HAWASSA UNIVERSITY

INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

DEPARTMENT OF CIVIL ENGINEERING

IDENTIFICATION OF COLLAPSIBILITY POTENTIAL OF SOILS ALONG SHASHEMENE – AJE ROAD CORRIDOR: AMBURE SITE

BY:

ASHEBIR TAJEBE BEDADA

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF HAWASSA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING (SPECIALIZATION: GEOTECHNICAL ENGINEERING)

DECEMBER, 2018

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "**IDENTIFICATION OF COLLAPSIBILITY POTENTIAL OF SOILS ALONG SHASHEMENE – AJE ROAD CORRIDOR: AMBURE SITE"** submitted in partial fulfillment of the requirements for the degree of **Master's** with specialization in **Geotechnical Engineering,** the Graduate Program of the **School of Civil Engineering and Built Environment,** and has been carried out by **Ashebir Tajebe Bedada** Id. No **PGGE/034/07**, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the school.

Dr. Tensay Gebremedhin	______________________	______________
Name of major advisor	Signature	Date
Bereket Bezabih (MSc)	______________________	______________
Name of co - advisor	Signature	Date

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

EXAMINERS' APPROVAL SHEET

We, the undersigned, members of the Board of Examiners of the final open defense by **Ashebir Tajebe Bedada,** have read and evaluated his thesis entitled "**IDENTIFICATION OF COLLAPSIBILITY POTENTIAL OF SOILS ALONG SHASHEMENE – AJE ROAD CORRIDOR: AMBURE SITE**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of **Master's** with specialization in **Geotechnical Engineering**.

____________________	____________________	________________
Name of the Chairperson	Signature	Date
____________________	____________________	________________
Name of Major Advisor	Signature	Date
____________________	____________________	________________
Name of Internal Examiner	Signature	Date
____________________	____________________	________________
Name of External Examiner	Signature	Date
____________________	____________________	________________
SGS Approval	Signature	Date

Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the School of Graduate Studies (SGS) through the School Graduate Committee (SGC) of the candidate's of the School.

Stamp of SGS

Date: ____________

HAWASSA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

EXAMINERS' APPROVAL SHEET

As members of the Board of Examiners of the final Master's degree open defense, we certify that we have read and evaluated the thesis prepared by **Ashebir Tajebe Bedada** under the title "**IDENTIFICATION OF COLLAPSIBILITY POTENTIAL OF SOILS ALONG SHASHEMENE – AJE ROAD CORRIDOR: AMBURE SITE**" and recommend that it be accepted as fulfilling the thesis requirement for the degree of **Master's of Science** in **Civil Engineering** with **Specialization** in **Geotechnical Engineering.**

________________	________________	_____________
Name of the Chairperson	Signature	Date
________________	________________	_____________
Name of Internal Examiner	Signature	Date
________________	________________	_____________
Name of External Examiner	Signature	Date

Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the SGS through the SGC of the candidate's of the School.

Thesis approved by

______________________	______________________	___________________
SGC	Signature	Date

Certification of the final Thesis

I hereby certify that all the corrections and recommendation suggested by the Board of Examiners are incorporated into the final thesis entitled "**IDENTIFICATION OF COLLAPSIBILITY POTENTIAL OF SOILS ALONG SHASHEMENE – AJE ROAD CORRIDOR: AMBURE SITE**" by **Ashebir Tajebe Bedada.**

______________________	________________	________________
Name of the Designate	Signature	Date

Stamp of SGS

Date: ________________________

Declaration

I hereby declare that this MSc Specialty thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

Name: Ashebir Tajebe Bedada

Signature: ______________________

Date of submission: ______________

This MSc Specialty thesis has been submitted for examination with my approval as Thesis advisor.

Name: Dr. Tensay Gebremedhin

Signature: ____

Place and Date of Submission: _____________________________

Acknowledgment

First and for most, I would like to show my deepest gratitude to my advisor Dr.Tensay Gebremedhin for the continuous support and guidance he has given me, for being there full of support at times when I need motivation and encouragement the most, and for giving me invaluable inspirations in class sessions to read different journals. Next, I would like to thank Ethiopian Roads Authority for offering me this scholarship. Finally, I would like to thank Mr. Bereket Bezabih for his role as co-advisor and assisting and guiding me as brotherhood; and the Schools of Civil Engineering and Built Environment of Hawassa University and Arba Minch University for their excellent welcome to conduct the laboratory tests of this study and invaluable support.

Dedication

I would like to dedicate all my life achievements, including this master's study, to my mother Aselefech Mekonnen, to my wife Genet Dessie, and my kids Mussie and Aaron. I can see it in my mind, the pride you all would have felt to see me standing at this point in my life. May God bless you more to live long enough to see me achieve more, and that is truly the only way I can repay you back the reward.

Abbreviations

ASTM: American Society for Testing Materials

AASHTO: American Association of Highway and Transportation Officials

HU - IoT: Hawassa University – Institute of Technology

AMU – IoT: Arba Minch University - Institute of Technology

ERA: Ethiopian Roads Authority

LL: Liquid Limit

PL: Plastic Limit

PI: Plasticity Index

OMC: Optimum Moisture Content

MDD: Maximum Dry Density

SPT: Standard penetration Test

CPT: Cone Penetration Test

LHS: Left Hand Side

RHS: Right Hand Side

SL and SR: Sample from the left and right sides, respectively

NP: Non Plastic

NMC: Natural Moisture Content

TABLE OF CONTENTS

List of Tables

List of figures and illustrations

List of Appendices

Abstract

Large areas of the earth's surface are covered by soils that are susceptible to large reduction in bulk volume when they are near to saturation (Knodel, 1992). These soils are termed as collapsing soils and are common in many parts of Ethiopia. This study is concerned with the identification of the collapsibility potential of soil found along Shashemene – Aje road corridor, in particular at Ambure site, Western Arsi zone, Ethiopia. The study was carried out as relatively little is known about the collapse phenomenon in the silty/ fine sands. The aim of the study was achieved through the experimental work which included engineering tests like: one dimensional oedometer testing and direct shear strength testing; and classification tests like: moisture content, bulk dry density, compaction, particle size analysis, and specific gravity tests. To provide a better understanding of the collapse behavior of the soil, classification tests were performed to make indicator analyses and evaluation of collapsibility potential using various empirical correlations. Based on the obtained results, soils from the area of study have a collapse potential values ranges from 3.81% to 5.89% and found moderate to highly collapsible types of soils in nature. From the findings of this study, as the Ethiopian Roads Authority site investigation manual does not consider the significant effects of these types of soils, the author suggests a direction in providing the design and construction specifications of problematic soils of collapsible types.

Keywords: Collapsible soils, collapsibility criteria, silty/ fine sands, Ambure site

CHAPTER 1: INTRODUCTION

Soil is one of the most important civil engineering materials in the construction process and determination of its geotechnical properties is the most important first phase of work for every type of civil engineering structures. For instance; index and engineering properties of soil are very important for empirical and analytical evaluation of geotechnical problems caused by problematic soils.

Soil that causes additional problems as a result of the circumstances of its composition or change in environmental conditions is termed as problematic soils. Soils in geotechnical engineering can be problematic especially if they become expansive, collapsible, dispersing easily, undergoing excessive settlement upon loading, having a distinct lack of strength or are soluble. These characteristics are likely as a result of their composition, the nature of their pore fluids, their mineralogy or their fabric. The design and construction of foundations on these kinds of soils poses serious challenges in which the conventional methods become impossible to adopt both from an engineering or economic point of view and hence requires an extensive understanding of the soil characteristics. There are a number of these types of soils which tend to be problematic and they include but not limited to expansive soils, dispersive soils, and collapsible soils (Mohsen et. al, 2012). For the purpose of this study, emphasis was placed on the collapsible soils.

Collapsible soils are classified as one sort of problematic soil, which are moisture sensitive because increase in moisture content is the primary triggering mechanism for its volume reduction (Mohsen et. al, 2012). These types of soils are typically silt and sand size with a small amount of clay and low plasticity index (Pawlak, 1983). These soils exhibit large decreases in strength at moisture contents approaching saturation, and rapidly undergo a significant change of volume due to a major readjustment of the soil structure upon wetting, or under the influence of the applied boundary forces or both (Rogers,1994). Collapsible soils are basically characterized as soils which consist of high void ratio, low initial bulk density and water content, great dry strength and stiffness, high percentage of fine grained particles and zero or slight plasticity (Howayek et. al, 2011). Soil collapse can occur in soil that exists above the groundwater level and due to this reason it is referred as the properties of unsaturated soils. The process of saturation weakens the existing engineering soil properties, or eliminates clay bonds holding the soil grains together through water tension (Mulvey, 1992). When this soil becomes saturated with water it collapses and so creates problems in buildings, roads, airfields, railways, and earth dams and reservoirs (Schwartz, 1985). One of the most important problems concerning collapsible soils in roads construction is instability and considerable settlement due to minor changes in the water content which can cause remarkable damages to overlying road prism instability and poor pavement performance.

Collapse and consolidation settlements are different in their characteristics. Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability owing to drainage of some of the pore water, the process continuing until the excess pore water pressure set up by an increase in total stress has completely dissipated. Consolidation settlement will result, for instance, if a structure is built over a layer of saturated clay or if the water table is lowered permanently in layer overlying clay (Craig, 2004). However, collapse is very different from traditional consolidation as no water is being expelled and in actual fact the soil will be absorbing water and progressively losing strength (Dudley, 1970). The problem is associated with a change in the compression characteristics of the soil effectuated by capillary forces resulting from partial saturation (Jennings and Knight, 1975).

The main structure of residual granitic soils consists of bulky sized quartz particles, silts, fine sands and colloids. Through intense leaching of the clays, silts and colloidal matter, a structure similar to a honeycomb develops. This structure becomes very unstable when saturated and is as a result susceptible to collapse and large bulk volume decrease (Koerner, 1984). A soil with a collapsible fabric can withstand moderately large imposed stresses with small settlements at low in-situ moisture content. When wetting up occurs, a decrease in volume and associated settlement will take place with no increase in the applied stress. The change in volume is associated with collapse of the soil structure (Schwartz, 1985). Collapse may occur in any open textured silty or sandy soil with a high void ratio which yet has moderately high shear strength at low moisture content owing to colloidal or other coatings around the individual grains (Brink et. al, 1982).

Even though sample disturbance is a major problem when dealing with undisturbed samples of problematic or collapsing soils, some geotechnical properties of problematic soils must be identified so that prediction of collapse potential can be also projected from results obtained both from classification and engineering tests through experimental correlations and direct measurement from collapse test before the commencement of any design or construction activities. For this reason, we should investigate the nature and properties of soil before starting the design and construction of civil engineering structures, especially of large scales like roads and dams, in an attempt to predict and minimize the collapse that may occur due to the presence of problematic soils later on.

In Ethiopia, from physical observations, the researcher presumed that problematic soils are most abundantly found in rift valley strip and southern part of the country where in Borena, Konso and Omo basins. According to Giacomo Corti, the typical Ethiopian rift morphology is 500 km long segment that starts from the eastern Ethiopia of Afar depression (latitude $9°40'N$) extended to the regions of southern Ethiopia of lakes Abaya and Chamo (latitude $5°30'N$). Hence, this study primarily targets Shashemene - Aje road corridor which is one of the federal roads found in the rift valley strip of Ethiopia and extends from Shashemene town to Shala woreda - Aje town. This road corridor is well known for its complicated geotechnical and geological problems with frequent heavy maintenance operations for pavement and road side protection works. However, this study only dealt with the occurrence of major collapse (surface rupture) at Ambure site in particular. The study included collection of soil samples, sample preparation, performing relevant laboratory tests, making analysis for the assessment of collapsibility potential of soils, and there upon conclusions and recommendations were made from the results obtained.

1.1 Background of the Study

The government of Ethiopia has paid off much effort to rehabilitate, expand and restoring the road networks to be sustainably established throughout the country. As part of its reform, the government assigned administration of rural roads to the regional governments and main roads to ERA as part of the federal government's authority. Hence, ERA administers the construction and maintenance of plenty of road projects of different design standards throughout the country at different district offices.

Collapsible soils are moisture sensitive in that increase in moisture content is the primary triggering mechanism for the volume reduction of these soils. One result of urbanization in arid regions is an increase in soil moisture content. Therefore, the impact of development induced changes in surface and groundwater regimes on the engineering performance of moisture sensitive arid soils, including collapsible soils, becomes a critical issue for continued sustainable population expansion into arid regions. These days developments in all aspects of life have resulted in the construction of modern cities and large structures and this clearly establishes the need for an in depth study of the subject of subsidence in collapsible soils. For instance; due to the expansion of urbanization, water consumption patterns were quite different from those past times. As a rapid advancement of civilization and increasing use of water for irrigation, industrial and domestic purposes may give rise to lowering of groundwater level and cause severe damage to an overlaying nearby structures on collapsible soils. Hence; a safe, reliable and economic method for predicting areas of likely soil subsidence is considerably important and an extensive amount of effort has been performed by different researchers in the past to enumerate parameters that meet the criteria of collapsibility potential of soils associated with metastability (Zapata et al. 2001).

Therefore, studying the identification of collapsibility potential of soils at Shashemene - Aje road corridor, Ambure site in particular is believed to figure out possible geotechnical engineering solutions especially in an attempt to minimize the construction cost overrun due to unforeseen geotechnical problems. Simultaneously, it will enhance the organization's capacity to complete roads projects within the estimated project completion periods and can be used as a benchmark for different road projects which are administered under the authority with similar scrutiny.

1.2 Geologic Classification of Soil Formations

Soils are classified geologically by their origin as residual, colluvial, alluvial, aeolian, glacial, or secondary soils. Conditions in arid and semi-arid climates favor the formation of the most collapsible soils. The mechanisms that account for almost all naturally occurring collapsible soil deposits are debris flows, rapid alluvial depositions, and wind-blown deposits (loess). This fact helps explain why geologic, geographic, and geomorphological information can be helpful in anticipating the location of collapsible soil deposits (Huth, 2007).

1.3 Geological, Geomorphological, Hydrogeological and Hazardous Conditions

The study area is located around 5.0 kilometers from Shashemene town, and the author considered the area would have the same geological, geomorphological and hydrogeological conditions with South - Western part of Shashemene area for only the case of this study.

1.3.1 Geology and Geomorphology of the Area of the Study

Identification of collapsible soil is best accomplished by testing specimens. However, geologic and geomorphologic information can be useful in anticipating collapsible soil deposits. Geomorphological considerations are quite valuable as a first step towards identification of a naturally occurring collapsible soil deposit (Rezaei et. al, 2012). For example, Holocene alluvial fan deposits in the South - Western regions of the United States should be assumed to be collapsible unless a comprehensive testing program demonstrates otherwise (Beckwith, 1995). Loessial deposits in northwestern China are known to exhibit significant collapse potential. Chinese researchers have found that geographical and geomorphological information is strongly correlated with collapsibility and collapse potential (Liu, 1998; and Lin, 1995). Thus; geological reconnaissance, coupled with experience with similar depositional environments, forms an important part of the process of collapsible soil identification.

According to the report by Geological Survey of Ethiopia (2011), Shashemene town was built on mid Pleistocene Corbetti ignimbrites overlain by two units of younger pyroclastic deposits: yellowish phreatomagmatic tuff and Wendo - Koshe pumice. The area Shashemene is located within the central part and the Eastern edge of the main Ethiopian rift and is dominated by the eastern part of the lower Pleistocene Hawassa caldera. The subsequent volcanic sequence of the middle Pleistocene age, Corbetti ignimbrites, is exposed in the eastern half, whereas to the west it is covered by Holocene pyroclastic deposits. Polygenetic sediments (re - sedimented pyroclastic, alluvial sediments, lacustrine sediments) fill the basin of the former Lake Cheleleka in the South - Western corner. The character of the sediments is a result of tectonic, volcanic and exogenous processes and fluctuations of water levels in the lake. They represent a mixture of re - sedimented loose pumice or mixtures with a low portion of silt, sand or soil. Grayish to black mud rich in organic compounds is locally developed. Local clasts of weathered ignimbrites and tuffs also occur.

Figure 1.1: Satellite map of the area of study (Source: Google Earth ©2018)

According to the report by Ethiopian Geological Survey (2011), there are two main geomorphological regions therefore joining each other in the area of the Shashemene. The western part, where the study area is located, belongs to the rift floor region; whereas the eastern part belongs to highlands. The eastern highlands region with altitudes exceeding 2,000 meters above sea level differs significantly from the rift floor area with altitudes varying between 1,700 and 1,900 meters above sea level. Hence, the surface runoffs at the eastern highlands flow down to the western lowlands.

1.3.2 Geological Hazards in the Area of Study

Prominent geological hazards in the study area are major surface rupture, gully formations and surface erosion. The major surface rupture that was happened in March 2016 gave rise to the formation of deepest gully. Understanding historical and present gully development is essential when addressing the causes and consequences of land degradation, especially in vulnerable dry land environments (Frankl et al, 2013). The study area vulnerable to gully erosion is associated with ephemeral streams with gentle slopes upon Corbetti ignimbrites overlain by phreatomagmatic tuffs. The erosion cuts through the lateritic and loose non - welded pyroclastic deposits being stopped on the top of the welded facies of the Corbetti ignimbrite (EGS, 2011).

1.3.3 Hydrology and Climatic Conditions of the Area of the Study

Topography and drainage play a major role in the development of a collapsible soil structure. Under topographical conditions which favor easy internal drainage, much of the clay is washed out and the characteristic structure of a collapsing soil develops (Weinert, 1980). According to climate-data.org, the climate is warm and temperate in Shashemene and around. According to the National Meteorology Agency of Ethiopia data from 2012 to 2017, the average maximum temperature of the area is about 32.2 °C and the average minimum temperature of about 10.3°C. From table 1.1, the warmest month is March with the average maximum temperature of 30°C of the six years. Whereas; from the last six years collected meteorological data, the area accommodated the maximum of 711.7 mm and 510.2 mm of annual precipitation falls in 2013 and 2016, respectively. Most precipitation falls between the months April to September with an average precipitation of 408.8 mm. However, in March 2016, the area accommodated the highest amount of precipitation of the month in six years with monthly precipitation of 42.4 mm. The following table summarizes the historical climate data of Shashemene and the area around.

Table 1.1: Monthly Average Maximum Temperature (°C) of Shashemene area

Year	Monthly Average Maximum Temperature (°C)											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
2012	N/A	N/A	N/A	N/A	28.0	26.2	23.2	24.7	24.2	27.2	N/A	N/A
2013	N/A	N/A	N/A	N/A	N/A	19.1	18.6	20.6	21.1	21.7	21.9	21.0
2014	20.1	19.8	N/A	26.2	26.3	26.7	25.6	26.5	26.4	N/A	27.6	26.4
2015	26.0	N/A	32.2	31.2	28.2	28.0	27.7	28.3	28.1	28.0	28.1	27.8
2016	28.3	28.3	29.0	29.5	28.9	28.5	28.4	28.2	28.1	28.2	28.3	N/A
2017	28.3	NA	28.9	27.9	27.9	28.3	28.2	28.6	28.1	N/A	N/A	N/A

Table 1.2: Monthly Average Minimum Temperature (°C) of Shashemene area

Year	Monthly Average Minimum Temperature (°C)											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
2012	N/A	N/A	N/A	N/A	13.3	14.0	14.4	13.9	13.0	10.7	N/A	N/A
2013	N/A	N/A	N/A	N/A	N/A	11.0	10.3	10.6	11.0	10.9	12.8	12.0
2014	11.5	11.0	N/A	14.4	13.9	14.1	13.6	13.9	13.9	N/A	14.1	14.0
2015	14.5	N/A	13.7	15.9	14.2	14.0	13.9	13.8	13.2	13.6	13.4	12.9
2016	13.3	13.6	13.4	13.5	13.8	13.2	13.3	13.5	13.4	13.4	13.3	N/A
2017	13.2	N/A	13.2	13.3	13.1	13.3	13.2	13.1	13.0	N/A	N/A	N/A

Table 1.3: Monthly Total Rainfall (mm) of Shashemene area

Year	Monthly Total Rainfall (mm)											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
2012	0.0	0.0	6.2	87.3	34.5	67.3	118.8	59.1	125.2	20.6	N/A	N/A
2013	N/A	N/A	N/A	103.2	74.1	70.8	93.4	163.6	146.3	26.0	34.3	0.0
2014	0.0	38.8	N/A	38.2	134.1	52.9	35.6	52.7	44.0	N/A	12.4	0.0
2015	1.0	N/A	20.5	36.6	91.3	49.2	62.2	72.5	49.3	9.4	5.4	24.0
2016	44.6	0.0	42.4	135.0	65.8	30.4	27.4	21.2	39.2	66.4	37.8	N/A
2017	0.0	N/A	30.0	31.6	107.0	20.0	45.8	24.6	42.4	N/A	5.2	0.0

1.4 Problem Statement

Shashemene - Aje road is one of the major road networks that are administered under ERA, Shashemene district maintenance office, serving more than a decade as a main gate to Southern nations and nationalities region major towns like Wolaita Soddo, Arba Minch and Jinka; as well as, mega projects like Gilgel Gibe III hydropower project and Kuraz sugar factory.

The author used to observe various geotechnical and geological problems along the mentioned road corridor. In particular, after the rainy seasons, the most likely to observe are the formation of deep gorges along either side of the road ditches at considerable stretches; thick silt deposit (as shown in figure 1.2) overlay on pavement and farmlands at most locations resulting from flood; formation of many spot holes; and corrugation and cracks on pavement.

Figure 1.2: Thick silt deposit overlay on pavement resulting from flood blocked the traffic (Photo by Author; August 24, 2017)

Collapsing behavior has also been reported for some alluvial deposits, specifically water deposited loose sediments. Alluvial type collapsible soils are mainly formed by flash flood or mudflows (debris flows) which come from huge precipitation in irregular intervals. Under these conditions, loose and metastable structures are induced due to particles which are deposited suddenly and locally (Howayek et al, 2011). To fix these geotechnical problems, ERA - Shashemene district maintenance office regularly operates light and heavy maintenance operations of different scope.

In March 2016, major surface rupture occurred at around 5.0 kilometers away from Shashemene town, Ambure village across the asphalt road corridor. This incident separated the road into two ends and blocked the traffic flow until a detour was constructed by the district maintenance office. From the physical observation, the extended rupture across the road soil strata nearly 600 meters in length with 50 meters of deep depth. The location where this major surface rupture occurred is the end point at which large quantity of floods from the Shashemene town and nearby villages was collected in natural pond and overflowed to the existing miter drain.

Figure 1.3: The surface rupture in March 2016 - RHS (Ambure, West Arsi zone, Ethiopia; Photo by Author; March 14, 2018)

As collapsible soils are highly sensitive to moisture variations, the researcher suspected that the aridity of the local environment in winter season as well as the moisture variations in summer season may give rise to the collapse nature of soil. Basically the author also has two extra reasons enforcing his interest to study on identifying the collapsibility potential of soils at this particular site.

Figure 1.4: The surface rupture in March 2016 - LHS (Ambure, West Arsi zone, Ethiopia; Photo by Author; March 14, 2018)

Firstly, there were abundant anthills at the local farmlands and over the pavement of some rural access roads during dry season. These anthills believed to be one of the main indicators of the presence of sinkholes below the ground surface that may give rise to the soil to collapse while nears to saturation. Second, the major surface rupture at Ambure village lets ERA to expend more than 37 million birr for maintenance operations.

1.5 Objectives of the Study

1.5.1 General Objectives

The main objective of this study is identifying collapsibility potential of soils along Shashemene - Aje road corridor, Ambure site in particular, and to develop a better understanding of the behavior of collapsible soils regarding road design and construction of Ethiopian perspective.

1.5.2 Specific Objectives

The specific objectives of this study are:

- To identify the collapsibility potential of soils using both existing experimental relations from soil parameters obtained from classification tests, and direct measurement from one – dimensional consolidation for collapse test.
- To identify the degree or severity of collapsibility potential of soil.
- To make an indicator analysis of soil from general properties of collapsible soils and compare the collapse potential results obtained.
- To understand stress – strain and compaction – moisture relationships of soils in the area of study.
- To make conclusions and recommendations up on the soil parameters obtained from classification and one - dimensional odometer tests.

1.6 Scope of the Study

This study was only focus on the major surface rupture of soil strata across the road at 5.0 kilometers away from Shashemene town, at Ambure site in particular. The boundary for the experimental unit of this study was limited to the total width of 50 meters where 25 meters wide from the center to both sides of the road. Similarly, the soil samples were taken only from the naturally existing soils 0.5 to 6.5 meters depth from the ground surface and limited to investigation at laboratory level only, i.e. this study did not include field tests and investigation of materials which are imported from other sources for the road construction.

Soil samples were collected from soil strata where major surface rupture exactly occurred through a combination of judgmental non - random and systematic random methods of sampling at the selected 8 (eight) sampling locations. Four samples at either sides of the road were taken where one sample at every 2.0 meters depth, starting from 0.5 meters from the ground surface, using manual workers and hand tools. The reason that the number of samples were limited to 8 (eight) is due to the reinstatement of gorge and only remain about 7.0 meters, and hence the author can't go further depth to collect extra samples. The direction of sample pits excavation for the soil sampling was parallel to the ground surface (perpendicular to soil profile) within the stated depth based on the soil stratification that likely for sampling.

Based on the availability of apparatuses and laboratory centers, ASTM and AASHTO standard manuals were interchangeably used for the laboratory procedures to conduct the designated soil laboratory tests.

1.7 Description of Research Questions

At the end of this study, it will come up with answering the following questions:

- Do the soil layers at the area of study have a potential to collapse?
- What is the severity or degree of collapsibility potential?
- What are the most significant soil properties that lead the soil susceptible to collapse?

1.8 Significance and Outcomes of the Study

The significance of this study is to offer the considerable cautions that should be taken regarding sampling and testing of soil, design, and construction of roads on collapsible soils that may cause damage by sudden and often large induced settlements when the soils are saturated after construction. Hence, identifying and predicting collapse potential of soils is very important to the design and construction of roads on the soil with collapsible nature.

In Ethiopia; for instance, the site investigation carried out at different road projects did not give a part to evaluate or identify potentially collapsible soils before the commencement of any design or construction works. Most of the soil tests, both in the field and laboratory level, were also conducted under partially saturated condition. However; as indicated by Jennings and Knight (1975), errors in assessment of engineering properties like: bearing capacity or compressibility of soil has been made since partially saturated or dry condition will often give a potentially collapsible soils dense or stiff consistency. On the other way also the recommendations given by the geotechnical engineer have not been critically evaluated, or totally may have been ignored by the client, designer or contractor mostly to tackle project cost and time overruns. So, under this study, the researcher made all efforts to scrutinize the topic of the study and offer the stakeholders of the construction sector (especially in roads) additional options to deal with site investigation reports in detail before the commencement of any construction activities. Because, detailed site investigation reports have a tendency to amend the scope of the intended project design, construction procedures and project completion cost as well.

As the author didn't find any published journals or studies directly related with this topic of study in Ethiopia, the outcomes of the study may bring initiation for other interested researchers helping to identifying the research gap on the topic of study; and moreover, the author strongly believe that this study helps to advance the knowledge on understanding the nature and properties of collapsible soils in civil engineering services of any.

1.9 Limitations of the Study

The author took some remedies to overcome things that may create challenges while in studying as listed below.

- Allocating extra budget for the purpose of completing this study.
- Travelling to AMU – IoT, which is 275 kilometers away from HU – IoT, to conduct consolidation test, direct shear strength test, and hydrometer tests.
- The author used polyvinyl chloride (PVC) pipes as sampling tools instead of shelvy tubes to collect undisturbed samples.

1.10 Organization of the Study

This study is organized under six chapters. Chapter one deals with the introduction which in turn contains the background of the study, statement of the problem, basic research questions, objectives and scope of the study, significance and outcomes of the study, and limitations of the study. Chapter two presents concepts and definition, literature review and framework of the study. Chapter three is about materials and methods used in this study like: research approach, research and sample design, data sources and instruments, method of the research analysis. Chapter four provides the summery of test results and graphs. Chapter five provides discussions and interpretations of the test results summarized in previous chapter with respect to the corresponding research questions of the study. Finally, chapter six constitutes conclusions and recommendation so as to solve based on the findings based on practices in Ethiopian construction standards and practices.

CHAPTER 2: LITERATURE REVIEW

2.1 Definitions and Properties of Collapsible Soils

2.1.1 Definition of Collapsible Soils

Numerous definitions describing potentially collapsible soils or the collapse phenomenon exist. Rogers (1995) discusses a number of definitions appearing randomly in the literatures, which are included below.

"A collapsing soil is a soil which rapidly undergoes an appreciable loss of volume due to a major readjustment of the soil structure upon wetting or under the influence of an applied boundary force." (Frank J. Anderson, 1968)

"Collapsible soil is a soil that undergoes an appreciable amount of volume change upon wetting, load application or a combination of both." (Sultan, 1969)

"Collapsible soil is any unsaturated soil that goes through a radical rearrangement of particles and a great loss of volume upon wetting with or without additional loading." (Dudley, 1970)

"There will be a gradual increase in compressibility as well as gradual decrease in shear strength of a collapsible soil during the saturation process." (Jenning and Burland, 1969 and Baden et.al, 1973)

"Collapsibility of soil can withstand a large applied vertical stress with small amount of compression, but also shows much larger settlement upon wetting of partially saturated soils, without any increase in applied vertical stress."(Jennings and Knight, 1975)

“Collapsibility of soil is settlement in partially saturated soil due to an increase in the degree of consolidation.” (Booth, 1977)

“Collapsible soil is a state of under consolidation related to apparent cohesive strength of unsaturated soils.” (Handy, 1973)

“Collapsible soil is one type of problematic soil which can withstand relatively large imposed stresses with small settlements at low in situ moisture content but will exhibit a decrease in volume and associated settlement with no increase in the applied stress if wetting up occurs.” (Schwartz, 1985)

“During wetting - induced collapse, under a constant vertical load and under oedometer conditions, a soil specimen undergoes an increase in horizontal stresses.” (Maswoswe, 1985)

“Collapsible soils are moisture sensitive and susceptible to large reductions in volume with increase in moisture content.” (Radhey et.al, 2006)

2.1.2 Properties of Collapsible Soils

Collapsible soils are generally associated with an open structure formed by sharp grains, low initial density, low natural water content, low plasticity, relatively high stiffness and strength in the dry state, and often by particle size in the silt to fine sand range (Mitchell and Soga, 2005). As their name indicates, these soils can exhibit a large volume change upon wetting, with or without extra loading, thus posing significant challenges to the geotechnical profession. Numerous soil types can fall in the general category of collapsible soils, including aeolian deposits, alluvial deposits, colluvial deposits, residual deposits, and volcanic tuff. A well-known aeolian deposit, known to often exhibit collapsing behavior, is loess, a yellow to reddish brown silt size soil, which is characterized by relatively low density and cohesion, but appreciable strength and stiffness in the dry state. Aeolian deposits with significant tendency to collapse are often found in arid regions where the water table is low (Gildenhuys, 2010). However, even in environments with medium rainfall, fine aeolian deposits can still present high collapsible potential, particularly if an impermeable surface crust has protected them from water infiltration (Clemence and Finbarr, 1981).

Collapsible soils present significant challenges to the engineering profession during construction, during the service life, and to a lesser degree during the design period. Challenges related primarily to differential settlements are encountered in the construction of roads on collapsible soils. Differential collapse settlement across roadway sections comes from two major factors. The first one is non-homogenous subgrade that encompasses materials with different degree of collapse potential, and the second non-uniform distribution of wetting in subgrade materials (Houston et al, 2002).

The main geotechnical problem associated with collapsible soils is the significant loss of shear strength and volume reduction occurring when they are subjected to additional water from rainfall, irrigation, broken water or sewer lines, moisture increase due to capillarity or ‘‘pumping’’ as a result of traffic loading, and the rising of ground water level. Generally, collapsible soils are under unsaturated conditions in the dry state, with negative pore pressure resulting in higher effective stresses and greater shear strength. Upon wetting, the pore pressure become less negative and the effective stresses are reduced causing a decrease in shear strength. Additionally, the water can dissolve or soften the bonds between the particles, allowing them to take a denser packing. This mechanism, referred to as wetting - induced collapse, or hydro - compression, can take place with or without extra loading (Howayek et al, 2011). Prevention of the problems induced by collapsible soils requires consideration of the identification and characterization of collapsible soils; assessment of collapse potential and settlement; estimation of the distribution and the degree of wetting in the deposit; and evaluation of design alternatives and mitigation strategies (Gildenhuys, 2010).

2.2 Conditions for Collapse Settlement to Occur

The factors that produce collapse as follows (Pereira et al, 2000):

- An open, partially unstable, unsaturated soil fabric;
- A high enough net total stress that will cause the structure to be metastable;
- A bonding or cementing agent that stabilizes the soil in the unsaturated condition; and
- The addition of water to the soil, which causes the bonding or cementing agent to be reduced and the inter-aggregate or inter-granular contacts to fail in shear, resulting in reduction in total volume of the soil mass.

The following conditions must be satisfied before collapse settlement can occur (Gildenhuys, 2010):

- A collapse fabric must be present in the soil.
- Partial saturation is essential. When soils are below the water table, collapse settlement will not occur;
- An increase in moisture content is essential. When the moisture content increases the bridging colloidal materials experience a loss of strength and the soil grains are forced into a denser state of packing associated with a reduction in void ratio;
- Most of the soils with a collapse fabric must be subjected to an imposed pressure which is greater than their overburden pressure before collapse will take place.

2.3 Problems Associated with Construction on Soils with a Collapsible Fabric

There are numerous recorded instances of problems associated with construction on soils with a collapsible fabric (Gildenhuys, 2010). Taking into consideration the modern knowledge with regard to these soils however, it appears reasonable to conclude that problems with construction take place under one or more of the following circumstances:

- No geotechnical investigation was done;
- Construction was carried out prior to the identification of the collapse phenomenon. This is mainly the case with settlement and distortion occurring within many older structures;
- During the investigation potentially collapsible soils within the profile were not correctly evaluated or identified (Schwartz, 1985). These errors in the assessment of compressibility or bearing capacity have been made, given that a partially saturated condition will frequently give a potentially collapsible soil a dense or stiff consistency (Jennings and Knight, 1975);
- The client, designer or contractor ignored the recommendations made by the geotechnical engineer (Schwartz, 1985).

2.4 Distribution of Soils with a Collapsible Fabric

Collapsible soils are covering vast areas of many arid and semi-arid countries. These soils have been found in all types of areas, however, within restricted geographic areas the identification of a source of a soil deposit or type of land shape (Morsy et al., 2017).

2.4.1 Transported Soils

The properties of transported soils are influenced by the mechanisms of transportation and deposition (McCarthy, 2006). It is noticeable that these types of transported deposits, with their associated problems due to collapse, can be found anywhere in the South Africa (Schwartz, 1985).

2.4.2 Residual Soils

The collapsible character of the residual soils derived from these ancient granites is associated with the deeply weathered soil profiles found in the humid regions in the eastern part of South Africa (Schwartz, 1985). In the humid regions chemical decomposition is the prevailing mode of rock weathering, producing soils with medium to high compressibility and low shear strengths (Zeevaert, 1983).

2.5 Sampling Procedures in Soils with a Collapsible Fabric

The consistency of the test procedures clearly dependent on the tests being carried out on representative undisturbed samples (Schwartz, 1985). According to Jennings and Knight (1975) one should use block samples cut by hand from a test pit hole or take samples in the field directly into consolidometer rings.

Specimens may be remolded or compacted or taken from undisturbed soil samples that are prepared in accordance with guidelines of ASTM - D5333. The minimum specimen diameter shall be 50 mm. The minimum initial specimen height shall be 12 mm, but shall be not less than ten times the maximum particle diameter. The sampling method to collect soil samples for this study was discussed in chapter three.

2.6 Measurement of Collapsibility Potential of Soils

Identification of collapsibility of soil was emphasized by many researchers in the past through laboratory tests (Holtz and Hilf, 1961; Jennings and Knight, 1975; Jasmer and Ore, 1987; Anderson and Reimer, 1995; Reznik, 2007; Gaaver, 2012; Kalantari, 2012; Rezaei et al., 2012) and field test tests (Reznik, 1993; Houston et al., 1995). Field tests are undoubtedly expensive in ground investigations and most of these laboratory procedures involved performing tests on undisturbed soil samples through direct shear tests and oedometer tests, which is very difficult to sample in particularly the cohesionless soils.

Collapse potential is used to estimate settlement that may occur in a soil layer at a particular site from different empirical equations using a predetermined applied vertical stress and saturation applied to a soil specimen taken from the soil layer. Procedures for estimating potential for collapse are uncertain because no single criterion can be applied to all collapsible soils. Index of collapse potential of soils for smaller applied vertical stress may be estimated assuming that the soil does not swell after inundation at smaller applied vertical stress.

Laboratory tests quantitatively study the collapse on wetting. Still, for tests to be accurate enough for design purposes, laboratory experiments need to follow stress paths and other in situ conditions very accurately. This raises questions about the design value of some tests (De Wet, 2009).

Gibbs and Bara (1962) stated low unit weights indicate a loose structure; the in-place dry unit weight is a good parameter for collapse prediction. Other properties used have included unit weight, water content, void - ratio, degree of saturation, liquid limit, plastic limit, plasticity index, and porosity. An easily applied criterion requiring only dry unit weight and liquid limit values was successfully used to delineate potentially collapsible soils. This criterion states that the soil voids in a soil mass must be sufficient to contain enough water for the soil to be at its liquid limit. When the soil has a low unit weight such that its void space is sufficiently large to hold the liquid limit water content or more, saturation can easily cause a liquid limit consistency at which the soil offers little resistance to deformation. If the voids are greater than this amount, saturation would result in water content in excess of the liquid limit and the potential for collapse would be high. If collapse did not occur, the soil would surely be in a very sensitive condition (Knodel, 1980). Although this criterion does not directly consider the time effects of cementation it is still very useful because as mentioned previously, all collapsible soils are weakened by wetting, whether immediately or eventually.

According to Schwartz (1985), during 1955 the sudden large settlement of portions of a steel-framed building near Witbank drew attention to a phenomenon which has become known as that of 'collapsing soils'. These are soils which decrease in bulk volume when water is introduced into them. A research project on these soils was initiated at the University of the Witwatersrand and Knight developed a theory explaining the mechanism of collapse and a laboratory procedure for predicting collapse settlement. It is of interest to note that an article published by Lund indicates that the phenomenon of 'collapse' was identified as early as 1948 during an investigation carried out to determine the reasons for failure of a reservoir constructed to the north west of Rustenburg. Schwartz (1985) made review on the initial research work carried out by Knight to understand how the mechanism of collapse was identified and used both laboratory (double-oedometer and classification tests) and field (Sausage test) testing procedures were used to predict and quantify collapse settlement of collapsible soils in South Africa. The development and interpretation of this testing procedure was carried out using conventional soil mechanics principles and consideration was given to the significance of the effective stress concept in the evaluation and interpretation of the collapse phenomenon and the proposed testing procedure.

Knodel (1992) stated variety of factors and conditions that are present in collapsible soils. Collapse settlement will not occur in soils which lie below the water table, because the condition of partial saturation is an essential prerequisite to collapse. If the soil is silty or clayey, it will likely have a stiff or hard consistency due to partial saturation. Therefore, during site inspection the in - situ water content must be considered and judgment made on a sample which has been wetted. Errors in assessment of collapse susceptibility mostly have been made simply because the engineer examining a dry profile has forgotten that the subsoil will become wetted after completion of the structure. Specific qualities have been found for specific collapsing soils in a given area, but frequently these qualities do not apply to other collapsing soils. Since the quantity of settlement that will be destructive varies from one structure to another, it is necessary to determine the amount of possible settlement to promote efficient design at a specific site or for a given project. The amount and rate of collapse to be affected by mineralogy, percentage of clay, shape of grains, grain size distribution, moisture content, void ratio, pore sizes and shapes, cementing agents, and others. However; Atterberg limits values, in combination with other soil properties, are widely used in identifying these soils. Many collapsing soils have LL < 45 and PI < 25, and usually much lower, often in the non-plastic range. Water content in place that is well below 100 percent saturation is required for collapse but the OMC for maximum collapse is usually between about 13 and 39 percent. Some soils may even initially gain strength as the water content increases; whereas, some soils collapse when wetted without additional loading other than the added water, but will decrease in volume even more with added load surcharge. Other soils require additional loading for any collapse to occur. Simple routine tests can indicate whether soils are collapse-susceptible and more complex tests provide data to determine the amount and rate of collapse. None of the tests, however, replicate field conditions; and correlations and corrections may need to be made as experience and additional data are accumulated. These factors very likely will not be directly transferrable from one area to another.

The greatest problems with collapsible soils arise when the existence and extent of the collapse potential are not recognized prior to construction because moisture - sensitive soils exhibiting collapse upon wetting. Therefore, the identification of collapsible soils and estimation of the collapse potential are major components in appropriate engineering for moisture - sensitive soil sites. Geologic and geomorphologic information can be also helpful in anticipating collapsible soil deposits. Geotechnical and geological engineers know from experience that alluvial and colluvial deposits in arid regions are likely to exhibit some collapse potential (Zapata et al, 2001). Alluvial fans in southwestern regions of the U.S should be assumed to be collapsible unless a comprehensive testing program demonstrates otherwise (Beckwith, 1995). Chinese researchers have found that geographical and geomorphologic information can be strongly correlated with collapsibility (Lin, 1995). Both qualitative and semi-quantitative correlations between collapse potential and various index properties have been developed and reported. Low initial density is of course a fair indicator of collapse potential, but some soils with moderately high density have also exhibited significant collapse. Another index property, which relates closely to density, is the water content at saturation. When the water content corresponding to full saturation significantly exceeds the liquid limit, substantial collapse potential is indicated. Correlations with seismic velocity, SPT N-values, and CPT tip resistance have also been attempted with low to moderate success. A disadvantage of all these correlations with index properties is that the correlations 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. Therefore, the most effective use of available site characterization funds is to perform actual collapse tests, either laboratory or in - situ. The advantage of these tests over other indirect correlations is that they provide not only identification data but also quantitative data for later assessment of settlements (Zapata et al, 2001).

Rafie et al. (2008) uses different correlations developed by Abelev criteria (1948), Feda criteria (1960), Denisov criteria (1964), Clevenger criteria (1985), and Lin and Wang criteria(1988) for the evaluation of collapsibility potential of problematic soil under case study on railway station. As these various soil collapsibility evaluation criterion offer different judgment as compared to Jennings and Knight (1975) collapse scale values; therefore, the complementary laboratory tests such as pinhole or double oedometer tests are recommended for correct recognition of problem. However, after reviewing the obtained results it seems that the most recent criterion of the Lin and Wang, the collapse problem could be regarded as one of the jobsite soil challenges.

Khelifa et al. (2010) studied the prediction of collapsible soils by CPT and Ultrasonic tests. The results obtained clearly show the influence of certain parameters such as kaolin content, water content and energy of compaction on the collapse potential, the limit penetration and the ultrasonic speed. The collapse potential can be excessive if the initial water content is low. For water content lower than the OMC of compaction test, there exists the energy of compaction beyond which collapse does not occur. Also, the possibility of using the CPT as identification means of the collapsible soils makes it possible to follow the evaluation of collapse and to propose a limit penetration, separating the collapsible soils from the non - collapsible soils. A new experimental approach to the prediction of collapsible soils based on ultrasonic tests, easy and fast, is proposed, and the results obtained depend on grain - size distribution, compactness of soil and water content. Ultrasonic speeds are limited as the speed less than or equal to 400 m/s indicates the collapse may appears, and the speed greater than 1000 m/s has no risk of the soil to collapse.

Gildenhuys (2010) studied to determine the occurrence and extent of collapse settlement as well as the shear behavior of the in residual granite soils from double odometer and shear strength tests. The Atterberg limits and particle size distributions of all the soil samples were determined according to the ASTM method, to describe and identify the soils. Direct shear tests are carried out to determine the consolidated - drained shear strength of a soil. The tests were performed on the samples collected from the area of research to determine the shear strength parameters of the soils as well as the effect of shear strength on collapsibility. It was further performed to determine the volume change of the soil samples during shear. Out of fourteen soil samples collected from the demarcated study area, ten were found to be collapsible at various degree of severity. Five of the ten soils that showed collapse have low dry densities. These soils conform in terms of the definition of a collapsible soil. The other five soils that illustrated collapse have high dry densities. Collapse settlement in these soils was not expected and contradicts typical collapse behavior. Of the four soils that did not collapse, one has a high dry density and three have low dry densities. The three soils with the low dry densities were expected to collapse. Jennings and Knight (1975) warn of the danger of assuming that all soils with a low dry density will show collapse or, vice versa that all soils with a high dry density will not collapse. However, soils with a collapsible fabric very often have a low dry density, and thus the behavior of some of the soils is unusual. In view of the identified typical collapse behavior, there may be factors other than dry density that may influence collapse settlement.

While, Gildenhuys (2010) studied the effect of collapsibility on the shear behavior of the soils, he stated the following findings:

- The presence of a collapsible fabric does not always result in an immediate decrease of shear strength if saturation occurs.
- Shear strength testing does not provide an adequate method for the determination of a collapsible soil.
- Soils can be very unpredictable and therefore one can very rarely make assumptions about the behavior of a soil without doing the necessary testing.

Howayek et al. (2011) stated collapse potential is an indication of the degree of bulk volume change soils exhibit due to the combined effects of load and water infiltration. It is generally expressed as the volumetric strain associated with wetting. There are a variety of approaches to measure the collapse potential of soils including laboratory methods and field methods. The most common way is to conduct laboratory tests using the oedometer apparatus. The main advantage of this approach is that three most important factors which affect collapse potential: degree of saturation, dry density, and overburden stress, can be controlled and measured. Typically results of oedometer tests are used for one - dimensional analysis. Similar measurements can be conducted in the triaxial apparatus, although it has been shown that little additional information on collapse is gained from these tests (Lawton, 1989). Two types of oedometer tests can be employed to determine collapse potential: the single oedometer test and the double oedometer test. In the single oedometer test a soil specimen is placed in the oedometer, and the desired overburden stress is gradually applied in increments until strain equilibrium is achieved. The soil specimen is then flushed with water under the applied stress. The collapse strain measured after water infiltration is termed collapse potential. ASTM - D5333 describes the procedure for the single oedometer test. It also classifies the degree of specimen collapse based on the collapse index Ie, the wetting induced strain under a surcharge of 200 kPa. The difference in the deformations measured from the two tests is the collapse due to wetting at any given stress level. 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. Again, various empirical correlation criterions for judging the collapse potential of a soil (as shown in Table 2.1 below) examine the applicability of a few of these criteria to the data collected for the soil of the study. In general, it is found that none of the criteria examined are able to fully capture the behavior of these soils. For example, the criteria proposed by Clevenger (1985) are based on fixed threshold values of the natural dry density, with no consideration of soil type or water content variation.

Ayadat et al. (2012) stated various collapse criteria like methods based on voids ratio, density and water content relationship; methods based on water content and Atterberg limits relationship; methods based on density and Atterberg limits; and methods based on particle size distribution of soils - where the first three categories do not take into account the influence of soil particle distribution. Finally, the researcher adapted to develop other criteria of collapse evaluation criteria from Markin and Feda (1966) that was rearranged to include only values for the bulk unit weight and the unit weight of soil. This expression can be presented on a chart of bulk unit weight against unit weight of soil constituents, and developed a standard chart that is used to predict the susceptibility of soil to collapse directly by knowing its bulk and soil constituents unit weight.

Noutash (2013) studied the use of LL and dry density test method for Collapsible Soil identification. He used various criteria developed by different researchers for the evaluation of soil collapse potential like Ableve (1948), Feda (1960), Denisov (1964), Lin and Wang (1988), Gibbs and Bara (1962), and Clevenger (1985).

2.6.1 Collapse Measurement using Experimental Criteria

For the determination of index properties of soils, there are different laboratory tests to be conducted at laboratory level. However; for the purpose of this study, the following tests were conducted at laboratory level for the measurement of collapse potential of the soil.

Water content determination test - This test is performed to determine the water (moisture) content of soils. For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil (Reddy, 2002).

Density (unit weight) determination test - This test is performed to determine the in-place density of undisturbed soil (Reddy, 2002).

Specific gravity determination test - The specific gravity materials of tells how much the material is heavier or lighter than distilled water. This test is performed to determine the specific gravity of soil by using a pycnometer (Cheng Liu, 1998).

Grain size analysis - It provides the grain size distribution, and it is required in classifying the soil (Reddy, 2002).

Atterberg limits determination tests - These tests are is performed to determine the plastic and liquid limits of a fine grained soil (Reddy, 2002).

Moisture – density relation test - Most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density. The optimum water content is the water content that results in the greatest density for a specified compactive effort (Reddy, 2002).

Table 2.1: Some existing criteria for collapse potential of soils **(**Ayadat et.al (2011), Howayek et.al (2011) and Briaud (2013))

Reference	Expression	Collapse criteria	Year
Clevenger	$\rho d < 1.28 \frac{g}{cm3}$, $\rho d > 1.44 \frac{g}{cm3}$ and $1.28 \frac{g}{cm3} \le \rho d \le 1.44 \frac{g}{cm3}$	• If $\rho d < 1.28 \frac{g}{cm3}$ then the soil will have large collapse settlement after minor water content change. • If $\rho d > 1.44 \frac{g}{cm3}$ then the small settlement and lesser collapse settlement could be expected. For medium range of soil density. • If $1.28 \frac{g}{cm3} \le \rho d \le 1.44 \frac{g}{cm3}$ then the medium collapse settlement could be evaluated.	1985
Handy	Clay contents (< 0.002mm)	• <16%, High probability of collapse • 16 – 24%, Probability of collapse • 24 – 32%, Less than 50 percent probability of collapse	1973
Feda	$Cp = \frac{\left[\left(\frac{\omega o}{S}\right) - PL\right]}{[LL - PL]}$ and $\eta o > 40\%$	• If $C_p > 0.85$, Soil to collapse. • If $\eta_o > 40\%$, Soil is susceptible to collapse.	1966

Reference	Expression	Collapse criteria	Year
Denisov	$Cp = \frac{eLL}{eo}$	• If C_p = 0.5 - 0.75, highly collapsing soils. • If C_p =1.0, non collapsible loams. • If C_p =1.5 - 2.0, non collapsible soils.	1964
Gibbs and Bara	$\gamma d = \frac{25.5}{(1+0.026LL)}$ or $eo > \frac{2.6LL}{100}$	• If the values of γ_d is less than or e_o is greater than the right side empirical equation, collapse will occur.	1962
Priklonski	$Cp = \frac{(\omega LL-\omega o)}{(LL-PL)}$	• If $C_p < 0$, highly collapsible soils. • If $C_p > 0.5$, non-collapsing soils. • If $C_p > 1.0$, swelling soils.	1952
Abelev	$Cp = \frac{\Delta e}{(1 + eo)} \times 100$	• $C_p > 2$, collapsible soils • $C_p < 2$, not collapsible soils	1948
Czechoslovak Standard	Percentage of fines, porosity, degree of saturation, liquid limit and natural moisture content	Collapse may occur when: • Silt < 60% • Clay < 15% • S < 60% and LL < 32% • $\eta > 40$ % • Natural moisture, $\omega_o < 13\%$	N/A

Where, C_p = collapse potential of the soil; e_o= natural void ratio; e_{LL} = void ratio at liquid limit state; Δe = void ratio reduction during soil saturation; ω_o = natural moisture content (%); ω_{LL} = moisture content at liquid limit state (%); η = natural porosity; ρ_d = natural dry density (g/cm^3); γ_d = natural dry density of soil (kN/m^3); S = natural degree of saturation; LL = liquid limit (%); PL = plastic limit (%).

The magnitude of soil collapsibility usually depends on initial porosity. The basic characteristics of collapsible soils are categorized as: high porosity (more than 40%), low saturation (less than 60%), high silt content (more than 30% and sometimes 90%), and rapid softening in the water (Rafie et.al, 2008).

In order to classify the collapsibility of soils, a variety of criteria for collapse potential have been proposed. Most of these criteria determine the critical condition of collapse based on void ratio, dry unit weight, Atterberg limits, natural water content, percentage of fine grain soils, and degree of saturation.

2.6.2 Collapse Measurement from Consolidation Test

In view of the wide range of soils which exhibit collapse properties it is obvious that the following tests are helpful in the identification of potentially collapsible soils and possibly the depth to which these soils occur in the soil profile (Schwartz, 1985).

Double oedometer test - The double oedometer test involves preparing two specimens, which are hopefully identical, for testing in the oedometer. However, this study didn't consider applying this collapse test method for collapse measurement as it is very difficult to find two identical undisturbed samples at the same time.

Single oedometer test - It is called a single specimen test because one specimen is used to get the wetted curve (Houston, 1995). Considering the difficulties associated with the interpretation of the double oedometer test, it would appear fitting to consider using a method which would require the testing of only one undisturbed sample. The sample is loaded at natural moisture content to the expected stress from the structure and then soaked. The consolidation at natural moisture content and the additional settlement due to collapse could then be calculated (Byrne et al., 1995).

An advantage of the test is that an attempt is being made to trail the loading and moisture content paths to which the soil will be subjected in the field. However, an over-prediction of settlement will result from this method seeing that no correction can be made for the regeneration of lateral stresses in the consolidometer while the soil is saturated (Schwartz, 1985).

Single oedometer test for collapse test – This test used the same testing apparatus with single oedometer test. It uses both saturated and unsaturated states for a single soil specimen. It is performed with the sample confined in a consolidometer ring. Typically, it consists of loading the soil sample to a vertical stress equal to the vertical total stress that the soil will experience at a chosen depth, recording the vertical strain versus time curve. Once the compression is completed, inundate the sample while continuing to record the vertical strain versus time curve. Once also the collapse is completed, the consolidation test can be resumed by increasing the vertical stress (Briaud, 2013).

The collapse potential test is a special case of the single consolidometer test in which the sample is saturated at a load of 200 kPa (Schwartz, 1985). According to Jennings and Knight (1975) the collapse potential is not a design parameter, but is an index figure providing a guide to the collapse situation and good reason for further investigation.

The following figure 2.1 illustrates the typical stress - strain graph of collapsible soils from single oedometer for collapse test (Schwartz, 1985).

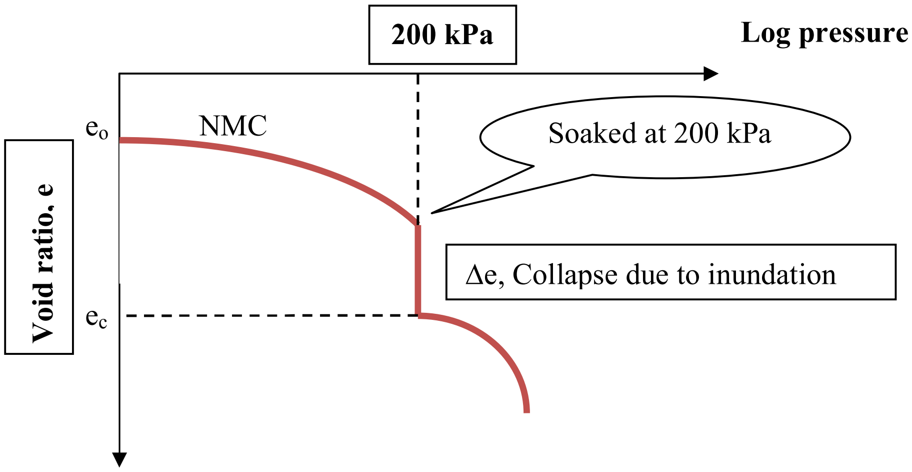


Figure 2.1: Typical single oedometer for collapse test result (Schwartz, 1985)

Once it is recognized that a soil may be collapsible, the extent of the collapse and its severity can be gauged by the scale proposed by Jennings and Knight (1975). Their scale is based on the collapse potential index C_P:

$$Cp = \frac{eo - ec}{(1 + eo)} \times 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; (1)$$

Where e_o is the void ratio of the soil at its natural water content under 200 kPa of vertical pressure in the consolidation test before wetting and e_c is the void ratio after soaking under 200 kPa of vertical pressure.

Table 2.2: Severity of collapsible soils scale (Jennings and Knight, 1975)

Severity of the problem	Collapse Potential, C_p, %
No Collapsibility	0 to 1
Medium Collapsibility	1 to 5
High Collapsibility	5 to 10
Very High Collapsibility	10 to 20
Extremely Collapsible	>20

(Source: Rafie et.al, 2008)

According to Schwartz (1985); “In view of the difficulties associated with the interpretation of the double oedometer test it would appear convenient to consider using a method which would require the testing of only one undisturbed sample. The sample could then be loaded at natural moisture content to the anticipated stress that will be imposed by the structure and then saturated. The consolidation at natural moisture content and the additional settlement due to collapse could both then be calculated. The test has the advantage that an attempt is being made to follow the loading and moisture content paths to which the soil will be subjected in the field. Knight indicates however that an over-prediction of settlement will result from this method since no correction can be made for the regeneration of lateral stresses in the consolidometer when the soil is saturated”.

According to ASTM D5333, collapse is decrease in height of a confined soil following wetting at a constant applied vertical stress. A collapsible soil may withstand relatively large applied vertical stress with small settlement while at a low water content, but this soil will exhibit settlement (that could be large) after wetting with no additional increase in stress. Large applied vertical stress is not necessary for collapse.

According to Youventharan et.al (2010), the best way to determine the amount of collapse that may occur is to perform a consolidation test (ASTM D5333) and simulate what would happen to the soil in the field. For this, the sample at its natural water content is placed in the consolidometer, the sample height is recorded, and the vertical pressure is increased in steps. For each step, the change in height of the sample is recorded every 30 minutes and the curve of stress vs. strain is plotted. Each pressure is kept on the sample until the rate of strain is less than 0.1 percent per an hour. When the vertical pressure reaches the pressure anticipated in the field and at the end of that load step, the sample is inundated and the readings of strain continue during the collapse as a function of time. The end of the collapse step is when the strain has become less than 0.1 percent per an hour. The next pressure step is applied, and so on, until the curve is completed (Briaud, 2013).

Collapse potential (I_c) is percent relative magnitude of soil collapse determined at any stress level. For the test is conducted as a one-dimensional test as follows:

$$Ic = \frac{\Delta h}{ho} \times 100\text{ , or }Ic = \frac{(df - di)}{ho} \times 100\text{ , or }Ic = \frac{\Delta e}{(1 + eo)} \times 100 \text{ ………………...………….. (2)}$$

Where, Δh = change in specimen height resulting from wetting; h_o = initial specimen height; d_f = dial reading at the appropriate stress level after wetting, mm; d_i = dial reading at the appropriate stress level before wetting, mm Δe = change in void ratio resulting from wetting; and e_o = initial void ratio.

Collapse index (I_e) is percent relative magnitude of collapse determined at 200 kPa of applied stress level. The test method consists of placing a 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 fluid shall be distilled and deionized water when evaluating the collapse index, I_e. The fluid may simulate pore water of the specimen or other field condition as necessary when evaluating collapse potential, I_c.

According to ASTM D5333, procedures for estimating potential for collapse are uncertain because no single criterion can be applied to all collapsible soils. For example, some soils may swell after fluid is added to the specimen until sufficient vertical stress has been applied. Collapse may then occur after additional vertical stress is applied. This test method may be used to determine the collapse potential, I_c, of soil at a particular vertical stress or the collapse index, I_e, at an applied vertical stress of 200 kPa. I_c for smaller applied vertical stress may be estimated assuming that the soil does not swell after inundation at smaller applied vertical stress.

Table 2.3: Classification of collapse index, I_e (ASTM: D 5333)

Degree of specimen collapse	Collapse index, I_e, %
None	0
Slight	0.1 to 2.0
Moderate	2.1 to 6.0
Moderate severe	6.1 to 10.0
Severe	>10

In general; different researchers have done various studies on the characteristics, identification, and prediction of collapsibility potential of collapsible soils at different countries with different project scopes.

Here the author enumerated the research gap from the literature review that most of the aforementioned researchers were used either the existing experimental correlations or double oedometer tests for the identification of collapsibility potential of soils with respect to Jennings and Knight (1975) collapse scale values, not both at once. Hence; in this study, the author carried out the identification of collapsibility potential values of soils at the area of study using both the existing empirical correlations (shown in Table 2.1) and direct measurement of collapse potential of soils from single oedometer for collapse test (ASTM D5333). Once the collapse values determined, the severity or degree of collapsibility potential were determined from the collapse scale values given by both Jennings and Knight (1975) and ASTM - D5333.

CHAPTER 3: MATERIALS AND METHODS

3.1 Description of the Area of Study

The study area is named as Ambure village and located around 5.0 kilometers away from Shashemene town, West Arsi zone, Ethiopia. According to Geological Survey of Ethiopia report; the Shashemene area is located on the eastern edge of the Main Ethiopian Rift in southern Ethiopia, some 190 km south of the capital Addis Ababa by air and 250 km by road. The area is located also at latitude 7°N on the south and 7.25°N on the north and by longitudes 38.5°E on the west and 38.75°E on the east and 1,924 meters above sea level.

The topography of the area of study lies between flat to rolling terrain conditions and more susceptible to heavy surface runoff and soil erosion. The topography of the area make more suitable for mechanized agricultural works and well-known for products like teff, sorghum, wheat, corn, potatoes etc.

3.2 Research Methodology and Sampling Design

3.2.1 Description of the Proposed Study Design

This section presents the methods employed for the experimental program and summarizes soil sample locations and sampling procedures, and relevant information for the soils used for the tests including water content, density/ unit weight, particle size analyses, specific gravity, Atterberg limits, compaction test, and one-dimensional consolidation for collapse test.

The flow chart (shown in figure 3.1) describes methodological approach for the overall sequences of activities to the identification of collapsibility potential of problematic soils for the area of study.

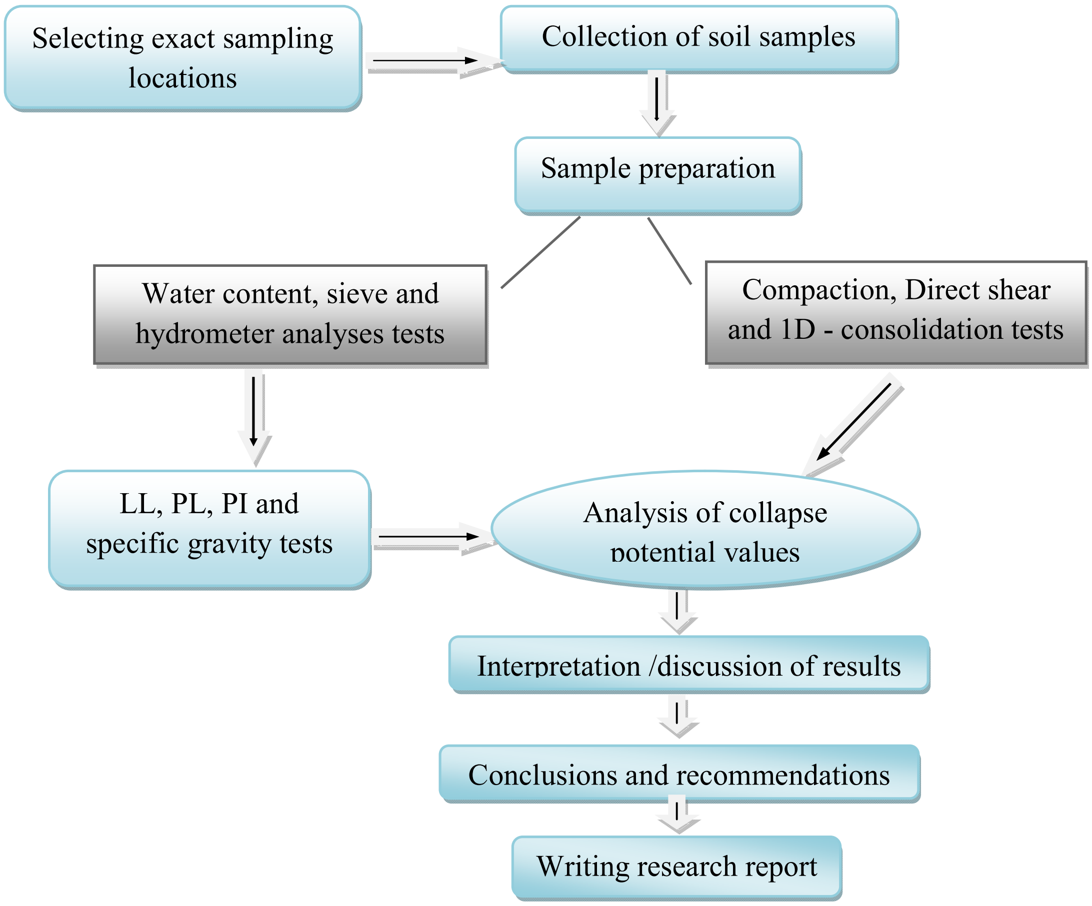


Figure 3.1: Flow chart of methodological approach for the assessment of collapsibility potential analysis of problematic soils.

3.2.2 Sample Site Selection and Sample Types

Both disturbed and undisturbed soil samples were collected from either sides of the road at the randomly identified and selected eight different locations of soil layers, i.e. four samples from either side of the soil strata. A truly undisturbed sample is nearly impossible and as such "undisturbed sample" refers to a sample where some precautions are taken to minimize disturbance or remolding effects. Hence, the researcher imposed the best efforts to collect undisturbed samples for density and collapse measurement tests. Disturbed soil samples were also collected and used for direct shear and classification tests.

3.2.3 Methods of Soil Samples Collection

As mentioned in section 1.6, soil samples were collected from soil strata where major crack exactly occurred through a combination of judgmental non - random and systematic random sampling method at the selected 8 (eight) sampling locations. Four samples at either sides of the road were collected; and one sample at every 2.0 meters starting 0.5 meters from ground surface using manual workers and hand tools. The reason that the number of samples were limited to 8 (eight) is due to the reinstatement of gorge and only remain about 7.0 meters, and hence the author can't go further depth to collect extra samples. The direction of pit excavation for the soil sampling was parallel to the ground surface (perpendicular to the soil profile) within the stated depth, based on the soil stratification very likely for sampling. Hence, disturbed samples were collected from at all sample locations where each sample pits by buckets within 0.5 to 6.5 meters from the ground surface by manual workers and hand tools.

Figure 3.2: Collection of disturbed sample from RHS (March 14, 2018)

However; for density and collapse measurement tests, undisturbed block samples were taken by a method of constructing a shelf, and excavating the sample by hand using a sharp knife, a polyvinyl chloride (PVC) pipes covered inside with an insulating thin plastic cover.

The PVC to provide confinement, permits undercutting of the sample, and ensure safe handling, transportation and storage of the soil samples.

Figure 3.3: Bundling of disturbed sample from LHS (March 14, 2018)

Figure 3.4: Undisturbed sample collection method from LHS (March 14, 2018)

Figure 3.5: Polyvinyl chloride (PVC) pipes to collect undisturbed samples (March 14, 2018)

3.2.4 Resources to be Used for Sample Preparation and Testing

Table 3.1 illustrates and summarizes the standard tests and procedures that were used all through this study under the designation code of AASHTO and ASTM material testing manuals while preparing soil specimens and conducting relevant laboratory tests.

Table 3.1: Summary of resources to be used for sample preparation and soil testing

Activity description	AASHTO designation code	ASTM designation code	Type of sample
Preparation of soil sample	T 87-86 (1996)		Disturbed and undisturbed
Moisture content test	T 265-93	D 2216	Disturbed
Specific gravity test	T 100-95	D 854	Disturbed
Density test		D 2937	Undisturbed

Activity description	AASHTO designation code	ASTM designation code	Type of sample
Measurement of collapse potential of soils from 1D consolidation	T 216	D 5333 & D 2435	Undisturbed
Particle size analysis test	T 88	D 422	Disturbed
Compaction test	T 180	D 1557	Disturbed
Atterberg limit (LL) test	T 89	D 4318	Disturbed
Atterberg limit (PL) test	T 90	D 4318	Disturbed
Direct shear test	T 236	D 3080	Disturbed

3.2.5 Methods of Data Analysis

The identification of collapsibility potential values of problematic soil at the area of study were using the existing empirical correlations as shown in Table 2.1 jointly with single oedometer test for collapse and compare the results. Once the collapse values have determined the severity or degree of collapsibility potential were determined from the range collapse values given by Jennings and Knight (1975) and ASTM - D5333 designation.

Method of analysis for the laboratory soil test results were done also using formulas provided in each test designation code of AASHTO and ASTM soil testing standard manuals based on the principles of geotechnical engineering. For better descriptions of parameters and interpretations, the plots of some tests were analyzed using Microsoft-excel as required.

CHAPTER 4: SUMMARY OF TEST RESULTS

This chapter summarizes the obtained soil sample test results of experimental works, both engineering and classification tests, which were carried out to reach the objectives of the study and planned to determine the collapsibility potential of problematic soils at the area of study. The discussions and interpretations of the obtained soil test results were stated in the next section.

Note that the following symbols were stands for the corresponding specimen's specific location and position at the area of study.

SR - 1: Soil sample from the right - hand side (RHS) of the road at 0.5 m below the surface.

SR - 2: Soil sample from the right - hand side (RHS) of the road at 2.5 m below the surface.

SR - 3: Soil sample from the right - hand side (RHS) of the road at 4.5 m below the surface.

SR - 4: Soil sample from the right - hand side (RHS) of the road at 6.5 m below the surface.

SL - 1: Soil sample from the left – hand side (LHS) of the road at 0.5 m below the surface.

SL - 2: Soil sample from the left – hand side (LHS) of the road at 2.5 m below the surface.

SL - 3: Soil sample from the left – hand side (LHS) of the road at 4.5 m below the surface.

SL - 4: Soil sample from the left – hand side (LHS) of the road at 6.5 m below the surface.

Soil tests of all the soil samples were conducted according to the ASTM and AASHTO method of standards. However, under this study all the classification tests except hydrometer analysis were conducted at HU–IoT soil test laboratory where as engineering and hydrometer analysis were conducted at AMU – IoT soil test laboratory. The author performed these laboratory tests to identify the collapsibility potential of problematic soils both empirically and direct measurement of soil at the area of study as illustrated and summarized below.

4.1 Classification Tests

In order to determine the characteristics of the soil samples, an extended range of classification tests were considered. These tests are named as classification tests as they are mostly used for soil classification purposes. Classification tests like moisture content, specific gravity, Atterberg limits, density/ unit weight, sieve and hydrometer analyses, and compaction tests were conducted under this study. Index properties of soils obtained from these classification tests were used for empirical correlations to determine the collapse potential of soils.

4.1.1 Natural Moisture Content, Specific Gravity, Dry Density/ Unit Weight and Atterberg's Limits

Table 4.1: Summary of results of natural moisture content, specific gravity, dry density/ unit weight and Atterberg limits of all soil samples

Sample code	Natural moisture content (ω %)	Specific gravity, G_s	Natural dry density, ρ_d (g/cm^3)	Natural dry unit weight, γ_d (kN/m^3)	PI	Remark
SL-1	6.48	2.46	1.25	12.22	NP	
SL-2	8.71	2.49	1.26	12.40	NP	
SL- 3	7.53	2.51	1.28	12.53	NP	
SL- 4	8.31	2.51	1.28	12.58	NP	
SR-1	9.26	2.49	1.25	12.24	NP	
SR-2	6.59	2.51	1.27	12.49	NP	
SR-3	9.37	2.52	1.29	12.63	NP	
SR-4	8. 50	2.59	1.32	12.97	NP	

4.1.2 Particle Size Analysis (Sieve and Hydrometer Analysis)

Table 4.2: Summary of results of average grain size of sample from LHS based on USCS (ASTM) classification system

Soil type	Average grain size, mm (ASTM)	Percentage (%)				Remark
		SL-1	SL-2	SL-3	SL-4	
- **Gravel**						
Coarse Gravel	75 to 19	1.29	1.21	0.69	1.23	
Fine Gravel	19 to 4.75	6.00	4.45	4.13	4.45	
- **Sand**						
Coarse Sand	4.75 to 2.0	3.50	3.25	3.54	3.44	
Medium Sand	2.0 to 0.425	16.11	24.11	29.45	22.75	
Fine Sand	0.425 to 0.075	50.51	47.52	45.53	48.36	
- Silt	0.075 to 0.002	22.34	19.24	16.47	19.56	
- Clay	< 0.002	0.27	0.23	0.19	0.23	

Table 4.3: Summary of results of average grain size of sample from LHS based on AASHTO classification system

Soil type	Average grain size, mm (AASHTO)	Percentage (%)				Remark
		SL-1	SL-2	SL-3	SL-4	
Gravel	19 to 2.0	9.50	7.70	7.67	7.89	
Sand	2.0 to 0.075	66.61	71.63	74.98	71.10	
Silt	0.075 to 0.002	22.34	19.24	16.47	19.56	
Clay	<0.002	0.27	0.23	0.19	0.23	

Table 4.4: Summary of soil classification of sample from LHS

Description	Expression	Values				Remark
		SL-1	SL-2	SL-3	SL-4	
Uniformity coefficient, C_u	$C_u=D_{60}/D_{30}$	1.89	2.60	3.33	2.50	USCS
Coefficient of gradation, C_c	$C_c=D_{30}^2/(D_{60}*D_{10})$	1.59	0.96	0.86	1.14	USCS
Plasticity	Non plastic	-				
Soil type based on USCS (ASTM) classification system	Silty sand (SM)					
Soil type based on AASHTO classification system	Fine sand (A- 3)					

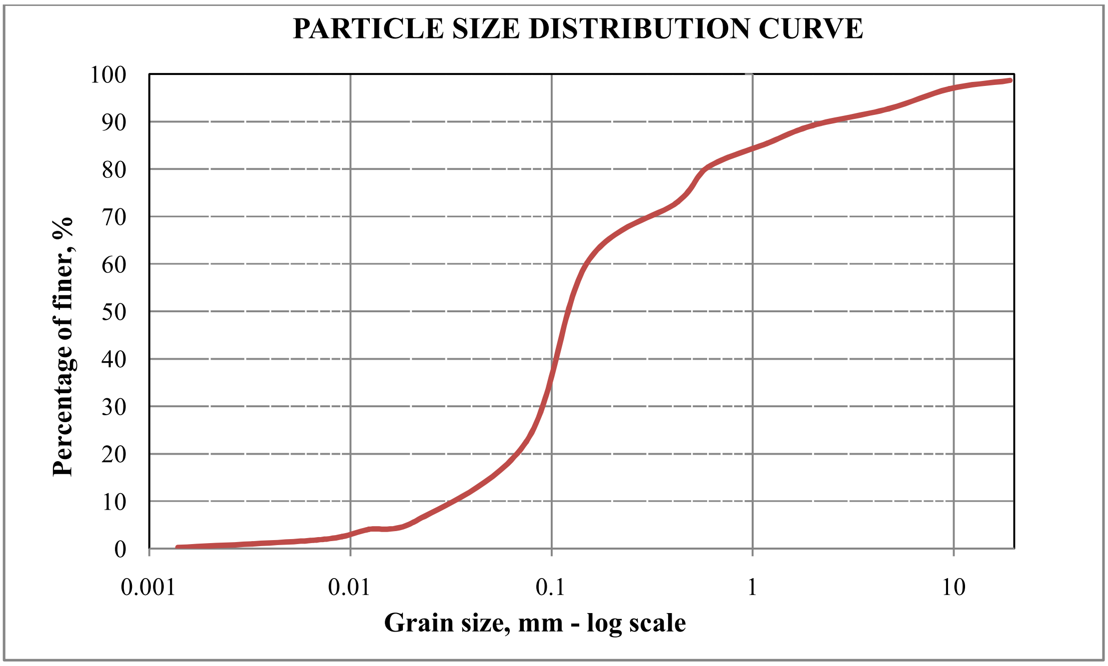


Figure 4.1: Particle size distribution curve for SL-1

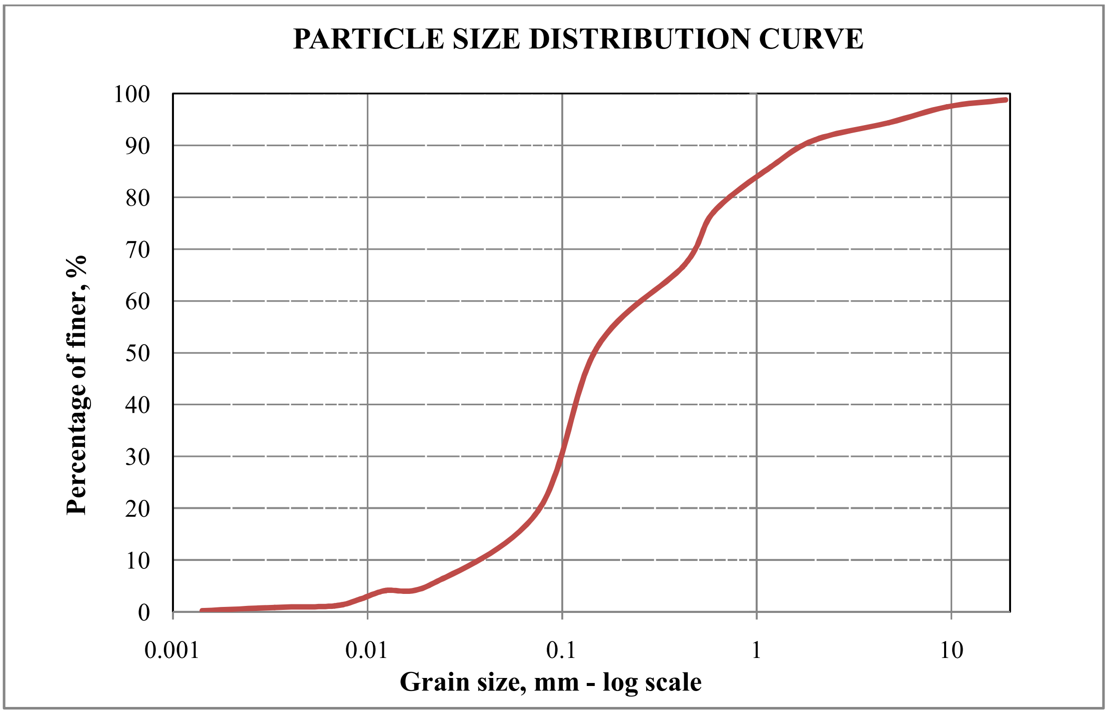


Figure 4.2: Particle size distribution curve for SL-2

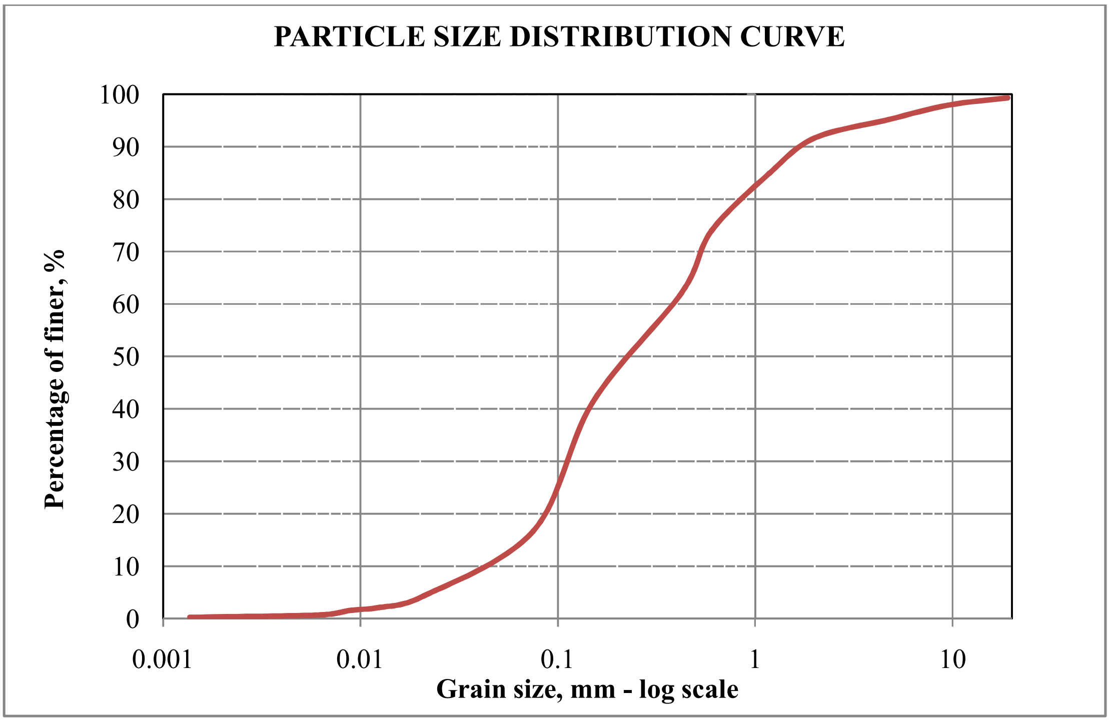


Figure 4.3: Particle size distribution curve for SL-3

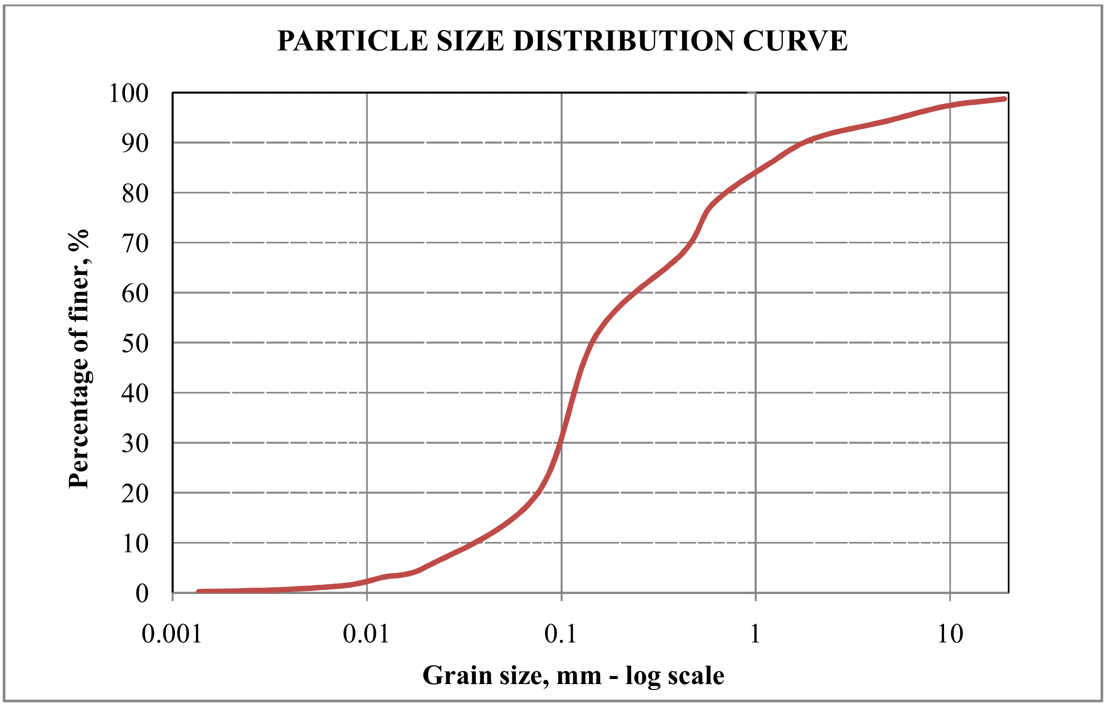


Figure 4.4: Particle size distribution curve for SL-4

Table 4.5: Summary of results of average grain size of sample from RHS based on USCS (ASTM) classification system

Soil type	Average grain size, mm (ASTM)	Percentage (%)				Remark
		SR-1	SR-2	SR-3	SR-4	
- **Gravel**						
Coarse Gravel	75 to 19	1.13	1.20	0.18	0.87	
Fine Gravel	19 to 4.75	2.90	4.09	3.80	3.49	
- **Sand**						
Coarse Sand	4.75 to 2.0	3.00	3.81	3.84	3.51	
Medium Sand	2.0 to 0.425	32.11	33.14	34.79	33.33	
Fine Sand	0.425 to 0.075	44.54	42.34	43.54	43.59	
- **Silt**	0.075 to 0.002	15.87	15.25	13.69	15.04	
- **Clay**	<0.002	0.46	0.18	0.16	0.17	

Table 4.6: Summary of results of average grain size of sample from RHS based on AASHTO classification system

Soil type	Average grain size, mm (AASHTO)	Percentage (%)				Remark
		SR-1	SR-2	SR-3	SR-4	
Gravel	19 to 2.0	5.90	7.90	7.64	7.00	
Sand	2.0 to 0.075	76.65	75.48	78.33	76.91	
Silt	0.075 to 0.002	15.87	15.25	13.69	15.04	
Clay	<0.002	0.46	0.18	0.16	0.17	

Table 4.7: Summary of soil classification of sample from RHS

Description	Expression	Values				Remark
		SR-1	SR-2	SR-3	SR-4	
Uniformity coefficient, C_u	$C_u=D_{60}/D_{30}$	3.33	4.00	2.19	3.46	USCS
Coefficient of gradation, C_c	$C_c=D_{30}^2/(D_{60}*D_{10})$	0.80	0.63	1.35	0.75	USCS
Plasticity	Non plastic	-	-	-	-	
Soil type based on USCS (ASTM) classification system	Silty sand (SM)					
Soil type based on AASHTO classification system	Fine sand (A- 3)					

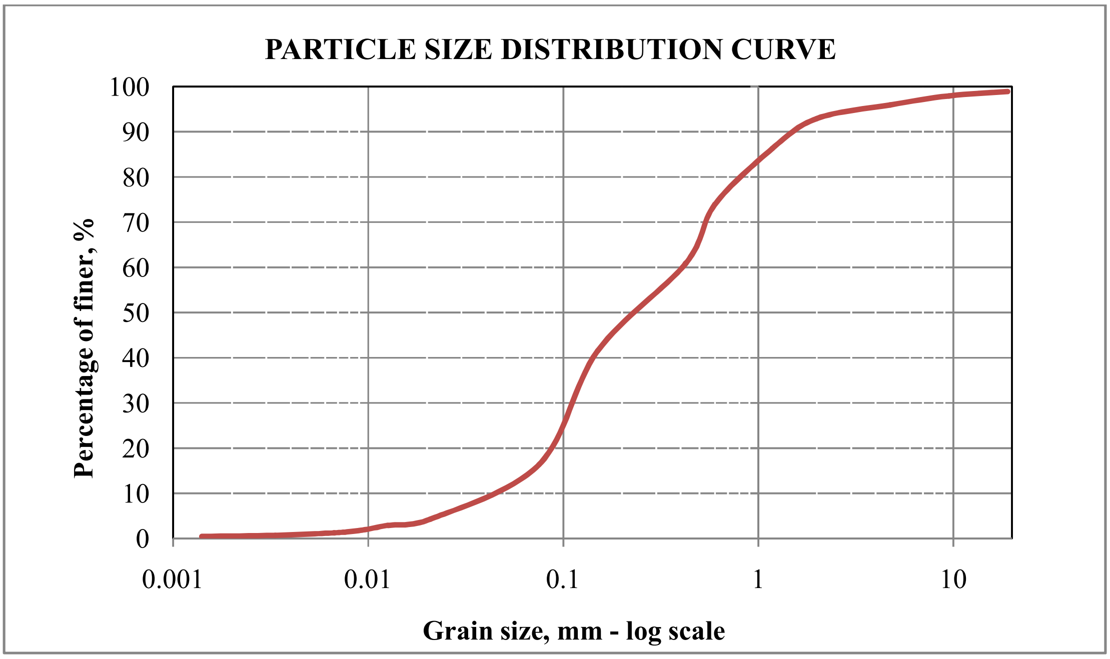


Figure 4.5: Particle size distribution curve for SR-1

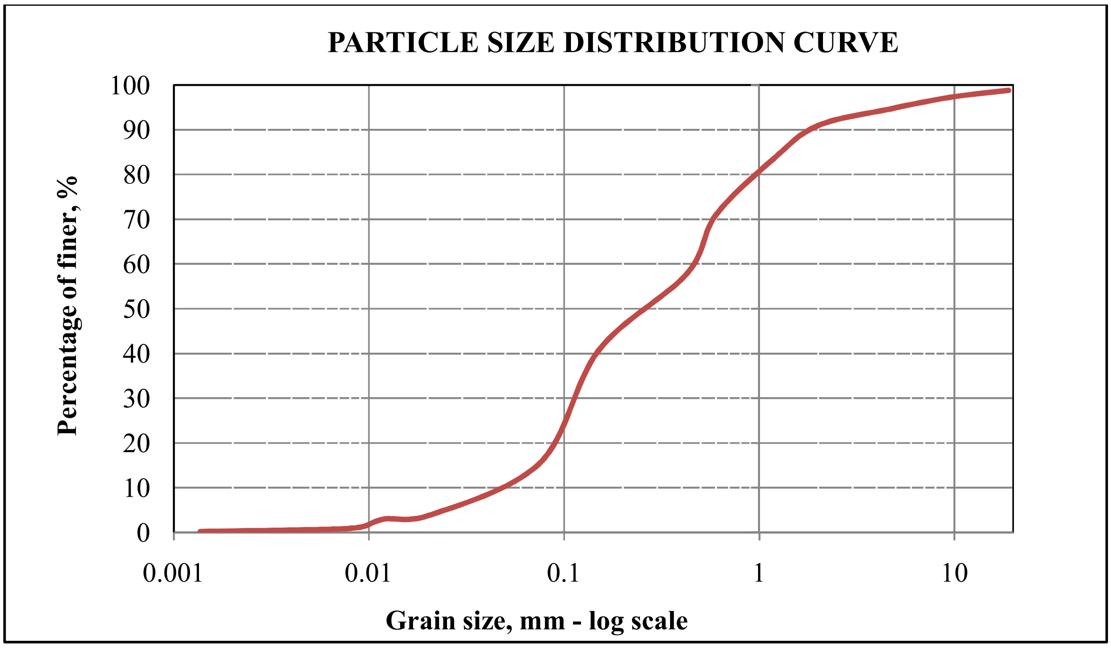


Figure 4.6: Particle size distribution curve for SR-2

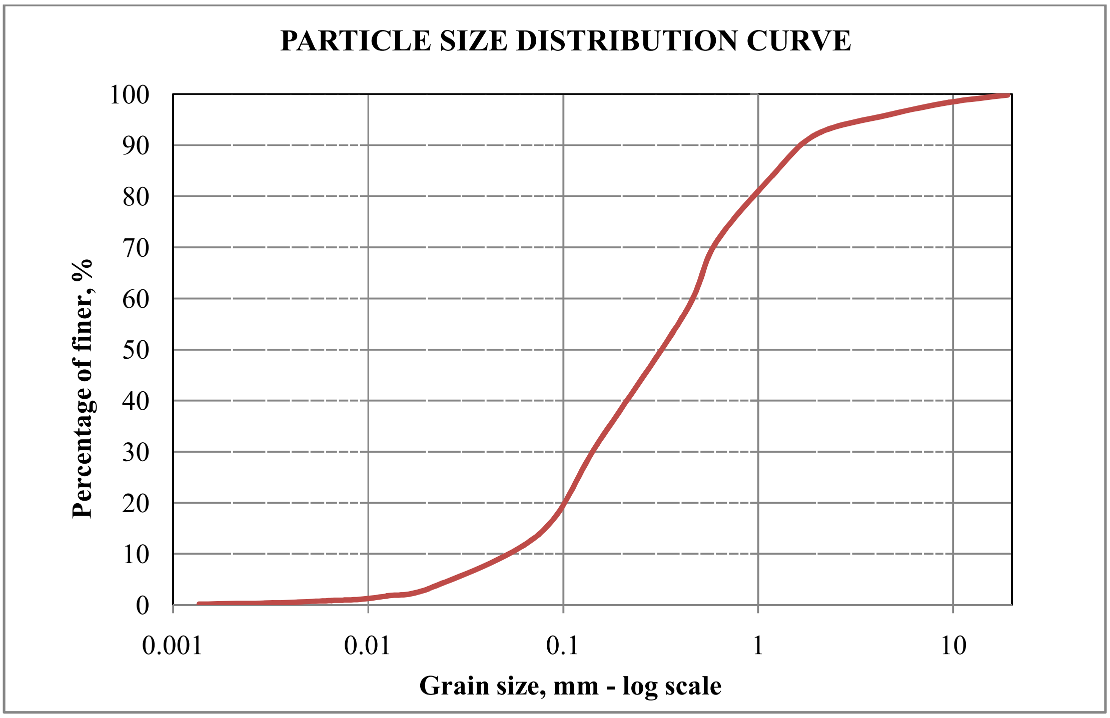


Figure 4.7: Particle size distribution curve for SR-3

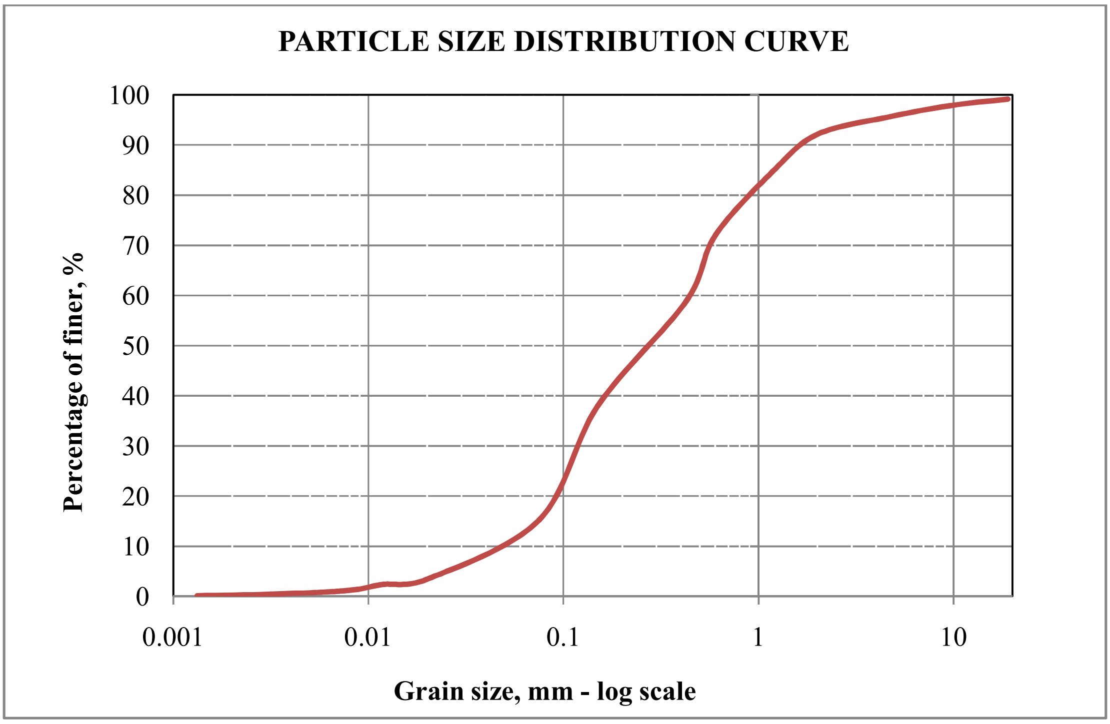


Figure 4.8: Particle size distribution curve for SR-4

4.1.3 Moisture – Density Relations

Table 4.8: OMC and MDD results from Modified Proctor Test

Sample code	OMC (%)	MDD (g/cm^3)	Remark
SL-1	21.00	1.40	
SL-2	21.00	1.38	
SL-3	22.90	1.34	
SL-4	22.80	1.39	
SR-1	27.20	1.36	
SR-2	28.00	1.33	
SR-3	30.00	1.34	
SR-4	25.50	1.35	

4.1.4 Void Ratio, Degree of Saturation and Porosity

Table 4.9: Summary of natural void ratio, degree of saturation and porosity results

Sample code	Natural void ratio, e_o	Natural degree of saturation, S (%)	Natural Porosity, η (%)	Remark
SL-1	0.98	19.00	49.40	
SL-2	0.97	23.26	49.16	
SL-3	0.97	23.64	49.13	
SL-4	0.96	22.77	48.94	
SR-1	0.99	24.52	49.82	
SR-2	0.97	18.05	49.31	
SR-3	0.96	20.44	49.02	
SR-4	0.96	21.94	48.96	

4.1.5 Summary of Empirical Method of Identification of Collapse Potential of Soil from its Index Properties

Table 4.10: Summary of empirical method of identification of collapse potential of SL-1

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SL-1	Clevenger (1985)	1.25 g/cm^3	Collapsible soil	High	ρ_d < 1.28 g/cm^3
	Handy (1973)	0.27 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.40 %	Collapsible soil	Not stated	η > 40% and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	4.71 %	Collapsible soil	High	C_p > 2%
	Czechoslovak Standard	Silt = 22.34 % Clay = 0.27 % S = 19 % LL = N/A η = 49.40 % ω_o = 6.48 %	Collapsible soil	Not stated	Silt < 60 % Clay < 15% S < 60 % LL < 32% η > 40 % ω_o < 13%

Table 4.11: Summary of empirical method of identification of collapse potential of SL-2

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SL-2	Clevenger (1985)	1.26 g/cm^3	Collapsible soil	High	ρ_d < 1.28 g/cm^3
	Handy (1973)	0.23 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.16 %	Collapsible soil	Not stated	η > 40% and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	5.56 %	Collapsible soil	High	C_p > 2%
	Czechoslovak Standard	Silt = 19.24 % Clay = 0.23 % S = 23.26 % LL = N/A η = 49.16 % ω_o = 8.71 %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% η > 40 % ω_o < 13%

Table 4.12: Summary of empirical method of identification of collapse potential of SL-3

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SL-3	Clevenger (1985)	1.28 g/cm^3	Collapsible soil	Medium	$1.28 \leq \rho_d \leq 1.44$ g/cm^3
	Handy (1973)	0.19 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.13 %	Collapsible soil	Not stated	$\eta > 40\%$ and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	5.64 %	Collapsible soil	High	$C_p > 2\%$
	Czechoslovak Standard	Silt = 16.47 % Clay = 0.19 % S = 23.64 % LL = N/A $\eta = 49.13$ % $\omega_o = 7.53$ %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% $\eta > 40$ % $\omega_o < 13\%$

Table 4.13: Summary of empirical method of identification of collapse potential of SL-4

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SL-4	Clevenger (1985)	1.28 g/cm^3	Collapsible soil	Medium	$1.28 \le \rho_d \le 1.44$ g/cm^3
	Handy (1973)	0.23 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	48.94 %	Collapsible soil	Not stated	η > 40% and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	5.78 %	Collapsible soil	High	C_p > 2%
	Czechoslovak Standard	Silt = 19.56 % Clay = 0.23 % S = 22.77 % LL = N/A η = 48.94 % ω_o = 8.31 %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% η > 40 % ω_o < 13%

Table 4.14: Summary of empirical method of identification of collapse potential of SR-1

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SR-1	Clevenger (1985)	1.25 g/cm^3	Collapsible soil	High	ρ_d < 1.28 g/cm^3
	Handy (1973)	0.46 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.82 %	Collapsible soil	Not stated	η > 40% and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	4.87 %	Collapsible soil	High	C_p > 2%
	Czechoslovak Standard	Silt = 15.87 % Clay = 0.46 % S = 24.52 % LL = N/A η = 49.82 % ω_o = 9.26 %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% η > 40 % ω_o < 13%

Table 4.15: Summary of empirical method of identification of collapse potential of SR-2

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SR-2	Clevenger (1985)	1.27 g/cm^3	Collapsible soil	High	ρ_d < 1.28 g/cm^3
	Handy (1973)	0.18 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.31 %	Collapsible soil	Not stated	η > 40% and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	4.70 %	Collapsible soil	High	C_p > 2%
	Czechoslovak Standard	Silt = 15.25 % Clay = 0.18 % S = 18.05 % LL = N/A η = 49.31 % ω_o = 6.59 %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% η > 40 % ω_o < 13%

Table 4.16: Summary of empirical method of identification of collapse potential of SR-3

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SR-3	Clevenger (1985)	1.29 g/cm^3	Collapsible soil	Medium	$1.28 \leq \rho_d \leq 1.44$ g/cm^3
	Handy (1973)	0.16 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	49.02 %	Collapsible soil	Not stated	$\eta > 40\%$ and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	5.41 %	Collapsible soil	High	$C_p > 2\%$
	Czechoslovak Standard	Silt = 13.69 % Clay = 0.16 % S = 20.44 % LL = N/A $\eta = 49.02$ % $\omega_o = 9.37$ %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% $\eta > 40$ % $\omega_o < 13\%$

Table 4.17: Summary of empirical method of identification of collapse potential of SR-4

Sample code	Reference	Collapse Index values	Conditions of soil collapsibility	Degree of collapsibility	Remark
SR-4	Clevenger (1985)	1.32 g/cm^3	Collapsible soil	Medium	$1.28 \leq \rho_d \leq 1.44$ g/cm^3
	Handy (1973)	0.17 %	Collapsible soil	High	Clay < 16%
	Feda (1966)	48.96 %	Collapsible soil	Not stated	$\eta > 40\%$ and C_p not applicable for NP.
	Denisov (1964)	-	Not applicable	-	NP
	Gibbs and Bara (1962)	-	Not applicable	-	NP
	Priklonski (1952)	-	Not applicable	-	NP
	Abelev (1948)	5.67%	Collapsible soil	High	$C_p > 2\%$
	Czechoslovak Standard	Silt = 15.04 % Clay = 0.17 % S = 21.94 % LL = N/A $\eta = 48.96$ % $\omega_o = 8.50$ %	Collapsible soil	Not stated	Silt < 60% Clay < 15% S < 60% LL < 32% $\eta > 40$ % $\omega_o < 13\%$

4.2 Engineering Tests

Both single oedometer for collapse test and direct shear test, which were conducted under this study, are categorized as engineering tests used for characterizing engineering behavior of soils.

Results from these tests used for direct measurement of collapse potential and characterize engineering behavior of soils.

4.2.1 Standard Method for Measurement of Collapse Potential of Soils

Table 4.18: Collapse index (C_p) computation from compression curve of sample from RHS of the road using Jennings and Knight (1975)

Description	Unit	Values				Remark
		SR-1	SR-2	SR-3	SR-4	
Initial void ratio at its natural water content before wetting, e_o		1.584	1.592	1.318	1.495	
Initial void ratio at its natural water content under 200 kPa of vertical pressure before wetting, e_i		1.339	1.341	1.094	1.255	
Final void ratio after soaking under 200 kPa of vertical pressure after wetting, e_f		1.213	1.229	1.003	1.160	
Collapse Potential, $C_p = 100^* \Delta e_c/(1+e_o)$	%	4.89	4.31	3.96	3.81	

Table 4.19: Collapse index (I_e) computation from compression curve of sample from RHS of the road ASTM - D5333

Description	Unit	Values				Remark
		SR-1	SR-2	SR-3	SR-4	
Initial specimen height, h_o	mm	20	20	20	20	
Dial reading at 200 kPa stress level before wetting, d_e	mm	1.894	1.944	1.928	1.924	
Dial reading at 200 kPa stress level after wetting, d_f	mm	2.872	2.806	2.72	2.686	
Collapse Index, $I_e = 100*(d_f - d_e)/h_o$	%	4.89	4.31	3.96	3.81	

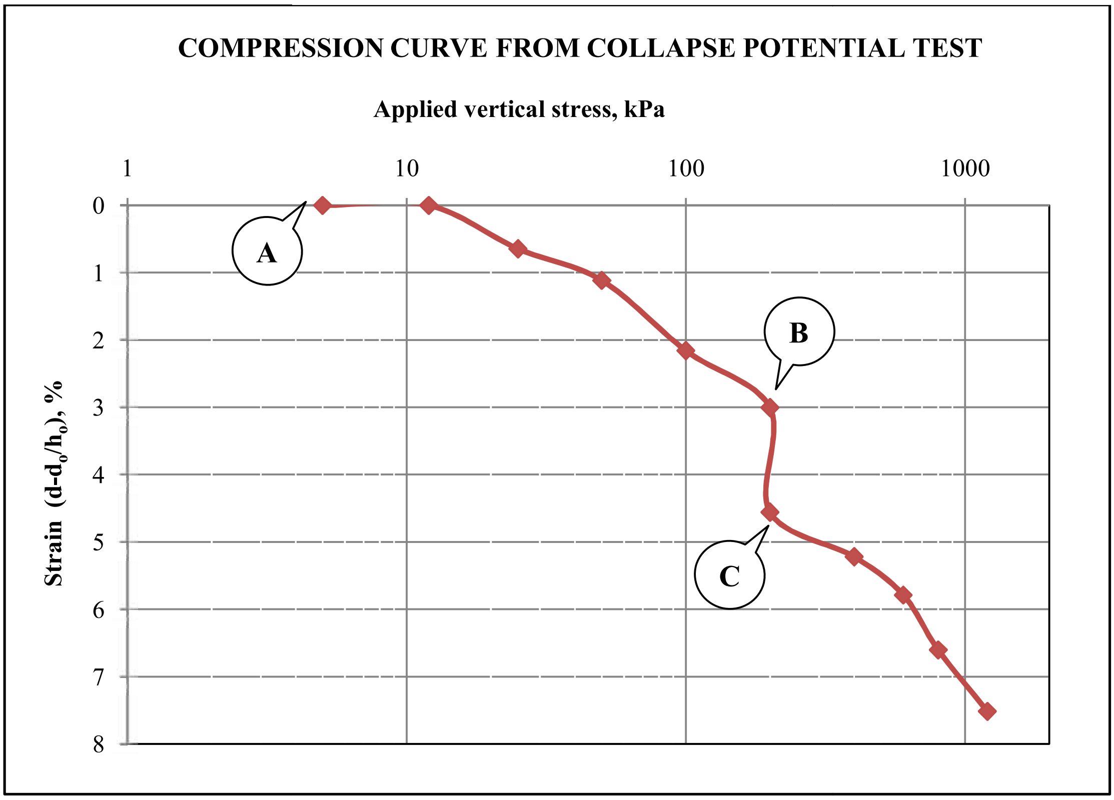


Figure 4.9: Compression curve of collapse test for SR-1 (ASTM - D 5333)

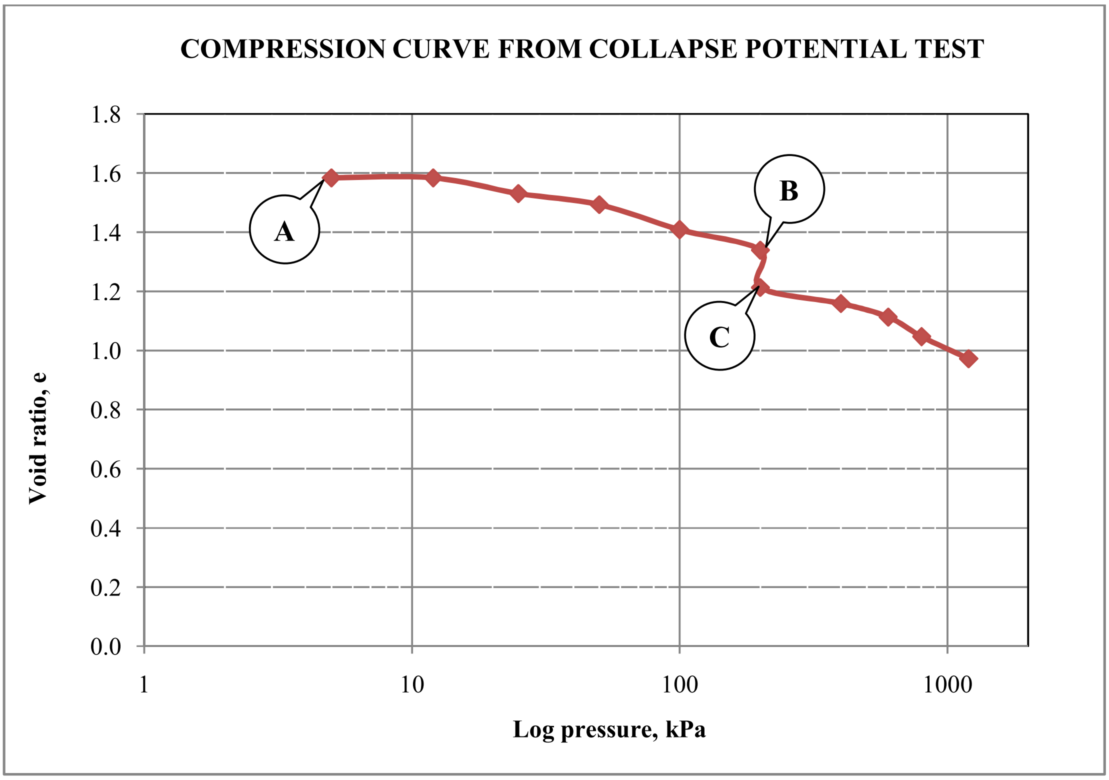


Figure 4.10: Compression curve of collapse test for SR-1 (Schwartz, 1985)

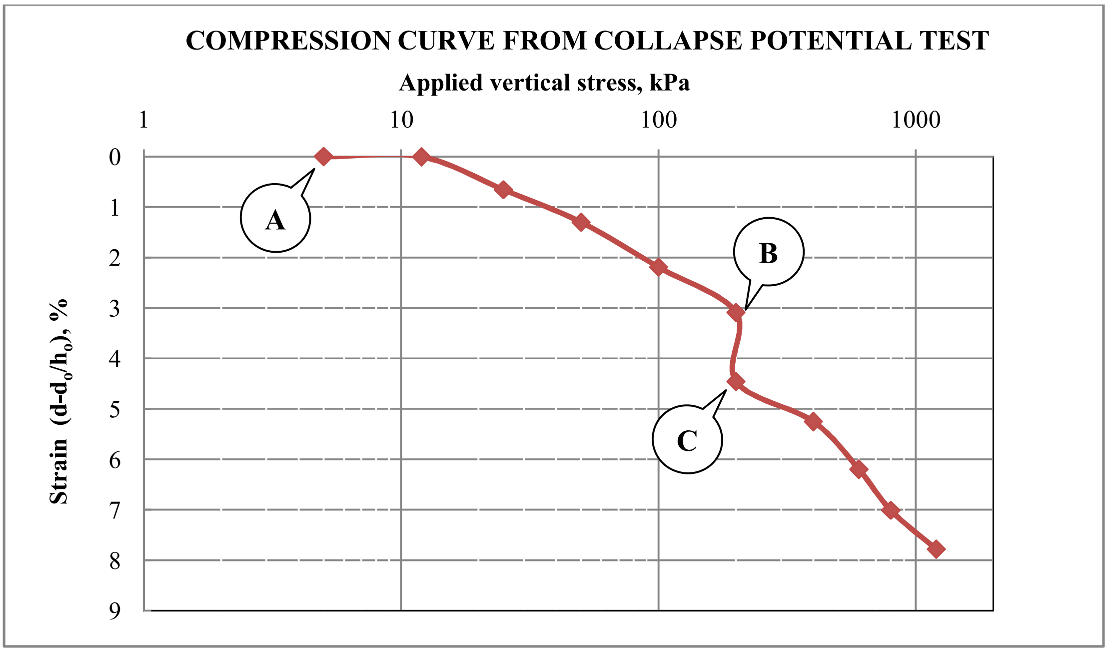


Figure 4.11: Compression curve of collapse test for SR-2 (ASTM - D 5333)

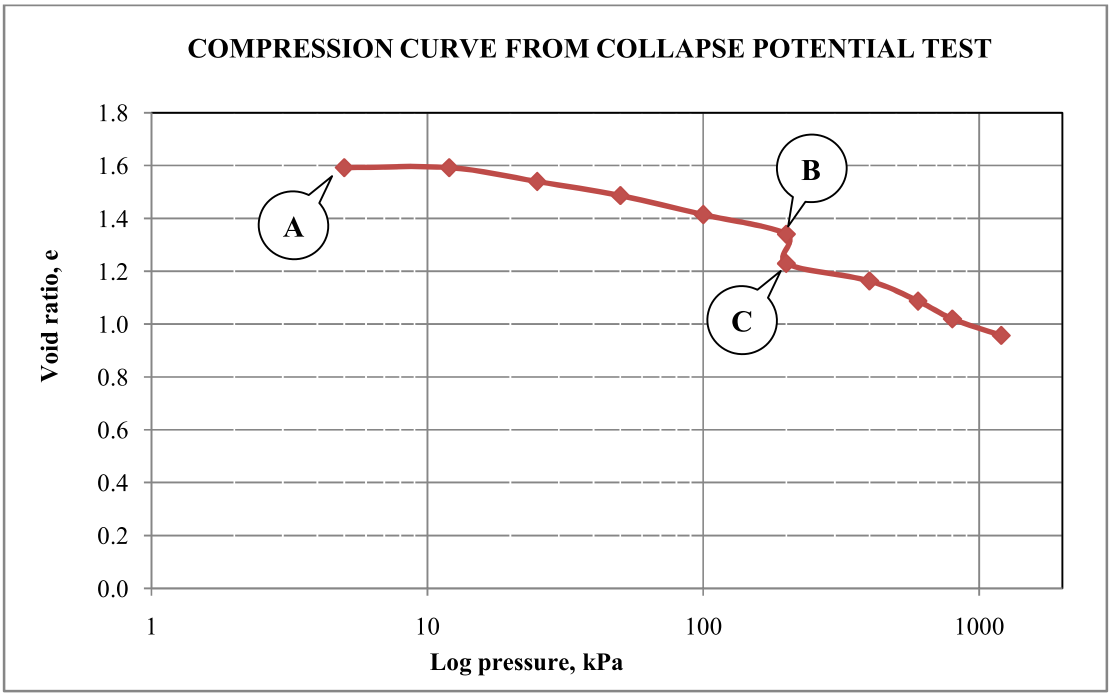


Figure 4.12: Compression curve of collapse test for SR-2 (Schwartz, 1985)

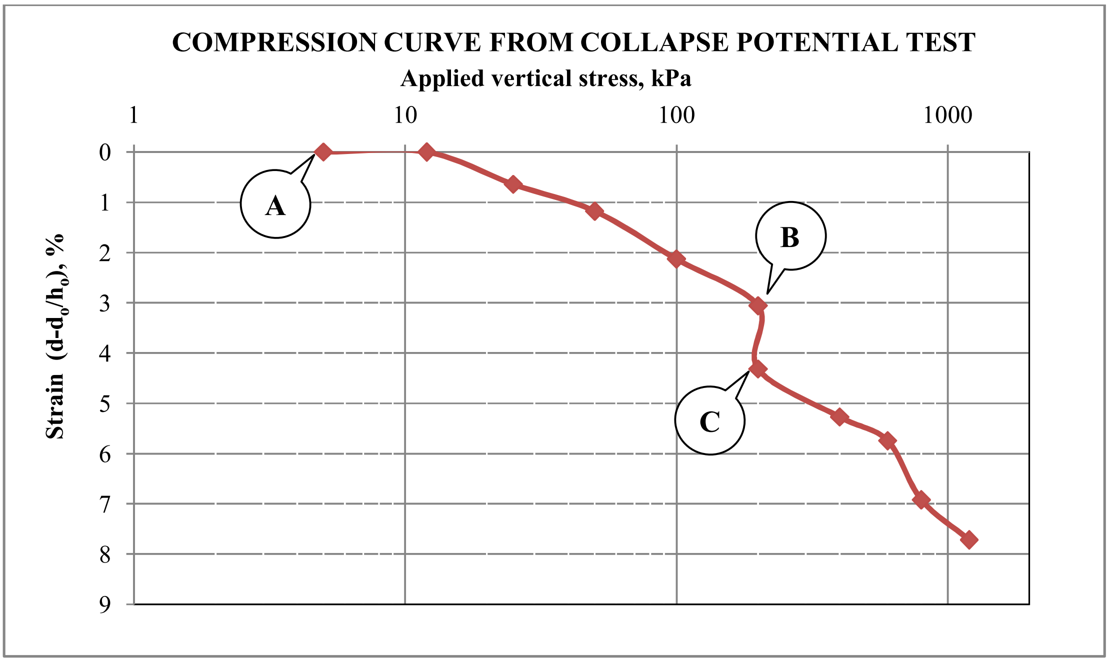


Figure 4.13: Compression curve of collapse test for SR-3 (ASTM - D 5333)

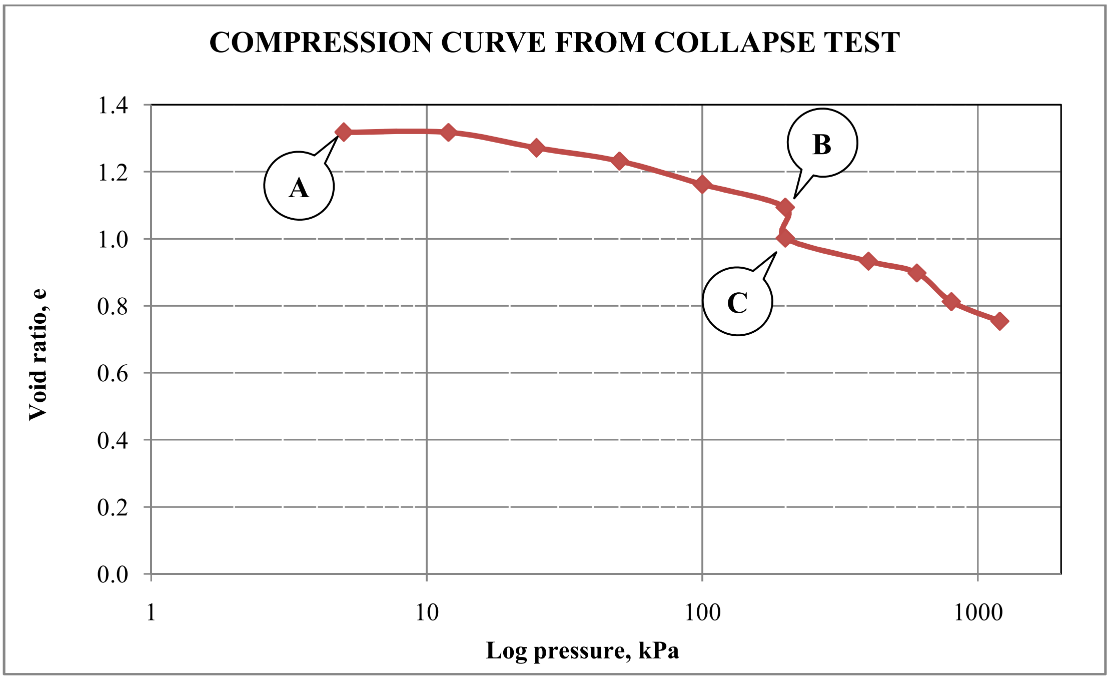


Figure 4.14: Compression curve of collapse test for SR-3 (Schwartz, 1985)

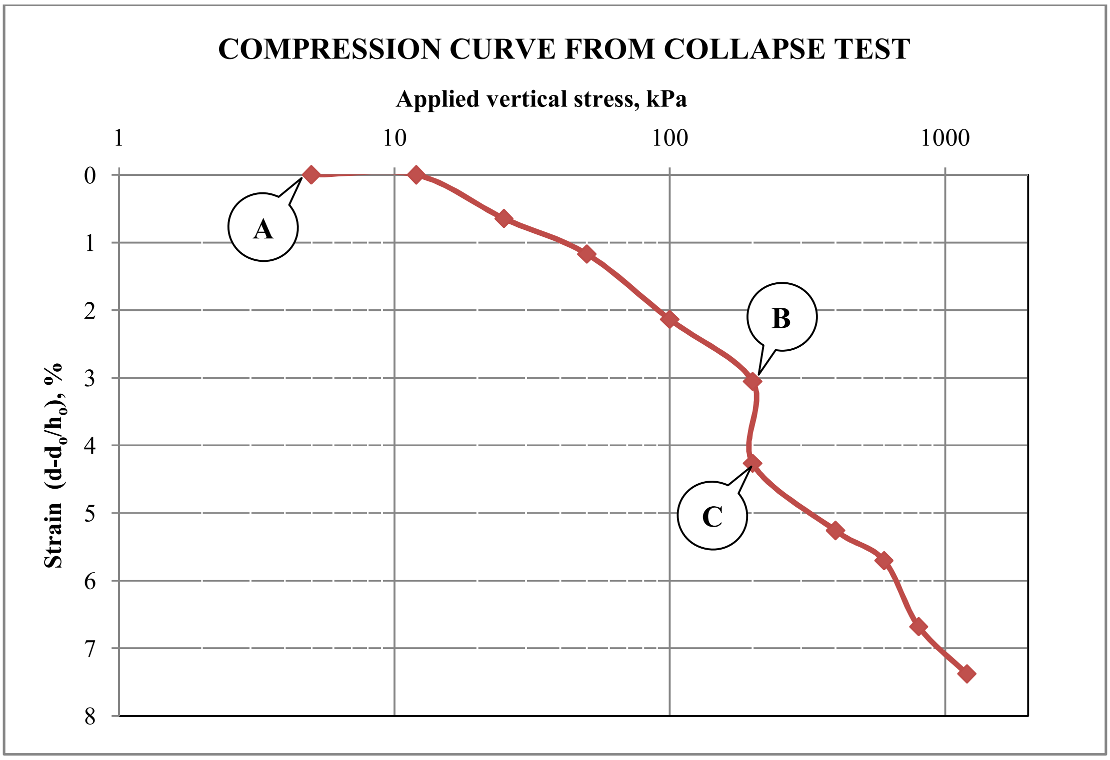


Figure 4.15: Compression curve of collapse test for SR-4 (ASTM - D 5333)

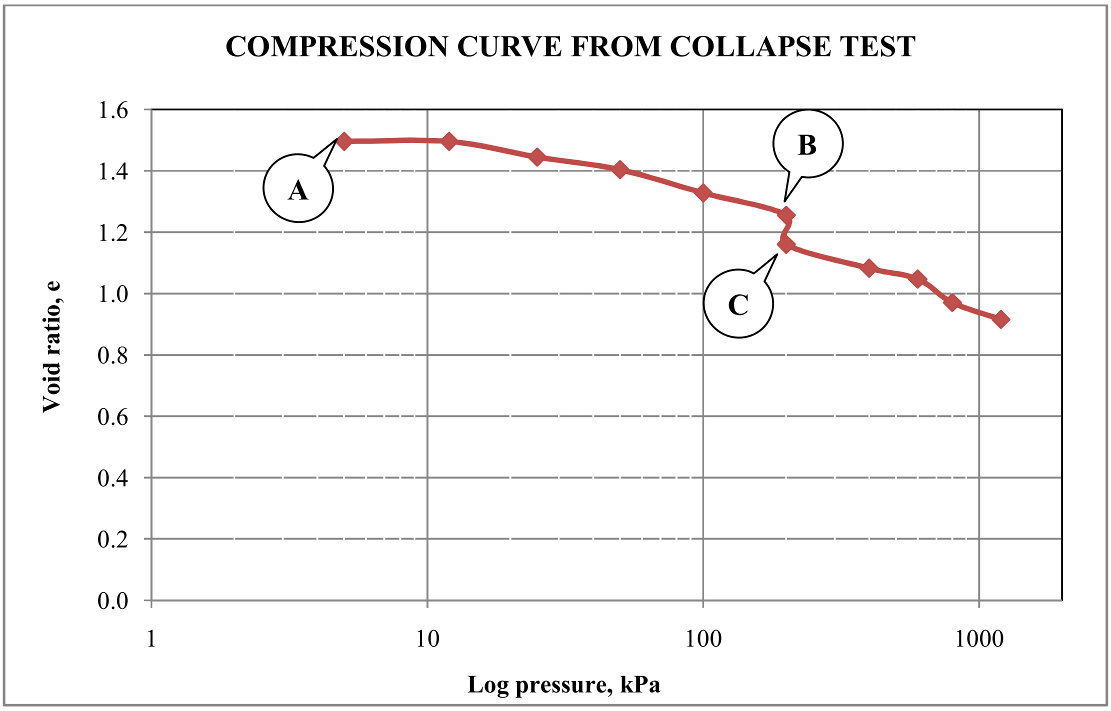


Figure 4.16: Compression curve of collapse test for SR-4 (Schwartz, 1985)

Table 4.20: Collapse index (I_e) computation from compression curve of sample from LHS of the road using ASTM - D5333

Description	Unit	Values				Remark
		SL-1	SL-2	SL-3	SL-4	
Initial specimen height, h_o	mm	20	20	20	20	
Dial reading at 200 kPa stress level before wetting, d_e	mm	1.941	1.973	1.909	1.783	
Dial reading at 200 kPa stress level after wetting, d_f	mm	3.119	3.101	2.903	2.687	
Collapse Index, $I_e = 100*(d_f - d_e)/h_o$	%	5.89	5.64	4.97	4.52	

Table 4.21: Collapse index (C_p) computation from compression curve of sample from LHS of the road using Jennings and Knight (1975)

Description	Unit	Values				Remark
		SL-1	SL-2	SL-3	SL-4	
Initial void ratio at its natural water content before wetting, e_o		1.376	1.356	1.394	1.257	
Initial void ratio at its natural water content under 200 kPa of vertical pressure before wetting, e_i		1.145	1.124	1.166	1.055	
Final void ratio after soaking under 200 kPa of vertical pressure after wetting, e_f		1.005	0.991	1.047	0.953	
Collapse Potential, $C_p = 100^* \Delta e_c/(1+e_o)$	%	5.89	5.64	4.97	4.52	

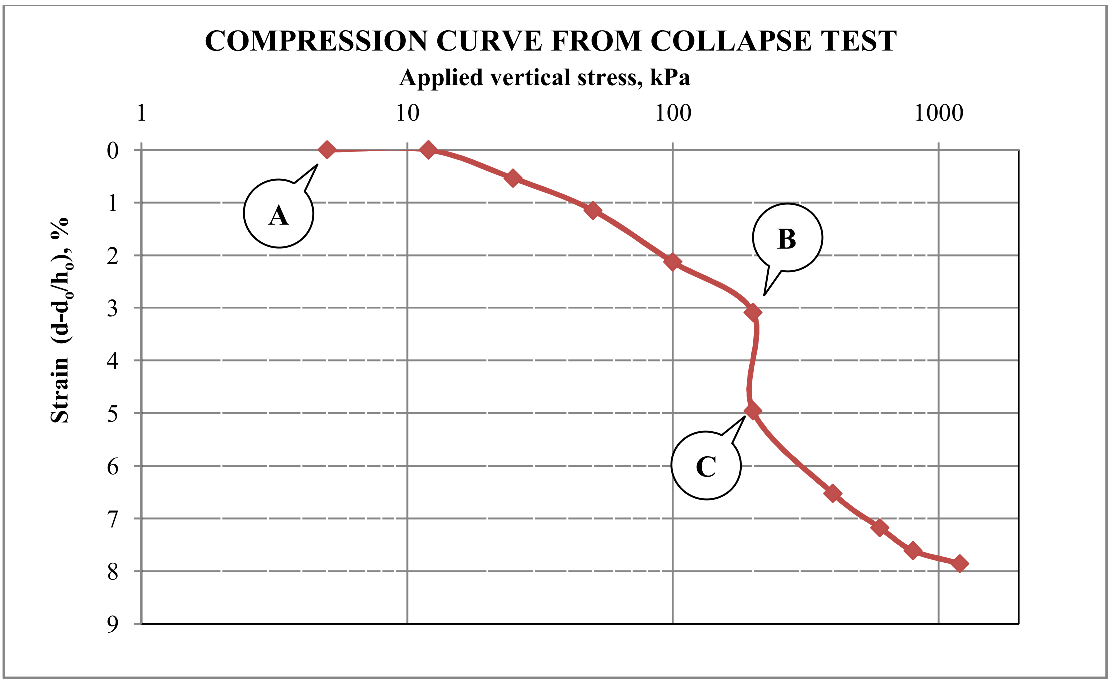


Figure 4.17: Compression curve of collapse test for SL-1 (ASTM - D 5333)

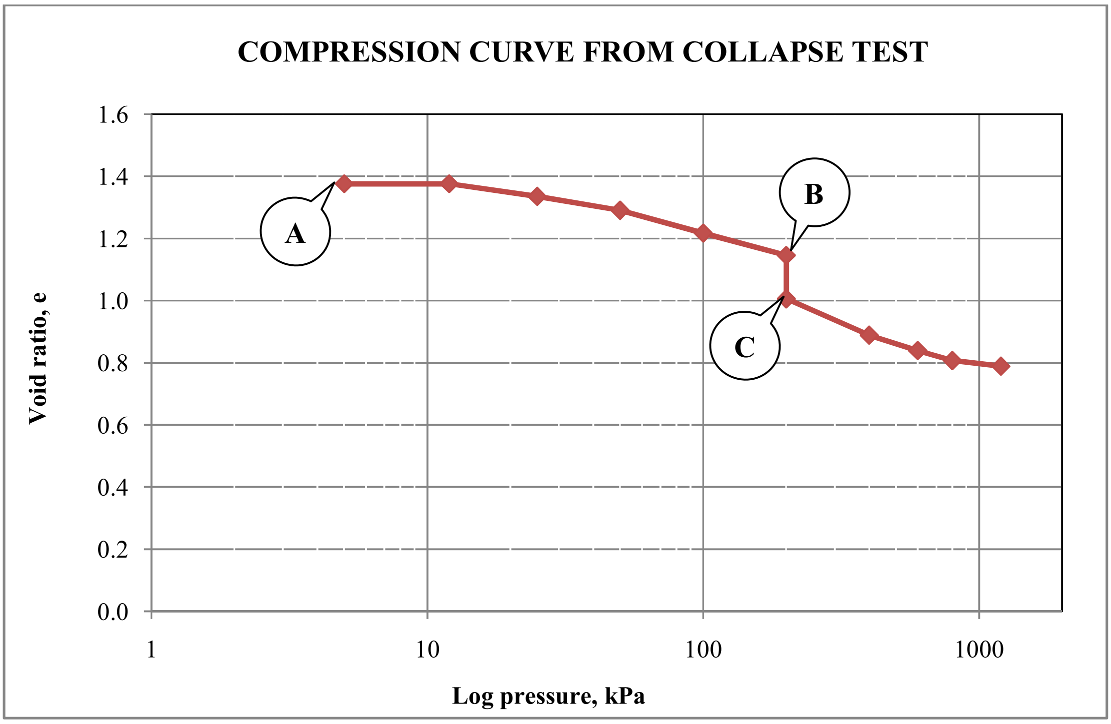


Figure 4.18: Compression curve of collapse test for SL-1 (Schwartz, 1985)

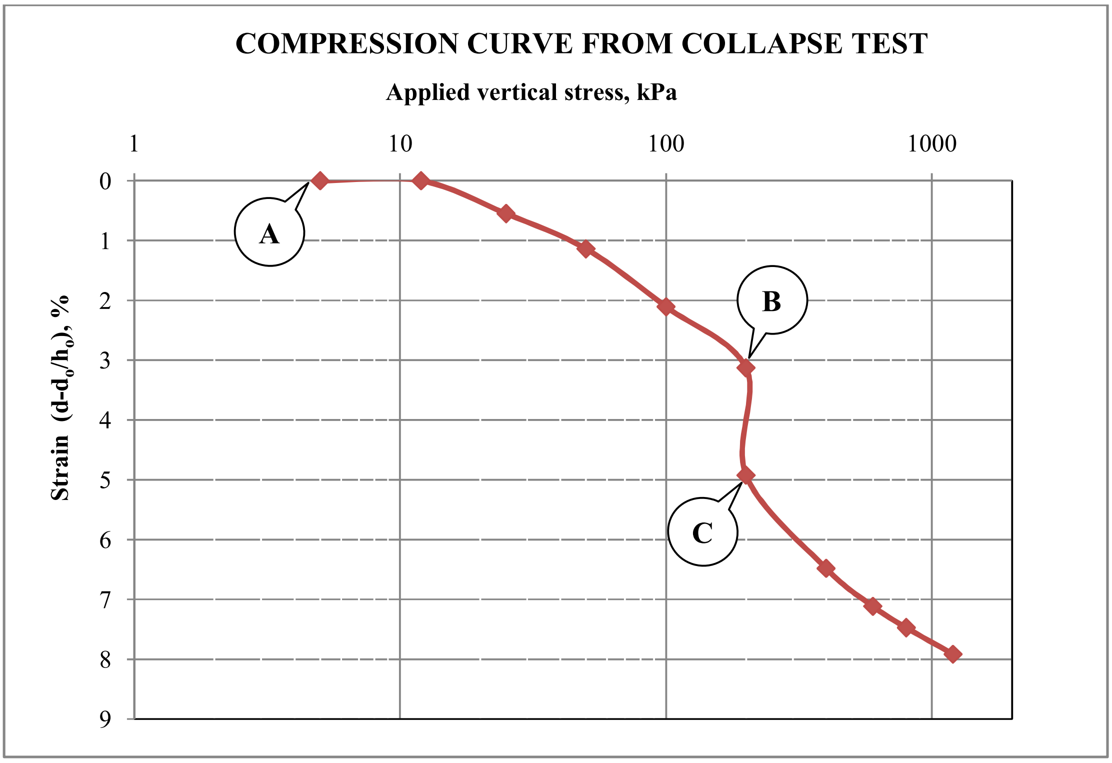


Figure 4.19: Compression curve of collapse test for SL-2 (ASTM - D 5333)

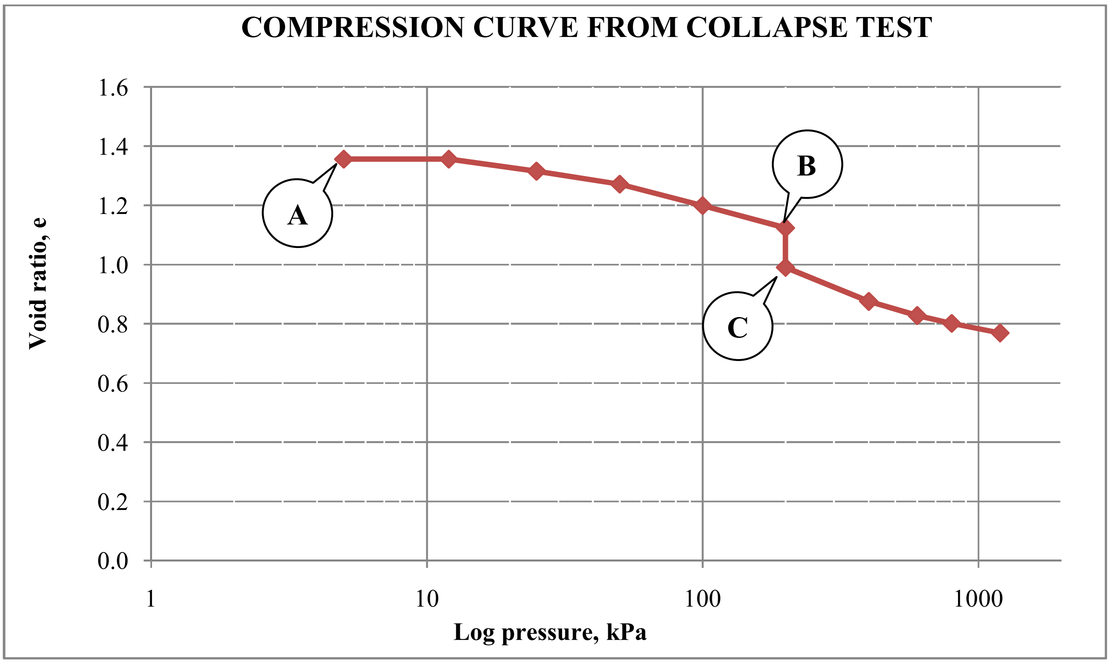


Figure 4.20: Compression curve of collapse test for SL-2 (Schwartz, 1985)

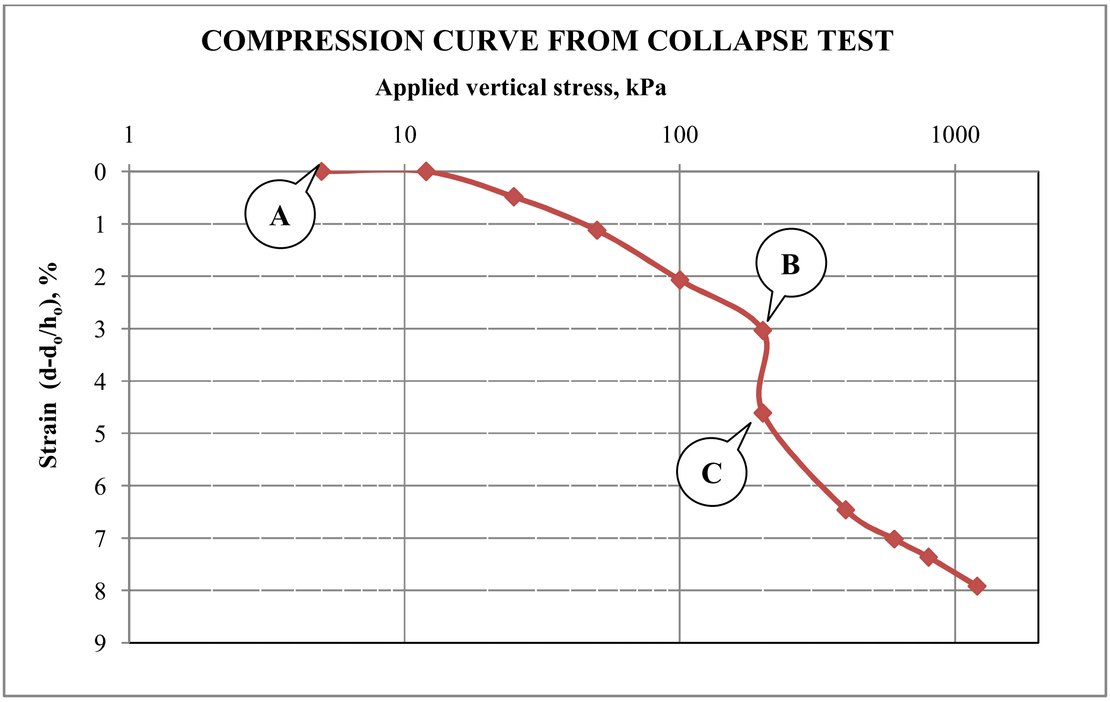


Figure 4.21: Compression curve of collapse test for SL-3 (ASTM - D 5333)

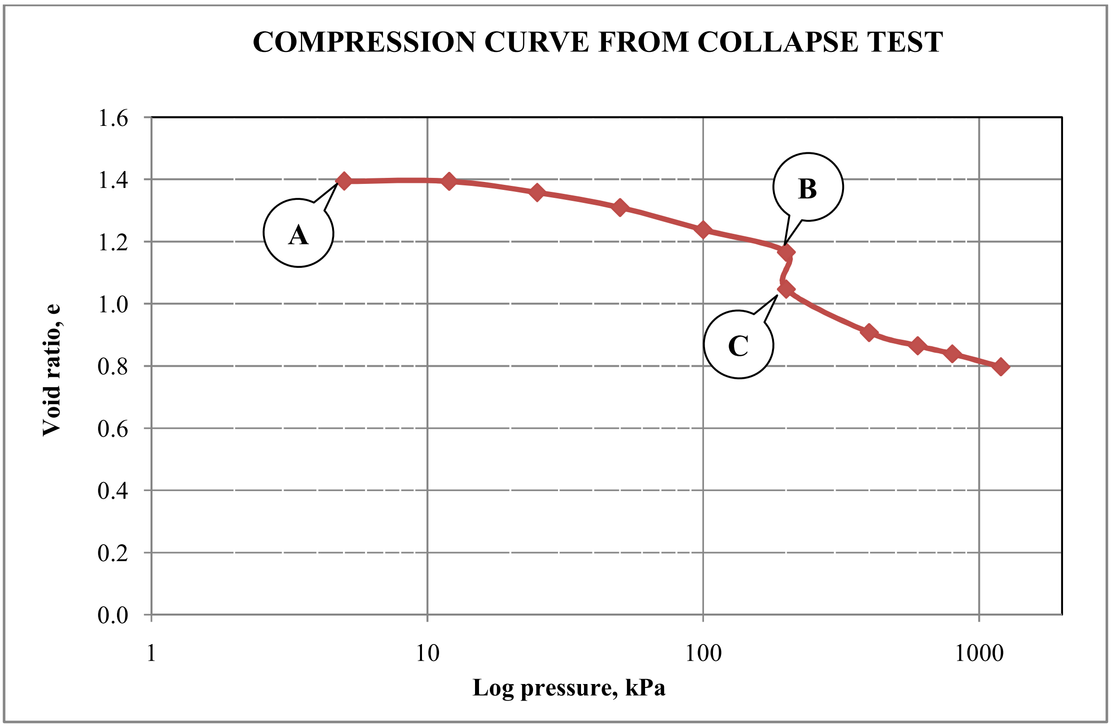


Figure 4.22: Compression curve of collapse test for SL-3 (Schwartz, 1985)

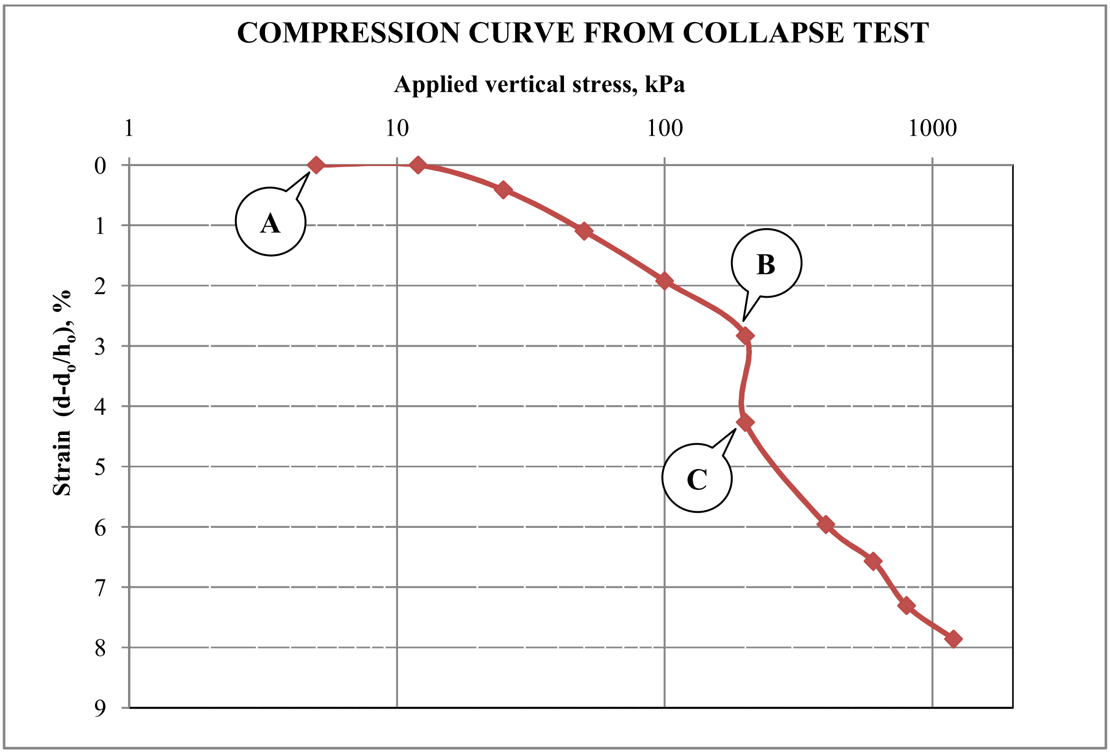


Figure 4.23: Compression curve of collapse test for SL-4 (ASTM - D 5333)

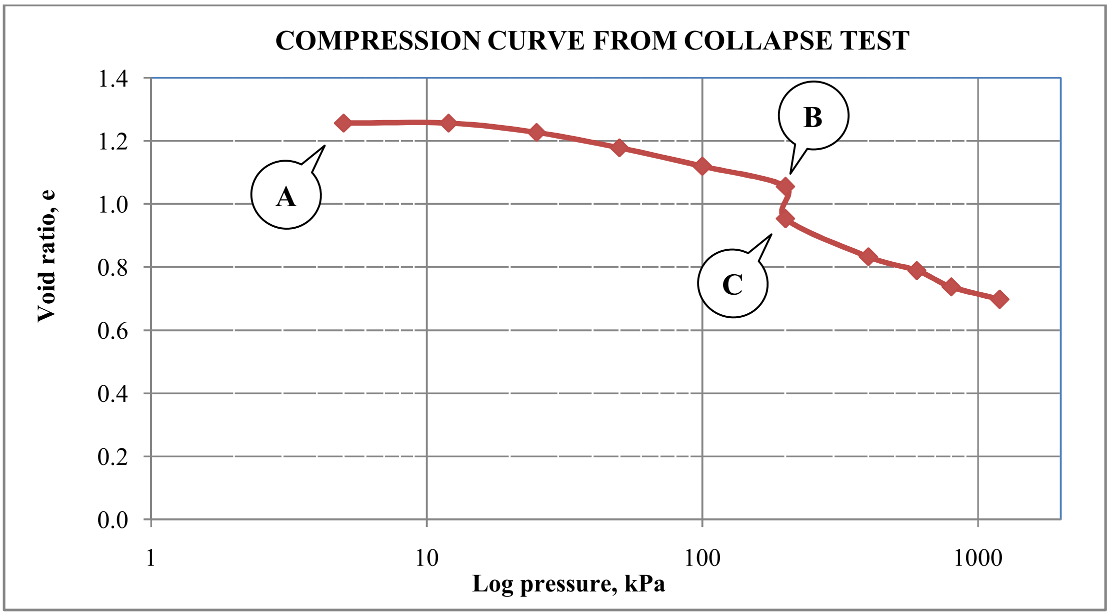


Figure 4.24: Compression curve of collapse test for SL-4 (Schwartz, 1985)

4.2.2 Direct Shear Test

Direct shear test is useful to study the stress – strain properties of cohesionless soil like sand and gravel. From this test we can obtain shear strength parameters like angle of internal fiction (ϕ) and cohesion (c). However, cohesion value for sand is generally taken as zero. The normal loads applied for conducting the direct shear test were 27.25 kPa, 54.50 kPa, and 81.75 kPa. This is due to the reason that the available direct shear test apparatus at AMU – IoT was equipped with the mentioned loads.

Table 4.22: Maximum shear stress and angle of internal friction for unsaturated samples

Data for Maximum Shear Stress for Unsaturated Soil Samples								
Normal Stress, kPa	Maximum Shear Stress, kPa							
	SR_1	SR_2	SR_3	SR_4	SL_1	SL_2	SL_3	SL_4
27.25	20.51	21.15	21.30	21.47	19.88	20.59	20.83	20.91
54.50	37.20	37.81	39.09	39.56	36.61	36.89	38.01	38.30
81.75	56.71	57.57	58.76	60.37	57.03	57.77	59.05	60.23
Cohesion (c), kPa	-	-	-	-	-	-	-	-
Angle of internal friction, ϕ	33.58	33.74	34.49	35.49	34.25	34.29	35.03	35.79

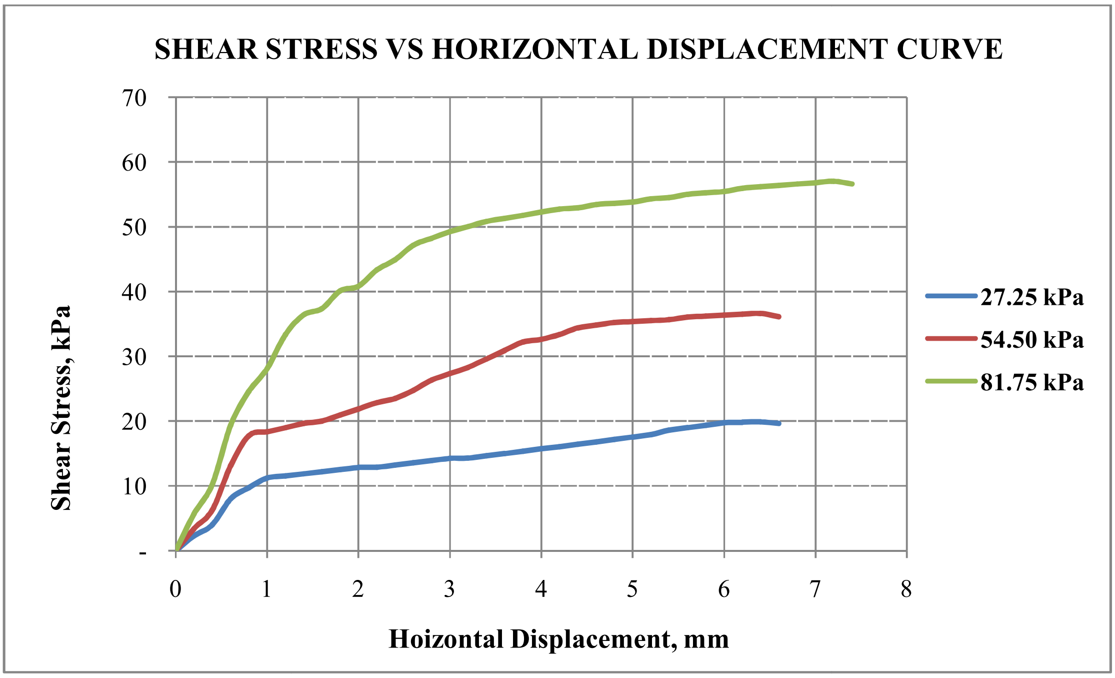


Figure 4.25: Shear stress vs. horizontal displacement curve of unsaturated sample (SL-1)

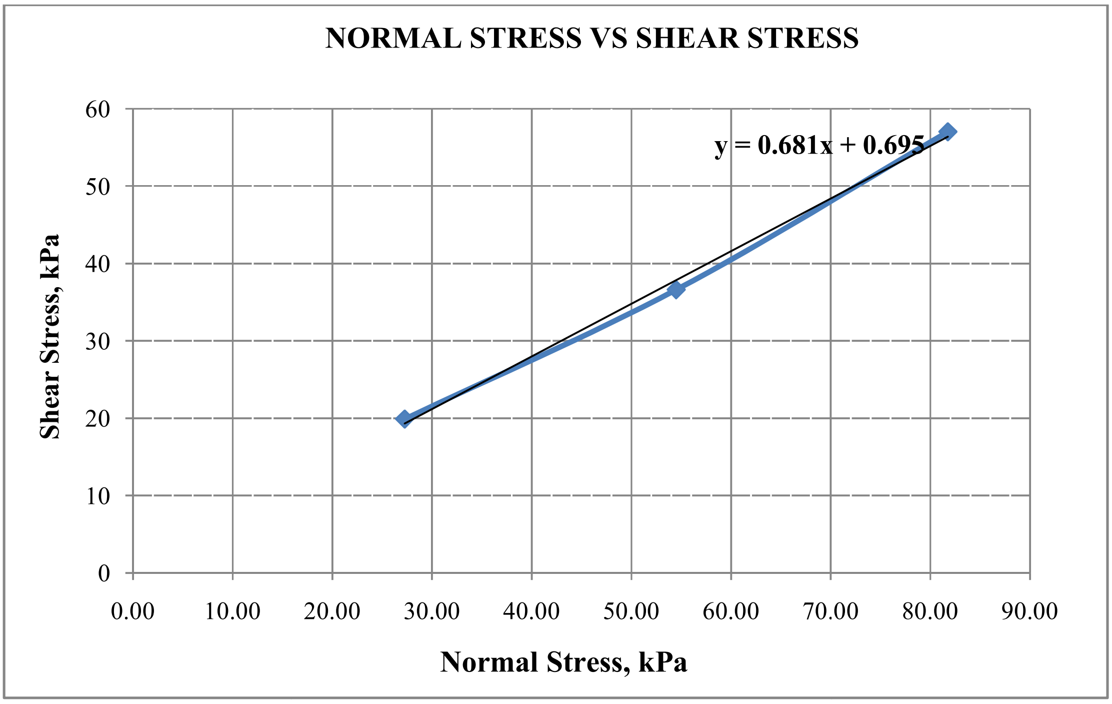


Figure 4.26: Normal stress vs. shear stress graph of unsaturated sample (SL-1)

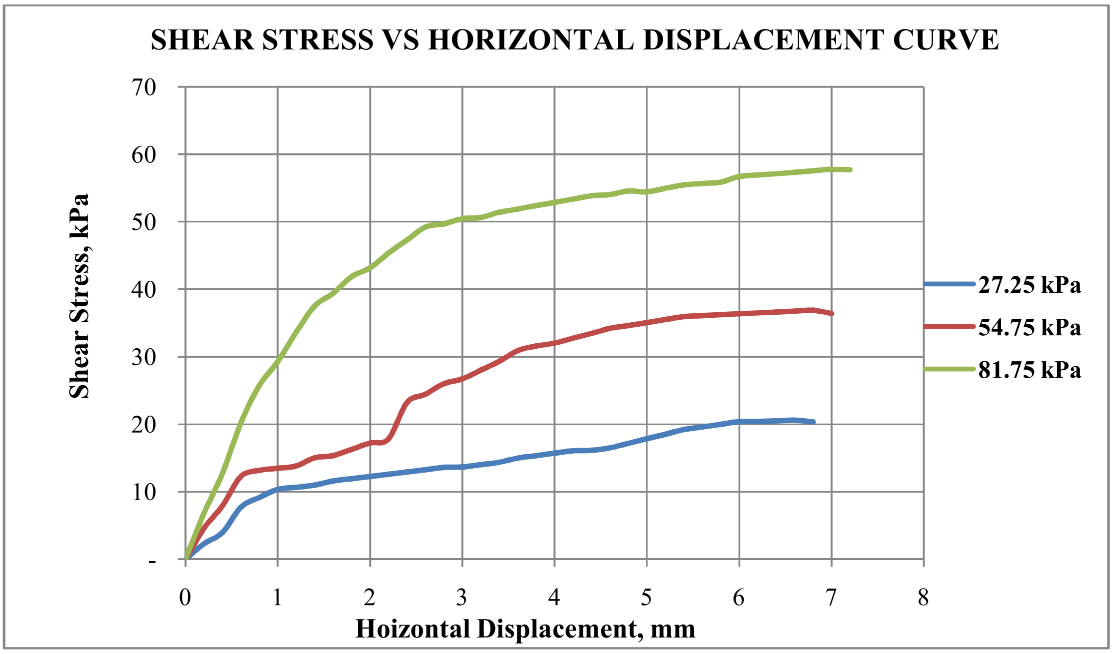


Figure 4.29: Shear stress vs. horizontal displacement curve of unsaturated sample (SL-2)

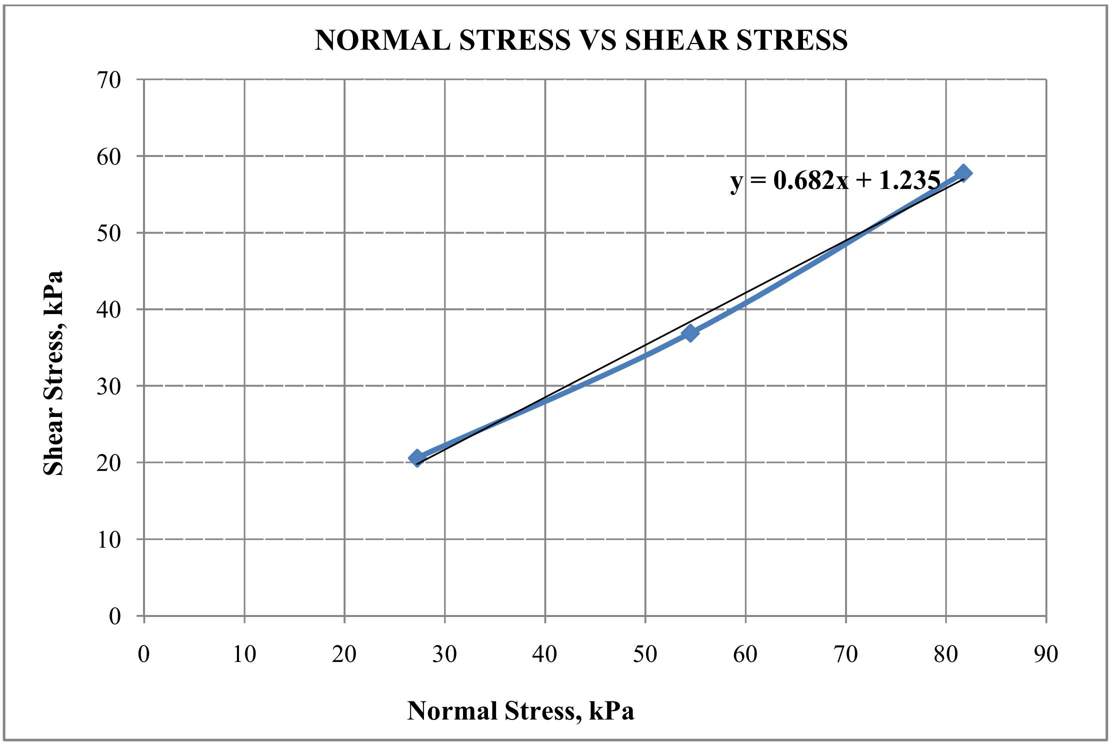


Figure 4.30: Normal stress vs. shear stress graph of unsaturated sample (SL-2)

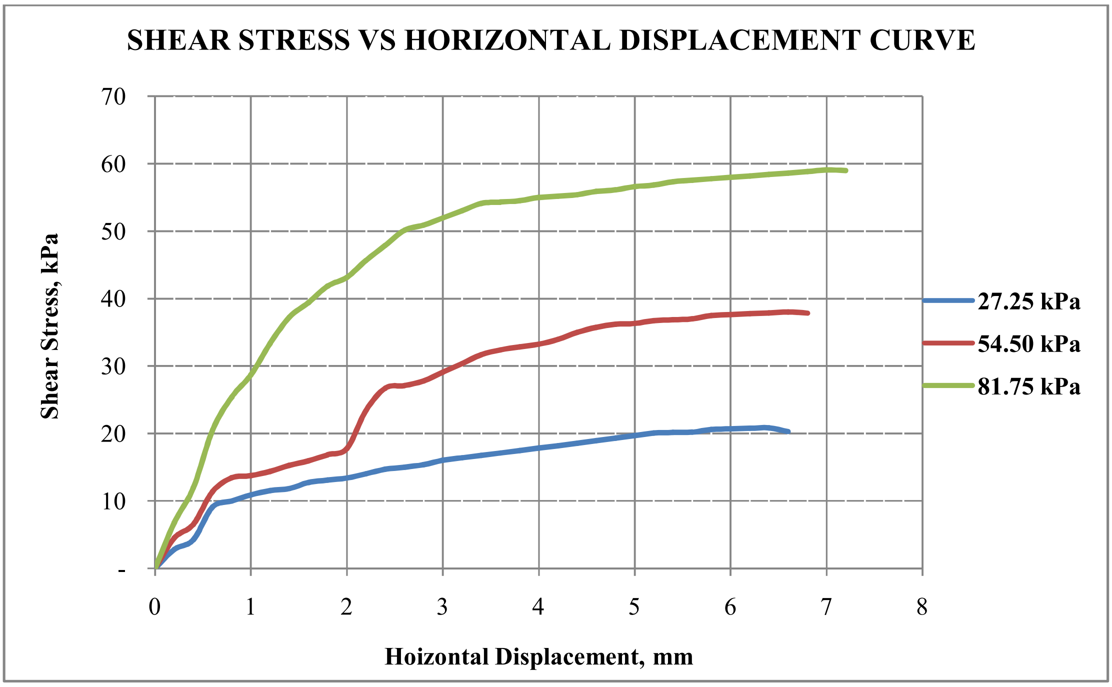


Figure 4.33: Shear stress vs. horizontal displacement curve of unsaturated soil (SL-3)

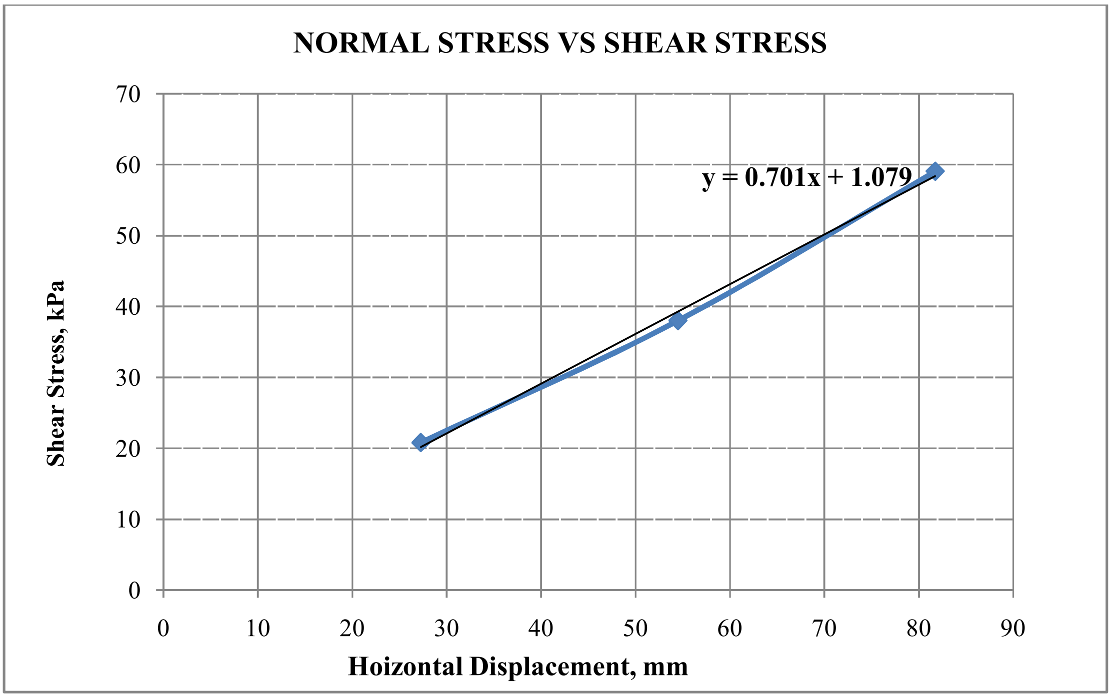


Figure 4.34: Normal stress vs. shear stress graph of unsaturated sample (SL-3)

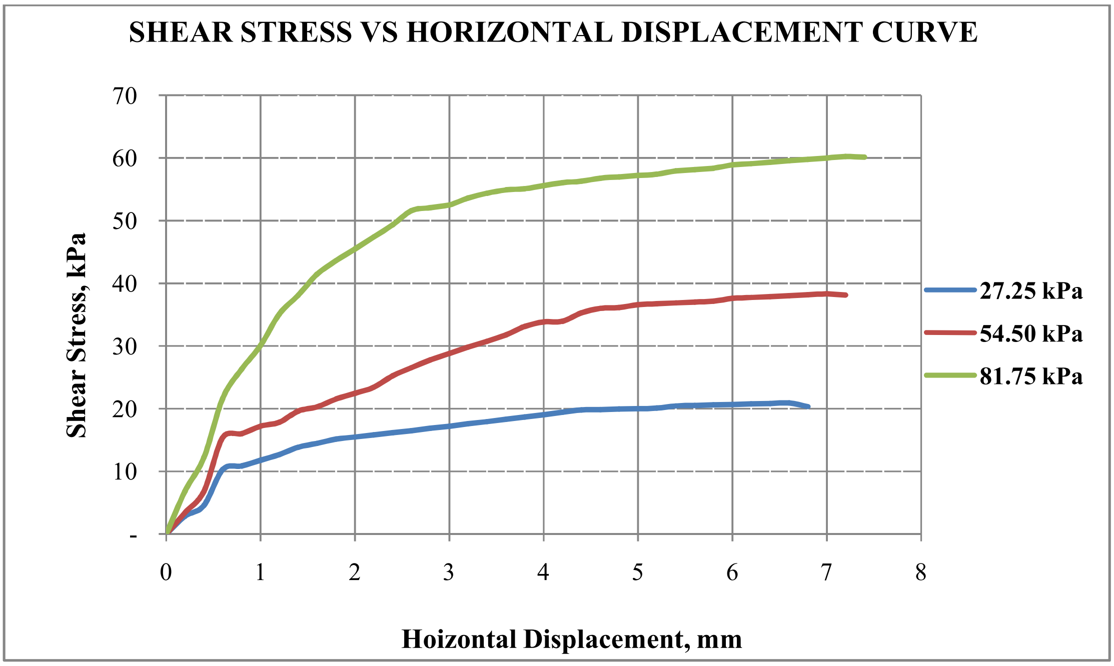


Figure 4.37: Shear stress vs. horizontal displacement curve of unsaturated sample (SL-4)

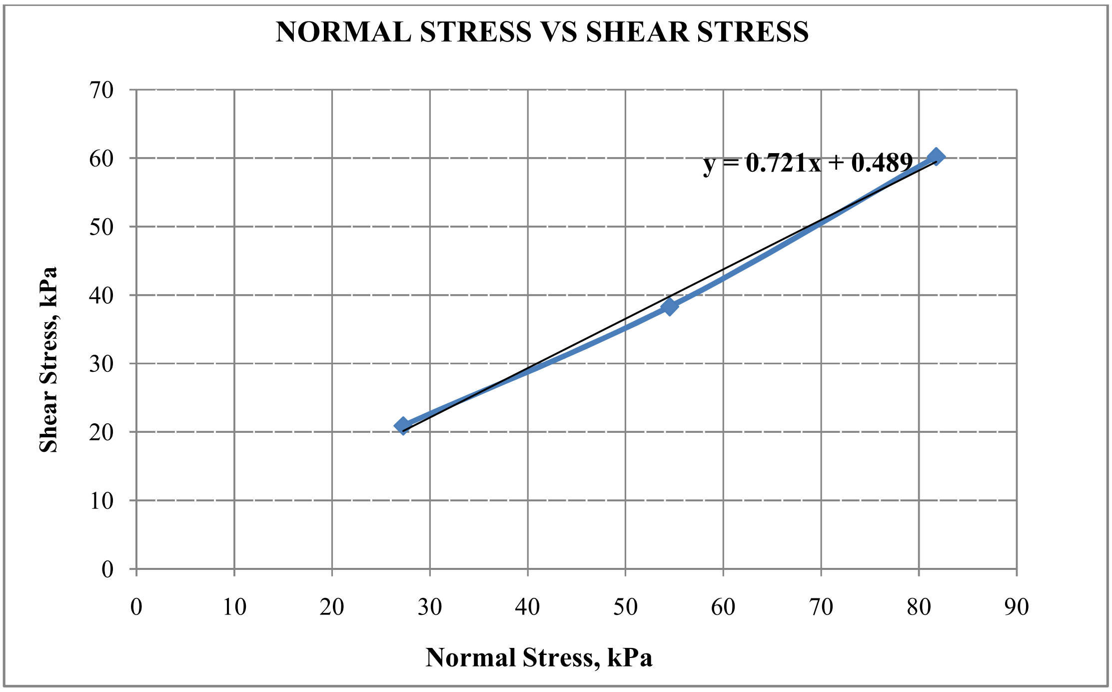


Figure 4.38: Normal stress vs. shear stress graph unsaturated sample (SL-4)

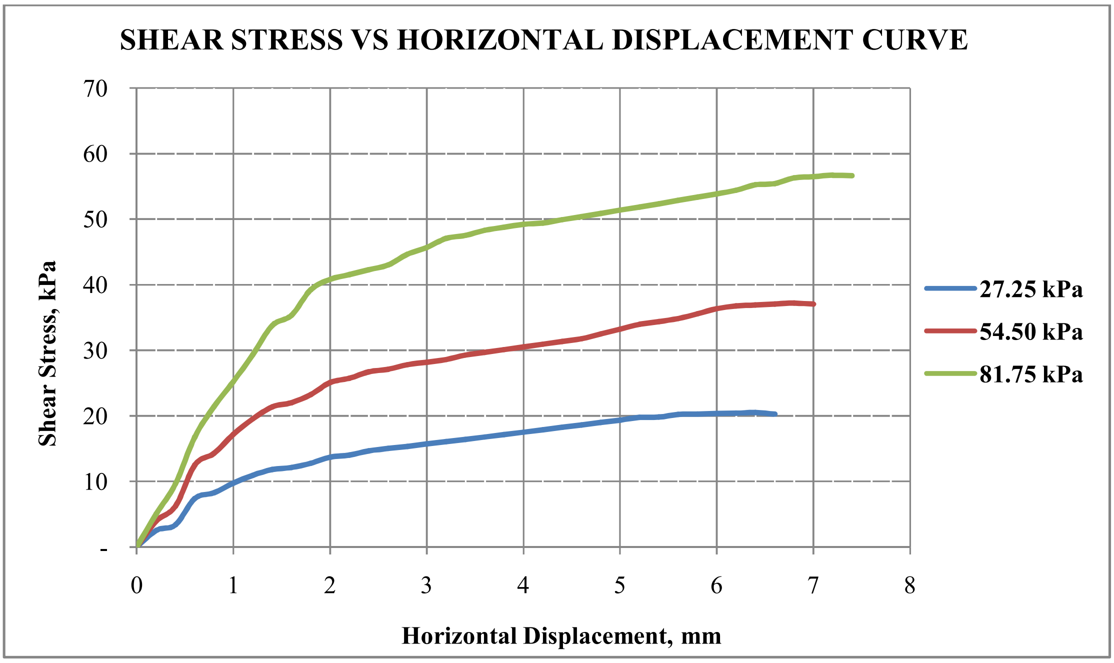


Figure 4.41: Shear stress vs. horizontal displacement curve unsaturated soil (SR-1)

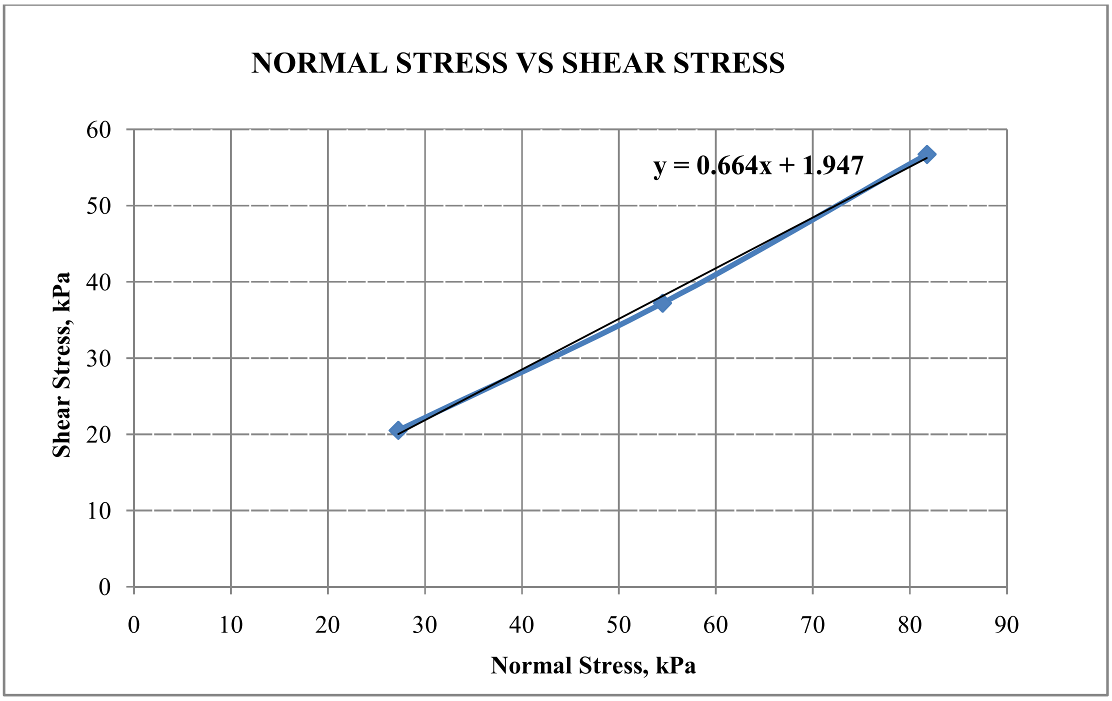


Figure 4.42: Normal stress vs. shear stress graph unsaturated soil (SR-1)

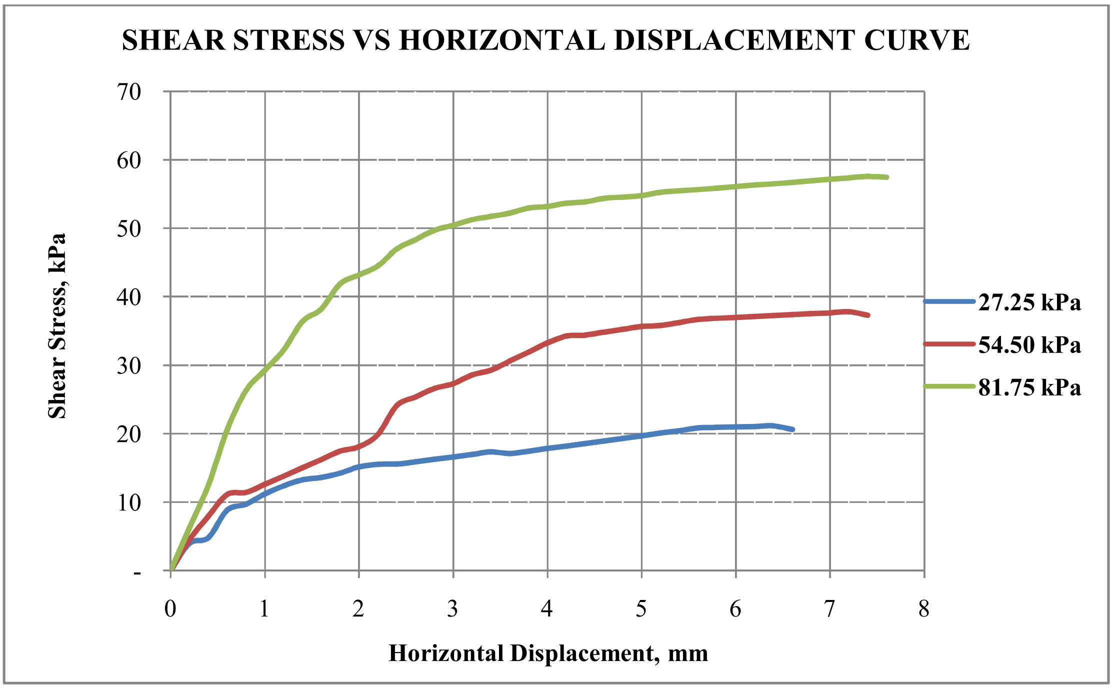


Figure 4.45: Shear stress vs. horizontal displacement curve unsaturated soil (SR-2)

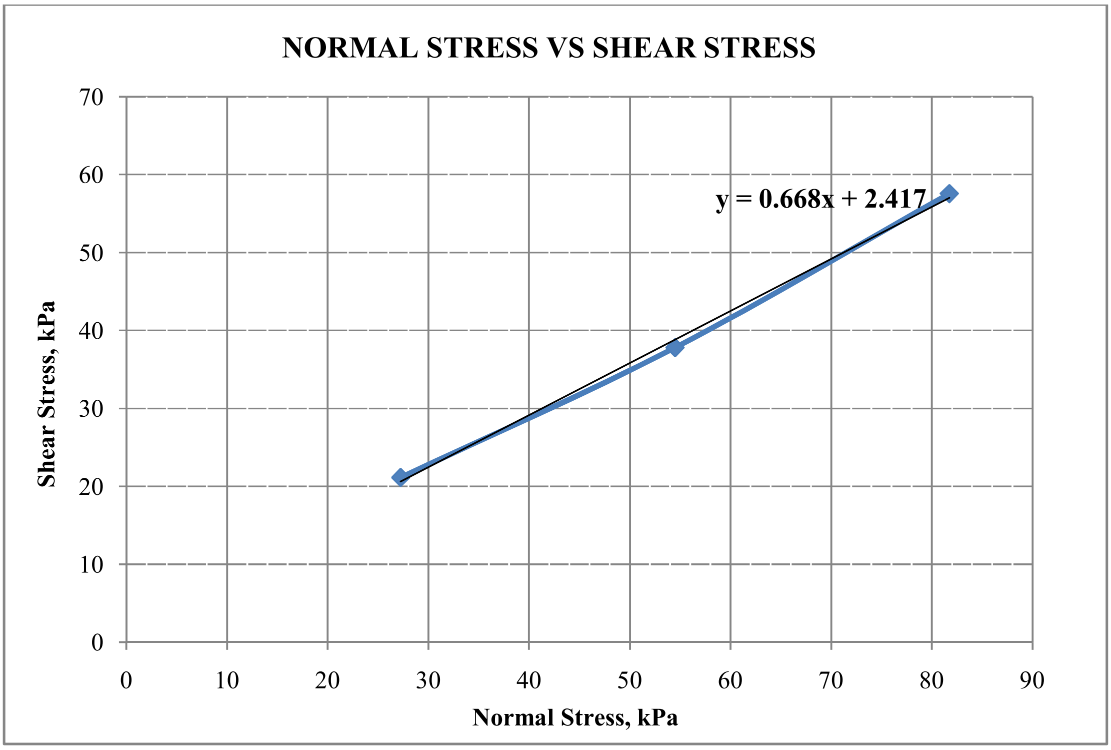


Figure 4.46: Normal stress vs. shear stress graph unsaturated soil (SR-2)

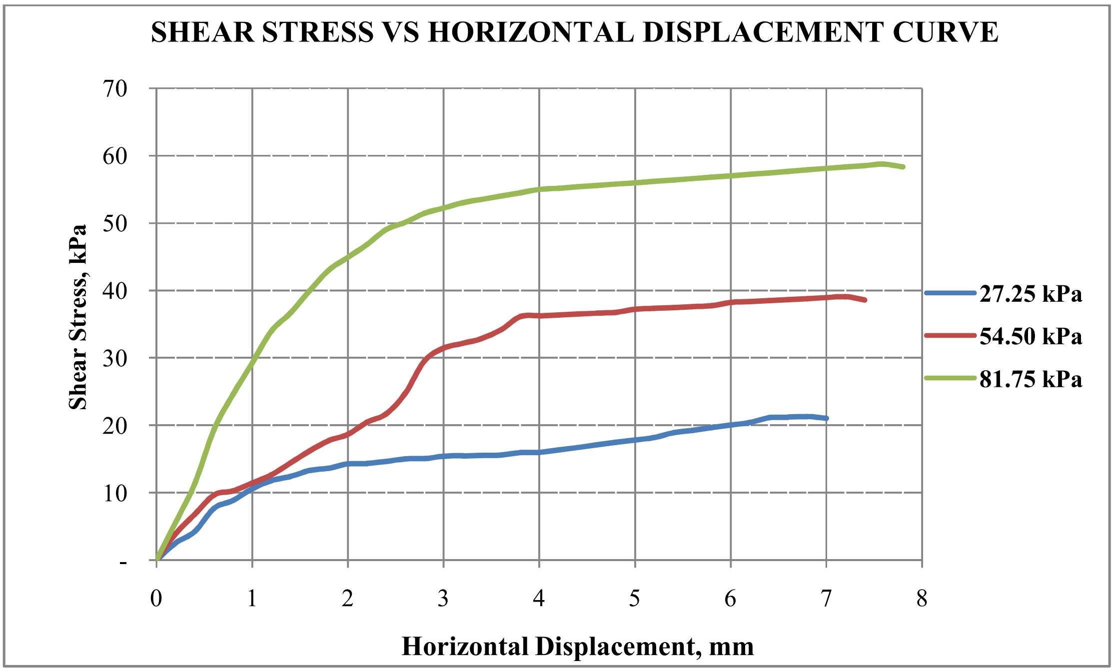


Figure 4.49: Shear stress vs. horizontal displacement curve unsaturated soil (SR-3)

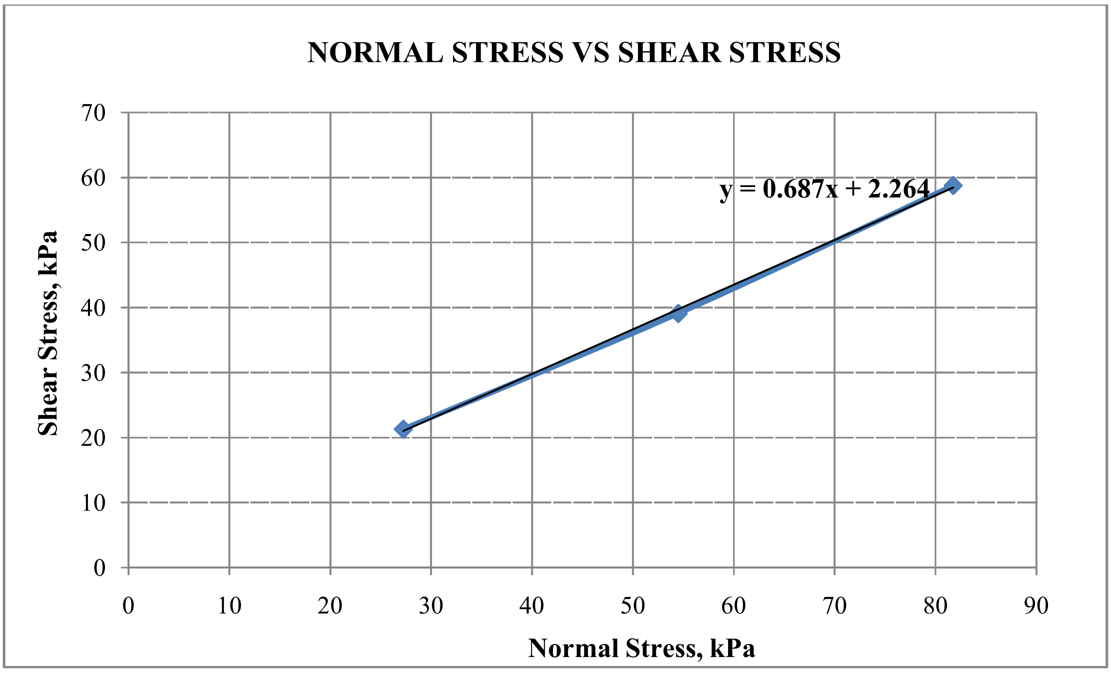


Figure 4.50: Normal stress vs. shear stress graph unsaturated soil (SR-3)

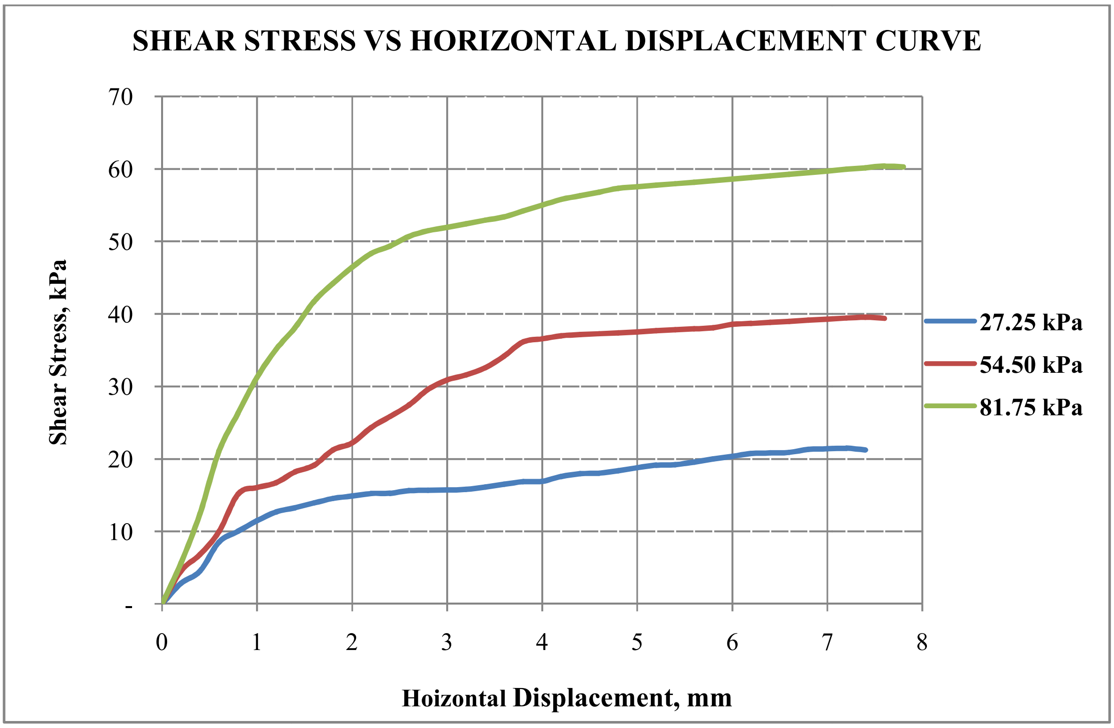


Figure 4.53: Shear stress vs. horizontal displacement curve unsaturated soil (SR-4)

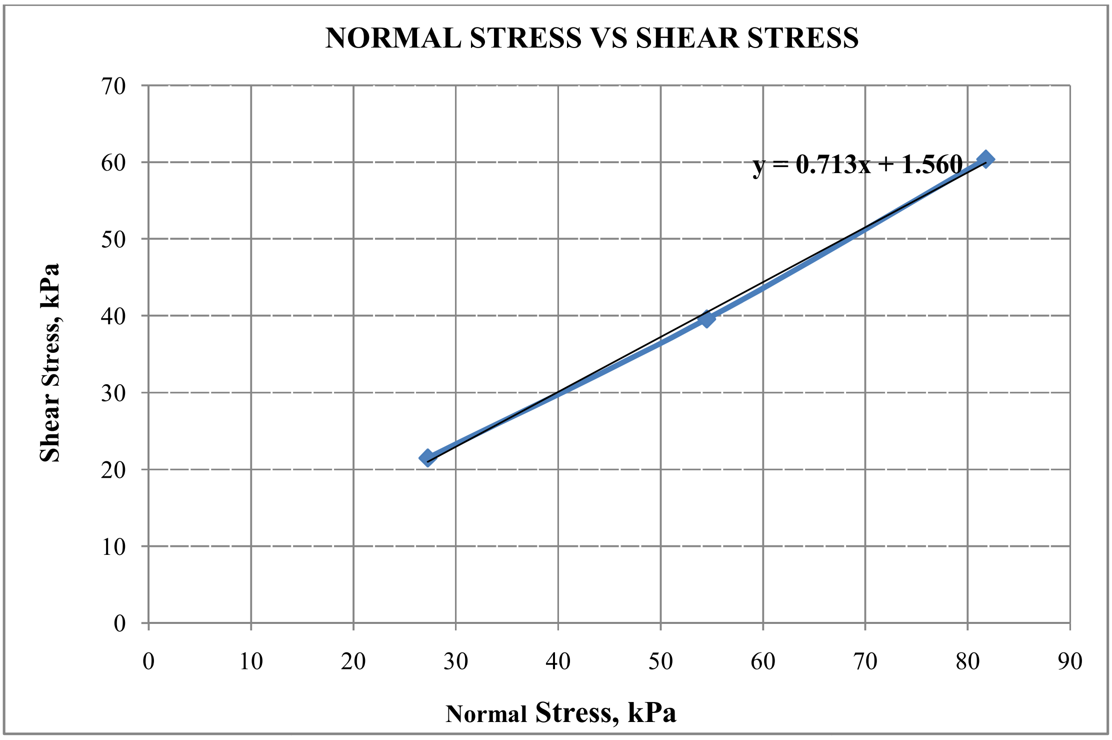


Figure 4.54: Normal stress vs. shear stress graph unsaturated soil (SR-4)

Table 4.23: Maximum shear stress and angle of internal friction for saturated samples

Data for Maximum Shear Stress for Saturated Soil Samples								
Normal Stress, kPa	Maximum Shear Stress, kPa							
	SR_1	SR_2	SR_3	SR_4	SL_1	SL_2	SL_3	SL_4
27.25	18.80	19.18	19.88	20.20	19.11	19.42	20.05	20.44
54.50	35.48	35.09	36.75	38.16	35.71	35.53	36.16	37.24
81.75	54.26	55.22	56.81	58.41	55.33	56.28	57.55	59.05
Cohesion (c), kPa	-	-	-	-	-	-	-	-
Angle of internal friction, ϕ	33.02	33.46	34.10	35.03	33.58	34.06	34.53	35.30

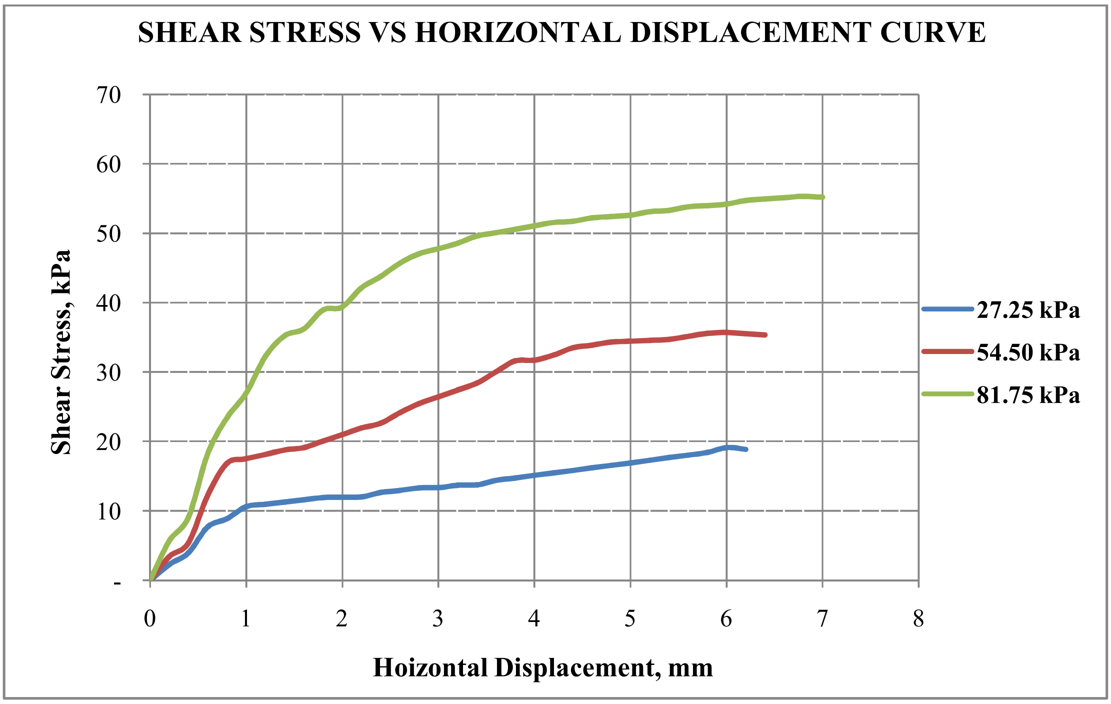


Figure 4.27: Shear stress vs. horizontal displacement curve of saturated sample (SL-1)

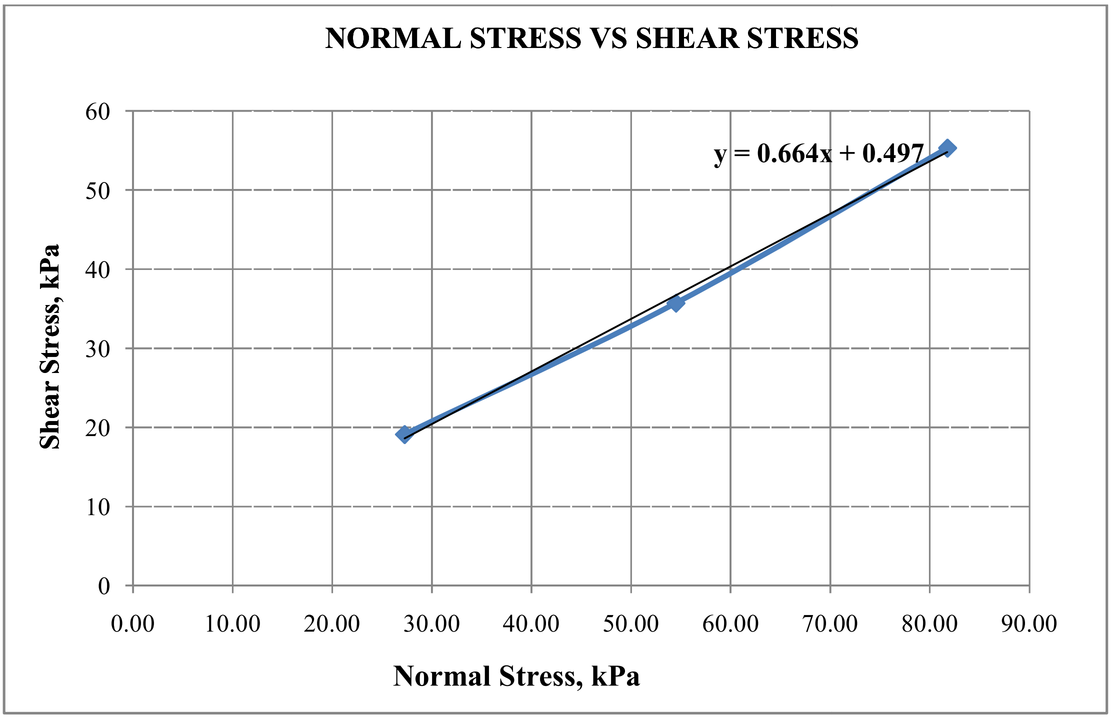


Figure 4.28: Normal stress vs. shear stress graph of saturated sample (SL-1)

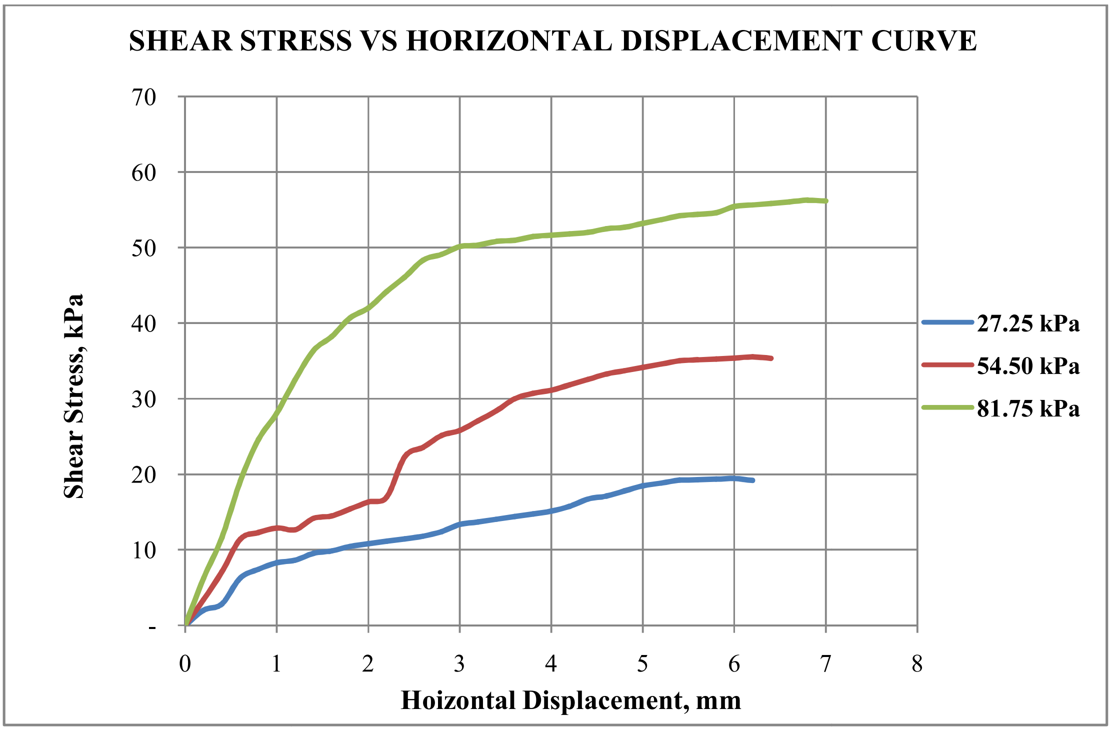


Figure 4.31: Shear stress vs. horizontal displacement curve of saturated sample (SL-2)

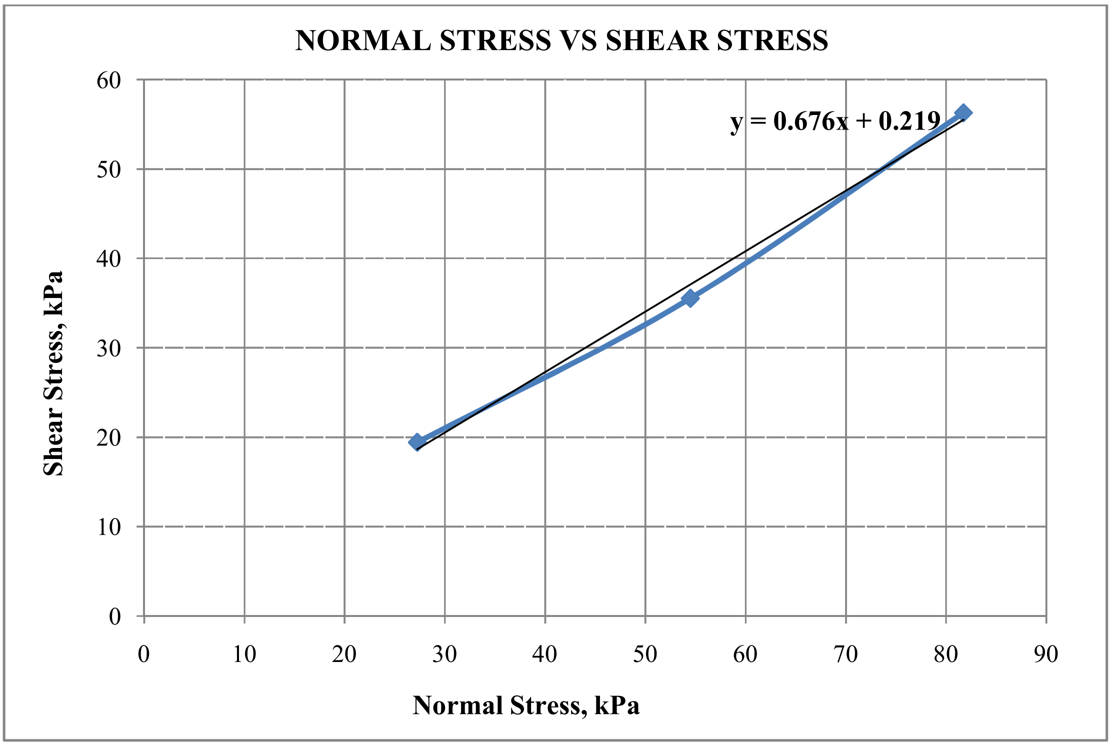


Figure 4.32: Normal stress vs. shear stress graph of saturated sample (SL-2)

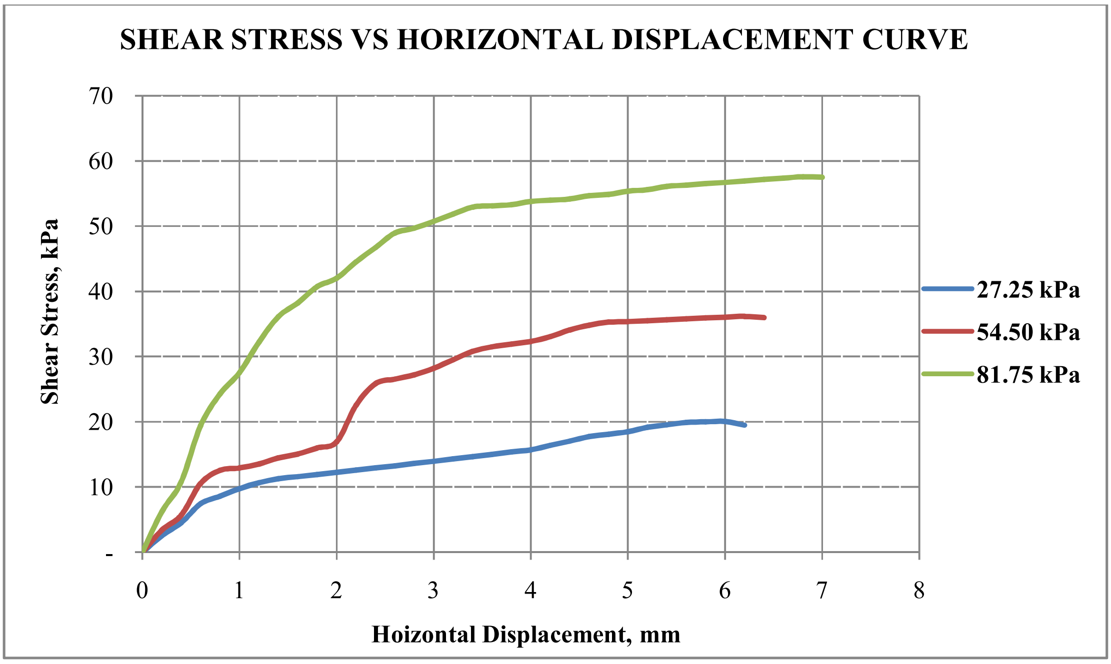


Figure 4.35: Shear stress vs. horizontal displacement curve of saturated sample (SL-3)

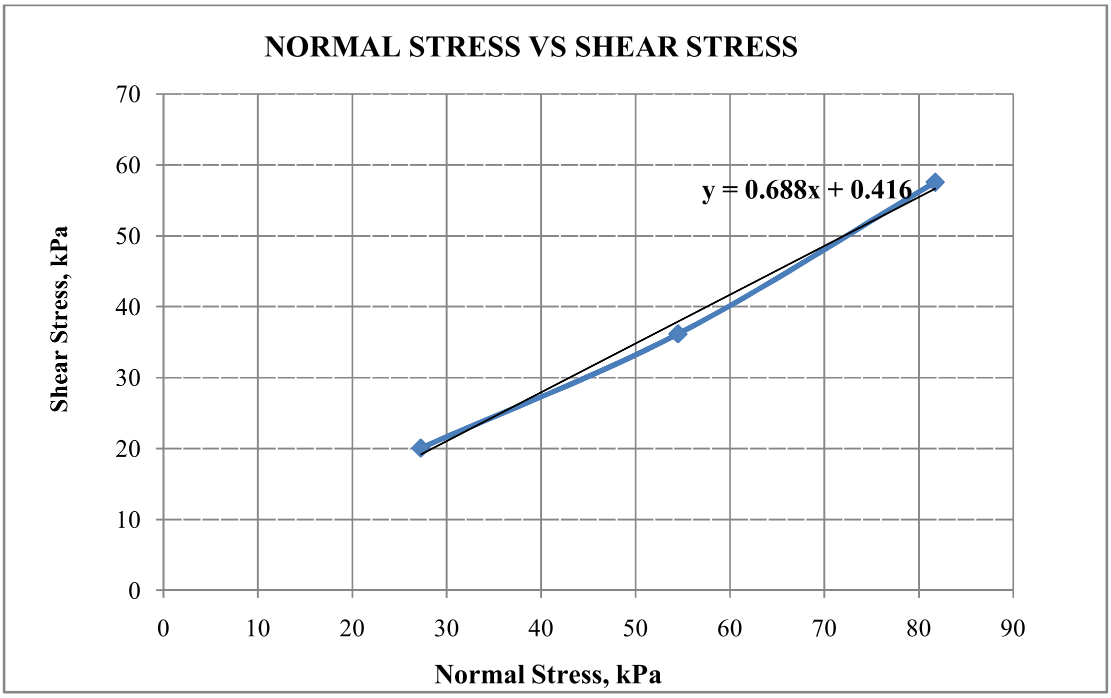


Figure 4.36: Normal stress vs. shear stress graph of saturated sample (SL-3)

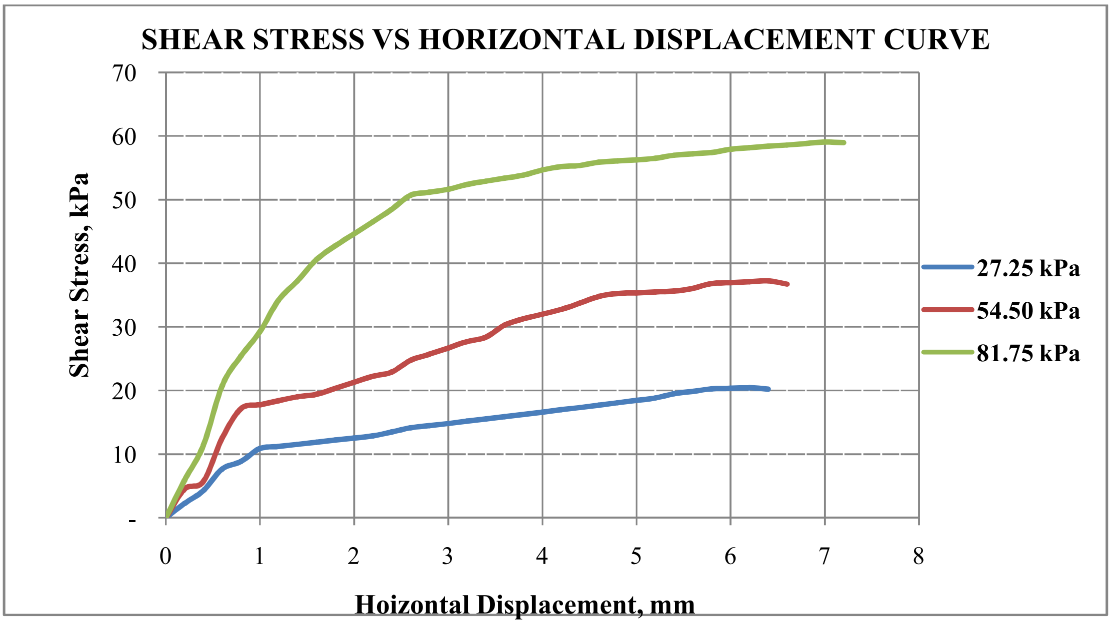


Figure 4.39: Shear stress vs. horizontal displacement curve of saturated sample (SL-4)

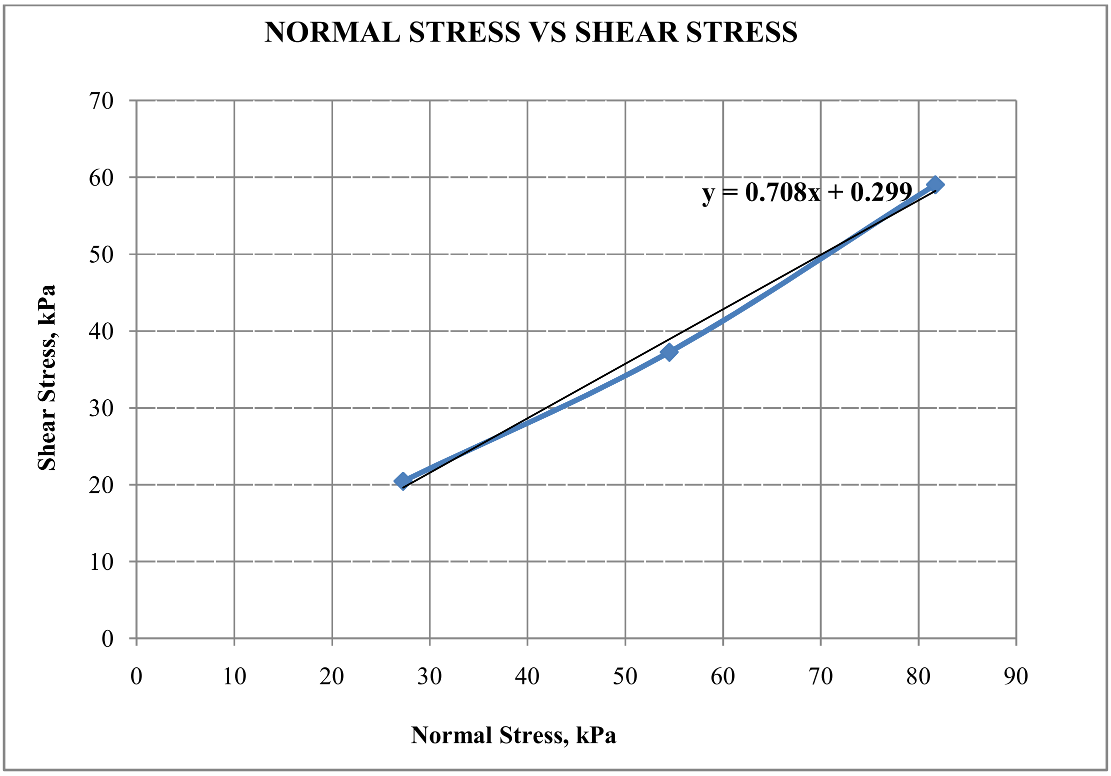


Figure 4.40: Normal stress vs. shear stress graph of saturated sample (SL-4)

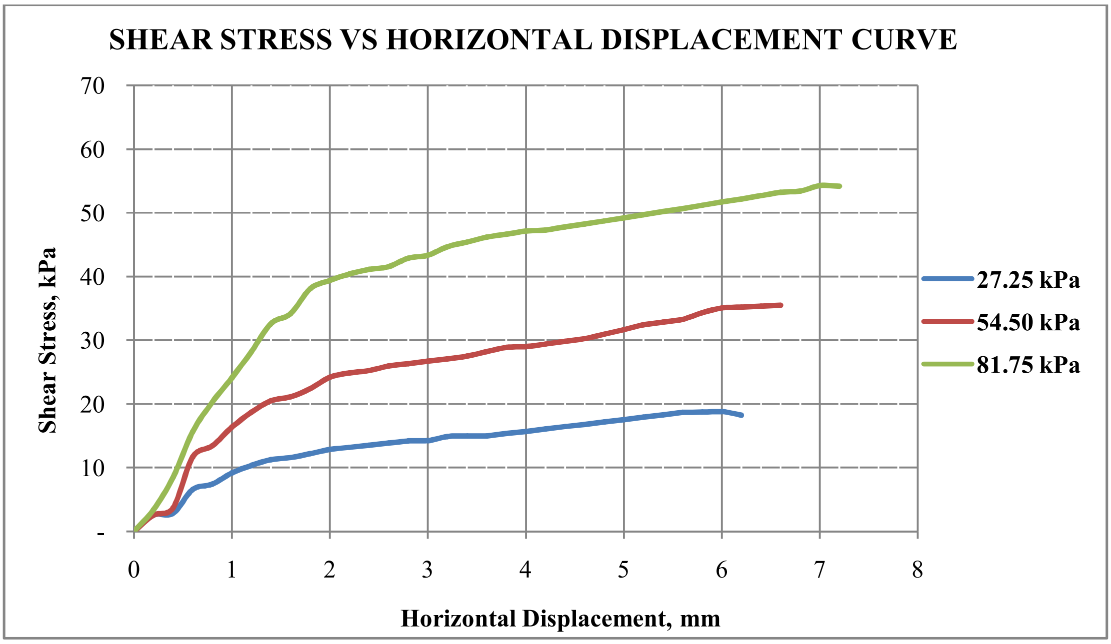


Figure 4.43: Shear stress vs. horizontal displacement curve saturated soil (SR-1)

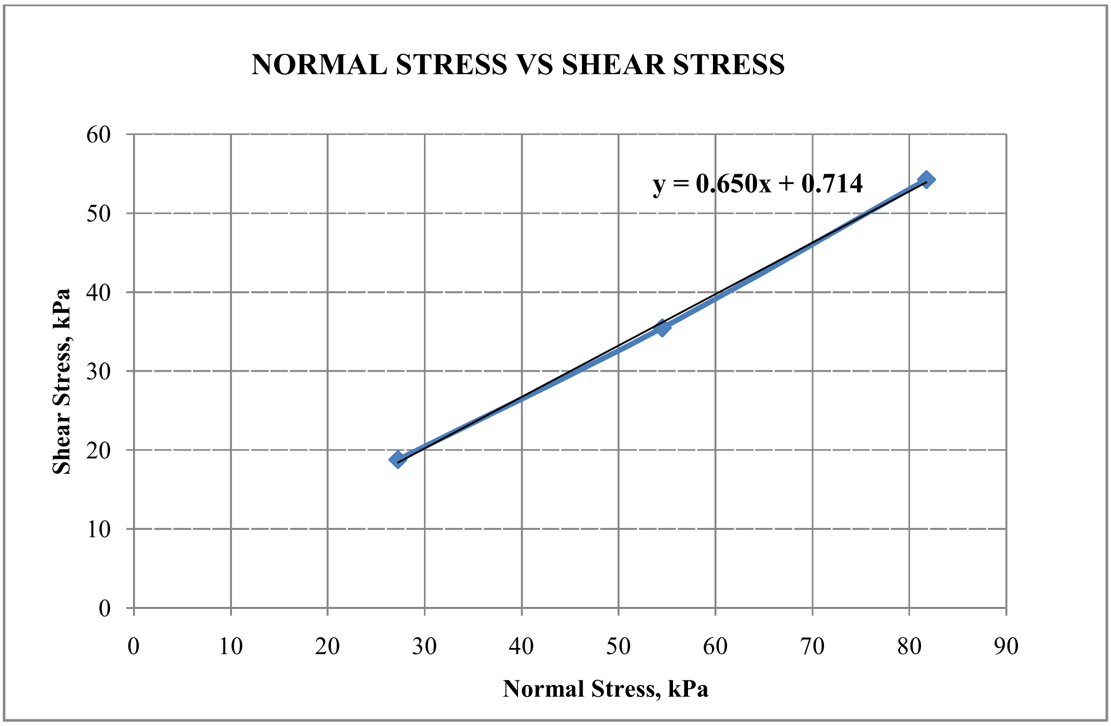


Figure 4.44: Normal stress vs. shear stress graph saturated soil (SR-1)

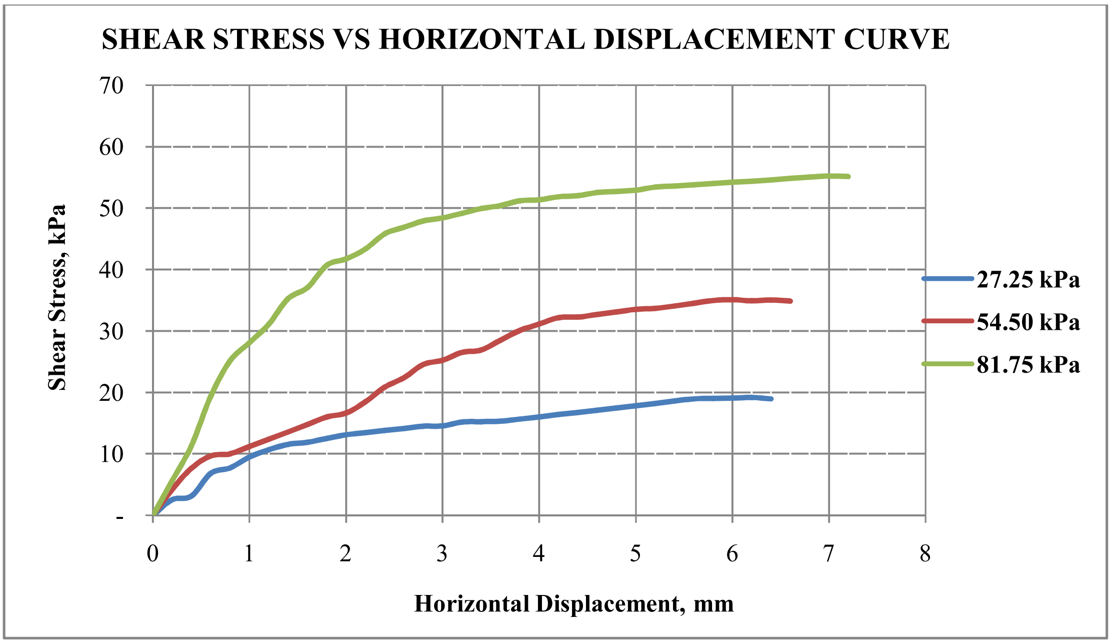


Figure 4.47: Shear stress vs. horizontal displacement curve saturated soil (SR-2)

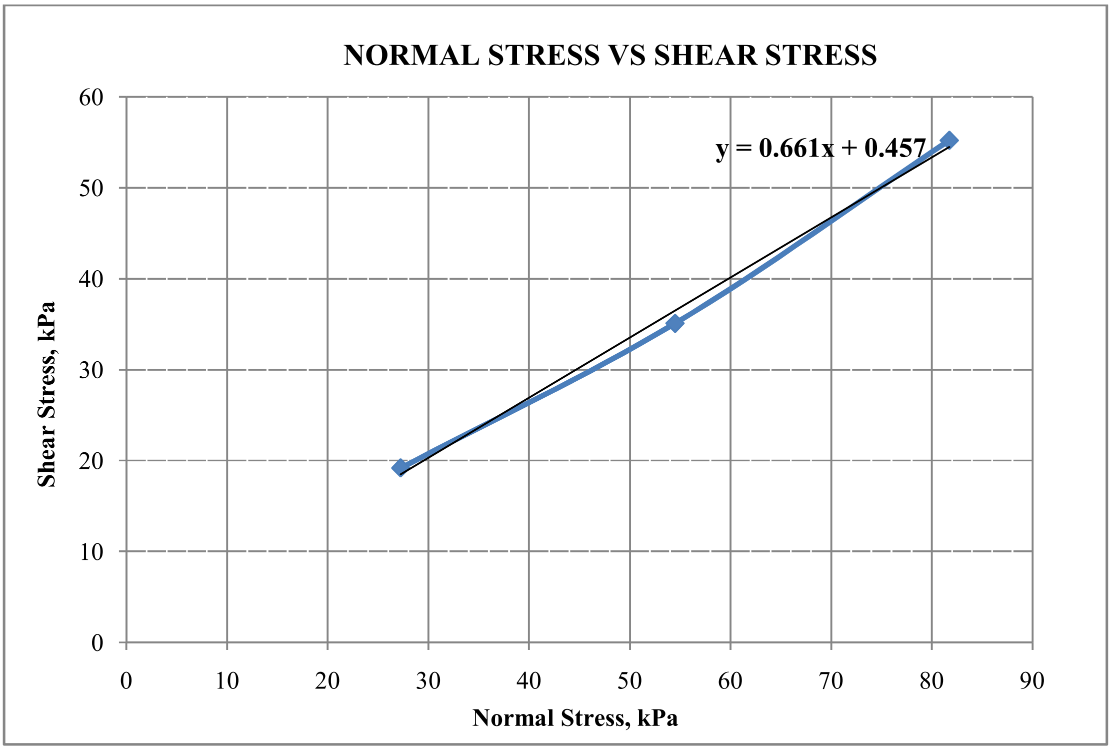


Figure 4.48: Normal stress vs. shear stress graph saturated soil (SR-2)

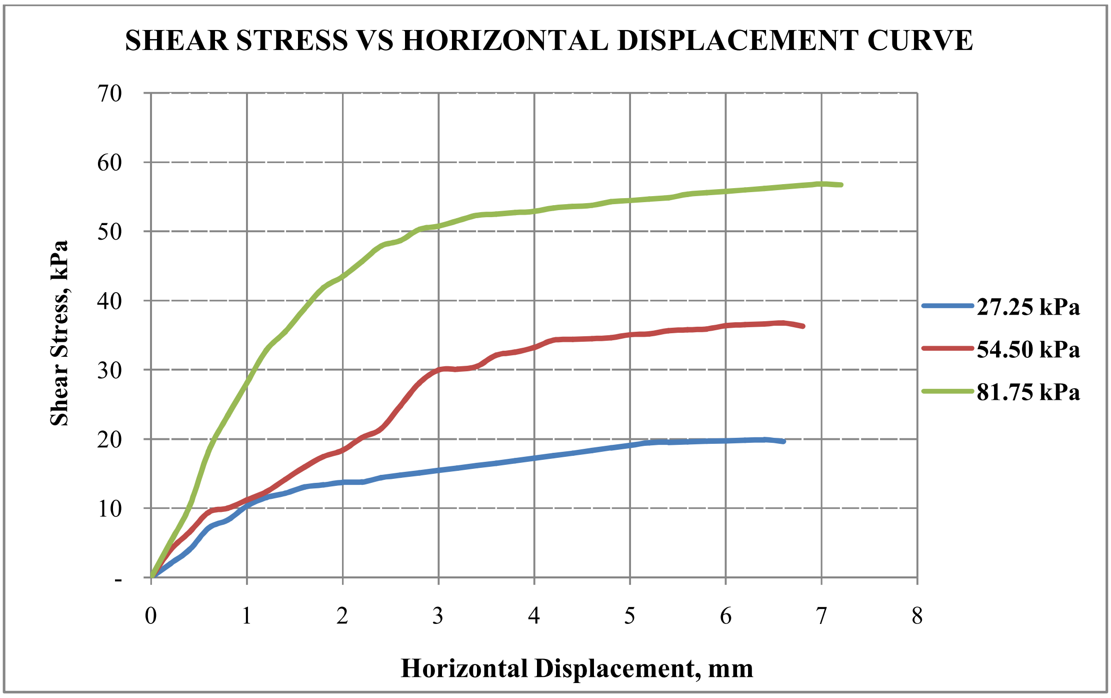


Figure 4.51: Shear stress vs. horizontal displacement curve saturated soil (SR-3)

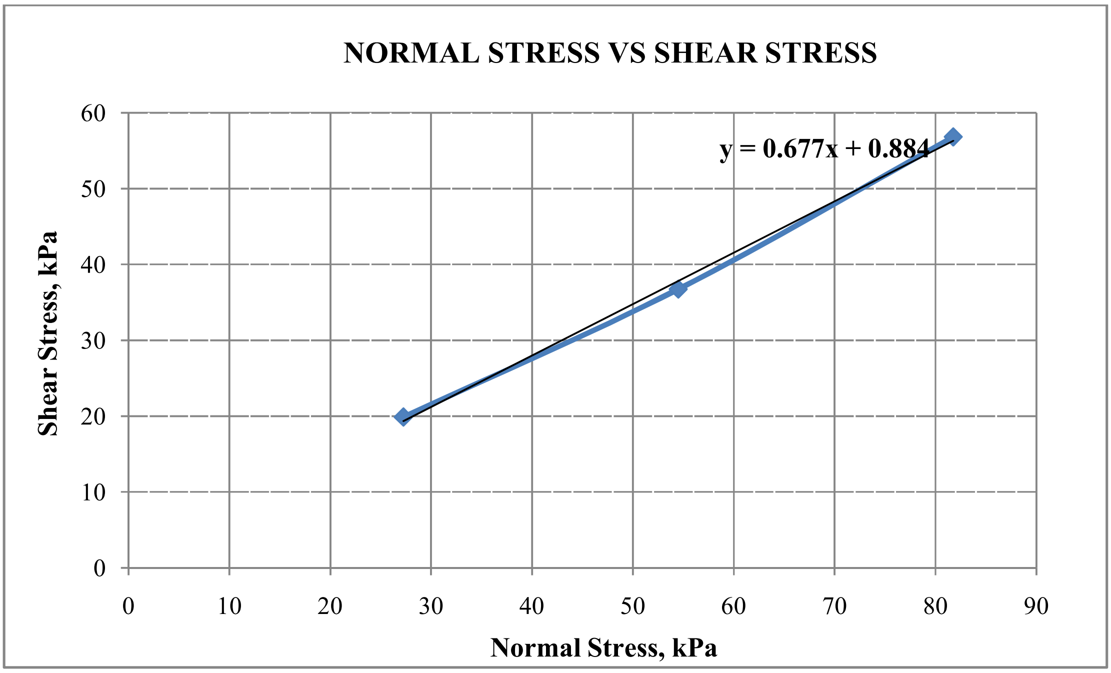


Figure 4.52: Normal stress vs. shear stress graph saturated soil (SR-3)

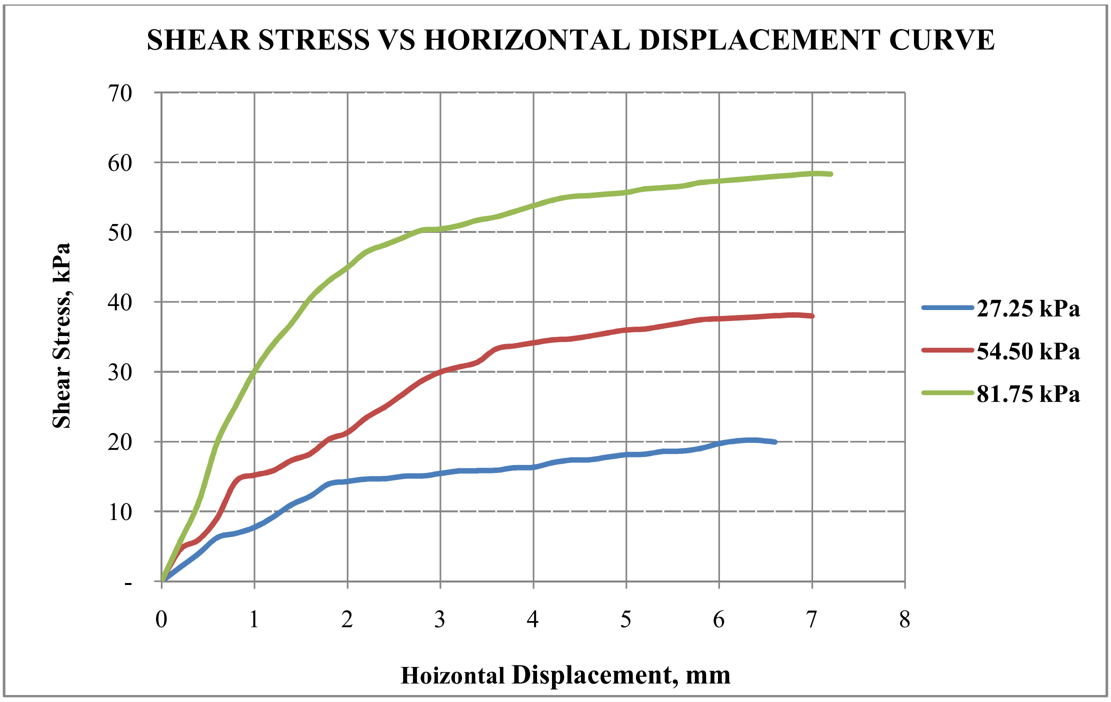


Figure 4.55: Shear stress vs. horizontal displacement curve saturated soil (SR-4)

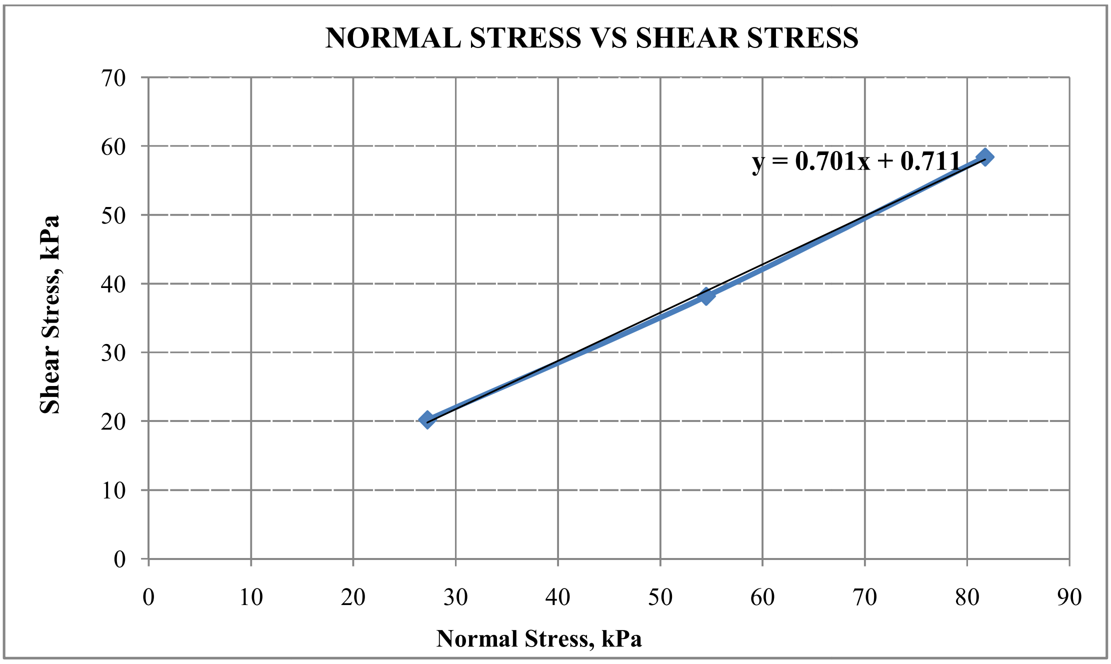


Figure 4.56: Normal stress vs. shear stress graph saturated soil (SR-4)

CHAPTER 5: DISCUSSIONS AND INTERPRETATIONS OF TEST RESULTS

This chapter discusses and interprets the soil samples test results shown in chapter 4.0 aiming to rendering how each soil samples match with the recognized collapse properties of soils. The discussion and evaluation of data is based on all of the available results and observations made during the testing procedure of the collapsible materials. The results are evaluated and discussed (individually or together) the section below.

5.1 Evaluating the Soil Samples from Index Parameters Based on General Properties of Collapsible Soils

As discussed in second chapter, collapsible soils are generally associated with an open structure formed by sharp grains (high void ratio and porosity), low initial density, low natural water content, low plasticity, relatively high stiffness and strength in the dry state, and often by particle size in the silt to fine sand range (Mitchell and Soga, 2005). As problems with collapse are generally associated with silty or sandy soils of low clay content, the identification of the soils will provide a better understanding of the collapse behavior of the soils (Brink et al., 1982).

Now let us summarize the results obtained at section 4.1.2 in terms of grain size as shown in the table below and evaluate the characteristics of soil samples accordingly.

Table 5.1: Summary of percentage of grain sizes of soil samples based on USCS (ASTM) classification system

Sample code	Soil type	Gravel content (%)	Sand content (%)	Silt content (%)	Clay content (%)
SL-1	Silty sand (SM)	7.29	70.11	22.34	0.27
SL-2	Silty sand (SM)	5.66	74.88	19.24	0.23
SL-3	Silty sand (SM)	4.82	78.52	16.47	0.19
SL-4	Silty sand (SM)	5.67	74.54	19.56	0.23
SR-1	Silty sand (SM)	4.03	79.65	15.87	0.46
SR-2	Silty sand (SM)	5.29	79.28	15.25	0.18
SR-3	Silty sand (SM)	3.98	82.17	13.69	0.16
SR-4	Silty sand (SM)	4.36	80.42	15.04	0.17

Table 5.2: Summary of percentage of grain sizes of soil samples based on AASHTO classification system

Sample code	Soil type	Gravel content (%)	Sand content (%)	Silt content (%)	Clay content (%)
SL-1	Fine sand (A-3)	9.50	66.61	22.34	0.27
SL-2	Fine sand (A-3)	7.70	71.63	19.24	0.23
SL-3	Fine sand (A-3)	7.67	74.98	16.47	0.19
SL-4	Fine sand (A-3)	7.89	71.10	19.56	0.23
SR-1	Fine sand (A-3)	5.90	76.65	15.87	0.46
SR-2	Fine sand (A-3)	7.90	75.48	15.25	0.18
SR-3	Fine sand (A-3)	7.64	78.33	13.69	0.16
SR-4	Fine sand (A-3)	7.00	76.91	15.04	0.17

Table 5.3: Void ratio, moisture content, and dry unit weight for some typical soils in a natural state (Braja M. Das, "Principles of Geotechnical Engineering", 7th Edition ©2010 - Table 3.2: "Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural Stat")

Soil type	Natural void ratio, e	Saturated moisture content (%)	Natural dry unit weight, γ_d (kN/m^3)	Remark
Silty sand	0.65	25.0	16.0	

From the above summary tables we can characterize the collapse behavior of the soil samples as shown below. The discussions of these results were done all but as a package; however, individual discussions may be incorporated whenever necessary.

5.1.1 Indicator Analysis from NMC, OMC and Plasticity

According to Clemence (1981), collapsible soils are soils that compact and collapse after they get wet. The soil particles are originally loosely packed and barely touch each other before moisture soaks into the ground. As water is added to the soil in quantity and moves downward, the water wets the contacts between soil particles and allows them to slip past each other to become more tightly packed. Water also affects clay between other soil particles so that it first expands, and then collapses like a pack of cards. Another term for collapsible soils is "hydro-compactive soils" because they compact after water is added.

Zainal et.al (2016), Collapsible soils can generally maintain a metastable structure, which can carry a moderate to relatively high overburden stress at its dry to relatively low moisture content. These meta-structural bonds between particles of unsaturated soils are maintained by the cementation of the clay like fines and also due to matric suction. These types of bonds are sensitive to water and with the increase in moisture content, matric suction decreases and cementation breaks between particles and eventually collapse upon saturation. The obtained natural moisture contents of all the soil samples have relatively low values (range from 6.48 to 9.37 percent) at their in situ condition, which are less than 10 percent. That means for the given volume of soil sample the amount of dry soil particles significantly greater than the available amount of water. Moreover, the results are well behind the average values of natural moisture content stated in Czechoslovak Standard. This standard stated a soil with natural moisture content below 13 percent have a potential to collapse. So it can be understood that all the soil samples can be designated as moderately dry at in situ condition. Therefore, the low moisture content (dryness) of the soil samples is one of the factors that offers the soils little resistance to deformation while nears to saturation.

Many collapsing soils have LL < 45 and PI < 25, and usually much lower, often in the non plastic range. Water content in place that is well below 100 percent saturation is required for collapse but the OMC for maximum collapse is usually between about 13 and 39 percent (Knodel, 1992). Hence, the summarized OMC results show that the non plastic soil samples with the percentage of OMC range from 21 to 30 percent and lie in the stated collapse range. Therefore, these properties are one of the indicators of the soil samples more susceptible to collapse while near to saturation.

5.1.2 Indicator Analysis from Void Ratio, Porosity, and Degree of Saturation

According to Rafie et.al (2008), the magnitude of soil collapsibility usually depends on initial porosity. The basic characteristics of collapsible soils are categorized as: high porosity (more than 40 percent), low saturation (less than 60 percent), high silt content (more than 30 percent and sometimes 90 percent), and rapid softening in the water. However, Schwartz (1985) stated the typical values for the critical degree of saturation above which collapse will not occur will be 61 percent for silty sand and 50 to 60 percent fine silty sands, after Jennings and Knight (1975). In addition, Zainal et.al (2016) stated the collapse rate (settlement/time) which is the slope of the settlement - time curve varies for each degree of saturation and its get steeper with increasing degree of saturation.

As stated in the above Table 5.3, the average value of natural void ratio of silty sands is 0.65; whereas, the summarized results show that void ratio the test results of the soil samples range from 0.96 to 0.98. This means that the obtained initial void ratios are much higher than the mean value. Again, the result of porosity of the soil samples range from 48.94 to 49.82 percent which is greater than 40 percent as stated by Czechoslovak standard and Rafie et.al (2008). Here we can see that as the value of void ratio decreases down to the depth then the value of porosity of soils also decreases. If the soil has high void ratio and porosity means it may have less tendency to control volume change, fluid conductivity and particles movement while subjected to saturation and/ or loading.

Similarly, the result of degree of saturation of the soil samples range from 18.05 to 24.52 percent which is very less of 60 percent which is stated by Czechoslovak standard and Rafie et.al (2008); and it is below critical degree of saturation stated by Jennings and Knight (1975) for silty or fine sands.

Therefore; the void ratio, porosity and degree of saturation values of all the soil samples can be taken as one of the indicators for the soil layers susceptible to collapse while nears to saturation.

5.1.3 Indicator Analysis from Natural Dry Density or Unit Weight

Clevenger (1985) proposed criterion for collapsibility evaluation is based on the soil dry density. He declares if the soil dry density is lesser than $1.28\frac{\mathrm{g}}{\mathrm{cm3}}$ then the soil will collapse after minor water content change. On the other hand, if the soil density is more than $1.44\frac{\mathrm{g}}{\mathrm{cm3}}$ then the lesser collapse settlement could be expected. For medium range of soil density, the medium collapse settlement could be evaluated.

Collapsible soils are basically characterized as soils which have great dry strength and stiffness (Howayek et al, 2011). Collapsible soil deposits share two main features, they are loose, cemented deposits, and they are naturally quite dry (Day, 2000). Collapsible soil can withstand a large applied vertical stress with small amount of compression, but then showed much larger settlement upon wetting, with no increase in vertical stress. Therefore, dry density is one of the crucial geotechnical engineering factors that decide the strength of soils.

The natural dry density and unit weight of all the soil samples range from $1.25\frac{\mathrm{g}}{\mathrm{cm3}}$ to $1.32\frac{\mathrm{g}}{\mathrm{cm3}}$, and $12.22\frac{\mathrm{kN}}{\mathrm{m3}}$ to $12.97\frac{\mathrm{kN}}{\mathrm{m3}}$; respectively. Using Clevenger (1985) collapse potential evaluation criteria, all the soil samples can be categorized as collapse soil with the degree of collapsibility range from medium to high as indicated from Table 4.10 to Table 4.17. Clevenger (1958) also stated that if the soil sample with dry unit weight is less than $12.6\frac{\mathrm{kN}}{\mathrm{m3}}$ have a large collapse settlement; whereas, if the soil sample with dry unit weight is greater than $14\frac{\mathrm{kN}}{\mathrm{m3}}$ have a small collapse settlement. This means that all the soil samples can be categorized as collapsible types of soil.

Again, the values are lower than the mean values stated in Table 5.3. Therefore, we can interpret and evaluate that all soil samples lie under collapsible soil category.

Low unit weights indicate a loose structure, the in place dry unit weight is a good parameter for collapse prediction (Knodel, 1962). However, soils with a collapsible fabric very often have a low dry density (Schwartz, 1985). Brink, Partridge and Williams indicate that these dry densities fall in the range of 0.90 to $1.60 \frac{\mathrm{g}}{\mathrm{cm3}}$. Jennings and Knight (1975), however, warn of the danger of assuming that all soils with a low dry density exhibit a tendency to collapse or, vice versa, which all soils with a high dry density will not collapse. This is also confirmed by Dudley (1970) who indicates that many stable soils have very low dry densities.

In case of this study the dry densities of all soil samples range from $1.25 \frac{\mathrm{g}}{\mathrm{cm3}}$ to $1.32 \frac{\mathrm{g}}{\mathrm{cm3}}$, which are nearly lie in the stated range. Therefore, the natural dry density and unit weight of the soil samples can be one of the indicators of the soil layers have a potential to collapse while nears to saturation.

5.1.4 Indicator Analysis from Soil Classification, Silt Content and Clay Content

Collapsible soils are typically silt and sand size with a small amount of clay and low plasticity index (Pawlak, 1983). Many collapsing soils have usually much lower plasticity, often in the non plastic range (Knodel, 1992). One of basic characteristics of collapsible soils is high silt content and rapid softening in the water (Rafie et.al. 2008). Problems with collapse are generally associated with silty or sandy soils of low clay content. Particle size distribution and Atterberg Limits will assist in identifying these soil types. It is important to take into consideration; however, that high clay content does not necessarily mean that collapse will not occur (Schwartz, 1985).

From sieve analysis of soil samples it was obtained the percentage by mass of fines range from 13.86 to 22.61 passing No. 200 sieve (0.075 mm). As all the soil samples have percentage of fines greater than 12 percent, it was used a single symbol. It was clearly shown (Table 5.1) that the percentage by mass of sand is higher than gravel, silt and clay contents. The soil samples are classified using USCS classification system as non plastic silty sand (SM) with gravel contents range from 3.98 to 7.29 percent, sand content range from 70.11 to 82.17 percent, silt contents range from 13.69 to 22.34 percent, and clay contents range from 0.16 to 0.46 percent.

Again, the soil samples were classified using AASHTO classification system as fine sand (A-3) with gravel contents range from 5.90 to 9.50 percent, sand content range from 66.61 to 78.33 percent, silt contents range from 13.69 to 22.34 percent, and clay contents range from 0.16 to 0.46 percent. The clay content (Table 5.2) of each soil samples is less than one percent by mass.

Therefore; relying on the aforesaid different findings and discussions made by various authors, we can categorize the type of the soil is one of the foremost indicators that the soil layers have the potential to collapse while nears saturation.

Identifying the specific type of soil is one of the indicators for soil susceptible to metastability (Schwartz, 1985 and Pawlak, 1983). But, identifying the soil type depends on the soil classification system used. In USCS, the gravelly and sandy soils clearly are separated; whereas in the AASHTO system, they are not. Symbols that are used in the USCS are more descriptive of the soil properties than the symbols used in the AASHTO system. The classification of organic soils is provided in the USCS; and under the AASHTO system, there is no place for organic soils. So, it is highly recommended to use USCS than AASHTO classification system to classifying soils.

5.1.5 Indicator Analysis from Compaction Behavior

According to ERA Specifications (2002), for earth structures the field compaction is usually call for a minimum of 93 to 95 percent of maximum dry density of that of obtained at the laboratory. Budhu (2011), the following figure illustrated the level of compaction can be attained at two water contents one before the attainment of the maximum dry unit weight (point - D), or dry of optimum (towards Point - A); and the other after the attainment of the maximum dry unit weight (point - E), or wet of optimum (towards point - B). Compact the soil wet of optimum for collapsible and expansive soils where soil volume changes from changes in moisture conditions are intolerable.

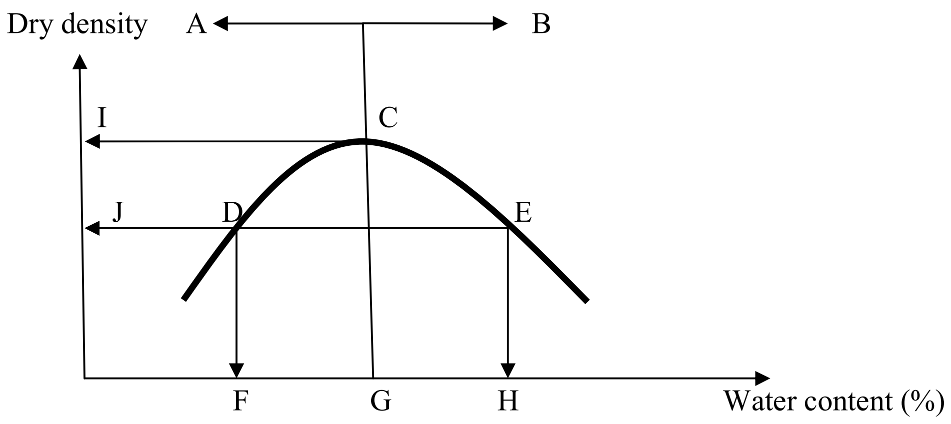


Figure 5.1: Typical illustration of soil compaction curve

From figure 5.2, the optimum moisture content and maximum dry density of soil test occur at point - C and projected straight downward to point - G for OMC and straight to leftward at point - I for MDD. However, the acceptable range of dry density is a minimum of 93 to

95 percent of MDD within point I and point – J; and the acceptable range of water content is plus or minus 2 percent of OMC within point - F and point - H.

The following table evaluate the in situ dry densities and moisture content versus the maximum dry density and moisture contents of the soil samples to interpret whether the soil samples may used as a roadbed material over which road prism to be constructed at in - situ conditions.

Table 5.4: Evaluating MDD from ERA Specification 2002 (Series 4000: Earthworks)

Sample code	OMC (%)	NMC (%)	Bulk dry density (g/cm^3)	MDD (g/cm^3)	93% of MDD (Roadbed)	Remark
SL-1	21.00	6.48	1.25	1.40	1.30	
SL-2	21.00	8.71	1.26	1.38	1.28	
SL-3	22.90	7.53	1.28	1.34	1.25	
SL-4	22.80	8.31	1.28	1.39	1.29	
SR-1	27.20	9.26	1.25	1.36	1.26	
SR-2	28.00	6.59	1.27	1.33	1.24	
SR-3	30.00	9.37	1.29	1.34	1.25	
SR-4	25.50	8. 50	1.32	1.35	1.26	

After Wagner (1957); silty sand (SM) soil groups have semi - pervious to previous permeability when compacted, good shearing strength when compacted and saturated, low compressibility when compacted and saturated, and fair workability as a construction material. According to AASHTO classification system also the rate of fine sand (A-3) soil groups as subgrade material is excellent to good.

From Table 5.4, it can be evaluated that the bulk dry density versus maximum/ minimum required dry densities of the soil samples; as well as, the natural and optimum moisture contents of soil samples were compared and analyzed based on ERA specification (2002) so as to use as roadbed construction material at in - situ conditions. The following table illustrates the evaluation of soil samples suitability as roadbed material.

Table 5.5: Evaluating OMC from ERA Specification 2002 (Series 4000: Earthworks)

Sample code	Acceptable moisture content of in-situ compaction (OMC $\pm$ 2%)	Status for roadbed at in-situ conditions	Remark
SL-1	Not OK	Not Good	
SL-2	Not OK	Not Good	
SL-3	Not OK	Good	At margin
SL-4	Not OK	Not Good	
SR-1	Not OK	Not Good	
SR-2	Not OK	Good	At margin
SR-3	Not OK	Good	At margin
SR-4	Not OK	Good	At margin

Soils with high silt contents are a common occurrence and can exhibit low strengths, and minimal bearing capacity, causing widespread construction and performance problems. These soils are highly moisture sensitive and their stability is greatly influenced by the degree of densification achieved during compaction. The strength and stiffness of silty subgrade soils is also greatly reduced when moisture infiltrates the compacted soil during post construction period. When considered for road subgrade, wet silts or those located in areas with a high water table, soil compaction efforts and construction traffic can produce detrimental pumping action, caused by the redistribution of water due to an uplifting effort. A modification of the moisture content produces negative effects on construction parameters such as soils strength and dry unit weight. This can be seen in the field by observing the behavior of the soil immediately under the compactor or the wheels of heavily loaded equipment. When the soil is too wet and the applied compaction energy is too great, pumping or weaving will occur as the wheel shoves the weaker soil ahead of its motion. In several cases reports indicated the fabric separating the subgrade from the sub base, was pushed up by an uplifting effort created by pumping phenomenon. The high silt soils when wet and located under pavement subgrade or are used as pavement embankments can constitute a real challenge during road construction phase (Bogdan, 2005).

From Table 5.5; soil samples SL-3, SR-2, SR-3 and SR-4 are rated as good to be used as roadbed material as they are marginally fulfilling the minimum required dry densities at in-situ compaction conditions; whereas, SL-1, SL-2, SL-4 and SR-1 are not. However, the in-situ moisture contents of all the soil samples are well behind the limits. This means that there is still a chance for the soil samples to undergo either linear or volumetric deformations for any applied stress.

As soils with high silt contents have also a potential to heave while further increase in moisture content, it is very difficult to use all the soil samples as roadbed material at their in-situ compaction conditions. While a soil mass (near to maximum dry density with moisture content well below/ above OMC) is sheared, it tends to expand (dilate) and gets looser. Usually this expansion is not uniform; some parts of the soil mass are looser than other parts. The flow rate of water in the soil will increase as water can easily (compared to the intact one) flow through the looser parts, possibly leading to collapse.

Therefore, characterizing the in - situ compaction conditions and their respective moisture content are vital for the identification of and prediction of collapsibility potential of problematic soils.

5.2 Identification of Collapsibility Potential of Soil from Empirical Correlations

5.2.1 Clevenger (1985)

Howayek et.al (2011) stated that previously, in 1958, Clevenger developed an experimental soil collapsibility potential identification criterion from their dry unit weight. If the dry unit weight of a soil is less than$12.6\frac{\mathrm{kN}}{\mathrm{m3}}$, large collapse settlement will occur. If dry unit weight of the soil is larger than$14\frac{\mathrm{kN}}{\mathrm{m3}}$, small collapse settlement will occur. Rafie et.al (2008) stated also Clevenger, in 1985, proposed criterion for collapsibility evaluation based on the soil dry density. He declares if the soil dry density is lesser than $1.28\frac{\mathrm{g}}{\mathrm{cm3}}$ then the soil will collapse after minor water content change. On the other hand, if the soil density is more than $1.44\frac{\mathrm{g}}{\mathrm{cm3}}$ then the lesser collapse settlement could be expected. For medium range of soil density, the medium collapse settlement could be evaluated.

The author used the later criteria to identify the collapsibility potential of the soil samples, because the earlier one did not stated the degree or severity of the collapse problem that will occur. Therefore, as of Clevenger (1985), the soil samples SL-1, SL-2, SR-1 and SR-2 are highly collapsible soils; while, soil samples SL-3, SL-4, SR-3 and SR-4 are moderately (medium) collapsible soils.

5.2.2 Handy (1973)

According to Howayek et.al (2011), Handy (1973) studied Iowa loess with clay and determined that when clay content is less than 16 percent, soils are subject to a high probability of collapse. For clay content in the 16 to 24 percent range, soils are likely to collapse. For clay content in the 24 to 32 percent range, the probability of collapse is 50 percent. If the clay content is greater than or equal to 32 percent, soils are usually safe from collapse. Therefore; according to Handy (1973), all the soil samples are identified as highly collapsible soils.

5.2.3 Feda (1966)

According to Briaud (2013), Feda (1966) developed an experimental soil collapsibility potential identification criterion from natural water content, soil saturation ratio, porosity, plastic limit, and plasticity index of properties soil as shown in Table 2.2.

This method is not applicable for the identification of the collapsibility potential of non plastic soils. However; as all the obtained values of porosity greater than 40 percent, the soil samples can be classified as collapsible soils but still the degree or severity of collapse problem was not stated.

5.2.4 Denisov (1964)

According to Rafie et.al (2008), Denisov (1964) proposed criterion if the ratio of void ratio in natural state to the ratio of void ratio at liquid limit water contents is greater than 1, then the soil is collapse susceptible. However; this method is not applicable for the identification of the collapsibility potential of non plastic soils.

5.2.5 Gibbs and Bara (1962)

According to Briaud (2013), Gibbs and Bara (1962) proposed criterion based on dry density, natural void ratio, and liquid limit of the soil as shown in Table 2.2. However; this method is also not applicable for the identification of the collapsibility potential of non plastic soils.

5.2.6 Priklonski (1952)

According to Howayek et.al (2011), Priklonski (1952) criterion utilizes the liquidity index to estimate the degree of collapsibility. Specifically, if the liquidity index is less than zero, soils have a high collapsibility because they are in a very dry state and thus susceptible to water infiltration. When the liquid index is larger than 0.5, soils are not likely to collapse. However; this method is also not applicable for the identification of the collapsibility potential of non plastic soils.

5.2.7 Abelev (1948)

According to Rafie et.al (2008), Abelev (1948) was the first researcher who proposed the criterion for evaluation of soil collapsibility potential due to variation of soil void ratio before and after saturation. If the collapse index (C_p) is greater than 2 percent, then the soil will be susceptible to collapse.

Therefore; according to Abelev (1948), all the soil samples are identified as highly collapsible soils. Here it is observed that the collapse potential of soil layers was increasing though depth, and looks like the collapse potential of soil layers more likely depend on the amount of silt content and degree of saturation. As the year (1948) indicates of Abelev can be named as a founder of characterizing the behaviour collapsible soils, but he did not mentioned the critical state of stress where collapse will be considered to happen.

5.2.8 Czechoslovak Standard (1994)

According to Howayek et.al (2011), the Czechoslovak Standard (Klukanova and Frankovaska, 1994) applied six critical conditions to classify the collapse potential. Specifically, the standard defines the following threshold values for collapse: silt content greater than 60 percent, clay content less than 15 percent, degree of saturation less than 60 percent, liquid limit below 32 percent, porosity greater than 40 percent, and natural water content less than 13 percent. Therefore; according to the Czechoslovak Standard (1994), all the soil samples are identified as collapsible soils. But, the degree or severity of collapse problem was not stated.

5.3 Identification of Collapsibility Potential of Soil from Collapse Test

Collapse and consolidation settlements are different in their characteristics. Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability owing to drainage of some of the pore water, the process continuing until the excess pore water pressure set up by an increase in total stress has completely dissipated. Consolidation settlement will result, for instance, if a structure is built over a layer of saturated clay or if the water table is lowered permanently in layer overlying clay (Craig, 2004). However, collapse settlement is very different from traditional consolidation as no water is being expelled and in actual fact the soil will be absorbing water and progressively losing strength (Dudley, 1970). The problem is associated with a change in the compression characteristics of the soil effectuated by capillary forces resulting from partial saturation (Jennings and Knight, 1975).

This study used single oedometer test from procedures stated on ASTM - D5333 standard test method for measurement of collapsible soils. The results obtained from this test were analyzed and interpreted the collapse problem with respect to the collapse index (I_e) and collapse potential (C_p) of soil given on Table 2.3 of soil given in Table 2.2, respectively.

It was shown (Figure 4.9 to 4.24) that the compression curves of all soil samples after collapse test. According to ASTM - D5333, Point – A was describing the starting point at which seating stress (5 kPa) applied on the soil specimen; Point – B describes the point at which the soil specimen is subjected to critical stress (200 kPa) before inundating; and Point – C describes the point at which the soil specimen is subjected to critical stress (200 kPa) after inundating. The collapse index and collapse potential of soil specimens was measured, however, at this critical state of loading by taking the difference of vertical strain or void ratio of soil samples before and after wetting. The collapse index and collapse potential of soil specimens for each load increment can be measured by drawing straight line from Point – A to Point – C, then measure the vertical difference.

According to ASTM - D5333, within 5 min of applying the seating stress, apply load increments each hour at natural water content until the appropriate vertical stress (200kPa) is applied to the soil.

The following two tables (Table 5.6 and 5.7) illustrate grand summary of results of degree of collapse, collapse index/ potential and influential soil properties.

Table 5.6: Summary of results of degree of collapse, collapse index/ potential and influential soil properties of RHS soil specimens

Description	**Values**			
	SR-1	**SR-2**	**SR-3**	**SR-4**
I_e	4.89	4.31	3.96	3.81
Severity	Moderate	Moderate	Moderate	Moderate
C_p	4.89	4.31	3.96	3.81
Severity	Medium collapsibility	Medium collapsibility	Medium collapsibility	Medium collapsibility
e_o	0.99	0.97	0.96	0.96
η (%)	49.82	49.31	49.02	48.96
ρ_d (g/cm^3)	1.25	1.27	1.29	1.32
ω_o (%)	9.26	6.59	9.37	8.50
S (%)	24.52	18.05	20.44	21.94
Silt, (%)	15.87	15.25	13.69	15.04
Clay, (%)	0.46	0.18	0.16	0.17

Table 5.7: Summary of results of degree of collapse, collapse index/ potential and influential soil properties of LHS soil specimens

Description	**Values**			
	SL-1	**SL-2**	**SL-3**	**SL-4**
I_e	5.89	5.64	4.97	4.52
Severity	Moderate	Moderate	Moderate	Moderate
C_p	5.89	5.64	4.97	4.52
Severity	High collapsibility	High collapsibility	Medium collapsibility	Medium collapsibility
e_o	0.98	0.97	0.97	0.96
η (%)	49.40	49.16	49.13	48.94
ρ_d (g/cm^3)	1.25	1.26	1.28	1.28
ω_o (%)	6.48	8.71	7.53	8.31
S (%)	19.00	23.26	23.64	22.77
Silt, (%)	22.34	19.24	16.47	19.56
Clay, (%)	0.27	0.23	0.19	0.23

Load increments should be 12, 25, 50, 100, 200, etc. kPa. The duration between load increments prior to wetting is limited to 1 hour to prevent excessive evaporation of moisture from the specimen that would cause erratic results. In soils with high permeability, collapse may occur rapidly and time dependency may be difficult to measure.

It is observed from Table 5.6 and 5.7 (consistent with data available in the literature) that the most significant collapse is observed in the soil specimens that notably have values of higher void ratio (range from 0.99 down to 0.96), higher porosity (range from 49.82 down to 48.94 percent), and lower dry densities (range from 1.25 up to $1.32 \frac{g}{cm3}$). It was clearly shown that when it goes from soil specimen code SL-1 to SL-4 or SR-1 to SR-4, the amount of collapse index or collapse potential of soil layers constantly deceased (range from 5.89 down to 3.81 percent) as the values of void ratio and porosity constantly deceased, and the dry density getting increased. This means; as the depth at which the soil samples were taken increases, the resistance to collapse settlement or deformation of soil layers increases.

The other important factors that influence the soil layers susceptible to collapse (like natural moisture content, degree of saturation, silt content and clay content) still have significant contribution on collapse problems. In this study, however, it can clearly stated on summary Table 5.6 and 5.7 that when it goes from soil specimen code SL-1 to SL-4 or SR-1 to SR-4, regardless of the constant decline of amount of collapse index or collapse potential, void ratio and porosity, as well as the constant increment of dry density of soil layers; the values of natural moisture content, degree of saturation, silt content and clay content of soil samples did not justify their influence as much significant than the void ratio, porosity and dry density of soil samples.

For instance, the degree of saturation of soil samples SR-1 (24.52 percent) and SL-3 (23.64 percent) are greater than soil samples SR-2 (18.05 percent) and SL-4 (22.77 percent), respectively. But, the collapse index or collapse potential value of soil samples SR-1 (4.89 percent) and SL-3 (4.97 percent) are lower than soil samples SR-2 (4.31 percent) SL-4 (4.52 percent), respectively. Rafie et.al (2008) stated the basic characteristics of collapsible soils are categorized as: low saturation (less than 60 percent) and rapid softening in the water. However, Schwartz (1985) stated the typical values for the critical degree of saturation above which collapse will not occur will be 61 percent for silty sand and 50 to 60 percent fine silty sands, after Jennings and Knight (1975). In addition, Zainal et.al (2016) stated the collapse rate varies for each degree of saturation and its get steeper with increasing degree of saturation. This implies that the higher degree of saturation, the lesser will be the value of collapse potential of soil samples. But, the above findings of this study did not justify this.

Similarly, the silt and clay contents of soil samples SR-1 (silt = 15.87 percent and clay = 0.46 percent), SR-2 (silt = 15.25 percent and clay = 0.18 percent), SL-3 (silt = 16.47 percent and clay = 0.19 percent) and SL-4 (silt = 19.56 percent and clay = 0.23 percent) have different percentages of silt and clay contents as shown. Schwartz (1985) stated problems with collapse are generally associated with silty or sandy soils of low clay content. It is important to take into consideration however that high clay content does not necessarily mean that collapse will not occur. Rafie et.al (2008) also stated one of basic characteristics of collapsible soils is high silt content and rapid softening in the water. Therefore, this study justifies the whether the higher or the lower the silt and clay contents of the soil samples did not guarantee the occurrence and degree of collapse potential of soil samples. Howayek (2011) stated that Czechoslovak Standard stated soils with natural moisture content below 13 percent have a potential to collapse. According to this study the natural moisture contents of all the soil samples vary erratically and less than 10 percent; and only justify that the dryness of the soil layers.

5.3.1 Comparing ASTM - D5333 and Jennings & Knight (1975) Methods

The author of this study identifies the following similarities and differences between ASTM - D5333 and Jennings & Knight (1975) methods of measuring the collapsibility potential of soils.

Similarities:

Both methods:

- Use oedometer test for collapse measurement.
- Use undisturbed and remolded soil specimens for testing.
- Use 200 kPa as the critical stress state of loading.
- Stated the severity or degree of collapse problem.
- Can be expressed in terms of typical compression curves.
- Based on Abelev's (1948) collapse criteria.

Differences:

Table 5.8: Comparison of ASTM - D5333 and Jennings & Knight (1975) methods

ASTM - D5333	Jennings & Knight (1975)
- Uses single oedometer test for testing of specimens.	- Uses double oedometer test for testing of specimens.
- Uses log pressure versus vertical strain compression curve.	- Uses void ratio versus log pressure compression curve.
- Collapse index can be measured both from vertical displacement and void ratio.	- Collapse index can be measured from void ratio.

ASTM - D5333	Jennings & Knight (1975)
- Soils with collapse index values less than 0.1 percent are non collapsible soils.	- Soils with collapse potential values less than or equal to 1 percent are non collapsible soils.
- Soils with collapse index values greater than 10 percent are severely collapsible soils.	- Soils with collapse index values greater than 20 percent are extremely collapsible soils.
- Relatively modern (Developed in 2003).	- Primitive (Developed in 1975).
- Fair prediction of collapse settlement.	- Over prediction of collapse settlement.

5.4 Stress – Strain Properties of Soil Samples from Direct Shear Test

Direct shear test is useful to study the stress – strain properties of cohesionless soil like sand and gravel. From this test we can obtain shear strength parameters like angle of internal fiction (ϕ) and cohesion (c). However, cohesion value for sand is generally taken as zero.

The shear strength is estimated as:

$$\tau = c + \sigma' \tan\phi \text{ -- } (3)$$

Where: τ = shear strength of soil (kPa); c = cohesion of soil (kPa); σ' = confining stress (kPa); and ϕ = angle of internal fiction (degree)

From equation (3), we can understand that sand typically has negligible shear strength at zero confining stress and as it is generally modelled with a cohesion value of zero. But, still if we go through direct shear test of this study, it gave values range from 0.219 to 2.417 kPa which is an erroneous estimated caused by combination of normal load and angle of internal friction. The angle of internal friction value is typically estimated based on the density of sand as very loose, loose, dense, medium dense, or very dense.

As shown in Figure 4.25 to 4.56, graphical representations of shear stress versus horizontal displacement and normal stress versus maximum shear stress were plotted for all soil samples to characterize and determine the shear strength parameters of soils. This test was done both on saturated and unsaturated soil samples to identify the influence shear parameters on the collapsibility potential of the soils. All tests were performed in a 60 x 60 x 25 mm direct shear device. The tests were carried out with normal pressures of 27.25 kPa, 54.50 kPa and 81.75 kPa. Eight tests were thus carried out for each saturation conditions to determine the shear strength parameters (Maximum shear, c and ϕ) under unconsolidated undrained conditions. The following two tables summarize the shear parameters and collapse index/ potential values of unsaturated and saturated soil samples.

Table 5.9: Summary shear parameters of unsaturated samples vs. collapse index values

Sample code	**Collapse index, (I_e/C_p)**	**Maximum shear at 27.25 kPa**	**Maximum shear at 54. 50 kPa**	**Maximum shear at 81.75 kPa**	**Cohesion, c (kPa)**	**Angle of internal fiction, ϕ°**
SL-1	5.89	19.88	36.61	57.03	-	34.25
SL-2	5.64	20.59	36.89	57.77	-	34.29
SL-3	4.97	20.83	36.01	59.05	-	35.05
SL-4	4.52	20.91	38.30	60.23	-	35.79
SR-1	4.89	20.51	37.20	56.71	-	33.58
SR-2	4.31	21.15	37.81	57.57	-	33.74
SR-3	3.96	21.30	39.09	58.76	-	34.49
SR-4	3.81	21.47	39.56	60.37	-	35.03

Table 5.10: Summary shear parameters of saturated samples vs. collapse index values

Sample code	Collapse index, (I_c/C_p)	Maximum shear at 27.25 kPa	Maximum shear at 54. 50 kPa	Maximum shear at 81.75 kPa	Cohesion, c (kPa)	Angle of internal fiction, $\phi°$
SL-1	5.89	19.11	35.71	55.33	-	33.58
SL-2	5.64	19.42	35.53	56.28	-	34.06
SL-3	4.97	20.05	36.16	57.55	-	34.53
SL-4	4.52	20.44	37.24	59.05	-	35.30
SR-1	4.89	18.80	35.48	54.26	-	33.02
SR-2	4.31	19.18	35.09	55.22	-	33.46
SR-3	3.96	19.88	36.75	56.81	-	34.10
SR-4	3.81	20.20	38.16	58.41	-	35.03

The presence of a collapsible fabric would result in an immediate decrease of shear strength if saturation occurs (Brink, 1985). The relationship between dilatancy indexes and normalized of shear stress is quite unique. They are not depending on the net normal stress and initial water content. This means that the effect of different water content on the shear strength directly reflect the changing of the dilatancy. It is also found that even the collapsibility is more without shearing and the horizontal movement causes higher shear on soaking (Hormdee, 2005). Therefore, we can observe that more attention should be paid in designing with the collapsibility under shear stress. However, a direct correlation between the collapsibility and the shear strength of the soils could not be found. Specially, for the analysis of rain - induced slope failures, it is important to measure unsaturated shear strength parameters at low suction where slopes become unstable (Gallage et.al, 2010).

The correlation between the shear strength of the soils and a number of variables such as moisture content, clay content and collapsibility and the angle of internal friction of the soil could not be found. The shear strength of a soil can thus not be determined by studying only the moisture content, clay content and collapsibility (Gildenhuys, 2010).

As illustrated in Figure 4.25 to 4.56, the shear stress increases gradually with horizontal shear displacement. It also increases in strength due to the confining pressure (Table 5.9 and 5.10). The shear strength of saturated soil samples are also less than that of unsaturated soil samples for the same test pit and confining pressure. Though, the values of shear strength parameters between saturated and unsaturated stress - strain behavior of soils is small, the results shown (Table 5.9 and 5.10) suggest that the specimen in lower water content has slightly higher peak shear strength than that in higher water content.

By nature clays soils used as cementing agents in soil particle masses. Clay soils also have high plasticity property and the tendency to lose their strength after wetting. As the results from soil classification shown (Table 5.1 and 5.2), the clay content of all soil samples are insignificant (less than 1 percent). Majority of components (more than 70 percent) in the soil specimens are sands, and the fiction between sand particles is due to sliding and rolling fiction and interlocking action. Therefore; when the soil specimens are subject to wetting and shear, the horizontal slip surface of the saturated soil specimens may not have significant difference in developing the resistance to shearing as compared to unsaturated soil specimens.

The shear stress and angle of internal friction of soil samples at both saturation conditions also increases as the collapse index/ potential of soil samples decreases down to depth. In contrast to Gallage et.al (2010) and Gildenhuys (2010), this signifies the shear resistances of soils have a significant factor for collapsibility potential of soils.

Yet again, down to the depth, the overburden pressure (it can be simulated as confining pressure on the field for the beneath soil layers) also increases and give rise to increasing the shear strength and angle of internal fiction while collapsibility potential of soil layers decreases. This also contradicts: “Many collapsing soils exist to considerable depth, often 30 meters or more and up to 200 meters (Knodel, 1992)”.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

While studying the collapsibility potential of the soils from the study area, the following conclusions were reached:

- The soil properties of the study area classified as yellowish brown silty or fine sands with medium to highly collapsible nature.
- The collapsible nature of the soil layers is one of the major factors that caused the major ground crack (surface rupture) to occur at the area of study. The collapse problem of soil layers was significantly from volume reduction which may be inspired by infiltration of water from the surface runoff due to the highest of the previous six years annual precipitation falls (510.2 mm) in March 2016. Hence, the infiltration of water gave rise to the increment of the in – situ stress (internal stress from self weight) of the soil layers; as well as, increase the ground water table at deep depth that leads collapse to occur from volume reduction.
- The soil layers at shallow depths have relatively lager collapsibility potential than soil layers exist at deep depths.
- Collapsible soils are susceptible to collapse when subjected to variations in moisture and amount of loading.
- Sampling disturbance is the major concern in collapse test. Therefore, hand-cut samples are more preferable for oedometer collapse test.
- Considering the importance of soil type in collapsibility potential of soil, then Unified Soil Classification System (USCS) is better describes the specific type of soil than AASHTO classification system.

- Non - plasticity nature of soil can prohibit the direct use of some existing experimental correlations for the identification of collapsibility potential of problematic soil. Whereas, direct measurement of collapsibility potential of soil from one - dimensional consolidation for collapse test affected by degree of saturation and the amount of loading. Therefore, one - dimensional consolidation for collapse test is the best describes collapse potential of soil for correlations.
- The direct shear test results revealed that soil in wetting is more contractive than the soil in drying at the same normal or confining stress. This means that the stress – stress properties of soils have a crucial effect on the collapsibility potential of soil layers.

6.2 Recommendations for Future Studies

This work presents a study toward understanding the identification of collapsibility potential of soils. However; the collapsibility potential of soil is influenced by numerous factors, this study cannot address all these factors. The future research is recommended to address issues that would require further consideration were given and summarized below.

- The study carried out in this thesis was based on the application of constant/ static loading conditions. As roads are mostly exposed to dynamic loads from traffic; therefore, further studies should be carried out to identify the response of collapsible soils to dynamic loading conditions before and after wetting; and strength of collapsible soils after wetting.
- This study incorporated analytical analysis of test results using geotechnical engineering formulas. However; in future studies, the application of numerical method analysis from constitutive soil model development considering the role of cementation, including soil

suction or pore water pressure, in collapsibility potential of problematic soils should be further identified.

- Mostly, in most roadway designs, much attention was given to the strength behaviors like: CBR and compaction of the materials. However; from this study, it is observed that collapsible soils are stiff at dry conditions but less resistance to deformation (collapse settlement) while subjected to significant moisture variations. Therefore; additional field investigations (like sausage test, ultrasonic test etc.) should be incorporated for the determination of collapsibility potential of soils to protect further damage on roads during and after construction.
- Once the collapse problems identified then the next step should be selecting the most economical techniques used to enhance the strength characteristics. Therefore, in future studies, it is very important to make studies on how to upgrade the engineering properties and performance of collapsible soils like chemical stabilization, Vibroflotation (grouting), Dynamic deep compaction, In - situ densification by surface rolling, Excavation and compaction, Soil replacement etc.
- Other factors like tectonic effects from earthquake should be further analyzed and investigated.

References

1. Ali, N.A. 2016. Controlling Collapsibility Potential by Partial Soil Replacement. King Marrito Institute for Engineering, Egypt. Online ISSN: 2249-4596 & Print ISSN: 0975-5861.

2. Al-Juari, K.A.K. 2009. Volume Change Measurement Of Collapsible Soil Stabilized With Lime And Waste Lime. University of Mosul, Libya. Tikrit Journal of Eng. Sciences Sciences/Vol.16/No.3: (38-54).

3. Assallay, A.M. 1998. Structure and hydro-collapse behavior of loess. PhD Thesis. Loughborough University. United Kingdom.

4. American Society for Testing Materials Committee, 2003. Designation: D 5333. 4 p.

5. Anderson, F.J. 1968. Collapsing soils and their basic parameters in an area in the Tucson, Arizona vicinity. M.Sc Thesis. University of Arizona, Arizona. USA. 104 p.

6. Ayadat,T. and A.M. Hanna. 2012. Assessment of Soil Collapse Prediction Methods. Concordia University, Quebec. Canada. 10.5829/idosi.ije.2012.25.01b.03

7. Barbu. B. 2004. Identification and Stabilization of Problematic Silts. PhD Thesis. University of New Orleans. Romania. 225 P.

8. Briaud, J.L. 2013. Geotechnical Engineering: Unsaturated and Saturated Soils. Published by John Wiley & Sons, Inc., Hoboken, New Jersey. P 424 – 425.

9. Brink, G.E. 2011. The Influence of Soil Suction on the Collapse Settlement of Different Soils in South Africa. M.Sc Thesis. University of Pretoria, Petoia. South Afica. 98 p.

10. Bureau of Reclamation, Denver. USA. 1992.

11. Das, B.M. 2010. Principles of Geotechnical Engineering. Published by Global Engineering: Christopher M. Shortt. Seventh Edition. P 1- 126.

12. Das, B.M. 2008. Advanced Soil Mechanics. Published by Taylor & Francis 270 Madison Ave, New York, USA. Third Edition. P 276 – 352.

13. Fall, M., R. Azzam and M. Sarr. 2006. Characterization of problematic soils for slope and foundation stability: case study from Dakar. University of Quebec, Canada. Aachen University, Germany. Gifu University, Japan. IAEG2006 - 103.

14. Frankl, A., et.al, (2013). Quantifying long-term changes in gully networks and volumes in dryland environments: The case of Northern Ethiopia. Geomorphology 201 (2013): p 254–263.

15. Gallage, C.P.K and T. Uchimura. 2010. Direct shear testing on unsaturated silty soils to investigate the effects of drying and wetting on shear strength parameters at low suction. © The Japanese Geotechnical Society.

16. Geological Survey of Ethiopia, Addis Ababa, Ethiopia. 2014.

17. Gildenhuys, N. 2010. MSc Eng Thesis. The Occurrence and Extent of Collapse Settlement in Residual Granite in the Stellenbosch Area. University of Stellenbosch, Cape Town, South Africa.178 p.

18. Hormdee, D., H. Ochiai and N. Yasufuku. 2005. Direct Shea and Compression Behaviors for an unsaturated compacted soil with water content and matric suction measurement. Taylor and Francis Group, London, ISBN 0415383374.

19. School of Graduate Studies. 2012. SGS Guideline, Hawassa University, Hawassa, Ethiopia. 88 p.

20. Indiana Department of Transportation and Purdue University. 2011. Joint Transportation Research Program. West Lafayette, Indiana, USA. 75 p.

21. International Journal of Geosciences. 2012. Scientific Research, Iran. 6 p.

22. International Conference on Case Histories in Geotechnical Engineering. Chicago, Apr 29th - May 4th, 2013.

23. International Scholarly and Scientific Research & Innovation. 2014. International Journal of Environmental, Chemical, Ecological, Geological and Geophysical Engineering. P 849 - 854.

24. International Journal of Scientific & Engineering Research. 2017. Volume 8, Issue 8, ISSN 2229-5518. P 125 – 128.

25. Khelifa,A., L.M. Salah and M. Farid. 2010. Prediction of Collapsible Soils by Cone Pentameter and Ultrasonic Tests. University of Batna, Studia Geotechnica et Mechanica, Algeria. Vol. XXXII, No. 2.

26. Multi-Regional Training and Certification Task Group. 2006. Soil Sampling and Testing for Transportation Engineering Technicians. 352 p.

27. National Metrology Agency of Ethiopia, SNNPRG Service Center, Hawassa, Ethiopia

28. Owolabi, T.A. and Samuel O.A. 2014. Geotechnical Properties of a Typical Collapsible Soil in South-Western Nigeria. Federal University of Technology Akure, Nigeria. Vol. 19.

29. Pan-Am CGS Geotechnical conference. Amundaray, J.I. and Alvaro Boiero. Caracas, Venezuela, 2011.

30. Rafie, B.M.A., R.Z. Moayed. 2008. Evaluation of Soil Collapsibility Potential: A Case Study of Semnan Railway Station. University of Qazvin and University of Science and Technology of Tehran, Iran. Vol. 13, Bund. G

31. Reddy, K.R. 2002. Engineering Properties of Soils Based on Laboratory Testing. University of Illinois at Chicago. 174p.

32. Schwartz, K. 1985. Collapsible soils. Problems of Soils in South Africa – State of Art.

33. Zapata, C.E., S.L. Houston and W.N. Houston. 2001. Geotechnical engineering practice for collapsible soils. Arizona State University, Tempe, Arizona. USA. 85287-5306.

Website Sources:

1. http://www.motr.gov.et/web/amhaic/civil-avi

2. http://www.era.gov.et/web/guest/about-us

3. http://en.climate-data.org/location/3655/

APPENDICES

Appendix – A: Natural moisture contents sample log pit

LABORATORY DETERMINATION OF MOISTURE CONTENT OF SOILS Test Method: AASHTO - T 265				
Sample No.	SL-1	Depth	0.5 m	
Date Tested	March 14th to 15th, 2018	Date Sample	March 14th, 2018	
Test No.		**1**	**2**	**Remark**
Container No.		FG_4	CA_5	
Mass of container + moist soil (W_1)	Gm	35.50	38.80	
Mass of container + oven dried soil (W_2)	Gm	34.60	37.60	
Mass of container (W_c)	Gm	20.40	19.50	
Moisture content (w) = $[(W_1 - W_2)/ (W_2 - W_c)]$ x 100 %	%	6.34	6.63	
Average Moisture Content (w %)	**%**	**6.48**		

Appendix – B: Dry density and unit weight and others sample log pits

LABORATORY DETERMINATION OF UNIT WEIGHT SOILS Test Method: ASTM - D 2937			
Sampling Station:	Ambure	Purpose	
Sample No.	SL-2	Depth:	2.5 m
Date Tested:	May 2, 2018	Date Sample	March 14th, 2018

Test No.		1	Remark
Container No.		G_8	
Mass of container + moist soil (W_1)	Gm	42.40	
Mass of container + oven dried soil (W_2)	Gm	40.60	
Mass of container (Wc)	Gm	20.70	
Moisture content (ω) = $[(W_1 - W_2)/ (W_2 - W_c)]$ x 100 %	**%**	9.05	

Trial Number		1	Remark
Mass of undisturbed soil sample , M_t	gm	797.80	
Length of undisturbed soil sample, L_t	cm	13.10	
Diameter of undisturbed soil sample, D_t	cm	7.50	
Volume of undisturbed soil sample, V_t	cm^3	578.74	
Bulk density of the soil, $\rho_b = M_t/V_t$	g/cm^3	1.38	
Dry density of soil, $\rho_d = [\rho_\beta/(1+\omega)]$	g/cm^3	1.26	
Dry unit weight of soil, $\gamma_d = [(\rho_d*g)]$	kN/m^3	12.40	

Unit weight of water, γ_w	kN/m^3	9.81	
Densityof water, ρ_w	g/cm^3	1.00	
Specific gravity of soil, G_s		2.49	
Void ratio, $e_o = (G_s\gamma_w/\gamma_d) - 1$		0.97	
Porosity, $\eta = e_o/(1+e_o)$	%	49.16	
Natural moisture content, ω_o	%	9.05	
Degree of satuation, $S = \omega G_s/e_o$	%	23.26	
Volume of voids, $V_v = \eta * V_t$	cm^3	284.52	
Volume of soil, $V_s = V_t - V_v$	cm^3	294.22	
Total mass of soil, $M_s = \rho_d * V_s$	Gm	371.95	
Initial mass of water, $M_{iw} = \omega_o * M_s$	Gm	33.64	
Volume of water, $V_w = M_{iw} * \rho_w$	cm^3	33.64	
Volume of air, $V_a = V_v - V_w$	cm^3	250.87	
Saturated density of soil, $\rho_{sat} = \rho_w * (G_s+e_o)/(1+e_o)$	g/cm^3	1.76	
Mass of water needed per cm^3 for full saturation of soil, $M_{add} = (\rho_{sat} - \rho_b)*V_a$	Gm	94.65	
Total mass of water needed for full saturation of soil, $M_{tw} = M_i + M_{add}$	Gm	128.29	
Moisture content at full saturation of soil, $\omega_f = 100*(M_{tw}/M_s)$	%	34.49	
Void ratio at full saturation, $e_f = \omega_f * G_s$		0.86	
Collapse potenatial, $C_p = 100*\Delta e/(1+e_o)$	%	5.56	

Appendix – C: Specific gravity sample log pit

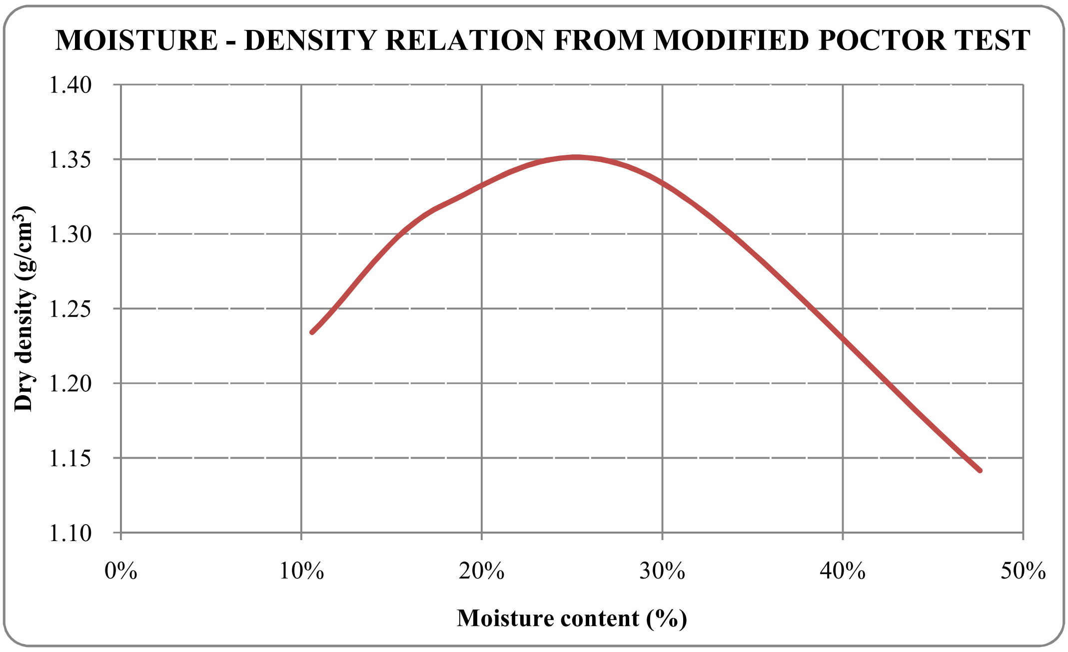
LABORATORY DETERMINATION OF SPECIFIC GRAVITY OF SOILS Test Method: AASHTO - T 100			
Sampling Station	Ambure	Purpose	Specific gravity
Sample No.	SL-3	Depth	4.5
Date Tested	Mar 15th, 2018	Date Sample	March 14th, 2018

COMPUTATIONS			REFERENCE TABLE		
Determination No.	1	2	**Temp (ºC)**	**Rel. Density**	**K**
Pycnometer No.	P_1	P_2	18	0.9986244	1.0004
Correction factor at T_x, K	0.9996	0.9996	19	0.9984347	1.0002
Weight of dry soil, W_s (gm)	25	25	20	0.9982343	1
Observed temprature of water ºC, T_i	20	20	21	0.9980233	0.9998
Desired temprature ºC, T_x	22	22	22	0.9978019	0.9996
Relative density of water at T_x, RD_x	0.9978019	0.9978019	23	0.9975702	0.9993
Relative density of water at T_i, RD_i	0.9982343	0.9982343	24	0.9973286	0.9991
Relative density of water at T_x / Relative density of water at T_i	0.9995668	0.9995668	25	0.9970770	0.9989
Weight of dry, clean pycnometer, W_p (gm)	99.5	90.5	26	0.9968156	0.9986
Weight of pycnometer + dry soil + water, W_{pws} (gm)	363.00	353.70	27	0.9965451	0.9983
Weight of pycnometer + water at T_x , W_{pw} (gm)	347.90	338.70	28	0.9962652	0.998
			29	0.9959761	0.9977
Specific gravity of soil at 20°C, G_s $G_s = K*W_s/(W_s + W_{pw}\text{ (at } T_x) - W_{pws})$ Where; $W_{pw}\text{ (at } T_x) = (RD_x / RD_i)*(W_{pw}\text{ (at } T_i) - W_p) + W_p$	2.52	2.50	30	0.9956780	0.9974
Average specific gravity of soil, G_s	**2.51**				

Appendix – D: Compaction test sample log pit

LABORATORY DETERMINATION OF MOISTURE - DENSITY RELATIONS OF SOILS						
Test Method: AASHTO - T 99/ T180 **Modified Proctor Test**						
Sampling station	Ambure		Depth		6.5	
Sample no.	SR-4		Date sample		March 14th, 2018	
Date tested	16-Mar-18					
Blows per layer	**56**	No. of layers		**5**	Rammer weight (Kg)	**4.5**
Mould diameter (mm)	**152**	Volume of mould (cm^3)		**2123**	Mould height	**117**
Trial number		**1**	**2**	**3**	**4**	
Mass of moist soil + Mass of mould, A	Gm	8,442.46	8,848.74	9,218.60	9,121.71	NMC
Mass of mould, B	Gm	5,545.10	5,545.10	5,545.10	5,545.10	
Mass of moist soil, C = [A-B]	gm	2,897.36	3,303.64	3,673.50	3,576.61	
Density of moist soil, $D = C/V_{mould}$	gm/cm^3	1.36	1.56	1.73	1.68	
Moisture can no.		**Z_{11}**	**E_2**	**A_6**	**CA_5**	
Mass of moist soil + Mass of can, E	gm	71.11	86.24	80.70	120.90	
Mass of dry soil + Mass of can, F	gm	66.30	76.40	67.20	88.20	
Mass of can, G	gm	20.90	21.50	21.00	19.50	
Mass of water, H = E – F	gm	4.81	9.84	13.50	32.70	

Mass of dry soil, I = F – G	gm	45.40	54.90	46.20	68.70	
Moisture content, J = (H/I)*100%	(%)	10.59%	17.92%	29.22%	47.60%	
Dry density, K = D/(1+J)	gm/cm^3	1.23	1.32	1.34	1.14	



Appendix – E: Particle size analysis sample log pit

LABORATORY DETERMINATION OF GRAIN SIZE OF SOILS									
Test Method: ASTM - D 422 / AASHTO - T 88									
SIEVE ANALYSIS									
Sample no.			SR-1						
Sieve no.	**Sieve size (mm)**	**Weight of sieve (gm)**	**Weight of sieve with soil retained (gm)**	**Mass of retained soil (gm)**	**Percentage retained (%)**	**Cumulative mass retained (gm)**	**% of cumulative mass retained (%)**	**% passing (%)**	**Remark**
3/4'	19	729.0	751.6	22.6	1.1	22.6	1.13	98.87	
3/8'	9.5	608.5	627.0	18.5	0.9	41.1	2.06	97.95	
4	4.75	408.4	447.8	39.4	2.0	80.5	4.03	95.98	
10	2	498.0	558.0	60.0	3.0	140.5	7.03	92.98	
16	1.18	451.0	585.3	134.3	6.7	274.8	13.74	86.26	
30	0.6	435.2	682.0	246.8	12.3	521.6	26.08	73.92	
40	0.425	390.9	652.0	261.1	13.1	782.7	39.14	60.87	
100	0.15	372.6	759.9	387.3	19.4	1170.0	58.50	41.50	
200	**0.075**	**357.6**	**861.0**	**503.4**	**25.2**	**1673.4**	**83.67**	**16.33**	
Pan	0	381.1	707.7	326.6	16.3	2000.0	100.00	-	
			Total	**2000.0**	**100.0**				

LABORATORY DETERMINATION OF GRAIN SIZE OF SOILS Test Method: ASTM - D 422 / AASHTO - T 88 HYDROMETER ANALYSIS									
Sample no.		SR-1			Apparatus				
Time (min)	**Hydrometer Reading (R_A)**	**Temperature (ºC)**	**Composite Correction**	**Corrected Hydrometer Reading (R_C)**	**Effective Depth, L (cm)**	**Coefficient K**	**Grain Size, D (mm)**	**% of Soil in Suspension, P**	**Adjusted % Finer, P_A (%)**
5	1.017	28	0.0013	1.0157	11.8	0.0135	0.02074	26.25	4.29
15	1.012	28	0.0013	1.0107	13.1	0.0135	0.01262	17.89	2.92
30	1.008	28	0.0013	1.0067	14.2	0.0135	0.00929	11.20	1.83
60	1.006	28	0.0013	1.0047	14.7	0.0135	0.00668	7.86	1.28
250	1.004	28	0.0013	1.0027	15.2	0.0135	0.00333	4.51	0.74
1440	1.003	28	0.0013	1.0017	15.5	0.0135	0.00140	2.84	0.46

Grain size (mm)	**Percentage Finer (%)**
19	98.87
9.5	97.95
4.75	95.98
2	92.98
1.18	86.26
0.6	73.92
0.425	60.87
0.15	41.50
0.075	16.33
0.021	4.29
0.013	2.92
0.009	1.83
0.007	1.28
0.003	0.74
0.001	0.46

Soil type	**Average grain size, mm (ASTM D 2487)**	**Percentage (%)**	**Remark**
Gravel		**4.03**	
Coarse Gravel	75 to 19	1.13	
Fine Gravel	19 to 4.75	2.90	
Sand		**79.65**	
Coarse Sand	4.75 to 2.0	3.00	
Medium Sand	2.0 to 0.425	32.11	
Fine Sand	0.425 to 0.075	44.54	
Silt	0.075 to 0.002	15.87	
Clay	<0.002	0.46	

Soil type	**Average grain size, mm (AASHTO)**	**Percentage (%)**	**Remark**
Gravel	19 to 2.0	5.90	
Sand	2.0 to 0.075	76.65	
Silt	0.075 to 0.002	15.87	
Clay	<0.002	0.46	

D_{60} = Diameter corresponding to 60% finer, mm	0.40
D_{30} = Diameter corresponding to 30% finer, mm	0.12
D_{10} = Diameter corresponding to 10% finer, mm	0.04
Uniformity coefficient, $C_u = D_{60}/D_{30}$	3.33
Coefficient of gradation, $C_c = [D_{30}^2/(D_{60}*D_{10})]$	0.80

SOIL CLASSIFICATION

CASE -I: UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) - ASTM: D 2487

Classification Criteria Description	**% Value**	**Major Category**	**Group Name**	**Group Symbol**	**Remark**
STEP - 1: Identifying major category and group symbol (Das, Table 4.2)					
Percentage retained at No. 200 sieve	83.67	Coarse grained soils	Gravel or Sand	G or S	<50% passed No. 200
Percentage passing through No. 200 sieve	16.33				
STEP - 2: Identifying a specific group symbol (Das, Table 4.2)					
Percentage retained above No. 4 sieve	4.03	Coarse grained soils	Sand	S	> 50% passed No. 4
Percentage passing through No. 4 sieve	95.98				
STEP - 3: Identifying a specific group symbol (Das, Table 4.2)					
Percentage passing through No. 200 sieve, F_{200}	16.33	Coarse grained soils	Sand	SM	F_{200} > 12 and PI < 4
Percentage retained at No. 200 sieve, R_{200}	83.67				0.05 < 0.5 (Ok!)
Percentage retained at No. 4 sieve, R_4	4.03				
$R_4/R_{200} \leq 0.5$ (Sands)	0.05				
Uniformity coefficient, C_u	3.33				Not satisfy the criteria
Coefficient of gradation, C_c	0.80				

STEP - 4: Identifying a group name					
Percentage passing 76.2 mm	100.00	Coarse grained soils	Silty Sand	SM	<15% gravel and F_{200} > 12%
Percentage passing No. 4	95.98				
Percentage passing No. 200	16.33				
Pan	-				
Percentage of Gravel	4.03				
Percentage of Sand	79.65				
Percentage of Silt	27.39				
Percentage of Clay	0.46				
Hence, the soil can be classified as **Non - Plastic Silty Sand (SM)**					

Classification Criteria Description	**% Value**	**General Classification**	**Group Classification**	**Group Symbol**	**Remark**
STEP - 1: Identifying major category and group symbol (Das, Table 4.1)					
Percentage of sample passing No. 200 sieve	16.33	Granular materials	Gravel or Sand	A–1, A–2, or A–3	< 35% passed No. 200
STEP - 2: Characteristics grain size & plasticity (Das, Table 4.1)					
Plasticity Index, PI	NP	Granular materials	Fine Sand	A–3	Non plastic material
Percentage of Gravel (75 mm to 2 mm sieve)	7.03				
Percentage of Sand (2 mm to 0.075 mm sieve)	76.65				
Percentage of Silt and Clay (< 0.075 mm)	16.33				
Hence, the soil can be classified as **Non - Plastic Fine Sand (A-3)**					

Appendix – F: Collapse test analysis log pits

LABORATORY DETERMINATION OF MEASUREMENT OF COLLAPSE POTENTIAL OF SOILS ASTM: D 5333			
STANDARD METHOD FOR MEASUREMENT OF COLLAPSE POTENTIAL OF SOILS			
Sample No.	SR - 2	Date of testing	May 08th to 15th, 2018
Dial at reading seating stress, d_o in mm	0.0	Ring diameter, mm	63.0
Initial specimen height, h_o in mm	20.0	Ring height, mm	20.0
Lever arm ratio for loading	9.0		

COMPUTATION FOR STRESS - STRAIN (COMPRESSION) CURVE

Wetting	Load Increment (Kpa)	Time	Dial reading at seating stress, d_o (mm)	Dial reading, d (mm)	d - d_o (mm)	h_o (mm)	Strain (%)
Before saturation	5	5 min	0.00	-	-	20	-
	12	1 h	0.00	0.001	0.001	20	0.01
	25	1 h	0.00	0.412	0.412	20	2.06
	50	1 h	0.00	0.820	0.820	20	4.10
	100	1 h	0.00	1.380	1.380	20	6.90
	200	1 h	0.00	1.944	1.944	20	9.72
After saturation	200	24 h	0.00	2.806	2.806	20	14.03
	400	24 h	0.00	3.310	3.310	20	16.55
	600	24 h	0.00	3.904	3.904	20	19.52
	800	24 h	0.00	4.416	4.416	20	22.08
	1200	24 h	0.00	4.903	4.903	20	24.52

COLLAPSE INDEX (Ie) COMPUTATION FROM COMPRESSION CURVE

Description	Unit	
Initial specimen height, H_o	mm	20
Dial reading at 200 kPa stress level before wetting, d_e	mm	1.944
Dial reading at 200 kPa stress level after wetting, d_f	mm	2.806
Collapse Index, $I_e = 100*(d_f - d_e)/h_o$	**%**	**4.31**

COMPUTATION FOR COMPRESSION CURVE FROM VOID RATIO							
Load Increment (Kpa)	**Time**	**d_f (mm)**	**H_o (mm)**	**H_s (mm)**	**$\Sigma\Delta H$ (mm)**	**$H = (H_o - \Sigma\Delta H)$**	**$e = (H - H_s)/H_s$**
5	5.0 min	0.000	20	7.71	0.0	20.000	1.592
12	1.0 h	0.001	20	7.71	0.001	19.999	1.592
25	1.0 h	0.412	20	7.71	0.412	19.588	1.539
50	1.0 h	0.820	20	7.71	0.820	19.180	1.486
100	1.0 h	1.380	20	7.71	1.380	18.620	1.414
200	**1.0 h**	**1.944**	**20**	**7.71**	**1.944**	**18.056**	**1.341**
200	**24.0 h**	**2.806**	**20**	**7.71**	**2.806**	**17.194**	**1.229**
400	24.0 h	3.310	20	7.71	3.310	16.690	1.163
600	24.0 h	3.904	20	7.71	3.904	16.096	1.086
800	24.0 h	4.416	20	7.71	4.416	15.584	1.020
1200	24.0 h	4.903	20	7.71	4.903	15.097	0.957

COLLAPSE POTENTIAL OF SOILS (Cp) COMPUTATION FROM VOID RATIO					
Description	**Unit**				
Mass of can, M_c	gm	24			
Mass of ring, M_r	gm	66			
Mass of dry soil+ring+can after test, M_D	gm	180.7			
Mass of dry soil, $M_d = (M_D - M_c - M_r)$	gm	150.4			
Mass of dry soil, M_d	gm	60.40			
Specific gravity of the soil, G_s		2.51			
Diameter of ring, D_r	mm	63			
Initial specimen height, H_o	mm	20			
Density of water, ρ_w	g/cm^3	1			
Volume of soil, $V_s = M_d/G_s\rho_w$	cm^3	24.05			
Specimen area, A (Constant)	cm^2	31.17			
Equivalent height of soil, $H_s = V_s/A$	mm	7.71			
Initial void ratio at its natural water content pressure before wetting, e_o		1.592			
Initial void ratio at its natural water content under 200 kPa of vertical pressure before wetting, e_i		1.341			
Final void ratio after soaking under 200 kPa of vertical pressure after wetting, e_f		1.229			
$\Delta e_c = e_i - e_f$		0.112			
Collapse Potential, $C_p = 100^* (\Delta e_c)/(1+e_o)$	**%**	**4.31**			

Appendix – G: Direct shear test analysis sample log pit

LABORATORY DETERMINATION OF SHEAR STRENGTH OF SOILS AASHTO: T – 236 DIRECT SHEAR TEST (UU Test)										
Sample No.		SR-3				Length of sample			60 mm	
		27.25 kPa Applied Vertical Stress			54.5 kPa Applied Vertical Stress			81.75 kPa Applied Vertical Stress		
Horizontal Displacement (mm)	Corrected Area (mm^2)	Proving Ring Reading	Shear Load (N)	Shear Stress (kPa)	Proving Ring Reading	Shear Load (N)	Shear Stress (kPa)	Proving Ring Reading	Shear Load (N)	Shear Stress (kPa)
0.0	3600	-	-	-	-	-	-	-	-	-
0.2	3588	3.5	7.11	1.98	7.0	14.2	3.96	9.0	18.3	5.09
0.4	3576	7.0	14.21	3.97	11.5	23.3	6.53	18.0	36.5	10.22
0.6	3564	12.5	25.38	7.12	16.5	33.5	9.40	32.0	65.0	18.23
0.8	3552	14.5	29.44	8.29	17.5	35.5	10.00	41.0	83.2	23.43
1.0	3540	18.0	36.54	10.32	19.5	39.6	11.18	49.0	99.5	28.10
1.2	3528	20.0	40.60	11.51	21.5	43.6	12.37	57.0	115.7	32.80
1.4	3516	21.0	42.63	12.12	24.5	49.7	14.15	61.5	124.8	35.51
1.6	3504	22.5	45.68	13.04	27.5	55.8	15.93	67.0	136.0	38.82
1.8	3492	23.0	46.69	13.37	30.0	60.9	17.44	72.0	146.2	41.86
2.0	3480	23.5	47.71	13.71	31.5	63.9	18.38	74.5	151.2	43.46
2.2	3468	23.5	47.71	13.76	34.5	70.0	20.19	78.0	158.3	45.66
2.4	3456	24.5	49.74	14.39	36.5	74.1	21.44	81.5	165.4	47.87
2.6	3444	25.0	50.75	14.74	42.0	85.3	24.76	82.5	167.5	48.63
2.8	3432	25.5	51.77	15.08	47.5	96.4	28.10	85.0	172.6	50.28
3.0	3420	26.0	52.78	15.43	50.5	102.5	29.98	85.5	173.6	50.75
3.2	3408	26.5	53.80	15.78	50.5	102.5	30.08	86.5	175.6	51.52
3.4	3396	27.0	54.81	16.14	51.0	103.5	30.49	87.5	177.6	52.30
3.6	3384	27.5	55.83	16.50	53.5	108.6	32.09	87.5	177.6	52.49
3.8	3372	28.0	56.84	16.86	54.0	109.6	32.51	87.5	177.6	52.68
4.0	3360	28.5	57.86	17.22	55.0	111.7	33.23	87.5	177.6	52.86
4.2	3348	29.0	58.87	17.58	56.5	114.7	34.26	88.0	178.6	53.36
4.4	3336	29.5	59.89	17.95	56.5	114.7	34.38	88.0	178.6	53.55
4.6	3324	30.0	60.90	18.32	56.5	114.7	34.51	88.0	178.6	53.74
4.8	3312	30.5	61.92	18.69	56.5	114.7	34.63	88.5	179.7	54.24
5.0	3300	31.0	62.93	19.07	57.0	115.7	35.06	88.5	179.7	54.44
5.2	3288	31.5	63.95	19.45	57.0	115.7	35.19	88.5	179.7	54.64

5.4	3276	31.5	63.95	19.52	57.5	116.7	35.63	88.5	179.7	54.84
5.6	3264	31.5	63.95	19.59	57.5	116.7	35.76	89.0	180.7	55.35
5.8	3252	31.5	63.95	19.66	57.5	116.7	35.89	89.0	180.7	55.56
6.0	3240	31.5	63.95	19.74	58.0	117.7	36.34	89.0	180.7	55.76
6.2	3228	31.5	63.95	19.81	58.0	117.7	36.47	89.0	180.7	55.97
6.4	3216	31.5	63.95	19.88	58.0	117.7	36.61	89.0	180.7	56.18
6.6	3204	31.0	62.93	19.64	58.0	117.7	36.75	89.0	180.7	56.39
6.8	3192				57.0	115.7	36.25	89.0	180.7	56.60
7.0	3180							89.0	180.7	56.81
7.2	3168							88.5	179.7	56.71
Maximum Shear Stress		**19.88**			**36.75**			**56.81**		

Appendix – H: Soil Sampling and Testing Activities Photographs















797.8