particles sizes, atterberg limits to find the liquid limit, and plasticity index, specific gravity, direct shear, and CBR to achieve the objective of the research. Undisturbed soil samples collected were also subjected to collapsible potential test at the bulk condition.

4.4 Sample Preparation

Considering the main purpose of the research and assessing the effects of different contents of pumice on properties of the soil under investigation several treatments were prepared and tested. Prior to laboratory tests and sample treatment, the sample was prepared in accordance with the method described in AASHTO T87-86. This method involves

- Air drying of samples and/or oven drying at 60ºC or less;
- Breaking up the soil aggregates by rubber covered mallet. Then, sieve analysis is performed to separate the dried soils into groups.

4.5 Standard Laboratory Testes

4.5.1 Grain- Size Analysis

The grain-size analysis test is carried out to determine the relative proportions of different grain sizes which make up a given soil mass. The particle size distribution curve of the material was established using the sieve and hydrometer methods. Sieving combined with hydrometer method is the standard method to determine quantitatively the particle size distribution of soils. The test includes the determination of the particle size distribution for the natural soil.

Wet Sieve Analysis

Wet sieve analysis was employed to determine the grain size distribution of the soil samples in accordance with ASTM D 2217-85 Test Method for particle size analysis of soils.

Hydrometer Analysis

To determine the distribution of fine particles (silt and clay) 50g of air -dried soil sample passing sieve 75 µm is used. The soil sample is socked in the chemical solution (Sodium hexa-meta phosphate) for 24 hours. Hydrometer analyses indirectly measure the diameter of the fine-grained soil based on their sedimentation time. After complete grain size analysis of both, the relative proportion of different size groups in soil sample can be determined. The test was conducted according to AASHTO T-11 96(2000).

4.5.2 Specific Gravity

Specific gravity which is the measure of the heaviness of the soil particles is determined by using the pychnometer method. It is the ratio of the mass in air of a given volume of soil particles to the mass in air of an equal volume of gas -free distilled water at a stated temperature (20°C). Determination of the specific gravity for the natural soil were done. The test is conducted in accordance with ASTM D 854-98 testing procedure.

4.5.3 Atterberg Limits

The test is a consistency Limit identification test on the basis of moisture content. It includes the determination of; the liquid limits, plastic limits, and the plasticity index for the natural soil. The tests are conducted in accordance with ASTM D 4318 testing procedures.

Liquid limits

The liquid limit is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. The liquid limit (LL) is the water content at which soil in a standard cup cut by a groove of standard dimensions will flow together for a distance of 13 mm when subjected to 25 shocks dropped from 10mm at a rate of two shocks per second.

Plastic limit

The plastic limit is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible state) state. A portion of the soil mixes used for the liquid limit test was retained for the determination of plastic limit. A small portion of the soil sample was put on a flat glass plate and mixed thoroughly enough to be shaped into a small ball. The ball was then moulded between fingers and then rolled on the glass plate with palm of hand into thread of about 3mm diameter when the thread crumbles by shearing. The crumbled threads were immediately put in a weighing pan for moisture content determination.

Plasticity index

The plasticity index of the natural soil and the soil–pumice mixture is the difference between the liquid limits and their corresponding plastic limits. The plasticity indexes of the samples are calculated as:

PI =LL –PL

4.5.4 Moisture Content of the Soil

This test was conducted according to (ASTM D 2216-98). The moisture content of disturbed and undisturbed soil samples was determined using the oven-drying method. Small representative natural soil specimens are obtained and oven-dried at 105°C for 24 hours. The sample is the reweighted, and difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, recorded as the initial moisture content for the natural soil.

4.5.5 Compaction

This test includes the determination of the maximum dry density and the optimum moisture content in accordance with ASTM D 698-91testing procedures. The test is conducted for both the natural and soil-pumice mixture.

The determination of the dry density – moisture content relationship of the soil sample was carried out using the standard proctor method. The air -dried soil samples were prepared and made to pass through No. 4 (4.75mm) sieve for each test. In this method, the soil is compacted by a 5.5 lb hammer falling a distance of one foot into a soil -filled mold. The mold is filled with three equal layers of soil and each layer is subjected to 25 drops of the hammer.

The blows are uniformly distributed over the surface of each layer. The collar is then detached, and the compacted sample is leveled off with a straight edge at the top of the mould. The leveled sample is then weighted to the nearest 1g in the mold. One small representative sample is then taken from the compacted soil for the determination of moisture content. The same procedure is repeated until a minimum of five sets of samples is taken for moisture content determination. The values of the dry densities are plotted against their respective moisture contents and MDD is deduced as the maximum point on the resulting curves.

4.5.6 Collapse Index/Potential

In order to determine the collapse property of the soil, one -dimensional Oedometer test has been carried out in accordance with testing procedures described in ASTM D 5333. The test method consists of placing the soil specimen at natural water content in a consolidometer, applying a predetermined applied vertical stress to the specimen, and inundating the specimen with fluid to induce the potential collapse in the soil specimen.

The collapsible potential is defined as the change in sample height (h) up on wetting compared to the original sample height (h_o).

The collapsible Index (I_c) of the soil is calculated as:

$I_c = \Delta h/(1+h_o)$ (4.1)

Δh is the change in the height of specimen upon flooding

h_o is the original height of the specimen

4.5.7 CBR (California Bearing Ratio) Test

The CBR test is conducted in accordance with AASHTO T193-93 for the natural soils and soil-pumice mixture. The test is performed by measuring the force required to penetrate a soil sample with a plunger of standard area.; it is aimed at determining the relationship between force and penetration. 5.0kg of the natural soil and the soil-pumice mixture are mixed at their respective optimum moisture contents in 2124 cubic centimeters mould. The samples are compacted in three layers with 56 blows from the 2.5kg rammer. The CBR is a measure of shearing resistance of the material under controlled density and moisture conditions. The CBR value for 2.54mm and 5.08mm are recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value. One -point CBR test have been done for all samples to determine the strength character of the soil alone and in the stabilized case.

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value and the CBR swell of the soil.

4.5.8 Direct Shear Test

The direct shear test is conducted in accordance with ASTM D 3080-03for the natural soils and soil- pumice mixture. This test is conducted on a soil specimen in a shear box which is split into two halves along a horizontal plane. The shear box is made of brass or gunmetal. It is square or circular in plan. In situations which need the shearing strength of a soil, direct shear test applied. The test can be performed on an undisturbed sample or remolded samples. The main advantage of the direct shear test is its simplicity and smoothness in operation and the rapidity of the testing program. But this test has the disadvantage that lateral pressure and stress on planes other than the plane of shear are not known during the test.

CHAPTER FIVE: TEST RESULTS AND DISCUSSIONS

5.1 Introduction

This chapter is concerned with the analyses and discussion of the results of the field and laboratory experiments. The experimental results of two series of soils: natural collapsible soil and natural collapsible soil mixed with different percentages of pumice were investigated. The results are analyzed and discussed to give the insight of the research in terms of engineering properties of collapsible soil. The tests include Atterberg limits, grain size analysis, moisture density relationship (compaction), direct shear test, and CBR.

Additionally, it presents the analysis of the one dimensional single oedometer test results as direct method for collapse index evaluation on undisturbed soil specimens at their natural moisture content and on compacted disturbed/improved soil specimens at moisture content equal to optimum moisture content. The results of these tests are discussed in this chapter.

5.2 Properties of Material Used in the Study

5.2.1 Natural Soil (Collapsible Soil)

The results of the tests conducted for identification and/or determination of properties of the natural collapsible soil are presented in Table 5.1. As shown in Figure 5.1 in the particle size distribution curve almost 49.7% of the soil is passing through No. 200 sieve. For repeated trials in the liquid limit test, the soil closes in 11 or 13 blows. No trial requiring less than 15 or more than 35 blows shall be recorded, according to the AASHTO T 089-96 testing procedure. And in the plastic limit test, the soil starts to crumbles before it rolls to into a thread of 3.2mm diameter. Accordingly, AASHTO T 090-96 recommends reporting a material non plastic when the plastic limit or liquid limit cannot be determined. Hence, the soil can be described as non-plastic (NP) silty sand with the symbol SM according to the USCS classification. According to AASHTO, the soil falls under the A-4(3) soil class.

The average specific gravity of the soil solids was 2.57, which is in agreement with the approximate value used for most soil computations. It is known that the specific gravity of the soil depends on its predominant soil materials. In the mineralogy section, it will be shown that the predominant mineral is potassium feldspar, the specific gravity of which is 2.57 (Das and

Sobhan, 2017). They are major constituents of the sand and silt fractions of soil. Therefore, the soil sample which is sandy and silty in nature according to a value of specific gravity.

The in situ dry density of the soil sample was determined by using the in-situ sand replacement method as described in Chapter 4. The value was found to be 1.172 gm/cm^3.The value is closer to the lower limit for the dry density as indicated by (Cleavenger, 1958). He proposed a criterion based on dry density, that is, if the density is less than 1.28 gm/cm^3, then the soil is liable to a significant settlement. And also Page-Green (2008) reported that typical collapsible soils have densities of less than 1600 kg/m^3 (mostly in the range of 1000 to 1585 kg/m^3). So, small value for natural dry density indicates that the soil is undesirable for foundation and liable to further investigation. Without further investigation, the result may be questionable and may require improvement before it can be used in any engineering project.

The in- situ water content of the soil sample was determined as part of the in-situ dry density test. The average was 12.46%. The soil has a maximum dry density of 1.36 g/cm^3, optimum moisture content of 25%, soaked CBR value of 20.7%.

Shear strength parameters from the graph of shear stress versus displacement and maximum shear stress versus applied vertical load respectively gives an angle of internal friction (∅) and cohesion(C). From these results, the angle of internal friction and cohesion of the soil are (ϕ= 6.73° & c = 0.68kN/m^2) at natural (bulk) condition and (ϕ= 31.48° & c = 9.23kN/m^2) at MDD respectively.

Finally, the results of single oedometer tests were carried out to investigate the behavior between the vertical stress - strain as well as the collapse Index. The results of single oedometer show that the collapse value (I_c= 6.72%), settlement=1.944 and (e_0= 1.192) of natural untreated soil and, the collapse value (Ic= 2.235%), settlement=0.950 and (e_0= 0.919) of compacted untreated soil at MDD. The values of the collapse index (I_c) classify the soil used from slight to severe disorder soils, according to the classification of ASTM (D5333). Hence, these values of the collapse potential (I_c) indicate that the soil is moderately severe disorder soil at natural(bulk) condition and moderate disorder soil compacted at MDD. So, it is important to use soil improvement methods to keep the soil stable and to reduce the magnitude of the collapse potential to an acceptable level. For this purpose, laboratory tests on soil samples treated with pumice, have been carried out in order to determine the effect of adding these products on the collapse potential (I_c).

Table 5. 1: Geotechnical properties of the natural soil

Property	Soil Sample 1	Soil Sample 2
Natural moisture content, (%)	12.46	11.0
Specific gravity	2.57	2.59
Grain size Analysis		
Percentage passing No. 200 sieve, (%)	49.7	91.0
Atterberg Limit		
Liquid limit, (%)	NP	NP
Plastic limit , (%)	NP	NP
Plasticity index , (%)	NP	NP
Classification		
AASHTO soil classification	A-4	A-4
USCS	SM	ML
Compaction Characteristics		
Maximum dry density, (g/cm^3)	1.36	1.26
Optimum moisture content, (%)	25	27
CBR		
Soaked CBR value , (%)	20.7	12
Shear strength of soil		
Angle of internal friction at bulk, (^{0})	6.72^o	2.31
Angle of internal friction at MDD, (^{0})	31.48	22
Cohesion at bulk, (kN/m^2)	0.68	0
Cohesion at MDD, (kN/m^2)	9.23	4.5
Collapsible potential		
Collapsible potential at bulk, (%)	6.72	4.28
Collapsible potential at MDD, (%)	2.235	1.08
Settlement at bulk	1.944	1.188
Settlement at MDD	0.950	0.312

As seen on the table 5.1, Of the two test pits, test pit one was selected for this thesis for further improving work because of the laboratory test result as shown in table 2, soil in test pit one exhibit slightly more collapse severity than soil in test pit two.

.

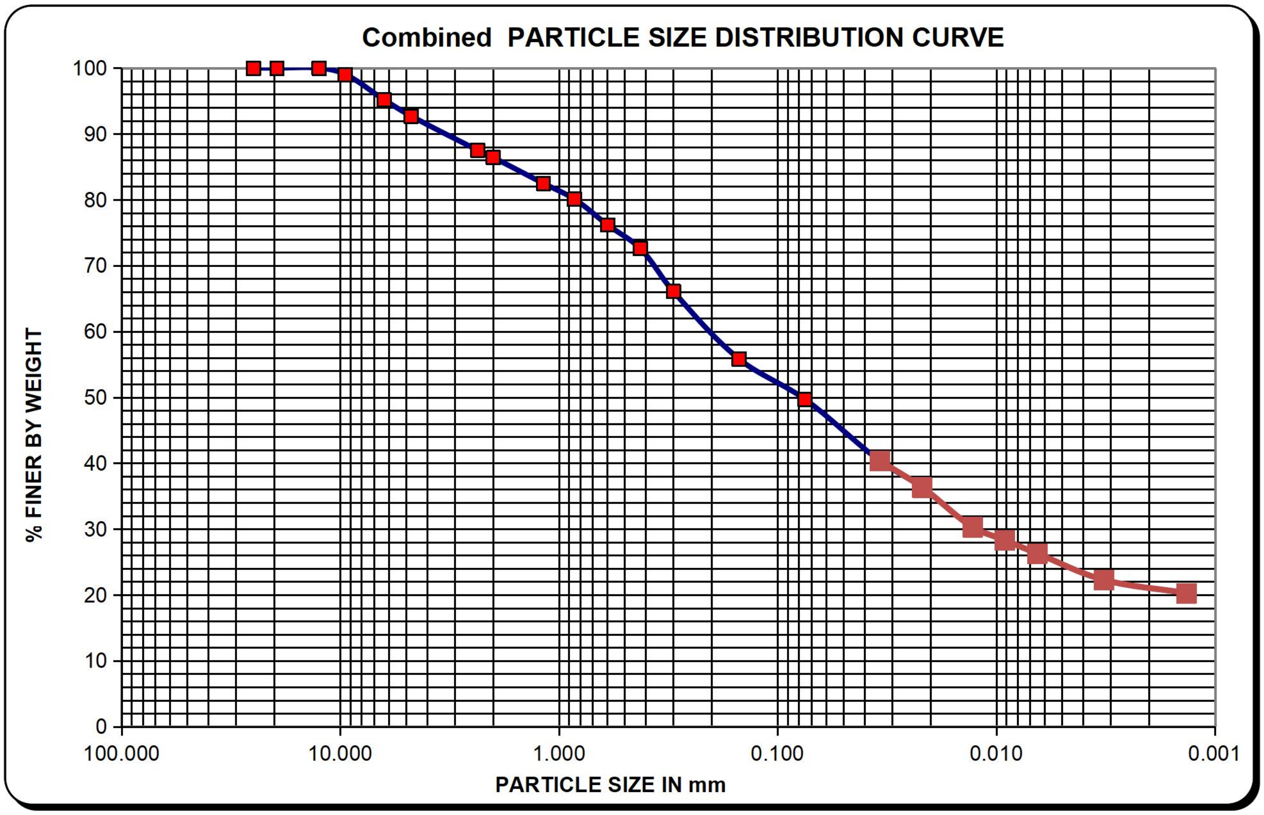


Figure 5. 1: Particle size distribution curve of the collapsible soil

5.2.2 Pumice

Pumice is a kind of glass formed and not a mixture of minerals. As shown in table 5.3 pumice is composed of 70.74% SiO_2, 17.81% Al_2O_3, 3.96% Na_2O and 2.92 K_2O.

The geotechnical properties of the pumice were used in the experimental program are shown in Table 5.2. The pumice used in the present study is light in color. As shown in table the results for the pumice are Atterberg limits test of NP, specific gravity of 2.023, % passing Sieve No.200 of 3.4, Optimum moisture content of (OMC) 59.7% and, maximum dry density (MDD) of 0.74.

Table 5. 2: Geotechnical properties of the pumice

Properties	Value
Specific gravity	2.023
Grain size Analysis	
% passing Sieve No.200	3.4
Atterberg Limit	
Liquid limit, (%)	NP
Plastic limit , (%)	NP
Plastic index, (%)	NP
Compaction Characteristics	
Optimum Moisture Content (%)	59.7
Maximum Dry Density, (g/cm^3)	0.74

Table 5. 3: Chemical composition of pumice

Description	Symbol	Percentage composition (%)
Silica	SiO_2	70.74
Alumina	Al_2O_3	17.81
Iron	Fe_2O_3	<0.01
Calcium	CaO	<0.01
Magnesium	MgO	<0.01
Sodium	Na_2O	3.96
Potassium	K_2O	2.92

5.3 Effect of Pumice on Engineering Properties of Collapsible Soil

5.3.1 Effect of Pumice on Compaction Characteristics

Standard compaction tests were performed to determine the optimum water contents and maximum dry densities of natural and pumice treated soils. Table 5.4 shows the effects of pumice on OMC and MDD of natural soil sample for uncured case. It was found that the optimum moisture content, OMC, of the sample increases and, maximum dry density, MDD, decreases as pumice content increases.

Table 5. 4: Summary of MDD and OMC with application of different pumice contents (uncured case)

Pumice Content (%)	0%	5%	10%	15%	20%	25%	30%
MDD(g/cm^3)	1.36	1.31	1.26	1.24	1.2	1.19	1.16
OMC (%)	25	26.36	28.00	28.88	29.5	30.14	31

Maximum Dry density

The effect pumice on maximum dry density of the collapsible soil is shown in Figure 5.2. As indicated in in the figure, the addition of pumice decreases the maximum dry density of the natural soil from 1.36 g/cm^3 to 1.16 g/cm^3 for soil sample with increased pumice content from 0% to 30% for uncured case. This result assures that there was a 15% decrease in the MDD of the soil sample.

The decrease in the maximum dry density is mainly due to;

- The dominance of the low weight and specific gravity of the pumice, hence the total dry weight of soil mixtures decreases.
- Pumice (with lower specific gravity) fills the soil voids and it contributes to a decrease in density.

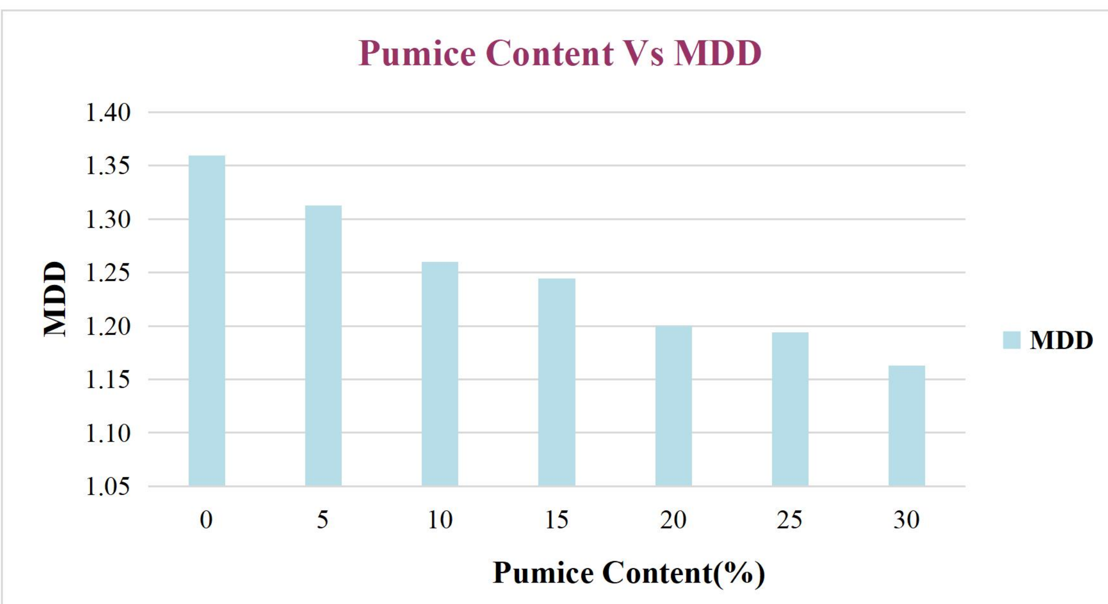


Figure 5. 2 : Pumice content vs. MDD (uncured case)

Optimum moisture content

The effect pumice on optimum moisture content of the collapsible soil is shown in Figure 5.3. The optimum moisture content increase from 25% to 31% for soil samples with increased pumice content from 0% to 30% for uncured case. This result implied that there is a maximum increase in the optimum moisture content of natural soil by 25%.

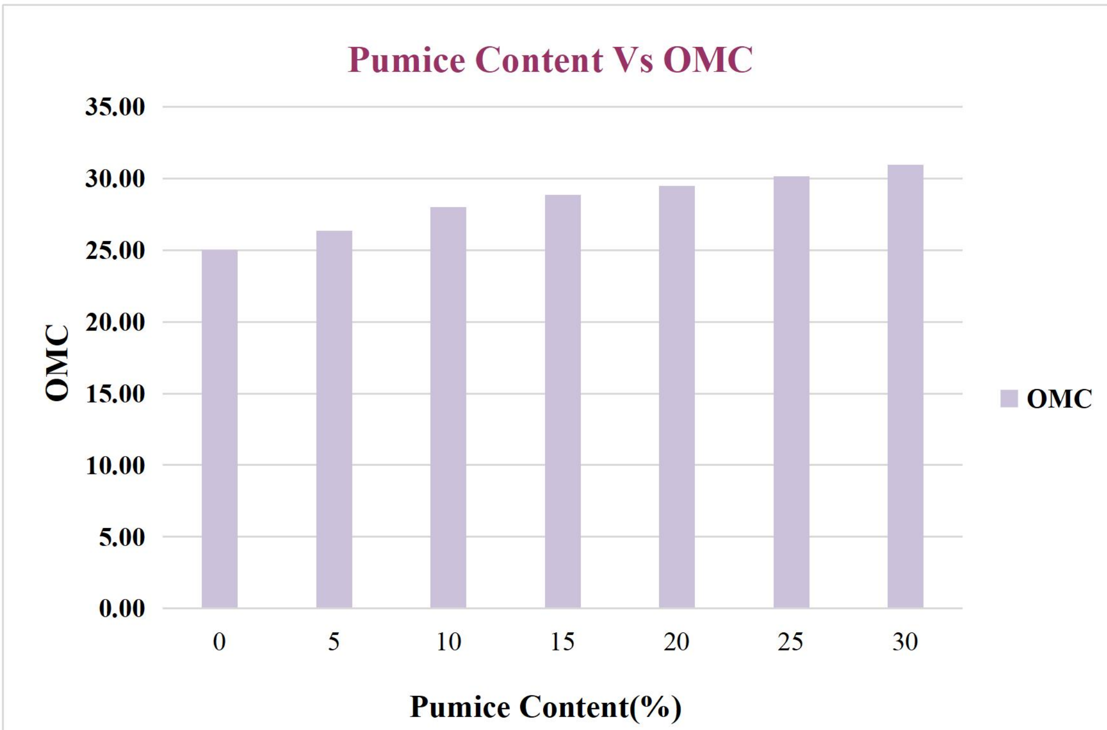


Figure 5. 3: Pumice content vs. OMC (uncured case)

The increasing in the optimum moisture content was mainly due to;

- The increase in OMC due to the addition of pumice caused by high absorption of water by pumice.
- Increasing OMC will always lead to a decrease in the maximum dry density as the specific gravity of soil and additives particles are more than water.
- Insufficient time required to complete reaction between the pumice with soil particle.
- Additional water is required to wet the large surface area of the pumice particles, or it is absorbed by the fine pumice particles. The advantage of the increase in OMC and a corresponding decrease in MDD of the soil is that it allows compaction to be easily

achieved with wet soil. As a result, drying the soil prior to compaction to minimize moisture content is no longer necessary.

Table 5. 5: Summary of MDD and OMC with application of different pumice contents (cured case)

Pumice Content (%)	15%	20%	25%	30%
MDD(g/cm^3)	1.22	1.18	1.16	1.13
OMC (%)	29.0	30.1	31.24	32.4

Cured case

For cured case, as shown in table 5.5 the OMC of the treated soil for 7 days curing of pumice blended with collapsible soil increase and, MDD decrease in some extent. As indicated in the Table, the addition of pumice decreases the maximum dry density of the natural soil from 1.36 g/cm3 to 1.13 g/cm3 and increases the optimum moisture content of the natural soil from 25% to 32.4 for 7 days cured case with increased pumice content from 0% to 30%.

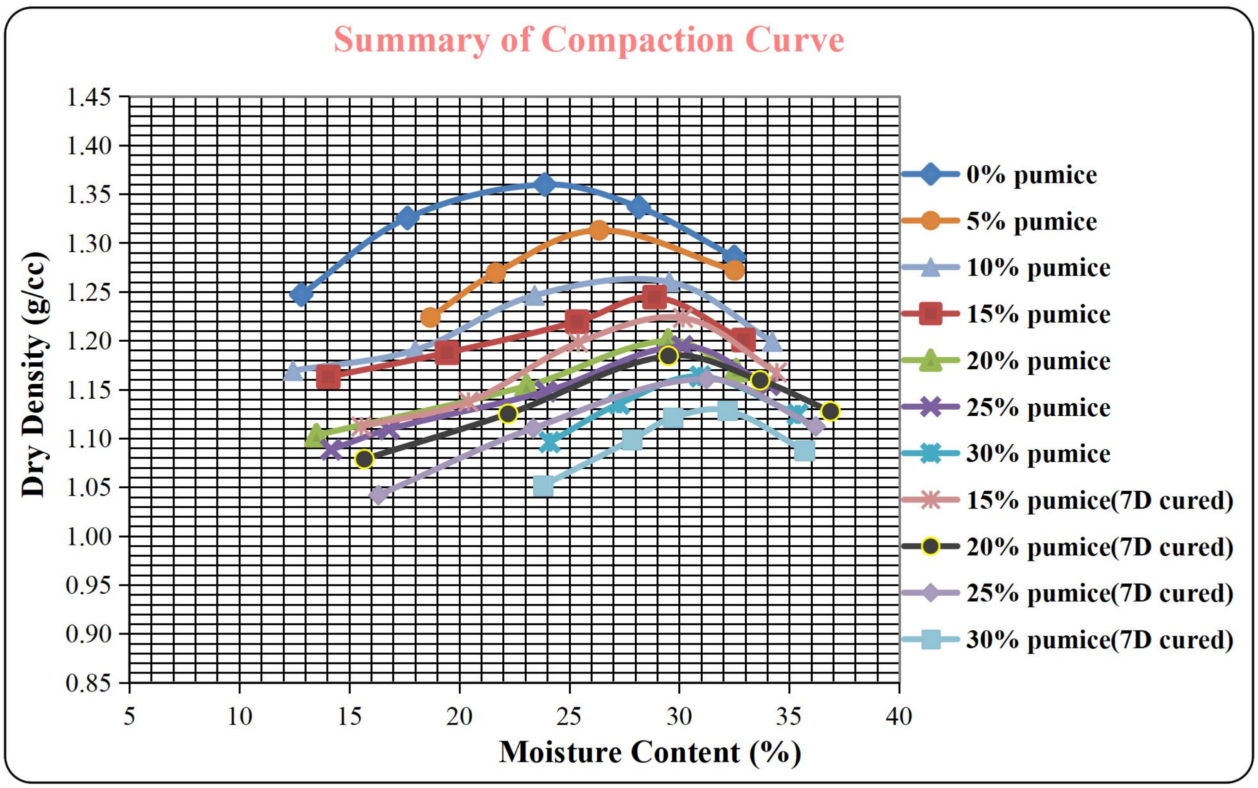


Figure 5. 4: Summery of compaction curves with application of different pumice contents (uncured & cured case)

5.3.2 Effect of Pumice on CBR Characteristics

The variations of California Bearing Ratio (CBR) with different percentages of soil and soil-pumice combinations are shown in table 5.6 for uncured case and table 5.7 for cured case. As shown in the same tables, as the percentage of pumice increases the CBR value also increases. The maximum California Bearing Ratio (CBR) value of 42.94 for uncured case and 43.7 for 7 days cured case at the combination of 30% pumice content under soaked condition. The maximum California Bearing Ratio (CBR) value shows that the load- bearing capacity of the soil sample increases with pumice treatment and this may be due to the pozzolanic reactions of pumice with soil. As pumice is mixed with soil in the presence of water, a series of reactions occurs, resulting in the dissociation of ($CaCO_3$, one of the collapsible soil's bonding agents), and (SiO_2 & Al_2O_3, the most dominant mineral present in pumice) in the binders. As a result, the formation of cementations and pozzolanic gels [calcium silicate hydrate gel (CSH) and calcium aluminate silicate hydrate gel (CASH)]. These reactions are referred to as cementations and/or pozzolanic reactions that result in the formation of cementation gels.

Table 5. 6 : Summary of CBR with application of different pumice content (uncured case)

Pumice Content (%)	**0**	**10**	**15**	**20**	**25**	**30**
CBR	20.74	23.47	27.8	36.03	40.6	42.94

Table 5. 7: Summary of CBR with application of different pumice contents (cured case)

Pumice Content (%)	**15**	**20**	**25**	**30**
CBR	30.9	37.5	41.3	43.7

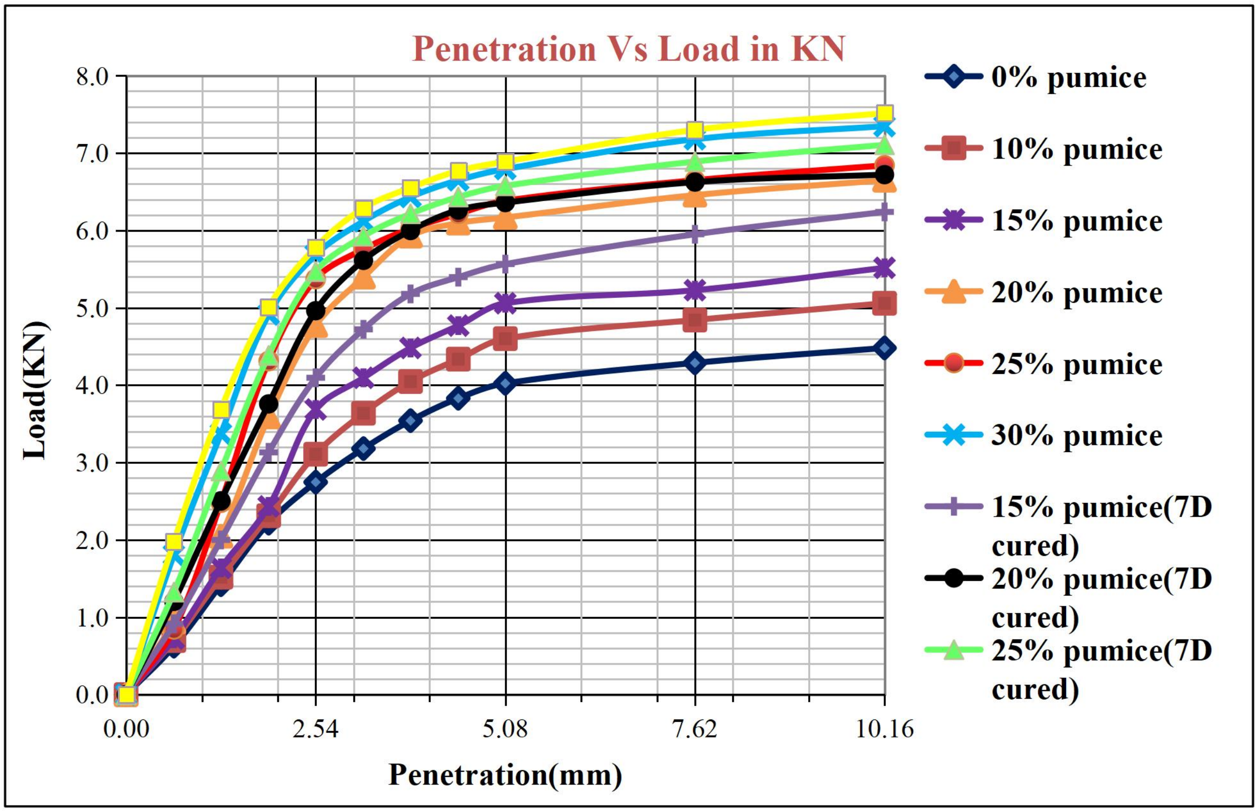


Figure 5. 5: Variation of soaked CBR with application of different pumice contents (uncured & cured case)

5.3.3 Effect of Pumice on Collapsible Potential the Soil

The results of the single oedometer tests are studied to determine the collapse potential value (C_p [%]) for undisturbed natural collapsible soils, natural compacted soil and soil-pumice stabilized sample as shown in Figure 5.7 for uncured and cured case.

The collapse potential of a single oedometer test for natural un-compacted soil ((C_p= 6.72) is higher than when compared with natural compacted soil (C_p= 2.24) and, the natural compacted soil has a low void ratio (e_0= 0.919) compared with natural un-compacted soil (e_0= 1.193). This is due to the loose of structure or weak structure and large spaces between the particles of un-compacted soil. This result shown that the compaction of collapsible soil affects its behavior and reduces the problem of collapsibility and the results of collapse value of natural untreated compacted soil decrease with range (2.17, 1.6, 1.2, 0.33, and 0.17) when mixed with 10, 15, 20, 25 & 30% pumice respectively for uncured case. For 7 days cured case, the results of collapse value of natural untreated compacted soil decrease with range (1.35, 0.85, 0.19, 0.12) when mixed with 15, 20, 25 & 30% pumice respectively.

The decreasing in collapse Index (C_p) results indicating a decrease of deformation due to wetting of the treated soil and thus improving the physical and mechanical characteristics of the treated soil or due to pozzolonic reaction of pumice with soil. The pozzolanic reaction takes place as the silica and alumina present in pumice with calcium from collapsible soil to form cementitious products such as calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH).

Table 5. 8: Results of single oedometer test with the application of different pumice contents (uncured case)

Pumice Content (%)	**0 at bulk**	**0 at MDD**	**10**	**15**	**20**	**25**	**30**
Collapsible Index	6.72	2.235	2.17	1.6	1.2	0.33	0.17

Table 5. 9: Results of single oedometer test with the application of different pumice contents (cured case)

Pumice Content (%)	**15**	**20**	**25**	**30**
Collapsible Index	1.35	0.85	0.19	0.12

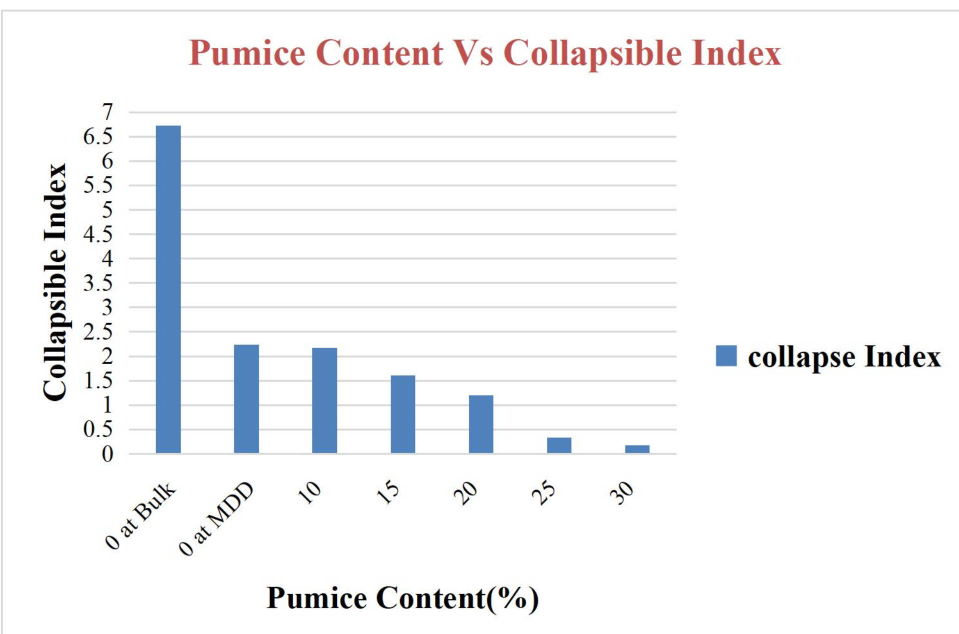


Figure 5. 6: Variation of collapsible potential with the application of different pumice contents (uncured case)

From the finding of test results, Collapse Index(I_c) of treated collapsible soil reduced significantly when the Pumice content is 25%, which indicates that 25% pumice content is the optimum amount to achieve the acceptable level of collapse index which is slight collapse Potential according to ASTM D5333. Also this the optimum amount of pumice achieves the highest strength and decrease of deformation due to wetting.

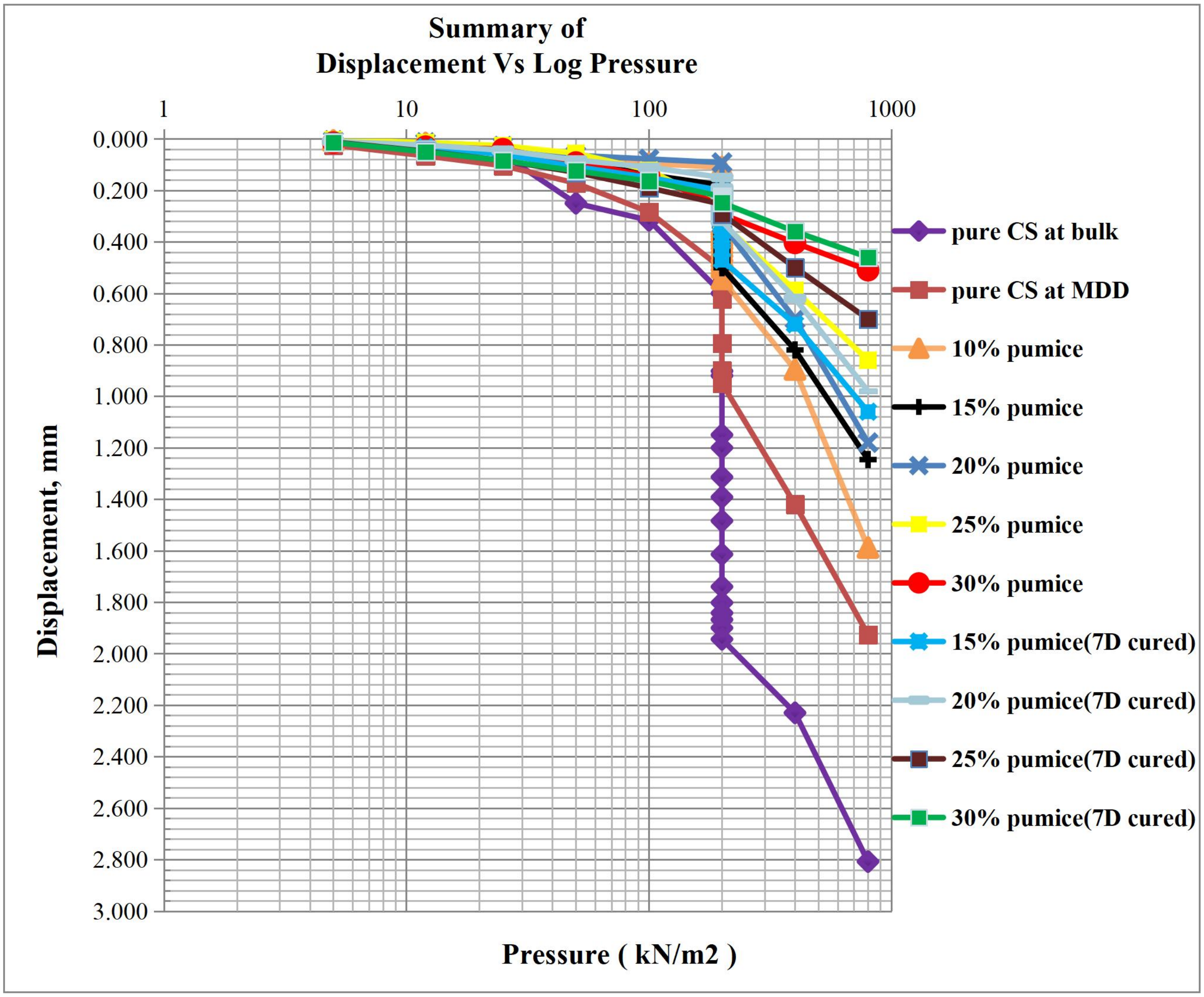


Figure 5. 7: Summary of collapsible potential with the application of different pumice contents (uncured & cured case)

5.3.3.1 Settlement

The settlement obtained from single oedometer test for natural un-compacted soil (S_{col} = 1.944) is higher than when compared with natural compacted soil (S_{col} = 0.950). As it was expected, compaction of collapsible soil affects its behavior and reduces the problem of settlement. The results of settlement of natural untreated compacted soil decrease with (0.544, 0.5, 0.332, 0.326, 0.292) when mixed with 10, 15, 20, 25 & 30% pumice respectively for uncured case. For 7 days curing, the results of settlement decrease with (0.471, 0.320, 0.292, 0.248) when the natural soil mixed with 15, 20, 25 & 30% pumice respectively.

It can be observed that the collapse settlement (S_{col}) for treated soil (uncured and cured case) is considerably less than that for natural compacted soil. The results indicated that as the collapse potential decreases, the settlement decreases. The results also confirmed that adding of pumice on the collapsible soil improves its behavior and reduces the collapse settlement.

Table 5. 10: Summary of settlement with the application of different pumice contents (uncured case)

Pumice Content (%)	**0 at bulk**	**0 at MDD**	**10**	**15**	**20**	**25**	**30**
Settlement	1.944	0.950	0.544	0.5	0.332	0.326	0.292

Table 5. 11: Summary of settlement with the application of different pumice contents (cured case)

Pumice Content (%)	**15**	**20**	**25**	**30**
Settlement	0.471	0.320	0.292	0.248

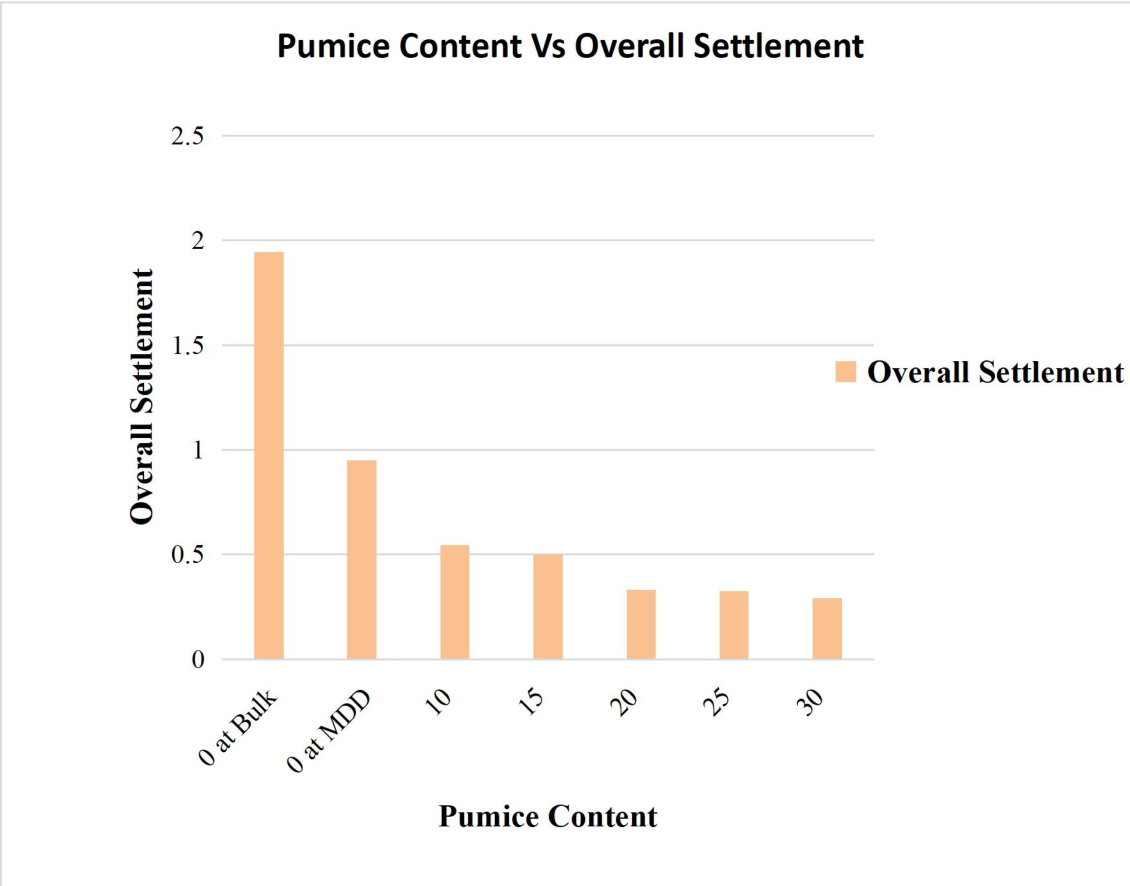


Figure 5. 8: Settlement with the application of different pumice contents (uncured case)

5.3.3.2 Settlement Versus Collapsible Index

The collapse index is directly proportional to settlement. The test results in Figure 5.9. Show that as the collapse potential decreases, the settlement also decreases. Due to the increased in surcharge pressure, the soil mixture settles gradually prior to inundation. This phenomenon can be seen in the early stages of the oedometer test, where the addition of the pressure has a significant effect on the settlement of the soil. As a result, the sudden settlement of the soil during inundation (collapse) is less significant. It can be easily observed that the collapse settlement (S_{col}) and the collapse index (I_e) for improved soil is considerably less than that for natural soil.

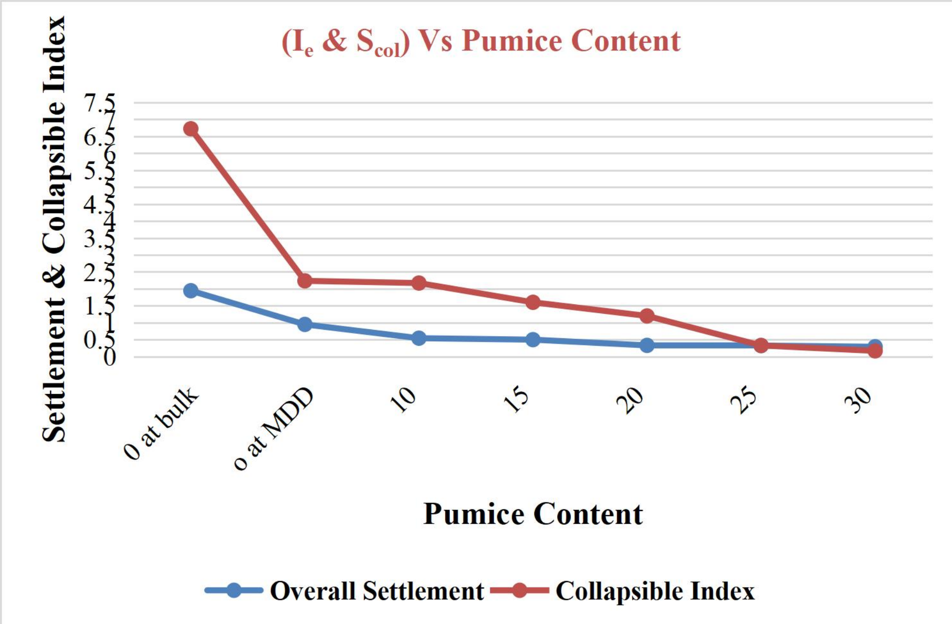


Figure 5. 9: Settlement and collapse index with the application of different pumice contents (uncured case)

5.3.4 Effect of Pumice on Shear Strength the Soil

The direct shear tests were performed following ASTM D 6528 and were conducted on untreated and pumice-treated samples compacted at the maximum dry densities and optimum moisture contents. The effect of pumice on angle of internal friction and cohesion of collapsible soil is shown in Table 5.12 and 5.13 for uncured and cured case respectively. The Addition of pumice has a significant effect on shear strength on samples containing pumice.

Table 5. 12: Variation of the angle of internal friction & cohesion with an application of different pumice contents (uncured case)

Pumice Content (%)	**0 at bulk**	**0 at MDD**	**10**	**15**	**20**	**25**	**30**
Angle of internal friction	6.72	31.48	35.95	37.37	38.70	39.81	40.9
Cohesion	0.68	9.23	19.50	22.10	24.3	26.39	28

Table 5. 13: Variation of the angle of internal friction & cohesion with an application of different pumice contents (cured case)

Pumice Content (%)	**15**	**20**	**25**	**30**
Angle of internal friction	37.6	39.15	39.94	41.01
Cohesion	23.59	25.45	27.6	29.15

The angle of internal friction for natural un-compacted soil ((ϕ= 6.72^0) is lower than when compared with natural compacted soil (ϕ= 31.48^0). This is due to the loose structure or weak structure and large spaces between the particles of natural soil. There is a considerable increase in the angle of internal friction from 31.48^0 to 40.9^0 for uncured treated soil and from 31.48^0 to 41.01^0 for 7 days cured treated soil samples with increased pumice content from 0% to 30%.

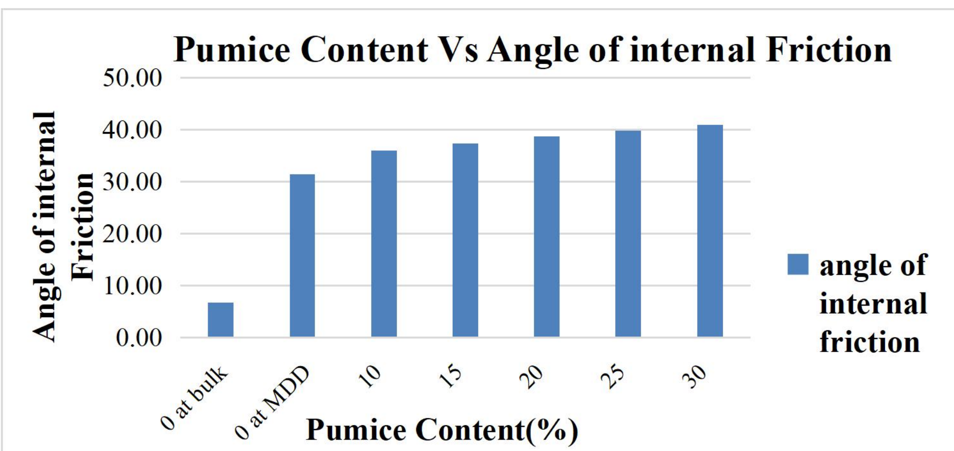


Figure 5. 10: Variation of angle of the internal friction with an application of different pumice contents (uncured case)

The variation of cohesion with different percentage of soil and pumice combination is shown in Figures 5.11 for uncured case. Addition of pumice had a significant effect on cohesion. There is a considerable increase in cohesion in sample containing different percentage of pumice. The improvement in the cohesion and internal friction angle value may be due to pozzolanic activities and cementitious characteristics of pumice.

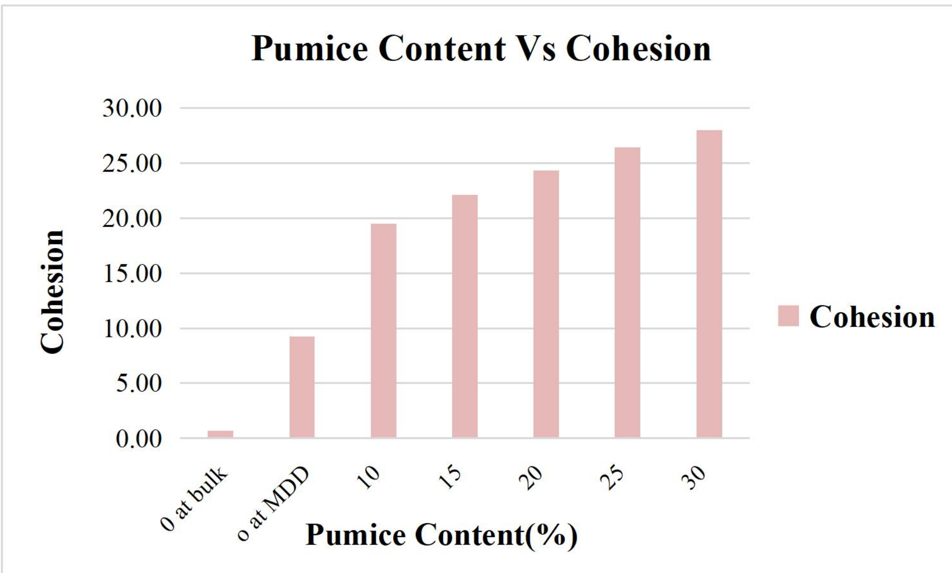


Figure 5. 11: Variation of cohesion with the application of different pumice contents (uncured case)

5.3.5 Evaluation of Collapse Potential by Indirect Method (Using Physical Property of the Soil)

The evaluation of collapse potential or susceptibility of soil to collapse using the physical properties of soil was reported in the literature as an indirect method for collapse potential evaluation. Several collapse criteria have been proposed for predicting whether a soil is liable to collapse upon saturation. The results of using these criteria are shown in Tables 5.14,5.15 & 5.16.

(Abelev, 1948) developed the first criteria for determining if the tested soils are collapsible. This criterion uses variations in soil void ratio before and after saturation to determine the susceptibility of soil to collapse. In terms of the above criteria, if δs is greater than 2 percent then the soil will be susceptible to collapse. (The test results of Tables 5.14) shown that the studied soils have δs>2 at (natural soil at bulk &, MDD, 10% pumice & 15% pumice) and this indicate that Collapse is probable in the soils. The soil is not susceptible to collapse at 20, 25 & 30% pumice content.

Table 5. 14: Evaluation of collapse potential using (Abelev, 1948) Criteria

Investigator, year	**Criteria**	**Value & Result**	**Pumice Content(%)**	
Abelev, 1948	$\delta s = (\Delta e / (e_L + 1)$ % Collapse is probable when: $\delta s > 2\%$	δs	0% at Bulk	9.72
			0% at MDD	4.75
			10%	2.72
			15%	2.50
			20%	1.66
			25%	1.63
			30%	1.46

The test result described according to Lin and Wang (1988) criterion is shown in the table 5.15. below. The results of single oedometer show that the collapse value (i_{cz} = 6.72%) natural untreated soil and the collapse value (i_{cz} = 2.235%) of compacted untreated soil at moisture content equal to optimum moisture content. The collapse value of natural untreated compacted soil decreases with range (i_{cz} = 2.17,1.602, 1.2, 0.329, 0.17) when mixed with 10, 15,20,25 & 30% pumice respectively. This indicated the soil is highily collapsible when natural soil at bulk condition, medium collapsible when natural soil at MDD ,10 15, & 20% pumice content and, the soil is not susceptible to collapse at 25 & 30% pumice content.

Table 5. 15: Evaluation of collapse potential using (Lin and Wang, 1988) Criteria

Investigator, year	Criteria	Value & Result	Pumice Content(%)	
Lin and Wang criteria (1988)	$i_{cz} = h_z - h_{zs}/h_1$ when: (0-1) % - No Collapsibility (1-5) % - Medium Collapsibility (5-10) %- High Collapsibility (10-20) % -Very High Collapsibility (>20) % - Extremely Collapsible	i_{cz}	0% at Bulk	6.72
			0% at MDD	2.235
			10%	2.17
			15%	1.602
			20%	1.2
			25%	0.329
			30%	0.17

The test result described according to Sabbagh (1982) criterion is shown in the table 5.16. below. The results of single oedometer show that the collapse value (C_p = 6.72%) natural untreated soil and the collapse value (C_p = 2.235%) of compacted untreated soil at moisture content equal to optimum moisture content. The collapse value of natural untreated compacted soil decreases with range (C_p = 2.17, 1.602, 1.2, 0.329, 0.17) when mixed with 10,15 20,25 & 30% pumice respectively. This indicated the soil is highly collapsible when natural soil at bulk condition, medium collapsible when natural soil at MDD & 10% pumice content and, the soil is not susceptible to collapse at 15, 20 25 & 30% pumice content.

Table 5. 16: Evaluation of collapse potential using (Sabbagh , 1982) Criteria

<table>
<tr><th>Investigator, year</th><th>Criteria</th><th>Value & Result</th><th colspan="2">Pumice Content(%)</th></tr>
<tr><td rowspan="7">Sabbagh (1982)</td><td rowspan="7">$C_p = \Delta H/H_0$
when:
If $0 < C_p \leq 2\%$, the soil is low collapsible (LC)
If $2\% < C_p \leq 5\%$, the soil is moderately collapsible (MC)
If Cp > 5%, the soil is highly collapsible (HC)</td><td rowspan="7">C_p</td><td>0% at Bulk</td><td>6.72</td></tr>
<tr><td>0% at MDD</td><td>2.235</td></tr>
<tr><td>10%</td><td>2.17</td></tr>
<tr><td>15%</td><td>1.602</td></tr>
<tr><td>20%</td><td>1.2</td></tr>
<tr><td>25%</td><td>0.329</td></tr>
<tr><td>30%</td><td>0.17</td></tr>
</table>

CHAPTER SIX: CONCLUSION AND RECOMMENDATION

6.1 Conclusion

Based on the study and results of the investigation, the following conclusions are drawn:

- Field and laboratory experiments on the collected soil samples showed that the soils have low density at their natural state, mostly made up of sands and silts, and have a non-plastic nature
- The increase of the pumice contents in the natural soil causes the reduction of the maximum dry density and increases the optimum water content. The decrease in maximum dry density is due to the domination of the low weight and specific gravity of pumice, hence the total dry weight of soil mixtures decreases. The increase of the OMC is due to additional water is required to wet the large surface area of the pumice particles, or it is absorbed by the fine pumice particles.
- One-dimensional collapse potential tests performed on remolded samples at their bulk densities and subjected to 200 kPa stress levels as well as in-situ stress levels anticipated at site revealed that the samples are moderately severe disorder collapsible soil, according to ASTM classification.
- One-dimensional collapse potential tests performed on samples remolded at their maximum dry densities and subjected to 200 kPa stress levels as well as in-situ stress levels anticipated at the site revealed that the samples are moderate collapsible soil, according to ASTM classification. This shows that the collapse potential of the collapsible soil sample slightly decreases with compaction.
- Results of One-dimensional collapse potential tests show that the collapse value of natural untreated compacted soil decreases when mixed with different percentage of pumice content. This may be attributed to changes in the soil's physical and chemical properties due to the pozzolanic reactions of pumice with soil. As pumice is mixed with soil in the presence of water, a series of pozzolanic reactions occurs. The pozzolanic reaction takes place as the silica and alumina present in pumice with calcium from collapsible soil to form cementitious products such as calcium-silicate-hydrates (CSH)and calcium-aluminate-

hydrates (CAH). The decrease in collapse value and increase in strength were found to be roughly related to the type and quantity of possible reaction products.

- Mixing the collapsible soil with a certain percent of pumice content results in an improvement of the collapsibility characteristics, and reduction of the collapse settlement upon wetting.
- The addition of pumice improved the CBR value. Hence, pumice can strongly improve the strength of the collapsible soil. This may be due to the pozzolanic reactions of pumice with soil.
- The angle of internal friction for natural un-compacted natural soil is slower than when compared with natural compacted soil. This is due to the natural soil's loose or weak structure and large gaps between the particles. And there is a considerable increase in the angle of internal friction for soil samples with increased pumice content. Addition of pumice had a significant effect on cohesion. There is a considerable increase in cohesion in sample containing different percentage of pumice. The improvement in the cohesion and internal friction angle value may be due to pozzolanic activities and cementitious characteristics of pumice.
- Effect of 7 days curing on compaction, CBR, direct shear and collapsible potential tests for treated soil sample have been observed.
- From the finding of test results, Collapse Index(Ic) of treated collapsible soil reduced significantly when the Pumice content is 25%, which indicates that 25% pumice content is the optimum amount to achieve the acceptable level of collapse index which is slight collapse Potential according to ASTM D5333. Also this the optimum amount of pumice achieves the highest strength and decrease of deformation due to wetting.

6.2 Recommendation

The present work has attempted to obtain the optimum blending proportion of Pumice material with collapsible soil which results a reduction in the collapsing behavior of these soil samples collected from specific area of Ziway around Adamitulu Woreda. However, due to financial constraints and time limitations the present research work did not cover the whole aspects of collapsible soil treatment with pumice. As a result, the following recommendations for better use of pumice material as a stabilizing agent for collapsible soils should be considered.

- Due to geological formation and some other factors collapsible soils may have different properties from place to place. Thus, it is recommended that the performance of pumice as stabilizing agent should be studied on collapsible soils of different origins.
- Intensive studies on the alternative treatment methods of collapsible soils based on the magnitude of the collapsibility, the effectiveness of the methods and the cost to be incurred have to be undertaken.

REFERENCES

1. Abdelmohsen, H., & Ali, N. (2014), improvement of collapsible soils, The eighth Alexandria international conference on Structural and Geotechnical Engineering Department of Structural Engineering, Faculty of Engineering, Alexandria University, Egypt 14-16.
2. Alfi, A. (1984). Mechanical and Electron Optical Properties of a Stabilized Collapsible Soil in Tucson Arizona (Microscopy, Lime-Stabilization). Phd thesis, University of Arizona.
3. Ali, N. A. (2016). Controlling collapsibility potential by partial replacement. International Conf. on Structural and Geotech. Eng. 14-16 April. Alex. Egypt. Global Journal of Researches in Engineering, Volume XVI, Issue I Version I.
4. Al-Rawas, A. (2000). State-of-the-Art Review of Collapsible Soils. Science and Technology, Special Review, Department of Civil Engineering, College of Engineering, Sultan Qaboos University, Sultanate of Oman,115-135.
5. Alshaba, A., Abdelaziz T., & Ragheb A. (2018). Treatment of collapsible soils by mixing with iron powder, Alexandria Engineering Journal, 57 (3737-3745).
6. ASTM. (1993). American society for testing materials. Standard test method for measurement of collapse potential of soils, ASTM D 5333 – 92 (Reapproved 1996), West Conshohocken, PA 19428-2959, United States.
7. Barden, L., McGown, A., & Collins, K., (1973). The collapse mechanism in partly saturated soil. Engineering Geology, 7, 49 – 60.
8. Basma, A., & Tuncer, E. R. (1992). Evaluation and control of collapsible soils. Journal of Geotechnical Engineering, 118(10), 1491-1504.
9. Bell, F.G., & Culshaw, M.G. (2001). Problem soils: a review from a British perspective, In: Proceeding of Problematic Soils Conference. Nottingham,1–37.
10. Bellil, S., Abbeche K., & Bahloul O., (2018). Treatment of collapsible soil using a bentonite–cement mixture. StudiaGeotechnica et Mechanica, 40(4), 233–243, published by sciendo.
11. Bigdeli, A. (2018). Evaluation and Control of Collapsible Soils in Okanagan-Thompson Region, Phd Thesis, University of British Columbia(Okanagan).
12. Charles, W.W. Ng., & Bruce, M. (2007). Advanced Unsaturated Soil Mechanics and Engineering, First Edition.
13. Coduto, D. P. (2001). Foundation Design Principles and Practices, third Edition.
14. Danait, H. (2015). Investigation on the impact of surface and ground water use for irrigation on soil salinity development and its effect on plant productivity in the central rift valley region around lake ziway. MSc thesis, Addis Ababa, Ethiopia.
15. Das, B. M. (2011). Principles of Foundation Engineering, Seventh Edition.
16. Das, B. M. and Sobhan, K. (2017). Principles of Geotechnical Engineering, Ninth Edition
17. ERA. (2013). Ethiopian Roads Authority. Geotechnical Design Manual. Addis Ababa, Ethiopia.
18. Esmaeli, M., Rafie B. M., & Moayed R. Z. (2008). Evaluation of Soil Collapsibility Potential: A Case Study of Semnan Railway Station. Electronic Journal of Geotechnical Engineering, Volume 13.

19. Evans, R.D., Jefferson, I., Northmore, K.J., & Jackson, P. (2004). In-situ investigation of problematical soils. In: Advances in Geotechnical Engineering: The Skempton Conference on 29–31 March. London, UK: Thomas Telford Publishing,1269–1279.
20. Feda, J. (1995). Mechanisms of collapse of soil structure. In: Derbyshire, E. et al. eds. Genesis and Properties of Collapsible Soils. Kluwer Academic Publishers, Netherlands,149–172.
21. Gaaver, K. (2012). Geotechnical properties of Egyptian collapsible soils. Alexandria Engineering Journal, 51(3), 205 - 210.
22. Hamid, O. (2012). A Thesis Submitted in Partial Fulfillment for the Degree of Master of Science in Building Technology. MSc Thesis, University of Khartoum.
23. Hosseini, A., Haeri, S.M., Mahvelati, S., & Fathi, A. (2019). Feasibility of using electrokinetics and nanomaterials to stabilize and improve collapsible soils. Journal of Rock Mechanics and Geotechnical Engineering ,1055-1065.
24. Houston, S., Houston, W., & Lawrence, C. (2002). Collapsible soil engineering in highway infrastructure development. Journal of Transportation Engineering, 128 (3), 295 – 300.
25. Houston, S., Houston, W., W.N., Zapata, C.E., & Lawrence, C. (2001). Geotechnical engineering practice for collapsible soils. Geotechnical and Geological Engineering, 19,333–355.
26. Kalantari, B. (2013), Foundation on collapsible soil: A review. proceeding of the institution of civil engineers, forensic engineering, Civil Engineering Department, University of Hormozgan, 166, 57-63.
27. Knodel, P. C. (1992), Characteristics and Problems of Collapsible Soils. U.S. Department of the Interior Bureau of Reclamation Denver Office Research and Laboratory Services Division Materials Engineering Branch.
28. Lawton, E., Fragaszy, R., & Hardcastle, J. (1989). Collapse of compacted clayey sand. Journal of Geotechnical Engineering, 115(9), 1257 - 1267.
29. Lawton, E. C., Fragaszy, R. J., and Hetherington, M. D. (1992). Review of Wetting-Induced Collapse in Compacted Soil. Journal of Geotechnical Engineering, 118(9), 1376–1394.
30. Li, P., Vanapalli, S., & Li, T. (2016). Review of collapse triggering mechanism of collapsible soils due to wetting. Journal of Rock Mechanics and Geotechnical Engineering 8(2), 256–274.
31. Meron, N. (2007). Characterization of salt affected soils in the central Rift Valley and assessing salt tolerance of different plants: A case study at the southwestern shore of Lake Ziway, MSc thesis, Addis Ababa, Ethiopia.
32. Moayed, Z., & Kamalzare M. (2015). Improving physical characteristics of collapsible soil (Case Study: Tehran-Semnan Railroad). Journal of Engineering Geology, 9(2),2869–2890. https://doi. org/10.18869/acadpub.jeg.9.2.2869.
33. Pereira, J., & Fredlund, D. (2000), Volume change behavior of collapsible compacted gneiss soil. Journal of Geotechnical and Geo-Environmental Engineering, 126(10), 907–916.
34. Rogers, C.D.F. (1995). Types and distribution of collapsible soils. In: Derbyshire, E.et al. Genesis and properties of collapsible soil. NATO ASI Series, Netherlands: Kluwer Academic Publishers, 468(1–17).

35. Rollins, K., & Rogers, G. (1994). Mitigation Measures for small structures on collapsible alluvial soils. Journal of Geotechnical Engineering, 120(9).
36. Tewodros, D. (2019). Index and Mechanical Properties of Collapsible Soils: A Case Study on Ziway-Arsi Negele Highway Project. MSc Thesis, Addis Ababa, Ethiopia.
37. Yohannes, M. (2019). Investigation on the Suitability of Pumice and Scoria Aggregates in Ribbed-Slab Construction. MSc thesis, Addis Ababa, Ethiopia.
38. Zamani, M., & Badv, K. (2019). Assessment of the Geotechnical Behavior of Collapsible Soils: A Case Study of the Mohammad-Abad RailwayStation Soil in Semnan. Geotech Geol Eng, Department of Civil Engineering, Faculty of Engineering, Urmia University, Urmia, Iran Springer Nature Switzerland.
39. Zerai, H. (2015). The potential use of cinder gravel as a base course material when stabilized by volcanic ash and lime (the case of metehara area). MSc thesis, Addis Ababa, Ethiopia.
40. Ziani, H., Abbèche K., Messaoudene I., & Pais A. (2019). Treatment of Collapsible Soils by Additions of Granulated Slag and Natural Pozzolan. KSCE Journal of Civil Engineering, pISSN 1226-7988, eISSN 1976-3808.

APPENDIX

APPENDIX A: SUMMARY OF TEST RESULTS OF THE STUDY

1. Natural Collapsible Soil

1.1 Specific Gravity Test

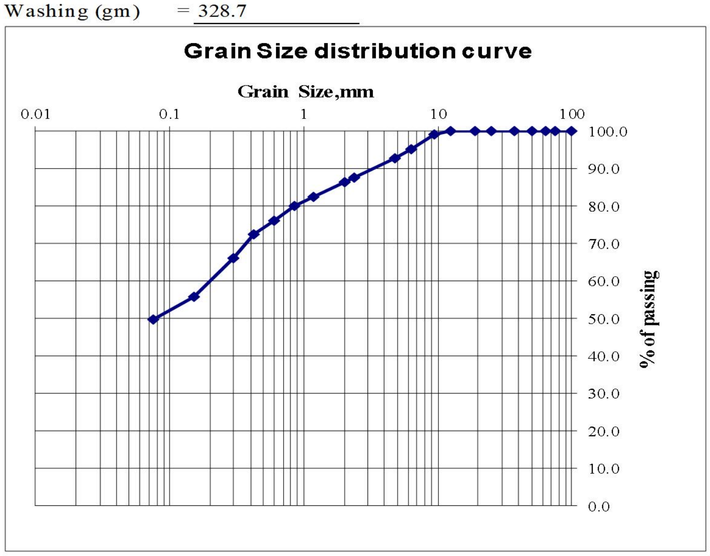
Specific Gravity: ASTM D854-98			
Trial	1	2	3
Pycnometer #	2	4	3
A. Mass of Pycnometer (empty, clean)	63.91	63.68	63.7
B. Mass of empty pycno + Dry soil	83.9	83.65	83.68
C. Mass of Dry Soil (No. 10 sieve)	19.99	19.97	19.98
D. Mass of Pycnometer full of water	172.65	165.96	165.98
E. Mass of pycno + Soil +full of water (boiled)	184.88	178.17	178.2
F. Volume of Sample=C+D-E	7.76	7.76	7.76
G. Specific Gravity=C/F	2.576	2.573	2.575
Average Specific Gravity ((1+2+3)/3)	2.57		

1.2 Gradation

Wet Sieve Analysis

Weight of Total Sample Before Washing (gm) = 328.7

Sieve Size(mm)	Wt. Ret (gm)	% Ret.	% pass
100	0	0.00	100.0
75	0	0.00	100.0
63.5	0	0.00	100.0
50	0	0.00	100.0
37.5	0	0.00	100.0
25	0	0.00	100.0
19	0	0.00	100.0
12.5	0	0.00	100.0
9.5	3.1	0.94	99.1
6.3	12.7	3.86	95.2
4.75	8.1	2.46	92.7
2.36	17	5.17	87.6
2.0	3.7	1.13	86.4
1.18	12.9	3.92	82.5
0.85	7.9	2.40	80.1
0.60	12.8	3.89	76.2
0.425	11.8	3.59	72.6
0.30	21.4	6.51	66.1
0.15	33.8	10.28	55.8
0.075	20.1	6.11	49.7
Sub Total	165.3		
Pan	163.4	49.7	
Total	328.7	100.00	



1.3 Hydrometer Analysis

Wt Dry soil, Ws =	50.00 g
Specific Gravity, Gs =	2.575
Hydrometer Type :	152H *
Correction factor for Gs ,(a=)	1.015

Hydrometer Analysis, ASTM D422/AASHTO-T88

Elapsed time (Minutes)	Actual Hydrometer Reading (Ra)	Testing Temprature (°C)	Composte correction	Corrected Hydrometer Reaqding (Rc)	Effective depth,L (cm)	K	Diameter of the particle, D, mm	Percent of Soil in Suspension (%)	Total percentage finer
2	27.000	20	6.9000	20.1	11.90	0.01397	0.0341	40.803	40.35
5	25.000	20	6.9000	18.1	12.20	0.01397	0.0218	36.743	36.34
15	22.000	20	6.9000	15.1	12.70	0.01397	0.0129	30.653	30.32
30	21.000	20	6.9000	14.1	12.90	0.01397	0.0092	28.623	28.31
60	20.000	20	6.9000	13.1	13.00	0.01397	0.0065	26.593	26.30
250	18.000	20	6.9000	11.1	13.30	0.01397	0.0032	22.533	22.29
1440	17.000	20	6.9000	10.1	13.50	0.01397	0.0014	20.503	20.28

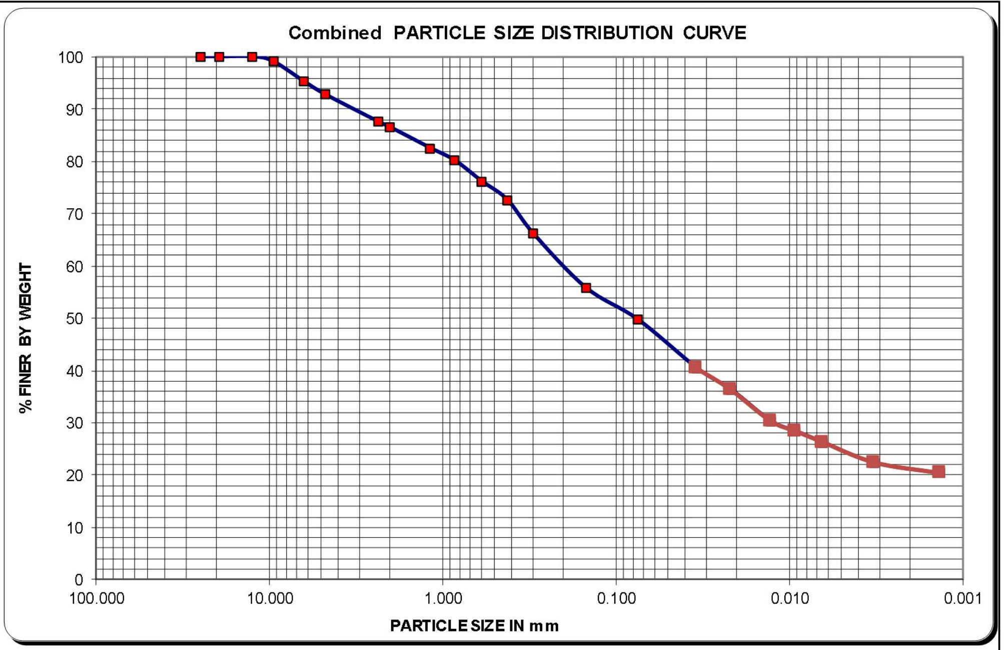
Hygroscopic moisture		
Mass of Can + Air Dried Soil=	102.95	gm
Mass of Can + Oven Dry Soil=	98.5	gm
Mass of Can=	20.36	gm
Moisture=	5.69	%

To correct the weight of air-dried sample for hygroscopic moisture, the given value shall be multiplied

$$\frac{100}{100 + \text{Percentage of hygroscopic moisture}} = 47.306$$

1.4 Grain Size Analysis (Sieve and Hydrometer Analysis) Summary Result

Grain size, mm	Percent Finer Than
25.00	100
19.50	100
12.50	100
9.50	99.06
6.30	95.19
4.75	92.73
2.36	87.56
2	86.43
1.18	82.51
0.85	80.10
0.6	76.21
0.425	72.62
0.300	66.11
0.15	55.83
0.075	49.71
0.0341	40.35
0.0218	36.34
0.0129	30.32
0.0092	28.31
0.0065	26.30
0.0032	22.29
0.0014	20.28



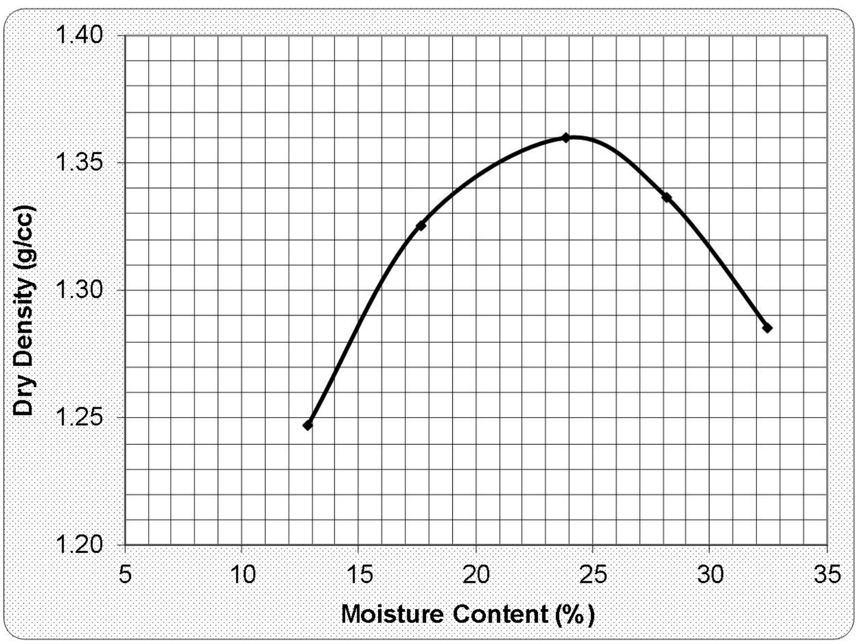
Proportions of particles	
Sand (2.00mm-0.075mm),%=	50.3
Silt (0.075mm-0.002mm), %=	27.7
Clay (less than 0.002), % =	22.0

1.5 Moisture Density Relationship (Procter Test)

Moisture Content and Density Relationship

AASHTO T-99, Method A

Trial No	1	2	3	4	5	6
Mold weight + sample (gm)	5749	5893	6011	6038	6029	
Mold weight (gm)	4421	4421	4421	4421	4421	
Sample weight (gm)	1328	1472	1590	1617	1608	
Mold volume (Cm3)	944	944	944	944	944	
Bulk density(g/cm3)	1.41	1.56	1.68	1.71	1.70	
	Moisture Test Results					
Wet weight +can	151.00	160.00	188.00	189.00	157.43	
Dry weight + can	136.00	138.70	156.00	152.30	123.82	
Weight of can	19.00	18.00	22.00	22.00	20.40	
Weight of moisture	15.00	21.30	32.00	36.70	33.61	
Weight of dry soil	117.00	120.70	134.00	130.30	103.42	
Moisture content (%)	12.82	17.65	23.88	28.17	32.50	
Dry density (g/cm3)	1.25	1.33	1.360	1.34	1.29	



MDD (g/cc) **1.36**

OMC (%) **25.0**

1.6 Natural Moisture Content

Natural Moisture Content		
AASHTO T-89 & T- 90		
Container No.		
Wt of wet soil + container (M_{CMS}) , gm	591.00	654.00
Wt of dry soil + container (M_{CDS}) , gm	580.00	636.50
Wt of container (M_C), gm	493.00	494.00
Wt of water (M_W), gm	11.00	17.50
Wt of dry soil (M_S), gm	87.00	142.50
Water content, % (W)	12.64	12.28
Average NMC (%)	12.46	

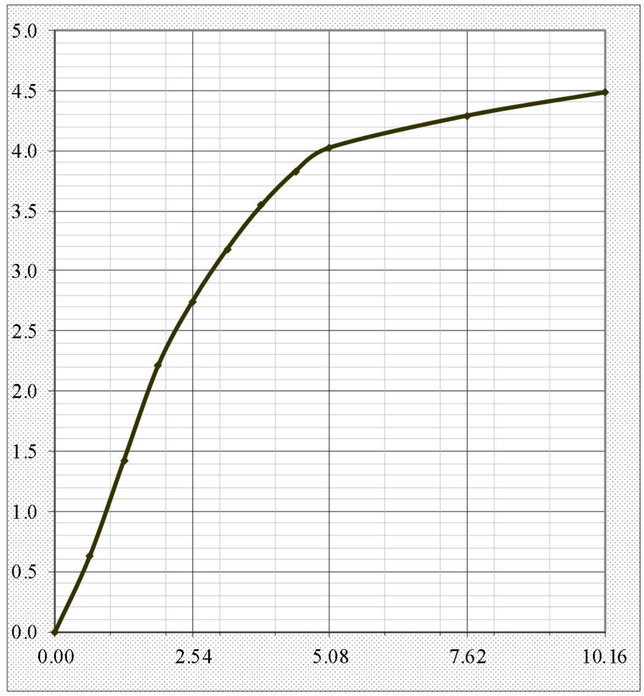
1.7 Field density

In Situ Density Determination	
Using Sand Replacement Method(AASHTO T-191)	
wt. of sand in cone Wsc(gm)	1388
Density of sand Ds(gm/cc)	1.248
Depth of hall	15cm
wt. of soil Wws	2686.5
Initial Wt. of sand in bottle (Wsb1)	6740
Final Wt. of sand in bottle (Wsb2)	2808
Wt. of sand in bottle & Cone (Wsbc=Wsb1-Wsb2)	3932
Wt. of sand in cone(Wsc)	1388
Wt. of sand in hole(Wsh=Wsbc-Wsc))	2544
Volumeof sand in hole(vsh=Wsh/Ds)	2038.462
Bulk density of Wet soil(WD=Wws/Vsh)	1.318

1.8 Californian Bearing Ratio (CBR) Tests

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	26.0	0.63
1.27	59.0	1.42
1.91	92.0	2.22
2.54	114.0	2.75
3.18	132.0	3.18
3.81	147.0	3.54
4.45	159.0	3.83
5.08	167.0	4.02
7.62	178.0	4.29
10.16	186.0	4.48

CBR %	
2.54mm	5.08mm
20.7	20.2



1.9 Direct Shear Tests at bulk

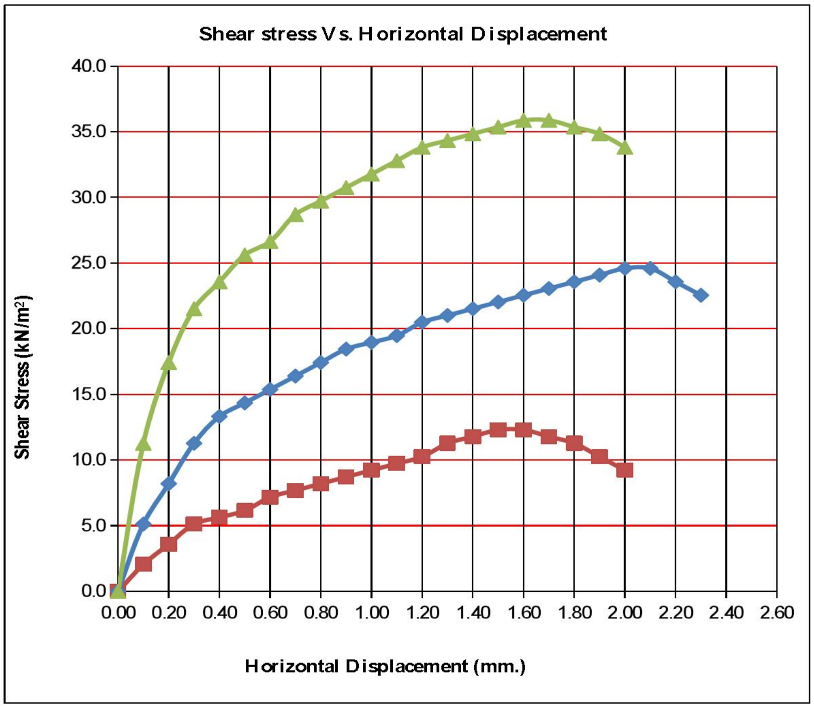
Direct Shear Test, ASTM-D 3080

SHEAR BOX DATA

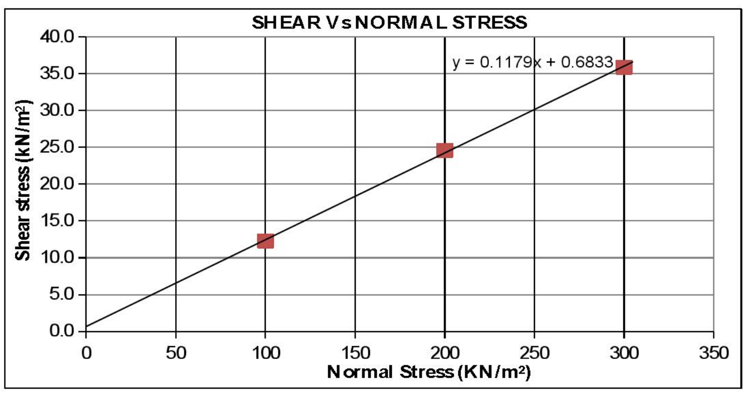
Length= 60 mm
Width= 60 mm
Depth= 20 mm
Area= 0.0036 m^2

Shear test Type : CD
Proving Ring Factor= 0.0037kN
Proving Ring Factor= 0.01mm
Laoding Rate= 2mm/min

H.D(MM.)	Specimen 1 shear str. (kN/m^2)	Specimen 2 shear str. (kN/m^2)	Specimen 3 shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	2.05	5.13	11.28
0.20	3.59	8.20	17.43
0.30	5.13	11.28	21.53
0.40	5.64	13.33	23.58
0.50	6.15	14.35	25.63
0.60	7.18	15.38	26.65
0.70	7.69	16.40	28.70
0.80	8.20	17.43	29.73
0.90	8.71	18.45	30.75
1.00	9.23	18.96	31.78
1.10	9.74	19.48	32.80
1.20	10.25	20.50	33.83
1.30	11.28	21.01	34.34
1.40	11.79	21.53	34.85
1.50	12.30	22.04	35.36
1.60	12.30	22.55	35.88
1.70	11.79	23.06	35.88
1.80	11.28	23.58	35.36
1.90	10.25	24.09	34.85
2.00	9.23	24.60	33.83
2.10		24.60	
2.20		23.58	
2.30		22.55	



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	12.30	24.60	35.88



Cohesion, C (kN/m^2) = 0.68

Angle of internal friction, Ø° =6.72°

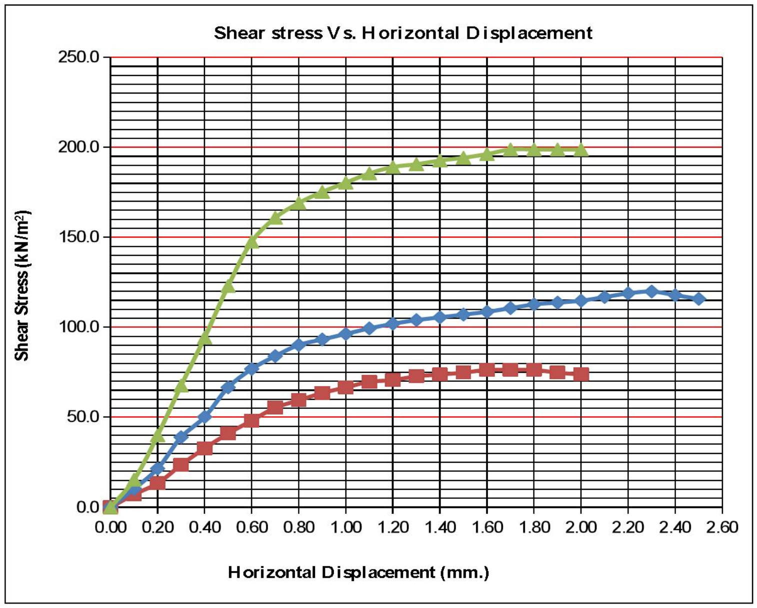
1.10 Direct Shear Tests at MDD

Direct Shear Test, ASTM -D 3080

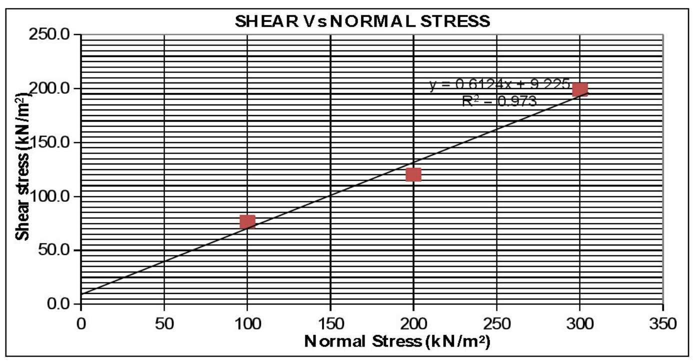
Shear test Type : CD
Proving Ring Factor= 0.00369
Proving Ring Factor= 0.01mm
Laoding Rate= 2mm/min

Length= 60 mm
Width= 60 mm
Depth= 20 mm
Area= 0.0036 m^2

H.D(MM.)	Specimen 1 shear str. (kN/m^2)	Specimen 2 shear str. (kN/m^2)	Specimen 3 shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	7.18	10.25	15.38
0.20	13.33	21.53	39.98
0.30	23.58	38.95	67.65
0.40	32.80	50.23	94.30
0.50	41.00	66.63	123.00
0.60	48.18	76.88	147.60
0.70	55.35	84.05	160.93
0.80	59.45	90.20	169.13
0.90	63.55	93.28	175.28
1.00	66.63	96.35	180.40
1.10	69.70	99.43	185.53
1.20	70.73	101.99	189.11
1.30	72.78	104.04	190.65
1.40	73.80	105.58	192.70
1.50	74.83	107.11	194.24
1.60	76.36	108.65	196.29
1.70	76.36	110.70	198.85
1.80	76.36	112.75	198.85
1.90	74.83	113.78	198.85
2.00	73.80	114.80	198.85
2.10		116.85	
2.20		118.90	
2.30		119.93	
2.40		117.88	
2.50		115.83	



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	76.36	119.93	198.85



Cohesion, C (kN/m^2) = 9.23   Angle of internal friction, Ø° =31.48°

1.11 Collapsible potential Tests at Bulk

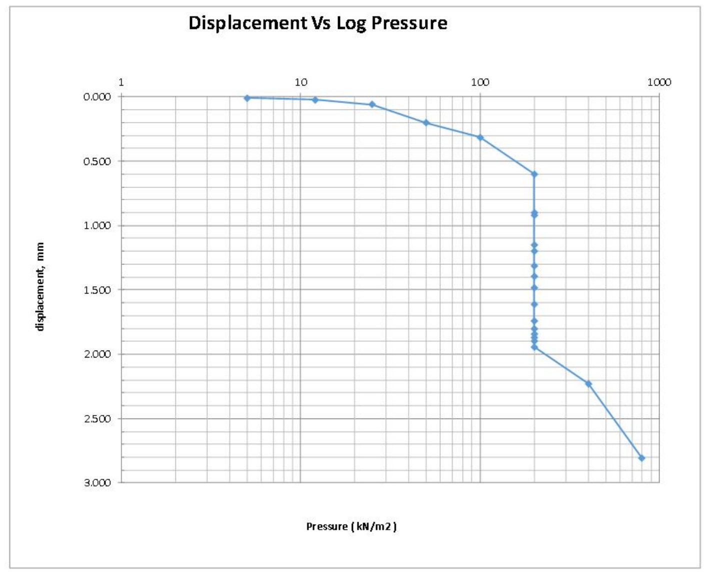
ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	1.944	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	17.908	Volume Change	3.82	Thickness of sample	20
Vvi=Vt-Vs	21.362	Final volume	35.45	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.193	Final bulk density	1.485	Volume of Sample	39.27
intial So= Vw/Vvi	26.840	Final Dry Density	1.29		
Volume change factor=F	0.110	Final void ratio	0.98		
		Vvf=Vtf-Vs	17.54		
		Final saturation Sf, %	39.90		

Moisture Determination

Before Test		After Test	
Wt.of can +wet sample(gm)	107.51	Wt.of ring +wet sample(gm)	124.8
Wt.of can +Dry sample(gm)	96.85	Wt.of ring +Dry sample(gm)	117.8
Wt.can(gm)	21.21	Wt.ring(gm)	72.14
Wt.of dry sample	75.64	Wt.of dry sample	45.66
Wt.of wet sample	86.3	Wt.of wet sample	52.66
Wt.of water(gm)	10.66	Wt.of water(gm)	7
Moisture Content(%)	14.09	Moisture Content(%)	15.33
Bulk Density	1.318		
Dry density	1.172		

Elapsed Time h	Time, Min.	d Stress, Kpa	Dial Reading, mm	ve Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	4.0	0.008	0.001	1.193
1		12	11.0	0.022	0.002	1.190
2		25	30.0	0.060	0.007	1.186
3		50	102.0	0.204	0.022	1.170
4		100	158.0	0.316	0.035	1.158
5		200	300.0	0.600	0.066	1.127
	0.1	200	451.0	0.902	0.099	1.094
	0.25	200	460.0	0.920	0.101	1.092
	0.5	200	575.0	1.150	0.126	1.067
	1	200	600.0	1.200	0.132	1.061
	2	200	657.0	1.314	0.144	1.049
	4	200	696.0	1.392	0.153	1.040
	8	200	742.0	1.484	0.163	1.030
	15	200	807.0	1.614	0.177	1.016
	30	200	870.0	1.740	0.191	1.002
1		200	901.0	1.802	0.198	0.995
2		200	921.0	1.842	0.202	0.991
4		200	934.0	1.868	0.205	0.988
8		200	950.0	1.900	0.208	0.985
24		200	972.0	1.944	0.213	0.980
32		400	1115.0	2.230	0.245	0.948
38		800	1404.0	2.808	0.308	0.885



Collapse Index, Ie Computation

d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	1.944
d_i, dial reading at 200KPa before wetting, mm	0.600
collapse Index $I_e=((d_f - d_i)/h_o)*100$	6.72

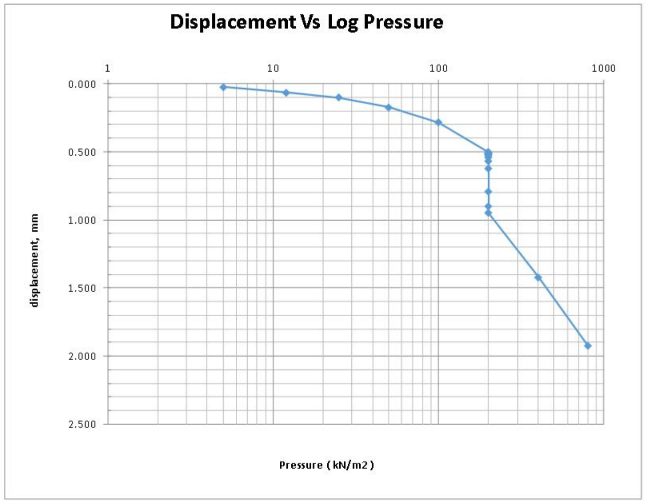
1.12 Collapsible potential Tests at MDD

ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.950	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	20.158	Volume Change	1.87	Thickness of sample	20
Vvi=Vt-Vs	19.112	Final volume	37.40	Area of sample	1963
intial Void ratio e0=Vv/Vs	0.948	Final bulk density	1.775	Volume of Sample	39.27
intial So= Vw/Vvi	71.284	Final Dry Density	1.39		
Volume change factor=F	0.097	Final void ratio	0.86		
		Vvf=Vtf-Vs	17.25		
		Final saturation Sf, %	84.07		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	190	Wt.of ring +wet sample(gm)	136.4
Wt.of can +Dry sample(gm)	159.6	Wt.of ring +Dry sample(gm)	121.9
Wt.can(gm)	44	Wt.ring(gm)	70
Wt.of dry sample	115.6	Wt.of dry sample	51.9
Wt.of wet sample	146	Wt.of wet sample	66.4
Wt.of water(gm)	30.4	Wt.of water(gm)	14.5
Moisture Content(%)	26.30	Moisture Content(%)	27.94
Bulk Density	1.666		
Dry density	1.319		

Elapsed Time h	Time, Min.	Stress, Kpa	Dial Reading, mm	ve Compres	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	12.0	0.024	0.002	0.948
1		12	33.0	0.066	0.006	0.942
2		25	52.0	0.104	0.010	0.938
3		50	86.0	0.172	0.017	0.931
4		100	142.0	0.284	0.028	0.920
5		200	251.5	0.503	0.049	0.899
	0.1	200	254.0	0.508	0.049	0.899
	0.25	200	256.0	0.512	0.050	0.898
	0.5	200	257.0	0.514	0.050	0.898
	1	200	257.5	0.515	0.050	0.898
	2	200	258.0	0.516	0.050	0.898
	4	200	258.5	0.517	0.050	0.898
	8	200	261.0	0.522	0.051	0.897
	15	200	264.0	0.528	0.051	0.897
	30	200	270.0	0.540	0.053	0.896
1		200	283.0	0.566	0.055	0.893
2		200	311.0	0.622	0.061	0.888
4		200	397.0	0.794	0.077	0.871
8		200	450.0	0.900	0.088	0.860
24		200	475.0	0.950	0.093	0.856
32		400	710.0	1.420	0.138	0.810
38		800	963.0	1.926	0.188	0.760



Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.950
d_i, dial reading at 200KPa before wetting, mm	0.503
collapse Index $I_e=((d_f - d_i)/h_o)*100$	2.235

2.Pumice

2.1 Chemical composition of Pumice

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

Issue Date:- 17/03/2022

Customer Name:- Yehamleshet Bekele

Request No:- GLD/RQ/742/22

Report No:- GLD/RN/271/22

Sample type :- Pumice

Sample Preparation: - 200 Mesh

Date Submitted:-01/03/2022

Number of Sample:-One (01)

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	Weight Of Sample
YEHAM-001	70.74	17.81	<0.01	<0.01	<0.01	3.96	2.92	0.12	0.01	<0.01	0.78	3.60	800.00gm

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

Analysts	Checked By	Approved By	Quality Control
Elisa Fiseha Lidet Endeshaw Nigist Fikadu Yirgalem Abraham	Tizita Zemene	Yohannes Getachew	Gosa Haile

2.2 Specific Gravity (Pumice)

SPECIFIC GRAVITY

TEST METHOD: ASTM D854-98

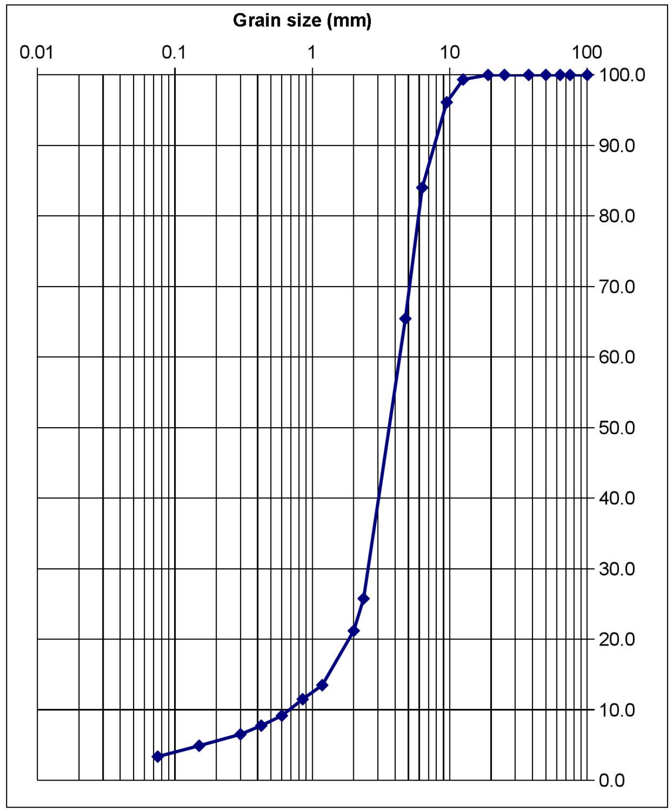
Trial	1	2
Pyknometer #	4	3
A. Mass of Pycnometer (empty, clean)	63.41	65.68
B. Mass of empty pycno + Dry soil	73.41	75.63
C. Mass of Dry Soil (No. 10 sieve)	10	9.95
D. Mass of Pycnometer full of water	165.96	174.9
E. Mass of pycno + Soil +full of water (boiled)	171	179.95
F. Volume of Sample=C+D-E	4.96	4.9
G. Specific Gravity=C/F	2.016	2.031
Average Specific Gravity ((1+2+3)/3)	2.023	

2.3 Gradation (Pumice)

Dry Sieve Analysis

Weight of Total Sample Before Washing (gm) = 708.2

Sieve Size(mm)	Wt. Ret (gm)	% Ret.	% pass
100	0	0.00	100.0
75	0	0.00	100.0
63.5	0	0.00	100.0
50	0	0.00	100.0
37.5	0	0.00	100.0
25	0	0.00	100.0
19	0	0.00	100.0
12.5	4.6	0.65	99.4
9.5	22.8	3.22	96.1
6.3	85.7	12.10	84.0
4.75	131.6	18.58	65.4
2.36	281	39.68	25.8
2.0	32.3	4.56	21.2
1.18	54.6	7.71	13.5
0.85	14.1	1.99	11.5
0.60	16.5	2.33	9.2
0.425	10	1.41	7.8
0.30	8.7	1.23	6.5
0.15	11.5	1.62	4.9
0.075	10.9	1.54	3.4
Sub Total	684.3		
Pan	23.9	3.4	
Total	708.2	100.00	

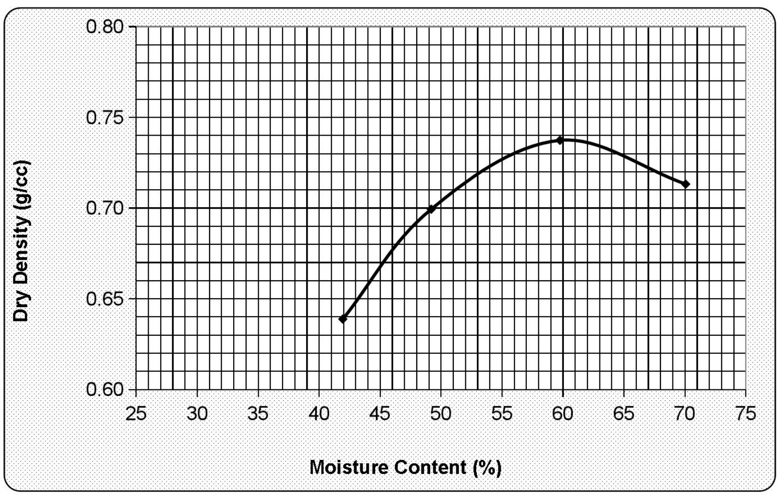


2.4 Compaction (Pumice)

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4
Mold weight + sample (gm)	5253	5382	5509	5542
Mold weight (gm)	4397	4397	4397	4397
Sample weight (gm)	856	985	1112	1145
Mold volume (Cm3)	944	944	944	944
Bulk density(g/cm3)	0.91	1.04	1.18	1.21
Moisture Test Results				
Wet weight + can	66.00	89.00	73.00	86.00
Dry weight + can	53.00	67.90	54.30	58.40
Weight of can	22.00	25.00	23.00	19.00
Weight of moisture	13.00	21.10	18.70	27.60
Weight of dry soil	31.00	42.90	31.30	39.40
Moisture content (%)	41.94	49.18	59.74	70.05
Dry density (g/cm3)	0.64	0.70	0.74	0.71



MDD (g/cc) 0.74

OMC (%) 59.7

APPENDIX B: SUMMARY OF TEST RESULTS WITH VARIFING PERCENTAGE OF PUMICE

3. Pumice Blended Sample

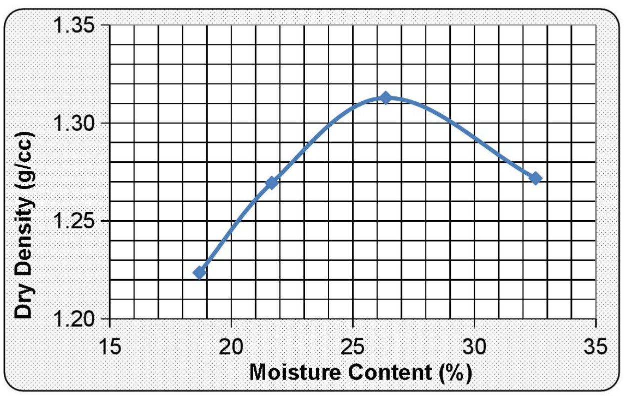
3.1 Moisture Density Relationship (Procter Test)

5% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4
Mold weight + sample (gm)	5775	5862	5970	5995
Mold weight (gm)	4404	4404	4404	4404
Sample weight (gm)	1371	1458	1566	1591
Mold volume (Cm3)	944	944	944	944
Bulk density(g/cm3)	1.45	1.54	1.66	1.69
	Moisture Test Results			
Wet weight +can	148.00	165.00	185.00	185.00
Dry weight + can	128.00	139.00	151.00	145.00
Weight of can	21.00	19.00	22.00	22.00
Weight of moisture	20.00	26.00	34.00	40.00
Weight of dry soil	107.00	120.00	129.00	123.00
Moisture content (%)	18.69	21.67	26.36	32.52
Dry density (g/cm3)	1.22	1.27	1.31	1.27



MDD (g/cc) **1.31**

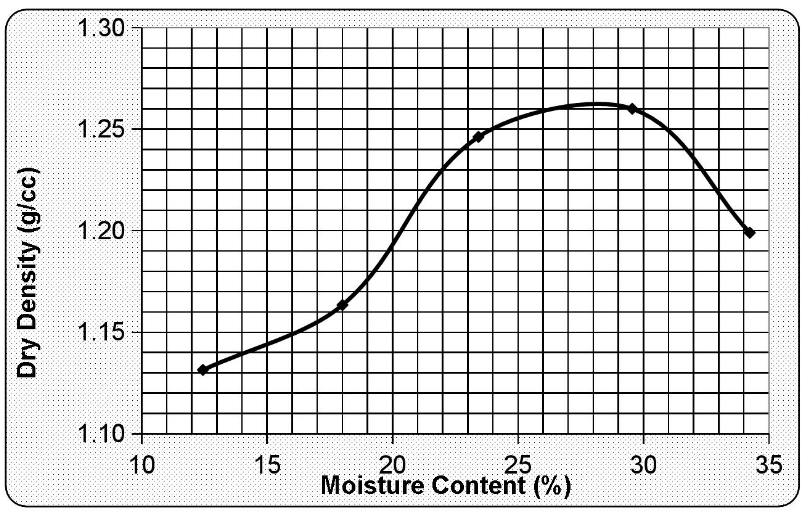
OMC (%) **26.4**

10% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4	5
Mold weight + sample (gm)	5618	5713	5869	5958	5938
Mold weight (gm)	4417	4417	4417	4417	4417
Sample weight (gm)	1201	1296	1452	1541	1521
Mold volume (Cm3)	944	944	944	944	945
Bulk density(g/cm3)	1.27	1.37	1.54	1.63	1.61
	Moisture Test Results				
Wet weight +can	191.00	155.00	171.00	255.00	182.70
Dry weight + can	172.30	134.10	145.00	207.10	141.97
Weight of can	22.00	18.00	34.00	45.00	23.00
Weight of moisture	18.70	20.90	26.00	47.90	40.73
Weight of dry soil	150.30	116.10	111.00	162.10	118.97
Moisture content (%)	12.44	18.00	23.42	29.55	34.24
Dry density (g/cm3)	1.13	1.16	1.25	1.26	1.20



MDD (g/cc) **1.26**

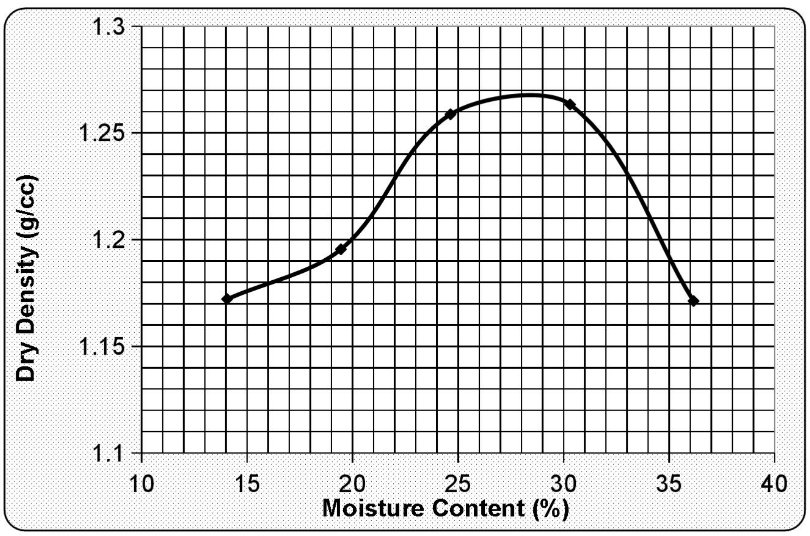
OMC (%) **28.0**

15% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4	5
Mold weight + sample (gm)	5669	5756	5860	5931	5925
Mold weight (gm)	4417	4417	4417	4417	4417
Sample weight (gm)	1252	1339	1443	1514	1508
Mold volume (Cm3)	944	944	944	944	945
Bulk density(g/cm3)	1.33	1.42	1.53	1.60	1.60
	Moisture Test Results				
Wet weight +can	161.00	151.00	195.00	196.50	201.50
Dry weight + can	144.00	130.00	160.00	157.40	156.67
Weight of can	23.00	22.00	22.00	22.00	20.50
Weight of moisture	17.00	21.00	35.00	39.10	44.83
Weight of dry soil	121.00	108.00	138.00	135.40	136.17
Moisture content (%)	14.05	19.44	25.36	28.88	32.92
Dry density (g/cm3)	1.16	1.19	1.22	1.24	1.20



MDD (g/cc) 1.24

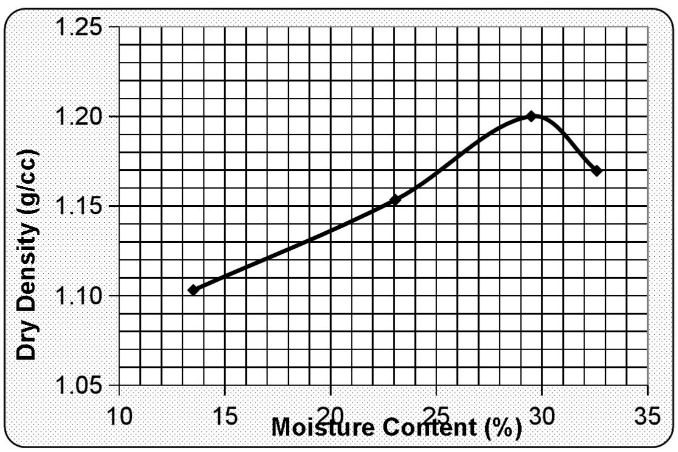
OMC (%) 28.88

20% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4
Mold weight + sample (gm)	5575	5733	5860	5857
Mold weight (gm)	4393	4393	4393	4393
Sample weight (gm)	1182	1340	1467	1464
Mold volume (Cm3)	944	944	944	944
Bulk density(g/cm3)	1.25	1.42	1.55	1.55
	Moisture Test Results			
Wet weight +can	131.00	143.60	151.90	152.10
Dry weight + can	117.91	121.00	122.50	120.10
Weight of can	21.00	23.00	22.80	21.90
Weight of moisture	13.09	22.60	29.40	32.00
Weight of dry soil	96.91	98.00	99.70	98.20
Moisture content (%)	13.51	23.06	29.49	32.59
Dry density (g/cm3)	1.10	1.15	1.20	1.17



MDD (g/cc) 1.20

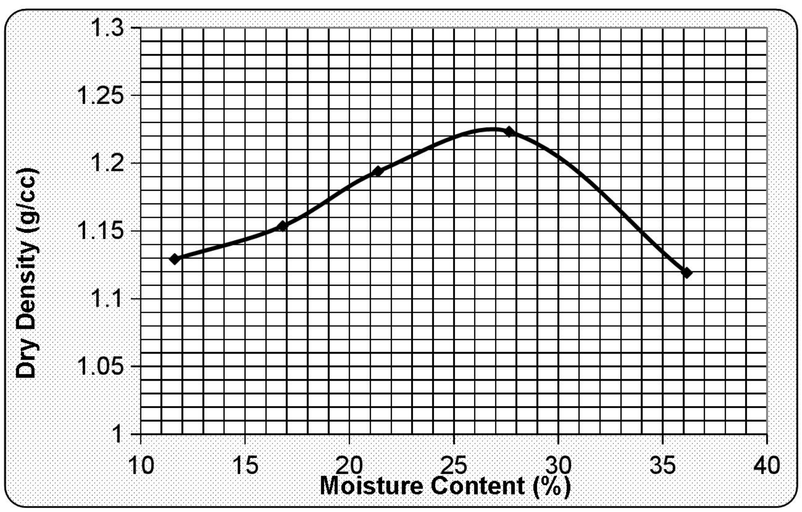
OMC (%) 29.5

25% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4	5
Mold weight + sample (gm)	5590	5640	5760	5884	5880
Mold weight (gm)	4417	4417	4417	4417	4417
Sample weight (gm)	1173	1223	1343	1467	1463
Mold volume (Cm3)	944	944	944	944	945
Bulk density(g/cm3)	1.24	1.30	1.42	1.55	1.55
	Moisture Test Results				
Wet weight +can	168.00	169.00	167.00	176.90	199.00
Dry weight + can	150.00	148.00	139.00	140.10	153.50
Weight of can	23.00	23.00	22.00	18.00	20.00
Weight of moisture	18.00	21.00	28.00	36.80	45.50
Weight of dry soil	127.00	125.00	117.00	122.10	133.50
Moisture content (%)	14.17	16.80	23.93	30.14	34.08
Dry density (g/cm3)	1.09	1.11	1.15	1.19	1.15



MDD (g/cc) 1.19

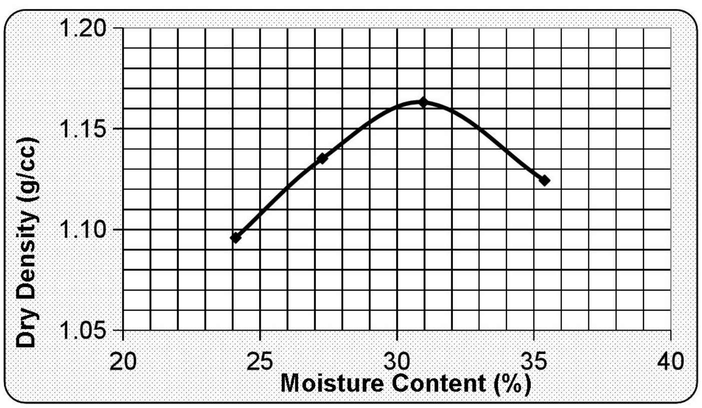
OMC (%) 30.14

30% pumice

Moisture Content and Density Relationship

AASHTO T-99, Method C

Trial No	1	2	3	4
Mold weight + sample (gm)	5686	5766	5840	5839
Mold weight (gm)	4402	4402	4402	4402
Sample weight (gm)	1284	1364	1438	1437
Mold volume (Cm3)	944	944	944	944
Bulk density(g/cm3)	1.36	1.44	1.52	1.52
	Moisture Test Results			
Wet weight +can	161.00	193.00	241.70	162.37
Dry weight + can	134.00	157.00	190.00	125.68
Weight of can	22.00	25.00	23.00	22.00
Weight of moisture	27.00	36.00	51.70	36.69
Weight of dry soil	112.00	132.00	167.00	103.68
Moisture content (%)	24.11	27.27	30.96	35.39
Dry density (g/cm3)	1.10	1.14	1.16	1.12



MDD (g/cc) 1.16

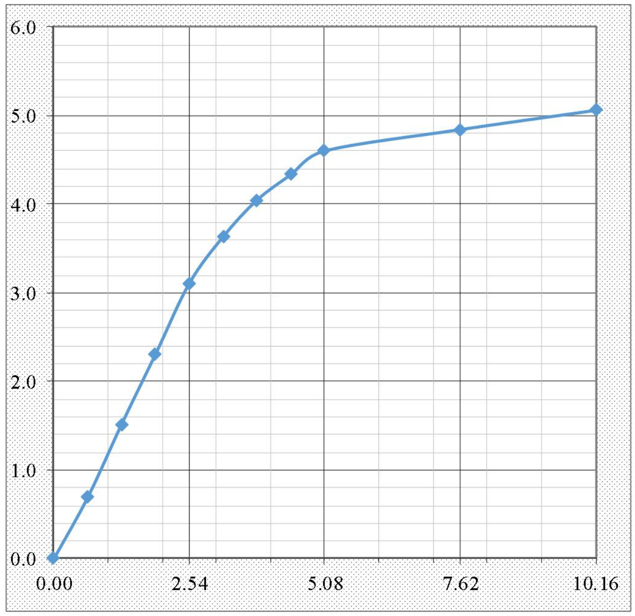
OMC (%) 31.0

3.2 CBR Test

10% Pumice

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	29.0	0.70
1.27	63.0	1.52
1.91	96.0	2.31
2.54	129.0	3.11
3.18	151.0	3.64
3.81	168.0	4.05
4.45	180.0	4.34
5.08	191.0	4.60
7.62	201.0	4.84
10.16	210.0	5.06

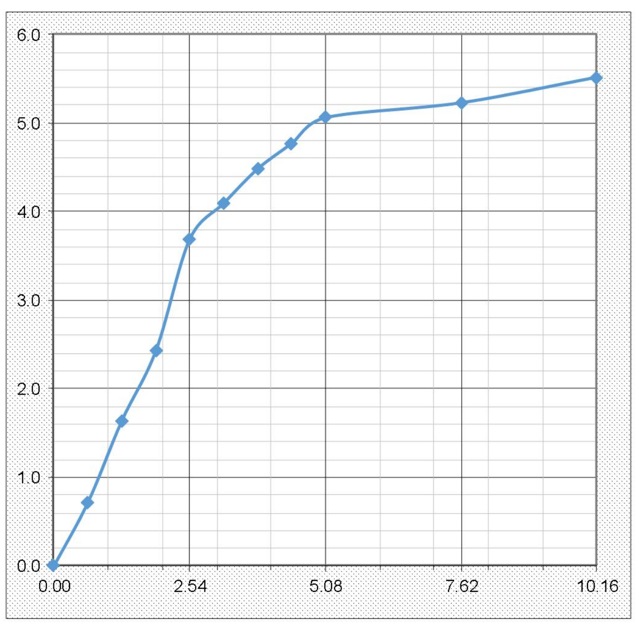
CBR %	
2.54mm	5.08mm
23.5	23.1



15% pumice

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	30.0	0.72
1.27	68.0	1.64
1.91	101.0	2.43
2.54	153.0	3.69
3.18	170.0	4.10
3.81	186.0	4.48
4.45	198.0	4.77
5.08	210.0	5.06
7.62	217.0	5.23
10.16	229.0	5.52

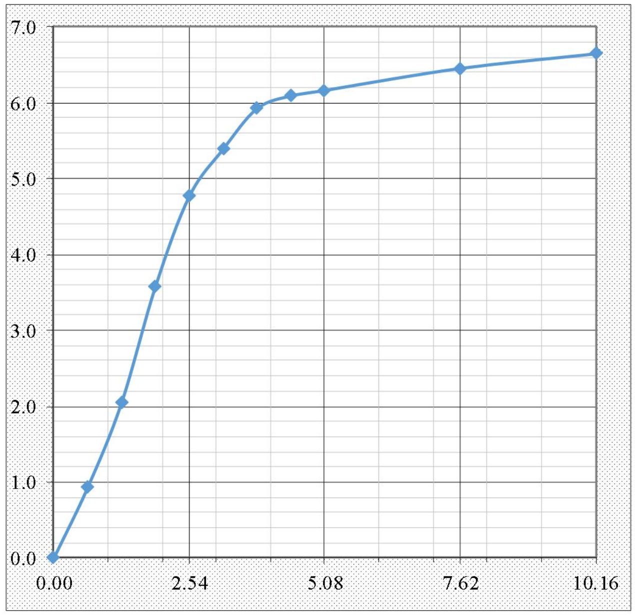
CBR %	
2.54mm	5.08mm
27.8	25.3



20 % pumice

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	39.0	0.94
1.27	85.0	2.05
1.91	149.0	3.59
2.54	198.0	4.77
3.18	224.0	5.40
3.81	246.0	5.93
4.45	253.0	6.09
5.08	256.0	6.17
7.62	268.0	6.46
10.16	276.0	6.65

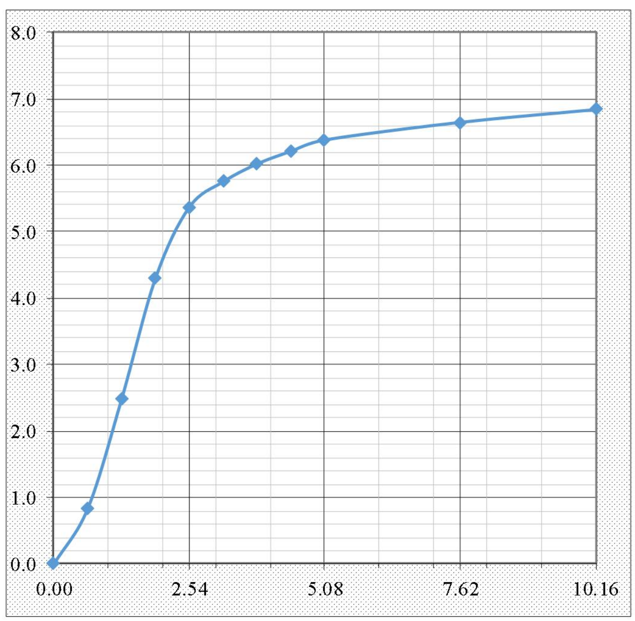
CBR %	
2.54mm	5.08mm
36.0	30.9



25%pumice

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	35.0	0.84
1.27	103.0	2.48
1.91	179.0	4.31
2.54	223.0	5.37
3.18	239.0	5.76
3.81	250.0	6.02
4.45	258.0	6.22
5.08	265.0	6.38
7.62	276.0	6.65
10.16	284.0	6.84

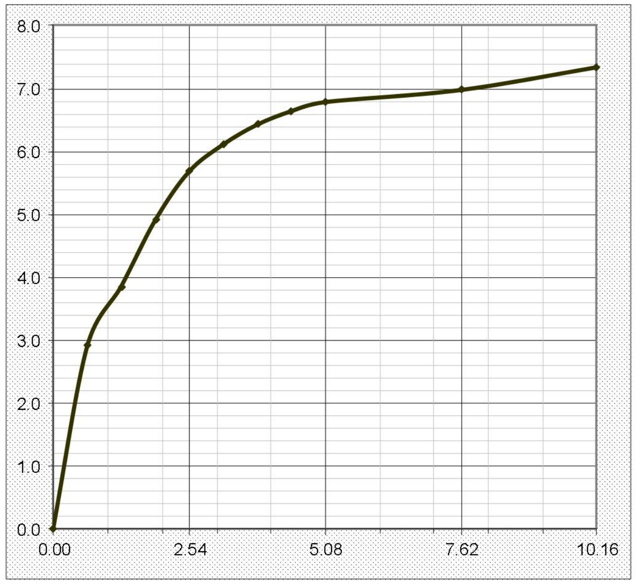
CBR %	
2.54mm	5.08mm
40.6	32.0



30% pumice

PENETRATION DATA		
Ring Factor (kN/DIV) =		0.02409
Pen.(mm)	Dial Reading	Load (kN)
0.0	0.0	0.00
0.64	75.0	1.81
1.27	140.0	3.37
1.91	204.0	4.91
2.54	236.0	5.69
3.18	254.0	6.12
3.81	267.0	6.43
4.45	276.0	6.65
5.08	282.0	6.79
7.62	298.0	7.18
10.16	305.0	7.35

CBR %	
2.54mm	5.08mm
42.9	34.0



3.3 collapsible potential test

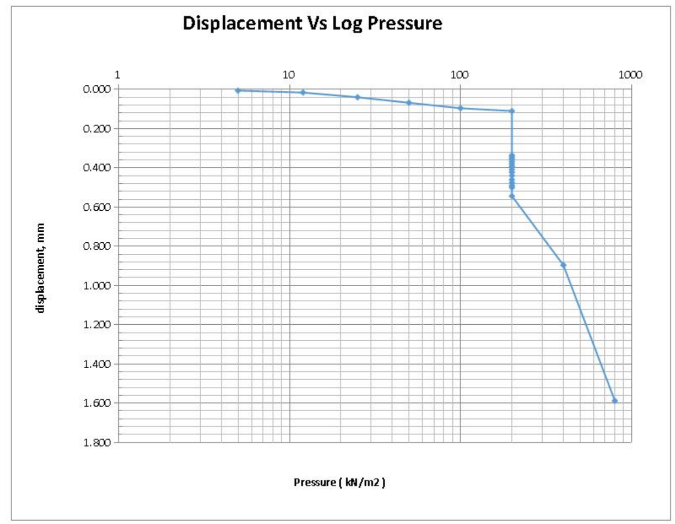
10% pumice

ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.544	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	19.903	Volume Change	1.07	Thickness of sample	20
Vvi=Vt-Vs	19.367	Final volume	38.20	Area of sample	1963
intial Void ratio e0=Vv/Vs	0.973	Final bulk density	1.718	Volume of Sample	39.27
intial So= Vw/Vvi	66.556	Final Dry Density	1.32		
Volume change factor=F	0.099	Final void ratio	0.92		
		Vvf=Vtf-Vs	18.30		
		Final saturation Sf, %	82.14		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	132.4	Wt.of ring +wet sample(gm)	135.3
Wt.of can +Dry sample(gm)	110.4	Wt.of ring +Dry sample(gm)	120.3
Wt.can(gm)	23.1	Wt.ring(gm)	69.66
Wt.of dry sample	87.3	Wt.of dry sample	50.61
Wt.of wet sample	109.3	Wt.of wet sample	65.64
Wt.of water(gm)	22	Wt.of water(gm)	15.03
Moisture Content(%)	25.20	Moisture Content(%)	29.70
Bulk Density	1.631		
Dry density	1.303		

Elapsed Time h	Time, Min.	Applied Stress, Kpa	Dial Reading, mm	Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	3.0	0.006	0.001	0.973
1		12	8.0	0.016	0.002	0.972
2		25	20.0	0.040	0.004	0.969
3		50	34.0	0.068	0.007	0.966
4		100	48.0	0.096	0.009	0.964
5		200	55.0	0.110	0.011	0.962
	0.1	200	168.0	0.336	0.033	0.940
	0.25	200	173.0	0.346	0.034	0.939
	0.5	200	178.0	0.356	0.035	0.938
	1	200	184.0	0.368	0.036	0.937
	2	200	190.0	0.380	0.037	0.936
	4	200	196.0	0.392	0.039	0.934
	8	200	204.0	0.408	0.040	0.933
	15	200	211.0	0.422	0.042	0.931
	30	200	220.0	0.440	0.043	0.930
1		200	230.0	0.460	0.045	0.928
2		200	238.0	0.476	0.047	0.926
4		200	245.0	0.490	0.048	0.925
8		200	250.0	0.500	0.049	0.924
24		200	272.0	0.544	0.054	0.919
32		400	448.0	0.896	0.088	0.885
38		800	794.0	1.588	0.157	0.816



Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.544
d_i, dial reading at 200KPa before wetting, mm	0.110
collapse Index $I_e=((d_f - d_i)/h_o)*100$	2.17

15% pumice

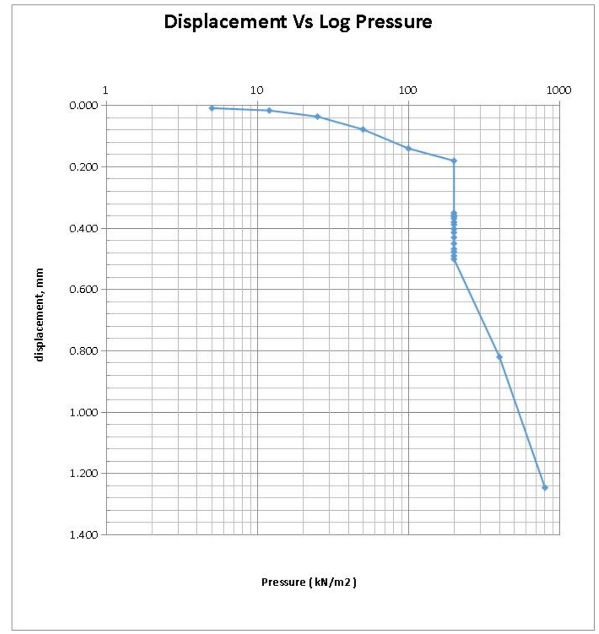
ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.500	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	19.345	Volume Change	0.98	Thickness of sample	20
Vvi=Vt-Vs	19.925	Final volume	38.29	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.030	Final bulk density	1.677	Volume of Sample	39.27
intial So= Vw/Vvi	63.259	Final Dry Density	1.29		
Volume change factor=F	0.102	Final void ratio	0.98		
		Vvf=Vtf-Vs	18.94		
		Final saturation Sf, %	78.66		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	156	Wt.of ring +wet sample(gm)	137.6
Wt.of can +Dry sample(gm)	129.1	Wt.of ring +Dry sample(gm)	122.7
Wt.can(gm)	23	Wt.ring(gm)	73.4
Wt.of dry sample	106.1	Wt.of dry sample	49.3
Wt.of wet sample	133	Wt.of wet sample	64.2
Wt.of water(gm)	26.9	Wt.of water(gm)	14.9
Moisture Content(%)	25.35	Moisture Content(%)	30.22
Bulk Density		1.587	
Dry density		1.266	

Elapsed Time h	Time, Min.	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	4.2	0.008	0.001	1.030
1		12	8.0	0.016	0.002	1.028
2		25	18.0	0.036	0.004	1.026
3		50	38.9	0.078	0.008	1.022
4		100	70.0	0.140	0.014	1.016
5		200	90.0	0.180	0.018	1.012
	0.1	200	175.0	0.350	0.036	0.995
	0.25	200	179.0	0.358	0.036	0.994
	0.5	200	181.0	0.362	0.037	0.993
	1	200	184.0	0.368	0.037	0.993
	2	200	190.0	0.380	0.039	0.991
	4	200	194.0	0.388	0.039	0.991
	8	200	201.0	0.402	0.041	0.989
	15	200	207.0	0.414	0.042	0.988
	30	200	215.0	0.430	0.044	0.986
1		200	225.0	0.450	0.046	0.984
2		200	233.9	0.468	0.047	0.983
4		200	238.5	0.477	0.048	0.982
8		200	245.0	0.490	0.050	0.980
24		200	250.2	0.500	0.051	0.979
32		400	410.0	0.820	0.083	0.947
38		800	623.0	1.246	0.126	0.904

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.500
d_i, dial reading at 200KPa before wetting, mm	0.180
collapse Index $I_e=((d_f - d_i)/h_o)*100$	1.602



20% pumice

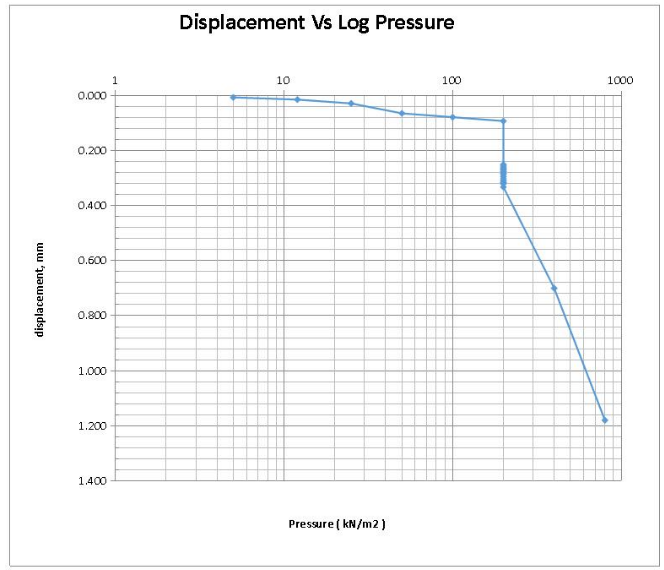
ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.332	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	18.4763	Volume Change	0.65	Thickness of sample	20
Vvi=Vt-Vs	20.7937	Final volume	38.62	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.1254	Final bulk density	1.580	Volume of Sample	39.27
intial So= Vw/Vvi	54.9015	Final Dry Density	1.22		
Volume change factor=F	0.1063	Final void ratio	1.09		
		Vvf=Vtf-Vs	20.14		
		Final saturation Sf, %	68.17		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	142.47	Wt.of ring +wet sample(gm)	133.1
Wt.of can +Dry sample(gm)	123.4	Wt.of ring +Dry sample(gm)	119.4
Wt.can(gm)	44.08	Wt.ring(gm)	72.1
Wt.of dry sample	79.32	Wt.of dry sample	47.27
Wt.of wet sample	98.39	Wt.of wet sample	61
Wt.of water(gm)	19.07	Wt.of water(gm)	13.73
Moisture Content(%)	24.04	Moisture Content(%)	29.05
Bulk Density	1.500		
Dry density	1.209		

Elapsed Time h	Time, Min.	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	3.0	0.006	0.001	1.125
1		12	7.0	0.014	0.001	1.124
2		25	14.0	0.028	0.003	1.122
3		50	32.0	0.064	0.007	1.119
4		100	39.0	0.078	0.008	1.117
5		200	46.0	0.092	0.010	1.116
	0.1	200	125.0	0.250	0.027	1.099
	0.25	200	128.0	0.256	0.027	1.098
	0.5	200	131.0	0.262	0.028	1.098
	1	200	133.0	0.266	0.028	1.097
	2	200	135.0	0.270	0.029	1.097
	4	200	138.0	0.276	0.029	1.096
	8	200	141.0	0.282	0.030	1.095
	15	200	143.0	0.286	0.030	1.095
	30	200	147.0	0.294	0.031	1.094
1		200	151.0	0.302	0.032	1.093
2		200	155.0	0.310	0.033	1.092
4		200	158.0	0.316	0.034	1.092
8		200	160.0	0.320	0.034	1.091
24		200	166.0	0.332	0.035	1.090
32		400	350.0	0.700	0.074	1.051
38		800	590.0	1.180	0.125	1.000

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.332
d_i, dial reading at 200KPa before wetting, mm	0.092
collapse Index $I_e=((d_f - d_i)/h_o)*100$	1.2



25% pumice

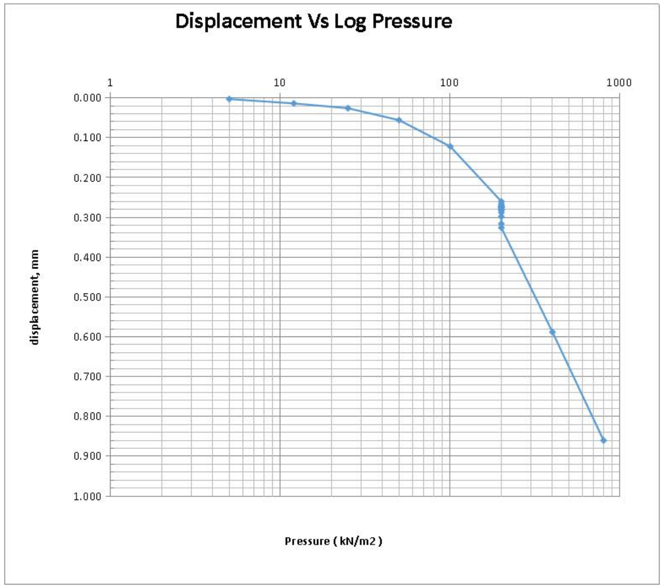
ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.326	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	19.082	Volume Change	0.64	Thickness of sample	20
Vvi=Vt-Vs	20.188	Final volume	38.63	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.058	Final bulk density	1.641	Volume of Sample	39.27
intial So= Vw/Vvi	62.463	Final Dry Density	1.25		
Volume change factor=F	0.103	Final void ratio	1.02		
		Vvf=Vtf-Vs	19.55		
		Final saturation Sf, %	76.73		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	120.2	Wt.of ring +wet sample(gm)	135.6
Wt.of can +Dry sample(gm)	99.5	Wt.of ring +Dry sample(gm)	120.6
Wt.can(gm)	19	Wt.ring(gm)	72.2
Wt.of dry sample	80.5	Wt.of dry sample	48.4
Wt.of wet sample	101.2	Wt.of wet sample	63.4
Wt.of water(gm)	20.7	Wt.of water(gm)	15
Moisture Content(%)	25.71	Moisture Content(%)	30.99
Bulk Density	1.570		
Dry density	1.249		

Elapsed Time h	Time, Min.	Applied Stress, Kpa	Dial Reading, mm	Cumulative Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	1.5	0.003	0.000	1.058
1		12	7.0	0.014	0.001	1.057
2		25	13.0	0.026	0.003	1.055
3		50	28.0	0.056	0.006	1.052
4		100	61.0	0.122	0.013	1.045
5		200	130.0	0.260	0.027	1.031
	0.1	200	133.0	0.266	0.027	1.031
	0.25	200	135.0	0.270	0.028	1.030
	0.5	200	136.0	0.272	0.028	1.030
	1	200	136.5	0.273	0.028	1.030
	2	200	137.0	0.274	0.028	1.030
	4	200	137.5	0.275	0.028	1.030
	8	200	138.0	0.276	0.028	1.030
	15	200	139.0	0.278	0.029	1.029
	30	200	141.0	0.282	0.029	1.029
1		200	141.0	0.282	0.029	1.029
2		200	144.0	0.288	0.030	1.028
4		200	149.0	0.298	0.031	1.027
8		200	158.0	0.316	0.033	1.025
24		200	162.9	0.326	0.034	1.024
32		400	294.0	0.588	0.061	0.997
38		800	430.0	0.860	0.088	0.970

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.326
d_i, dial reading at 200KPa before wetting, mm	0.260
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.329



30% pumice

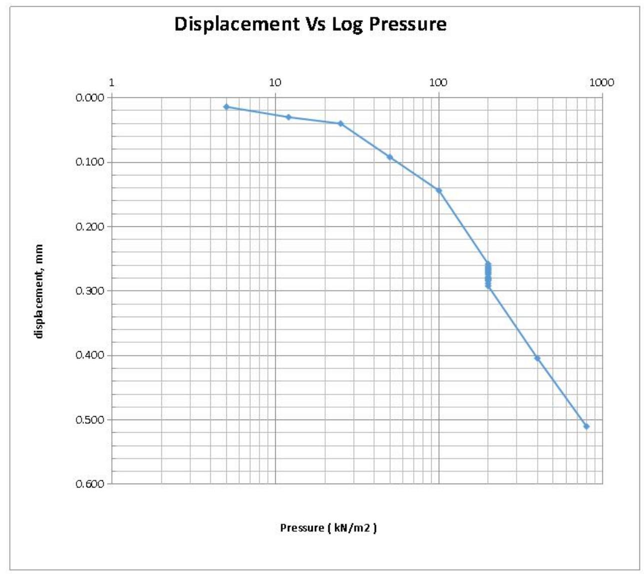
ASTM D 5333 Measurement of Collapse Potential of Soil

Specific Gravity	2.57	Overall settlement	0.292	Diameter of sample	50
Vs=DD/Gs*Vt(cm3)	18.431	Volume Change	0.57	Thickness of sample	20
Vvi=Vt-Vs	20.839	Final volume	38.70	Area of sample	1963
intial Void ratio e0=Vv/Vs	1.131	Final bulk density	1.627	Volume of Sample	39.27
intial So= Vw/Vvi	63.836	Final Dry Density	1.22		
Volume change factor=F	0.107	Final void ratio	1.10		
		Vvf=Vtf-Vs	20.27		
		Final saturation Sf, %	77.96		

Moisture Determination			
Before Test		After Test	
Wt.of can +wet sample(gm)	130	Wt.of ring +wet sample(gm)	132.6
Wt.of can +Dry sample(gm)	106.1	Wt.of ring +Dry sample(gm)	116.8
Wt.can(gm)	21	Wt.ring(gm)	69.65
Wt.of dry sample	85.1	Wt.of dry sample	47.15
Wt.of wet sample	109	Wt.of wet sample	62.95
Wt.of water(gm)	23.9	Wt.of water(gm)	15.8
Moisture Content(%)	28.08	Moisture Content(%)	33.51
Bulk Density		1.545	
Dry density		1.206	

Elapsed Time h	Time, Min.	Applied Stress, Kpa	Dial Reading, mm	Compression, mm	Void Ratio Δe=F*ΔH	e1=eo-Δe
	5	5	7.0	0.014	0.001	1.131
1		12	15.0	0.030	0.003	1.127
2		25	20.0	0.040	0.004	1.126
3		50	46.0	0.092	0.010	1.121
4		100	72.0	0.144	0.015	1.115
5		200	129.0	0.258	0.027	1.103
	0.1	200	131.0	0.262	0.028	1.103
	0.25	200	132.0	0.264	0.028	1.103
	0.5	200	133.0	0.266	0.028	1.102
	1	200	134.0	0.268	0.029	1.102
	2	200	135.0	0.270	0.029	1.102
	4	200	136.0	0.272	0.029	1.102
	8	200	137.0	0.274	0.029	1.101
	15	200	139.0	0.278	0.030	1.101
	30	200	140.0	0.280	0.030	1.101
1		200	140.5	0.281	0.030	1.101
2		200	141.0	0.282	0.030	1.101
4		200	142.0	0.284	0.030	1.100
8		200	144.0	0.288	0.031	1.100
24		200	146.0	0.292	0.031	1.100
32		400	202.0	0.404	0.043	1.088
38		800	255.0	0.510	0.054	1.076

Collapse Index, Ie Computation	
d_o, *dial reading at seating stress, mm*	0.0
h_o, initial specimen height, mm	20
d_f, dial reading at 200KPa after wetting ,mm	0.292
d_i, dial reading at 200KPa before wetting, mm	0.258
collapse Index $I_e=((d_f - d_i)/h_o)*100$	0.17



3.4 Direct shear test

10% pumice

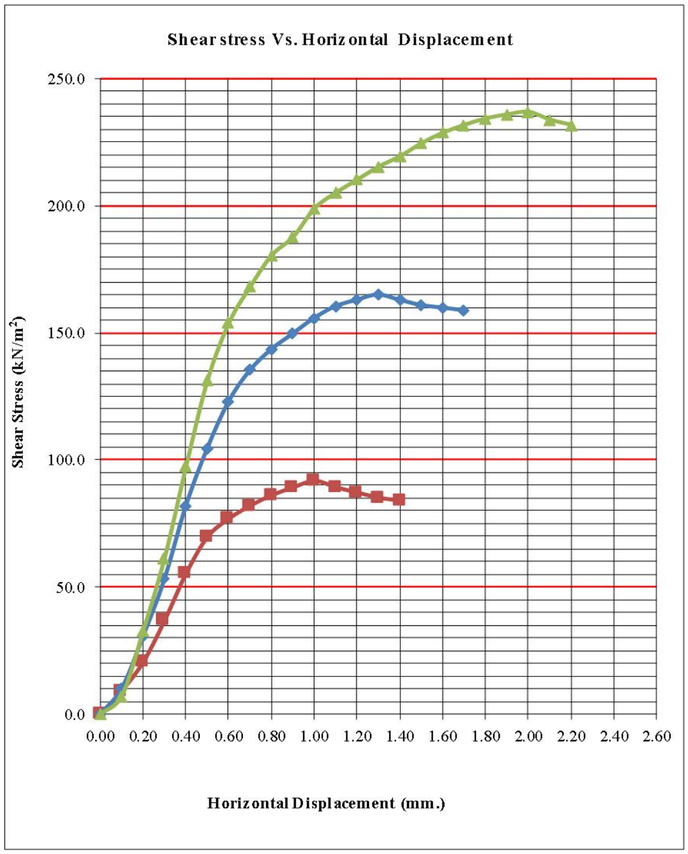
Direct Shear Test, ASTM-D 3080

SHEAR BOX DATA

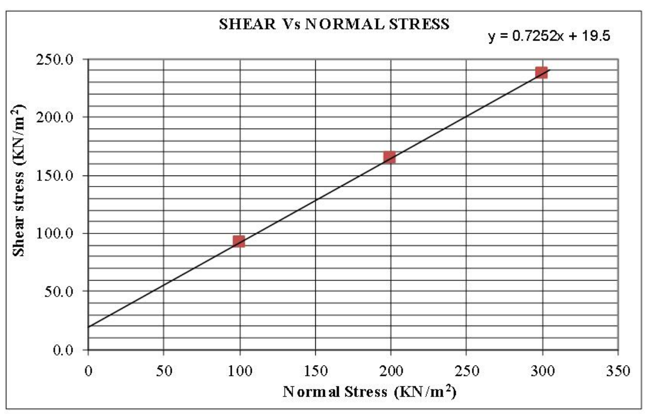
Length=	60	mm
Width=	60	mm
Depth=	20	mm
Area=	0.0036	m^2

Shear test Type :	CD	
Proving Ring Facto	0.0037	kN
Proving Ring Facto	0.01	mm
Laoding Rate=	2mm/min	

H.D(MM.)	Specimen 1 shear str. (kN/m^2)	Specimen 2 shear str. (kN/m^2)	Specimen 3 shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	9.23	10.25	7.18
0.20	20.50	30.75	32.80
0.30	36.90	53.30	61.50
0.40	55.35	82.00	97.38
0.50	69.70	104.55	131.20
0.60	76.88	123.00	153.75
0.70	82.00	135.30	168.10
0.80	86.10	143.50	180.40
0.90	89.18	149.65	187.58
1.00	91.74	155.80	198.85
1.10	89.18	160.41	205.00
1.20	87.13	162.98	210.13
1.30	85.08	165.03	215.25
1.40	84.05	162.98	219.35
1.50		160.93	224.48
1.60		159.90	228.58
1.70		158.88	231.65
1.80			234.21
1.90			235.75
2.00			236.78
2.10			233.70
2.20			231.65



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	91.74	165.03	236.78



Cohesion, C (kN/m^2) = 19.50 **Angle of internal friction, Ø° =35.95**

15% pumice

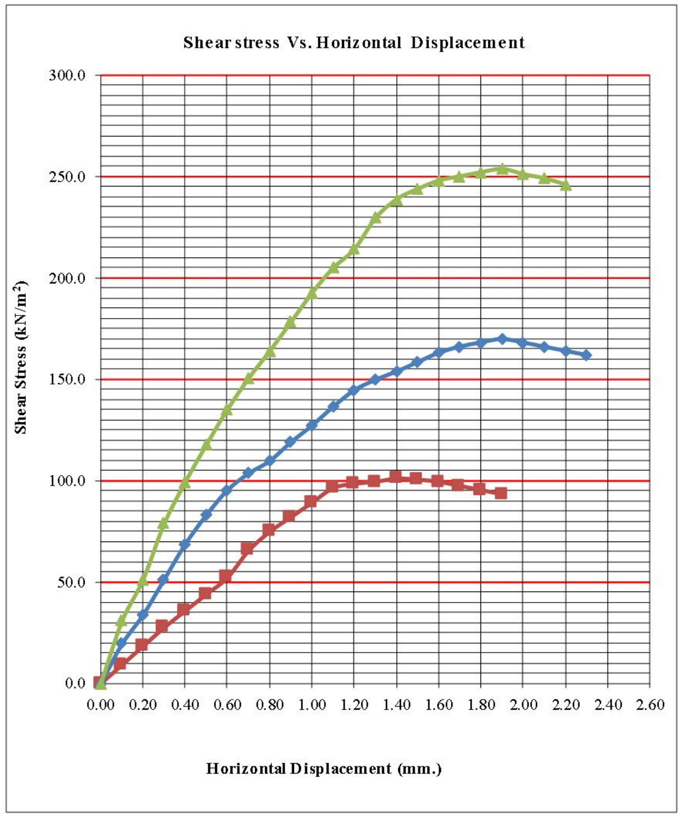
Direct Shear Test, ASTM-D 3080

SHEAR BOX DATA

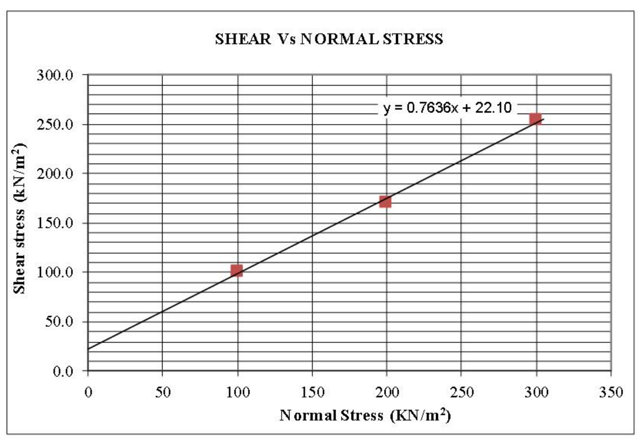
Length=	60	mm
Width=	60	mm
Depth=	20	mm
Area=	0.0036	m^2

Shear test Type :	CD	
Proving Ring Facto	0.0037	kN
Proving Ring Facto	0.01	mm
Laoding Rate=	2mm/min	

	Specimen		
	1	2	3
H.D(MM.)	shear str. (kN/m^2)	shear str. (kN/m^2)	shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	9.23	19.48	30.75
0.20	18.45	33.83	51.25
0.30	27.68	51.25	78.93
0.40	35.88	68.68	99.43
0.50	44.08	83.03	117.88
0.60	52.28	95.33	135.30
0.70	65.60	103.53	150.68
0.80	74.83	109.68	164.00
0.90	82.00	118.90	178.35
1.00	89.18	127.10	192.70
1.10	96.35	136.33	205.00
1.20	98.40	144.53	214.23
1.30	99.43	149.65	229.60
1.40	100.99	153.75	238.83
1.50	100.45	158.36	243.95
1.60	99.43	162.98	248.05
1.70	97.38	166.05	250.10
1.80	95.33	168.10	252.15
1.90	93.28	169.80	253.70
2.00		168.10	251.13
2.10		166.05	249.08
2.20		164.00	246.00
2.30		161.95	



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	100.99	169.80	253.70



Cohesion, C (kN/m^2) = 22.10   Angle of internal friction, Ø = 37.37°

20% pumice

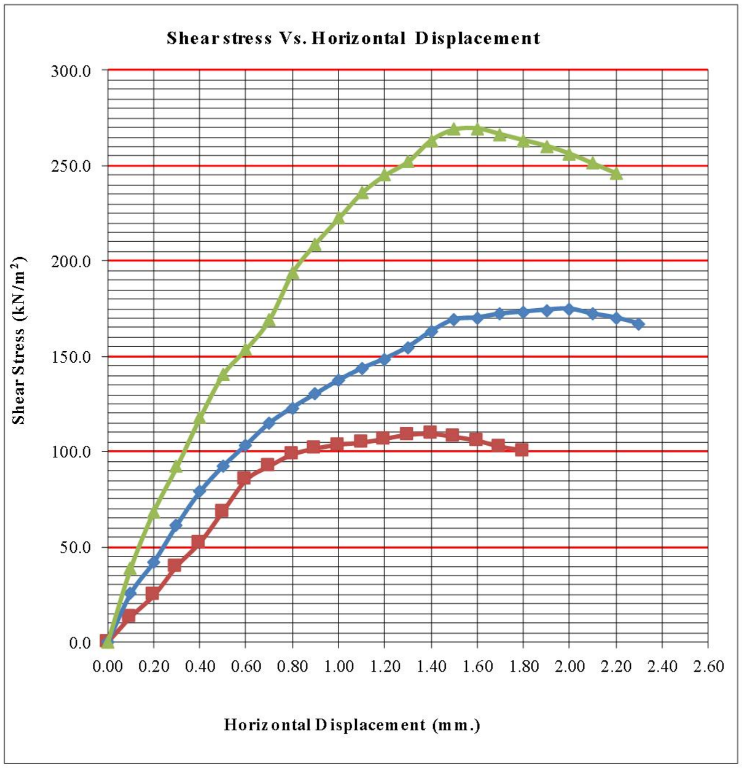
Direct Shear Test, ASTM-D 3080

SHEAR BOX DATA

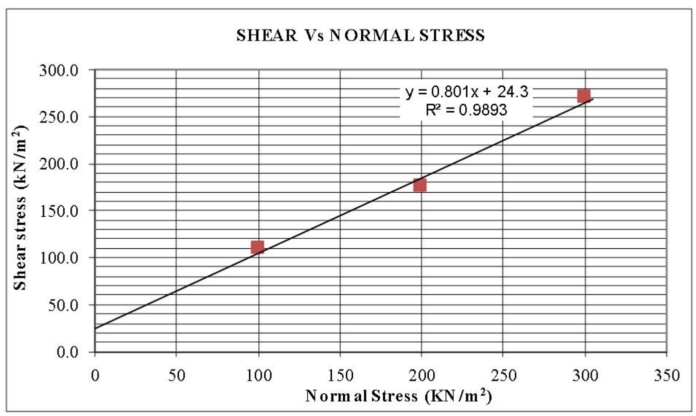
Length=	60	mm
Width=	60	mm
Depth=	20	mm
Area=	0.0036	m^2

Shear test Type :	CD	
Proving Ring Factor=	0.00369	kN
Proving Ring Factor=	0.01	mm
Laoding Rate=	2mm/min	

H.D (MM.)	Specimen 1 shear str. (kN/m^2)	Specimen 2 shear str. (kN/m^2)	Specimen 3 shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	13.33	25.63	38.95
0.20	24.60	42.03	68.68
0.30	39.98	61.50	92.25
0.40	52.28	78.93	117.88
0.50	68.68	92.25	140.43
0.60	85.08	103.53	153.75
0.70	92.25	114.80	169.13
0.80	98.40	123.00	193.73
0.90	101.48	130.18	209.10
1.00	103.53	137.35	222.43
1.10	104.55	143.50	235.75
1.20	106.60	148.63	244.98
1.30	108.65	154.78	252.15
1.40	109.16	162.98	263.43
1.50	107.63	169.13	269.06
1.60	105.58	170.15	269.58
1.70	102.50	172.20	266.50
1.80	100.45	173.23	263.43
1.90		174.25	260.35
2.00		174.76	256.25
2.10		172.20	251.13
2.20		170.15	246.00
2.30		167.08	



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	109.20	174.90	269.40



Cohesion, C (kN/m^2) = 24.30

Angle of internal friction, Ø° =38.70^0

25% Pumice

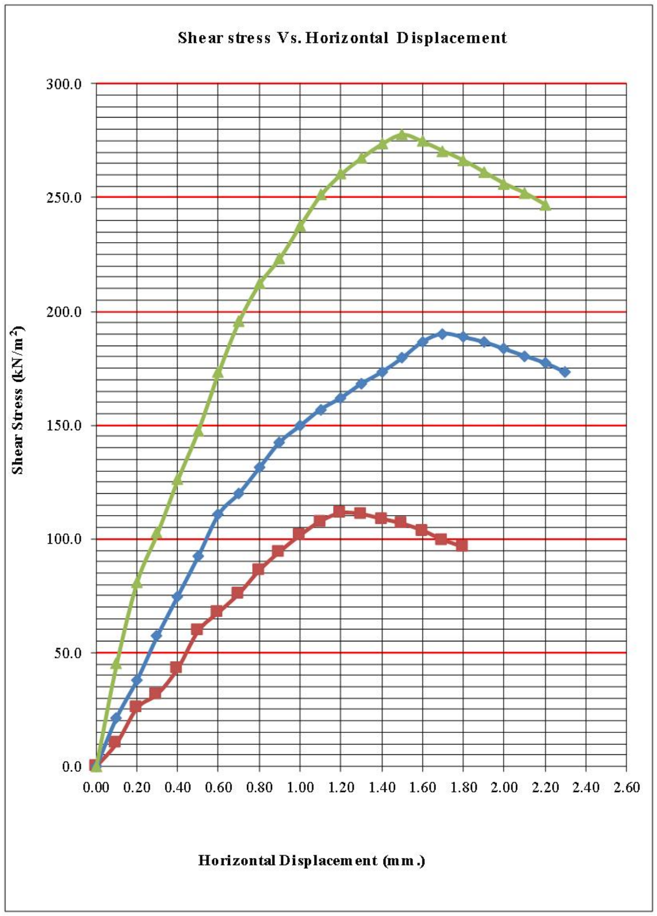
Direct Shear Test, ASTM-D 3080

SHEAR BOX DATA

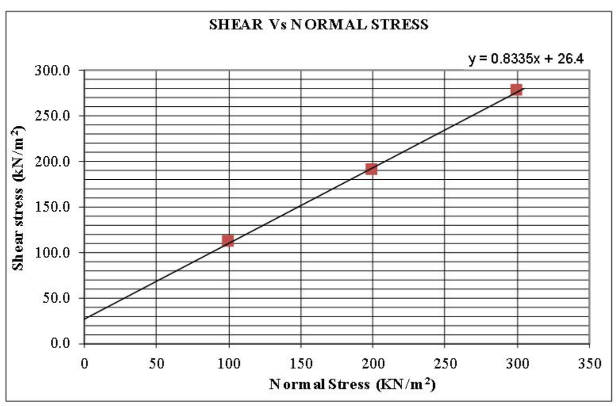
Length= 60 mm
Width= 60 mm
Depth= 20 mm
Area= 0.0036 m^2

Shear test Type : CD
Proving Ring Facto 0.0037 kN
Proving Ring Facto 0.01 mm
Laoding Rate= 2mm/min

	Specimen		
	1	2	3
H.D(MM.)	shear str. (kN/m^2)	shear str. (kN/m^2)	shear str. (kN/m^2)
0.00	0.00	0.00	0.00
0.10	10.25	21.53	45.10
0.20	25.63	37.93	80.98
0.30	31.78	57.40	102.50
0.40	43.05	74.83	126.08
0.50	59.45	92.25	147.60
0.60	67.65	110.70	173.23
0.70	75.85	119.93	195.78
0.80	86.10	131.20	212.18
0.90	94.30	142.48	223.45
1.00	101.48	149.65	237.80
1.10	107.63	156.83	251.13
1.20	111.21	161.95	260.35
1.30	110.70	168.10	267.53
1.40	108.65	173.23	273.68
1.50	106.60	179.38	277.78
1.60	103.53	186.55	274.70
1.70	99.43	190.14	270.60
1.80	96.35	188.60	266.50
1.90		186.55	261.38
2.00		183.48	256.25
2.10		180.40	252.15
2.20		177.33	247.03
2.30		173.23	



Speciemen	A	B	C
Normal Stress (kN/m^2)	100.00	200.00	300.00
Shear Stress (kN/m^2)	111.20	190.20	277.90



Cohesion, C (kN/m^2) = 26.39

Angle of internal friction, Ø= 39.81°

APPENDIX C: SUMMARY OF TEST CURVES WITH VARIFING PERCENTAGE OF PUMICE

4. Summary of test results

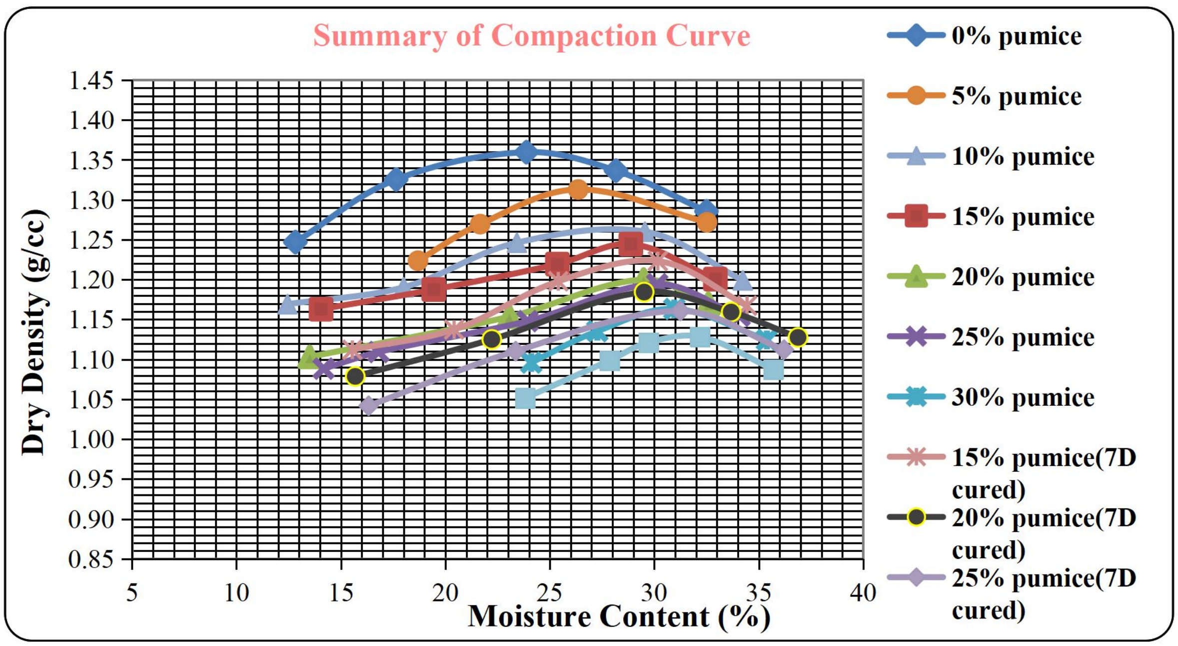
4.1 Compaction test

Uncured case

Pumice Content (%)	0%	5%	10%	15%	20%	25%	30%
MDD(g/cm^3)	1.36	1.31	1.26	1.24	1.2	1.19	1.16
OMC (%)	25	26.36	28.00	28.88	29.5	30.14	31

Cured case

Pumice Content (%)	15%	20%	25%	30%
MDD(g/cm^3)	1.22	1.18	1.16	1.13
OMC (%)	29.0	30.1	31.24	32.4



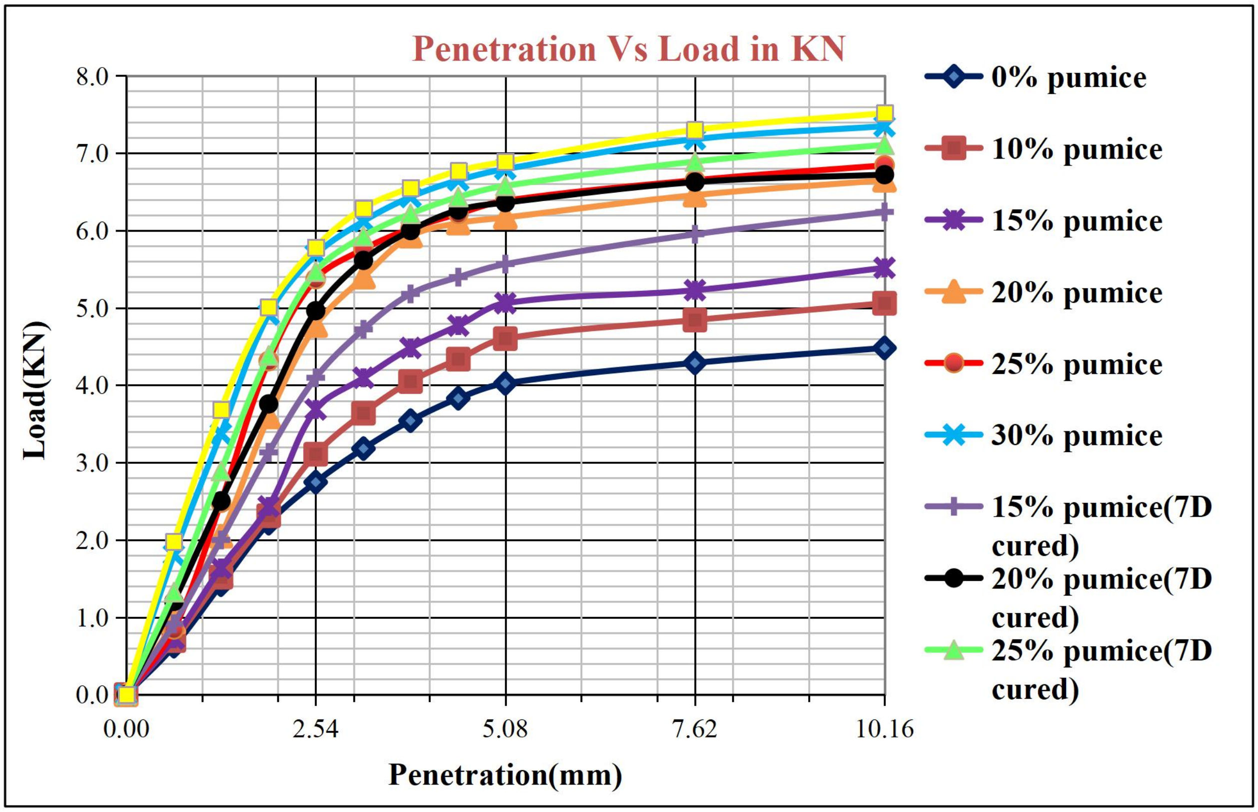
4.2 CBR test

Uncured case

Pumice Content (%)	**0**	**10**	**15**	**20**	**25**	**30**
CBR	20.74	23.47	27.8	36.03	40.6	42.94

Cured case

Pumice Content (%)	**15**	**20**	**25**	**30**
CBR	30.9	37.5	41.3	43.7



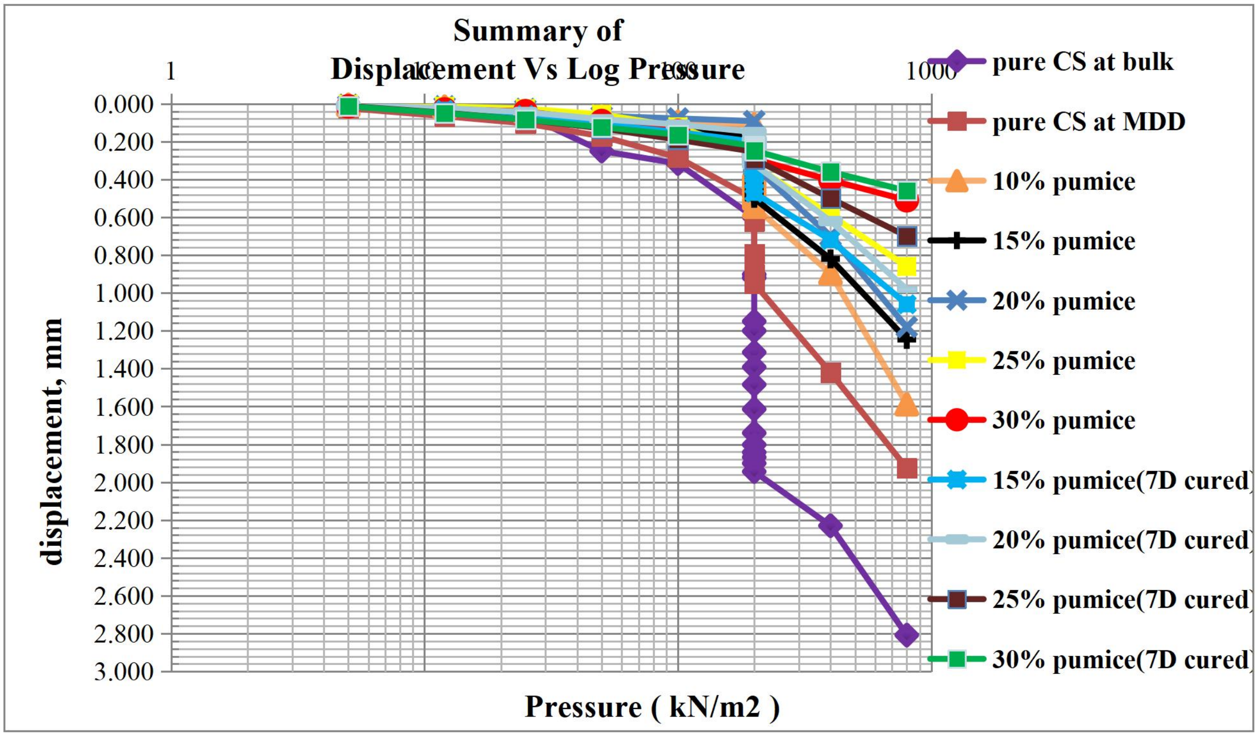
4.3 Collapsible Potential test

Uncured case

Pumice Content (%)	**0 at bulk**	**0 at MDD**	**10**	**15**	**20**	**25**	**30**
Collapsible Potential	6.72	2.235	2.17	1.6	1.2	0.33	0.17

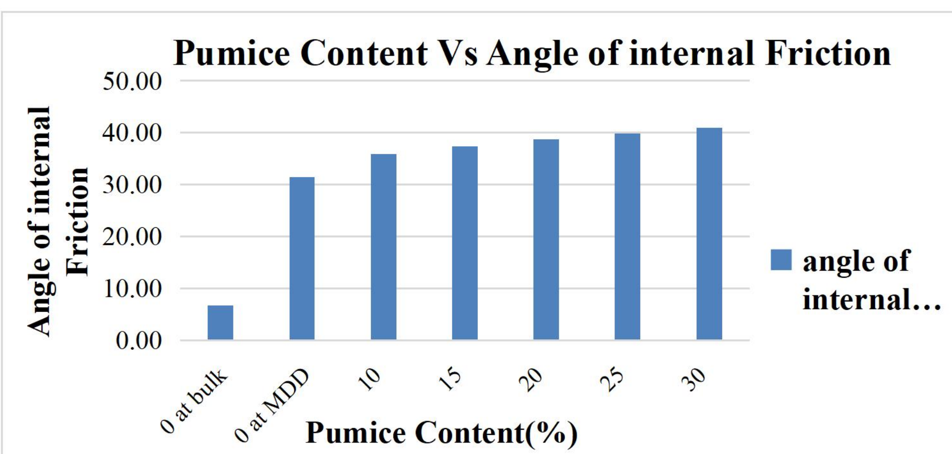
Cured case

Pumice Content (%)	**15**	**20**	**25**	**30**
Collapsible Index	1.35	0.85	0.19	0.12



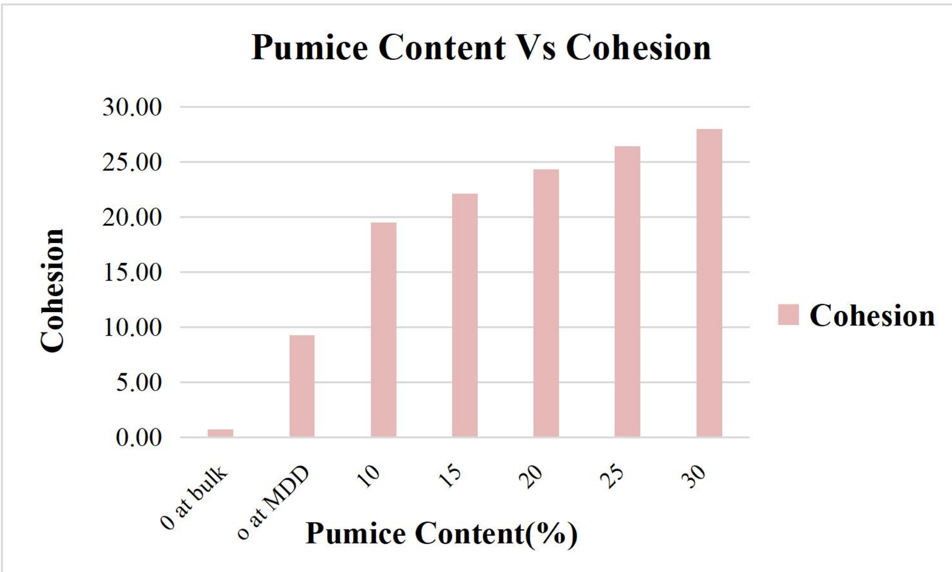
4.4 Direct shear test
Angle of internal friction

Pumice Content (%)	0 at bulk	0 at MDD	10	15	20	25	30
Angle of internal friction	6.72	31.48	35.95	37.37	38.70	39.81	40.9

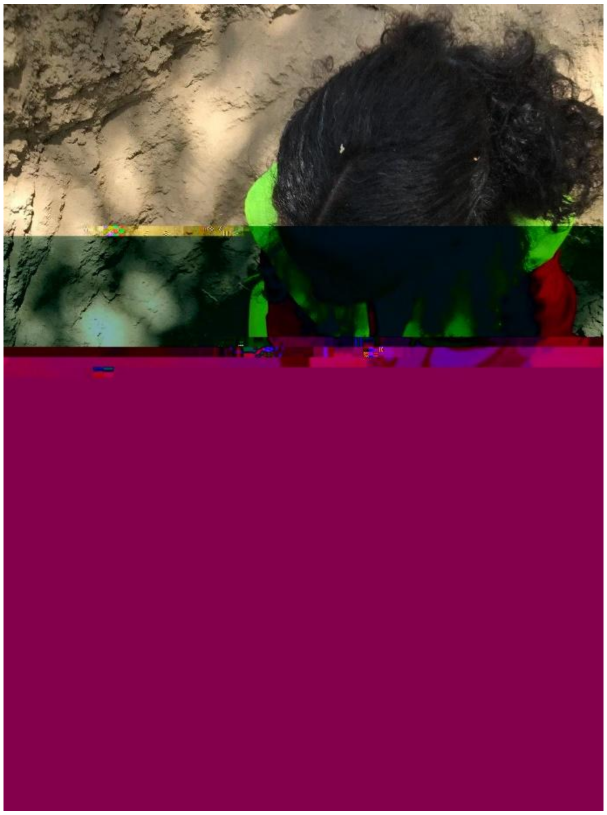


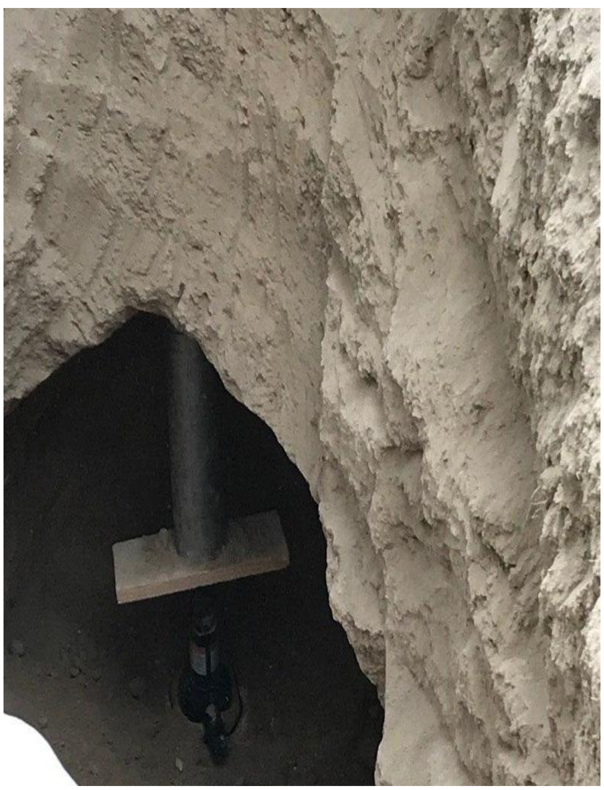
Cohesion

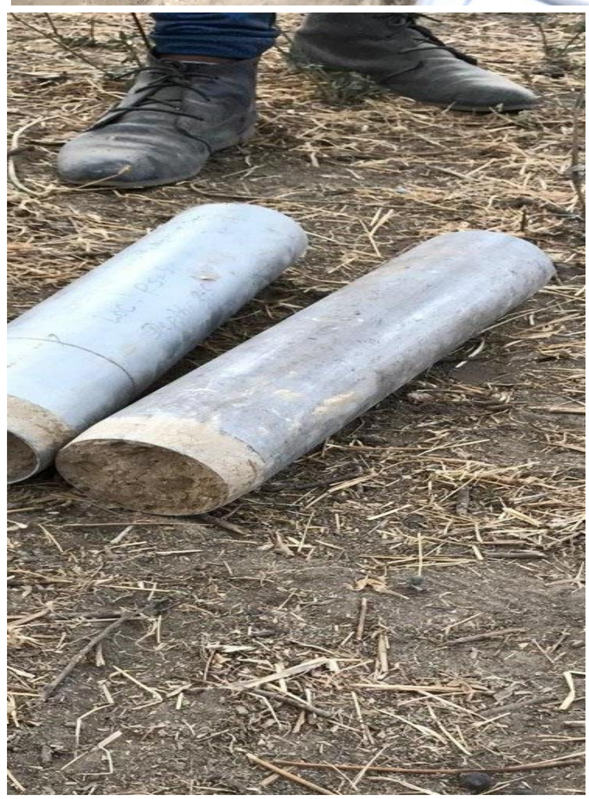
Pumice Content (%)	**0 at bulk**	**0 at MDD**	**10**	**15**	**20**	**25**	**30**
Cohesion	0.68	9.23	19.50	22.10	24.3	26.39	28



APPENDIX D: PHOTOS











100
ELE


ELE


TECNO
CAMON